APPENDIX

Appendix Contents

Appendix A Marietta

Appendix A-1 Marietta Geotechnical Report

GeoEngineers

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Memorandum

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Marietta Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Marietta reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at Marietta Site, located as shown in the Vicinity Map, Figure 1. The Marietta reservoir is a steel reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Bellingham Drift. Glacial outwash soils are mapped nearby. Although not mapped, the conditions encountered are representative of glacially consolidated soils.

Surface Conditions

The project site is located approximately 500 feet to the west of Wynn Road and 350 feet north of Marietta Avenue. The reservoir is located on top of a small hill and the site drops off in all directions. The site is bounded by residential houses in all directions. A small gravel roadway leads to the site from the south.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-10 (2019)—on March 25, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 40½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan, Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil units encountered at the project site is provided below. Our interpreted soil conditions are based on soil conditions encountered during our project specific geotechnical boring(s), review of any previously completed explorations (none available for this site), and our experience at nearby project sites.

- **Glacially Consolidated Soils** – Glacially consolidated soils were encountered at the surface of the exploration and extended to completion. The boring was completed at 40½ feet. The soil consists of very dense sand with variable silt and gravel content.

Groundwater

Groundwater seepage was not observed at final depth of boring. Glacially consolidated soils commonly have isolated saturated sandier zones or "pods" at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Very dense glacially consolidated soils were encountered at our boring location and extended to the full depth explored (40 feet bgs). We conclude that the reservoir foundation is founded on very dense soils.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic

scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on very dense glacially consolidated soils which are not at risk of liquefaction.

AWWA/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on publication D100-11 of the American Water Works Association (AWWA) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. AWWA AND ASCE 7-10 PARAMETERS

AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
AWWA Seismic Use Group	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	95.7
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.6
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.398
Seismic design value, S_{DS}	0.649
Seismic design value, S_{D1}	0.357

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the M 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	12	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location.

These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	18	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.36	0.65	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.20	0.36	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and our observations, we anticipate that the existing reservoir is bearing directly on glacially consolidated soils. We recommend that the structure be evaluated based on an allowable bearing pressure of 5,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral resistance is not necessary for an above grade steel reservoir.

Global Stability

Based on review of publicly available LiDAR for the site, there are slopes inclined at 40 percent or steeper to the north, east, and west that are approximately 30 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on very dense glacially consolidated soils. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

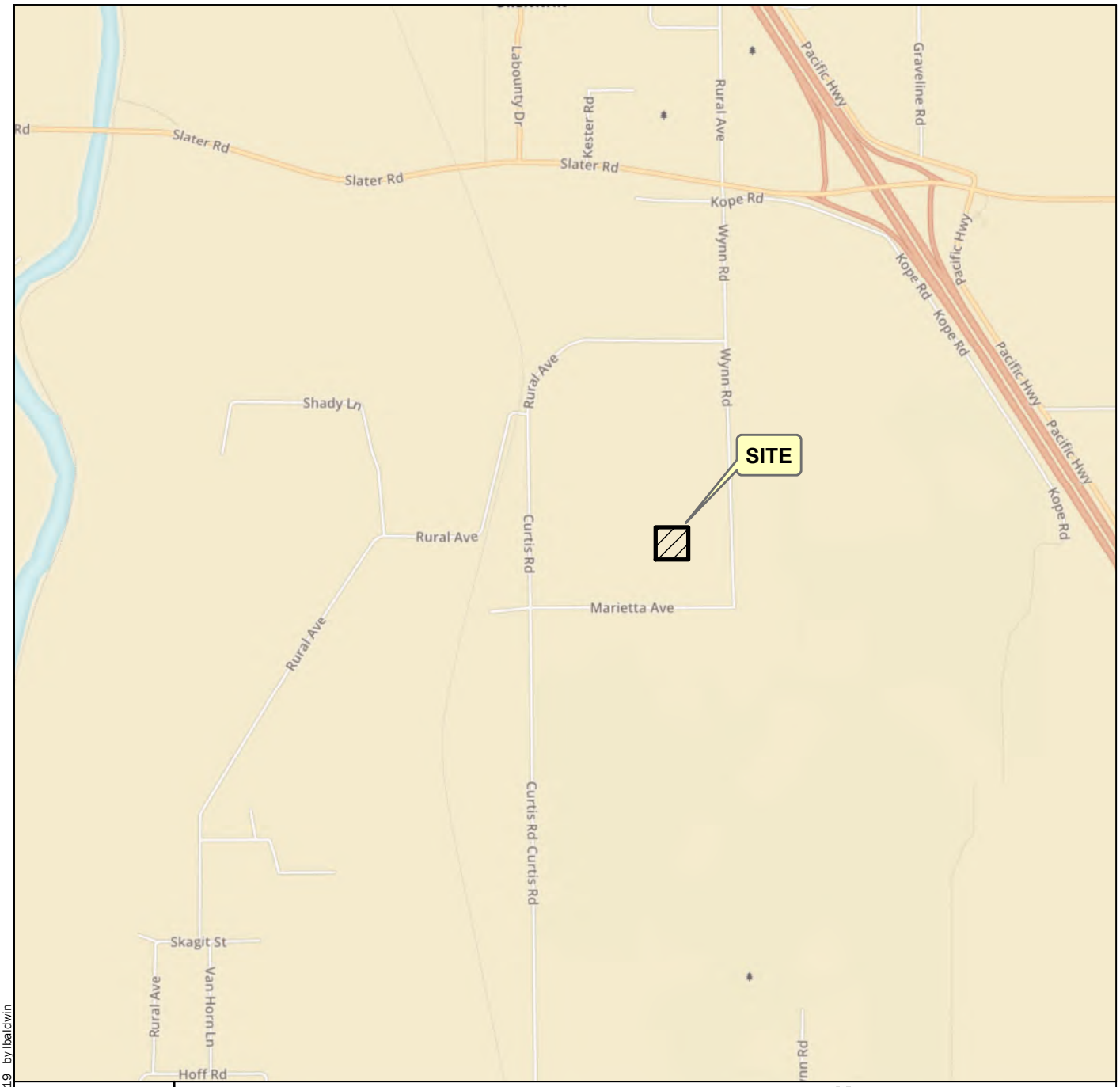
Figure 2 – Site Plan

Figure 3 – Key to Exploration Logs

Figure 4 – Log of boring B-10

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault



1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Marietta Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_Marietta.mxd Date Exported: 04/16/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

100 0 100

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Marietta Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled	Start 3/25/2019	End 3/25/2019	Total Depth (ft)	40.25	Logged By / Checked By	BWS / AJH	Driller	Boretec1, Inc.	Drilling Method	Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum	230 NAVD88				Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop			Drilling Equipment	EC-95
Easting (X) / Northing (Y)	1225100 663820				System Datum	WA State Plane North NAD83 (feet)			Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SP-SM	Gray-brown fine to coarse sand with silt, gravel and cobbles (very dense, moist) (glacially consolidated soils)			
		SPT-Auger		50/0"		1						
225	5		12	50/6"*		2						*Blow count overstated
			3	50/3"		3						
220	10			50/6"		4 %F		SM	Gray silty fine to medium sand (very dense, moist)	5	35	
215	15		6	83		5 MC				9		
210	20		3	50/6"		6 MC				9		
								SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
205	25		10	50/4"		7 %F				7	26	
								SP-SM	Brown fine to coarse sand with silt and gravel (very dense, moist)			
200	30		3	50/3"		8 %F				3	14	
								SM	Gray silty fine to medium sand with gravel (very dense, moist)			
195	35											

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-10

GeoEngineers

Project: COB Reservoir Inspection and Repair - Marietta Avenue
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		50/4"		9 %F				1	28	
190	40		50/1"		10						

Log of Boring B-10 (continued)

GeoEngineers

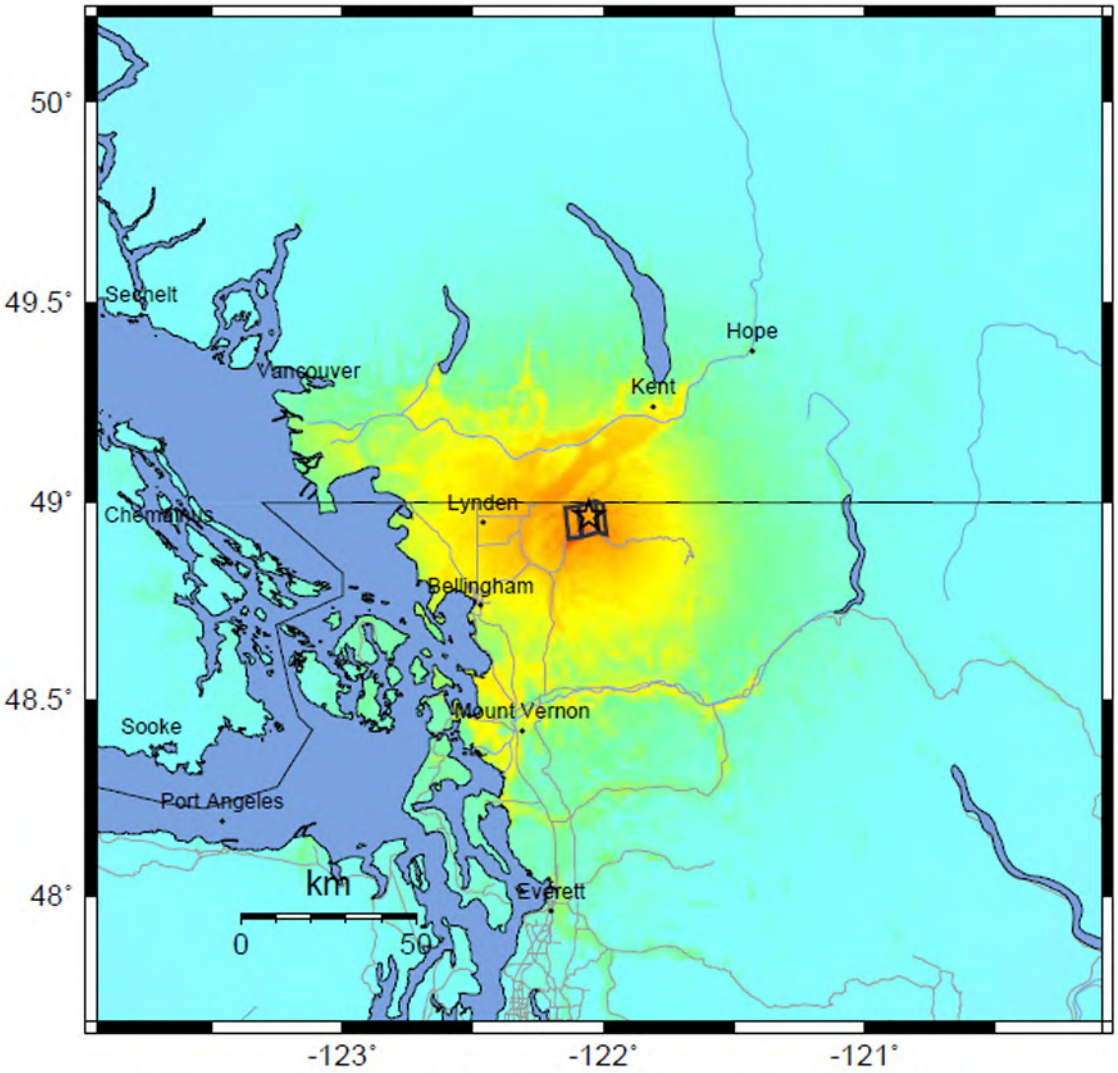
Project: COB Reservoir Inspection and Repair - Marietta Avenue
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 2 of 2

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

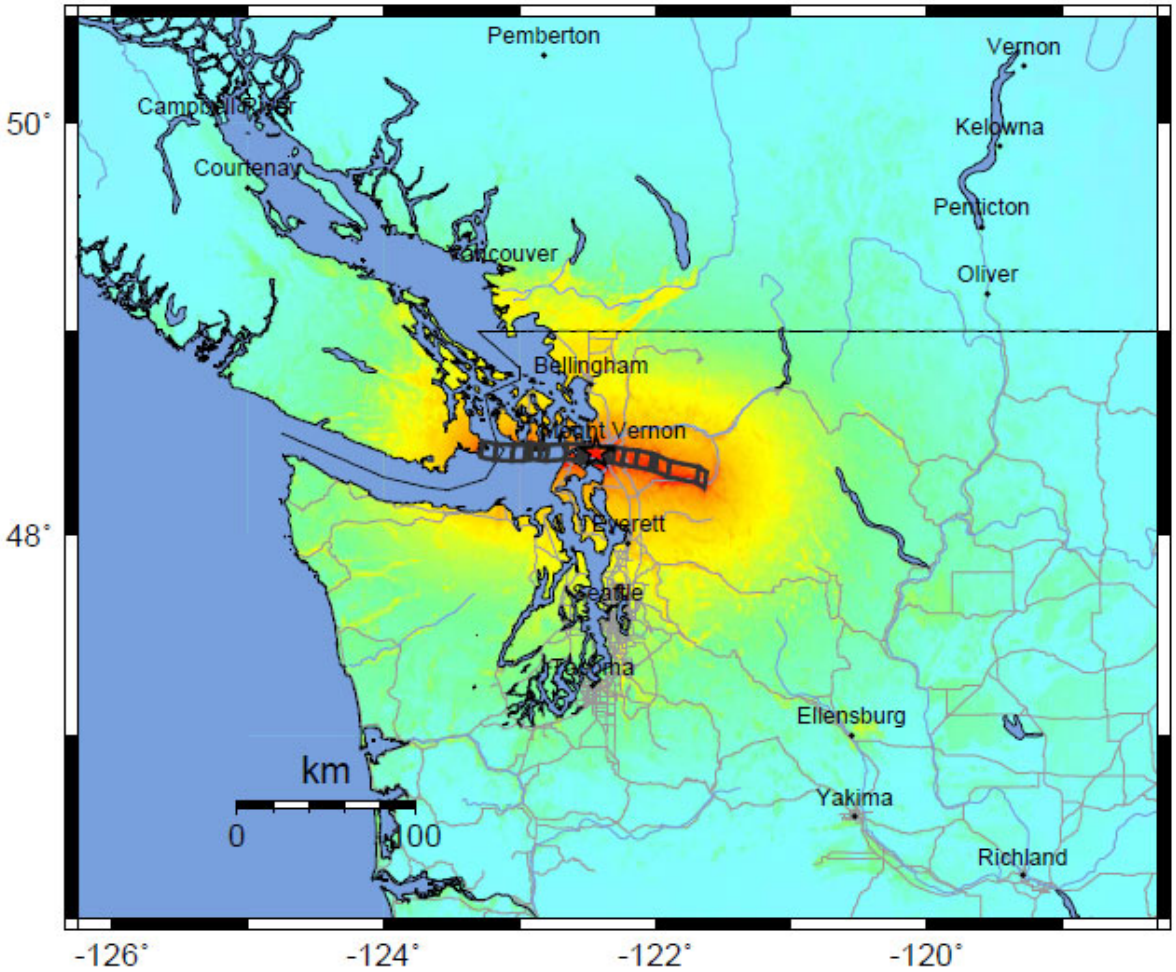
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix A-2 Marietta Corrosion and Coatings Report

February 14, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Marietta Tank Corrosion and Coatings Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an internal and external corrosion and coatings evaluation for the City of Bellingham's Marietta steel water storage tank. Specific tasks completed during this evaluation include:

1. Evaluate observed corrosion on the tank's steel surfaces.
2. Complete an assessment of both the interior and exterior coating.
3. Measure remaining steel wall thickness using ultrasonic thickness testing equipment at accessible locations. Thickness testing was completed on each shell course and the tank roof.
4. Measure the depth of any accessible noted pitting.
5. Evaluate coating losses and corrosion on visible surfaces.
6. Field test for presence of lead using lead-check swabs. Collection of external paint samples for leachable lead analysis (TCLP).

BACKGROUND INFORMATION

Marietta Tank

Height 50-ft tall, 100-ft diameter, welded steel constructed in approximately 1970. Interior equipment includes a PVC inlet piping supported using stainless steel straps connected to the column supports, and 1 center and 8 outer ring roof support columns. The tank does not have an internal ladder.

The interior coating appears to be the original coal tar enamel, the exterior surfaces were likely overcoated in 2013. An impressed current cathodic protection system comprised of mixed metal oxide wire anodes is supported from the roof.

COATING AND STEEL EVALUATION METHODS

A series of field tests were completed on the interior and exterior surfaces of the reservoirs during our site visit. A description of each test is provided below.

Dry Film Thickness

The thickness of the existing coating system was measured using a portable electromagnetic dry film thickness gauge (Type 2 gauge). This gauge measures the thickness of the coating between the gauge probe and the metallic substrate.

Steel Thickness

Steel wall thickness was measured using a General Electric model DM5E ultrasonic thickness gauge calibrated for carbon steel. When measuring steel thickness, this unit uses an echo-echo function that allows for measurement of the steel without removing the coating material.

Lead Testing

A field lead check swab was used to test for the presence of lead on the exterior prime coat. A sample of coating material was removed from the roof exterior and submitted to an analytical laboratory to test for the presence of leachable lead (Toxic Characteristic Leaching Procedure - TCLP). This test is conducted to determine if the coating material is classified as hazardous requiring specialized handling, containing, and disposing when removed from the reservoir.

Cathodic Protection System Evaluation

As the reservoir is empty, the impressed current cathodic protection system was turned off. Our inspection of this system was limited to a visual observation of the installed equipment.

TEST RESULTS AND ANALYSIS

Exterior Coating Thickness

The exterior coating of the tank appears to be a three coat system with a red primer, light blue intermediate coating, and light green topcoat. According to the Coating Specifications (Section 09870 – Coating System for Steel Tanks" issued for tank coating work in 2013, the tank was to have been overcoated with an acrylic polymer. The prime coating was noted to be a modified polyamidoamine epoxy. The presence of lead was noted in the specification.

The measured total thickness of the external coating ranged from 8 – 16 mils. Typical high performance coatings for the exterior surfaces of water storage tanks are on the order of 12 – 16 mils.

Exterior Observed Assessment

The exterior surfaces of the tank had only a few visible minor instances of small pits, all less than 1/32" deep. These defects appear to have originated during the fabrication or construction process as opposed to active corrosion.

The concrete ring wall is in generally good condition. There was a small amount of chipping noted and moss growth at several locations. The tank chime (steel lip in contact with ring wall) has not shown signs of corrosion loss. Coating defects on the edges of the chime have exposed the steel to the environment resulting in minor general surface corrosion and rust staining.

The visible exterior welds on the sheet plates and lap joints are sound with minor instance of weld splatter but do not display any signs of corrosion degradation.

Exterior Coating Assessment

The exterior sidewalls of the tank are dirty with streaks of organic matter at locations where water runs off the roof. There are isolated locations of top coat damage with the underlying prime coat visible. Most of the coating damage is limited to quarter-sized defects with one location that appears to have been cut with a knife. Coating damage near the bottom of the tank is likely due to rocks being through during mowing or brush cutting operations.

The top coat material is moderately tight to the surface. Using a sharp knife, only small pieces of coating could be removed when pressure was applied at failed top coat locations.

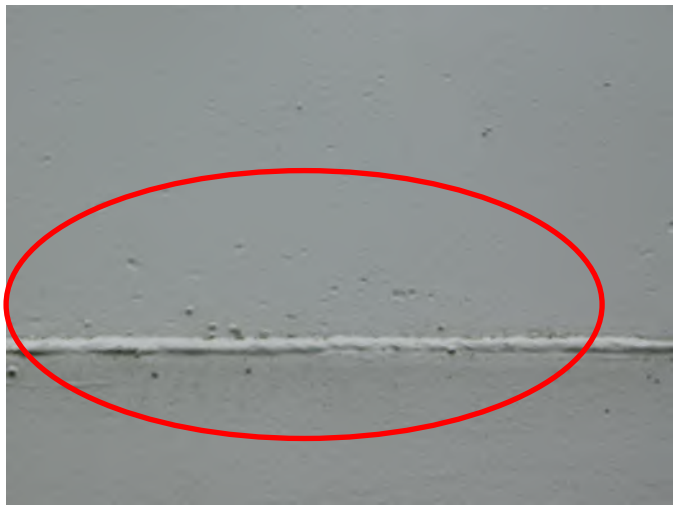
Marietta Tank, exterior surfaces are dirty with area of organic material accumulation

The roof coating is in very good condition. The coating is well adhered to the substrate and only one location of failure to the prime coat was noted near the roof access hatch. The configuration of the roof plates is such that at this location rainwater will accumulate creating a ponding condition where water can stagnate, weakening the coating.

The overlapping of the roof plates was done such that in many locations, the plate on the lower slope would be seal welded on top of the upper plate (see photograph). In this manner, water will collect in these areas and eventually resulting in coating damage. The most appropriate means for lessening the effect of this condition would be to have city personnel perform an annual inspection on the roof and complete any minor coating repair work at that time.

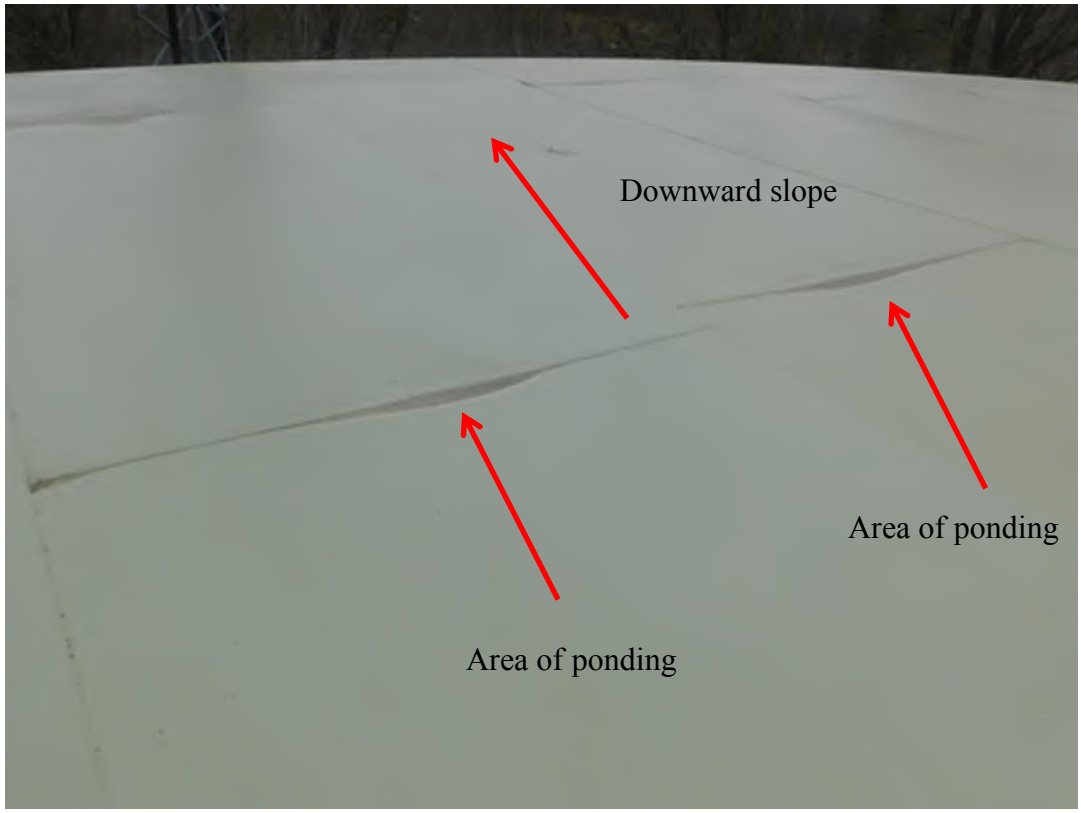
The roof vent and collar are constructed of aluminum. There is no evidence of corrosion or rust staining on this equipment.

Top coat damage on side wall

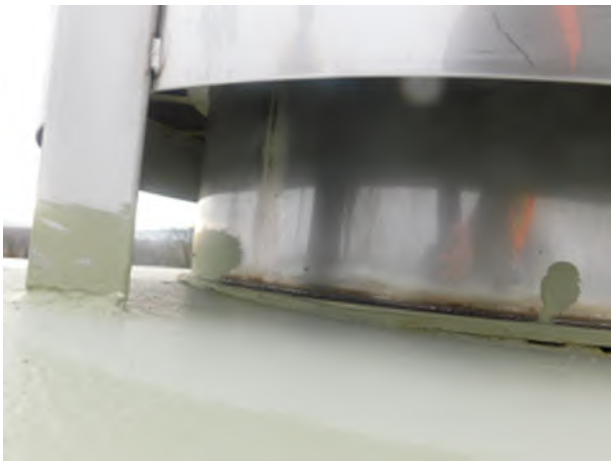


Small fabrication dimples and weld splatter

Heavy organic material growth



Overlapping of roof plates to allow for water ponding



Aluminum vent, no corrosion

Coating failure on roof near access hatch

Interior Observed Corrosion

A visual inspection was completed from the floor of the drained reservoir and from the roof access hatch. The roof and roof support members show areas of rust staining, particularly on the edges of the beams and rafters. However, when viewed from the roof access hatch, there was no observed significant metal deterioration. A more comprehensive evaluation of the roof surfaces can be conducted from a raft once the tank is filled. Aside from slight corrosion on the inlet piping, there was no corrosion observed on the interior below-water surfaces.

Overlapped roof plate, not seal welded

Painters rail, no corrosion on edges

Interior Coating Assessment

The interior coating consists of coal tar enamel ranging in thickness from 100 – 300 mils. It is likely that this material is the original coating applied during construction. The coating was mop applied to the sidewalls and portions of the floor were flood coating (coating poured on and then smoothed out). The coating is fairly brittle and cracks in places when walked on. At sound coating locations, the material continues to be tightly adhered to the walls, floor, and columns.

Overall, the interior coating is in very good condition and when coupled with the cathodic protection system, will provide a minimum of 10 years of additional service life.

Floor – flood coated, dirt covers much of floor

Floor – flood coated

Lead Test

The field lead-check swab showed positive results for the presence of lead in the prime coating material. A sample of coating was removed from the tank roof hatch area and submitted for TCLP (leachable lead) testing. This information will be necessary when a full exterior blast and recoat is performed. Results of the TCLP test will be appended to this report once they become available.

Ultrasonic Thickness Testing

The remaining steel wall thickness was measured using a General Electric model DM5E ultrasonic thickness tester calibrated for carbon steel. Thickness data was collected on each shell course and the roof plates. Results of the ultrasonic thickness tests are presented in Table 1.

Table 1: Marietta Steel Thickness Measurements

Location	Design Thickness, in.	Measured Thickness, in.
Bottom Course	0.688	0.663 – 0.667
2nd Course	0.568	0.558
3rd Course	0.469	0.443
4th Course	0.375	0.378
5th Course	0.250	0.240
6th Course	0.250	0.249
Roof	0.188	0.200 – 0.208

* The floor plate was not measured but design thickness was 0.250 inches.

A comparison of the individual course reservoir steel thickness collected during this survey and as part of work conducted in 2004 shows very similar results. Design thickness was not available. However, it does not appear that there has been appreciable steel deterioration at the measured locations.

Cathodic Protection Equipment

The transformer rectifier was turned off during our evaluation. However, it was noted that the unit is a constant voltage model. This type of rectifier will provide a constant voltage output regardless of the level of water within the tank. As the level of water decreases, protective current demands decrease (less submerged surface area). The amount of voltage necessary to provide the protective current also decreases in accordance with Ohm's law. However, with constant voltage rectifiers, the increased voltage gradients with a lowering of the water level have the potential to cause coating damage.

To accommodate for fluctuating water levels, an autopotential rectifier is normally used. This type of unit adjusts current output based upon water levels using an installed stationary

reference electrode. Replacement of the rectifier unit will not require replacement of the existing anodes which likely have several decades of remaining life.

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the reservoirs.

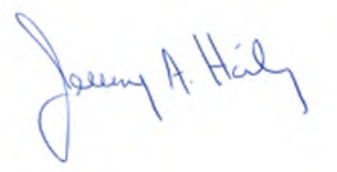
1. According the documents reviewed, the exterior surfaces of the tank were overcoated in 2013.
2. The exterior surface of the tank is dirty and has several areas of organic material accumulation on the roof and sidewalls.
3. There are isolated locations of top coat failure on the roof and sidewalls. There was no exposed steel observed.
4. The exterior welds are in good condition with observable weld splatter at many locations.
5. The interior coating below the water line is in very good condition. This portion of the interior coating has a minimum 10 years of remaining life.
6. Instance of rust staining and coating loss were observed on the roof and roof support members. However, there was no extensive corrosion damage.
7. The exterior prime coat tested positive for the presence of lead. Laboratory TCLP testing is currently being conducted on a collected paint sample and results will be appended to this report when available.
8. A majority of the steel plate thicknesses were consistent with design requirements.
9. The existing transformer rectifier is a constant voltage model. Rectifier units for water tanks should be autopotential in order to account for fluctuations in water level.

RECOMMENDATIONS

1. The exterior surfaces need to be pressure washed to remove dirt and other debris. This will help extend the life of the coating.
2. Repair the areas of top coat damage, particularly near the roof access hatch where water is able to pond. This will preserve the life of the prime coat material and keep the underlying steel from corroding.
3. City personnel should perform an annual visual inspection of the tank roof and make any minor coating repairs as required.
4. Install a stationary reference electrode in the water tank. Replace the existing constant voltage rectifier with an autopotential unit.

We appreciate the opportunity to work with you on this project. If you have any questions or would like assistance with the implementation of the report recommendations, please do not hesitate to contact our office.

Sincerely,
Northwest Corrosion Engineering



Jeremy A. Hailey, P.E.

Appendix A-3 Marietta Structural Report

CITY OF BELLINGHAM

CH 3: MARIETTA RESERVOIR

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

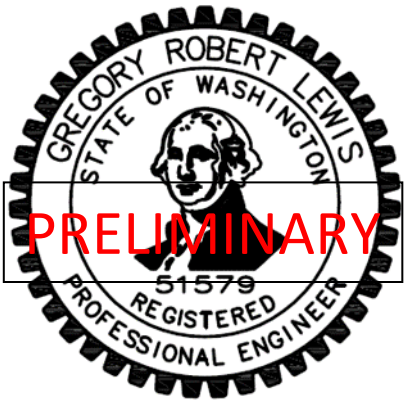
Table of Contents

1 Introduction

The following report has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Marietta, 3.0 Million Gallon (MG) steel reservoir. The reservoir is located near 1404 Marietta Ave, Bellingham, WA (Lat. 48.8071, Long. -122.5566), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to inspect and visually evaluate the reservoir on January 24th, 2017 by Peterson Structural Engineers (PSE), Murraysmith, Inc., and Northwest Corrosion. The reservoir has been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Marietta Steel Reservoir – 3.0 MG

2.1 Description & Background

The original reservoir was designed by American Pipe & Construction Co. – Northwest Division - of Portland, OR, and the original construction drawings provided have an issue date of October 1969. The identification plate on the tank indicates a construction year of 1969. The reservoir is a ground-supported welded steel reservoir with a 100-foot inside diameter. The reservoir shell plates consist of six courses and are a total height of 50'-3" (50-feet of shell plate with a 3-inch angle at the top). The overflow is located at the base of the angle and within the rafter line. The overflow is at a height of 50-feet which is 3-inches below the roof plate.

The reservoir roof is supported by a central 10-inch diameter column and 1 additional ring of column-supported girders which are located approximately 21'-10" out from the central column. There are a total of 8 additional 10-inch diameter columns supporting the secondary ring. The roof plate is approximately 0.204-inches thick and sloped at an angle of 0.75:12. The roof consists of 8-foot x 20-foot plates which are lapped and welded along their external seam. The interior roof was observed from the hatch. From this observation, it was noted that the plates were not welded along their interior seams where the roof plates were lapped. Further, there is no apparent connection (welded or otherwise) between the roof plates and rafters. Roof plates are welded at the shell-to-wall interface.

Per the original drawings the reservoir is shown supported on a 12-inch-wide by 4-foot deep footing. The drawings show the footing contains (8) #7 circumferential bars on each face with #4 ties at 1-foot on center. Outlet piping is shown to run 2-feet below the base of the footing rather than through the footing. However, measurements onsite found the exposed exterior portion of the footing to be 15.5-inches wide beyond the face of the tank shell. This measurement, by itself, is wider that the drawings indicate and assuming the anchors are centered on the footing, this would indicate the footing is actually closer to 30-inches wide. The shell is anchored to the footing using (33) 3/8-inch by 6-inch wide anchor straps. Measurements taken onsite indicated the visual portion of the strap assembly conforms to the drawing details.

2.1.1 Description of Additional Site Structures and Features

The site included two additional structures adjacent to the reservoir related to its operation. The first is a 10-foot by 10-foot CMU block building, with a wood-frame roof which is founded on a concrete slab. The CMU building houses electrical systems associated with the reservoir's corrosion protection system, but it is our understanding that system is not currently in use.

The second structure is a 12-foot by 7-foot below-grade concrete equipment vault. The vault is 7'-6" deep and houses the piping for the 16-inch diameter tank outlet and water main lines. Drawings provided indicate the vault was installed as part of tank retrofits around 2012 which included the installation of the new inlet piping system for the reservoir.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a single site visit to observe the as-built current condition of the reservoir. The site visit occurred while the reservoir was drained allowing PSE to evaluate the interior floor, manway hatch, and

the interior structural systems. It is our understanding that the reservoir had been drained and has been out of service for around five years due to low demand in the area. The site visit was performed on January 24th, 2019.

Steel Roof: The existing reservoir roof was found to be in generally good condition. Along the roof edge, there were noted to be areas between the rafters which had low spots which allowed for ponding. A few areas on the west and south side, where ponding was prevalent, were beginning to show signs of coating failure. Additionally, the roof plate lapping was done in such a way that waterflow off of the roof was hampered and which allowed for small areas of ponding. These instances were noted throughout the roof surface but did not appear to be associated with any coating failures. Overall areas of staining, due to water or instances of corrosion, were noted to be minimal.

The roof hatch opening was 2-feet by 2-feet and was generally clear of notable corrosion. The body consisted of a welded box section which is welded to the tank roof where it penetrates the roof plates. The hatch itself is a standard shoe-box style with a lock and foam gasket. Minimal loss of paint and staining was noted around the hatch and roof interface. No interior ladder system was installed at the hatch location which is atypical of most reservoirs PSE has observed. However, an inside tank ladder is not required by the associated design standard (AWWA D100-11 Section 5.4.2.6)

Reservoir Floor and Walls: The interior walls and floor were covered in a thick (approx. ¼-inch) layer of coal-tar coating. While PSE generally checks for the impact corrosion has had on steel structural members, this layer obscured the steel from view. However, the coal-tar layer appeared to be intact with very few areas of cracking or delamination. As a result, while the steel could not be observed directly, it is unlikely that there are any major areas of corrosion due to the intactness of the coal-tar layer. The underside of roof plates and the roof plate/rafter contact zone were not seal welded which provides locations where corrosion can occur. In the roof and rafter zone the coal-tar coating was much thinner and evidence of corrosion staining was noted. However, and likely helped by the reservoir being out of operation for years, the corrosion noted appeared to be generally surface corrosion. In the areas observed from the hatch, no visible section loss of roof members was observed.

In addition to the coal-tar coating the City maintains a corrosion protection (CP) system within the reservoir. While it is unclear the overall effectiveness of the CP system the combination of the CP system, coal-tar, and being empty has helped limit any observable issues and generally the reservoir interior was noted to be in good condition. Should the tank be put back into service, the areas that appear to be more susceptible to corrosion issues should be periodically monitored. These areas include the roof and rafter system as signs of corrosion are already present. Additionally, the columns, where the new inlet pipe has been installed, should also be periodically monitored. Corrosion at the inlet pipe anchor points appears to be due to the bracket system which looks to have compromised the coal-tar layer by “crushing” the stiff coating. This has resulted in the coal-tar delaminating around the attachment points and exposing the underlying steel.

The exterior walls of the reservoir were noted to be in generally good condition with minor chipping and pock-marking in the coating. It was noted that along the north side, staining due to roof run-off was

evident as vertical green striations. While these areas should be cleaned as part of standard upkeep, they did not appear to currently have any impact on the underlying coating or wall. Further, the external appurtenances such as the ladder and manways were found to be in good condition.

Foundation and Site: The reservoir is supported on a concrete ring foundation, which was found to be in good condition with no settlement issues or major cracking visibly apparent. Strap anchors are located at about 9'-6" on center to anchor the reservoir. The concrete around the straps appeared to be competent. Per our assessment it was noted that there was no grout layer between the shell and top of concrete ring foundation. Per the relevant design standard (AWWA D100-11, Section 3.8.5.2(6)), when anchors are used, a 1-inch thick grouted layer should be placed between the base of the reservoir and top of concrete foundation.

Generally, the overall site appeared to be well graded but standing water was noted around the base of the foundation on the northwest side of the reservoir. Prior to the day of the inspection, there had been a rain event. However, additional grading could be used to better direct water runoff away from the foundation. The top of the reservoir's concrete foundation averaged approximately 6-inches above adjacent grade around the majority of the reservoir. This is in conformance with the applicable design standards (AWWA D100-11 Section 12.7.1). This height is recommended to keep materials from collecting along the base of the shell.

On the west side, where the foundation height differential was at its lowest and where the standing water was noted, grass clippings and other debris were noted to be building up along the base of the shell. These debris should be removed as part of the site maintenance activities. The site itself is bounded by a fence which results in a cleared space of approximately 10 to 15-feet around the reservoir. This zone has been cleared of trees and brush. Outside of the fence line, some larger trees were noted to be growing in relatively close proximity to the reservoir. Tree shading can slow water evaporation and increase contact time between water and steel resulting in an increased corrosion potential. Trees and/or branches can also come loose during a storm event and cause impact damage to the roof and shell. For these reasons, trees should be cut back away from the reservoir.

2.2.1 Visual Condition of Additional Site Structures and Features

The additional structures on the site were noted to be in generally good condition. Neither structure was observed to have any observed visible signs of structural failures, settlement issues, or major areas of corrosion or decay during our site inspection.

2.3 Structural Analysis

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoirs under the current applicable Codes and standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code "Welded Carbon Steel Tanks for Water Storage", AWWA D100-11 was utilized. The evaluation was based on original construction documents and site visit observations.

Per the City, the average operating level was between 45 and 50-feet when the reservoir was in service. In order to develop recommendations pertinent to the typical operating ranges, the analysis of the reservoir and the following discussion is based on a maximum operating level of 50-feet, where the top of the overflow is located. Per discussion with the City, this reservoir's operating height, when in service, is often up to the overflow level and so using the maximum operating height as the analysis and design maximum appears to be appropriate. This operating level is 3-inches below the roof plate and at the top of wall. PSE also evaluated the reservoir at 45.5-feet, the point at which roof slosh is alleviated.

Reservoir Shell – Material Thickness: While the reservoir drawings detail the shell thickness, the actual measured shell thicknesses are used for PSE's analysis. As multiple measurements are taken, PSE will typically utilize the average thickness. The exception is in instances where a shell course has been measured in multiple locations and any plates are found to deviate from the average by greater than 5%. In such a case, PSE will use the lower bound value as the controlling steel thickness for that shell course.

Reservoir Steel Thicknesses				
Component	Drawing (in)	Average (in)	Minimum (in)	Value Used (in)
Shell Course 1	0.6875	0.665	0.663	0.665
Shell Course 2	0.5687	0.558	0.558	0.558
Shell Course 3	0.4688	0.443	0.443	0.443
Shell Course 4	0.375	0.378	0.378	0.378
Shell Course 5	0.25	0.240	0.240	0.240
Shell Course 6	0.25	0.249	0.249	0.249
Floor Plate	0.25	N/A	N/A	0.250
Roof Plate	0.1875	0.204	0.200	0.204

2.3.1 Hydrostatic and Gravity Analysis

Roof Framing: The roof plate thicknesses and layout are adequate to meet current code requirements for the required live and snow loads. While the rafters are not attached to the roof plate, D100 allows for a rafter with a depth less than 15-inches to be evaluated as continuously braced. When this allowance is taken into effect, all rafters passed design checks. The interior 10-inch diameter center column and 10-inch diameter exterior columns, supporting the girders upon which the rafters bear, were found to be sized appropriately per current code. The girders themselves were found to be sized correctly for a braced condition. However, unlike the rafters, the D100 assumed braced condition is not applicable and physical restraints are required. As shown in the drawing and observed from the hatch, the bearing seats present provide the necessary restraint and the braced condition assumption is acceptable. Overall the roof structure appears to be in good condition.

Reservoir Shell – Hydrostatic Stress: A review of the drawings found General Notes on Drawing No. D-7066 in which it was written "Const. per AWWA D100-67 & Cust. Spec. Appendix C". This indicates the

design was conducted per Appendix C which provides an alternative design methodology compared to the typical Section 3 design. Appendix C of the 1967 edition of D100-67 corresponds to the modern D100-11 Section 14. This section allows for the design of reservoirs with higher tensile strength materials. The use of these materials are limited by requirements for additional testing and temperature limits. Per AWWA D100 Figure 23 the lowest one-day mean temperature for this area is 0°F. Comparing this value to the tables, ASTM A36 Steel (AWWA allowable tensile stress of 19,330psi) is acceptable for areas with an allowable temperature low of 0°F and a steel thickness up to 1-inch.

Appendix C/Section 14 also requires additional special inspection of the welds versus those outlined in Section 3. This additional special inspection allows for an increased efficiency factor of 100% to be used in design rather than Section 3's 85% efficiency factor. Per Drawing No. D-7063 the shop X-ray plan, to be performed in conformance with Appendix C, is outlined on the *Shell Layout From Inside* detail.

As a back-check, values from Appendix C/Section 14 are utilized at an operating height of 50-feet (as the initial design should always be for the overflow level), using the assumed tensile strength based on values provided in Section 14 and the assumed efficiency increase allowed when the special inspection of welds are implemented per Section 14, these values can be incorporated into the following shell thickness equation to provide a pseudo-verification of the steel type likely used in the design:

$$Shell\ Thickness = \frac{2.6(Operating\ Height)(Diameter)}{(Plate\ Tensile\ Stress)(Efficiency)} = \frac{2.6 \times 50ft \times 100ft}{19330psi \times 1.0} = 0.673\ inch$$

This result matches with the first course shell plate thickness which is listed as 11/16 or 0.6875-inch on the drawings. Therefore, an A36 or a similar grade steel, is assumed to be the probable grade of steel that was used in the fabrication.

Using these results, and the assumed steel grade noted, the reservoir plates would be acceptable for the anticipated hydrostatic stresses, if the thickness were as shown on the drawings. However, the in-situ measured plate thicknesses found that plates averaged about 10 mil below the required design values. Due to this, the plates were found to be between 1 to 3% overstressed at the overflow operating level. While this overstress is relatively small it is indicative of a reservoir without much built-in reserve capacity.

Foundations: Per the Geotech report, the allowable soil bearing capacity for the site 5,000-psf. For the reservoir the calculated gravity load bearing pressure was determined to be 7,800-psf for a 12-inch wide footing, as is shown in the drawings. This bearing pressure exceeds the allowable. However, based off the 15.5-inch field measurement and assuming the shell is approximately centered on the foundation, the footing is likely closer to 30-inches wide. Taking this larger footing into account reduces the gravity load bearing pressure to 3,100-psf, which is an acceptable bearing pressure.

2.3.2 Hydrodynamic and Seismic Analysis

Reservoir Shell – Hydrodynamic Stress: As noted in the hydrostatic section, analysis of the shell was conducted following the methods allowed in AWWA D100 Section 14. See the previous hydrostatic section for additional notes. When utilizing these assumptions, the hydrostatic stress values were found to be 1 to 3% greater than allowable when using the as-measured steel thicknesses as noted above. For design,

when evaluating for the seismic hydrodynamic stresses case, the allowable shell design tensile stress can be increased by a third for short-duration loads. Taking this allowable tensile stress increase into account, only shell courses 5 was found to have issues with hydrodynamic loads. Evaluated at the 50-foot overflow, shell course 5's capacity was exceeded by 1%. While the reservoir is overstressed at the 50-foot operating level, in general, this tensile stress overage is relatively small.

Freeboard/Slosh: The AWWA describes the freeboard height as the distance between the top of the overflow and base of the rafters. For analysis, PSE typically utilizes a freeboard based on the distance between the top maximum operating level and the base of the rafters. Per the City, this reservoir's maximum operating level and overflow are at the same elevation. As the overflow extends into the rafter line this, technically, results in a negative freeboard condition. During a seismic event, operating at overflow would result in a slosh wave that is nearly completely constrained. For a constrained slosh wave, the wave generated by the ground motion is forced up into the roof. This slosh impact wave can damage the roof and rafters and dislodge roof plates. Additionally, constraining the slosh wave increases the total overturning force as the wave motion is restrained and forced up into the roof, increasing the potential for overturning. For this reservoir, the slosh wave height was determined to be 4.5-feet. This height is a function of the geometry of the reservoir and local seismic conditions. As a result, the reservoir would need to have its operating height reduced to 44.5-feet to meet freeboard requirements per Code and alleviate all slosh related loads on the roof. This level is based on a 50-foot overflow level less the slosh and rafter depth (4.5-feet + 1-foot = 5.5-feet) as freeboard is taken to the base of the rafters.

Should a higher operating level be required, the roof would need to be retrofitted to resist slosh impact and uplift loads. While the roof plates and rafters are adequate for gravity loads, they do not have the capacity to resist the anticipated slosh loads. To resist these loads the roof plates would need to be welded to the rafters to provide a load path between the plates and rafter elements. A slosh wave resulting from a 50-foot operating level would exceed the roof plate and rafter's capacities if they were not either replaced or heavily retrofitted and reinforced. At lower operating levels, the roof zone effected by the slosh wave is reduced and there is potential to identify an intermediate operating level which minimizes retrofit requirements. In this case, it appears that an operating level of 45.5-feet would prevent slosh from impacting the roof plates which would have the larger destabilizing effect. While the slosh wave would still impact on the rafters it's impact would be greatly reduced and the existing rafter configuration would have the necessary capacity need to resist the slosh loads.

Due to the lapped-plate configuration of the roof, a pre-evaluation of the plate joints should be conducted prior to any welding or seal-welding of the roof to increase their capacity to resist slosh loads. This would be to determine if welding is possible or if the lap joints are potentially corroded. While a full seal weld of the rafters might be preferred, crevasse corrosion resulting from the lap might restrict this or limit this option. In such cases, skip welding can be used to meet structural requirements. While these retrofits would likely be the simplest, if they are unable to be performed due to corrosion, more involved retrofit methods could be considered to stabilize the roof members against a slosh wave.

Overturning and Anchorage: The existing reservoir is of a configuration that is typically considered a non-standpipe reservoir (i.e. a ground supported reservoir). In this configuration the diameter of the reservoir

exceeds its operating height by such a margin that it is less likely to require anchorage to resist a seismic event. However, due to the high local seismicity, the reservoir has been installed with anchor straps.

Additional updates to the seismic code and new information on the effects of overturning due to slosh have resulting in increased overturning effects due to a seismic event. In this case, at a 50-foot operating level, the constrained slosh results in a 25% increase to the base overturning load. As a result, the existing anchor strap's capacity are exceeded by 2.5 times their design capacity.

Overturning and bearing on the foundation are also increased when considering constrained slosh and the soil bearing pressure, at a 50-foot operating level, is 20,000 psf (for a 12-inch wide footing), when constrained slosh is accounted for versus 16,000 psf, when it is not. As noted previously, this foundation is likely 30-inches wide rather than the 12-inches shown in the available drawings. For the 30-inch wide foundation these bearing values decrease to 8,000-psf and 6,400-psf, respectively. Per the geotechnical analysis, the bearing pressure resulting from overturning due to constrained slosh would exceed the allowable pressure by 18%. To limit the adverse effects of slosh induced overturning would require a reduction in operating height. At a height of 45.5-feet, for the 30-inch wide foundation assumption, this bearing pressure is reduced to within acceptable limits for both static and seismic loads.

Per current Code, anchors also need to be ductile relative to their connections (i.e. the main steel section should yield in a ductile manner prior to a non-ductile concrete breakout failure or welded attachment rupture). Per PSE's analysis, loads to the anchors can be significantly reduced by reducing the operating level and PSE checked the anchors at 45.5-feet. However, at 45.5-feet the anchors would still require modification in order to meet ductility requirements. For strap anchors this could be achieved by notching the anchors to effectively reduce their capacity below that of the associated concrete breakout or weld rupture. Unfortunately notching the straps would require additional straps be added to make-up for the overall reduced capacity. Alternatively, when the reservoir is operated at a 45.5-foot operating level, the strap anchors are not even required to resist uplift as the reservoir is considered to be self-anchored. Rather than upgrading the footing, PSE would recommend reducing the operating level to 45.5-feet and then detaching and removing the anchors from the reservoir.

Should the anchors be removed, the design vertical displacements would remain the same while horizontal design displacement requires as outlined in ASCE 7-10 Table 15.7-1 would increase from 0.5 to 2-inches. Increased horizontal movement would require flexible pipe fittings to accommodate the minimum code listed displacements. However, if anchors are kept, they must meet requirements for capacity and ductility. This means the inclusion of additional anchors and as they would still undergo an uplift load, any new and existing anchors would need to conform to ACI ductility requirements.

2.4 Summary

The reservoir appears to be in generally good condition for static and dynamic operational loadings. Typically, a primary driver of structural failure during normal operations is unchecked corrosion. The competency of the coal-tar coating and the fact the reservoir is currently out of operation has prevented corrosion from propagating. However, it was noted that some of the roof elements did show signs of past corrosion and these areas should be recoated and periodically monitored should the reservoir be put back

into service. Although PSE has made some assumptions in our analysis, these assumptions seem reasonable and depict a reservoir whose structure is largely adequate for gravity and hydrostatic loads. Alternately the reservoir appears to be under-designed for current seismic loads. This is exacerbated by an exceptionally high overflow and operating level. Operating at a 50-foot overflow, the roof, foundation, and anchorage would all be susceptible to failure during a seismic event.

Based upon the evaluation and aforementioned conditions, retrofit of the reservoir to function at the 50-foot operating level would be extensive. PSE would recommend a reduction in the maximum operating level in order to bring this reservoir into compliance with current codes.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to potentially bring the reservoir into partial or substantial compliance with current code.

Reduced Storage Volume

The first option is to perform no structural upgrades and lower the operating level to 45.5-feet so that slosh effects, foundation loads, anchorage requirements, and shell issues are all addressed by reducing the operating level and therefore the applied loads. As noted previously, the anchor straps would need to be removed from the reservoir to prevent damage to either the reservoir shell or foundation in the event of an earthquake. Additionally, the foundation should be evaluated to verify it has a minimum width of 30-inches.

Maintain Maximum Storage Volume

The second option is to continue to operate the reservoir at an operating height of 50-feet. In such a case PSE would recommend a more comprehensive evaluation of the roof to determine if the roof can be reinforced and seal welded. This would involve an evaluation of the roof where plates are lapped to determine if there is any concern regarding corrosion. If the roof is determined to be competent, the plates can be seal welded as well as welded to the rafters to ensure a positive connection again slosh-induced uplift. This would require a full roof load analysis as the entire roof support structure, from roof plates to column bases, would need to be reinforced. These retrofits would help to reinforce the roof against any slosh loads. Note, this may not be feasible, or it may require uneconomical upgrades.

Once the roof is reinforced, the foundation would need to be retrofitted with an expanded footing and new anchor bolts (rather than straps) and anchor bolt chairs. For the anticipated loads a pile system (helicals or micropiles), tied into a new expanded ring foundation, may be the best option to limit sitework and construction under the reservoir.

Intermediate Storage Volume

The third option is to operate at an intermediate level to limit slosh impact loads on the roof and lower the associated overturning and bearing loads. Alternately, the roof could also be raised, and new shell course installed so as to alleviate the potential for constrained slosh. However, based on the analysis

results, even if constrained slosh is eliminated, PSE still anticipates that retrofit work will be required to strengthen the foundation and anchorage if the reservoir is to be operated above a 46.5-foot elevation.

General Site Recommendations

For the overall site, PSE recommend trimming trees around the reservoir to reduce the potential for damage from trees during storm events. Trimming trees will also increase air movement around the reservoir and to reduced conditions which are deleterious to the coating. Where standing water has the potential to occur, the site should be regraded to direct surface water way from the footing or something similar to a French drain or drainage trench may be installed.

Ancillary structures observed on the site appeared to be in generally good condition. Further their proximity to the reservoir was not such that they could damage or weaken the structure should they fail. No repairs are proposed for these structures beyond typical maintenance.

2.6 Scans of Select Construction Documents

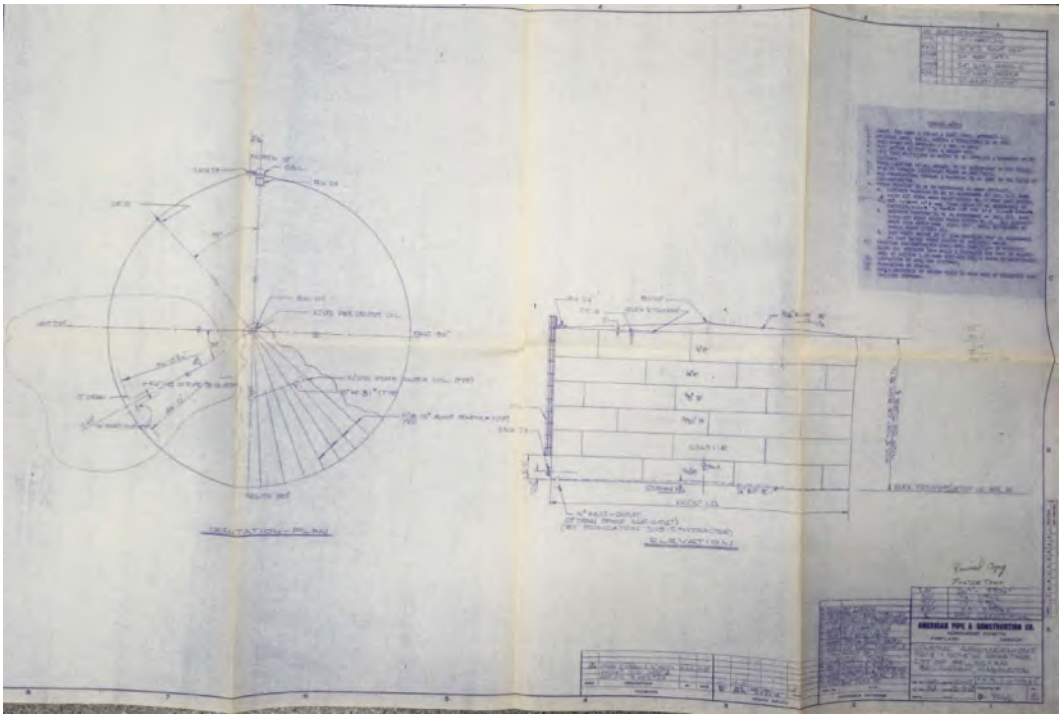


Figure 2-1: Plan and Elevation Views

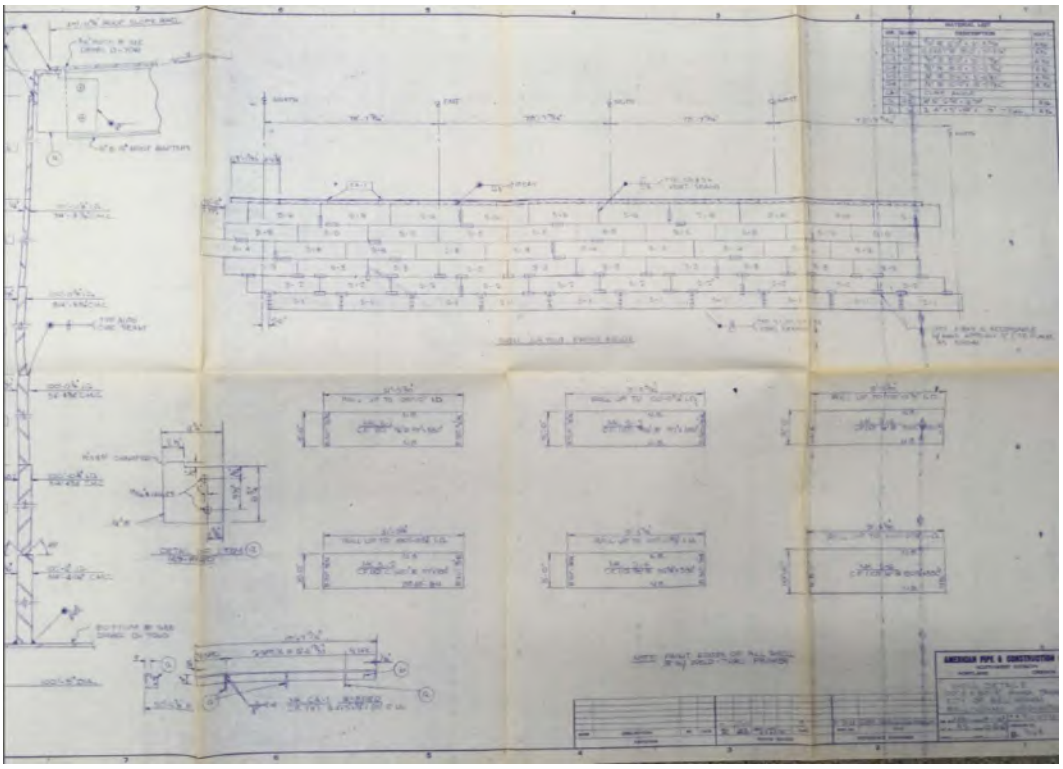


Figure 2-2: Wall Plate Layout and Wall Elevation

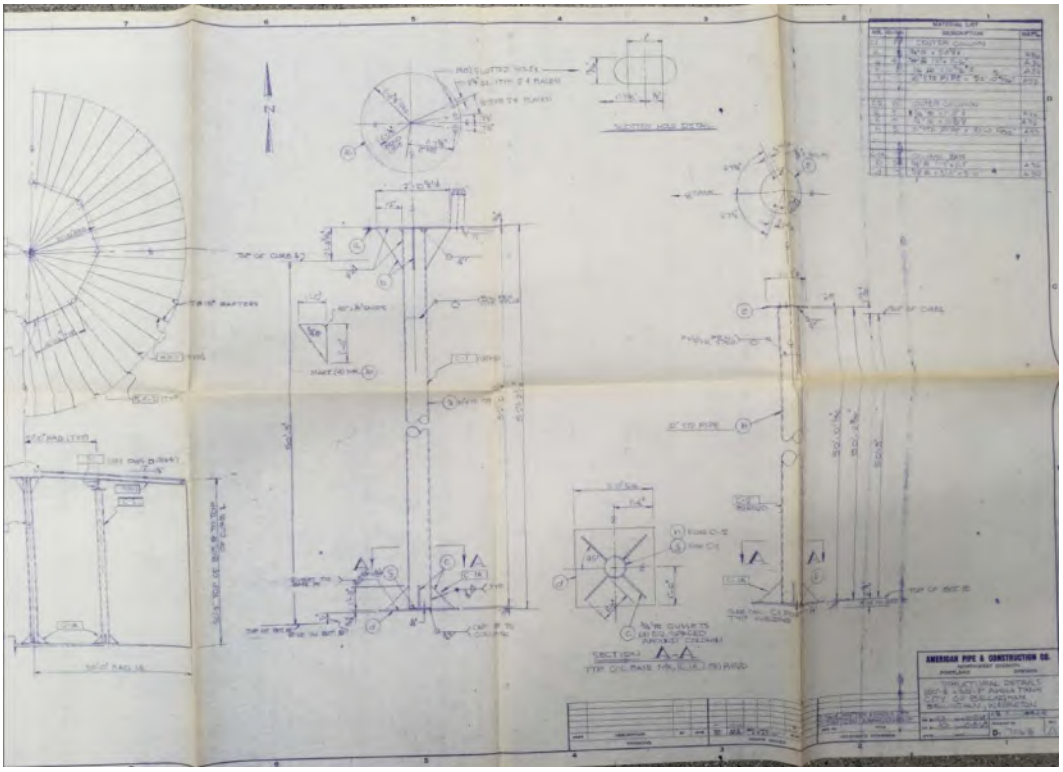


Figure 2-3: Roof Section and Column Assembly

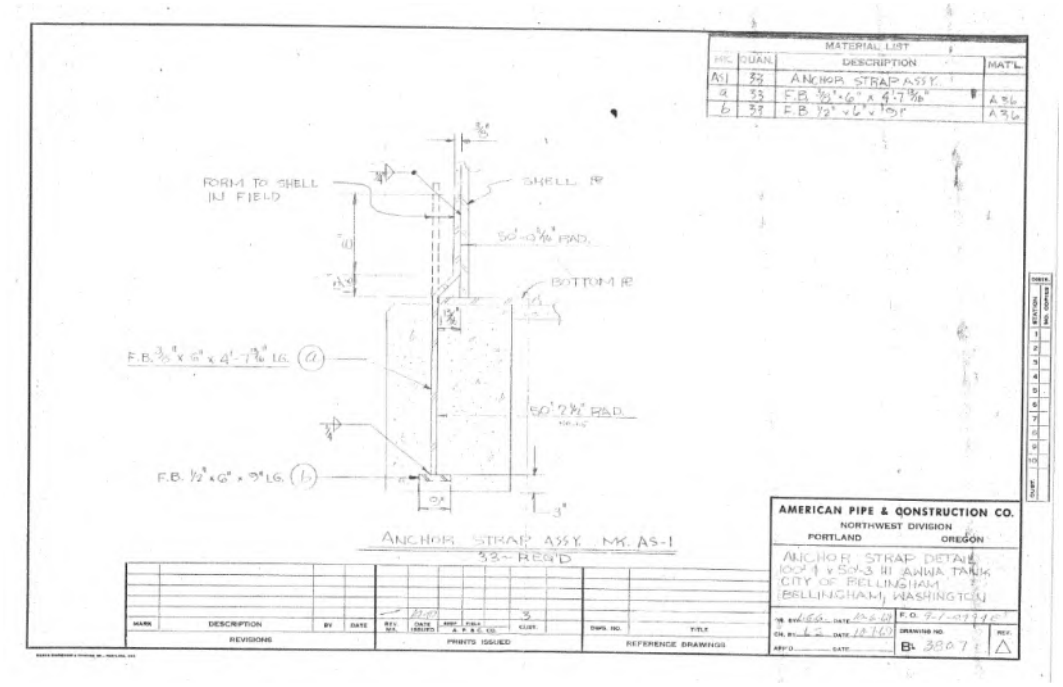


Figure 2-4: Anchor Strap Assembly

2.7 Observations Pictures

Figure 2-5:: Marietta Reservoir – Elevation

Figure 2-6:: Marietta Reservoir – 10x10 CMU Building Elevation

Figure 2-7: Marietta Reservoir – Piping Vault

Figure 2-8: Marietta Reservoir – Access Ladder and Wall Staining

Figure 2-9: Marietta Reservoir – Footing, Strap Anchor, and base of shell

Figure 2-10: Marietta Reservoir – Roof

Figure 2-11: Marietta Reservoir – Roof - Areas of ponding at edge depressions

Figure 2-12: Marietta Reservoir – Roof - Areas of water retention due to plate lapping

Figure 2-13: Marietta Reservoir – Roof hatch

Figure 2-14:Marietta Reservoir – Rafter-to-roof edge connection

Figure 2-15: Marietta Reservoir – Overflow Pipe to outside

Figure 2-16: Marietta Reservoir – Column and rafter support girders

Figure 2-17:Marietta Reservoir – Column base plate

Figure 2-18: Marietta Reservoir – Failure of coal-tar coating on column due to inlet pipe attachment

Figure 2-19: Marietta Reservoir – Manway opening

Figure 2-20: Marietta Reservoir – Pipe floor penetration – outlet pipe

Figure 2-21: Marietta Reservoir – Exterior shell – Non-smooth well line and pockmarking on plate surface

Figure 2-22: Marietta Reservoir – Standing water along foundation edge

2.8 Field Notes

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Built: 1969
Overflow: 50'
Typ. Operation: 45' - 50'

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: 1802-0019 Project Name: Marietta 3.0 MG

Site Visit Date: 1/24/19 Reservoir Type: Steel

Site Conditions: Overcast, cloudy wet ground

1404 Marietta Ave

Exterior Inspection 50' wall height

Number of Steel Shell Courses: 6 Standard Height of Shell Courses: 8'-0" (top 10'-0")

Shell Course Height: (top) 10', 8', 8', 8', 8', 8', ____ (bottom)

Knuckle yes (no) Radius N/A Thickness N/A

Shell Course Thicknesses: (top) 11/16, 0.5687, 15/32, 3/8, 1/4, 1/4, ____ (bot)

Condition of Exterior Shell and Coatings (check for location of paint delamination and rust):
Gen good, north side shedded an has staining evident, incidental coating loss from rock/branch impact. Weld lines not smooth in most locations and appear rough

List location of all external items (pipes, manways, ladders, etc.) on drawing page. Locate items using pace count method.

