Feasibility Study Report

Feasibility Study for Water Storage Improvements

City of Manzanita, OR

May 3, 2022

Funded By:
Federal Emergency Management Agency (FEMA)
Partnered with
Oregon Emergency Management (OEM)
$75,000

and

City of Manzanita
$25,000

Contents

Tables

Figures

Appendices

Appendix A. Shannon & Wilson Geotechnical Report

Appendix B. 10% Design Drawings

Appendix C. Construction Cost Estimate

EXPIRES: 06/30/2023

Acronyms and Abbreviations

AACE	American Association of Cost Engineering
ANSI	American National Standards Institute
ATS	automatic transfer switch
AWWA	American Water Works Association
City	City of Manzanita
CMU	concrete masonry unit
CSZ	Cascadia Subduction Zone
DEQ	Oregon Department of Environmental Quality
DOGAMI	Department of Geology and Mineral Industries
EI&C	electrical, instrumentation, and control
HDPE	high-density polyethylene
hp	horsepower
IBC	International Building Code
KVA	kilovolt-ampere
kW	kilowatt
MG	million-gallon
OPCC	opinion of probable construction cost
ORP	Oregon Resilience Plan
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Structural Specialty Code
PVC	polyvinyl chloride
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
WMP	Water Master Plan

1 Introduction

Due to serious concerns about the seismic vulnerability of its three existing reservoirs, the City of Manzanita (City) commissioned a feasibility study to evaluate viable mitigation strategies to improve the City's water storage system resilience. The three project alternatives for evaluation included:

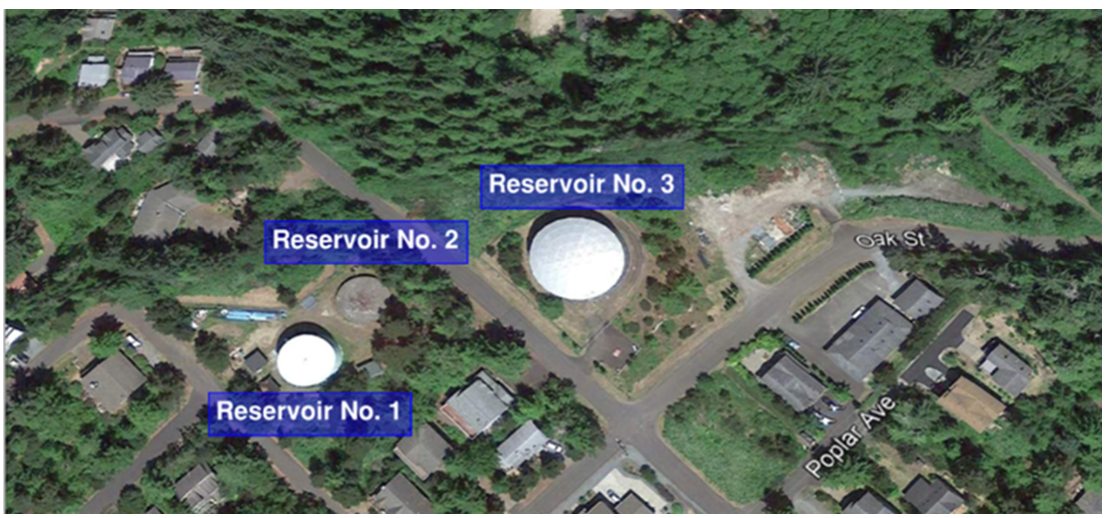
Alternative 1: Minimal upgrades

Alternative 2: Retrofit Reservoir No. 1

Alternative 3: Complete Replacement of Existing Reservoirs

After collecting relevant background and research information and evaluating the alternatives, the City decided Alternative 3, Complete Replacement of Existing Reservoirs, is the best solution and value for the community. This decision was based on a comparison of several factors such as seismic resilience, cost, operations and maintenance, and constructability. With the preferred alternative identified, the design and cost estimate have been advanced to a 10-percent level of design to provide the City with a feasible design concept for the replacement tanks and a detailed construction cost estimate providing a strong foundation for final design. This report also provides the background, basis of design, and project details that may be needed as content and justification for subsequent project funding applications or endeavors. Figure 1 shows the location and orientation of the existing reservoirs proposed for replacement.

Figure 1. Existing Reservoirs



1.1 Background

Relevant background information was collected and research conducted through discussions with the City, review of previous studies and reports, review of record drawings, and site observations. Applicable codes and standards, best practices, mapping resources, and environmental resources also were completed. HDR prepared a Background and Research Memorandum and submitted to the City on April 22, 2021.

An Alternatives Evaluation also was conducted to compare the three alternatives identified in the introductory paragraph. Key components of the evaluation included development of schematic design concepts for each alternative, an associated cost estimate, and identification of the pros and cons. HDR prepared the Alternatives Evaluation Technical Memorandum and submitted to the City on June 7, 2021. An alternatives selection workshop was subsequently held with the City on October 8, 2021.

This report, the 10-percent drawings, and detailed construction cost estimate represent completion of the Feasibility Study project.

1.1.1 Project Purpose and Drivers

As identified in the 2018 Water System Seismic Resilience Study completed by BergerABAM, there are several hazards and issues that create vulnerabilities for the existing reservoirs. The City is located adjacent to the Pacific Ocean, making it particularly vulnerable to a Cascadia Subduction Zone (CSZ) seismic event (magnitude 9.0). Community hazards may include ground accelerations (shaking), tsunami, landslides, and liquefaction. These hazards pose a significant threat to the water delivery system, reservoirs, and City recovery efforts.

Built primarily on sandy soils, the City's three existing reservoirs may be susceptible to liquefaction and ground movement. The largest and most recently constructed water reservoir tank (1.5-million-gallon [MG] Reservoir No. 3) is at risk for structural failure because it is located adjacent to a steep slope designated as a landslide hazard. The remaining two reservoirs risk structural failure due to a lack of lateral reinforcement and the effects from ground subsidence.

In addition, the complex network of yard piping, connections, and valving between the three existing reservoirs creates additional vulnerability for the community's water storage and delivery. Even if the reservoir structures survive a seismic event, the contents are likely to spill because of sheared pipe connections or joint separations. Losing water storage would impede the City's ability for first response and recovery actions such as firefighting and delivery of potable water to residents and critical facilities after a seismic event.

The 2013 Oregon Resilience Plan (ORP) is a key guidance document for planning and designing resilient water infrastructure. The ORP recommends that communities improve the backbone of their water system, of which reservoir storage is a key component. The ORP also states that the water delivery system should be capable of supplying key community needs, including fire suppression, health and emergency response, and community drinking water distribution points while more wide-spread damage to the larger components of the system are being addressed.

1.1.2 City's Project Goals

A project kickoff meeting was held on March 17, 2021 and attended by Dan Weitzel – City Public Works Director, Dan Johnston – HDR Project Manager, Andy Fortner – HDR Structural Engineer, and Don Best – HDR Electrical Engineer. The City identified a range of goals for this project including:

- Mitigating landslide and seismic risks associated with the reservoirs

- For the retrofit or replacement options, providing the City a two-reservoir system allowing one to be taken offline for maintenance
- Simplifying the pipe network and hydraulics between the reservoirs
- Retaining water storage during and after a seismic event for fighting fires and community use during the disaster recovery period

2 Existing Conditions and Facilities

2.1 Water Tank Structures

The existing system consists of three reservoirs, or water storage tanks (Table 1).

Table 1. Existing Reservoirs

Reservoir No.	Material	Capacity (MG)	Construction Date	Notes
1	Welded Steel	0.50	1979	Recoated in 2003
2	Concrete	0.25	1960	Storage is not useful
3	Glass-fused, Bolted Steel	1.60	1997	Repairs needed for settlement issues

Reservoir No. 1 appears to be in fair condition; however, the exterior coating is beginning to fail. Some areas are showing surface corrosion where the paint has flaked off. If the tank remains in service, cleaning and coating of local corrosion areas would be needed. Recoating of the entire tank may be needed if it remains in service for more than 5 years from the date of this study. Exterior corrosion can also be a likely indicator of interior corrosion issues. A full inspection of the tank should be performed to determine if any areas showing corrosion have excessive section loss and require repair. No guardrails are present around the top of the tank, which does not meet Occupational Safety and Health Administration (OSHA) requirements.

Record drawings from 1979 appear complete and provide design details typical for the time period. As part of BergerABAM's 2018 Water System Resilience Study, a high-level evaluation of this tank was conducted and revealed seismic load inadequacies primarily because the steel plates used for the tank shell were relatively thin and susceptible to buckling. Furthermore, the tank is not anchored to the foundation which could lead to tank displacement during a seismic event.

Reservoir No. 2 is in fair condition considering its age. Some small surface spalls and bugholing in the surface matrix of the concrete was noted. No significant cracks or evidence of leakage, such as efflorescence, was noted. No guardrails are provided around the top of the tank, which does not meet OSHA standards. The low volume of storage is not of much value to the water system; however, the tank remains in service as it is integral to the piping layout and hydraulics of the water system in its current configuration. The City has been unable to locate record drawings for this tank, which would have likely included tank reinforcing design details. Without drawings, designing a retrofit alternative is more complicated. It is, however, unlikely that a retrofit would be productive due to the age, material, and low capacity of Reservoir No. 2.

Reservoir No. 3 shows numerous water seepage trails on the surface of the tank at bolt holes and panel joints. As reported by the City, these leaks have been sealed and the sealant appears effective at this time. The geodesic dome roof shows no signs of obvious distress. Per City-provided information, no serious issues have been noted on the interior of the tank. The access ladder to the top of the tank appears in serviceable condition and to meet current standards. Current OSHA regulations require that by 2036, all caged ladders must be retrofitted with a ladder climbing device or replaced with a less hazardous access, such as stairs.

This type of tank (glass-lined steel) generally performs poorly during a seismic event. Furthermore, Reservoir No. 3 is perched at the top of a slope that is highly susceptible to a landslide during an earthquake event. The City has been unable to locate record drawings for the tank but indicated that a consultant (HGE) had been engaged with the storage tank vendor for the original design.

2.2 Geotechnical Conditions

Shannon and Wilson (2021) conducted a geotechnical feasibility study and prepared a report documenting its findings (Appendix A). This section summarizes those findings, which identified the following geotechnical hazards:

- Ground shaking
- Landslides and slope instability, particularly at the Reservoir No. 3 site
- Liquefaction and related phenomena, such as settlement, lateral spread, and post-seismic soil strength reduction

As per the geotechnical report, the risk of tsunami and fault rupture are low near all three tanks. The north facing slope near Reservoir No. 3 would likely experience a landslide during a seismic event. This is based on a history of ground deformation at nearby Epoh Avenue, as well as large quantities of undocumented and uncontrolled fill placed at the site over the course of many years. According to the City, up to 20 feet of uncompacted and undocumented fill was placed on this site. The bulk of the material is believed to have come from the Neahkahnie Landslide, however the composition of the fill and compactive effort provided during placement is unknown.

Data from the Department of Geology and Mineral Industries (DOGAMI) and the Oregon Statewide Geohazards Viewer indicates there are existing landslides in the region north of Reservoir No. 3. Based on this, the seismic slope hazards at this location are extremely high and would not be economically feasible to mitigate due to the large size of the unstable area. For this reason, the Reservoir No. 3 site area is not being considered for new water storage facilities.

The parcel containing Reservoir Nos. 1 and 2 is a relative high point in the area. The nearest slope is northeast more than 100 feet away and no evidence of slope instability was noted during the site visit. DOGAMI maps this area as a moderate landslide hazard, which is based on an analysis of spatial statistics and intended for planning purposes only. Additional site-specific geotechnical studies are required to fully characterize the soils and identify hazards. However, based on the required mitigation for liquefaction, discussed below, the risk of a slope stability issue jeopardizing the new tanks is low.

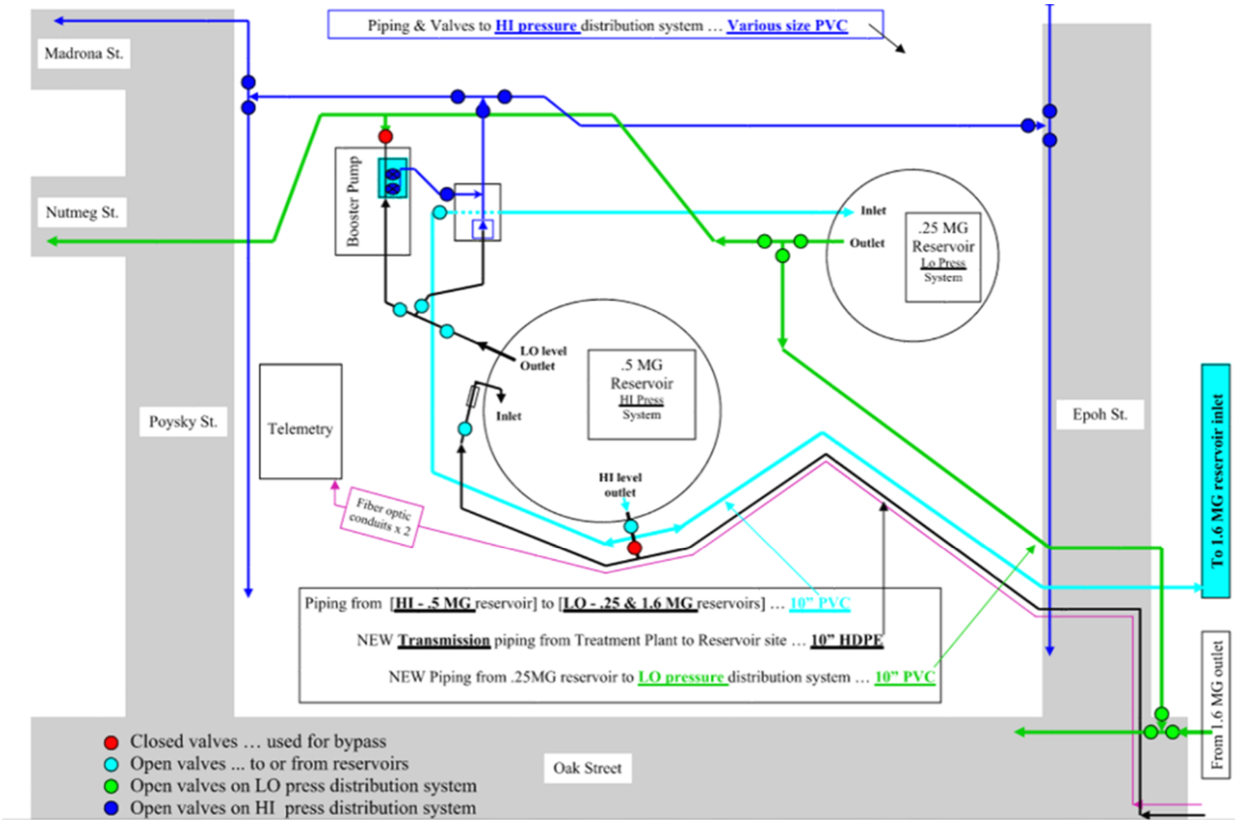
DOGAMI also classifies the parcel with Reservoir Nos. 1 and 2 as having a moderate to high probability of liquefaction. Much like the landslide hazard classification, this classification is based on the soil type and an analysis of spatial statistics. The presence or absence of groundwater is required to fully characterize the liquefaction risk. An absence of groundwater reduces the risk of liquefaction. Additional geotechnical investigation, such as site-specific borings, cone penetrometer tests, or a combination of the two, will be required to determine the location of groundwater and assess the density of the sands the site is founded on to fully characterize the liquefaction hazard. The current cost estimate conservatively assumes that liquefaction will occur. Ground improvements (i.e., soil mixing) have been assumed for mitigation of the liquefaction risk.

2.3 Pipelines and Connections

Finished water from the treatment plant comes to the south reservoir site through a 10-inch high-density polyethylene (HDPE) transmission pipeline and connects to Reservoir No. 1 through a visible inlet at the base of the reservoir. This section of HDPE pipeline was built in 2008 and reflects the most recent upgrades made to the yard pipe network (Figure 2). There are two outlets from Reservoir No. 1 that connect to the larger network. There is a high level outlet that connects to Reservoir Nos. 2 and 3 through a 10-inch polyvinyl chloride (PVC) pipeline and serves the low pressure distribution zone. There also is a 10-inch low level outlet that feeds into the adjacent pump station, serving the high pressure zone. Reservoir No. 2 has a single buried inlet and outlet connection with the outlet splitting at a tee in the yard and feeding the low pressure zone to the north and south. The 10-inch PVC distribution pipeline to the north connects with the outlet from Reservoir No. 3 to feed the low pressure zone.

Because of the pipeline age and limited records, it is generally assumed that all buried pipeline and tank connections, including valves and pumps, are rigidly connected, thereby representing seismic risk for pipeline breaks and leaks during a seismic event. At the inlet to Reservoir No. 1, there is a ball joint coupling, which would typically provide some needed flexibility, however, it does not appear to be effectively mounted and maintained. A single, 10-inch swing check valve is located on the low level outlet from Reservoir No. 1 feeding into the high-pressure zone.

Figure 2. Yard Pipe Network



2.4 Electrical, Instrumentation, & Control

The Reservoir Nos. 1 and 2 site receives overhead electrical service at 120/240 Volts, 200 Amps from the Tillamook Public Utility District. Standby power is provided by a trailer-mounted, diesel engine generator set with a standby rating of 15 kilowatts (kW; Figure 3). The standby power system must be manually started and manually transferred upon loss of utility power. The transfer to standby power requires manual operation of mechanically linked circuit breakers in the service panelboard. The generator includes a belly tank with the storage capacity to operate the generator for approximately 24 hours as reported by City staff.

Figure 3. Trailer-Mounted Generator

The electrical, instrumentation, and control (EI&C) loads at the reservoir site include:

- Booster pump (5 horsepower [hp]) and associated control panel (across the line-type motor starter)
- Uninstalled booster pump with integral variable frequency drive (1.5 hp)
- Programmable logic controller control panel
- Classical radio equipment
- Emergency services radio equipment (Ultra/Very High Frequency)
- Treated water analytical equipment (pH, temperature, and chlorine)
- Treated water turbidity analyzer
- Reservoir level transmitters
- Radio/Control building air conditioning
- Genset heater and battery charger
- Outlet receptacles
- Lighting

The reservoir telemetry cabinet includes a Koyo – DirectLogic 205 programmable logic controller that is monitored by a computer-based, human machine interface software package (Wonderware InTouch) via fiber optic connection. The computer is in the Public Works Building and monitors/controls the groundwater wells, Manzanita Water Treatment Plant, and Wheeler Station (via DSL communications and City-owned fiber optic cable). The computer includes alarm notification software and can be remotely accessed. The City's system integrator for the programmable logic controller and human machine interface is Camtronics (Terry Nelson) from Camas Valley, Oregon.

2.5 Site Conditions and Topography

Reservoir No. 3 site conditions are composed of a flat gravelly surface surrounded by trees. The grade slopes significantly downward on the north side of the property. The surface elevation of the property is approximately 210 feet above mean sea level. The lot for Reservoir No. 3 abuts Oak Street to the southeast and Epoh Avenue to the southwest with no immediate neighbor to the north. There is vehicle access around the outside of the tank with a chain link fence around the perimeter of the property. There are no other buildings or storage facilities at this location.

Reservoir Nos.1 and 2 share the same property southwest of Reservoir No. 3 and sit approximately 20 feet higher at a surface elevation of 230 feet above mean sea level. The parcel for the Reservoir Nos. 1 and 2 site abuts Epoh Avenue to the northeast, Poysky Avenue to the southwest, and residential properties to the southeast and northwest. The site is relatively flat at the base of both tanks with access around each tank. The tank area is generally elevated from the adjacent streets and residential property, with naturally treed and vegetated downward slopes in each direction. Three existing buildings house the pump station, controls, and yard equipment.

The topography of the existing reservoir site varies significantly and limits or complicates construction beyond the existing developed area. The northeast, southeast, and southern margins of the parcel slope downward significantly to daylight with existing surrounding grades.

