Memo

Date:	Monday, July 20, 2020
Project:	Seismic Resilience Assessment
To:	Brett Musick, PE, City of Newberg
From:	Andy McCaskill, P.E.; Katie Walker, P.E.
Subject:	Executive Summary

Introduction

The City of Newberg (City) operates a water system consisting of a wellfield, raw water transmission pipelines, a water treatment plant, three water storage reservoirs, one pump station, and distribution system pipelines. In support of the 2017 Water Master Plan and Oregon Health Authority (OHA) guidelines, the City conducted a water system seismic resilience assessment (SRA). The purpose of the SRA is to define level-of-service (LOS) goals, evaluate the expected performance of the system during a Cascadia Subduction Zone (CSZ) earthquake, and identify recommended mitigation measures to address deficiencies. The SRA included the following studies:

- Seismic Resiliency Goals – during this study, goals and retrofit performance criteria were defined (see Appendix A).
- Geotechnical Engineering Report (GER) – during this study, geotechnical conditions were reviewed to identify seismic hazards (see Appendix B).
- Vulnerabilities Assessments – the purpose of this report was to assess the vulnerabilities of the City's water system and the pipeline bridge (see Appendix C).
- Mitigation Recommendations – mitigation strategies were recommended and developed at a conceptual level to address some system vulnerabilities (see Appendix D).
- Recommendations for Future Studies – additional studies were identified to clarify and confirm the City's seismic mitigation needs (see Appendix E).

This executive summary presents the purpose and key findings from each study.

Seismic Recovery Goals

In this study, the water system level of service goals were established to define performance expectations after a CSZ earthquake. A collaborative workshop was conducted to identify the restoration priorities for the City with short-term (no disruption) needs including fire suppression and the Providence Newberg Medical Center. Using guidelines in the Oregon Resilience Plan (ORP) tailored to the City's needs, recovery goals were identified for all major components of the water system (see Attachment A).

The study also identified the backbone of the City's water system, which are the components required to meet the short-term needs outlined in the recovery goals (see Attachment B). These components should be designed or modified to experience only minor damage during a CSZ earthquake.

In addition to defining goals and identifying the system backbone, objectives for retrofitting existing water system components were identified based on how quickly they could be restored.

Geotechnical Engineering Report

The GER included a review of the existing geologic and geotechnical conditions to develop seismic ground motion, seismic hazard, and permanent ground deformation hazard maps. At the WTP, the following was conducted:

- One boring
- Evaluation of liquefaction potential and liquefaction-induced settlement
- Evaluation of potential for slope failure
- Evaluation of seismically induced ground movement and potential for lateral spread

Vulnerabilities Assessment

In the Vulnerabilities Assessments, water system components were compared against the seismic hazard maps developed in the GER showing peak ground velocity, probability of liquefaction, and landslide induced permanent ground deformation. In addition to a desktop review, a site visit was conducted to inspect the water system and interview City personnel. Based on the assessment, the following vulnerabilities were identified:

Pipeline Bridge
A desktop assessment was conducted to review the bridge, but record drawings were not available. The assessment concluded that the bridge and transmission main are unlikely to survive a CSZ earthquake. A retrofit, likely costing in the tens-of-millions, would be required with additional studies and inspections needed to clarify and confirm the bridge conditions.

Wellfield
In general, the wells are likely at risk for liquefaction and lateral spread. During a CSZ earthquake, differential settlement could occur between the well casing and pipe connection, the well screen could be plugged, and the seismic shaking could cause groundwater levels to fluctuate. Additional vulnerabilities include lack of backup power and lack of reliable access across the river.

30-inch HDPE Transmission Main
Based on a review of the geotechnical documents from the construction of the main, the transmission main is susceptible to liquefaction induced settlement on the southern side of the river, and at the shallowest section on the northern side of the river. These conditions would likely result in differential settlement causing pipe separation or damage during a CSZ earthquake.

Water Treatment Plant
Studies conducted at the WTP indicate up to two feet of lateral spread displacements at a distance of approximately 300 feet from the crest of the slope during a CSZ earthquake. Stability analyses also showed seismically induced ground displacements in the range of approximately 7.5 feet. In addition, the review of the slope indicated that it is only marginally stable under static conditions and not stable in seismic or post-seismic conditions.

A site visit was conducted to assess components at the WTP. In general, the review of the structures indicated that none meet either the structural or non-structural performance objectives outlined as part of the Seismic Recovery Goals. Significant work is required at the WTP to meet recovery goals, and it was recommended that further evaluation be conducted to compare the cost of upgrading the WTP versus building a new WTP. However, it should be noted that while the buildings will not withstand a CSZ event, the plant site itself is not susceptible to a landslide into the river.

Water System Backbone
The seismic hazard maps prepared under the GER were applied against pipeline information, such as age, corrosion, and material, to identify the estimated number of pipeline breaks and length of repair. For the non-landslide areas, it is estimated that 245 breaks will occur (see Attachment C, Table 1). For the landslide prone areas, a range of 84 to 626 breaks will occur (see Attachment C, Table 2).

Water Distribution Pipelines
The water distribution network is considered a lower priority for seismic resilience based on the LOS goals established by the City. For the non-landslide areas, it is estimated that 1,159 water breaks will occur (see Attachment C, Table 3). For the landslide prone areas, a range of 336 to 2,518 breaks will occur (see Attachment C, Table 4).

WTP Yard Piping
Several vulnerabilities exist at the WTP including:

- Lack of isolation valves at the WTP to prevent water loss or cross contamination, or preserve water storage at the WTP
- Lack of a WTP bypass line to supply water from the wellfield to the distribution for firefighting or domestic use (boiling required for potable use)
- Lack of seismic couplings at building pipeline penetrations to prevent pipe separation

Water Storage Tanks Yard Piping
Vulnerabilities at the Corral Creek Site include:

- Flexible couplings may need to be replaced with seismic couplings to provide more movement during an earthquake
- Lack of seismic couplings on the pipeline to prevent pipe separation
- Lack of a hydraulic control valve to quickly protect water storage if a loss of power or SCADA occurs

Vulnerabilities at the North Valley Water Storage Tanks include:

- Unknown capabilities of couplings at pipe penetrations
- Inlet/outlet line will be subject to landslide movements and pipeline separation
- Lack of a hydraulic control valve to quickly protect water storage if a loss of power or SCADA occurs

Water System Operations
Vulnerabilities and observations related to water system operations include:

- No fire flow or pressure deficiencies were identified that could affect system recovery after a CSZ earthquake
- No deficiencies in water system storage capacity
- SCADA system could be improved or expanded to include greater centralized monitoring and control of the system, with backup power and communications improved at identified locations
- Lack of a redundant water supply, which is currently being investigated under another study
- Ensure GIS mapping is adequately detailed to locate critical isolation valves and facilities in an emergency.

Mitigation Recommendations

The Vulnerabilities Assessment identified areas where the City needs to improve or retrofit the water system. The following five mitigation strategies were identified as top priorities for the City. Mitigation strategies were presented in two separate memos: one for recommendations at the WTP and one for recommendations within the distribution and storage system.

Rehabilitation of Existing WTP
The existing WTP is susceptible to liquefaction, ground deformation, and lateral spreading. The goal of rehabilitation is to address the deficiencies identified in previous studies by installing ground improvements between the WTP site and the shoreline to prevent lateral movement and strengthening structural components to withstand a CSZ event. The range of construction cost estimates could be from $3.3M to $13M.

Construction of Greenfield WTP
Since several structures at the existing WTP are nearing the end of their useful life, an alternative strategy is to replace the existing plant with a seismically resilient one. The range of construction cost for a new plant could be from $12.3M to $49.2M.

Emergency Connection and Control at the WTP
As identified in the vulnerability assessment, the WTP poses several risks if a CSZ earthquake occurs. By adding a point for emergency cross-connection and installing hydraulic control valves, the plant could be isolated during an earthquake event, allowing raw water to continue

into the distribution system. The construction cost for these improvements is approximately $500K.

Improvements to Water Storage
The vulnerability assessment identified the potential for water loss at the storage tanks during a CSZ earthquake. By adding hydraulic control valves and replacing a portion of the pipe at North Valley Water Storage Tanks, water storage at the tanks could be preserved. The construction for the improvements at the Corral Creek Site is approximately $300K, and $750K at the North Valley Water Storage Tanks.

Cast Iron and Concrete Pipe Replacement
Based on the evaluation of pipeline in the City's backbone, old cast iron and concrete pipe poses the greatest risk for damage during a CSZ earthquake. The construction costs for the replacement of pipe is approximately $12.5M and represents the replacement of more than 37,000 linear feet of pipe.

Recommendations for Future Studies

To further refine mitigation strategies, additional studies are required. Studies recommended include the following list (Note that this list is not all-inclusive as other efforts will likely be identified):

- Develop new engineering standards to address seismic resiliency needs in new infrastructure or buildings
- Identification of alternative water demands that could impact water storage available within the system
- Additional geotechnical investigations to better classify the seismic hazards that the water system may experience and allow the City to focus on the most hazardous areas.
- Investigate specific structural recommendations for structures at the WTP and other City facilities
- Evaluate specific mitigation strategies for the pipeline bridge
- Investigate additional mitigation strategies that address remaining vulnerabilities

Attachment A:

Water System Recovery Goals

City of Newberg Water System Recovery Goals
(adapted from OSSPAC 2013 and NIST 2015)

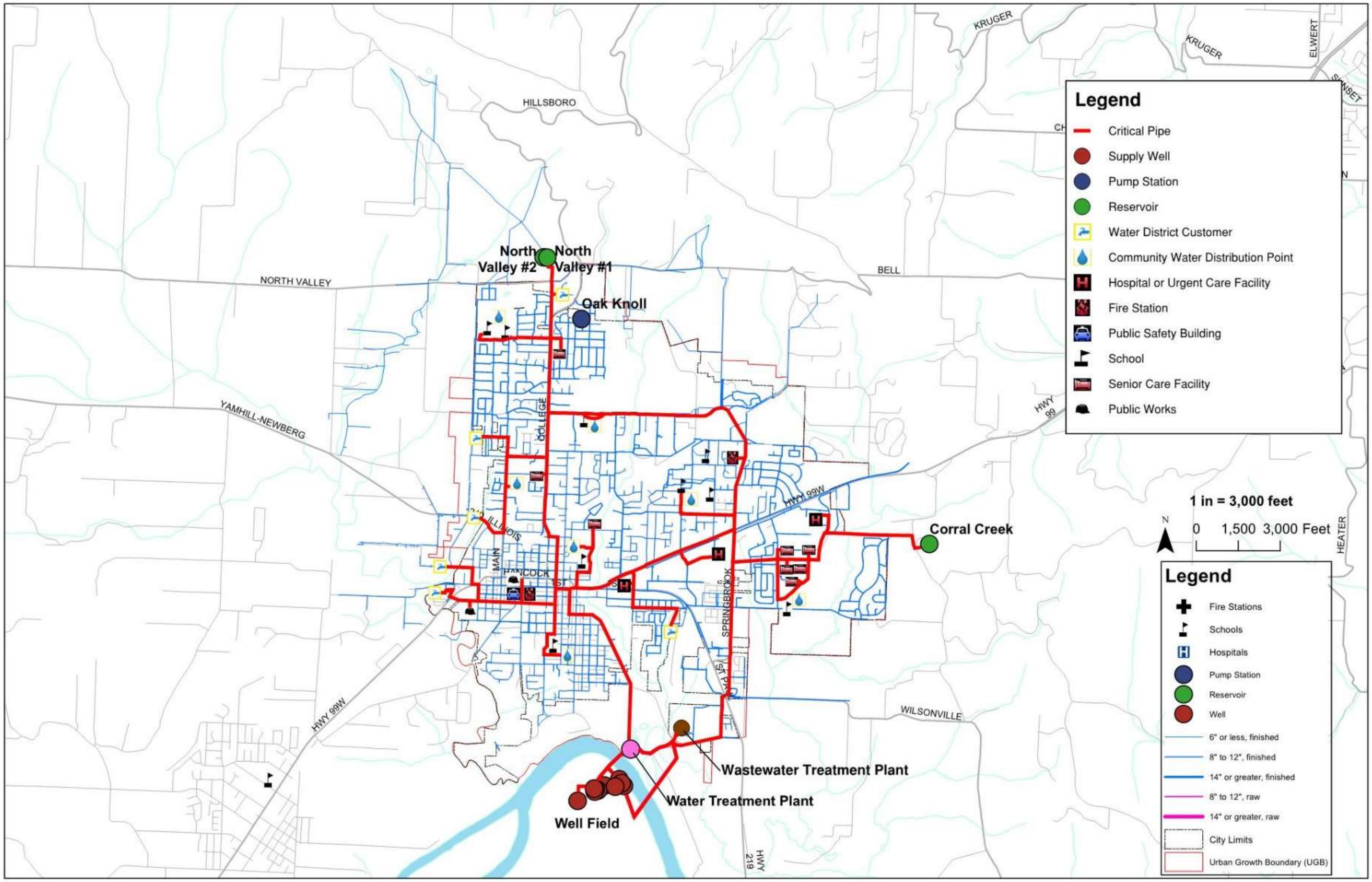
Water Systems	Target Timeframe for Recovery							
	Phase 1: Short-Term			Phase 2: Intermediate			Phase 3: Long-Term	
	Days			Weeks			Months	
	0-1	1-3	3-7	1-2	2-4	4-12	3-6	6-12
Source								
Raw or source water and terminal reservoirs	R	Y		G				
Raw water conveyance (pump stations and piping to WTP)	R	Y		G				
Water Production	R	Y		G				
Well and/or Treatment operations functional	R	Y		G				
Transmission (including Booster Stations)								
Backbone transmission facilities (pipelines, pump station, and tanks)	G							
Water for fire suppression at key supply points (to promote redundancy)	G							
Control Systems								
SCADA and other control systems	G							
Distribution								
Critical Facilities								
Wholesale Users (other communities, rural water districts)	G							
Hospitals	G							
EOC, Police Stations, Fire Stations, Public Works Buildings	Y	G						
Emergency Housing								
Emergency Shelters	Y	G						
Housing/Neighborhoods								
Potable water available at community distribution centers		Y	G					
Water for fire suppression at fire hydrants			R	Y	G			
Community Recovery Infrastructure								
All other clusters			R	Y	G			

Key to Table

Desired time to restore components to 30% operational	R
Desired time to restore components to 60% operational	Y
Desired time to restore components to 90% operational	G

Attachment B:

Water System Backbone Map


Legend
Critical Pipe
Supply Well
Pump Station
Reservoir
Water District Customer
Community Water Distribution Point
Hospital or Urgent Care Facility
Fire Station
Public Safety Building
School
Senior Care Facility
Public Works
1 in = 3,000 feet
0 1,500 3,000 Feet
Legend
Fire Stations
Schools
Hospitals
Pump Station
Reservoir
Well
6" or less, finished
8" to 12", finished
14" or greater, finished
8" to 12", raw
14" or greater, raw
City Limits
Urban Growth Boundary (UGB)
North Valley #2
North Valley #1
Oak Knoll
Corral Creek
Wastewater Treatment Plant
Water Treatment Plant
Well Field
NORTH VALLEY
HILLSBORO
BELL
KRUGER
ELWERT
YAMHILL-NEWBERG
COLLEGE
ILLINOIS
MAIN
HANCOCK
SPRINGBROOK
HWY 99W
WILSONVILLE
HWY 219
HEATER

Attachment C:

Water System Summary Tables

Table 1. Water System Backbone Summary, Non-Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard (ft)	Percentage of Backbone Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft)
Cast Iron	23,860	25%	89	4	268
Ductile Iron	58,433	62%	109	2	536
RCC	12,592	13%	47	4	268
Grand Total	**94,884**	**100%**	**245**	**3**	**387**

Table note: Estimated Number of Breaks Due to peak ground velocity (PGV) and peak ground deformation (PGD) (non-landslide) by Pipe Material

Table 2. Water System Backbone Summary, Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard(ft.)	Percentage of Backbone Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft.)
Cast Iron	1,193	1%	30-228	25-191	5-39
Ductile Iron	2,922	3%	37-279	13-95	10-79
RCC	630	1%	16-120	25-191	5-39
Grand Total	**4,744**	**5%**	**84-626**	**64-477**	**5-79**

Table note: Estimated Number of Breaks Due to PGD (landslide) by Pipe Material

Table 3. Water Distribution System Summary, Non-Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard (ft)	Percentage of Distribution Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft)
C-900	11,713	3%	35	3	336
CI	106,470	23%	397	4	268
DI	296,271	63%	553	2	536
PVC	28,707	6%	85	3	336
Other	23,905	5%	89	4	268
Grand Total	**467,065**	**100%**	**1,159**	**2**	**403**

Table note: Estimated Number of Breaks Due to PGV and PGD (non-landslide) by Pipe Material

Table 4. Water Distribution System Summary, Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard(ft.)	Percentage of Distribution Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft.)
C-900	586	3%	12-89	20-153	7-49
CI	5,324	23%	135-1,016	25-191	5-39
DI	14,814	63%	188-1,413	13-95	10-79
PVC	1,435	6%	29-219	20-153	7-49
Other	1,195	5%	30-228	25-191	5-39
Grand Total	**23,353**	**100%**	**336-2,518**	**59-439**	**5-79**

Table note: Estimated Number of Breaks Due to PGD (landslide) by Pipe Material

Appendix A:
Seismic Resiliency Goals

WATER SYSTEM SEISMIC RESILIENCE STUDY

CITY OF NEWBERG PUBLIC WORKS DEPARTMENT
NEWBERG, OREGON

Final Technical Memorandum: Seismic Recovery Goals

August 16th, 2019
SEFT Project Number: B19009.00

SEFT Consulting Group
4800 SW Griffith Drive, Suite 100
Beaverton, OR 97005

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1.0 Introduction and Background

1.1 City of Newberg Water System Description

The City of Newberg water system currently consists of the City's wellfield, raw water transmission pipelines, water treatment plant, three water storage reservoirs, one pump station, and distribution system pipelines. The entire water service area is one pressure zone, except for approximately 40 customers that are served by the Oak Knoll booster pump station. The system uses approximately 56 miles of distribution pipelines to provide water to business and residential customers within the City of Newberg service area and six small water district wholesale customers. The primary water supply is the City's well field located on the south side of the Willamette River in Marion County. Two raw water transmission mains cross the river to the treatment plant. An under river 30-inch diameter high density polyethylene transmission main can supply 100% of the treatment plant capacity. An older 24-inch diameter cast iron transmission main is supported by a decommissioned highway bridge. The City's water treatment plant is a conventional filtration facility with a nominal capacity of 9 million gallons per day (MGD). The current average day demand for the water system is approximately 2.4 MGD and summertime demands can increase to approximately 4.5 MGD.

1.2 Seismic Resilience Study

Based on recommendations contained in the 2017 City of Newberg Water Master Plan and requirements of the Oregon Health Authority, the City of Newberg is conducting a water system seismic resilience study. This study will evaluate the expected performance of the City water system following a Magnitude 9.0 (M9.0) Cascadia Subduction Zone (CSZ) earthquake and identify preliminary recommendations for improvements that should be implemented to enable the City to more rapidly restore water service after a major earthquake, to meet community social and economic needs. The scope of this seismic resilience study includes:

1. Define water system level of service (LOS) goals for the City water system following a major seismic event;
2. Identify key backbone system components that are required to achieve these LOS goals, including the locations of key supply points for water for fire suppression and community water distribution;
3. Define performance criteria for individual system components that are required to achieve these LOS goals;
4. Conduct a limited geotechnical seismic hazards evaluation for the City water system and slope stability analysis at the water treatment plant site (Shannon & Wilson);
5. Conduct a limited well/pipeline (HDR), and structural/nonstructural (SEFT/HDR) vulnerability assessment to determine estimated system performance following a M9.0 CSZ earthquake;

6. Identify gaps between the LOS goals and current performance estimates; and
7. Develop preliminary mitigation recommendations to close these gaps utilizing new or retrofit infrastructure, changes to design standards, enhancements in emergency response planning, and recommendations for further study.

This Technical Memorandum (TM) presents the HDR team recommendations related to scope items 1 through 3.

1.3 Resilience Planning by Other Metro Region Agencies

The resilience planning effort being undertaken by the City of Newberg is similar to the planning activities undertaken by several Portland metro region agencies. Additionally, numerous other agencies on the west coast of the United States and Canada are actively conducting resilience planning and resilience-based capital improvement projects.

Tualatin Valley Water District, City of Hillsboro Water Department, and Willamette Water Supply Program

TVWD and the City of Hillsboro Water Department have each completed a water system resilience plan and they are partnering to complete the billion-dollar Willamette Water Supply Program (WWSP) to provide an additional water supply for the region. When complete, the WWSP will greatly enhance the ability of the partner agencies to deliver water to their customers immediately after a major earthquake by providing a resilient and reliable water supply for the region, designed to meet stringent seismic performance goals.

City of Portland

The Portland Water Bureau has completed a water system resilience planning project and is beginning to incorporate recommendations from the plan into their capital improvement projects. The Bureau of Environmental Services has completed a wastewater system seismic resilience master plan and has already begun to incorporate early action item recommendations into practice.

City of Gresham

The City of Gresham has completed resilience planning projects for both their water and wastewater systems and are beginning to incorporate recommendations from these plans into their capital improvement projects. They have successfully leveraged their water system resilience plan to obtain Federal Emergency Management Agency pre-disaster mitigation grant funding to implement seismic improvements at one of their water reservoirs.

2.0 Community Resilience

Events like Hurricane Katrina in 2005, the Great East Japan M9.0 Earthquake and Tsunami in 2011, and Hurricane Sandy in 2012 have underscored the devastating impacts that natural disasters can inflict at a local, regional, state, and multi-state level. The Federal government has defined the National Preparedness Goal as: "A secure and resilient Nation with the capabilities required across the whole community to prevent, protect against, mitigate, respond to, and recover from the threats and hazards that pose the greatest risk" (FEMA, 2015).

One strategy to achieve this National Preparedness Goal is to plan for and implement programs and strategies to improve disaster resilience at the local, regional, state, and national level. Oregon is a national leader in community resilience. In February of 2013, the Oregon Seismic Safety Policy Advisory Commission submitted a report to the 77th Legislative Assembly entitled the *Oregon Resilience Plan: Reducing Risk and Improving Recovery for the Next Cascadia Earthquake and Tsunami* (OSSPAC, 2013). The report discussed the risk that is faced by the citizens of Oregon from an impending Cascadia Subduction Zone earthquake and accompanying tsunami, and the gaps that exist between the current state of Oregon's infrastructure and where it needs to be. In addition to life safety impacts, the report also highlighted the economic vulnerabilities to individuals and communities from such an event. The *ORP* went on to outline steps that can be taken over the next 50 years to bring the state closer to resilient performance through a systematic program of vulnerability assessments, capital investments in public infrastructure, new incentives to engage the private sector, and policy changes that reflect current understanding of the Cascadia threat. While the *ORP* specifically addresses improving resilience in the aftermath of a major earthquake, implementation of the plan is also expected to improve resilience for other hazards.

A primary focus of the *ORP* goals is to minimize the long-term economic damage associated with the potential out-migration of businesses and population that would be expected to occur following a major disaster if basic services cannot be restored rapidly enough to meet the communities social and economic needs. Resilience of the water system will be key to the region's economic recovery. For example, the fundamental goal of quickly restoring the supply of safe drinking water to homes and businesses will help to enable residents to shelter-in-place and businesses to resume operation as quickly as possible after the event. Small businesses are particularly vulnerable to being closed for an unplanned amount of time and many may not be able to re-open if closed for more than a month. Each business closing negatively impacts employment, tax revenue, and the long-term economic and social viability of the City. The more rapidly that businesses are able to reopen, the quicker revenue will normalize, and money will circulate within the region's economy. At a fundamental level, the water system must be functioning at a certain level for service fees to be collected to provide revenue for the City of Newberg to sustain everyday functions and to help fund the recovery process.

2.1 Definition

In the field of community disaster planning, a common definition of "resilience" has been put forth by Presidential Policy Directive (PPD). PPD-8 [2011] defines resilience as "the ability to adapt to changing conditions and withstand and rapidly recover from disruption due to emergencies." PPD-21 [2013] refined the definition to "…the ability to prepare for and adapt to changing conditions and to withstand and recover rapidly from disruptions. Resilience includes the ability to withstand and recover from deliberate attacks, accidents, or naturally occurring threats or incidents."

2.2 Planning Process

While varied forms of community disaster preparedness planning have been taking place for decades, a specific focus on community resilience has developed over about the last 10 years. In 2015, the National Institute of Standards and Technology (NIST) published NIST Special Publication 1190, *Community Resilience Planning Guide for Buildings and Infrastructure Systems* (NIST, 2015). The *Guide* outlines a consistent framework for a six-step resilience planning process (see Figure 2.1) that is designed to be conducted at a community level, involving broad representation from local and regional government, building owners, infrastructure system owner/operators, and community representatives. The *Guide* process can also be adapted to resilience planning for a specific infrastructure system (e.g. water system), with some limitations. One of the main limitations of an individual infrastructure system planning approach is that it requires assumptions to be made that can't be tested with community stakeholders and other infrastructure system providers. For instance, operation of water pump stations requires commercial electrical power or emergency generators with adequate fuel supplies. The timeline for restoration of commercial electrical power or availability of fuel for generators is largely controlled by stakeholders that aren't involved in a water system only planning scenario.

2.3 Seismic Hazard

One of the initial steps in the resilience planning process involves determining the specific hazards to be safeguarded against. Consistent with Oregon Health Authority requirements, the City of Newberg has selected a M9.0 Cascadia Subduction Zone scenario earthquake as the hazard to be explicitly considered for this seismic resilience study.

The geologic and seismologic information available for identifying the potential seismicity throughout the State of Oregon is continually evolving, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes. The available information indicates the potential seismic sources that may affect the state can be grouped into three categories:

- Subduction zone events related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate,
- Subcrustal events related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and
- Local crustal events associated with movement on shallow, local faults.

A major contributor to the seismic hazard in western Oregon is the Cascadia Subduction Zone (CSZ) that lies off the coast of Oregon, Washington, Northern California, and British Columbia. The CSZ is an active plate boundary along which the remnants of the Farallon Plate (the Gorda, Juan de Fuca and Explorer plates) are being subducted beneath the western edge of the North American continent. Figure 2.2 shows that the subduction zone off the coast of Oregon is a mirror image of the subduction zone off the coast of Northern Japan that produced the deadly Magnitude 9.0 Tohoku earthquake in 2011. Seismologists anticipate that the strong shaking from a CSZ earthquake will last from 3 to 5 minutes, much longer than the 30-second strong shaking experienced in a typical California earthquake.

Seismologists' understanding of the damaging earthquakes produced by the CSZ has steadily increased over the past 25 years. Research by the Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon State University, and others has provided evidence of the timeline of historic great CSZ earthquakes. The timeline of these 41 earthquakes over the last 10,000 years is provided in Figure 2.3, showing that past earthquakes have occurred at highly variable intervals, and can range widely in size and in which parts of the Pacific Northwest they affected. The rupture distance for these CSZ earthquakes varies from a short rupture along the Northern California and Southern Oregon Coast, to a rupture along the entire length of the subduction zone from Northern California to British Columbia. There is about a 37 percent chance in the next 50 years of a Magnitude 8+ earthquake originating on the southern portion of the CSZ and up to a 15 percent chance in the next 50 years of a great earthquake affecting the entire Pacific Northwest. The scenario involving rupture of the Northern Oregon portion would significantly impact all Western Oregon, including Newberg.

Figure 2.1 – Six-Step Process to Planning for Community Resilience (NIST, 2015)

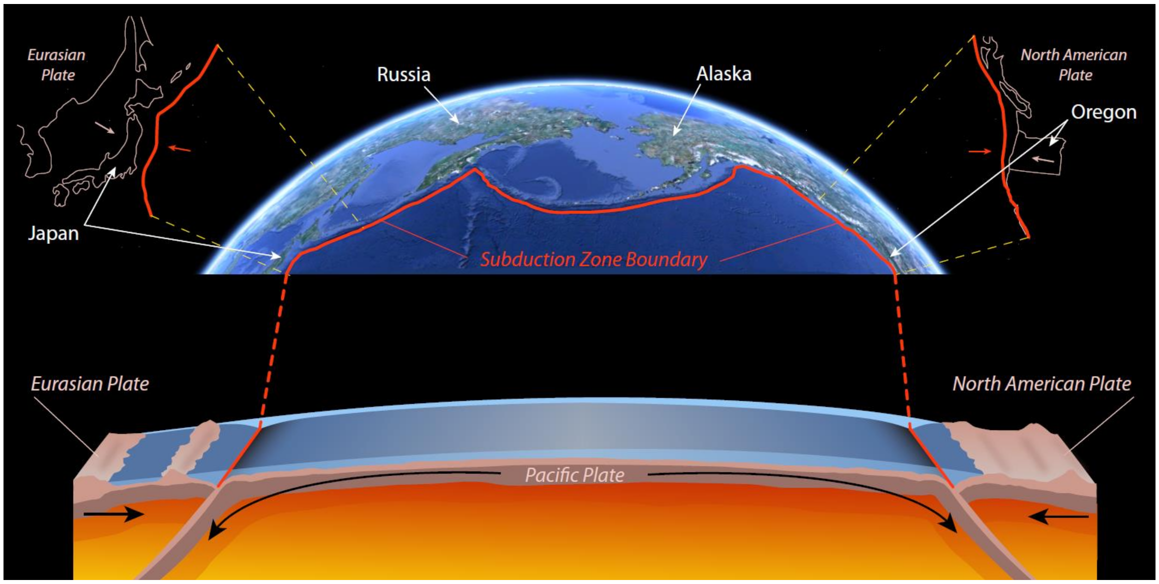

Figure 2.2 – Oregon and Northern Japan Mirror Image Subduction Zones (OSSPAC, 2013)

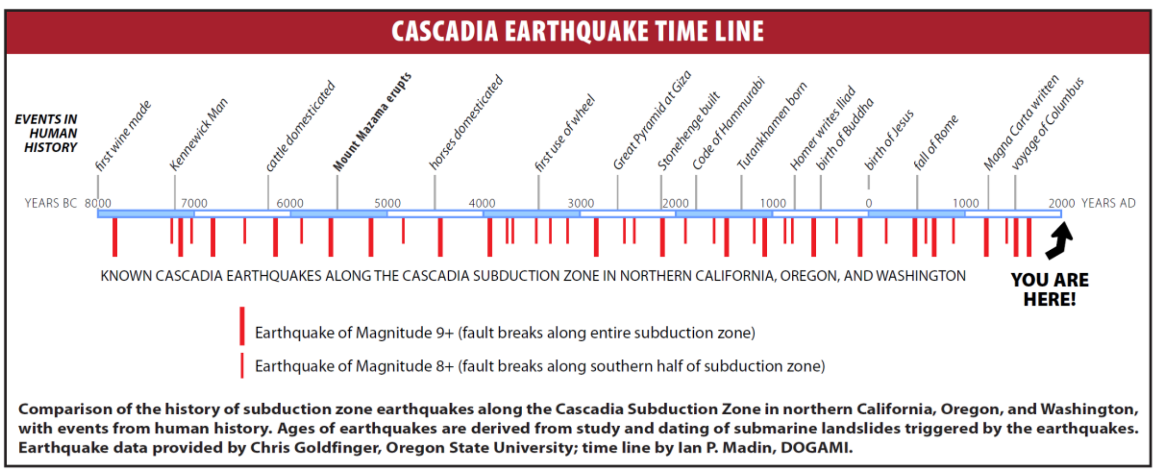

Figure 2.3 – Historic Cascadia Subduction Zone Earthquake Timeline (DOGAMI, 2010)

3.0 Level of Service Goals

Resilience planning involves establishing level of service (LOS) goals to define system performance expectations after being impacted by the hazard under consideration. These LOS goals could be simple, such as maintain service for 100 percent of customers during a routine winter storm that disrupts commercial electrical power for 24 hours, or they may be more complex for more damaging hazards like major earthquakes. This section presents examples of LOS goals included in other plans and then describes the LOS goals suggested for adoption by the City of Newberg for the water system.

3.1 SPUR Resilient City

In one of the first studies of its kind, the San Francisco Planning + Urban Research Association (SPUR) developed a series of policy papers aimed at raising awareness of how San Francisco's buildings and lifeline infrastructure are likely to perform in an expected earthquake and identifying actions that could be implemented before an earthquake to improve the City's resilience. The report outlined the importance of how the restoration timeline for water, wastewater, electrical power, and other lifeline systems impacts the speed with which a community can return to normal after a major disruption (SPUR, 2009). The report established the goals of restoring lifeline services to: 1) 90 percent of customers within 72 hours, 2) 95 percent of customers within one month, and 3) 100 percent of customers within four months after an expected level earthquake. It is assumed that critical facilities (e.g., hospitals, emergency operations centers, etc.) would be included in the 90 percent of customers restored within 72 hours. For buildings, the SPUR report defines the expected level earthquake as one having a 10 percent probability of occurring in a 50-year period and compares it to a magnitude 7.2 earthquake on the peninsula segment of the San Andreas Fault. The SPUR report also indicated that for lifeline systems, that typically have a longer design life than buildings, a larger expected level earthquake should be considered.

3.2 Oregon Resilience Plan

The threat of a Cascadia earthquake is a significant enough physical, economic, and social risk in the Pacific Northwest that in 2012 and 2013, at the request of the State of Oregon Legislative Assembly, the Oregon Seismic Safety Policy Advisory Commission (OSSPAC) and a team of volunteer professionals developed the *Oregon Resilience Plan: Reducing Risk and Improving Recovery for the Next Cascadia Earthquake and Tsunami* (OSSPAC, 2013). The *ORP* outlines steps that can be taken over a 50-year period to bring the state closer to resilient performance through a systematic program of vulnerability assessments, capital investments in buildings and infrastructure systems, new incentives to engage the private sector, and policy changes that reflect current understanding of the Cascadia threat to our community and economy.

OSSPAC assembled eight task groups, comprising over 160 volunteer subject-matter experts from government, universities, the private sector, and the general public. Task Groups included: (1) Cascadia earthquake scenario, (2) business and workforce continuity, (3) coastal communities, (4) critical and essential buildings, (5) transportation, (6) energy, (7) information and communications, and (8) water and wastewater. Task Group activities were overseen by OSSPAC and an Advisory Group. Each Task Group was charged to:

- Determine the likely impacts of a Magnitude 9.0 Cascadia earthquake and tsunami on its assigned sector, and estimate the time required to restore functions in that sector if the earthquake were to strike under present conditions;
- Define acceptable timeframes to restore functions after a future Cascadia earthquake to fulfill expected resilient performance; and
- Recommend changes in practice and policies that, if implemented during the next 50 years, will allow Oregon to reach the desired resilience targets.

The various task groups used estimates of the seismic hazard and expected ground motions developed by the Cascadia Earthquake Scenario Task Group in combination with knowledge of the construction era and condition of existing infrastructure to estimate the expected performance and service restoration times if the scenario event were to occur at the time the *ORP* was being developed.

The *ORP* used the SPUR model as a starting point for developing LOS goals (target timelines for restoration of services) after a Cascadia earthquake. These restoration targets were established assuming system resilience enhancements would be implemented over the following 50 years. These targets were set for three levels of service:

- Minimal level of service restored for the use of emergency response;
- Functional level of service up to 50 percent of capacity that is sufficient to get the economy moving again, and an
- Operational level of service where restoration is up to 90 percent of capacity (which may still rely on temporary fixes).

Table 3.1 summarizes the *ORP's* goals for the restoration of water service for the Willamette Valley (after 50 years of resilience improvements) and compares it to the expected performance if the earthquake were to have occurred at the time the *ORP* was written. The time differences between the *ORP* restoration target (LOS) goal and expected performance illustrates the resilience gaps that require investment in infrastructure improvements, and public policy enhancements over the coming years.

Table 3.1 – ORP Water System Recovery Goals: Valley Zone (adapted from OSSPAC 2013)

	0-24 hours	1-3 days	3-7 days	1-2 weeks	2-4 weeks	1-3 months	3-6 months	6-12 months	1-3 years	3+ years
Potable water available at supply source (WTP, wells, impoundment)	R	Y		G			X			
Main transmission facilities, pipes, pump stations, and reservoirs (backbone) operational	G					X				
Water supply to critical facilities available	Y	G				X				
Water for fire suppression – at key supply points	G		X							
Water for fire suppression – at fire hydrants			R	Y	G			X		
Water available at community distribution centers/points		Y	G	X						
Distribution system operational		R	Y	G				X		

Key to Table

Target Timeframe for Recovery:

Desired time to restore components to 20-30% operational	R
Desired time to restore components to 50-60% operational	Y
Desired time to restore components to 80-90% operational	G
Current state (90% operational)	X

3.3 NIST Community Resilience Planning Guide

The authors of the NIST *Guide* built upon the framework established by SPUR and the *ORP* in developing recommendations for community resilience planning. The categories, for which restoration timeline goals should be set, were further expanded to consider additional system components and to clarify that restoration timelines will likely vary based on the building cluster that is being supported (critical facilities, emergency housing, housing/neighborhoods, etc.). The *Guide* does not make recommendations for recovery timelines but provides a framework that communities can use to collectively establish these recovery timeline goals. The expanded *Guide* performance goal table along with the restoration timeline goals established by the *ORP* have been used in developing level of service goals for this project. Further description of the recommended City of Newberg water system level of service goals developed as part of this project is provided in Section 3.8.

3.4 San Francisco Public Utilities Commission

The San Francisco Public Utilities Commission (SFPUC) outlines seismic design requirements in an agency specific engineering standard, *General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities* (SFPUC, 2014). The purpose of the Standard is "to set forth consistent criteria for the seismic design and retrofit of San Francisco's water and wastewater infrastructures. These systems comprise buildings, aboveground and underground piping, retaining walls, underground structures, tanks and basins, dams and reservoirs, special structures, and equipment under the jurisdiction of the SFPUC."

The SFPUC Standard establishes that the water system basic level of service goal is to deliver winter day demand (WDD) within 24 hours after a major earthquake. For critical and non-redundant structures and components, this major earthquake is defined as having a 5% probability of exceedance in 50 years (975-year return period). The basic level of service goal also considers several supplemental criteria that include (SFPUC, 2014):

- Deliver WDD to at least 70% of SFPUC wholesale customers' turnouts within each of the three customer groups;
- Achieve a 90% confidence level of meeting the above goal, given the occurrence of a major earthquake;
- To achieve the basic level of service, the SFPUC shall rely on the wholesale customer's own water systems and supply or other regional water purveyor's systems. SFPUC will work with customers to assess their ability to contribute to their own system reliability;
- The SFPUC shall consider a facility to have failed if it cannot be brought back to its intended purpose within 24 hours without secondary damage resulting; and
- To achieve the basic level of service, the SFPUC shall assume that power supplies are available, whether from the grid or from standby sources.

The SFPUC shall assume that no significant repairs are performed in the first 24 hours following a major earthquake. Possible operations that might occur during the first 24 hours include valve operations, temporary bypasses, and restoration of minor planned outages, if regional infrastructure remains intact.

3.5 Community Needs Following a Major Earthquake

To support the region's economic and community recovery after a major disaster, infrastructure services are required to be restored as the building clusters that rely on these services come back online (i.e., a building that will take six months to reopen due to repair of structural damage doesn't need water service until the end of that six months). In some cases, like that for smaller businesses, an outage of critical services like water for more than a few weeks may mean a business cannot return to a location. The current expectation of many Oregonians is that water service will be restored within one month after a major earthquake (City Club, 2017). The water system recovery goals suggested in the *ORP* are generally consistent with this public expectation. The *ORP* also sets goals for partial recovery in the initial days and weeks after a major earthquake with the aim of supporting rapid economic and social recovery.

Given that it would be cost prohibitive to eliminate all earthquake damage, a fundamental short-term community need will be to provide water for fire suppression and for use by hospitals, emergency shelters, and other similar facilities. Immediately after the event, it is anticipated that the City of Newberg will focus on repairing any damage to the water system supplying these critical customers and then quickly transition to restoring water service to other customers. This goal for rapid restoration of the water service will help support the Newberg Community's desire that residents will be able to shelter-in-place in their homes immediately after a major earthquake and that they will be able to resume a semi-normal daily routine after two to four weeks by returning to school/work, shopping at their local grocery store, receiving medical care at their local clinic, etc. All these normal activities involve the use of water. At first it is expected that temporary measures will be required to distribute water, but as the weeks progress more permanent fixes will be implemented and the temporary measures will slowly disappear. The City may also be challenged by an influx of people displaced from coastal communities that were severely impacted by the earthquake and associated tsunami. Therefore, the post-disaster emergency water demand could increase to support additional short-term residents.

Table 3.2 provides a breakdown of restoration priorities for City customers that was jointly developed in a collaborative workshop conducted with the HDR team and City of Newberg staff. The table links social/economic needs to restoration timeline goals [short-term (no disruption), short-term (1-3 days), intermediate-term (within 4 weeks), and long-term (months)]. Note that these restoration timeline goals have been established based on our current understanding of the community's social and economic needs, without consideration or knowledge of the current expected seismic performance of these existing community facilities. In order to support community social and economic needs on a timeline that is similar to that proposed for the water system, many of these community facilities may need to be seismically retrofit or replaced with new buildings designed with a higher structural and nonstructural performance objective. If a facility that is critical to supporting community short- and intermediate-term social/economic needs is relocated, site selection criteria for the new location should consider proximity to the water system backbone or the water system backbone should be appropriately modified to include the location of the new facility.

Table 3.2 – City of Newberg Social/Economic Recovery Goals

Response/Recovery Phase	Social/Economic Needs
Short-Term (no disruption)	• Water Supply Points for Fire Suppression ○ North Valley and Corral Creek Reservoirs ○ Newberg High School ○ Chehalem Valley Middle School ○ Edwards and Joan Austin Elementary Schools ○ George Fox University ○ Portland Community College ○ Rogers Landing (drafting from Willamette River) • Providence Newberg Medical Center
Short-Term (1-3 days)	• Newberg Public Safety Building (Police Station, City EOC) • Fire stations ○ TVF&R Station #20 and #21 • Community Water Distribution Points ○ Calvary Chapel Newberg ○ Chehalem Glenn Golf Course ○ Church of Jesus Christ of Latter-Day Saints ○ Family Life Church ○ First Presbyterian Church ○ Grace Baptist Church ○ George Fox University ○ Newberg Christian Church ○ Newberg Friends Church ○ Northside Community Church ○ River Street Church of God ○ Seventh Day Adventists ○ Zion Lutheran Church • Urgent Care Centers ○ Newberg Urgent Care ○ Providence Express Care • Dialysis Center (Fresenius Kidney Care) • Emergency shelters ○ Newberg High School ○ Chehalem Valley and Mountain View Middle Schools ○ Edwards Elementary School ○ George Fox University (locations TBD) • Senior Care Facilities ○ Arbor Oaks Terrace ○ Astor House at Springbrook ○ Avamere Newberg ○ Brookdale Newberg ○ Friendsview Retirement Community ○ Friendsview Springbrook Meadows ○ Marquis Newberg ○ Willow Place

Table 3.2 – City of Newberg Social/Economic Recovery Goals (cont.)

Response/Recovery Phase	Social/Economic Needs
Short-Term *(cont.)* (1-3 days)	• Sportsman Airpark (supplied by Sam Whitney Water District) • Wastewater Treatment Plant (pump seal water) • Public Works Department buildings • Newberg School District Office
Intermediate-Term (within 4 weeks)	• Water District Customers ○ Chehalem Terrace ○ Chehalem Valley ○ NW Newberg ○ Sunny Acres ○ West Sheridan • City of Newberg facilities • Remaining Newberg School District facilities • Medical office buildings • 90% of customer connections • 90% of fire hydrants
Long-Term (months)	• Remaining 10% of customer connections • Remaining 10% of fire hydrants

3.6 Water Supply Points for Fire Suppression

Table 3.2 and Figure 3.1 identify the potential location of nine key supply points distributed throughout the city where tanker trucks could obtain water for fire suppression if the hydrant system is down following a major earthquake. At the two reservoir sites, it may be necessary to install seismic shutoff valves to preserve water storage, install segments of hardened pipe, and upgrade roadway access to the reservoirs. At the fire water distribution points within the city, it is anticipated that hydrants will be installed that are connected to the hardened backbone system and are designed to accommodate any expected permanent ground deformation. The Rogers Landing Boat Launch is proposed as an alternative site where fire trucks could draft water from the Willamette River.

3.7 Community Water Distribution Points

Table 3.2 and Figure 3.2 identify the potential location of 12 community water distribution points throughout the city where city residents could obtain potable water following a major earthquake. The City of Newberg Public Works Department is working with faith-based organizations to provide the manpower necessary to operate these water distribution sites. At the community water distribution points, it is recommended that hydrants be installed that are connected to the hardened backbone system and are designed to accommodate any expected permanent ground deformation.

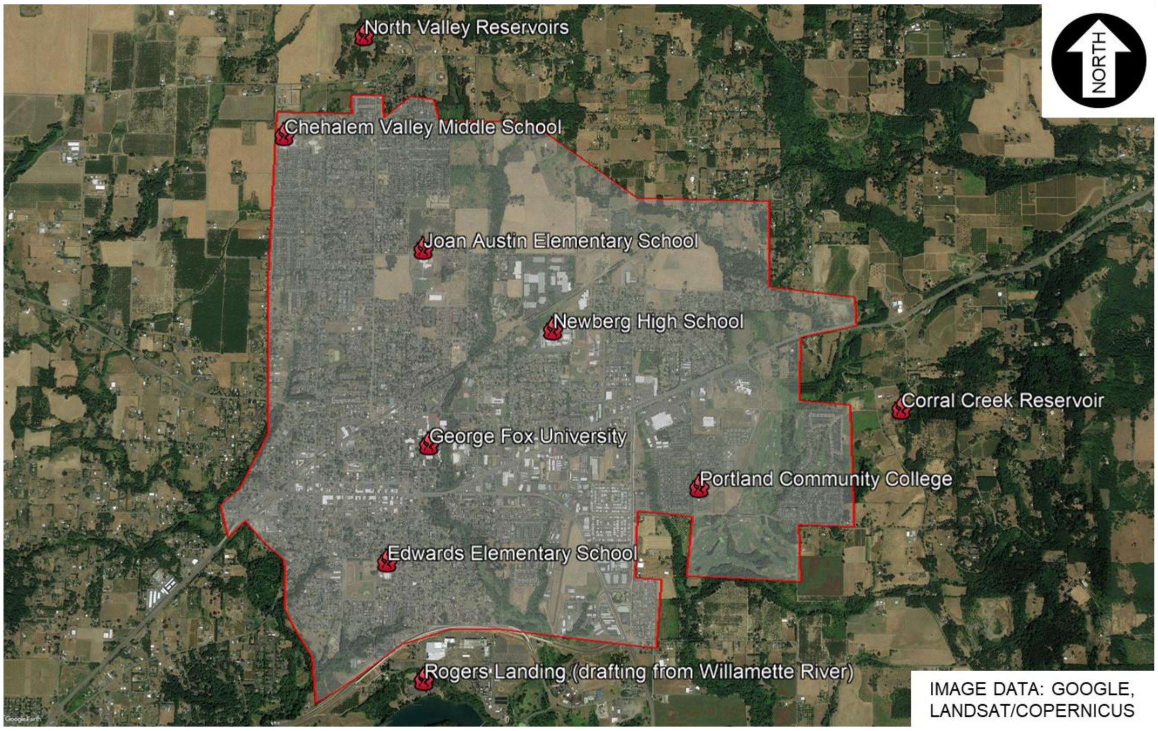

Figure 3.1 – Potential Water Supply Points for Fire Suppression

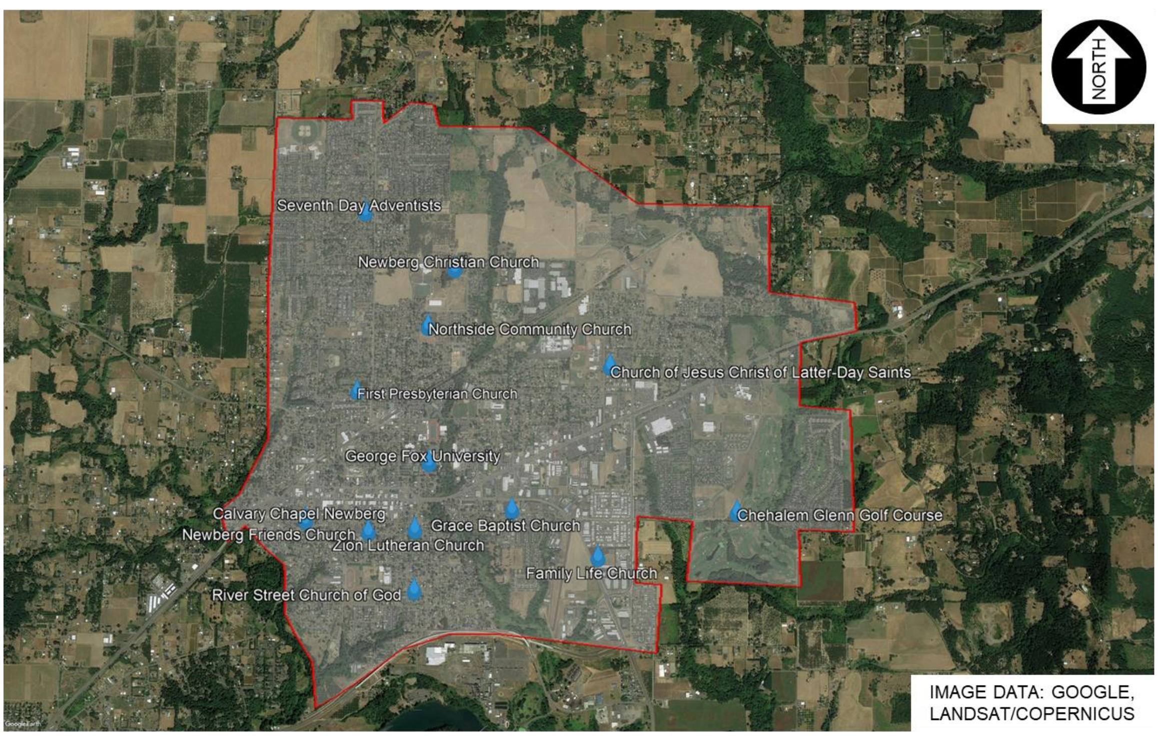

Figure 3.2 – Potential Community Water Distribution Points

3.8 City of Newberg Water System Level of Service Goals

The *ORP* was developed assuming a three-tiered LOS goal approach to implement a phased restoration of services and help define the speed of recovery for a community's infrastructure systems. The *ORP* recommended a timeline for these three-tiered LOS goals but provided the flexibility for an individual utility to define how the levels of functional restoration are to be achieved for their specific system. The LOS (i.e., restoration timeline) goals proposed for adoption by the City of Newberg align with those presented in the *ORP* and are augmented by additional considerations suggested by the NIST *Guide*. Table 3.3 summarizes these goals for the City of Newberg water system broken down in terms of specific goals for source, transmission, control systems, and distribution. All goals are based on providing water meeting minimum regulatory requirements, although a boil water notice may be in effect due to damage throughout the distribution system. Table 3.3 provides additional information about the recommended definition of 30%, 60%, and 90% operational for City of Newberg water system infrastructure. For example, the 90% operational goal for hospital facilities has been defined to mean that the City of Newberg water system is capable of delivering 90% of their average winter day demand of water meeting minimum regulatory requirements to hospital facilities within the City of Newberg service area.

Table 3.3 – City of Newberg Water System Recovery Goals (adapted from OSSPAC 2013 and NIST 2015)

Water Systems	Target Timeframe for Recovery							
	Phase 1: Short-Term			Phase 2: Intermediate			Phase 3: Long-Term	
	Days			Weeks			Months	
	0-1	1-3	3-7	1-2	2-4	4-12	3-6	6-12
Source								
Raw or source water and terminal reservoirs	30% AWDD[a]	60% AWDD		90% AWDD				
Raw water conveyance (pump stations and piping to WTP)	30% AWDD	60% AWDD		90% AWDD				
Water Production	30% AWDD	60% AWDD		90% AWDD				
Well and/or Treatment operations functional	30% AWDD	60% AWDD		90% AWDD				
Transmission								
Backbone transmission facilities (pipelines, pump station, and tanks)	90% AWDD							
Water for fire suppression at key supply points (to promote redundancy)	90% of required fire flow and duration available							
Control Systems								
SCADA and other control systems	90% of components required for normal operation are functional							
Distribution								
Critical Facilities								
Hospitals	90% of AWDD							
EOC, Police Stations, Fire Stations, Public Works Buildings	60% of AWDD	90% AWDD						
Emergency Housing								
Emergency Shelters	60% of emergency water for drinking/sanitation	90% of emergency water for drinking/sanitation						
Housing/Neighborhoods								
Potable water available at community distribution centers		60% of emergency water for drinking/sanitation	90% of emergency water for drinking/sanitation					
Water for fire suppression at fire hydrants			30% of hydrants restored	60% of hydrants restored	90% of hydrants restored			
Community Recovery Infrastructure								
All other clusters			30% of customer connections restored	60% of customer connections restored	90% of customer connections restored			

[a] AWDD = Average Winter Day Demand

Key to Table

Desired time to restore components to 30% operational	R
Desired time to restore components to 60% operational	Y
Desired time to restore components to 90% operational	G

4.0 City of Newberg Backbone System Supporting Short-Term Community Needs

Satisfying short-term LOS restoration timeline goals requires critical components of the water production, treatment, transmission, and distribution system to remain operational or experience only minor damage after a major earthquake. These critical system components usually include: small diameter distribution pipelines and associated reservoirs/pump stations that connect to critical and essential facilities (hospitals, emergency shelters, etc.), large diameter transmission pipelines and associated pump stations, treatment plant structures, and certain support facilities (laboratories, maintenance shops, etc.). If an assessment of these critical system components reveals any gaps between the expected performance and that required to achieve the LOS goals, then these deficient components should be seismically retrofit or replaced, as appropriate.

The HDR team has collaborated with the City of Newberg to identify the proposed backbone for the City water system shown in Figure 4.1. The backbone system provides water distribution system connections between the well field, raw water transmission pipelines, water treatment plant, finished water reservoirs, and distribution system pipelines that serve facilities that are required to meet short-term community needs (see Table 3.2). The backbone systems proposed for the City of Newberg water system is consistent with that envisioned during the development of the *ORP*. The backbone includes elements of the water system that are required to meet short-term LOS restoration timeframe goals in the initial days after a major earthquake. Since it would be challenging to implement any significant repairs to the backbone system in the initial days after an earthquake, the elements of the backbone system should be designed or retrofit such that they experience only minor or no geotechnical, structural, and nonstructural related damage during a major earthquake.

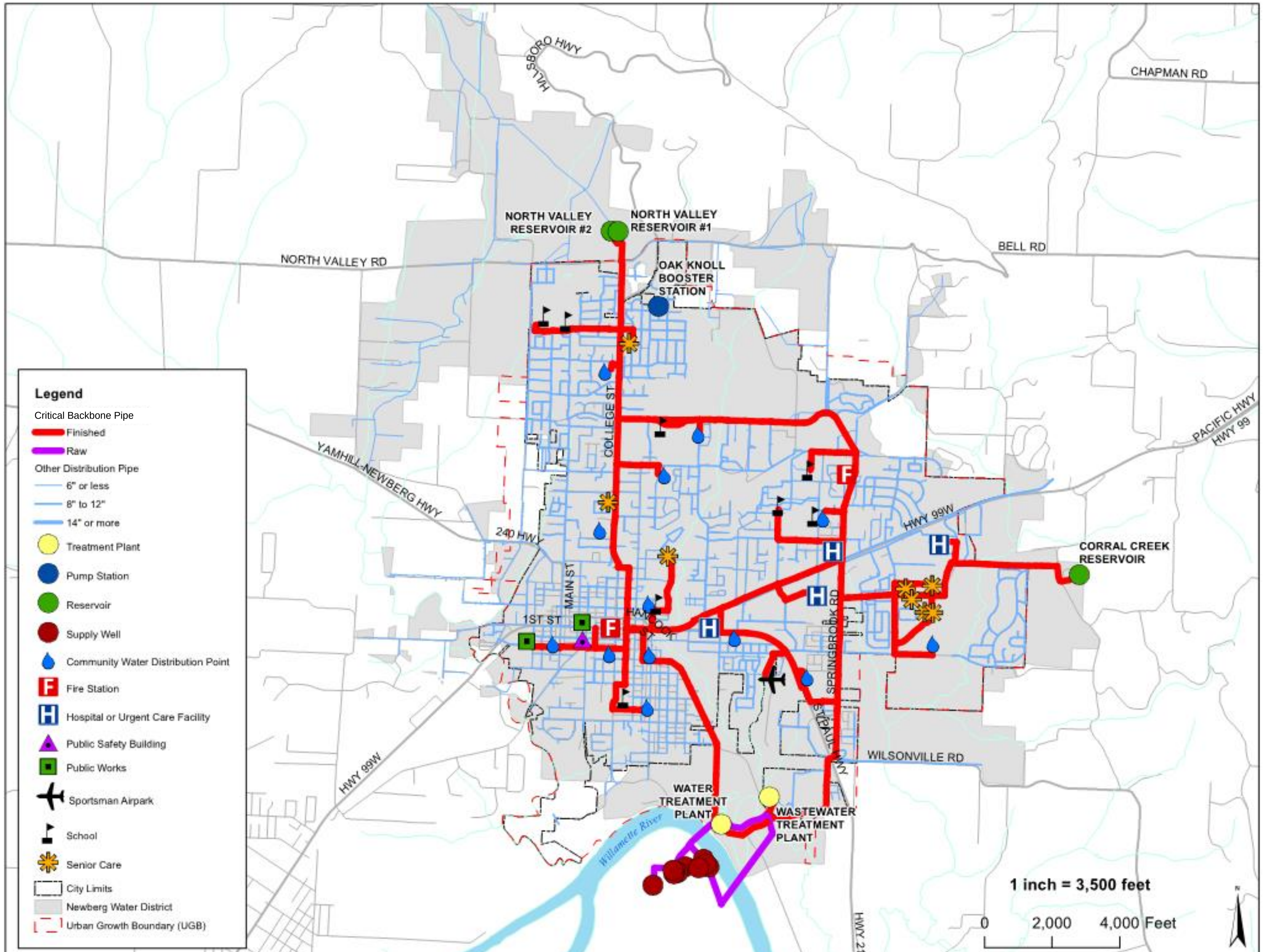

Figure 4.1 – City of Newberg Water System Backbone

5.0 Translation of Level of Service Goals into System Performance Requirements

Several factors need to be taken into consideration when translating the City of Newberg LOS goals into performance requirements for the seismic design or retrofit of water system components. Section 5.1 describes several of the factors that have been considered in developing the recommended general performance requirements detailed in Section 5.2.

5.1 Considerations

The following subsections describe factors considered in developing performance requirements for the various components of the City of Newberg water system. For future water system projects, these factors should also be evaluated on a project-specific basis to determine if there are any unique features of the project that require modification of the general seismic resilience-based performance requirements.