Condition of Ladder/Vents/Hatch/Welds: Ladder (External) Good condition w/ 7 attachment points. Hatch, 1 hatch 24" diameter w/ 4 lip good cond. with staining. Welds most are fairly rough but no corrosion. Ladder

Manway Dia: 24", Ladder Dimensions 16" wide 1" rung at 12" o.c.

Question:
- ☐ Operating range
- ☐ Roof size
- ☐ No ladder by hatch, will need witch for entry

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

Exposure and Condition of Foundation: Exp. agg finish, minor cracking but no differential ← 15½" → exposed portion of footing wider than drawing

No Grout Layer → Footing

Grade Relative to top of foundation 4-5 on North side remaining closer to 6" +, List max/min on drawing page.

Pothole at footing. Depth of Footing _____ Dist from anchor CL to footing edge

Top Surface Roof Plates & Coating Condition (check for paint declamation and rust):

Gen good, min areas of delam, plate lapping is such that flow off roof is inhibited - many areas where water retained, waviness in surface due to no seal weld, areas of larger water detainment at outer edge where dips between rafters occur.

Thickness of roof plates 3/16, Slope of Roof* 3/4:12

0.200-0.208 measured in-situ

Interior Inspection

Tank Diameter 100', Distance from Wall to Mid-span Rafter Support ≅ 30'-0" at mid-span of girder

Column Diameter/Size: 10" STD Pipe

Interior Footing: None

Column Spacing/Configuration: Ext. column 16' from each other. Crc column to ext 20.8'. Ext. to wall 28.2'. 1 int and 8 ext column

Condition of Interior Shell and Coatings: Good w/ coal tar coating

Bottom of Roof Plate/Framing/Coatings Condition: Good w/ coal tar coating in tact

2

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WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

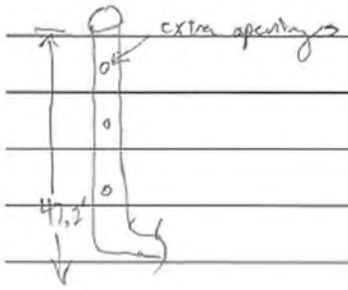
Floor Plate Condition/Thickness/Coatings: 1/4" Floor PL per dwg, viewed for exterior. Interior good coal tar coating

Ladder/Pipes/Overflow Conditions: No ladder. Outlet good cond. Overflow is a PVC pipe, 12" dia with overflow near center

Diameter of Inlet ~14.5", Outlet ~8", Overflow 12"

Distance of Top of Overflow from base of roof elements 3" below roof

Other Comments: Inlet runs up ext. column and have 3-holes in pipe that allow circulation



Outlet is near rod plate & does not have a line, vents through mesh onto ground.

3

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

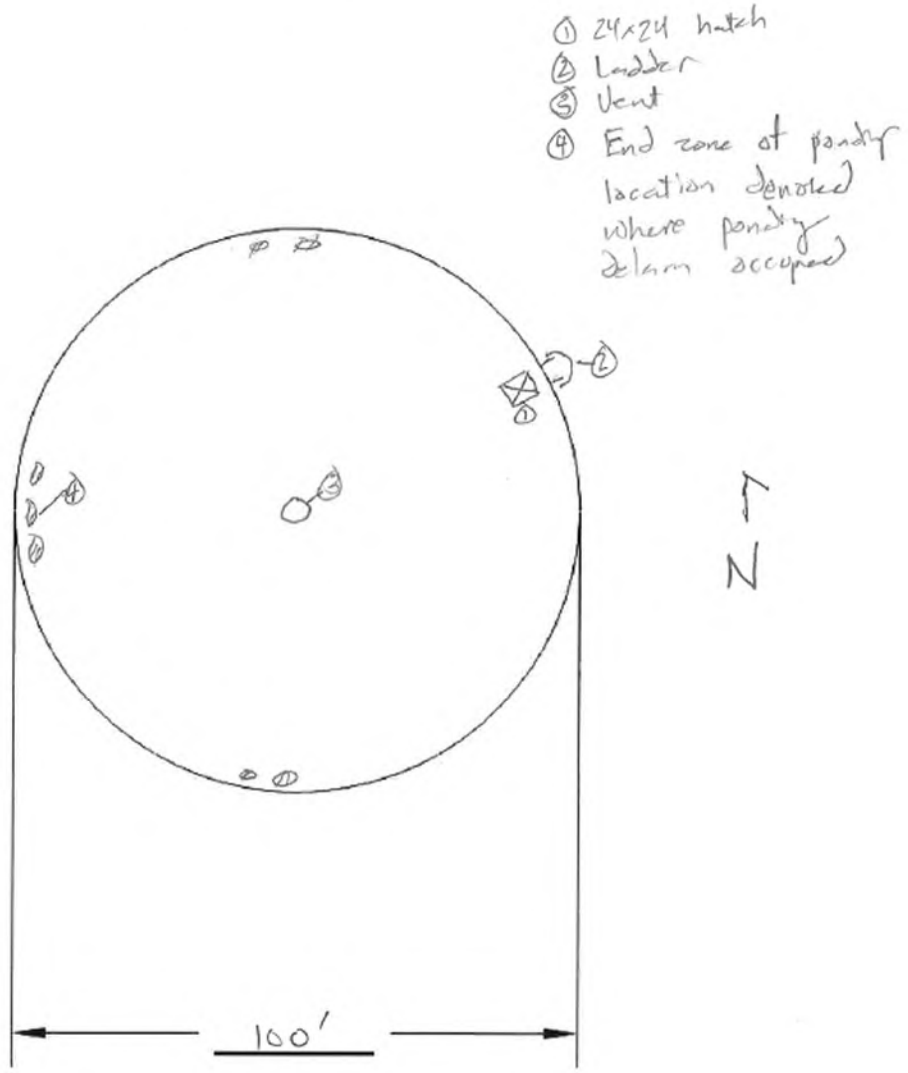


Figure 1: Reservoir Overview – Note location of hatches, ladders, and manways. List given and measured diameter.

5

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

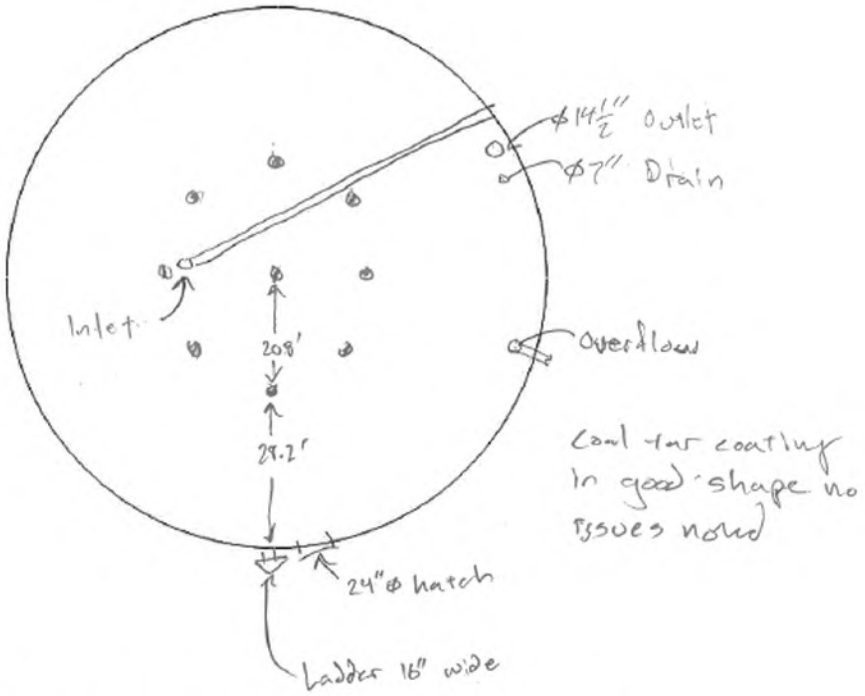


Figure 2: Reservoir Roof – Sketch location of roof supports, hatches, vent, columns, etc.

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

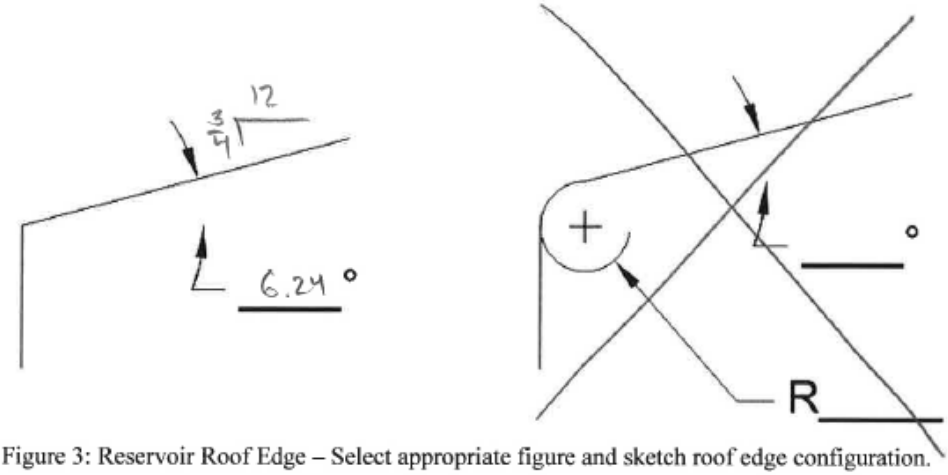


Figure 3: Reservoir Roof Edge – Select appropriate figure and sketch roof edge configuration.

Sketch Roof Type

Thickness: _____
Height: _____

Thickness: _____
Height: _____

Thickness: 0.25
Height: 10'

Thickness: 0.25
Height: 8'

Thickness: 0.375
Height: 8'

Thickness: 0.4688
Height: 8'

Thickness: 0.5687
Height: 8'

Thickness: 0.6875
Height: 8'

Figure 4: Exterior shell information and notes.

7

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WELDED STEEL RESERVOIR SITE INSPECTION

Project #: _____ Site Visit Date: _____

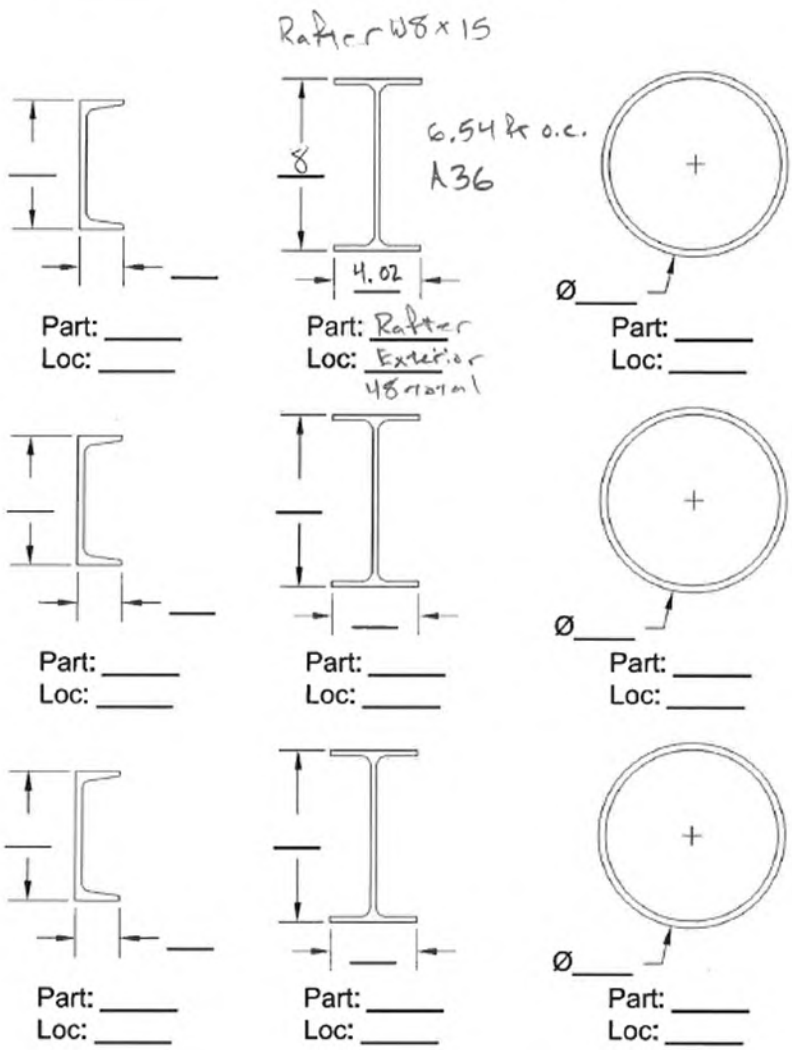


Figure 4: Component detail notes.

8

END OF SECTION

Appendix A-4 Marietta General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Marietta Reservoir *General Info*

Field Visit Date: 1/24/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	1/24/2019
Reservoir Name and Location:	Marietta - 1404 Marietta Ave
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis, Jeremy Hailey
Client Staff Present:	Steve Bradshaw, Shayla Francis, Aaron
Year Constructed:	1969
Overflow Destination:	Drainage ditch SW of reservoir property
Discharge Destination/Zone:	276 North Zone
Fill Location:	276 North Zone - Curtis Rd Main
Reservoir Material:	Welded Steel

Measurement Type	Measurement	Unit
Volume:	2.5	MG
Diameter:	100	ft
Height	50	ft
Overflow Elevation:	275	ft AMSL
Bottom Elevation:	TBD	ft AMSL
Level of Overflow	TBD	ft
Minimum Normal Operating Level:	45	ft
Maximum Normal Operating Level:	50	ft
Notes: Reservoir is currently not being used.		

Marietta Reservoir *Exterior Inspection*

Field Visit Date: 1/24/2019

Exterior Ladder:	
Present on site?	Yes
Material:	Welded Steel
Condition:	Good
Corrosion:	No
Cage:	Yes
Security Type:	Locked access hatch
Security Condition:	Good
Wall Attachment Type:	Welded
Wall Attachment Condition:	Good
Rung Shape:	Circular

Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	6	in
Back Clearance:	24	in

Notes: Seven attachment points. Welds are fairly rough, but no corrosion noted.

Exterior Fall Prevention System:	
Present at Site:	Yes
Type:	Dual carabiner ladder straps
Fall Protection System Condition:	Good
Notes:	

Side Vents and Screens:	
Present at Site:	No
Condition:	N/A
Notes:	

Entry Hatch:		
Hatch Location:	North, roof	
Material:	Steel	
Condition:	Fair	
Gasketed:	No	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	North	
Measurement Type	Measurement	Unit
Size:	24	in
Curb Height:	6	in
Notes: Bee hives noticed in vicinity of roof hatch, some minor corrosion. No interior ladder.		

Entry Hatch:		
Hatch Location:	North, side	
Material:	Steel	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	No	
Lock:	Yes	
Frame Drain Location:	N/A	
Measurement Type	Measurement	Unit
Size:	24	in
Curb Height:	N/A	in
Notes:		

Roof Vents and Screen:		
Material:	Wire mesh	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	6	in
Notes: One 36" roof vent. Screen in good condition.		

Roof:	
Roof Sloped:	Yes
Downspouts:	No
Ponding on Roof:	Yes
Roof Finish:	Painted
Slope of roof	3/4 : 12

Measurement Type	Measurement	Unit
Overhang Distance:	0	in
Thickness of roof slab	0.200 - 0.208	in

Notes: The configuration of the roof plates is such that at this location rainwater will accumulate creating a ponding condition where water can stagnate, weakening the coating.

Railing:	
Present at Site:	No

Grating:	
Present at Site:	No

Foundation:	
Material:	Concrete
Condition:	Good
Anchoring Condition:	Good

Measurement Type	Measurement	Unit
Grade (North)	4-5	In
Grade (Other areas)	6	In

Notes: Minor cracks in foundation, but no indication of adverse differential settlement. Footing measurements indicate wider than original design drawings.

Other:	
Photo of Anchoring System:	Yes
Flexible Couplings at Foundation:	No
Notes:	

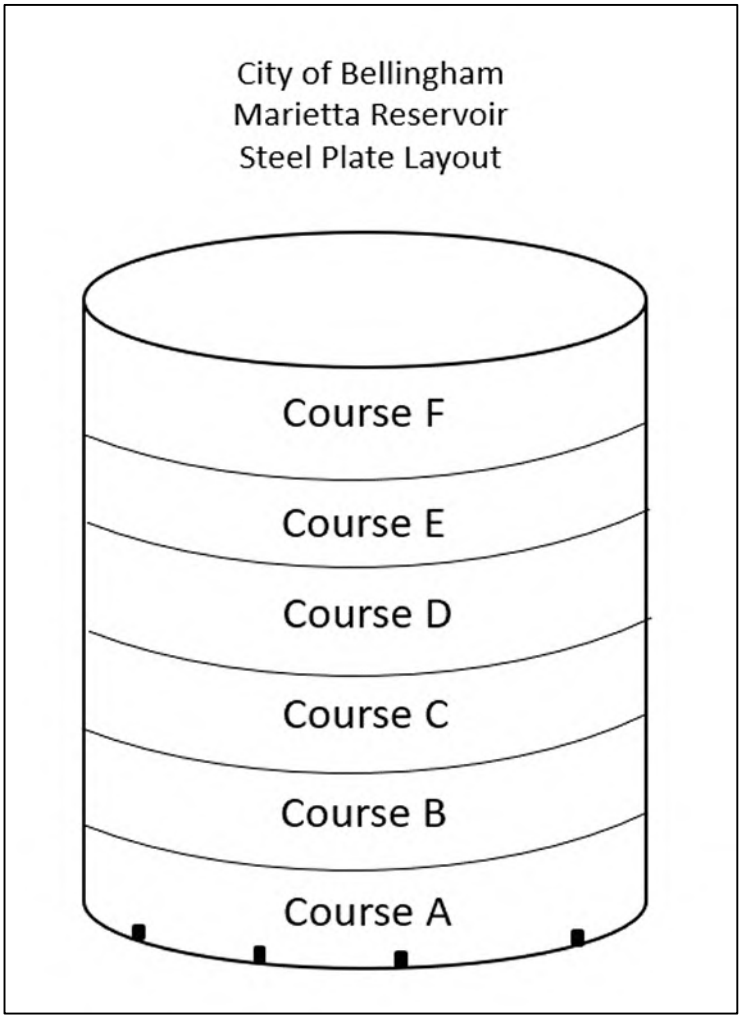
Exterior Coating	
Exterior Walls:	3 coat system (See notes)
Exterior of Roof:	Paint
Exterior Piping:	Paint
Exterior Coating System Lead Concerns:	Yes
Paint Samples Collected:	Yes
Soil Samples Collected:	
Exterior Coating DFT Testing Results:	8 - 16 mils
Exterior Coating Adhesion Testing Results:	Good
Notes: Swab of exterior coating tested positive for lead. Leachable lead test forthcoming. Prime coat is polyamidoamine epoxy - lead noted in specification. Three coat system appears to be red primer, light blue intermediate coating, and light green topcoat. The exterior surfaces of the tank had only a few visible minor instances of small pits, all less than 1/32” deep.	

Marietta Reservoir *Steel Plates*

Field Visit Date: 1/24/2019

Course	Average Steel Plate Height (Feet)	Steel Plate Thickness (Inches)	Notes
F	10.0	0.249	
E	8.0	0.24	
D	8.0	0.378	
C	8.0	0.443	
B	8.0	0.558	
A	8.0	0.663-0.667	

Notes: Weld lines are not smooth in most locations and appear rough.



Marietta Reservoir *Interior Inspection*

Field Visit Date: 1/24/2019

Interior Ladder:	
Present at Site:	No

Interior Fall Prevention System:	
Present at Site:	No

Interior Roof:		
Material:	Welded Steel	
Condition:	Good	
Measurement Type	Measurement	Unit
Wall to Mid-span rafter support	30	ft
Notes: The roof and roof support members show areas of rust staining, particularly on the edges of the beams and rafters. However, when viewed from the roof access hatch, there was no observed significant metal deterioration.		

Columns:		
Material:	Steel	
Condition:	Good	
Measurement Type	Measurement	Unit
Width/diameter	12	in
Base width	3	ft
Notes:		

Floor	
Material:	Welded Steel
Condition:	Good
Leaks:	None apparent
Approximate Location:	N/A
Severity:	N/A
Notes:	

Walls:	
Painters Rings Present:	Yes
Condition:	Good
Interior Wall Material:	Welded Steel
Notes:	

Interior Coating	
Interior Walls:	Coal tar
Interior Floor:	Coal tar
Interior of Roof:	Coal tar
Interior Ladder:	N/A
Interior Piping:	N/A
Interior Coating System Lead/Coal Tar Concerns:	Yes
Interior Coating DFT Testing Results:	100 – 300 mils
Interior Coating Adhesion Testing Results:	Good
Notes: The coating was mop applied to the sidewalls and portions of the floor were flood coating (coating poured on and then smoothed out). The coating is fairly brittle and cracks in places when walked on.	

Field Visit Date: 1/24/2019

Piping		
Inlet Piping:	Size (Inches OD):	12
	Condition:	Good
	Material:	DI/PVC
	Notes: Inlet enters side of reservoir w/insulated cover.	
Outlet Piping:	Size (Inches OD):	16
	Condition:	Good
	Material:	Steel
	Lip (Inches)	13
	Notes: Level transmitter located off outlet piping.	
Overflow Piping:	Size (Inches OD):	12
	Condition:	Fair
	Air Gap:	Yes
	Screened:	Yes
	Material:	Unknown
	Outlet Location:	Top of reservoir, NW side
	Erosion Evident:	No
	Screen Condition:	Fair
	Overflow to roof (inches)	3
	Notes: Overflow discharge near top of reservoir, screen is present, but unable to inspect closely.	
Drain Piping:	Size (Inches OD):	8
	Condition:	Fair
	Outlet Location:	Drainage ditch SW of reservoir property
	Screened:	Unknown
	Material:	Unknown
	Silt Stop Type:	Unknown
	Air Gap:	Unknown
	Screen Condition:	Unknown
	Notes: Unable to locate drain piping outlet to inspect.	

Piping Facilities		
Exterior Valving:	Type:	Ductile Iron
	Condition:	Very Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	Good
	Secured:	Yes
Washdown Piping	Location:	No
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	Not apparent
Notes: Inlet/Outlet piping improvements in 2013.		

Electrical	
Cathodic Protection:	Yes
Impressed Current:	Yes
Anodes:	Yes
Notes: Ceiling hung anodes - 5 in middle, 14 half way to perimeter. Visual inspection only as system was not turned on. Constant voltage model.	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	No
Check Valves:	Yes
Common Inlet/Outlet:	No
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	No
Sediment Build-Up Height Above Floor (ft)	No
Water Quality Sample Taps?	Yes
Notes:	

Appendix A-5 Marietta Condition Assessment Score Sheet

Marietta Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No Camera. Evidence of Vandalism
	Vegetation Separation	0	0	0	0	0	0	3	0	Trees too close, almost overhanging; no noted debris
	Site Drainage	0	0	0	0	0	0	2	0	Standing water
Walls	Exterior Walls	3	4	3	3	0	0	5	0	Walls need cleaning; Spot repair only
	Interior Walls	5	5	3	3	4	0	5	0	Coal Tar Coating is not best practice (?)
Floor/ Foundation	Foundation	4	5	3	2	0	0	5	0	Organic material buildup on foundation
	Interior Floor	4	5	3	2	4	0	5	0	Some sediment buildup on floor
	Anchors (Steel) / Seismic Cables (PSC)	0	4	5	2	0	0	0	0	Anchors are not required. Current undersized and can cause foundation failure
Roof	Exterior Roof	3	4	5	2	5	0	2	0	Roof susceptible to slosh induced failure at the max operating level. Ponding
	Interior Roof and Supports	5	5	5	2	0	0	0	0	
	Columns	5	4	5	2	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	4	0	0	0	1	3	0	Difficult to get up to ladder. Needs resting platform. Cage compliant through 2036.
	Interior Ladders/Fall Protection	0	0	0	0	0	0	0	0	No int ladder; none required
	Access Hatches	2	4	0	0	2	3	2	0	Needs second side hatch; exist. is too small. Roof hatch has bug issues, high maint., and no railing
	Railings and Roof Fall Protection	0	0	0	0	0	2	0	0	System required for roof workers.
	Vents	5	5	0	0	2	0	1	0	Screen too large and failed design checks.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	5	5	0	2	5	0	5	0	No flex coupling
	Outlet Piping	5	5	0	2	5	0	5	0	No flex coupling
	Drain Piping	0	4	0	2	3	0	2	0	No flex coupling. Dechlorination and energy dissipation unknown
	Overflow Piping	0	4	0	0	4	0	2	0	Does not extend to ground and hard to inspect. Screen appears compliant
	Washdown Piping	0	0	0	0	0	0	3	0	Unknown washdown piping. Needs update
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	5	5	0	0	5	0	5	5	New valves
	Isolation Valving	5	5	0	0	5	0	5	5	New valves
Misc.	Cathodic Protection System	0	0	0	0	0	0	3	3	Constant voltage model
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	5	0	5	0	Retrofitted inlet/outlet
	Categorical Score	4.4	4.5	4.0	2.2	4.1	2.0	3.7	4.3	

Overall Score
3.3

Appendix B Padden

Appendix B-1 Padden Geotechnical Report

GeoEngineers

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Padden Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Padden reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the Padden site, located as shown in the Vicinity Map, Figure 1. The Padden reservoir is a steel reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) maps for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by undifferentiated glacial deposits in a close approximately of the Chuckanut Formation deposits.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift. Based on previous experience in the area, Bellingham (glaciomarine) Drift overlies the bedrock in this area.

Previous Studies

We reviewed the original report for the project titled, "Foundation Investigation, Treatment Plant and James Street and Lake Padden Reservoirs, Bellingham, Water System" completed by Shannon & Wilson (S&W) dated April 7, 1966. A single boring was completed at the site to a depth of 19 feet. The boring log indicates medium dense to dense sandy gravel with variable silt and clay content. The report describes the reservoir as having a "dish" shaped bottom set into the hillside. Excavation of 14 to 22 feet was required to install the reservoir.

Surface Conditions

The project site is located approximately 450 feet to the south of the intersection of 28th and Broad Street at the bottom of a hill, in an open grass field. The site is bounded by Lake Padden park to the east, south, and west and residential properties to the north. The site slopes downward from north to south (Elevation 450 to 430 feet) until it reaches the reservoir. South of the reservoir the slope continues to Elevation 408 feet and a stream.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-1 (2019)—on March 22, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 71½ feet below the existing ground surface (bgs). The location of the boring is shown in the Site Plan Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations and our experience at nearby project sites.

- **Fill** – A layer of fill was encountered in the boring. The thickness of the fill was found to be 6½ feet. The fill was comprised of brown dense gravel with sand.
- **Undifferentiated Glacial Drift** – Native undifferentiated glacial drift was encountered underlying the fill to the termination depth of 71½ feet. The character of the undifferentiated drift unit was quite variable: most of this unit was a dense sand with silt and gravel and silty sand, with a stiff silt layer from approximately 14 to 19 feet bgs.

Groundwater

Groundwater seepage was observed at a depth of 19 feet bgs in the boring. The undifferentiated glacial drift unit commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Dense/stiff undifferentiated glacial drift soils were encountered at our boring location and extended to the full depth explored (71½ feet bgs). We reviewed the original geotechnical report prepared by Shannon & Wilson (1966). The base of the reservoir is approximately 14 to 22 feet below grade. Therefore, we conclude that the reservoir foundation is founded on very dense soils.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on dense glacial drift deposits which are not at risk of liquefaction.

AWWA/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on publication D100-11 of the AWWA and the ASCE 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the

maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	D
Soil Profile Type	Stiff Soil
Average Field Standard Penetration Resistance	$15 \leq N_{ave} \leq 50$
AWWA Seismic Use Group	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	96.8
1-Second Period Spectral Response Acceleration, S_1 (percent g)	38.1
Seismic Coefficient, F_a	1.11
Seismic Coefficient, F_V	1.64
MCE_G peak ground acceleration, PGA	0.400
Seismic design value, S_{DS}	0.718
Seismic design value, S_{D1}	0.416

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek,

are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8050 to 7250 calendar years before present (cy B.P.), 3190 to 2980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.24	0.43	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.08	0.14	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeter per second, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends more than 78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.36	0.65	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and review of original geotechnical report previously referenced, we understand that the existing reservoir is bearing on dense native sand and gravel deposits. We recommend that the structure be evaluated based on an allowable bearing pressure of 3,000 pounds per

square foot (psf) which is consistent with local practice. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral resistance is not necessary for an above ground steel reservoir.

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the west that is approximately 30 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on dense to very dense undifferentiated glacial drift deposits. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:tln

Attachments

Figure 1 – Vicinity Map

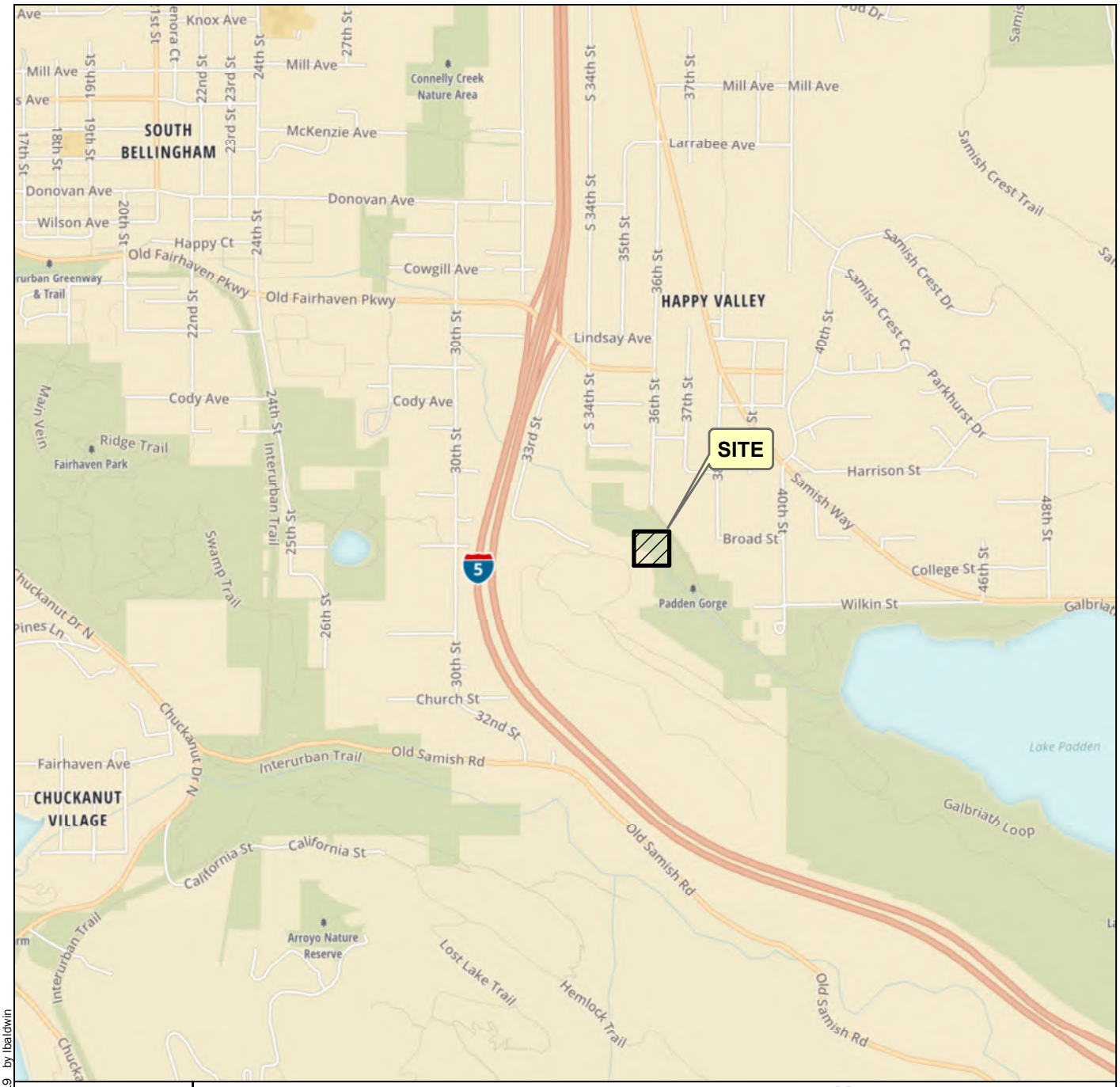
Figure 2 – Site Plan

Figure 3 – Key to Exploration Logs

Figure 4 – Log of Boring B-1 (2019)

Figure 5 – BSSC2014 Scenario Catalog – Mw 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – Mw 7.5 Devils Mountain Fault



Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Padden Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_Padden.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

100 0 100

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Padden Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled	Start 3/22/2019	End 3/22/2019	Total Depth (ft) 71.5	Logged By BWS / Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum	430 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) / Northing (Y)	1245750 / 626840		System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0					GP	Brown-gray fine to coarse gravel with sand (dense, moist) (fill)			
		3	46	1						
425	5	0	50/5"	2						
						SP-SM	Brown fine to coarse sand with silt and gravel (dense to very dense, moist) (undifferentiated glacial drift)	7		
		9	52	3 MC						
420	10	6	29	4 MC		SM	Brown silty fine to coarse sand with gravel (dense, moist)	14		
						ML	Gray silt with sand and gravel (stiff, moist)			
415	15	18	9	5 MC				12		
						SM	Gray silty fine to coarse sand with gravel (dense, moist)			Groundwater seepage observed at 19 feet during drilling
410	20	18	33	6						
405	25	12	50/6"	7			Grades to very dense			
400	30	9	86	8						
395	35									

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-1

GeoEngineers

Project: COB Reservoir Inspection and Repair - Lake Padden
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35	X	12	60		9						
390	40	X	0	50/5"		10						
385	45	X	18	51		11						
380	50	X	15	33		12 / MC				9		
375	55	X	12	66		13						
370	60	X	15	41		14			Grades to dense			
365	65	X	18	22		15 / MC			Increased silt content	9		
360	70	X	18	37		16						

Log of Boring B-1 (continued)

GeoEngineers

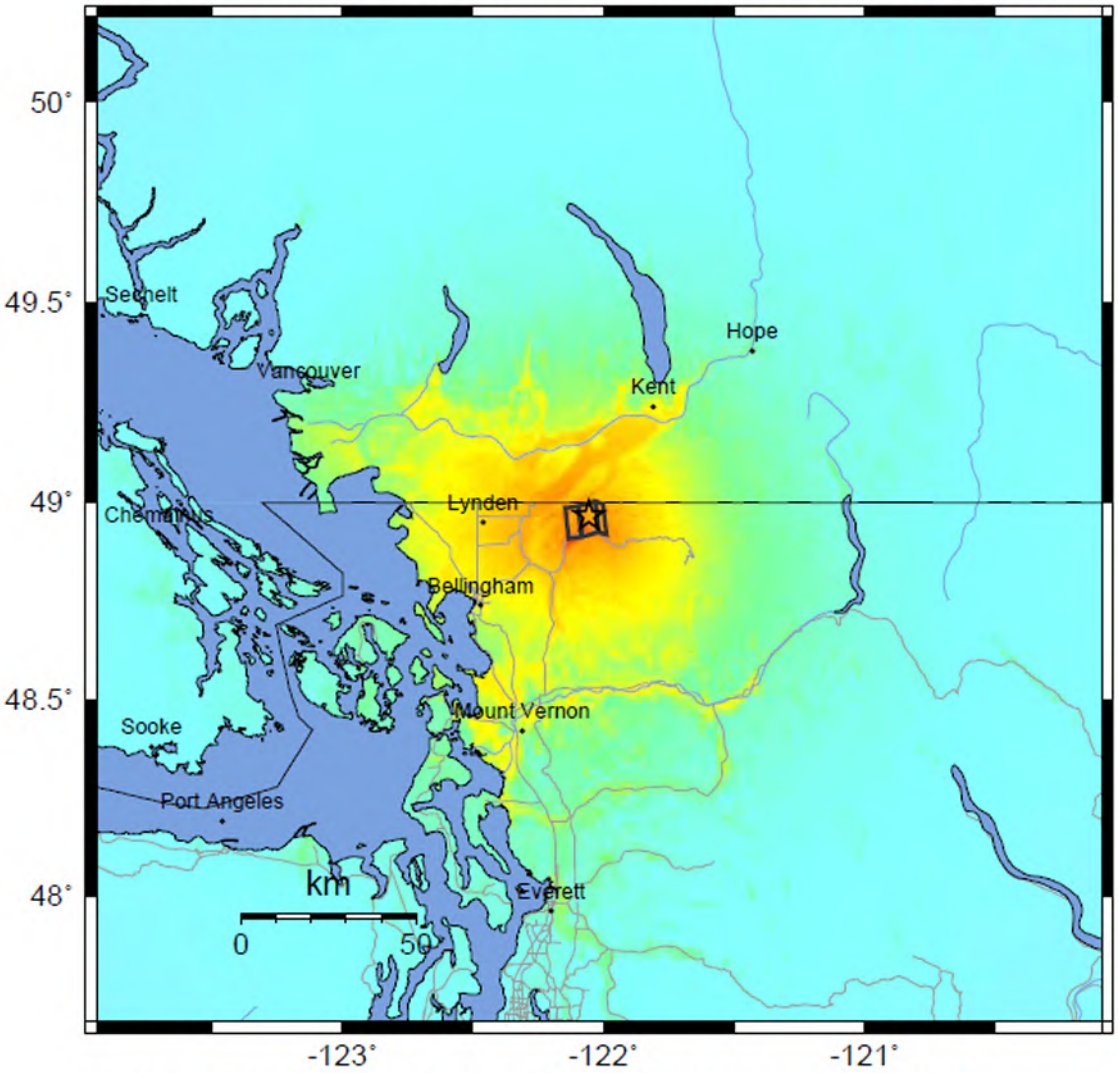
Project: COB Reservoir Inspection and Repair - Lake Padden
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 2 of 2

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

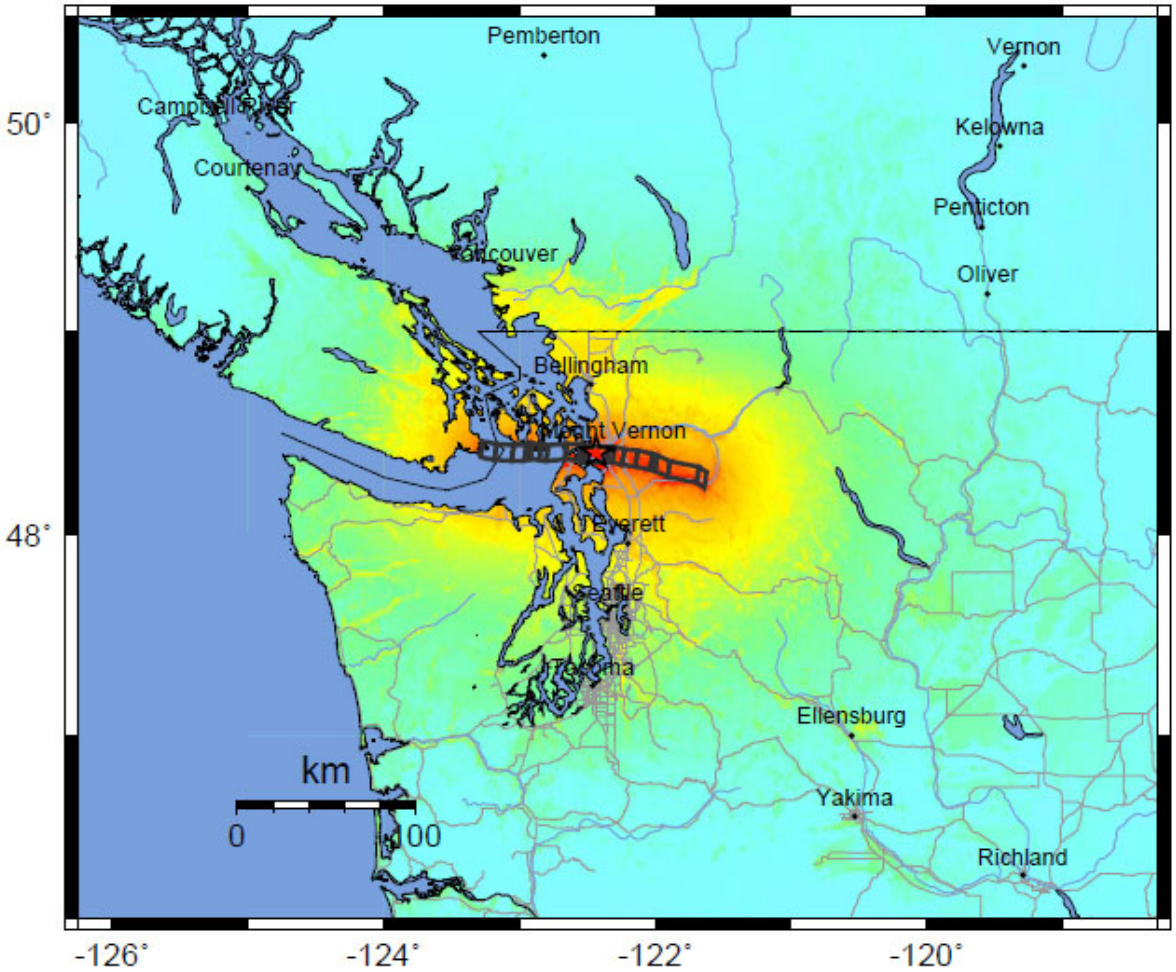
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix B-2 Padden Corrosion and Coatings Report

May 17, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Lake Padden Tank Corrosion and Coatings Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an internal and external corrosion and coatings evaluation for the City of Bellingham's Lake Padden steel water storage tank. Specific tasks performed during this evaluation included:

1. Evaluate observed corrosion on the tank's steel surfaces.
2. Complete an assessment of both the interior and exterior coatings. The interior floor and lower wall inspection was completed while the tank was empty. A raft was used to inspect the upper wall and roof surfaces with the water level near the height of the overflow.
3. Measure remaining steel wall thickness using ultrasonic thickness testing equipment at accessible locations.
4. Measure the depth of any accessible noted pitting.
5. Evaluate coating losses and corrosion on visible surfaces.
6. Test for presence of lead using field lead-check swabs.
7. Perform a checkout of the impressed current cathodic protection system.

BACKGROUND INFORMATION

Lake Padden Tank

Height 22-ft tall, 59-ft diameter, welded steel constructed in 1967. The exterior coating is comprised of a four coat system, with the top layer appearing to be an overcoat completed at a late date. An impressed current cathodic protection system comprised of 15 mixed metal oxide wire anodes is supported from the roof (3 anodes around the center and 12 anodes in the outer ring).

COATING AND STEEL EVALUATION METHODS

A series of field tests were completed on the interior and exterior surfaces of the tank during our site visit. A description of each test is provided below.

Dry Film Thickness

The thickness of the existing coating system was measured using a portable electromagnetic dry film thickness gauge (Type 2 gauge). This gauge measures the thickness of the coating between the gauge probe and the metallic substrate.

Steel Thickness

Steel wall thickness was measured using a General Electric model DM5E ultrasonic thickness gauge calibrated for carbon steel. When measuring steel thickness, this unit uses an echo-echo function that allows for measurement of the steel without removing the coating material. Measurements were recorded on each shell course, roof knuckle, roof plate, and interior bottom floor plate.

Lead Testing

A field lead check swab was used to test for the presence of lead on the exterior prime coat. This test is used to determine if lead based coatings are present. If lead is detected, a coating sample is collected and submitted to an analytical laboratory to test for the presence of leachable lead using the Toxic Characteristic Leaching Procedure (TCLP). This test is conducted to determine if the coating material is classified as hazardous requiring specialized handling, containment, and disposal when removed from the tank.

Cathodic Protection System Evaluation

A checkout of the impressed current cathodic protection system was performed while the tank was filled. The test included measuring on, instant off, and depolarized potentials to determine if the interior submerged surfaces meet corrosion control criteria. As part of this survey, the transformer rectifier was inspected and tested for proper operation.

TEST RESULTS AND ANALYSIS

Exterior Coating Thickness

The exterior coating of the tank sidewalls appears to be a four coat system with a red primer, green first intermediate coat, silver second intermediate coat, and blue topcoat. City employees stated the blue topcoat was applied years after the original three coat system.

The measured total thickness of the external coating on the sidewalls ranged from 6.6 – 14.3 mils. Readings taken on areas of missing top coat ranged from 6.9 – 9.5 mils. Typical high performance coatings for the exterior surfaces of water storage tanks are on the order of 12 – 16 mils.

The coating on the exterior tank roof appears to be a two coat system with a red primer and a blue top coat. Dry film thickness measurements taken on the exterior roof coating ranged from 3.5 – 9.6 mils.

Exterior Coating Assessment

The exterior sidewalls of the tank are dirty with streaks of organic matter at locations where water runs off the roof. The top coat is lightly adhered and easily peels off. 10-15% of the top

coat is missing, mostly centered in one location along the bottom course adjacent to the ladder. The underlying coatings are in better condition with moderate adhesion.

The surfaces of the tank roof are very dirty. There are no visible coating defects and the coating is moderately adhered. The roof vent is on the side of the roof as opposed to the center and with the exception of a dirty cover, the vent is in good condition. There is an excessive amount of paint loss and corrosion on the roof access hatch.

Exterior Observed Corrosion Assessment

The exterior sidewall surfaces of the tank had no visible pitting and no damage was observed on the exterior welds.

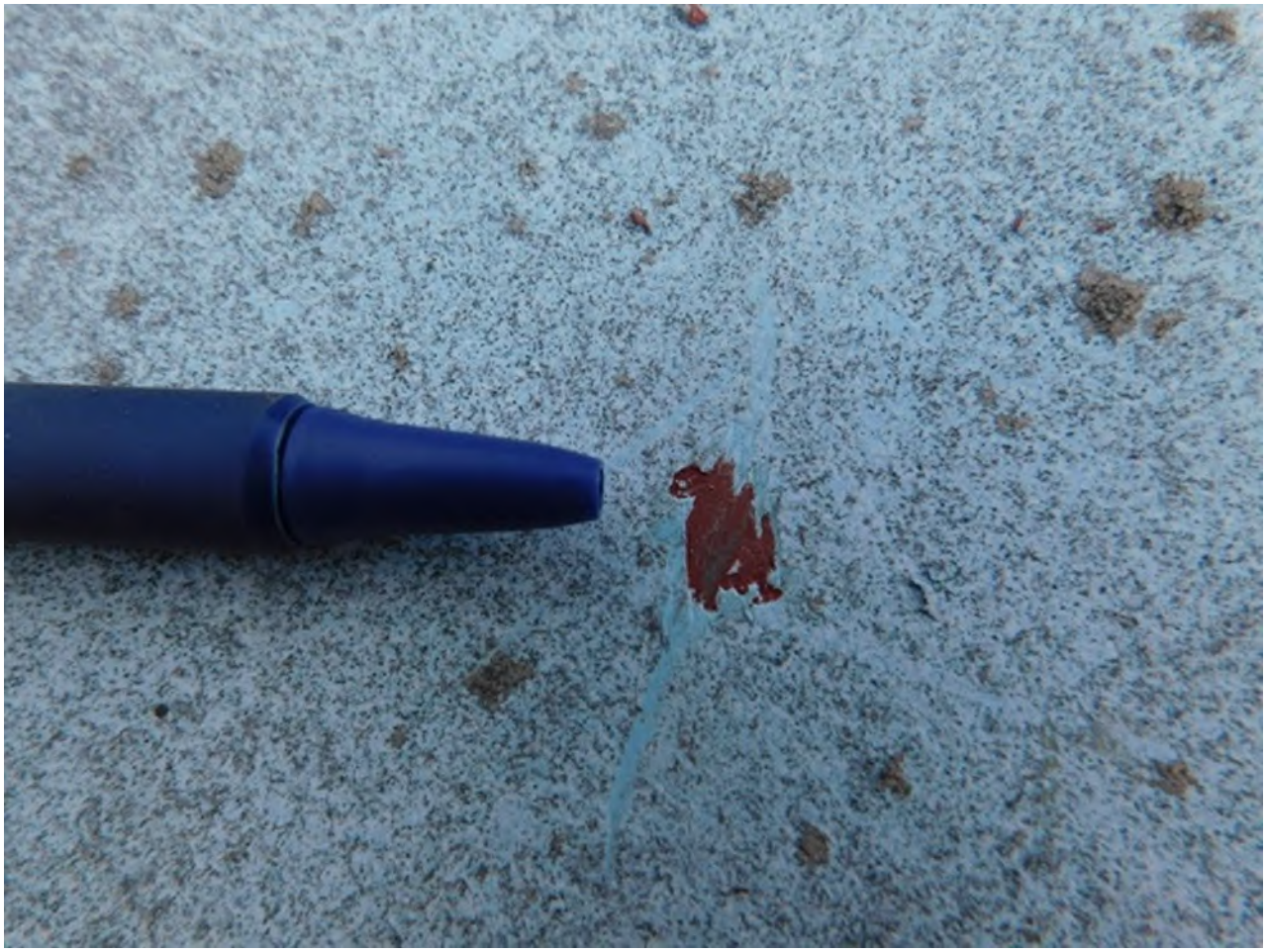
The tank chime is in good condition with only minor surface rust at areas of coating loss. The concrete grouting between the tank chime and the ring wall has failed in several locations and should be repaired

The roof welds are in good condition and are oriented on the steel plates such that rain water will run off as opposed to collect at the lap joint areas. No visible signs of pitting are present.

Lake Padden Tank

Missing top coat on sidewall

Organic growth on sidewall



Area of top coat easily scraped off with a knife

Tank exterior roof

Roof vent

Coating failure on roof access hatch

Sealing failure at chime/ring wall transition

Interior Coating Thickness

The measured total thickness of the internal coating on the sidewalls ranged from 5.2 – 10 mils. Typical high performance coatings for the interior surfaces of water storage tanks are on the order of 12 – 20 mils. Dry film thickness measurements on the floor ranged from 5.5 – 14.5 mils.

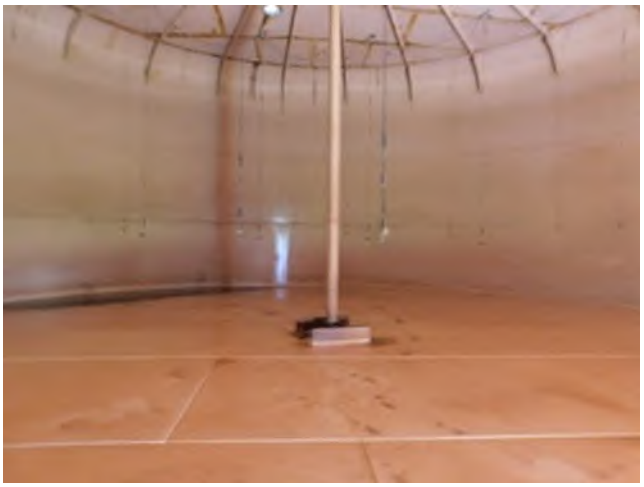
Interior Coating Assessment

There is blistering in the coating on the floor underneath each anode (likely due to higher than required voltage operation of the cathodic protection system) and along the weld seams. The coating on the overflow pipe is in good condition. There is blistering on the bottom end of the center column, but the coating on the center column plate is in sound condition.

There is significant coating blistering on the bottom 4-inches of the interior wall. There is also delaminated coating on areas of the knuckle sheet as it transitions to the roof plates. The interior surfaces of the roof hatch has significant coating loss over much of the area. Areas of sound coating are moderately adhered.

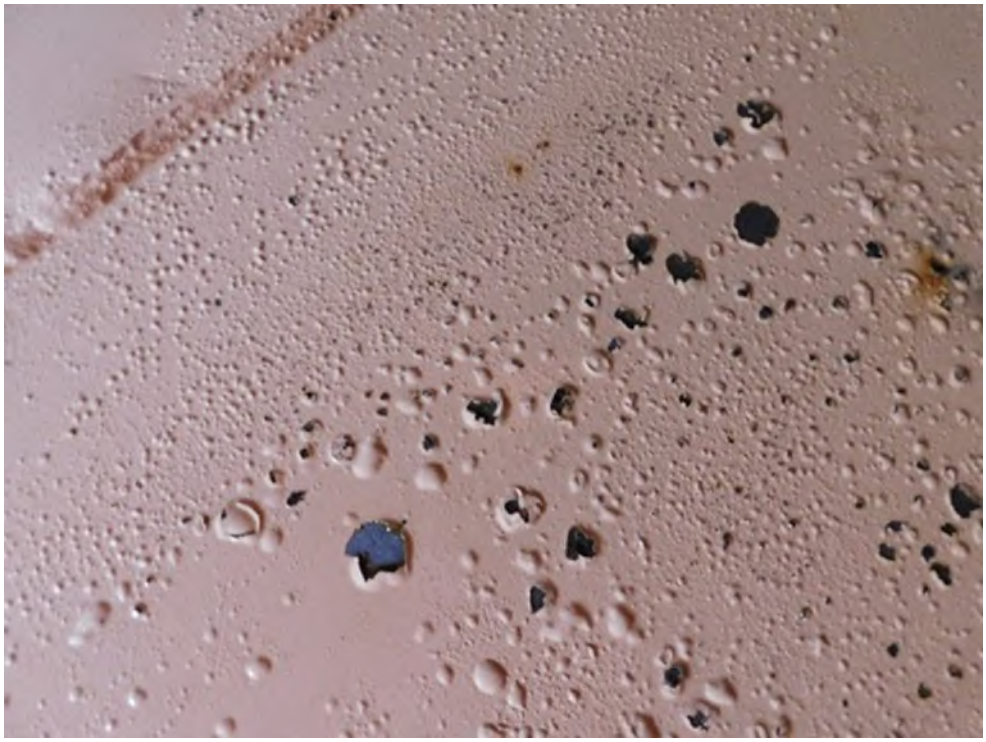
Interior Observed Corrosion

The roof shows rust staining at all roof plate overlaps, typical for these non-seal welded areas. The structural components surrounding the access hatch and areas above the water level are experiencing general surface corrosion and crevice corrosion at steel overlap areas. Exposed steel locations below the waterline are in very good condition with no observed pitting. This is due to the operation of the cathodic protection system.



Tank interior floor, sidewalls and center column

Interior roof, rust staining at all non-seal welded overlaps



Floor – blistering in the coating, steel under blisters shows no losses

Sidewall – blistering on the first 4” of coating, typical around entire circumference

Coating loss and corrosion at roof hatch

Corrosion at roof plate overlap near access hatch

Crevice corrosion at knuckle/roof plate

Corrosion around vent opening

Roof beams at center column

Closeup of beams

Coating loss at roof plate overlap

Corrosion and coating loss at anode hanger

As shown in the above pictures, there is a significant amount of coating losses and crevice corrosion at the non-seal welded locations. Without repair, this condition will continue resulting in significant damage to the steel, particularly in-between roof plates and the overlap areas between the roof beams and steel roof plate.

Lead Test

Lead tests were performed at two locations around the tank on the prime coat material. Both lead check swab kits tested negative for the presence of lead materials.

Ultrasonic Thickness Testing

Thickness data was collected on each shell course and the roof plates. Results of the ultrasonic thickness tests are presented in Table 1. Design thickness data was not available.

Table 1: Lake Padden Steel Thickness Measurements

Location	Measured Thickness, in.
Bottom Course	0.307 – 0.311
2nd Course	0.262
3rd Course	0.258
Knuckle	0.276
Roof	0.189
Interior Floor	0.250

Cathodic Protection Equipment

The impressed current cathodic protection system utilizes 15 mixed metal oxide anodes. A checkout of the rectifier and testing of the cathodic protection system was completed during a separate site visit while the tank was full.

The installed rectifier is a constant voltage model that does not internally adjust power output based upon water levels in the tank. For example, a decrease in water level results in less submerged surface area and less current needed for protection. An autopotential rectifier will adjust itself and reduce current output by decreasing voltage to account for the lower current demand. Without this type of automation, cathodic disbondment of the coating can occur, as was observed under each anode and on the lower interior sidewall surfaces.

While cathodic disbondment will not normally result in damage to the steel, coating life will be lessened leading to increased protective current demands. The installation of an autopotential rectifier with a stationary reference electrode positioned within the tank will provide for the correct method of cathodic protection.

Results of the rectifier checkout and potential measurements are included in Tables 2 and 3 at the end of this report. Initial testing showed that corrosion protection criteria was not being

met at all tested locations. The rectifier output was then increased to provide additional current. After adjustment, the collected data indicates that corrosion control criteria is being met at all locations with the exception of the bottom 4-feet of the tank. After an appropriate polarization period, typical 24 – 48 hours, the tank should meet criteria at all locations.

Mixed metal oxide anode strings suspended from roof

Constant voltage rectifier installed in pump station

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the tank.

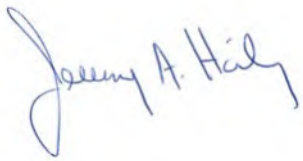
1. All exterior surfaces of the tank are dirty with several areas of organic material accumulation on the roof and sidewalls.
2. The exterior top coat is loosely adhered and easily peels off.
3. The interior coating has multiple areas of blistering on the floor and lower sidewall. The remaining submerged areas on the sidewall are in decent condition.
4. A significant amount of rust staining was observed on the roof and roof support members at all non-seal welded locations.
5. Sealing material between the tank chime and ring wall is cracking and broken in several locations around the tank.
6. Field lead checks swabs tested negative for the presence of lead.
7. During the checkout of the cathodic protection system, rectifier current output was increased in order to provide additional corrosion control.
8. The existing transformer rectifier is a constant voltage model. Rectifier units for water tanks should be autopotential in order to account for fluctuations in water level.