2.6 Environmental Resources and Permitting

A desktop analysis of environmental resources at the project site was completed to identify potential constraints.

2.6.1 Endangered Species

The U.S. Fish and Wildlife Service (USFWS) Information for Planning and Consultation lists the following species as potentially affected at the project location:

- Marbled murrelet
- Northern spotted owl
- Short-tailed albatross
- Western snowy plover
- Leatherback sea turtle
- Loggerhead sea turtle
- Olive ridley sea turtle

The presence of or suitable habitat for endangered species within the project area is considered unlikely. As the project progresses into future phases, additional analysis may be required to determine if suitable habitat may be affected by project activities.

2.6.2 Wetlands and Waters

A search of the USFWS National Wetlands Inventory and aerial images resulted in no indicators of any wetlands or waters within the project site. While permits are unlikely, future phases of this project should fully confirm lack of wetlands or waters within the project site.

2.6.3 Historic and Cultural Resources

If the project is federally funded, Section 106 Consultation would be required during a future design phase for historic and cultural resources that may potentially be located within the project area. A search of the Oregon Historic Sites Database resulted in no historic properties located within the project area. Archaeological resources are not publicly available online, however, given the proximity to the coastline, there is potential for archaeological resources to be located within or near the project area. Historic and archaeological surveys would be required before any construction activities could occur.

2.6.4 Hazardous Materials

A review of the Oregon Department of Environmental Quality's (DEQ) Facility Profiler-Lite resulted in no documented hazardous materials at or near the project site. Additional confirmation of presence or absence of hazardous materials could be established with a Phase I Hazardous Materials Study.

2.6.5 Zoning and Land-use

The existing site is within the City's R-2 zone (medium density residential). The northwest corner of the site with the two existing reservoir tanks is within the Beaches and Dunes Overlay Zone.

Figure 4. Zoning

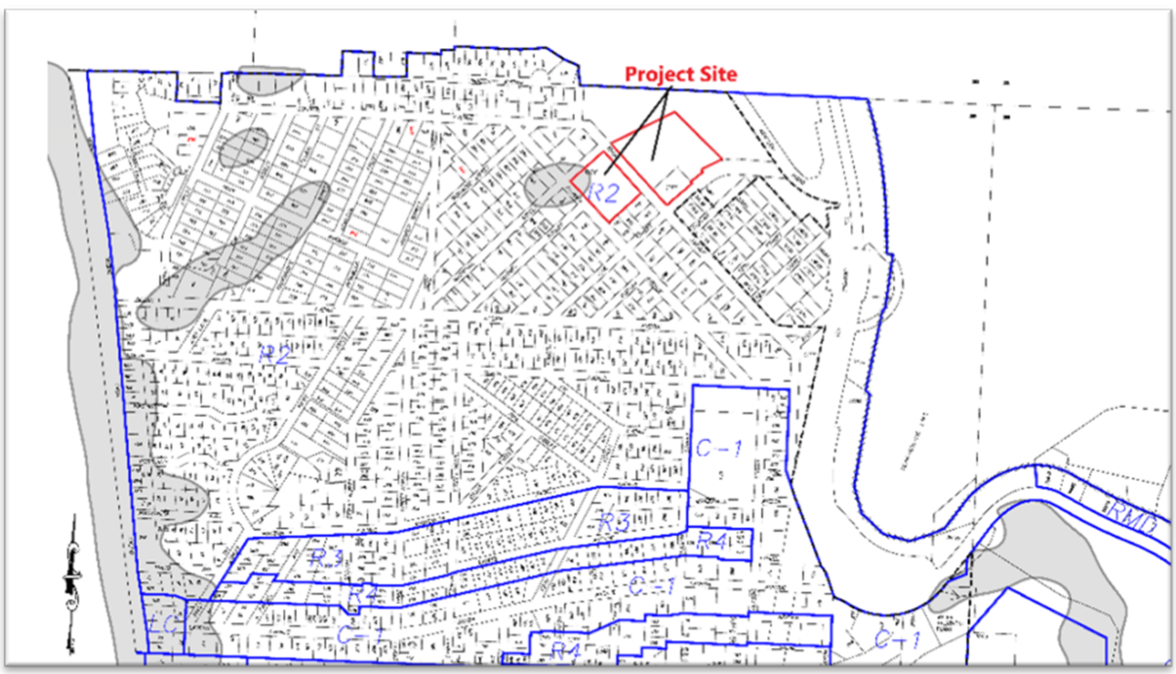


Table 2 summarizes potential constraints and anticipated permits required for the applicable zones.

Table 2. Potential Permits

Permit/ Approval	Agency	Applicability	Timeline	Additional Analysis	Risks/ Challenges
Federal					
Section 106 Historic/Cultural Review	Oregon State Historic Preservation Office (SHPO)	Projects with federal nexus	~5 months for fieldwork, baseline report, and SHPO review	Historic/Cultural Resource Survey & Report	Timeline assumes no resources will be affected
Section 7 Endangered Species Act	USFWS	Projects with federal nexus. Project will likely result in no effect to ES- listed species	1 month	Assumed Endangered Species Act No Effect Memo	Timeline assumes no endangered species would be affected by the project
NEPA	Federal Emergency Management Agency	Projects with federal nexus	--	--	--
State					
NPDES 1200-C Stormwater Permit	DEQ	Construction that will disturb more than 1 acre of land or may discharge stormwater to surface waters	Submit 30 days before soil disturbance. 14-day public notice required after DEQ review is completed	--	--
Local					
Conditional Use Permit	City of Manzanita	Public utility facility in residential zone	--	Site plan; pre-application meeting	--
Tree Removal Permit	City of Manzanita	Removal of any tree larger than 6 inches in diameter or 4.5 feet in height	--	Site plan; justification statement; replacement plan	--
Building Permit	City of Manzanita	All commercial and residential development proposals	--	Site Plan Approval; Site Investigation and Hazards Report by professional geologist required for Beaches and Dunes Overlay Zone	Burden of proof to show construction feasibility in potentially hazardous areas (steep slopes)

2.7 Mapping Resources

The following mapping resources were used for the feasibility study. Additionally, the CAD files provided by the City from the 2018 HGE 10-inch water main project were used as backgrounds for drawings and supplemented information from the resources listed below.

- *Oregon HazVu: Statewide Geohazard Viewer* (https://gis.dogami.oregon.gov/maps/hazvu/) can identify hazards such as flood, CSZ, coastal erosion, volcano, earthquake, and landslide.
- *Tsunami Inundation Map (TIM)* (https://www.oregongeology.org/pubs/tim/p-TIM-overview.htm) shows modeled tsunami inundation zones and is the basis for Oregon tsunami evacuation protocol.
- *Tillamook County Public GIS* (https://www.co.tillamook.or.us/gov/GIS/Default.htm) contains aerial imagery and public GIS data.
- *Google Earth Pro 2021:* Useful for aerial imagery and an approximate definition of existing topography.

3 Design Standards and References

3.1 Water System and Resilience

Current state and federal regulations, design standards, and hazard mitigation guidance include:

- ORP (2013) provides guidance to determine vulnerable infrastructure, recovery timelines, and mitigation associated with the CSZ earthquake.
- *Tillamook County Multi-Jurisdictional Hazard Mitigation Plan Update* (2017) identifies regionally specific hazards and mitigation plans.
- U.S. Geological Survey's (USGS) *ShakeAlert Earthquake Early Warning System* (2021) is a new program that detects earthquakes that have already begun, offering seconds of advance warning that would allow people and systems to take actions to protect life and property from destructive shaking.

American National Standards Institute (ANSI) and American Water Works Association (AWWA) standards apply to water treatment and supply, including distribution pipe, valves, and fittings. The ORP generally recommends strengthening backbone water system facilities including major transmission and distribution pipelines as well as storage facilities. This includes incorporating seismic considerations into construction and rehabilitation of all water system components.

3.2 Tank Structures and Resilience

The following codes and standards are applicable to the design of new reservoirs or retrofit of existing reservoirs:

- Oregon Structural Specialty Code (OSSC) and the International Building Code (IBC). The State of Oregon currently adopts the 2018 IBC via the 2019 OSSC. The 2021 IBC is expected to be adopted in October 2022. No significant changes to seismic loadings are anticipated between the currently adopted 2018 IBC and the upcoming 2021 IBC.
- American Concrete Institute 350-06: Code Requirements for Environmental Engineering Concrete Structures
- American Concrete Institute 350.3-06: Seismic Design of Liquid-Containing Concrete Structures
- AWWA D110: Wire and Strand Wound Circular Prestressed Concrete Water Tanks
- AWWA J100: Risk and Resilience Management for Water and Wastewater Systems
- ORP (2013)

Building codes, such as IBC and OSSC, are primarily intended to address basic life safety (Risk Category II). The ORP, however, recommends designing for a higher level of resilience, which would be the equivalent of a Risk Category IV design for critical facilities. Risk Category IV structures have more stringent detailing requirements, such that they will remain operational post-earthquake.

3.3 Electrical and I&C and Resilience

The design will conform to the latest edition of electrical codes adopted by the State of Oregon (in effect as of the anticipated permitting date) and in accordance with the approved codes and standards listed below:

- Oregon Electrical Specialty Code (2021)
- Oregon Energy Efficiency Specialty Code (2021)
- National Fire Protection Association 70 National Electrical Code (2020)
- National Electrical Manufacturers Association - MG1, Motors and Generators
- Life Safety Code, National Fire Protection Association101-HB85
- Underwriters Laboratories

4 Alternatives Evaluation Summary

An alternatives evaluation was previously performed to determine the preferred alternative for design. Three design options were originally identified as the most promising alternatives for this study and are summarized in the sections that follow. As previously stated, the City ultimately selected Alternative 3 – Complete Replacement of the Reservoirs.

4.1 Alternative 1 – Minimal Upgrades

Alternative 1 consisted of completing a minimal number of upgrades to enhance the remaining service life of the existing facilities. This alternative represented the lowest cost alternative and considered utilizing existing infrastructure as-is. With this being the lowest cost alternative, it becomes more attractive if believed that there is not enough value gained from the higher cost alternatives.

Pros for Alternative 1

- Least costly
- Least disruptive
- Utilizes remaining life of existing infrastructure

Cons for Alternative 1

- Only minor improvements to seismic resilience
- Risks of losing water storage during/after a seismic event remain
- Reservoir No. 3 continues to present risk of failure as slope continues to settle
- As existing tanks continue to age, more repairs would be required

4.2 Alternative 2 - Retrofit Reservoir No. 1

Alternative 2 presented a hybrid approach by incorporating construction of a new reservoir, while also retrofitting Reservoir No. 1. In this Alternative, Reservoir Nos. 2 and 3 would be demolished or abandoned.

Pros for Alternative 2

- Improves seismic resilience by installing a new larger tank designed for full seismic resilience and retrofitting the existing Reservoir No. 1 to increase its seismic resilience
- Eliminates Reservoir No. 3 which presents a high risk of failure due to slope settlement and landslide potential during a seismic event
- Replaces or upgrades most of the water distribution system and components, further enhancing the seismic resilience of the overall storage and distribution system
- Integrates the seismic early warning equipment with the system and provides improved resiliency of water storage

Cons for Alternative 2

- Retrofitting a 40+-year old tank may not provide desired life expectancy when compared to the required overall construction cost
- True cost to repair/retrofit Reservoir No. 1 is unknown until a thorough inspection is performed
- Design and constructability challenges because the grade around Reservoir No. 1 must be maintained, thereby constraining options for the new tank design

4.3 Alternative 3 – Complete Replacement of Existing Reservoirs

The final alternative included complete replacement of existing reservoirs and on-site facilities. Alternative 3 consisted of replacing existing Reservoirs No. 1 and 2, and abandoning or removing Reservoir No. 3.

Pros for Alternative 3

- Provides highest level of seismic resilience between the three, proposed alternatives
- Allows for total re-calibration of operations and hydraulics, including the pump station
- Allows for optimal placement of isolation and control valves
- Provides longest life expectancy for facilities
- Allows complete integration of telemetry and controls, including the ShakeAlert system
- Eliminates the high risk of failure associated with Reservoir No. 3
- Allows for complete reconstruction of the existing reservoir site for optimal grading, layout, and design
- Eliminates constructability issues associated with trying to preserve Reservoir No. 1
- Eliminates unknown costs associated with retrofitting/rehabilitating existing Reservoir No. 1

Cons for Alternative 3

- Potential operational challenges while depending on Reservoir No. 3 during construction
- Highest overall construction cost of the three proposed alternatives

5 Design and Approach for Selected Alternative

As previously indicated, the City selected Alternative 3, Complete Replacement of Existing Reservoirs, as the best solution and value for the community. Appendix B provides the 10-percent design drawings for this alternative.

5.1 Site Layout and Improvements

The site for the selected alternative utilizes the same City-owned property where the existing Reservoirs No. 1 and 2 are located. The existing storage tanks and yard pipelines currently located within this lot will be demolished and removed from the site completely. The existing facility structures will be demolished and replaced.

During demolition of the existing structures, the site will be levelled to accommodate the new reservoirs, facilities, and vehicle access. Site elevations are expected to be similar to existing site elevations. In addition to new tanks and buildings, the proposed site improvements include construction of new gravel driveways for vehicle access and storage. Clearances are maintained around the storage tanks so that vehicles and equipment can maneuver through and around the site. All yard space will be surfaced with gravel for access and storage.

Retaining walls are needed on the northern and southern project area boundaries to stabilize existing slopes and increase the usable space. Associated landscaping will be necessary to revegetate disturbed areas. Security fencing will be provided along the perimeter of the project area with associated gate access points.

5.2 Demolition and Temporary Facilities

Tank Nos. 1 and 2 will be demolished. The concrete and reinforcing steel in Tank No. 1 and steel shell of Tank No. 2 can be recycled, as they have little to no salvage value, particularly once labor costs for demolition and costs to transport the waste is considered.

As an alternative to full demolition and disposal of Tank No. 3, the tank could be modified with doors and repurposed as a storage silo. It could also be disassembled and sold for use elsewhere.

Demolition and replacement of Reservoir Nos.1 and 2 will require careful planning and construction sequencing to maintain water service through the remaining infrastructure. Reservoir No. 3 has a capacity of 1.6 MG, water service elevation of 237 feet, and can be utilized as the single network storage tank and point of distribution when the other two are taken offline. There are two existing pipe connections at Reservoir No.3 - an existing 10-inch inlet from the water treatment plant through a normally closed valve as well as a 10-inch direct connection to Reservoir No. 2. Prior to demolition, the connection between Reservoirs No. 2 and 3 would need to be re-routed to connect the HI and LO pressure systems to Reservoir No. 3. A temporary pump station will also need to be installed for the HI pressure system. During the design phase, this plan can be further coordinated and refined through discussions with City Operations staff to ensure adequate level of service.

5.3 Tank Structures

The selected alternative consists of two identical AWWA D110, Type I, prestressed concrete tanks. These types of tanks generally have the lowest maintenance cost and good seismic performance due to their ductile nature. Prestressed concrete tanks are common in the region. Generally, these tanks are designed and constructed by a specialty tank vendor, such as DN Tanks.

Prestressed tanks were chosen at this time as they provide greater flexibility in placement. Unlike a steel tank, a concrete tank can be partially buried. Based on the site constraints, taller tanks are necessary because of the limited space for tank diameter. The ability to bury the tanks is beneficial, as it reduces the overall tank height above grade. If desired, a steel bolted or welded tank can be further explored during the design phase.

New standards being adopted (NSF 61) require that all tanks be constructed of materials certified for contact with drinking water. This can be achieved in one of two ways: 1) all materials used in construction, including cement, aggregate, concrete admixtures, reinforcing, etc., must be certified and approved by NSF, or 2), by the application of an NSF certified and approved coating system. Application and maintenance of a coating system can be a significant cost over time. Therefore, for this study, it is assumed that all materials used in tank construction that are in contact with water will be NSF 61 certified.

As the geotechnical conditions at the site are not yet fully characterized, it is assumed for this feasibility study that ground improvements will be required to mitigate for liquefaction. Ground improvements would be placed directly under the tanks and auxiliary structures, creating a mass of improved soil to build on.

The most cost-effective approach to ground improvements is deep soil mixing, where a large drill is used to inject a binding slurry into the ground. This helps to solidify the soft soils and improve their engineering characteristics. Additionally, with the proper ground improvement design, standard shallow foundations could be used to support the tanks and auxiliary buildings, without the need for expensive deep foundations (driven piles or drilled shafts). This will further reduce cost and simplify construction.

5.4 Auxiliary Structures

Two auxiliary structures are required; a generator enclosure and a pump station and telemetry building. The generator enclosure will likely be a prefabricated structure supplied with the generator, with the generator and enclosure placed on a concrete equipment slab. If ground improvements are required for mitigation of seismic hazards, the ground improvements should extend such that the standby generator and its enclosure are included within the extents..

The new pump station building will be of concrete masonry unit (CMU) construction, with prefabricated timber roof trusses. A small electric heater and passive cooling will be provided to prevent temperatures from reaching extremes. A recent change to the Oregon Energy Code requires insulation of small CMU structures. This can be accomplished via the use of insulated blocks or panels on the exterior of the building. Insulated exterior panels are generally the most cost-effective option, while providing easy mounting of wall-mounted equipment via embeds directly into the CMU walls.

Much like the generator, ground improvements, if required, should extend to protect the pump station building as well. This would allow the building to be founded on a typical spread footing and stem wall type foundation.

5.5 Pipelines and Connections

Construction of new storage tanks will require all new inlet and outlet connections for the transmission and distribution system, including new yard pipelines, valves, and fittings. The goal for newly constructed pipes and connections is to be simply configured for operation and maintenance and seismically resilient for longevity. In addition, strategically located valves and an inter-tank connection are included for isolation. Tank inlet and outlet connections will include restrained flexible couplings. Individual inlet and outlet valves for each tank will be linked to the Shake Alert system to close in the case of a seismic event. This will preserve water storage even if upstream or downstream pipelines rupture. Altitude valves are provided on the inlet lines to each tank and will control reservoir levels. Pressure reducing valves and flow meters are provided on the outlet lines for each tank. New pipe and fittings will be Class 52 ductile iron with restrained joints. All new valves will be ductile iron body, as well, with joints specified by the design. All components should be NSF 61 certified and conform to ANSI/AWWA.

5.6 Water Storage Volume

Per the 2021 Water Master Plan (WMP), the recommended minimum storage capacity should be equal to [(3 x Average Day Demand) + (fire flow)]. For the planning period of approximately 50 years (2070) the minimum required volume is 1.77 MG. The WMP also recommends a total storage volume of 2.0 MG and indicates that extra storage may benefit the City with regard to fire flow volumes, as well as the ability to provide emergency water to Neahkahnie and Nehalem through interties. The benefits of extra water storage is offset by potential water quality ageing issues and greater construction cost.

There is little published guidance with respect to the provision of additional water storage related to longer disaster recovery periods. However, the ORP (2013) suggests that communities with more resilient water storage will be afforded a longer recovery time with the ability to rely on stored water in lieu of producing more treated water.

5.7 Water Storage Hydraulics

Current documentation shows that more than 90 percent of the City’s demand is directed via gravity to the lower pressure zone. The remaining 10 percent of water demand is directed to the higher elevations which is supported by a pump station. Assuming distribution to the pressure zones stays the same, the proposed tanks are intended to perform similarly with regard to existing hydraulic profiles. As such, the minimum water service elevation for the proposed tanks is 257 feet.

5.8 Electrical, Instrumentation & Control

The selected alternative includes the construction of new auxiliary structures as described above. A new electrical service from Tillamook Public Utility District will be installed; it is assumed that a 120/240-Volt, 200-Amp service would be adequate for the new structures. The structures will house new electrical distribution equipment, a new automatic transfer switch, booster pump variable frequency drives, telemetry control panel, water quality instruments, and relocated radio equipment.

A description of some of the key EI&C elements is provided below.