5.1.1 Geotechnical Hazards

Observations from past earthquakes have indicated that geotechnical hazards are a major contributing factor to the expected post-earthquake performance of water systems. Infrastructure that is exposed to liquefaction, lateral spreading, or landslide geotechnical hazards requires special design considerations that include either mitigation measures to address the geotechnical hazard or predetermined work-arounds to bypass components that may fail during an earthquake. Water treatment plants can be particularly vulnerable to damage from earthquake-induced liquefaction and lateral spreading because these facilities are often constructed in low-lying areas near water sources. These areas correspond with those at high risk for liquefaction and lateral spreading. Transmission and distribution piping that crosses creeks our other low-lying areas are also particularly vulnerable to damage from earthquake-induced liquefaction and lateral spreading.

5.1.2 Effects of Aftershocks

Major earthquakes are often accompanied by numerous aftershocks. In the 2011 Tohoku Japan earthquake two major aftershocks caused additional damage to infrastructure systems, resulting in relapses in the number of customer outages (Nojima, 2012). It may be necessary to reevaluate system components or perform additional repairs after major aftershocks.

5.1.3 Repair Difficulty

Certain water system components (like large diameter transmission mains) may be very difficult to repair after an earthquake. If a component is anticipated to be difficult to repair and it is also important to system performance, then it should be designed to minimize any potential earthquake damage that would impact the functionality of the component. Other assets of this type could include pipes under railroad tracks or highways.

5.1.4 Availability of Public Works Department Staff

The first priority for many City of Newberg Public Works Department staff in the initial hours and days following a major earthquake will be to ensure the health and safety of their families. Once those critical needs are addressed, City of Newberg Public Works Department staff will, ideally, be available to report to work. However, even after they return to work, it is possible that the City Emergency Manager may assign Public Works Department staff to work on non-water system related tasks that are deemed more critical to the City's disaster response activities. This scenario suggests that Public Works Department staff may have limited ability to perform repairs or implement predetermined work-arounds in the initial hours and days after an earthquake. Critical components of the water system that are required to be operational within the first 3-7 days after an earthquake should be designed or seismically retrofitted to remain operational during and immediately after a major earthquake.

5.1.5 Availability of Design Professionals and Contractors

The restoration timeline goals and required repairs must be in line with the anticipated availability of qualified design professionals and contractors to design and implement the repairs. It is anticipated that the design and construction of major repairs to a pump station or treatment plant structure would take between 6-12 months. It is anticipated that the design and construction that replaces a pump station or treatment plant structure would take a minimum of 18 months. These timeframes may increase if the City decides to rebuild the pump stations to a higher standard of performance, i.e., a resilient design, which may require more planning and design time.

5.1.6 Availability of Repair Materials or Replacement Equipment

The City of Newberg maintains limited supplies of emergency repair materials, but these supplies are not anticipated to be adequate for the number of repairs that may be necessary after a major earthquake. For disasters that impact a relatively small geographic region, it is possible that other nearby utilities could lend repair supplies. However, a CSZ earthquake will impact the entire Pacific Northwest (from Northern California to British Columbia) and relying on neighboring utilities as a potential source for repair materials is likely impractical.

Additionally, some equipment used in pump stations and treatment plants is not available from manufacturer's stock and has a long lead time for production. Special consideration must be given to this difficult-to-source equipment to ensure that it is either not damaged during an earthquake, a predetermined work-around has been established, or the equipment manufacturing lead time aligns with restoration timeline goals.

5.1.7 Infrastructure Dependencies

The restoration of water system infrastructure is highly dependent on other infrastructure systems. Examples of these dependencies include:

- Co-location with and damage to other lifeline systems (roads, bridges, wastewater pipes, etc.);
- Liquid fuel availability for trucks, generators, and equipment;
- Commercial electrical power;
- Transportation system for delivery of repair materials and mutual aid assistance crews; and
- Cellular communications system for coordination of City of Newberg staff and contractors.

The level of service goals and performance requirements suggested in this report assume that all lifeline service providers will be making significant investments in the earthquake resilience of their systems in the next 45 years. If one or more lifeline sectors do not make these system improvements, then the speed of community recovery could be greatly impacted because of the dependencies between all infrastructure systems. Figure 5.1 shows an example of the complicated dependency relationships among lifelines in the San Francisco Bay Area (City and County of San Francisco Lifelines Council, 2014). Heavy and light lines widths depict the relative level of dependencies anticipated to occur between the various lifelines systems following a scenario M7.9 earthquake on the San Andreas fault.

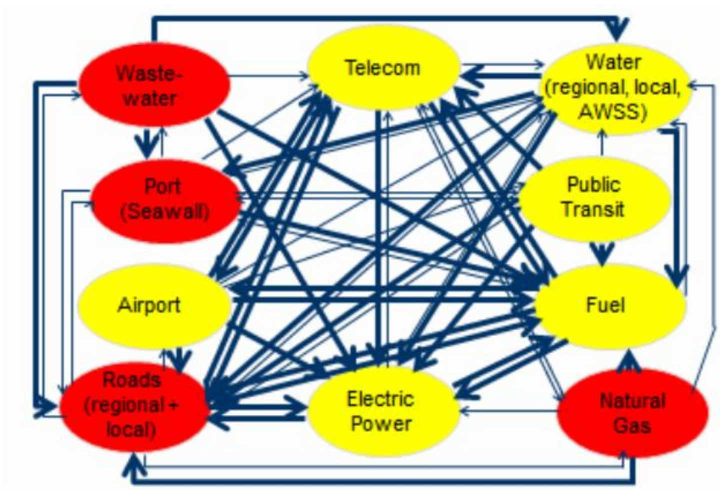

Figure 5.1 – Lifeline Interdependencies in the San Francisco Bay Area (City and County of San Francisco Lifelines Council, 2014)

5.2 Water System Structures

Water system structures (reservoirs, pump stations, etc.) required to maintain water pressure for fire suppression are designated as Risk Category IV structures and water system structures not required to maintain water pressure for fire suppression are designated as Risk Category III structures according to the requirements of the latest edition of the *Oregon Structural Specialty Code* (OSSC, 2014). For new structures, the construction cost increase associated with elevating the design standard from Risk Category III to Risk Category IV is typically relatively minor. Therefore, it is recommended that all new water system structures should be designed per the more stringent *Oregon Structural Specialty Code* seismic design requirements for Risk Category IV structures. Also, since geotechnical hazards (e.g., liquefaction and lateral spreading, etc.) can significantly impact the performance of water system structures following a major earthquake, it is recommended that site-specific geotechnical investigations and analysis be conducted to characterize these potential hazards. Water system structure designs should include appropriate measures to mitigate these potential site-specific geotechnical hazards. Equipment associated with water system structures should be adequately braced and seismically certified, per the requirements of the latest edition of ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2017a), so that it could remain operational after a design level earthquake, as long as dependent systems are also functional [e.g., electrical power (emergency generator or commercial), etc.]. Piping entering or exiting water system structures should be designed to accommodate the anticipated earthquake-induced relative movement between the structure and surrounding soil.

In order to meet the target LOS goals, water system structures need to meet or exceed defined levels of structural and nonstructural seismic performance. ASCE 41-17, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2017b), presents several structural and nonstructural seismic performance objectives and describes the expected level of earthquake damage associated with each performance objective. Also included are expectations about the operability and reparability of earthquake damage for these various performance objectives. The ASCE 41-17 descriptions of these performance objectives are provided below and summarized in Figure 5.2. Table 5.1 provides a comparison between these performance objectives and the intended performance associated with *Oregon Structural Specialty Code* Risk Categories.

Table 5.1 – Comparison of Seismic Performance Objectives with OSSC Risk Categories

Risk Category	**Performance Objective[a]**	
	Structural	**Nonstructural**
IV	Immediate Occupancy	Operational
III	Damage Control	Position Retention
I & II	Life Safety	Position Retention

[a] For the BSE-1N seismic hazard level as defined by ASCE 41-17

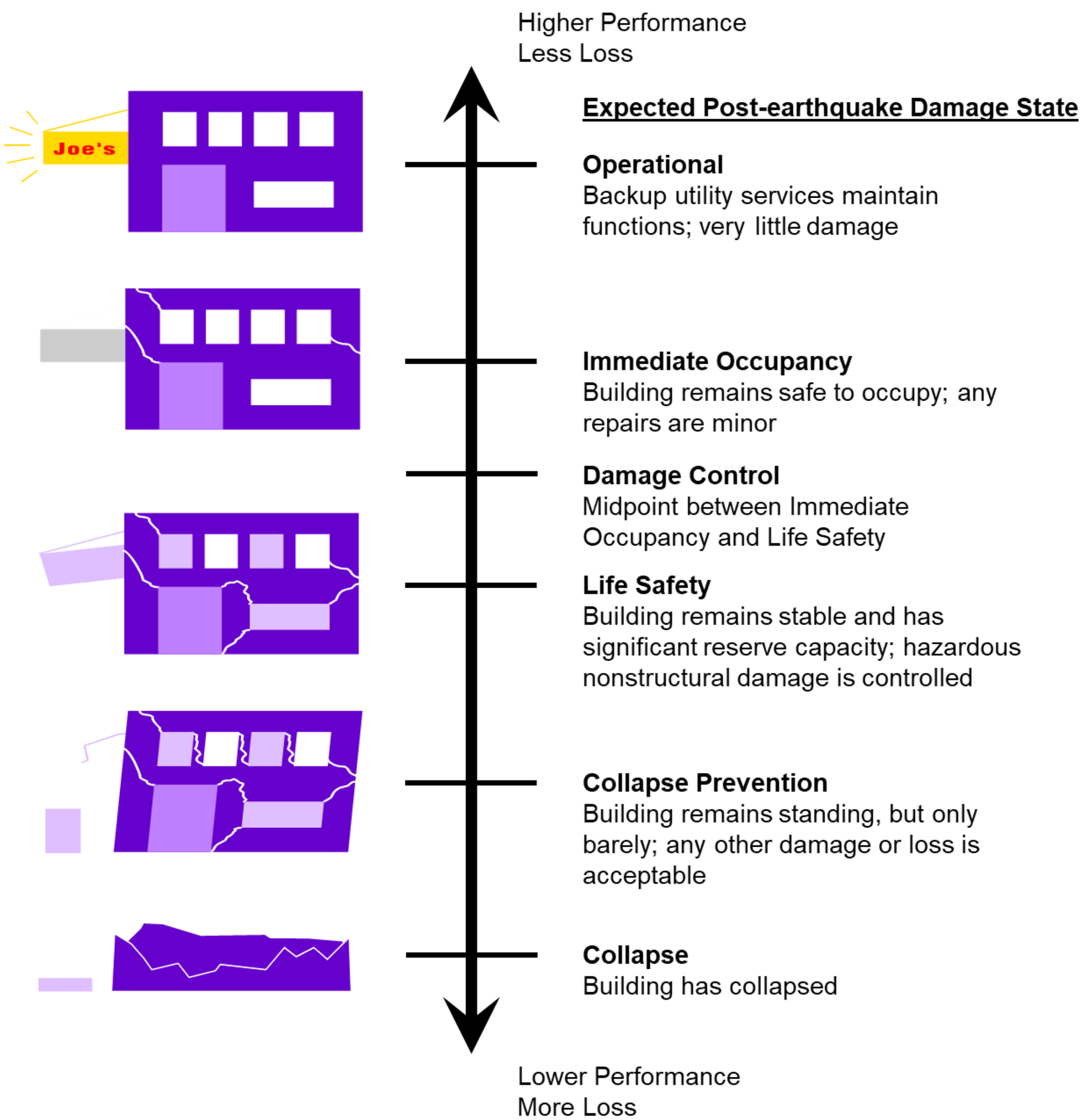

Figure 5.2 – Building Performance Objectives (adapted from ASCE, 2017b)

Structural Performance Objectives

Immediate Occupancy: "Immediate Occupancy" refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury from structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.

Damage Control: "Damage Control" refers to a midway point between Life Safety (see next description) and Immediate Occupancy (see previous description). This performance objective is intended to provide a structure with a greater reliability of resisting collapse and being less damaged than a typical structure, but not to the extent required of a structure designed to meet the Immediate Occupancy Performance Level. Although this level is a numerically intermediate level between Life Safety and Immediate Occupancy, the two performance objectives are essentially different from each other. The primary consideration for Immediate Occupancy is that the damage is limited in such a manner as to permit reoccupation of the building, with limited repair work occurring while the building is occupied. The primary consideration for Life Safety is that a margin of safety against collapse be maintained and that consideration for occupants to return to the building is a secondary impact to the Life Safety objective being achieved. The Damage Control Performance Level provides for a greater margin of safety against collapse than the Life Safety Performance Level would. The level might control damage in such a manner as to permit return to function more quickly than the Life Safety Performance Level, but not as quickly as the Immediate Occupancy Performance Level does.

Life Safety: "Life Safety" refers to the post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside the building. Injuries might occur during the earthquake; however, the overall risk of life-threatening injury from structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before re-occupancy.

Nonstructural Performance Objectives

Operational: "Operational" refers to the performance level where most nonstructural systems required for normal use of the building are functional, although minor cleanup and repair of some items might be required. Achieving the Operational nonstructural performance level requires considerations of many elements beyond those that are normally within the sole province of the structural engineer's responsibilities. For Operational nonstructural performance, in addition to ensuring that nonstructural components are properly mounted and braced within the structure, it is often necessary to provide emergency standby equipment to provide utility services from external sources that might be disrupted. It might also be necessary to perform qualification testing to ensure that all necessary equipment will function during or after strong shaking.

Position Retention: "Position Retention" refers to the nonstructural condition of a building after an event where, presuming that the building is structurally safe, occupants can occupy the building safely, with some limitations: normal use might be impaired, some cleanup might be needed, and some inspection might be warranted. In general, building equipment is secured in place and might be able to function if the necessary utility service is available. However, some components might experience misalignments or internal damage and be inoperable. Power, water, natural gas, communications lines, and other utilities required for normal building use might not be available. Cladding, glazing, ceilings, and partitions might be damaged but would not present safety hazards or un-occupiable conditions. For this performance level, the risk of life-threatening injury caused by nonstructural damage is very low.

Detailed geotechnical and structural seismic evaluations should be conducted for existing water system structures to determine if their anticipated seismic performance will enable LOS goals to be achieved. To satisfy the target water system restoration timeline, structures that must be operational soon after a major earthquake should be evaluated and if required, seismically retrofit to a more stringent structural and nonstructural performance level than those that are not required until later in the recovery phase. Table 5.2 provides the seismic retrofit criteria proposed for adoption by the City of Newberg for water system infrastructure in terms of the structural and nonstructural performance objectives presented in ASCE 41. These performance objectives are for the Basic Safety Earthquake-1 for use with the Basic Performance Objective Equivalent to New Building Standards (BSE-1N). This BSE-1N seismic hazard level is consistent with that used to design new structures per the *Oregon Structural Specialty Code*. Note that the proposed LOS goals require that the water system has essentially been restored to a 90% operational level within 2-4 weeks after a M9.0 CSZ earthquake. This would suggest that the majority of system components are capable of achieving Immediate Occupancy structural performance and Operational nonstructural performance. Table 5.2 also includes alternative (less stringent) retrofit performance objectives for system components that might not be required to be returned to service until 1-6 months or 6-12 months after the earthquake. For example, the City of Newberg may decide that one of the reservoirs is not required to achieve short- and intermediate-term LOS goals and may elect to relax the restoration timeline goals for that particular water system structure.

Table 5.2 – Water System Seismic Retrofit Performance Objectives

Restoration Timeline	Retrofit Performance Objective[a]	
	Structural	Nonstructural
0-1 months	Immediate Occupancy	Operational
1-6 months	Immediate Occupancy	Position Retention[b]
6-12 months	Damage Control[c]	Position Retention[b]

[a] For the BSE-1N seismic hazard level as defined by ASCE 41-17.
[b] Assumes lead time for delivery and installation of damaged equipment falls within restoration timeline goals, otherwise equipment should be seismically certified per the requirements of the latest edition of ASCE 7.
[c] Assumes that the structural damage can be repaired within restoration timeline goals. For earthquake damage that may be especially difficult to repair within the target timeline, structure should be retrofit to satisfy the Immediate Occupancy performance objective.

6.0 Limitations

The opinions and recommendations presented in this report were developed with the care commonly used as the state of practice of the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report. This report has been prepared for the City of Newberg to be used solely in its evaluation of the seismic safety of the water system referenced. This report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or uses.

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Appendix B:
Geotechnical Engineering Report

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GEOTECHNICAL ENGINEERING REPORT

City of Newberg Water System Seismic Resilience Study

NEWBERG, OREGON

July 2020

Shannon & Wilson No: 101895

SHANNON & WILSON

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Submitted To: HDR, Engineering Inc.
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Attn: Joe Miller

Subject: GEOTECHNICAL ENGINEERING REPORT, CITY OF NEWBERG WATER SYSTEM SEISMIC RESILIENCE STUDY, NEWBERG, OREGON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to HDR Engineering, Inc. (HDR). Our scope of services was specified in the Geotechnical Subconsultant Agreement dated April 29, 2019. This report presents results of our geotechnical seismic hazard assessment for the City of Newberg's (the City) water system and service area for use in assessing the vulnerability of the City's critical infrastructure. The assessment was performed utilizing Geographic Information System (GIS) data and is based on the magnitude 9.0 Cascadia Subduction Zone (CSZ) scenario defined in the Oregon Resilience Plan (OSSPAC, 2013). Along with evaluating the seismic hazard within the City, we were also tasked with evaluating the seismic hazard and slope stability at the Water Treatment Plant (WTP).

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

David Jacobson, GIT
Staff Geologist

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1 SCOPE OF SERVICES

The purposes of the HDR team's seismic hazard assessment are to define water system level-of-service goals, assess the existing system with respect to the levels of service, and develop recommended mitigation measures to address deficiencies. Shannon & Wilson's task is to prepare and provide GIS maps of:

- probability of liquefaction
- probability of earthquake-induced landslides
- liquefaction-induced permanent ground deformation
- earthquake-induced-landslide permanent ground deformations

To achieve these purposes, our scope of services included the following:

- Review existing geologic and geotechnical information;
- Develop seismic ground motion, seismic hazard, and permanent ground deformation hazard maps;
- Perform one boring at the WTP;
- Evaluate liquefaction potential and liquefaction-induced settlement at the WTP;
- Evaluate potential for slope failure for static, seismic, and post-seismic (liquefied) conditions using a limit equilibrium analyses and Slope-W software at the WTP;
- Evaluate seismically induced ground movement using Newmark-type analyses at the WTP;
- Evaluate potential for lateral spread using empirical methods at the WTP, and;
- Summarize the geotechnical evaluations at the WTP and provide maps for the seismic hazard assessment in a Technical Memorandum.

To support the team's structural vulnerability assessment, we also included maps of peak ground acceleration, 0.3- and 1-second spectral accelerations, peak ground velocity, and liquefaction-induced settlement in addition to the maps listed above.

2 SEISMIC HAZARD MAPPING

2.1 Approach

The GIS map layers developed for this project are primarily based on published geologic maps; variations from actual site conditions should be expected. Also, the analyses, methods and approaches applied herein were developed and used by the Oregon Department of Geology and Mineral Industries (DOGAMI) and the Federal Emergency Management Agency (FEMA) for planning purposes only. They are not the same as those used for site-specific, code-based geotechnical design.

2.2 Existing Information Review

2.2.1 Regional Seismological Setting

Earthquakes in the Pacific Northwest occur largely as a result of the subduction of the Juan de Fuca plate beneath the North American plate along the Cascadia Subduction Zone (CSZ). The CSZ is located approximately parallel to the coastline from northern California to southern British Columbia. The compressional forces that exist between these two colliding plates cause the oceanic Juan de Fuca plate to descend, or subduct, beneath the continental plate at a rate of about 1.5 inches per year. This process leads to volcanism in the North American plate and stresses and faulting in both plates throughout much of the western regions of southern British Columbia, Washington, Oregon, and northern California. Stress between the colliding plates is periodically relieved through great earthquakes at the CSZ plate interface.

Within the regional tectonic framework and historical seismicity, three broad earthquake sources are identified:

- **Subduction Zone Interface Earthquakes** originate along the CSZ, which is located 25 miles beneath the coastline. Paleoseismic evidence and historic tsunami records from Japan indicate that the most recent subduction zone interface event was in 1700 AD and was an approximately magnitude 9 earthquake that likely ruptured the full length of the CSZ.
- **Deep-Focus, Intraplate Earthquakes** originate from within the subducting Juan de Fuca oceanic plate as a result of the downward bending and tension in the subducted plate. These earthquakes typically occur 28 to 38 miles beneath the surface. Such events on the CSZ are estimated to be as large as magnitude 7.5. Historic earthquakes include the 1949 magnitude 7.1 Olympia earthquake, the 1965 magnitude 6.5 earthquake between Tacoma and Seattle, and the magnitude 6.8 2001 Nisqually earthquake. The highest rate of CSZ intraslab activity is beneath the Puget Sound area, with much lower rates observed beneath western Oregon.
- **Shallow-Focus Crustal Earthquakes** are typically located within the upper 12 miles of the earth's surface. The relative plate movements along the CSZ cause not only east-west compressive strain but dextral shear, clockwise rotation, and north-south compression of the leading edge of the North American Plate (Wells and others, 1998), which is the cause of much of the shallow crustal seismicity of engineering significance in the region. The largest known crustal earthquake in the Pacific Northwest is the 1872

North Cascades earthquake with an estimated magnitude of about 7. Other examples include the 1993 magnitude 5.6 Scotts Mill earthquake and magnitudes 5.9 and 6.0 Klamath Falls earthquakes.

2.2.2 Oregon Resilience Plan

The Oregon Resilience Plan is a result of Oregon House Resolution 3, adopted in April 2011. The House Resolution directed the Oregon Seismic Safety Policy Advisory Commission "to lead and coordinate preparation of an Oregon Resilience Plan that reviews policy options, summarizes relevant reports and studies by state agencies, and makes recommendations on policy direction to protect lives and keep commerce flowing during and after a Cascadia earthquake and tsunami" (OSSPAC, 2013). A task group then developed a Cascadia Earthquake Scenario for use by other work groups as a basis for assessing the effects of the scenario on various sectors of society or parts of the built environment.

This assessment is for a magnitude 9.0 CSZ earthquake, as defined in the Oregon Resilience Plan. Other magnitudes of CSZ events and earthquakes from other sources are not considered.

2.2.3 Geology

The City of Newberg is located in the Willamette Valley physiographic province (Orr and others, 1992). The local geology has been mapped by numerous authors, including Schlicker and Deacon (1967), Frank and Collins (1978), Burns and others (1997), O'Connor and others (2001), and Wells and others (2018). A simplified geologic map of the City is presented in Figure 1 and is based on DOGAMI publications OGDC-6 (Smith and Row, 2015) and SLIDO 3.4 (Burns and Watzig, 2017).

Published mapping suggests that the city is underlain at depth by oceanic sandstone of the Scappoose Formation and basalt of the Columbia River Basalt Group (CRBG), which flowed in the area between about 17 million and 6 million years ago. These units are exposed at the ground surface along the northeast side of the city with smaller outcrops on the east and west sides of the city (see Figure 1).

Based on maps and cross sections prepared by Frank and Collins (1978), the CRBG in the project area is overlain by Pliocene (5.3 to 2.6-million-year-old) Troutdale Formation, which locally consists of silt and clay with occasional beds of sand and gravel. These sediments have historically been referred to by several names, including Troutdale Formation (Schlicker and Deacon, 1967; Frank and Collins, 1978), Sandy River Mudstone equivalent (Madin, 1990), and Hillsboro Formation (Wilson, 1998). These sediments, referred to in this report as Pliocene Alluvium, were deposited in local sub-basins that had been created by extensive faulting and folding of the CRBG and underlying basement rocks (Schlicker and Deacon, 1967). In the vicinity of the City, small outcrops are mapped to the northeast and north (see Figure 1).

Throughout most of the City, the Pliocene Alluvium is concealed at the surface by Pleistocene flood sediments (see Figure 1). The Pleistocene flood sediments were deposited during repeated glacial outburst floods (O'Connor and others, 2001). During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley, and the lake refilled, leading to 40 or more repetitive outburst floods, at intervals of decades (Allen and others, 2009). The floods are collectively known as the Missoula Floods, and during each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge.

When the floodwater emerged from the western end of the gorge, it deposited a tremendous load of boulders, cobbles, and gravel nearest the mouth of the gorge and along the main channel of the Columbia River. Floodwaters stretched along most of the Willamette Valley, creating a temporary lake known as Lake Allison (Orr and others, 1992). Once spread out, the lower-energy waters deposited variable thicknesses of micaceous sand and silt throughout the Willamette Valley, as far south as Eugene (Allen and others, 2009). Within the vicinity of the City, several authors, including Schlicker and Deacon (1967) and Frank and Collins (1978), refer to the fine-grained sediments as Willamette Silt. In this report, we have adopted the name Fine-Grained Missoula Flood Deposits, after more recent mapping by O'Conner and others (2001). In Figure 1, the Fine-Grained Missoula Flood Deposits are mapped as Missoula Flood Deposits.

Additional, more recent geologic units, which appear throughout the project site, and are included on Figure 1, are Landslide Deposits, Floodplain Deposits, and Alluvium of Smaller Streams. The Landslide Deposits were added to the site geologic map based on mapping from SLIDO 3.4 (Burns and Watzig, 2017). Landslide deposits typically consist of a mix of unconsolidated rock, soil, sediment, and colluvium. Only a single landslide deposit was added to the geologic map of the project site in the northeast corner of Figure 1. Within the southern portion of the project site, Holocene and upper Pleistocene Floodplain Deposits are mapped around the Willamette River. These units, which were mapped by O'Connor and Others, 2001, consist of unconsolidated silt, sand, and gravel. This unit incorporates both active channels and modern floodplains. In some areas, this unit can reach 15 meters in thickness. The Alluvium of Smaller Streams, which is also in the southern section of the project site, is predominantly made up of unconsolidated clay, silt, sand, and some gravel. This unit is differentiated from the Floodplain Deposits based on the size of the stream which deposited it.

2.2.4 Available Mapping

DOGAMI developed a publication based on the Oregon Resilience Plan CSZ scenario for the state of Oregon. The publication, Open-File Report O-13-06, primarily consists of GIS data of site conditions, ground motions, ground deformations, and other hazards associated with a magnitude 9.0 event on the CSZ (Madin and Burns, 2013). Datasets of interest for this project include the following:

- Shear Wave Velocity within 30 meters of the Ground Surface (Vs30)
- Bedrock and Site Peak Ground Acceleration (PGA)
- Bedrock and Site 1-second Spectral Acceleration (SA1)
- Bedrock and Site Peak Ground Velocity (PGV)
- Liquefaction Susceptibility, Probability, and Permanent Ground Deformation (PGD)
- Earthquake-Induced Landslide Susceptibility, Probability, and PGD

The provided methodology indicates that, within the project area, the majority of these datasets were derived based on the Relative Earthquake Hazard Map of the Portland Metro Region (IMS-1; Mabey and others, 1997); the Oregon Geologic Data Compilation Release 5 (OGDC-5; Ma and others, 2009); and the Statewide Landslide Information Database for Oregon Release 2 (SLIDO-2; Burns and others, 2011). The bedrock ground motions included in the publication were provided to DOGAMI by the U. S. Geological Survey (USGS) and are based on the USGS Cascadia M 9.0 scenario ShakeMap®.
Following the publication of O-13-06, DOGAMI published the Oregon Geologic Data Compilation Release 6 (OGDC-6; Smith and Roe, 2015) and Release 3.4 of the Statewide Landslide Information Database for Oregon (SLIDO-3.4; Burns and Watzig, 2017). These recent publications have not yet been incorporated into DOGAMI's CSZ scenario datasets.

Bedrock 0.3-second spectral acceleration data were downloaded from the USGS website for the Cascadia M 9.0 scenario ShakeMap® (USGS, 2011). Data for the 0.2-second spectral acceleration, as used in building codes, were not available. For preliminary planning purposes, the 0.2-second spectral acceleration can be approximated as the 0.3-second spectral acceleration.

2.3 Modifications to Published Geologic Mapping

Our geologic study draws on data from the O-13-06 document which characterizes the geologic hazards for the Cascadia Subduction Zone event, but also incorporates landslide data from SLIDO 3.4 and new geologic information from the OGDC-6. The OGDC dataset combines the best-known geologic mapping of the entire state into a single database. While more recent mapping of the area has been completed, most notably USGS Open-File Report 2018-1044, the digital files were not made available when both DOGAMI and the USGS were contacted. Minor modifications were made to the OGDC-6 layer based on metadata within the file.

Using the OGDC-6 as the geologic base map, we overlaid and added in deposits from SLIDO-3.4 that were not included in the geologic map. Within the entire study area, only a single landslide deposit had to be added in the northeast portion of the study area. The resulting final map is shown on Figure 1.

2.4 Seismic Hazard Maps

The purpose of the maps is to delineate the ground shaking and permanent ground deformation hazard across the service area based on a magnitude 9.0 CSZ earthquake. Ground shaking hazard is delineated in terms of the following:

- Peak ground acceleration (PGA)
- 0.3-second spectral acceleration (SA0.3)
- 1-second spectral acceleration (SA1)
- Peak ground velocity (PGV)

Permanent ground deformation (PGD) hazard is delineated by the following:

- Probability of liquefaction
- Liquefaction-induced lateral spread PGD
- Liquefaction-induced settlement PGD
- Probability of earthquake-induced sliding in both wet and dry conditions
- Landslide-induced PGD in both wet and dry conditions

These maps were derived using the same approach as the published DOGAMI O-13-06 magnitude 9.0 CSZ scenario maps but using more recently published background information and more targeted assumptions about local conditions. We provide maps of the updated information (i.e., most recent geologic map in Figure 1) and maps developed as intermediate steps (i.e., Figure 3, Liquefaction Hazard, and Figures 4 and 5, Landslide Susceptibility in both wet and dry conditions) in deriving the final hazard maps.

Modifications to both the O-13-06 methodology and additional input maps are summarized below.

2.5 Shear Wave Velocity, Vs30

For the study area around Newberg, there are published DOGAMI maps which show Vs30 values. However, because multiple methodologies were used across the area, the data lacks uniformity. Additionally, there are no 3D shear wave velocity models such as exist for the Portland metropolitan area. Therefore, due to the limited availability of Vs30 data throughout the project study area, values were assigned based on NEHRP site classes. In our opinion, this was the best way to create a unified map. To do this, Vs30 values from Holzer and others (2005), which are adapted from BSSC (2001), were assigned to each geologic unit based on its site class. In the determination of site classes, both published classes in O-13-06 as well as interpretation of geologic units were used. Both the site class and Vs30 values assigned to each geologic category are shown below. These values should be considered estimates and assume that the material in the upper 100 feet is uniform.

- Columbia River Basalt: Site Class B, 1130 m/s
- Troutdale and Scappoose Formations: Site Class B/C Boundary, 760 m/s
- Landslide deposits overlying rock: Site Class C, 540 m/s
- Landslide deposits overlying flood deposits: Site Class D, 270 m/s
- Missoula Flood Deposits: Site Class D, 270 m/s
- Floodplain Deposits and Alluvium of Smaller Streams: Site Class D to E, 180 m/s

While some published DOGAMI maps classify landslide deposits as Site Class F, it is our opinion that the deposits do not meet the criteria of Site Class F material, as defined in the Hazus® -MH 2.0 Technical Manual (FEMA, 2011). The final Vs30 map is shown on Figure 2.

2.6 Liquefaction Hazard

The liquefaction susceptibility map provided in O-13-06 is a compilation of liquefaction susceptibility maps from other DOGAMI publications. Within the Newberg area, this includes both IMS-7 and IMS-24. Explanatory texts for both of these interpretive map series indicate that susceptible units were assumed to be saturated. This was believed to be a conservative approach as the majority of highly liquefiable sediment is restricted to alluvial deposits in areas of low relief and high rainfall. However, comparison of the maps revealed that different methodologies were used to determine liquefaction susceptibility. This meant that susceptibility within the same unit could vary significantly across the boundary between IMS-7 and IMS-24. Therefore, we used our updated geologic map (Figure 1) and employed the Youd and Perkins (1978) methodology, as well as knowledge of regional liquefaction susceptibility, to assign new liquefaction susceptibilities and create a unified map. The resulting map is shown on Figure 3.

2.7 Landslide Susceptibility

We generally followed the methodology and Geologic Group assignments as described in O-13-06, using the compiled geologic map shown on Figure 1 and discussed above, as the base map. We assigned Geologic Group C (relatively weak material) to areas mapped as Alluvial of Smaller Streams, Missoula Flood Deposits, Floodplain Deposits, and Landslide Deposits. All other geologic units, including Columbia River Basalt, Scappoose Formation, and Troutdale Formation, were assigned Geologic Group B. We calculated a slope map from bare earth lidar data of the area to complete the landslide susceptibility map because DOGAMI's slope map was not included in O-13-06. In order to give what we believe are upper and lower limits of landslide susceptibility, maps accounting for both dry and wet conditions were generated. Dry conditions assume that the groundwater is below the level of sliding, while wet conditions assume that the groundwater level is at ground surface. The landslide susceptibility maps are shown on Figures 4 and 5.

2.8 PGA, SA1, SA0.3, and PGV

The site amplification factors in O-13-06 were calculated based on site class and the appropriate Vs30 value for each site, as determined when creating the Vs30 map as described above. We calculated the PGA and SA1 site amplification factors for the Newberg area from the Vs30 raster described above using the approach referenced in O-13-06 (Boore and Atkinson, 2008) and applied them to the bedrock PGA and SA1 maps provided with O-13-06 to produce PGA, SA1, and PGV maps modified for Site Class.

Maps of Peak Ground Acceleration, 1-Second Spectral Acceleration, and Peak Ground Velocity are shown on Figures 6, 8, and 9, respectively. The same methodology was used for the 0.3-Second Spectral Acceleration map, shown in Figure 7, using the bedrock SA0.3 map from the USGS scenario. It should be noted that current USGS & DOGAMI mapping does not include mapping for the 0.2-second spectral acceleration, but it does include spectral acceleration for a period of 0.3 seconds. For preliminary planning purposes the 0.2-second spectral acceleration can be approximated as the 0.3-second spectral acceleration.

2.9 Probability of Liquefaction

We used the refined liquefaction hazard map described above and followed the methods presented in O-13-06 to develop a map of liquefaction probability. Because we assigned a liquefaction susceptibility of "Low to Moderate" for Missoula Flood Deposits, its Pml value, which is defined as the proportion of a map unit susceptible to liquefaction, had to be interpreted. Because geologic units with low and moderate susceptibilities have Pml values of 0.05 and 0.1 respectively. Therefore, Missoula Flood Deposits were assigned a Pml of 0.075. The resulting map is shown on Figure 10.

2.10 Liquefaction-Induced PGD

2.10.1 Lateral Spreading

We used the refined liquefaction hazard map described above and followed the methods presented in O-13-06 to calculate permanent ground deformations from liquefaction-induced lateral spreading. The map of estimated PGD due to lateral spreading is included on Figure 11.

2.10.2 Settlement

DOGAMI did not include a map of predicted ground settlement associated with liquefaction in O-13-06. We calculated estimated liquefaction-induced settlements following the methodology in Chapter 4 of the Hazus® -MH 2.0 Technical Manual (FEMA, 2011), using the refined liquefaction hazard map discussed above.

The FEMA method associates each susceptibility category with a unique settlement amplitude value. Each of the values is assumed to have an uncertainty with a uniform probability distribution from one-half to two times the respective value. The map of estimated PGD due to liquefaction-induced settlement is included on Figure 12.

2.11 Probability of Earthquake-Induced Landslides

We used the refined landslide susceptibility and PGA maps described above and followed the methods presented in O-13-06 to calculate and map the probability of earthquake-induced landslides. To give what we believe are upper and lower limits of the probability of earthquake-induced landslides, we calculated probabilities in both wet and dry conditions. This was done by populating tables 4.17 and 4.18 in Chapter 4 of the Hazus® - MH 2.0 Technical Manual (FEMA, 2011). The resulting maps are shown on Figures 13 and 14.

2.12 Earthquake-Induced Landslide PGD

The earthquake-induced landslide PGD map is based on the methodology in Hazus® -MH 2.0 Technical Manual (FEMA, 2011), which is referenced in O-13-06. We retained the acceleration term that DOGAMI chose to remove from FEMA equation 4-25 because the acceleration is in "decimal fraction of g's," not cm/sec2, as DOGAMI indicated.

Additionally, we observed that the equation given by DOGAMI for the displacement factor did not produce a curve similar to the FEMA Figure 4.14 relationship. In examining the DOGAMI equation, we saw that if the first constant was made negative, a curve similar to the FEMA Figure 4.14 relationship was seen. Therefore, we based our calculations on this slightly amended and corrected relationship to match the source FEMA publication. As we did for all landslide maps, we generated permanent ground deformation maps for both wet and dry conditions. These maps were based on probability inputs generated when calculating the probability of earthquake-induced landslides. Our maps of estimated earthquake-induced landslide permanent ground deformation are shown on Figures 15 and 16.

2.13 Seismic Hazards at Critical Infrastructure

The locations of selected infrastructure have been provided by HDR. The approximate locations of the selected infrastructure are shown on Figures 1 through 16 and a summary of the GIS map results for seismic hazards at these specific locations are shown on an attached Table 1.

3 WATER TREATMENT PLANT SLOPE EVALUATION

3.1 Background

The existing WTP is adjacent to a steep slope that is north of the Willamette River. The site also contains a pipe bridge that extends from the crest of the north slope to the well fields south of the Willamette River. We understand based on existing information that the north slope has had periods of instability. Most notably, a slide occurred along the north slope in the spring of 1996 and was documented in a report prepared by Squier Associates dated June 24, 1999. A repair to the slope consisting of a rock buttress was designed and documented by Squier Associates in a summary report dated June 28, 2002. According to the summary report, the slope repair was completed on October 26, 1999.

An additional slope evaluation was performed by Northwest Geotech, Inc. (NGI), and was documented in a summary letter dated November 8, 2016. According to the findings in the NGI summary letter, recent and historic landslides have been observed along the riverbank near the existing pipe bridge. We understand that there are two inclinometers installed along the north slope. One inclinometer is located near the existing pipe bridge and the other is south of the existing WTP. However, the data from the two inclinometers was not made available at the time of this report.

The approximate location of the WTP site is shown on Figure 17, Vicinity Map and the current explorations and slope stability section are shown on Figure 18.

3.2 Subsurface Conditions

The field exploration program for the project included two geoprobes, designated P-1 and P-2, and two cone penetration tests (CPTs), designated CPT-1 and CPT-2. The approximate locations of the explorations are shown on Figure 18. The explorations were performed on May 20, 2019. The two geoprobes were advanced to depths ranging from 30 to 68 feet and the two CPTs were advanced to depths ranging from 68 to 83 feet below the existing ground surface (bgs). Details of the field explorations, including techniques used to advance and sample the geoprobes and cone penetration tests, are presented in Appendix A, Field Explorations.

We grouped the materials encountered in our field explorations into three geotechnical units, as described below. This interpretation of the subsurface conditions is based on the explorations and regional geologic information from published sources. The geological units are as follows:

- **Fill:** Silty Gravel with Sand (GM) to Silt with Sand (ML), wood debris also encountered;
- **Fine-Grained Missoula Flood Deposits:** Silt (ML), Silt with Sand (ML), Sandy Silt (ML), Silty Sand (SM), Lean Clay (CL), Fat Clay (CH); and
- **Hillsboro Formation:** Fat Clay (CH).

These geological units were grouped based on their engineering properties, geologic origins, and distribution in the subsurface.

3.3 Groundwater

The depth to groundwater was estimated from a dissipation test performed within CPT-1. According to the results of the dissipation test, the depth to groundwater is approximately 35 feet bgs.

3.4 Seismic and Geologic Hazards

The seismic hazard evaluation for this project was conducted in accordance with the American Society of Civil Engineer's (ASCE) Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE 7-16), which is based on earthquake ground motions with a 2,475-year return period.

3.5 Strong Ground Motion

ASCE 7-16 requires that geotechnical hazard analyses (liquefaction, specifically) be performed for Maximum Considered Earthquake Geometric Mean (MCE_G) ground motions and adjusted for site class effects. Specifically, the peak ground acceleration used in the liquefaction-related hazard analyses, PGA_M, is defined as:

Exhibit 1: Site-Adjusted Peak Ground Acceleration

Equation	Variable and Definition	
$PGA_M = F_{PGA} \times PGA$	PGA_M	MCE_G Peak Ground Acceleration Adjusted for Site Class Effects
	F_{PGA}	Site Coefficient from ASCE 7-16 Table 11.8-1
	PGA	MCE_G Peak Ground Acceleration of Site Class B/C Boundary Conditions

Reference: ASCE 7-16, Equation 11.8-1

For this project, we obtained a PGA_M of 0.474g using a PGA of 0.392g and an F_{PGA} of 1.208. PGA is shown in ASCE 7-16 Figure 22-9 and is derived from the most recent USGS National Seismic Hazard Mapping Project ground motion hazard analyses results by Petersen and others (2014). F_{PGA} is a function of site class and PGA as indicated in ASCE 7-16 Table 11.8-1. The shear wave velocities measured in CPT-1 correspond to Site Class D.

3.6 Liquefaction

Liquefaction is a phenomenon in which excess pore pressure of loose to medium dense, saturated, nonplastic to low plasticity silts and granular soils increases during ground shaking. The increase in excess pore pressure results in a reduction of soil shear strength and a quicksand-like condition.

Soil behavior under seismic loading is the primary factor in determining the susceptibility of a soil to liquefaction. Important factors in evaluating soil behavior are relative density, the fines content (percent of soil by weight smaller than 0.075 millimeter, passing the No. 200 sieve), and the plasticity characteristics of the fines. Relative density is estimated based on methods including Standard Penetration Test (SPT) N-values, CPT tip resistances, and shear wave velocity.

The second major component of a liquefaction study is the design earthquake motions. Seismogenic sources that contribute to the seismic hazards at the site include the CSZ interface, CSZ Benioff zone, and local shallow crustal faults. Because the maximum earthquake magnitudes for sources vary significantly, we used a mean maximum magnitude of 7.5 for ground motions with a 2,475-year return period for liquefaction analyses.

3.7 Liquefaction Analysis and Liquefaction-Induced Settlement

Shannon & Wilson evaluated liquefaction potential of the soils by performing liquefaction analyses on the CPTs using the Boulanger and Idriss (2014) method. The liquefaction analysis for CPT soundings was accomplished using the computer program CLiq Version 2 by GeoLogismiki, which incorporates the Boulanger and Idriss (2014) method. Shannon & Wilson used the ground motion parameters described above (i.e., PGA of 0.474g at the surface and moment magnitude 7.5). Soil layers identified as potentially liquefiable in the explorations are summarized in Exhibit 2.

Exhibit 2: Summary of Liquefaction-Induced Settlement

Location	Approximate Ground Surface Elevation (feet)	Approximate Groundwater Elevation (feet)	Approximate Liquefiable Layer Depth (feet)	Approximate Settlement at Ground Surface (inches)
CPT-1	170	135	35 to 45	1.5
CPT-2	170	135	36 to 46	1

Exhibit 2 also presents total estimated liquefaction-induced settlement at the ground surface. Liquefaction-induced settlement magnitudes based on CPT soundings were estimated using Zhang et al. (2002).

3.8 Lateral Spreading

Lateral spreading hazards can exist in areas with mild slopes adjacent to a much steeper slope or vertical face. Lateral spreading failure can occur if soil liquefaction develops during a seismic event and the ground acceleration (inertial force) briefly surpasses the yield acceleration (shear strength) of the liquefied soil. This can cause both the liquefied soil and an overlying non-liquefied crust of soil to displace laterally down mild slopes or towards an embankment face. The displacements are cumulative and permanent in nature.

Shannon & Wilson performed a preliminary screening of lateral spreading hazards at the site using the Zhang et al. (2004) methodology. The results of the Zhang et al. (2004) analyses at the project site indicate lateral spread displacements may be up to approximately 2 feet at a distance of approximately 300 feet from the crest of the slope, which would impact existing infrastructure at the WTP site. More accurate assessments of the liquefaction-related hazards present at the site may be made using non-linear time history numerical models that explicitly model the buildup of excess pore water pressure in the soil and associated soil strain (e.g. 2-dimensional FLAC analyses). However, these analyses are beyond the scope of this project.

3.9 Slope Stability

We performed a slope stability analysis at one cross-section through the slope adjacent to the existing pipe bridge, based on available topographic information (i.e. LiDAR), and our subsurface explorations. The subsurface groundwater was based on the water level estimated from our CPT explorations and the water level within the Willamette River was based on the gage height measured from the nearest river gage. Also, the riverbed elevation was estimated from a USGS bathymetric survey performed in 2002.

3.9.1 Approach

Slope stability is influenced by various factors, including the following: (1) the geometry of the soil mass and subsurface materials; (2) the weight of soil materials overlying a potential failure surface; (3) the shear strength of soils and/or rock along a potential failure surface; and (4) the hydrostatic pressure (groundwater levels) present within the soil mass and along a potential failure surface.

The stability of a slope can be expressed in terms of a factor of safety, which is defined as the ratio of resisting forces to driving forces. At equilibrium, the factor of safety is equal to 1.0, and the driving forces are balanced by the resisting forces. Slope movement is predicted when the driving forces exceed the resisting forces, i.e., the factor of safety is less than 1.0.

An increase in the factor of safety greater than 1.0, whether by increasing the resisting forces or decreasing the driving forces, reflects a corresponding increase in the stability of the mass. The actual factor of safety may differ from the calculated factor of safety, due to variations or uncertainty in the soil strength, subsurface geometry, potential failure surface location and orientation, groundwater level, and other factors that are not completely known.

Shannon & Wilson performed the slope stability analysis using the computer program SLOPE/W, Version 10.0.0.17401 (Geo Slope International, 2018). The Morgenstern-Price method was used for rotational and irregular surface failure mechanisms. We utilized information from the closest explorations to estimate material strength and unit weight parameters for the geologic units assumed to underlie the slope. Specifically, strength correlations based on the CPTs were used. Liquefied strength parameters were developed from CPT correlations.

The slope stability was evaluated for the static, seismic, and post-seismic (liquefied soil) conditions. See discussions of these various conditions below and Exhibit 3 for tabulations of the results of our slope stability analyses.

3.9.2 Static

For slopes supporting or impacting essential facilities, a minimum factor of safety of 1.5 is recommended for the static condition.

3.9.3 Seismic

A minimum factor of safety of 1.1 is recommended for the seismic case. Shannon & Wilson performed pseudo-static analyses to evaluate the seismic slope stability using a horizontal seismic coefficient of 0.237, which is equal to one-half of the PGA_M. If the factor of safety of the critical failure surface was less than 1.1, potential displacements were estimated using the procedures in the National Cooperative Highway Research Program (NCHRP) document NCHRP 611 (NCHRP, 2008).

3.9.4 Post-Seismic

A minimum factor of safety of 1.1 is recommended for the post-seismic (liquefied) condition. A failure surface with a factor of safety less than 1.1 indicates the potential for a flow failure caused by a loss of strength within a liquefied soil layer. A flow failure is initiated when a shear failure occurs along a failure surface and is often characterized by large rapid ground movement of the soil mass inside the failure zone.

3.9.5 Results of the Slope Stability Analysis

We evaluated the stability of the slope for static, seismic, and post-seismic conditions. Based on our analysis, the slope is marginally stable under static conditions and is not stable in seismic or post-seismic conditions. The slope stability results are summarized in Exhibit 3 and plots of the results are shown in Appendix B.

Exhibit 3: Summary of Slope Stability Results

Condition	Factor of Safety
Static	1.02
Seismic	0.65
Post-Seismic	0.75

Stability analyses performed for the seismic and post-seismic case indicated that the slope had a factor of safety less than 1.1. Therefore, based on the results, seismically induced displacements and/or flow failures could occur at this site during and after a seismic event. As mentioned previously, lateral spreading (i.e. flow failure) displacements could be in the range of approximately 2 feet at a distance of approximately 300 feet from the crest of the slope. Seismically induced ground deformations using the methods outlined in NCHRP (2008) could be in the range of approximately 7.5 feet.

4 LIMITATIONS

This report, data collection, and hazard mapping has been completed for the exclusive use of HDR, Inc., and the City of Newberg for specific application to the Water System Seismic Resiliency project.

No interpretations between exploration locations are included in this report. The interpretations, conclusions, and recommendations that are contained in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no warranty, either express or implied.

The scope of our geotechnical services described in this report has not included an environmental evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site for evaluation or disposal of contaminated soils or groundwater, should they be encountered, except as noted in this report.

The subsurface explorations were performed to characterize soil conditions at limited locations at the site and our observations are specific to the locations and depths noted on the explorations and in this report. No amount of subsurface exploration can precisely predict the characteristics, quality, or distribution of subsurface site conditions. Potential variation includes but is not limited to the following: varying conditions between borings, changes to the site and subsurface conditions due to the passage of time or intervening causes (natural and manmade), and seasonal or recharge source-influenced fluctuations of groundwater conditions.

Shannon & Wilson has prepared a document, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of this document. This document is attached to the end of this report.

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Table 1 - Seismic Hazards Mapped at Selected Infrastructure Locations

Locations	Site Class	PGA (g)	0.3-Second SA (g)	1-Second SA (g)	Liquefaction-Induced Settlement (inches)	Liquefaction-Induced Lateral Spreading (inches)	Earthquake-Induced Landslide PGD (Wet) (feet)
North Valley Reservoir #1	D	0.163	0.486	0.301	0.5-1.5	0-0.1	~2 near slope 150 feet from reservoir
North Valley Reservoir #2			0.482				
Water Treatment Plant	D	0.163	0.599	0.297	0.5-1.5	~16 near slope 120 feet from plant	~20 near slope 120 feet from plant
Corral Creek Reservoir	B	0.133	0.251	0.107	0	0-0.1	~0.5 near slope 100 feet from reservoir

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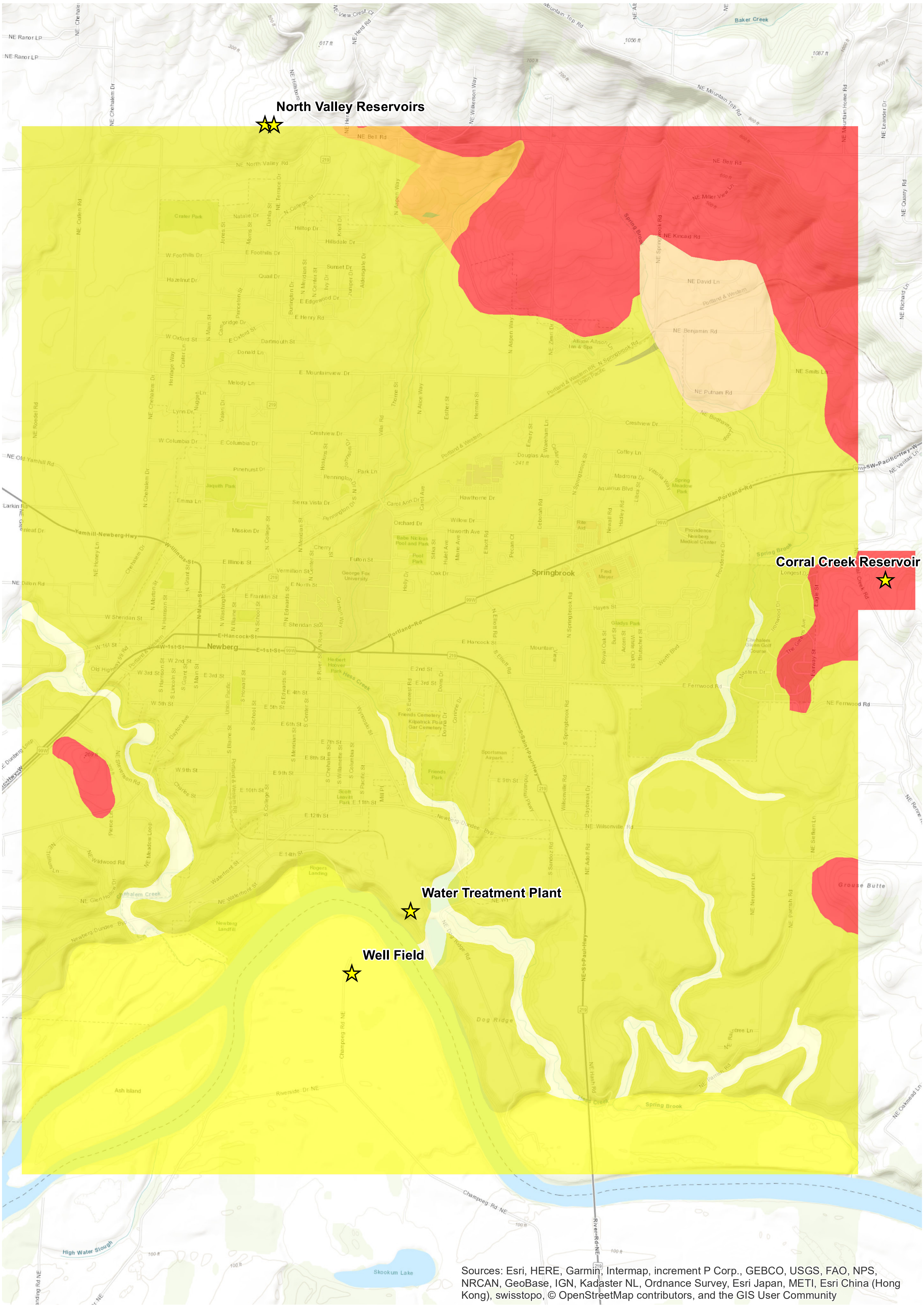

0 0.25 0.5 1
Miles

LEGEND

☆ City Properties

Map Unit Name

- Alluvium of smaller streams
- Floodplain deposits
- Landslide deposits
- Missoula Flood Deposits
- Troutdale Formation
- Scappoose Formation
- Columbia River Basalt Group

NOTES

1. Geologic mapping modified from DOGAMI publicaions OGDC-6 and SLIDO-3.4. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

GEOLOGIC MAP

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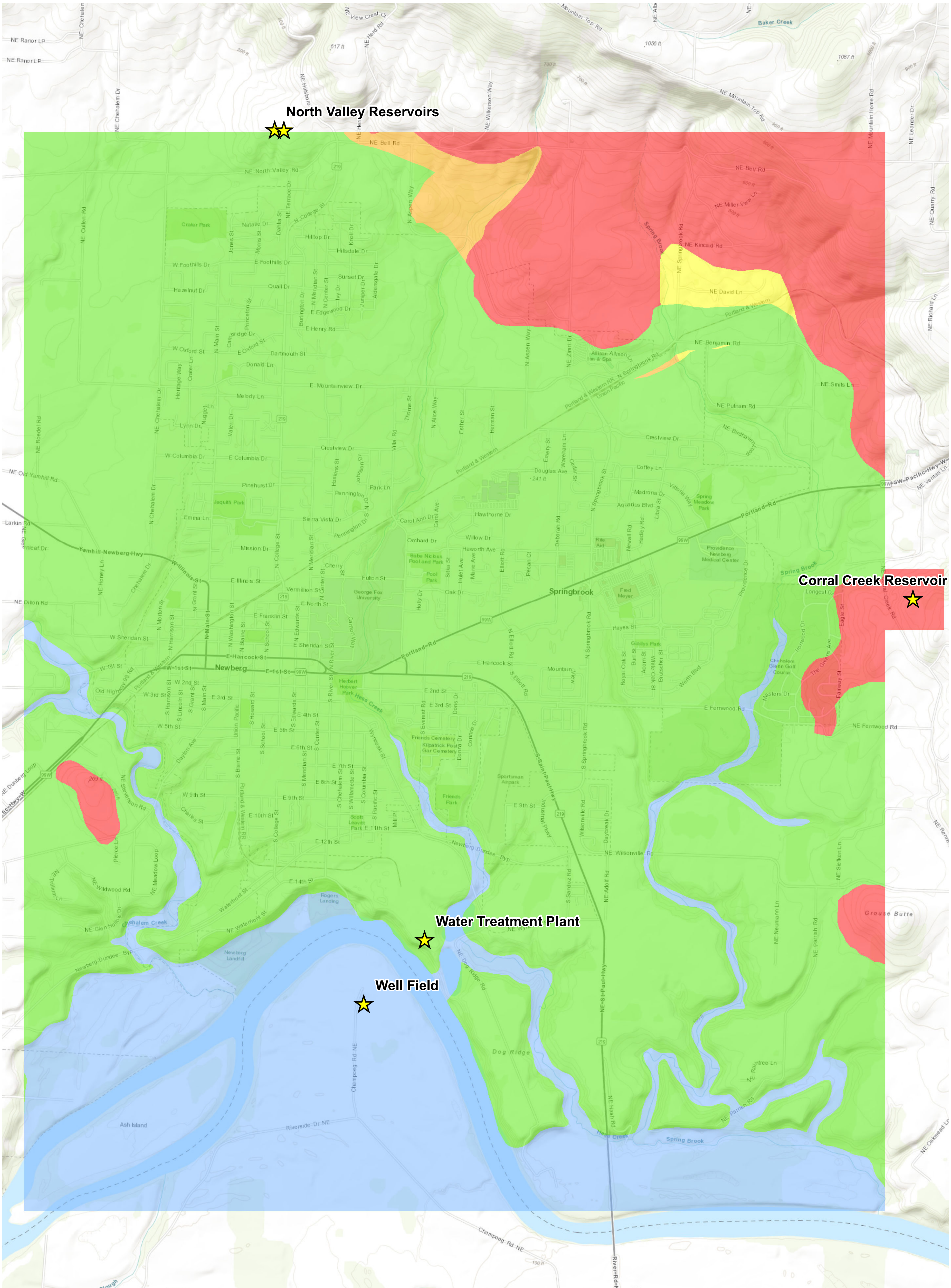
FIG. 1

FIG. 1



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ City Properties

Shear Wave Velocity, Vs30 (m/s)

- 180
- 270
- 540
- 760
- 1130

0 0.25 0.5 1
Miles

NOTES

1. Vs30 based on NEHRP site class and estimated from geologic descriptions. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