RECOMMENDATIONS

1. The exterior surfaces need to be pressure washed to remove dirt and other debris. This will help extend the life of the coating providing approximately five more years of useful service life. During pressure washing operations, the removal of top coat material can be expected at several locations on the tank due to low adhesion qualities.
2. A full exterior recoat should be completed by 2025.
3. Given the condition of the interior floor and roof, it is recommended to remove all existing coating, seal-weld all overlaps, abrasively blast all surfaces, and apply a new protective coating. To minimize any additional steel losses, this work should be completed within the next two years.
4. Repair damaged grout at the tank chime/ring wall transition.
5. Install an autopotential rectifier to properly adjust cathodic protection current output based upon water levels in the tank. This task will also include installation of a stationary reference electrode positioned within the tank with its lead wire routed to the rectifier.

We appreciate the opportunity to work with you on this project. If you have any questions or would like any additional information, please feel free to contact our office.

Sincerely,
Northwest Corrosion Engineering



Jeremy A. Hailey, P.E.

TABLE 12: Rectifier Data – Lake Padden

Manufacturer:	Goodall
Model:	CSAYSA 18-4 FNSZ
Serial No:	8602314

Name Plate Data:

AC Volts 120 DC Volts 18
AC Amps 0.997 DC Amps 4
60 Hertz, Single Phase, Amb. Temp 45ºC
Max Tap Setting – Coarse D, Fine 5
Shunt Rating – 50mV/5A

As-Found

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	2.0	2.02
Milliamps DC	0.0	17.3
Tap Setting	Course A Fine 2	

As-Adjusted

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	2.5	2.97
Milliamps DC	0.0	68
Tap Setting	Course A Fine 3	

NOTES:

1. Rectifier output was increased to provide additional protective current.
2. The rectifier is a constant voltage model and is not capable of automatic current output adjustment.

TABLE 3: Structure-To-Electrolyte Potential Data

At Roof Access Hatch

	Potential, recorded in millivolts		
Ref. Cell Depth (ft)	**ON**	**Instant Off**	**Depolarized**
Water Level	-1362	-817	-709
2	-1358	-820	-708
4	-1348	-819	-708
6	-1324	-816	-707
8	-1287	-814	-706
10	-1240	-810	-704
12	-1169	-804	-702
14	-1069	-797	-702
16	-923	-775	-705

NOTES:

1. All data was collected using a portable copper-copper/sulfate reference electrode and calibrate digital multimeter.
2. Corrosion control criteria is met when the instant off potential is at or more negative than -850 millivolts. Alternatively, criteria is also established if the depolarized potential is 100 millivolt or more than the instant off value (National Association of Corrosion Engineers Standard Practice *SP0388 Impressed Current Cathodic Protection of Internal Submerged Surfaces of Carbon Steel Water Storage Tanks.*

Appendix B-3 Padden Structural Report

CITY OF BELLINGHAM

CH 4: PADDEN LAKE RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

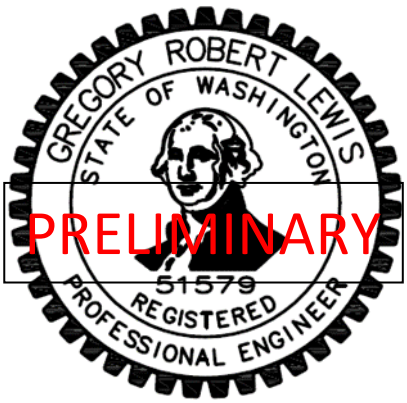
Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Padden Lake, 0.5 Million Gallon (MG) steel reservoir. The reservoir is located near 2606 38th St, Bellingham, WA (Lat. 48.7070, Long. -122.4680), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to inspect and visually evaluate the reservoir on April 9th, 2019 and again on April 30th, 2019 by Peterson Structural Engineers (PSE), Murraysmith, Inc., and Northwest Corrosion. The reservoir was drained during the initial inspection which facilitated the interior inspection and filled during the subsequent inspection to facilitate and evaluation of the interior roof. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Padden Lake Steel Reservoir – 0.5 MG

2.1 Description & Background

The original reservoir was designed by the engineering and planning firm of Stevens, Thompson, Runyan & Ries, Inc. in 1966 and fabricated by the Pittsburgh-Des Moines Steel Company. The identification plate on the tank indicates a construction year of 1967. The reservoir is a ground-supported welded steel reservoir with a 59-foot inside diameter. The wall consists of three 7'-5" shell courses for a total height of 22'-3". The overflow is at a height of 25-feet and located about 3-inches below the roof plate. This is above the wall-line and within the roof knuckle zone, approximately 2'-9" higher than the top of the adjacent vertical wall.

The reservoir roof is supported by a central 6-inch diameter column. The roof plate was measured to be 0.189-inches thick and has an approximate slope of 1:12, which is greater than the code minimum of 0.75:12. The roof is comprised of 6-foot by 18-foot plates which are lapped and welded along their exterior seams. The interior roof plates were not observed to welded along their interior lap seams which is typical of this type and age of reservoir. Further, there is no connection (welded or otherwise) between the roof plates and rafters. The roof plates are welded along the exterior seem of the roof plate-to-knuckle interface.

A single original drawing sheet was available for this reservoir, *Lake Padden Reservoir* "File: 65-P-329" (listed as *Padden Plan.jpg* in the City's e-files), see Figure 2-1. This drawing shows the reservoir supported on a 12-inch-wide by 3.5-foot-deep footing. The drawing shows the footing containing (6) #6 circumferential bars on each face with #4 ties at 12-inches on center. Outlet piping is shown to run approximately 2-feet below the base of the footing rather than through the footing. Measurements onsite found the exposed exterior portion of the footing to be consistent with the drawing's dimensions. The shell is unanchored and is positioned so that the shell-wall bears on the center of the footing.

2.1.1 Description of Additional Site Structures and Features

The site included two additional structures adjacent to the reservoir related to its operation. The first is a 10-foot x 12-foot concrete altitude valve vault. This vault is located to the south of the reservoir and the reservoir's 16-inch outlet-inlet-drain pipe is run through the vault. The altitude vault is partially buried with the upper 3-feet above grade.

The second structure is a 513-square foot pump station designed by Pool Engineering Inc. in 1983. The pump station is a brick building with a concrete slab-on-grade floor. The pump station contains booster pumps and per the drawings, was originally sized for an additional 105-foot diameter reservoir on the site (not built at the time of this evaluation). Drawings provided for the pump station indicate the construction of the pump station coincided with piping upgrades to the site and altitude valve vault.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed two site visits to observe the as-built current condition of the reservoir. The first site visit occurred on April 9th, 2019, while the reservoir was drained, allowing PSE to evaluate the interior floor, manway hatch, and the interior structural systems. The second site visit occurred on April 30th, 2019

while the reservoir was full to allow PSE to evaluate the roof framing and vent opening for corrosion and structural issues.

Steel Roof: The exterior reservoir roof was found to be in generally good structural condition. The roof coating was found to be competent although some organic growth was observed to be developing on the surface of the roof plate. Throughout the roof it was noted that the plate had a "springiness" to it where the roof plate did not sit in direct contact with the rafters below. As one walks across the roof, the reservoir roof can deflect suddenly and there is a bounce to the roof as one walks across its surface. This springiness can to occur in steel reservoirs of all sizes but can be particularly noticeable in smaller reservoirs. This may occur for two primary reasons. First, the roof rafters are radially sloping and arrayed in a circle, as opposed to the roof plates which are flat rectangular sheets. As the plates are placed, the differential between the interior and exterior curve of the roof can result in the plates being offset upwards towards the interior. As the plates are not welded to the rafters these gaps allow for the noted springiness. Secondly, thermal expansion can cause the roof to expand and this forces the plates upwards in the exterior edge is constrained by the wall. While this springiness can be alarming to people walking on the roof, it is not a structural issue of concern as the rafters are adequate for supporting the resulting differential loading.

The roof hatch opening was measured to be 2-feet by 2-feet. The body consists of a welded box section which is welded to the tank roof where it penetrates though the roof plates. The hatch itself is a standard 'shoe-box' style with a lock and foam gasket. The exterior coating appeared to be intact but various organic growths were noted on the roof surface and found on the hatch. The interior of the hatch and lid was noted to have instances of corrosion and the coating was observed to be failing. Rust-burst was observed to be developing along the interior corner of the hatch. Rust-burst is an issue that causes the corroded layer to expand and fan apart at the edges and weakens the underlying plate. Other areas were noted to have some preliminary section loss along the plate and weld lines. An interior ladder system was installed below the hatch and appeared to be in generally good condition with some minor coating loss and corrosion at the very top of the ladder rails where it is in close contact with the roof.

The interior reservoir roof was noted to be in fair condition with multiple instance of crevasse corrosion, coating failure, and section loss along plate lap lines. Rafter connections were noted to be corroded with fasteners beginning to fail. Adjacent to the hatch, the corrosion had impacted one of the rafter's bolts such that it had lost its nut. Other bolting issues (non-corrosion related) were noted at the center of the reservoir where there were multiple locations where the rafters were observed to be missing the bolts that connected them to the center support plate. The missing bolts at the center hub are likely a result poor detailing which limited construction clearances resulting in these bolts being omitted during the original construction. All rafters at the center were noted to have at least one fastener attaching them to the support plate.

Reservoir Floor and Walls: The exterior wall of the reservoir was noted to be in generally good structural condition although issues were noted in the coating which could ultimately impact the underlying steel. Minor chipping and pockmarking in the coating have exposed the steel and caused some incidental corrosion. Overall, the coating was stained and there were multiple locations of failure which has exposed

the underlying coating layers. Along the west side of the reservoir a large mat of moss was growing on the upper portions of the shell. This area also showed signs of rub marks from adjacent tree branches. External appurtenances such as the ladder and manways were found to be in fair condition with incidental coating loss and corrosion noted. Above the manway hatch the identification plate was beginning to come loose due to a corrosion induced failure of its attachment point.

The interior walls (below the typical water level) and floor steel were found to be in generally good condition. Where the coating was compromised, and the base steel exposed the reservoir corrosion protection system appeared to have prevented any further notable corrosion. Although instance of coating failure and bubbling were noted throughout the reservoir, overall corrosion appeared to be minor. Above the typical water level, there were noted to be many areas of corrosion along roof knuckle and leading into roof.

Foundation and Site: The reservoir is supported on a concrete ring foundation, which was found to be in good condition with no settlement issues or major cracking visibly apparent. The gap between the bottom of the floor plate and top of the footing varied from direct contact on the south side to an approximately 1-inch gap on the north side. The smaller gap was sealed with and joint caulking material while the north side's gap was filled with grout. The grout was observed to have failed in multiple locations.

The site slopes from the north towards the reservoir and then away from the reservoir to the south. There had been recent rains and the general site was noted to have instances of standing water. However, the ground directly surrounding the reservoir appeared firm and no standing water was noted. The reservoir's concrete footing height above adjacent grade averaged approximately 6-inches around the majority of the reservoir. This meets the recommended by code requirements outlined in AWWA D100-11 Section 12.7.1. The site is bounded by a fence which results in a clear space of approximately 12 to 15-feet around the reservoir. This zone has been cleared of trees and brush. Outside of the fence line, some larger trees were found to be growing so their branches were in contact with and rubbing against the roof and wall of the reservoir along the western side. This rubbing can damage the coating and exposed the underlying steel, making is suspectable to corrosion. Tree shading also can slow water evaporation and increase contact time between water and metal resulting in an increased corrosion potential. In this case areas of the shell were observed to be developing heavy coatings of moss and organic growth. Trees and/or branches can also break during a storm event and cause impact damage to the roof and shell of an adjacent reservoir. For these reasons, tree branches should be cut back from the reservoir where practical.

2.2.1 Visual Condition of Additional Site Structures and Features

The additional structures on the site were noted to be in generally good condition. Although not investigated in detail, a cursory review found that neither structure appear to have any visible signs of major structural failures, settlement issues, or other large-scale defects during our site inspection.

2.3 Structural Analysis

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoirs under the current applicable Codes and standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE

7-10). In addition, the American Water Works Associate (AWWA) code AWWA D100-11 was utilized. The evaluation was based on original construction documents and site visit observations.

Per the City, the average operating level is between 16.5 and 24.5-feet while its overflow is at 25-feet. The overflow is located above the top of the wall and within the knuckle, approximately 3-inches below the roof plate. In order to develop recommendations pertinent to the typical operating ranges the analysis of the reservoir was conducted at overflow (the maximum operating level) to determine the reservoir's structural adequacy based on current code. The reservoir was then re-analyzed at lower operating levels to determine if a reduction in the operating level could result in the lessening or elimination of any of the noted issues.

Reservoir Shell – Material Thickness: As only a single reservoir drawing was available (covering the footing), the shell thicknesses were measured in-field via ultrasonic testing for use with PSE's analysis. As multiple measurements are taken, PSE will typically utilize the average thickness reading. The exception is in instances where a plate section within a given shell course has been found to deviate from the average by greater than 5%. In such a case, PSE will use the lower bound value as the controlling steel thickness for the analysis of that shell course.

Reservoir Steel Thicknesses				
Component	Drawing (in)	Average (in)	Minimum (in)	Value Used (in)
Shell Course 1	N/A	0.310	0.307	0.310
Shell Course 2	N/A	0.262	0.262	0.262
Shell Course 3	N/A	0.258	0.258	0.258
Floor Plate	N/A	0.250	0.250	0.250
Roof Plate	N/A	0.189	0.189	0.189
Roof Knuckle	N/A	0.276	0.276	0.276

2.3.1 Hydrostatic and Gravity Analysis

Roof Framing: The roof plate thicknesses and layout are adequate under current code for the required live and snow loads. Based on Whatcom County and City of Bellingham requirements, the ground snow load used for design was determined to be 35-psf while the design snow load was determined to be 32-psf. This is higher than the typical design snow load minimum value of 25-psf, set by AWWA D100 and by most local building departments. While the rafters are not attached to the roof plate, D100 allows for a rafter with a depth less than 15-inches to be evaluated as continuously braced. When this allowance is used, all rafters passed design checks for the 32-psf snow load. The interior 6-inch diameter center column was found to be sized appropriately per current code for this load. Overall the roof structure appears to be structurally adequate for the site's higher than typical snow loads under current code.

Reservoir Shell – Hydrostatic Stress: No documentation was available that indicated the reservoir's design method. Further the reservoir's identification plate did not indicate any potential design method of steel

grade. As a result, the reservoir is assumed to be designed per Section 3 of AWWA D100-65. This section would be similar to the design methodology as outlined in the current Section 3 of AWWA D100-11. This section uses reduced tensile strengths for the shell plates and reduced weld efficiency owing to the less-stringent inspection criteria of the section. Overall this results in a more conservative design with thicker shell courses and welds sizes.

As a back-check, values from Section 3 are utilized to check the minimum required plate thickness required for an operating height of 25-feet (as the initial design should always be for the overflow level). This check uses an AWWA defined Class 1 plate steel (Grade 15 ksi).

$$Shell\ Thickness = \frac{2.6(Operating\ Height)(Diameter)}{(Plate\ Tensile\ Stress)(Efficiency)} = \frac{2.6 \times 25ft \times 59ft}{15000psi \times 0.85} = 0.301\ inch$$

This result matches the first course shell plate thickness which was measured as 0.310-inch and the Section 3 assumption appears to be appropriate. Continuing to using Section 3 to check the remaining shell courses it was found that all plates are acceptable for the anticipated hydrostatic stresses.

Foundations: Per PSE's analysis it was determined that the reservoir has a resulting bearing pressure of 2700-psf for gravity and hydrostatic loads. Based on the Geotechnical investigation, this site has an allowable bearing capacity of 3000-psf and therefore the footing is adequate for the calculated loads. This is further confirmed by the foundation itself which was not observed to have any differential settlement or foundation cracking issues.

While the existing foundation appears to be adequate for the current operational loads, it is likely undersized for seismic loads, as will be discussed further in the Overturning section.

2.3.2 Hydrodynamic and Seismic Analysis

Reservoir Shell – Hydrodynamic Stress: As noted in the hydrostatic section, analysis of the shell was conducted following the methods allowed in AWWA D100 Section 3. See the previous hydrostatic section for additional notes. When utilizing these assumptions, the hydrostatic and hydrodynamic stress values were found to be within acceptable levels when using the as-measured steel thicknesses, and assumed steel grade, as noted above for an operating level of 25-feet. Per the current code, the reservoir's shell is adequate for the site's anticipated seismic loads.

Freeboard/Slosh: The AWWA describes the freeboard height as the distance between the top of the overflow and base of the rafters. For analysis, PSE typically utilizes a freeboard based on the distance between the top maximum operating level and the base of the rafters. Per the City, this reservoir's maximum operating level is at 24.5-feet, about 6-inches below the 25-foot overflow. As the overflow is located above the wall and extends into the rafter line this technically results in a negative freeboard condition. During a seismic event, operating at overflow would result in a slosh wave that is nearly completely constrained. For a constrained slosh wave, the wave generated by the ground motion is forced up into the roof. This slosh impact wave can damage the roof and rafters, dislodging roof plates, and damage hatches and vents. Additionally, constraining the slosh wave increases the potential for

overturning and can result in the need for anchors on a reservoir that would not otherwise need anchors for stability.

For this reservoir, the code calculated slosh wave height was determined to be 4.0-feet. This height is a function of the geometry of the reservoir and local seismic conditions. As a result, the reservoir would need to have its operating height reduced to 19.75-feet to alleviate all slosh related loads on the roof components and roof rafter supports. The reason the water level must be reduced below 21-feet is because the overflow water level is located above the wall line. The point at which the slosh wave will have an increased chance to destabilize the roof occurs at an operating height of 23.75-feet (the mid-point of the roof knuckle). As the overflow is located above this height, the 4-foot reduction would be taken from a height of 23.75-feet rather than from the 25-foot overflow level.

Should a higher operating level be required, the roof would need to be retrofitted to resist slosh impact uplift loads. While the roof plates and rafters are adequate for gravity loads, they do not have the capacity to resist the anticipated slosh loads. To resist these loads the roof plates would need to be welded to the rafters to provide a load path between the plate and rafter elements. A slosh wave resulting from a 24.5-foot operating level would exceed the roof plate and rafter's capacity if they were not replaced or heavily retrofitted and reinforced. At lower operating levels, the roof zone affected by the slosh wave is reduced and there is potential to identify an intermediate operating level which minimizes retrofit requirements. It was determined that operating at a 20.5-foot operating level would limit a slosh wave impact to the roof knuckle portion of the roof and thus would limit the required upgrades needed to the main body of the roof.

Overturning and Anchorage: The existing reservoir is in a configuration that is typically considered a non-standpipe reservoir. In this configuration the diameter of the reservoir exceeds its operating height by such a margin that it is less likely to require anchorage to resist a seismic overturning event. While additional updates to the seismic code and new information on the effects of slosh have further exacerbated the potential overturning effects due to a seismic event, this reservoir was determined to be stable against overturning even without anchors.

For a 25-foot operating level, the constrained slosh results in a 65% increase to the base overturning load. As a result, the seismic bearing on the foundation resulting from overturning is increased to 6600-psf, 2.4 times greater than the static bearing and in exceedance of the allowable short-duration-bearing pressure of 4000-psf determined by the Geotechnical investigation. Limiting the adverse effects of slosh induced overturning would require a reduction in operating height. At a reduced operating height of 22.25-feet (equal to the wall height) the bearing pressure drops to within acceptable levels.

2.4 Summary

Structurally the reservoir was determined to be adequate for current code gravity and hydrostatic loads, except as noted for slosh below. The shell plate, roof members, and footing were all determined to be adequate in size and layout. The primary structural concerns noted were a result of corrosion. The exterior of the reservoir was noted to have compromised coating and organic growth while the interior roof and rafter system was observed to have multiple instances of failed coating and corrosion. While the

reservoir's design does make it adequate for gravity and hydrostatic loads, maintenance is required before corrosion adversely impacts these structural systems, reducing their capacity and performance.

Evaluated per current seismic codes, the main body of the reservoir was found to perform reasonably well. However, unaccounted for in the original design is the impact that slosh can have on the reservoir's performance. As a result, many of the failures that were identified were noted to either be caused or exacerbated by the slosh loads. Operating at a 24.5-foot water level, the roof system and foundation would both be susceptible to failure during a seismic event.

Based upon the evaluation and aforementioned conditions, retrofit of the reservoir to function at the 24.5-foot operating level would be extensive. At a minimum, PSE would recommend a reduction in the maximum operating level in order to bring this reservoir into compliance with current seismic codes and limit required upgrades.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to potentially bring the reservoir into partial or substantial compliance with current code.

Reduced Storage Volume

The first option is to perform no structural upgrades and lower the current maximum operating level by 4.75-feet to 19.75-feet. Lowering the operating level by this amount would ensure that slosh will not impact roof elements. By eliminating slosh this would reduce foundation loads and loads on the roof. No structural upgrades would be needed at this operating level; however, corroded elements should still be repaired. Depending on the City's storage requirements, a lower operating level could result in a cost-effective way to meet current code structural requirements with minimal upgrade efforts.

Maintain Maximum Storage Volume

The second option is to continue to operate the reservoir at an operating height of 24.5-feet. In such a case PSE would recommend an evaluation of the roof to determine if the roof can be reinforced and seal welded. Based on the site observations, it appears that the roof plates have been significantly impacted by corrosion along their seams. Before any retrofit work is undertaken the lapped roof plates should be evaluated to determine the extent of corrosion along these edges. If the roof is determined to be competent, the plates can be seal welded as well as welded to the rafters to ensure a positive connection again slosh-induced uplift. This would require a full roof load analysis as the entire roof support structure, from roof plates to column base, would need to be reinforced. These retrofits would help to reinforce the roof against any slosh loads. However, please note that this retrofit may ultimately not be feasible, or it may require uneconomical upgrades.

Once the roof is reinforced, the foundation would need to be retrofitted with an expanded footing to handle the increased bearing loads resulting from the constrained slosh. For the anticipated loads, a pile system (helical or micro piles), tied into a new expanded ring foundation, may be an option to limit sitework and construction under the reservoir.

Alternately, the roof could also be raised, and new shell course installed. Adding more shell height would result in more freeboard and eliminate the impacts of constrained slosh. For this upgrade the existing roof could be saved and only the interior column would need to be replaced with a taller column. Slosh impacts would be mitigated by moving the roof out of the impact zone and thus limiting upgrade requirements. Or, as the roof has already been noted to have significant corrosion issues, the roof could be removed and replaced. Since this is a smaller diameter reservoir, it would be a good candidate for a self-supporting dome roof. A self-supporting dome roof does not require internal rafters or supports. By eliminating these features, this would remove many of the areas that are prone to developing corrosion related issues.

Intermediate Storage Volume

The third option is to operate at an intermediate level to limit slosh impact loads on the roof and lower the associated overturning and bearing loads. In this case the roof could also be replaced rather than retrofitted. A domed roof, with its curved shape, is better able to accommodate and dissipate slosh loads. A combination of a reduced operating level in conjunction with large-radius dome roof could be sufficient to meet storage demands with fewer upgrades.

In lieu of replacing the roof, the operating level could be reduced to 20.5-feet. This would be sufficient to alleviate any increased overturning effects on the foundation and limit the slosh impact on the roof to a 4-foot zone around the edge of the roof. As this zone primary consists of the roof knuckle, which is already fully welded to the top of the wall and the adjacent roof plate, loads on this area would transfer through the knuckle rather than through the rafters. This would further reduce the need for rafter upgrades and limit the extent to which upgrades would be needed on the rafter ends. Note, this wouldn't meet current code requirements since adequate freeboard wouldn't be provided so the potential for damage to the roof would still exist but would be substantially reduced.

General Site and Structural Repair Recommendations

For the overall site, PSE recommend trimming trees around the reservoir to reduce contact from the trees and the potential for damage from trees during storm events. Trimming trees will also increase air movement around the reservoir and help to reduce conditions which are deleterious to the coating.

As organic materials (moss and lichen) have built up on the exterior shell, the shell should be pressure washed to removed growth and debris. Poorly adhered paint should be removed, and the reservoir recoated.

Around the foundation the failing grout should be removed back to either competent grout or the compacted oil sand base under the reservoir. Once the gap is cleared, it should be refilled with a non-shrink grout mixture, which meets the minimum thickness requirements as outlined by the manufacturer. Where grout cannot be placed, the gap should be packed with a backer material and sealed using an exterior grade elastomeric flexible caulking.

On the interior of the reservoir, plates with compromised sections, such as observed around the vent and hatch, should be removed and replaced. Any welds with a "jagged" profile, such as around the hatch,

should be ground smooth and repaired. Rafter ends should be re-checked and where bolts are missing, or they are missing nuts, new nuts or bolts installed.

Ancillary structures observed on the site appeared to be in generally good condition. Further, their proximity to the reservoir was not such that they could damage or weaken the structure should they fail. No repairs are proposed for these structures beyond typical maintenance.

2.6 Scans of Select Construction Documents

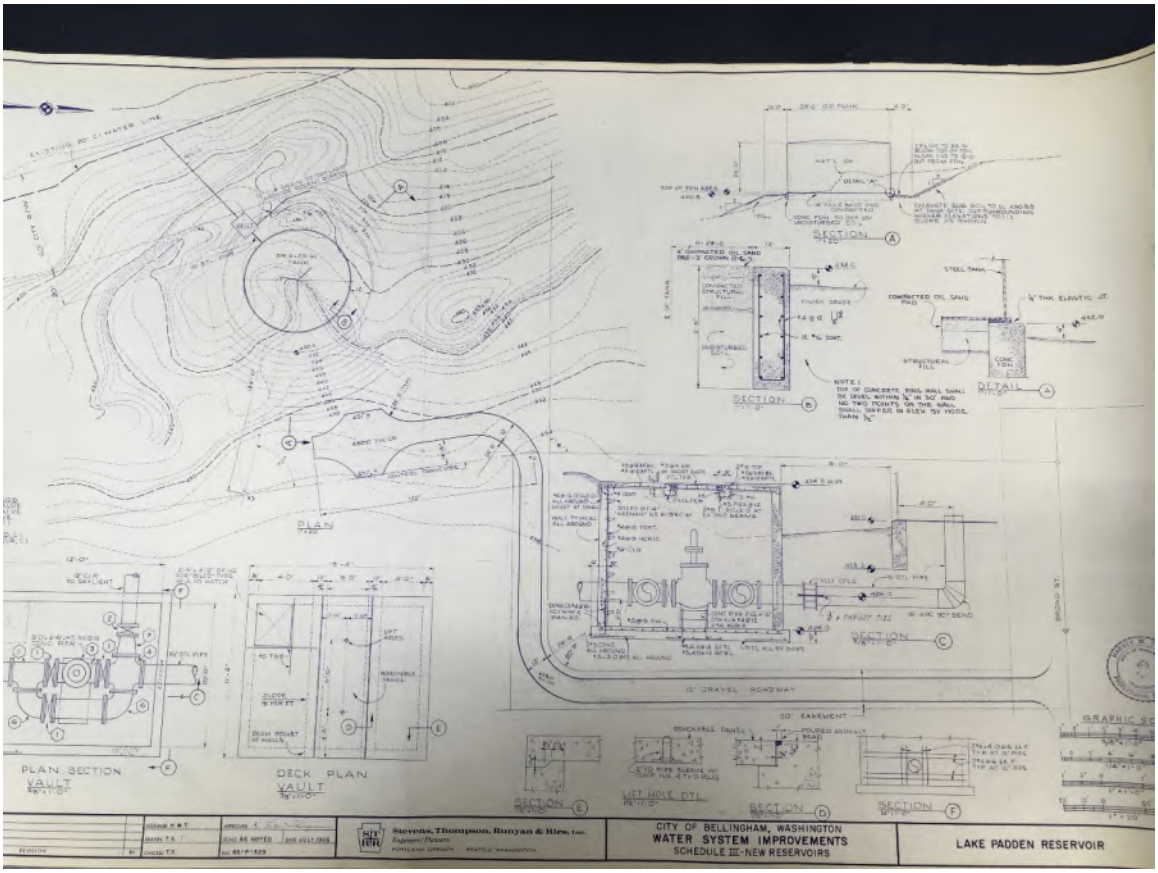


Figure 2-1: Padden – Lake Padden Reservoir (only drawing sheet available, this sheet has footing information)

2.7 Observations Pictures

Figure 2-2: Padden Reservoir – Elevation

Figure 2-3: Padden Reservoir – ID Plate

Figure 2-4: Padden Reservoir – Pump Station

Figure 2-5: Padden Reservoir – Altitude Vault

Figure 2-6: Padden Reservoir – Access Ladder and Wall Staining (South Side)

Figure 2-7: Padden Reservoir – Close-up of Ladder Enclosure

Figure 2-8: Padden Reservoir – Staining, Moss Growth, and Branch Rubbing (West Side)

Figure 2-9: Padden Reservoir – Overflow Outlet Pipe

Figure 2-10: Padden Reservoir – Shell Staining (North Side)

Figure 2-11: Padden Reservoir – Failure of Grout at Base (multiple locations noted)

Figure 2-12: Padden Reservoir – Coating Delamination from Base Layer (Southeast Side)

Figure 2-13: Padden Reservoir – Roof Vent

Figure 2-14: Padden Reservoir – Roof Access Hatch and Corrosion

Figure 2-15: Padden Reservoir – Rafter End by Roof Hatch, Corrosion and Missing Nut

Figure 2-16: Padden Reservoir – Roof Layout, Corrosion Noted Along Plate Seams

Figure 2-17: Padden Reservoir – Inlet/Outlet/Drainpipe Corrosion (Pipe Collar to Right)

Figure 2-18 Padden Reservoir – Overflow Pipe

Figure 2-19: Padden Reservoir – Floor Plate, Coating Bubbling (multiple locations noted)

Figure 2-20 Padden Reservoir – Wall Plate, Coating Bubbling (multiple locations noted)

2.8 Field Notes

WELDED STEEL RESERVOIR SITE INSPECTION

Project #: A1802-0019 Project Name: Padden Bellingham Reservoir Inspection

Site Visit Date: 4/9/19 Reservoir Type: Steel 2606 38th St, Bellingham, WA 48.7070, -122.4680

Site Conditions: Sunny 48°F, wet from recent rains but clear

Exterior Inspection

Number of Steel Shell Courses: 3 Standard Height of Shell Courses: 7'-5" (89")

Shell Course Height: (top) ____, ____, ____, ____, 89", 89", 89" (bottom)

Knuckle (yes/no) Radius 3' Thickness 0.276

Shell Course Thicknesses: (top) ____, ____, ____, ____, 0.258, 0.262, 0.311 (bot) 0.307 0.311

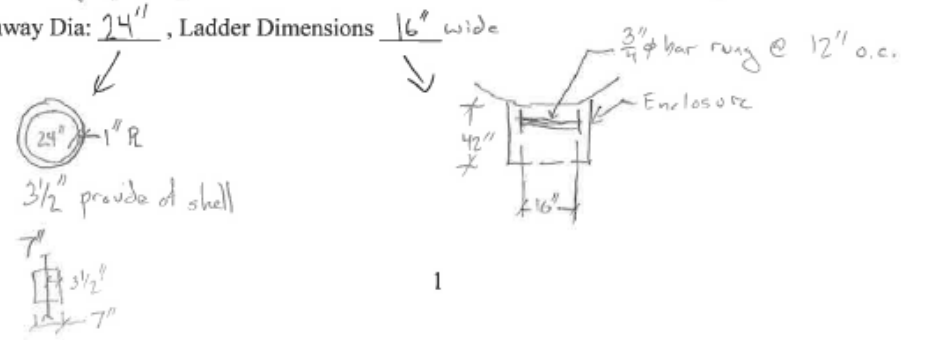
Condition of Exterior Shell and Coatings (check for location of paint delamination and rust):

Poor. Significant amount of moss on W. & N. side with loss of primary coating to base on E. & S. sides, multiple areas of delam. and chipping

List location of all external items (pipes, manways, ladders, etc.) on drawing page. Locate items using pace count method.

Condition of Ladder/Vents/Hatch/Welds: Overflow: Exterior coating intact, functional but w/ discolored; Welds: lower quality, more overlap and double lines, few notes of burrs or edges; Manway: signs of corrosion at base on at locking point; Ladder: corrosion at flats otherwise gen. good.

Manway Dia: 24", Ladder Dimensions 16" wide



1

WELDED STEEL RESERVOIR SITE INSPECTION

Exposure and Condition of Foundation: Varries from 8" ex. on NW side to 3" ex. on S. Exposed agg. footing w/ moss growth. Some cracks but no differential cracking/settlement. Combo of caulking & grout used between bottom PL and top of foundation.

Grade Relative to top of foundation 3 to 8, List max/min on drawing page.

Pothole at footing. Depth of Footing ____ Dist from anchor CL to footing edge N/A

Top Surface Roof Plates & Coating Condition (check for paint declamation and rust):

Good Condition although many instances of moss or other growth

Thickness of roof plates 0.187, Slope of Roof* 4.5°

Interior Inspection

Tank Diameter 59', Distance from Wall to Mid-span Rafter Support N/A

Column Diameter/Size: OD 6.7"

Interior Footing: (4) C-Channels ← configuration

8x8 PL

Column Spacing/Configuration: (1) column c+c

Condition of Interior Shell and Coatings: CP system too high causing paint to blister but underlying steel good.

Bottom of Roof Plate/Framing/Coatings Condition: Multiple loc. of corrosion at joint lines. Rafter ends have corroded/missing bolts. Condensation/corrosion at vent.

Floor Plate Condition/Thickness/Coatings: Coating is blistering but underlying steel good.

PSE
PETERSON STRUCTURAL ENGINEERS

WELDED STEEL RESERVOIR SITE INSPECTION

Ladder/Pipes/Overflow Conditions: Ladder: good; Overflow: good; Outlet: corrosion in pipe

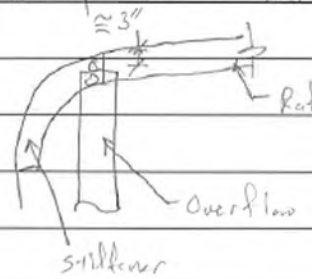
Same Pipe

Diameter of Inlet 15" ID, Outlet 15" ID Overflow 8.75 OD

Distance of Top of Overflow from base of roof elements 3" to bot. of roof plate

Other Comments: Operating Range 16.5'-24.5', overflow 25'

Overflow operates in rafter line.



3

WELDED STEEL RESERVOIR SITE INSPECTION

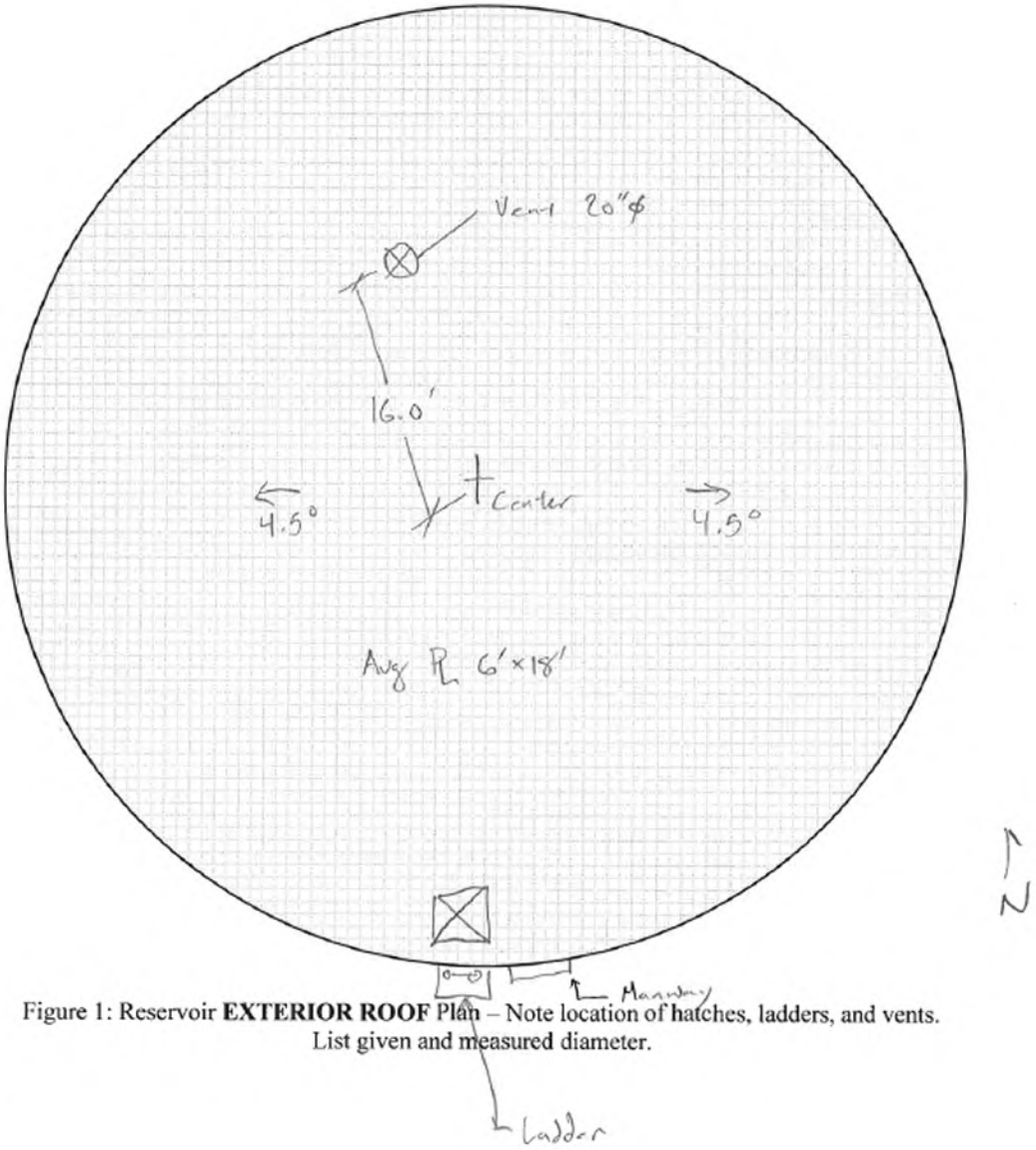


Figure 1: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and vents. List given and measured diameter.

WELDED STEEL RESERVOIR SITE INSPECTION

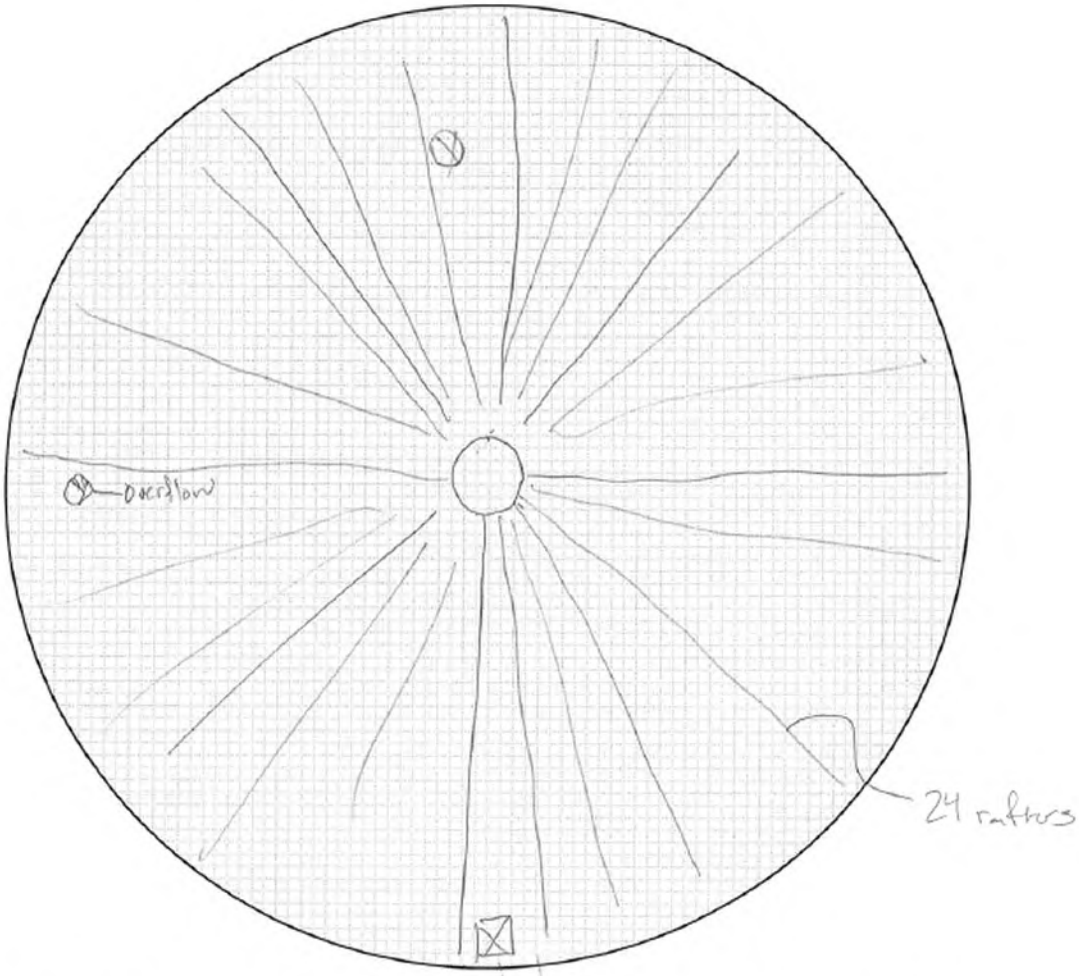


Figure 2: Reservoir **REFLECTED ROOF AND RAFTER** Plan – Sketch location of roof supports, hatches, vent, columns, etc. List given and measured diameters.

Use Figure 5 for roof edge notes.
Use Figure 6 for component dimensions

WELDED STEEL RESERVOIR SITE INSPECTION

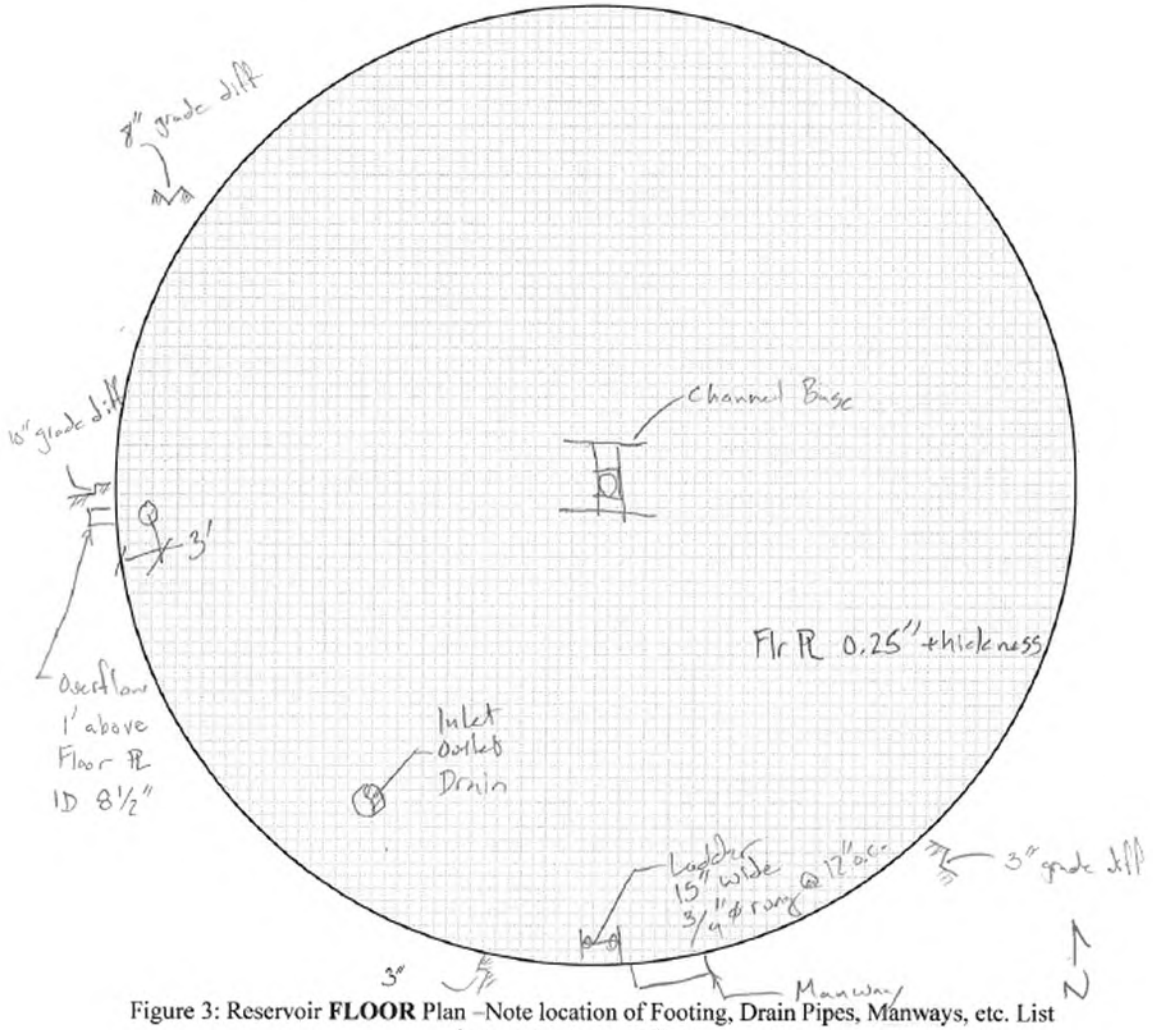


Figure 3: Reservoir **FLOOR** Plan –Note location of Footing, Drain Pipes, Manways, etc. List given and measured diameter.

WELDED STEEL RESERVOIR SITE INSPECTION

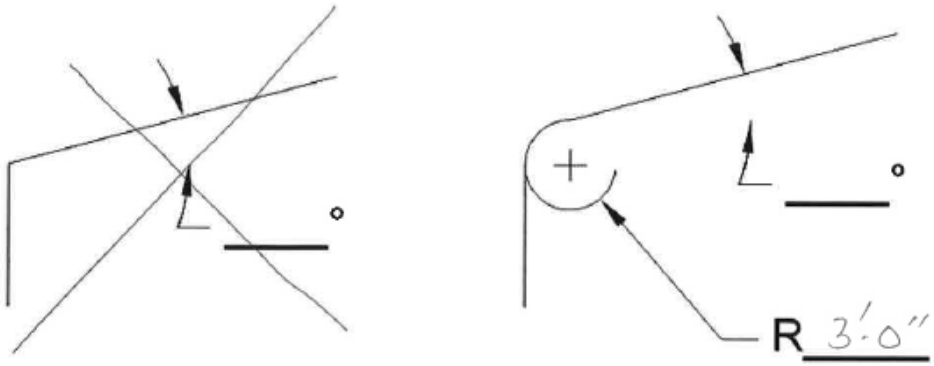
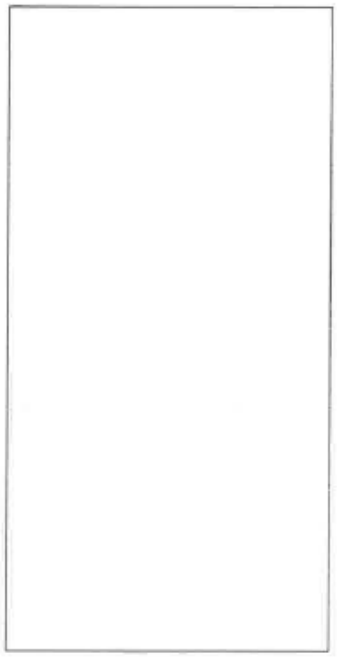


Figure 4: Reservoir Roof Edge – Select appropriate figure and sketch roof edge configuration.

Sketch Roof Type



Thickness: _____
Height: _____

Thickness: _____
Height: _____

Thickness: _____
Height: _____

Thickness: _____
Height: _____

Thickness: _____
Height: _____

Thickness: _____
Height: 7'-5"

Thickness: _____
Height: 7'-5"

Thickness: _____
Height: 7'-5"

Figure 5: Exterior shell information and notes.

WELDED STEEL RESERVOIR SITE INSPECTION

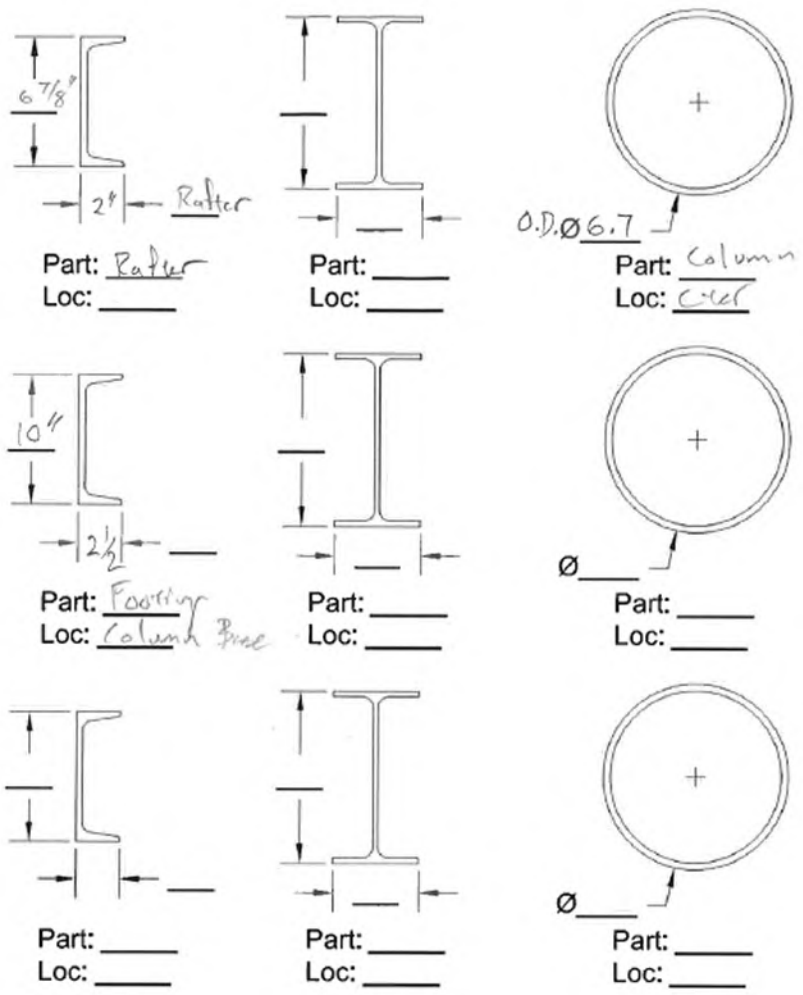
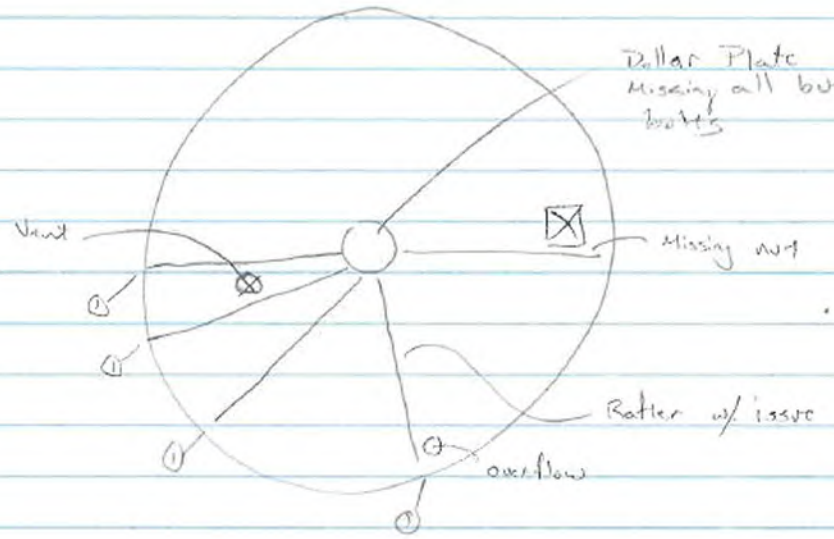


Figure 6: Component detail notes.

Padden – Float Notes 4/30/19

Rafters, Bolt issues near rafters by vent & overflow

overflow
2 1/8" clear



① Ends are short, require long slots for bolts

END OF SECTION

Appendix B-4 Padden General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Padden Reservoir *General Info*

Field Visit Date: 4/8/2019

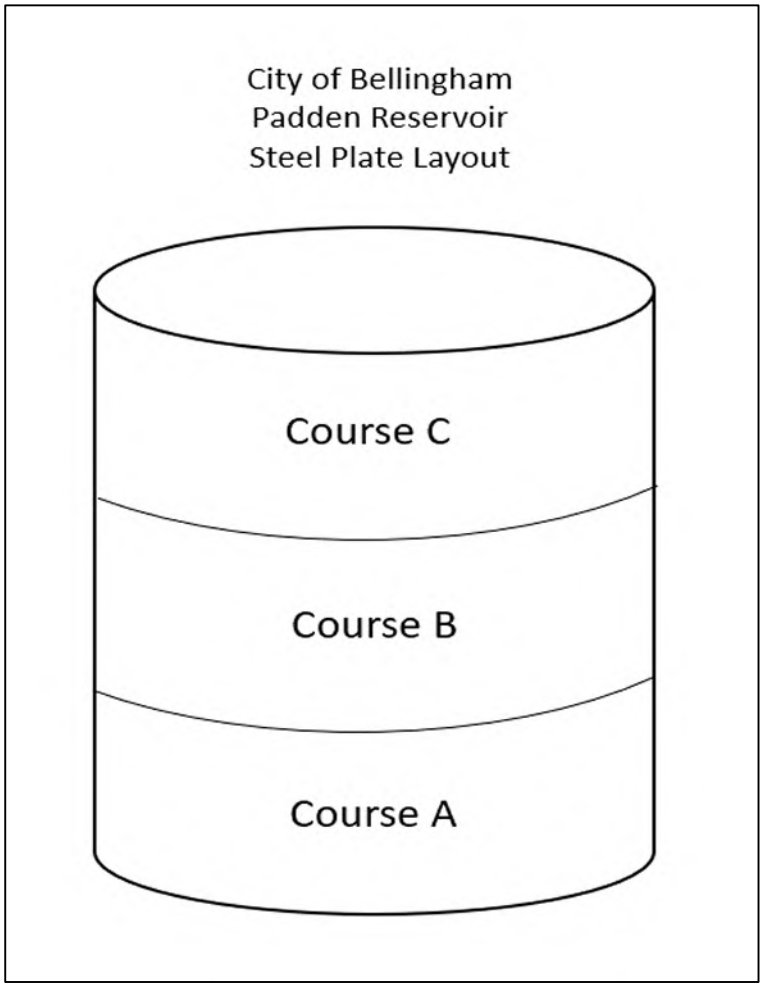
Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	4/8/2019
Reservoir Name and Location:	Padden - Behind/SW of 3810 Broad St 98229
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis
Client Staff Present:	Shayla Francis, Steve Bradshaw, Nick Leininger, Jenny Eakins
Year Constructed:	1958
Overflow Destination:	Padden Creek
Discharge Destination/Zone:	West - from vault W to water line, common w/ fill; Drains S to 457 Zone
Fill Location:	West - from vault W to water line, common w/ discharge
Reservoir Material:	Welded Steel

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	60	ft
Height	25	ft
Overflow Elevation:	457	ft AMSL
Bottom Elevation:	432	ft AMSL
Level of Overflow	25	ft
Minimum Normal Operating Level:	16.5	ft
Maximum Normal Operating Level:	24.5	ft
Notes:		

Padden Reservoir *Field Notes: Steel Plates*

Field Visit Date: 4/8/2019

Course	Average Steel Plate Height (inches)	Steel Plate Thickness (Inches)	Notes
Roof		0.189	
Knuckle		0.276	3 foot radius
C	89.0	0.258	
B	89.0	0.262	
A	89.0	0.311	
Notes:			



Padden Reservoir *Exterior Inspection*

Field Visit Date: 4/8/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Welded Steel	
Condition:	Fair	
Corrosion:	Yes	
Cage:	Yes	
Security Type:	locked access panel	
Security Condition:	Good	
Wall Attachment Type:	Welded	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	15	in
Rung Spacing:	12	in
Side Clearance:	9.5	in
Front Clearance:	7.5	in
Back Clearance:	2.5	in
Notes: Some corrosion on handles near roof.		

Exterior Fall Prevention System:	
Present at Site:	Yes
Type:	Cage
Fall Protection System Condition:	Fair
Notes: Cages no longer OSHA compliant.	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:	
Hatch Location:	Roof - South
Material:	Welded Steel
Condition:	Fair
Gasketed:	Yes
Intrusion Alarm:	Yes
Lock:	No
Frame Drain Location:	South

Measurement Type	Measurement	Unit
Size:	24	in
Curb Height:	6	in
Notes:		

Entry Hatch:	
Hatch Location:	Side - south
Material:	Welded Steel
Condition:	Fair
Gasketed:	Yes
Intrusion Alarm:	No
Lock:	No
Frame Drain Location:	Downward

Measurement Type	Measurement	Unit
Size:	2	ft
Curb Height:	3	in
Notes: Signs of corrosion at base at locking point. No alarm as it is held in place with water.		

Roof Vents and Screen:	
Material:	Steel
Condition:	Good
Prevents Water Intrusion:	Yes

Measurement Type	Measurement	Unit
Structure Size:	18	in
Screen Size:	1/24	in
Notes: top overhang 2ft 8in. Bird and bug screen. Vent located off center. Organic growth on top.		

Roof:	
Condition:	Fair
Roof Sloped:	Yes
Downspouts:	No
Ponding on Roof:	Yes
Roof Finish:	Coated
Slope of roof	4.5 degrees

Measurement Type	Measurement	Unit
Overhang Distance:	0	in
Notes: Moss growing on roof. Ponding in one or two locations. Coating peeling.		

Railing:	
Present at Site:	No

Grating:	
Present at Site:	No

Foundation:		
Able to be inspected?	Yes	
Condition:	Good	
Anchoring Condition:	N/A	
Photo of Anchoring System:	No	
Flexible Couplings at Foundation:	No	
Measurement Type	Measurement	**Unit**
Exposed foundation (northwest)	8	in
Exposed foundation (south)	3	in
Notes: Foundation extends 3.5' into subsurface. Moss growth present. Some cracks, but no differential cracking/settlement. Missing grout.		

Walls:	
Condition:	Fair
Notes: Welds are lower quality. Overlap and double lines present. Burrs and edges also shown.	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	Unknown
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	Yes
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	Walls: 6.6 - 14.3 mils Roof: 3.5 - 9.6 mils
Exterior Coating Adhesion Testing Results:	Top coat lightly adhered, underlying moderate adhesion
Notes: Coating condition is poor. Significant amount of moss on west and north sides. Primary coating is exposed to base on east and west sides. Delamination and chipping present throughout. Lead test negative.	

Padden Reservoir *Interior Inspection*

Field Visit Date: 4/8/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Welded Steel	
Condition:	Fair	
Corrosion:	Yes	
Cage:	No	
Security Type:	locked hatch	
Security Condition:	Good	
Wall Attachment Type:	Welded	
Wall Attachment Condition:	Fair	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	15	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	N/A	in
Back Clearance:	N/A	in
Notes: Coating blisters near wall attachment points. Corrosion noted near top of ladder.		

Interior Fall Prevention System:	
Present at Site:	No

Interior Roof:		
Condition:	Poor	
Measurement Type	Measurement	Unit
N/A	N/A	N/A
Notes: A high amount of corrosion, located mostly at sheet metal joins. Also present at vent. 24 rafters. Rafter ends have corroded/missing bolts.		

Columns:		
Present at Site:	Yes	
Material:	Welded Steel	
Condition:	Good	
Measurement Type	Measurement	Unit
Width/Diameter	6.7	in
Base width	See notes	in
Column Spacing/Configuration: One column in middle of reservoir. I-beam dimensions: 35 in tall, 36 in wide, 10 inches off the ground		

Floor	
Condition:	Fair
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: Water ponding/sedimentation on north side. Top coat wear at inlet/outlet	

Walls:	
Condition:	Fair
Painters Rings Present:	Yes
Notes: painter's ring present at overflow. some staining	

Interior Coating	
Interior Walls:	Unknown
Interior Floor:	Unknown
Interior of Roof:	Unknown
Interior Ladder:	Unknown
Interior Piping:	Unknown
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	Walls: 5..2 - 10 mils Floor: 5.5 - 14.5 mils
Interior Coating Adhesion Testing Results:	Walls and floor: fair. Roof: poor
Notes: Coating blisters near ladder, floor, seams due to elevated cathodic protection. Lead test negative. Roof major coating failure.	

Padden Reservoir *Miscellaneous*

Field Visit Date: 4/8/2019

Piping		
Inlet Piping:	Size (Inches OD):	16
	Condition:	Fair
	Material:	Cast Iron
	Notes: Combined inlet/outlet/drain	
Outlet Piping:	Size (inches OD):	16
	Condition:	Fair
	Material:	Cast Iron
	Lip (Inches)	0.5
	Notes:	
Overflow Piping:	Size (inches OD):	8
	Condition:	Fair
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	Padden Creek
	Erosion Evident:	No
	Screen Condition:	Poor
	Overflow to roof (feet)	3 in
	Notes:1/4 in screen.	
Drain Piping:	Size (inches OD):	16
	Condition:	Fair
	Outlet Location:	Padden creek via pipe under path
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	Removable ring
	Air Gap:	No
	Screen Condition:	N/A
	Notes: Outlet pipe at creek. Screen missing. Silt stop 5.5 inches, but it was not flush with floor.	