5.8.1 Standby Power Generator

A fix-mounted diesel generator will be provided to provide standby power to the facility. The standby power generator would include a sub-base diesel fuel tank with the capacity to operate the generator at full load for 96 hours. The installed generator should also include an automatic transfer switch (ATS). Upon loss of utility power, the ATS would call for the generator to start and transfer the station power to the generator. The proposed generator would be installed outdoors with a weather-proof enclosure. Because there are residences nearby, the generator enclosure should also include Level 1 sound attenuation with a generator silencer to provide good sound reduction properties (e.g., super-critical silencer).

Preliminary calculations indicate that a generator set with a 30 kilowatt (kW) rating is required to provide standby power to the following loads:

Table 3. Load Rating Summary

Description	Rating
UPS Load – Classical Music Radio Gear	1 KVA
UPS Load – Emergency Services Radio Gear	1 KVA
UPS Load – Telemetry	0.5 KVA
General – House Loads (Lights, Receptacles, Fans)	1 KW
Booster Pump 1 (Variable Frequency Drive)	5 hp
Booster Pump 2 (Variable Frequency Drive)	5 hp

KVA = kilovolt-ampere

5.8.2 ShakeAlert Integration

A ShakeAlert seismic early warning system will be used to preemptively close valves or restrict flow to help protect water storage. The ShakeAlert system has been developed through a coordinated effort between the USGS, California Office of Emergency Services, Caltech, University of Washington, University of California at Berkeley, and University of Oregon. It depends on an integrated network of USGS seismometers distributed throughout the west coast. The system is tied into the USGS network and uses information provided to “identify and characterize an earthquake a few seconds after it begins, calculates the likely intensity of ground shaking that will result, and deliver alerts to people and infrastructure in harm’s way.”

There are a limited number of private partners providing the ShakeAlert technology; RH2 Engineering and Varius, Inc. These systems include a small controller that listens for ShakeAlert messages through an internet connection. The controller could potentially provide up to 3 to 4 minutes warning of a seismic event. It includes outputs for interconnection to a SCADA system. Varius, Inc. indicated that a normal range of installed costs for their system is $25,000 to $35,000 with an approximately $150 monthly service cost. Varius will provide the ShakeAlert system to be installed at the facility. Varius will provide commissioning service as needed to place the system into operation. The system will provide an early warning signal to the SCADA system that may be used to automatically close an isolation valve.

5.8.3 Instrumentation

Installation will include new instrumentation and controls as described below:

- Treated water analytical equipment (pH/temperature and chlorine)
- Treated water turbidity analyzer
- Reservoir level transmitters
- Reservoir hatch door position switches.
- Flowmeter(s)
- Reservoir isolation valve(s).

5.9 Operations and Maintenance

5.9.1 Tank Structures

Prestressed concrete tanks require little maintenance once they are constructed. Regular monitoring inspections of the tank interior and exterior should be performed over the life of the tank to monitor for issues and make repairs as required. Shrinkage cracking is more likely to occur early in the life of the tank, so inspections to monitor for such should be performed more frequently early in the life of the tank and larger cracks sealed as required. AWWA D110 recommends an inspection after the first year that the tank is in service, with routine inspections performed every 5 to 10 years after, with the inspection schedule modified as needed based on findings.

If a coating is used to meet NSF 61 requirements, the coating will need to be regularly inspected and repaired if damage is found. However, most coatings for concrete tanks are durable and rarely require significant maintenance. Minor coating repairs can be done as part of the routine inspection.

Connections to the tanks should be regularly inspected and repaired as necessary. This will be addressed in more detail in subsequent sections.

5.9.2 Water System

In general, upgrading the water system with new pipe, fittings, and valves will require standard operation and maintenance such as valve exercising and pipe flushing. In addition, the new tank configuration and larger reservoirs will require appropriate water quality monitoring.

5.9.3 Electrical, Instrumentation & Control

5.9.3.1 Standby Generator

Standby generators require preventative and predictive maintenance and regular testing to provide reliable operation. The operation and maintenance cost for standby generators varies depending on the size of the generator, usage, and type of fuel used.

The facility standby power system will require maintenance activities that should be performed weekly, monthly, semi-annually, and annually to provide safe and efficient operation during an electrical outage. These activities should be as required by the generator manufacturer and recorded in a generator logbook. A representative listing of these activities is listed below.

Weekly Maintenance Activities:

- Run the generator for several minutes to achieve operational temperature
- Inspect engine, radiator, and generator for debris and loose fittings and leaks
- Check fuel tank levels
- Perform visible emissions observation
- Verify the generator circuit breaker is closed and the generator is in auto mode

Monthly Maintenance Activities:

- Check oil levels
- Check engine coolant level

Semi-Annual Maintenance Activities:

- Sample oil
- Check coolant lines and connections
- Clean battery connections and apply corrosion inhibitor
- Check battery electrolyte level and specific gravity
- Clean crankcase breather and inspect system components
- Inspect exhaust system, muffler, and exhaust pipes
- Clean out electrical boxes and inspect wiring
- Inspect air intake system and replace filter if needed

Annual Generator Maintenance Activities:

- Run the system for 1 hour under full building load initiated by the ATS
- Perform load bank test
- Inspect fuel system for leaks, corrosion, damage
- Replace fuel filter(s)
- Grease bearing if needed
- Replace oil and oil filter and oil hoses if needed
- Check coolant condition and replace if needed
- Inspect and adjust belts, verify engine block heater is operating, check radiator cap, clean exterior of radiator, inspect fan shroud
- Inspect and clean engine speed timing sensor
- Inspect and clean ATS contacts and wiring

The condition of the diesel fuel should also be monitored for contamination. It may be necessary to replace or treat the fuel if not consumed before becoming degraded.

5.9.3.2 Instrumentation

The pH analyzers and turbidity analyzer will require regular cleaning and calibration to produce accurate recording of water quality parameters.

6 Cost Evaluation

6.1 Cost Estimate Methodology

For the selected alternative, a Class 4 budgetary opinion of probable construction cost (OPCC) was developed per the American Association of Cost Engineering (AACE) International Recommended Practice No. 18R-97 Cost Estimate Classification System. This detailed construction cost estimate is provided in Appendix C.

6.2 Opinion of Probable Construction Cost

Although additional geotechnical data is needed to confirm, ground improvements can be costly and are conservatively included in this OPCC (Table 4). Further geotechnical investigation could lead to a reduction or elimination of ground improvements. For this OPCC, a volume of deep soil mixing was calculated based on an assumed 50-foot depth of improvement at a 50 percent replacement ratio. Deep soil mixing was selected as it generally provides the best balance of cost and performance, and is commonly used for this type of project. Furthermore, it is assumed that 33 percent of the replacement volume will require disposal as spoils. Once additional geotechnical investigation is performed, this OPCC should be revisited and the quantity of ground improvements refined as needed.

Table 4. OPCC Summary

Alternative 3 - Complete Tank Replacement			Section Totals
Demolition and Temporary Facilities			$50,000
Site Work			$301,000
Yard Pipelines, Vaults, Connections			$751,700
Tanks, Foundations, Ground Improvements			$7,120,000
Testing and Commissioning			$40,000
Buildings & Mechanical			$181,100
Electrical, Instrumentation, Generator, Monitoring, Control			$269,500
		Subtotal	**$8,713,300**
General Conditions, Mobilization, Insurance & Bonds		10%	$871,400
Miscellaneous Items and Contingencies		15%	$1,307,000
Alternative 3 Construction Cost			**$10,892,000**
AACE Estimate Range (Class 4)	**Low**	**15%**	**$9,259,000**
	High	**25%**	**$13,615,000**

7 Conclusion

The City's Water Storage Improvements project is an important initiative that addresses seismic-related vulnerabilities in its potable water storage system. Having a reliable and seismically resilient water storage system means that the City can have greater confidence that water will be available for the community to fight fires and provide drinking water while working to recover from a seismic-related natural disaster. Through discussions with the City about project needs and requirements, gathering and analysis of data, comparing and contrasting alternatives, and formulation of preliminary design plans, a clearer picture of the project scope and budget has been developed. The project is ready to advance into the next design stages.

There are some issues to consider as the project progresses into the next stages of design:

- A fully developed geotechnical investigation and study are important to define the geotechnical hazards such as liquefaction. Once completed, the appropriate mitigation measures can be incorporated into the design and the OPCC adjusted accordingly.
- Permitting and land use planning could affect the design. Additionally, community involvement may influence the design and direction of the project.

8 References

Oregon Seismic Safety Policy Advisory Commission (OSSPAC)

2013 The Oregon Resilience Plan Reducing Risk and Improving Recovery for the Next Cascadia Earthquake and Tsunami Report to the 77th Legislature Assembly, Salem, Oregon. February 2013.

Shannon and Wilson

2021 *Geotechnical Feasibility Assessment: Feasibility Study for Water Storage Improvements* (Report 106549-001). April 2021.

Appendix A.
Shannon & Wilson Geotechnical Report

SUBMITTED TO:
HDR, Inc.
1050 SW 6th Ave, Suite 1800
Portland, Oregon, 97204

BY:
Shannon & Wilson, Inc.
3990 Collins Way, Suite 100
Lake Oswego, Oregon,
97035

503-210-4750
www.shannonwilson.com

GEOTECHNICAL FEASIBILITY ASSESSMENT

Feasibility Study for Water Storage Improvements

MANZANITA, OREGON

April 2021

Shannon & Wilson No: 106549-001

SHANNON & WILSON

PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Submitted To: HDR, Inc.
1050 SW 6th Ave, Suite 1800
Portland, Oregon, 97204
Attn: Dan Johnston, PE

Subject: GEOTECHNICAL FEASIBILITY ASSESSMENT, FEASIBILITY STUDY FOR WATER STORAGE IMPROVEMENTS , MANZANITA, OREGON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to HDR, Inc. Our scope of services was specified in our agreement with HDR, Inc. dated March 25, 2021. This report presents our geotechnical feasibility assessment and was prepared by the undersigned.

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.

REGISTERED PROFESSIONAL ENGINEER
77330PE
OREGON
Jan. 10, 2006
ELLIOTT CARL MECHAM
EXPIRES: 12/31/2022

Elliott Mecham, PE
Senior Associate

Cody Sorensen, CEG
Associate

DSJ:CKS:ECM/las:mmb

CONTENTS

CONTENTS

Exhibits

Figures

Appendices

1 INTRODUCTION

The City of Manzanita (City) has requested a geotechnical hazard evaluation for their reservoirs and a feasibility analysis of potential improvements. The alternatives under consideration include doing nothing, retrofitting the existing reservoirs, or replacing the reservoirs with two new seismically resilient reservoirs on existing project sites. The project area which includes three existing reservoirs in northern Manzanita, as shown in Figure 1, Vicinity Map. The project area consists of two City-owned properties opposite Epoh Avenue, and located just northwest of the intersection of Epoh Avenue and Oak Street, and approximately 0.18 miles west of Highway 101. The area included in the feasibility study is presented on Figure 2, Site Map. Shannon & Wilson's geotechnical evaluation included the following tasks:

- Gather existing geologic information from readily available publicly available sources;
- Perform a site reconnaissance of the project area;
- Summarize the geologic and seismic setting within the project area;
- Discuss the seismic hazard within the project area; and
- Discuss methods that could be used to mitigate the seismic hazard.

The existing information sources that were used as a basis for this evaluation are as follows:

- Oregon Department of Geologic and Mineral Industries (DOGAMI) Interactive GIS maps, and
- Shannon & Wilson, 2013, US 101 Neahkahnie Creek Culvert Replacement Project.

2 BACKGROUND.

Through the Oregon Health Authority's Sustainable Infrastructure Planning Project, the City obtained a study grant to evaluate the seismic resilience of their water system. Assessing resilience is a high priority for the State of Oregon because of the risk of a Cascadia Subduction Zone (CSZ) event. For cities on the Oregon Coast, this is particularly important due to the heightened risk based on its geographic location. Because of the proximity of the coast to the subduction zone, significant ground accelerations, landslides, liquefaction, and tsunami are all hazards posed by a CSZ event.

The study grant received by the City was a direct response to the 2013 Oregon Resilience Plan. This plan outlined not only the likely impacts of a magnitude 9.0 CSZ event, but timeframes in which it would take to restore functions around the state, and ways in which practices and policies could be changed over the next 50 years to increase resilience. This document highlights vulnerabilities of buildings, businesses, transportation, energy, communications, and water/wastewater systems. Also outlined was how capital investment and new incentives could lessen the impact a CSZ event would have on Oregon's economy and communities.

The City currently has three reservoirs located on the north side of town, near its highest point. The reservoirs are of varying size and age. Two of the reservoirs, Reservoir No.1 and No.2, sit adjacent to one another and have a combined total capacity of 750,000 gallons. Reservoir No. 1, built in 1979 of welded steel, has a capacity of 500,000 gallons. The tank of Reservoir No. 1 was recoated in 2003. Reservoir No. 2, constructed with concrete in 1960, is partially buried, and has a capacity of 250,000 gallons. The third reservoir, Reservoir No. 3, was constructed with glass-fused bolted steel in 1997, and has a capacity of 1.6 million gallons. Reservoir No. 3 is located on a different plot of land from Reservoirs No. 1 and No. 2 and is across Oak Street from the City's Public Works office. Since its construction in 1997, Reservoir No. 3 has experienced settlement issues of up to 8 inches and has required repairs. In response to the problems associated with this tank, the City is evaluating the feasibility of constructing two new 1-million-gallon reservoirs on one or both of the City owned properties shown on Figure 2, Site Map.

3 SITE RECONNAISSANCE

Shannon & Wilson conducted a site reconnaissance with HDR and the City on April 2, 2021 in accordance with our scope of work. A Shannon & Wilson geologist and senior geotechnical engineer performed the site reconnaissance to observe and evaluate current site conditions around the existing three reservoirs.

3.1 Reservoir No. 3

The site reconnaissance began at Reservoir No. 3, on the north side of the intersection of Epoh Avenue and Oak St. According to a City representative onsite during the reconnaissance, the southwest portion of the reservoir is underlain by native material, and the northeast portion of the reservoir is underlain by undocumented reworked native material and sand fill. In conversations with the City, minimal compaction effort was applied to the fill beneath Reservoir No. 3.

Exhibit 3-1: Reservoir No. 3, view north.

According to the City representative, the area east of the tank consists of approximately 20 feet of uncompacted, undocumented fill. The City representative indicated the undocumented fill was landslide debris imported to the site from the Neahkahnie Landslide prior to the construction of Reservoir No. 3. Available aerial photos showing site disturbance, which was likely caused by fill placement are included in Appendix A. On the north side of the reservoir is an access road which also is composed of this undocumented fill. Settlement of the undocumented fill material is exposed in some areas around the reservoir, most notably at the eastern extent of the lot, where in 2016-2017 an area subsided approximately 12 inches over the course of one year (See Exhibit 3-3).

Exhibit 3-2: Gravel lot east of Reservoir No. 3. Lot is built on approximately 20-feet of uncompacted, undocumented fill.

Exhibit 3-3: Subsidence area on the east side of the City property.

During the reconnaissance of Reservoir No. 3, the City identified numerous areas in which leaking of the tank had occurred. The most notable of these leaks is on the east side of the tank, marked by streaks in the coating of the reservoir as shown in Exhibit 3-4.

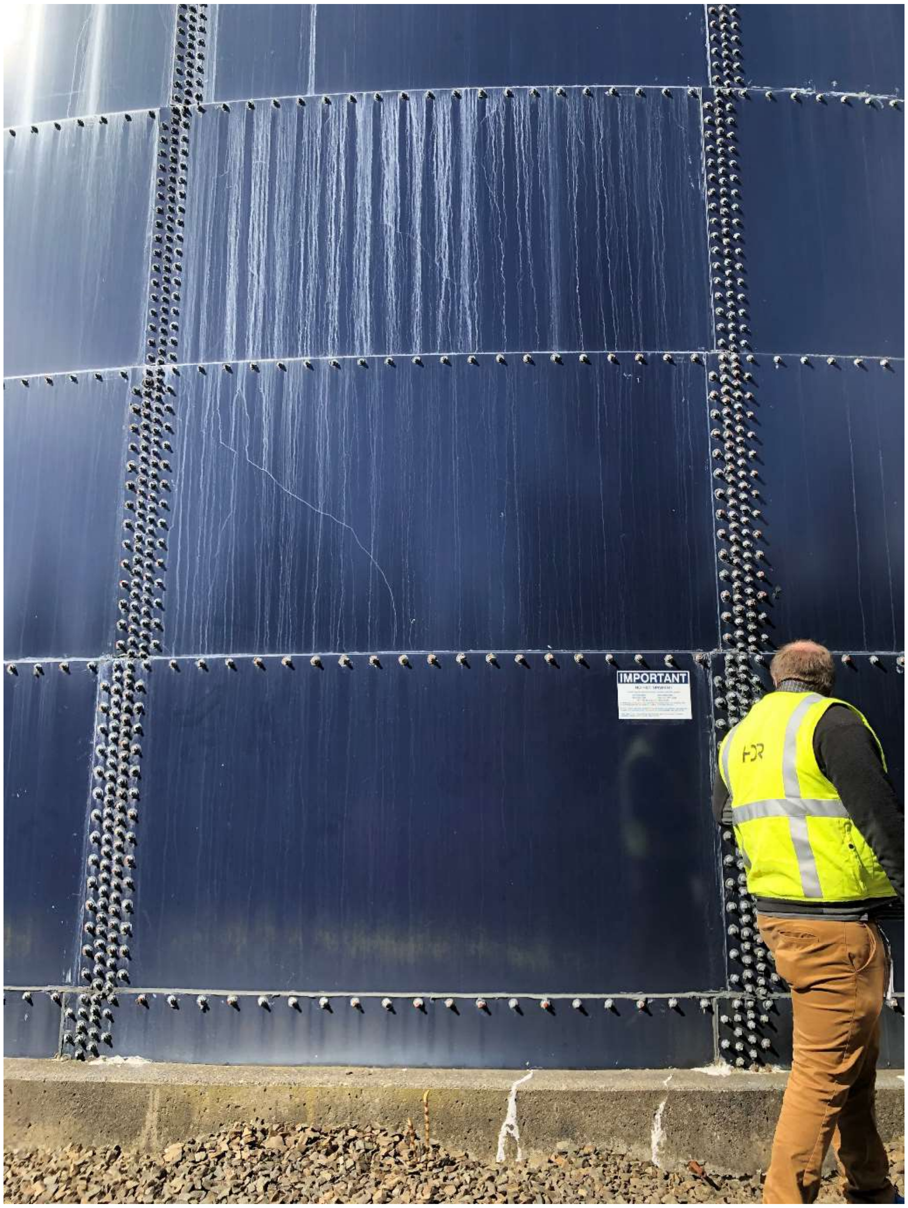


Exhibit 3-4: Streaking along the side of Reservoir No. 3 marking the location of leaks in the tank.

3.2 Reservoir No. 1 and Reservoir No. 2

The property containing Reservoir No. 1 and Reservoir No. 2 is located just west across Epoh Avenue from Reservoir No. 3. The property which Reservoirs No. 1 and No. 2 is on appears to be approximately 20 feet higher in elevation than the property on which Reservoir No. 3 is located. The property encompassing Reservoirs No. 1 and No. 2 also contains a pumphouse. Reservoir No. 2 is partially-buried, with about 8-10 feet of the tank visible above ground surface. According to the City, there is another 10-12 feet of tank below ground surface. Reservoir No. 1 is entirely above-ground and has a capacity of 500,000 gallons.

Exhibit 3-5: Reservoir No. 2, View east at the west side of the tank.

Exhibit 3-6: Reservoir No. 1, View west at the east side of the tank.