SHEAR WAVE VELOCITY, Vs 30

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FIG. 2

FIG. 2

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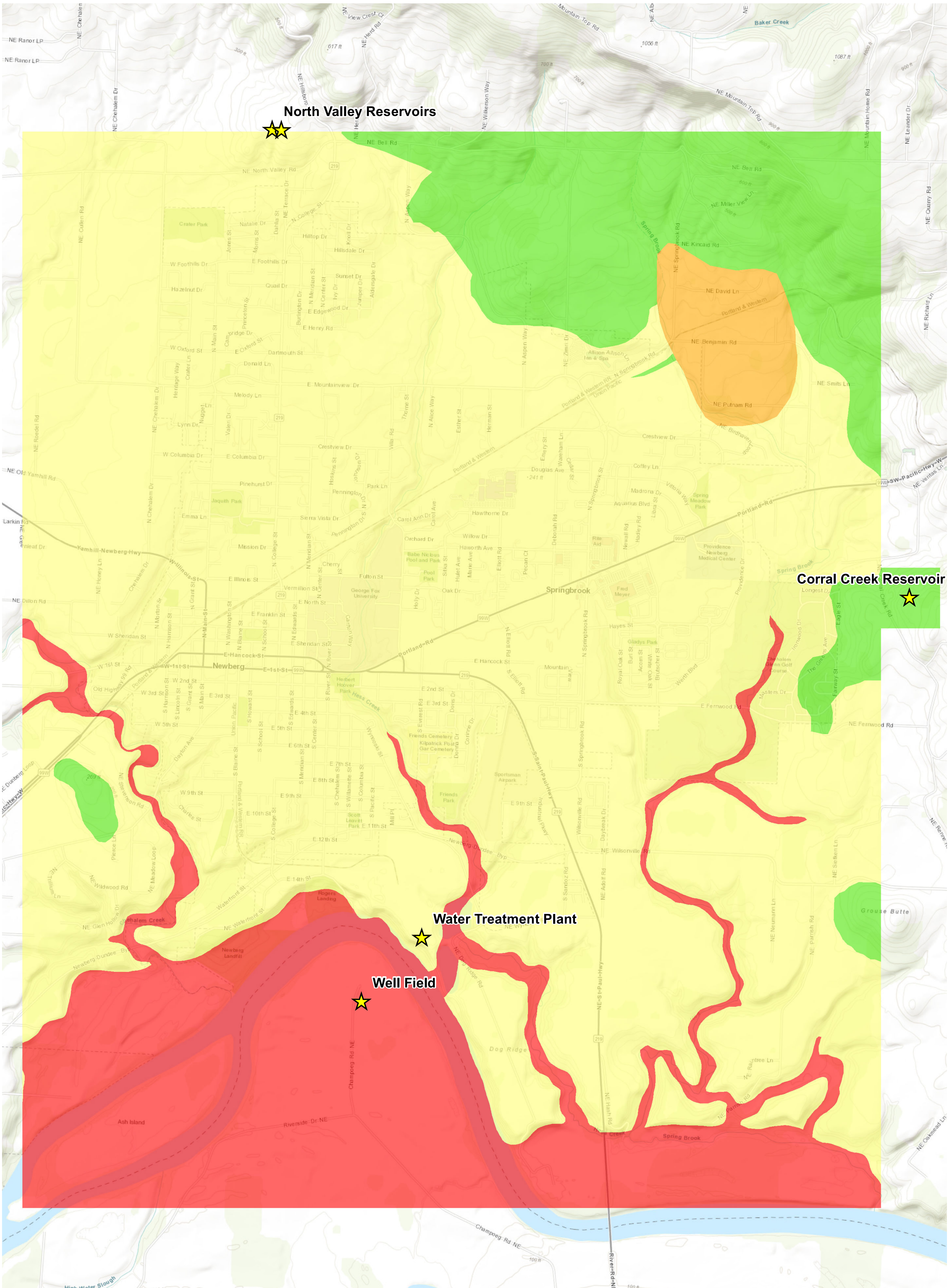

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

★ City Properties

Liquefaction Hazard

- Very Low
- Low to Moderate
- Moderate
- High

0 0.25 0.5 1

Miles

NOTES

1. Liquefaction hazard map developed from data provided with DOGAMI publication OGDC-6, the Youd and Perkins, 1978 methodology, and knowledge of regional liquefaction hazards.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

LIQUEFACTION HAZARD

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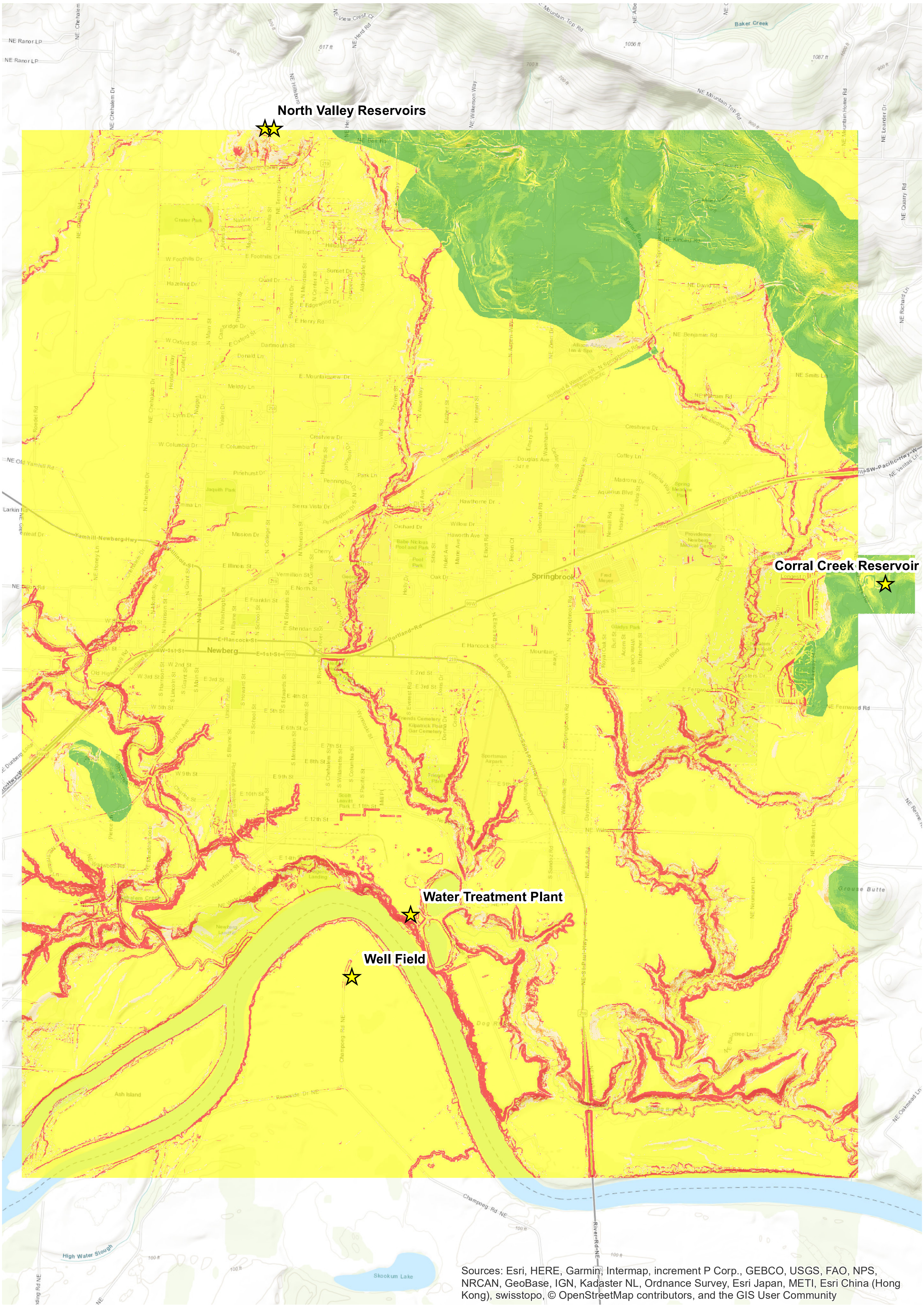
FIG. 3

FIG. 3



LEGEND

Landslide Susceptibility in Dry Conditions

(Scale of 0 - 10)

- 0
- 3
- 4
- 5
- 6
- 7
- 9

☆ City Properties

0 0.25 0.5 1

Miles

NOTES

1. Landslide susceptibility calculated from data provided with DOGAMI publications SLIDO-3.4, O-12-02, OGDC-6 and LiDAR. Methodology taken from HAZUS. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

LANDSLIDE SUSCEPTIBILITY (DRY CONDITIONS)

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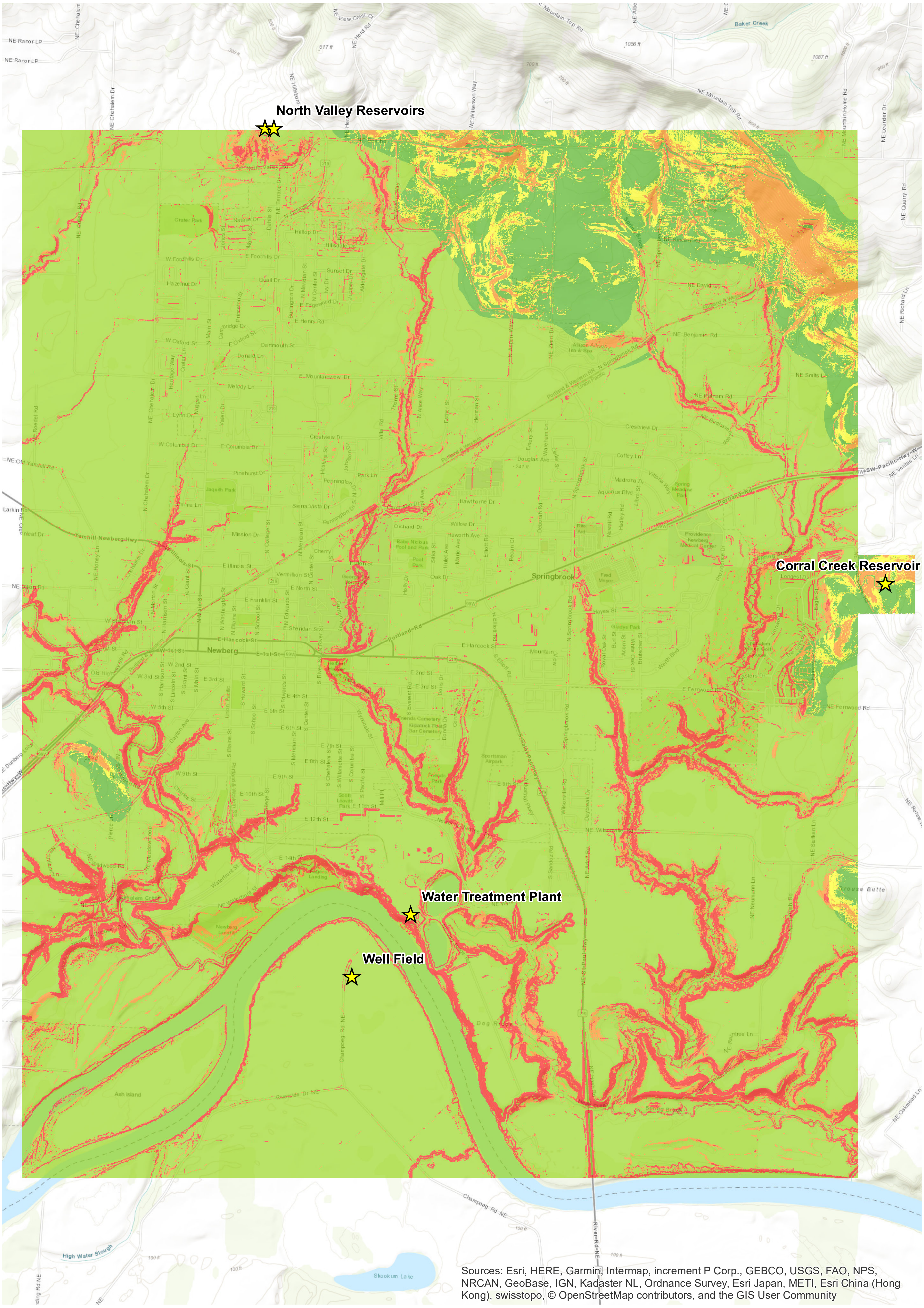
FIG. 4

FIG. 4

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\Landslide Susceptibility (Wet).mxd Date: 9/11/2019 Login: DSJ



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Landslide Susceptibility in Wet Conditions

(Scale of 0 - 10)

- 5
- 7
- 8
- 9
- 10

City Properties

0 0.25 0.5 1

Miles

NOTES

1. Landslide susceptibility calculated from data provided with DOGAMI publications SLIDO-3.4, O-12-02, OGDC-6 and LiDAR. Methodology taken from HAZUS. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

LANDSLIDE SUSCEPTIBILITY (WET CONDITIONS)

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FIG. 5

FIG. 5

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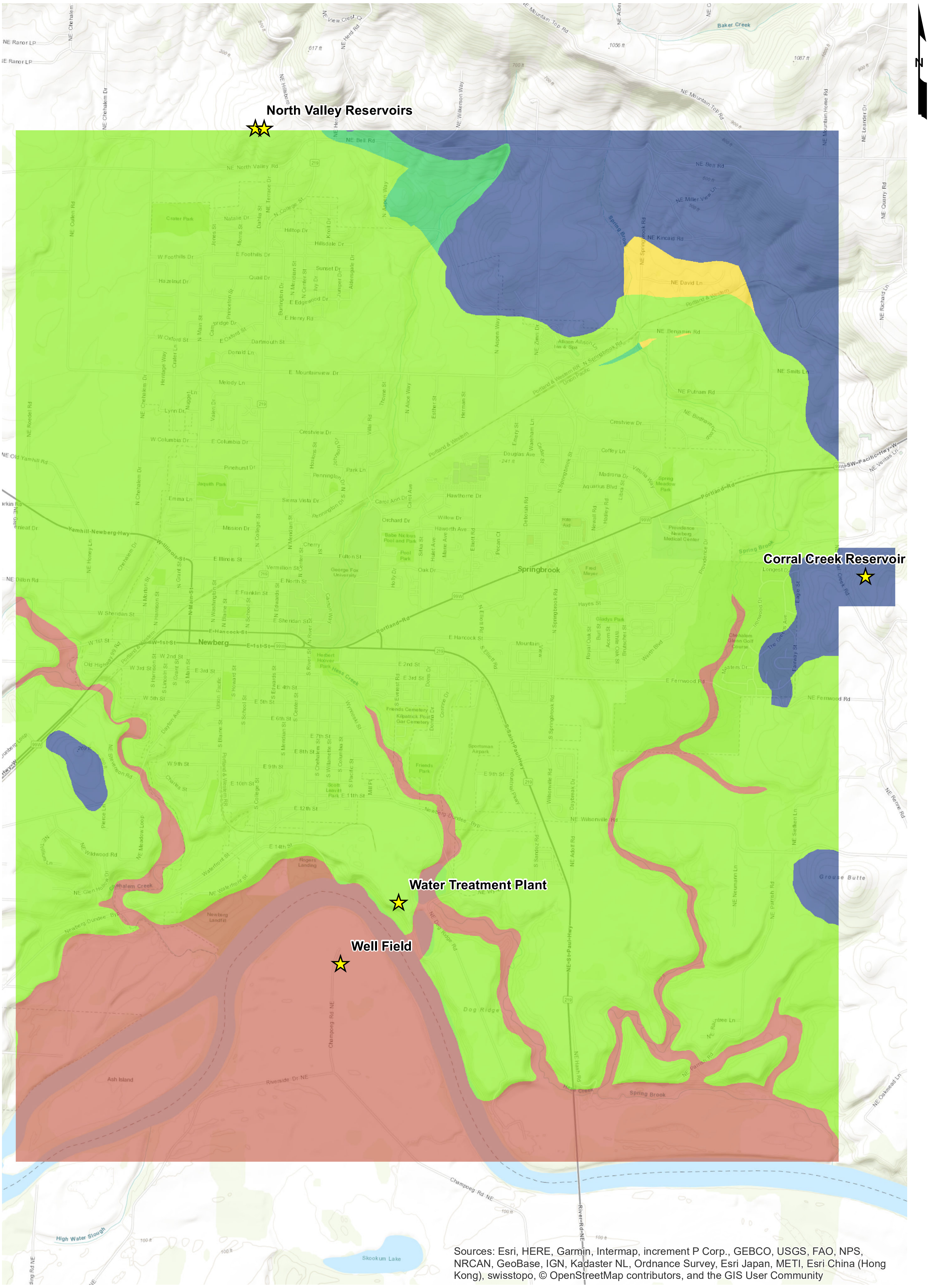

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LEGEND

Site PGA (g)

- 0.13 - 0.14
- 0.14 - 0.15
- 0.15 - 0.16
- 0.16 - 0.17
- 0.17 - 0.18
- 0.18 - 0.19
- 0.19 - 0.2

☆ City Properties

0 0.25 0.5 1
Miles

NOTES

1. PGA map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publication O-13-06 and methodology in Boore and Atkinson, 2008. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

PEAK GROUND ACCELERATION, PGA

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FIG. 6

FIG. 6

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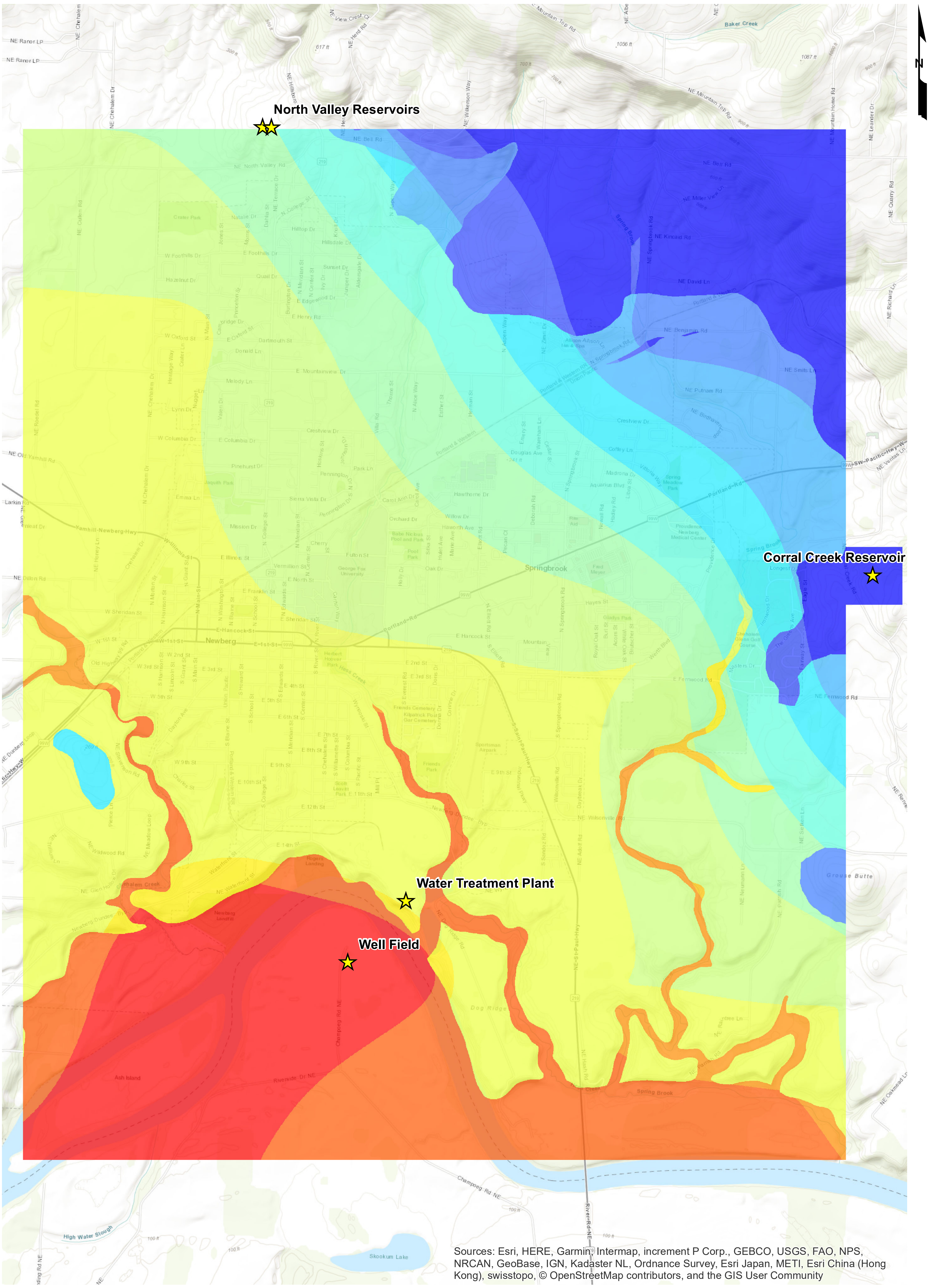

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ City Properties

Site SA 0.3 (g)

- 0.24 - 0.28
- 0.28 - 0.32
- 0.32 - 0.36
- 0.36 - 0.4
- 0.4 - 0.44
- 0.44 - 0.48
- 0.48 - 0.52
- 0.52 - 0.56
- 0.56 - 0.6
- 0.6 - 0.64
- 0.64 - 0.68
- 0.68 - 0.72
- 0.72 - 0.76
- 0.76 - 0.8

0 0.25 0.5 1
Miles

NOTES

1. SA0.3 map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with the USGS Scenario published September 20, 2011, and DOGAMI publications O-12-02 and OGDC-6. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

0.3-SECOND SPECTRAL ACCELERATION, SA0.3

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FIG. 7

FIG. 7

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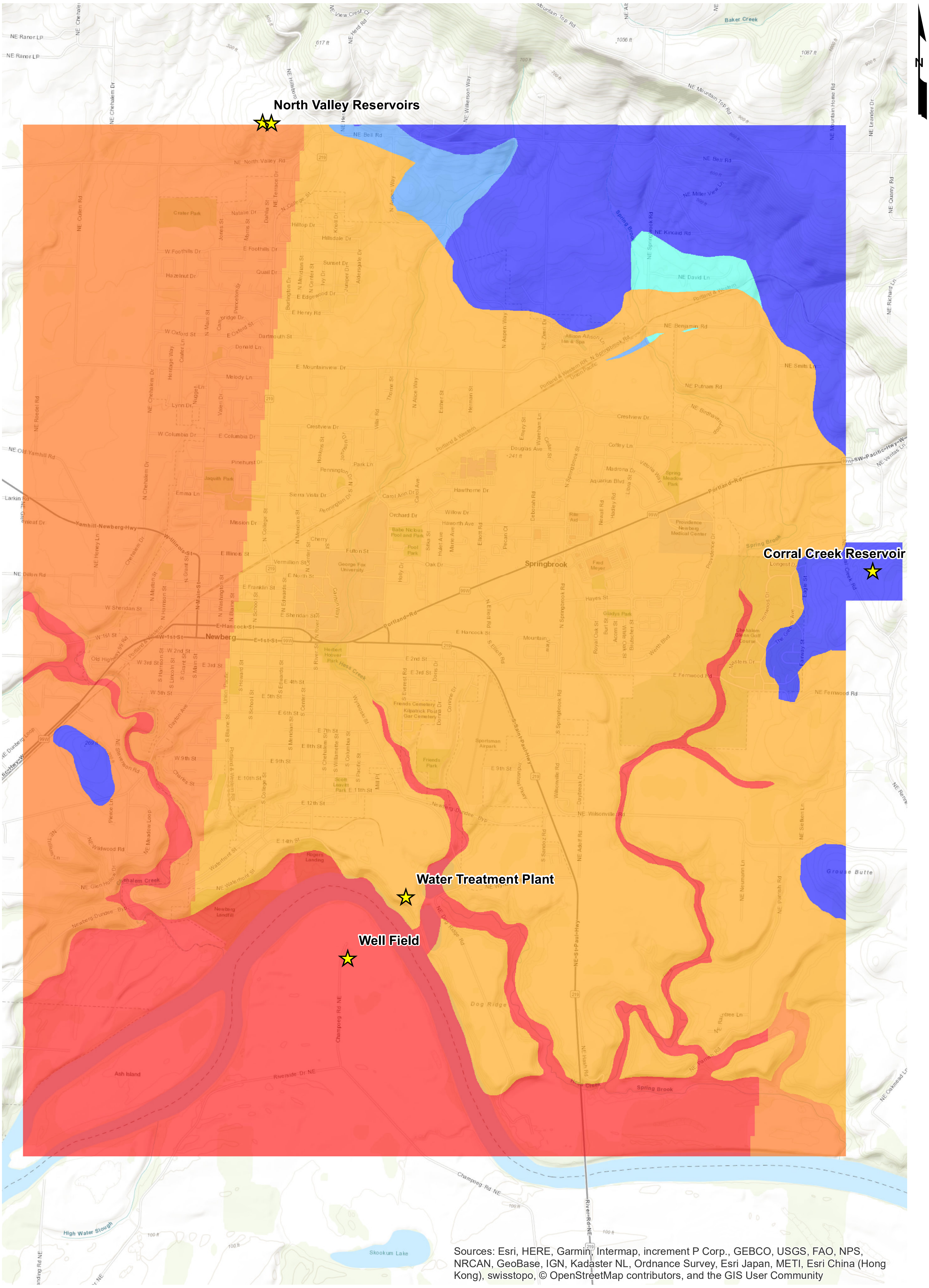

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LEGEND

City Properties

Site SA1 (g)

- 0.10 - 0.12
- 0.12 - 0.14
- 0.14 - 0.16
- 0.16 - 0.18
- 0.18 - 0.2
- 0.2 - 0.22
- 0.22 - 0.24
- 0.24 - 0.26
- 0.26 - 0.28
- 0.28 - 0.3
- 0.3 - 0.32
- 0.32 - 0.34

0 0.25 0.5 1

Miles

NOTES

1. SA1 map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publications O-13-06 and OGDC-6, and methodology in Boore and Atkinson, 2008. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

1-SECOND SPECTRAL ACCELERATION, SA1

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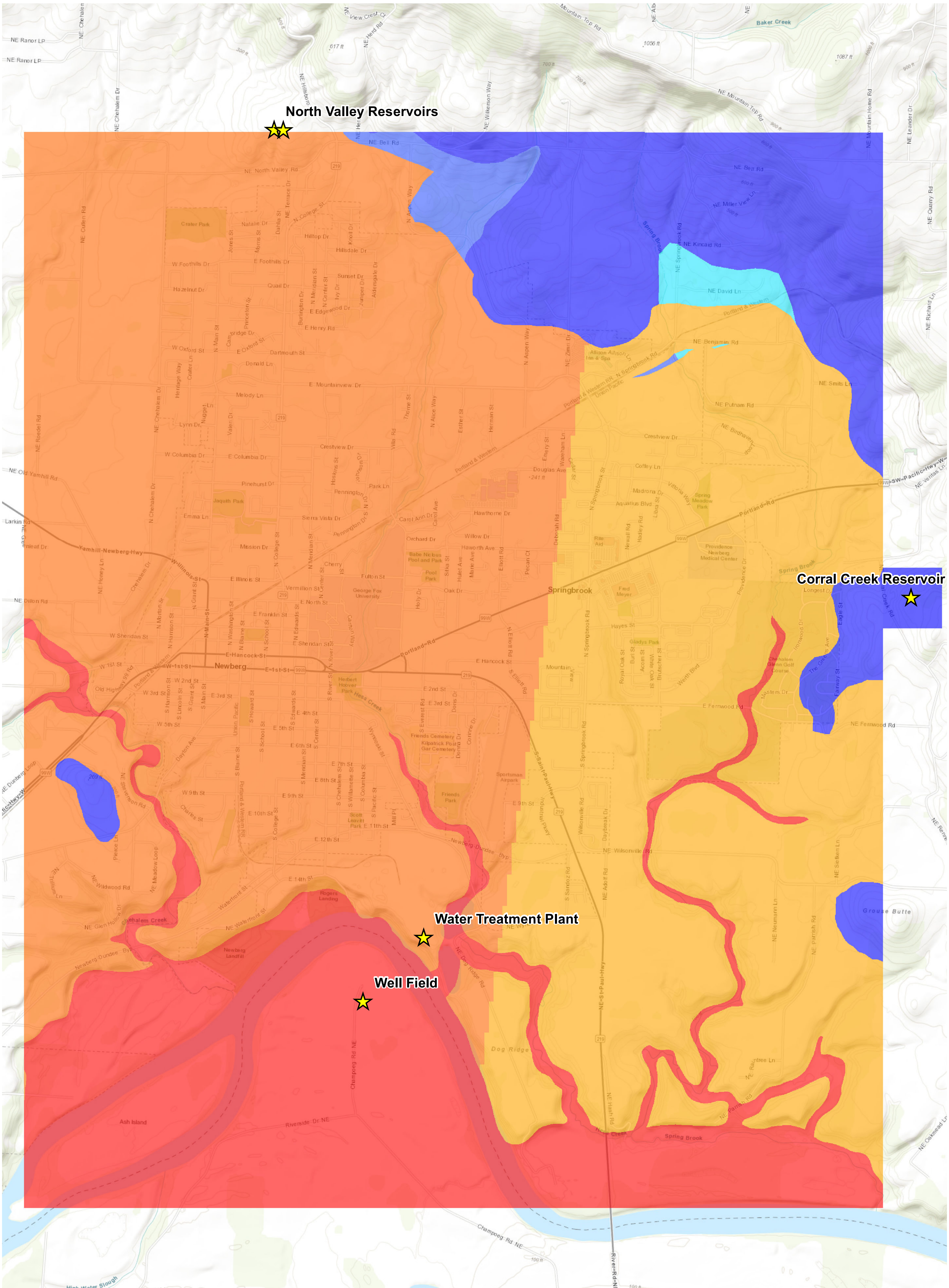
FIG. 8

FIG. 8

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\PGV.mxd Date: 9/11/2019 Login: DSJ



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LEGEND

Site PGV (cm/sec)

- 10 - 12
- 12 - 14
- 14 - 16
- 16 - 18
- 18 - 20
- 20 - 22
- 22 - 24
- 24 - 26
- 26 - 28
- 28 - 30
- 30 - 32

☆ City Properties

0 0.25 0.5 1
Miles

NOTES

1. PGV map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publications O-13-06 and OGDC-6, and methodology in Boore and Atkinson, 2008. See text for details.

City of Newberg Seismic Resiliency
Yamhil County, Oregon

PEAK GROUND VELOCITY, PGV

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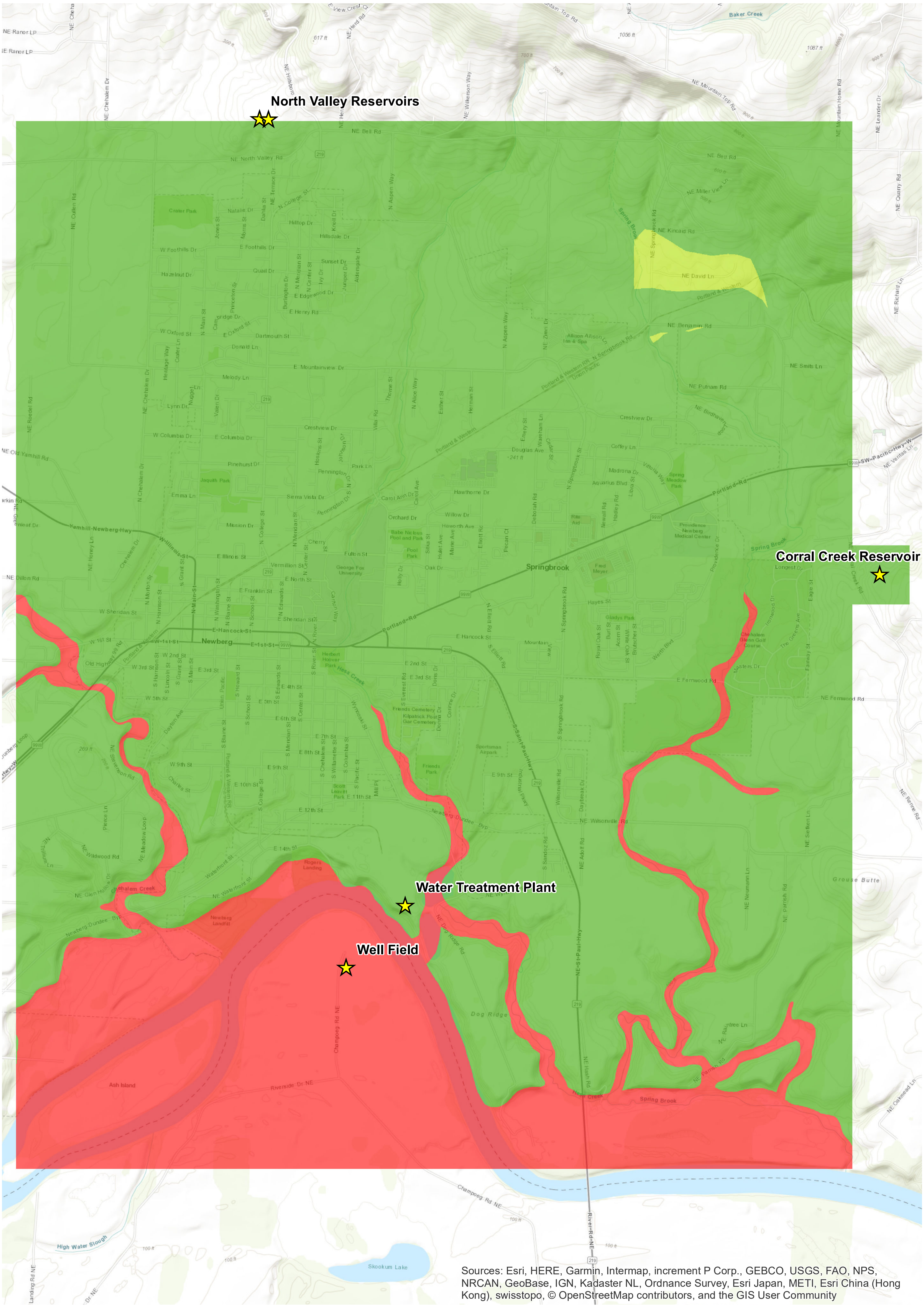
FIG. 9

FIG. 9



LEGEND

Liquefaction Probability (%)

(Proportion of Area Expected to Liquefy)

- 0 - 1
- 1 - 5
- 5 - 10
- 10 - 15

★ City Properties

0 0.25 0.5 1
Miles

NOTES

1. Probability of liquefaction for magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02 and OGDC-6. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

PROBABILITY OF LIQUEFACTION

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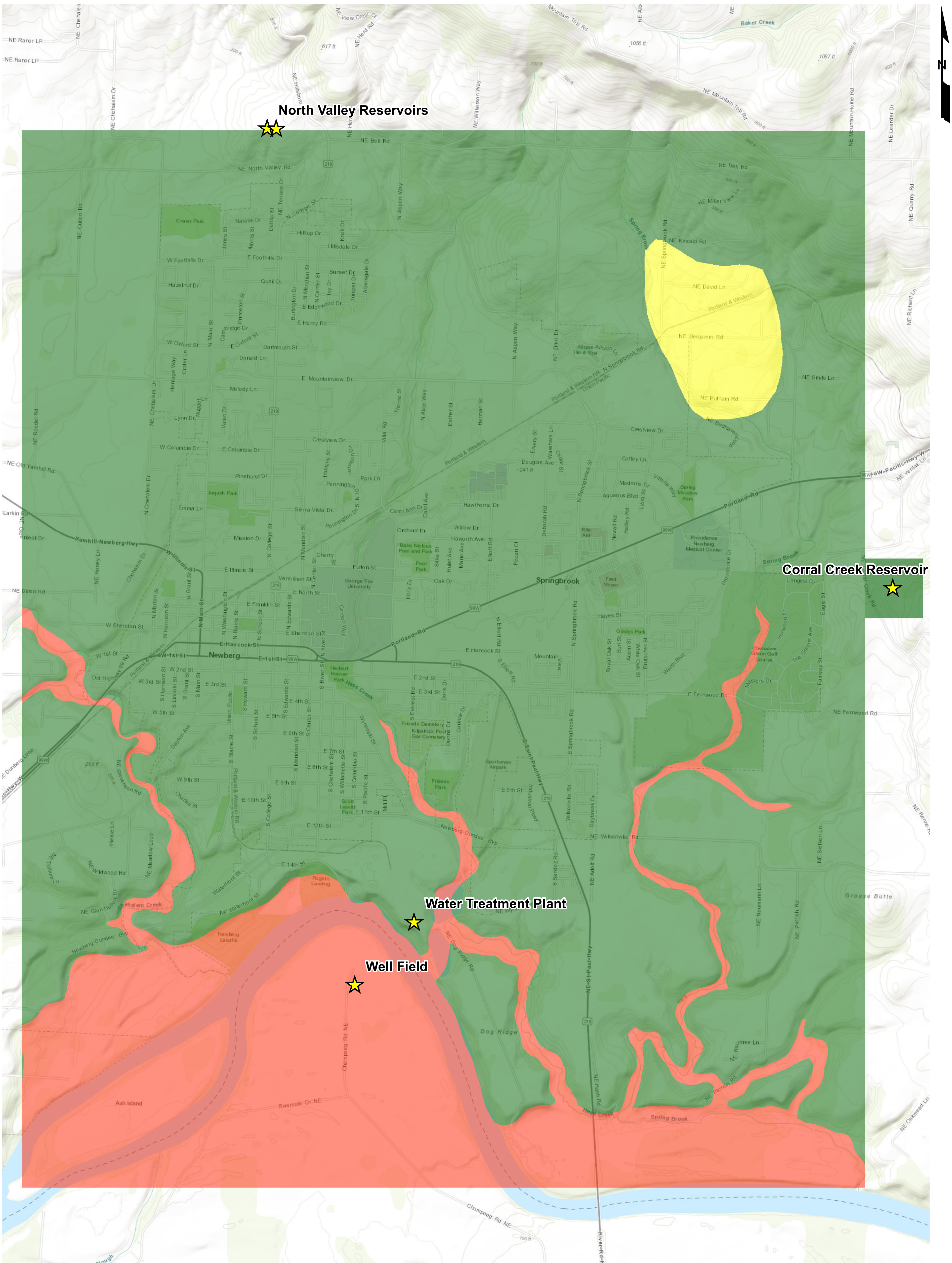
FIG. 10

FIG. 10



LEGEND

★ City Properties

Liquefaction-Induced Lateral Spreading PGD (in)

- 0 - 0.1
- 0.1 - 2
- 2 - 6
- 6 - 12
- 12 - 24

0 0.25 0.5 1
Miles

NOTES

1. Liquefaction-induced lateral spreading PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6 and FEMA publication Hazus-MH 2.0 Technical Manual. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

LIQUEFACTION-INDUCED LATERAL SPREADING PERMANENT GROUND DEFORMATION, PGD

July 2020 101895

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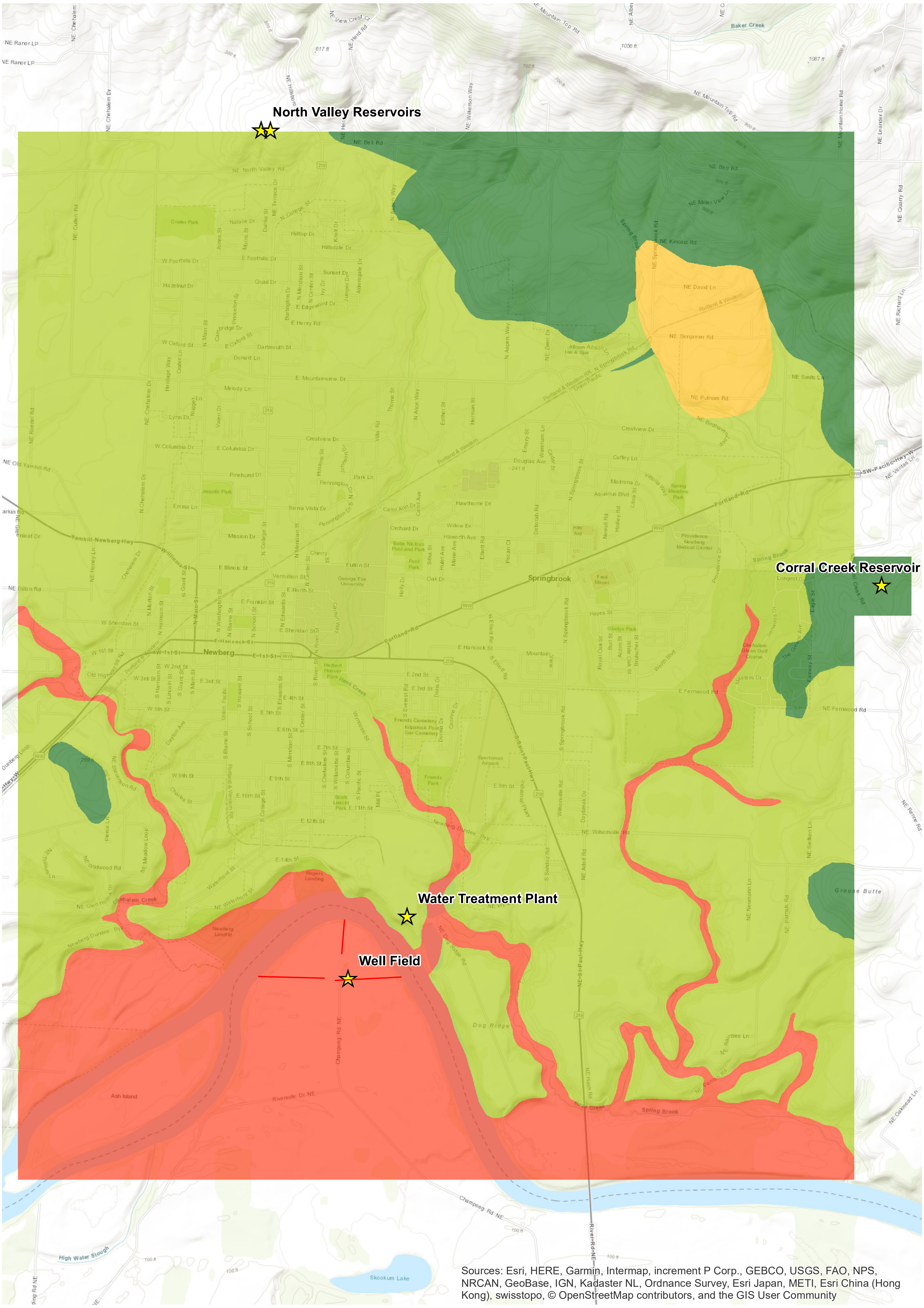
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 11

FIG. 11

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\Liquefaction Settlement.mxd Date: 9/11/2019 Login: DSJ



LEGEND

☆ City Properties

Liquefaction-Induced Settlement PGD (in)

- 0
- 0.5 - 1.5
- 1.5 - 2
- 2 - 6

0 0.25 0.5 1

Miles

NOTES

1. Liquefaction-induced settlement PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6 and FEMA publication Hazus-MH 2.0 Technical Manual. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

LIQUEFACTION-INDUCED SETTLEMENT PERMANENT GROUND DEFORMATION, PGD

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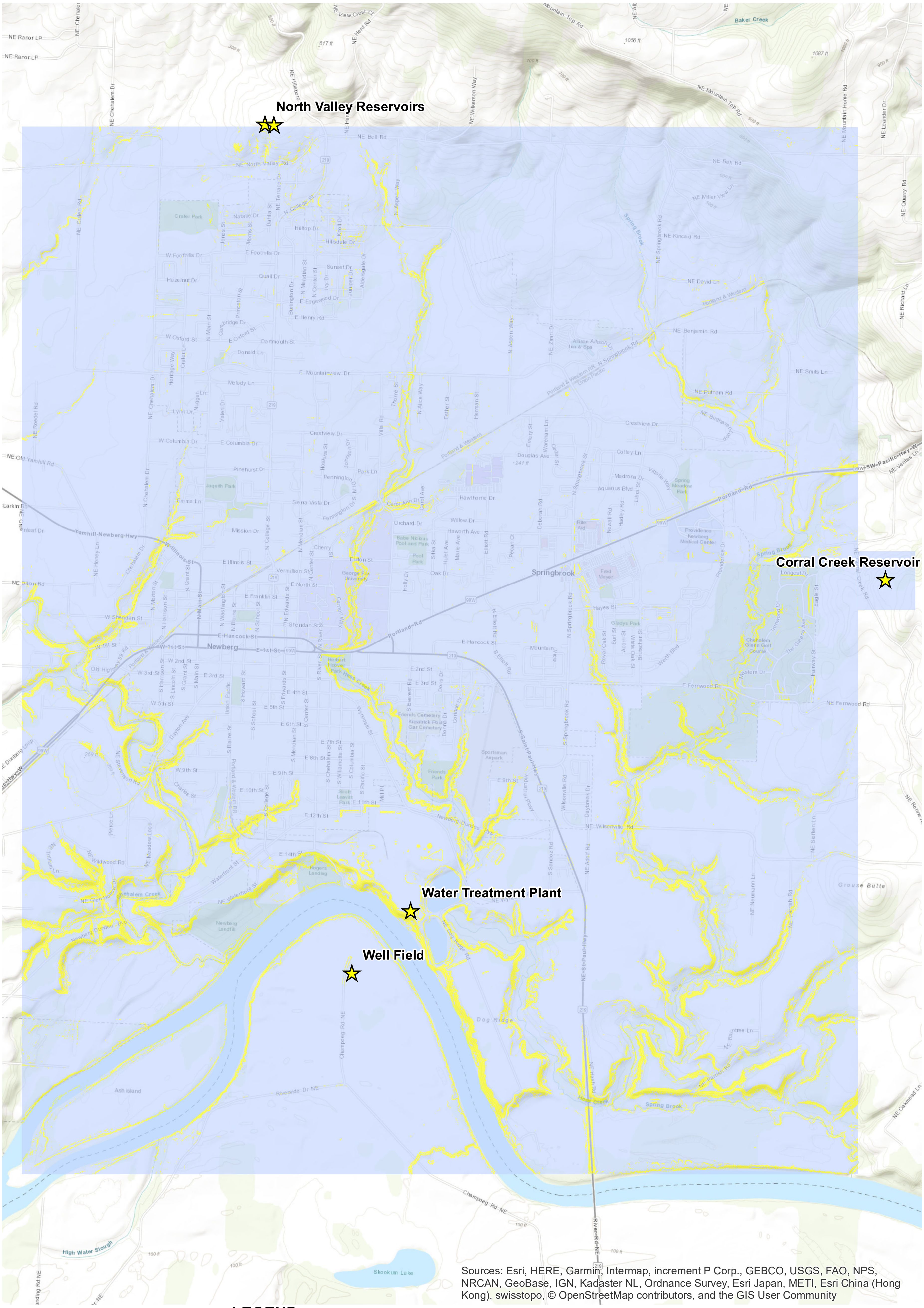
FIG. 12

FIG. 12

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\Landslide Probability (Dry).mxd Date: 9/11/2019 Login: DSJ



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ City Properties

Earthquake-Induced Lanslide Probability in Dry Conditions (%)

(Proportion of Area Expected to Fail)

- 0
- 25

0 0.25 0.5 1

Miles

NOTES

1. Earthquake-induced landslide probability for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-3.4 and LiDAR. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

PROBABILITY OF EARTHQUAKE-INDUCED LANDSLIDES (DRY)

July 2020 101895

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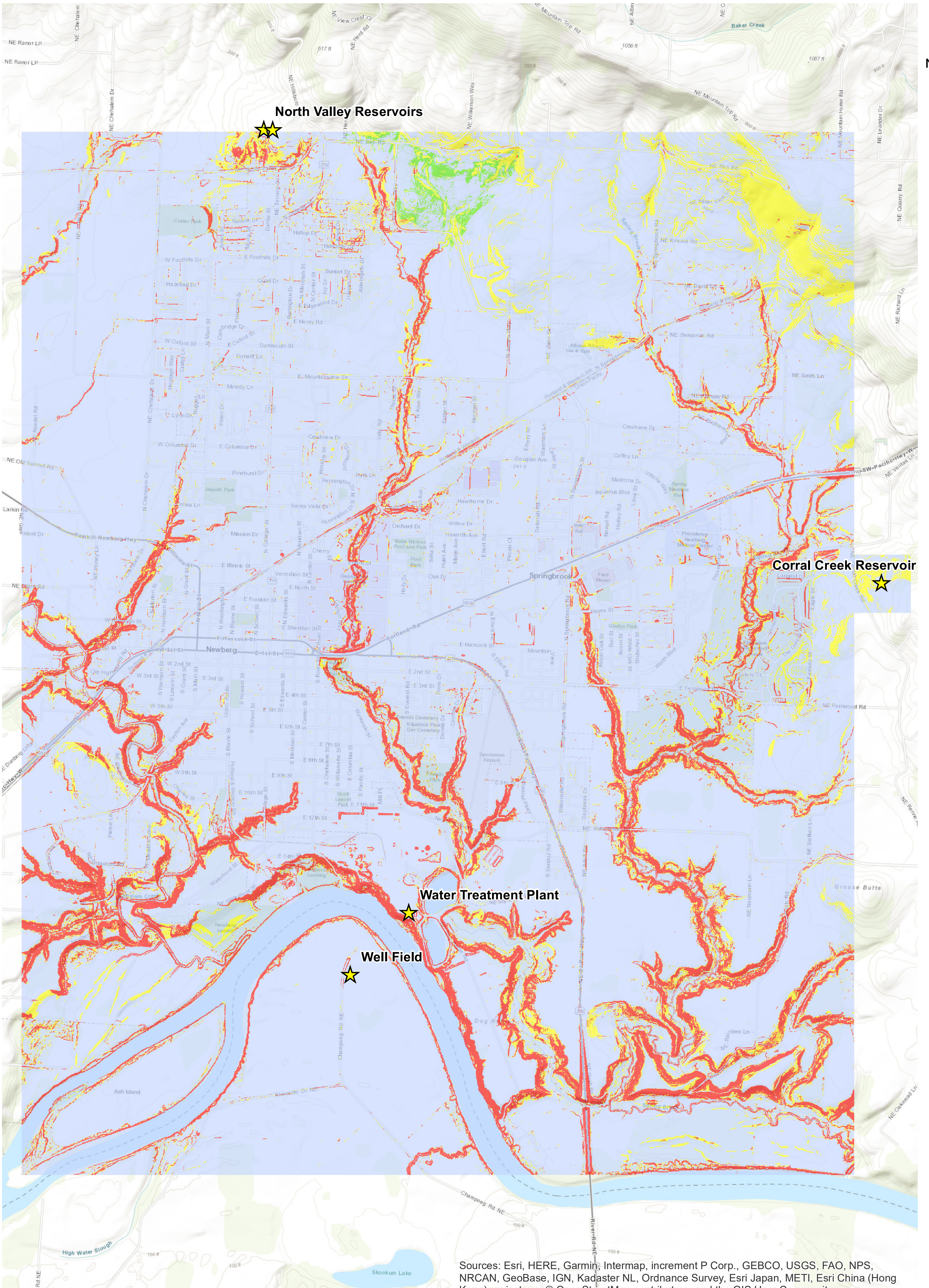
FIG. 13

FIG. 13

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\Landslide Probability (Wet).mxd Date: 9/11/2019 Login: DSJ



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ City Properties

Earthquake-Induced Landslide Probability in Wet Conditions (5)

(Proportion of Area Expected to Fail)

- 0
- 20
- 25
- 30

0 0.25 0.5 1

Miles

NOTES

1. Earthquake-induced landslide probability for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-3.4 and LiDAR. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

PROBABILITY OF EARTHQUAKE-INDUCED LANDSLIDES (WET)

July 2020 101895

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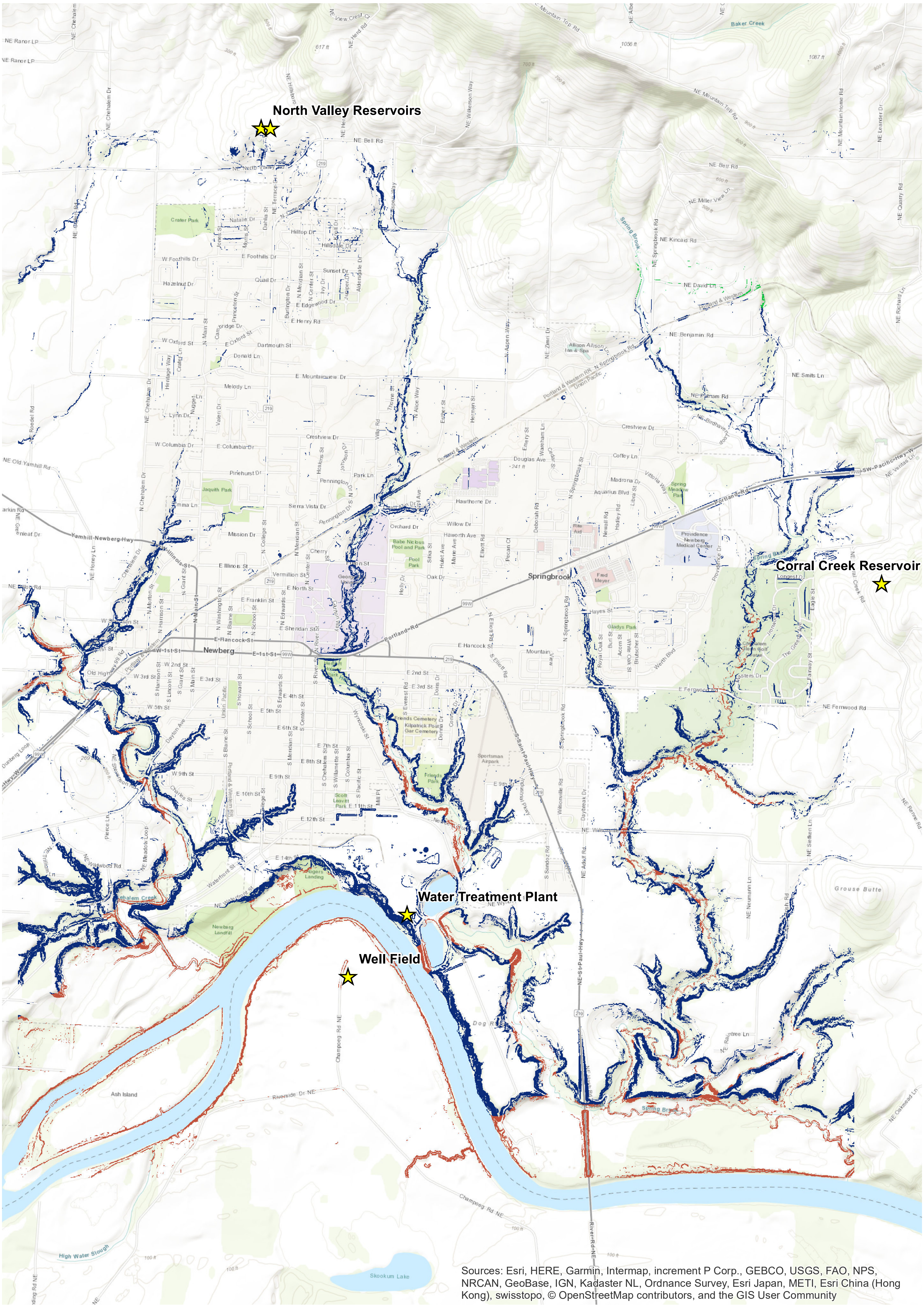
FIG. 14

FIG. 14

Filename: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\11x17 Files\Landslide PGD (Dry).mxd Date: 9/11/2019 Login: DSJ



LEGEND

Earthquake-Induced Landslide PGD (ft)

- Negligible
- < 2
- 2 - 3
- 2 - 4
- 4+

☆ City Properties

0 0.25 0.5 1
Miles

NOTES

1. Earthquake-induced landslide PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-3.4, and LiDAR. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

EARTHQUAKE-INDUCED LANDSLIDE PERMANENT GROUND DEFORMATION, PGD (DRY)

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FIG. 15

FIG. 15

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Earthquake-Induced Landslide PGD (ft)
(Wet Conditions)

- Negligible
- < 1
- 1 - 2
- 2 - 5
- 5 - 10
- 10 - 20
- 20+

★ City Properties

0 0.25 0.5 1
Miles

NOTES

1. Earthquake-induced landslide PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-3.4, and LiDAR. See text for details.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

EARTHQUAKE-INDUCED LANDSLIDE PERMANENT GROUND DEFORMATION, PGD (WET)

July 2020 101895

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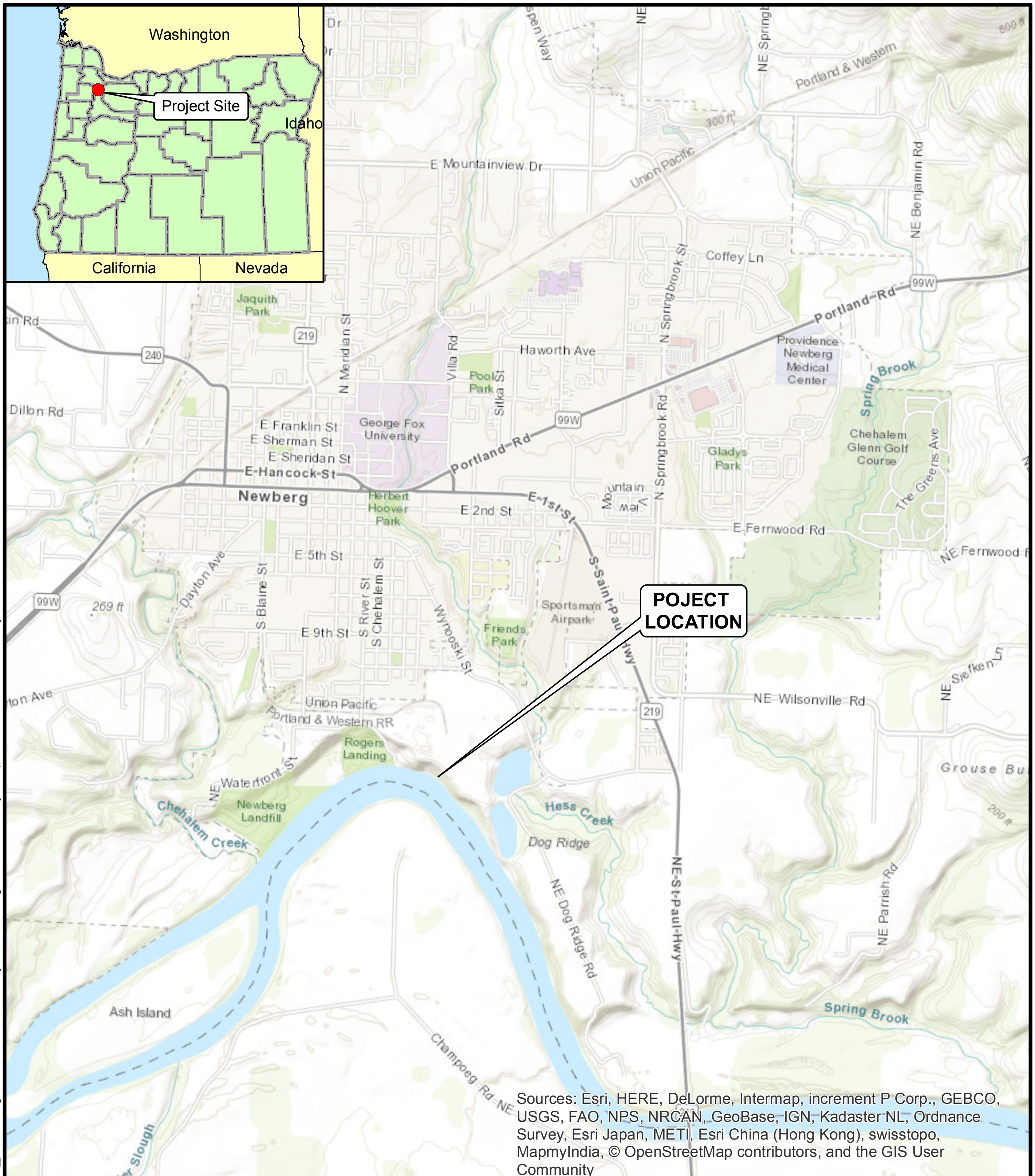
FIG. 16

FIG. 16



City of Newberg Water System Resilience
Yamhill County, Oregon

WATER TREATMENT PLANT VICINITY MAP

July 2020 101895

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Geotechnical and Environmental Consultants

FIG. 17

Path: T:\Projects\PDX\101000s\101895_Newberg Seismic Resiliency\Avmxd\Figure 1 - Vicinity Map.mxd 10/11/2018 kjw

LEGEND

B-1 Designation and Approximate Location of Boring

Approximate Location of Slope Stability Cross Section

0 25 50 100

Scale in Feet

Note: Not shown are the historical explorations for the slope repair performed by Squier Associates in 1999 and the slope evaluation performed by Northwest Geotech, Inc. in 2016

City of Newberg Water System Resilience
Yamhill County, Oregon

WATER TREATMENT PLANT SLOPE SITE AND EXPLORATION PLAN

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 18

APPENDIX A: FIELD EXPLORATIONS

Appendix A
Field Explorations

CONTENTS

Figures

Attachments

APPENDIX A: FIELD EXPLORATIONS

A.1 GENERAL

The field exploration program included two Cone Penetration Tests (CPTs) and two geoprobe explorations. The exploration locations were not surveyed but were referenced to nearby existing structures and should be considered approximate. Approximate CPT locations are shown on the Site and Exploration Plan, Figure 18. The CPTs and geoprobes were completed on May 20, 2019, by Oregon Geotechnical Explorations, Inc. (OGE), of Keizer, Oregon. This appendix describes general exploration methods and presents logs of the materials encountered.