Piping Facilities		
Exterior Valving:	Type:	Butterfly valve
	Condition:	Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	No
	Secured:	N/A
Washdown Piping	Location:	Near ladder
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	Unknown
Notes: Fire hydrant used for washdown		

Electrical	
Cathodic Protection:	Yes
Impressed Current:	Yes
Anodes:	Yes
Notes: Not variable voltage	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	Yes
Common Inlet/Outlet:	Yes
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	none
Sediment Build-Up Height Above Floor (in)	0.2
Water Quality Sample Taps?	Yes
Notes: Altitude valve serves as check valve. A small pipe next to inlet/outlet/drain pipe for WQ sampling.	

Appendix B-5 Padden Condition Assessment Score Sheet

Padden Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No Camera. Evidence of Vandalism
	Vegetation Separation	0	0	0	0	0	0	2	0	Trees overhang and are hitting walls
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	1	4	5	5	0	0	5	0	Top coat peeling, prime coat exposed in some areas
	Interior Walls	4	5	5	5	5	0	5	0	Blistering no corrosion Below WL; Corrosion is worse on roof
Floor/ Foundation	Foundation	5	4	5	2	0	0	5	0	Const. slosh increases overturning- anchors needed. Foundation loads higher than acceptable
	Interior Floor	4	4	5	2	4	0	5	0	
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	2	0	0	0	0	
Roof	Exterior Roof	3	4	5	2	5	0	3	0	Slosh will impact and damage the roof. Needs cleaning
	Interior Roof and Supports	2	1	5	2	0	0	0	0	No seal welds - lots of coating loss and corrosion
	Columns	4	4	5	2	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	3	3	0	0	0	3	5	0	25.25 ft tall - fall protection required. Old style cage
	Interior Ladders/Fall Protection	3	3	0	0	0	1	2	0	25.25 ft tall - fall protection required. Evaluate corrosion on ceiling weld.
	Access Hatches	2	2	0	0	3	3	2	0	Sig. corrosion on interior of roof hatch edge. Needs second side hatch; exist. hatch is too small. Roof hatch high maint. And needs roof hatch railing
	Railings and Roof Fall Protection	0	0	0	0	0	2	0	0	System required for roof workers.
	Vents	3	5	0	0	5	0	4	0	Passed design checks, but not in middle of reservoir. Screen looked compliant. Corroded on int.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	2	3	0	0	4	0	5	0	Comb in/out/drain. Pipe very corroded
	Outlet Piping	0	0	0	0	0	0	5	0	Comb in/out/drain.
	Drain Piping	0	0	0	0	3	0	3	0	Comb in/out/drain. Silt stop not fully functional. Missing screen. Enters Padden Creek
	Overflow Piping	5	4	0	0	4	0	5	0	Non compliant screen
	Washdown Piping	0	0	0	0	0	0	5	0	Fire hydrant on site.
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	4	4	0	0	5	0	5	3	Altitude valve
	Isolation Valving	5	5	0	0	5	0	5	3	
Misc.	Cathodic Protection System	0	0	0	0	0	0	3	3	Constant voltage model
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	3.3	3.7	5.0	2.8	4.3	2.3	4.2	3.0	

Overall Score
3.6

Appendix C Whatcom Falls I

Appendix C-1 Whatcom Falls I Geotechnical Report

GEOENGINEERS

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Whatcom Falls I Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Whatcom Falls I reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at Whatcom Falls I site, located as shown in the Vicinity Map, Figure 1. The Whatcom Falls I reservoir is a steel reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geologic Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. Undifferentiated glacial deposits are mapped nearby.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift.

Previous Studies and As-built Information

We reviewed the original report for the Whatcom Falls I reservoir project titled, "Subsurface Exploration and Geotechnical Engineering Study, Proposed Whatcom Creek Reservoir, Bellingham, Washington" completed by Hart Crowser & Associates Inc. (HC) dated March 24, 1982. Five test pits (TP-1 through TP-5) were completed near the proposed Whatcom Falls I reservoir footprint to depths of 9 to 14.5 feet below ground surface (bgs). The test pits encountered loose topsoil/weathered horizon overlying dense silty sand interpreted to be glacial till. Seismic refraction was also completed indicating bedrock would likely be encountered near Elevation 255 feet (which is near the proposed foundation elevation as described below).

The as-builts for the Whatcom Falls I reservoir are dated May 1982 and indicate that the grade at the reservoir footprint was excavated a significant depth to subgrade elevation 260 feet and footings extending to approximately elevation 254 feet (City Datum).

Surface Conditions

The project site is located approximately 250 feet to the south of Arbor Court. The reservoir is located on top of a small hill and the site drops off in all directions, located within Whatcom Falls Park. The site is bounded by a wooded area in all directions. A small dirt road accesses the site from the east.

Subsurface Exploration

No new explorations were completed as part of this study for this site. The locations of the previous explorations are shown in the Site Plan, Figure 2. The test pits logs from the previous study are presented in Appendix A.

Subsurface Conditions

A general description of each of the soil units encountered at the project site is provided below.

- **Forest Duff/Topsoil** – Forrest duff, topsoil and/or an upper weathered soil horizon was encountered at the surface of all explorations. This upper layer extended to 1.5 to 3 feet bgs. The weathered soil is described as loose silty sand with roots and organics. In all excavations a slight to moderate seepage was found in this layer.
- **Glacial Till** –The glacial till consists of very dense gravely silty sand.
- **Chuckanut Sandstone** – Chuckanut sandstone was not encountered in the test pits at the reservoir location, however it was identified using geophysical methods near the planned base of the excavation as part of construction of the Whatcom Falls I reservoir.

Groundwater

The test pits noted perched groundwater seepage was observed above the native till between 1.5 to 3 feet bgs. The Chuckanut sandstone unit and glacial till commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Based on review of the exploration logs and the as-built drawings, overburden soil units (topsoil/weathered horizon and glacial till) encountered in the test pits were excavated as part of site grading for the Whatcom Falls I reservoir. The seismic refraction results indicated that bedrock was encountered just above the finished floor elevation. Therefore, we conclude that the Whatcom Falls I reservoir is founded on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds

of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located within the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on bedrock which is not at risk of liquefaction.

AWWA/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on publications D100-11 of the AWWA and the ASCE 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. AWWA AND ASCE 7-10 PARAMETERS

AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
AWWA Seismic Use Group	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	95.0
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.3
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.427
MCE_G peak ground acceleration, PGA	0.393
Seismic design value, S_{DS}	0.646
Seismic design value, S_{D1}	0.355
MCE_G peak ground acceleration, PGA	0.393
Seismic design value, S_{DS}	0.646
Seismic design value, S_{D1}	0.355

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are near the

Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 3 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the M 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey

Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 4 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

We anticipate that the existing Whatcom Falls I reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral resistance is not necessary for an above grade steel reservoir.

Below Grade Walls

The Whatcom Falls I reservoir does not include below grade walls.

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the south that is approximately 20 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

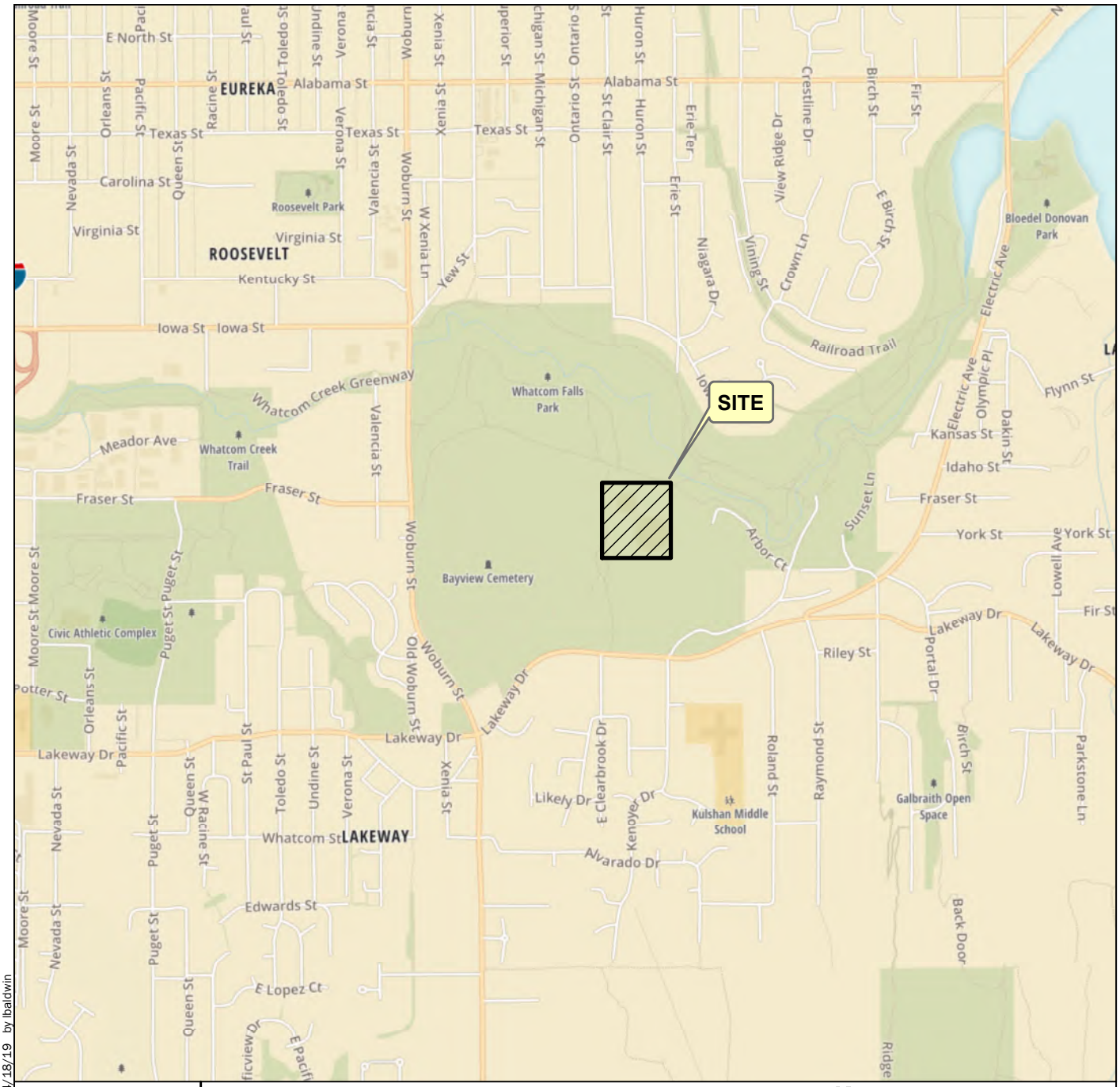
Figure 2 – Site Plan

Figure 3 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 4 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault

Appendix A. Log of Explorations

Figures A-1 through A-3 – TP-1 through TP-6 (Hart-Crowser & Associates, Inc.)



N
W E
S

1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

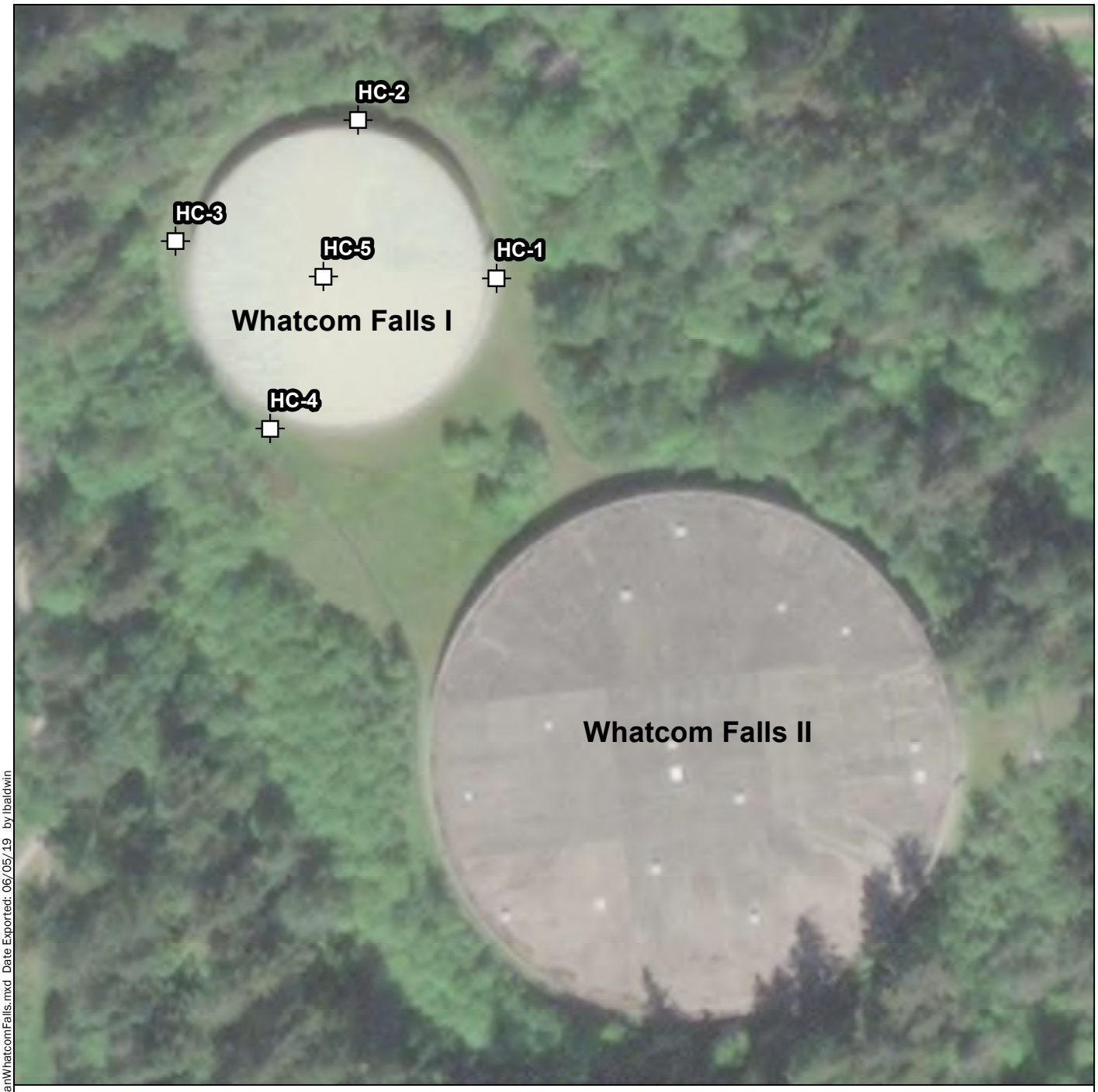
Whatcom Falls Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_WhatcomFalls.mxd Date Exported: 04/18/19 by lbaldwin



Legend

Test Pit by Hart Crowser (1982)

100 0 100

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Whatcom Falls I Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

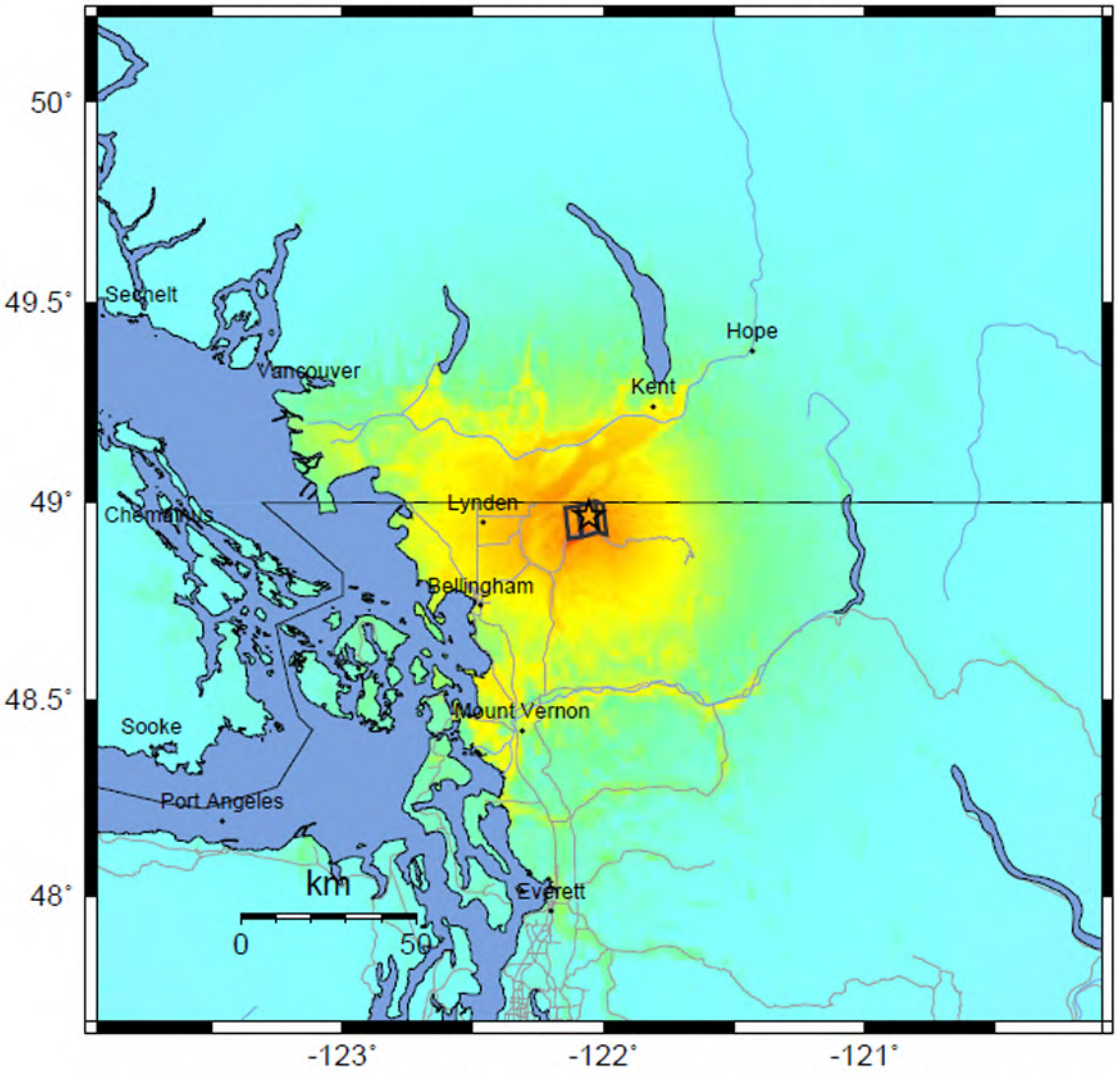
GeoEngineers

Figure 2

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

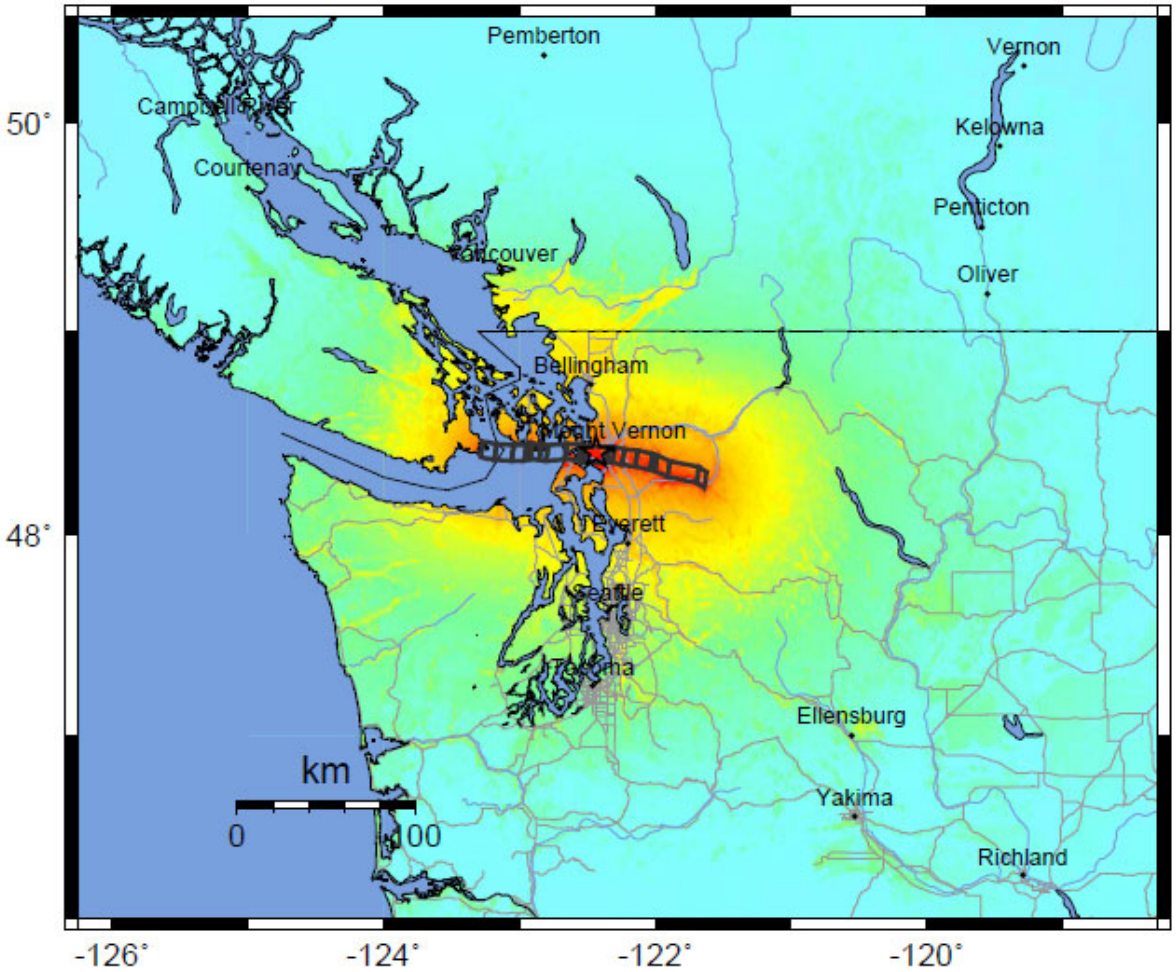
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

APPENDIX A
Log of Explorations

Test Pit Log TP-1

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
			0	Ground Surface Elevation Approximately 260 Feet
				Forest duff
			1	Loose, wet, mottled, very silty SAND with many roots, root casts, up to 4 inches diameter. (TOPSOIL)
S-1	16	#200 wash	2	
			3	Moderate seepage.
				Dense, moist, brown, gravelly silty SAND (TILL).
S-2	13		4	
			5	
			6	—Density increases below 6 feet.
			7	
			8	—Dense to very dense, moist, brown, gravelly very silty SAND with cobbles up to 8 inches diameter.
S-3	11	#200 wash	9	
				Refusal.
			10	Bottom of Test Pit at 9.0 Feet.
			11	Completed 2/27/82.
			12	
			13	
			14	
			15	

Test Pit Log TP-2

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
			0	Ground Surface Elevation Approximately 274 Feet
				Forest duff
S-1	23		1	Loose, wet, mottled brown, silty SAND with many thin roots, (TOPSOIL).
			2	Weathered to 2 feet, slight seepage at 2.5 feet.
			3	Dense, moist, gray, brown, gravelly silty SAND with cobbles up to 3 inches diameter (TILL).
S-2	16		4	
			5	— Below 5 feet density increases.
			6	
			7	
			8	
			9	
			10	
			11	
			12	— Dense to very dense, moist, gray-brown, very silty SAND, with cobbles.
S-3	11	MA	13	
				Bottom of test pit at 13.0 feet.
			14	Completed 2/22/82.
			15	

J-1089 March 1982
HART-CROWSER & associates inc.
Figure A-1

Test Pit Log TP-3

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
				Ground Surface Elevation Approximately 268 Feet
			0	Forest Duff.
			1	Loose, wet, mottled, brown, very silty SAND with many roots up to 1 inch diameter (TOPSOIL).
S-1	12		2	
			3	Dense, moist, brown-gray, gravelly, silty SAND with boulders up to 3 feet diameter (TILL).
S-2	12		4	
			5	
			6	— Slow, but not difficult excavation, friable soil.
			7	
			8	
			9	
			10	
			11	— Dense to very dense, moist, brown-gray, gravelly, very silty SAND.
S-3	11	#200 wash	12	Bottom of test pit at 12.0 feet. Completed 2/22/82.
			13	
			14	
			15	

Test Pit Log TP-4

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
				Ground Surface Elevation Approximately 276 Feet
			0	Forest Duff
			1	Loose, wet, mottled, brown, very silty SAND with many roots up to 1 inch diameter. (TOPSOIL)
S-1	29		2	Moderate seepage at 2.5 feet.
			3	Dense, moist, gray-brown, gravelly, silty SAND. (TILL)
S-2	12		4	
			5	
			6	
B-1	12	Std. Compaction	7	
			8	—Cobbles up to 8 inch diameter.
			9	
			10	
			11	Dense to very dense, moist, blue-gray, gravelly, silty SAND.
			12	
			13	
S-3	9		14	
			15	Bottom of test pit at 14.5 feet. Completed 2/22/82.

J-1089 March 1982
HART-CROWSER & associates inc.
Figure A-2

Test Pit Log TP-5

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
				Ground Surface Elevation Approximately 275 Feet.
			0	Disturbed forest duff.
S-1	36		1	Very Loose, saturated, mottled brown, organic sandy SILT with many thin roots. (TOPSOIL). Moderate seepage at 2.0 feet.
			2	Dense, moist, gray-brown, gravelly, silty SAND (TILL).
S-2	13		3	
			4	
			5	
			6	
			7	
			8	
			9	
			10	Dense to very dense, moist, gray, gravelly, silty SAND.
			11	
			12	Slight seepage at 11 feet.
S-3	10		13	
			14	Bottom of Test Pit at 13.5 feet. Completed 2/22/82.
			15	

Test Pit Log TP-6

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
				Ground Surface Elevation Approximately 241 Feet.
			0	Silty forest duff.
S-1	30		1	Loose, wet, dark brown, organic, sandy SILT with many roots up to 1 inch diameter. (TOPSOIL).
			2	Medium Dense, moist, brown, slightly silty, fine SAND with localized cuttings of sandstone (weathered sandstone?).
S-2	18	MA	3	
S-3	—		4	
			5	Soft to medium hard SILTSTONE.
			6	Refusal. Bottom of Test Pit at 4.3 feet. Completed 2/23/82.
			7	
			8	
			9	
			10	
			11	
			12	
			13	
			14	
			15	

J-1089 March 1982
HART-CROWSER & associates inc.
Figure A-3

Appendix C-2 Whatcom Falls I Corrosion and Coatings Report

June 30, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Whatcom Falls #1 and #2 Tank Cathodic Protection System Checkout

Mr. Hardy,

Northwest Corrosion Engineering completed a checkout of the cathodic protection systems associated with the City of Bellingham's Whatcom Falls #1 steel water storage tank and Whatcom Falls #2 concrete water storage tank.

BACKGROUND INFORMATION

Whatcom Falls #1 Tank

Whatcom Falls Tank #1 was built in 1982 and is 17-ft tall, 200-ft in diameter and constructed of welded steel. The impressed current cathodic protection system is comprised of an autopotential rectifier and 60 high silicon cast iron anodes supported from the roof. The anodes are arranged in three rings with diameters of 40, 120, and 180-ft. Each ring contains 20 anodes which are connected to a common header cable.

Whatcom Falls #2 Tank

The Whatcom Falls #2 Tank, constructed in 1995, is 22-ft tall, 350-ft diameter and consists of spiral reinforced concrete. An impressed current cathodic protection system is installed for corrosion control of submerged and concrete embedded metallic tank components. The configuration and type of anode material is unknown.

TEST PROCEDURES AND ANALYSIS

Current Output

System current output is recorded by measuring the voltage drop across the calibrated current measuring shunt installed on the rectifier panel board. As an additional check, a portable clamp-on ammeter is used to measure the current flow in the anode header cable.

Structure-to-Electrolyte Potentials

Structure-to-Electrolyte potentials were recorded between the tank structures and a portable copper-copper/sulfate (CSE) reference electrode. Data was collected along the interior of each tank from the water surface level to the bottom of the tank at two-foot intervals using the roof access hatches. Potentials were measured using a high impedance digital voltmeter. Potentials recorded during this survey included:

ON – The ON potential is recorded between the structure and a reference electrode while the cathodic protection system is in operation. This value gives an indication as to whether the structure is receiving current from the anodes. However, this measurement includes an error introduced into the circuit as a result of current flow. Because of this error, these values are only used when it is not possible to completely disconnect all current flow sources or when comparing previously established ON and Instant Off readings.

Instant Off – This measurement is the actual electrical potential between the structure and reference electrode. To measure this value, the cathodic protection system is momentarily switched off, resulting in a polarized potential, free of error that establishes if adequate cathodic protection is being provided.

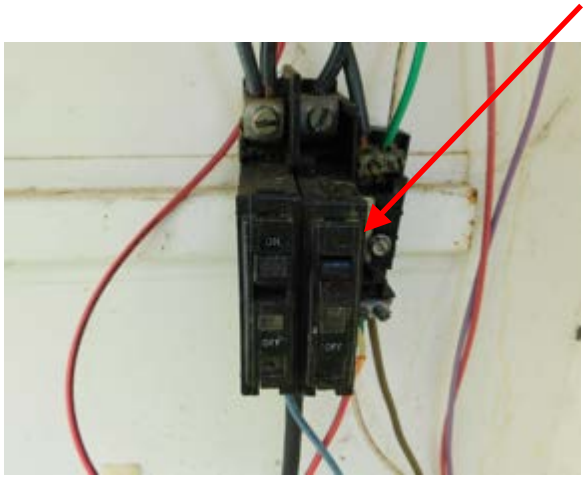
Rectifier and Structure-to-Electrolyte data collected during this survey is presented in Tables I and II at the end of this report.

RESULTS

Whatcom Falls #1 Tank

Using a portable reference electrode, all potential measurements meet NACE criteria for effective corrosion control[1]. The stationary reference electrode used to operate the rectifiers autopotential circuitry appears to be failing as its readings are significantly more positive than normal.

Upon our arrival, it was noted that the rectifier was not functioning. Troubleshooting resulted in determining that the AC breaker for the unit was defective. As a temporary measure, the AC conductors for the rectifier were landed on an adjacent breaker in order to test the cathodic protection equipment. After testing, the rectifier was left terminated to the operating breaker, however, a dedicated breaker needs to be provided for the unit.



Defective rectifier breaker

Rectifier panelboard

[1] SP0388 Impressed Current Cathodic Protection of Internal Submerged Surfaces of Carbon Steel Water Storage Tanks

As part of the internal inspection of the Whatcom Falls #1 tank, it was noted that one of the anode support hangers had become detached from the roof. This should be replaced in the near future to ensure that the anode does not electrically short to the bottom of the tank.

Broken anode hanger, should be suspended from the roof

Whatcom Falls #2 Tank

The Whatcom Falls #2 tank is constructed of concrete and as such, the electrical potential requirements for corrosion control are less than the typical (-)850 instant off requirement for carbon steel. The application of excessive current to metallic materials embedded in concrete can result in damage to the reinforcing steel and spiral-wound cables. Cathodic protection current output should be limited to that which allows for approximately 100 millivolts of polarization (typically in the -500 to -600 millivolt range versus a CSE).

The Whatcom Falls #2 tank does not have readily accessible areas to make metallic connection for testing purposes. Normally, a structure connection is made to the interior ladder, roof hatch lid, or other electrically continuous component of the tank. In this case, the installed stainless steel ladder in the #1 hatch is not continuous with the reinforcing steel. When performing the potential profile survey, the negative lead connection was made to the negative terminal of the rectifier. This does not pose a problem so long as Instant Off potentials are measured as recording On readings will result in additional measurement error due to the negative terminal being a current carrying conductor.

The Whatcom Falls #2 tank does not include provisions for autopotential circuitry. The rectifier is adjusted to a maintain a constant voltage output regardless of the level of water in the tank. This can become problematic if the water level fluctuates on a regular basis as the embedded steel will be subjected to higher voltage gradients which could result in damage to these tank structural components.

RECOMMENDATIONS

1. Replace defective AC breaker for the Whatcom Falls #1 tank rectifier.
2. Replace aging stationary reference electrode used to operate the autopotential circuitry of Whatcom Falls #1 tank.
3. Repair defective anode hanger on Whatcom Falls #1 tank.
4. Install a new autopotential rectifier, stationary reference electrode, and sensing ground for the Whatcom Falls #2 tank.
5. All cathodic protection systems should be inspected by a qualified Corrosion Engineer on an annual basis.

We appreciate the opportunity of assisting you with this project. Please feel free to contact our office if you have any questions or would like additional information.

Sincerely,
Northwest Corrosion Engineering

Jeremy A. Hailey, P.E

TABLE 1A: Rectifier Data – Whatcom Falls #1

Manufacturer: Goodall
Model: CRAYSA 60-22 FNPRSZ-0
Serial No: 8601683

Name Plate Data: AC Volts 115 DC Volts 60
AC Amps 17.7 DC Amps 22
60 Hertz, Single Phase, Amb. Temp 45ºC
Max Tap Setting – Coarse E, Fine 5
Shunt Rating – 50mV/30A

As-Found

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	5.0	3.78
Amps DC	1.0	0.320
Set Potential	-1000	-1207 ON
Read Potential	-1250	-502 Instant Off
Tap Setting	Course A Fine 3	-320 Depol. In 30 seconds

TABLE 1B: Structure-To-Electrolyte Potential Data

Ref. Cell Depth, ft	ON, mV	Instant Off, mV	Ref. Cell Depth, ft	ON, mV	Instant Off, mV
Top of Water	-1918	-1028	8	-1911	-1007
2	-1933	-1024	10	-1839	-1008
4	-1932	-1026	12	-1771	-999
6	-1931	-1007	14	-1753	-998

NOTES:

1. The AC breaker for the rectifier is defective and needs to be replaced.
2. The instant off potential of the stationary reference electrode is more positive than normal, however, using a portable reference electrode, all potentials are within normal range.
3. No adjustments to the rectifier output were made.
4. The rectifier was left operating in autopotential mode.
5. All potentials meet criteria for effective corrosion control.

TABLE 2A: Rectifier Data – Whatcom Falls #2

Manufacturer: RTS
Model: CSAYSA 60-8 Z
Serial No: C-991376

Name Plate Data: AC Volts 115 DC Volts 60
AC Amps 6.35 DC Amps 8
60 Hertz, Single Phase, Amb. Temp 45ºC
Max Tap Setting – Coarse E, Fine 5
Shunt Rating – 50mV/10A

As-Found

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	34	34.2
Amps DC	3.0	3.02
Tap Setting	Course C Fine 2	

TABLE 2B: Structure-To-Electrolyte Potential Data

Ref. Cell Depth, ft	ON, mV	Instant Off, mV	Ref. Cell Depth, ft	ON, mV	Instant Off, mV
Top of Water	-910	-596	10	-934	-607
2	-913	-587	12	-939	-613
4	-917	-591	14	-944	-622
6	-922	-596	16	-941	-618
8	-928	-601			

NOTES:

1. Potential data was collected at hatch #1.
2. The rectifier is a constant voltage model and is not capable of automatic current output adjustment.
3. The stainless steel ladder at hatch #1 is not continuous with the embedded steel within the tank. For testing purposes, the structure lead was connected to the rectifiers negative terminal.
4. All potentials meet criteria for effective corrosion control.

June 30, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Whatcom Falls #1 Tank Corrosion and Coatings Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an external corrosion and coatings evaluation and internal corrosion and coatings evaluation of the roof surfaces for the City of Bellingham's Whatcom Fall's #1 steel water storage tank. A checkout of the cathodic protection equipment associated with this tank is provided in a separate report. Specific tasks performed during this evaluation included:

1. Evaluate observed corrosion on the tank's steel surfaces.
2. Complete an assessment of both the exterior coatings and interior roof coatings. The interior roof inspection was completed while the tank was full. A raft was used to inspect the upper wall and roof surfaces with the water level near the height of the overflow.
3. Measure steel wall thickness using ultrasonic thickness testing equipment at accessible locations.
4. Measure the depth of any accessible noted pitting.
5. Evaluate coating losses and corrosion on visible surfaces.
6. Test for presence of lead using field lead-check swabs.

BACKGROUND INFORMATION

Whatcom Falls Tank #1 was built in 1982 and is 17-ft tall, 200-ft in diameter and constructed of welded steel. The exterior coating is comprised of a two coat system consisting of a red primer and green topcoat. It is not known if the existing tank coating is original. An impressed current cathodic protection system is used to provide corrosion control to the interior submerged steel surfaces.

COATING AND STEEL EVALUATION METHODS

A series of field tests were completed on the interior and exterior surfaces of the tank during our site visit. A description of each test is provided below.

Dry Film Thickness

The thickness of the existing coating system was measured using a portable electromagnetic dry film thickness gauge (Type 2 gauge). This gauge measures the thickness of the coating between the gauge probe and the metallic substrate.

Steel Thickness

Steel wall thickness was measured using a General Electric model DM5E ultrasonic thickness gauge calibrated for carbon steel. When measuring steel thickness, this unit uses an echo-echo function that allows for measurement of the steel without removing the coating material. Measurements were recorded on each shell course, roof knuckle, roof plate, and interior bottom floor plate.

Lead Testing

A field lead check swab was used to test for the presence of lead on the exterior prime coat. This test is used to determine if lead based coatings are present. If lead is detected, a coating sample is collected and submitted to an analytical laboratory to test for the presence of leachable lead using the Toxic Characteristic Leaching Procedure (TCLP). This test is conducted to determine if the coating material is classified as hazardous requiring specialized handling, containment, and disposal when removed from the tank.

TEST RESULTS AND ANALYSIS

Exterior Coating Thickness

The exterior coating of the tank sidewalls appears to be a two coat system with a red primer and green topcoat. There are multiple locations where a separate overcoat was applied to cover graffiti.

The measured total thickness of the external coating on the sidewalls ranged from 5.8 – 11.5 mils, with an average of 7.63 mils. Typical high performance coatings for the exterior surfaces of water storage tanks are on the order of 12 – 16 mils.

The coating on the exterior tank roof appears to be a two coat system with a red primer and a green top coat. Dry film thickness measurements taken on the exterior roof coating ranged from 5.4 – 11.1 mils, with an average of 8.04 mils.

Exterior Coating Assessment

Areas of debris accumulation are present on the exterior sidewalls, with heavy organic growth around the north side downspout. There is less than 1% total top coat loss, mostly concentrated in isolated locations. Both the prime coat and top coats are tightly adhered and the coated welds are in good condition. There are several areas of coating sags and runs, however this is not affecting the integrity of the coating or the underlying steel. Several locations have been overcoated to cover graffiti and the overcoat is blistered and peeling off. The underlying original coating at these locations is sound.

There is an accumulation of dirt on the roof coating that should be removed. The handrails are experiencing approximately 10% topcoat damage with the exposed steel showing rust staining.

The coating on the exterior roof plates is in good condition with less than 1% topcoat loss noted.

Whatcom Falls #1 Tank

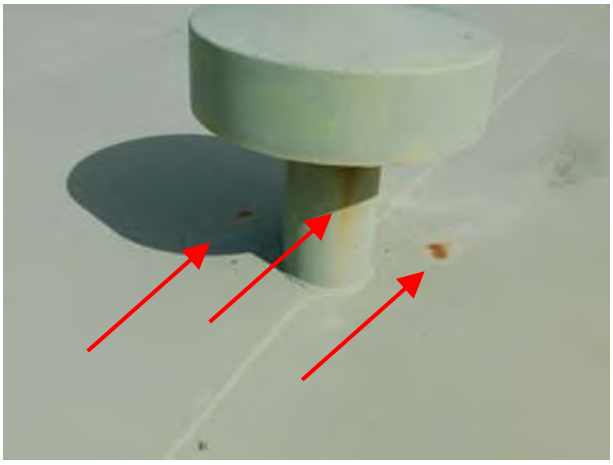
Missing top coat on sidewall

Areas of overcoat to cover graffiti

Overcoat to cover graffiti blistered and peeling off of original coating

Top coat loss on Chime

Tank exterior roof, most surfaces covered with debris

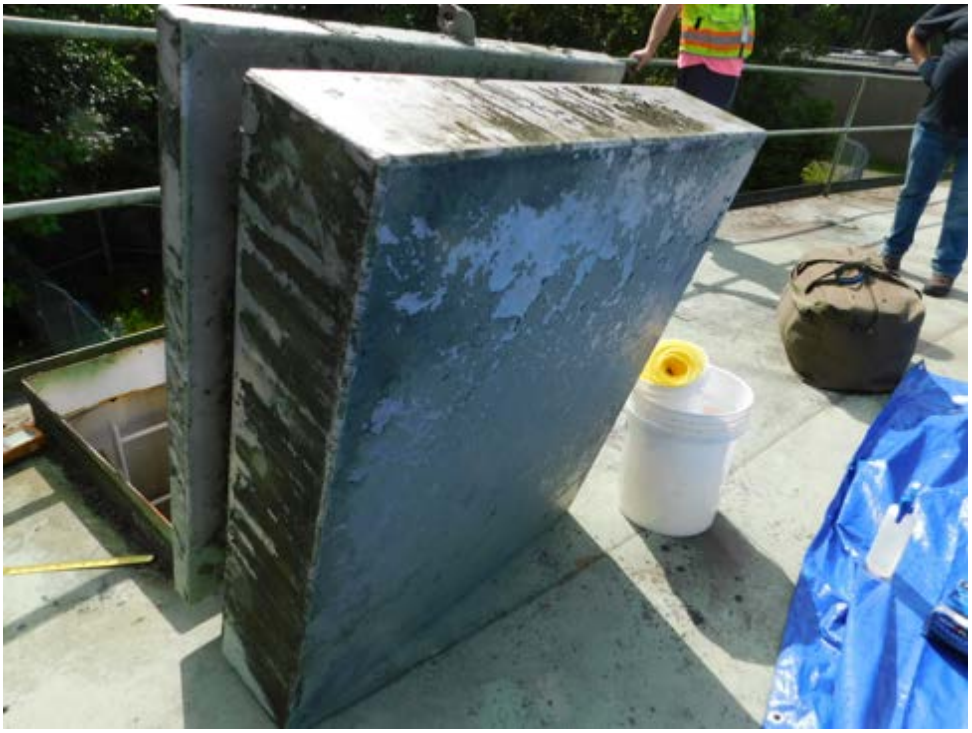


Roof vent rust staining

Coating loss on roof railing with exposed steel

Coating delamination at roof ladder

Significant corrosion damage at roof vent



Coating disbondment on aluminum roof access hatch

Exterior Observed Corrosion Assessment

The exterior sidewall surfaces of the tank had no visible pitting and no corrosion damage was observed on the exterior welds. The chime surfaces are experiencing approximately 15% top coat loss with no visible metal loss.

The roof welds are in good condition and are oriented on the steel plates such that rain water will run off as opposed to collecting at the lap joint areas. No visible signs of pitting are present.

The coating on the aluminum roof access hatch is disbanding. This is common for coating applied to aluminum or galvanized steel surfaces without proper surface preparation. The underlying metal is in good condition with no corrosion damage observed.

The center vent screen cover is experiencing significant corrosion losses. Corrosion is occurring as the bare carbon steel is reacting with the bare stainless steel mesh resulting in galvanic coupling. In order to eliminate this occurrence, the carbon steel cover should be replaced (or repaired) and recoated.

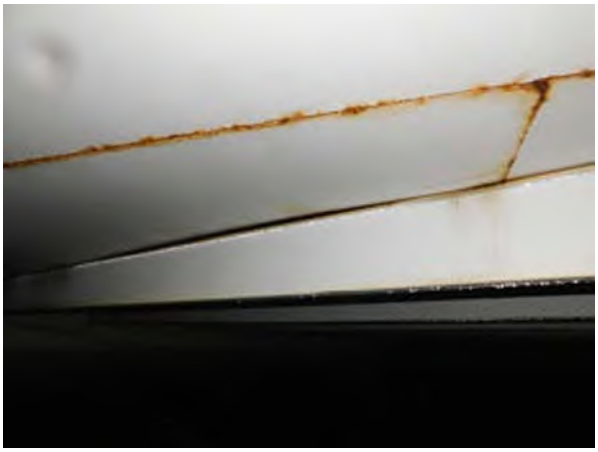
Interior Coating Thickness

The measured total thickness of the internal coating on the sidewalls ranged from 6 – 14 mils. Typical high performance coatings for the interior surfaces of water storage tanks are on the order of 12 – 20 mils.

Interior Coating Assessment

Coating losses are evident on most of the support beam edges and edges of steel plating. In addition, areas of roof plate overlap show coating losses at most locations. These sites are prone to corrosion if a stripe coating is not applied during the coating process. As wet coatings cure, they tend to pull back from the corners and edges exposing the underlying steel. The application of a stripe coat provides and additional layer of coating thickness and protection against these losses.

Coating loss at beam and wall edges



Coating loss at roof plate edges

Rust staining of roof plates

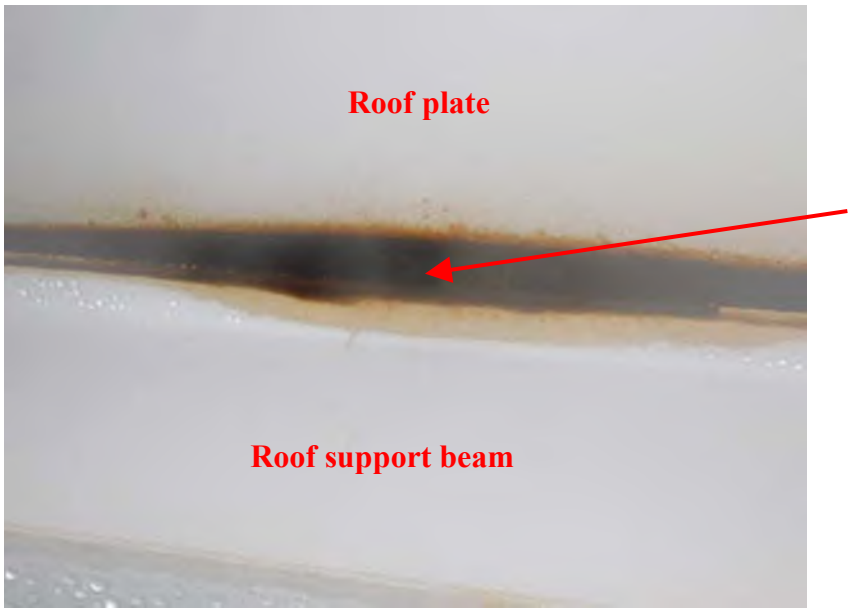
Rust staining of roof plates

Deflected roof plate

Rust staining at center column

Interior Observed Corrosion

The roof shows rust staining at all roof plate overlaps, typical for these non-seal welded areas. The structural components surrounding the access hatch and areas above the water level are experiencing general surface corrosion and crevice corrosion at steel overlap areas.



Areas of large gaps between the roof plate beams and roof plates exist throughout the tank – this will eventually lead to steel thinning at the overlap sites

Fasteners drilled through roof with corrosion damage

Pack rust between beam and roof plate

Gap between beam and roof plate, area is not coated, bare steel has surface corrosion

Surfaces below the water line will be inspected at a later date after the tank is drained.

Ultrasonic Thickness Testing

Thickness data was collected on each shell course and the roof plates. Results of the ultrasonic thickness tests are presented in Table 1. Design thickness data was provided

Table 1: Whatcom #1 Tank Steel Thickness Measurements

Location	Measured Thickness, in.	Nominal Design Thickness*, in.
Chime Plate	0.252	0.250
1st Course	0.465	0.438
2nd Course	0.397	0.375
Roof	0.188	0.188

* From Material Certification Test Reports, Pool Engineers, Inc. September 14, 1982.

Lead Test

A lead test was performed the prime coat material. The lead check swab kit tested negative for the presence of lead materials.

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the tank.

1. All exterior surfaces of the tank are dirty with several areas of organic material accumulation on the sidewalls and north side drain spout.
2. Sealing material between the tank chime and ring wall is no longer effective in most locations.
3. Field lead check swab tested negative for the presence of lead.
4. Coating losses on the exterior and roof surfaces is limited to isolated locations.
5. The handrails, roof access hatch cover, and exterior surface within the ladder shroud have coating disbondment occurring.
6. The carbon steel center vent screen support has significant corrosion on its surface.
7. A significant amount of interior rust staining was observed on the roof and roof support members at all non-seal welded locations.
8. Measured steel thickness met or exceeded the thicknesses described on the material certification documentation.

RECOMMENDATIONS

1. The exterior surfaces need to be pressure washed to remove dirt and other debris.
2. Spot repair exterior coating damage as necessary.
3. Replace or repair the center vent screen support. The screen support should have a well-adhered two part epoxy coating in order to limit galvanic corrosion activity between it and the stainless steel mesh.

4. Seal chime area around the base of the tank. Sealant can be mortar or urethane caulking such as Sikaflex 1A. Slope sealing material such that water runs away from the tank.
5. Replace defective anode hanger as noted in the cathodic protection checkout report.
6. Replace stationary reference electrode used to operate the autopotential circuitry of the transformer rectifier.
7. Given the condition of the interior roof, it is recommended to remove all existing roof coatings, seal-weld all overlaps, abrasively blast all surfaces, and apply a new protective coating. To minimize any additional steel losses, this work should be completed within the next five years. The timing of this recommendations may change once the remaining interior surfaces are inspected.

We appreciate the opportunity to work with you on this project. If you have any questions or would like any additional information, please feel free to contact our office.

Sincerely,
Northwest Corrosion Engineering

Jeremy A. Hailey, P.E.

Appendix C-3 Whatcom Falls I Structural Report

CITY OF BELLINGHAM

CH 5: WHATCOM FALLS 1 STEEL RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

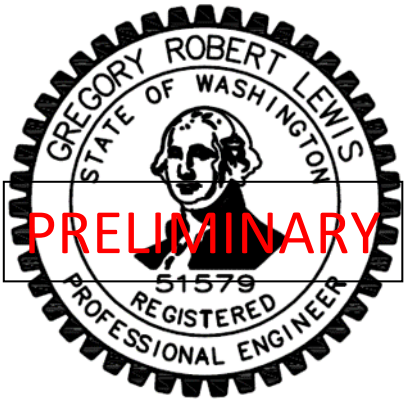
Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Whatcom Falls 1, 4.1 Million Gallon (MG) steel reservoir. The reservoir is located near 3504 Arbor Ct, Bellingham, WA (Lat. 48.7516, Long. -122.4365), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to inspect and visually evaluate the reservoir on June 12th, 2019 and again on November 7th, 2019 by Peterson Structural Engineers (PSE), Murraysmith, Inc., and Northwest Corrosion. The reservoir was filled during the initial inspection to facilitate an evaluation of the interior roof and drained during the subsequent inspection to facilitate the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Whatcom Falls 1 Steel Reservoir – 4.1 MG

2.1 Description & Background

The original reservoir was designed in 1982 by the engineering firms of Tectonics, Inc. (who designed the reservoir itself) and CRS Group Engineers, Inc. (who designed grading, foundation, piping, and cathodic protection). The reservoir was built by P.R. & S., Inc. Contractors and the identification plate on the tank indicates a construction year of 1982. As-builts annotations were finalized in January 1984. The reservoir is a ground-supported welded steel reservoir with a 201-foot inside diameter. The wall consists of two shell courses, with a bottom course height of 10'-0" and an upper course height of 7'-6" for a total height of 17'-6". This reservoir does not include an overflow pipe which is atypical of standard reservoir designs. Without an overflow pipe the reservoir can be operated at a 17.5-foot operating level, equating to a storage volume of 4.1MG which is the volume the City lists for this reservoir. The identification plated indicates the storage is 3.8MG which corresponds to a maximum operating height of 16-feet. This appears to indicate the reservoir is operated above its intended design operating level when the water level is over 16-feet.

The reservoir roof is supported by two rings of girders. Girders are supported on (16) 5-inch pipe columns while the roof center is support by an 8-inch pipe column. The roof plate was measured to be 0.188-inches thick and has an approximate slope of 0.75:12 as required by code. The roof is comprised of 6-foot by 30-foot plates which are lapped and welded along their exterior-side seams. The interior of the roof plates were not observed to be welded along their interior lap seams which is typical of this type and age of reservoir. Further, there is no connection (welded or otherwise) between the roof plates and rafters. The roof plates are welded along the exterior seem of the roof plate-to-knuckle interface.

Drawing are available for this reservoir and were used to determine the foundation configuration. The CRS Group Engineers, Inc. plan set titled *Grading, Foundation and Piping*, sheet 4 of 7, has the foundation details, which are shown in Figure 2-7, in detail 1/4. Per those drawings, the reservoir is supported on a ring wall footing which is 24-inch-wide by 3.5-foot-deep. Measurements onsite found the exposed exterior portion of the footing to be consistent with the drawing's dimensions. The shell is unanchored and is positioned so that the shell-wall bears on the center of the footing. The drawing shows the ring wall footing contains (6) #7 circumferential bars on each face with #5 ties at 9-inches on center. Per Detail 2/4, the reservoir piping runs under the footing and is encased in lean concrete fill.

2.1.1 Description of Additional Site Structures and Features

The overall site includes the Whatcom Falls 2 prestress reservoir, approximately 150-feet to the southeast of Whatcom Falls 1. The structural adequacy of Whatcom Falls 2 is discussed in a separate report. The site also includes a pump station, located about an eighth of a mile to the east. The pump station was not evaluated by PSE as part of the evaluation. Neither of these structures are near enough to the reservoir to adversely impact Whatcom Falls 1 during a seismic event.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed two site visits to observe the as-built current condition of the reservoir. The first site visit occurred on June 12th, 2019, while the reservoir was full, allowing PSE to evaluate the roof framing and

vent opening for corrosion and structural issues in addition to exterior systems. The second site visit occurred on November 7th, 2019 while the reservoir was empty to allow PSE to evaluate the floor, manway hatch, and the interior structural systems.

Steel Roof: The exterior reservoir roof was found to be in generally good structural condition. The roof coating was found to be competent although some organic growth and debris build-up was noted to be occurring. Higher instances of debris build-up were found to be occurring along the western side closest to the surrounding forested area. This debris build-up was further exacerbated by a lip around the exterior edge of the reservoir. While this lip was intended to direct water to a set of roof drains, it appears to be impeding the overall roof drainage. This lip had been trimmed near the roof drains in an effort to ensure that water can flow off of the roof, even if a drain is clogged.

The roof hatch opening was measured to be 3-feet by 3-feet. The body consists of a welded box section which is welded to the top of the tank roof at the opening. The hatch is aluminum and in a standard 'shoe-box' style with a lock and a screened vent measuring 1.5-foot by 1.5-foot. The reservoir hatch does not have a gasket along the edge where the hatch bears on the body of the hatch body. Despite the hatch and vent coating appearing to be in poor condition, the underlying aluminum appears to be in good condition. On the interior of the hatch, along the body/roof interface, corrosion was noted with some minor section loss. An interior ladder system was installed below the hatch and appeared to be in generally good condition with some minor coating loss and incidental corrosion spotting along its length.

The interior reservoir roof was noted to be in fair condition with multiple instances of crevasse corrosion, coating failure along member corners, and minor section loss along plate lap lines. Rafter connections were noted to be in fairly good condition with some minor corrosion observed. A majority of the roof components appeared to be in structurally acceptable condition.

Reservoir Floor and Walls: The exterior wall of the reservoir was noted to be in generally good structural condition with some isolated instance of coating failure or organic growth. Overall construction of the shell and welds were found to be of good quality. As noted previously, the roof drain configuration has created some roof issues but the drains have been effective in minimizing debris staining and build-up on the reservoir side walls. External appurtenances such as the ladder and manways were found to be in good condition. As the ladder is contained within an enclosure, the coating in this zone was noted to have higher instances of delamination, likely due to higher temperature and moisture in the enclosure. Above the manway hatch the identification plate identified the tank type and manufacturer.

The floor steel and interior walls below the typical water level were found to be in generally good condition although heavy corrosion staining was noted on the walls. Where the coating was compromised, exposing the base steel, the reservoir's corrosion protection system appears to have limited said failures to isolated failure points. Although these instances of coating failure and bubbling were noted at various locations throughout the reservoir floor, the overall extent of corrosion below the waterline appeared to be limited in effect. Above the typical water level, corrosion was more endemic and was noted along the interior roof edge, at plate laps and joints, and within the body of the plates themselves, in the form of coating failure and delamination.

Foundation and Site: The reservoir is supported on a concrete ring foundation, which was found to be in good visual condition with no settlement issues or major cracking visible. The gap between the bottom of the floor plate and top of the footing varied from direct contact on the south side to approximately 1-inch on the north side. The gap was sealed with joint caulking material where it is narrower and filled with grout where it is at its widest on the north side. The grout was observed to have cracked and failed in multiple locations.

The overall site is generally flat with a drop-off towards the north. The site is located in a forested area and a marsh-like zone is located to the south of the reservoir between Whatcom Falls 1 and 2. However, the ground directly surrounding the reservoir appeared firm and no standing water was noted. The reservoir's concrete footing height above the adjacent grade averaged approximately 6-inches around the majority of the reservoir. This meets the recommended minimum outlined in AWWA D100-11 Section 12.7.1. The site is bounded by a fence which results in a clear space of 15 to 16-feet around the reservoir which has been cleared of trees and brush. Outside the fence line the area is forested. Generally, trees were kept back from the reservoir outside of the fence line. The trees on the west side of the reservoir encroach the closest, though they are still well away from the reservoir. This proximity corresponds with the largest build-up of roof debris as the height of the trees allows more leaf-litter to make it onto the roof.

2.2.1 Visual Condition of Additional Site Structures and Features

As noted above, no additional site structures are discussed in this report.

2.3 Structural Analysis

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoirs under the current applicable Codes and standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D100-11 was utilized. The evaluation was based on original construction documents and site visit observations.

Per the City, the average operating level is between 15.5-feet in winter and 16.5-feet in summer. The reservoir's design volume appears to be 16-feet. However, the reservoir lacks an overflow and could be operated to the top of the wall, at 17.5-feet, or higher. The City has listed the reservoir's storage volume as 4.1-MG, which corresponds with a 17.5-foot operating level. Using current code requirements, this higher operating level has been used in PSE's structural adequacy checks.

Reservoir Shell – Material Thickness: In addition to the reservoir drawing, in-situ shell thickness readings were taken in-field via ultrasonic testing for use in PSE's analysis. As multiple measurements are taken, PSE will typically utilize the average thickness reading. The exception is in instances where a plate section within a given shell course has been found to deviate from the average by greater than 5%. In such a case, PSE will use the lower bound value as the controlling steel thickness for the analysis of that shell course.

Reservoir Steel Thicknesses				
Component	Drawing (in)	Average (in)	Minimum (in)	Value Used (in)
Shell Course 1	0.438	0.465	0.465	0.465
Shell Course 2	0.375	0.397	0.397	0.397
Floor Plate	0.250	0.252	0.252	0.252
Roof Plate	0.188	0.189	0.189	0.189

2.3.1 Hydrostatic and Gravity Analysis

Roof Framing: The roof plate thickness and layout are adequate under current code for the required live and snow loads. Based on Whatcom County and City of Bellingham requirements, the ground snow load used for design was determined to be 20-psf while the design snow load was determined to be 25-psf. This value conforms to the minimum design snow load value of 25-psf, set by AWWA D100 and by most local building departments. While the rafters are not attached to the roof plate, D100 allows for a rafter with a depth less than 15-inches to be evaluated as continuously braced. When this allowance is used, all rafters passed design checks for the 25-psf snow load. The interior 8-inch diameter center column was found to be sized appropriately for this load per current code. Overall the roof structure appears to be structurally adequate for the anticipated gravity loads.

Reservoir Shell – Hydrostatic Stress: No documentation was available that indicated the reservoir's design method. Further the reservoir's identification plate did not indicate a potential design method but did indicate the steel grade is A516 Grade 60. Per AWWA D100-79 the reservoir could be designed per either Section 3 of AWWA D100-79 or Appendix C (now known as Section 14). Section 3 requires less documentation and inspection and produces a more conservative reservoir design, while Appendix C requires better documentation and special inspection but allows for thinner shell components.

As a back-check, values from Section 14 are utilized to check the minimum required plate thickness required for an operating height of 16-feet, as the initial design should always be for the overflow level. As this reservoir has no overflow, this height is based on the operating level associated with the 3.8-MG storage volume listed on the identification plate. This check assumes AWWA defined A516 Grade 60 steel with a maximum tensile stress of 19,200-psi per AWWA Table 34 and an efficiency factor of 100%, owing to an assumed weld special inspection program used throughout construction as required per Section 14 design.

$$Shell\ Thickness = \frac{2.6(Operating\ Height)(Diameter)}{(Plate\ Tensile\ Stress)(Efficiency)} = \frac{2.6 \times 16ft \times 201ft}{19200psi \times 1.0} = 0.436\ inch$$

This result matches the first course shell plate thickness which is listed as 7/16-inches (0.4375-inches) on the drawings and the Section 14 assumption appears to be appropriate. Both shell courses were then re-checked using these design assumptions along with the actual measured shell thickness (which are thicker than what was specified on the drawings). At the maximum operating level of 17.5-feet, the shell's tensile

capacity was only exceeded by 2.5%. At the current 16.5-foot maximum operating level the stresses were found to be within acceptable levels.