3.3 Epoh Avenue

In 2017, the City observed cracking and deformation of Epoh Avenue on the southwest side of Reservoir No. 3. A site reconnaissance was performed by OTAK and their geotechnical subconsultant Geotech Solutions, Inc., were they observed cracks approximately 130 feet long which were approximately 3/4-inch wide. Additionally, the guardrail along the north side of Epoh Avenue was observed to be tilting outward. Based on City documentation, the cracks grew throughout the year, most notably after winter rainfall events. Following recommendations by OTAK and their geotechnical subconsultant Geotech Solutions, Inc. the road was repaved and today there are no further signs of slope instability. The movement of the slope was attributed to creep within sand fill placed in the roadway by Geotech Solutions, Inc.

Exhibit 3-7: 2017 Surface cracking and deformation along Epoh Avenue.

Exhibit 3-8: Surface restoration of Epoh Avenue.

3.4 North Facing Slope

On the north side of Reservoir No. 3 is a large north facing slope, which according to DOGAMI, has a high landslide susceptibility. During our site reconnaissance, we walked the slope to observe any surficial indications of slope instability. Surficial features and vegetation generally indicated stable conditions on the north slope.

Exhibit 3-9: Slope just north of Reservoir No.3.

4 REGIONAL GEOLOGY

The project site lies on the west side of the Coast Range Physiographic Province, on the west side of a northeast-plunging anticline (Wells and others, 1994). According to mapping by Wells and others (1994), the geology of the project area generally includes Quaternary (about 2.6 million years to present) fine-grained eolian (wind-blown) sediments, river and coastal terraces, and landslide deposits. A generalized Geologic Map is presented in Figure 3. It is based on the Oregon Department of Geology and Mineral Industries (DOGAMI) Oregon Geologic Data Compilation, version 6.

Underlying much of these Quaternary sediments is the Alsea formation, which Wells and others (1994) describe as thick bedded to massive tuffaceaous siltstone containing abundant white tuff beds, calcareous concretions, and sparse sandstone beds. The Alsea formation is underlain by the older Nestucca Formation, which Wells and others (1994) describe as thin bedded dark gray tuffaceous mudstone with fine- to coarse-grained sandstone interbeds. Both units were deposited in low-energy marine environments, with local volcanoes periodically contributing fine ash to the sediment load.

Over time, the Alsea and Nestucca Formations were uplifted by tectonic forces. Within the last 10,000 years, beach, dune, fluvial, and estuarine deposits have accumulated on their weathered surfaces. In more recent times, humans have locally placed fill during land or roadway development.

Previous geotechnical borings performed by Shannon & Wilson which encountered sedimentary rock consistent with the Alsea formation and located approximately 0.4 miles from the site are included in Attachment B.

5 SEISMIC SETTING

Earthquakes in the Pacific Northwest occur largely as a result of the region's close proximity to an active convergent plate boundary. At this boundary, dense oceanic crust is subducting beneath less dense continental crust. At this subduction plate boundary, known as the Cascadia Subduction Zone (CSZ), the Explorer, Juan de Fuca, and Gorda Oceanic Plates are subducting beneath the overriding, westward-moving North American Plate. Oblique convergence of these plates not only results in east-west compressive strain, but also in dextral (right-lateral) shear, clockwise rotation, and north-south compression of accreted crustal blocks that form the leading edge of the North American Plate (Wells and others, 1998). The CSZ extends about 750 miles from northern California to southern British Columbia and lies approximately 74 miles west of the project site. Within the present understanding of the regional tectonic framework and historical seismicity, three broad seismogenic sources have been identified:

- A mega-thrust source at an interface between the North American and Juan de Fuca plates in the CSZ;
- A deep subcrustal zone (intraslab) in the subducted Juan de Fuca Plate in the CSZ; and
- A shallow crustal zone within the North American Plate.

All three sources potentially produce earthquakes that impact the ground motion hazards at the project site. Offshore, elastic release of strain accumulated in the locked plate interface of the CSZ produces great megathrust earthquakes (greater than moment magnitude [Mw]

8.0) about every 500 years (Atwater and Hemphill-Haley, 1997; Clague, 1997; Goldfinger and others, 2003 and 2012); the most recent rupture occurred in A.D. 1700 (Satake and others, 1996; Atwater and Hemphill-Haley, 1997; Clague, 1997; Yamaguchi and others, 1997; Goldfinger and others, 2003 and 2012). Onshore, migration and rotation of tectonic blocks produce deformation along shallow faults within the upper part of the crust. At depth, rupture within the subducting slab, referred to as the intraslab, has produced some of the largest recorded earthquakes (Mw 6.5 to 7) to strike the Pacific Northwest in the northern California Coast region and Western Washington. However, over the past century, intraslab earthquakes have been markedly infrequent in Oregon. The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

5.1.1 Cascadia Subduction Zone: Mega-Thrust Interface Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project area and potential rupture surface range from about 18 to 75 horizontal miles. Focal depths for mega-thrust earthquakes are commonly on the order of about 15 to 25 miles. Rupture of the interface could result in earthquakes with moment magnitudes on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (e.g., Atwater, 1987; Atwater and Hemphill-Haley, 1997) as a result of tectonic movement associated with mega-thrust earthquakes on the CSZ. It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996), the most recent interplate event on the CSZ was an Mw 9 event on January 26, 1700.

5.1.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates as a result of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. The nearest seismogenic intraslab portion of the Juan de Fuca plate is approximately 30 to 60 miles below the Portland area. Ludwin and others (1991) estimate that the maximum Mw from this source zone would be about 7.5. Ground shaking produced by intraplate earthquakes would be less intense and less prolonged in the Portland area than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 Mw 6.7 Olympia earthquake, the 1965

Mw 6.7 earthquake between Tacoma and Seattle, and the 2001 Mw 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

5.1.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within Pacific Northwest, typically within the upper 4 to 19 miles of the continental crust. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake at approximate Mw 6.5 to 7.0. Other examples include the 1993 Mw 5.6 Scotts Mill earthquake and the 1993 Mw 6.0 Klamath Falls earthquake.

5.1.4 Local Quaternary Faults and Folds

Quaternary crustal faults and folds throughout Oregon and Washington have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.58 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database (USGS, 2021), there are no Class A features within approximately 10 miles of the project site. The CSZ itself is approximately 74 miles west of the project site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year and the most recent deformation occurring about 300 years ago (Personius and Nelson, 2006).

6 POTENTIAL GEOLOGIC/GEOTECHNICAL HAZARDS

The potential geologic hazards within the study area are expected to be related to seismicity and are primarily ground shaking, landslides, liquefaction, and liquefaction-related phenomena such as settlement, lateral spreading, and post-seismic soil strength reduction. The risk of other seismic hazards, such as fault rupture and tsunami, is low within the study area. The following sections include a discussion of the relevant seismic hazards present within the study area.

6.1 Landslides

According to the Department of Geology and Mineral Industries (DOGAMI), the existing reservoirs are located within zones of moderate to high landslide hazard. According to the Oregon Statewide Geohazards Viewer (HazVu), much of the area to the east and north of the existing tanks has a very high landslide hazard indicating there are existing landslides. As discussed in the background section, ground deformation has been observed in Epoh Avenue which was attributed creep in fill materials placed near the crest of the slope. The Statewide Landslide Information Database for Oregon (SLIDO) shows no mapped landslides in the immediate vicinity of the three reservoirs.

Based on the proximity of Reservoir No. 3 to the north facing slope, the history of ground deformation in fill materials at Epoh Avenue, and the large areas of undocumented fill on the property containing Reservoir No. 3, in our opinion the risk of a seismic slope hazard is high.

Most of the property containing Reservoirs No. 1 and No. 2 is mapped as a moderate landslide hazard by DOGAMI (as is much of the town of Manzanita), as shown on Figure 4, Landslide Susceptibility. However, the site is relatively flat, and the existing reservoirs are located over 100 feet away from the nearest significant slope to the northeast. We did not observe any evidence of slope instability on the property containing Reservoirs No. 1 and No. 2.

The relative landslide hazard risk was developed by DOGAMI by creating a generalized geology-landslide intersect map and a percent slope map. Spatial statistics were then used to determine the mean and standard deviation of slope angles within landslides per geologic unit. Thirty percent of the area within the statewide hazard map consists of High or Very High hazard slopes and 80 percent of the landslides are located within this area. Limitations of the input and modeling mean that the map should only be used for general planning purposes, and the map cannot be used as a substitute for geotechnical explorations and detailed site-specific analyses. During design, geotechnical explorations and site-specific analysis should be used to further assess the risk. If a detailed analysis does show a slope stability risk at Reservoirs No. 1 and No. 2, then the slope stability issue will need to be addressed during design. Design solutions could include replacing soft or weak materials with a rock buttress, ground improvements to increase soil strength, or installation of structural elements such as a soldier pile and tie-back or secant wall.

6.2 Liquefaction and Lateral Spread

Soil liquefaction occurs in susceptible subsurface soils below the groundwater level. It is a phenomenon in which excess pore water pressure of loose to medium dense, saturated, granular soils increases during ground shaking to a level near the initial effective stress. The increased excess pore pressure results in a reduction of soil shear strength. Given that sands were observed at the ground surface and likely underlie a large portion of the project area, liquefaction is a potential hazard within the project area. A map of liquefaction susceptibility prepared by DOGAMI, and included as Figure 5, indicates the project area has a moderate to high risk of liquefaction. The effects of liquefaction typically include lateral spreading, slope instability, ground settlement, and strength reductions, such as lower allowable soil bearing.

We note that the DOGAMI hazard assessment is based solely on soil type and does not consider ground water presence or the absence of groundwater. If groundwater is not present at the site, the DOGAMI hazard map is likely overestimating the liquefaction potential. The relative density also impacts the liquefaction potential of the sands. Obtaining site specific borings or Cone Penetrometer Tests (CPTs) to assess the density of the sand was outside the scope of this study, but we recommend that they be performed during design to further assess the extent of the liquefaction hazard.

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 towards an embankment face, or banks of streams, rivers, and other bodies of water. The displacements are cumulative and permanent in nature. If liquefaction occurs there is risk of post seismic slope instability and potential lateral displacement towards the existing slope to the northeast.

6.2.1 Liquefaction Induced Post-Seismic Settlement

Settlement will likely occur in cohesionless soil below the groundwater table that undergo liquefaction and pore pressure development during ground shaking. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change, as the excess pore pressure dissipates after ground shaking. Seismic ground settlement does not typically occur uniformly over an area, and differential settlement may impact existing or proposed structures and infrastructure supported by liquefied soil and/or within the liquified zones. Differential settlement is often estimated to range between 50 and 80 percent of the total settlement. Consequences of seismic-induced settlement would be subsequent settlement of shallow foundations overlying the liquefied soil.

6.3 Tsunami

The reservoir sites are located near the high point in the town at an elevation of approximately 225 feet. The sites are outside of the statuary inundation line and at an elevation over 175 feet higher than portions of the town within the inundation line. The tsunami inundation maps do not directly address other Tsunami related risks such as tsunami related erosion, or saturation of the existing terrain and slopes that could lead to slope instability. However, the tank foundations are over approximately 1,500 feet inland from the edge of the tsunami inundation zone. The available DOGAMI liquefaction potential and landslide maps have been presented in earlier sections of this report.

6.4 Uncompacted, Undocumented Fill

As mentioned above, according to the City, there is approximately 20 feet of uncompacted, undocumented fill on the property Reservoir No. 3 was built. While much of the material came from the Neahkahnie Landslide, its overall composition and effort of compaction of the material during placement is unknown. Therefore, due to the uncertainty of the fill, we anticipate that both static and dynamic differential settlement could occur within this lot. This could be due to material differences within the fill, or potential voids as it is uncompacted.

7 FEASIBILITY OPTIONS AND CONCEPTUAL DESIGN AND MITIGATION CONSIDERATIONS

7.1 Reservoir No. 1 and No. 2 Site Feasibility

Based on a review of publicly available information and geologic mapping it is our preliminary opinion that it is geotechnically feasible to construct new seismically resilient reservoirs at the Reservoir No. 1 and No. 2 Site. This should be confirmed with site specific geotechnical explorations such as geotechnical borings or Cone Penetrometer Tests (CPTs) during a pre-design phase in preparation for final design phase.

The primary seismic hazard risk identified in the DOGAMI mapping and our site reconnaissance at Reservoir Sites No. 1 and No. 2 is the potential for liquefaction, and associated permanent ground deformation from settlement and potential for liquefied soils to move towards the face of the slope to the north causing post-seismic slope stability issues. As discussed in the liquefaction hazard section of this report, the potential for liquefaction is dependent on the presence of ground water and the density of the soil. The DOGAMI hazard maps are developed based solely on the age and soil type of the mapped surface geology. The DOGAMI hazard maps indicate that the liquefaction risk is high. It is possible that the publicly available mapping over-estimates the liquefaction hazard. However, for planning purposes we suggest including provisions to mitigate the liquefaction risk until subsurface explorations can be performed to further characterize the risk. Shannon & Wilson can provide these explorations upon request.

7.2 Reservoir No. 1 and No. 2 New Construction Options

A potential strategy for mitigating liquefaction could include ground improvement to reduce or eliminate the hazard. For new construction, a number of different ground improvement technologies are available. The type of ground improvement most suitable for the site depends on the depth of the liquefiable layer and fines content of the liquefiable material.

Rammed aggregate piers can be constructed using several different construction methods. One such method consists of displacing existing soils and compacting aggregate piers using proprietary systems in a grid pattern beneath the new reservoir. Rammed aggregate piers may be a feasible alternative if liquefiable soils are relatively shallow (a maximum of 20 to 30 feet deep). Rammed aggregate piers may be most effective when the fines content of the displaced soils is relatively low.

Liquefaction mitigation can also be achieved through mass-mixing, which is performed by blending soils with a cement binder material using a horizontal-axis rotating drum mounted on the boom of an excavator. The mixing drum is advanced into the native soils while pumping a cement slurry through the mixing drum and mixing the soil to the target depth. The mixing is performed in continuous rectangular cells, which contain liquefiable soils and mitigate the liquefaction hazard.

Where liquefiable soils extend to depths greater than 20 to 30 feet below ground surface, ground improvement methods such as stone columns or deep soil mixing may be more feasible than rammed aggregate piers of mass soil mixing. Stone columns may be constructed using a vibratory tool that is installed below the liquefiable layer to densify the surrounding layer while installing the stone columns in a grid pattern. The presence of fine-grained material in the liquefiable layer can limit the amount of densification that occurs in the native soil and thus limit the effectiveness of the liquefaction mitigation technique.

An alternative to stone columns when fine grained soils are present is deep soil mixing. Deep soil mixing requires a temporary grout plant to be constructed on-site, to allow for the mixing of native soils with cement grout. To construct the deep soil mix columns, a revolving hollow shaft with mixing paddles is advanced below the liquefiable layer. As the mixing tool is advanced, cement grout is pumped through the hollow stem and mixing with the native soil creating a reinforced soil column. Cells of deep soil mix columns can be installed beneath the reservoir to mitigate liquefaction and liquefaction related slope instability (if present).

Exhibit 7-1: Photo of deep soil mixing paddles at OR 38 Scottsburg Bridge.

Exhibit 7-2: Photo of deep soil mixing plant at OR 38 Scottsburg Bridge.

Mitigation methods for new reservoirs may also include designing the structure with stiffened shallow continuous mat-type foundation systems or deep foundations extending to stable soils if no liquefaction related slope hazard is present. If the liquefiable soil is shallow, it may also be possible to over-excavate the liquefiable soils and replace them with crushed rock. The mitigation approach selected should be based on site specific geotechnical information such as borings and CPTs.

7.3 Reservoir No. 1 Retrofit Options

We understand the steel reservoir, Reservoir No. 1, is supported by a concrete ring foundation. Because older reservoirs are often designed for lower seismic loading than required by current seismic design standards, structural and geotechnical analysis performed during design may reveal that seismic loads on the ring foundation exceed the allowable bearing capacity. This type of deficiency can be addressed by enlarging the foundation and doweling a new foundation into the existing foundation or underpinning the existing foundation with intermediate foundations.

If potentially liquefiable soils are present beneath the tank and the predicted liquefaction induced settlement exceeds the allowable structural tolerance inside the tank, retrofitting only the tank may be problematic. On previous evaluations, Shannon & Wilson has evaluated the feasibility of coring through structural slabs to mitigate liquefaction beneath the structure by performing a type of ground improvement referred to as jet-grouting. While jet grouting can be performed from inside of structures, the process can be technically challenging, and the cost of the retrofit often exceeds the cost of demolishing and building a new seismically resilient structure.

7.4 Reservoir No. 1 and No. 2 Seismic Slope Stability Options

If a lateral spread or seismic slope instability hazard is identified based on-site specific analysis, remediation will be required to achieve a seismically resilient reservoir system. Design solutions could include replacing soft or weak materials with a rock buttress, ground improvements to increase soil strength, or installation of structural elements such as a soldier pile and tie-back or secant wall on the northeast portion of the property closest to the slope. On previous projects where a liquefaction related slope instability issue were identified, ground improvement was determined to be the most cost-effective solution. However, the costs for each system will be a function of the amount soil that needs to be improved, the geometry of the slope and availability of specialty contractors.

7.5 Reservoir No. 3 Feasibility

Several geotechnical and geologic hazards were identified at Reservoir No. 3 which significantly increase the seismic hazard risk and could negatively affect the long-term performance of the reservoir. As mentioned above, the existing reservoir is located in an area of a high slope hazard and has experienced significant differential settlement. Outside of the reservoir footprint, undocumented fill to depths of 20 feet has been placed. Based on our reconnaissance, large boulders were used to construct the fill. If not well mixed with smaller material these boulders could create voids that manifest as surface settlement during static or seismic loading. Additionally, buried wood was observed in the fill, which could also result in surface settlement as the wood decomposes with time.

Exhibit 7-3: Photo of boulders from mass fill at Reservoir No. 3 site.

Exhibit 7-4: Photo of organics from mass fill at Reservoir No. 3 site.

In our opinion the risk of long term seismic and non-seismic geohazards negatively impacting the performance of a new reservoir at Reservoir No. 3 is high and the cost associated with partial mitigation of the hazards would be high. We recommend against pursuing significant site preparation and construction of a new reservoir at this location if other lower risk sites are available.

8 LIMITATIONS

Our interpretations, conclusions and geotechnical considerations are based on a desktop study including review of publicly available information prepared by others, and a single site visit. No explorations were performed to evaluate geotechnical site conditions and make interpretations. Should proposed development of sites within the study area occur, we recommend that appropriate explorations and site characterization testing and evaluation be done, a detailed site-specific geotechnical study be performed, and geotechnical firms with experience in both static and seismic conditions perform the work.

Within the limitations of scope, schedule, and budget, the conclusions presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. Shannon & Wilson makes no other warranty, either express or implied. These conclusions were based on Shannon & Wilson's understanding of the project as described in this report and the site conditions as observed at the time of our field reconnaissance.

This report was prepared for the exclusive use of HDR Inc. and City of Manzanita, Oregon. The scope of Shannon & Wilson's present work did not include environmental assessments or evaluations regarding the presence or absence of hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this sites, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

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

9 REFERENCES

Atwater, B.F., 1987, Evidence for great Holocene earthquakes along the outer coast of Washington State: Science, v. 236, p. 942-944.

Atwater, B.F., and Hemphill-Haley, E., 1997, Recurrence intervals for great earthquakes of the past 3500 years at Northeastern Willapa Bay, Washington: U.S. Geological Survey Professional Paper 1576.

Clague, J.J., 1997, Evidence for Large Earthquakes at the Cascadia Subduction Zone: Reviews of Geophysics, v. 35, no. 4, p. 439-460.

Goldfinger, C., Nelson, C.H., and Johnson, J.E., 2003, Deep-Water Turbidites as Holocene Earthquake Proxies: The Cascadia Subduction Zone and Northern San Andreas Fault Systems: Annali Geofisica, v. 46, p. 1169-1194.

Goldfinger, C., Nelson, C.H., Morey, A., Johnson, J.E., Gutierrez-Pastor, J., Eriksson, A.T., Karabanov, E., Patton, J., Gracia, E., Enkin, R., Dallimore, A., Dunhill, G., and Vallier, T., 2012, Turbidite Event History: Methods and Implications for Holocene Paleoseismicity of the Cascadia Subduction Zone: USGS Professional Paper 1661-F, 184 p, 64 Figures.