A.1.1 Cone Penetration Testing

OGE pushed CPT-1 and CPT-2 using a track-mounted CPT rig, which uses helical anchors, drilled into the ground, to help the rig to push down with a force greater than its weight. CPT-1 and CPT-2 were advanced to depths of 83 and 68 feet, respectively.

During a CPT, a specialized cone assembly at the end of a steel probe is hydraulically pushed down through the subsurface. The cone assembly contains load cells and associated strain gauges which monitor the deformation of the load cells. One set of load cells deforms with increasing resistance to cone tip penetration. Another set of load cells deforms with increasing frictional resistance encountered on a sleeve on the outside of the assembly. The cone assembly also contains a piezometer which measures pore pressure. Data from the strain gauges and from the piezometer are transmitted from the cone assembly back through extension rods to a CPT recording device via a cable. Analysis software using industry standard calculations then converts the raw data signals from the instruments into cone resistance, sleeve friction, and pore pressure.

Pore pressure is useful in estimating soil behavior type because penetration has varying effects on pore pressure, depending on the type of material being penetrated. Dissipation of pore pressure can also be measured if the cone advance is temporarily halted. Pore pressure dissipation tests were performed at one depth in CPT-1 and can be used to estimate the static groundwater level and to estimate the soil hydraulic conductivity at the test location. Twenty-five shear wave velocity tests were performed in CPT-1.

A.1.2 CPT Logs

All raw CPT data was reduced by OGE into values of cone resistance, sleeve friction, and pore pressure. Shannon & Wilson prepared graphic plots of the reduced data, along with several interpreted engineering parameters. The plots are presented in Figures A1 and A2, and include cone resistance (q_t) in tons per square foot (tsf), sleeve friction (f_s) in tsf, friction ratio (f_s/q_t) expressed as a percentage, pore pressure in tsf, estimated soil behavior type (SBT), undrained shear strength in pounds per square foot (psf), and estimated SPT N-value (N_{60}) in blows per foot (bpf). Plots of the pore pressure dissipation tests, prepared by OGE, are enclosed at the end of this attachment.

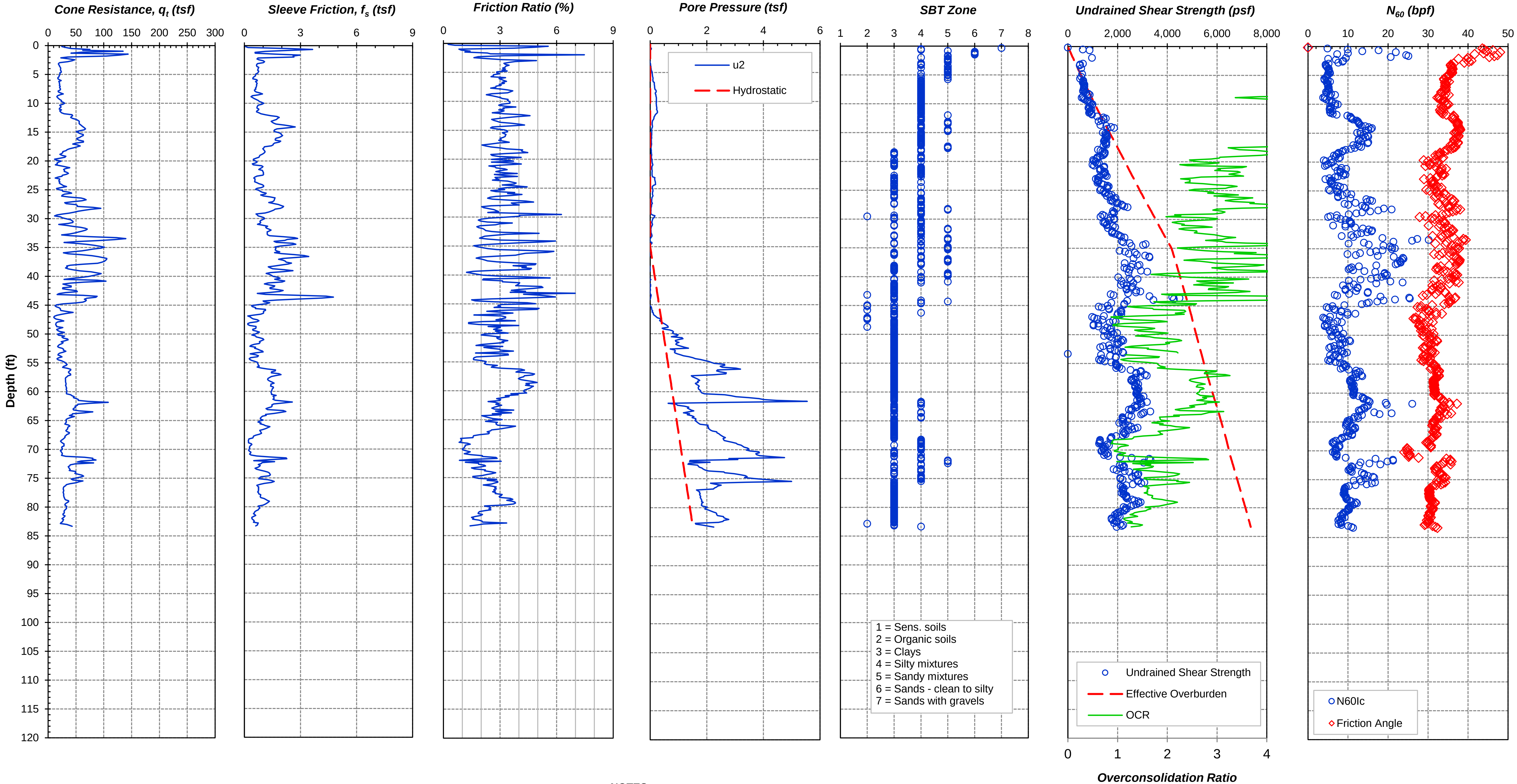
A.1.3 Geoprobe Explorations

Geoprobe explorations P-1 and P-2 were advanced to depths of 68 and 30 feet, respectively. Samples were not able to be recovered from approximately 10 to 40 feet during exploration P-1. Therefore, an additional geoprobe P-2 was performed to obtain samples from the zone that was not recovered from P-1.

The probes were advanced using a track-mounted Geoprobe™ drill rig capable of continuous push probe sampling. Soil sampling was performed using a track-mounted, direct push probe rig equipped with 2.5-inch-outside-diameter casing. Samples were collected by advancing casings lined with 4-foot plastic sleeves using percussive force to remove soils in their path.

A.1.4 Exploration Backfill

All holes were backfilled in accordance with Oregon Department of Ecology regulations. No wells or other instruments were installed in the holes. The holes were backfilled from the bottom up to the existing ground surface using bentonite chips.



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 170 ft.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

INTERPRETED CPT SOUNDING
CPT-1

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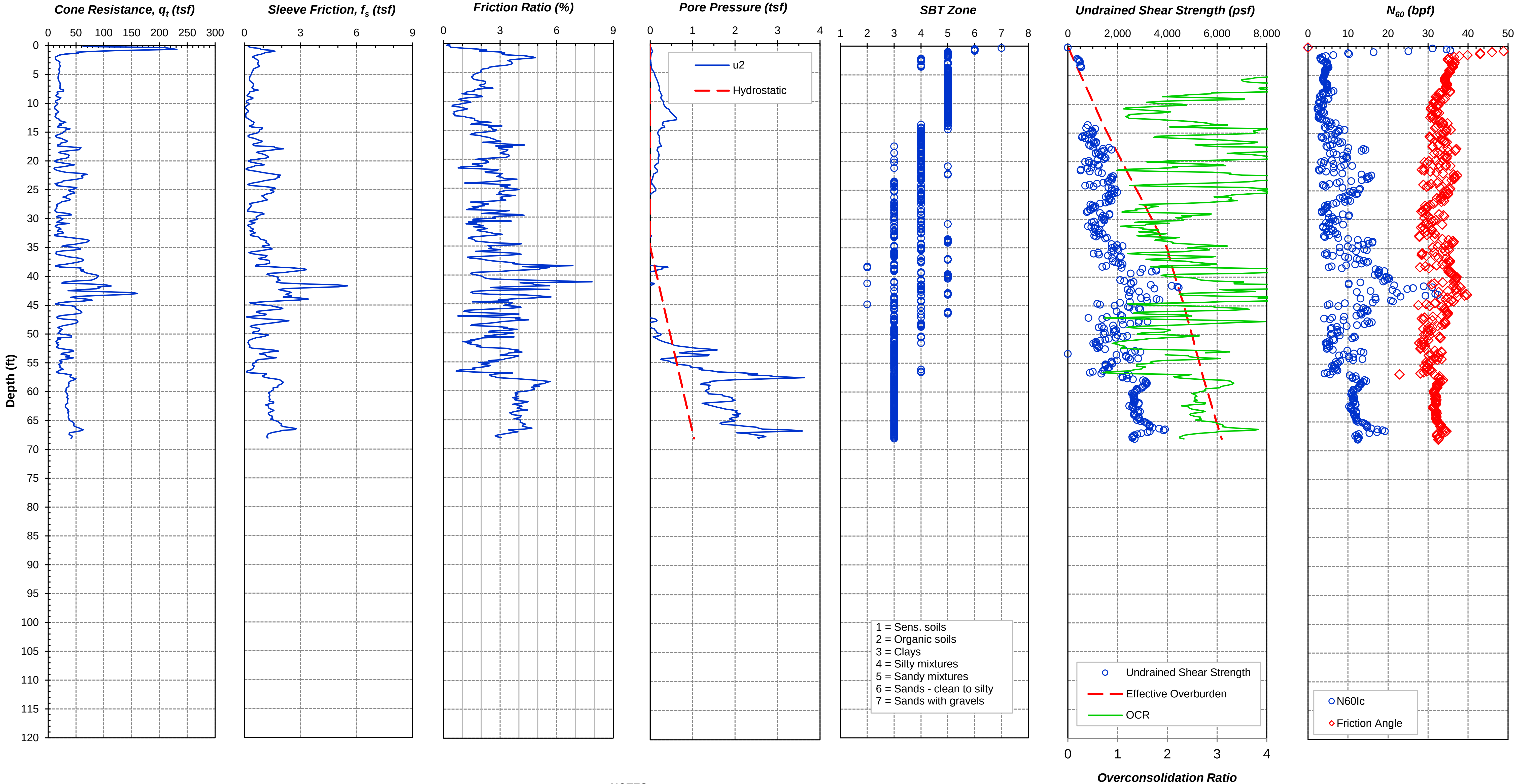
Geotechnical and Environmental Consultants

FIG. A1



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 170 ft.

City of Newberg Seismic Resiliency
Yamhill County, Oregon

INTERPRETED CPT SOUNDING
CPT-2

July 2020 101895

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FIG. A2

Total Depth:	68 ft.	Northing:	~	Drilling Method:	Direct Push
Top Elevation:	~	Easting:	~	Drilling Company:	Oregon Geotechnical
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 6622 Track Rig
Horiz. Datum:		Offset:	~	Other Comments:	

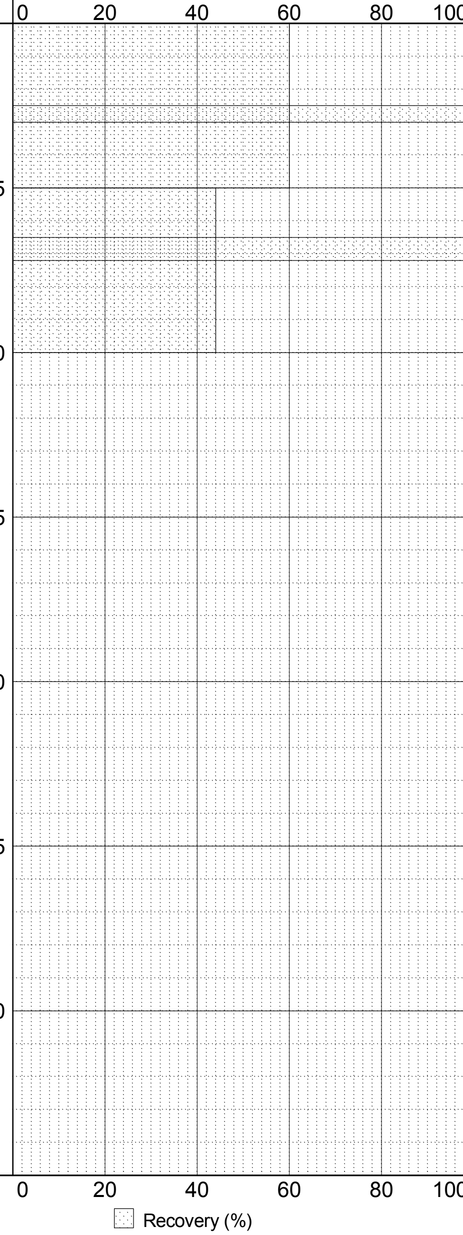
Hole Diam.: 2.5 in.
Rod Type: N/A
Hammer Type: N/A

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples	Ground Water	Depth, ft.
Brown and gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; nonplastic fines; trace organics and wood debris. **FILL**	1.0			
Red-brown rotten wood encountered from 1.0 to 1.2 feet	1.2	S-1 G		
Dark brown mottled orange-brown, *Silt with Sand (ML)*; moist; fine sand; low plasticity; trace rootlets and organics; disturbed texture.	3.5			5
		S-2 G		
Brown mottled orange-brown, *Silt (ML)*; moist; trace fine sand; low plasticity; micaceous; slight iron oxidation and staining. **FINE-GRAINED MISSOULA FLOOD DEPOSITS**		*		10, 15
		*		
Brown, *Silt with Sand (ML)*; moist; fine sand; nonplastic; micaceous; description based on material encountered in adjacent boring P-2. *Possible rock in probe tip preventing sample recovery, driller indicates it should clear*	18.2	*		20
Brown, *Sandy Silt (ML)*; moist; fine sand; nonplastic; micaceous; description based on material encountered in adjacent boring P-2.	24.5	*		25
Brown, *Silty Sand (SM)*; moist; fine sand; nonplastic fines; micaceous; occasional zones of slight iron oxidation with red-brown mottling; description based on material encountered in adjacent boring P-2.	28.0	*		30

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- 1" Plastic Sheath
- G Grab Sample

● % Water Content
Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

City of Newberg Seismic Resiliency Plan
Newberg, Oregon

LOG OF BORING P-1

June 2020 101895

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 1 of 2

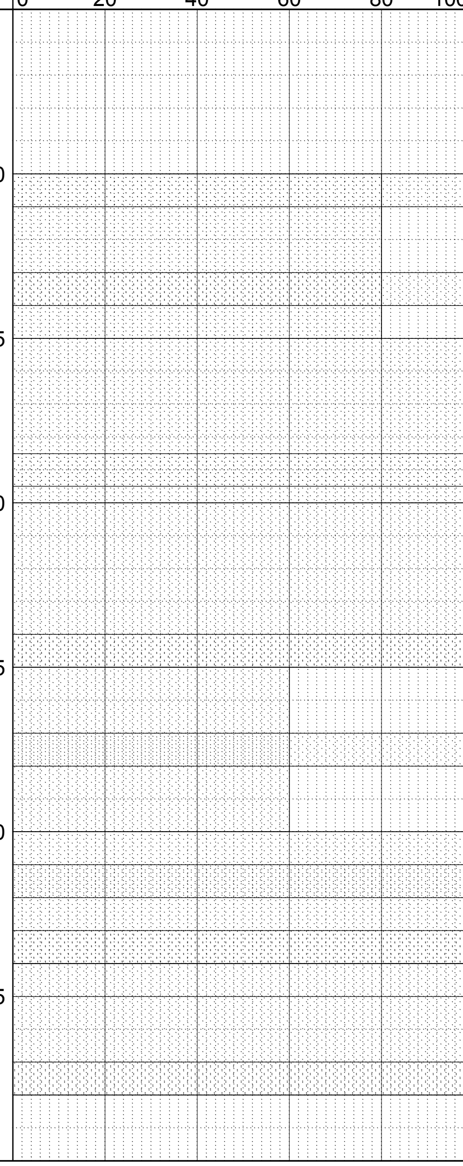
REV 2

MASTER_LOG_E 101895 GINT.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 7/9/19 Log: CKS Rev: Typ: CKS

Total Depth:	68 ft.	Northing:	~	Drilling Method:	Direct Push	Hole Diam.:	2.5 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Oregon Geotechnical	Rod Type:	N/A
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 6622 Track Rig	Hammer Type:	N/A
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Gray, *Silty Sand (SM)*; moist to wet; fine sand; nonplastic fines; trace wood fragments and wood debris; micaceous. *Driller to pull rods to clear obstruction in probe tip* **FINE-GRAINED MISSOULA FLOOD DEPOSITS**	35.0		* S-3		40
Gray, *Silt (ML)*; wet; trace fine sand; nonplastic to low plasticity; micaceous; stratified with trace to few interbeds of *Silty Sand (SM)* and low to medium plasticity *Silt (ML)*. *Silty Sand (SM)* interbed from 46 to 46.5 feet	42.0		S-4 S-5		45 50
Gray, *Lean Clay (CL)*; moist to wet; trace fine sand; medium plasticity; micaceous.	53.0		S-6		55
Gray, *Fat Clay (CH)*; moist; trace fine sand; high plasticity.	56.5		S-7		
Green-gray mottled orange-brown, *Fat Clay (CH)*; moist; trace fine sand; high plasticity; stratified with few interbeds of relict, decomposed, fine to coarse sand; few hard, fine to coarse sand-sized iron oxide deposits; slight to moderate iron oxidation and staining. **HILLSBORO FORMATION**	60.0		S-8 S-9 S-10		60 65
Completed: May 20, 2019	68.0				

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



● % Water Content
Plastic Limit ├──┤ Liquid Limit

LEGEND
- * Sample Not Recovered
- 1" Plastic Sheath
- Grab Sample

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

City of Newberg Seismic Resiliency Plan
Newberg, Oregon

LOG OF BORING P-1

June 2020 101895

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 2 of 2

REV 2

MASTER_LOG_E 101895 GINT.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 7/9/19 Log: CKS Rev: Typ: CKS

Total Depth:	30 ft.	Northing:	~	Drilling Method:	Direct Push	Hole Diam.:	2.5 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Oregon Geotechnical	Rod Type:	N/A
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 6622 Track Rig	Hammer Type:	N/A
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Depth, ft.
Dark brown, *Silty Gravel with Sand (GM)*; moist; fine to coarse, angular to subangular gravel; fine to coarse sand; nonplastic to low plasticity fines; trace organics and wood debris. **FILL**	1.5	S-1	
Brown mottled orange-brown, *Silt with Sand (ML)*; moist; fine sand; nonplastic; trace roots; micaceous; disturbed texture.	3.5	S-2	5
Brown and tan-brown mottled orange-brown, *Silt (ML)*; moist; trace fine sand; nonplastic; trace fine organics; micaceous; stratified with trace to few interbeds of *Sandy Silt (ML)*; slight iron oxidation and staining. **FINE-GRAINED MISSOULA FLOOD DEPOSITS**		S-3	10, 15
Brown, *Silt with Sand (ML)*; moist; fine sand; nonplastic; micaceous.	18.2	S-4	20
Brown, *Sandy Silt (ML)*; moist; fine sand; nonplastic; micaceous.	24.5	S-5	25
Brown, *Silty Sand (SM)*; moist; fine sand; nonplastic fines; micaceous; occasional zones of slight iron oxidation with red-brown mottling.	28.0	S-6	30
Completed: May 20, 2019	30.0		

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

LEGEND

2" Plastic Sheath

Grab Sample

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

City of Newberg Seismic Resiliency Plan
Newberg, Oregon

LOG OF BORING P-2

June 2020 101895

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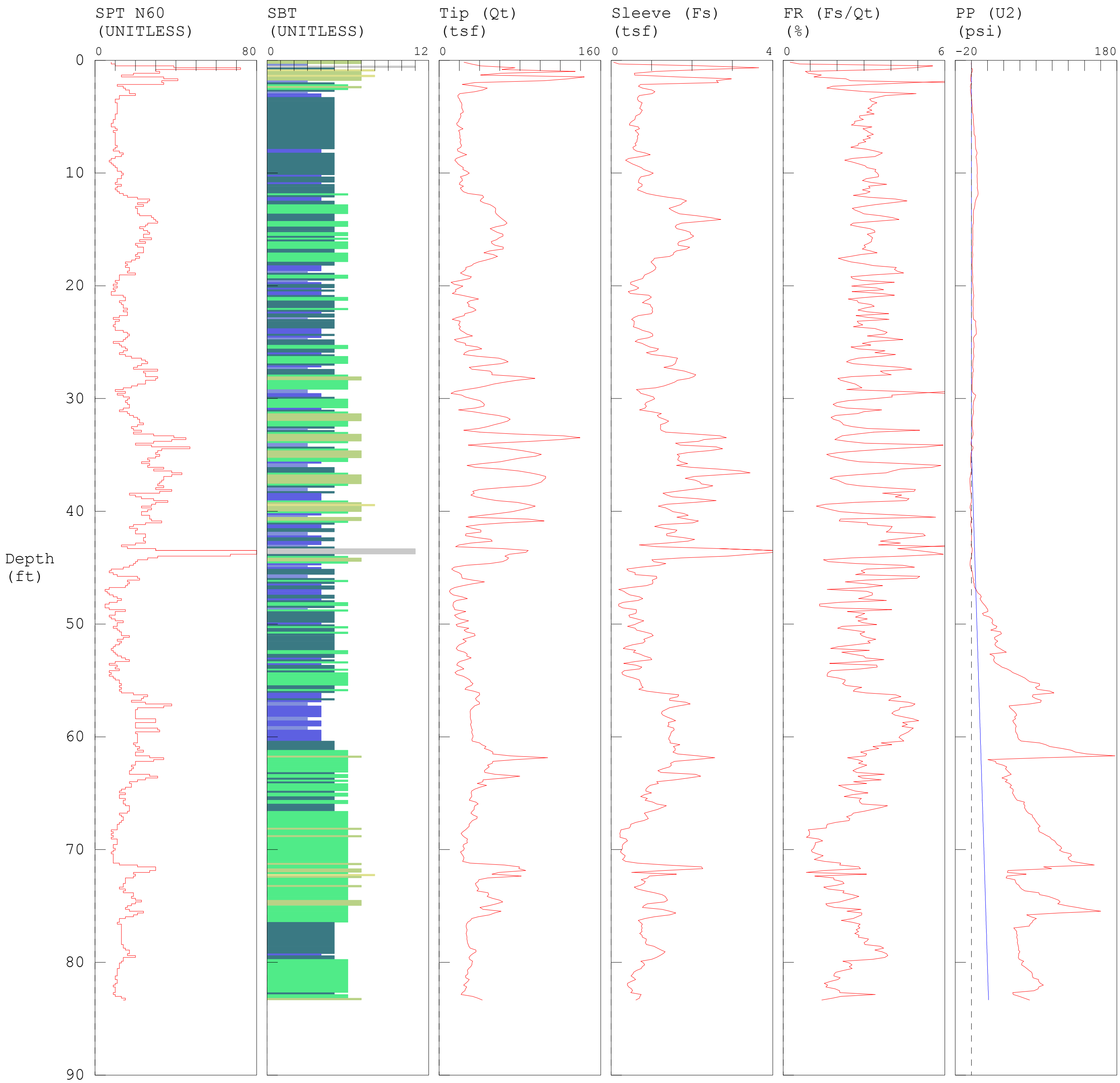
FIG. A4

REV 2

Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg

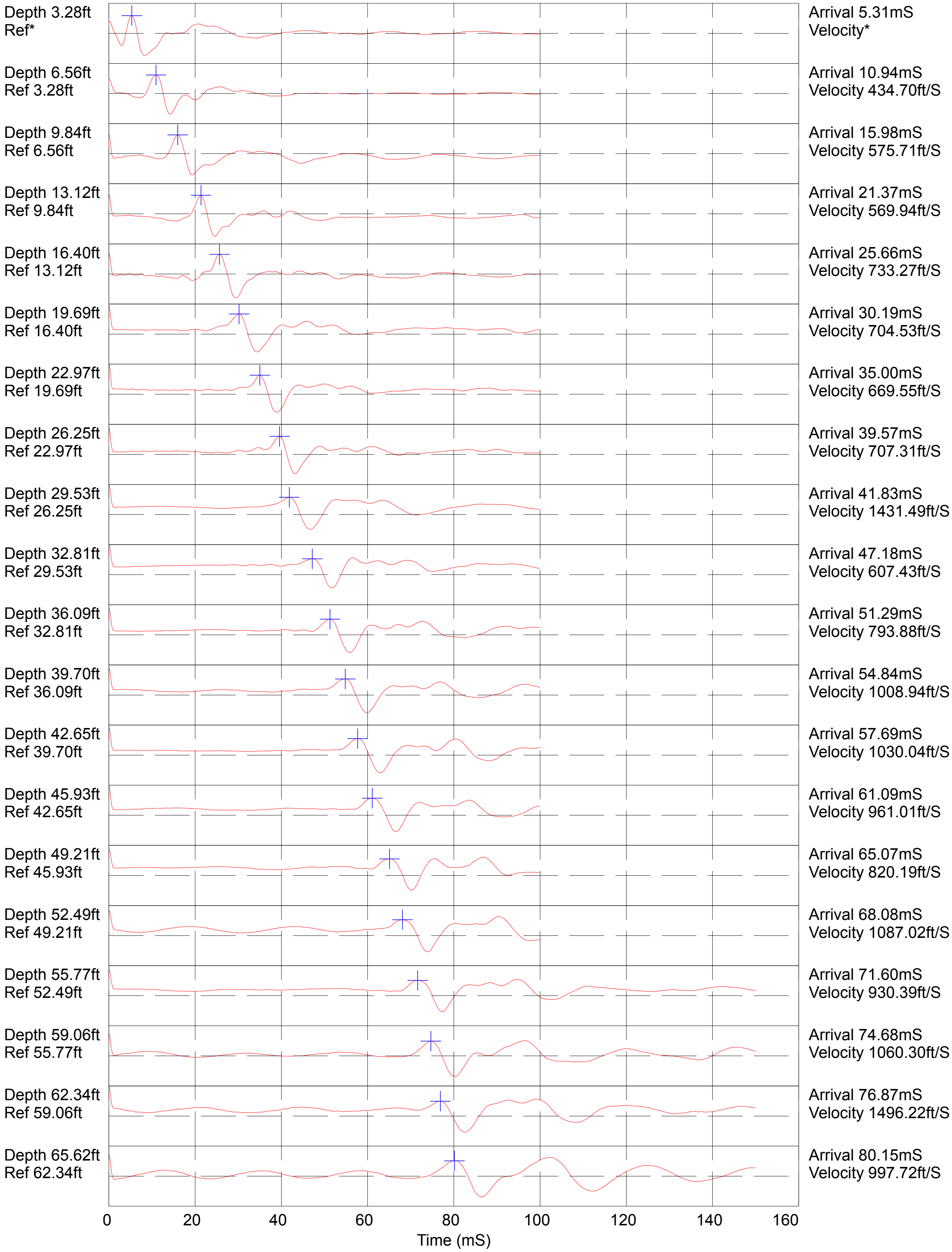
OPERATOR: OGE DMM
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 5/20/2019 8:53:04 AM
TOTAL DEPTH: 83.333 ft



1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy sil	10	gravelly sand to sand
2	organic material	5	clayey silt to silty cl	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey si	9	sand	12	sand to clayey sand (*)

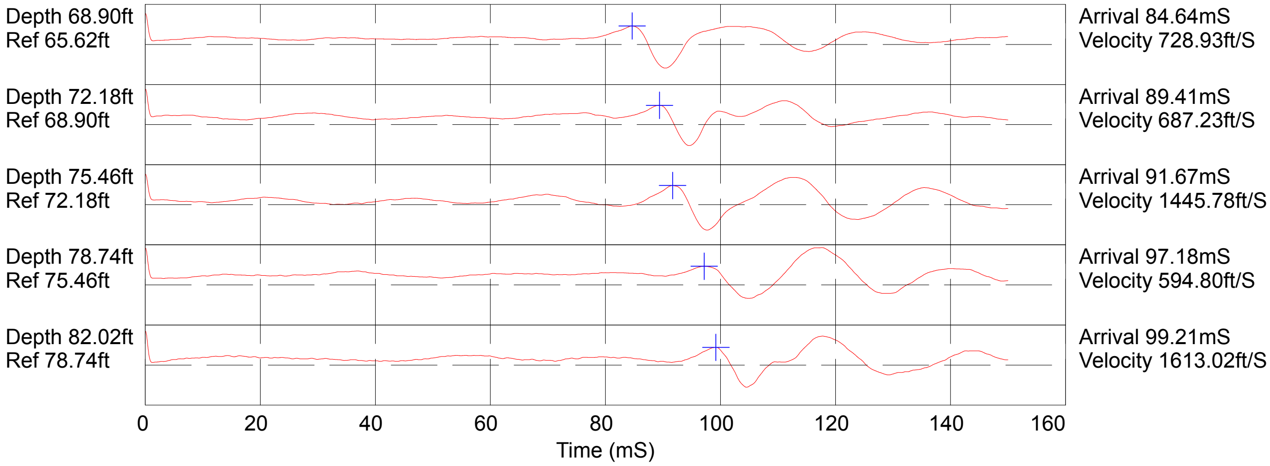
*SBT/SPT CORRELATION: UBC-1983

COMMENT: Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

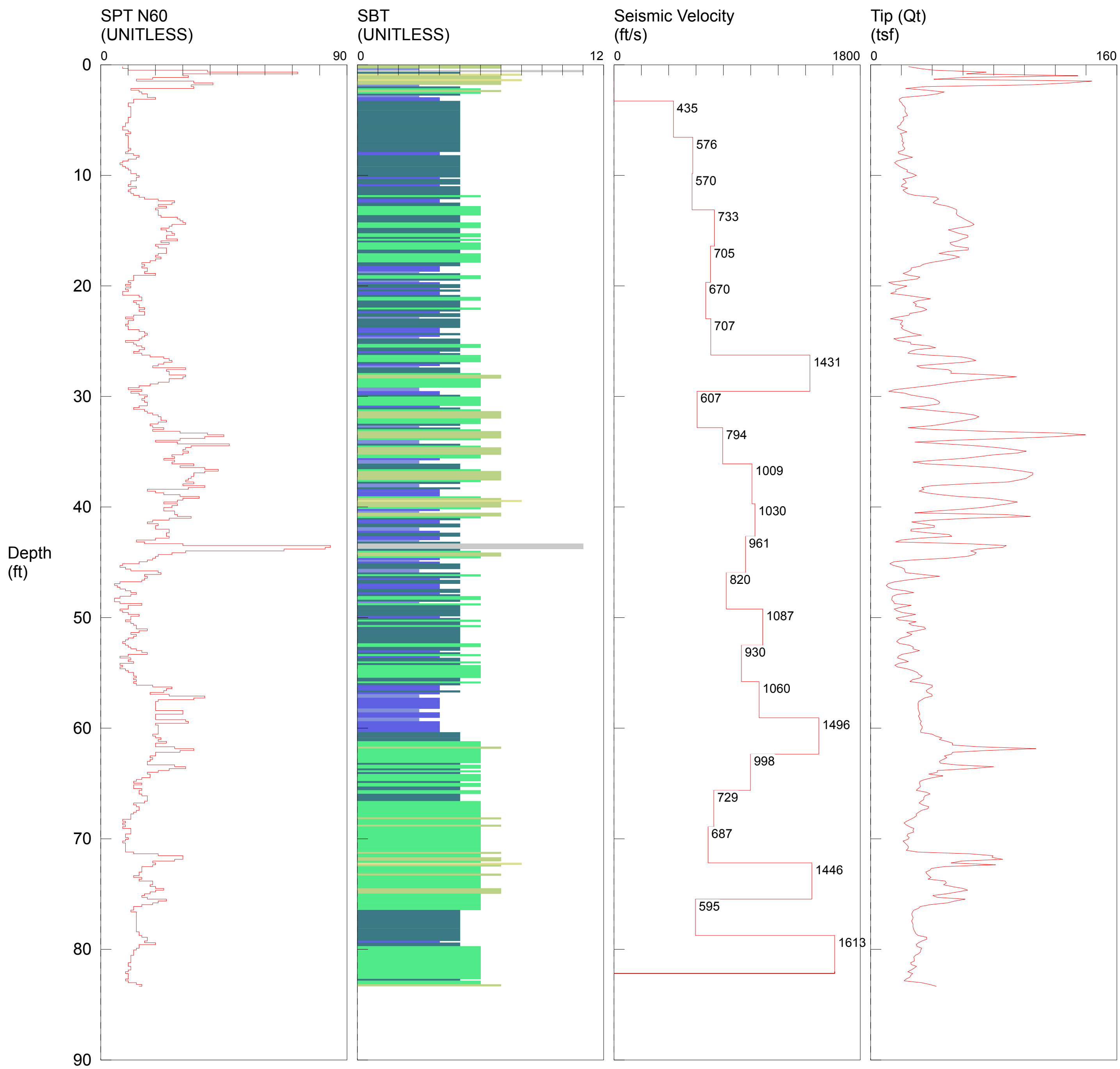
COMMENT: Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg

OPERATOR: OGE DMM
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 5/20/2019 8:53:04 AM
TOTAL DEPTH: 83.333 ft

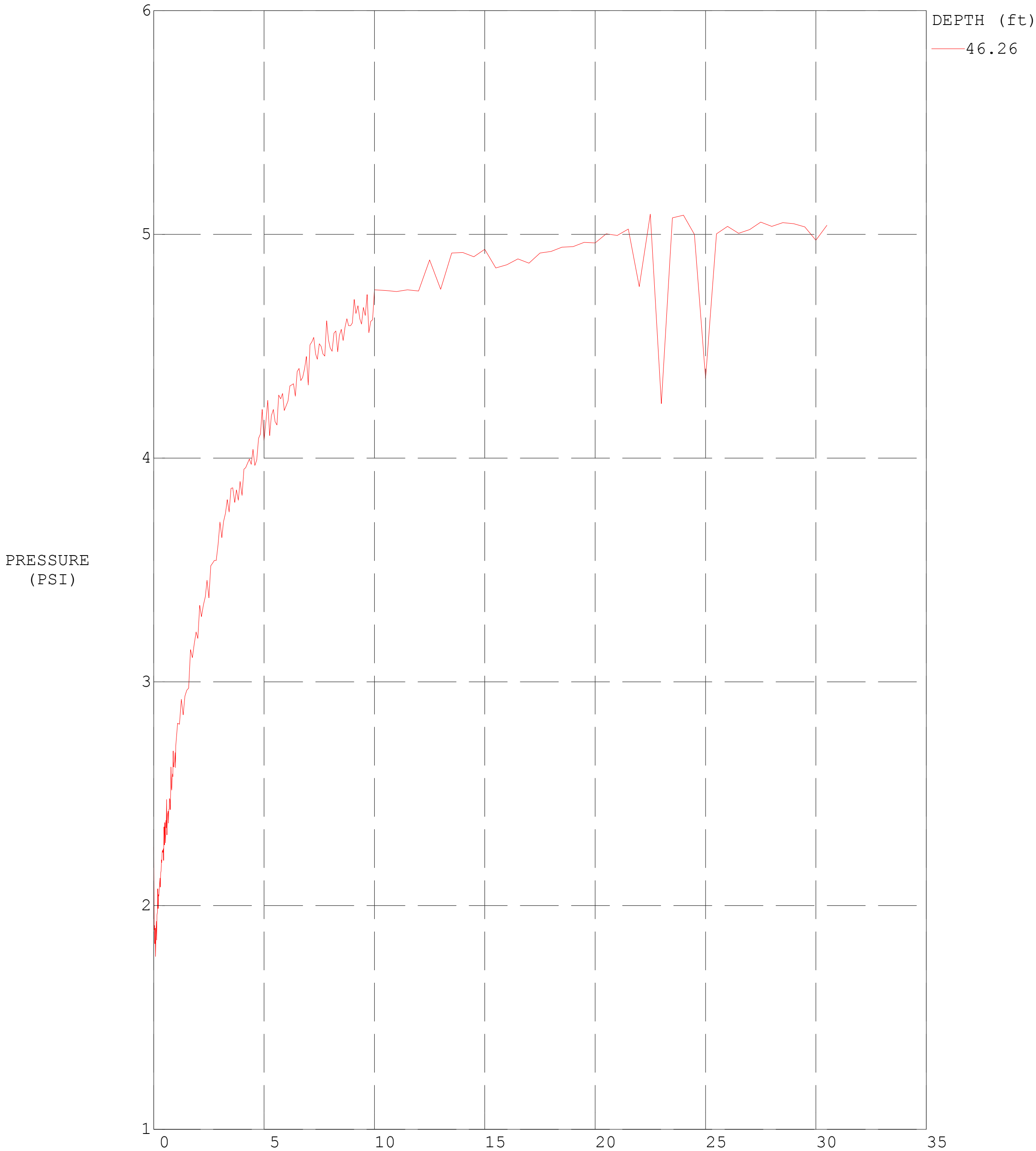


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg

TEST DATE: 5/20/2019 8:53:04 AM



MAXIMUM PRESSURE = 5.091 (PSI)IME: (MINUTES)

HYDROSTATIC PRESSURE = 5.153 (PSI), WATER TABLE: 34.37 ft

Shannon & Wilson / CPT-1 / 1400 Wynooski St Newberg

OPERATOR: OGE DMM
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 5/20/2019 8:53:04 AM
TOTAL DEPTH: 83.333 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
0.164	24.60	0.0622	0.253	-0.062	8	7	silty sand to sandy silt
0.328	31.82	0.1930	0.607	-0.227	10	7	silty sand to sandy silt
0.492	41.08	2.2819	5.554	0.041	39	3	clay
0.656	75.02	3.6534	4.870	-0.017	72	11	very stiff fine grained (*)
0.820	62.25	2.3299	3.743	1.319	30	5	clayey silt to silty clay
0.984	134.71	1.1113	0.825	1.109	32	8	sand to silty sand
1.148	61.01	0.5736	0.940	-0.083	19	7	silty sand to sandy silt
1.312	41.08	0.5810	1.414	-0.513	13	7	silty sand to sandy silt
1.476	143.70	1.6389	1.140	-0.766	34	8	sand to silty sand
1.640	128.15	2.9762	2.322	-0.907	41	7	silty sand to sandy silt
1.804	104.20	2.6030	2.498	-1.076	33	7	silty sand to sandy silt
1.969	35.50	2.6523	7.471	0.172	34	3	clay
2.133	22.80	0.7309	3.206	-0.864	11	5	clayey silt to silty clay
2.297	36.56	0.6723	1.839	-1.295	14	6	sandy silt to clayey silt
2.461	47.71	0.7708	1.615	-1.033	15	7	silty sand to sandy silt
2.625	43.23	1.0040	2.322	-1.279	17	6	sandy silt to clayey silt
2.789	35.10	1.0828	3.085	-1.143	17	5	clayey silt to silty clay
2.953	20.56	1.0142	4.933	-0.678	20	3	clay
3.117	18.45	0.7020	3.805	-0.370	12	4	silty clay to clay
3.281	18.81	0.6579	3.498	-0.229	12	4	silty clay to clay
3.445	20.24	0.6489	3.206	0.303	10	5	clayey silt to silty clay
3.609	20.72	0.6783	3.273	0.444	10	5	clayey silt to silty clay
3.773	22.04	0.7664	3.477	0.520	11	5	clayey silt to silty clay
3.937	22.58	0.7460	3.304	0.768	11	5	clayey silt to silty clay
4.101	22.23	0.7273	3.271	0.844	11	5	clayey silt to silty clay
4.265	22.46	0.7055	3.141	1.011	11	5	clayey silt to silty clay
4.429	23.39	0.7696	3.290	1.090	11	5	clayey silt to silty clay
4.593	23.91	0.7414	3.100	1.176	11	5	clayey silt to silty clay
4.757	20.97	0.7089	3.381	1.939	10	5	clayey silt to silty clay
4.921	21.57	0.6108	2.832	2.142	10	5	clayey silt to silty clay
5.085	20.99	0.5954	2.836	2.090	10	5	clayey silt to silty clay
5.249	19.68	0.5855	2.976	2.374	9	5	clayey silt to silty clay
5.413	19.40	0.5142	2.650	2.502	9	5	clayey silt to silty clay
5.577	17.34	0.4606	2.656	2.634	8	5	clayey silt to silty clay
5.741	17.74	0.4483	2.528	2.846	8	5	clayey silt to silty clay
5.906	20.34	0.6701	3.294	3.120	10	5	clayey silt to silty clay
6.070	23.53	0.6957	2.957	3.178	11	5	clayey silt to silty clay
6.234	19.24	0.5943	3.089	2.996	9	5	clayey silt to silty clay
6.398	20.20	0.6450	3.193	3.204	10	5	clayey silt to silty clay
6.562	20.36	0.6811	3.345	3.342	10	5	clayey silt to silty clay
6.726	20.76	0.6454	3.108	4.635	10	5	clayey silt to silty clay
6.890	20.32	0.6577	3.237	4.564	10	5	clayey silt to silty clay
7.054	21.59	0.6393	2.961	4.735	10	5	clayey silt to silty clay
7.218	20.71	0.6350	3.067	4.840	10	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
7.382	21.13	0.5897	2.791	4.921	10	5	clayey silt to silty clay
7.546	22.20	0.6171	2.780	5.041	11	5	clayey silt to silty clay
7.710	21.38	0.5368	2.511	5.189	10	5	clayey silt to silty clay
7.874	17.92	0.5527	3.084	5.403	9	5	clayey silt to silty clay
8.038	18.40	0.6279	3.412	6.429	12	4	silty clay to clay
8.202	22.65	0.8339	3.681	6.913	14	4	silty clay to clay
8.366	27.18	0.9667	3.557	6.599	13	5	clayey silt to silty clay
8.530	21.65	0.7373	3.406	5.666	10	5	clayey silt to silty clay
8.694	16.97	0.4347	2.562	5.911	8	5	clayey silt to silty clay
8.858	15.55	0.3548	2.282	6.508	7	5	clayey silt to silty clay
9.022	17.16	0.4414	2.573	6.508	8	5	clayey silt to silty clay
9.186	18.81	0.5287	2.812	6.823	9	5	clayey silt to silty clay
9.350	21.02	0.6379	3.035	7.002	10	5	clayey silt to silty clay
9.514	23.89	0.7437	3.113	7.009	11	5	clayey silt to silty clay
9.678	23.44	0.8060	3.439	7.042	11	5	clayey silt to silty clay
9.843	27.18	0.9241	3.400	7.040	13	5	clayey silt to silty clay
10.007	29.74	1.0398	3.496	7.307	14	5	clayey silt to silty clay
10.171	26.71	0.9453	3.539	6.880	13	5	clayey silt to silty clay
10.335	20.89	0.7377	3.532	6.885	13	4	silty clay to clay
10.499	22.61	0.6975	3.085	7.052	11	5	clayey silt to silty clay
10.663	23.06	0.6785	2.942	7.033	11	5	clayey silt to silty clay
10.827	20.70	0.6746	3.259	7.135	10	5	clayey silt to silty clay
10.991	19.94	0.7672	3.848	7.474	13	4	silty clay to clay
11.155	24.02	0.7609	3.168	7.493	11	5	clayey silt to silty clay
11.319	20.98	0.6767	3.226	7.190	10	5	clayey silt to silty clay
11.483	22.16	0.6516	2.940	7.727	11	5	clayey silt to silty clay
11.647	24.04	0.7749	3.224	8.113	12	5	clayey silt to silty clay
11.811	28.76	0.8801	3.060	8.242	14	5	clayey silt to silty clay
11.975	42.82	1.1127	2.598	7.970	16	6	sandy silt to clayey silt
12.139	44.11	1.4028	3.180	6.737	21	5	clayey silt to silty clay
12.303	41.80	1.7399	4.163	5.725	27	4	silty clay to clay
12.467	40.59	1.8652	4.596	5.103	26	4	silty clay to clay
12.631	44.08	1.7726	4.021	4.514	21	5	clayey silt to silty clay
12.795	50.15	1.7265	3.443	4.335	24	5	clayey silt to silty clay
12.959	53.51	1.5832	2.958	4.060	20	6	sandy silt to clayey silt
13.123	55.95	1.4325	2.560	3.631	21	6	sandy silt to clayey silt
13.287	56.01	1.4940	2.668	2.643	21	6	sandy silt to clayey silt
13.451	55.56	1.5843	2.851	2.467	21	6	sandy silt to clayey silt
13.615	56.44	1.6915	2.997	2.331	22	6	sandy silt to clayey silt
13.780	58.70	2.1163	3.605	2.307	28	5	clayey silt to silty clay
13.944	60.71	2.4573	4.048	2.247	29	5	clayey silt to silty clay
14.108	63.16	2.7152	4.299	2.538	30	5	clayey silt to silty clay
14.272	65.49	2.2713	3.468	2.586	31	5	clayey silt to silty clay
14.436	67.25	1.7454	2.595	2.450	26	6	sandy silt to clayey silt
14.600	64.66	1.6142	2.497	1.823	25	6	sandy silt to clayey silt
14.764	56.27	1.5834	2.814	1.699	22	6	sandy silt to clayey silt
14.928	50.51	1.6273	3.222	1.443	24	5	clayey silt to silty clay
15.092	53.95	1.8281	3.388	1.691	26	5	clayey silt to silty clay
15.256	57.40	1.9561	3.408	1.694	27	5	clayey silt to silty clay
15.420	63.06	1.9666	3.118	1.761	24	6	sandy silt to clayey silt
15.584	63.17	2.0417	3.232	1.656	24	6	sandy silt to clayey silt
15.748	59.42	1.9729	3.320	1.694	28	5	clayey silt to silty clay
15.912	56.78	1.8623	3.280	1.629	22	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
16.076	51.58	1.6901	3.277	1.522	25	5	clayey silt to silty clay
16.240	52.19	1.6505	3.163	1.653	20	6	sandy silt to clayey silt
16.404	55.21	1.6575	3.002	1.622	21	6	sandy silt to clayey silt
16.568	63.47	1.9421	3.060	1.226	24	6	sandy silt to clayey silt
16.732	63.41	1.8617	2.936	1.272	24	6	sandy silt to clayey silt
16.896	50.04	1.6180	3.233	1.293	24	5	clayey silt to silty clay
17.060	44.34	1.5462	3.487	1.338	21	5	clayey silt to silty clay
17.224	53.33	1.5380	2.884	1.572	20	6	sandy silt to clayey silt
17.388	57.76	1.4418	2.496	1.462	22	6	sandy silt to clayey silt
17.552	52.51	1.0729	2.043	1.291	20	6	sandy silt to clayey silt
17.717	46.38	1.0428	2.248	1.152	18	6	sandy silt to clayey silt
17.881	38.98	0.9845	2.525	0.949	15	6	sandy silt to clayey silt
18.045	34.20	1.0301	3.012	1.040	16	5	clayey silt to silty clay
18.209	32.15	1.0746	3.342	1.042	15	5	clayey silt to silty clay
18.373	26.40	1.1092	4.202	1.331	17	4	silty clay to clay
18.537	26.39	1.0903	4.131	1.241	17	4	silty clay to clay
18.701	24.38	1.0311	4.229	1.283	16	4	silty clay to clay
18.865	21.09	0.9431	4.472	1.367	20	3	clay
19.029	24.99	0.8323	3.331	1.558	12	5	clayey silt to silty clay
19.193	31.91	0.8014	2.512	1.741	12	6	sandy silt to clayey silt
19.357	30.19	0.7788	2.580	1.470	12	6	sandy silt to clayey silt
19.521	21.43	0.6272	2.927	1.353	10	5	clayey silt to silty clay
19.685	11.70	0.4823	4.124	1.307	11	3	clay
19.849	14.84	0.4800	3.235	1.813	9	4	silty clay to clay
20.013	23.44	0.5968	2.545	1.930	11	5	clayey silt to silty clay
20.177	19.75	0.6841	3.463	2.016	9	5	clayey silt to silty clay
20.341	16.28	0.6064	3.724	2.042	10	4	silty clay to clay
20.505	16.48	0.4210	2.554	2.307	8	5	clayey silt to silty clay
20.669	13.07	0.4769	3.648	2.505	8	4	silty clay to clay
20.833	22.43	0.9265	4.131	2.987	14	4	silty clay to clay
20.997	31.09	0.9946	3.200	2.834	15	5	clayey silt to silty clay
21.161	38.92	0.9388	2.412	2.164	15	6	sandy silt to clayey silt
21.325	30.77	0.8459	2.749	1.997	12	6	sandy silt to clayey silt
21.490	27.72	0.7732	2.790	1.997	13	5	clayey silt to silty clay
21.654	29.69	0.9653	3.252	1.987	14	5	clayey silt to silty clay
21.818	29.04	0.9813	3.379	2.068	14	5	clayey silt to silty clay
21.982	34.01	1.0231	3.008	2.240	16	5	clayey silt to silty clay
22.146	36.43	1.0117	2.777	2.142	14	6	sandy silt to clayey silt
22.310	32.87	1.0274	3.125	2.056	16	5	clayey silt to silty clay
22.474	25.43	0.9953	3.914	2.080	16	4	silty clay to clay
22.638	24.99	0.6760	2.705	2.142	12	5	clayey silt to silty clay
22.802	17.86	0.5661	3.170	2.333	9	5	clayey silt to silty clay
22.966	12.83	0.5039	3.928	2.696	12	3	clay
23.130	20.78	0.5702	2.744	5.327	10	5	clayey silt to silty clay
23.294	20.25	0.5671	2.800	5.096	10	5	clayey silt to silty clay
23.458	19.18	0.5660	2.952	5.177	9	5	clayey silt to silty clay
23.622	21.02	0.5469	2.602	5.437	10	5	clayey silt to silty clay
23.786	19.87	0.6769	3.406	5.740	10	5	clayey silt to silty clay
23.950	21.39	0.7768	3.631	5.942	14	4	silty clay to clay
24.114	24.70	0.9534	3.859	6.000	16	4	silty clay to clay
24.278	27.27	1.0255	3.761	6.100	17	4	silty clay to clay
24.442	33.01	1.0320	3.126	4.003	16	5	clayey silt to silty clay
24.606	23.56	0.9135	3.877	2.579	15	4	silty clay to clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
24.770	15.12	0.6720	4.443	2.262	14	3	clay
24.934	17.85	0.5940	3.327	2.724	9	5	clayey silt to silty clay
25.098	25.99	0.7628	2.935	3.008	12	5	clayey silt to silty clay
25.262	26.13	0.8502	3.254	3.065	13	5	clayey silt to silty clay
25.427	38.00	0.9549	2.513	3.149	15	6	sandy silt to clayey silt
25.591	42.16	1.1705	2.777	2.686	16	6	sandy silt to clayey silt
25.755	29.57	1.1182	3.781	2.550	14	5	clayey silt to silty clay
25.919	25.58	0.8702	3.402	2.486	12	5	clayey silt to silty clay
26.083	24.24	1.0096	4.166	2.801	15	4	silty clay to clay
26.247	37.82	1.3721	3.628	3.099	18	5	clayey silt to silty clay
26.411	58.97	1.6422	2.785	2.712	23	6	sandy silt to clayey silt
26.575	64.95	1.6129	2.483	1.956	25	6	sandy silt to clayey silt
26.739	68.38	1.6044	2.346	1.889	26	6	sandy silt to clayey silt
26.903	60.84	1.5816	2.600	1.665	23	6	sandy silt to clayey silt
27.067	43.77	1.5107	3.451	1.689	21	5	clayey silt to silty clay
27.231	29.87	1.3056	4.372	1.546	19	4	silty clay to clay
27.395	32.52	1.5536	4.777	2.133	31	3	clay
27.559	50.27	1.8122	3.605	2.531	24	5	clayey silt to silty clay
27.723	52.41	1.9482	3.718	1.520	25	5	clayey silt to silty clay
27.887	52.15	2.0945	4.016	1.582	25	5	clayey silt to silty clay
28.051	80.75	2.0431	2.530	1.491	31	6	sandy silt to clayey silt
28.215	94.79	1.9232	2.029	1.255	30	7	silty sand to sandy silt
28.379	76.95	1.6433	2.136	1.033	25	7	silty sand to sandy silt
28.543	65.02	1.5683	2.412	0.878	25	6	sandy silt to clayey silt
28.707	54.05	1.4457	2.675	0.995	21	6	sandy silt to clayey silt
28.871	46.74	1.3080	2.798	0.813	18	6	sandy silt to clayey silt
29.035	37.73	1.1023	2.921	0.830	14	6	sandy silt to clayey silt
29.199	26.55	0.6245	2.353	0.818	10	6	sandy silt to clayey silt
29.364	15.83	0.6953	4.392	1.042	15	3	clay
29.528	11.71	0.7334	6.261	2.505	11	3	clay
29.692	22.19	0.8950	4.034	5.317	14	4	silty clay to clay
29.856	25.87	1.0467	4.047	4.838	17	4	silty clay to clay
30.020	32.83	1.0763	3.279	4.067	16	5	clayey silt to silty clay
30.184	40.43	0.9766	2.416	2.972	15	6	sandy silt to clayey silt
30.348	43.05	0.8651	2.010	2.557	16	6	sandy silt to clayey silt
30.512	44.90	0.8311	1.851	2.314	17	6	sandy silt to clayey silt
30.676	44.26	0.8723	1.971	2.047	17	6	sandy silt to clayey silt
30.840	35.80	0.8375	2.339	1.959	14	6	sandy silt to clayey silt
31.004	19.24	0.7027	3.652	2.185	12	4	silty clay to clay
31.168	33.30	0.9977	2.996	2.615	16	5	clayey silt to silty clay
31.332	44.91	1.2435	2.769	2.269	17	6	sandy silt to clayey silt
31.496	58.41	1.1391	1.950	1.746	19	7	silty sand to sandy silt
31.660	66.86	1.1863	1.774	1.214	21	7	silty sand to sandy silt
31.824	70.36	1.3122	1.865	0.949	22	7	silty sand to sandy silt
31.988	67.74	1.4215	2.098	0.868	22	7	silty sand to sandy silt
32.152	61.58	1.3438	2.182	0.854	24	6	sandy silt to clayey silt
32.316	54.44	1.2182	2.238	0.945	21	6	sandy silt to clayey silt
32.480	47.79	1.2267	2.567	0.811	18	6	sandy silt to clayey silt
32.644	38.93	1.2187	3.130	1.095	19	5	clayey silt to silty clay
32.808	24.36	1.2348	5.069	1.629	23	3	clay
32.972	40.25	1.3508	3.356	1.856	19	5	clayey silt to silty clay
33.136	75.54	2.0677	2.737	2.152	29	6	sandy silt to clayey silt
33.301	122.83	2.6863	2.187	1.751	39	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
33.465	139.48	2.8496	2.043	0.792	45	7	silty sand to sandy silt
33.629	119.94	2.2982	1.916	0.074	38	7	silty sand to sandy silt
33.793	91.38	2.1639	2.368	-0.219	29	7	silty sand to sandy silt
33.957	52.68	1.5924	3.023	-0.439	20	6	sandy silt to clayey silt
34.121	28.84	1.7126	5.937	-0.427	28	3	clay
34.285	48.78	2.5983	5.326	1.970	47	3	clay
34.449	68.50	2.7565	4.024	1.949	33	5	clayey silt to silty clay
34.613	82.08	2.4348	2.966	0.542	31	6	sandy silt to clayey silt
34.777	92.49	1.9056	2.060	-0.253	30	7	silty sand to sandy silt
34.941	101.12	1.6294	1.611	-0.685	32	7	silty sand to sandy silt
35.105	95.30	1.6938	1.777	-0.871	30	7	silty sand to sandy silt
35.269	80.33	1.6628	2.070	-1.150	26	7	silty sand to sandy silt
35.433	70.10	1.6484	2.351	-1.102	27	6	sandy silt to clayey silt
35.597	60.46	1.7217	2.848	-0.971	23	6	sandy silt to clayey silt
35.761	42.87	1.8923	4.414	-0.985	27	4	silty clay to clay
35.925	27.57	1.6144	5.856	-0.842	26	3	clay
36.089	35.86	1.9770	5.513	-0.494	34	3	clay
36.253	60.84	2.6494	4.355	0.396	29	5	clayey silt to silty clay
36.417	78.74	3.1976	4.061	0.119	38	5	clayey silt to silty clay
36.581	89.95	3.4395	3.824	-0.506	43	5	clayey silt to silty clay
36.745	98.65	2.7649	2.803	-0.942	38	6	sandy silt to clayey silt
36.909	105.48	2.0601	1.953	-1.202	34	7	silty sand to sandy silt
37.073	105.25	1.8285	1.737	-1.813	34	7	silty sand to sandy silt
37.238	103.02	2.0758	2.015	-1.861	33	7	silty sand to sandy silt
37.402	101.14	2.1933	2.169	-1.894	32	7	silty sand to sandy silt
37.566	95.81	2.2338	2.331	-1.687	31	7	silty sand to sandy silt
37.730	89.07	2.5177	2.826	-1.591	34	6	sandy silt to clayey silt
37.894	61.70	2.3272	3.772	-1.510	30	5	clayey silt to silty clay
38.058	39.95	1.9602	4.907	-1.388	38	3	clay
38.222	33.24	1.6147	4.858	-0.971	32	3	clay
38.386	34.70	1.2804	3.690	-0.389	17	5	clayey silt to silty clay
38.550	31.43	1.3809	4.394	-0.210	20	4	silty clay to clay
38.714	36.37	1.5129	4.160	-0.138	23	4	silty clay to clay
38.878	45.84	2.1410	4.671	-0.103	29	4	silty clay to clay
39.042	56.95	2.5994	4.564	-0.005	36	4	silty clay to clay
39.206	77.21	2.0706	2.682	-0.239	30	6	sandy silt to clayey silt
39.370	86.90	1.3783	1.586	-0.904	28	7	silty sand to sandy silt
39.534	95.21	1.1668	1.225	-1.570	23	8	sand to silty sand
39.698	89.27	1.3477	1.510	-1.727	28	7	silty sand to sandy silt
39.862	81.06	1.4491	1.788	-0.942	26	7	silty sand to sandy silt
40.026	72.43	1.5291	2.111	-0.835	23	7	silty sand to sandy silt
40.190	59.38	1.7874	3.010	-0.902	23	6	sandy silt to clayey silt
40.354	42.54	1.9077	4.485	-0.828	27	4	silty clay to clay
40.518	28.71	1.6242	5.658	-0.589	27	3	clay
40.682	88.33	1.8752	2.123	0.856	28	7	silty sand to sandy silt
40.846	103.89	2.1595	2.079	-0.439	33	7	silty sand to sandy silt
41.011	64.58	2.0212	3.130	-0.749	25	6	sandy silt to clayey silt
41.175	40.24	1.2596	3.130	-1.000	19	5	clayey silt to silty clay
41.339	26.57	1.0749	4.045	-0.615	17	4	silty clay to clay
41.503	33.12	1.3238	3.997	0.358	21	4	silty clay to clay
41.667	41.34	1.6260	3.934	0.482	20	5	clayey silt to silty clay
41.831	41.31	1.5810	3.827	0.456	20	5	clayey silt to silty clay
41.995	26.39	1.3696	5.190	-0.076	25	3	clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
42.159	26.07	1.3757	5.277	-0.088	25	3	clay
42.323	37.22	1.7734	4.765	0.079	24	4	silty clay to clay
42.487	51.04	2.0687	4.053	0.236	24	5	clayey silt to silty clay
42.651	52.59	1.8933	3.600	-0.157	25	5	clayey silt to silty clay
42.815	31.44	1.3683	4.352	-0.021	20	4	silty clay to clay
42.979	19.67	0.6953	3.535	-0.386	13	4	silty clay to clay
43.143	16.25	1.1342	6.980	0.265	16	3	clay
43.307	62.86	2.6705	4.248	1.023	30	5	clayey silt to silty clay
43.471	88.14	4.3772	4.966	1.038	84	11	very stiff fine grained (*)
43.635	85.77	4.7623	5.553	0.604	82	11	very stiff fine grained (*)
43.799	69.66	4.1405	5.944	0.394	67	11	very stiff fine grained (*)
43.963	65.06	2.8777	4.423	0.231	31	5	clayey silt to silty clay
44.127	68.67	1.7510	2.550	-0.468	26	6	sandy silt to clayey silt
44.291	67.16	1.0060	1.498	-1.071	21	7	silty sand to sandy silt
44.455	62.69	1.0552	1.683	-1.589	20	7	silty sand to sandy silt
44.619	50.22	1.3561	2.700	-1.777	19	6	sandy silt to clayey silt
44.783	26.59	1.1804	4.440	-1.703	17	4	silty clay to clay
44.948	14.98	0.7302	4.873	-1.202	14	3	clay
45.112	12.46	0.3855	3.095	-0.253	8	4	silty clay to clay
45.276	14.77	0.4274	2.894	0.577	7	5	clayey silt to silty clay
45.440	19.59	0.5934	3.030	1.272	9	5	clayey silt to silty clay
45.604	21.97	0.5897	2.683	1.496	11	5	clayey silt to silty clay
45.768	22.14	1.1226	5.071	1.730	21	3	clay
45.932	22.77	1.1297	4.962	1.982	22	3	clay
46.096	33.16	1.0166	3.066	2.872	16	5	clayey silt to silty clay
46.260	44.85	1.0325	2.302	2.135	17	6	sandy silt to clayey silt
46.424	33.23	1.0222	3.076	3.502	16	5	clayey silt to silty clay
46.588	23.70	0.8736	3.687	3.507	15	4	silty clay to clay
46.752	15.59	0.4744	3.042	4.158	7	5	clayey silt to silty clay
46.916	11.41	0.1857	1.628	5.644	5	5	clayey silt to silty clay
47.080	10.13	0.3142	3.102	6.837	6	4	silty clay to clay
47.244	11.33	0.3757	3.316	8.113	7	4	silty clay to clay
47.408	14.04	0.4648	3.310	11.038	9	4	silty clay to clay
47.572	22.60	0.6969	3.083	12.205	11	5	clayey silt to silty clay
47.736	27.96	0.7814	2.795	10.594	13	5	clayey silt to silty clay
47.900	17.30	0.6627	3.831	11.873	11	4	silty clay to clay
48.064	14.29	0.2979	2.085	13.359	7	5	clayey silt to silty clay
48.228	13.89	0.1869	1.346	15.282	5	6	sandy silt to clayey silt
48.392	13.64	0.1842	1.350	17.269	5	6	sandy silt to clayey silt
48.556	14.95	0.2856	1.910	18.796	7	5	clayey silt to silty clay
48.720	15.37	0.6200	4.033	20.189	15	3	clay
48.885	26.31	0.6264	2.381	20.325	10	6	sandy silt to clayey silt
49.049	20.05	0.6077	3.031	13.665	10	5	clayey silt to silty clay
49.213	15.26	0.4382	2.872	15.120	7	5	clayey silt to silty clay
49.377	17.00	0.4567	2.686	22.420	8	5	clayey silt to silty clay
49.541	20.51	0.6408	3.124	25.466	10	5	clayey silt to silty clay
49.705	29.13	0.8073	2.771	26.375	14	5	clayey silt to silty clay
49.869	23.09	0.6827	2.957	25.321	11	5	clayey silt to silty clay
50.033	16.65	0.5776	3.469	26.928	11	4	silty clay to clay
50.197	18.24	0.4015	2.201	31.585	9	5	clayey silt to silty clay
50.361	29.48	0.5944	2.017	30.294	11	6	sandy silt to clayey silt
50.525	24.68	0.6305	2.555	23.906	12	5	clayey silt to silty clay
50.689	26.89	0.8155	3.033	36.084	13	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
50.853	34.90	0.9952	2.851	37.117	13	6	sandy silt to clayey silt
51.017	35.61	1.0403	2.921	32.551	17	5	clayey silt to silty clay
51.181	29.92	0.9293	3.106	28.548	14	5	clayey silt to silty clay
51.345	23.42	0.8057	3.440	28.897	11	5	clayey silt to silty clay
51.509	26.83	0.8410	3.135	32.606	13	5	clayey silt to silty clay
51.673	24.39	0.7881	3.231	29.727	12	5	clayey silt to silty clay
51.837	19.97	0.5289	2.649	29.033	10	5	clayey silt to silty clay
52.001	17.95	0.3668	2.044	31.330	9	5	clayey silt to silty clay
52.165	16.94	0.2982	1.760	35.056	8	5	clayey silt to silty clay
52.329	18.75	0.5998	3.199	38.840	9	5	clayey silt to silty clay
52.493	25.30	0.5495	2.172	43.260	10	6	sandy silt to clayey silt
52.657	27.75	0.7104	2.560	22.756	11	6	sandy silt to clayey silt
52.822	27.38	0.8186	2.989	27.549	13	5	clayey silt to silty clay
52.986	31.73	0.9711	3.060	28.512	15	5	clayey silt to silty clay
53.150	26.87	1.0057	3.742	27.463	17	4	silty clay to clay
53.314	20.38	0.5987	2.939	28.038	10	5	clayey silt to silty clay
53.478	17.91	0.3091	1.726	30.710	7	6	sandy silt to clayey silt
53.642	17.52	0.6089	3.474	34.398	11	4	silty clay to clay
53.806	21.66	0.7555	3.488	37.675	10	5	clayey silt to silty clay
53.970	24.49	0.7032	2.871	43.950	12	5	clayey silt to silty clay
54.134	18.33	0.3276	1.788	48.189	7	6	sandy silt to clayey silt
54.298	15.68	0.2611	1.665	51.164	8	5	clayey silt to silty clay
54.462	17.50	0.2859	1.634	56.805	7	6	sandy silt to clayey silt
54.626	23.71	0.3866	1.630	61.474	9	6	sandy silt to clayey silt
54.790	27.29	0.5617	2.058	66.128	10	6	sandy silt to clayey silt
54.954	30.32	0.6981	2.302	69.101	12	6	sandy silt to clayey silt
55.118	31.19	0.7112	2.280	72.264	12	6	sandy silt to clayey silt
55.282	33.48	0.7381	2.205	84.159	13	6	sandy silt to clayey silt
55.446	31.89	0.7442	2.334	84.457	12	6	sandy silt to clayey silt
55.610	27.15	0.7923	2.919	79.285	13	5	clayey silt to silty clay
55.774	25.58	0.6817	2.665	81.382	12	5	clayey silt to silty clay
55.938	34.13	0.8871	2.599	97.222	13	6	sandy silt to clayey silt
56.102	39.91	1.2276	3.076	102.198	19	5	clayey silt to silty clay
56.266	40.05	1.6705	4.171	76.126	26	4	silty clay to clay
56.430	37.77	1.6509	4.371	75.289	24	4	silty clay to clay
56.594	36.02	1.4514	4.029	79.197	23	4	silty clay to clay
56.759	36.84	1.4424	3.915	86.289	18	5	clayey silt to silty clay
56.923	39.72	1.8423	4.638	85.702	25	4	silty clay to clay
57.087	39.96	1.9557	4.894	72.030	38	3	clay
57.251	35.11	1.6788	4.782	46.533	34	3	clay
57.415	33.27	1.4429	4.337	47.805	21	4	silty clay to clay
57.579	31.33	1.3501	4.309	50.383	20	4	silty clay to clay
57.743	30.71	1.2652	4.120	52.800	20	4	silty clay to clay
57.907	30.81	1.2413	4.028	54.856	20	4	silty clay to clay
58.071	30.87	1.2822	4.153	56.030	20	4	silty clay to clay
58.235	30.83	1.3761	4.464	53.501	20	4	silty clay to clay
58.399	31.08	1.4960	4.813	54.465	30	3	clay
58.563	31.67	1.5922	5.028	51.094	30	3	clay
58.727	31.90	1.4141	4.433	52.321	20	4	silty clay to clay
58.891	30.98	1.4277	4.608	54.239	20	4	silty clay to clay
59.055	31.72	1.3988	4.410	55.472	20	4	silty clay to clay
59.219	32.04	1.5475	4.829	54.470	31	3	clay
59.383	32.96	1.5587	4.729	54.551	32	3	clay