Foundations: Per PSE's analysis it was determined that the reservoir footing has a resulting bearing pressure of 2700-psf for gravity and hydrostatic loads. Based on the Geotechnical investigation, this site has an allowable bearing capacity of 6000-psf and therefore the footing is adequate for the calculated loads. This is further confirmed by the foundation itself which was not observed to have any differential settlement or foundation cracking issues.

2.3.2 Hydrodynamic and Seismic Analysis

Reservoir Shell – Hydrodynamic Stress: As noted in the hydrostatic section, analysis of the shell was conducted following the methods allowed in AWWA D100 Section 14. See the previous hydrostatic section for additional notes. When utilizing these assumptions, the hydrostatic and hydrodynamic stress values were found to be within acceptable levels when using the as-measured steel thicknesses and assumed steel grade, as noted above, for an operating level of 17.5-feet. Per the current code, the reservoir's shell is adequate for the site's anticipated seismic loads, when neglecting constrained slosh, see the next section for additional information on the slosh analysis.

Freeboard/Slosh: The AWWA describes the freeboard height as the distance between the top of the overflow and base of the rafters. For this analysis the freeboard checks included a check based on the AWWA requirements as well as a check based on the maximum operating level. Per the City, this reservoir's maximum operating level is 16.5-feet, about 12-inches below the roof plate. As there is no overflow, the reservoir can be operated up to the bottom of the roof (and possibly higher), so the reservoir was evaluated for slosh at both the 16.5 and 17.5-foot levels. During a seismic event, operating at these levels would result in a constrained slosh wave . For a constrained slosh wave, the wave generated by the ground motion impacts the bottom of the roof. This slosh impact wave can damage the roof, rafters, hatches, and vents, and dislodge roof plates. Additionally, constraining the slosh wave can increase the potential for overturning.

For this reservoir, the code calculated slosh wave height was determined to be 3.6-feet. This height is a function of the geometry of the reservoir and local seismic conditions. As a result, the operating height would need to be reduced to 14-feet to alleviate all slosh related loads on the roof components and roof rafter supports.

Should a higher operating level be required, the roof would need to be retrofitted to resist slosh impact uplift loads. While the roof plates and rafters are adequate for gravity loads, they do not have the capacity to resist the upward direct force resulting from the anticipated slosh wave. To resist these loads the roof plates would need to be welded to the rafters in order to provide a load path between the plate and rafter elements. A slosh wave resulting from a 16.5-foot operating level would exceed the roof plate's and rafter's capacity and so additional and extensive retrofit work also need to be performed to further strengthen these elements.

Overturning and Anchorage: Due to the large diameter of this reservoir, there is limited concern with uplift. Even accounting for a fully restrained slosh-wave, which can greatly increase overturning potential,

this reservoir is stable and does not require anchors per the referenced code requirements. Further the foundation bearing capacity is well within acceptable load limits.

2.4 Summary

Structurally the reservoir was determined to be adequate for current code gravity, hydrostatic, and hydrodynamic loads, except for slosh loads noted below. The shell plate, roof members, and footing were all determined to be adequate in size and layout. Atypical of most reservoirs, this reservoir did not have and overflow pipe. An overflow is required per the governing design standards and without an overflow pipe the reservoir is susceptible to damage if it is overfilled, which can result in excessive interior pressures on the roof and wall.

The primary structural concerns noted were a result of corrosion. The exterior of the reservoir was noted to have organic growth and debris build-up on the west and northwest portions of the wall and roof. The interior roof and rafter system was observed to have instances of failed coating and corrosion. While the reservoir's design is adequate for gravity and hydrostatic loads, maintenance is required before corrosion adversely impacts structural systems, reducing their capacity and performance.

Evaluated per current seismic codes, the main body of the reservoir was found to perform reasonably well. However, slosh wave impact was unaccounted for in the original design and can have adverse impacts on performance. As a result, many of the failures that were identified were in the roof due to the anticipated slosh wave. Operating at a 16.5 to 17.5-foot water level the roof system would be susceptible to failure during a seismic event.

Based upon the evaluation and aforementioned conditions, retrofit of the reservoir to function at the 16 to 17.5-foot operating level would be extensive. At a minimum, PSE recommends a reduction in the maximum operating level in order to bring this reservoir into compliance with current seismic codes and to limit required upgrades.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code.

<u>Reduced Storage Volume</u>

The first option is to perform no structural upgrades and lower the current maximum operating level by 3.5-feet to 14-feet. Lowering the operating level by this amount would provide adequate freeboard for the expected slosh. With adequate freeboard a slosh wave would not impact roof elements and no roof structural upgrades would be needed at this operating level. Repairs would primarily be non-structural and include corrosion removal, re-coating, and the installation of an overflow pipe.

<u>Maintain Maximum Storage Volume</u>

The second option is to continue using an operating height of 16.5-feet or higher. In such a case PSE would recommend an evaluation of the roof to determine if it can be reinforced and seal welded. Based on the

site observations, it appears that the roof plates have been impacted by corrosion along their seams. Before any retrofit work is undertaken the lapped roof plates should be evaluated to determine the extent of corrosion along these edges. If the roof is determined to be competent, the plates can be seal welded and welded to the rafters to ensure a positive connection against slosh-induced uplift. This would require a full roof load analysis as the entire roof support structure, from roof plates to column bases, would need to be reinforced. These retrofits would help to reinforce the roof against any slosh loads. However, please note that this retrofit may ultimately not be feasible or the final upgrade configuration might be uneconomical.

Alternately, the roof could also be raised, and new shell course installed. Adding more shell height would result in more freeboard and eliminate the impacts of constrained slosh. For this upgrade the existing roof could be saved and only the interior column would need to be replace with a taller column. Slosh impacts would be mitigated by moving the roof out of the impact zone which would limit roof upgrade requirements and allowing for the reservoir to be operated at a higher level. This higher level could be achieved as one of the ways to add a new shell course is to lift the other courses and then replace the first shell course with a new thicker shell course.

Intermediate Storage Volume

Due to the diameter of this reservoir and wave height associated with the slosh wave, no effective intermediate storage volume exists that would eliminate roof upgrades. At a minimum, operating at 14.5-feet would require retrofits to the exterior 10-foot section of the roof as slosh loads would exceed the roof's design capacity and the roof could potential fail. An operating level of 15.5-feet would require retrofits to the exterior 20-feet of roof as well as to the exterior columns. An operating level of 16.5-feet would require retrofits to the exterior 30-feet of roof and 17.5-feet would require retrofits to the exterior 40-feet of roof. The operating level would need to be reduced to 14-feet in order to achieve the full freeboard requirements and eliminating all potential roof retrofit requirements.

General Site and Structural Repair Recommendations

As the reservoir lacks an overflow pipe, an overflow pipe should be sized and installed in the reservoir. This is covered in AWWA D100 Section 5.3 and is required to protect the tank against over-pressure and overload. The overflow pipe should be sized according to the geometry of the tank.

On the interior of the reservoir the roof plates, wall plates, and floor plates should be monitored. Issues with corrosion at the time of observation did not appear to be excessive or to have resulted in major section loss. As a result, no retrofit upgrades to replace corroded structural members are recommended at this time.

Around the exterior of the reservoir, PSE recommends a cleaning of the reservoir's shell and removal of any vegetation growing between the floor plate and top of footing. PSE would recommend adding additional roof scuppers to further facilitate roof drainage and limit the collection of debris which could be deleterious to the roof coating.

2.6 Scans of Select Construction Documents

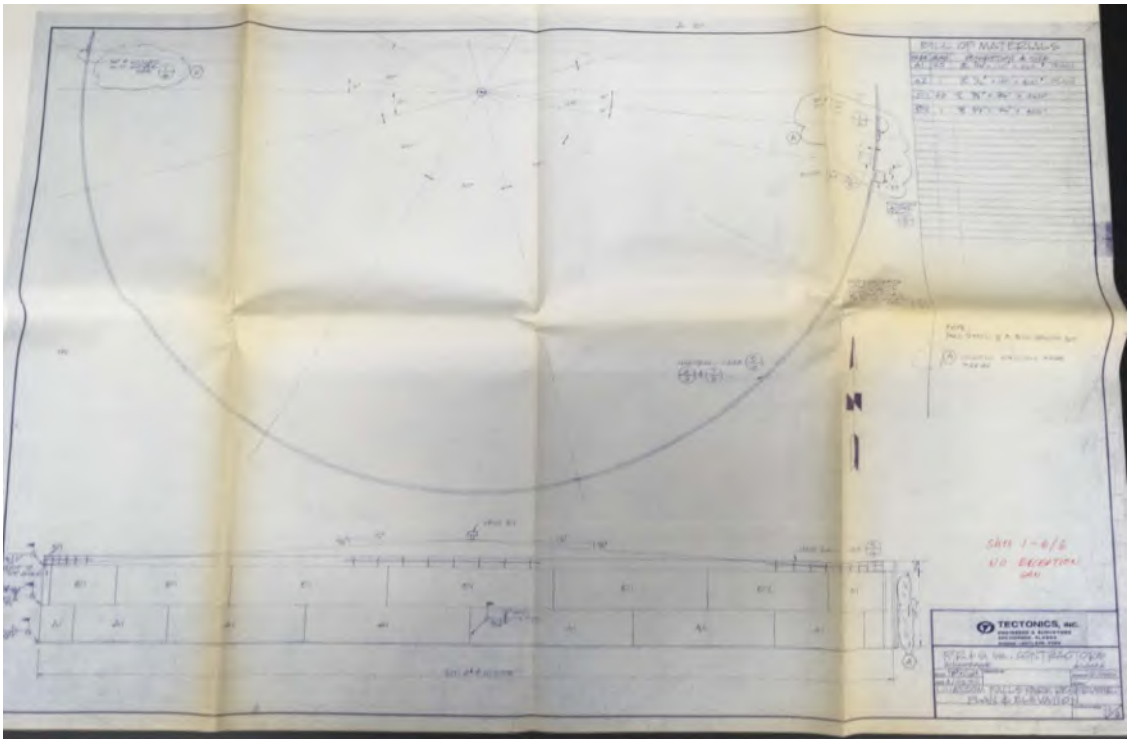


Figure 2-1: Whatcom Falls 1 – Plan and Elevation

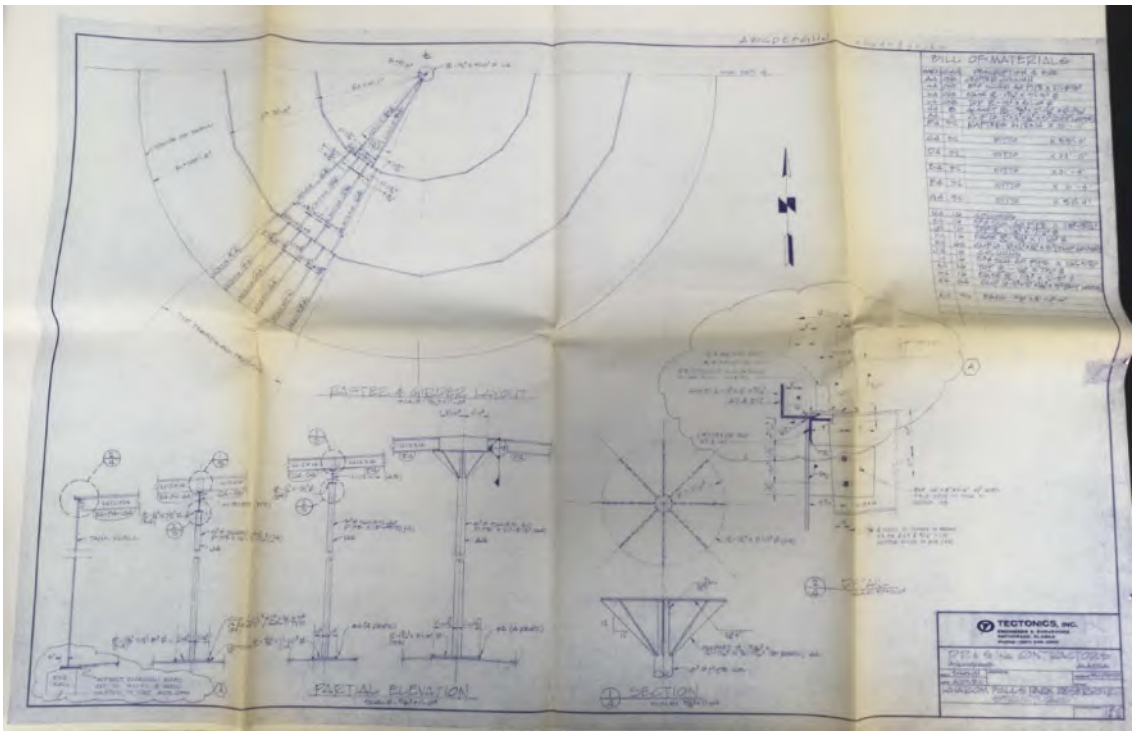


Figure 2-2: Whatcom Falls 1 – Roof Framing Plan

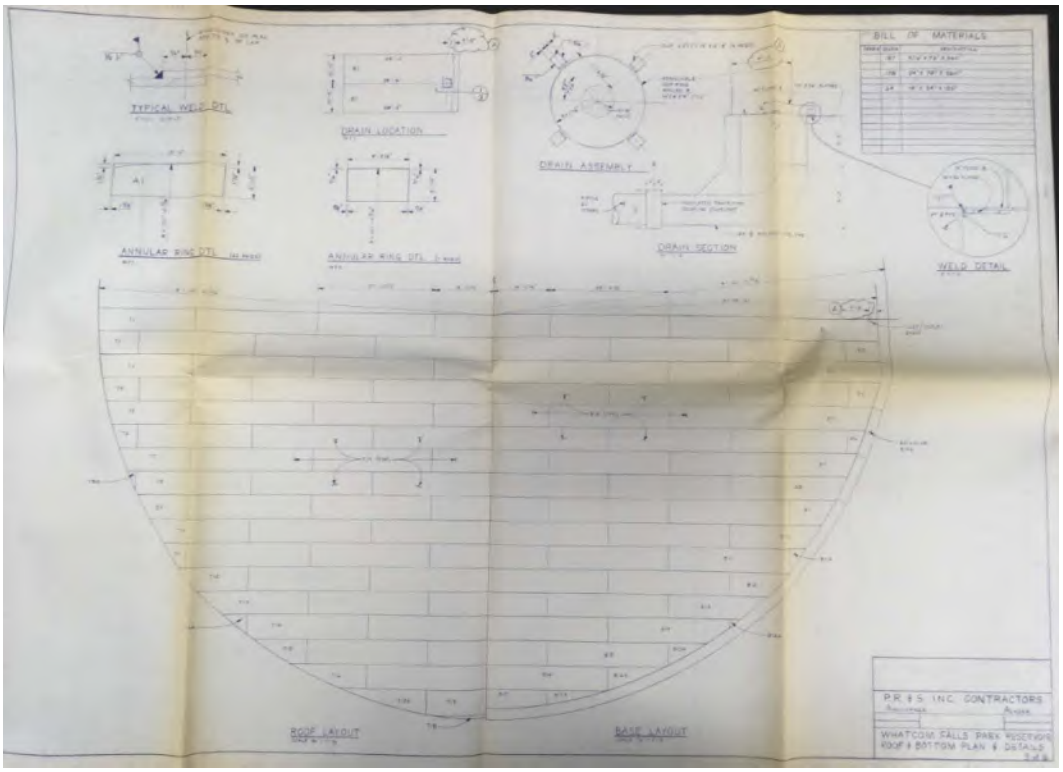


Figure 2-3: Whatcom Falls 1 – Roof and Bottom Plan and Details

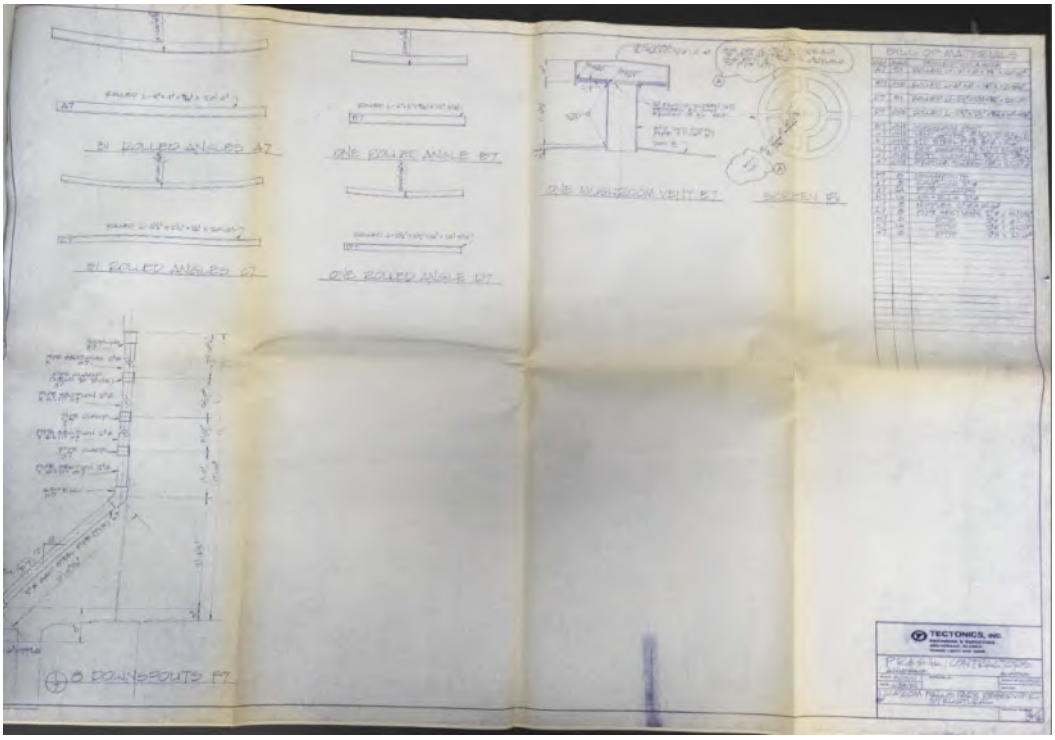


Figure 2-4: Whatcom Falls 1 – Vent and Drain Details

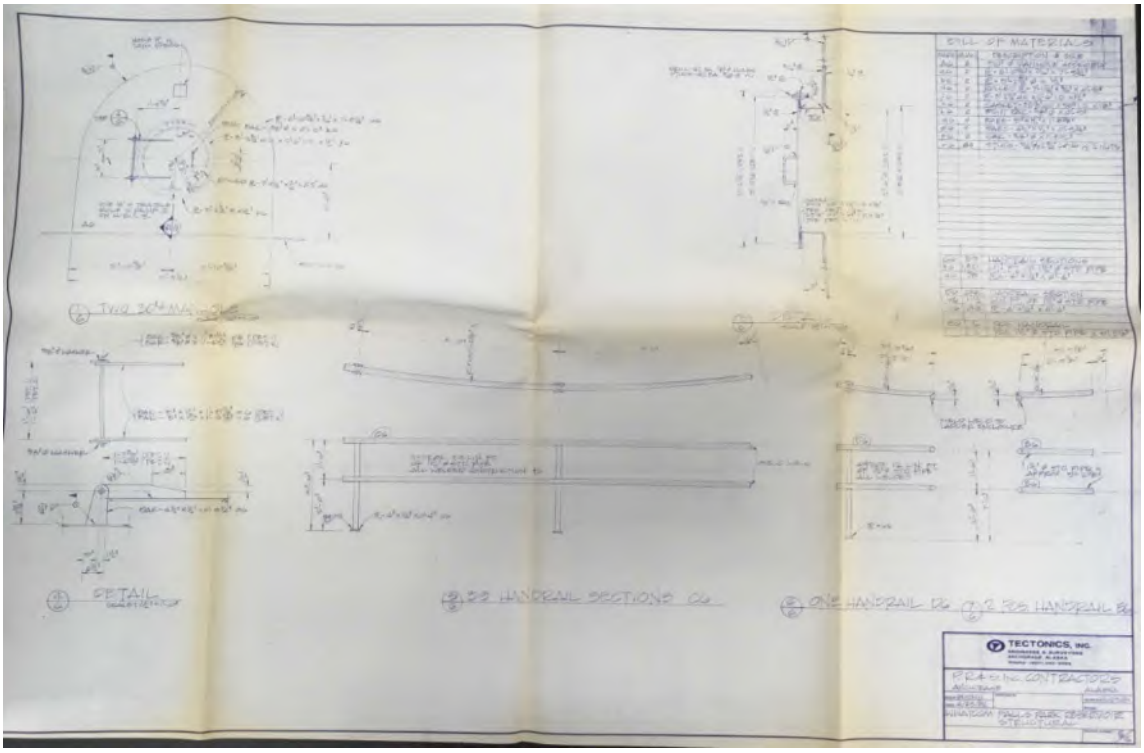


Figure 2-5: Whatcom Falls 1 – Manway and Railing Details

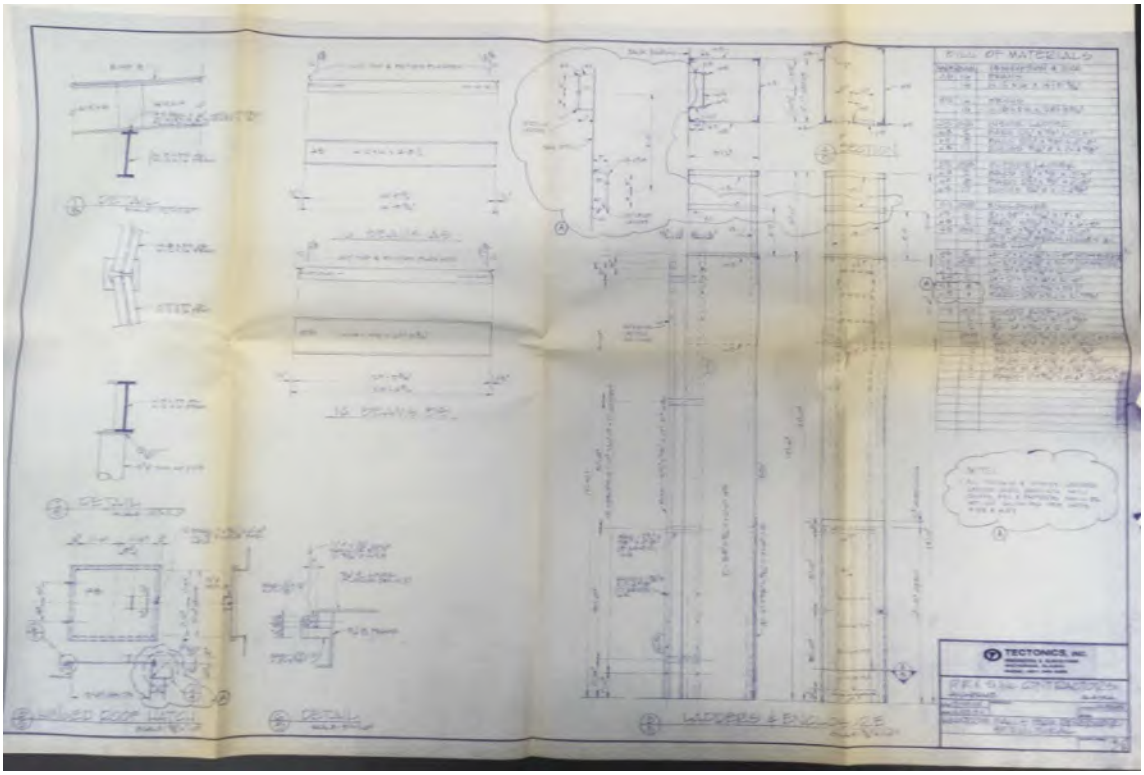


Figure 2-6: Whatcom Falls 1 – Ladder and Hatch Details

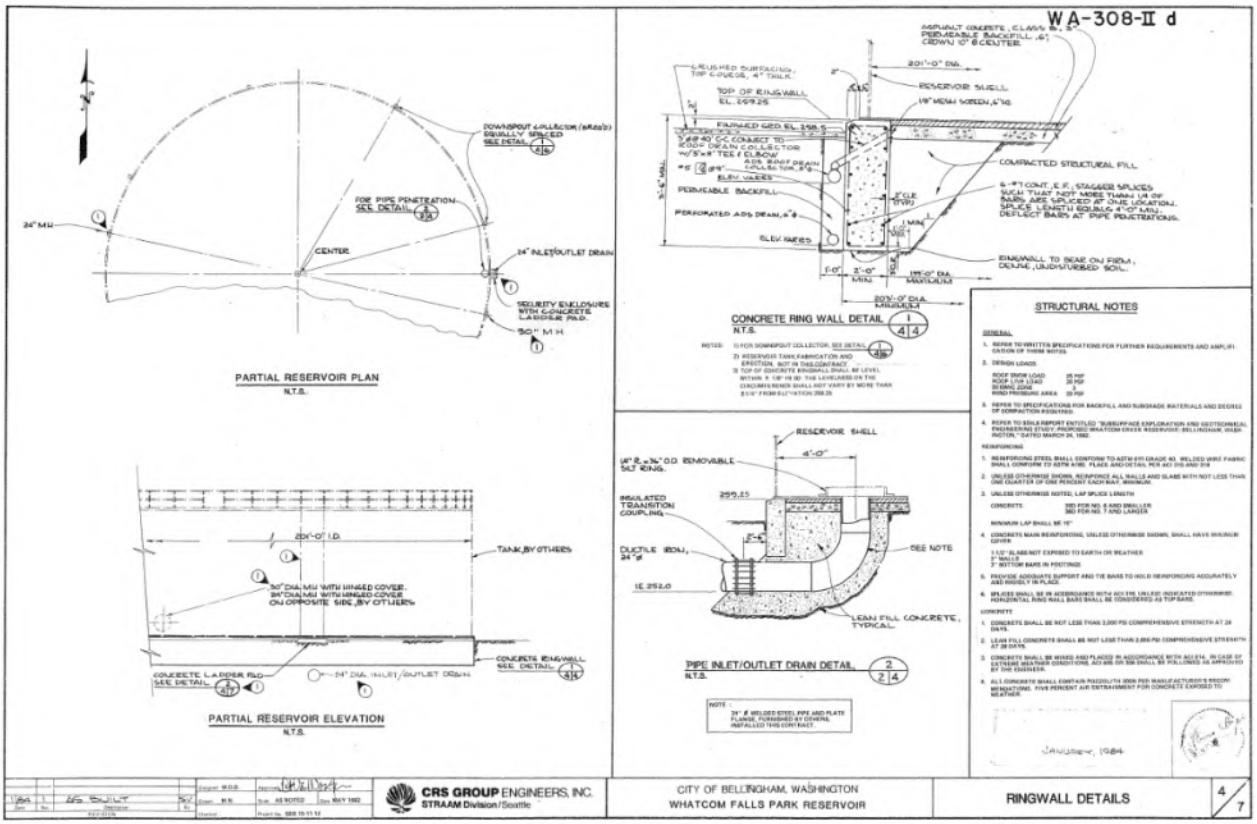


Figure 2-7: Whatcom Falls 1 – Ringwall Details

2.7 Observations Pictures

Figure 2-8: Whatcom Falls 1 Reservoir – Elevation

Figure 2-9: Whatcom Falls 1 Reservoir – ID Plate

Figure 2-10: Whatcom Falls 1 Reservoir – Manway Hatch (south side)

Figure 2-11: Whatcom Falls 1 Reservoir –Ladder Enclosure

Figure 2-12: Whatcom Falls 1 Reservoir – Sidewall (east side)

Figure 2-13: Whatcom Falls 1 Reservoir – Sidewall with scratch in coating (north side)

Figure 2-14: Whatcom Falls 1 Reservoir – Footing and Drain

Figure 2-15: Whatcom Falls 1 Reservoir –Roof Drain with Staining

Figure 2-16: Whatcom Falls 1 Reservoir – Roof Drain. Side plate has been notched to facilitate drainage if a pipe is blocked.

Figure 2-17: Whatcom Falls 1 Reservoir – Roof with debris build-up (west side)

Figure 2-18: Whatcom Falls 1 Reservoir – Roof Vent

Figure 2-19: Whatcom Falls 1 Reservoir – Access Hatch with Built in Vent

Figure 2-20: Whatcom Falls 1 Reservoir – Roof Rafter Connection

Figure 2-21: Whatcom Falls 1 Reservoir – Center Column and Support Plate

Figure 2-22: Whatcom Falls 1 Reservoir – Roof Plate, example of coating failure

Figure 2-23: Whatcom Falls 1 Reservoir – Overview of Roof Framing and Support Structure.

Figure 2-24 Whatcom Falls 1 Reservoir – Combination Inlet, Outlet, and Drain (with removeable Silt Stop Ring)

Figure 2-25: Whatcom Falls 1 Reservoir – Center Column 45-inch square base plate

Figure 2-26 Whatcom Falls 1 Reservoir – Ladder and Wall (both wall and ladder show signs of heavy corrosion staining)

Figure 2-27 Whatcom Falls 1 Reservoir – Instance of Coating Failure and Corrosion on Floor

Figure 2-28 Whatcom Falls 1 Reservoir – Instance of Coating Failure and Corrosion on Floor at Plate Joint

Figure 2-29 Whatcom Falls 1 Reservoir – Instance of Coating Failure and Corrosion on Wall

2.8 Field Notes

WELDED STEEL RESERVOIR SITE INSPECTION

3.8MG per ID plate

Project #: A1802-0019 Project Name: Bellingham Reservoir Evaluation

Site Visit Date: 1st 6/12/19 2nd 11/7/19 Reservoir Type: Non standpipe Steel – 4.1 MG Whatcom I. 3201 Arbor Ct, Bellingham, WA (48.7502, -122.4359)

Site Conditions:
1st Clear, dry, sunny 70°F, ring road overgrown, low-lands outside of fence line and berm and east-ward slope
2nd Clear, wet, sunny 35°F

Exterior Inspection

Number of Steel Shell Courses: 2 Standard Height of Shell Courses: N/A

Shell Course Height: (top) ____, ____, ____, ____, ____, 8', 9'6" (bottom)

Knuckle yes (no) Radius N/A Thickness N/A

Shell Course Thicknesses: (top) ____, ____, ____, ____, ____, ____, ____ (bot)

Condition of Exterior Shell and Coatings (check for location of paint delamination and rust):

Name plate Material A516-60. Paint in gen. good cond. w/ touch-ups, Areas of incidental failure & delam. Weld quality is fair to good w/ min. burrs or issues. On north side below, roof lip litterbins are present. Drains have corrosion but have helped keep sides of reservoir clean.

List location of all external items (pipes, manways, ladders, etc.) on drawing page. Locate items using pace count method.

Condition of Ladder/Vents/Hatch/Welds: Hatches – Fair, flat surfaces have coating loss & corrosion on bolts. Roof drains – Fair, run-off material & corrosion on bolts. Ladder – Good, inside of enclosure has some paint delam. issues

Manway Dia: 36", Ladder Dimensions 16" wide, 1"ϕ rung @ 12" o.c., side ∎ 2½ × ¼ In a enclosure

Operating Winter 6.5'-15.5'
Summer 7.0'-16.5'
Avg 11'

1

WELDED STEEL RESERVOIR SITE INSPECTION

Exposure and Condition of Foundation: Foundation, 11" proud of wall. (24" total width?) Top of concrete sloped & in good condition. 6" above adjacent grade but overgrown. No grout, black mastic used & failing

Grade Relative to top of foundation 6" and larger in some areas, List max/min on drawing page.

Pothole at footing. Depth of Footing N/A Dist from anchor CL to footing edge

Top Surface Roof Plates & Coating Condition (check for paint declamation and rust):

Toward crown, gen. good w/ staining. At edge more material retained and more instances of coating failure & delamination. Scuppers added to roof by drain but material still build-up

Thickness of roof plates _____, Slope of Roof* ≈ 2 - 4° depending on loc. roof plate is wavy

Interior Inspection

Tank Diameter 202', Distance from Wall to Mid-span Rafter Support 38' → 33' - 30'

Column Diameter/Size: 5.5" OUTER (2) COLUMN ROWS, ≈ 9" CENTER

Interior Footing: _____

Column Spacing/Configuration: RADIAL w/ (2) COLUMN ROWS.

Condition of Interior Shell and Coatings: COATINGS GENERALLY GOOD UNDER STAINED LAYER

Bottom of Roof Plate/Framing/Coatings Condition: SOME STAINING, GENERALLY WORSE ON ROOF W/IN INNER RAFTER REGION, STAINING @ ROOF EDGE & WELD SEAMS, RAFTERS GENERALLY OK, SOME STAINING @ EDGE OF ROOF & @ GIRDER SUPPORTS, TOP FLANGE STAINING, BOTTOM FLANGE OK, KNIFE PLATES A BIT STAINED

Floor Plate Condition/Thickness/Coatings: GENERALLY GOOD, SOME STAINING @ SEAMS

2

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WELDED STEEL RESERVOIR SITE INSPECTION

Ladder/Pipes/Overflow Conditions: Ladder - good w/ staining but little corrosion. Pipes - None internal, Overflow - None
Combo Inlet/Outlet/Drain - sedimentation present and staining but minimal corrosion.

Diameter of Inlet 23½", Outlet 23½", Overflow No overflow
Same

Distance of Top of Overflow from base of roof elements No overflow

Other Comments: Reservoir has single inlet/outlet. No overflow observed.
"Bath tub" ring of corrosion staining around wall of res. stain appears to fade about 2' below rafters, likely where typical max. opperation level located.

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WELDED STEEL RESERVOIR SITE INSPECTION

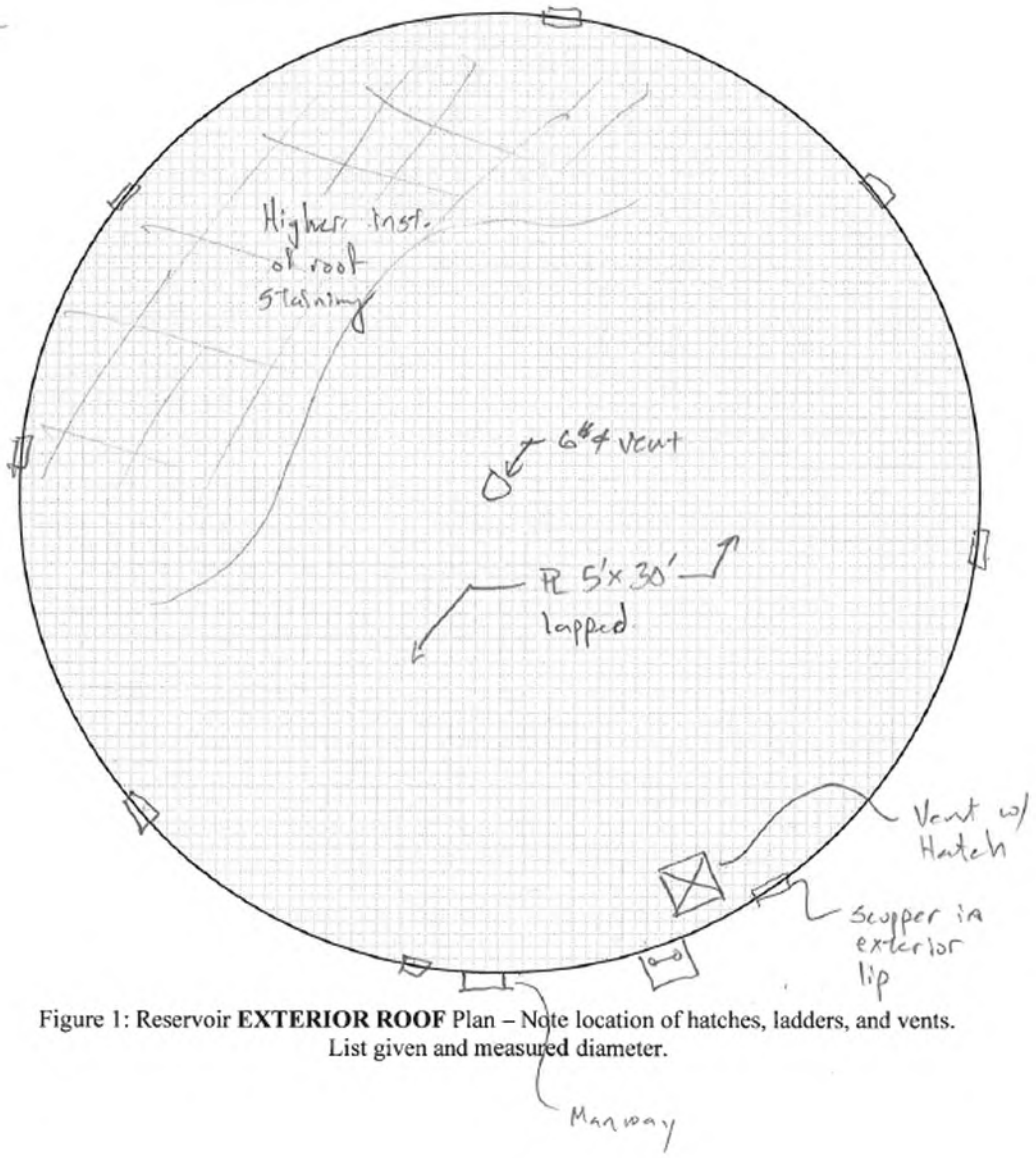

Figure 1: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and vents. List given and measured diameter.

WELDED STEEL RESERVOIR SITE INSPECTION

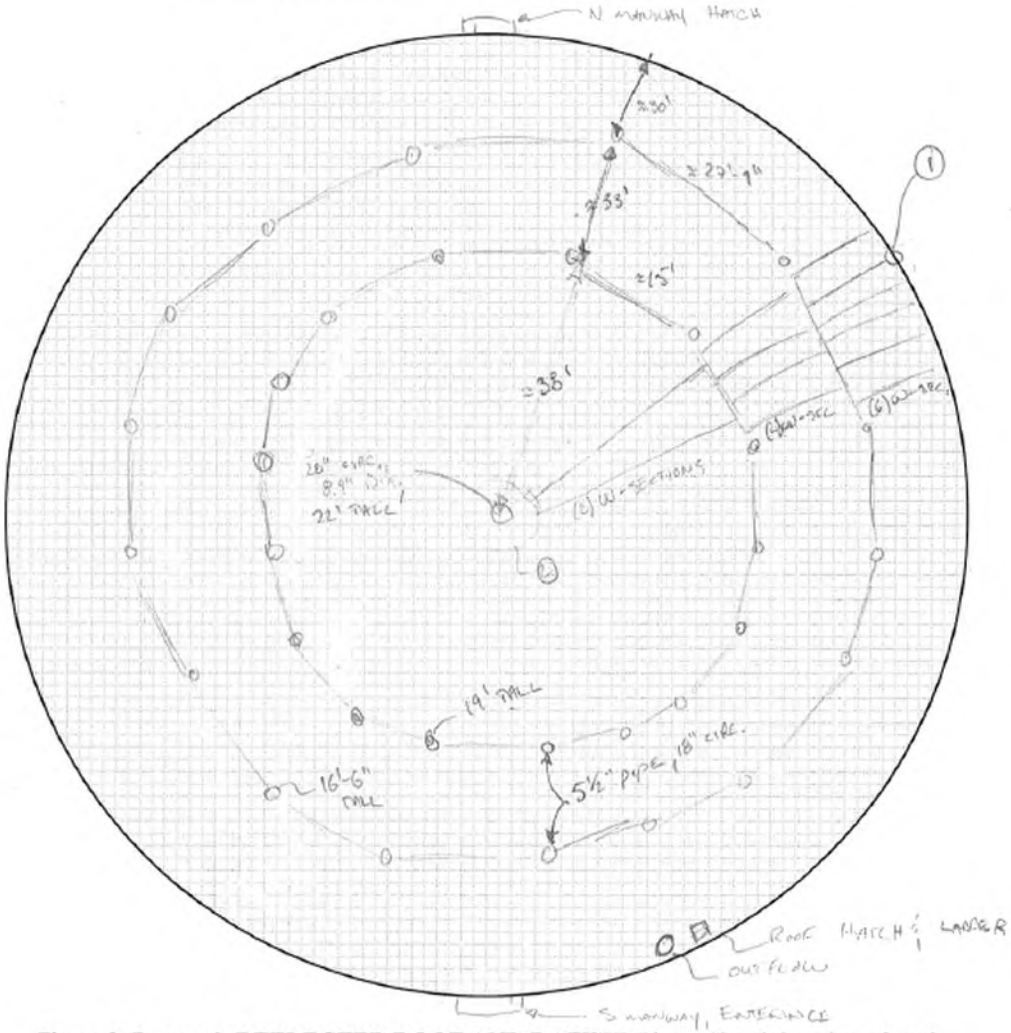

Figure 2: Reservoir **REFLECTED ROOF AND RAFTER** Plan – Sketch location of roof supports, hatches, vent, columns, etc. List given and measured diameters.

Use Figure 5 for roof edge notes.
Use Figure 6 for component dimensions

(1) KNEE PLATE w/(2) BOLTS TO WEB
(2) COLUMN TOP PLATE w/ (8) ▽ STIFFENERS, ≈ 55" DIA.

5

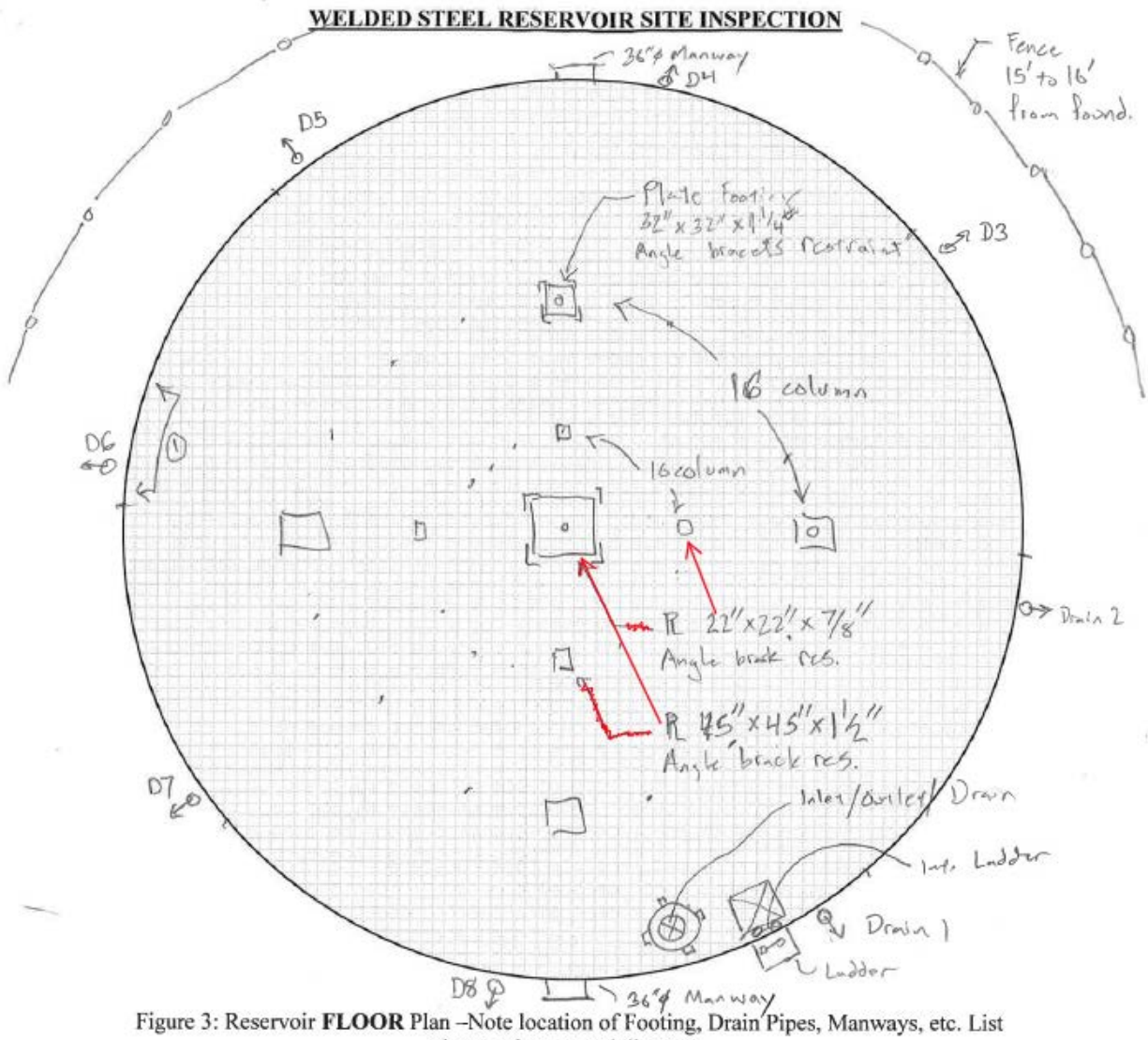

Figure 3: Reservoir **FLOOR** Plan –Note location of Footing, Drain Pipes, Manways, etc. List given and measured diameter.

N

① Approx. location of wall corrosion? Nodules. About 3' o.c. Unclear what cause is about. About 20" above floor. (4) total

Incidental floor plate corrosion. Mostly along seams & edges of plates.

Overall floor good/clear of sediment. Until you get to drain where more sed. is present.

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WELDED STEEL RESERVOIR SITE INSPECTION

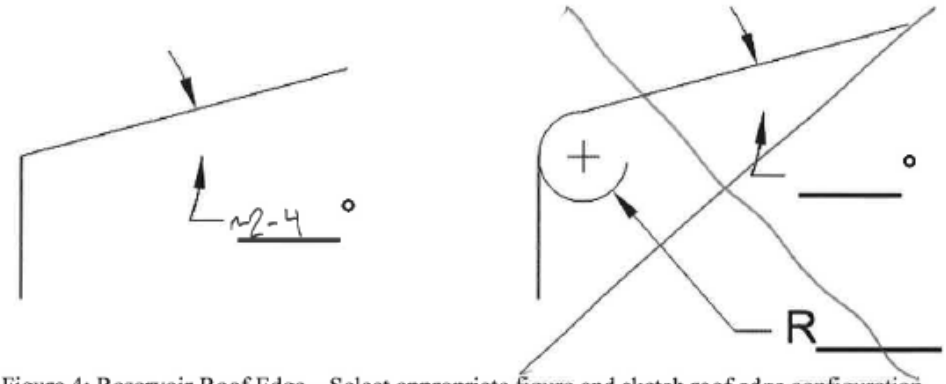
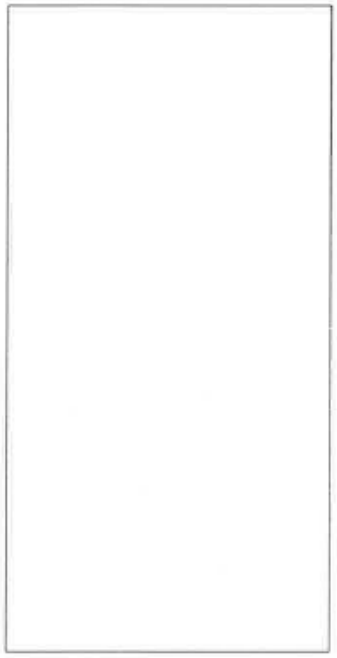


Figure 4: Reservoir Roof Edge – Select appropriate figure and sketch roof edge configuration.

Sketch Roof Type



Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: ____

Thickness: ____
Height: 9'-6"

Figure 5: Exterior shell information and notes.

7

WELDED STEEL RESERVOIR SITE INSPECTION

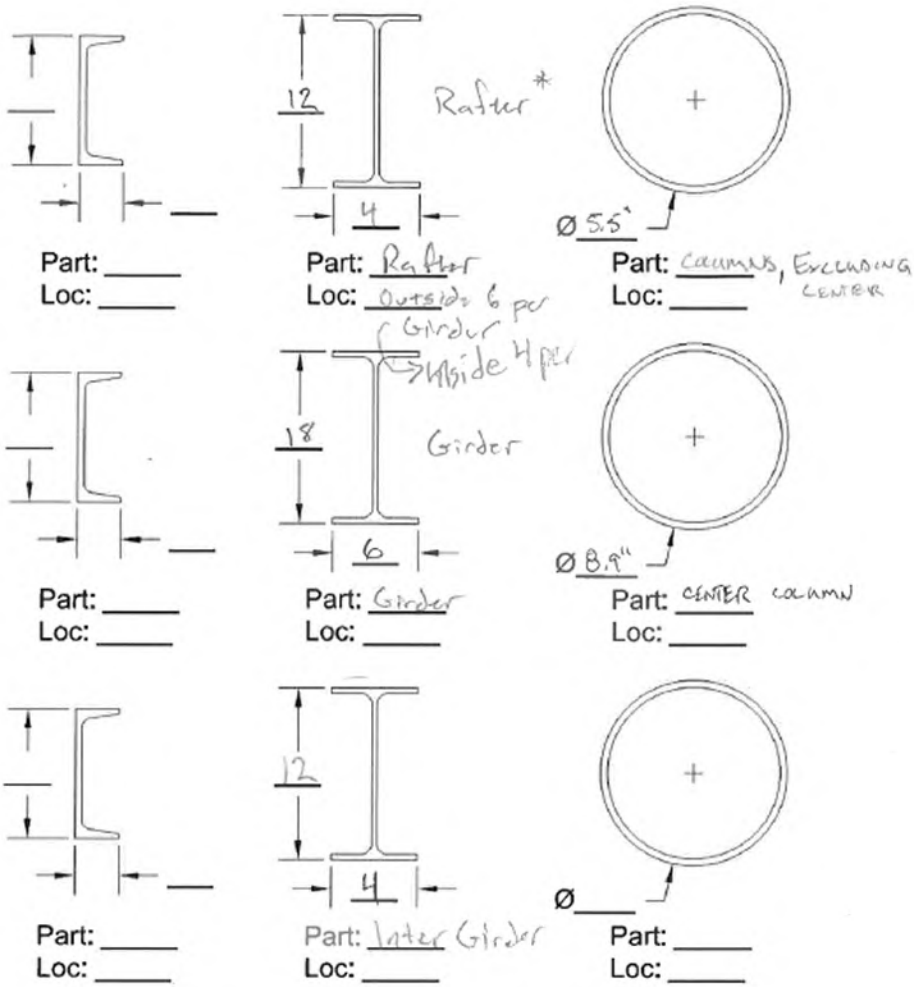


Figure 6: Component detail notes.

* Rafter welded to tops of girders

8

END OF SECTION

Appendix C-4 Whatcom Falls I General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Whatcom Falls I Reservoir *General Info*

Field Visit Date: 6/12/ & 11/7/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	6/12/ & 11/7/2019
Reservoir Name and Location:	Whatcom Falls I - SSE of 3201 Arbor Court 98229
Inspected by:	Nate Hardy, Corey Poland, Chris Hiatt, Greg Lewis, Nick Welling, Jeremy Hailey
Client Staff Present:	Shayla Francis, Jenny Eakins
Year Constructed:	1984
Overflow Destination:	N/A
Discharge Destination/Zone:	276 North Zone, Dakin & Yew Pump
Fill Location:	E side, common with outlet from Whatcom Falls II
Reservoir Material:	Welded Steel

Measurement Type	Measurement	Unit
Volume:	4	MG
Diameter (or other dimensions - see notes):	200	ft
Height	17	ft
Overflow Elevation:	276.5	ft AMSL
Bottom Elevation:	259.3	ft AMSL
Level of Overflow	17.2	ft
Minimum Normal Operating Level:	7	ft
Maximum Normal Operating Level:	16.5	ft
Notes: Operating levels are for summer. Winter levels range from 6.5' to 15.5'. Overflow via clearwell @ Whatcom Falls II.		

Whatcom Falls I Reservoir *Exterior Inspection*

Field Visit Date: 6/12/ & 11/7/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Welded Steel	
Condition:	Good	
Corrosion:	No	
Cage:	Yes	
Security Type:	Lockable access door	
Security Condition:	Good	
Wall Attachment Type:	Good	
Wall Attachment Condition:	Welded	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	16	in
Rung Spacing:	13	in
Side Clearance:	7.5	in
Front Clearance:	6.75	in
Back Clearance:	29	in
Notes: 16 in ring. 16.75 with side rails. Corrosion noted on access door.		

Exterior Fall Prevention System:	
Present at Site:	No
Type:	
Fall Protection System Condition:	
Notes: Used dual carabiner straps	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Side: west	
Material:	Welded Steel	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	No	
Lock:	Yes	
Frame Drain Location:	N/A	
Measurement Type	Measurement	Unit
Size:	30	in
Curb Height:	5.5	in
Notes: Lid 38in across. Holds water in lieu of alarm.		

Entry Hatch:		
Hatch Location:	Side: East	
Material:	Welded Steel	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	No	
Lock:	Yes	
Frame Drain Location:	N/A	
Measurement Type	Measurement	Unit
Size:	30	in
Curb Height:	5.5	in
Notes: Lid 38in across. Holds water in lieu of alarm.		

Entry Hatch:		
Hatch Location:	Roof	
Material:	Welded Steel	
Condition:	Fair	
Gasketed:	No	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	perimeter	
Measurement Type	Measurement	Unit
Size:	30x36	in
Curb Height:	6	in
Notes: has 1/4 in screen. corrosion. lid 34inx40in. Screen: 24x20 inches w/ 7 inch tall mesh.		

Roof Vents and Screen:		
Material:	Welded Steel	
Condition:	Fair	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	1/4	in
Notes: Corrosion on underside. 26in top, 28in diameter pipe. Coating peeling.		

Roof:	
Condition:	Good
Roof Sloped:	Yes
Downspouts:	Yes
Ponding on Roof:	Yes
Roof Finish:	smooth
Slope of roof	2-4 degrees

Measurement Type	Measurement	Unit
Overhang Distance:	2.5	in
Thickness of roof slab	N/A	in

Notes: 2in pipe openings for drain. 12in steel cutout. Coating peeling on roof. Corrosion near vent. Downspouts clogged.

Railing:	
Present at Site:	Yes
Material:	Welded Steel
Condition:	Fair
Corrosion present?	No
Mid-rail:	Yes
Attachment Condition:	Good
Attachment Type:	welded

Measurement Type	Measurement	Unit
Toe Guard Height:	4	in
Top Height:	42	in

Notes: Mid rail at 24in. Has organic buildup.

Grating:	
Present at Site:	No

Foundation:	
Able to be inspected?	Yes
Condition:	Good
Anchoring Condition:	N/A
Photo of Anchoring System:	No
Flexible Couplings at Foundation:	Yes

Measurement Type	Measurement	Unit

Notes: Black mastic layer failing between foundation and structure.

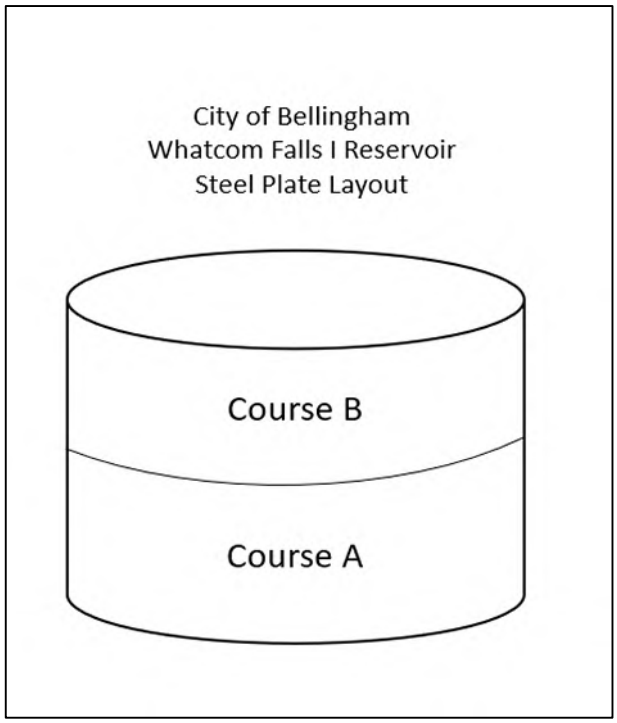
Walls:	
Condition:	
Notes: NE side has heavy organic growth, especially on downspouts. NE side has scratches. E side has moss growing on top courses.	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	Unknown
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	Yes
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	5.4 to 11.5 mils
Exterior Coating Adhesion Testing Results:	prime and top coats tightly adhered, except graffiti cover up areas.
Notes: Red primer and green topcoat. Lead check swab tested negative for lead.	

Whatcom Falls I Reservoir *Steel Plates*

Field Visit Date: 6/12/ & 11/7/2019

Course	Average Steel Plate Height (Feet)	Steel Plate Thickness (Inches)	Notes
Roof	3.0	0.188	
B	7.5	0.397	
A	10.0	0.465	
Chime	N/A	0.252	



Whatcom Falls I Reservoir *Interior Inspection*

Field Visit Date: 6/12/ & 11/7/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Welded Steel	
Condition:	Fair	
Corrosion:	Yes	
Cage:	No	
Security Type:	Lockable access hatch	
Security Condition:	Good	
Wall Attachment Type:	Welded	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	7	in
Back Clearance:	N/A	in
Notes: minor blistering		

Interior Fall Prevention System:	
Present at Site:	No
Type:	
Fall Protection System Condition:	
Notes: 17' ladder	

Interior Roof:		
Condition:	Fair	
Measurement Type	Measurement	Unit
		ft
Notes: Rusting of roof plates, deflecting roof plates, gaps between roof plate and roof support beam. Corrosion at fastener spots.		

Columns:		
Present at Site:	Yes	
Material:	Welded Steel	
Condition:	Good	
Measurement Type	Measurement	Unit
Width/Diameter	5.5	in
Base width	32	in
Column Spacing/Configuration: Center, 16 middle, 16 outer. Column 9in column. bases: center 22in, second 45in. Outside 32in plate. crevice corrosion on bases.		

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes:	

Walls:	
Condition:	Fair
Painters Rings Present:	No
Notes: rust stain comes off when scrubbed. South quadrant: coating coming off. Minor pitting - all 1/16" or shallower	

Interior Coating	
Interior Walls:	Unknown
Interior Floor:	Unknown
Interior of Roof:	Unknown
Interior Ladder:	Unknown
Interior Piping:	Unknown
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	11 to 20 mils. 7.3 mils @ holidays. 7.2 mils on column.
Interior Coating Adhesion Testing Results:	Well adhered.
Notes: Minor blistering, scratches. Areas of rust.	

Whatcom Falls I Reservoir

Miscellaneous

Field Visit Date: 6/12/ & 11/7/2019

Piping		
Inlet Piping:	Size (Inches OD):	24
	Condition:	Fair
	Material:	Ductile Iron
	Notes: Combined inlet/Outlet/drain. some corrosion	
Outlet Piping:	Size (inches OD):	24
	Condition:	Fair
	Material:	Ductile Iron
	Lip (Inches)	8
	Notes: 1/4" height 36" OD removable silt ring	
Overflow Piping:	Size (inches OD):	
	Condition:	
	Air Gap:	
	Screened:	
	Material:	
	Outlet Location:	
	Erosion Evident:	
	Screen Condition:	
	Overflow to roof (feet)	
	Notes: Overflow on clear well of Whatcom Falls II	
Drain Piping:	Size (inches OD):	24
	Condition:	Fair
	Outlet Location:	Sanitary Sewer per ops sheet
	Screened:	Yes
	Material:	Ductile Iron
	Silt Stop Type:	ring
	Air Gap:	Yes
	Screen Condition:	Poor
	Notes: Rock Rip Rap after 54-in diameter base. silt stop has gap. 1/16" mesh backed w/ 1/4". Actually drains to Whatcom Creek	

Piping Facilities		
Exterior Valving:	Type:	BFV
	Condition:	Good
	Secured:	No
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	FH outside
	Size (Inches OD):	4
Roof/Wall Piping Penetrations	Sealed:	N/A
	Leaks:	N/A
Notes: Valves in ground		

Electrical	
Cathodic Protection:	Yes
Impressed Current:	Yes
Anodes:	Yes
Notes: Constant voltage syst, AC breaker defective.	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	No
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	Yes
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	0
Water Quality Sample Taps?	Yes
Notes: Graffiti. Recently cleaned interior.	

Appendix C-5 Whatcom Falls I Condition Assessment Score Sheet

WFI Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	3	0	No Camera. Evidence of Vandalism
	Vegetation Separation	0	0	0	0	0	0	1	0	Organic material accumulating on roof - negative impact on roof drains
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	3	5	4	5	0	0	5	0	Top coat peeling, dirty, scratches. With no overflow, walls inadequate for max. fill, but ok for max op level
	Interior Walls	4	4	4	5	0	0	5	0	Rust discoloration below water line comes off when rubbed. Isolated bubbles and small chips
Floor/ Foundation	Foundation	4	4	4	5	0	0	4	0	Missing grout layer. Some organic buildup and vegetation growing.
	Interior Floor	5	4	5	5	0	0	5	0	Recently cleaned
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	No anchors needed, even accounting for slosh
Roof	Exterior Roof	3	4	5	2	0	0	4	0	Organic material buildup, some coating loss. Ponding. Expected slosh wave will damage the roof at current operating level
	Interior Roof and Supports	3	3	1	2	0	0	0	0	No seal welds - loss of coating and corrosion.
	Columns	3	5	5	2	0	0	0	0	Design traps material. Slosh is expected to upset columns.
Appur-tenances	Exterior Ladders/Fall Protection	3	4	0	0	0	5	5	0	Coating peeling. no fall protection required.
	Interior Ladders/Fall Protection	4	5	0	0	0	5	5	0	no fall protection required.
	Access Hatches	3	5	0	0	2	3	3	0	Coating peeling. Screen too coarse and high maintenance frame. Needs roof hatch railing.
	Railings and Roof Fall Protection	3	3	0	0	0	5	0	0	Coating peeling and rust staining. Meets OSHA Requirements.
	Vents	3	3	0	0	2	0	3	0	Passed design checks w. ex screen, #24 would fail. Corroded.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	5	5	0	3	3	0	5	0	Comb in/out/drain. Only one sheet shows a pipe with Flex Coupling (CRS Sheet 6 of 7), unclear type and adequacy per current code.
	Outlet Piping	0	0	0	3	0	0	0	0	Comb in/out/drain.
	Drain Piping	0	0	0	3	3	0	3	0	Comb in/out/drain. Silt stop not fully functional.
	Overflow Piping	0	0	0	0	0	0	0	0	No overflow
	Washdown Piping	0	0	0	0	0	0	5	0	Fire hydrant on site.
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	0	0	0	0	0	0	5	5	
	Isolation Valving	0	0	0	0	0	0	5	5	
Misc.	Cathodic Protection System	0	0	0	0	0	0	1	4	Not currently functioning
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	3	0	
	Categorical Score	3.5	4.2	4.0	3.5	2.5	4.5	4.0	4.7	

Overall Score
3.8

Appendix D Dakin II

Appendix D-1 Dakin II Geotechnical Report

GEOENGINEERS

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Dakin I and Dakin II Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Dakin I and Dakin II reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the Dakin I and Dakin II site, located as shown in the Vicinity Map, Figure 1. The Dakin I reservoir is a round reinforced concrete structure with a hopper base and the Dakin II reservoir is a prestressed concrete reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone.