Ludwin, R.S., Weaver, C.S., and Crosson, R.S., 1991, Seismicity of Washington and Oregon in Slemmons, D.B., E.R. Engdahl, M.D. Zoback, and D.D. Blackwell (eds.), Neotectonics of North America, p. 77-98.

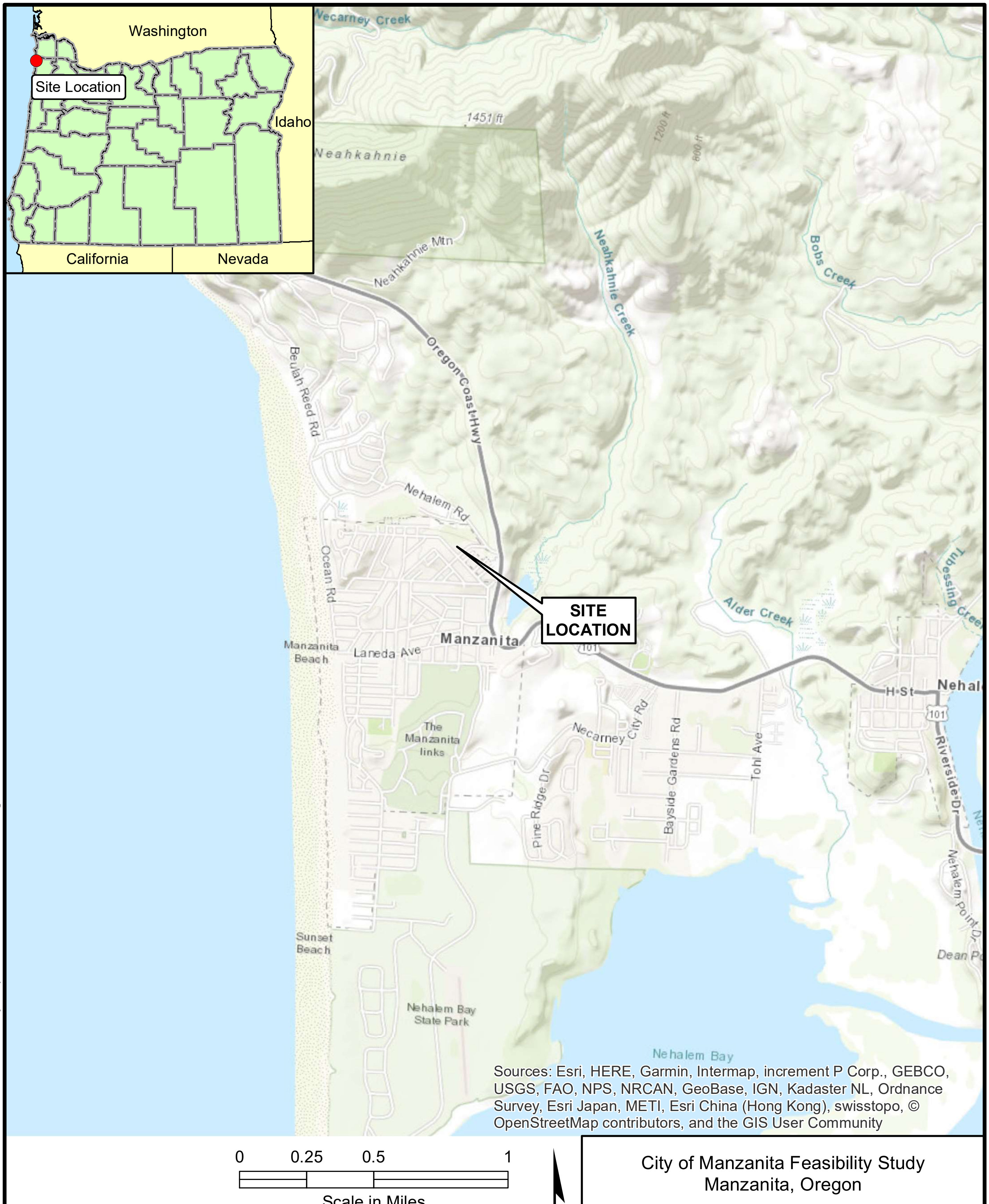
Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., 1996, Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700, Nature, 379, p. 246-249.

United States Geological Survey, 2021, Quaternary fault and fold database of the United States: U.S. Geological Survey website, https://earthquake.usgs.gov/hazards/qfaults/, accessed 4/22/21 6:39 PM.

Wells, R. E., Weaver, C. S., and Blakely, R. J., 1998, Fore arc migration in Cascadia and its neotectonic significance; Geology, v. 26, p. 759-762.

Wells, R.E., Snavely, P.D. Jr., MacLeod, N.S., Kelly, M.M., and Parker, M.J., 1994, Geologic Map of the Tillamook Highlands, Northwest Oregon Coast Range (Tillamook, Nehalem, Enright, Timber, Fairdale, and Blaine 15 minute Quadrangles): United States Geological Survey Open File Report 94-21, scale 1:62,500.

Yamaguchi, D.K., Atwater, B.F., Bunker, D.E., Benson, B.E., Reid, M.S., 1997, Tree-ring Dating the 1700 Cascadia Earthquake: Nature, v.389, p. 922-923.



City of Manzanita Feasibility Study
Manzanita, Oregon

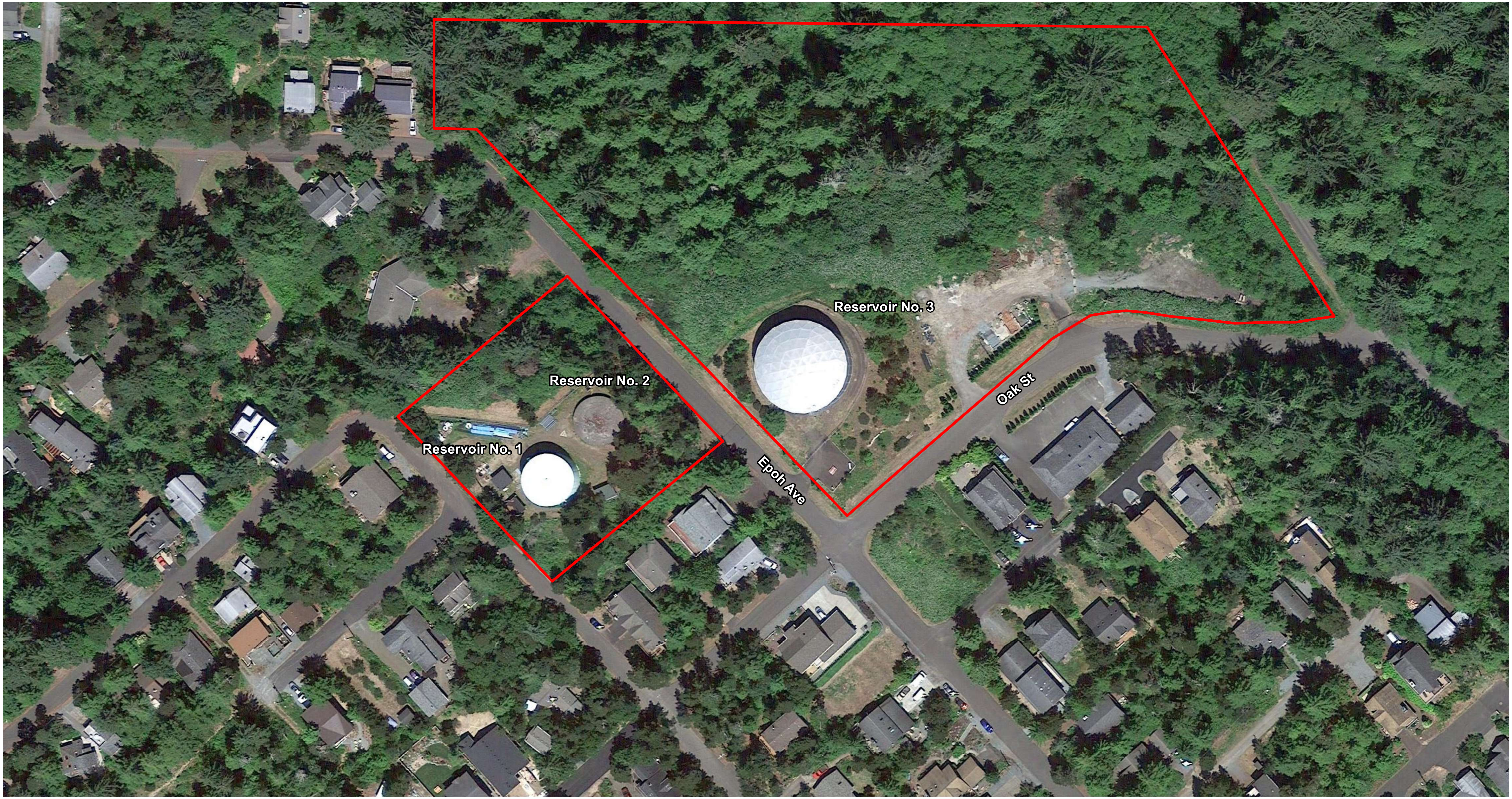
VICINITY MAP

April 2021 106549

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: T:\Projects\PDX\106000s\106549 - Manzanita\Vicinity Map.mxd Date: 4/23/2021 Login: DSJ



LEGEND

Approximate City-Owned Taxlots

0 50 100 200

Scale in Feet

City of Manzanita Feasibility Study
Manzanita, Oregon

SITE MAP

April 2021 106549

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2

Filename: T:\Projects\PDX\106000s\106549 - Manzanita\Site Map.mxd Date: 4/23/2021 Login: DSJ

LEGEND

Geologic Map Units

- Alluvial deposits
- Alsea Formation
- Eolian deposits
- Landslide deposits

0 100 200 400

Scale in Feet

NOTES

1. Geologic units from Oregon Geologic Data Compilation release 6 (OGDC-6)

City of Manzanita Feasibility Study
Manzanita, Oregon

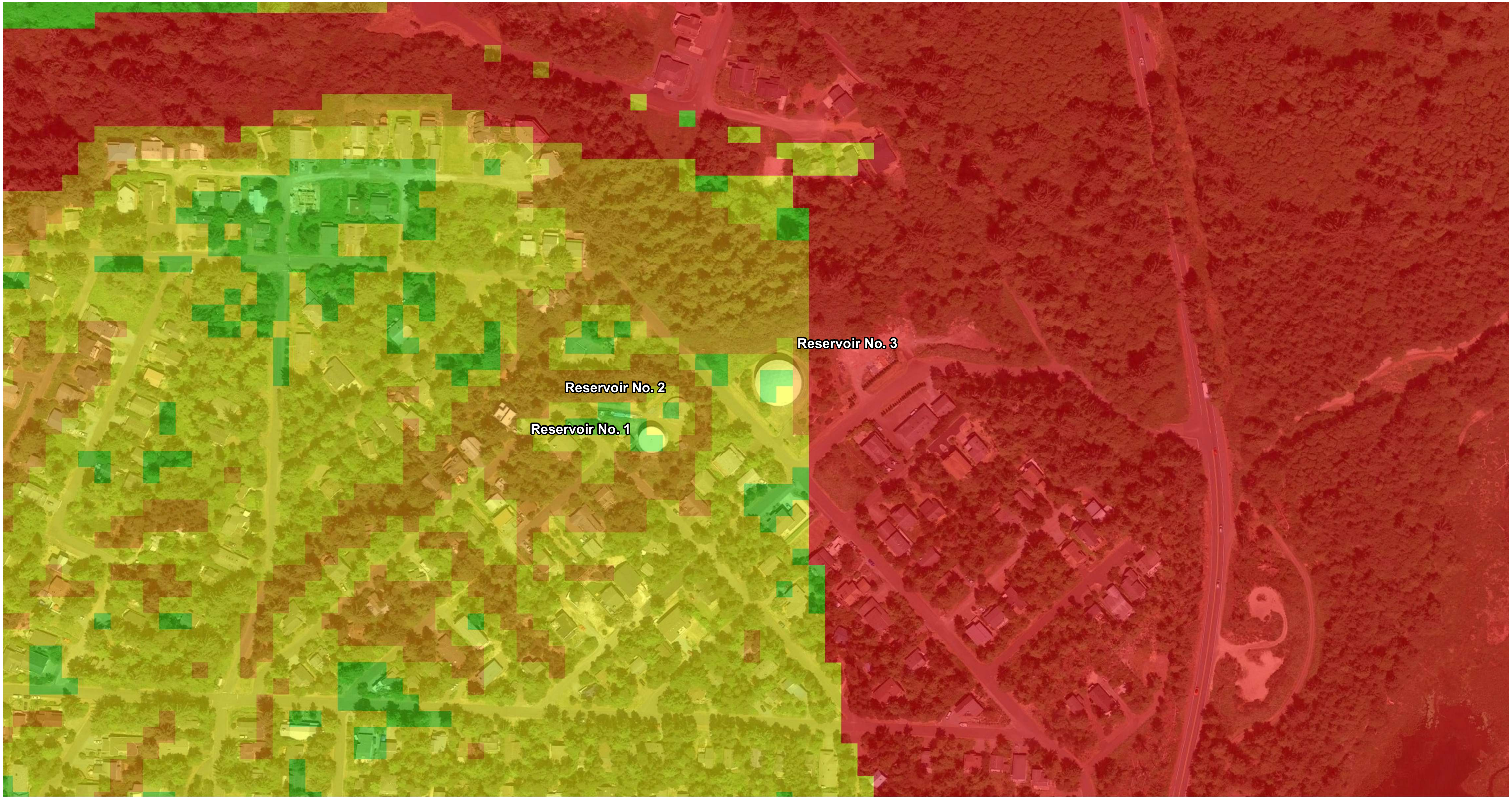
GEOLOGIC MAP

April 2021 106549

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 3

Filename: T:\Projects\PDX\106000s\106549 - Manzanita\GEOLOGIC mAP 2.mxd Date: 4/22/2021 Login: DSJ



LEGEND

Landslide Susceptibility

- Low
- Moderate
- High
- Very High

0 100 200 400
Scale in Feet

NOTES
1. Landslide susceptibility from DOGAMI publication O-16-2.

City of Manzanita Feasibility Study
Manzanita, Oregon

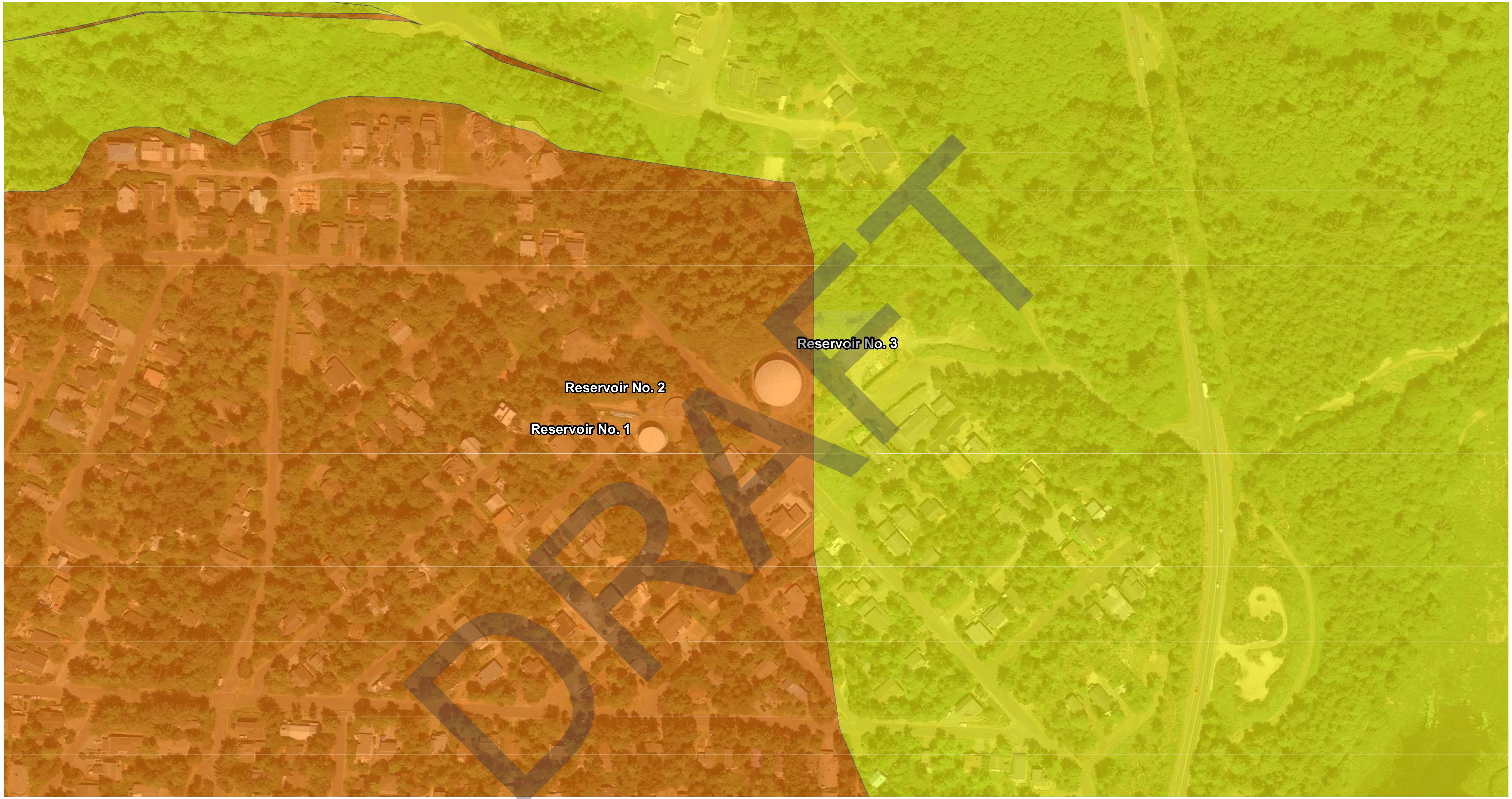
LANDSLIDE SUSCEPTIBILITY

April 2021 106549

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5

Filename: T:\Projects\PDX\106000s\106549 - Manzanita\Landslide Susceptibility.mxd Date: 4/23/2021 Login: DSJ