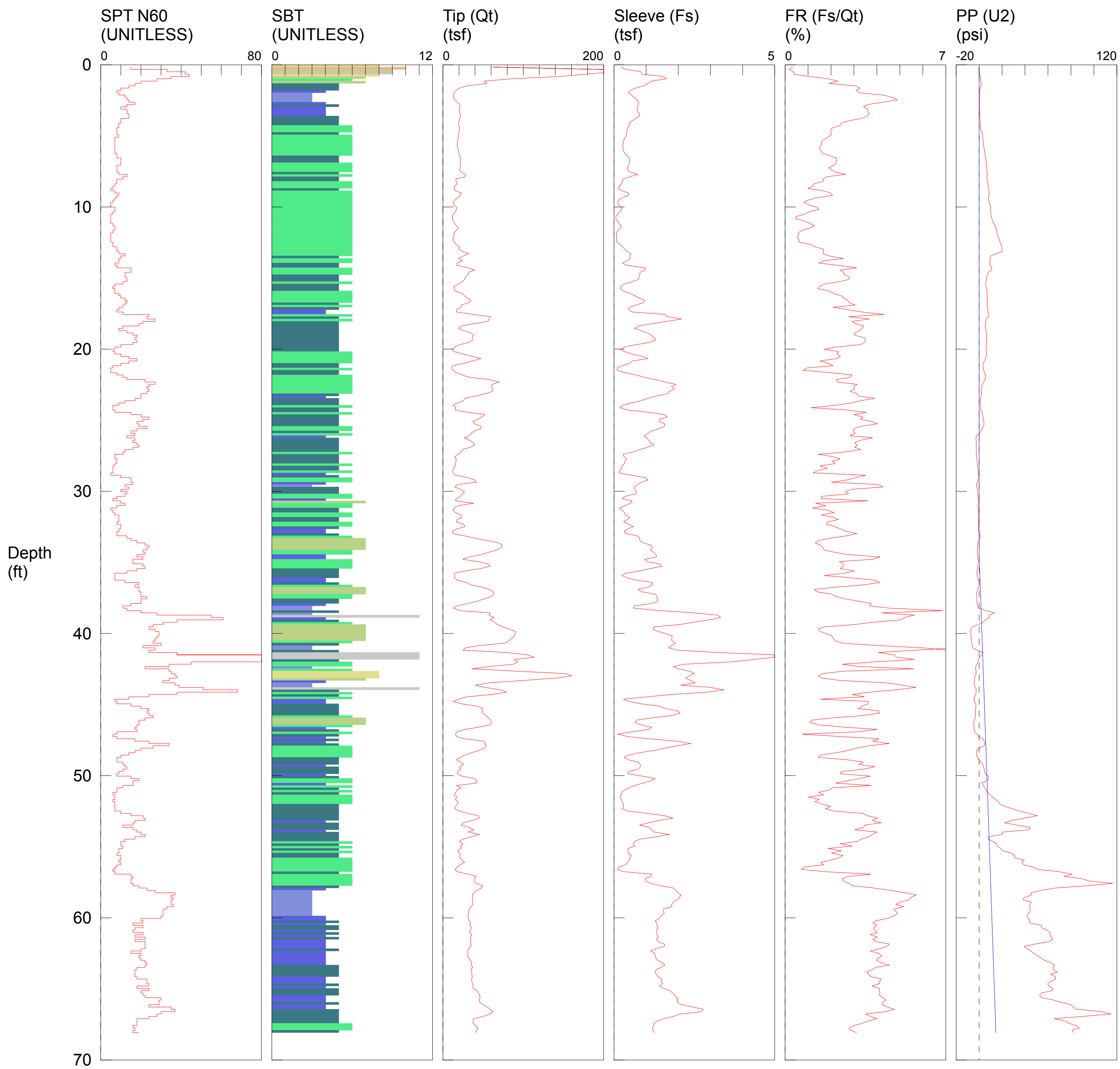
Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
59.547	32.10	1.4922	4.648	54.790	20	4	silty clay to clay
59.711	32.65	1.4739	4.515	56.095	21	4	silty clay to clay
59.875	33.55	1.4614	4.356	57.199	21	4	silty clay to clay
60.039	33.35	1.4333	4.299	57.357	21	4	silty clay to clay
60.203	32.41	1.4213	4.386	58.363	21	4	silty clay to clay
60.367	32.63	1.4613	4.479	61.414	21	4	silty clay to clay
60.532	40.04	1.4552	3.634	84.466	19	5	clayey silt to silty clay
60.696	42.16	1.6986	4.029	91.773	20	5	clayey silt to silty clay
60.860	46.28	1.5414	3.330	97.148	22	5	clayey silt to silty clay
61.024	44.62	1.5837	3.549	111.710	21	5	clayey silt to silty clay
61.188	49.67	1.5857	3.193	123.337	24	5	clayey silt to silty clay
61.352	53.13	1.5277	2.876	128.678	20	6	sandy silt to clayey silt
61.516	53.05	1.6461	3.103	143.087	20	6	sandy silt to clayey silt
61.680	70.20	2.1418	3.051	177.800	27	6	sandy silt to clayey silt
61.844	107.35	2.5619	2.387	90.483	34	7	silty sand to sandy silt
62.008	75.11	2.0736	2.761	20.351	29	6	sandy silt to clayey silt
62.172	53.20	1.5707	2.952	29.918	20	6	sandy silt to clayey silt
62.336	52.52	1.4678	2.795	30.655	20	6	sandy silt to clayey silt
62.500	47.96	1.4893	3.105	39.028	18	6	sandy silt to clayey silt
62.664	49.51	1.4468	2.923	43.766	19	6	sandy silt to clayey silt
62.828	46.96	1.2791	2.724	42.203	18	6	sandy silt to clayey silt
62.992	44.71	1.1564	2.586	44.508	17	6	sandy silt to clayey silt
63.156	45.22	1.2324	2.725	46.466	17	6	sandy silt to clayey silt
63.320	55.36	2.0795	3.757	49.312	27	5	clayey silt to silty clay
63.484	79.90	2.2161	2.773	43.456	31	6	sandy silt to clayey silt
63.648	65.46	1.9192	2.932	37.718	25	6	sandy silt to clayey silt
63.812	43.39	1.5764	3.633	44.567	21	5	clayey silt to silty clay
63.976	43.26	1.1280	2.607	41.977	17	6	sandy silt to clayey silt
64.140	37.76	1.1674	3.091	46.994	18	5	clayey silt to silty clay
64.304	46.91	0.9646	2.056	49.534	18	6	sandy silt to clayey silt
64.469	40.38	0.9050	2.241	47.523	15	6	sandy silt to clayey silt
64.633	35.24	0.8590	2.438	51.507	13	6	sandy silt to clayey silt
64.797	32.08	0.8564	2.670	51.724	12	6	sandy silt to clayey silt
64.961	31.71	0.9986	3.149	50.691	15	5	clayey silt to silty clay
65.125	32.10	0.8250	2.570	52.082	12	6	sandy silt to clayey silt
65.289	32.28	0.7296	2.260	53.320	12	6	sandy silt to clayey silt
65.453	30.80	0.8938	2.902	54.747	15	5	clayey silt to silty clay
65.617	29.70	0.8391	2.825	56.514	14	5	clayey silt to silty clay
65.781	36.25	1.0293	2.839	65.668	14	6	sandy silt to clayey silt
65.945	38.62	1.1280	2.921	64.773	15	6	sandy silt to clayey silt
66.109	35.31	1.3677	3.874	66.097	17	5	clayey silt to silty clay
66.273	34.58	1.2528	3.623	65.658	17	5	clayey silt to silty clay
66.437	34.57	1.1421	3.303	67.054	17	5	clayey silt to silty clay
66.601	32.75	0.9857	3.009	68.220	16	5	clayey silt to silty clay
66.765	31.44	0.8181	2.602	70.546	12	6	sandy silt to clayey silt
66.929	33.75	0.8568	2.538	75.339	13	6	sandy silt to clayey silt
67.093	37.75	0.8890	2.355	75.601	14	6	sandy silt to clayey silt
67.257	37.17	0.9199	2.475	77.763	14	6	sandy silt to clayey silt
67.421	33.05	0.8069	2.442	79.101	13	6	sandy silt to clayey silt
67.585	31.27	0.6087	1.947	80.697	12	6	sandy silt to clayey silt
67.749	29.62	0.4842	1.635	82.632	11	6	sandy silt to clayey silt
67.913	29.76	0.4777	1.605	85.091	11	6	sandy silt to clayey silt
68.077	28.10	0.4503	1.603	82.629	11	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
68.241	25.96	0.2421	0.933	83.054	8	7	silty sand to sandy silt
68.406	23.43	0.2293	0.978	83.848	9	6	sandy silt to clayey silt
68.570	21.73	0.2117	0.974	88.553	8	6	sandy silt to clayey silt
68.734	23.10	0.2231	0.966	92.713	9	6	sandy silt to clayey silt
68.898	25.22	0.2162	0.857	97.327	8	7	silty sand to sandy silt
69.062	28.08	0.3695	1.316	98.780	11	6	sandy silt to clayey silt
69.226	27.49	0.4017	1.461	103.043	11	6	sandy silt to clayey silt
69.390	27.84	0.3305	1.187	104.620	11	6	sandy silt to clayey silt
69.554	24.14	0.2885	1.195	106.321	9	6	sandy silt to clayey silt
69.718	23.65	0.2723	1.151	110.333	9	6	sandy silt to clayey silt
69.882	25.01	0.2831	1.132	111.748	10	6	sandy silt to clayey silt
70.046	24.21	0.2521	1.041	108.830	9	6	sandy silt to clayey silt
70.210	21.13	0.2270	1.074	113.203	8	6	sandy silt to clayey silt
70.374	22.30	0.2811	1.261	123.003	9	6	sandy silt to clayey silt
70.538	23.51	0.3483	1.481	123.692	9	6	sandy silt to clayey silt
70.702	24.58	0.3267	1.329	119.661	9	6	sandy silt to clayey silt
70.866	24.27	0.2794	1.151	120.591	9	6	sandy silt to clayey silt
71.030	23.19	0.3539	1.526	123.988	9	6	sandy silt to clayey silt
71.194	30.64	0.6536	2.133	137.161	12	6	sandy silt to clayey silt
71.358	64.43	1.3532	2.100	152.062	21	7	silty sand to sandy silt
71.522	79.46	2.2325	2.810	89.218	30	6	sandy silt to clayey silt
71.686	78.92	2.2674	2.873	99.178	30	6	sandy silt to clayey silt
71.850	85.79	1.3118	1.529	45.240	27	7	silty sand to sandy silt
72.014	59.12	0.5060	0.856	44.524	19	7	silty sand to sandy silt
72.178	52.29	1.6172	3.093	67.722	20	6	sandy silt to clayey silt
72.343	81.25	0.9492	1.168	44.000	19	8	sand to silty sand
72.507	48.12	0.9024	1.875	42.876	15	7	silty sand to sandy silt
72.671	39.33	0.7400	1.881	53.742	15	6	sandy silt to clayey silt
72.835	38.50	0.8458	2.197	54.444	15	6	sandy silt to clayey silt
72.999	35.85	0.8098	2.259	55.520	14	6	sandy silt to clayey silt
73.163	36.51	0.7261	1.989	58.652	14	6	sandy silt to clayey silt
73.327	37.79	0.5689	1.505	59.942	12	7	silty sand to sandy silt
73.491	39.14	0.6522	1.666	62.118	15	6	sandy silt to clayey silt
73.655	37.40	0.8443	2.258	66.181	14	6	sandy silt to clayey silt
73.819	48.50	1.0727	2.212	83.488	19	6	sandy silt to clayey silt
73.983	49.14	1.2560	2.556	84.438	19	6	sandy silt to clayey silt
74.147	47.45	1.3337	2.810	95.223	18	6	sandy silt to clayey silt
74.311	52.69	1.3658	2.592	101.716	20	6	sandy silt to clayey silt
74.475	59.87	1.3947	2.330	107.790	23	6	sandy silt to clayey silt
74.639	63.00	1.3197	2.095	109.706	20	7	silty sand to sandy silt
74.803	56.70	0.8943	1.577	106.817	18	7	silty sand to sandy silt
74.967	49.21	0.8098	1.646	117.595	16	7	silty sand to sandy silt
75.131	40.35	0.7711	1.911	130.639	15	6	sandy silt to clayey silt
75.295	43.42	1.2556	2.892	143.237	17	6	sandy silt to clayey silt
75.459	61.38	1.4495	2.362	160.106	24	6	sandy silt to clayey silt
75.623	54.86	1.6013	2.919	108.324	21	6	sandy silt to clayey silt
75.787	49.82	1.4502	2.911	68.395	19	6	sandy silt to clayey silt
75.951	38.80	1.0677	2.752	73.660	15	6	sandy silt to clayey silt
76.115	32.24	0.8367	2.595	80.327	12	6	sandy silt to clayey silt
76.280	31.09	0.6699	2.154	76.420	12	6	sandy silt to clayey silt
76.444	28.14	0.7213	2.563	75.162	11	6	sandy silt to clayey silt
76.608	27.09	0.7733	2.855	72.670	13	5	clayey silt to silty clay
76.772	26.71	0.7425	2.780	71.625	13	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
76.936	26.90	0.7662	2.849	52.378	13	5	clayey silt to silty clay
77.100	27.62	0.7477	2.708	54.203	13	5	clayey silt to silty clay
77.264	27.30	0.7638	2.797	56.178	13	5	clayey silt to silty clay
77.428	26.59	0.7057	2.654	55.730	13	5	clayey silt to silty clay
77.592	27.17	0.7644	2.813	56.731	13	5	clayey silt to silty clay
77.756	27.59	0.7683	2.785	55.961	13	5	clayey silt to silty clay
77.920	27.47	0.8691	3.164	56.705	13	5	clayey silt to silty clay
78.084	27.73	0.8645	3.117	57.204	13	5	clayey silt to silty clay
78.248	28.11	0.8438	3.002	56.722	13	5	clayey silt to silty clay
78.412	28.76	0.8724	3.033	57.409	14	5	clayey silt to silty clay
78.576	29.38	1.0574	3.599	57.962	14	5	clayey silt to silty clay
78.740	30.74	1.1461	3.728	58.525	15	5	clayey silt to silty clay
78.904	36.38	1.2372	3.400	58.187	17	5	clayey silt to silty clay
79.068	36.46	1.3367	3.667	59.942	17	5	clayey silt to silty clay
79.232	33.42	1.2820	3.836	58.349	16	5	clayey silt to silty clay
79.396	31.03	1.2044	3.882	58.554	20	4	silty clay to clay
79.560	29.58	1.1155	3.771	57.953	14	5	clayey silt to silty clay
79.724	30.18	1.0002	3.314	59.270	14	5	clayey silt to silty clay
79.888	33.13	0.7342	2.216	64.609	13	6	sandy silt to clayey silt
80.052	32.55	0.7993	2.456	62.533	12	6	sandy silt to clayey silt
80.217	32.46	0.8288	2.553	62.004	12	6	sandy silt to clayey silt
80.381	31.24	0.7658	2.451	65.737	12	6	sandy silt to clayey silt
80.545	29.09	0.7439	2.557	68.514	11	6	sandy silt to clayey silt
80.709	28.03	0.6300	2.248	71.274	11	6	sandy silt to clayey silt
80.873	28.36	0.5549	1.957	74.487	11	6	sandy silt to clayey silt
81.037	28.33	0.5359	1.891	75.298	11	6	sandy silt to clayey silt
81.201	26.04	0.4889	1.878	75.298	10	6	sandy silt to clayey silt
81.365	26.45	0.5587	2.113	76.308	10	6	sandy silt to clayey silt
81.529	29.74	0.6031	2.028	82.947	11	6	sandy silt to clayey silt
81.693	28.32	0.4937	1.743	83.350	11	6	sandy silt to clayey silt
81.857	26.28	0.4006	1.524	85.270	10	6	sandy silt to clayey silt
82.021	24.16	0.3984	1.649	89.032	9	6	sandy silt to clayey silt
82.185	27.22	0.4318	1.586	83.376	10	6	sandy silt to clayey silt
82.349	26.04	0.5021	1.928	82.429	10	6	sandy silt to clayey silt
82.513	25.40	0.5312	2.091	77.422	10	6	sandy silt to clayey silt
82.677	22.97	0.4833	2.104	51.090	9	6	sandy silt to clayey silt
82.841	21.68	0.7404	3.415	51.715	10	5	clayey silt to silty clay
83.005	33.13	0.7104	2.144	59.465	13	6	sandy silt to clayey silt
83.169	38.26	0.7002	1.830	65.394	15	6	sandy silt to clayey silt
83.333	42.66	0.6102	1.430	71.994	14	7	silty sand to sandy silt

Shannon & Wilson / CPT-2 / 1400 Wynooski St Newberg

OPERATOR: OGE DMM
CONE ID: DDG1415
HOLE NUMBER: CPT-2
TEST DATE: 5/20/2019 7:45:54 AM
TOTAL DEPTH: 68.077 ft



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Shannon & Wilson / CPT-2 / 1400 Wynooski St Newberg

OPERATOR: OGE DMM
CONE ID: DDG1415
HOLE NUMBER: CPT-2
TEST DATE: 5/20/2019 7:45:54 AM
TOTAL DEPTH: 68.077 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
0.164	62.94	0.2117	0.336	0.014	15	8	sand to silty sand
0.328	207.74	0.3627	0.175	-0.248	33	10	gravelly sand to sand
0.492	220.35	0.8408	0.382	-0.029	42	9	sand
0.656	231.36	0.8648	0.374	0.329	44	9	sand
0.820	145.86	1.5280	1.048	1.031	35	8	sand to silty sand
0.984	87.35	1.6293	1.865	1.021	28	7	silty sand to sandy silt
1.148	51.36	1.1799	2.297	1.515	20	6	sandy silt to clayey silt
1.312	54.76	1.0444	1.907	1.866	17	7	silty sand to sandy silt
1.476	29.87	0.9160	3.066	1.472	14	5	clayey silt to silty clay
1.640	20.43	0.6653	3.257	1.011	10	5	clayey silt to silty clay
1.804	16.42	0.5084	3.096	0.680	8	5	clayey silt to silty clay
1.969	13.49	0.4385	3.251	0.496	9	4	silty clay to clay
2.133	12.81	0.5471	4.272	0.482	12	3	clay
2.297	13.16	0.6182	4.697	0.413	13	3	clay
2.461	14.52	0.7087	4.882	0.310	14	3	clay
2.625	17.77	0.7615	4.286	0.284	17	3	clay
2.789	20.94	0.7576	3.618	0.355	13	4	silty clay to clay
2.953	21.20	0.7237	3.414	0.370	10	5	clayey silt to silty clay
3.117	20.13	0.7204	3.578	0.475	13	4	silty clay to clay
3.281	20.32	0.7316	3.600	0.542	13	4	silty clay to clay
3.445	21.65	0.7936	3.666	0.553	14	4	silty clay to clay
3.609	21.93	0.7844	3.577	0.613	14	4	silty clay to clay
3.773	20.64	0.6861	3.324	0.661	10	5	clayey silt to silty clay
3.937	19.90	0.5532	2.780	0.840	10	5	clayey silt to silty clay
4.101	19.32	0.4716	2.440	0.947	9	5	clayey silt to silty clay
4.265	19.83	0.4765	2.403	1.159	9	5	clayey silt to silty clay
4.429	20.93	0.4308	2.058	1.345	8	6	sandy silt to clayey silt
4.593	20.36	0.4053	1.990	1.472	8	6	sandy silt to clayey silt
4.757	19.74	0.3924	1.988	2.455	8	6	sandy silt to clayey silt
4.921	19.07	0.3857	2.023	2.774	9	5	clayey silt to silty clay
5.085	19.00	0.3532	1.859	3.085	7	6	sandy silt to clayey silt
5.249	18.38	0.3134	1.705	3.333	7	6	sandy silt to clayey silt
5.413	18.07	0.2856	1.580	3.540	7	6	sandy silt to clayey silt
5.577	17.85	0.2851	1.597	3.750	7	6	sandy silt to clayey silt
5.741	18.04	0.2705	1.499	4.022	7	6	sandy silt to clayey silt
5.906	18.48	0.2819	1.525	4.394	7	6	sandy silt to clayey silt
6.070	19.14	0.2979	1.557	4.769	7	6	sandy silt to clayey silt
6.234	19.91	0.3333	1.674	5.072	8	6	sandy silt to clayey silt
6.398	21.24	0.4116	1.938	5.387	8	6	sandy silt to clayey silt
6.562	21.56	0.4805	2.229	5.651	10	5	clayey silt to silty clay
6.726	21.75	0.4860	2.235	5.950	10	5	clayey silt to silty clay
6.890	21.49	0.4778	2.223	6.088	10	5	clayey silt to silty clay
7.054	21.30	0.4557	2.139	6.286	8	6	sandy silt to clayey silt
7.218	21.65	0.4116	1.901	6.599	8	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
7.382	20.52	0.4121	2.008	6.737	8	6	sandy silt to clayey silt
7.546	22.85	0.5029	2.201	6.959	9	6	sandy silt to clayey silt
7.710	27.86	0.7335	2.633	7.126	13	5	clayey silt to silty clay
7.874	27.68	0.4992	1.803	6.882	11	6	sandy silt to clayey silt
8.038	16.07	0.2661	1.655	6.651	8	5	clayey silt to silty clay
8.202	16.15	0.2694	1.668	7.340	8	5	clayey silt to silty clay
8.366	17.82	0.2242	1.258	7.794	7	6	sandy silt to clayey silt
8.530	14.72	0.1796	1.220	7.741	6	6	sandy silt to clayey silt
8.694	13.77	0.1368	0.993	8.199	5	6	sandy silt to clayey silt
8.858	15.24	0.2524	1.657	8.590	7	5	clayey silt to silty clay
9.022	22.90	0.4549	1.986	8.965	9	6	sandy silt to clayey silt
9.186	21.69	0.4480	2.066	8.171	8	6	sandy silt to clayey silt
9.350	18.40	0.2562	1.393	8.087	7	6	sandy silt to clayey silt
9.514	14.47	0.1451	1.003	8.381	6	6	sandy silt to clayey silt
9.678	13.00	0.1061	0.816	8.810	5	6	sandy silt to clayey silt
9.843	13.48	0.1353	1.004	9.483	5	6	sandy silt to clayey silt
10.007	17.04	0.2337	1.371	10.222	7	6	sandy silt to clayey silt
10.171	17.19	0.2515	1.463	9.998	7	6	sandy silt to clayey silt
10.335	15.27	0.1644	1.077	10.022	6	6	sandy silt to clayey silt
10.499	12.67	0.1082	0.854	10.217	5	6	sandy silt to clayey silt
10.663	11.91	0.0583	0.489	10.623	5	6	sandy silt to clayey silt
10.827	11.85	0.0541	0.457	11.131	5	6	sandy silt to clayey silt
10.991	12.51	0.1058	0.846	11.880	5	6	sandy silt to clayey silt
11.155	14.72	0.1589	1.079	12.596	6	6	sandy silt to clayey silt
11.319	17.12	0.2189	1.278	14.211	7	6	sandy silt to clayey silt
11.483	19.13	0.2065	1.079	14.707	7	6	sandy silt to clayey silt
11.647	16.80	0.1254	0.747	14.931	6	6	sandy silt to clayey silt
11.811	13.41	0.0759	0.566	15.475	5	6	sandy silt to clayey silt
11.975	14.13	0.0827	0.585	16.155	5	6	sandy silt to clayey silt
12.139	13.43	0.0698	0.520	16.816	5	6	sandy silt to clayey silt
12.303	12.36	0.0760	0.615	17.541	5	6	sandy silt to clayey silt
12.467	14.49	0.0858	0.592	18.223	6	6	sandy silt to clayey silt
12.631	16.00	0.1774	1.109	19.101	6	6	sandy silt to clayey silt
12.795	20.63	0.2637	1.278	19.741	8	6	sandy silt to clayey silt
12.959	19.96	0.3330	1.668	19.891	8	6	sandy silt to clayey silt
13.123	24.58	0.4110	1.672	20.046	9	6	sandy silt to clayey silt
13.287	31.76	0.5274	1.660	15.003	12	6	sandy silt to clayey silt
13.451	25.60	0.4893	1.911	10.134	10	6	sandy silt to clayey silt
13.615	19.69	0.4989	2.534	9.328	9	5	clayey silt to silty clay
13.780	24.18	0.4108	1.699	9.654	9	6	sandy silt to clayey silt
13.944	17.91	0.2623	1.464	9.177	7	6	sandy silt to clayey silt
14.108	17.20	0.3924	2.281	9.688	8	5	clayey silt to silty clay
14.272	31.47	0.9786	3.109	10.893	15	5	clayey silt to silty clay
14.436	39.44	0.9471	2.401	11.024	15	6	sandy silt to clayey silt
14.600	31.75	0.8078	2.544	7.801	12	6	sandy silt to clayey silt
14.764	31.12	0.8036	2.582	7.431	12	6	sandy silt to clayey silt
14.928	27.94	0.7848	2.808	6.517	13	5	clayey silt to silty clay
15.092	27.00	0.7488	2.774	6.310	13	5	clayey silt to silty clay
15.256	23.62	0.5596	2.369	5.864	11	5	clayey silt to silty clay
15.420	19.17	0.3332	1.738	5.628	7	6	sandy silt to clayey silt
15.584	13.57	0.2046	1.507	5.804	6	5	clayey silt to silty clay
15.748	14.01	0.1997	1.426	6.355	7	5	clayey silt to silty clay
15.912	14.97	0.2900	1.936	6.706	7	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
16.076	24.35	0.5088	2.090	7.059	9	6	sandy silt to clayey silt
16.240	26.61	0.6479	2.435	6.775	10	6	sandy silt to clayey silt
16.404	30.58	0.7787	2.547	7.107	12	6	sandy silt to clayey silt
16.568	34.87	0.9498	2.724	7.078	13	6	sandy silt to clayey silt
16.732	31.74	0.8913	2.808	6.882	12	6	sandy silt to clayey silt
16.896	22.96	0.7003	3.050	6.854	11	5	clayey silt to silty clay
17.060	22.38	0.4684	2.093	7.283	9	6	sandy silt to clayey silt
17.224	16.78	0.4557	2.716	7.343	8	5	clayey silt to silty clay
17.388	16.73	0.5611	3.355	7.677	11	4	silty clay to clay
17.552	36.87	1.5895	4.310	8.159	24	4	silty clay to clay
17.717	59.16	1.6428	2.777	8.490	23	6	sandy silt to clayey silt
17.881	57.26	2.1018	3.671	5.713	27	5	clayey silt to silty clay
18.045	56.05	1.6295	2.907	5.749	21	6	sandy silt to clayey silt
18.209	39.02	1.1964	3.066	5.351	19	5	clayey silt to silty clay
18.373	22.81	0.7812	3.425	5.153	11	5	clayey silt to silty clay
18.537	19.19	0.6423	3.347	5.475	9	5	clayey silt to silty clay
18.701	26.28	0.8534	3.247	6.143	13	5	clayey silt to silty clay
18.865	34.44	1.0382	3.014	6.329	16	5	clayey silt to silty clay
19.029	38.25	1.1612	3.036	6.463	18	5	clayey silt to silty clay
19.193	36.10	1.2423	3.441	6.687	17	5	clayey silt to silty clay
19.357	37.04	1.2955	3.497	6.677	18	5	clayey silt to silty clay
19.521	32.53	1.1270	3.465	6.505	16	5	clayey silt to silty clay
19.685	21.01	0.6927	3.297	6.036	10	5	clayey silt to silty clay
19.849	14.94	0.3303	2.211	6.050	7	5	clayey silt to silty clay
20.013	11.92	0.2041	1.711	6.067	6	5	clayey silt to silty clay
20.177	13.86	0.3224	2.327	6.219	7	5	clayey silt to silty clay
20.341	25.05	0.5649	2.255	6.346	10	6	sandy silt to clayey silt
20.505	37.09	0.8931	2.408	6.126	14	6	sandy silt to clayey silt
20.669	47.14	1.0614	2.252	5.754	18	6	sandy silt to clayey silt
20.833	36.81	0.5619	1.526	4.318	14	6	sandy silt to clayey silt
20.997	26.05	0.4860	1.866	3.717	10	6	sandy silt to clayey silt
21.161	12.18	0.2529	2.076	3.237	6	5	clayey silt to silty clay
21.325	10.69	0.1002	0.937	3.597	5	5	clayey silt to silty clay
21.490	12.17	0.0948	0.779	3.970	5	6	sandy silt to clayey silt
21.654	16.88	0.3339	1.977	4.814	8	5	clayey silt to silty clay
21.818	23.64	0.6882	2.911	5.442	11	5	clayey silt to silty clay
21.982	33.02	0.9393	2.845	5.606	13	6	sandy silt to clayey silt
22.146	58.21	1.2928	2.221	5.205	22	6	sandy silt to clayey silt
22.310	69.96	1.5876	2.269	4.055	27	6	sandy silt to clayey silt
22.474	61.54	1.9306	3.137	2.603	24	6	sandy silt to clayey silt
22.638	59.97	1.7852	2.977	2.522	23	6	sandy silt to clayey silt
22.802	62.42	1.8879	3.025	2.426	24	6	sandy silt to clayey silt
22.966	61.33	1.7348	2.829	2.018	23	6	sandy silt to clayey silt
23.130	53.21	1.6713	3.141	1.997	20	6	sandy silt to clayey silt
23.294	41.36	1.3125	3.173	1.710	20	5	clayey silt to silty clay
23.458	27.10	1.0576	3.903	1.212	17	4	silty clay to clay
23.622	25.48	0.8380	3.290	1.126	12	5	clayey silt to silty clay
23.786	18.52	0.5359	2.894	0.973	9	5	clayey silt to silty clay
23.950	12.49	0.2729	2.185	1.064	6	5	clayey silt to silty clay
24.114	16.28	0.1831	1.124	1.353	6	6	sandy silt to clayey silt
24.278	14.93	0.3954	2.649	1.398	7	5	clayey silt to silty clay
24.442	30.56	1.0814	3.539	2.557	15	5	clayey silt to silty clay
24.606	51.65	1.5488	2.999	2.887	20	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
24.770	49.08	1.6649	3.392	3.309	24	5	clayey silt to silty clay
24.934	43.00	1.4064	3.271	3.822	21	5	clayey silt to silty clay
25.098	37.27	1.4063	3.773	4.072	18	5	clayey silt to silty clay
25.262	39.42	1.5817	4.012	4.299	19	5	clayey silt to silty clay
25.427	47.25	1.5639	3.310	3.650	23	5	clayey silt to silty clay
25.591	47.11	1.4160	3.006	2.355	18	6	sandy silt to clayey silt
25.755	39.18	1.1565	2.952	1.436	15	6	sandy silt to clayey silt
25.919	35.21	1.0746	3.051	0.697	17	5	clayey silt to silty clay
26.083	32.78	0.9296	2.835	-2.219	13	6	sandy silt to clayey silt
26.247	26.70	1.0170	3.809	-2.856	17	4	silty clay to clay
26.411	33.22	1.0886	3.276	-2.679	16	5	clayey silt to silty clay
26.575	37.42	1.1342	3.031	-2.588	18	5	clayey silt to silty clay
26.739	39.68	1.2471	3.143	-2.512	19	5	clayey silt to silty clay
26.903	34.15	1.0349	3.031	-2.407	16	5	clayey silt to silty clay
27.067	24.29	0.8021	3.302	-2.514	12	5	clayey silt to silty clay
27.231	22.38	0.5844	2.611	-2.483	11	5	clayey silt to silty clay
27.395	19.21	0.2783	1.449	-2.400	7	6	sandy silt to clayey silt
27.559	14.78	0.3063	2.072	-2.350	7	5	clayey silt to silty clay
27.723	16.41	0.3911	2.383	-1.947	8	5	clayey silt to silty clay
27.887	16.47	0.3381	2.053	-1.811	8	5	clayey silt to silty clay
28.051	15.14	0.3297	2.177	-1.610	7	5	clayey silt to silty clay
28.215	15.73	0.2398	1.525	-1.310	6	6	sandy silt to clayey silt
28.379	13.36	0.2450	1.834	-1.102	6	5	clayey silt to silty clay
28.543	12.23	0.1723	1.409	-0.825	6	5	clayey silt to silty clay
28.707	12.85	0.1571	1.222	-0.735	5	6	sandy silt to clayey silt
28.871	15.49	0.5428	3.505	-0.542	10	4	silty clay to clay
29.035	31.17	0.9619	3.086	-0.024	15	5	clayey silt to silty clay
29.199	39.29	1.0580	2.693	-0.754	15	6	sandy silt to clayey silt
29.364	41.68	0.8405	2.016	-1.429	16	6	sandy silt to clayey silt
29.528	18.21	0.7370	4.047	-2.555	12	4	silty clay to clay
29.692	14.74	0.6291	4.269	-2.056	14	3	clay
29.856	21.47	0.6302	2.935	-1.429	10	5	clayey silt to silty clay
30.020	26.87	0.7024	2.614	-1.298	13	5	clayey silt to silty clay
30.184	24.10	0.6654	2.761	-0.933	12	5	clayey silt to silty clay
30.348	21.54	0.3429	1.591	-0.969	8	6	sandy silt to clayey silt
30.512	16.62	0.2593	1.560	-0.816	6	6	sandy silt to clayey silt
30.676	15.58	0.5606	3.597	-0.670	10	4	silty clay to clay
30.840	38.40	0.5080	1.323	-0.389	12	7	silty sand to sandy silt
31.004	19.87	0.3514	1.768	-1.004	8	6	sandy silt to clayey silt
31.168	13.87	0.1656	1.194	-0.964	5	6	sandy silt to clayey silt
31.332	12.66	0.2345	1.852	-0.778	6	5	clayey silt to silty clay
31.496	15.40	0.3281	2.130	-0.499	7	5	clayey silt to silty clay
31.660	22.37	0.3540	1.582	-0.432	9	6	sandy silt to clayey silt
31.824	23.12	0.4856	2.100	-0.675	9	6	sandy silt to clayey silt
31.988	17.85	0.4148	2.323	-0.756	9	5	clayey silt to silty clay
32.152	16.13	0.2985	1.851	-0.518	8	5	clayey silt to silty clay
32.316	26.10	0.4669	1.789	-0.615	10	6	sandy silt to clayey silt
32.480	27.32	0.5922	2.167	-0.563	10	6	sandy silt to clayey silt
32.644	17.75	0.4262	2.401	-0.615	9	5	clayey silt to silty clay
32.808	11.85	0.3140	2.650	-0.429	8	4	silty clay to clay
32.972	12.03	0.3757	3.124	0.346	8	4	silty clay to clay
33.136	26.74	0.7134	2.668	1.007	13	5	clayey silt to silty clay
33.301	40.19	0.7977	1.985	0.608	15	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
33.465	55.72	0.8002	1.436	-0.239	18	7	silty sand to sandy silt
33.629	69.87	0.9174	1.313	-1.229	22	7	silty sand to sandy silt
33.793	73.78	1.1441	1.551	-1.481	24	7	silty sand to sandy silt
33.957	72.30	1.1919	1.649	-1.594	23	7	silty sand to sandy silt
34.121	67.30	1.1344	1.686	-1.658	21	7	silty sand to sandy silt
34.285	57.30	1.2234	2.135	-1.508	22	6	sandy silt to clayey silt
34.449	48.26	1.3049	2.704	-1.345	18	6	sandy silt to clayey silt
34.613	32.22	1.3293	4.126	-1.291	21	4	silty clay to clay
34.777	24.83	0.9484	3.820	-1.083	16	4	silty clay to clay
34.941	42.95	1.0137	2.360	-0.499	16	6	sandy silt to clayey silt
35.105	55.80	1.4088	2.525	-0.840	21	6	sandy silt to clayey silt
35.269	58.66	1.4897	2.540	-1.355	22	6	sandy silt to clayey silt
35.433	46.94	1.1250	2.397	-1.543	18	6	sandy silt to clayey silt
35.597	26.95	0.8109	3.009	-1.811	13	5	clayey silt to silty clay
35.761	14.46	0.3781	2.615	-2.030	7	5	clayey silt to silty clay
35.925	14.41	0.2454	1.703	-1.722	7	5	clayey silt to silty clay
36.089	14.08	0.4104	2.915	-1.241	7	5	clayey silt to silty clay
36.253	18.07	0.7063	3.909	-0.592	12	4	silty clay to clay
36.417	29.33	1.2077	4.118	-0.036	19	4	silty clay to clay
36.581	34.54	1.1805	3.418	-0.129	17	5	clayey silt to silty clay
36.745	48.76	1.0733	2.201	-0.363	19	6	sandy silt to clayey silt
36.909	58.52	0.7415	1.267	-1.143	19	7	silty sand to sandy silt
37.073	62.61	0.8821	1.409	-1.970	20	7	silty sand to sandy silt
37.238	63.10	1.2851	2.037	-2.176	20	7	silty sand to sandy silt
37.402	60.12	1.3359	2.222	-2.183	23	6	sandy silt to clayey silt
37.566	51.89	1.3458	2.594	-1.942	20	6	sandy silt to clayey silt
37.730	42.25	1.3583	3.215	-1.925	20	5	clayey silt to silty clay
37.894	31.09	1.1687	3.759	-1.727	15	5	clayey silt to silty clay
38.058	17.24	0.6329	3.671	-1.481	11	4	silty clay to clay
38.222	13.15	0.5965	4.535	-0.208	13	3	clay
38.386	20.99	1.4438	6.879	6.203	20	3	clay
38.550	59.11	2.4877	4.209	13.443	28	5	clayey silt to silty clay
38.714	57.27	3.2200	5.622	8.676	55	3	clay
38.878	63.74	3.3161	5.203	8.879	61	11	very stiff fine grained (*)
39.042	58.83	2.9721	5.052	5.332	38	4	silty clay to clay
39.206	64.41	2.4638	3.825	3.395	31	5	clayey silt to silty clay
39.370	67.76	2.0729	3.059	-0.448	26	6	sandy silt to clayey silt
39.534	74.07	1.2450	1.681	-5.859	24	7	silty sand to sandy silt
39.698	83.32	1.2075	1.449	-7.324	27	7	silty sand to sandy silt
39.862	90.38	1.3784	1.525	-7.286	29	7	silty sand to sandy silt
40.026	89.85	1.6360	1.821	-7.286	29	7	silty sand to sandy silt
40.190	88.67	1.8123	2.044	-7.092	28	7	silty sand to sandy silt
40.354	85.42	1.7939	2.100	-6.954	27	7	silty sand to sandy silt
40.518	81.86	1.8000	2.199	-6.770	26	7	silty sand to sandy silt
40.682	77.30	1.9106	2.472	-6.389	30	6	sandy silt to clayey silt
40.846	44.11	1.7919	4.062	-6.088	21	5	clayey silt to silty clay
41.011	28.17	1.6981	6.029	-5.687	27	3	clay
41.175	24.79	1.9524	7.875	-1.491	24	3	clay
41.339	78.77	3.2032	4.067	3.316	38	5	clayey silt to silty clay
41.503	105.42	4.9791	4.723	2.495	101	11	very stiff fine grained (*)
41.667	113.31	5.5155	4.868	0.396	109	11	very stiff fine grained (*)
41.831	89.34	5.0338	5.634	-0.685	86	11	very stiff fine grained (*)
41.995	93.38	3.6769	3.937	-0.897	45	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
42.159	87.50	2.1952	2.509	-1.932	34	6	sandy silt to clayey silt
42.323	58.24	1.8486	3.174	-2.920	22	6	sandy silt to clayey silt
42.487	35.98	2.0145	5.599	-3.159	34	3	clay
42.651	92.29	2.3604	2.558	-2.603	35	6	sandy silt to clayey silt
42.815	153.20	2.4998	1.632	-2.698	37	8	sand to silty sand
42.979	160.44	2.3360	1.456	-3.624	38	8	sand to silty sand
43.143	140.78	2.2298	1.584	-3.979	34	8	sand to silty sand
43.307	97.58	2.3884	2.448	-4.342	31	7	silty sand to sandy silt
43.471	57.50	2.5248	4.391	-4.270	37	4	silty clay to clay
43.635	40.57	2.0766	5.119	-4.020	39	3	clay
43.799	52.90	3.0128	5.695	-3.213	51	3	clay
43.963	71.07	3.4163	4.807	-2.550	68	11	very stiff fine grained (*)
44.127	79.17	2.9161	3.683	-2.531	38	5	clayey silt to silty clay
44.291	61.72	1.8044	2.924	-3.244	24	6	sandy silt to clayey silt
44.455	29.86	1.0312	3.453	-4.039	14	5	clayey silt to silty clay
44.619	19.27	0.2954	1.533	-4.571	7	6	sandy silt to clayey silt
44.783	12.70	0.4616	3.634	-3.993	8	4	silty clay to clay
44.948	23.01	0.8302	3.608	-3.068	15	4	silty clay to clay
45.112	42.55	1.4290	3.358	-2.677	20	5	clayey silt to silty clay
45.276	49.86	1.5868	3.183	-3.113	24	5	clayey silt to silty clay
45.440	48.55	1.9701	4.058	-3.445	23	5	clayey silt to silty clay
45.604	49.96	2.0537	4.110	-3.213	24	5	clayey silt to silty clay
45.768	53.59	1.8214	3.399	-3.199	26	5	clayey silt to silty clay
45.932	56.74	1.1899	2.097	-3.359	22	6	sandy silt to clayey silt
46.096	59.41	0.6959	1.171	-4.327	19	7	silty sand to sandy silt
46.260	60.47	0.6512	1.077	-4.714	19	7	silty sand to sandy silt
46.424	57.90	0.8463	1.462	-4.671	18	7	silty sand to sandy silt
46.588	44.78	1.1633	2.598	-4.628	17	6	sandy silt to clayey silt
46.752	27.49	1.0975	3.993	-4.256	18	4	silty clay to clay
46.916	17.75	0.5651	3.183	-3.803	8	5	clayey silt to silty clay
47.080	15.45	0.1181	0.764	-1.639	6	6	sandy silt to clayey silt
47.244	16.96	0.4103	2.420	-0.654	8	5	clayey silt to silty clay
47.408	26.59	1.0806	4.064	3.123	17	4	silty clay to clay
47.572	51.03	1.9474	3.816	4.394	24	5	clayey silt to silty clay
47.736	52.71	2.3855	4.526	5.198	34	4	silty clay to clay
47.900	53.55	1.9617	3.663	4.473	26	5	clayey silt to silty clay
48.064	51.31	1.5884	3.096	-0.146	20	6	sandy silt to clayey silt
48.228	44.58	1.2232	2.744	-2.080	17	6	sandy silt to clayey silt
48.392	35.65	0.7900	2.216	-2.536	14	6	sandy silt to clayey silt
48.556	27.26	0.4202	1.541	-2.142	10	6	sandy silt to clayey silt
48.720	21.69	0.3138	1.447	-1.539	8	6	sandy silt to clayey silt
48.885	16.81	0.4609	2.743	-0.258	8	5	clayey silt to silty clay
49.049	22.06	0.7511	3.404	1.684	11	5	clayey silt to silty clay
49.213	25.52	0.8178	3.205	3.397	12	5	clayey silt to silty clay
49.377	21.05	0.8227	3.908	3.545	13	4	silty clay to clay
49.541	22.02	0.7556	3.432	4.392	11	5	clayey silt to silty clay
49.705	18.78	0.4607	2.453	4.843	9	5	clayey silt to silty clay
49.869	17.62	0.4247	2.410	5.255	8	5	clayey silt to silty clay
50.033	23.46	0.8705	3.711	8.051	15	4	silty clay to clay
50.197	39.12	1.2815	3.275	8.245	19	5	clayey silt to silty clay
50.361	42.15	1.1446	2.716	6.727	16	6	sandy silt to clayey silt
50.525	41.67	0.9145	2.194	2.197	16	6	sandy silt to clayey silt
50.689	17.06	0.6402	3.752	3.481	11	4	silty clay to clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
50.853	22.18	0.4588	2.069	5.255	8	6	sandy silt to clayey silt
51.017	17.25	0.3303	1.915	5.833	8	5	clayey silt to silty clay
51.181	15.96	0.2333	1.462	7.558	6	6	sandy silt to clayey silt
51.345	13.78	0.2298	1.668	9.523	7	5	clayey silt to silty clay
51.509	17.88	0.1818	1.017	11.699	7	6	sandy silt to clayey silt
51.673	15.32	0.2272	1.483	14.490	6	6	sandy silt to clayey silt
51.837	18.92	0.2465	1.303	17.040	7	6	sandy silt to clayey silt
52.001	18.19	0.2900	1.594	18.233	7	6	sandy silt to clayey silt
52.165	15.09	0.2996	1.985	22.424	7	5	clayey silt to silty clay
52.329	14.54	0.2574	1.770	28.844	7	5	clayey silt to silty clay
52.493	20.00	0.4255	2.127	34.892	10	5	clayey silt to silty clay
52.657	31.16	1.0568	3.392	43.797	15	5	clayey silt to silty clay
52.822	44.04	1.5819	3.592	50.527	21	5	clayey silt to silty clay
52.986	45.25	1.8241	4.031	39.667	22	5	clayey silt to silty clay
53.150	36.51	1.3890	3.805	28.357	17	5	clayey silt to silty clay
53.314	25.80	1.0815	4.191	22.205	16	4	silty clay to clay
53.478	22.82	0.7928	3.475	32.721	11	5	clayey silt to silty clay
53.642	32.53	1.1113	3.416	44.474	16	5	clayey silt to silty clay
53.806	38.35	1.1646	3.037	43.542	18	5	clayey silt to silty clay
53.970	31.35	1.2541	4.000	20.304	20	4	silty clay to clay
54.134	45.17	1.7139	3.795	20.132	22	5	clayey silt to silty clay
54.298	36.31	1.3244	3.648	8.633	17	5	clayey silt to silty clay
54.462	24.43	0.8072	3.305	7.980	12	5	clayey silt to silty clay
54.626	20.36	0.6445	3.166	11.825	10	5	clayey silt to silty clay
54.790	24.83	0.5956	2.399	15.103	10	6	sandy silt to clayey silt
54.954	22.14	0.6474	2.924	15.762	11	5	clayey silt to silty clay
55.118	22.97	0.4324	1.882	18.848	9	6	sandy silt to clayey silt
55.282	18.41	0.4474	2.430	20.590	9	5	clayey silt to silty clay
55.446	20.93	0.4243	2.027	28.002	8	6	sandy silt to clayey silt
55.610	21.30	0.5372	2.522	30.850	10	5	clayey silt to silty clay
55.774	20.95	0.4946	2.361	30.674	10	5	clayey silt to silty clay
55.938	23.98	0.4818	2.009	39.348	9	6	sandy silt to clayey silt
56.102	26.86	0.4152	1.546	36.976	10	6	sandy silt to clayey silt
56.266	21.00	0.3547	1.689	38.811	8	6	sandy silt to clayey silt
56.430	17.77	0.1508	0.848	46.734	7	6	sandy silt to clayey silt
56.594	14.89	0.1039	0.698	50.772	6	6	sandy silt to clayey silt
56.759	17.39	0.2993	1.721	69.418	7	6	sandy silt to clayey silt
56.923	31.53	1.1767	3.732	81.103	15	5	clayey silt to silty clay
57.087	41.23	1.1449	2.777	73.862	16	6	sandy silt to clayey silt
57.251	38.75	0.9680	2.498	89.757	15	6	sandy silt to clayey silt
57.415	38.70	0.9807	2.534	95.478	15	6	sandy silt to clayey silt
57.579	41.23	1.1432	2.773	116.364	16	6	sandy silt to clayey silt
57.743	49.54	1.4528	2.932	93.789	19	6	sandy silt to clayey silt
57.907	47.72	1.8471	3.871	66.283	23	5	clayey silt to silty clay
58.071	42.13	1.9317	4.585	54.024	27	4	silty clay to clay
58.235	38.13	1.9902	5.220	45.185	37	3	clay
58.399	36.42	2.0816	5.716	45.574	35	3	clay
58.563	37.27	2.0405	5.475	39.505	36	3	clay
58.727	36.38	1.9695	5.414	38.060	35	3	clay
58.891	36.74	1.8407	5.011	45.157	35	3	clay
59.055	37.13	1.7896	4.820	44.343	36	3	clay
59.219	34.94	1.8002	5.153	43.919	33	3	clay
59.383	32.51	1.5375	4.730	44.474	31	3	clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	FR (Fs/Qt) (%)	PP (U2) (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
59.547	31.86	1.5373	4.825	44.377	31	3	clay
59.711	32.03	1.5654	4.887	42.800	31	3	clay
59.875	30.88	1.4569	4.717	41.531	30	3	clay
60.039	31.57	1.4089	4.462	42.840	20	4	silty clay to clay
60.203	32.41	1.2831	3.958	44.021	21	4	silty clay to clay
60.367	34.42	1.3197	3.834	43.737	16	5	clayey silt to silty clay
60.532	33.65	1.3438	3.994	54.332	21	4	silty clay to clay
60.696	35.37	1.3642	3.856	54.651	17	5	clayey silt to silty clay
60.860	33.90	1.3197	3.893	58.740	16	5	clayey silt to silty clay
61.024	33.23	1.3337	4.014	61.875	21	4	silty clay to clay
61.188	35.98	1.3336	3.707	61.581	17	5	clayey silt to silty clay
61.352	34.31	1.3790	4.020	62.607	22	4	silty clay to clay
61.516	34.52	1.3187	3.820	64.141	17	5	clayey silt to silty clay
61.680	34.30	1.3584	3.960	60.830	22	4	silty clay to clay
61.844	34.64	1.5718	4.538	48.434	22	4	silty clay to clay
62.008	35.15	1.5636	4.449	39.279	22	4	silty clay to clay
62.172	31.62	1.3043	4.125	42.122	20	4	silty clay to clay
62.336	30.82	1.1413	3.704	44.656	15	5	clayey silt to silty clay
62.500	31.06	1.2667	4.079	47.771	20	4	silty clay to clay
62.664	30.45	1.2851	4.220	50.524	19	4	silty clay to clay
62.828	32.01	1.3000	4.062	53.902	20	4	silty clay to clay
62.992	34.17	1.3534	3.961	56.927	22	4	silty clay to clay
63.156	35.85	1.4993	4.182	62.576	23	4	silty clay to clay
63.320	34.85	1.5813	4.537	65.625	22	4	silty clay to clay
63.484	36.99	1.4635	3.956	64.470	18	5	clayey silt to silty clay
63.648	34.83	1.3150	3.776	64.921	17	5	clayey silt to silty clay
63.812	35.88	1.2788	3.565	68.232	17	5	clayey silt to silty clay
63.976	36.02	1.3100	3.636	62.288	17	5	clayey silt to silty clay
64.140	36.57	1.3685	3.742	66.441	18	5	clayey silt to silty clay
64.304	35.69	1.4965	4.193	66.765	23	4	silty clay to clay
64.469	35.84	1.4792	4.127	62.452	23	4	silty clay to clay
64.633	37.04	1.5192	4.102	59.771	24	4	silty clay to clay
64.797	37.21	1.4105	3.791	60.563	18	5	clayey silt to silty clay
64.961	37.69	1.5462	4.103	64.277	24	4	silty clay to clay
65.125	42.07	1.7310	4.115	63.633	20	5	clayey silt to silty clay
65.289	44.24	1.8263	4.128	54.508	21	5	clayey silt to silty clay
65.453	45.84	1.8815	4.104	52.674	22	5	clayey silt to silty clay
65.617	46.29	1.9723	4.261	54.310	30	4	silty clay to clay
65.781	44.98	1.9784	4.399	64.866	29	4	silty clay to clay
65.945	46.04	1.9568	4.251	71.047	29	4	silty clay to clay
66.109	49.77	2.1048	4.229	79.912	24	5	clayey silt to silty clay
66.273	55.05	2.4558	4.461	81.813	35	4	silty clay to clay
66.437	58.50	2.7750	4.744	84.719	37	4	silty clay to clay
66.601	62.41	2.7191	4.357	106.631	30	5	clayey silt to silty clay
66.765	58.47	2.0453	3.498	114.861	28	5	clayey silt to silty clay
66.929	50.00	1.8587	3.718	90.161	24	5	clayey silt to silty clay
67.093	38.51	1.5425	4.006	65.401	18	5	clayey silt to silty clay
67.257	37.38	1.3676	3.659	74.392	18	5	clayey silt to silty clay
67.421	37.17	1.2422	3.342	78.311	18	5	clayey silt to silty clay
67.585	41.14	1.2265	2.981	84.784	16	6	sandy silt to clayey silt
67.749	43.23	1.2056	2.789	87.288	17	6	sandy silt to clayey silt
67.913	41.93	1.2003	2.863	81.131	16	6	sandy silt to clayey silt
68.077	40.50	1.2603	3.112	82.193	19	5	clayey silt to silty clay