Surface Conditions

The project site is located approximately 550 feet to the east of Sylvan Street and 50 feet north of Balsam Lane. The reservoir is located on top of a small hill and the site drops off in all directions. The site is bounded by a wooded area, associated with Big Rock Park, in all directions. A small gravel roadway leads to the site from the southwest.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing two new geotechnical borings B-4 and B-5 (2019)—on March 25, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The borings were completed to a depth of 5½ to 17½ feet below the existing ground surface (bgs). The location of the explorations are shown in the Site Plan Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration logs are presented in Figures 4 and 5.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites. We also reviewed as-built drawings for this site.

- **Fill** – Fill was encountered at the surface of both explorations. Fill extended to 4 feet in B-4. The fill was significantly thicker in B-5 extending to 15 feet bgs. The fill generally consisted of medium stiff to stiff blue-brown silt with variable amounts of sand, gravel, and organic matter. Boring B-5 encountered a thin layer of wood at the interface between the fill and sandstone. As discussed subsequently, we expect that the fill is representative of backfill around the reservoir foundation and does not extend below the foundation.

- **Chuckanut Sandstone** – Chuckanut sandstone was encountered below the fill at 4 feet bgs in B-4 and 15 feet bgs in B-5. The fine to coarse grained sandstone was brown with weak cementation and occasional bedding planes.

Groundwater

Groundwater seepage was not observed at the final depth of borings. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Dakin I

Based on the conditions encountered in our boring, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. Available drawings and sketches do not provide any additional foundation details.

Dakin II

Based on review of drawings for the project by PEI Consulting Engineers & Surveyors dated 1991 and our boring, the existing Dakin II reservoir has a mixed bearing profile. A portion of the reservoir is supported on bedrock, and the southern portion is supported on lean concrete extending to bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a

Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest know faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structures bear on sandstone which is not at risk of liquefaction.

American Concrete Institute/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI), publication D110-13 of the American Water Works Association (AWWA) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI, AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group (Dakin I)	II
AWWA Seismic Use Group (Dakin II)	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	94.4
1-Second Period Spectral Response Acceleration, S_1 (percent g)	36.9
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.43
MCE_G peak ground acceleration, PGA	0.390
Seismic design value, S_{DS}	0.643
Seismic design value, S_{D1}	0.352

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_u 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the

two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 6 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	12	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeters per second, g = gravity

M_w 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends more than 78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 7 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	14	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Dakin I

Based on the conditions encountered in our boring, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure

of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Dakin II

Based on review of project drawings referenced previously and our boring, the existing Dakin II reservoir has a mixed bearing profile. The reservoir is supported on bedrock, although the southern portion is supported on lean concrete extending to bedrock. According to the as-built drawing, a design allowable bearing pressure of 4,000 psf was used for design. This structure could be evaluated based on the design bearing pressure of 4,000 psf; in our opinion, an allowable bearing pressure of 6,000 psf would also be appropriate for this tank. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

We understand that the existing reservoirs include below grade walls. Our recommendations for evaluating below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section and backfilled with structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Dakin I

As previously mentioned, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

Dakin II

Based on review of publicly available LiDAR for the site, there are slopes inclined at 30 to 40 percent to the southeast of Dakin II that is approximately 25 feet tall. The existing Dakin II reservoir has a mixed bearing profile between bedrock and lean concrete extending to bedrock. Based on the structure support extending to bedrock, it is our opinion that there is a low risk of slope instability that would impact the existing reservoir. Based on the soil conditions encountered in boring B-5 and site topography which includes slopes at 2H:1V or flatter, it is our opinion that there is a low risk of slope instability for the slope adjacent to the tank.

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AJH:HP:JRG:tln

Attachments-

Figure 1 – Vicinity Map

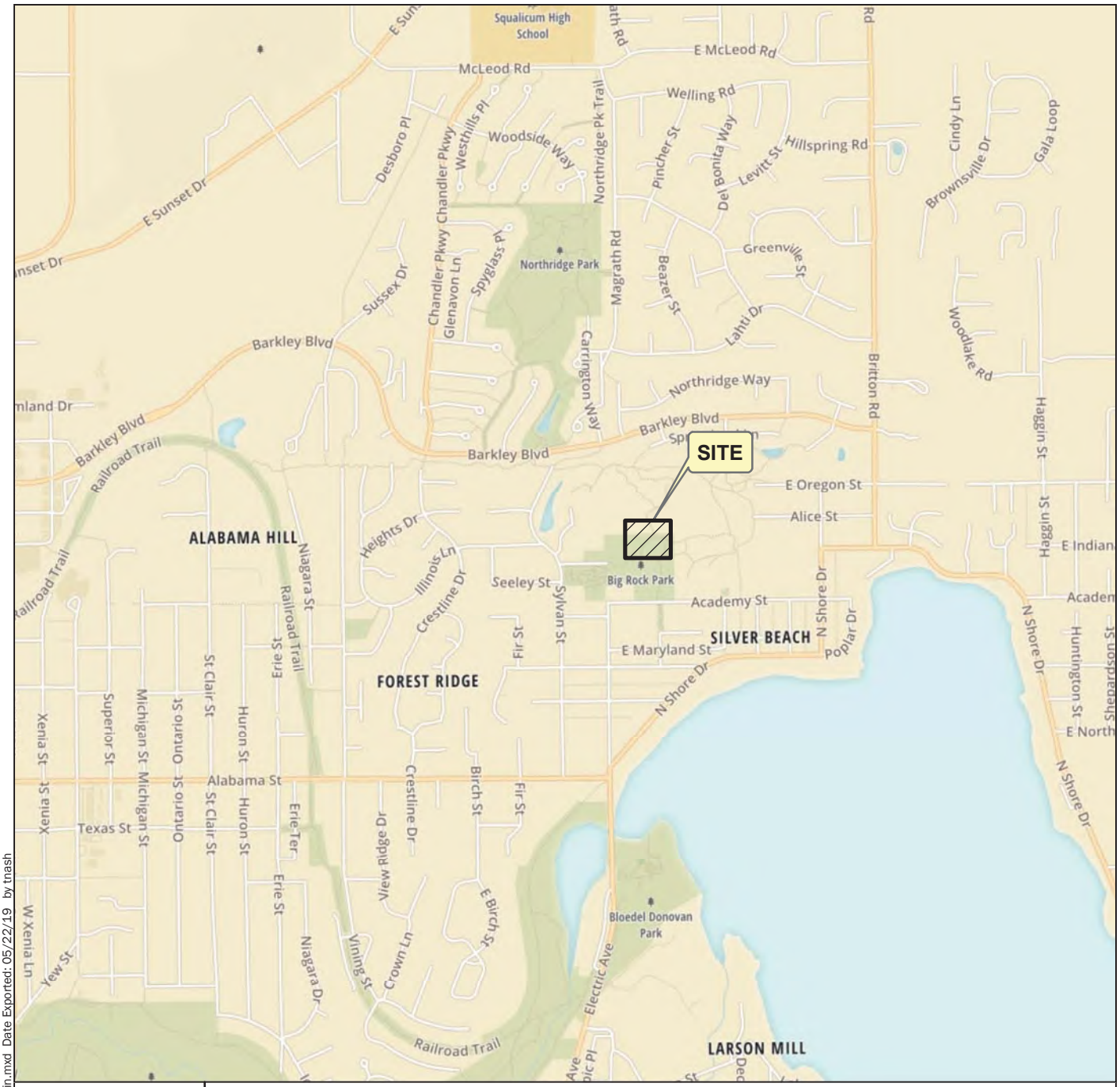
Figure 2 – Site Plan

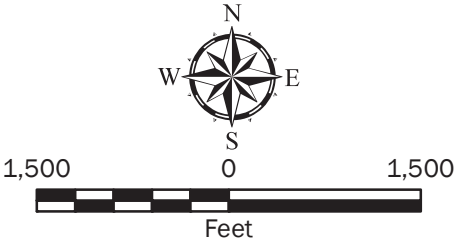
Figure 3 – Key to Exploration Logs

Figures 4 and 5 –Log of Borings B-4 and B-5

Figure 6 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 7 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault





Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Dakin I and II Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 1

\\geoengineers.com\WAN\Projects\0\0356159\GIS\035615900_F01_VM_Dakin.mxd Date Exported: 05/22/19 by tnash

Legend

Boring by GeoEngineers (2019)

75 0 75

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Dakin I and II Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled Start 3/25/2019 End 3/25/2019	Total Depth (ft) 5.5	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) 510 Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment EC-95	
Easting (X) 1257680 Northing (Y) 649290		System Datum WA State Plane North NAD83 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown silt with sand, gravel and organic matter (soft to medium stiff, moist) (fill)			
		X	12	53/11"		1						
505								Sandstone	Brown sandstone (Chuckanut Formation)			
	5	X	4	50/4"		2						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-4

GeoEngineers

Project: COB Reservoir Inspection and Repair - Dakin I
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

Drilled	Start 3/25/2019	End 3/25/2019	Total Depth (ft) 17.75	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	500 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) Northing (Y)	1257840 649280		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown silt with sand, gravel and occasional organic matter (stiff, moist) (fill)			
		☒	5	50/5"		1 MC				13		
495	5	☒	18	23		2 MC		ML	Blue-brown silt with sand, gravel and wood fibers (stiff, moist)	13		
		☒	18	7		3 MC			Becomes medium stiff	18		
490	10	☒	6	4		4 MC				20		
485	15	☒	5	50/5"		5		Wood	Wood fibers			
								Sandstone	Brown sandstone (Chuckanut Formation)			
		⊠	2	50/2"		6						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-5

GeoEngineers

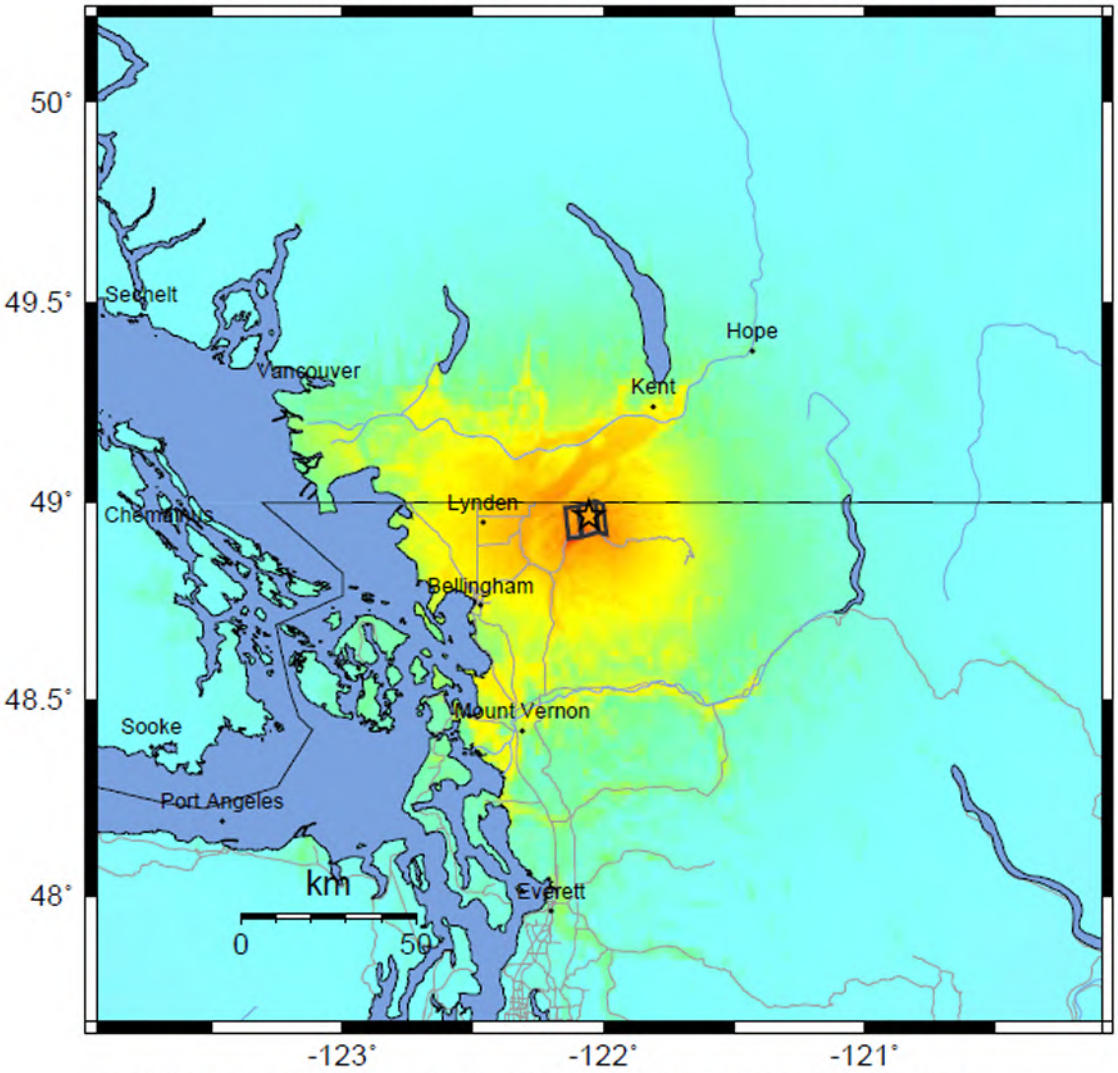
Project: COB Reservoir Inspection and Repair - Dakin II
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 5
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 6

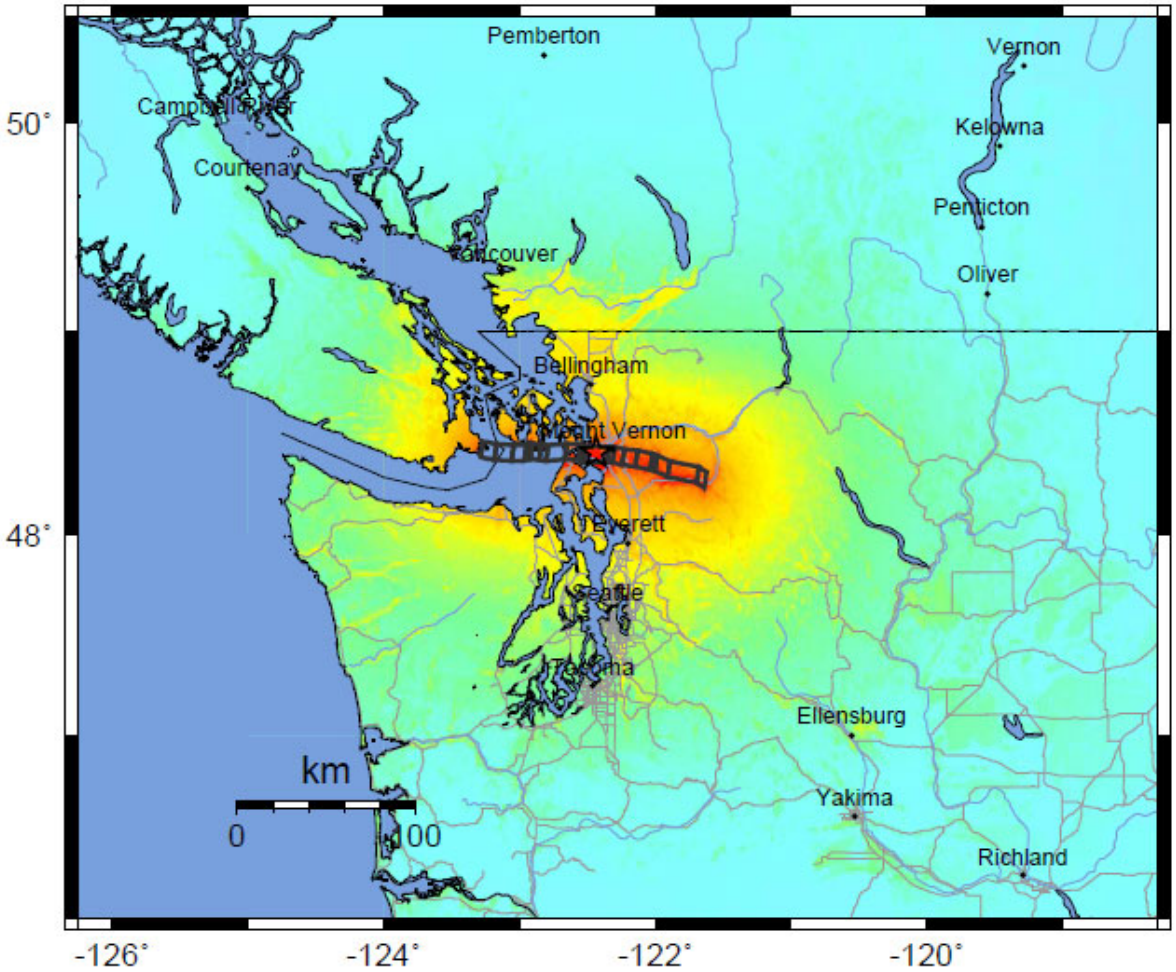
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 7

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix D-2 Dakin II Structural Report

CITY OF BELLINGHAM

CH 6: DAKIN 2 RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

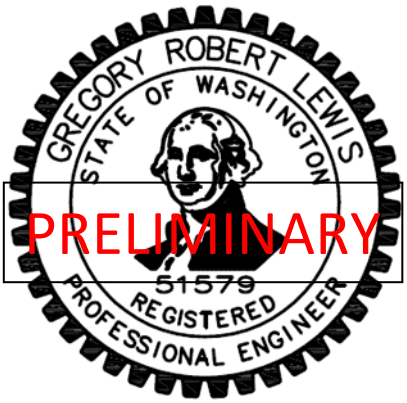
Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to identify potential problem areas that may be candidates for repair or upgrade at the Dakin 2, 0.5 Million Gallon (MG) prestressed concrete reservoir. The reservoir is located near 3800 Balsam Ln, Bellingham, WA (Lat. 48.769, Long. -122.420), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on April 30th, 2019 by Peterson Structural Engineers (PSE), and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Dakin 2 Prestress Reservoir – 0.5 MG

2.1 Description & Background

Based on the available documents, it appears that PEI Consulting Engineers & Surveyors prepared a performance specification for the reservoir's design. Based on this specification, DYK, now known as DN Tanks, prepared the design and performed prestressing of the reservoir's tendons and wire. PEI acted as the Civil Engineer of Record for the project. The original construction drawings are dated January 1990 with record drawings dated March 1991. The reservoir is a partially buried, 68-foot inside diameter by 19.5-foot high (to bottom of roof), strand-wrapped, pre-stressed concrete water reservoir with a self-supporting dome roof and parapet wall. The dome roof is 4-inches thick at the center with a thickened edge. No joints were observed in the roof slab as it appears to be a single, monolithic pour.

Per the provided as-built drawings the reservoir contains vertical pre-stressing strands in the core wall. The dome roof is keyed into the side of the wall below the top of the parapet. There are 52 total 1-1/4-inch diameter vertical tendons per the drawings. The wall base connection utilizes 104 seismic cable sets (2 per vertical tendon) each with (3) 3/8-inch diameter, 7-wire galvanized strands. The floor connections are detailed with a continuous elastomeric bearing pad while the dome roof to wall connection is detailed with a roughened concrete keyway.

The original drawings specify a roof live load of 30-psf load. This is sufficient to meet the current code requirements of either the 20-psf live roof or 25-psf snow design load. For the evaluation contained herein the controlling roof load used is 30-psf.

Per the drawings, the overflow height was listed as 18.5-feet, which is 1-foot below the dome roof-to-wall interface. Per the City, the reservoir's operating level averages between 12 and 15.5-feet, for a maximum storage volume of about 0.42 MG. PSE performed its evaluation based on the 18.5-foot maximum operating level.

2.1.1 Description of Additional Site Structures and Features

The site also includes the Dakin 1 reservoir. This reservoir is not within close proximity to Dakin 2 and it does not appear that it will have any impact on the Dakin 2 reservoir due to a seismic event. Results of a detailed Dakin 1 evaluation are provided in a separate report.

2.2 Visual Condition Assessment and Associated Recommendations

We performed a site visit to observe the as-built current condition of the reservoir's interior, exterior, and site conditions. The reservoir was drained for our inspection, which was performed on April 30th, 2019.

<u>Dome Roof:</u> In general, the main body of the dome roof and top roof surface were noted to be in good condition with only minimal circumferential pattern cracking girding the middle of the dome. No joints were observed in the roof slab and it appeared to be a single, monolithic, pour. The edge of the roof is thickened, resulting in a flattened exterior section. Further, the reservoir wall extends above the roof resulting in a parapet. Four roof drains are located along the perimeter but appear to be inadequate to handle the amount of debris which collects along the roof edge. When debris were cleared away from drain openings the drainpipes were observed to be clogged.

The roof has a single 6 by 8-foot access hatch and a 3-foot square vent opening at the center of the dome. The hatch, vent cover, and associated concrete curb all appeared to be in good condition with no apparent structural deficiencies. The vents appeared to meet current requirements for water quality. The access hatch, which has a gutter inset into the curb, is considered a high-maintenance design per the Washington State Department of Health. The gutter-drains for these hatches should be screened to further protect them.

The interior condition of the reservoir roof was found to be in generally good visual condition with minimal cracking and a single noted instance of corrosion. This occurrence of corrosion is likely a location where either the roof reinforcing did not have the necessary cover, or a concrete anchor was left in place from construction, but likely not a result of a structural failure. Located throughout the dome there appeared to be a mineralization developing. This mineralization did not appear to be resulting from or causing any structural issues and is discussed further in the reservoir floor section.

Prestress Walls: The exterior walls of the reservoir are covered in shotcrete to protect the strands wrapping around the core wall. The reservoir is partially buried, up to 8-feet on the west side and down to approximately 2-feet on the east. The site drops off towards the north, east, and south around the reservoir. The surrounding grade is an uneven grass field within the boundary of the site. Per the drawings, the reservoir was placed over bedrock which appears had to be removed to create a level bearing surface for the reservoir's base slab. This general stratum is confirmed in the geotechnical report in which Chuckanut Sandstone was found about 15 feet below ground surface, adjacent around Dakin 2's foundation.

The walls of the reservoir were found to be in generally good condition. One 3-foot long instance of efflorescence was noted along the east side. Additionally, minor alligator cracking was observed around the reservoir but while this type of cracking was extensive in its area of effect, the individual cracks that comprise the alligator cracking generally appeared to be minor.

In additional to our observation, PSE performed "sounding" of the walls around the perimeter of the reservoir. Sounding is a process of tapping the reservoir's exterior surface with a hammer to listen to the report from the strike. A sharp "ping" sound is generally indicative of a competent shotcrete layer while a dull or hollow "thud" sound can be indicative of delamination within the shotcrete layer. While areas of the reservoir were either below grade or out of reach and could not be sounded, the areas that were sounded were found to be competent in the areas inspected.

The tops of the wall extend up past the roof resulting in a parapet. Within the wall tops, vertical prestressing block-outs were visible. During construction, after the wall is built, these block-outs allow access to the high strength threadbars in the walls. These threadbars are tensioned in order to ensure the wall is under constant vertical compression, which improves performance of the wall under operating and seismic loads. After the completion of tensioning the block-outs are packed with high strength non-shrink concrete to seal them from the elements. These seals did not appear to be performing well and a majority of the sealed openings had adjacent stress cracking and efflorescence. Additionally, 8 caps were noted to have substantially or completely failed, exposing the top of the threadbars to the elements.

The main surface of the interior walls of the reservoir were found to be in good condition with very minimal instances of bug-holes or surface defects. Issues were noted along the dome roof-to-wall keyed joint in which sections of concrete were beginning to fail and spall. As reinforcing is near the surface in these zones, there is the potential for corrosion to occur resulting in further issues. The wall consists of 7 wall sections, each with 7 threadbars and a single, shorter section with only 3 threadbars. These sections match the number of sections listed in the provided as-built drawings. Formwork holes and epoxy injection ports looked to be adequately capped, sealed and in good condition.

Slab Floor: The interior floor slab was visibly found to be in good condition with no observed issues of cracking or signs of failure. As the floor slab appears to be a single, monolithic pour, no joints are present in the slab. Throughout the surface of the slab it was noted that there is extensive mineralization. This mineralization did not appear to be the cause of any noted structural issues. Instances were observed on the roof, the ladder and drain cover, and throughout the floor. It is unclear what is causing this, but it is likely a result of water chemistry and/or suspended minerals in the water. While the mineralization can be removed, it does not currently appear to be causing any issues.

Appurtenances: The reservoir piping consists of a 12-inch inlet, outlet, and drain line and an 8-inch overflow line located on the east side of the reservoir. Overall these appeared in good visual condition, but corrosion carbuncles were noted at joints along with instances of the aforementioned mineralization. The ladder was observed to be in good condition but also had instances of mineralization around its lower body.

2.2.1 Visual Condition of Additional Site Structures and Features

See the Dakin 1 report for additional information on other site features and structures.

2.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the reservoirs under the current adopted code and standards. Seismic loads were determined using the American Society for Civil Engineers, “Minimum Design Loads for Buildings and Other Structures” (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13, “Wire and Strand Wound Circular Prestressed Concrete Water Tanks” was utilized. Evaluation was based on the provided as-built original construction documents and site visit observations.

2.3.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof thickness of 4-inches exceeds the required AWWA D110 minimums of 3-inches. The additional thickness helps to resist cracking and ensures the appropriate concrete cover for the reinforcing. Based on the plans, reinforcing in the dome roof was found to meet the minimum requirements for spacing and density.

Vertical Wall Reinforcement: The reservoir’s as-built drawings show vertical pre-stressing within the walls spaced at 4’-1-7/8” on center. While PSE could not observe the vertical pre-stressing directly, we did observe the location of the block-outs set into the parapet walls, which matched the number and spacing

indicated on the plans. Per analysis, the size and spacing of the pre-stressing was determined to be adequate.

Columns: No columns are present as this reservoir has a dome roof.

Foundations: The foundations for wall footings were evaluated based on information obtained from the provided as-built drawings. The design was found to be adequate for current codes. Per the Geotechnical report the allowable bearing capacity for the site was given as 6,000-psf. Based on PSE's design checks, this bearing capacity is adequate to resist a maximum design soil bearing pressure of approximately 2,500 psf, which occurs at an overflow operating level of 18.5-feet.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Joints: Per the drawings there are 104 sets of seismic cables (two sets per vertical pre-stressing bar). Based upon an analysis it appears that this meets current code and provides the necessary lateral capacity for an operating level of up to 18.5-feet. Should the City require, the reservoir can be operated all the way up to the 18.5-foot design overflow level, which is at higher than current operation.

Strand Wrap: The circumferential pre-stressing requirements and wrapping diagram for the reservoir are detailed on the pre-stressing load distribution diagram on Drawing Sheet 9 of the original drawings dated January 1990. Based on PSE's evaluation under current code requirements for the anticipated static and seismic loads resulting from an operating height of 18.5-feet, the circumferential pre-stressing is slightly above is utilization capacity, by about 2% maximum. This assumes a final circumferential prestressing force requirement of 240-psi used in the wall design to account for the dome roof and as covered in AWWA D110 Section 3.5.2.1. However, the prestressing is adequate and not overstressed when evaluated for the current 15-foot maximum operating level.

Columns: No columns are present as this reservoir has a dome roof.

Freeboard/Slosh: The current freeboard at a maximum operating level of 15-feet is 4.5-feet. This is adequate to handle the anticipated 3-foot slosh wave. Should the reservoir be operated at 18.5-feet, which would reduce the available freeboard to 1-foot, PSE determined the existing roof has the requisite structural capacity to resist the slosh impact wave when considering the slosh wave height as an equivalent hydrostatic force based upon the calculated slosh height.

The roof hatch is located along the exterior edge of the dome and may blow out or be damaged in a seismic event if a slosh wave hits the underside of the hatch when operated at a level without adequate freeboard. This would likely be a localized failure or cause isolated damage to the hatch and is not expected to impact the overall structural performance of the reservoir.

2.4 Summary

The Dakin 2 Reservoir was constructed in 1990 and appears to be in substantial conformance to current design codes and standards. The reservoir was designed to operate at an 18.5-foot operating level and based on current code, it is only slightly out of compliance due to Code updates resulting in increases to

the design loads. At its current lower operating level, the potential seismic demands are further decreased and no structural resulted.

Issues noted with the reservoir include the failure of concrete plugs located in the threadbar block-outs, instances of efflorescence, mineralization build-up, instances of corrosion, concrete cracking at the dome roof bearing seat, and blocked roof drains. Major structural issues identified were few but without maintenance some of the above listed issues could result in further problems and become more difficult to repair in the future. As the reservoir appears to be in good condition making the appropriate fixes now will ensure a longer service life for the reservoir.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

The reservoir was observed to be in good condition and generally compliant with code at its current operating level. This is further supported by PSE's analysis and evaluation. PSE recommends the following repairs and upgrades.

1. The current condition of the concrete above the threadbar is in poor condition. Due to the observed failures, all of the block-outs should be cleared of concrete. Block-outs should be cleaned out by a competent contractor familiar with prestressed construction. After removal of the concrete, the top of the threadbars should be observed by a qualified structural engineer such as PSE or DN Tanks to determine if any additional issues are present which could adversely impact the performance of the vertical threadbar. Once inspected, any vertical threadbar issues should be corrected or repaired. After repair, all exposed surfaces (such as the concrete and the end of the threadbar) should be treated with a corrosion inhibitor/bonding agent (for example: Armetek EpoCem 100). After the corrosion inhibitor/bonding agent has set, the block-outs should be filled with a high-strength non-shrink grout.
2. Along the interior dome roof-to-wall interface, any cracked concrete should be removed, and the site observed for additional corrosion. Where corrosion is noted, it should be removed back to component material. These areas should be coated with a bonding agent and patched. The exterior top and interior bottom of the roof-to-wall interface should be coated to prevent water infiltration through this joint.
3. The existing drains are prone to blockage due to their relatively small diameter and the parapet configuration of the roof. Working with a qualified structural engineer experienced in prestress reservoirs, such as PSE or DN Tanks, to verify horizontal and vertical pre-stressing location, the existing roof drains should be expanded, and new drains added. In addition to roof drainpipes, the configuration should allow the water to bypass the vertical drain if clogged. See Figure 2-14 for an example of the approach recommended.
4. Instances of corrosion were observed on the underside of the roof. These areas should be cleaned back to component material and coated to prevent further corrosion.
5. As part of maintenance, any piping and ladder corrosion and mineralization should be removed.
6. The roof access hatch gutter drainage points should be screened to protect them. This will bring them into compliance with the Washington Department of Health requirements for the sanitary protection or reservoirs.

2.6 Scans of Select Construction Documents

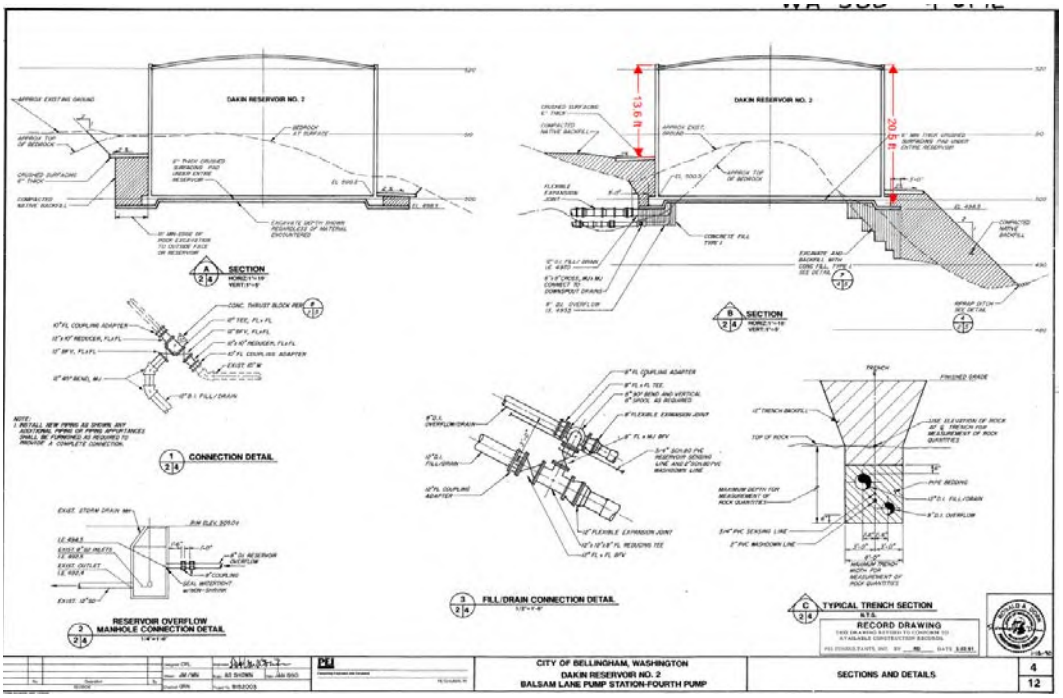


Figure 2-1: Dakin 2 – Section and Details, Sheet 4

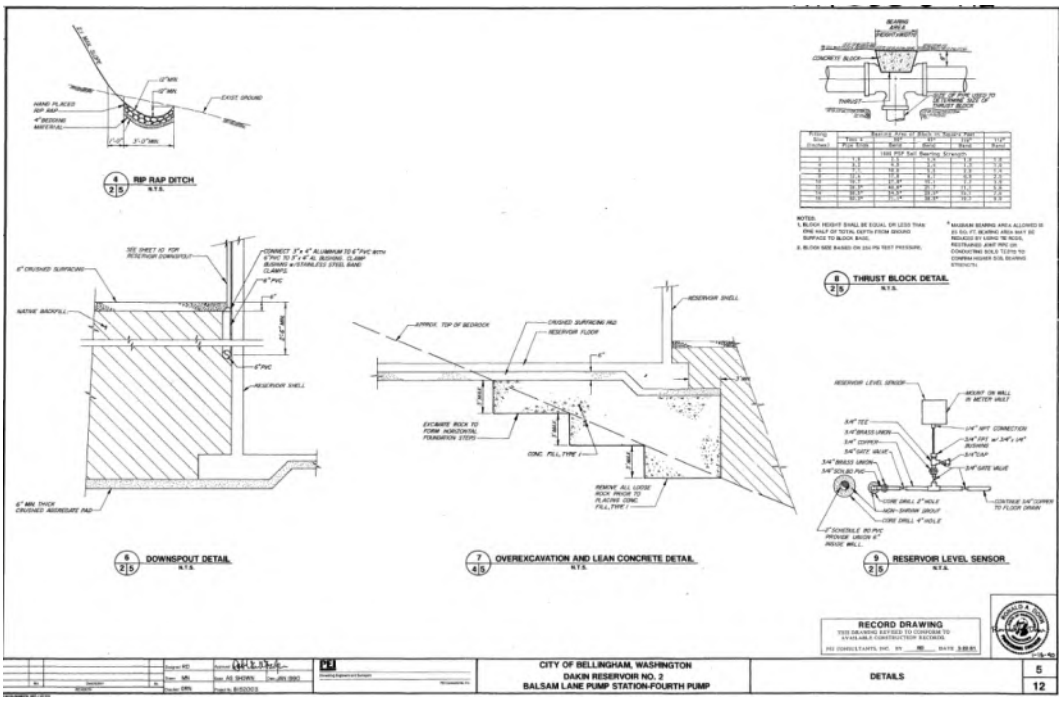


Figure 2-2: Dakin 2 –Details, Sheet 5

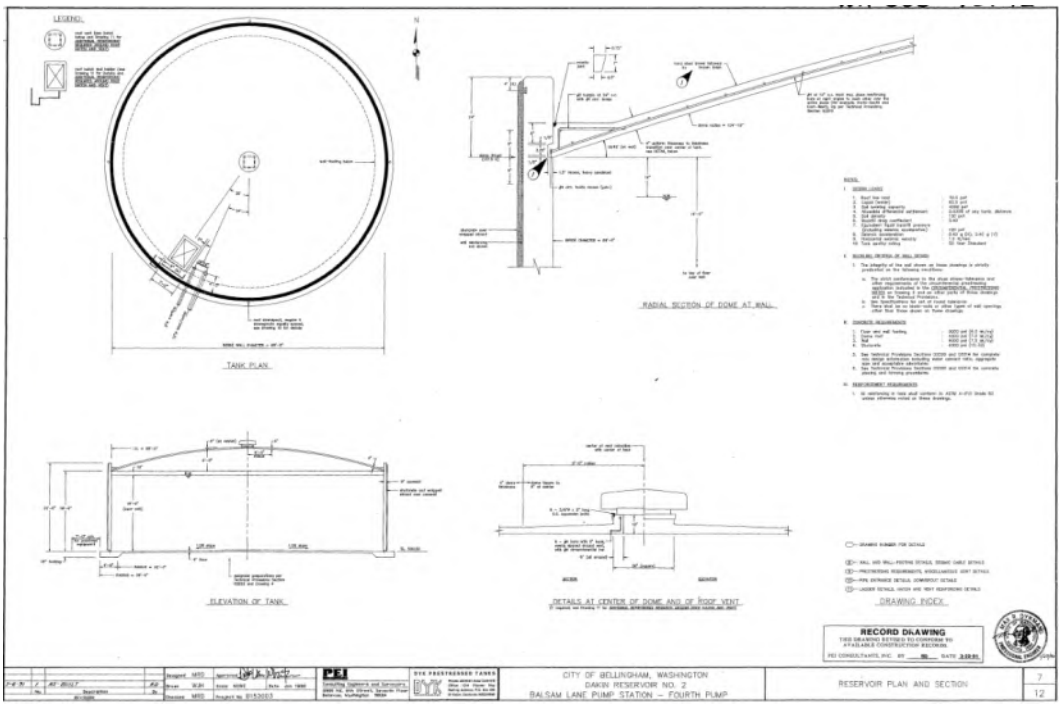


Figure 2-3: Dakin 2 – Reservoir Plan and Section, Sheet 7

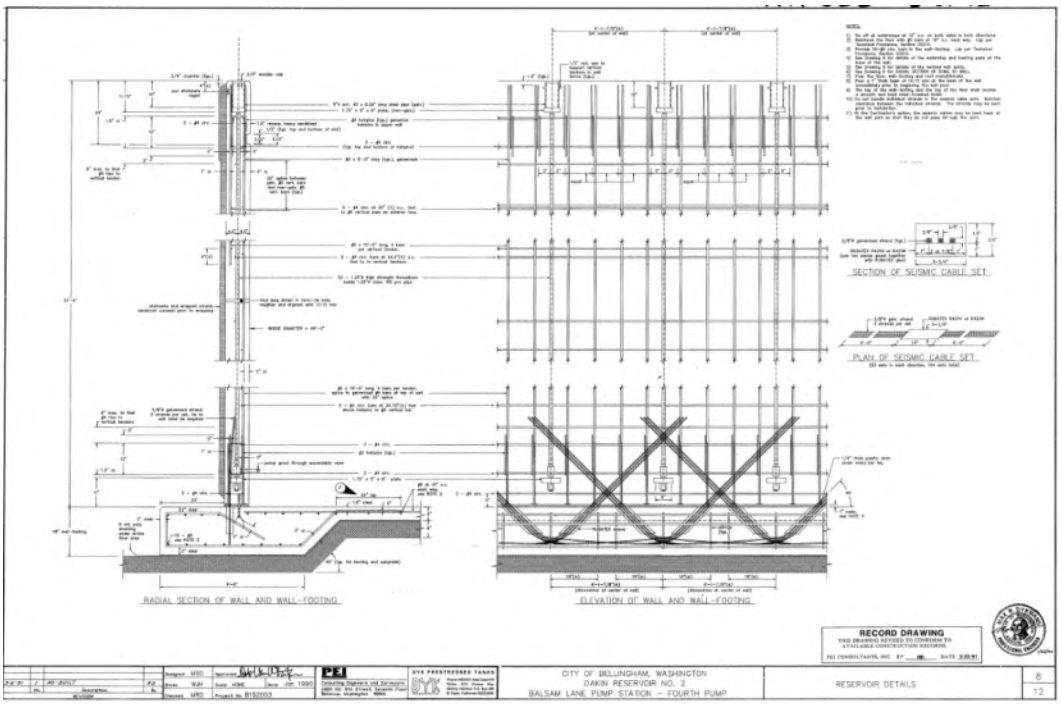


Figure 2-4: Dakin 2 – Reservoir Details, Sheet 8

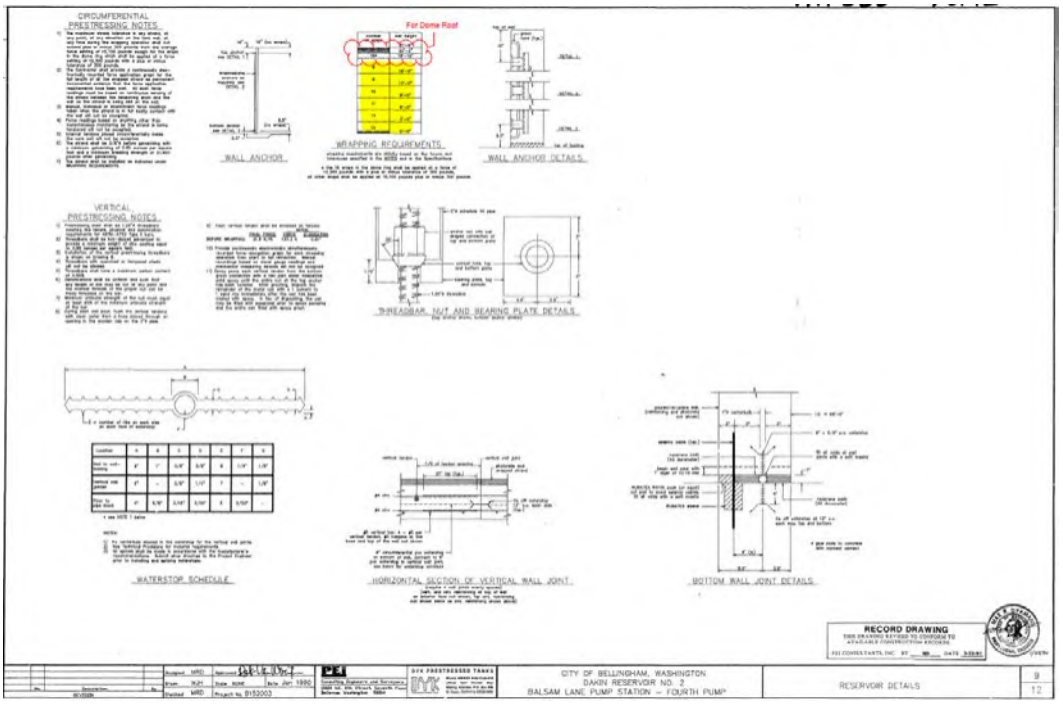


Figure 2-5: Dakin 2 – Reservoir Details, Sheet 9

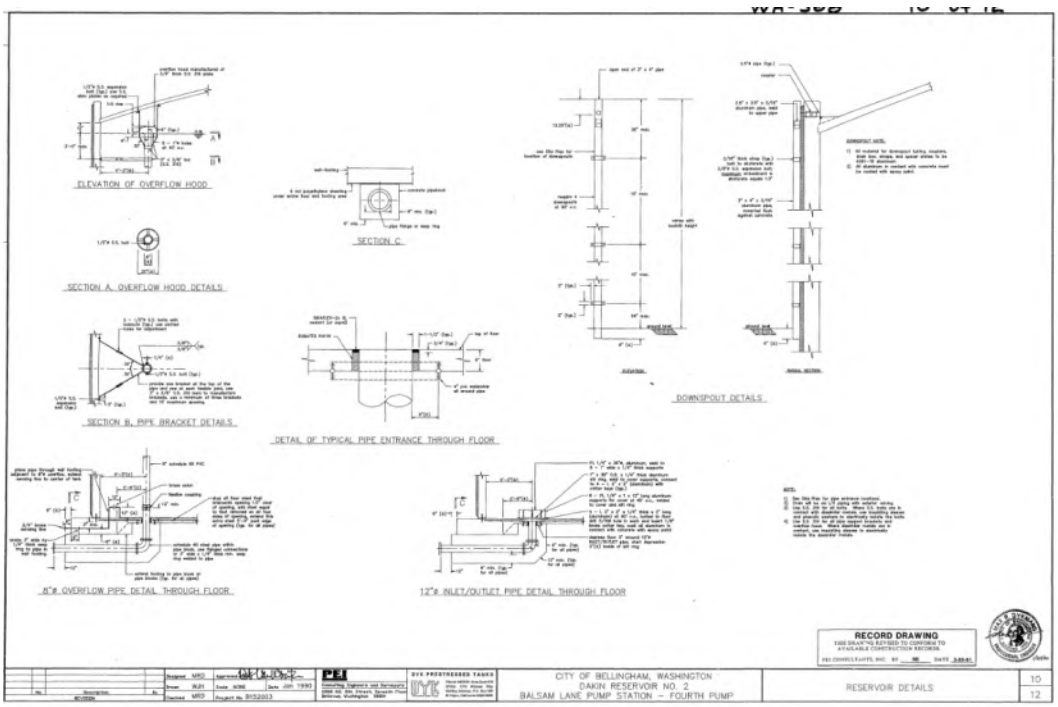


Figure 2-6: Dakin 2 – Reservoir Details, Sheet 10

2.7 Observations Pictures

Figure 2-7: Dakin 2 - West Elevation

Figure 2-8: Dakin 2 - Exterior Ladder

Figure 2-9: Dakin 2 - Example of Alligator Pattern Cracking observed around reservoir in shotcrete

Figure 2-10: Dakin 2 - Instance of Efflorescing noted in Shotcrete

Figure 2-11: Dakin 2 - Reservoir Dome Roof

Figure 2-12: Dakin 2 - Top of Parapet and Concrete Threadbar Cap

Figure 2-13: Dakin 2 - Failed Concrete Threadbar Cap, Threadbar can be seen in hole

Figure 2-14: Dakin 2 - Blocked Roof Drain. Retrofit drain should allow for overflow if its main pipe is blocked (see example on right)

Figure 2-15: Dakin 2 - Roof Hatch

Figure 2-16: Dakin 2 - Roof Vent

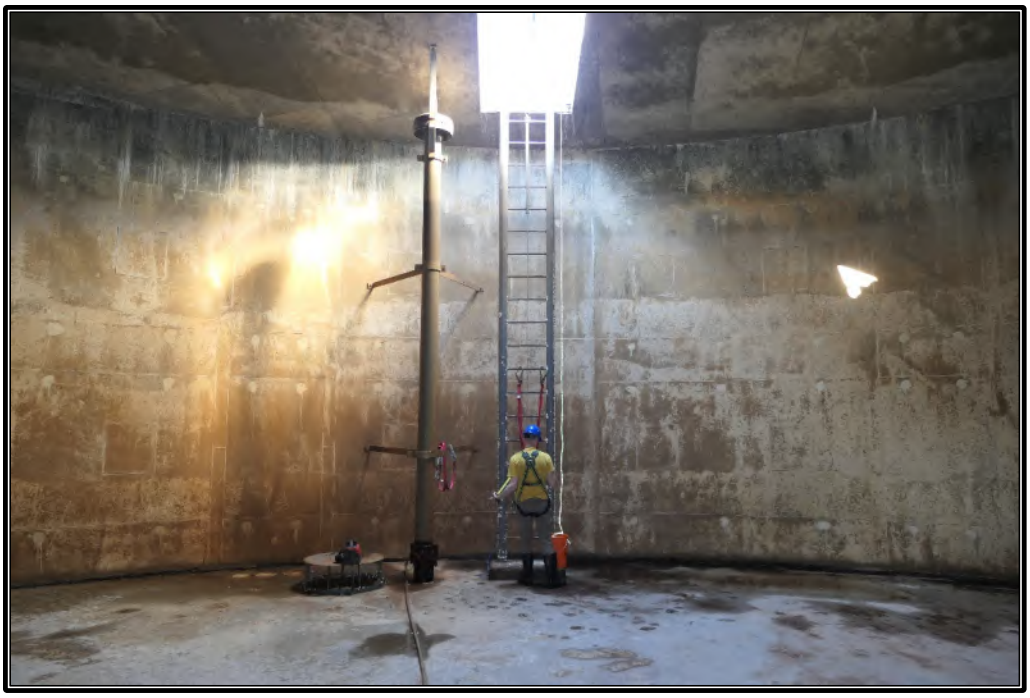


Figure 2-17: Dakin 2 - Entry Ladder, Overflow, and Inlet/Outlet/Drain

Figure 2-18: Dakin 2 - Wall – Short wall section center

Figure 2-19: Dakin 2 - Dome Roof and Square Vent Opening

Figure 2-20: Dakin 2 - Spalled Concrete along Dome Roof and Wall Bearing Interface

Figure 2-21: Dakin 2 - Mineralization along floor

Figure 2-22: Dakin 2 - Close-up of Floor Mineralization

Figure 2-23: Dakin 2 - Corrosion carbuncles at base of overflow pipe.

Figure 2-24: Dakin 2 - Mineralization on Ladder

2.8 Field Notes

PRESTRESSED RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Eval

PROJECT NUMBER: A1802-0019

PRESTRESSED RESERVOIR SITE INSPECTION

Reservoir Name: Dakin II Access 2918 Sylvan St, Bellingham 98226 Lat 48.7692, Long -122.4199

Site Visit Date: 4/30 Reservoir Type: Prestress w/ Dome Roof & Parapet

Temperature and weather: Sunny, clear 41°F

Site Conditions: Clear of trees and brush, short grass, site slopes east w/ severe drop once past fence-line.

PSE Staff: Greg Lewis

Client/Other Staff: Nathan Carey (Murray Smith) Jeremy (NW Corrosion) City Staff

Exterior Inspection

Backfill Dim. to Top of ~~Roof Slab~~ Parapet: (N) 19.3' (E) 20.5' (S) 19' (W) 13.6'

Roof Slab Thickness: 4" / 4" (from hatch) (drawings/measured) Roof Overhang Dimension: N/A / None (drawings/measured) parapet roof

Drip Groove? (Y/N): N/A / None (drawings/measured) Threadbar Pocket Spacing: 4'-1 7/8" / 49" approx (drawings/measured)

Top Surface Roof Slab Condition: Good, minor cracking around vent. Parapet has cracking & post-tension caps are falling in some areas

Ladder/Vents/Hatch/Joint Conditions: Ladder: 1" rungs, AL good condition, only (5) 3/8" bolts, many holes missing bolts.

Exterior Shotcrete Condition: Good, some patterning on surface but competant. One area (NW side) w/ noted efflorescences, only 2' long, 4' above grade, minor cracking

1

PRESTRESSED RESERVOIR SITE INSPECTION

Sound shotcrete at regular intervals and record results: Sounding good. Checked surface plus around downspouts & efflorescence patch, no issues noted.

Wire/Strand Wrapping Condition: Not viewed

Other Comments:

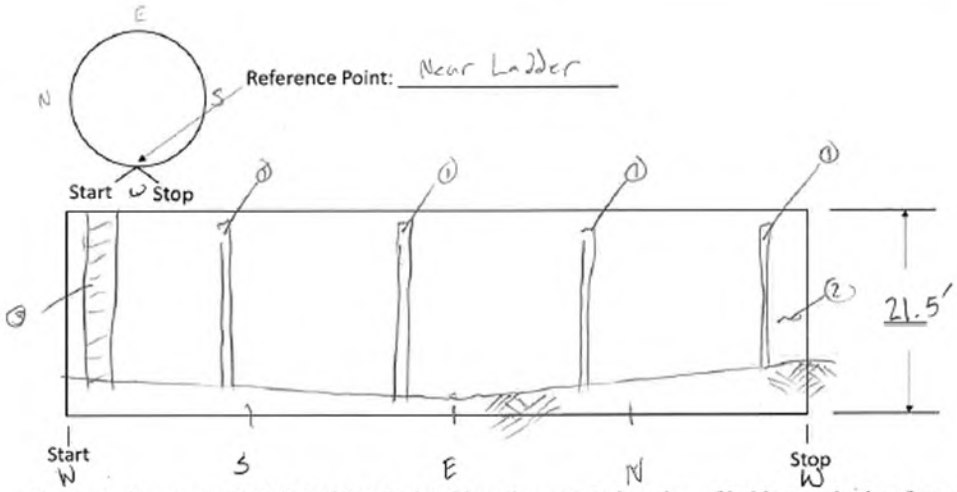


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

① Downspout ② Efflorescence ③ Ladder

One N/E/S side fence w/i about 12.5' from res. W side is Dakin 1

PRESTRESSED RESERVOIR SITE INSPECTION

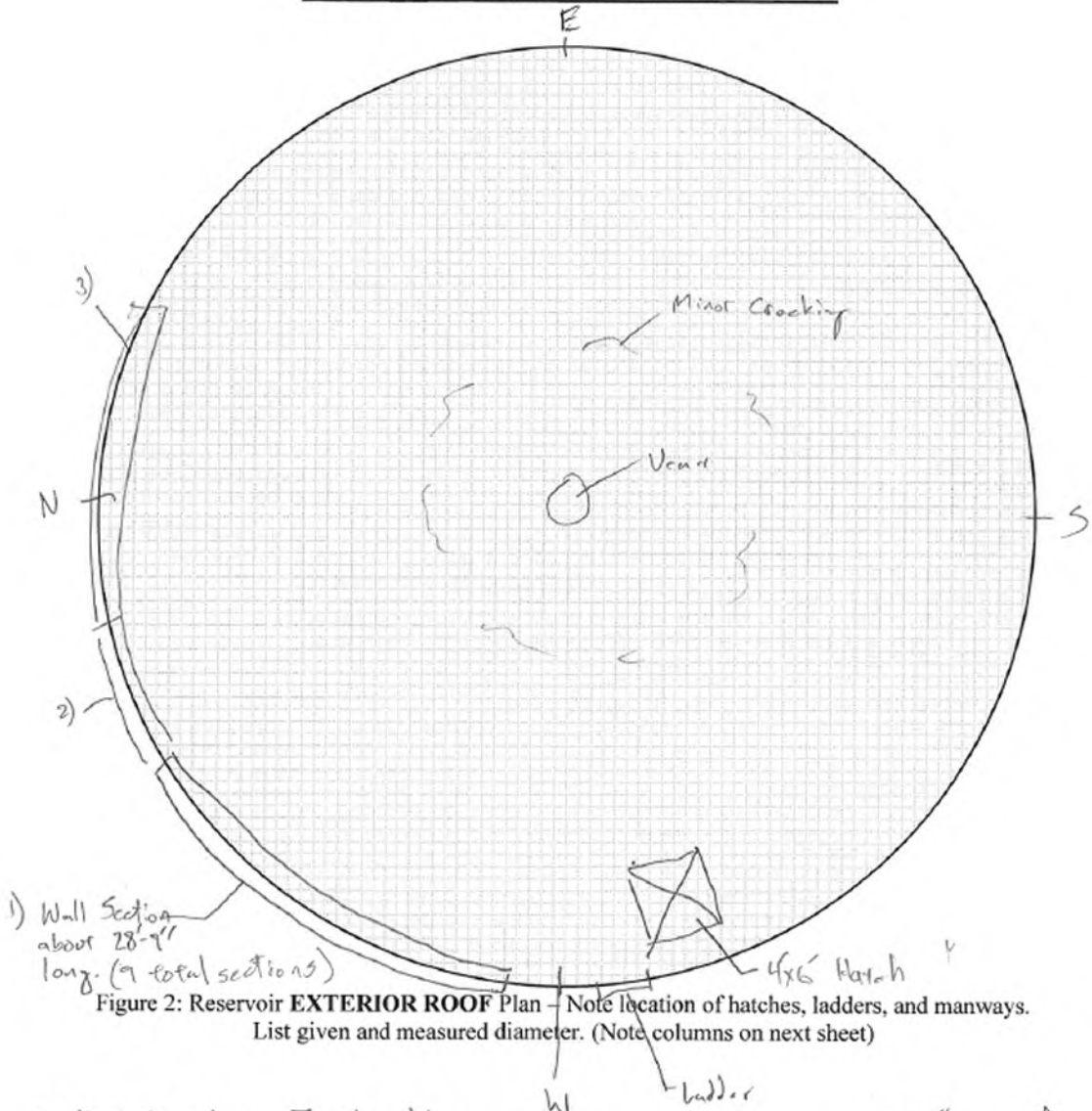


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

Wall Section has 7 threadbar pockets, pockets about 49" o.c. & 27" from ends

Section
1) 7 pocket
2) 3 pocket
3) 7 pocket, 1, 5, 7 pocket caps missing
4) 7 pocket
5) 7 pocket, 1, 3, 5 pocket caps damaged
6) 7 pocket, 1, 2 pocket caps damaged
7) 7 pocket
8) 7 pocket

PRESTRESSED RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Gen good, a couple notes (see below).
No major cracking noted. Mineralization seems to be forming on bottom surface (throughout) and depositing on floor.

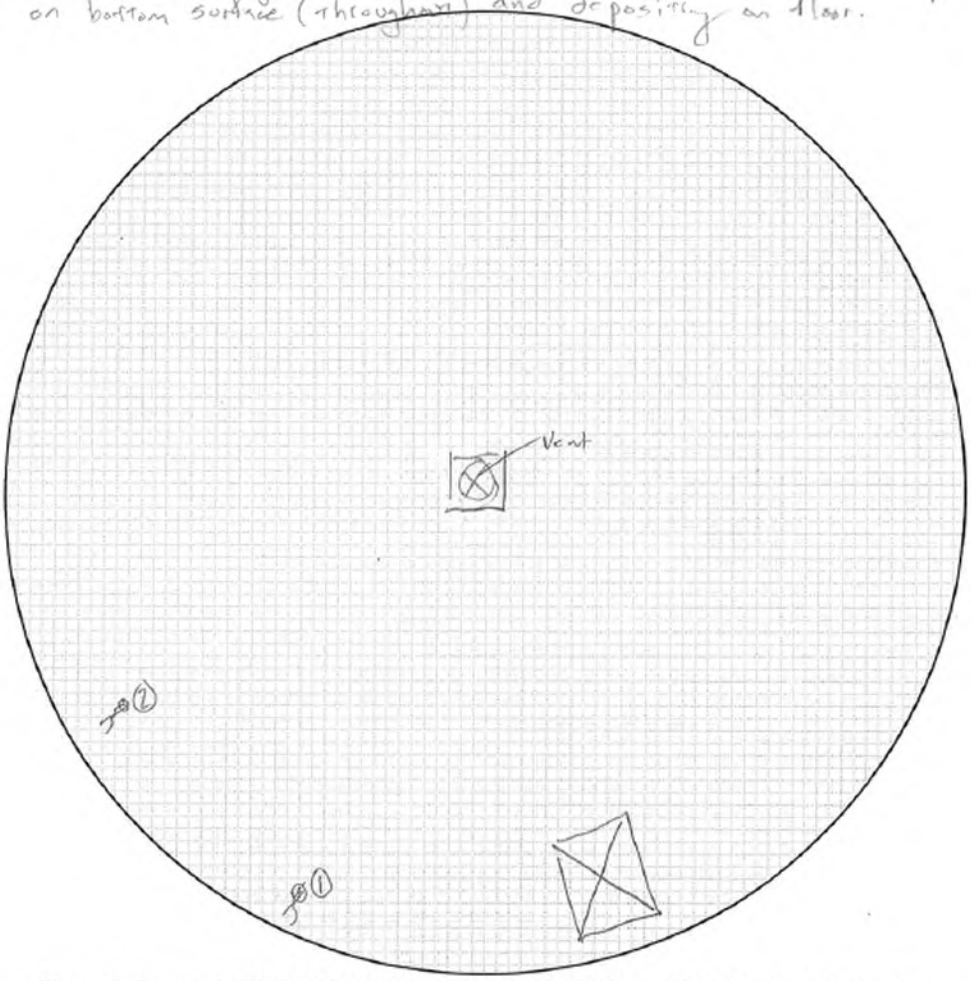


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

① Efflorescence point
② Corrosion point

PRESTRESSED RESERVOIR SITE INSPECTION

Column Diameter: N/A / None (drawings/measured) Footing Size/Thickness: 4'-8" x 18" / Not viewed (drawings/measured)

Column Spacing: N/A None (drawings/measured) Wall Curb Dimensions: N/A / None (drawings/measured)

Floor Slab Condition: Monolithic pour, not joint. Center of slab has echo when stepped on, might be localize settling near center??? Minerals on floor, very hard, looks like deposited from ceiling.