LEGEND

Liquefaction Susceptibility

Moderate

High

0 100 200 400

Scale in Feet

NOTES

1. Liquefaction susceptibility from DOGAMI publication O-13-6.

City of Manzanita Feasibility Study
Manzanita, Oregon

LIQUEFACTION SUSCEPTIBILITY

April 2021 106549

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 4

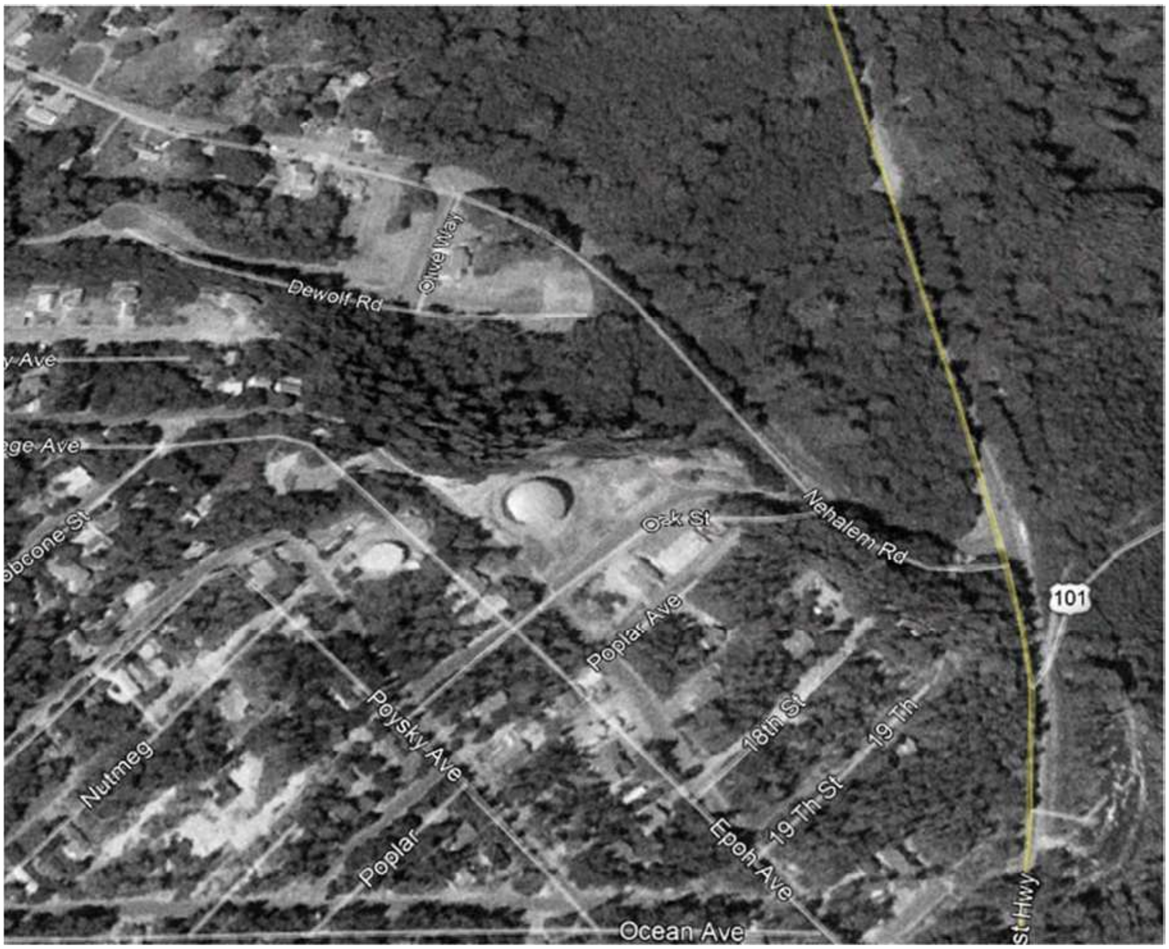
Filename: T:\Projects\PDX\106000s\106549 - Manzanita\Liquefaction Susceptibility.mxd Date: 4/22/2021 Login: DSJ

Appendix A

Aerial Photographs

CONTENTS

1994 Aerial Photograph of Site

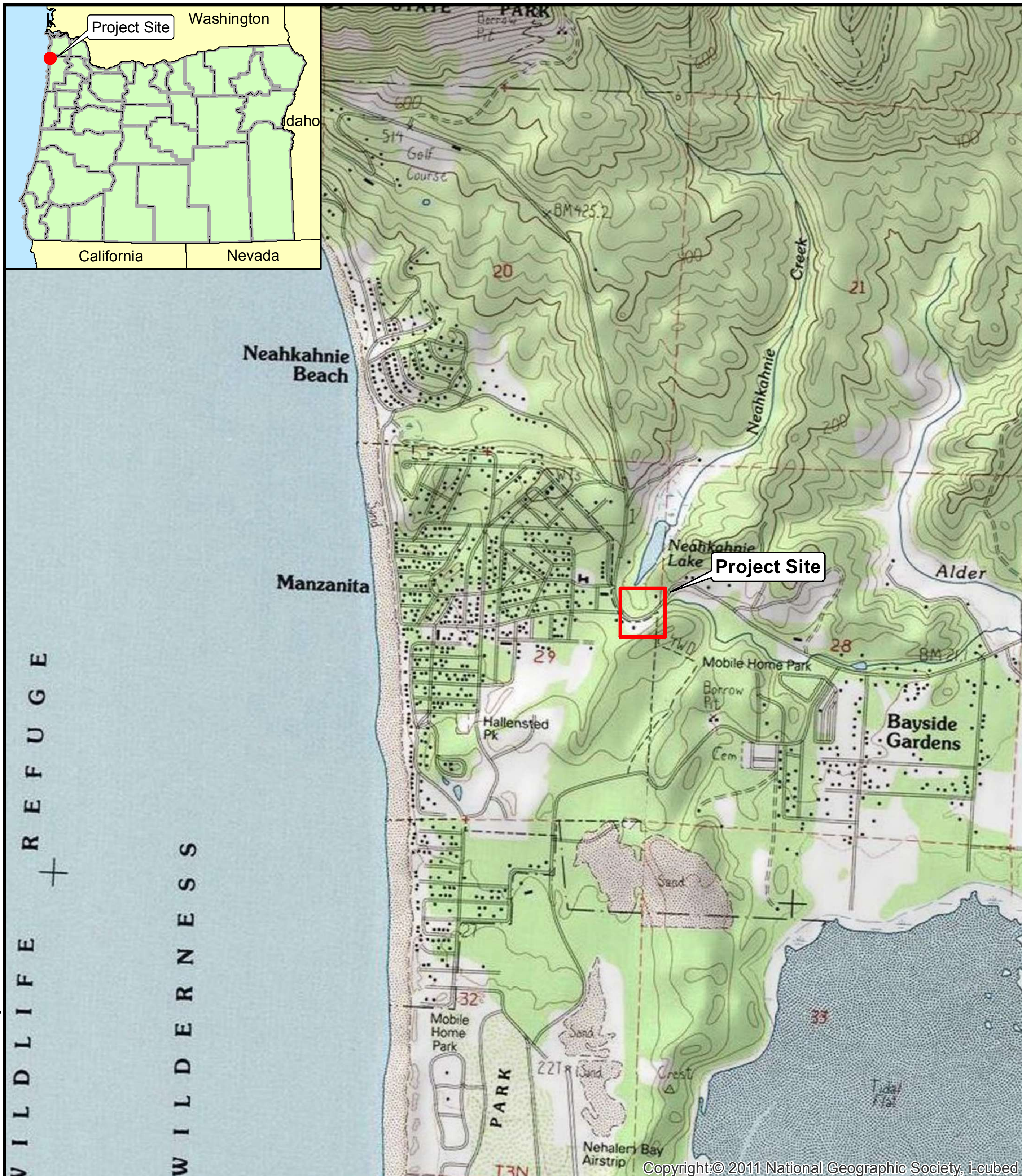


2000 Aerial Photograph of Site

Appendix B

Borings from Other Sites

CONTENTS



Path: T:\Projects\24-1\3749_OR101_Manzanita\Avmxd\Vicinity.mxd 2/14/2013 ath

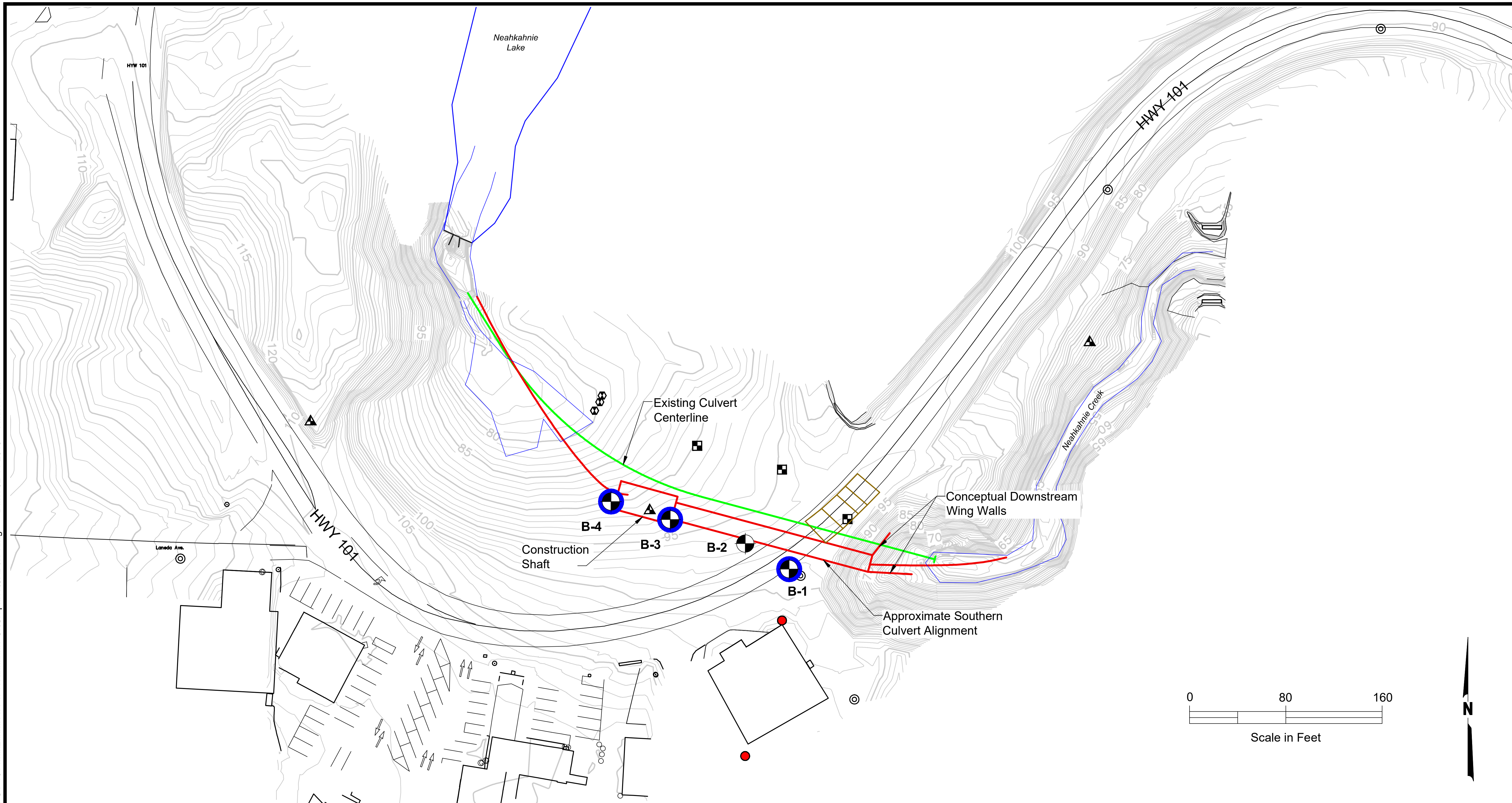
US 101 Neahkahnie Creek
Culvert Replacement
Manzanita, Oregon

VICINITY MAP

February 2013 | 24-1-03749-400

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1



EXPLANATION

B-2 Location and Designation of S&W Boring

B-1 Location and Designation of S&W Boring with Vibrating Wire Piezometer

Approximate Locations of Borings by Others:

ODOT (1992)

Wright/Deacon & Associates (2000)

CH2MHILL (2001)

Kleinfelder (2003)

Approximate Bent Locations of 1929 Trestle (may or may not have been demolished)

Note:
The horizontal datum is State Plane LDP and the vertical datum is NGVD 1929.

US 101 Neahkahnie Creek
Culvert Replacement
Manzanita, Oregon

SITE PLAN

February 2013 | 24-1-03749-400

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2

File: I:\WIP\Projects\24-1 Portland\3700\3749 US 101 Manzanita Ave to Neahkahnie Cr\Graphics\CAD\SP.dwg Date: 02-14-2013 Author: aah

			Hole No. **B-1**
Project **US 101 Neahkahnie Creek Culvert Replacement**		Purpose **Culvert**	E.A. No. **N/A**
Highway **US 101**		County **Tillamook**	Key No. **11258**
Hole Location Northing: **766,938.02**	Easting: **7,327,615.85**		Start Card No. **N/A**
Equipment **CME-850 Track (SPT Hammer Efficiency = 83.9%)**		Driller **Hardcore Drilling**	Bridge No. **N/A**
Project Geologist **Adrian A.J. Holmes**		Recorder **Cody K. Sorensen**	Ground Elev. **99.25 ft**
Start Date **December 20, 2012**	End Date **December 20, 2012**	Total Depth **75.00 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0					**0.00 - 0.80 ASPHALT CONCRETE**		Mud rotary drilling technique (5-inch hole)		
					0.80 - 1.70 BASE AGGREGATE - Occasional cobbles at 1.5 feet				
					1.70 - 24.00 SAND with trace silt; SP; Tan-brown; Nonplastic fines; Moist; Very loose to loose; Fine to medium sand; Homogeneous; (Sand Fill)				
5	N1	73	3-4-5						
10	N2	87	2-1-2						
15	N3	67	2-2-2	24					
20	N4	80	3-3-4						
					24.00 - 42.00 SAND with trace silt; SP; Tan-brown and gray; Nonplastic fines; Moist; Medium dense; Fine to medium sand; Stratified tan-brown and gray color; (Beach and Dune Deposits)				
25	N5	87	4-5-8	19					
30	N6	73	5-6-9						
35									

Project Name **US 101 Neahkahnie Creek Culvert Replacement** Hole No. **B-1**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
35	N7	87	7-12-14	20	**35.00 Grades to moist to wet with trace fine subrounded gravel**				
40	N8	93	6-10-13						
					42.00 - 55.50 SAND with trace silt; SP; Tan-brown and gray; Nonplastic fines; Moist; Dense to very dense; Fine to medium sand; Stratified tan-brown and gray color; (Beach and Dune Deposits)		Vibrating wire piezometer (Serial number 1239956) installed at 44.0 feet.		
45	N9	87	7-16-16	20					
50	N10	80	18-25-29		**50.00 Slight iron-oxide staining**				
					54.00 Grades to stratified with layers of silty SAND				
55	N11a N11b	100	11-6-7		**55.50 - 59.00 Clayey SILT with some fine to coarse sand and trace fine to coarse gravel; MH; Gray; Low to medium plasticity; Wet; Stiff; Rounded to subrounded gravel; Occasional organics; (Fluvial and Estuarine Deposits)**				
					59.00 - 62.00 Clayey SILT; MH; Gray and orange; Medium to high plasticity; Moist; Hard; Relict mudstone structure; (Residual Soil)				
60	N12	100	15-20-25						
					62.00 - 75.00 MUDSTONE; Gray; Slightly Weathered; Very soft to soft (R1-R2); Moderately close; Very thin to thinly bedded, high angle, rough, undulating joints with iron-oxide staining; Numerous SANDSTONE interbeds oriented at approximately 20º; (Marine Sedimentary Rock)				
65	N13	87	24-28-35						
	C1	100	RQD = 100				Switched to HQ3 drilling technique (4-inch hole)		
70	C2	40	RQD = 24						
75					**75.00 End of hole**				
80									
85									
88									

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-2**
Project **US 101 Neahkahnie Creek Culvert Replacement**	Purpose **Culvert**	E.A. No. **N/A**
Highway **US 101**	County **Tillamook**	Key No. **11258**
Hole Location Northing: **766,959.26** Easting: **7,327,579.48**		Start Card No. **N/A**
Equipment **CME-850 Track (SPT Hammer Efficiency = 83.9%)**	Driller **Hardcore Drilling**	Bridge No. **N/A**
Project Geologist **Adrian A.J. Holmes**	Recorder **Cody K. Sorensen**	Ground Elev. **96.84 ft**
Start Date **December 17, 2012** / End Date **December 17, 2012**	Total Depth **75.50 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0						**0.00 - 0.30 SOD**		Mud rotary drilling technique (5-inch hole)		
						0.30 - 23.00 SAND with trace silt; SP; Tan-brown; Nonplastic fines; Moist; Very loose grading to loose; Fine to medium sand; Homogeneous; (Sand Fill)				
5	N1	40	1-1-1			**5.00 Slight iron-oxide staining to 12 feet**				
10	N2	60	1-1-2		25	**10.00 Grades to wet**				
15	N3	33	1-2-1							
20	N4	60	2-2-3		24					
						23.00 - 27.00 SAND with some to trace silt and trace fine to coarse gravel; SP-SM/SP; Tan-brown; Nonplastic fines; Wet; Medium dense; Fine to medium sand; Rounded to subangular gravel; (Fluvial and Estuarine Deposits)				
25	N5	40	7-6-8							
						27.00 - 46.20 SAND with trace silt; SP; Tan-brown and gray; Nonplastic fines; Moist to wet; Medium dense to dense; Fine to medium sand; Stratified tan-brown and gray color; (Beach and Dune Deposits)				
30	N6	47	9-11-13		16					
35										

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
35	N7	73	8-8-16						
40	N8	67	9-15-29	19	**40.00 Slight iron-oxide staining to 46.2 feet**				
45	N9a	80	11-12-8						
	N9b			40	**46.20 - 52.00 Clayey SILT with trace fine sand; ML, MH; Dark brown to dark gray; Low plasticity grading to medium plasticity with depth; wet; stiff to medium stiff; occasional organic debris and flecks; (Fluvial and Estuarine Deposits)**				
50	U1	100		67					
	N10	100	2-2-3	85	**52.00 - 58.00 Clayey SILT with some fine to coarse sand to sandy clayey SILT with some fine gravel; MH; Light blue-white, brown and gray-brown; High plasticity; Wet; Medium stiff to stiff; Subangular to angular gravel composed of mudstone; (Residual Soil)**				
55	N11	67	3-5-6	55					
					58.00 - 64.50 Clayey SILT with trace fine sand; MH; Light gray and orange to orange-brown; High plasticity; Wet; Stiff; Moderate iron-oxide staining; Relict mudstone/sandstone structure; (Residual Soil)				
60	N12	100	1-6-8	62					
65	N13	100	7-25-50		**64.50 - 75.50 MUDSTONE; Gray; Slightly Weathered; Very soft to soft (R1-R2); Very thin to thinly bedded; Occasional thin interbeds of decomposed sandstone; (Marine Sedimentary Rock)**				
70	N14	100	37-36-41						
75	N15	100	50/1st 5"		**75.50 End of hole**				
80									
85									
88									

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-3**
Project **US 101 Neahkahnie Creek Culvert Replacement**	Purpose **Culvert**	E.A. No. **N/A**
Highway **US 101**	County **Tillamook**	Key No. **11258**
Hole Location Northing: **766,979.23** Easting: **7,327,517.04**		Start Card No. **N/A**
Equipment **CME-850 Track (SPT Hammer Efficiency = 83.9%)**	Driller **Hardcore Drilling**	Bridge No. **N/A**
Project Geologist **Adrian A.J. Holmes**	Recorder **Cody K. Sorensen**	Ground Elev. **93.65 ft**
Start Date **December 17, 2012** End Date **December 18, 2012**	Total Depth **75.00 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0					**0.00 - 14.00 Gravelly clayey SILT and silty GRAVEL; MH/GM; Gray and orange-brown; Low to medium plasticity fines; Moist; Medium stiff to stiff / loose; Subangular to angular gravel composed of mudstone; (Landslide Debris Fill)**		Mud rotary drilling technique (5-inch hole)		
5	N1	40	3-2-3						
10	N2	67	4-4-5	35					
15	N3	67	6-7-7	26	**14.00 - 18.00 Gravelly clayey SILT with some fine to coarse sand; MH; Brown and orange-brown; Low to medium plasticity; Moist; Stiff; Subangular to angular gravel composed of mudstone and basalt; Slight iron-oxide staining; (Landslide Debris Fill)**				
20	N4	73	4-2-3	52	**18.00 - 24.00 Clayey SILT with some fine to coarse gravel and some fine to coarse sand; MH; Dark gray and orange-brown; Low to medium plasticity; Moist; Medium stiff; Subangular to angular gravel; Occasional organics; (Landslide Debris Fill)**				
25	N5	47	5-4-6	22	**24.00 - 39.00 SAND with some silt to silty SAND with trace fine gravel; SP-SM, SM; Brown-gray; Nonplastic fines; Wet; Loose to medium dense; Mostly fine to medium sand; Subrounded to subangular gravel composed of mudstone; Scattered organic debris; (Fluvial and Estuarine Deposits)**				
30	N6	0	4-4-5						
35									