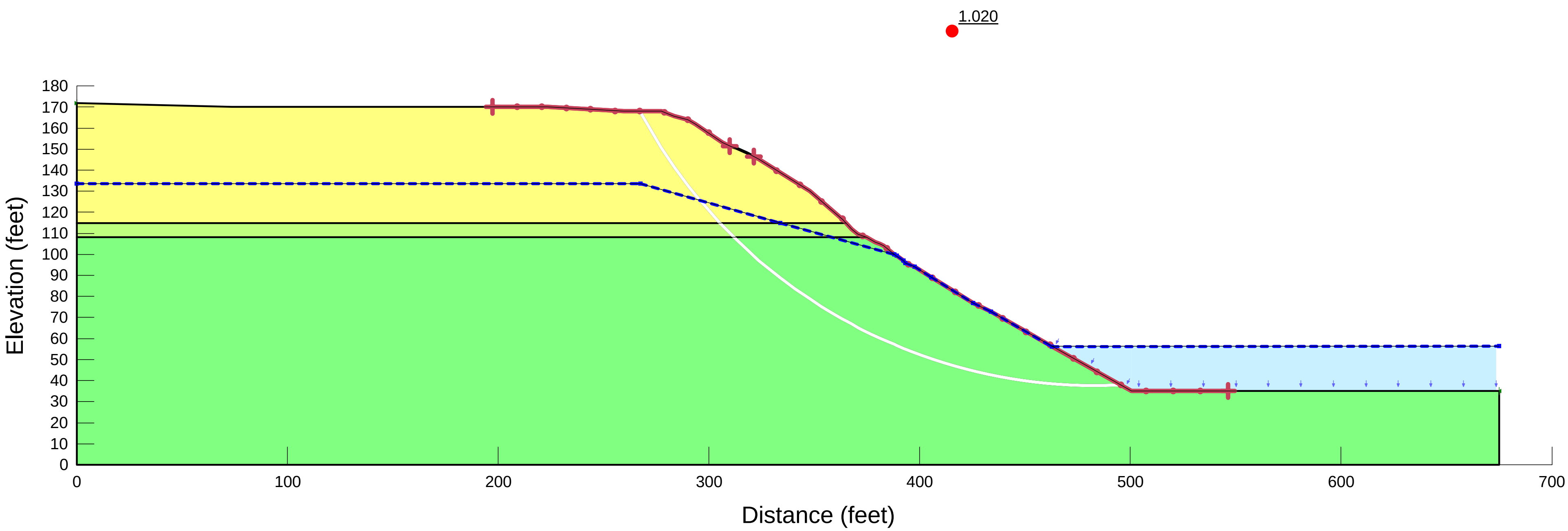
APPENDIX B: SLOPE STABILITY SUMMARY RESULTS

Appendix B

Slope Stability Summary Results

Figures

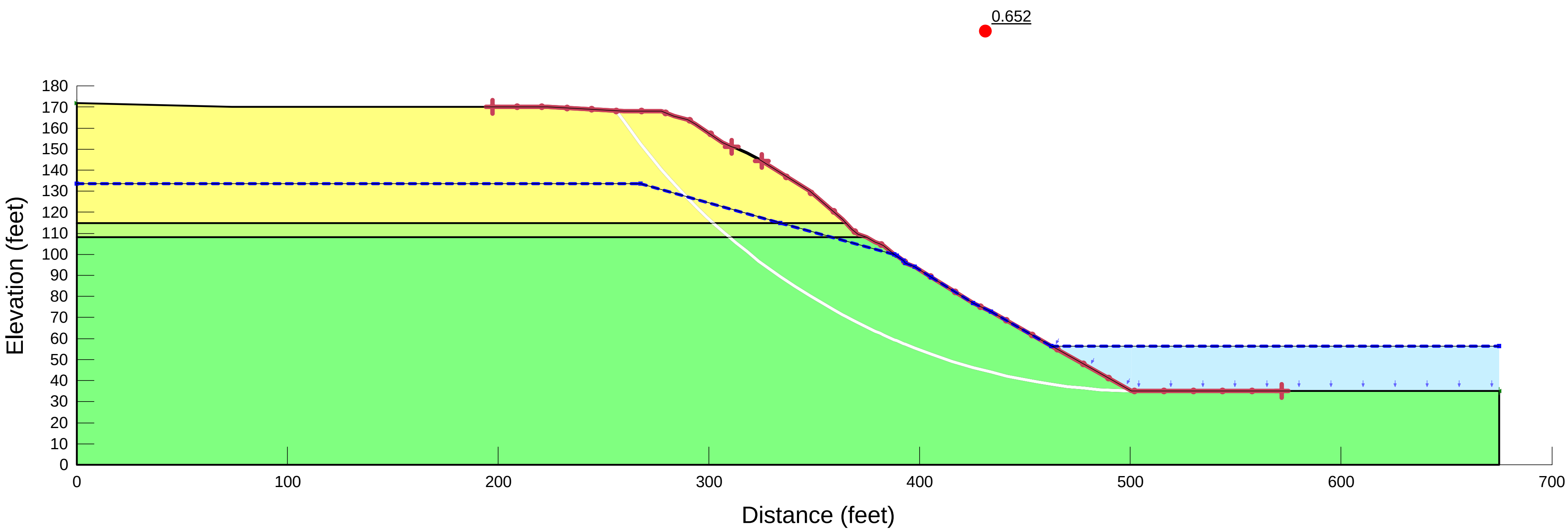
101895 - City of Newberg Water System Resilience
Figure B-1 - Static Slope Stability



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	FG-MFD	Mohr-Coulomb	110	100	28	0	1
	FG-MFD_Clay	Mohr-Coulomb	110	200	30	0	1
	Hillsboro	Mohr-Coulomb	120	600	32	0	1

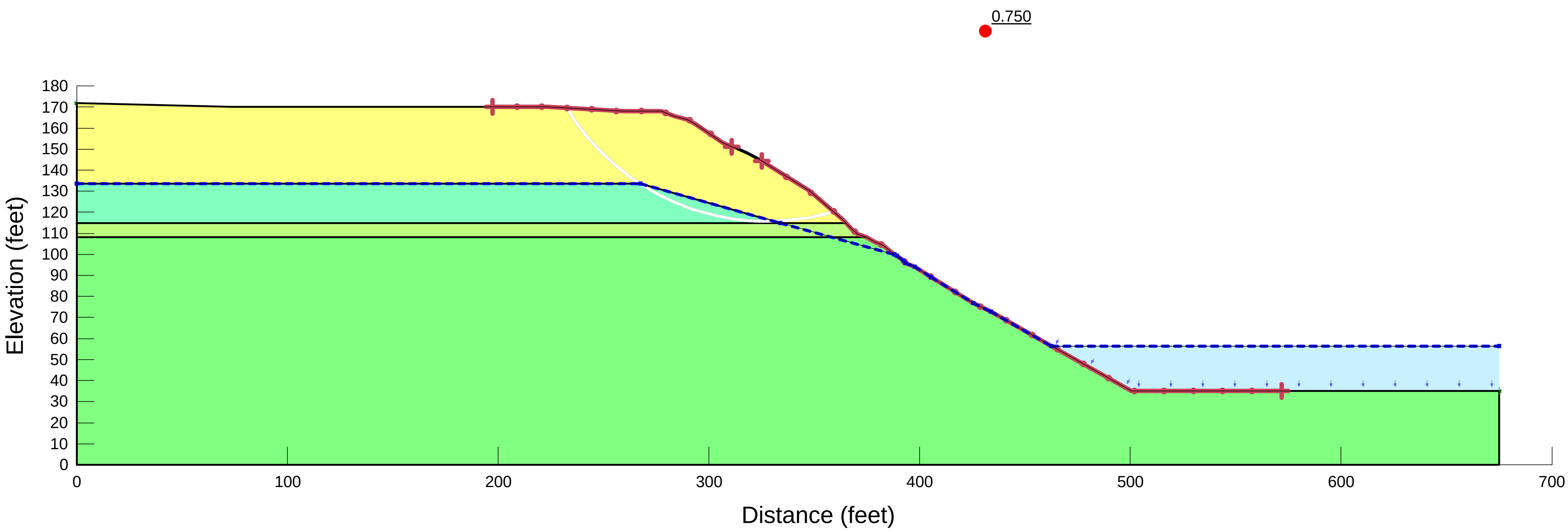
101895 - City of Newberg Water System Resilience
Figure B-2 - Seismic Slope Stability

Horz Seismic Coef.: 0.237



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	FG-MFD	Mohr-Coulomb	110	100	28	0	1
	FG-MFD_Clay	Mohr-Coulomb	110	200	30	0	1
	Hillsboro	Mohr-Coulomb	120	600	32	0	1

101895 - City of Newberg Water System Resilience
Figure B-3 - Post-Seismic Slope Stability



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	FG-MFD	Mohr-Coulomb	110	100	28	0	1
	FG-MFD_Clay	Mohr-Coulomb	110	200	30	0	1
	FG-MFD_Liquefied	Mohr-Coulomb	110	10	4	0	1
	Hillsboro	Mohr-Coulomb	120	600	32	0	1

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining

IMPORTANT INFORMATION

your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims

IMPORTANT INFORMATION

being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Appendix C:
Vulnerability Assessments

WATER SYSTEM SEISMIC RESILIENCE STUDY

CITY OF NEWBERG PUBLIC WORKS DEPARTMENT
NEWBERG, OREGON

Final Technical Memorandum: Seismic Vulnerability Assessment of Water System

July 2nd, 2020
SEFT Project Number: B19009.00

SEFT Consulting Group
4800 SW Griffith Drive, Suite 100
Beaverton, OR 97005

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1.0 Introduction and Background

1.1 City of Newberg Water System Description

The City of Newberg water system currently consists of the City's wellfield, raw water transmission pipelines, water treatment plant, three water storage reservoirs, one pump station, and distribution system pipelines. The entire water service area is one pressure zone, except for approximately 40 customers that are served by the Oak Knoll booster pump station. The system uses approximately 56 miles of distribution pipelines to provide water to business and residential customers within the City of Newberg service area and six small water district wholesale customers. The primary water supply is the City's well field located on the south side of the Willamette River in Marion County. Two raw water transmission mains cross the river to the treatment plant. An under river 30-inch diameter high density polyethylene transmission main can supply 100% of the treatment plant capacity. An older 24-inch diameter cast iron transmission main is supported by a decommissioned highway bridge. The City's water treatment plant is a conventional filtration facility with a nominal capacity of 9 million gallons per day (MGD). The current average day demand for the water system is approximately 2.4 MGD and summertime demands can increase to approximately 4.5 MGD.

1.2 Seismic Resilience Study

Based on recommendations contained in the 2017 City of Newberg Water Master Plan and requirements of the Oregon Health Authority, the City of Newberg is conducting a water system seismic resilience study. This study will evaluate the expected performance of the City water system following a Magnitude 9.0 (M9.0) Cascadia Subduction Zone (CSZ) earthquake and identify preliminary recommendations for improvements that should be implemented to enable the City to more rapidly restore water service after a major earthquake, to meet community social and economic needs. The scope of this seismic resilience study includes:

1. Define water system level of service (LOS) goals for the City water system following a major seismic event;
2. Identify key backbone system components that are required to achieve these LOS goals, including the locations of key supply points for water for fire suppression and community water distribution;
3. Define performance criteria for individual system components that are required to achieve these LOS goals;
4. Conduct a limited geotechnical seismic hazards evaluation for the City water system and slope stability analysis at the water treatment plant site (Shannon & Wilson);
5. Conduct a limited well/pipeline (HDR), and structural/nonstructural (SEFT/HDR) vulnerability assessment to determine estimated system performance following a M9.0 CSZ earthquake;

6. Identify gaps between the LOS goals and current performance estimates; and
7. Develop preliminary mitigation recommendations to close these gaps utilizing new or retrofit infrastructure, changes to design standards, enhancements in emergency response planning, and recommendations for further study.

This Technical Memorandum (TM) presents SEFT's findings related to scope item 5. The components of the water system that have been evaluated by SEFT as part of this effort are summarized in Table 1.1. The locations of these components are illustrated in Figure 1.1. To complete this scope of work, SEFT utilized the Task 2 TM (Seismic Recovery Goals) and Task 3 TM (Seismic Hazards Summary), completed as part of this project, and the as-built drawings indicated in Table 1.2.

Table 1.1 – Summary of Water System Components Evaluated by SEFT

Water System Component	Structure Type	Year of Original Construction
Corral Creek Road Reservoir		
4.0 MG Reservoir	Strand-Wound Circular Prestressed Concrete	2004
North Valley Reservoirs		
4.0 MG Reservoir No.1	Strand-Wound Circular Prestressed Concrete	1961
4.0 MG Reservoir No.2	Strand-Wound Circular Prestressed Concrete	1977
Water Treatment Plant		
Original Treatment/Control Building	Reinforced concrete	pre-1961
1961 Treatment/Control Building Addition	Reinforced concrete	1961
1970 Treatment/Control Building Addition	Reinforced concrete	1970
Sedimentation Basin No.1	Reinforced concrete	1961
Filters No.1 and 2, Filter Gallery, Pump Room, Clearwell, and Filters No. 3 and 4 Addition	Reinforced concrete	1970 1980 (Filters No. 3 and 4)
Sodium Hypochlorite Generation Building	Steel Moment Resisting Frame (North-South) and Steel Brace Frame (East-West)	2005

Table 1.2 – Evaluation Documents

As-Built Drawings	Water System Component
Corral Creek Road Reservoir	
"4.0 Million Gallon Corral Creek Road Reservoir (A2004001)" prepared by CH2MHill, dated April 2002	• Corral Creek Road Reservoir
North Valley Reservoirs	
"North Valley 4.0 MG West Reservoir (A600001)" prepared by Carl E. Green & Associates Consulting Engineers, dated August 1960	• North Valley Reservoir No.1
"Site Work For Reservoir No.2 (A770016)" prepared by Robert E. Meyer Engineers Inc., dated November 1977	• North Valley Reservoir No.2
"North Valley and Corral Creek Reservoirs Seismic Upgrades (A2016007)" prepared by Kennedy/Jenks Consultants, dated September 2015	• Modifications in North Valley Reservoir No.1 • Modifications and seismic upgrade of North Valley Reservoir No.2
Water Treatment Plant	
"Water Treatment Plant (A500002)" prepared by John Cunningham & Associates Consulting Engineers, dated December 1950	• Not applicable (1)
"Water Treatment Plant Addition (A610001)" prepared by Carl E. Green & Associates Consulting Engineers, dated April 1961	• Treatment/Control Building (1961 Addition) • Sedimentation Basin No.1
"Water Treatment Plant (A700004)" prepared by CH2M, dated July 1970	• Treatment/Control Building (1970 Addition) • Filters No.1 and 2, Filter Gallery, Pump Room, and Clearwell
"Water Treatment Plant Expansion (A800027)" prepared by Kramer, Chin & Mayo, Inc. Consulting Engineers, dated July 1980	• Filters No. 3 and 4
"Water Treatment Plant Improvements Project (A2002014)" prepared by MWH, dated September 2002	• Modifications to Filters No. 1 to 4 and Filter Gallery
"Water Treatment Plant Expansion to 9.5 MGD (A2007005)" prepared by CH2MHill, dated March 2005	• Sodium Hypochlorite Generation Building • Modifications to Filters No. 1 to 4, Treatment/ Control Building, and Sedimentation Basin No.1

Notes:
(1) The geometry and location of the structures shown in these drawings are inconsistent with current plant layout.

Figure 1.1 – City of Newberg Water System General Location Map

2.0 Evaluation Methodology and Seismic Performance Objectives

2.1 Seismic Hazard

This evaluation considered a single seismic hazard level associated with a M9.0 scenario earthquake originating on the Cascadia Subduction Zone (CSZ). As part of this project, Shannon and Wilson, Inc. conducted a geotechnical seismic hazard assessment (Shannon & Wilson, 2019). In their report, Shannon & Wilson provided estimates of the spectral acceleration and permeant ground deformation (PGD) for liquefaction-induced settlement, liquefaction-induced lateral spreading, and earthquake-induced landslide associated with the M9.0 CSZ scenario earthquake. This geotechnical data was used as the basis for SEFT's structural evaluation.

2.2 Seismic Performance Objectives

In the initial phase of this project, the HDR/SEFT team worked with the City of Newberg to establish proposed level of service (LOS) goals for the City of Newberg water system following a major earthquake as described in SEFT (2019). The structural and nonstructural performance objectives used for evaluation of water system components for the M9.0 CSZ scenario earthquake were based on these LOS goals and are described in Sections 2.2.1 and 2.2.2.

2.2.1 Structural Performance Objective

Immediate Occupancy: "Immediate Occupancy" refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury from structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.

2.2.2 Nonstructural Performance Objectives

Operational: "Operational" refers to the performance level where most nonstructural systems required for normal use of the building are functional, although minor cleanup and repair of some items might be required. Achieving the Operational nonstructural performance level requires considerations of many elements beyond those that are normally within the sole province of the structural engineer's responsibilities. For Operational nonstructural performance, in addition to ensuring that nonstructural components are properly mounted and braced within the structure, it is often necessary to provide emergency standby equipment to provide utility services from external sources that might be disrupted. It might also be necessary to perform qualification testing to ensure that all necessary equipment will function during or after strong shaking.

2.3 Water System Evaluation Methodology

The seismic structural evaluation of components within the City of Newberg water system was completed using the Tier 1 procedure of ASCE 41-17, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2017b). This Tier 1 procedure uses a checklist-based approach to identify potential seismic structural deficiencies that have been commonly observed in past earthquakes. The Tier 1 procedure also uses quick-check calculations to evaluate potential deficiencies in the primary components of the seismic load resisting system.

However, ASCE 41-17 does not include quick-check calculations and acceptance criteria that are directly applicable to the reservoirs evaluated as part of this study. Therefore, in place of these quick-check calculations, American Water Works Association (AWWA) standard design checks were evaluated for primary components of the seismic load path (circumferential strand, seismic cables, etc.). The calculation of seismic forces acting on the reservoirs has been based on the applicable AWWA standard. Concrete tank seismic loads were based on AWWA D110-13, *Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks* (AWWA, 2013).

Freeboard calculations where completed based on both the applicable AWWA design standard and ASCE 7-16, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2017a). The required freeboard calculated using ASCE 7-16 varies from that calculated using the AWWA standards. This study used the more conservative of the freeboard estimates calculated using both methods. The recommended freeboard calculations used a seismic importance factor equal to 1.0, as indicated in the applicable standards. In order to ensure Immediate Occupancy structural performance for the M9.0 CSZ event, we have increased the calculated freeboard values by a factor equal to 1.5.

The seismic nonstructural evaluation of components within the City of Newberg water system was completed using the nonstructural seismic evaluation checklists presented in ASCE 41-17 supplemented by TCLEE Monograph No. 22, *Seismic Screening Checklists for Water and Wastewater Facilities* (TCLEE, 2002). Similar to the ASCE 41 Tier 1 structural evaluation procedure, this checklist-based evaluation approach is used to identify potential seismic nonstructural deficiencies that have been commonly observed in past earthquakes.

3.0 Expected Seismic Structural and Nonstructural Performance

The expected structural and nonstructural seismic performance of the City of Newberg water system components has been evaluated for a M9.0 CSZ scenario earthquake. Sections 3.1 through 3.4 provide a short narrative description of the water system component evaluated, followed by a table that summarizes the potential seismic structural and nonstructural deficiencies identified by the seismic evaluation using the ASCE 41-17 Tier 1 and TCLEE Monograph No. 22 checklist-based procedures. These sections also include images from the as-built drawings where structural deficiencies are identified and selected photos taken during site visits conducted on August 9th and 16th, 2019.

3.1 Corral Creek Road Reservoir

The Corral Creek Road Reservoir, built in 2004, is a partially buried 4 million-gallon (MG) strand-wound circular prestressed concrete water tank with a nearly flat roof (see Figure 3.1). The tank is 138 ft. in diameter and approximately 40 ft. tall. The roof of the reservoir is supported by circular concrete columns. It is one of the three reservoirs that provide water storage for the city.

The circular concrete wall is reinforced with a combination of mild steel reinforcement, vertical post-tensioning bars and horizontal prestressing strands around the exterior surface to resist internal hydrostatic pressure and seismic forces. A continuous strip footing supports the exterior walls. The connection between the walls and footings is typically composed of a bearing pad and diagonal seismic cables that are anchored into the tank wall and foundation. The seismic cables are de-bonded at the wall to foundation interface. This connection allows the tank to shrink and swell radially, as needed to accommodate varying internal pressure due to changes in the water level inside the tank. The roof is connected to the walls using a series of shear keys constructed using vertical HSS posts designed to prevent the roof from sliding off the structure in an earthquake, but also allows the tank to shrink and swell radially.

An electrical panelboard and SCADA equipment is located adjacent to the reservoir in a metal electrical enclosure. The enclosure is covered by a canopy that is supported by steel tube section cantilever posts, as shown in Figure 3.2.

Table 3.1 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.1, the Corral Creek Road Reservoir is currently expected to achieve Immediate Occupancy structural performance but is not currently expected to achieve Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.1 – Corral Creek Road Reservoir Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Per Shannon & Wilson Report, minimal permanent ground deformation (PGD) is anticipated at the reservoir: 0 inches liquefaction induced settlement, 0-0.1 inches liquefaction-induced lateral spreading, and approximately 0.5 feet earthquake-induced landslide PGD near slope 100 feet from reservoir. This level of PGD is not anticipated to cause significant structural damage to the reservoir. However, the impact of earthquake-induced landslide PGD should be considered as a potential hazard for the buried pipelines that connect to the reservoir and are located in the potential landslide zone. • None Identified.
Nonstructural	• SCADA system backup batteries inside metal enclosure are not restrained. See Figure 3.3.

Figure 3.1 – Corral Creek Road Reservoir

Figure 3.2 – Electrical Panelboard and SCADA Equipment Enclosure and Canopy

Figure 3.3 – Unrestrained Backup Batteries

3.2 North Valley Reservoir No. 1

North Valley Reservoir No. 1, built in 1960, is a partially buried 4 MG strand-wound circular prestressed concrete water tank with a concrete dome roof, as shown in Figure 3.4. The tank is 144 ft. in diameter by approximately 52 ft. tall (at the dome center). At the middle of the reservoir, there is a 90 ft. diameter flat bottom slab that transitions to a sloped reservoir bottom (2 horizontal to 1 vertical) up to the top of the wall footing, approximately 13.5 ft. above the flat slab elevation, as can be observed in Figure 3.5. The maximum water surface is approximately 17 ft below the center of the dome, and 1 ft above the top of the walls. It is one of the three reservoirs that provide water storage for the city.

The circular concrete wall is reinforced with a combination of mild steel reinforcement, vertical post-tensioning bars and horizontal prestressing strand around the exterior surface to resist internal pressure. A continuous strip footing supports the exterior walls. The connection between the wall and footing is typically composed of a bearing pad and diagonal seismic cables that are anchored into the tank wall and foundation. The seismic cables are de-bonded at the wall to foundation interface. This connection allows the tank to shrink and swell radially, as needed to accommodate varying internal pressure due to changes in the water level inside the tank. The dome is anchored to the wall by 1 in diameter galvanized bolts (eight, equally spaced) with rubber pads in the interface.

An electrical panelboard, SCADA equipment, and analyzer equipment are located in the former Chlorination Building at the site, as shown in Figure 3.6. The building is a single-story minimally reinforced masonry wall structure with a straight-sheathed wood roof diaphragm.

Table 3.2 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.2, the North Valley Reservoir No.1 is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Additionally, based on the potential deficiencies identified in this assessment, the former Chlorination Building is not currently expected to achieve Life Safety performance and represents a safety hazard to City staff and contractors.

Table 3.2 – North Valley Reservoir No. 1 Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Per Shannon & Wilson Report, minimal permanent ground deformation (PGD) is anticipated at the reservoir: 0.5-1.5 inches liquefaction induced settlement, 0-0.1 inches liquefaction-induced lateral spreading, and approximately 2 feet earthquake-induced landslide PGD near slope 150 feet from reservoir. This level of PGD may cause structural damage to and/or leaking of the reservoir. Additionally, the impact of earthquake-induced landslide PGD should be considered as a potential hazard for the buried pipelines that connect to the reservoir and are located in the potential landslide zone. • The number of dome anchors (8 anchors) is insufficient to transfer the expected seismic forces from the dome to the reservoir walls. See Figure 3.7. • The existing capacity of the horizontal prestressing on the wall of the reservoir is insufficient to resist the combination of hydrostatic and expected hydrodynamic hoop forces during the earthquake. • The seismic cables provided at the base of the wall are insufficient to resist the expected hydrodynamic forces at the base of the reservoir during an earthquake.
Nonstructural	• Reservoir vertical inlet nozzles are not braced and may not be adequate to resist earthquake-induced hydrodynamic forces. See Figure 3.8. • SCADA system and chemical analyzer equipment that is used for monitoring of reservoirs is located in the former Chlorination Building that would likely not perform well during an earthquake. • SCADA system backup batteries in the former Chlorinator Building are not adequately restrained to prevent movement during an earthquake. See Figure 3.9. • Friction Clips are used to restrain the SCADA antenna, see Figure 3.10. However, friction clips are generally not considered to be reliable to resist earthquake-induced forces.

Figure 3.4 – North Valley Reservoir No. 1

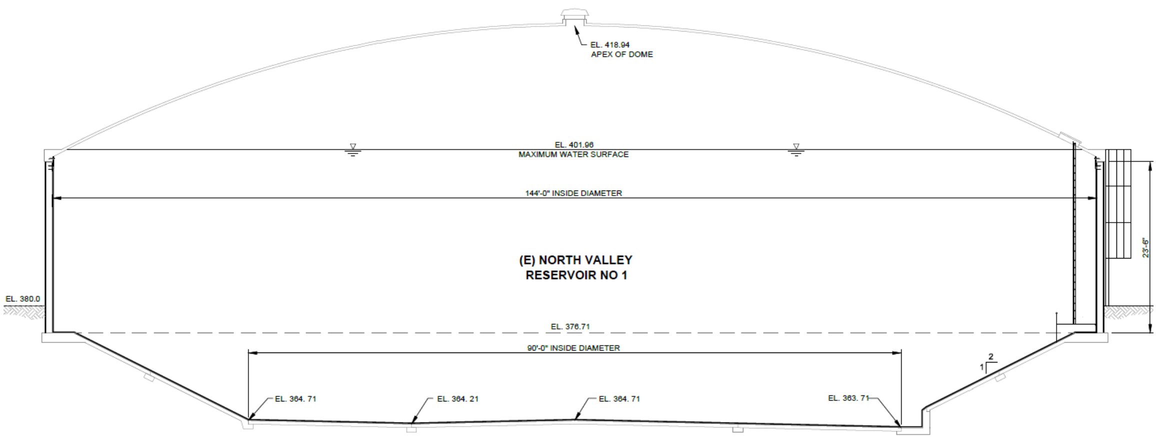


Figure 3.5 – North Valley Reservoir No. 1 Cross-Section
(Source Drawings: "North Valley and Corral Creek Reservoirs Seismic Upgrades (A2016007)")

Figure 3.6 – Former Chlorination Building

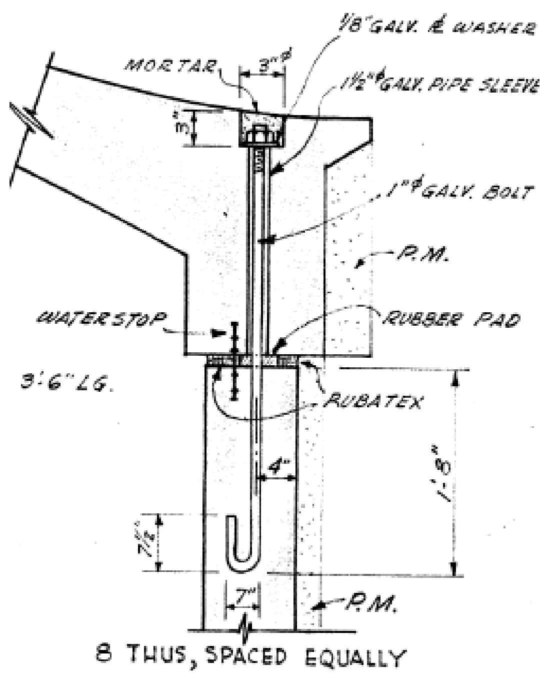


Figure 3.7 – Dome Anchor Detail
(Source Drawings: "North Valley 4.0 MG West Reservoir (A600001)")

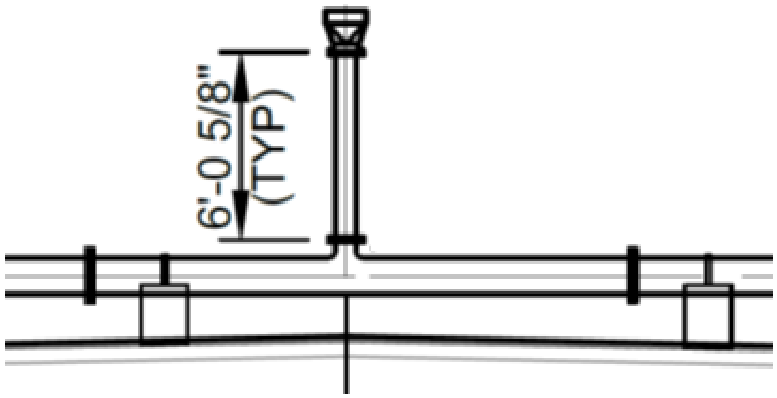


Figure 3.8 – Reservoir No. 1 Vertical Inlet Nozzles not Braced to Structure (Source Drawings: "North Valley and Corral Creek Reservoirs Seismic Upgrades (A2016007)")

Figure 3.9 – Backup Batteries not Adequately Restrained

Figure 3.10 – SCADA Antenna Supported with Friction Clips

3.3 North Valley Reservoir No. 2

North Valley Reservoir No. 2 is a partially buried 4 MG strand-wound circular prestressed concrete water tank with a concrete dome roof (see Figure 3.11). The reservoir was originally constructed in 1977 and seismically upgraded in 2015. The tank is 151 ft. in diameter by approximately 47 ft. tall (by the dome center). The maximum water surface is approximately 17 ft below the center of the dome. It is one of the three reservoirs that provide water storage for the city.

The circular concrete wall is reinforced with a combination of mild steel reinforcement, vertical post-tensioning bars and horizontal prestressing strand around the exterior surface to resist internal pressure. A continuous strip footing supports the exterior walls. The connection between the wall and footing is typically composed of a bearing pad and diagonal seismic cables that are anchored into the tank wall and foundation. The seismic cables are de-bonded at the wall to foundation interface. This connection allows the tank to shrink and swell radially, as needed to accommodate varying internal pressure due to changes in the water level inside the tank. The dome is connected to the walls through a continuous shear key to prevent the roof from sliding off the structure.

The recent seismic upgrade included providing additional horizontal prestress strands over the height of the ring beam at the top of the reservoir wall and strengthening the wall to foundation connection at 148 locations around the inside perimeter of the tank to prevent the reservoir from sliding during an earthquake. Design calculations from this 2015 seismic upgrade by Kennedy/Jenks were not available for SEFT's review as part of this seismic vulnerability assessment.

Table 3.3 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.3, the North Valley Reservoir No. 2 is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.3 – North Valley Reservoir No. 2 Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Per Shannon & Wilson Report, minimal permanent ground deformation (PGD) is anticipated at the reservoir: 0.5-1.5 inches liquefaction induced settlement, 0-0.1 inches liquefaction-induced lateral spreading, and approximately 2 feet earthquake-induced landslide PGD near slope 150 feet from reservoir. This level of PGD may cause structural damage to and/or leaking of the reservoir. Additionally, the impact of earthquake-induced landslide PGD should be considered as a potential hazard for the buried pipelines that connect to the reservoir and are located in the potential landslide zone. • The existing capacity of the horizontal prestressing on the wall of the reservoir is insufficient to resist the combination of hydrostatic and expected hydrodynamic hoop forces during the earthquake, when neglecting the contribution of the soil passive earth pressure.
Nonstructural	• Same as North Valley Reservoir No. 1, see Table 3.2. See Figure 3.12 related to the unbraced inlet nozzles inside the reservoir.

Figure 3.11 – North Valley Reservoir No.2

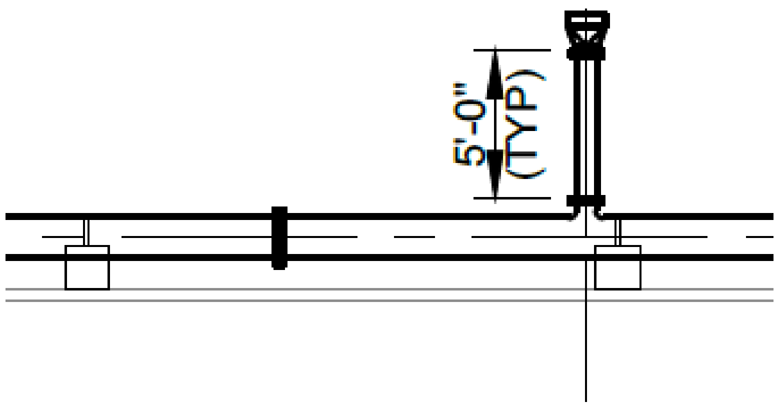


Figure 3.12 – Reservoir No. 2 Vertical Inlet Nozzles not Braced to Structure (Source Drawings: "North Valley and Corral Creek Reservoirs Seismic Upgrades (A2016007)")

3.4 Water Treatment Plant

The City of Newberg Water Treatment Plant (WTP) receives raw water from the well field located across the Willamette River, and after treatment, finished water is pumped to the distribution system and the City's three finished water reservoirs. The WTP is located approximately 2.5 miles southwest of Corral Creek Road Reservoir and approximately 3.4 miles south-southeast of North Valley Reservoirs.

The WTP consists of the following buildings and process units (those shown in bold text were included in the scope of the current seismic vulnerability assessment), as illustrated in Figure 3.13:

- **Original Treatment/Control Building (pre-1961)**
- **1961 Treatment/Control Building Addition**
- **1970 Treatment/Control Building Addition**
- **Sedimentation Basin No. 1 (North)**
- Sedimentation Basin No. 2 (South)
- **Filters No. 1 to 4, Filter Gallery, Pump Room, and associated Clearwell**
- Filter No. 5 and 6, and associated Clearwell
- **Sodium Hypochlorite Generation Building**
- Sodium Hydroxide Building
- Backwash Basin

The City of Newberg WTP was originally built prior to 1961. Available drawings from 1950 show structures with a geometry and layout that is inconsistent with the current plant configuration. Drawings from 1961 show a portion of the Treatment/Control Building and Sedimentation Basin No. 2 (south basin) as existing structures. It is assumed that these structures were constructed after 1950 and prior to 1961. The original plant had a capacity of approximately 1 MGD. Several plant upgrades and expansions have occurred since original construction to increase the plant capacity to 9.5 MGD. These upgrade and expansion projects have included:

- Treatment/Control Building Addition and Sedimentation Basin No. 1 (north basin) were constructed in 1961;
- A second Treatment/Control Building Addition, Filters No.1 and 2, Filter Gallery, Pump Room, and Clearwell were constructed in 1970;
- Filters No. 3 and 4 were constructed in 1980;
- Sodium Hydroxide Building was constructed in 2002; and
- Sodium Hypochlorite Generation Building and Filters No. 5 and 6 (with associated expansion of the Clearwell and Filter Gallery) were constructed in 2005.

A number of these treatment plant structures were constructed in close proximity to other structures and lack an adequate seismic joint (i.e., gap) to prevent potential pounding between the adjacent structures. Differential response of the adjacent structures during an earthquake would likely result in pounding between the structures that would cause localized damage to one or both adjacent structures. The seismic vulnerability assessment summaries in the following sections indicate where lack of an adequate seismic joint between adjacent structures has been identified as a potential deficiency.

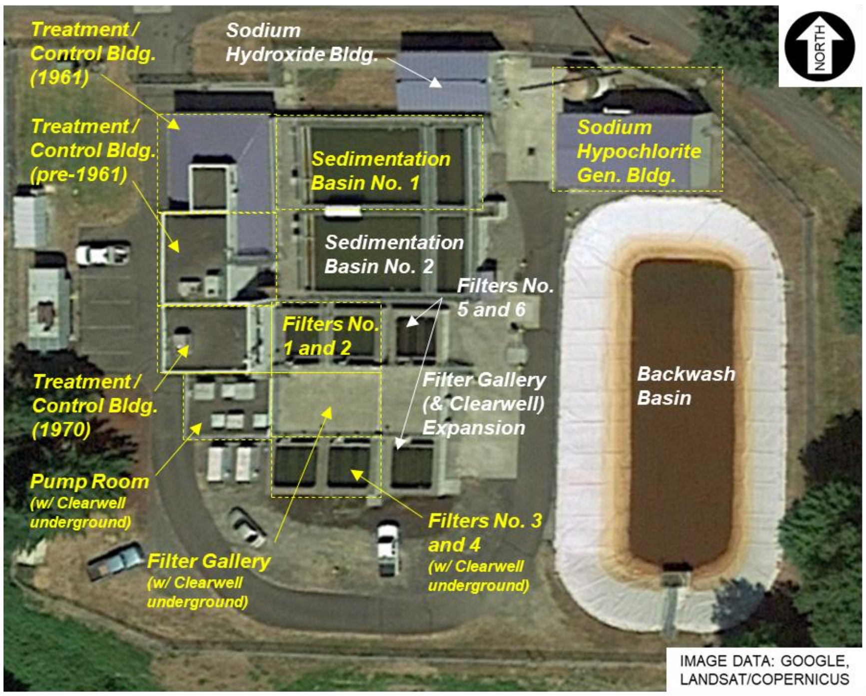


Figure 3.13 – Newberg Water Treatment Plant Location Map

3.4.1 Original Treatment/Control Building (pre-1961)

The Treatment/Control Building was originally constructed prior to 1961 and is located on the west side of the treatment plant. The Original Treatment/Control Building (pre-1961), is shown in Figure 3.14. The building is a two-story reinforced concrete shear wall building with reinforced concrete floor and roof diaphragms.

In 1961, an addition was constructed on the north side of the Original Treatment/Control Building (pre-1961). In 1970, a second addition was constructed, this time on the south side of the Original Treatment/Control Building (pre-1961). Both additions were constructed to be seismically independent of the Original Treatment/Control Building (pre-1961), however the joint width was specified to be ¾ inch or less.

Currently the ground level of the Original Treatment/Control Building (pre-1961) is used to house the polymer feed system, a pipe gallery for the raw water pipeline feeding Sedimentation Basin No. 2, and miscellaneous storage. The second level contains electrical equipment and motor control centers for the majority of the plant.

Structural drawings were not available for the Original Treatment/Control Building and development of as-built drawings was beyond the scope of this study. Potential structural deficiencies identified by this assessment have been based on field observations and general knowledge of typical construction practices during the era of original construction. Table 3.4 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.4, the Original Treatment/Control Building (pre-1961) is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Additionally, based on the potential deficiencies identified in this assessment, the Original Treatment/Control Building (pre-1961) is not currently expected to achieve Life Safety performance and represents a safety hazard to City staff and contractors.

Table 3.4 – Original Treatment/Control Building (pre-1961) Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Per Shannon & Wilson Report, significant permanent ground deformation (PGD) is anticipated near the WTP: 0.5-1.5 inches liquefaction induced settlement, approximately 16 inches liquefaction-induced lateral spreading near slope 120 feet from plant, approximately 20 feet earthquake-induced landslide PGD near slope 120 feet from plant. This level of PGD could potentially cause structural damage to WTP buildings and process units and also damage associated buried piping. Additional geotechnical and structural assessment is recommended to more accurately characterize the level of PGD anticipated to occur at the WTP and evaluate the ability of structures and buried pipelines to accommodate this level of PGD. • A large L-shaped diaphragm opening (stairs) is located at the northwest corner of the building adjacent to both the north and west shear walls. This opening significantly reduces the ability of the diaphragm to transfer seismic forces to the walls. See Figure 3.15. • Concrete columns are not likely to satisfy deformation compatibility requirements due to inadequate tie spacing. • It is likely that the diaphragm to shear wall connection does not have adequate capacity to develop the lesser of the shear strength of the walls or diaphragms. • Several potential deficiencies are likely that are associated with detailing requirements for reinforcing steel (reinforcing ratio, foundation dowels, and wall and diaphragm reinforcing at openings). • The width of the seismic joints between the Original Treatment/Control Building, and the 1961 and 1970 Additions are not adequate to prevent potential pounding between these adjacent structures. See Figure 3.16.

Table 3.4 – Original Treatment/Control Building (pre-1961) Seismic Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Pipes that penetrate concrete walls do not have adequate flexibility through the wall to accommodate the relative movement between the wall and the pipes. See Figure 3.17. • The raw water piping and valves are not adequately seismically braced. See Figure 3.18. • Vertical pipes are not adequately braced to the structure to resist seismic forces and do not have adequate flexibility to accommodate inter-story drift. See Figure 3.19. • Large chemical storage containers/drums are not restrained. See Figure 3.20. • Rolling carts are not restrained. See Figure 3.21. • A cabinet is improperly anchored to an electrical conduit with a U-bolt. See Figure 3.22. • Storage racks are not restrained. See Figure 3.23. • Mechanical ducts are unbraced. See Figure 3.24. • In-line fan unit is not braced in the direction parallel to the wall. See Figure 3.25. • It is unknown if adequate dowels are provided between the electrical cabinet housekeeping pads and floor slab. • Large diameter electrical conduits are not braced and flexible connections are not provided between the conduit and the top of the electrical cabinets. See Figure 3.26. • At least one of the electrical cabinets appears to be missing anchors at the base of the cabinet. See Figure 3.27. • Vertical cast iron roof drain in Electrical Room is not braced to structure and does not have adequate flexibility to accommodate inter-story drift. Potential failure could cause water intrusion and consequent damage to electrical equipment. See Figure 3.28. • Lights on pendant supports are not braced and may potentially swing and cause damage to other components. Some light fixtures do not include lens covers to prevent the light tubes from falling. See Figure 3.29. • Refrigerator and filing cabinets adjacent to walkway are not restrained. See Figure 3.30.

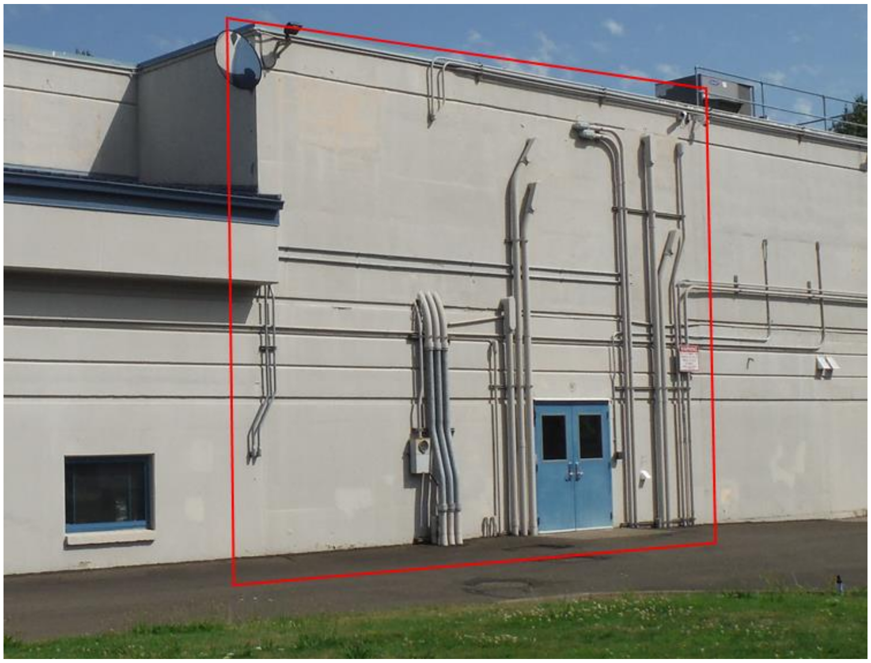


Figure 3.14 – Original Treatment/Control Building (pre-1961)

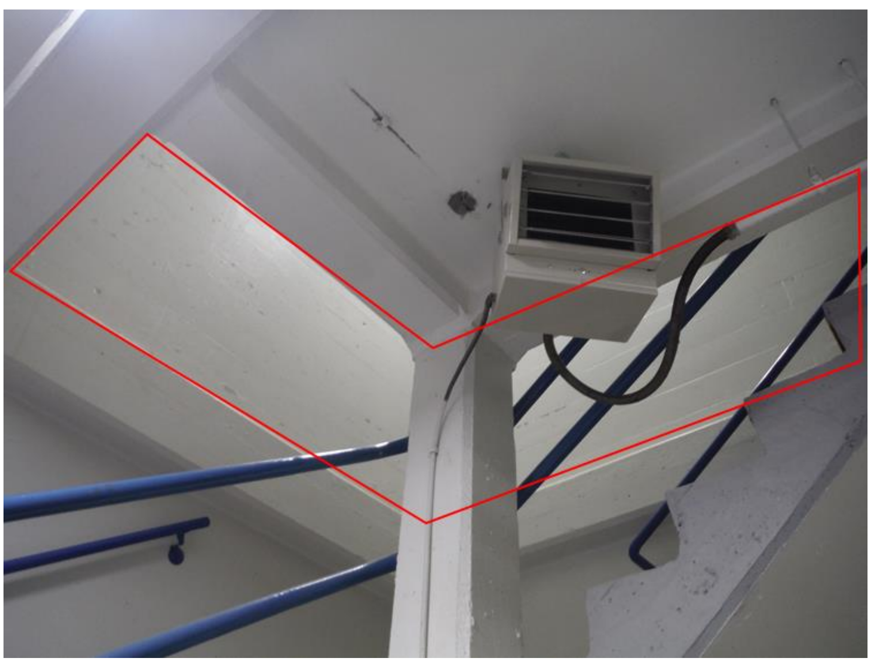


Figure 3.15 – Large Diaphragm Opening Adjacent to Shear Walls

Figure 3.16 – Seismic Joint Between Original Treatment/Control Building (pre-1961) and 1961 Addition

Figure 3.17 – Concrete Wall Penetration by Raw Water Pipe

Figure 3.18 – Raw Water Piping System without Adequate Bracing

Figure 3.19 – Vertical Pipe without Lateral Restraint

Figure 3.20 – Unrestrained Chemical Storage Containers

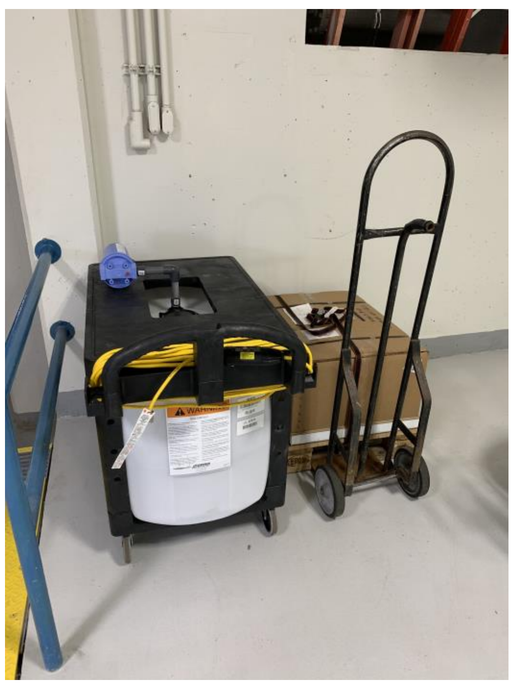


Figure 3.21 – Unrestrained Rolling Carts

Figure 3.22 – Storage Cabinet Restrained with U-Bolt to Electrical Conduits

Figure 3.23 – Unrestrained Storage Rack

Figure 3.24 – Mechanical Ducts not Braced to Structure

Figure 3.25 – In-Line Fan Unit Unrestrained to Movement Parallel to Wall

Figure 3.26 – Electrical Conduits not Seismically Braced and without Flexible Connections to Cabinets

Figure 3.27 – Electrical Cabinets with Missing Anchor at the Base

Figure 3.28 – Unbraced Cast Iron (Brittle) Vertical Pipe next to Electrical Cabinet

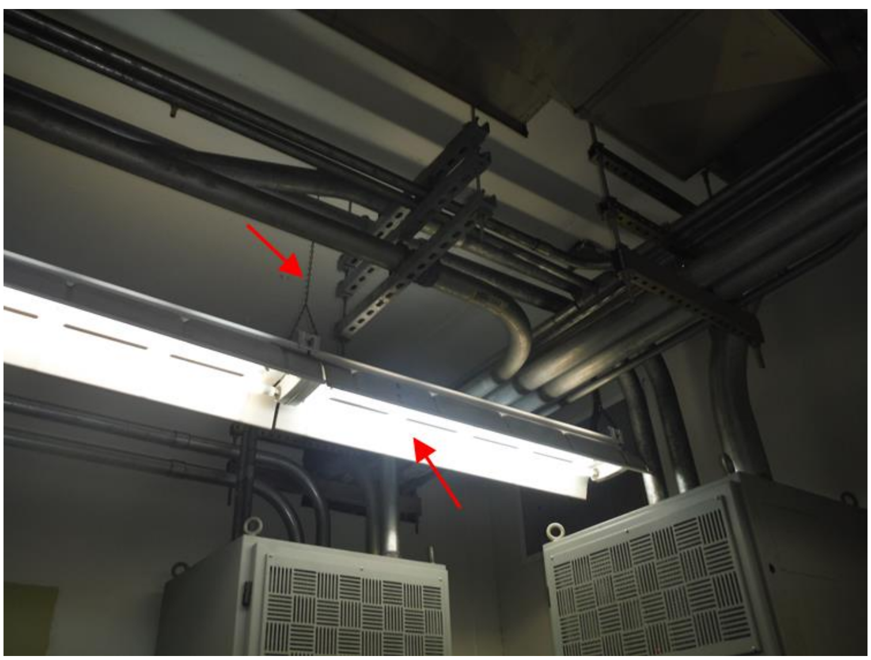


Figure 3.29 – Lights on Pendant Supports not Restrained and without Lens Covers

Figure 3.30 – Unrestrained Refrigerator and Filing Cabinets Adjacent to Walkway

3.4.2 1961 Treatment/Control Building Addition

In 1961, a Treatment/Control Building Addition was constructed on the north side of the Original Treatment/Control Building and west of Sedimentation Basin No. 1 (see Figure 3.31). The 1961 Treatment/Control Building Addition is a two-story reinforced concrete shear wall structure with reinforced concrete floor and roof diaphragms. The lower level of the structure is partially buried and supports abandoned coke beds (formerly used as part of the treatment process).

This 1961 Addition was constructed on the north side of the Original Treatment/Control Building (pre-1961). The addition was constructed to be seismically independent of the Original Treatment/Control Building (pre-1961), however the joint width was specified to be ¾ inch or less.

Currently the 1961 Treatment/Control Building Addition is used as a storage room/shop on the ground level, and an office area on the second floor.

Table 3.5 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.5, the 1961 Treatment/Control Building Addition is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Additionally, based on the potential deficiencies identified in this assessment, the 1961 Treatment/Control Building Addition is not currently expected to achieve Life Safety performance and represents a safety hazard to City staff and contractors.

Table 3.5 – 1961 Treatment/Control Building Addition Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4. • Second story concrete shear walls are not continuous to the foundation. See Figure 3.32 • Concrete columns do not satisfy deformation compatibility requirements due to inadequate tie spacing. • There is only one shear wall line in the east-west direction that is continuous to the foundation (Figure 3.32) resulting in deficient load path, lack of redundancy, potential torsional issues, and lack of adequate diaphragm chords. • The second floor level is comprised of a split-level diaphragm. See Figure 3.32. • The width of the seismic joint between the Original Treatment/Control Building and the 1961 Addition is not adequate to prevent potential pounding between these adjacent structures.
Nonstructural	• Storage racks and shelves are not anchored or braced. See Figure 3.33. • Heavy contents (porta-torch gas cylinders and small air compressor) are stored on top shelves (more than 4 feet above floor level) without restraint. See Figure 3.34. • Computer equipment is unrestrained. See Figure 3.35.

Figure 3.31 – 1961 Treatment/Control Building Addition

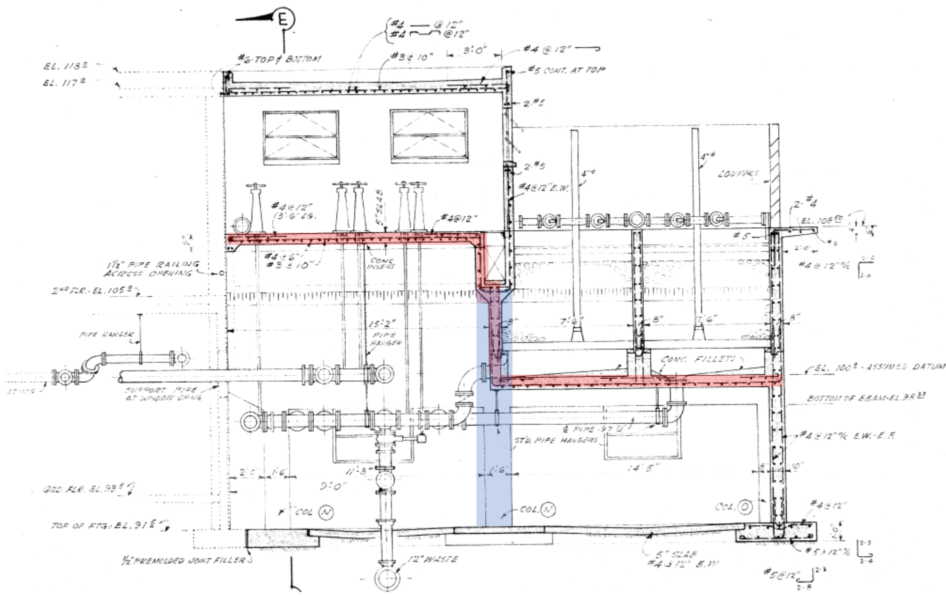


Figure 3.32 – Shear Wall not Continuous to Foundation (Blue Shaded) and with Split Level Diaphragms (Red Shaded)
(Source Drawings: "Water Treatment Plant Addition (A610001)")

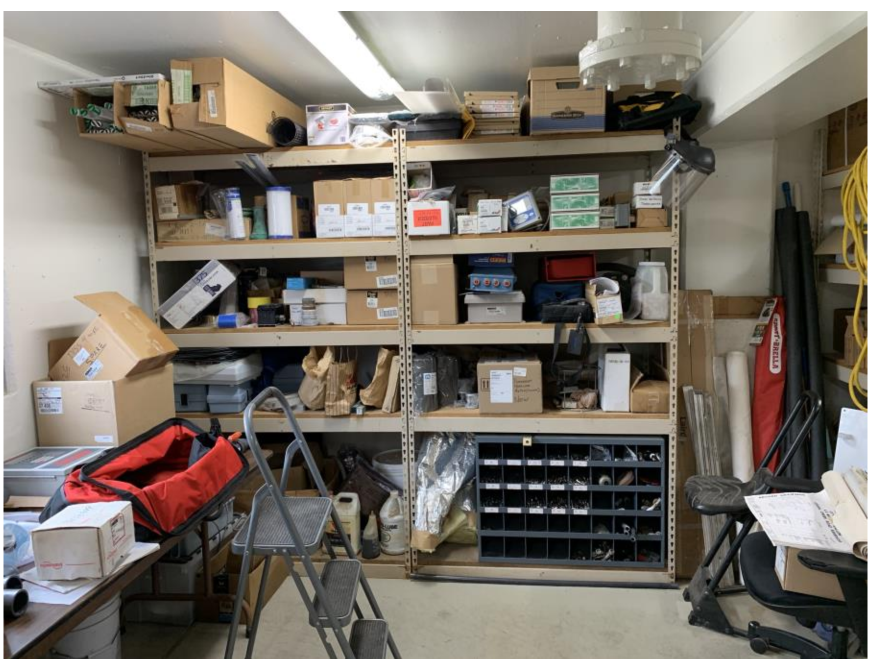


Figure 3.33 – Unrestrained Storage Rack

Figure 3.34 – Porta-Torch Gas Cylinders and Air compressor Stored on Top Shelf

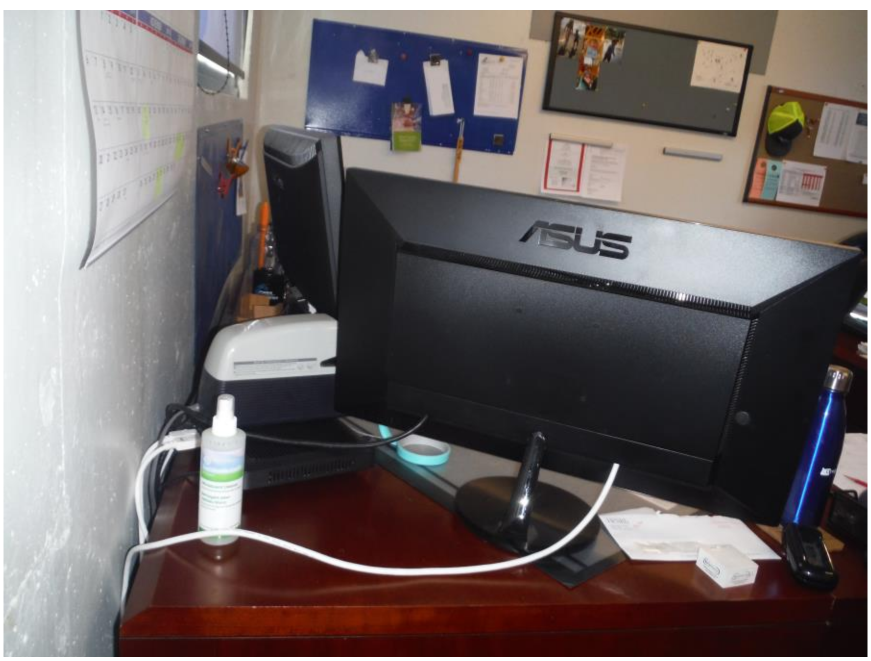


Figure 3.35 – Unrestrained Computer Equipment

3.4.3 1970 Treatment/Control Building Addition

In 1970, a Treatment/Control Building Addition was constructed on the south side of the Original Treatment/Control Building and west of Filters No. 1 and 2 (see Figure 3.36). The south wall of the 1970 Treatment/Control Building Addition is shared by the Pump Room, that was also constructed at the same time. The 1970 Treatment/Control Building Addition is a two-story reinforced concrete shear wall structure with a reinforced concrete diaphragm at the second floor level and a wood (straight-sheathed) roof diaphragm.