Floor Slab Joints Spacing/Condition: No joints

Column/Footing Conditions: N/A

5

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PRESTRESSED RESERVOIR SITE INSPECTION

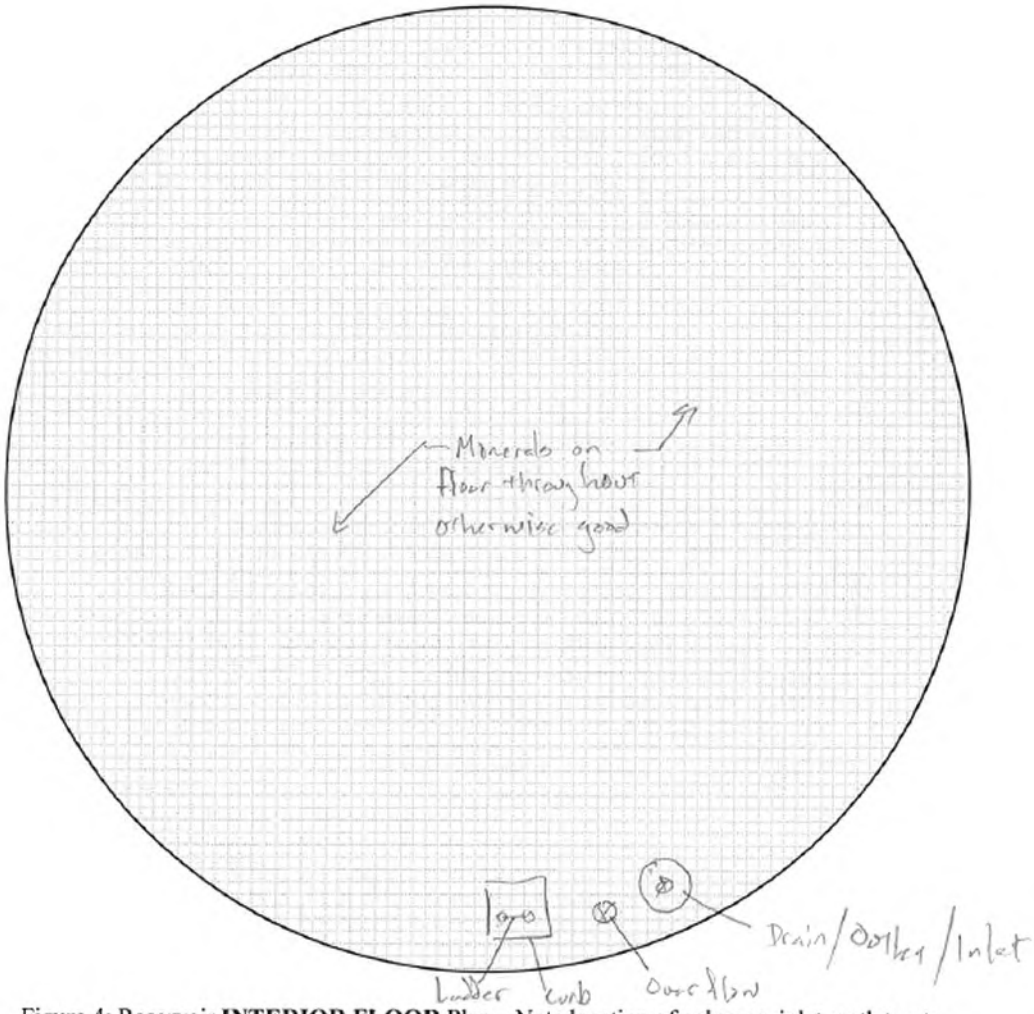

Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

PRESTRESSED RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Good. Some rock pockets at base over neoprene but otherwise good.

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): No major pitting issues but cracking noted at roof/wall interface where post-tension rods located.

# of wall sections: 8 (one shorter)

Ladder/Pipes/Overflow Conditions: Overflow & Ladder have mineral deposits

Overflow Height: 18.5'/ ≈ 18.5' (drawings/measured) Operating Height: 12'-15.5' (per City/PUD/other)

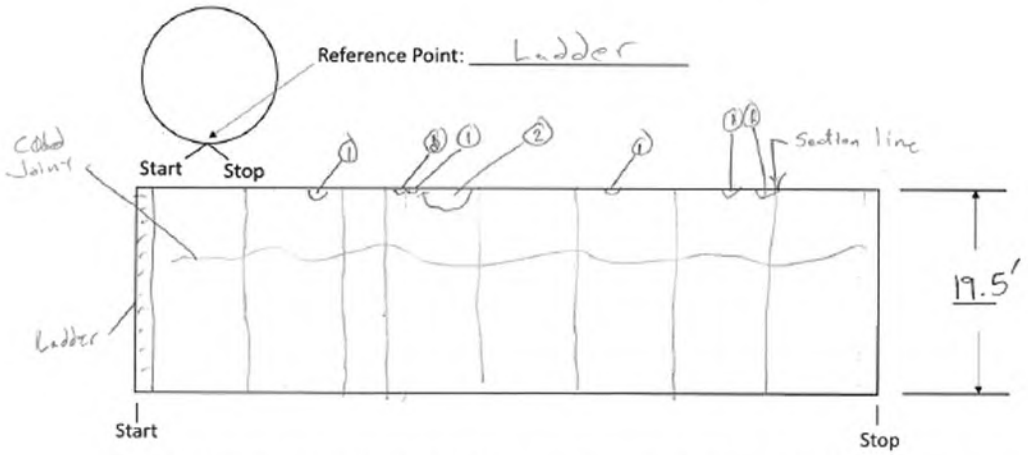

Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

① Concrete Breakout by threadbar
② Mass stress zone.

7

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END OF SECTION

Appendix D-3 Dakin II General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Dakin II Reservoir *General Info*

Field Visit Date: 4/30/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	4/30/2019
Reservoir Name and Location:	Dakin II - Balsam Ln, Parking ~540 ft E of 2918 Sylvan St; NE of Dakin I
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis
Client Staff Present:	Shayla Francis, Nick Leininger, Jenny Eakins
Year Constructed:	1990
Overflow Destination:	SW side of reservoir; Storm drain MH
Discharge Destination/Zone:	Common w/ fill; SW side of reservoir; To Dakin-Yew 519 Zone
Fill Location:	S, common w/ discharge
Reservoir Material:	Pre-Stressed Concrete

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	68	ft
Height	18.5	ft
Overflow Elevation:	519	ft AMSL
Bottom Elevation:	500.5	ft AMSL
Level of Overflow	18.5	ft
Minimum Normal Operating Level:	12	ft
Maximum Normal Operating Level:	15.5	ft
Notes:		

Dakin II Reservoir *Exterior Inspection*

Field Visit Date: 4/30/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	Enclosure	
Security Condition:	Very Good	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Fair	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	21	in
Rung Spacing:	12	in
Side Clearance:	5	in
Front Clearance:	7	in
Back Clearance:	N/A	in
Notes: Missing anchor bolt - right side, middle. Bolt size is 3/8". Five remaining anchor bolts for ladder. Vacant anchoring holes in enclosure apparatus. Ladder width is rung width.		

Exterior Fall Prevention System:	
Present at Site:	No
Type:	
Fall Protection System Condition:	
Notes:	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:	
Hatch Location:	Roof
Material:	Aluminum
Condition:	Good
Gasketed:	Yes
Intrusion Alarm:	Yes
Lock:	Yes
Frame Drain Location:	towards perimeter

Measurement Type	Measurement	Unit
Size:	4x6	ft
Curb Height:	10	in
Notes: Hatch curb height 10 inches on upper and 9 inches on lower (perimeter) side. Older type entry hatch. Gasket is not secured.		

Roof Vents and Screen:		
Material:	Aluminum	
Condition:	Very Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	1/16	in
Notes: Vent has 36" outer diameter and 24" inner diameter. Screen size approx.		

Roof:		
Condition:	Fair	
Roof Sloped:	Yes	
Downspouts:	Yes	
Ponding on Roof:	No	
Roof Finish:	N/A	
Slope of roof	Flat to 17 degrees	
Measurement Type	Measurement	Unit
Overhang Distance:	0	in
Thickness of roof slab	4	in
Notes: Parapet height is 13". Drain holes have a 2.5" diameter. The slab is thicker at edge . Trees overhang roof; significant organic debris noted. Ponding likely happening where drain holes are filled with debris. Parapet has cracks in it. Minor cracking around vent. Some parapet pocket caps missing.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Good	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Parapet Height:	14	in
Top Height:	4.75	ft
Notes: Railing only located near exterior ladder and reservoir entry hatch (not around entire reservoir perimeter). Mid rails at 2'4" and 3'6"		

Grating:	
Present at Site:	No

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Good
Notes: Minor areas of efflorescence - east side.	

Exterior Coating	
Exterior Walls:	No Coating
Exterior of Roof:	No Coating
Exterior Piping:	No Coating
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A
Notes:	

Dakin II Reservoir *Interior Inspection*

Field Visit Date: 4/30/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Fair	
Corrosion:	Yes	
Cage:	No	
Security Type:	Locked access hatch	
Security Condition:	Good	
Wall Attachment Type:	Anchored to floor/hatch sidewall	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	21	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	3	ft
Back Clearance:	N/A	in
21" rung width; 28" total w/ sides. Significant mineral deposits noted.		

Interior Fall Prevention System:	
Present at Site:	Yes
Type:	Cable/slider type
Fall Protection System Condition:	Very Poor
Notes: Slider and cable exhibit corrosion - unusable.	

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
N/A	N/A	ft
Notes: Minor cracks noted along perimeter. A point of efflorescence and a point of corrosion also present.		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: Single concrete pour (no joints). Mineral deposits noted throughout.	

Walls:	
Condition:	Fair
Painters Rings Present:	No
Notes: Mineral drips from ceiling. Concrete cracking along roof/wall interface.	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	No Coating
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Dakin II Reservoir *Miscellaneous*

Field Visit Date: 4/30/2019

Piping		
Inlet Piping:	Size (Inches OD):	12
	Condition:	Fair
	Material:	Ductile Iron
	Notes: Combined inlet/outlet/drain.	
Outlet Piping:	Size (inches OD):	12
	Condition:	Fair
	Material:	Ductile Iron
	Lip (Inches)	0
	Notes:	
Overflow Piping:	Size (inches OD):	8
	Condition:	Fair
	Air Gap:	No
	Screened:	No
	Material:	Stainless Steel
	Outlet Location:	Storm sewer
	Erosion Evident:	No
	Screen Condition:	N/A
	Overflow to roof (feet)	1
	Notes: Coupling near base appears to be uncoated and has heavy corrosion. Combines w/ Dakin I. Plans call for PVC	
Drain Piping:	Size (inches OD):	12
	Condition:	Fair
	Outlet Location:	Storm sewer
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	Silt ring
	Air Gap:	No
	Screen Condition:	N/A
	Notes: 2 in silt ring not flush with floor, but prevents larger debris from entering. Combines with Dakin I	

Piping Facilities		
Exterior Valving:	Type:	Butterfly valves
	Condition:	Good
	Secured:	No
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	Hydrant by ext. ladder
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	N/A
	Leaks:	N/A
Notes: Valves can be accessed from ground level. Fire hydrant for washdown covered in garbage bag.		

Electrical	
Cathodic Protection:	N/A
Impressed Current:	N/A
Anodes:	N/A
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	No
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	<0.1
Water Quality Sample Taps?	Yes
Notes: Reservoir sensor in meter vault near Balsam Lake Pump Station. Flex couplings on plans. Check valves not found on plans, but runs though pump station	

Appendix D-4 Dakin II Condition Assessment Score Sheet

Dakin II Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No Camera. Evidence of Vandalism
	Vegetation Separation	0	0	0	0	0	0	1	0	Overhanging with lots of accumulation problems
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	4	2	5	5	3	0	0	0	Staining on upper wall. Cracks in parapet w/ missing caps.
	Interior Walls	4	3	5	5	5	0	5	0	Cracked concrete below dome roof should be removed and patched. Staining on upper walls
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	4	4	5	5	5	0	5	0	Calcium carbonate deposits
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	5	0	0	0	0	
Roof	Exterior Roof	1	3	5	5	2	0	2	0	Roof drains very prone to blockage. Water appears to seep though. Debris. Drainage affecting WQ
	Interior Roof and Supports	5	4	5	5	0	0	0	0	
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	4	5	0	Missing bolt. 20 feet tall, so ladder fall protection not required
	Interior Ladders/Fall Protection	3	2	0	0	0	4	3	0	20 feet- fall protection not required. Exist fall protection not functional. Al Corroding
	Access Hatches	5	5	0	0	4	4	5	0	High maintenance design. Needs full roof hatch.
	Railings and Roof Fall Protection	5	5	0	0	0	3	0	0	System required for roof workers.
	Vents	4	5	0	0	3	0	5	0	Screen too big. Minor corrosion on screen. Passed design checks
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	3	4	0	0	4	0	5	0	Comb in/out/drain. Debris accumulation in pipe and a little corroded
	Outlet Piping	0	0	0	0	0	0	5	0	Comb in/out/drain. Silt stop does not appear functional.
	Drain Piping	0	0	0	0	2	0	5	0	Comb in/out/drain. No air gap
	Overflow Piping	3	3	0	0	2	0	5	0	No air gap. Lots of corrosion on lower pipe
	Washdown Piping	0	0	0	0	2	0	0	0	Unknown
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	0	0	0	0	5	0	5	5	Unknown valves
	Isolation Valving	0	0	0	0	5	0	5	5	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	3.8	3.8	5.0	5.0	3.5	3.8	4.4	5.0	

Overall Score
4.5

Appendix E Kearney

Appendix E-1 Kearney Geotechnical Report

GeoEngineers

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

Memorandum

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Kearney Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Kearney reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at Kearney site, located as shown in the Vicinity Map, Figure 1. The Kearney reservoir is a prestressed concrete reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Bellingham Drift. The maps also indicate the presence of Huntington Formation near the area, particularly on King Mountain.

The Bellingham Drift is a glaciomarine drift deposit which consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper 5 to 15 feet of this unit in upland areas is typically stiff. The stiff layer possesses relatively high shear strength and low compressibility characteristics. The stiff layer oftentimes grades to medium stiff or even soft, gray, clayey silt or clay with depth. The entire profile can stiff, likely from being partially glacially overridden, when it is a shallow profile over bedrock. The soft to medium stiff glaciomarine drift possesses relatively low shear strength and moderate to high compressibility characteristics.

The Huntington Formation is a geologically younger version of the nearby Chuckanut Formation, a mixture of sandstone, conglomerate, shale, and coal. The Huntingdon and Chuckanut Formations consist of interbedded sedimentary rocks ranging from claystone to coarse sandstone and includes conglomerate, shale and coal. The character of the rock can vary considerably over short distances.

Surface Conditions

The project site is located 500 feet to the west of James Street Road and 850 feet north of East Kellogg Road. The reservoir is located south of a small stormwater pond, north and east of a wooded area, and west of a residential area. The sight grades downward slightly to the southwest. A small gravel roadway leads to Kearney from the east.

Available Subsurface Information

Subsurface soil and groundwater conditions were evaluated by reviewing available geotechnical information. GeoEngineers performed site exploration and prepared the foundation design report titled, "Report, Geotechnical Engineering Services, Kearney Road Reservoir, Kearney Road and James Street Road, Bellingham, Washington" dated November 30, 2001. Explorations included two geotechnical borings (rock cores) B-1 and B-2 (2001) as well as seven test pits TP-1 through TP-7 (2000). The locations of the explorations are shown in the Site Plan Figure 2. The exploration logs are presented in Appendix A.

The borings were completed to depths of 18 to 28.3 feet below the existing ground surface (bgs). Bedrock was encountered at 1½ to 6 feet bgs in all the explorations. Bedrock consisted of sedimentary rocks, likely of the Huntington Formation as described above. The test pits encountered stiff clay and loose clayey sand overlying sedimentary bedrock identified as sandstone or pebble conglomerate.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on our review of previously completed explorations described above and our experience at nearby project sites.

- **Fill** – A thin layer of fill was observed at the surface of TP-2 to a depth of 1-foot bgs. The fill consisted of brown fine to coarse gravel with sand and organic material.
- **Glaciomarine Drift** – Glaciomarine drift was encountered in all explorations. The borings and test pits encountered glaciomarine drift between 0 to 5 feet bgs. The glaciomarine drift consists of gray to brown stiff clay with variable sand and gravel/brown loose clayey sand with gravel and cobbles.
- **Bedrock** – Bedrock of the Huntington Formation (similar to the Chuckanut Formation) was encountered in all of the explorations, with the exception of TP-4. The borings were completed at 18- 28.3 feet bgs. The bedrock varied from siltstone, fine to coarse grained sandstone, to pebble conglomerate.

Groundwater

Groundwater seepage was observed in B-1, B-2, and TP-3 at depths of 2, 3, and 7.6 feet bgs respectively. All other explorations did not encounter groundwater. The bedrock commonly has isolated saturated sandier zones or "pods" at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

GeoEngineers prepared a geotechnical design report and performed construction monitoring during foundation preparation for this reservoir. The design elevation would have put the reservoir foundation on bedrock and glaciomarine drift (stiff clay) which would have resulted in differential settlement. Therefore, the glaciomarine drift was excavated and replaced by a prism of roller compacted concrete (RCC) placed directly on the bedrock. The foundation condition is shown on the drawings for the project prepared by EarthTech, Sheets C5 and C6, dated February 2005.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on RCC and/or sedimentary rock which is not at risk of liquefaction.

AWWA/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on publication D110-13 of the American Water Works (AWWA) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. AWWA AND ASCE 7-10 PARAMETERS

AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	N_{ave}>50
AWWA Seismic Use Group	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	94.5
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.0
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.43
MCE_G peak ground acceleration, PGA	0.391
Seismic design value, S_{DS}	0.644
Seismic design value, S_{D1}	0.353

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

Mw 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 3 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	14	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T=0.2 sec
cm=centimeter, g=gravity

Mw 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 4 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.36	0.65	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T=0.2 sec

Shallow Foundations

We observed foundation construction for this reservoir and confirmed that it bears directly on bedrock and/or RCC extending to bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 8,000 pounds per square foot (psf) which is consistent with the value used in design. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The reservoir includes below grade walls. Our recommendations for evaluating below grade concrete walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section and the wall backfill consists of structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf
Allowable Passive Earth Pressure Coefficient (K_p)	3.5
Allowable Passive Earth Pressure[1]	350 pcf
Allowable Concrete Wall Foundation Coefficient of Sliding	0.45

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Based on review of as-builts for the site, slopes near the tank are at a 16½ foot offset and then a structural fill slope overlying the roller compacted concrete inclined at 50 percent to meet existing grades which are inclined at 10 to 20 percent. The existing reservoir is bearing directly on bedrock or RCC extending to bedrock. Therefore, it is our opinion that there is a low risk of slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

Figure 2 – Site Plan

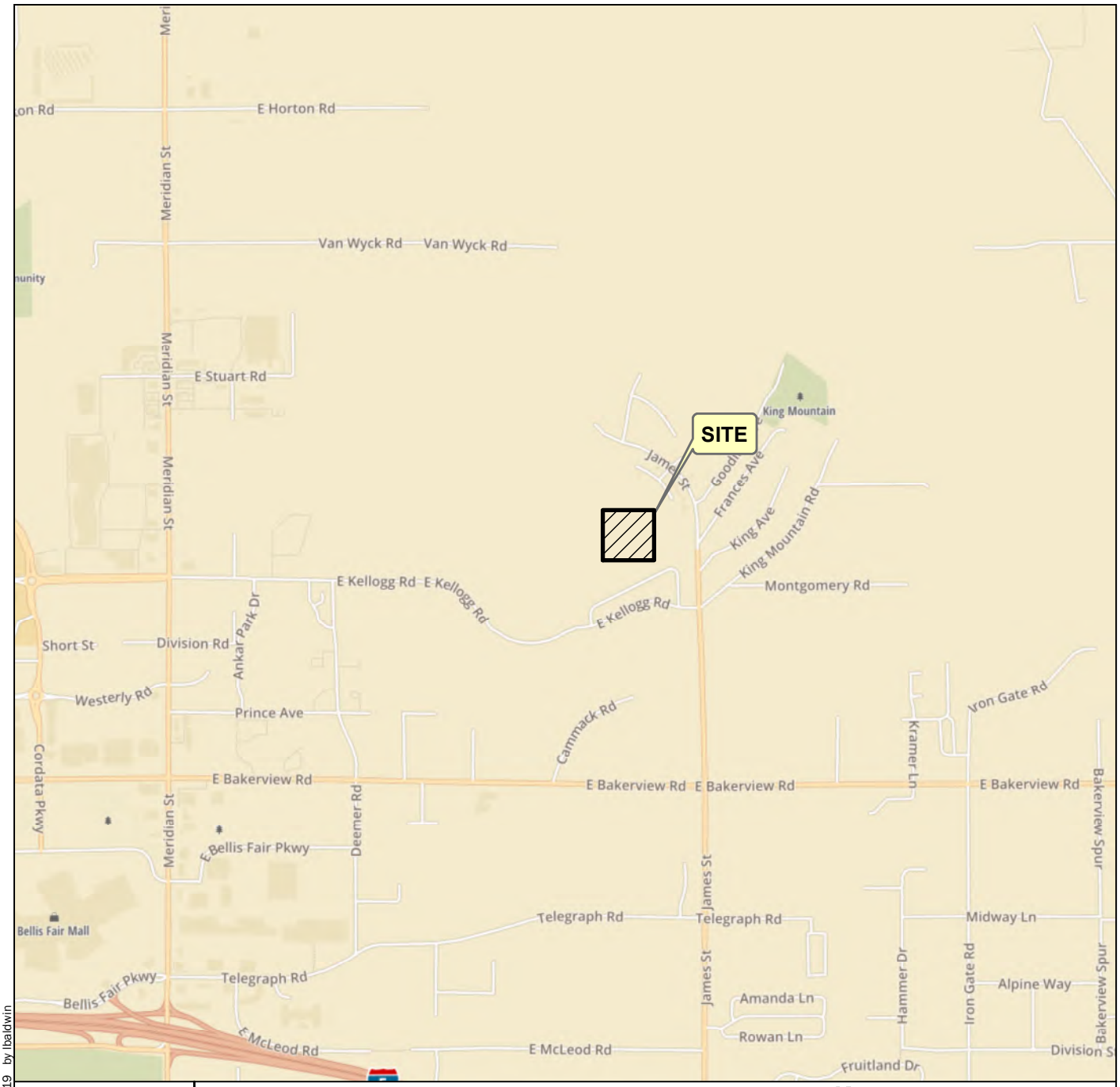
Figure 3 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

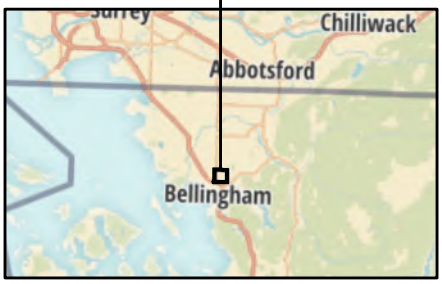
Figure 4 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault

Appendix A. Logs of Explorations

- Figures A-3 and A-4 – Logs of Rock Core B-1 and B-2
- Figures A-5 through A-11 – Logs of Test Pits TP-1 through TP-7

One copy submitted





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1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Kearney Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

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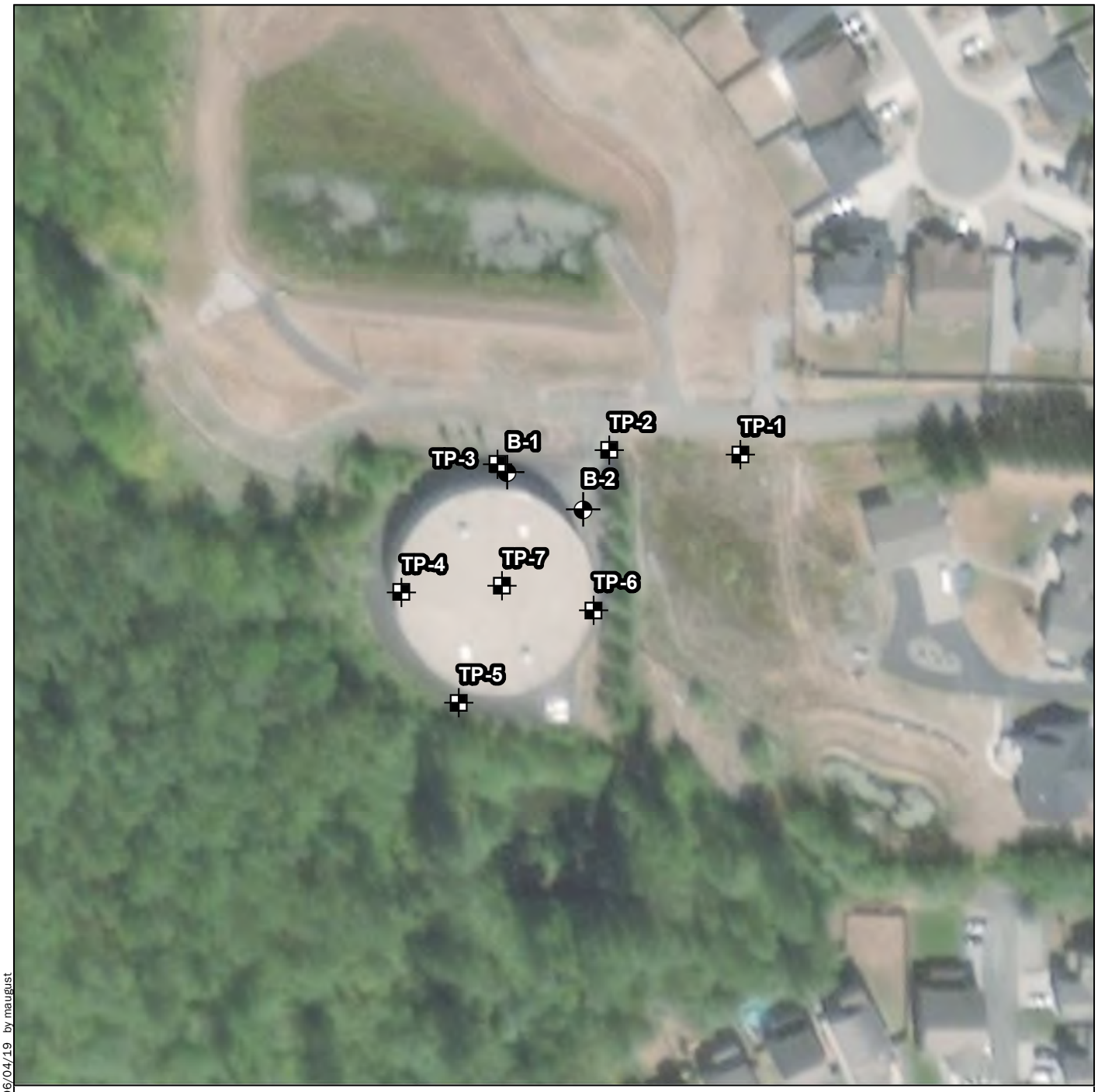

Legend

Boring by GeoEngineers (2001)

Test Pit by GeoEngineers (2001)

100 0 100

Feet

N W E S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Kearney Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

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APPENDIX A
Logs of Explorations

Date(s) Drilled	09/27/01	Logged By	RMB	Checked By	BEB
Drilling Contractor	Boretech	Drilling Method	Horizontal Coring	Inclination from Horizontal/Bearing	Vertical
Circulation Fluid	Water	Drill Bit Type	NX Core Barrel	Drilling Equipment	B-24 Drill Rig
Total Depth (ft)	28.3	Surface Elevation (ft)	+259	Ground Water Level (ft. bgs)	Not determined
Datum/ System		Easting		Northing	

Elevation feet	Depth feet	ROCK CORE: Run No.	% Recovery	RQD %	Fracture Drawing	Number	Graphic Log	MATERIAL DESCRIPTION	OTHER TESTS AND NOTES
	0							Soil overburden (see log of TP-3)	
255	5								
		1	80	0				Fine to coarse gravel	5 min/4" coring
		2	100	42				Pebble conglomerate, light gray to brown, visually fresh to stained state, fine gravel size clasts with fine to coarse sand matrix, pit quality, sub-horizontal breaks, moderate relative absorption, identified by fabric, sedimentary rock (CBBO to BBBO)	
		3	100	0					5 min/1' coring
250	10	4	60	29					15 min/2.8' coring
		5	100	55				Pebble conglomerate, light gray to gray, visually fresh state, fine gravel size clasts with fine to coarse sand matrix, pit quality, moderate relative absorption, sub-horizontal breaks, identified by fabric, sedimentary rock (BBBO)	10 min/2.1' coring
245	15	6	75	62					15 min/3.5' coring
		7	100	28		1 2		Sandstone, gray, visually fresh state, medium to coarse sand with occasional fine gravel size clasts, pit quality, slow relative absorption, sub-horizontal breaks and 45º fractures, identified by fabric, sedimentary rock (BBBO) 1. 45º open fracture with pyrite crystals 2. 45º open fracture	5 min/1.5' coring
240	20							Pebble conglomerate, light gray, visually fresh state, fine gravel size clasts with fine to coarse sand matrix, pit quality, slow relative absorption, sub-horizontal breaks, identified by fabric, sedimentary rock, (BBBO)	10 min/2.5' coring

Note: See Figure A-2 for explanation of symbols

LOG OF ROCK CORE B-1

Geo Engineers

Project: Kearney Reservoir
Project Location: Bellingham, Washington
Project Number: 0356-049-00

Figure: A-3
Sheet 1 of 2

Elevation feet	Depth feet	ROCK CORE: Run No.	% Recovery	RQD %	Fracture Drawing	Number	Graphic Log	MATERIAL DESCRIPTION	OTHER TESTS AND NOTES
	20	8	53	46					Partly decomposed state from 21.3 feet to 21.5 feet
		9	100	100					10 min/2.3' coring
235									15 min/4.5' coring (lost approximately 6" of core)
	25	10	100	100					
								Siltstone, gray, visually fresh state, fine grained, rebound quality, slow relative absorption, identified by color and fabric (BABE)	Sandstone filled void with 70º dip at 27.2 feet bgs
								Boring completed at 28.3 feet on 09/27/01 Ground water not determined during drilling due to presence of drilling fluid	Water at 2 feet bgs after completion of drilling
230									
	30								
225									
	35								
220									
	40								
215									

0356-049-00 GEI_ROCKCORE_2.1.0 W:\BELLIN~1\PROJECTS\0\0356049\00\FINALS\0356049B.GPJ GEIV2_2.GDT 11/5/01

LOG OF ROCK CORE B-1 (continued)

Geo Engineers

Project: Kearney Reservoir
Project Location: Bellingham, Washington
Project Number: 0356-049-00

Figure: A-3
Sheet 2 of 2

Date(s) Drilled	09/27/01	Logged By	RMB	Checked By	BEB
Drilling Contractor	Boretech	Drilling Method	Horizontal Coring	Inclination from Horizontal/Bearing	Vertical
Circulation Fluid	Water	Drill Bit Type	NX Core Barrel	Drilling Equipment	B-24 Drill Rig
Total Depth (ft)	18	Surface Elevation (ft)	+259	Ground Water Level (ft. bgs)	Not determined
Datum/ System					

Elevation feet	Depth feet	ROCK CORE: Run No.	% Recovery	RQD %	Fracture Drawing	Number	Graphic Log	MATERIAL DESCRIPTION	OTHER TESTS AND NOTES
	0							Soil overburden (see log of TP-3)	
		1	24	0				Sandstone, brown, stained state, medium to coarse grained with occasional fine gravel sized clasts, pit to dent quality, sub-horizontal breaks, moderate relative absorption, identified by fabric, sedimentary rock (CCBO to CBBO)	5 min/1' coring
255		2	79	0					5 min/2' coring
	5	3	100	15					10 min/3.3' coring
250		4	70	42				Pebble conglomerate, gray, visually fresh state, fine gravel size clasts with medium to coarse sand matrix, pit quality, moderate relative absorption, sub-horizontal breaks, identifed by fabric, sedimentary rock (BBBO)	15 min/3' coring
	10	5	100	21				Partly decomposed from 12.4 feet to 12.9 feet	10 min/1.9' coring
245		6	90	48				Brown, stained stated from 14 feet to 14.4 feet	
	15								10 min/9.5' coring
		7	13	0				Pebble conglomerate, brown, partly decomposed state, gravel sized clasts with fine to coarse sand matrix, pit quality, moderate relative absorption, identified by fabric, sedimentary rock (DCDO)	Losing water, bit stuck 10 min/2.2' coring
240	20							Boring completed at 18 feet on 09/27/01 Ground water level not determined due to presence of drilling fluid	Water at 3 feet bgs after completion of drilling

Note: See Figure A-2 for explanation of symbols

LOG OF ROCK CORE B-2

Geo Engineers

Project: Kearney Reservoir
Project Location: Bellingham, Washington
Project Number: 0356-049-00

Figure: A-4
Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +265

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
265	0						4 inch topsoil			
		☒	1			GP	Brown fine to coarse gravel with sand (medium dense, moist) (fill)	8		
						CL	Brown and gray clay with sand, gravel and cobbles (stiff, moist)			
		☒	2					20		
		☒	3					20		
260	5						Test pit completed at 4.5 feet below ground surface on 11/17/00 No ground water seepage observed No caving observed Disturbed soil samples obtained at 0.5, 2 and 4 feet			
255	10									
250	15									
245	20									
240	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-1

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-5

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +262

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0						3 inches topsoil			
		☒	1			GP	Dark brown fine to coarse gravel with sand and organic matter (medium dense, moist) (fill)	7		
						CL	Gray clay with sand and occasional cobbles (stiff, moist)			
260										
		☒	2					22		
		☒	3					19		
							Grades to very stiff			
							Test pit completed at 4 feet below ground surface on 11/17/00			
	5						No ground water seepage observed			
							No caving observed			
							Disturbed soil samples obtained at 0.5, 2.5 and 4 feet			
255										
	10									
250										
	15									
245										
	20									
240										
	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-2

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-6

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +260

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
260	0	☒	1			SC	Brown clayey fine to coarse sand with gravel and organic matter (loose, moist)	32		
						CL	Reddish brown fine to coarse sandy clay with gravel and occasional cobbles (stiff, moist)			
		☒	2					24		
		☒	3					27		
255	5	☒	4			RX	Pebble conglomerate, gray to brown	6		
		☒	5							
							Test pit completed at 8 feet below ground surface on 11/17/00 due to practical refusal in bedrock Rapid ground water seepage observed at 7.6 feet during drilling No caving observed			
250	10						Disturbed soil samples obtained at 0, 2, 4, 5 and 7 feet			
245	15									
240	20									
235	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-3

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-7

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +248

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0	☒	1			SC	Dark brown clayey sand with gravel, organic matter and occasional cobbles (loose, moist)	29		
						RX	Sandstone, light brown, fine grained			
		☒	2					17		
245										
		☒	3							
	5									
		☒	4							
240							Test pit completed at 8 feet below ground surface on 11/17/00 due to practical refusal in bedrock No ground water seepage observed No caving observed			
	10						Disturbed soil samples obtained at 0, 2, 4 and 7.5 feet			
235										
	15									
230										
	20									
225										
	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-4

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-8

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +243

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0	☒	1			SC	Dark brown clayey fine to coarse sand with organic matter, gravel, and occasional cobbles (loose, moist)	34		
						CL	Tan clay with fine sand and occasional gravel (stiff, moist)			
		☒	2					20		
240		☒	3			RX	Sandstone, brown, fine grained			
	5									
							Test pit completed at 6 feet below ground surface on 11/17/00 due to practical refusal in bedrock No ground water seepage observed No caving observed Disturbed soil samples obtained at 0, 1.5 and 3 feet			
235										
	10									
230										
	15									
225										
	20									
220										
	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-5

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-9

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +254

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0	☒	1			SC	Dark brown clayey fine to coarse sand with gravel, cobbles and organic matter (loose, moist)	38		
		☒	2				Grades to tan with gravel, cobbles and occasional boulders (medium dense, moist)	25		
250		☒	3			RX	Pebble conglomerate, brown	24		
	5	☒	4							
							Test pit completed at 6 feet below ground surface on 11/17/00 due to practical refusal in bedrock No ground water seepage observed No caving observed Disturbed soil samples obtained at 0, 2, 4 and 6 feet			
245										
	10									
240										
	15									
235										
	20									
230										
	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-6

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-10

Sheet 1 of 1

Date Excavated: 11/17/00

Logged by: AKM

Equipment: JD-590 Tracked Excavator

Surface Elevation (ft): +252

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0	☒	1			SC	Dark brown clayey fine to coarse sand with gravel, cobbles and organic matter (loose, moist)	31		
							Grades to tan (medium dense, moist)			
250		☒	2					22		
						RX	Pebble conglomerate, brown			
		☒	3							
	5	☒	4							
							Test pit completed at 5.5 feet below ground surface on 11/17/00 due to practical refusal in bedrock No ground water seepage observed No caving observed Disturbed soil samples obtained at 0, 1.5, 3 and 5 feet			
245										
	10									
240										
	15									
235										
	20									
230										
	25									

Note: See Figure A-2 for explanation of symbols

LOG OF TEST PIT TP-7

Geo Engineers

Project: Kearny Road Reservoir

Project Location: Bellingham, Wa

Project Number: 0356-049-00

Figure: A-11

Sheet 1 of 1

Appendix E-2 Kearney Structural Report

CITY OF BELLINGHAM

CH 7: KEARNEY RESERVOIR

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to identify potential problem areas that may be candidates for repair or upgrade at the Kearney, 2.49 Million Gallon (MG) prestressed concrete reservoir. The reservoir is located near 465 Kearney St, Bellingham, WA (Lat. 48.796, Long. -122.467), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on March 14th, 2019 by Peterson Structural Engineers (PSE), and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 Kearney Prestress Reservoir – 2.49 MG

2.1 Description & Background

The reservoir was design was prepared by Earth Tech and the original construction drawings provided are dated February 2005. The reservoir is a partially buried, 130-foot inside diameter by 27-foot high, strand-wrapped, pre-stressed concrete water reservoir with interior columns. The roof is 9-inches thick and is supported by (32) 18-inch diameter columns. No joints were observed in the roof slab as it is a single, monolithic pour.

Per the original drawings the reservoir contains vertical pre-stressing bars in the core wall. The roof slab bears on the tank shell wall with shear cans at each vertical tendon. There are 99 total vertical tendons—each 1-3/8-inch in diameter, per the drawings. The wall base connection utilizes 198 seismic cable sets each with (4) ½-inch diameter, 7-wire galvanized strands. Both the roof and the floor connections are detailed with a continuous elastomeric bearing pad.

The original drawings specify a roof live load of 20-psf, a roof snow load of 25-psf, and a collateral roof dead load of 10-psf for design. This meets the current code requirements of 25 psf for snow load design. For the evaluation contained herein the controlling roof load used is 25 psf.

The reservoir does not have an overflow pipe installed and the overflow discharge pipe is capped. Per the drawings, the anticipated overflow height was listed as 26-feet, 1-foot below the roof slab. Per the City, the reservoir's operating level is set by the remaining reservoirs in the service zone, typically averaging between 14.5 and 22-feet, for a maximum storage volume of about 2.1 MG.

2.1.1 Description of Additional Site Structures and Features

The site includes an adjacent, below-grade, pre-cast piping vault. The vault is located southeast of the reservoir and is 14-feet long by 13-feet wide and 7.5-feet deep. The top of the vault has a 3-foot square access hatch and two 4 by 8-foot hatches, for use in accessing the associated piping. The vault is reinforced concrete and the roof slab was measured to be 13.5-inches thick.

2.2 Visual Condition Assessment and Associated Recommendations

We performed a site visit to observe the as-built current condition of the reservoir's interior, exterior, and site conditions. The reservoir was drained for our inspection. The site visit was performed March 14th, 2019.

<u>Concrete Roof:</u> The exterior condition of the reservoir's roof showed no visible areas of ponding or any issues that could be associated with failure. In general, the roof and roof-top structure were noted to be in good condition with minimal, non-structural, pattern cracking being the extent of issues noted. No joints were observed in the roof slab and it appeared to be a single, monolithic, pour. The edge of the roof appeared to be in good condition. The roof overhang was measured from the ladder and found to be 8.5-inches. A drip edge was present located mid-way from the exterior edge.

The roof has two access hatches which have 6 by 8-foot openings and two vents with 3'-10" square openings. The hatches, vent covers, and associated concrete curbs all appeared to be in good condition

with no apparent structural deficiencies noted. While the vents appeared to meet current requirements for water quality the access hatches, which have a channel inset into the curb, are considered high-maintenance designs per the Washington State Department of Health. The gutter-drains for these hatches should be screened to further protect them.

The interior condition of the reservoir roof was found to be in generally good visual condition and the drop panels were observed to be in good condition. From the inside of the reservoir it was noted that there were areas of incidental cracking around the exterior edge of the roof. This cracking was more pronounced where column drop panels were located within a few feet of the wall. Of the (32) drop panels, a total of eight drop panels, near the wall where noted to have some cracking in-line with their edges. The cracking at these drop panels was observed to radiate out towards the wall from the corner of the drop panels. The roof itself is supported with 18-inch diameter columns on 7-foot square footings. The footings are oriented at 45 degrees to the drop panels. No visible issues were noted with either the columns or footings during our inspection.

Prestress Walls: The exterior walls of the reservoir are covered in shotcrete to protect the strand wrapping around the core wall. The reservoir is partially buried, up to 8-feet on the north side and down to ½-foot on the south. The surrounding grade is a paved access ring-road. Site drainage is to a stormwater retention pond located to the south of the reservoir. Per the drawings, the north side of the reservoir was to be excavated to an "assumed surface of rock" while the south side was backfilled to create a level grade. The change in stiffness between the excavated and filled sides can sometimes result in a crack down the center of the reservoir, where the change in foundation stiffness occurs; no such cracking was observed during our site visit. The reservoir site is located in a lightly forested/residential area. There were no visual indications of any major slope movement or settlement noted.

Efflorescence was noted to be concentrated on the south exterior face of the reservoir. The observed areas of efflorescence were located approximately 8-feet above grade, and approximately in-line with the rear-backfilled area. The localization of the efflorescence on the southern side could be a result of the differential backfill and "water-in-transit" which comes to the surface and evaporates on the south (sunward) face of the reservoir. Efflorescence is typically more of a cosmetic issue rather than a durability concern. Evaluation of the existing efflorescence did not note any additional associated issues such as delamination (see below), cracking, or other failure of the shotcrete layer.

In additional to our observation, PSE performed "sounding" of the walls around the perimeter of the reservoir. Sounding is a process of tapping the reservoir's exterior surface to listen to the report of the hammer strike. A sharp "ping" sound is generally indicative of a competent shotcrete layer while a dull or hollow "thud" sound can be indicative of delamination within the shotcrete layer. Two possible delamination areas were identified on the north side of the reservoir. These issues had occurred along lines where roof run-off was occurring. A driving force behind delamination is typically water infiltration between the core wall and shotcrete and subsequent freeze-thaw which not only causes the delamination but also makes these areas more suspectable to future water infiltration and delamination. It should be noted that a significant portion of the reservoir walls were either below grade or out of reach and could

not be sounded with a hammer. Therefore other locations of delamination could be present which were not identified during PSE's evaluation.

The interior walls of the reservoir were found to be in good condition with very minimal instances of bug-holes or surface defects. Overall the concrete was found to be competent and in good condition. Nine wall sections were counted which match the number of sections listed in the provided as-built drawings. Formwork holes and epoxy injection ports looked to be adequately capped and sealed and in good condition.

Slab Floor: The interior slab was visibly found to be in good condition with no observed issues of cracking or visual signs of failure. At the time of our evaluation the floor slab had not been cleaned and so the overall floor was coated in thin sediment layer. Section of this layer were manually swept aside to better view the slab. However, much of the floor remained obscured and it is likely that minor surface and/or shrinkage cracking, typical of a membrane floor slab, exist but were not directly observed. No major issues were identified during the evaluation. As the floor slab is a single, monolithic pour, no joints are present in the slab.

Appurtenances: The reservoir piping consists of a 20-inch inlet located in the northwest quadrant of the reservoir and a 30-inch outlet located in the southeast quadrant of the reservoir. Overall these appeared in good visual condition, but some corrosion carbuncles were noted to be beginning to form. While this reservoir does have a flange attachment for a 20-inch diameter overflow pipe in its southwest quadrant, no overflow pipe is currently installed. The overflow was observed to be capped and per the City, the reservoir's operating level is based on the overall zone's operating elevation. For the reservoir drain, rather than a single drainage point, the system is comprised of a series of drainage channels located throughout the reservoir's floor. These drains are supported by a concrete encasement and are linked to a single 6-inch pipe located to the south of the reservoir.

2.2.1 Visual Condition of Additional Site Structures and Features

The cast-in pipe vault was noted to be in good condition with no visible signs of cracking, structural failure, or leakage observed during our inspection. Components and systems appeared to be in good working order.

2.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the current adopted code and standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13, "Wire and Strand Wound Circular Prestressed Concrete Water Tanks" was utilized. Evaluation was based on the provided as-built original construction documents and site visit observations.

2.3.1 Hydrostatic and Gravity Analysis

Roof Slab: The roof slab thickness of 9-inches is above the code recommended minimum thickness of 6-3/4-inches for the spans provided. The additional thickness helps to resist cracking resulting from

potential long-term creep problems. Based on the plans, reinforcing in the roof slab was determined to be sufficient for both flexure and crack control for current code requirements.

During the observation, some cracking was noted at the edge of the column drop panels in areas where column drop panels were in close proximity to the walls. These issues are likely a result of the curvature of the reservoir's wall and stress concentrations which occur at the change in stiffness between the drop panel and the roof slab. Overall, the cracking observed appears to be relatively minor and should be monitored as part of routine maintenance to check for any further development or associated leaking.

As noted above, the reservoir does not have an overflow pipe installed. Currently, the water level in the reservoir is run to a level consistent with the zone's water level. This level is below the base of the roof, in the 22-foot range. However, if the water level exceeded the wall height, then water pressure would be exerted on the roof. The roof system is design for downward gravity loads, not upward pressure loads. This water pressure load could cause damage to and/or failure of the roof system. Depending on the City's overall system redundancy and the maximum possible operating level of the zone, it is recommended that an overflow pipe be installed as a redundancy measure and to meet the requirements of AWWA D110-13 Section 3.11.2.1. Should any system issues occur which cause the Kearney reservoir be filled faster than it can drain, an overflow would help to prevent water pressure loads on the roof and the ensuing damage or failure due to said overfilling.

Vertical Wall Reinforcement: The reservoir as-built drawings show vertical pre-stressing within the walls spaced at 4'-1-7/8" on center. While PSE could not observe the vertical pre-stressing directly, we did observe the location of the epoxy injection ports which matched the pre-stressing spacing indicated on the plans. Per analysis, the size and spacing of the pre-stressing was determined to be adequate. Based on the site visit, the injection ports appear to be in good condition and the walls were observed to be competent and generally crack free.

Columns: Per the original as-built drawings, the columns were found to be suitable for anticipated design loads when analyzed under current code requirements. During the site visit no visible defects or signs that would indicate the presence of any deficiencies, were observed in the columns. The columns meet current code requirements for static load resistance; see Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings were evaluated based on information obtained from the provided as-built drawings. The design was found to be adequate for current codes. Per the Geotechnical report the allowable bearing capacity for the site was given as 8,000-psf. Based on PSE's design checks, this is bearing capacity is adequate for a maximum design soil bearing pressure of approximately 3,900 psf, which occurs at an overflow operating level of up to 26-feet.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Joints: Per the drawings there are 99 sets of seismic cables (two sets per vertical pre-stressing bar). Based upon an analysis it appears that this meets current code and provides the necessary lateral capacity for an operating level of up to 26-feet. Should the City require, it appears that the reservoir can be operated at higher levels than it is currently, all the way up to the 26-foot design overflow level.

Strand Wrap: The circumferential prestressing requirements and wrapping diagram for the reservoir are detailed on the prestressing load distribution diagram on Drawing S7 of the original drawings dated February 2005. Based on PSE's evaluation under current code requirements, the circumferential prestressing is at capacity for the anticipated static and seismic loads resulting from an operating height of 26-feet. At the current maximum operating height of 22-feet, the existing wrapping is more than adequate for both hydrodynamic hoop stresses as well as temperature and shrinkage requirements.

Columns: The roof is supported by continuous bearing pads on top of the wall and is restrained by shear cans which limit the potential seismic deflection of the roof and columns to a maximum of 0.125-inches. The existing columns were determined to be adequate to resist the seismic and eccentric loads induced by this 0.125-inch deflection. Therefore, the columns are adequate to resist both seismic and static loads at the maximum operating level of 26-feet based upon current code requirements.

Freeboard/Slosh: The current freeboard at a maximum operating level of 22-feet is 5-feet. This is more than adequate to handle the anticipated 3-foot slosh wave. Should the reservoir be operated at 26-feet, which would reduce the available freeboard to 1-foot, PSE determined the existing roof has the requisite structural capacity to resist the slosh impact wave resulting from operating at the design overflow level when considering the slosh wave height as an equivalent hydrostatic force based upon the calculated slosh height.

The roof hatches are located in the exterior slab spans and may blow out or be damaged in a seismic event if a slosh wave hits the underside of the hatches when operated at a level without adequate freeboard. This would likely be a localized failure or isolated damage to the hatch and isn't expected to impact the overall structural performance of the reservoir.

2.4 Summary

The Kearney Reservoir was constructed in 2005 and appears to be in substantial conformance to current design codes and standards. The reservoir was designed to operate at a 26-foot operating level and if required, can still do so while generally meeting the current code requirements as noted above. At the lower operating level it is currently run at, potential seismic demands are further decreased.

Issues noted with the reservoir include the exterior shotcrete and the lack of an overflow pipe. To the south of the reservoir, efflorescence was noted on the surface of the shotcrete but did not appear to be accompanied by any actual failure or cracking in the shotcrete layout at the time of our inspection. On the north side of the reservoir, potential shotcrete delamination issues were identified through the process of sounding. These delaminated areas appear to be limited to two locations where roof runoff is occurring. The runoff may be allowing water to infiltrate between the concrete and shotcrete layers and thus causing delamination when freeze-thaw effects occur. For a reservoir of this age the strand should be galvanized which will help against short term corrosion resulting from delamination. However, these delamination sites should be monitored and their eventually repaired.

In the current operating condition, the reservoir does not get operated at a level that would utilize an overflow pipe. However, the overflow is not only for when operating near the maximum operating level

but also for issues that might overfill a reservoir (blocked drain, etc.). The overflow pipe should be installed for emergency and atypical operating conditions

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

As the reservoir was observed to be in good condition, which is also supported by PSE's analysis, recommendations are minimal. While efflorescence is primarily cosmetic it does consist of soluble salt-forming components. If these are allowed to build up for extended periods of time, these salts can result in a form of salt attack which can deteriorate and weaken the shotcrete. As efflorescence can be harder to remove over time, it is recommended that regular cleaning of the reservoir take place to removed existing efflorescence to both ensure the long-term adequacy of the shotcrete and to help remove any issues which might obscure any underlying cracking which would otherwise be identified for repair.

On the north side of the reservoir two areas have been identified as having potential delamination issues. PSE recommends that the zones identified continue to be monitored. Based upon the age of construction and the size of potential delamination observed, it is generally our experience that it is unlikely that the strands are damaged. Through the course of monitoring, if the delamination issues worsens, a more extensive evaluation can be undertaken to determine the full extent of the issue and to help define a course of repair.

PSE recommends the installation of an overflow pipe to meet the requirements of AWWA D110-13 Section 3.11.2.1. While the zone might generally be run at a level below the base of the roof, each individual reservoir should be able to vent water as necessary. Overfilling a reservoir can result in damage to the roof since they are not designed to withstand hydrostatic, uplift loads. As the overflow pipe is already in place, PSE would recommend attaching the remaining portions of the overflow assemble in order to alleviate any possibility of damage that might occur to the roof, due to overfilling of the reservoir.

Finally, the roof access hatch gutter drainage points should be screened to protect them. This will bring them into compliance with the Washington Department of Health requirements for the sanitary protection or reservoirs.