ODOT DRILL LOG - FOR RPTS - NO MATL DESC 24-1-3749-ODOTR1.GPJ ODOT_MAN.GDT 2/14/13

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Drilling Methods, Size and Remarks	Water Level/ Date
35	N7	80	4-6-7	24			
						Vibrating wire piezometer (Serial number 1239957) installed at 38.0 feet.	
40	N8	100	0-1-3		**39.00 - 44.00 Clayey SILT with trace fine to coarse sand and trace fine gravel; MH; Gray; Medium plasticity; Wet; Soft; Rounded, moderately weathered gravel; (Fluvial and Estuarine Deposits)**		
					42.00 Gravelly layer to 44.0 feet		
45	U1	0			**44.00 - 57.00 Clayey SILT with some fine to coarse gravel; MH; Gray and orange-brown; Medium to high plasticity; Moist to wet; Medium stiff to very stiff; Angular gravel composed of mudstone; Relict mudstone structure; (Residual Soil)**		
	N9	67	6-3-4	48			
50	N10	13	5-7-15				
55	N11	87	5-6-12				
					57.00 - 75.00 MUDSTONE; Gray; Slightly weathered; Very soft to soft (R1-R2); Very thin to thinly bedded with scattered to numerous SANDSTONE interbeds and 1- to 2-mm-thick seams of calcite; (Marine Sedimentary Rock)		
60	N12	100	34-39-50/5"				
65	N13	100	50/1st 5"			Switched to HQ3 drilling technique (4-inch hole)	
	C1	100	RQD = 100				
					69.00 Thin 1/4-in-thick SANDSTONE layer oriented at 50º		
70	C2	100	RQD = 100				
75					**75.00 End of hole**		
80							
85							
88							

ODOT DRILL LOG - FOR RPTS - NO MATL DESC 24-1-3749-ODOTR1.GPJ ODOT_MAN.GDT 2/14/13

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

Project **US 101 Neahkahnie Creek Culvert Replacement**	Purpose **Culvert**	Hole No. **B-4** / E.A. No. **N/A**
Highway **US 101**	County **Tillamook**	Key No. **11258**
Hole Location Northing: **766,994.19** Easting: **7,327,467.87**		Start Card No. **N/A**
Equipment **CME-850 Track (SPT Hammer Efficiency = 83.9%)**	Driller **Hardcore Drilling**	Bridge No. **N/A**
Project Geologist **Adrian A.J. Holmes**	Recorder **Cody K. Sorensen**	Ground Elev. **90.20 ft**
Start Date **December 19, 2012** / End Date **December 19, 2012**	Total Depth **41.50 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0					**0.00 - 0.40 SILT; ML; Dark brown; Nonplastic; Wet; Soft; Scattered to numerous roots and organics; (Fill)**		Mud rotary drilling technique (5-inch hole)		
					0.40 - 14.00 Clayey SILT with some fine to coarse sand and some fine to coarse gravel; MH; Brown-gray and orange-brown; Medium plasticity; Moist; Medium stiff to stiff; Subangular to angular gravel composed of mudstone; Occasional cobbles; (Landslide Debris Fill)				
5	N1	80	3-3-4	44					
10	N2	87	2-3-10	42					
					14.00 - 22.00 Sandy clayey SILT with some fine to coarse gravel to silty GRAVEL with some fine to coarse sand; MH/GM; Gray-brown and orange-brown; Low to medium plasticity fines; Moist to wet; Soft to stiff / very loose; Subrounded to angular gravel; Slight iron-oxide staining; (Landslide Debris Fill)				
15	N3	67	2-3-1						
20	N4	80	3-5-4						
					22.00 Scattered organics and wood debris				
					22.00 - 29.80 SAND with some silt and trace fine gravel; SP-SM; Gray-brown; Nonplastic fines; Wet; Medium dense; Fine to medium sand; Subrounded to subangular gravel; Occasional organics; (Fluvial and Estuarine Deposits)				
25	N5	53	4-9-13	24					
							Vibrating wire piezometer (Serial number 1239958) installed at 28.0 feet.		
30	N6	80	9-7-4	67	**29.80 - 37.00 Clayey SILT; MH; Gray, orange-brown and orange; High plasticity; Stiff to very stiff; Relict mudstone structure; Moderate iron-oxide staining; (Residual Soil)**				
35									

ODOT DRILL LOG - FOR RPTS - NO MATL DESC 24-1-3749-ODOTR1.GPJ ODOT_MAN.GDT 2/14/13

Project Name **US 101 Neahkahnie Creek Culvert Replacement** Hole No. **B-4**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
35	N7	100	8-9-15						
					37.00 - 41.50 MUDSTONE; Gray; Slightly weathered; Very soft to soft (R1-R2); Very thin to thinly bedded; Scattered interbeds of very soft to soft (R1-R2), fine- to medium-grained SANDSTONE; (Marine Sedimentary Rock)				
40	N8	100	17-22-31						
					41.50 End of hole				
45									
50									
55									
60									
65									
70									
75									
80									
85									
88									

ODOT DRILL LOG - FOR RPTS - NO MATL DESC 24-1-3749-ODOTR1.GPJ ODOT_MAN.GDT 2/14/13

NOTE: The interior length of each core box is 2.0 feet.

US 101 Neahkahnie Creek
Culvert Replacement
Manzanita, Oregon

BORING B-1
CORE PHOTOGRAPHS

February 2013 24-1-03749-400

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A5

File: I:\WIP\Projects\24-1 Portland\3700\3749 US 101 Manzanita Ave to Neahkahnie Cr\Graphics\CAD\Core Photo Figures.dwg Date: 02-14-2013 Author: aah

File: I:\WIP\Projects\24-1 Portland\3700\3749 US 101 Manzanita Ave to Neahkahnie Cr\Graphics\CAD\Core Photo Figures.dwg Date: 02-14-2013 Author: aah

NOTE: The interior length of each core box is 2.0 feet.

US 101 Neahkahnie Creek
Culvert Replacement
Manzanita, Oregon

BORING B-3
CORE PHOTOGRAPHS

February 2013 24-1-03749-400

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A6

IMPORTANT INFORMATION

Important Information

About Your Geotechnical/Environmental 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 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 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 B.
10% Design Drawings

Design Drawings For

City of Manzanita

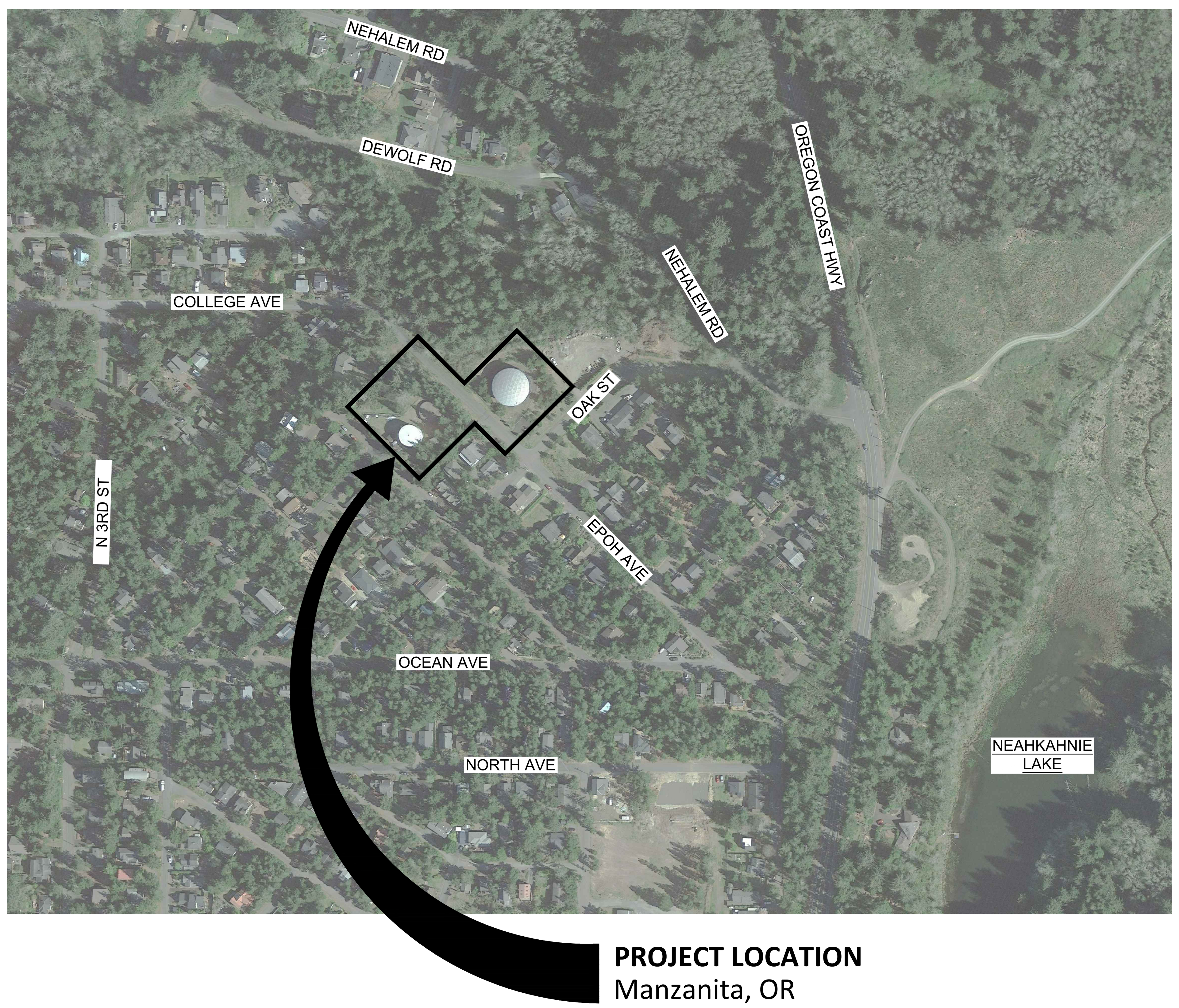
Replacement of Water Storage Tanks

10% Design Submittal

December 2021

Project No.

HDR 10291767



PROJECT LOCATION
Manzanita, OR

VICINITY MAP

0 250 500

SCALE IN FEET (APROX)

N

INDEX OF DRAWINGS

OWNER REPRESENTATIVE:

NAME	AGENCY	CONTACT
DAN WEITZEL PUBLIC WORKS DIRECTOR	CITY HALL 1090 OAK ST. MANZANITA, OR 97130	503-368-5347 DWEITZEL@CI.MANZANITA.OR.US

ENGINEER'S REPRESENTATIVES:

NAME	AGENCY	CONTACT
DAN JOHNSTON, PE	HDR INC. 1050 SW SIXTH AVE, SUITE 1800 PORTLAND, OR 97204	503-727-3946 DANIEL.JOHNSTON@HDRINC.COM

KNOW WHAT'S BELOW.
CALL BEFORE YOU DIG. 811
"LOOK UP AND LIVE"

10% DESIGN (NOT FOR CONSTRUCTION)

1 2 3 4 5 6 7 8

D C B A

GENERAL NOTES:

1. DURING DEMOLITION AND REPLACEMENT OF RESERVOIRS #1 AND #2, RESERVOIR #3 WILL BE ADJUSTED SO AS TO SUPPORT BOTH PRESSURE ZONES. A TEMPORARY PUMPSTATION AND TEMPORARY CONNECTIONS ARE REQUIRED FOR THIS OPERATION. ONCE THE NEW RESERVOIRS ARE COMPLETED, RESERVOIR #3 WILL BE TAKEN OFFLINE AND REPURPOSED OR DEMOLISHED.

0 20 40
SCALE IN FEET

N

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	RANDY HARRINGTON
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

THE CITY OF MANZANITA OREGON

CITY OF MANZANITA
WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

EXISTING CONDITIONS AND TEMPORARY FACILITIES

0 1" 2"

DATE	01 / 11 / 2022
FILENAME	02 _ C-01
SCALE	1" = 20'
SHEET	C-01

LEGEND:

- GRAVEL DRIVEWAY
- GRAVEL & DEEP SOIL MIXING
- FENCE —x—x—

KEY NOTES:

1. STANDBY GENERATOR W/ SUB-BASE DIESEL TANK AND SOUND ATTENUATING ENCLOSURE
2. CMU PUMPSTATION BLDG
3. CMU TELECOMMUNICATIONS BDLG

GENERAL NOTES:

1. THE INTENT FOR STORMWATER MANAGEMENT IS TO DIRECT ROOF RUNOFF TO SPLASHPADS, AND TO DISPERSE RUNOFF TO ADJACENT VEGETATED AREAS.
2. PROVIDE TEMPORARY EROSION CONTROL MEASURES AND RESTORE EXISTING VEGETATION WITHIN PROJECT LIMITS.

0 20 40
SCALE IN FEET

N

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	RANDY HARRINGTON
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

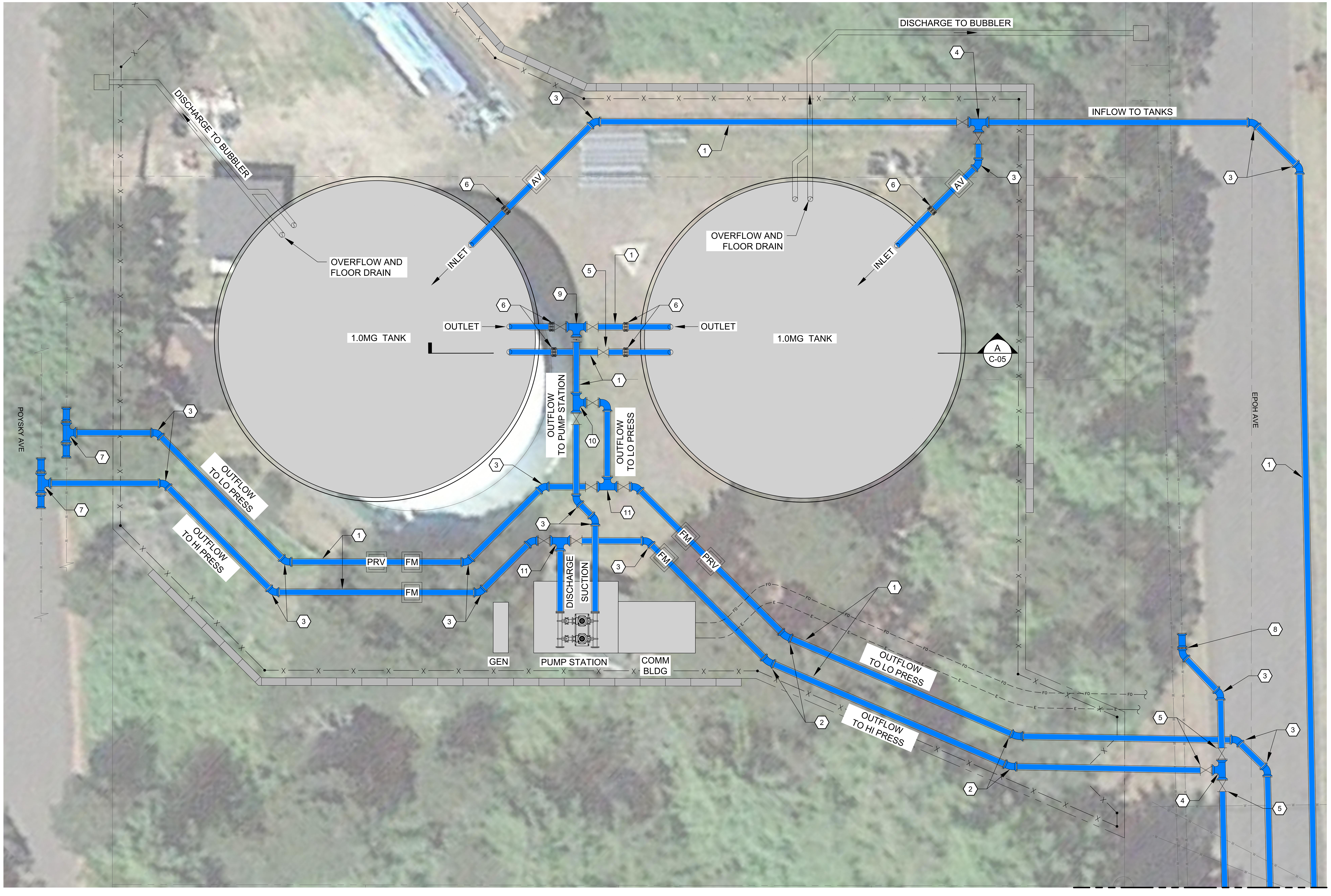
CITY OF MANZANITA
WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

SITE LAYOUT

0 1" 2"

DATE 01 / 11 / 2022
FILENAME 03 _ C-02
SCALE 1" = 20'
SHEET C-02



LEGEND:

- ROW
- WATER
- FENCE
- COMM
- ELEC

KEY NOTES:

1. 10" CLASS 52 DI PIPE W/ RESTRAINED JOINTS
2. (1) 10" DI 22.5° BEND (MJ)
3. (1) 10" DI 45° BEND (MJ)
4. (1) 10" DI TEE (MJ)
5. (1) 10" DI GATE VALVE (MJ)
6. (1) 10" RESTRAINED FLEXIBLE COUPLING (MJ)
7. CONNECT TO EXISTING WATER MAIN BY INSTALLING:
   (1) 10" DI TEE (MJ)
   (2) 10" DI 12" SPOOL W/ TRANSITION COUPLING (MJ)
8. CONNECT TO EXISTING WATER MAIN BY INSTALLING:
   (1) 10" DI 45° BEND (MJ)
   (1) 10" DI 12" SPOOL W/ TRANSITION COUPLING (MJ)
9. (1) 10" DI TEE (MJxFL)
   (2) 10" DI GATE VALVE (FLxMJ)
   (1) 10" DI 12" SPOOL
   (1) 10" DI BUTTERFLY VALVE (MJ)
10. (1) 10" DI TEE (MJxFL)
    (1) 10" DI GATE VALVE (FLxMJ)
    (1) 10" DI GATE VALVE (FL)
    (1) 10" DI 90° BEND (FLxMJ)
11. (1) 10" DI TEE (MJxFL)
    (1) 10" DI GATE VALVE (FLxMJ)
    (1) 10" DI GATE VALVE (FL)
    (1) 10" DI 45° BEND (FLxMJ)

0 10 20
SCALE IN FEET

N

HDR

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	RANDY HARRINGTON
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

THE CITY OF MANZANITA OREGON

CITY OF MANZANITA
WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

UTILITY PLAN NORTH

0 1" 2"

DATE 01 / 11 / 2022
FILENAME 03 _ C-03
SCALE 1" = 10'
SHEET C-03

MATCHLINE SEE SHEET C-03

LEGEND:

EOP
MISC. UG
ROW
WATER

KEY NOTES:

1. 10" CLASS 52 DI PIPE W/ RESTRAINED JOINTS
2. (1) 10" DI 45° BEND (MJ)
3. CONNECT TO EXISTING WATER MAIN BY INSTALLING:
(1) TRANSITION COUPLING (MJ)
4. CONNECT TO EXISTING WATER MAIN BY INSTALLING:
(1) 10" DI TEE (MJ) AND WET-TAP
(1) 10" DI 12" SPOOL W/ TRANSITION COUPLING (MJ)
5. (1) 10" DI GATE VALVE (MJ)

0 10 20
SCALE IN FEET

N

HDR

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	RANDY HARRINGTON
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