This 1970 Addition was constructed on the south side of the Original Treatment/Control Building (pre-1961). The addition was constructed to be seismically independent of the Original Treatment/Control Building (pre-1961), however the joint width was specified to be ¾ inch or less.

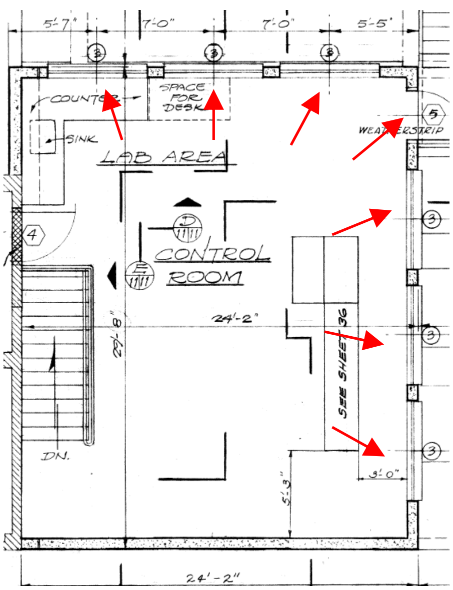
Currently the 1970 Treatment/Control Building Addition contains restrooms, and a hallway at the ground level and plant control room, office and laboratory spaces on the second floor.

Table 3.6 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.6, the 1970 Treatment/Building Addition is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Additionally, based on the potential deficiencies identified in this assessment, the 1970 Treatment/Control Building Addition is not currently expected to achieve Life Safety performance and represents a safety hazard to City staff and contractors.

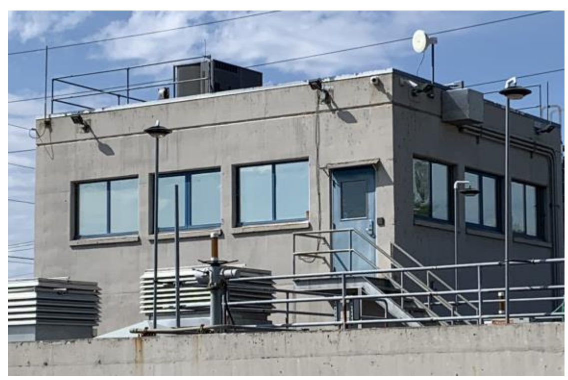
Table 3.6 – 1970 Treatment/Control Building Addition Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4. • Concrete columns do not satisfy deformation compatibility requirements due to inadequate tie spacing. • There is only one shear wall line in the east-west direction, resulting in a deficient load path, lack of redundancy, potential torsional issues, and lack of adequate diaphragm chords. • Between the second floor and the roof there is a significant reduction in the cross-sectional area of the south and east shear walls due to the existing windows and door. See Figure 3.37. • The roof diaphragm lacks adequate cross ties between flexible diaphragm chords. See Figures 3.38. • In the north-south direction (perpendicular to glulam members) there does not appear to be an adequate load path to transfer seismic forces from the roof diaphragm to the concrete shear walls. See Figure 3.39. • The roof diaphragm is not attached to the concrete shear walls with connections that are adequate to resist the expected out-of-plane forces. Additionally, the ledgers that supports the roof straight sheathing on the north and south sides of the buildings are potentially subjected to cross grain bending when resisting wall out-of-plane anchorage forces. See Figure 3.40. • The width of the seismic joint between the Original Treatment/Control Building and the 1970 Addition is not adequate to prevent potential pounding between these adjacent structures.
Nonstructural	• The CMU partition walls around the restrooms are constructed tight to the adjacent concrete beams and walls without an adequate separation to prevent them from unintentionally participating in resisting seismic loads. See Figure 3.41. • Computer equipment is unrestrained. See Figure 3.42. • Several pieces of equipment on the lab counter are unrestrained. See Figure 3.43. • Chemical cabinets doors are not properly latched to prevent accidental opening during an earthquake. See Figure 3.44. • Water heater is not adequately restrained. See Figure 3.45. • Light fixtures are supported by the ceiling grid and lack proper independent support. See Figure 3.46. • The suspended ceiling system is not adequately braced to the structure. See Figure 3.46.

Figure 3.36 – 1970 Treatment/Control Building Addition



(a) Architectural Plan View of Control Room (Source Drawings: "Water Treatment Plant (A700004)"



(b) Outside View of Control Room East and South Walls

Figure 3.37 – Reduction of Shear Walls Cross Section Due to Presence of Windows and Door

Figure 3.38 – Flexible Diaphragm Chords without Cross Ties

Figure 3.39 – Joist to Perpendicular Wall Connection

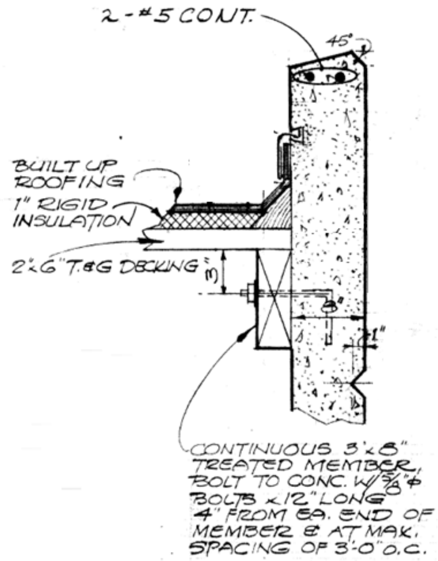
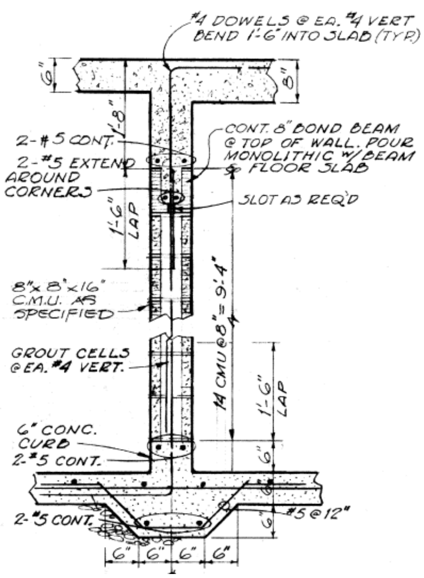
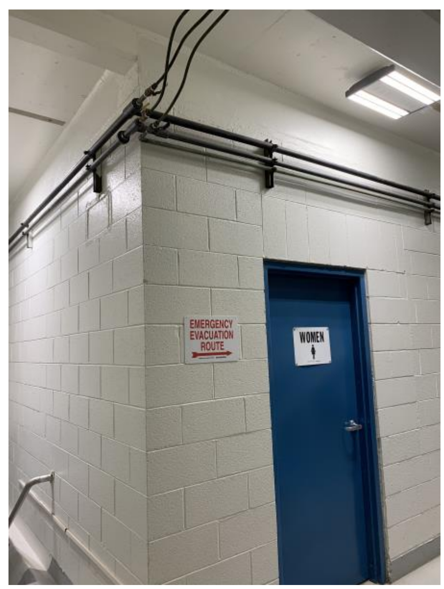


Figure 3.40 – Detail of Joist to Adjacent Wall Connection (Source Drawings: "Water Treatment Plant (A700004)")



(a) Detail of CMU Wall to RC Beam Connection (Source Drawings: "Water Treatment Plant (A700004)")



(b) CMU Wall Partitions

Figure 3.41 – CMU Wall Partitions not Isolated from Structure

Figure 3.42 – Unrestrained Computer Equipment

Figure 3.43 – Unrestrained Equipment on Lab Counter

Figure 3.44 – Chemical Cabinet Doors without Proper Latches

Figure 3.45 – Water Heater Tank not Adequately Restrained

Figure 3.46 – Light Fixture Supported by Ceiling Grid

3.4.4 Sedimentation Basin No. 1

Sedimentation Basin No.1, shown in Figure 3.47, was built in 1961 and is located north of Sedimentation Basin No.2. Sedimentation Basin No.1 has reinforced concrete shear walls around the perimeter. The center wall between Sedimentation Basin No. 1 and 2 is shared by both basins. In the basin, there are a wood baffle near the west end to still the flow into the basin and three steel weirs crossing the basin in the north-south direction near the east end to convey water to the collector trough.

Sedimentation Basin No. 1 was constructed around 1970 on the north side of Sedimentation Basin No. 2 (pre-1961). The addition was constructed to be seismically independent of the Original Treatment/Control Building (pre-1961), however the joint width was specified to be ½ inch.

Structural drawings were not available for Sedimentation Basin No. 2 (i.e. the structure that forms the south wall of Sedimentation Basin No. 1) and development of as-built drawings was beyond the scope of this study. Potential structural deficiencies identified by this assessment have been based on field observations and general knowledge of typical construction practices during the era of original construction. Table 3.7 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.7, Sedimentation Basin No.1 is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.7 – Sedimentation Basin No. 1 Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4. • The width of the seismic joint between Sedimentation Basins No. 1 and 2 is not adequate to prevent potential pounding between these adjacent structures. See Figure 3.48. • Insufficient freeboard (approximately7 in) to accommodate sloshing waves, which may potentially overtop the basin and enter the Sodium Hydroxide Building through air vents in the south wall of the building. See Figure 3.49. • Seismic joints were detailed to include a copper water stop, but potential water leaks may occur due to relative movement between Sedimentation Basins No. 1 and 2, and the effluent structure (built in 1970). See Figure 3.50. • The Basin perimeter walls are potentially overstressed by earthquake-induced hydrodynamic forces and will likely be damaged during an earthquake.
Nonstructural	• Wooden baffles may not have adequate strength to resist hydrodynamic forces. See Figure 3.51. • Small diameter anchors used to connect the weir troughs to the basin walls may not be adequate to resist hydrodynamic forces. See Figure 3.52. • Pipes that penetrate concrete walls may not have adequate flexibility to accommodate the relative movement between the wall and the pipes. See Figure 3.53.

Figure 3.47 – Sedimentation Basin No. 1 Structure

Figure 3.48 – Construction Joint Between Sedimentation Basins No. 1 (1961 Construction) and No. 2 (pre-1961 Construction)

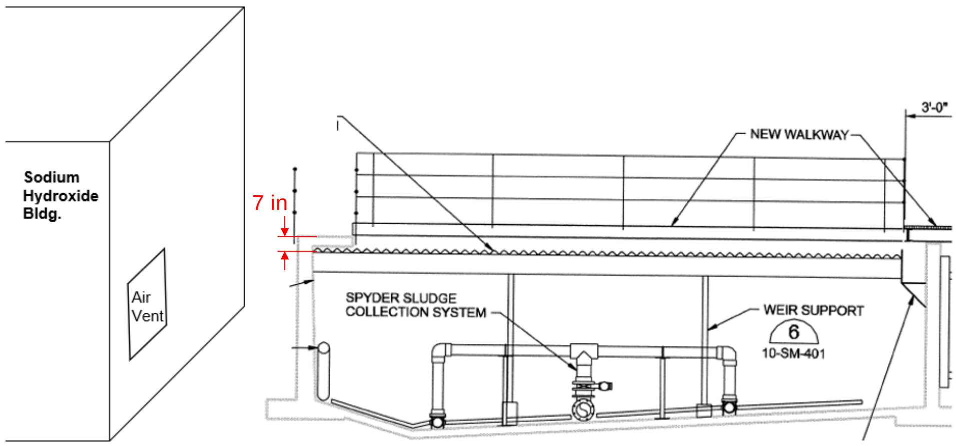


Figure 3.49 – Insufficient Freeboard (~7 in) to Accommodate Sloshing Waves in Sedimentation Basin Near Sodium Hydroxide Building

Figure 3.50 – Sedimentation Basins Effluent Structure (Outlet Basin Structure)

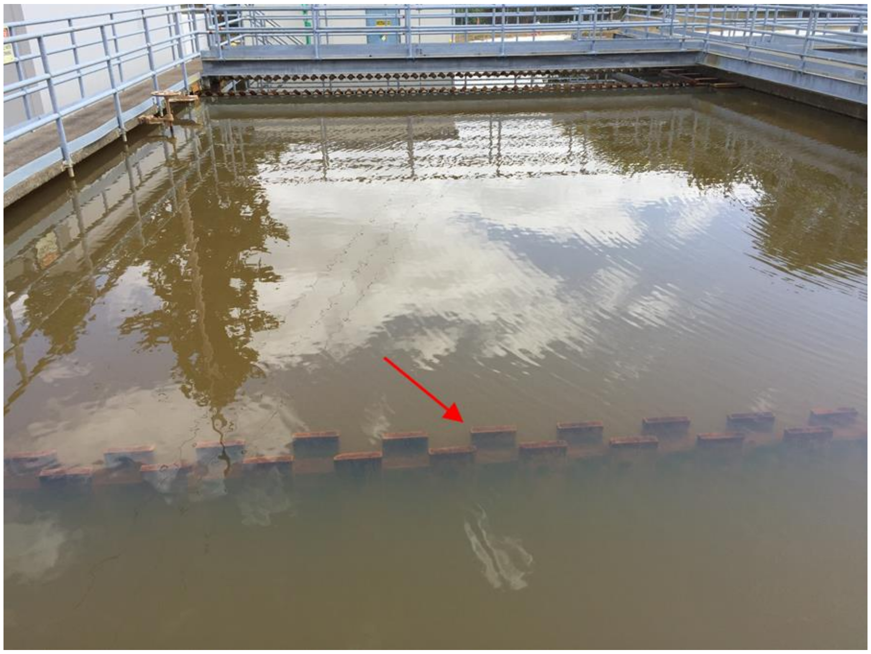


Figure 3.51 – Wooden Baffles in Sedimentation Basin No. 1

Figure 3.52 – Weir Trough to Basin Structure Connection Using Small Diameter Anchors

Figure 3.53 – Raw Water Pipes Penetrating Concrete Wall without Adequate Flexibility Through Wall

3.4.5 Filters No.1 to 4, Filter Gallery, Pump Room, and Associated Clearwell

Filters No.1 and 2, the Filter Gallery, the Pump Room, and the associated Clearwell were constructed in 1970. Filters No. 3 and 4 were added in 1980. Figure 3.54 shows the Filters No. 1 to 4 and the concrete roof slab over the Filter Gallery. Figure 3.55 shows the exterior of the partially buried Pump Room. Filters No. 1 and 2 are located east of the 1970 Treatment/Control Building Addition and south of Sedimentation Basin No. 2. The Filter Gallery is located south of Filters No.1 and 2 and north of Filters No. 3 and 4.

The Filters have reinforced concrete shear walls around their perimeter and reinforced concrete (Filters No. 1 and 2) or steel (Filters No. 3 and 4) wash troughs crossing the filters in the east-west direction. The Filter Gallery and Pump Room are located above the Clearwell and form a two-story reinforced concrete shear wall structure with reinforced concrete diaphragms, except at the Pump Room roof that consists of a wood (straight-sheathed) diaphragm. The Clearwell that was built in 1970 also extends under Filters No. 3 and 4 (which were considered as a future expansion during the 1970 design and construction).

In 2005, the Filter Gallery was extended towards the east, and two new filters (Filters No. 5 and 6) and a Clearwell expansion were constructed approximately 3 ft. east of the existing filters. At the Filter Gallery roof level, the slab for the Filter Gallery expansion extends towards the west to within 1 inch of the roof slab from the original Filter Gallery (1970 construction). Within the Filter Gallery, a short walkway section was added between the original Filter Gallery (1970 construction) and expansion Filter Gallery. A small expansion joint is provided between the walkway and original Filter Gallery. A single short section of 24-inch diameter pipe hydraulically connects the expansion Clearwell to the original Clearwell (1970 construction).

Table 3.8 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.8, the Filters No.1 to 4, Filter Gallery, Pump Room, and associated Clearwell structure is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Additionally, based on the potential deficiencies identified in this assessment, the Filters No.1 to 4, Filter Gallery, Pump Room, and associated Clearwell structure is not currently expected to achieve Life Safety performance and represents a safety hazard to City staff and contractors.

Table 3.8 – Filters No. 1 to 4, Filter Gallery, Pump Room, and Associated Clearwell Structure Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4. Filter Gallery and Clearwell • The south shear wall of the Filter Gallery is not continuous to the foundation. It is supported by concrete columns within the Clearwell. See Figure 3.56. • Clearwell concrete columns do not satisfy deformation compatibility requirements due to inadequate tie spacing. • The diaphragm to shear wall connection does not have adequate capacity to develop the lesser of the shear strength of the walls or diaphragms. • The width of the roof slab and walkway seismic joint between Filters No. 2 and 4, and Filters No. 5 and 6 is not adequate to prevent potential pounding between these adjacent structures. See Figure 3.57. • The width of the walkway slab seismic joint between Filters No. 1 and 2, and Sedimentation Basin No. 2 is not adequate to prevent potential pounding between these adjacent structures. Pump Room • The Pump Room is not seismically separated from the 1970 Treatment/Control Building Addition, but these structures are of different heights and their floor/roof levels are not aligned. See Figure 3.58. These split-level diaphragms impose seismic forces in the out-of-plane direction at mid-height of the shared wall. This configuration is not desirable for a structure intended to provide Immediate Occupancy structural performance after a major earthquake. • The roof diaphragm lacks adequate cross ties between flexible diaphragm chords. See Figure 3.59. • In the east-west direction (perpendicular to glulam members) there does not appear to be an adequate load path to transfer seismic forces from the roof diaphragm to the north concrete shear wall. See Figure 3.60. • The roof diaphragm is not attached to the concrete shear walls with connections that are adequate to resist the expected out-of-plane forces.

Table 3.8 – Filters No. 1 to 4, Filter Gallery, Pump Room, and Associated Clearwell Structure Seismic Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	<u>Filters</u> • The Filters are not seismically separated from the 1970 Treatment/Control Building Addition, but these structures are of different heights and their floor/roof levels are not aligned. See Figure 3.61. These split-level diaphragms impose seismic forces in the out-of-plane direction at mid-height of the shared wall. This configuration is not desirable for a structure intended to provide Immediate Occupancy structural performance after a major earthquake.
Nonstructural	<u>Filter Gallery</u> • The finished water, filter backwash, sodium hydroxide, and air scour pipes that cross the seismic joint between the 1970 Filter Gallery and 2005 Filter Gallery Addition do not appear to have adequate flexibility to accommodate potential differential displacements between these adjacent structures. See Figures 3.62 and 3.63. • The finished water, filter backwash, and air scour pipes are not adequately braced to the structure to resist seismic forces. See Figure 3.64. • Valves and valve operators installed in-line with the finished water and backwash pipes are not independently braced (arrows in Figure 3.64). • The air scour piping does not have adequate flexibility to accommodate potential relative movement between the blowers located in soundproofing enclosures outside the building and the Filter Gallery building. See Figure 3.65. • The air vent valve and muffler are not adequately braced to the structure to resist seismic forces. See Figure 3.66. <u>Pump Room</u> • The vertical air relief pipe is not adequately braced to the structure to resist seismic forces. See Figure 3.67. • Pump motors are not braced to the structure above their center of gravity. See Figure 3.68. • Flexible connections are not used between pump casing and piping to accommodate potential differential movement. See Figure 3.68. • The electrical transformer is not adequately braced to prevent movement parallel to the wall. See Figure 3.69. • Anchorage between rooftop HVAC units and roof curbs is potentially inadequate.

Table 3.8 – Filters No. 1 to 4, Filter Gallery, Pump Room, and Associated Clearwell Structure Seismic Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	<u>Filters</u> • Valve operators are not adequately anchored to the Filter structure to resist seismic forces. They are bolted to slotted base plates that appear to have been significantly modified. See Figure 3.70.

Figure 3.54 – Filters No. 1 to 4 and Filter Gallery Roof Slab

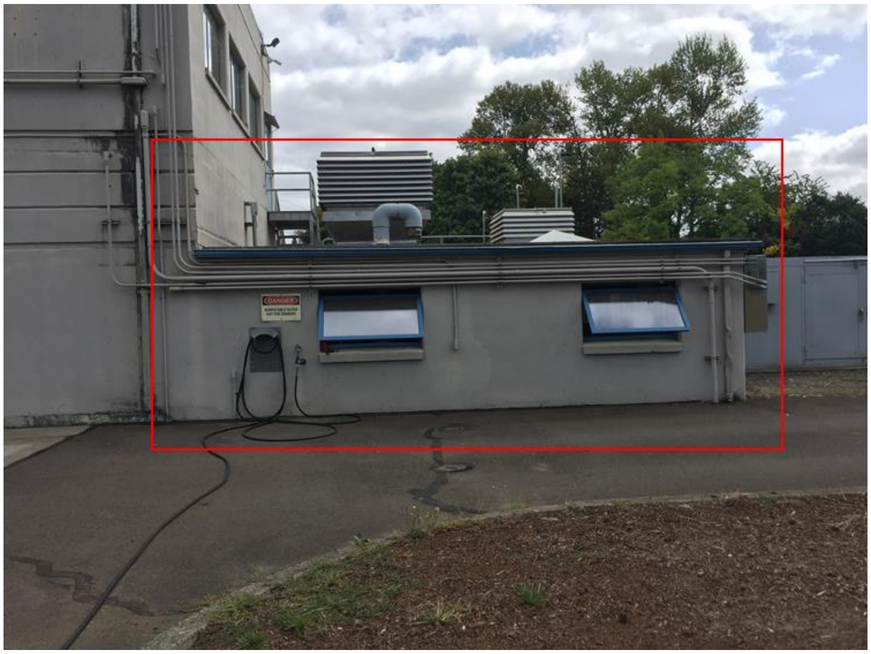


Figure 3.55 – Pump Room

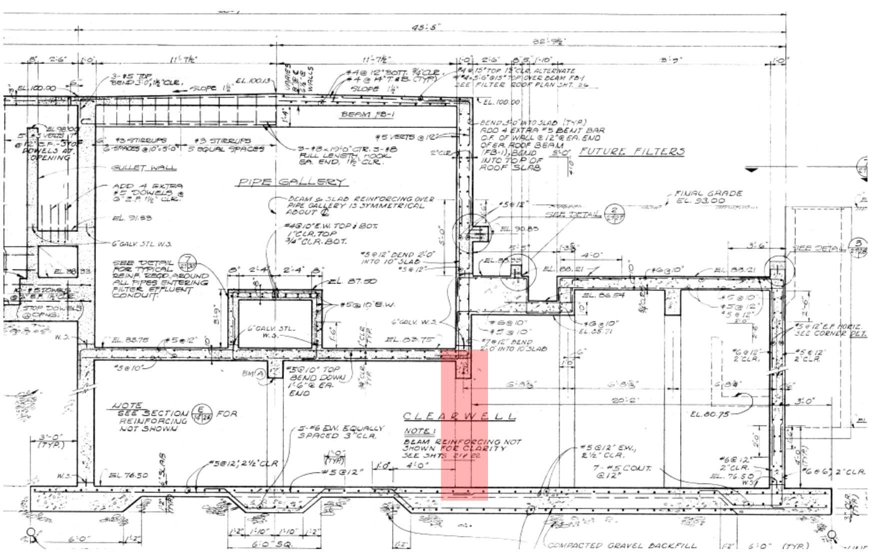


Figure 3.56 – Shear Wall not Continuous to Foundation (Source Drawings: "Water Treatment Plant (A700004)")

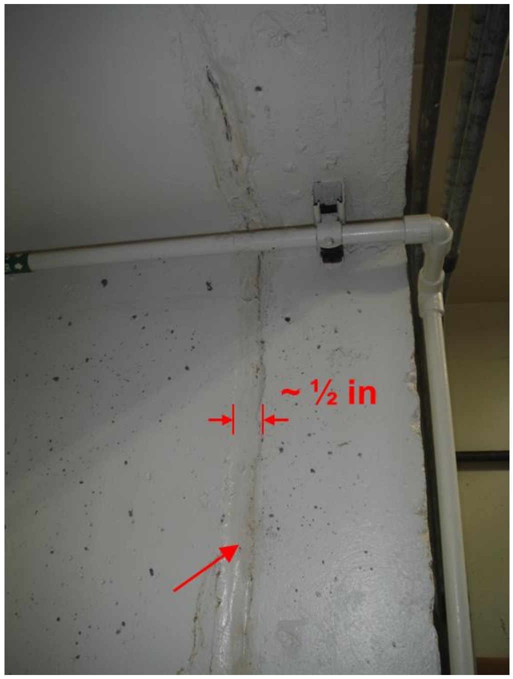


Figure 3.57 – Filter Gallery Seismic Joint (Between 1970 Construction and 2005 Expansion)

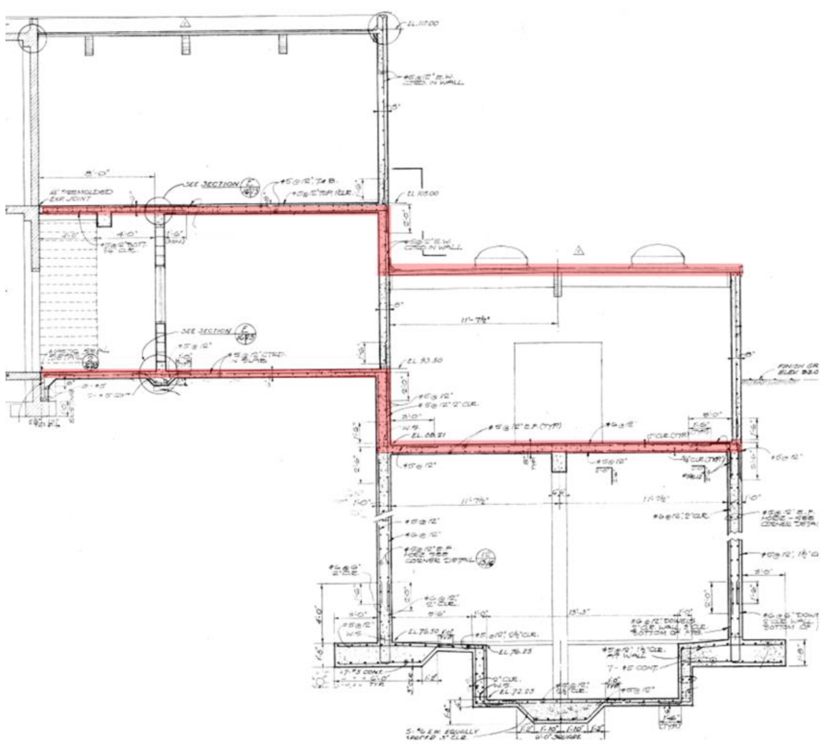


Figure 3.58 – Split Level Diaphragms
(Source Drawings: "Water Treatment Plant(A700004)")

Figure 3.59 – Flexible Diaphragm without Cross Ties

Figure 3.60 – Joist to Perpendicular Wall Connection

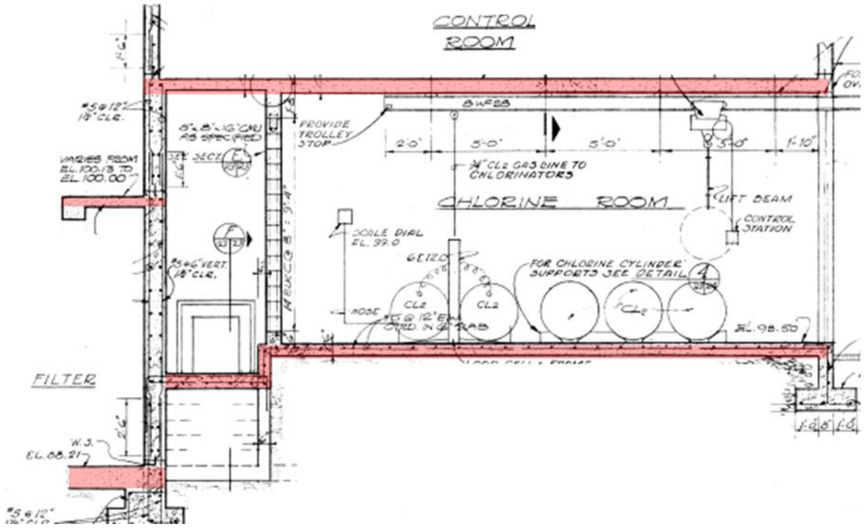


Figure 3.61 – Control/Treatment Building (1970) and Filter Floor/Roof Levels not Aligned (Source Drawings: "Water Treatment Plant (A700004)")

Figure 3.62 – Finished Water Sample Pipe and Filter Backwash Pipe Cross Seismic Joint without Adequate Flexibility

Figure 3.63 – Air Scour Pipe Crosses Seismic Joint without Adequate Flexibility

Figure 3.64 – Valves and Valve Actuators Installed In-Line with Piping Systems not Independently Braced

Figure 3.65 – Air Scour Piping from Blowers to Filter Gallery without Adequate Flexibility to Accommodate Differential Movement

Figure 3.66 – Air Vent Valve and Muffler not Adequately Braced

Figure 3.67 – Air Relief Piping Penetrating Laterally Unrestrained

Figure 3.68 – Pump Motors not Braced to Structure Above their Center of Gravity

Figure 3.69 – Electrical Transformer not Adequately Braced Against Movement Parallel to Wall

Figure 3.70 – Valve Actuators Installed on Significantly Modified Base Plates

3.4.6 Sodium Hypochlorite Generation Building

The Sodium Hypochlorite Generation Building is a steel frame metal building system constructed in 2005 (see Figure 3.71). The building is located at the northeast corner of the plant site. Immediately north of the building, there is a tank storing salt brine solution (NaCl) that is used in the generation of sodium hypochlorite.

The Sodium Hypochlorite Generation Building metal building system consists of steel moment resisting frames in the north-south direction and steel braced frames in the east-west direction (see Figure 3.72) and has a bare metal deck and tension rod flexible roof diaphragm.

Structural drawings were not available for the Sodium Hypochlorite Generation Building and development of as-built drawings was beyond the scope of this study. Potential structural deficiencies identified by this assessment have been based on field observations and general knowledge of typical construction practices. Table 3.9 provides a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.9, the Sodium Hypochlorite Generation Building is not currently expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.9 – Sodium Hypochlorite Generation Building Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4. • The lateral force resisting system lacks redundancy in both directions since there is only one lateral force resisting bay per frame line. See Figures 3.73 and 3.74. • The load path to transfer seismic forces from the roof diaphragm to the moment frame beam is not adequate since there is no blocking provided between purlins. See Figure 3.75. • The load path to transfer seismic forces from the roof diaphragm to the braced frame tension rod bracing involves indirect force transfer from the roof diaphragm to the purlins and then out-of-plane bending of the moment frame beam to column connection to transfer forces to the tension rod bracing. This indirect load path is not desirable for a building with an Immediate Occupancy structural performance objective. See Figure 3.76. • Steel beams and columns likely do not meet section compactness requirements for highly ductile member.

Table 3.9 – Sodium Hypochlorite Generation Building Seismic Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	• It is likely that the moment resisting connections do not have adequate capacity to develop the expected strength of the adjoining beam and column members and panel zones may not have adequate capacity to resist expected shear force demands. See Figure 3.77. • Purlin splices may not have adequate capacity to resist cross tie forces. See Figure 3.78. • Grout layer is not provided under column base plates and nuts on anchor rod are not tight. See Figure 3.79.
Nonstructural	• Pipes from the exterior salt brine tank into process equipment inside the building do not have adequate flexibility to accommodate the expected relative movement between the tank and building. See Figure 3.80. • Drain pipe from the exterior salt brine tank through the concrete slab does not have adequate flexibility to accommodate potential relative movement between tank and the slab. See Figure 3.81. • PVC Vent Piping is not braced to the structure either inside or outside the building. See Figure 3.82. • Pipes connecting the two sodium hypochlorite tanks do not have adequate flexibility to accommodate potential relative movement between the tanks. See Figure 3.83. • Piping connected to both the Sodium Hypochlorite Generation skid and the building does not have flexibility to accommodate the expected building movement. See Figure 3.84. • Anchorage of chemical feed pumps is potentially not adequate due to small diameter and missing anchors. See Figure 3.85. • Hot water heater is not adequately braced to the structure as it has only one strap restraining it instead of two. See Figure 3.86. • Storage barrel is not restrained. See Figure 3.86. • Water softener components are not restrained. See Figure 3.87. • Instant hot water heater is not adequately restrained (only restrained against movement in one direction). See Figure 3.88. • Control Panel is not adequately braced to the structure as it is attached only to the relatively flexible fiberglass handrail. See Figure 3.89. • Transformer on strut support is not adequately braced to the structure. See Figure 3.90. • Lights on pendant supports are not braced and may potentially swing and cause damage to other components. See Figure 3.91.

Figure 3.71 – Sodium Hypochlorite Generation Building

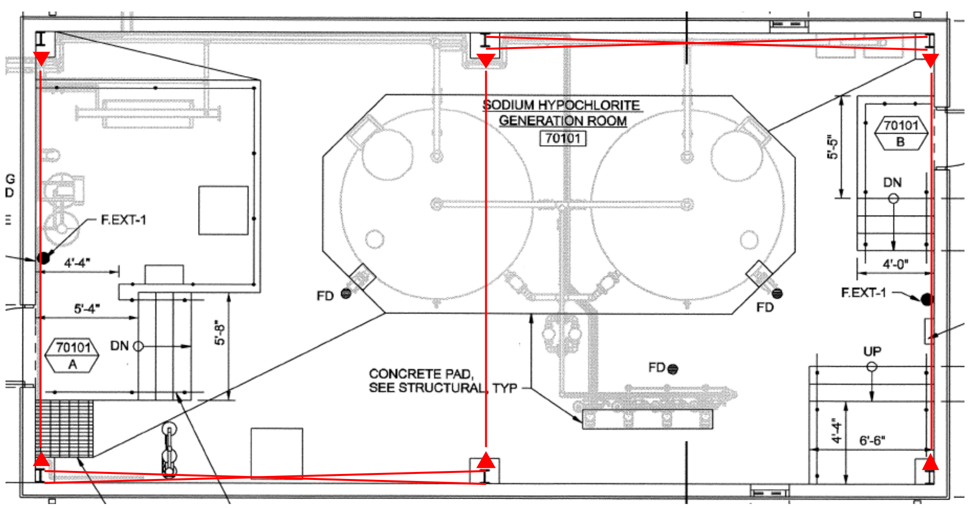


Figure 3.72 – Scheme of Building Lateral Force Resisting Systems (Source Drawings: Water Treatment Plant Expansion to 9.5 MGD (A2007005)")

(a) East Bay without Rod Bracing ***(b) West Bay with Rod Bracing***

Figure 3.73 – Single Lateral Force Resisting Bay in Frame Line along East-West Direction

Figure 3.74 – Single Lateral Force Resisting Bay in Frame Line along North-South Direction

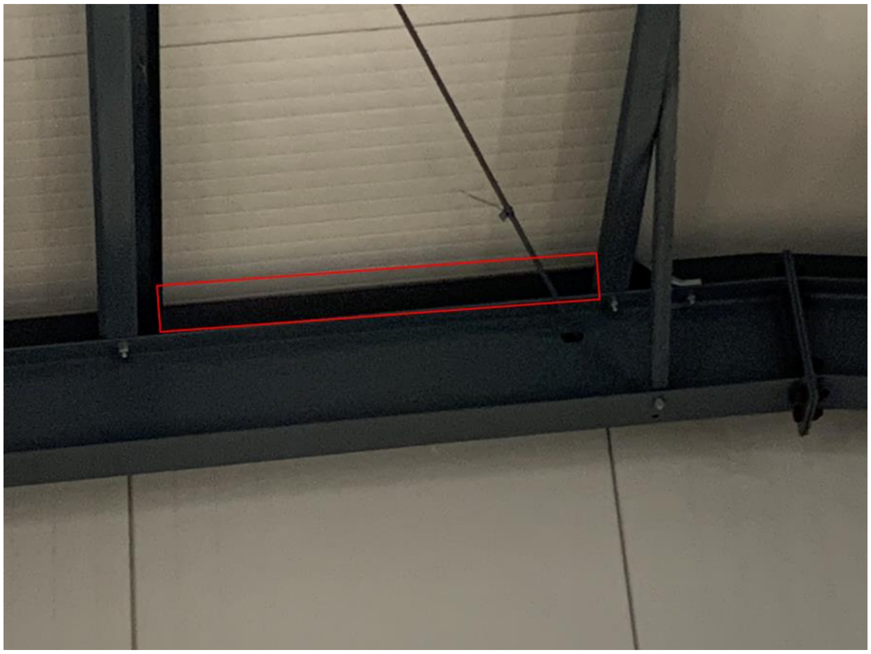


Figure 3.75 – Inadequate Load Path from Roof Diaphragm to Moment Frame Beams (no Blocking between Purlins)

Figure 3.76 – Indirect Load Path from Diaphragm to Brace Frame

Figure 3.77 – View of Moment Frame Connection and Panel Zone

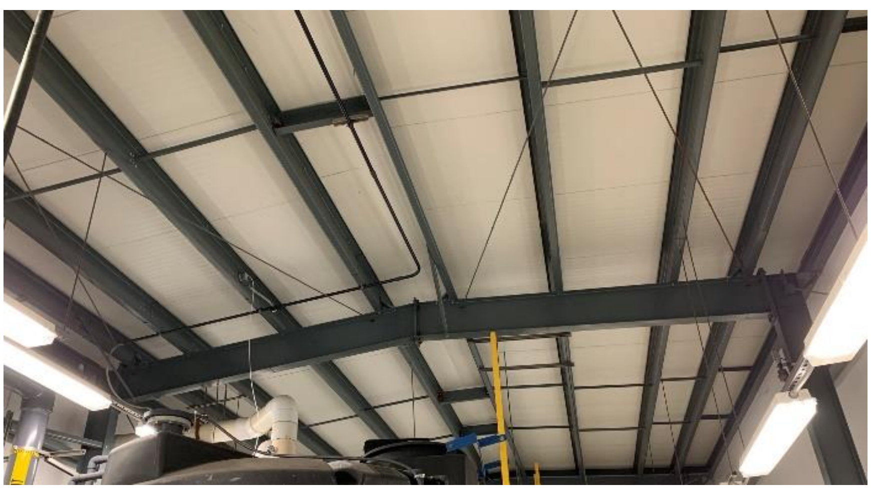


Figure 3.78 – Purlins Between Diaphragm Chords

Figure 3.79 – Ungrouted Base Plate and Nuts on Anchor Rods not Tight

Figure 3.80 – Piping Connecting Salt Brine Tank to Sodium Hypochlorite Generation Building without Adequate Flexibility

Figure 3.81 – Lack of Flexibility in Salt Brine Tank Drain Pipe

(a) Unbraced Piping Outside the Building ***(b) Unbraced Piping Inside the Building***

Figure 3.82 – Unbraced PVC Vent Piping

Figure 3.83 – Lack of Flexibility of Piping Connecting Sodium Hypochlorite Tanks

Figure 3.84 – Lack of Flexibility in Piping between Sodium Hypochlorite Generator and Attachment to Building

Figure 3.85 – Deficient Anchorage Between Chemical Feed Pumps and Concrete Support

Figure 3.86 – Water Heater not Adequately Restrained and Unrestrained Barrel

Figure 3.87 – Water Softener Components not Restrained

Figure 3.88 – Instant Hot Water Heater not Adequately Restrained

Figure 3.89 – Control Panel not Adequately Braced

Figure 3.90 – Transformer not Adequately Braced

Figure 3.91 – Unrestrained Light Fixtures

3.4.7 On-site Electrical Components

The seismic evaluation performed by SEFT also included consideration of the on-site electrical components that serve the water treatment plant (emergency generator, electrical switchgear and electrical transformer). These components are located west of the Treatment/Control Building and are shown in Figures 3.92 to 3.94. The emergency generator at the water treatment plant is a part of Portland General Electric's (PGE's) dispatchable generation program. PGE is responsible for performing routine maintenance and testing of the generator.

Table 3.10 provides a summary of potential seismic deficiencies identified by this evaluation. Based on the deficiencies identified in Table 3.10, the electrical components identified are not expected to support the Water Treatment Plant achieving Operational nonstructural performance following a M9.0 CSZ earthquake.

Table 3.10 – On-site Electrical Components Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• Permanent ground deformation – see first bullet of Table 3.4.
Nonstructural	• The stainless steel cabinet adjacent to the electrical switchgear is supported by both the original switchgear concrete pad and a concrete pad extension. This concrete pad extension may not be adequately attached to the original switchgear concrete pad and differential movement between the original pad and extension may damage the stainless steel cabinet. See Figure 3.93 • Electrical switchgear connection to the concrete pad appears to be missing an anchor and may not be adequate to resist the expected seismic loads. See Figure 3.95. • Electrical Transformer does not appear to be anchored to concrete pad. See Figure 3.96. • It is likely that starter batteries for the emergency generator are not adequately restrained.

Figure 3.92 – Emergency Generator

Figure 3.93 – Electrical Switchgear

Figure 3.94 – Electrical Transformer

Figure 3.95 – Missing Anchors on Switchgear to Concrete Pad Connection

Figure 3.96 – Electrical Transformer not Anchored to Concrete Pad

4.0 Next Steps

This report summarizes the results of SEFT's seismic structural and nonstructural evaluation of three reservoirs (Corral Creek Road, North Valley No. 1 and North Valley No. 2), and selected components of the City of Newberg Water Treatment Plant [Original Treatment/Control Building (pre-1961), 1961 Treatment/Control Building Addition, 1970 Treatment/Control Building Addition, Sedimentation Basin No. 1, Filters No.1 to 4, Filter Gallery, Pump Room, and Associated Clearwell Structure, and Sodium Hypochlorite Generation Building]. Based on the potential structural and nonstructural deficiencies observed, none of the evaluated structures are expected to achieve both the Immediate Occupancy structural performance objective and Operational nonstructural performance objective for a M9.0 CSZ scenario earthquake.

In order to continue to advance with City of Newberg water system resilience planning process, we recommend that a follow-up study be conducted that develops retrofit concepts for critical system components and includes consideration of dependency relationships required to sustain water system operation (diesel fuel for generator, salt for generation of sodium hypochlorite, etc.). The City of Newberg should also continue to evaluate and implement alternative options to provide water to customers in the event that the WTP and/or reservoirs are significantly damaged by a major earthquake and could take months to repair for more recently constructed structures to years to rebuild older structures. Additionally, for the safety of City staff and contractors, the City is strongly encouraged to implement a near-term seismic retrofit program to address Life Safety seismic deficiencies for the occupiable water system structures.

If an expansion of the plant is considered in the future to meet water production or operational goals, then there would be an opportunity to build more seismically resilient structures and associated support infrastructure that is capable of meeting the City's post-earthquake LOS goals. The location and foundation design for any new water system structures should include appropriate consideration of potential earthquake-induced permanent ground deformation, especially at the existing treatment plant site because of the steep slope of the riverbank located in close proximity to the plant.

5.0 Limitations

The opinions and recommendations presented in this report were developed with the care commonly used as the state of practice of the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report. This report has been prepared for the City of Newberg to be used solely in its evaluation of the seismic safety of the water system components referenced. This report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or uses.

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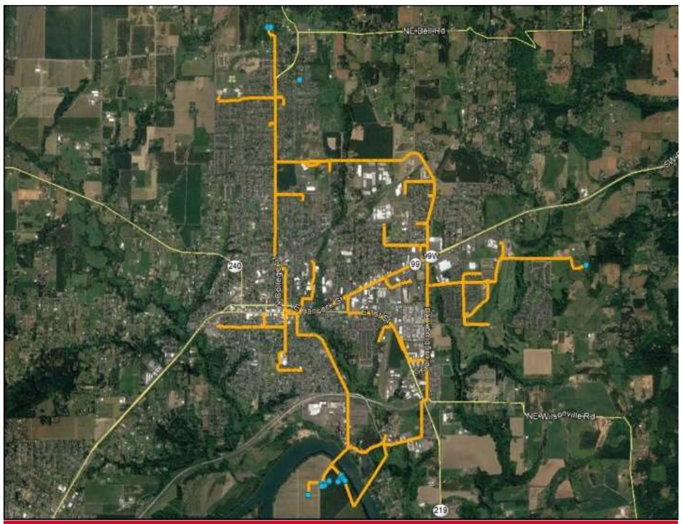

Water System Vulnerability Assessment

Water System Seismic Resilience Study

City of Newberg, OR

July 20, 2020

Contents

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Acronyms

ALA	American Lifelines Alliance
CSZ	Cascadia Subduction Zone
GIS	geographic information system
HDPE	high-density polyethylene
ODOT	Oregon Department of Transportation
PGD	permanent ground displacement
PGV	peak ground velocity
WTP	water treatment plant

1 Vulnerability Assessment

This report is a component of the overall vulnerability assessment that covers the non-structural aspects of the City of Newberg's (City) water system, with the exception of the pipeline bridge. As a subconsultant to HDR, SEFT prepared the vulnerability assessment of the water treatment plant (WTP) and water storage tanks. The following items are included in this report:

- Pipeline bridge
- Wellfield
- 30-inch high-density polyethylene (HDPE) transmission main
- Water system backbone
- Water distribution system
- Yard piping at the WTP and water storage tanks
- Water system operations

Prior to the completion of this vulnerability assessment, Shannon and Wilson completed a geotechnical engineering report summarizing seismic hazards from a Cascadia Subduction Zone (CSZ) magnitude 9.0 event. From this analysis, mapping was generated to identify zones of peak ground velocity, probability of liquefaction, and landslide induced permanent ground deformation. Based on this information, calculations and observations were made with respect to the impact on water system components listed above.

On August 9, 2019, a site visit was conducted to visually inspect the water system infrastructure and interview City operations personnel regarding system components, functionality, operability, and known deficiencies. The site visit focused on the more visible components of the water system such as the WTP, water storage tanks, pipeline bridge, wellfield, and some buried items (e.g., vaults and valves). The operations personnel provided extensive background information about system operations and composition, which is incorporated into this assessment where applicable.

This vulnerability assessment includes a combination of quantitative and qualitative evaluation techniques. American Lifelines Alliance (ALA) methodology was used for the Quantitative analysis to assess damage of buried pipelines. This method incorporates site-specific geotechnical data to predict the total number of pipeline breaks. Although this approach results in defined data points, it is theoretical and subject to high levels of variance. Qualitative evaluation techniques, such as review of record drawings and cross-referencing geotechnical observations, were used to evaluate other components such as the wellfield and 30-inch HDPE transmission main.

1.1 Structural Evaluation of Pipeline Bridge

As part of the Water System Seismic Resilience Study for the City of Newberg, HDR evaluated the pipeline bridge over the Willamette River based on the documents provided by the City, including past seismic evaluation reports and other public domain information available about this historic bridge.

The bridge is a three-span, cantilever deck truss, with a pony truss-type bridge making up the center span. The bridge was constructed in approximately 1917 by the Oregon State Highway Department (now known as the Oregon Department of Transportation [ODOT]). The central pony truss bears on the ends of the cantilever spans, which is a unique configuration. At some point, the structure was abandoned by ODOT and is now used by the City to carry its main water transmission line.

The structural evaluation was limited to a desktop study based on available information and noting general deficiencies and possible retrofits. As-built drawings are not currently available, therefore no numerical analysis was performed. If the City wishes to fully characterize the seismic hazards and investigate firm retrofit options, as-built drawings would be required.

1.1.1 Superstructure

The bridge superstructure (Figure 1) is constructed of a riveted truss with apparent pin bearing assemblies to the substructure. Because the photos do not show the abutments, their condition is unknown. Photos show the middle span bears on the cantilever arms, but the level of restraint is unclear. When the bridge was converted for waterline use, the deck was removed and waterlines and a catwalk installed on the existing floor beams. This helps the seismic performance of the bridge, as it reduces the seismic mass of the structure from its original configuration.

In general, older truss bridges were not designed for ductility and do not perform well in a seismic event. Retrofitting them to ensure ductile behavior is prohibitively expensive in most cases. A common retrofit procedure used with older truss bridges is replacing the bearings with isolation bearings. This method, also known as “base isolation," allows the superstructure to move independently of the substructure, and minimizes the earthquake forces being transmitted to the bridge. On this bridge, the waterline would need to be isolated, which could likely be accomplished by replacing the fixed bearing waterline assemblies with rollers. The truss would need to be checked for seismic forces, as some seismic loads may affect the superstructure. However, any required modifications would likely be less costly than those required if no base isolation was performed.

Figure 1. Pipeline Bridge Superstructure

1.1.2 Substructure

Based on photos and descriptions in the seismic evaluation performed by Montgomery-Watson in 2011, the in-water piers appear steel jacketed concrete. In a seismic event, these may perform well; however, the embedment depth is unknown. If the piers are not embedded deep enough into the soil, they will lack sufficient overturning resistance and could fail during a seismic event from inertial loading. The depth of the existing piers, and additional capacity required to meet seismic loading, will drive the required mitigation method. The most likely retrofit strategy is installation of additional piles or localized ground improvements below the existing pier to provide additional lateral stability.

The details of the end abutments are unknown, however drawings from the 1927 repair suggest that the end abutments, Piers 1 and 4, are of similar construction to the main in-water piers. The 2011 seismic evaluation suggests an additional abutment was constructed at the north end when the trestles were removed. Without specific details, no additional recommendations can be provided regarding seismic upgrades to the end abutments.

1.1.3 Geotechnical Hazards

As part of the Geotechnical Engineering Study, Shannon and Wilson performed two borings and two CPT (Cone Penetration Test) runs at the western approach of the pipe bridge. A slope stability study also was performed at the west edge of the bridge. Bore log results show the site is underlain by silts and clays.

Shannon and Wilson's preliminary analysis indicates the slope is not stable for seismic or post-seismic conditions and the site may experience on the order of 2 feet of lateral spread due to liquefaction. Additional as-constructed details on the foundation system are required to accurately determine what vulnerabilities exist at this particular site. In general, these foundations do not perform well in soils that are subject to liquefaction and lateral spread, as they do not have adequate capacity to remain standing under large lateral pressures induced by liquefaction. Typical mitigation strategies include installation of additional piles and/or drilled shafts to improve the lateral capacity of the foundation, or ground improvements to protect the foundation from additional lateral loads.

1.1.4 24-inch Transmission Main

The 24-inch ductile iron water transmission is approximately 2,085 linear feet, installed in 1980 (Figure 2). This transmission main parallels and serves the same function as the 30-inch HDPE transmission main, by conveying raw water from the wellfield to the City's WTP. The pipeline shares the bridge deck with other power and communication pipelines/conduits. Because the pipeline is solely supported by the bridge, the pipeline will be subject to any failure modes experienced by the bridge in a seismic event. Isolation valves are located on each side of the bridge, which can provide isolation of the damage. Depending on how the bridge fails, damage to the interconnecting system, water loss, and potential cross-contamination may also occur.

Figure 2. 24-inch Water Transmission Main

1.1.5 Summary

Based on review of the available data, the pipeline bridge is unlikely to withstand a CSZ magnitude 9.0 earthquake and will require significant retrofits. This could cost in the tens-of-millions. Before further investigation and analysis can be performed, review of as-built construction documents and a comprehensive physical inspection would be necessary. A dive inspection also is recommended to assess the condition of the exposed foundation elements underwater.

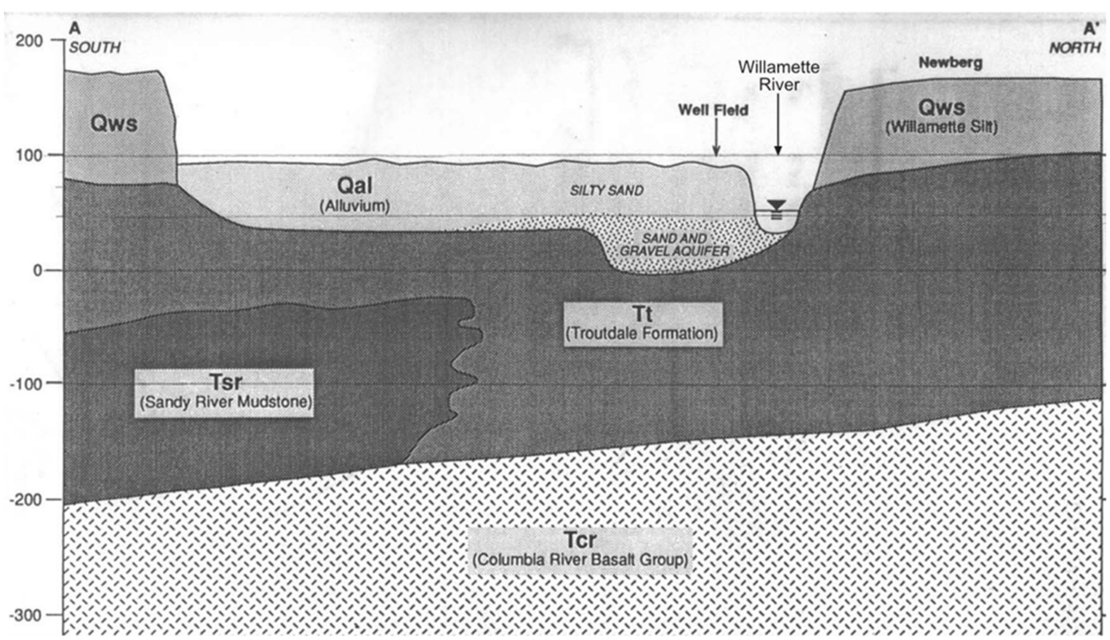
With regard to the 24-inch transmission main, it shares the same structural risks as the bridge. It is unlikely to survive a CSZ magnitude 9.0 seismic event. Because of its low resilience level, the water system is vulnerable to damage to the interconnecting system, water loss, and potential contamination. Isolation valves on either end of the bridge can be closed to minimize water loss if pipeline damage occurs, but they lack automation for quick closure and could be damaged during a CSZ event.

1.2 30-inch HDPE Transmission Main

In 2006, the 30-inch HDPE water transmission main was constructed using horizontal directional drilling under the Willamette River (Figure 3). It is approximately 2,600 linear feet, and extends several hundred feet beyond the river, ranging in depth from 50 feet directly under the river, to 175 feet below the west bank. As with the 24-inch transmission main, it conveys raw water from the City's wellfield to the WTP. Because of its unique construction and depth, Shannon and Wilson provided resilience observations specific to this transmission main crossing:

- According to geotechnical documents from the project, most of the undercrossing is within the Troutdale Formation. The Troutdale Formation is predominantly fine-grained (i.e., silts and clays), with medium to high plasticity. In general, material that is characterized as medium to high plasticity is not susceptible to liquefaction. The risk of liquefaction is likely low for most of the undercrossing.
- On the southern side of the river, the pipeline transitions into the surficial alluvial soils (i.e., wellfield area). This area may be susceptible to liquefaction induced settlement, which could induce differential settlement, especially where the pipeline transitions into the wellfield piping.
- Where the pipeline is at its shallowest on the northern side of the river, the pipeline is within approximately 400 feet of the bank of the Willamette River, and susceptible to lateral spreading. The magnitude of lateral spread at this distance is approximately 5 to 10 inches. Additional study, including explorations and laboratory testing would need to be performed to provide a more reliable estimate of the lateral spreading hazard at this location.

Figure 3. Soils at HDPE Crossing



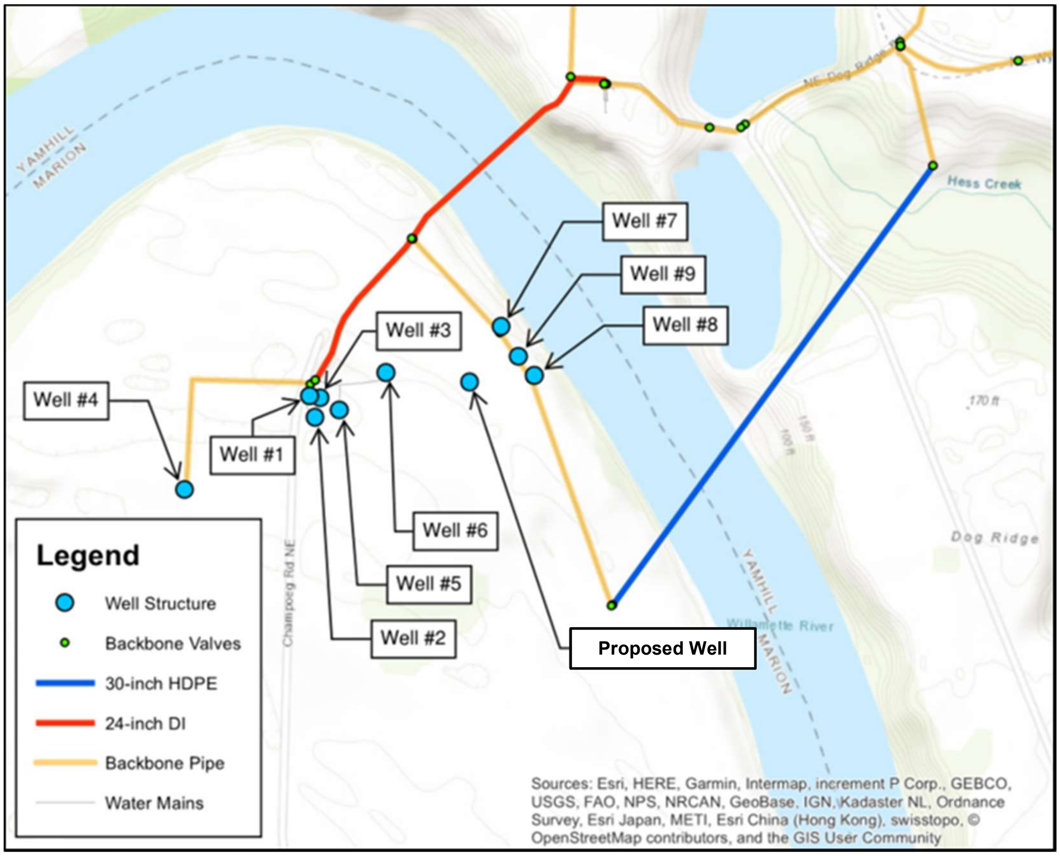
In summary, the majority of the crossing has a low risk of damage during a CSZ event. Vulnerabilities posed by the 30-inch HDPE transmission main are focused on the zone south of the river crossing in the wellfield area, and on the north side within 400 feet from the riverbank. In the wellfield area, differential settlement may occur between the HDPE line and wellfield lines, causing separation or damage. On the northern side, lateral spreading could cause pipe separation or damage.