THE CITY OF MANZANITA, OREGON

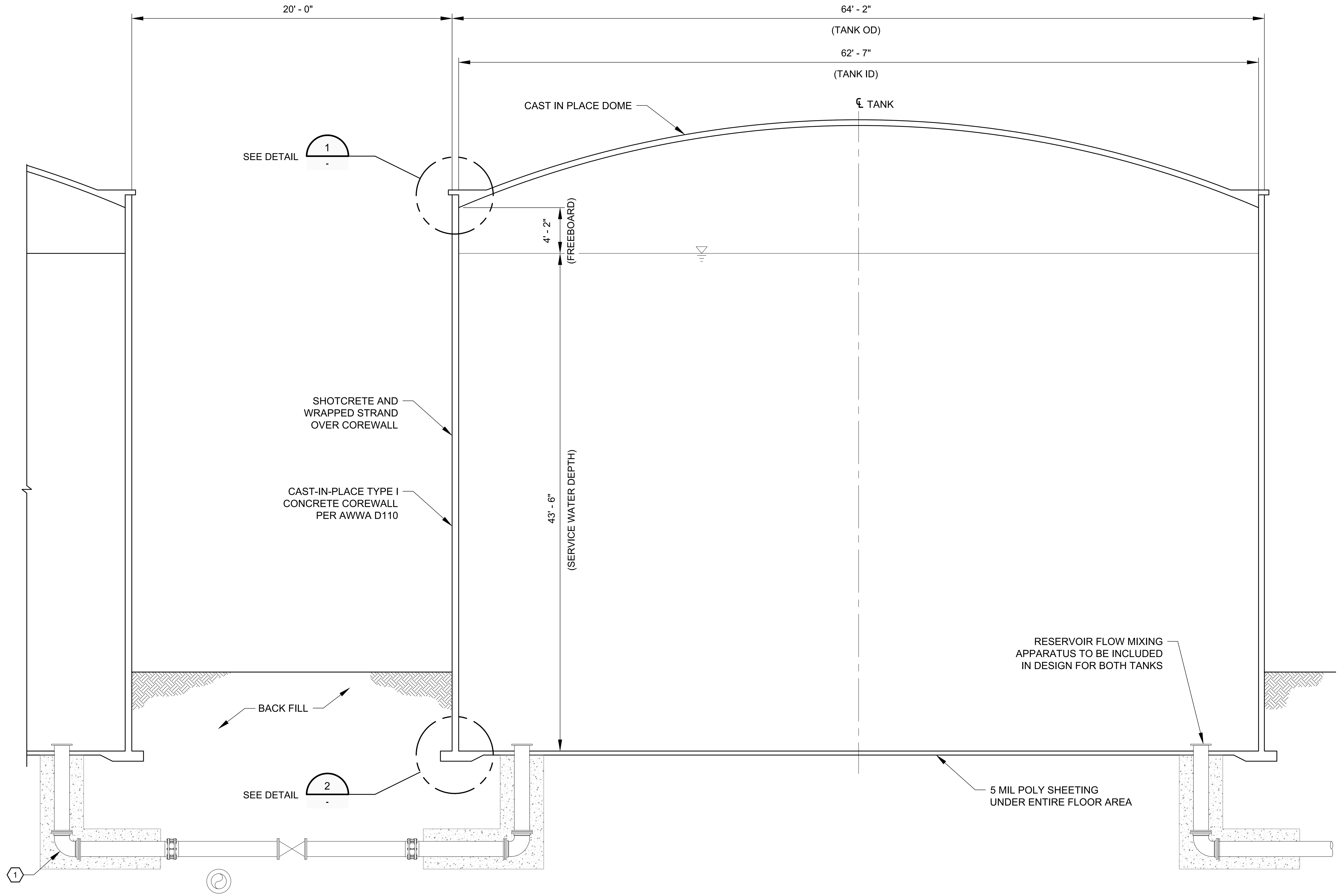
CITY OF MANZANITA
WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

UTILITY PLAN SOUTH

0 1" 2"

DATE	01 / 11 / 2022
FILENAME	04 _ C-04
SCALE	1" = 10'
SHEET	C-04

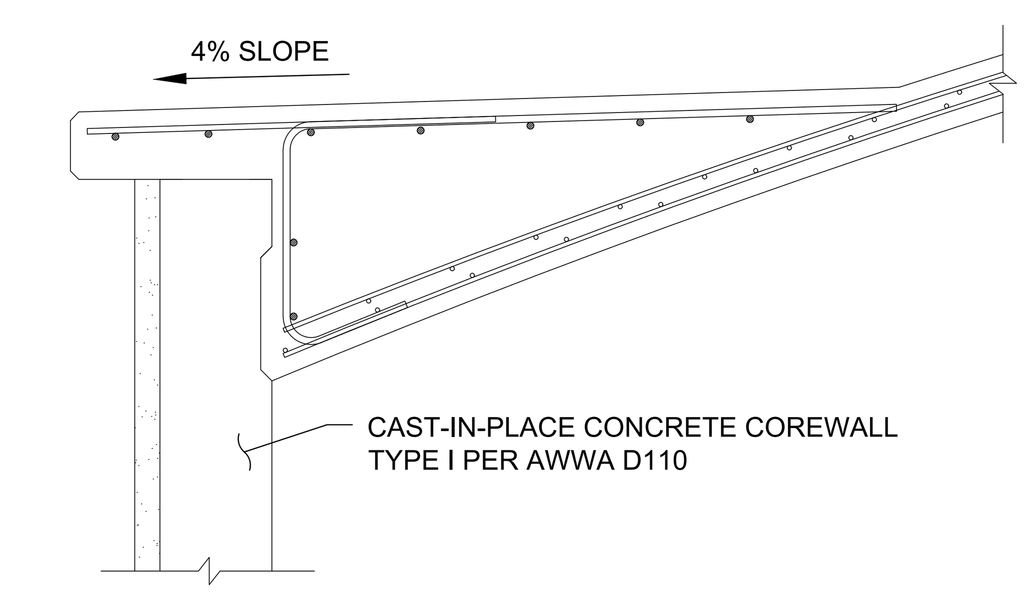


A C-03 SECTION

0 5 10
SCALE IN FEET

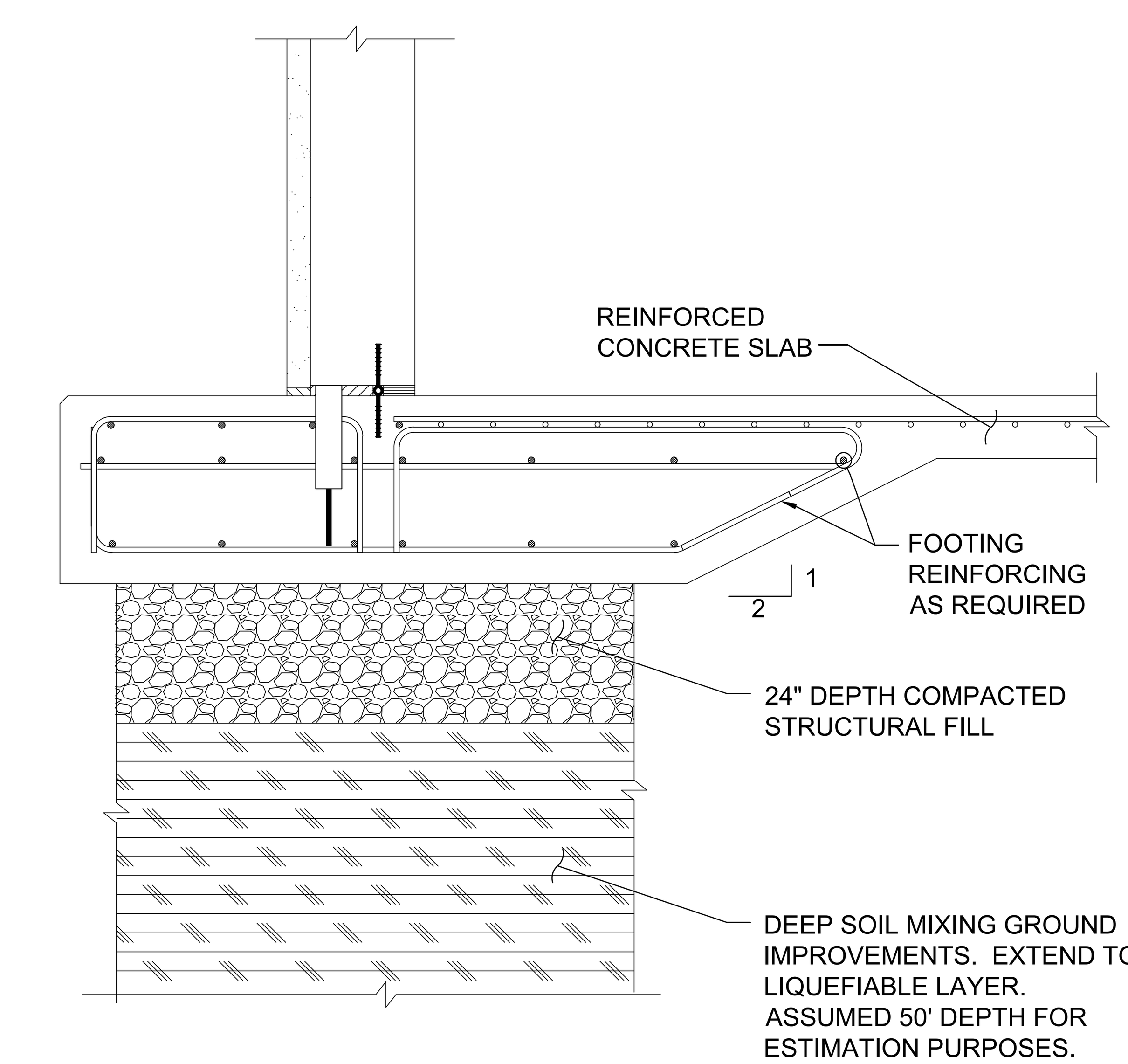
GENERAL NOTES:

1. NOTES, DIMENSIONS, AND ELEVATIONS APPLY TO BOTH TANKS.
2. SEE UTILITY PLAN SHEETS FOR PIPE CONTINUATION.



1 - DETAIL

NTS



2 - DETAIL

NTS

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	RANDY HARRINGTON
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

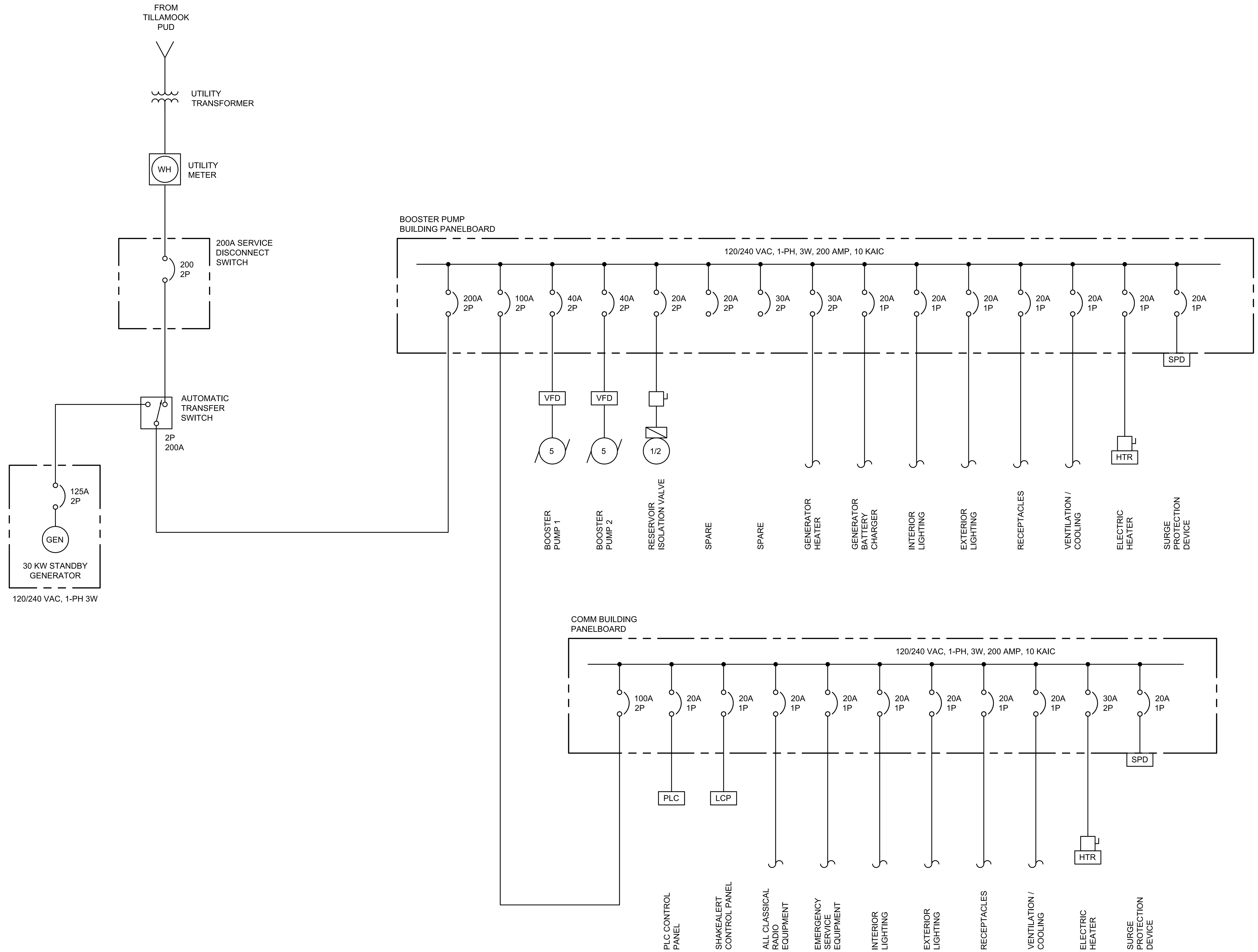
CITY OF MANZANITA WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

WATER TANK AND FOUNDATION SECTION

0 1" 2"

DATE	01 / 11 / 2022
FILENAME	06 _ C-05
SCALE	AS NOTED
SHEET	C-05



ELECTRICAL ONE-LINE DIAGRAM

GENERAL NOTES:

1. STANDBY GENERATOR SHALL HAVE TO CAPABILITY TO FULLY OPERATE THE RESERVOIR FACILITY AND INCLUDE FUEL STORAGE FOR 96 HOURS OF OPERATION UNDER FULL LOAD.
2. BOOSTER PUMPS ARE AUTOMATICALLY CONTROLLED BY THE PLC CONTROL PANEL.
3. PLC CONTROL PANELS SHALL INCLUDE A UPS WITH 10 MINUTE BATTERY RUN TIME.
4. PLC CONTROL PANELS SHALL PROVIDE UPS POWER TO THE RESERVOIR LEVEL TRANSMITTERS, RESERVOIR FLOW METERS, PH ANALYZER AND TURBIDIMETER.
5. RESERVOIR ISOLATION VALVE WILL AUTOMATICALLY CLOSE WHEN A SIESMIC EVENT WARNING IS RECEIVED FROM THE SHAKEALERT SYSTEM.
6. NOT ALL PANELBOARD CIRCUITS ARE SHOWN.

ISSUE	DATE	DESCRIPTION
0	12 / 23 / 2021	10% DESIGN

PROJECT MANAGER	DAN JOHNSTON
DESIGNED	DON BEST
DRAWN	SHAWN KUHNS
PROJECT NUMBER	10291767

CITY OF MANZANITA WATER STORAGE TANKS

10% DESIGN (NOT FOR CONSTRUCTION)

ELECTRICAL ONE-LINE DIAGRAM

DATE	01 / 11 / 2022
FILENAME	07 _ E-01
SCALE	NTS
SHEET	E-01

Appendix C.
Construction Cost Estimate

Feasibility Study for Water Storage Improvements
January 2022

Detailed Opinion of Probable Construction Cost

Alternative 3 - Complete Tank Replacement	Qty	Units	Price/Unit	Unit Totals	Category Total
Demolition and Temporary Facilities					
Temporary Pumpstation	1	LS	$15,000	$15,000	
Temporary Watermain	1	LS	$10,000	$10,000	
Tank Demolition	2	EA	$5,000	$10,000	
Watermain Removal	1	LS	$5,000	$5,000	
Building Removal	1	LS	$5,000	$5,000	
Site Demolition	1	LS	$5,000	$5,000	
Section Total					**$50,000**
Site Work					
Clearing and Grubbing	1	LS	$25,000	$25,000	
Erosion Control	1	LS	$10,000	$10,000	
Site Preparation and Grading	3112	SY	$25	$77,900	
Earthwork	556	CY	$25	$13,900	
Gravel Surfacing	1994	SY	$30	$59,900	
Storm Drainage	1	LS	$10,000	$10,000	
MSE Retaining Wall	330	LF	$100	$33,000	
Security Fencing	750	LF	$75	$56,300	
Landscape Restoration	1	LS	$15,000	$15,000	
Section Total					**$301,000**
Yard Pipelines, Vaults, Connections					
10-inch Dia. DI Potable Water Pipe	1380	LF	$190	$262,200	
12-inch Dia. DI Potable Water Pipe	175	LF	$225	$39,400	
Trench Surface Restoration	500	LF	$100	$50,000	
10-inch Connection to 10-inch Existing Main	6	EA	$7,000	$42,000	
10-inch Tee	9	EA	$850	$7,700	
10-inch 22.5-degree Bend	4	EA	$650	$2,600	
10-inch 45-degree Bend	24	EA	$1,000	$24,000	
10-inch 90-degree Bend	1	EA	$1,250	$1,300	
10-inch Gate Valve	11	EA	$3,500	$38,500	
10-inch Butterfly Valve	1	EA	$5,000	$5,000	
10-inch Restrained Flexible Coupling	6	EA	$1,500	$9,000	
Altitude Valve Vault	2	EA	$50,000	$100,000	
Flow Meter Vault	4	EA	$25,000	$100,000	
PRV Vault	2	EA	$35,000	$70,000	
Section Total					**$751,700**
Tanks, Foundations, Ground Improvements					
1.2MG AWWA D110 Type I Tank w/ Dome	2	EA	$1,800,000	$3,600,000	
Ground Improvements	16000	CY	$200	$3,200,000	
Spoils Disposal	5333	CY	$60	$320,000	
Section Total					**$7,120,000**
Testing and Commissioning					
Testing and Disinfection of Yard Pipelines	1	LS	$10,000	$10,000	
Testing and Disinfection of Tanks	1	LS	$15,000	$15,000	
Testing and Commissioning of PS and EI&C	1	LS	$15,000	$15,000	
Section Total					**$40,000**
Buildings & Mechanical					
Pumpstation CMU Building	236	SF	$170	$40,200	
Pumpstation Foundation	236	SF	$60	$14,200	
Generator Concrete Pad	100	SF	$10	$1,000	
Telecommunications Building	155	SF	$170	$26,400	
Telecommunications Foundation	155	SF	$60	$9,300	
Pumps and Mechanical	2	EA	$40,000	$80,000	
Building Mechanical & Plumbing	1	LS	$10,000	$10,000	
Section Total					**$181,100**
Electrical, Instrumentation, Generator, Monitoring, Control					
Panelboards, Conduit/Wire, LTG/RCPT	1	LS	$75,000	$75,000	
Communication Cables	1	LS	$10,000	$10,000	
Variable Frequency Drives (5hp)	2	LS	$5,000	$10,000	
Standby Genset and ATS (30kw)	1	LS	$40,000	$40,000	
ShakeAlert Integration	1	LS	$35,000	$35,000	
Level Transmitters (Ultrasonic)	2	LS	$3,500	$7,000	
pH Analyzer	1	LS	$5,000	$5,000	
Turbidimeter	1	LS	$5,000	$5,000	
Chlorine Analyzer	1	LS	$7,500	$7,500	
Flow Meter (10" line)	1	LS	$10,000	$10,000	
PLC Control Panel and Telemetry	1	LS	$30,000	$30,000	
System Integration/SCADA Programming	1	LS	$35,000	$35,000	
Section Total					**$269,500**
			Subtotal		**$8,713,300**
			General Conditions, Mobilization, Insurance & Bonds	10%	$871,400
			Miscellaneous Items and Contingencies	15%	$1,307,000
			Alternative 3 Construction Cost		**$10,892,000**
	AACE Estimate Range (Class 4)		Low	-15%	$ 9,259,000
			High	25%	$ 13,615,000