1.3 Wellfield

The wellfield area is composed of nine wells on the southern side of the river (Figure 4). Currently, five of the nine wells are in operation. Construction of the wells occurred from as early as 1948 up to the present. Because the wellfield is composed of different types of infrastructure at different depths, and could experience impacts to groundwater during a seismic event, Shannon and Wilson provided a focused assessment of this area with the following key observations:

- According to the surficial geology mapped within the region and the available subsurface exploration logs, the surface soils near the well field will be predominantly alluvial soils. The alluvial soils encountered in nearby explorations are characterized as loose sands and gravels and non-plastic to low plasticity silts and were encountered to a depth of 70 feet below the ground surface (approximate elevation 15 feet). Groundwater is indicated at a depth of 24 feet. In general, loose sands and non-plastic to low plasticity silts below the water table will be susceptible to liquefaction.
- Based on the well descriptions in the water system plan, wells 1 through 3 have been removed from operation. Descriptions of wells 4 through 9 indicate that the wells were installed to total depths ranging from 88.5 to 96 feet below the ground surface with the screens placed within a sand and gravel aquifer that appears to overlie the Troutdale Formation and is part of the surficial alluvial soils. Therefore, the wells are likely at risk for liquefaction and lateral spread.
- Some of the consequences of seismic activity within the wellfield include:
    - Based on the proximity to the Willamette River, lateral spreading is likely the primary risk especially for wells near the bank of the Willamette River. Lateral spreading could cause significant lateral displacement of the well casing near the ground surface and above the river bottom. Lateral spreading magnitudes could range from 12 to 24 inches in this area with higher magnitudes closer to the river and then tapering down as you get farther from the river. The well descriptions indicate that wells 4 through 9 were installed with cement surface seals that ranged from 20 to 46 feet in thickness. The existing cement surface seals could help provide some lateral capacity for the well casings.
    - Liquefaction induced settlement is likely a secondary risk that could cause differential settlement between the well casing and pipe connection.
    - Seismic shaking could cause sand and other coarse particles to flow toward the well and plugging of the well screen reducing the capacity of the well.
    - Seismic shaking could cause groundwater levels to fluctuate.

Figure 4. Wellfield



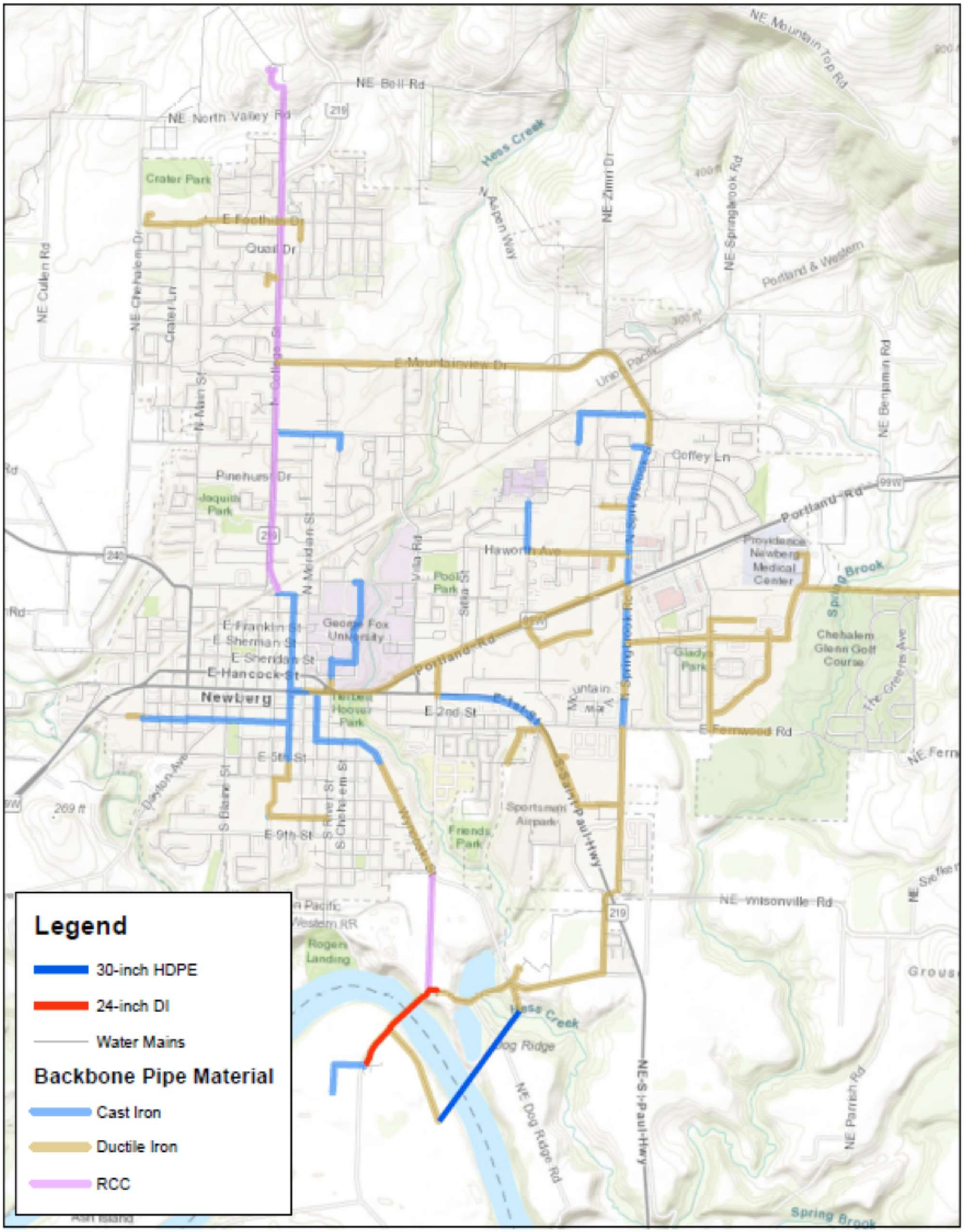
In summary, geotechnical vulnerabilities in the wellfield zone include significant lateral displacement for wells closest to the riverbank, differential settlement between wells and transmission pipelines, change in groundwater levels, and siltation of well screens. The following are additional vulnerabilities identified through discussion with operations personnel and review of record drawings:

- There is only one backup generator located at well 9. Considering that power may be disrupted for a long period of time, additional generators may be needed to provide adequate supply after a CSZ event.
- Because the wellfield is located on the other side of the Willamette River, City crews may not be able to access the wellfield quickly due to bridge failure or other access issues. This may make it difficult to access critical isolation valves (i.e., isolate 24-inch transmission main) or to provide fuel to the standby generator.

1.4 Water System Backbone

The water system backbone was identified in an early phase of this study in which level of service goals were established. Pipelines identified as part of the backbone are generally responsible for connecting all of the critical infrastructure such as the wells, WTP, primary transmission and distribution, and water storage tanks. The City's backbone water system consists of approximately 59 percent ductile iron, 24 percent cast iron, 13 percent concrete, 3 percent HDPE, and 2 percent other (Figure 5)

Figure 5. Water System Backbone by Pipe Material



A vulnerability assessment of the backbone was completed using the ALA procedure to evaluate the probability of earthquake damage. The ALA Pipeline Fragility Formulations consider the following factors that lead to damage of buried pipe in earthquakes:

- Ground shaking
- Landslides
- Liquefaction
- Settlement
- Fault crossings
- Continuous pipeline
- Segmented pipelines
- Appurtenances and branches
- Age and corrosion

The ALA outlines vulnerability functions focused on two specific mechanisms that cause pipe damage: *seismic wave passage* and *earthquake induced ground failure*. Wave passage is directly related to peak ground particle velocity (PGV) associated with ground shaking. Ground failure refers to permanent ground displacement (PGD) associated with landslides and liquefaction. The Geotechnical Engineering Report completed by Shannon & Wilson identifies the following related to PGV and PGD:

- Peak ground velocity (PGV)
- Liquefaction-induced lateral spread (PGD)
- Liquefaction-induced settlement (PGD)
- Landslide-induced PGD in both wet and dry conditions

This analysis applies the equations defined in the ALA with information provided in the geotechnical report. Non-geotechnical components, such as age and corrosion, are accounted for by applying a fragility curve modification factor. Key limitations of this analysis include quality of construction and consideration for pipeline restraint. Table 1 calculates the amount of damage for each significant pipe material:

Table 1. ALA Pipeline Results

Pipe Material	PGV	Liquefaction-induced lateral spread PGD	Liquefaction-induced settlement PGD	Landslide-induced PGD (dry)	Landslide-induced PGD (wet)
Cast Iron					
Hazard Score*	11.02 in/sec	2 in	1.5 in	24 in	180 in
Modification Factor	1.00	1.00	1.00	1.00	1.00
RR Score**	**0.02**	**2.12**	**1.59**	**25.44**	**190.80**
Est. Percentage of Pipe Impacted	100%	100%	100%	5%	5%
Est. Length of Pipe Impacted (ft.)	**23860**	**23860**	**23860**	**1193**	**1193**
Est. Total Breaks in Pipeline	**0.49**	**50.58**	**37.94**	**30.35**	**227.62**

Pipe Material	PGV	Liquefaction-induced lateral spread PGD	Liquefaction-induced settlement PGD	Landslide-induced PGD (dry)	Landslide-induced PGD (wet)
Ductile Iron					
Hazard Score*	11.02 in/sec	2 in	1.5 in	24 in	180 in
Modification Factor	0.50	0.50	0.50	0.50	0.50
RR Score**	**0.01**	**1.06**	**0.80**	**12.72**	**95.40**
Est. Percentage of Pipe Impacted	100%	100%	100%	5%	5%
Est. Length of Pipe Impacted (ft.)	**58433**	**58433**	**58433**	**2922**	**2922**
Est. Total Breaks in Pipeline	**0.60**	**61.94**	**46.45**	**37.16**	**278.72**
RCC					
Hazard Score*	11.02 in/sec	2 in	1.5 in	24 in	180 in
Modification Factor	1.00	1.00	1.00	1.00	1.00
RR Score**	**0.02**	**2.12**	**1.59**	**25.44**	**190.80**
Est. Percentage of Pipe Impacted	100%	100%	100%	5%	5%
Est. Length of Pipe Impacted (ft.)	**12592**	**12592**	**12592**	**630**	**630**
Est. Total Breaks in Pipeline	**0.26**	**26.69**	**20.02**	**16.02**	**120.13**

*Hazard Score estimated from Geotechnical Engineering Report (Shannon and Wilson)
** RR Score is calculated in breaks per 1,000 feet

The table shows that the amount of pipe damage is largely dependent on the pipe material and whether it is subject to liquefaction or landslide. Damage caused by PGV (shaking) is relatively minimal. Damage caused by liquefaction induced lateral spread or landslide induced deformation (dry) is comparable. If in wet soil conditions, the landslide induced deformation is magnitudes greater.

Table 2 and Table 3 further summarize the damage, separating non-landslide and landslide prone areas, respectively. The tables also include pipe length and material, with the majority of pipe located outside of landslide prone areas. For the non-landslide areas (Table 2), the total estimated number of pipeline breaks is 245, at a frequency of 3 per 1,000 feet (or an average of 387 feet between each break). As an example, if two repair crews could repair four locations per day, it would require a total of 60 days to repair the non-landslide backbone area. For the landslide prone areas, there is a dramatic difference between dry and wet conditions. Under the same scenario, repairs would take an additional 21 to 156 days to repair. In reality, those pipelines would require full replacement, whether it was wet or dry, because of the breakage frequency.

Table 2. ALA Summary Non-Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard (ft)	Percentage of Backbone Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft)
Cast Iron	23,860	25%	89	4	268
Ductile Iron	58,433	62%	109	2	536
RCC	12,592	13%	47	4	268
Grand Total	**94,884**	**100%**	**245**	**3**	**387**

Table note: Estimated Number of Breaks Due to PGV and PGD (non-landslide) by Pipe Material

Table 3. ALA Summary for Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard(ft.)	Percentage of Backbone Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft.)
Cast Iron	1,193	1%	30-228	25-191	5-39
Ductile Iron	2,922	3%	37-279	13-95	10-79
RCC	630	1%	16-120	25-191	5-39
Grand Total	**4,744**	**5%**	**84-626**	**64-477**	**5-79**

Table note: Estimated Number of Breaks Due to PGD (landslide) by Pipe Material

1.5 Water Distribution Pipelines (non-backbone)

The water system distribution network represents the highest quantity of water pipelines, but is also considered a lower priority for seismic resilience. In terms of composition, the network includes approximately 63 percent ductile iron, 23 percent cast iron, 9 percent PVC, and 5 percent other.

For simplicity of presentation, only the summary tables for non-landslide and landslide areas are provided (Table 4 and Table 5, respectively). For most of the distribution system (non-landslide), results show 1,159 water main breaks at a frequency of 2 per 1,000 feet (403 feet between each break; Table 4). Under the previously assumed scenario of repairing four locations per day (two crews at two repairs per day), repairs would require 290 days. For the landslide prone areas, a range of 336 to 2,518 breaks would occur and require a range of 84 to 630 days to repair. As in the case with the backbone system, those pipelines in the landslide prone areas would likely require full replacement instead of repair.

Table 4. ALA Summary Non-Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard (ft)	Percentage of Distribution Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft)
C-900	11,713	3%	35	3	336
CI	106,470	23%	397	4	268
DI	296,271	63%	553	2	536
PVC	28,707	6%	85	3	336
Other	23,905	5%	89	4	268
Grand Total	**467,065**	**100%**	**1,159**	**2**	**403**

Table note: Estimated Number of Breaks Due to PGV and PGD (non-landslide) by Pipe Material

Table 5. ALA Summary of Landslide Areas

Pipe Material	Total Material Length Within Geo-Hazard(ft.)	Percentage of Distribution Total	Est. Total No. of Breaks	Est. No. of Breaks per 1,000 ft.	Est. Space Between Breaks (ft.)
C-900	586	3%	12-89	20-153	7-49
CI	5,324	23%	135-1,016	25-191	5-39
DI	14,814	63%	188-1,413	13-95	10-79
PVC	1,435	6%	29-219	20-153	7-49
Other	1,195	5%	30-228	25-191	5-39
Grand Total	**23,353**	**100%**	**336-2,518**	**59-439**	**5-79**

Table note: Estimated Number of Breaks Due to PGD (landslide) by Pipe Material

1.6 Yard Pipeline Vulnerabilities

An important component of water system resilience is to evaluate how the critical structures are connected to the transmission/distribution system. This includes not only pipeline construction, but also placement of seismic couplings, isolation valves, pressure-regulating valves, and remote monitoring or control capability. For this evaluation, vulnerabilities were identified through site visit observations, interview of operations personnel, and review of record drawings. Evaluated locations included yard pipelines (exterior to the building) for the WTP and water storage tank sites.

1.6.1 Water Treatment Plant

WTP vulnerabilities and observations include the following:

- There is a remotely operable isolation valve at the inlet to the WTP, but not a remotely operable isolation valve on the discharge to the WTP. If a seismic event occurred, the WTP may not be immediately isolated from the water system, creating more potential for water loss or cross-contamination.
- There are no known control valves (hydraulic pressure sustaining valves) on the inlet or outlet sides of the WTP that would engage automatically to isolate the WTP, thereby preserving water storage in the WTP and preventing cross-contamination.
- There is no bypass line around the WTP that would connect raw water transmission from the wellfield to the distribution system. This means that supplying water after a seismic event would depend on repair and recovery of the WTP. A bypass would allow temporary raw water for firefighting and domestic use (boiling would be needed for drinking).
- Based on record drawings, there are couplings located at pipeline building penetrations that may allow minimal movement; however, they are not seismically resistant. Differential settlement could occur between the structure and outside pipelines. Lateral spreading may also cause pipe separation.

1.6.2 Water Storage Tanks

There are two water storage tank sites; the Corral Creek Road Reservoir east of the City and the North Valley water storage tanks north of the City. Vulnerabilities and observations include the following:

Corral Creek Site

- Pipeline connections along the exterior of the water tank are fitted with flexible couplings. Given the relatively low amount of liquefaction and lateral spreading predicted, these may be adequate for movement that may occur. These couplings, however, do not provide the amount of protection that a seismic coupling provides.
- A landslide may result in up to 6 inches of lateral spread approximately 100 feet from the reservoir. There are no seismic couplings in the pipeline that could accommodate this movement, which could lead to pipe separation.
- There is a remotely operable isolation valve on the inlet/outlet line to the water tank, allowing for quick isolation and protection of the water storage in the tank during and after an event. There is not, however, a hydraulic control valve, that could operate and close independently of the SCADA system (if down) to protect the water storage.

North Valley Water Storage Tanks

- This site location (Figure 6) is subject to higher magnitudes of permanent ground deformation. Differential settlement of approximately 0.5 to 1.5 inches could occur between structures and connecting pipelines. It is unknown if exterior couplings could absorb this movement.
- The inlet/outlet line to the site will be subject to landslide movement up to 2 feet. This is a significant range of movement that would require one or more seismic couplings to absorb. In its current state, pipeline separation likely would occur.
- There is a remotely operable isolation valve on the inlet/outlet line to the water tank, allowing for quick isolation and protection of the water storage in the tank during and after an event. There is not, however, a hydraulic control valve, that could operate and close independently of the SCADA system (if down) to protect the water storage.

Figure 6. North Valley Site

1.7 Water System Operations

From an operational perspective, the following vulnerabilities and observations were gathered from a number of sources including review of the most current water system plan, site visit, review of record drawings, and interviews with operations personnel.

- The City operates at relatively high average system pressures. There are no fire-flow or pressure deficiencies identified that could affect system recovery after a CSV event.
- There are no current deficiencies in water system storage capacity.
- The SCADA system could be improved or expanded to include greater centralized monitoring and control of the system. Identify locations without backup battery power. Engage power and communications utilities to gauge utility resilience and backup measures.
- Not having a redundant water supply in an alternate geographic location creates a significant vulnerability for the water system. It is understood the City is actively pursuing redundant water supply options.
- Ensure geographic information system (GIS) mapping is adequately detailed to locate critical isolation valves and facilities in an emergency.

1.8 Summary

This study identified several water system vulnerabilities associated with the pipeline bridge, 30-inch HDPE transmission main, wellfield, water system backbone, water distribution network, and system operations. The probability and magnitude of the damage that could occur depends on both qualitative and quantitative assessments; meaning that there are a wide range of possible outcomes. With careful consideration of these assessments, a picture of the potential damage can be drawn, and can then lead to development of priorities and improvements.

Table 6 summarizes the vulnerabilities for each water system component and includes an estimated recovery period for repair or replacement.

Table 6. Summary of Vulnerabilities

Component	Vulnerabilities	Estimated Recovery Period (days)
Pipeline Bridge	• Superstructure not designed for ductility • Substructure compromised by liquefaction and lateral spread • Pipeline will fail with the bridge and risk damage to connecting system, water loss, and contamination	Unlikely repairable and not cost effective to re-build
30-inch HDPE Line	• On northern side of river, pipe separation likely due to lateral spread • On southern side of river, liquefaction induced differential settlement with wellfield transmission lines	If the damage is isolated, repair could be in the range of two weeks. Access issues may prevent repair
Wellfield	• Insufficient backup power generation • Lateral spread and liquefaction could cause irreparable damage to deep wells • Potential siltation and changes to groundwater levels	Damage could be severe and require several months for new well construction
Water System Backbone	• Pipeline breaks due to lateral spread, settlement, and landslide	Approximately 60 days for non-landslide, and 21 to 156 days for landslide areas
Water Distribution	• Pipeline breaks due to lateral spread, settlement, and landslide	Approximately 290 days for non-landslide, and 84 to 630 days for landslide area
Yard Piping	• Loss of water storage due to absence of automated hydraulic control valves • Loss of storage due to absence of seismic couplings at structures or landslide zones • No bypass around WTP	Repair could be within a month, but water loss could be costly to the community during recovery

Appendix D:
Mitigation Recommendations

Memo

Date:	Friday, April 24, 2020
Project:	Seismic Resilience Assessment
To:	Brett Musick, PE, City of Newberg
From:	Andy McCaskill, P.E.; Chad Gipson, P.E.; Katie Walker, P.E.
Subject:	WTP Seismic Resiliency Cost Estimates

Introduction

Due to a potential Cascadia Subduction Zone event, the City of Newberg, OR is evaluating its water system to identify gaps in seismic resiliency. The existing water treatment plant (WTP) consists of vintage concrete structures not designed or detailed for current seismic codes. To mitigate this risk, significant work is required to perform a detailed seismic analysis of the existing structures and develop a structural retrofit and reinforcement scheme for the facility. The existing WTP site is also susceptible to lateral spreading during an earthquake, which would cause extensive damage to the plant without significant ground improvements. The purpose of this memorandum is provide information on the estimated cost to retrofit the existing WTP structures and perform ground improvements to mitigate lateral spreading at the existing plant, as well as the cost of building a new WTP.

Current Water Treatment Plant – Seismic Mitigation

The following cost estimate was developed primarily based on the seismic deficiency findings developed by SEFT (September 2019), using the ASCE41 Tier 1 seismic deficiency checklist method. Based on those findings, HDR developed rough order of magnitude cost estimates to perform seismic retrofits to address these deficiencies in order to meet the Basic Performance Objective for Existing Buildings (BPOE) criteria for a Risk Category IV essential facility in accordance with ASCE41 recommendations and guidelines.

The cost estimate is based solely on addressing seismic deficiencies identified in the Tier 1 assessment. It should be noted that some structures are approaching the end of their useful design life and there are potentially other deficiencies not addressed by the seismic retrofits.

It should be noted that the geotechnical investigation performed by Shannon and Wilson (July 2019) indicated that the existing plant is susceptible to liquefaction, ground deformation and lateral spreading. It is assumed that given the estimated level of settlement during a seismic event (approximately 1 inch), that most of the structures within the plant can tolerate this settlement with minimal impact to operations or life safety during a Cascadia Subduction Zone (CSZ) earthquake. As such, it is assumed that piles or deep foundation elements are not required at the existing plant to mitigate for liquefaction induced settlement.

However, the estimated seismic induced lateral spread movement is expected to be several feet. This is generally mitigated through the installation of ground improvements between the site and the shoreline to help buttress the site and prevent lateral movement. While detailed design of ground improvements is determined by the geotechnical engineer, HDR used unit costs based on past project experience with similar seismic hazards in order to estimate the magnitude of ground improvement costs for this site.

Table 1 presents the summary of the cost estimate for seismic mitigation improvements to the existing WTP based on the findings from the SEFT report.

Table 1: Existing WTP Seismic Mitigation Cost Estimate

Description	Cost
Original Control Building	$ 320,000
1961 Control Building Addition	$ 325,000
1970 Control Building Addition	$ 350,000
Sedimentation Basin #1	$ 205,000
Sedimentation Basin #2 (not in SEFT study)	$ 205,000
Filter Gallery and Clearwell	$ 245,000
Pump Room	$ 170,000
Filters	$ 150,000
Sodium Hypochlorite Building	$ 50,000
Subtotal Seismic Retrofits	**$ 2,020,000**
Nonstructural Seismic Mitigation (25%)	$ 505,000
Ground Improvements	$ 2,000,000
Subtotal	**$ 4,525,000**
Engineering and permitting (15%)	$ 680,000
Contingency (25%)	$ 1,300,000
Total	**$ 6,505,000**

Conceptual level cost estimates for an AACE Class 5 estimate can range from -50% on the low end and up to 100% on the high end. Using the cost estimate presented in Table 1, the range of the WTP construction cost estimate could be from approximately $3.3M to $13M.

New Water Treatment Plant

The cost estimate for a new water treatment plant is based on the design criteria outlined in Section 7 of the 2002 Water Treatment Facility Plan. The treatment process are identified as follows:

- Oxidation Contact Basins – use chlorine to oxidize iron
- Dissolved Air Flotation – removes iron solids
- Granular Media Filters – filtration
- Clearwell – storage and additional disinfection contact time
- Sludge Pump Station – sends solids from DAF to the sludge thickener
- Backwash Equalization Basin – stores backwash waste from the filter before sending to sanitary sewer

- Sludge Thickener – thickens solids before discharge to sanitary sewer

Table 2 presents the design criteria used in the cost estimate.

Table 2: New WTP Cost Estimate Design Criteria

Parameter	Design Value or Specification
Initial Maximum Design Flow	12 million gallons per day (MGD)
Oxidation Contact Basins	Number of units: 3, initially Design contact time: 15 minutes
Dissolved Air Flotation	Number of units: 3, initially Surface loading rate: 6 gallons per square foot (gpm/sf)
Granular Media Filters	Number of units: 4, initially Filter loading rate: 6 gpm/sf Area of each filter: 384 sf Depth of media: 5 feet (1 foot sand, 4 feet anthracite)
Clearwell	Storage: 1 million gallons
Sludge Pump Station	Pumps: 1 duty + 1 standby Horsepower: assumed 2 hp
Backwash Equalization Basin	Backwash flow rate: 20 gpm/sf Backwash duration: 10 minutes Filter to waste flow rate: 6 gpm/sf Filter to waste duration: 5 minutes Number of stored backwashes: 4
Backwash Supply Pump Station	Pumps: 1 duty + 1 standby Horsepower: assumed 125 hp
High Service Pump Station	Pumps: 5 duty + 1 standby Horsepower: assumed 100 hp
Chemical Systems	Coagulant: tank plus metering pumps (1 duty + 1 standby) Sodium Hydroxide (caustic): tank plus metering pumps (1 duty + 1 standby) Filter Aid Polymer: 1 tote with mixer, 1 blending skid Sludge Thickener Polymer: 2 tote with mixer, ` blending skid Chlorine: none (assumed City would transfer existing chlorine generation system to the new plant)
Administrative Building	Size: 3,750 feet

Table 3 presents the summary of the cost estimate for a new WTP. This estimate does not include any requirements for offsite work, such as installation new electrical lines, raw or finished water pipelines.

Table 3: New WTP Conceptual Cost Estimate

Description	Cost
Administration Building	$ 1,218,750
Chemical Systems	$ 421,000
Site Civil	$ 927,000
Seismic Mitigation	$ 927,000
Generators	$ 500,000
Oxidation Contact Basins	$ 329,500

Description	Cost
Dissolved Air Flotation	$ 1,841,000
Filtration	$ 1,143,000
Solids Handling	$ 899,750
Clearwell	$ 2,570,750
Piping	$ 842,000
Electrical/I&C	$ 2,156,000
Start-up Costs	$ 275,600
Subtotal	**$ 14,051,350**
Engineering and permitting (15%)	$ 2,108,000
Contractor OH/Profit/Mob/Insurance/GC	$ 3,513,000
Subtotal	**$ 19,672,350**
Contingency (25%)	$ 4,918,000
Total	**$ 24,590,350**

Conceptual level cost estimates can range from -50% on the low end and up to 100% on the high end. Using the cost estimate presented in Table , the range of the WTP construction cost estimate could be from approximately $12.3M to $49.2M.

Memo

Date:	Monday, June 22, 2020
Project:	City of Newberg Seismic Resilience Assessment
To:	Brett Musick, PE, City of Newberg
From:	Andy McCaskill, PE; Katie Walker, PE
Subject:	Seismic Resilience Assessment – Mitigation Recommendations

Introduction

The City of Newberg (City) is conducting a seismic resilience assessment (SRA) to assess vulnerabilities in their system and identify mitigation strategies to meet their level-of-service (LOS) goals during and after a Cascadia Subduction Zone (CSZ) event. Previous mitigation strategies identified as part of the SRA include the rehabilitation of the existing water treatment plant and construction of a greenfield water treatment plant. The purpose of this memorandum is to present the following three additional recommendations to mitigate seismic challenges:

1. Emergency Connection and Control at the Water Treatment Plant (WTP)
2. Seismic Improvements at Corral Creek and North Valley Water Storage Tanks (WSTs)
3. Cast Iron and Concrete Pipe Replacement

The following sections describe these recommendations in more detail and include a conceptual design and construction cost estimate.

Mitigation Recommendation 1 – Emergency Connection and Control at WTP

As documented in other studies, the WTP is susceptible to several seismic risks including slope instability, liquefaction, and lateral induced settlement. Since all water to the City's distribution system currently runs through the WTP and repairs at the plant will likely be needed following a CSV event, the installation of a WTP emergency connection point is recommended. This emergency connection would provide a point where the raw water line could be connected to the finished water line (see Appendix A), allowing raw water to be used in the community for firefighting and domestic use (must be boiled for potable consumption). To facilitate the connection, tees are to be added to the raw and finished water pipeline with isolation valves installed in a connection vault (see Figure 1). A spool piece would be added during an emergency to provide a cross-connection point. The conceptual cost for this item is approximately $200K. One future item for consideration includes modeling the City's system hydraulics and pressures to evaluate how to operate the emergency connection and if additional appurtenances are required.

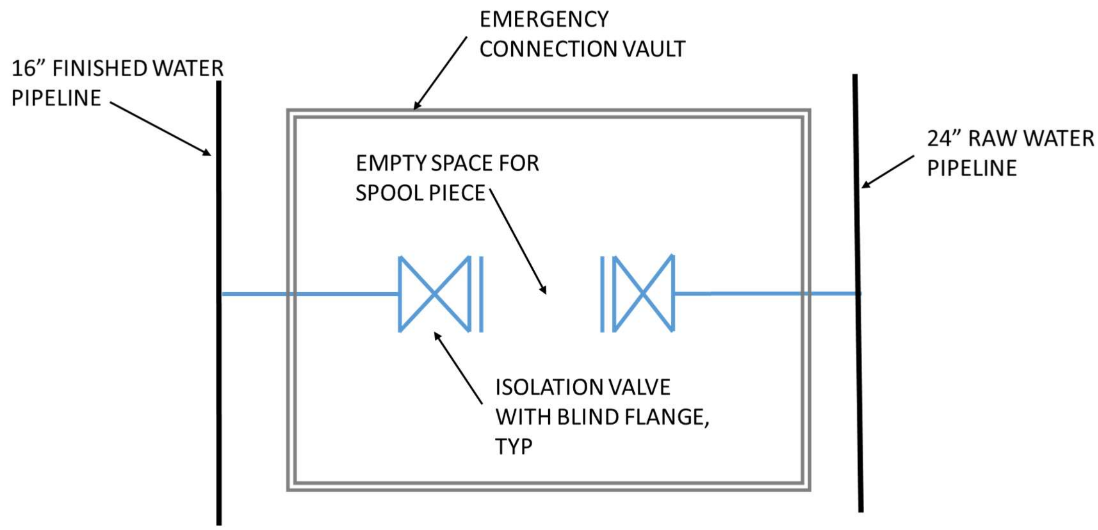


Figure 1. Raw Water Emergency Connection Vault

In addition, it is recommended that a hydraulically actuated pressure sustaining valve be installed on the raw water line that would close in the case of a pressure drop upstream, potentially due to a pipeline bridge failure or transmission main break. This valve would automatically close to prevent the water system from bleeding back into the river or wellfield area if there is a transmission main break. The conceptual cost for this item is approximately $300K. One future item for consideration includes modeling the City's system hydraulics and pressures to refine the pressure sustaining valve operation.

Mitigation Recommendation 2 – Seismic Improvements at Corral Creek and North Valley WSTs

Conceptual layouts for these improvements are presented in Appendix B.

Corral Creek WST Improvements

Pipeline separation, and subsequent water loss, was identified as a main vulnerability at the Corral Creek WST. It is recommended that a hydraulically actuated pressure sustaining valve be installed on the inlet/outlet to the tank to preserve water storage if a pipeline break occurs. The conceptual cost for this item is approximately $300K. Future items for consideration include modeling the City's system hydraulics and pressures to refine the pressure sustaining valve operation, and evaluating an option to retrofit the existing altitude vault.

North Valley WSTs Improvements

The North Valley WSTs have a similar vulnerability for water loss as the Corral Creek WST; a hydraulically actuated pressure sustaining valve is also recommended for installation on the inlet/outlet. The conceptual cost for this item is approximately $300K. One future item for consideration includes modeling the City's system hydraulics and pressures to refine the pressure sustaining valve operation.

In addition to the valve, it is recommended that the portion of the concrete pipeline from the tank to NE North Valley Road be replaced due to the potential for landslide in the area and the lack of seismic resiliency within the pipeline. Approximately 800 linear feet of 24" pipeline is recommended to be replaced with restrained joint ductile iron pipe at a conceptual cost estimate of $450K.

Mitigation Recommendation 3 – Cast Iron and Concrete Pipe Replacement

The survey of the City's backbone identified that it contains approximately 24% cast iron pipe and 13% concrete pipe (see Appendix C). The vulnerability assessment identified that a majority of the breaks in the system's backbone will occur in these pipe materials and will likely not be repairable following a CSZ event. Table 1 presents the breakdown of pipe sizes by pipe material.

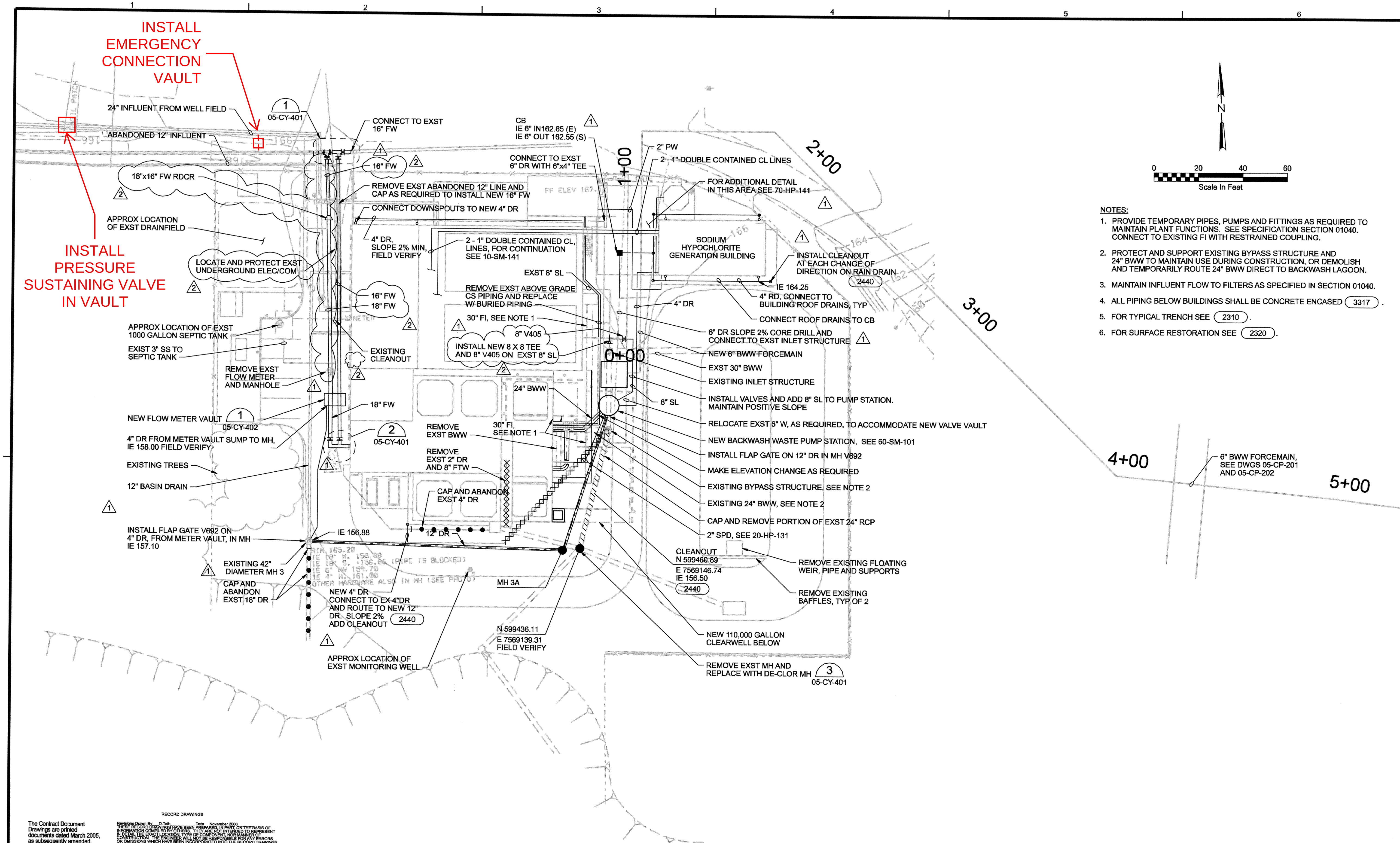
Table 1. Backbone Pipe Replacement by Pipe Size and Material

Pipe Diameter	Linear Feet of Pipe		Total Linear Feet of Pipe
	Cast Iron	Concrete	
6"	1,500	0	1,500
8"	7,979	0	7,979
10"	3,520	0	3,520
12"	6,850	17	6,867
14"	60	0	60
16"	0	2,600	2,600
18"	4,920	9,030	13,950
24"	0	950	950
Total			37,426

It is recommended that these pipes be replaced with restrained joint ductile iron pipe to reduce the recovery time for the water system backbone. A portion of the concrete pipe identified in this table is also recommended to be replaced under Mitigation Recommendation 2 – North Valley WSTs. The conceptual cost for this item is approximately $12.5M and assumes an additional 10% pipe replacement.

Appendix A:

Mitigation Recommendation 1 – Conceptual WTP Improvements



NOTES:

1. PROVIDE TEMPORARY PIPES, PUMPS AND FITTINGS AS REQUIRED TO MAINTAIN PLANT FUNCTIONS. SEE SPECIFICATION SECTION 01040. CONNECT TO EXISTING FI WITH RESTRAINED COUPLING.
2. PROTECT AND SUPPORT EXISTING BYPASS STRUCTURE AND 24" BWW TO MAINTAIN USE DURING CONSTRUCTION, OR DEMOLISH AND TEMPORARILY ROUTE 24" BWW DIRECT TO BACKWASH LAGOON.
3. MAINTAIN INFLUENT FLOW TO FILTERS AS SPECIFIED IN SECTION 01040.
4. ALL PIPING BELOW BUILDINGS SHALL BE CONCRETE ENCASED (3317).
5. FOR TYPICAL TRENCH SEE (2310).
6. FOR SURFACE RESTORATION SEE (2320).

The Contract Document Drawings are printed documents dated March 2005, as subsequently amended, which define the scope, extent, and character of the work. The original document drawing was sealed & signed by: Todd Bryan Perimon State of Oregon P.E. No. 18708

RECORD DRAWINGS

Revisions Drawn By: D.Toth Date: November 2006
THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT, NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

DSGN TB PERIMON	3	2007	RECORD DRAWING			
DR JL PFEIFER	2	4/05	ADDENDUM NO. 2	RFG	BJP	
CHK WB WAGNER	1	4/05	ADDENDUM NO. 1	TBP	RCF	
APVD RC FRANKENFIELD	NO.	DATE	REVISION	BY	APVD	

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CH2MHILL

CITY OF NEWBERG
NEWBERG, OREGON
WATER TREATMENT PLANT EXPANSION TO 9.5 MGD

SITE DEVELOPMENT

YARD PIPING PLAN AND DEMOLITION

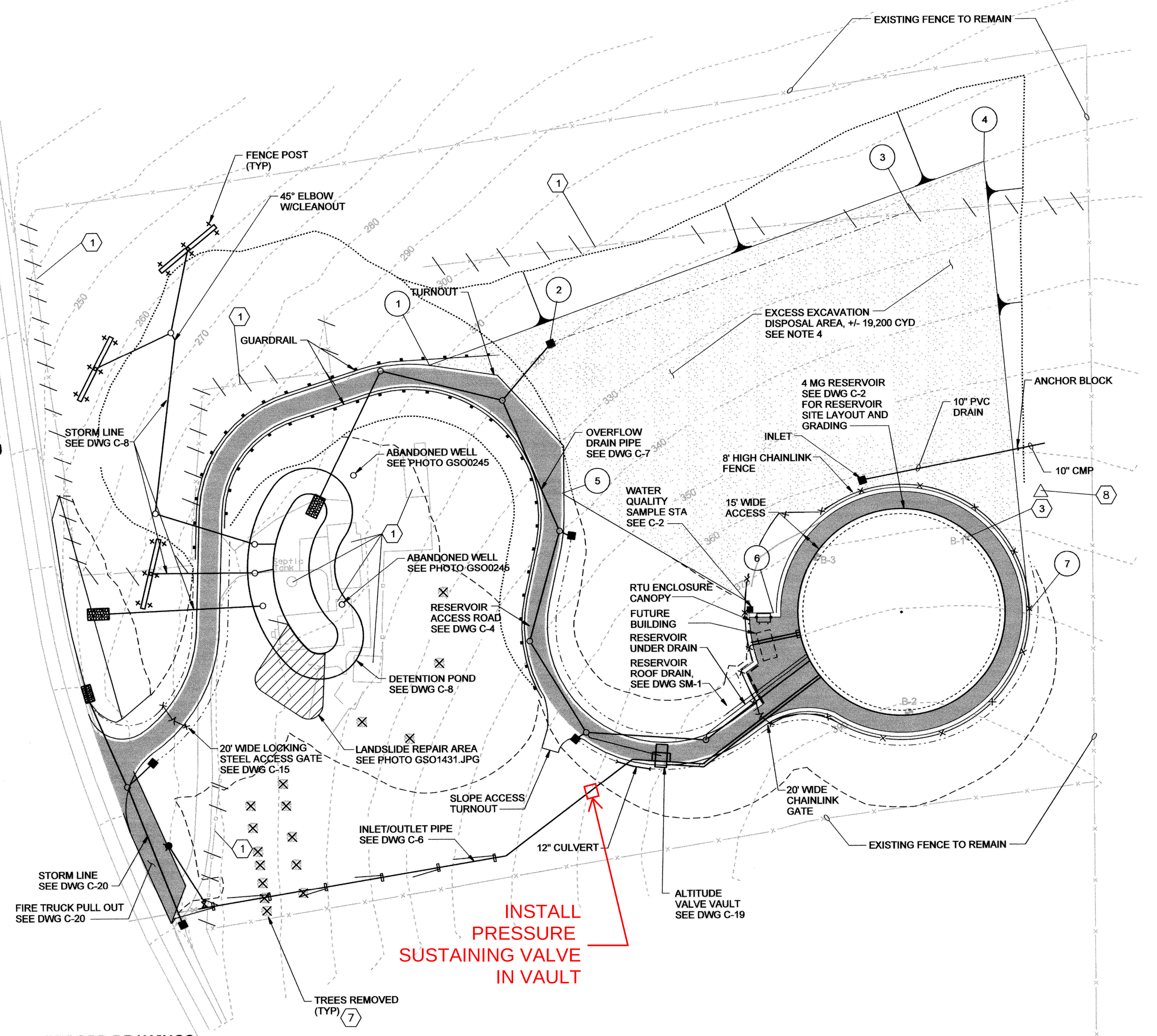
SHEET	17
DWG	05-CY-101
DATE	MARCH 2005
PROJ	320153

FILENAME: 05ncy101d_320153.dgn PLOT DATE: 5/29/2007 PLOT TIME: 11:46:54 AM

A2007005T

Appendix B:

Mitigation Recommendation 2 – Conceptual WSTs Improvements



0 40 80 120
Scale In Feet

DISPOSAL AREA LOCATION DATA

PT NO	COORDINATES		ELEV
1	N 607081.25	E 7579315.66	324.43
2	N 607096.02	E 7579399.95	323.00
3	N 607187.30	E 7579653.44	329.89
4	N 607221.41	E 7579704.92	335.00
5	N 606990.61	E 7579407.80	338.87
6	N 606905.65	E 7579555.30	378.72
7	N 606905.65	E 7579734.30	378.72

LEGEND:

- B-1 BORING LOCATIONS (CH2M HILL, JUNE 2000)
- ASPHALT PAVEMENT
- TOE OF FILL
- CUT SLOPE LIMIT
- BOTTOM OF DITCH
- FENCE DEMOLITION
- 370 EXISTING CONTOURS

NOTES:

1. MAN-MADE FEATURES TO BE DEMOLISHED INCLUDE, BUT ARE NOT LIMITED TO:
   A. HOUSE FOUNDATION *
   B. GARAGE BARN FOUNDATION *
   C. SEPTIC SYSTEM
   D. FENCE
   E. GRAVEL ACCESS ROAD
   F. CONCRETE VAULT STRUCTURE
   G. STONE SHED
   H. BRICKPATIO

   * WOOD STRUCTURES WERE BURNED 04/07/2002
2. CONTRACTOR STAGING AREA MAY BE ANYWHERE ON SITE, WITHIN CLEARING LIMIT FENCE SHOWN ON DWG C-9
3. ABANDON PIEZOMETER
4. ADJUST FINISHED ELEVATION OF DISPOSAL AREA TO WASTE ACTUAL QUANTITY OF EXCESS EXCAVATION MATERIAL. PROVIDE UNIFORM SURFACE THAT DRAINS TO DITCH INLET 2.
5. SEE ELECTRICAL PLAN FOR POWER POLE LOCATIONS.
6. BOUNDARY SURVEY WAS COMPLETED BY MATT DUNCKEL & ASSOC. IN APRIL 2002. SUCCESSFUL BIDDER MAY OBTAIN COPY OF SURVEY FROM THE CITY OF NEWBERG.
7. TREE REMOVED FOR CONSTRUCTION
8. CONSTRUCTION TBM-2 RR SPIKE IN 10" FIR TREE ± 50' N TANK ELEVATION 371.26 NGVD 1929

RECORD DRAWINGS

Revisions Drawn By Prisciliano Peralta Date Feb 2004

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

The Contract Document Drawings are printed documents dated April 2002, as subsequently amended, which define the scope, extent, and character of the work. The original document drawing was sealed and signed by Tim G. Yamada, State of Oregon, P.E. No. 62607PE, dated November 9, 1999.

DSGN	T. YAMADA
DR	H. GIFFORD
CHK	B. WAGNBR
APVD	BJ PHELPS

NO. | DATE | REVISION | BY | APVD

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CH2MHILL

CITY OF NEWBERG
NEWBERG, OREGON
CORRAL CREEK ROAD
4.0 MG RESERVOIR

CORRAL CREEK ROAD RESERVOIR
SITE WORK
SITE PLAN

A20040010

SHEET	4
DWG	C-1
DATE	APRIL 2002
PROJ	167877

FILENAME: 05cc101d_167877.dgn PLOT DATE: 25-AUG-2004 PLOT TIME: 10:15:19

NOTES:

1. CONTRACTOR IS RESPONSIBLE FOR MAKING IMPROVEMENTS TO EXISTING ACCESS ROAD THAT ARE NECESSARY FOR CONSTRUCTION ACTIVITIES DURING RESERVOIR IMPROVEMENTS.
2. MAINTAIN ACCESS ROAD FOR USE OF PRIVATE RESIDENTS AT ALL TIMES.
3. ROAD TO THE SOUTH SHALL NOT BE USED TO ACCESS THE SITE.
4. INSTALL SAMPLE STATION ON COMMON FILL LINE IN EXISTING VALVE VAULT. FINAL LOCATION OF SAMPLE STATION SHALL BE DETERMINED IN THE FIELD BY THE ENGINEER. SEE DETAIL 4 ON SHEET M-7.
5. CONTRACTOR SHALL SWEEP HWY 219 AT THE INTERSECTION OF THE ACCESS ROAD DAILY. DO NOT TRACK MUD, DIRT OR DEBRIS ONTO HIGHWAY.

SURVEY CONTROL:

TOPOGRAPHIC SURVEY DATUM FOR VERTICAL CONTROL IS NAVD 88. HORIZONTAL CONTROL IS BASED ON NAD 83, OREGON STATE PLANE NORTH.

CP#1: PK NAIL
N 617155.10
E 7565937.40
EL 386.17

CP#2: PK NAIL
N 617176.16
E 7566157.68
EL 396.88

CP#3: 5/8" IRON ROD
N 617106.16
E 7567007.16
EL 378.65

0 50 100
1"=50'

RECORD DRAWINGS

USE OF DOCUMENTS

THIS DOCUMENT, INCLUDING THE INCORPORATED DESIGNS, IS AN INSTRUMENT OF SERVICE FOR THIS PROJECT AND SHALL NOT BE USED FOR ANY OTHER PROJECT WITHOUT THE WRITTEN AUTHORIZATION OF KENNEDY/JENKS CONSULTANTS ©.

NO.	REVISION	DATE	BY
	RECORD DRAWINGS	JAN 2017	SCS

SCALES

0 1"
0 25mm
IF THIS BAR IS NOT DIMENSION SHOWN, ADJUST SCALES ACCORDINGLY.

THESE RECORD DRAWINGS HAVE BEEN PREPARED BASED ON INFORMATION PROVIDED BY THE CONTRACTOR AND OTHERS. KENNEDY/JENKS CONSULTANTS HAS NOT VERIFIED THE ACCURACY OR COMPLETENESS OF THE INFORMATION PROVIDED TO THEM AND DOES NOT WARRANT THE ACCURACY OR COMPLETENESS OF THESE RECORD DRAWINGS. USERS OF THESE RECORD DRAWINGS ASSUME ALL RISK OF LOSS RESULTING FROM THEIR USE.

DESIGNED SCS
DRAWN SLG
CHECKED MDL

CITY OF NEWBERG
NEWBERG, OREGON

NORTH VALLEY AND CORRAL CREEK RESERVOIRS SEISMIC UPGRADE AND INLET/OUTLET MODIFICATIONS

Kennedy/Jenks Consultants©
200 SW MARKET STREET, SUITE 500, PORTLAND, OREGON 97201

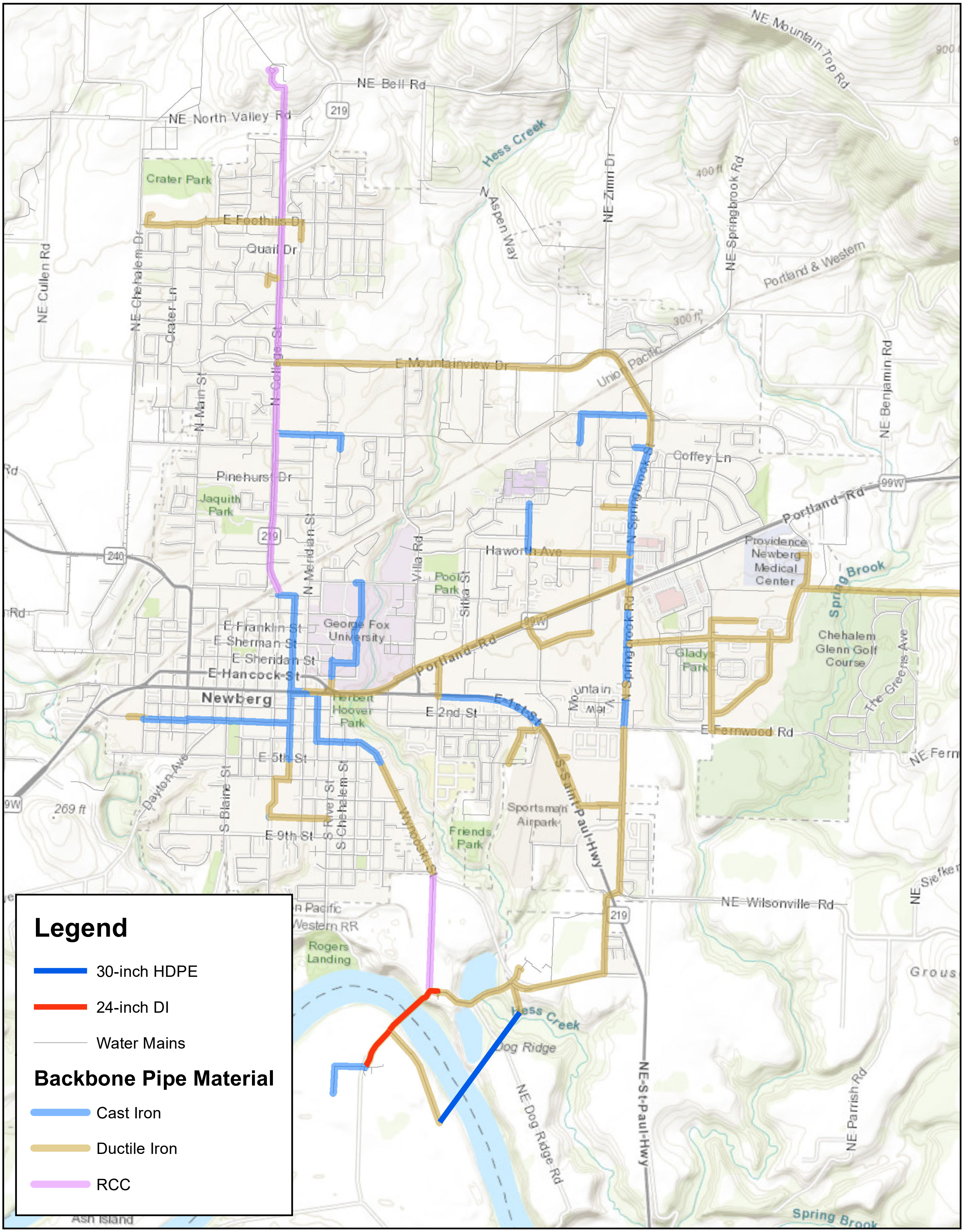
NORTH VALLEY RESERVOIRS NO1 & NO2 - SITE & EROSION CONTROL PLAN

FILE NAME 1091031C01_REC.DWG
JOB NO. 1091031.20
DATE SEPTEMBER 2015
SHEET C-1 OF -

\\Por2\cad\10\1091031.20-Newberg_NorthValley\1091031c01_REC.dwg
STEPHANIE GOTSCH 2/7/2017 1:58 PM

Appendix C:

Mitigation Recommendation 3 – Backbone Pipeline Replacements


Legend
30-inch HDPE
24-inch DI
Water Mains
Backbone Pipe Material
Cast Iron
Ductile Iron
RCC
Newberg
NE Bell Rd
NE North Valley Rd
Crater Park
E Foothills Dr
Quail Dr
E Mountainview Dr
Pinehurst Dr
Jaquith Park
Hess Creek
NE Zimri Dr
N Aspen Way
NE Springbrook Rd
Portland & Western
Union Pacific
Coffey Ln
Portland Rd
Providence Newberg Medical Center
Spring Brook
Haworth Ave
Chehalem Glenn Golf Course
George Fox University
E Franklin St
E Sherman St
E Sheridan St
E Hancock St
Herbert Hoover Park
E 2nd St
E 1st St
E 5th St
E 9th St
E Fernwood Rd
Sportsman Airpark
Friends Park
Gladys Park
Rogers Landing
NE Wilsonville Rd
NE St Paul Hwy
NE Dog Ridge Rd
NE Parrish Rd
Ash Island
NE Cullen Rd
NE Chehalem Dr
Crater Ln
N Main St
N College St
N Meridian St
Villa Rd
Sitka St
Dayton Ave
S Blaine St
S River St
S Chehalem St
NE Benjamin Rd
NE Mountain Top Rd
Pacific Western RR

Appendix E:

Recommendations for Future Studies

HDR

Memo

Date:	Monday, June 22, 2020
Project:	City of Newberg Seismic Resilience Assessment
To:	Brett Musick, PE, City of Newberg
From:	Andy McCaskill, P.E. and Katie Walker, P.E.
Subject:	Seismic Resilience Assessment – Recommendations for Future Studies

Introduction

The City of Newberg (Newberg) operates a water system consisting of a wellfield, raw water transmission pipelines, a water treatment plant, three water storage reservoirs, one pump station, and distribution system pipelines. In support of the 2017 Water Master Plan and Oregon Health Authority (OHA) guidelines, Newberg conducted a water system seismic resilience assessment (SRA). The purpose of this memorandum is to identify the additional recommended studies to further clarify and confirm the City's seismic mitigation needs.

Future Studies

Seismic Recovery Goals

During workshops, alternative demand strategies were discussed, such as a potential influx of residents from coastal areas. Additional studies could be conducted to identify additional demands that impact the water storage available within the system.

Geotechnical

Additional geotechnical studies are recommended to better classify the seismic hazards that the water system components may experience. Targeted field investigations will allow Newberg to focus on the most hazardous areas. These include:

- Investigate vulnerabilities of the horizontal directional drill transmission main under the river. The soil conditions in the south side of the alignment indicate liquefaction induced settlement, especially at the transition to the well field piping.
- Impacts of seismic activity to the well field, well infrastructure, and groundwater. It is likely, based on the soil information available, that significant liquefaction and lateral spreading will occur during a CSZ earthquake. This could cause separation between the well casing and the pipe connection, plug the screens and reduce the capacity of the well, and fluctuation in the groundwater levels.
- Review the effects of bank erosion due to the Willamette River on slope stability in the proximity of the WTP.

Structural

The SRA included high level assessments of structural components within the City's water system. Depending on the desire to retrofit or rehabilitate the pipeline bridge, additional studies should be conducted to identify the mitigation measures needed to maintain the structure and the pipeline during a CSZ event. Likewise, additional investigations should be conducted at the WTP to identify specific mitigation measures for individual structural components.

Mitigation Strategies

As part of the SRA, only five mitigation strategies were identified. Additional improvements need to be identified and implemented to achieve the LOS goals. Additional mitigation strategies to investigate include:

- Wellfield infrastructure improvements based on the recommended additional geotechnical investigations.
- Improvements to the seismic resiliency of the transmission system main to address the potential for pipe separation.
- Improvements to slope stability at the WTP to prevent landslides.
- Installation of pipeline bridge isolation valves to minimize water loss if the bridge or pipeline fails.
- Construct a seismic resilient well with backup generator away from the river to replace well 4.
- Install seismic raw waterline from new seismic well to existing 30" HDPE line.
- Install a raw water booster pump station with a connection to potable water system.
- Investigate locations where seismic joints can be added to protect the water system.

Other Studies

- Develop new engineering standards to address seismic resiliency needs including those for the backbone system and updates to water service connections
- Review SCADA and GIS mapping system to see where improvements can be made with helpful alarms and feedback.
- Review fiber optic and power supply to identify vulnerabilities, and how the outage of those items would impact the water system.