2.6 Scans of Select Construction Documents

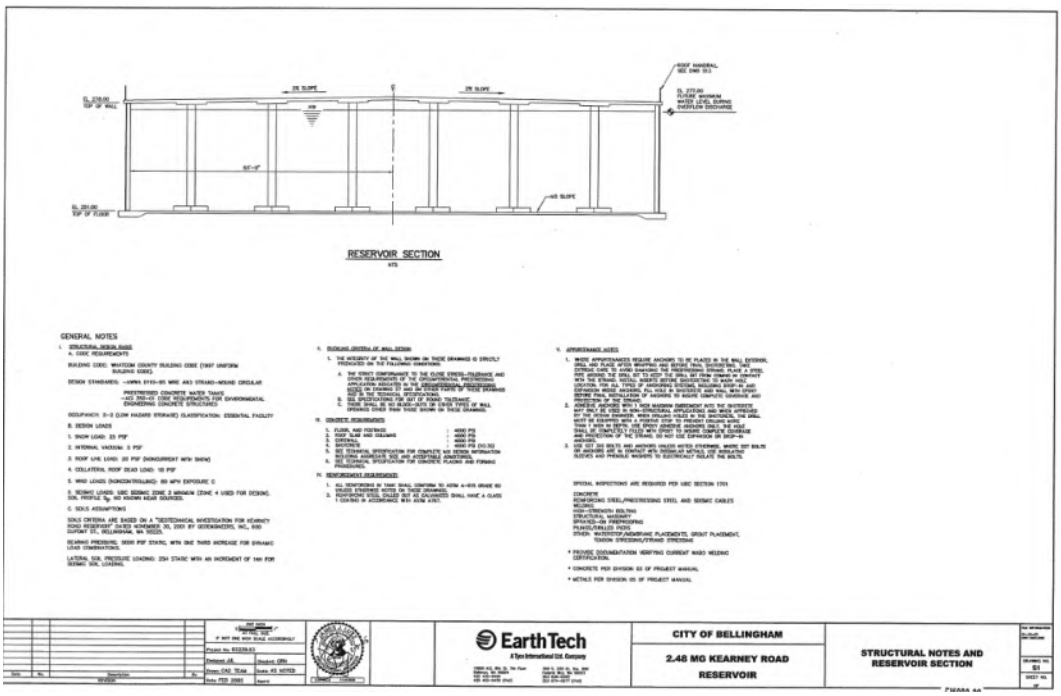


Figure 2-1: Kearney Structural Notes and Reservoir Section

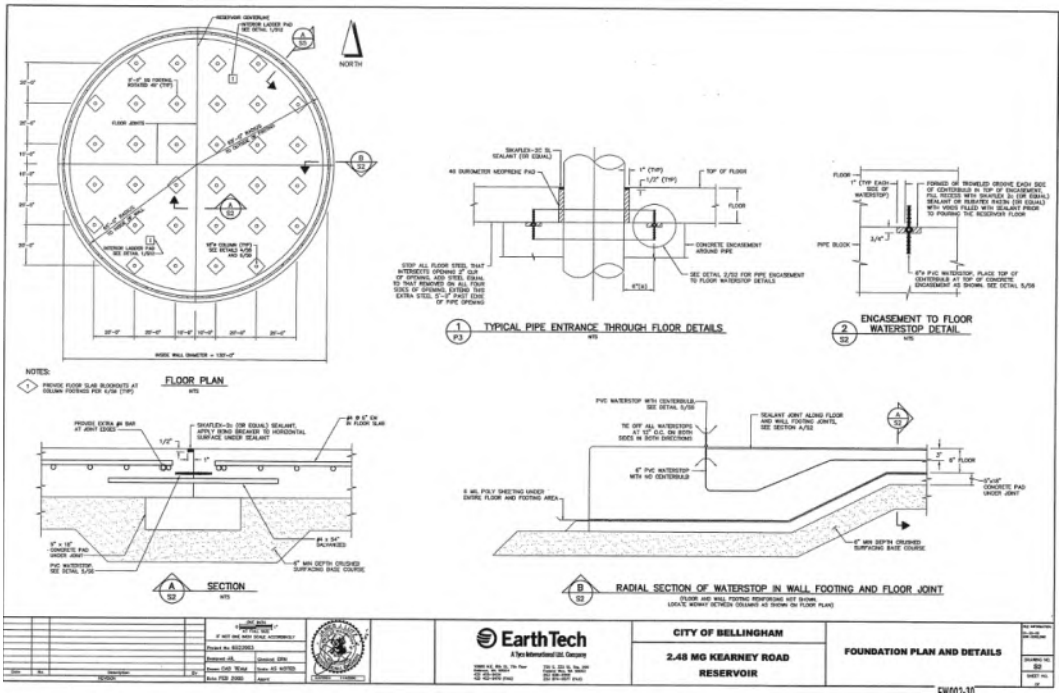


Figure 2-2: Kearney Foundation Plan and Details

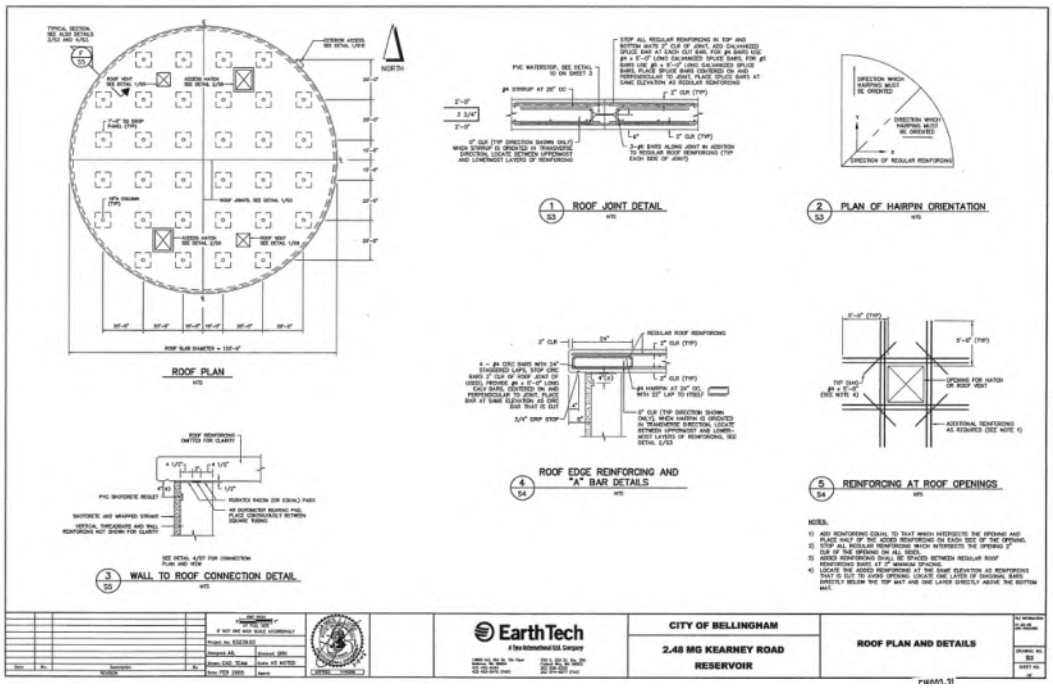


Figure 2-3: Kearney Roof Plan and Details

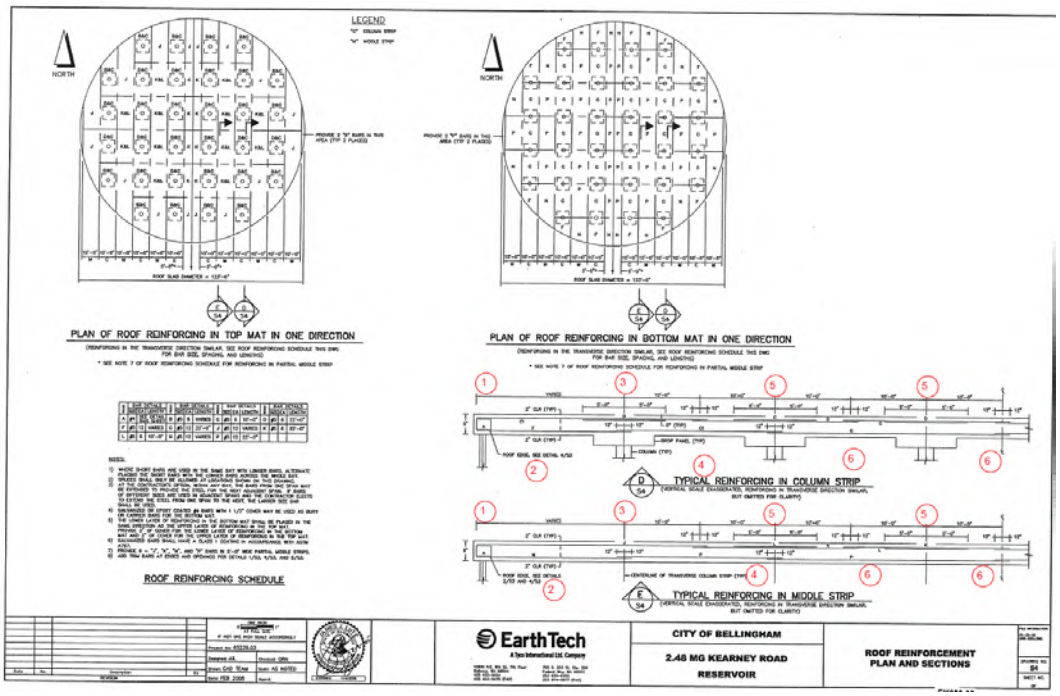


Figure 2-4: Kearney Roof Reinforcment Plan and Sections

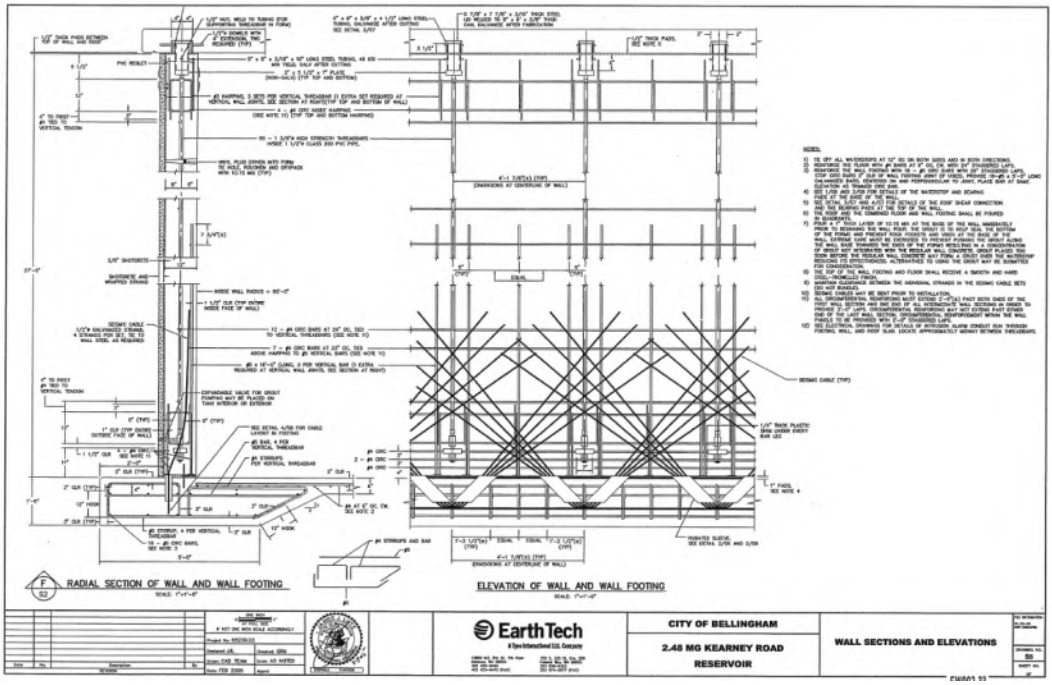


Figure 2-5: Roof Sections and Elevations

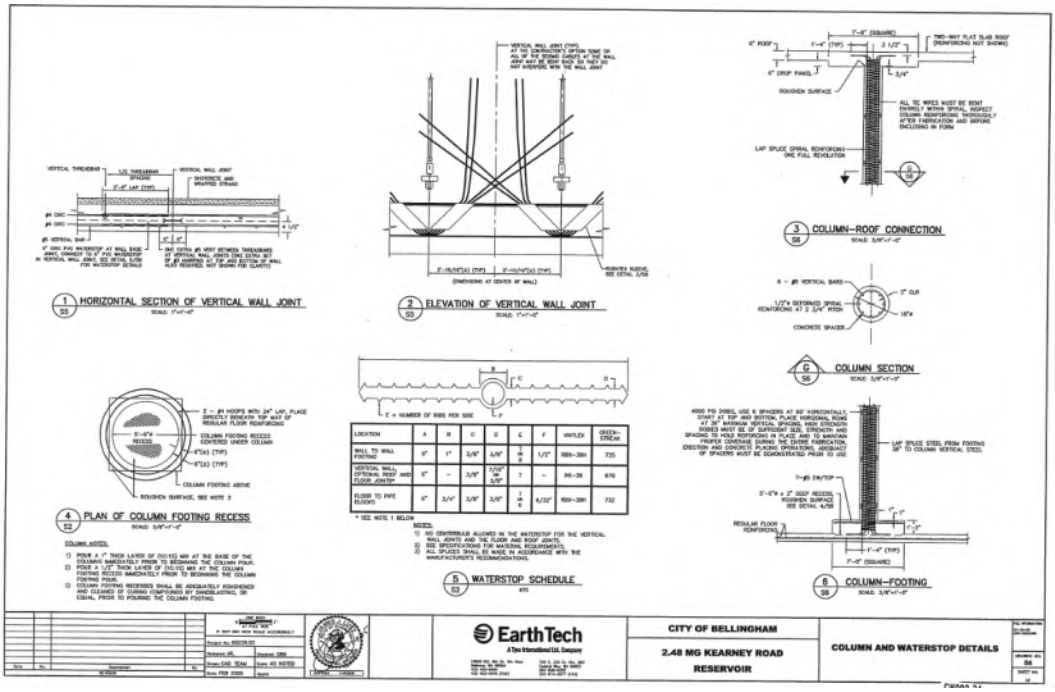


Figure 2-6: Column and Waterstop Details

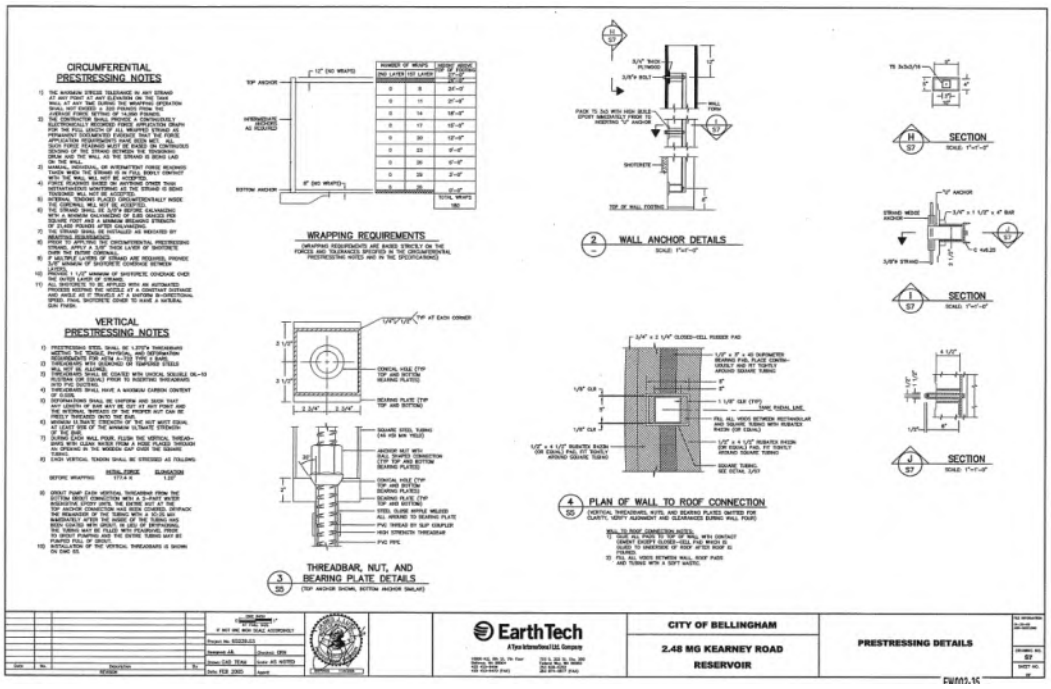


Figure 2-7: Prestressing Details

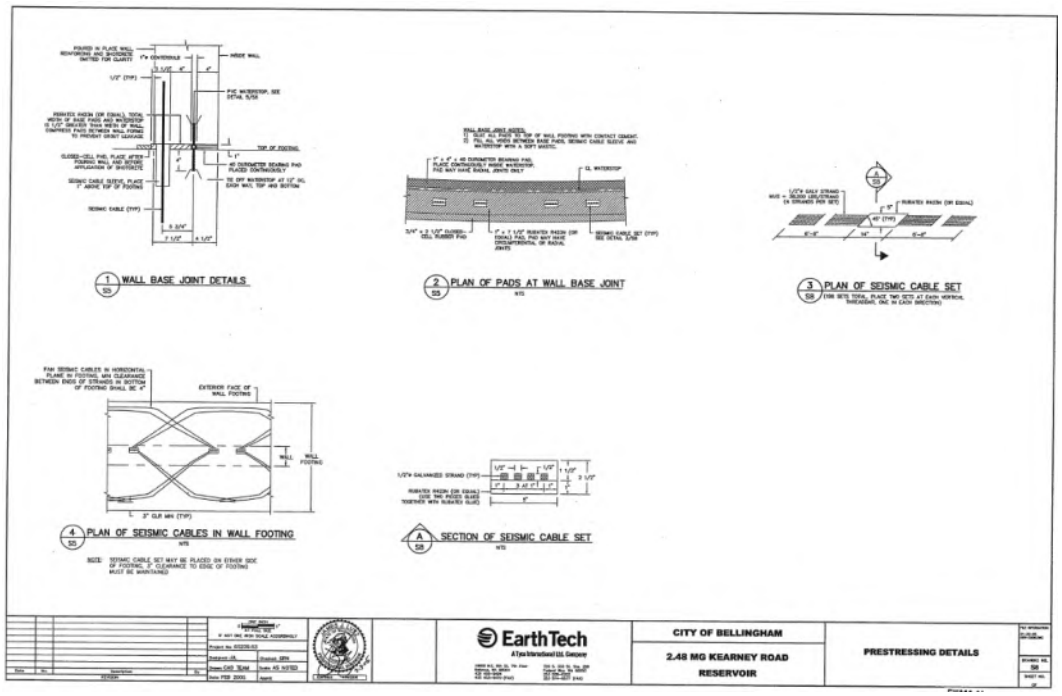


Figure 2-8: Prestressing Details

2.7 Observations Pictures

Figure 2-9: Reservoir North Elevation

Figure 2-10: Reservoir South Elevation & Exterior Ladder

Figure 2-11: Pipe Vault, Located southeast of reservoir

Figure 2-12: Typical Efflorescing in Shotcrete

Figure 2-13: Location of Potential Delamination (points 1 (near) and 2 (far) located at water stained areas)

Figure 2-14: Hatch

Figure 2-15:Interior Column Layout and Wall

Figure 2-16:Interior Drop Panels and Roof

Figure 2-17: Typical Roof Cracking Along Sides of Drop Panel Near Roof Edge

Figure 2-18: Capped Overflow

2.8 Field Notes

PRESTRESSED RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Eval.

PROJECT NUMBER: A1802-0019

PRESTRESSED RESERVOIR SITE INSPECTION

Reservoir Name: Kearney Res. – 465 Kearney St

Site Visit Date: 3/14/19 Reservoir Type: 2.48 MG

Temperature and weather: 49°F Clear / Overcast

Site Conditions: Paved w/ ring road. Overflow basin to the south

PSE Staff: Gray

Client/Other Staff: Nathan / Corey - Murraysmith

Exterior Inspection

Backfill Dim. to ~~Top~~ Bot. of Roof Slab: (N) 18' (E) 21.7' (S) 25.4' (W) 22.5'

Roof Slab Thickness: 9" / 9" (drawings/measured) Roof Overhang Dimension: 9" / 8½" (drawings/measured)

Drip Groove? (Y/N): Y / Y (drawings/measured) Threadbar Pocket Spacing: 4.2' / 4'-2" (drawings/measured)

Top Surface Roof Slab Condition: Good. Some cracking but most minor. No structural issues noted for Ert roof slab.

Ladder/Vents/Hatch/Joint Conditions: Ladder - Good (16" wide, 12" rung space, 11" rung to 12" side) Vent & Hatch Good

Exterior Shotcrete Condition: Gen. Good but on North side sounding found area along roof drain point with potential delam issue, Noted area 1 & 2, Crazing/cracking noted about 8' above grade in multiple locations

1

PRESTRESSED RESERVOIR SITE INSPECTION

Sound shotcrete at regular intervals and record results: Loc 1 & 2 on N. side of res. Sounded and are potential areas of delam. and should be repaired.

Wire/Strand Wrapping Condition: Not evaluated (see above - 2 zones of delam could impact)

Other Comments: None

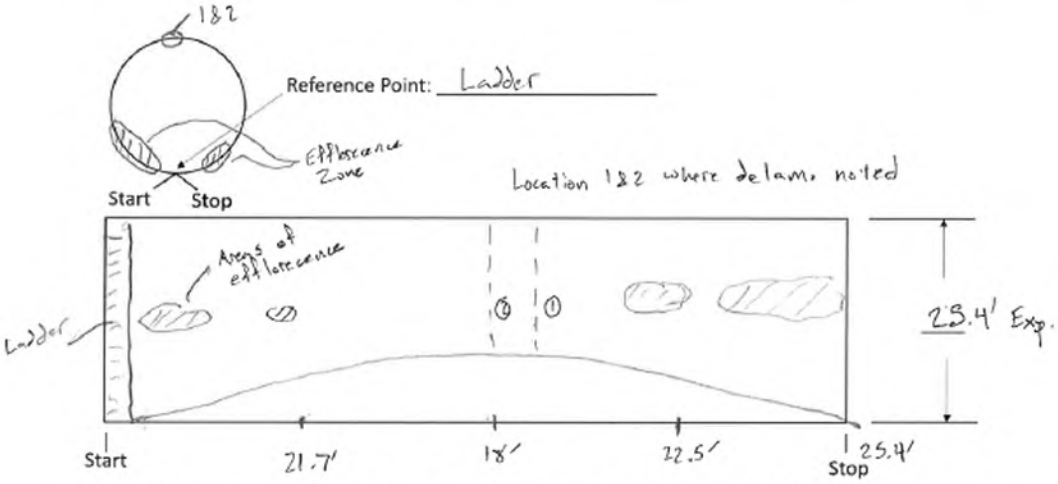
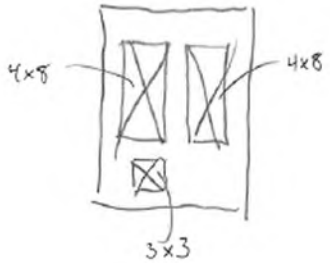


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

Valve Vault Notes:

14'x13'x7.5' Clr. Middle strip 28" wide. Roof Slab 13½" thick

Overall good condition. No issues w/ Crack of Water during obs.



2

PRESTRESSED RESERVOIR SITE INSPECTION

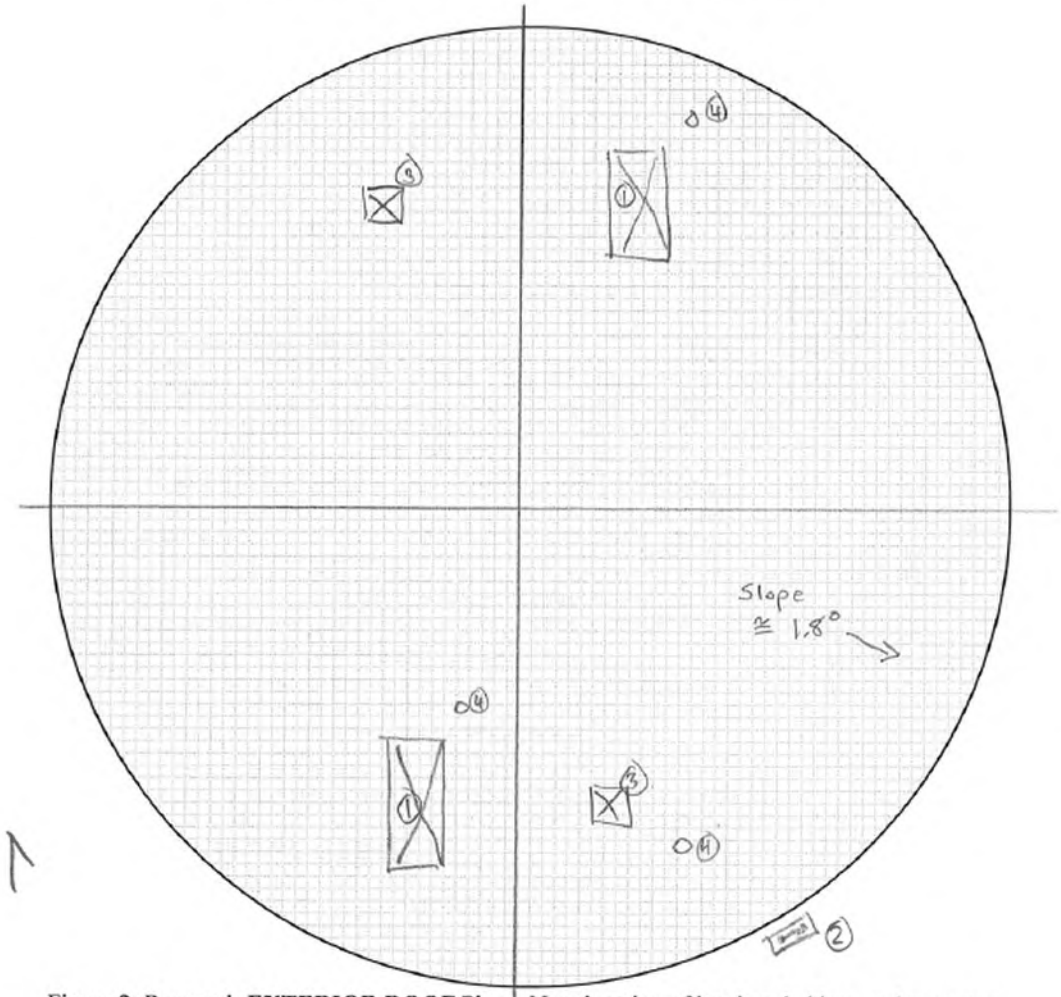


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

① Hatch Ext 8'x10', 12" curb. 6'-4" x 8'-4" opening
② Ladder see pg 1 notes
③ Vent Ext 4'-10" square, 16" curb w/ AL cap.
④ Light, sight lighting directed at ladder & hatches

3

PRESTRESSED RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Minimal crack, no major efflorescence. Primary cracking noted by drop panels nearest walls

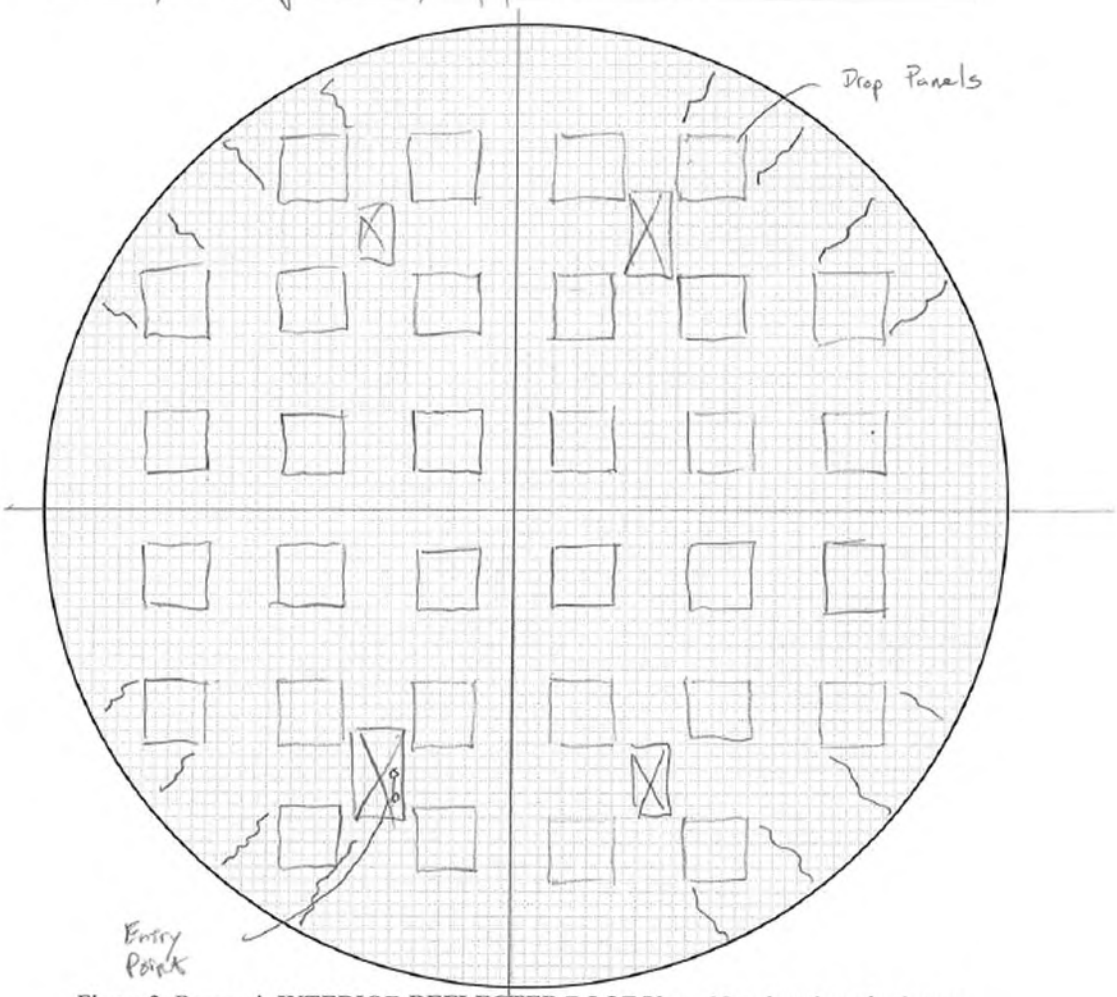


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

PRESTRESSED RESERVOIR SITE INSPECTION

Column Diameter: 18" / 18" (drawings/measured) Footing Size/Thickness: 7'x7' 1'-2" / 7' square 1'-2½" (drawings/measured)

Column Spacing: 20' (drawings/measured) Wall Curb Dimensions: N/A / N/A (drawings/measured)

Floor Slab Condition: Good, no major cracking noted. Slab does have cast-in drains

Floor Slab Joints Spacing/Condition: No joints

Column/Footing Conditions: Good no spall or cracking. Footing rotated 45° relative to drop panel

Note: No overflow pipe. Tank level set by zone.

5

PSE PETERSON STRUCTURAL ENGINEERS

PRESTRESSED RESERVOIR SITE INSPECTION

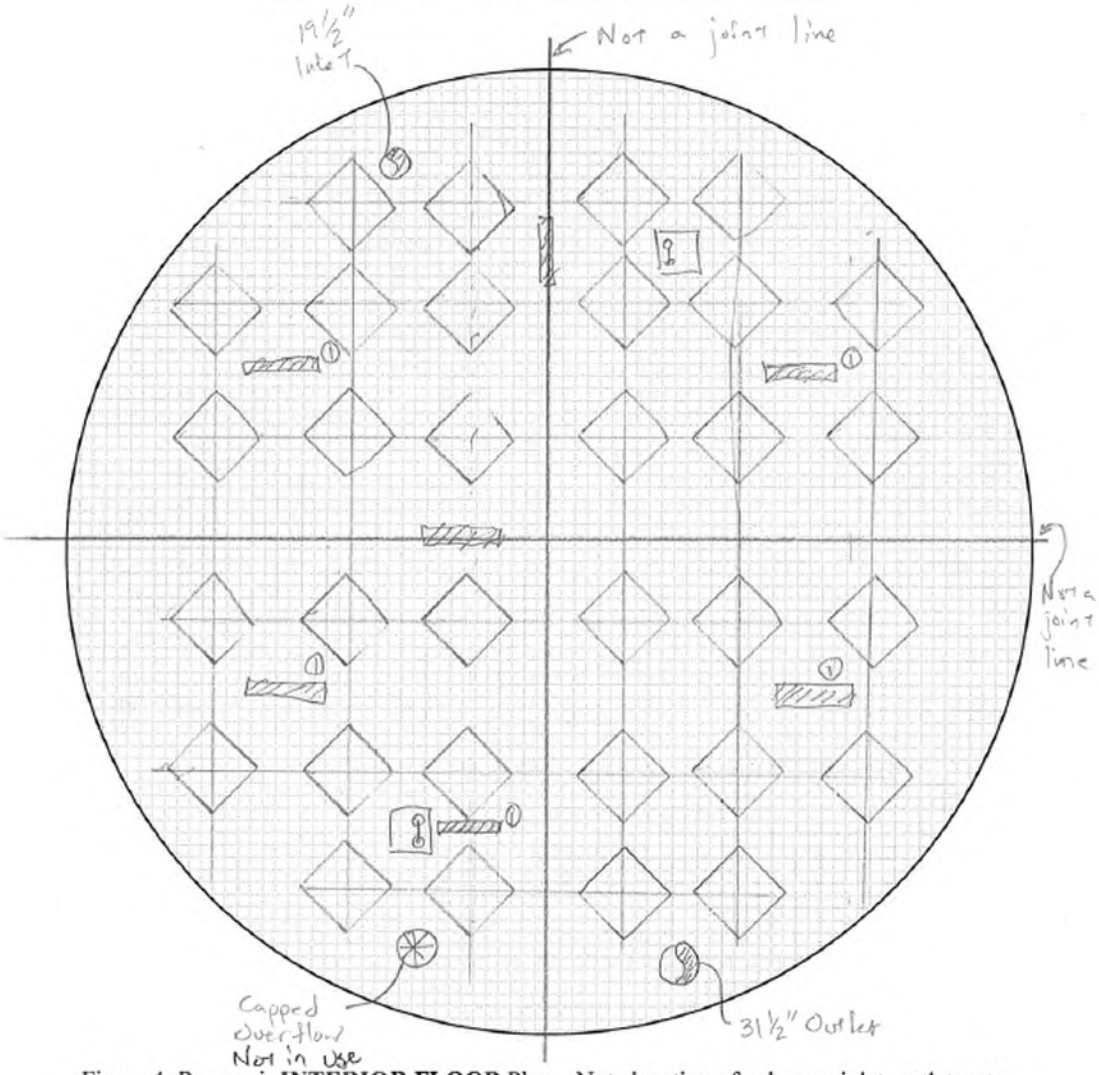


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

① Drain Channel 9'-10" x 5" wide

PRESTRESSED RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Good. A few areas where bearing pad is notched or proud of wall but no major issues

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): No really noted. Formwork and patching good.

# of wall sections: 9 (abou 45' ea.)

Ladder/Pipes/Overflow Conditions:

Overflow Height: 26', No Overflow (drawings/measured) capped

Operating Height: 24' low (per City/PUD/other) 9' Min Avg 14.5'-22' max

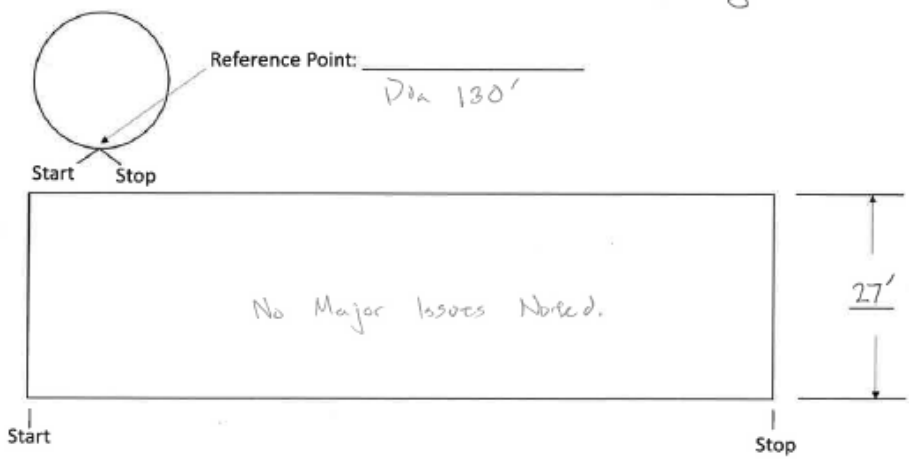


Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

7

END OF SECTION

Appendix E-3 Kearney General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Kearney Reservoir *General Info*

Field Visit Date: 3/14/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	3/14/2019
Reservoir Name and Location:	Kearney - 4281 James St 98226, near James St and Francis Ave
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis
Client Staff Present:	Shayla Francis, Steve Bradshaw, Alex
Year Constructed:	2006
Overflow Destination:	CT Reservoir (Whatcom Falls II)
Discharge Destination/Zone:	276 North Zone
Fill Location:	276 North Zone
Reservoir Material:	Pre-Stressed Concrete

Measurement Type	Measurement	Unit
Volume:	2.5	MG
Diameter (or other dimensions - see notes):	130	ft
Height	27	ft
Overflow Elevation:	276	ft AMSL
Bottom Elevation:	251	ft AMSL
Level of Overflow	25	ft
Minimum Normal Operating Level:	14.5	ft
Maximum Normal Operating Level:	22	ft
Notes: No overflow in reservoir. Overflow elevations are for Whatcom Falls II, acting as overflow.		

Kearney Reservoir *Exterior Inspection*

Field Visit Date: 3/14/2019

Exterior Ladder:	
Present on site?	Yes
Material:	Stainless Steel
Condition:	Very Good
Corrosion:	No
Cage:	No
Security Type:	Locking door
Security Condition:	Very Good
Wall Attachment Type:	Anchored
Wall Attachment Condition:	Very Good
Rung Shape:	Circular

Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	2	in
Front Clearance:	10	in
Back Clearance:	N/A	in
Notes:		

Exterior Fall Prevention System:	
Present at Site:	Yes
Type:	Rigid Rail
Fall Protection System Condition:	Very Good
Notes:	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:	
Hatch Location:	NE Roof
Material:	Aluminum
Condition:	Very Good
Gasketed:	Yes
Intrusion Alarm:	Yes
Lock:	Yes
Frame Drain Location:	N/A

Measurement Type	Measurement	Unit
Size:	8X10	ft
Curb Height:	12	in
Notes: Measurement of exterior. Interior: 6'4" x 8'4"		

Entry Hatch:		
Hatch Location:	SW Roof	
Material:	Aluminum	
Condition:	Very Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Exterior	
Measurement Type	Measurement	Unit
Size:	8X10	ft
Curb Height:	12	in
Notes: Measurement interior: 6'4" x 8'4"		

Roof Vents and Screen:		
Material:	Aluminum	
Condition:	Very Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	3/8	in
Notes: Screen size taken from drawings.		

Roof:		
Condition:	Very Good	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	None	
Slope of roof	1.7 deg	
Measurement Type	Measurement	Unit
Overhang Distance:	9	in
Thickness of roof slab	9	in
Notes: Some minor cracking.		

Railing:		
Present at Site:	Yes	
Material:	Stainless Steel	
Condition:	Very Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Very Good	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	6	in
Top Height:	43	in
Notes:		

Grating:	
Present at Site:	No

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Fair
Notes: Two areas of potential concrete delamination identified from sounding where roof drainage runs down side of reservoir. Minor efflorescence noted about 8' above grade in multiple locations.	

Exterior Coating	
Exterior Walls:	No Coating
Exterior of Roof:	No Coating
Exterior Piping:	N/A
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A
Notes:	

Kearney Reservoir *Interior Inspection*

Field Visit Date: 3/14/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Stainless Steel	
Condition:	Very Good	
Corrosion:	No	
Cage:	No	
Security Type:	locked access hatch	
Security Condition:	Very Good	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Very Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	N/A	in
Back Clearance:	N/A	in
Notes: Ladder has removable extension		

Interior Fall Prevention System:	
Present at Site:	Yes
Type:	Rigid Rail
Fall Protection System Condition:	Very Good
Notes:	

Interior Roof:		
Condition:	Very Good	
Measurement Type	Measurement	Unit
N/A		ft
Notes: Minimal cracking and no major efflorescence. Primary cracking noted by drop panels nearest walls. Roof is reinforced concrete.		

Columns:		
Material:	Concrete	
Condition:	Very Good	
Measurement Type	Measurement	Unit
Width/Diameter	18	in
Base width	7	ft
Notes: Column Spacing/Configuration: 32 columns w/ 7 sq. footing rotated 45 degrees.		

Floor	
Condition:	Very Good
Leaks:	No
Approximate Location:	
Severity:	
Notes:	

Walls:	
Condition:	Very Good
Painters Rings Present:	No
Notes: 130 ft diameter interior. No issues noted on interior walls	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	No Coating
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Kearney Reservoir *Miscellaneous*

Field Visit Date: 3/14/2019

Piping		
Inlet Piping:	Size (Inches OD):	20
	Condition:	Good
	Material:	Ductile Iron
	Notes: Inlet and outlet combine after piping vault.	
Outlet Piping:	Size (inches OD):	30
	Condition:	Good
	Material:	Ductile Iron
	Lip (Inches)	4
	Notes:	
Overflow Piping:	Size (inches OD):	20
	Condition:	N/A
	Air Gap:	N/A
	Screened:	N/A
	Material:	Ductile Iron
	Outlet Location:	N/A
	Erosion Evident:	N/A
	Screen Condition:	N/A
	Overflow to roof (feet)	N/A
	Notes: Overflow piping penetration present, but capped near floor.	
Drain Piping:	Size (inches OD):	6
	Condition:	Good
	Outlet Location:	Storm pond
	Screened:	Yes
	Material:	Ductile Iron
	Silt Stop Type:	N/A
	Air Gap:	Yes
	Screen Condition:	Fair
	Notes: Six rectangular floor drains present. Grates are stainless steel. Outlet screen size too coarse. Dechlorination on overflow only.	

Piping Facilities		
Exterior Valving:	Type:	Butterfly valves
	Condition:	Very Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	Two locations reservoir interior
	Size (Inches OD):	2
Roof/Wall Piping Penetrations	Sealed:	N/A
	Leaks:	N/A
Notes: Interior washdown piping hose bibs exhibit extensive corrosion.		

Electrical	
Cathodic Protection:	No
Impressed Current:	No
Anodes:	No
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	Yes
Common Inlet/Outlet:	No
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	Yes - minor
Hydraulic Mixing System Type and Mfg.:	No
Sediment Build-Up Height Above Floor (in)	0.1
Water Quality Sample Taps?	Yes
Notes: WQ tap located on outlet pipe. Tree has fallen on fence allowing unauthorized access. Float system weight had fallen to reservoir floor, but was removed by City staff as site visit was completed.	

Appendix E-4 Kearney Condition Assessment Score Sheet

Kearney Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	3	0	Fence damaged by tree
	Vegetation Separation	0	0	0	0	0	0	4	0	A few trees are close enough to damage reservoir if they fall.
	Site Drainage	0	0	0	0	0	0	3	0	Organic growth and staining near ring road
Walls	Exterior Walls	5	3	5	5	5	0	5	0	Areas of delamination noted should be investigated and repaired so that strand is not impacted
	Interior Walls	5	5	5	5	5	0	5	0	
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	3	5	5	5	4	0	5	0	Some sediment accumulating
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	5	0	0	0	0	
Roof	Exterior Roof	5	5	5	5	5	0	5	0	
	Interior Roof and Supports	5	4	1	5	0	0	0	0	The overflow is capped which can cause overpressure issues
	Columns	5	5	5	5	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	5	5	0	
	Interior Ladders/Fall Protection	5	5	0	0	0	5	4	0	Some difficulty getting into reservoir.
	Access Hatches	5	5	0	0	3	3	4	0	High maintenance design. Needs hatch railing
	Railings and Roof Fall Protection	5	5	0	0	0	5	5	0	
	Vents	5	5	0	0	3	0	5	0	Screen too coarse and hard to inspect
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	3	4	0	5	5	0	5	0	Inlet pipe is corroded - needs coating
	Outlet Piping	5	0	0	5	5	0	5	0	
	Drain Piping	5	5	0	5	3	0	3	0	Needs updated screen and dechlorination system
	Overflow Piping	0	4	0	0	0	0	2	0	Missing overflow
	Washdown Piping	3	1	0	0	0	0	2	0	Too corroded to use
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	5	5	0	0	5	0	5	5	
	Isolation Valving	5	5	0	0	5	0	5	5	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	4.6	4.5	4.4	5.0	4.4	4.5	4.3	5.0	

Overall Score
4.6

Appendix F Whatcom Falls II

Appendix F-1 Whatcom Falls II Geotechnical Report

GeoEngineers

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

Memorandum

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Whatcom Falls II Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Whatcom Falls II reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at Whatcom Falls II reservoir site, located as shown in the Vicinity Map, Figure 1. The Whatcom Falls II reservoir is a prestressed concrete reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. Undifferentiated glacial deposits are mapped nearby.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift.

Previous Studies and As-built

We reviewed the original report for the Whatcom Falls II reservoir project titled "Revised Preliminary Geotechnical Report, Preliminary Tank Siting Study, Gravity Zone CT and Operating Volume Reservoir, Bellingham, Washington" by Converse Consultants NW dated September 22, 1992. Six test pits were completed near the proposed Whatcom Falls II reservoir footprint to depths of 1 to 15 feet below ground surface (bgs). The test pits encountered various glacially consolidated soil layers including glacial till and advance outwash, and also encountered bedrock.

The as-builts for the Whatcom Falls II reservoir are dated May 1993 and indicate that the reservoir top of slab is Elevation 254.5 feet (City Datum).

Surface Conditions

The project site is located 250 feet to the south of Arbor Court. The reservoir is located on top of a small hill and the site drops off in all directions, located within Whatcom Falls Park. The site is bounded by a wooded area in all directions. A small dirt road accesses the site from the east.

Subsurface Exploration

No new explorations were completed as part of this study for this site. The locations of the previous explorations are shown in the Site Plan, Figure 2. The test pits logs from the previous study are presented in Appendix A.

Subsurface Conditions

A general description of each of the soil units encountered at the project site is provided below. Our interpreted soil conditions are based on soil conditions encountered during our current phase geotechnical borings, our previously completed borings and test pits and our experience at nearby project sites.

- **Forest Duff –** Approximately 0.5 to 1 foot of forest duff was encountered at all test pit locations.
- **Recent Alluvium –** Recent alluvium was only encountered in TP-5 and is described as dense silty sand.
- **Advance Outwash –** Advance outwash was encountered in TP-3, TP-4 and TP-5 and is described as medium dense to very dense sand and gravel with occasional cobbles.
- **Glacial Till –** Glacial till was encountered in TP-1 and TP-2. The till is described as very dense silty sand with gravel.
- **Chuckanut Sandstone- Siltstone –** Sandstone was encountered in TP-2 at 8 feet bgs, TP-4 at 14 feet bgs, and TP-6 at 1 foot bgs, respectively. Test pit TP-6 is located along the access road and not near the reservoir foundation.

Groundwater

Groundwater was observed in TP-4 at a depth of 9.5 feet and in TP-5 at a depth of 5.5 feet bgs. The Chuckanut sandstone unit and glacial till commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Based on our review of the explorations and the project as-built drawings, we conclude that the Whatcom Falls II reservoir is founded on dense glacial soils or structural fill extending to glacial soils. The alluvial soils encountered in the explorations were excavated and replaced with structural fill.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located within the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on dense glacial drift deposits/structural fill which are not at risk of liquefaction.

AWWA/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on publications D100-11 of the AWWA and the ASCE 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the

maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. AWWA AND ASCE 7-10 PARAMETERS

AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	D
Soil Profile Type	Stiff Soil
Average Field Standard Penetration Resistance	$15 \leq N_{ave} \leq 50$
AWWA Seismic Use Group	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	95.0
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.3
Seismic Coefficient, F_a	1.12
Seismic Coefficient, F_V	1.66
MCE_G peak ground acceleration, PGA	0.393
Seismic design value, S_{DS}	0.710
Seismic design value, S_{D1}	0.411
MCE_G peak ground acceleration, PGA	0.393
Seismic design value, S_{DS}	0.710
Seismic design value, S_{D1}	0.411

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 3 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the M 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and

paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 4 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on review of the explorations and as-built drawings, we conclude that the Whatcom Falls II reservoir is supported on dense glacially consolidated soils or structural fill extending to this layer. The drawings indicate that any recent alluvium such as encountered in TP-5 was removed and replaced with structural fill. The as-builts indicate that the design allowable bearing capacity was 3,500 pounds per square foot (psf) for wall footings and 4,000 psf for column footings. We recommend that the structure be evaluated with the same bearing pressures. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The Whatcom Falls II reservoir includes partially buried walls. Our recommendations for concrete below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section and the backfill is structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the south that is approximately 20 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing on glacial soil or

structural fill extending to glacial soils. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

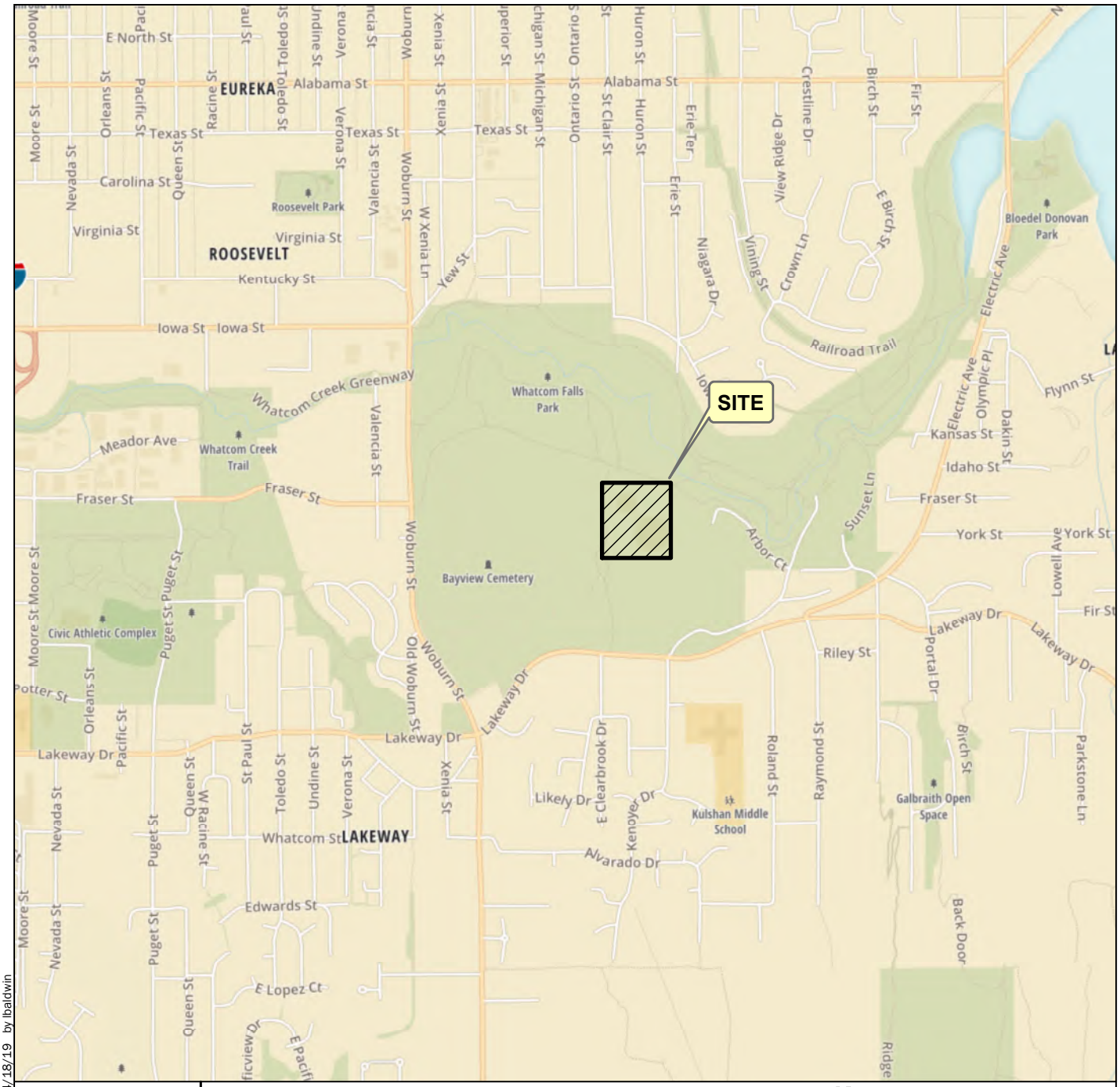
Figure 2 – Site Plan

Figure 3 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 4 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault

Appendix A. Log of Explorations

Figures A-1 through A-6 – Test Pits TP-1 through TP-6 (Converse Consultants NW)



N
W E
S

1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

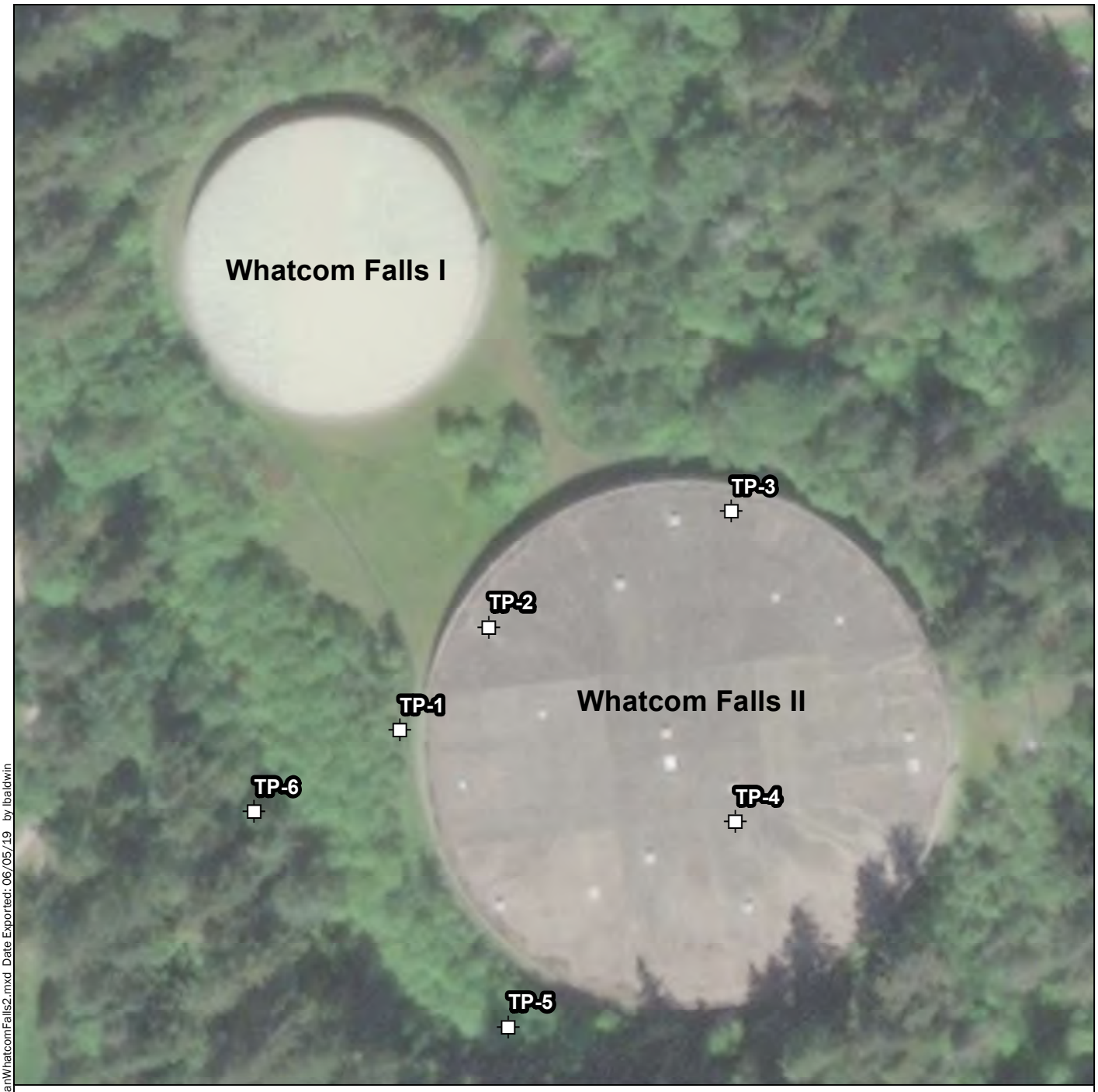
Whatcom Falls Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_WhatcomFalls.mxd Date Exported: 04/18/19 by lbaldwin



Legend

Test Pit by Converse Consultants (1992)

100 0 100

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Whatcom Falls II Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

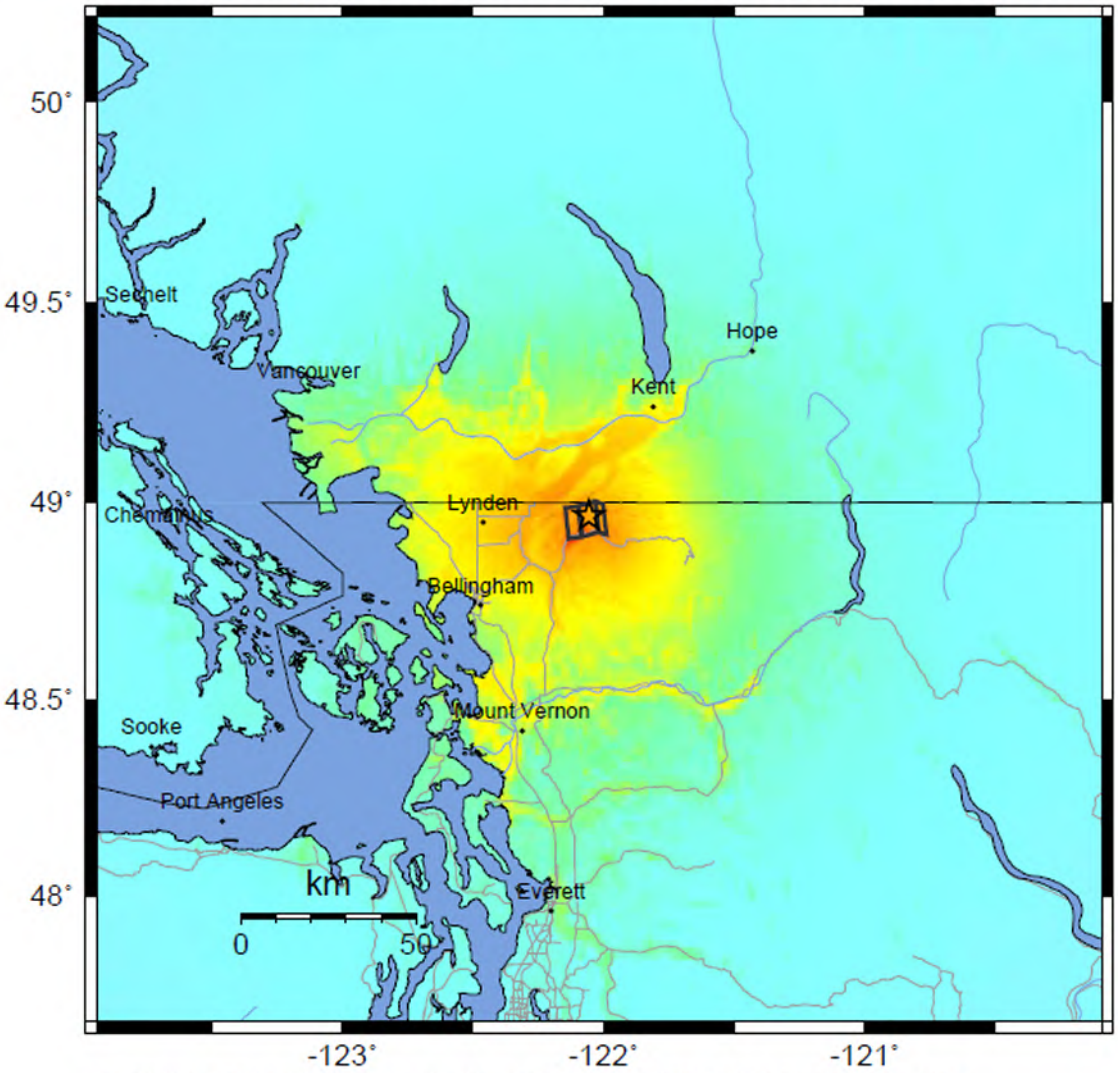
GeoEngineers

Figure 2

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

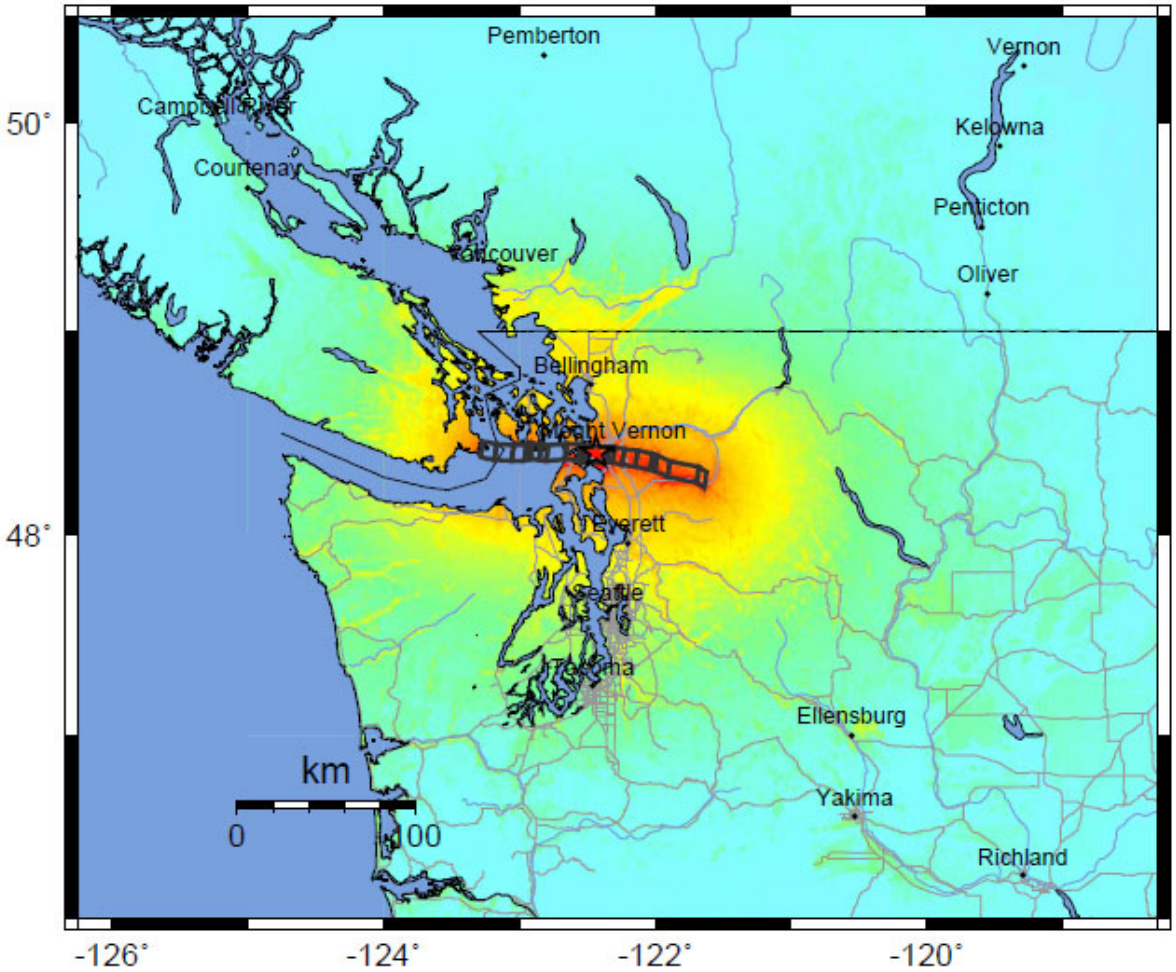
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

APPENDIX A
Log of Explorations

LOG OF TEST PIT NO. TP-1

Surface Conditions: **top of knoll, flat, low brush, conifers** Elevation (Approx.): **278**

This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	DESCRIPTION
						Forest duff, 6 inches
1						GLACIAL TILL (weathered) SILTY SAND WITH GRAVEL; gray-brown mottled rust, trace cobbles, occasional boulders to 2-foot diameter, trace roots; dense, slightly moist
2						
3	275					
4						becomes unweathered, grades to gray; very dense
5						
6						
7						
8	270					
9						
10						
11						
12						
13	265	1				
						Bottom of test pit at 13-1/2 feet; completed and backfilled on 7/23/92. No groundwater encountered.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington

for PEI/Barrett Consulting Group

92-35132-01

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No.
A-1

LOG OF TEST PIT NO. TP-2

Surface Conditions: **on a 3H:1V slope, east side of knoll, low brush, ferns, conifers**

Elevation (Approx.): **259**

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
						Forest duff; red-brown, organics; soft, moist
1						GLACIAL TILL (weathered)
2						SILTY SAND WITH GRAVEL; brown to gray-brown, mottled rust, roots to 3 feet, cobbles up to 4-inch diameter, trace boulders up to 3-foot diameter; dense, slightly moist
3						
4	255					
5						becomes unweathered, gray-brown to gray
6						
7		1				boulders up to 3-foot diameter
8		2				BEDROCK
						SANDSTONE; gray, fine to medium-grained, hard, slightly weathered; bedding is about 15 degrees from the horizontal, sloping down to the east
						Dug to refusal at 8 feet; completed and backfilled on 7/23/92. No groundwater encountered.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington

for PEI/Barrett Consulting Group

92-35132-01

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No.
A-2

LOG OF TEST PIT NO. TP-3

Surface Conditions: **flat, slight slope to northeast** Elevation (Approx.): **258**

This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	DESCRIPTION
						Forest duff, 6 inches, dark red-brown, organics; loose, moist
						ADVANCE OUTWASH DEPOSITS
1						SAND AND GRAVEL; brown, fine to coarse, some cobbles up to 8-inch diameter, trace silt; medium dense, slightly moist
2						
3	255					
4		1	7			
5						gravel and cobble layer, fine to coarse
6						
7						more coarse gravel
8	250					increasing number of boulders up to 2 feet gravel layer
9						
10						
11						
12						
13	245					
14						
15						Bottom of test pit at depth 15 feet; completed and backfilled on 7/23/92. Test pit terminated due to caving. No groundwater encountered.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington

for PEI/Barrett Consulting Group

92-35132-01

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No.
A-3

LOG OF TEST PIT NO. TP-4

Surface Conditions: **flat, open, with low vegetation** Elevation (Approx.): **252**

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
						Forest duff, red-brown, trace organics; soft, moist
1						ADVANCE OUTWASH DEPOSITS (unweathered)
2	250					SAND AND GRAVEL; gray-brown, fine to coarse, little silt, roots to depth 2.0 feet; very dense, slightly moist
3						
4						
5						grades less silt
6		1				
7	245					
8						
9						large boulder, size estimated at 6-feet by 3-feet
10						occasional boulders up to 2 feet; wet, dense
11						
12	240					
13						
14						BEDROCK
						SANDSTONE; hard, fine to medium-grained, slightly weathered Dug to refusal at depth 14 feet; completed and backfilled on 7/23/92. Groundwater observed at 9-1/2 feet.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington **92-35132-01**

for PEI/Barrett Consulting Group

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No. A-4

LOG OF TEST PIT NO. TP-5

Surface Conditions: **flat, low brush, ferns, south end of knoll, many conifers** Elevation (Approx.): **249**

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
						Forest duff, 6 inches; red-brown; soft, moist
						RECENT ALLUVIUM
1						SILTY SAND; red-brown, fine to coarse, trace to little fine to coarse gravel, roots to 1 foot; medium dense, moist
2						
3						
4	245	1				
5						
6		2				SANDY SILT; gray, little fine to coarse sand, trace fine to coarse gravel; stiff, very moist
7						ADVANCE OUTWASH DEPOSITS
8						SAND AND GRAVEL; brown, trace to little silt, grades to less silt; very dense, wet
9	240					
10						
11						boulder, size estimated at about 5-foot by 1-foot
12						boulders up to 4-foot diameter, angular and subangular
13						
14	235					Dug to limit of track hoe reach at depth 14 feet; completed and backfilled on 7/23/92. Groundwater encountered at depth 5-1/2 feet.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington

for PEI/Barrett Consulting Group

92-35132-01

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No. A-5

LOG OF TEST PIT NO. TP-6

Surface Conditions: **west side of knoll, flat, low brush and vegetation, conifers**

Elevation (Approx.): **248**

Depth, ft	Elev., ft	Samples	Moisture Content, %	Other tests	Graphic Symbol	This log is part of the report prepared by Converse Consultants NW for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this trench at the time of excavation. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
						Forest duff; red-brown, trace organics; soft, moist
1		1				BEDROCK
						SANDSTONE; gray, hard, slightly weathered
						Dug to refusal at depth 1 foot; completed and backfilled on 7/23/92. No groundwater encountered.

Project No.

BELLINGHAM PRELIMINARY TANK SITING

Bellingham, Washington

for PEI/Barrett Consulting Group

92-35132-01

Converse Consultants NW Geotechnical Engineering and Applied Earth Sciences

Figure No.
A-6

Appendix F-2 Whatcom Falls II Corrosion and Coatings Report

June 30, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Whatcom Falls #1 and #2 Tank Cathodic Protection System Checkout

Mr. Hardy,

Northwest Corrosion Engineering completed a checkout of the cathodic protection systems associated with the City of Bellingham's Whatcom Falls #1 steel water storage tank and Whatcom Falls #2 concrete water storage tank.

BACKGROUND INFORMATION

Whatcom Falls #1 Tank

Whatcom Falls Tank #1 was built in 1982 and is 17-ft tall, 200-ft in diameter and constructed of welded steel. The impressed current cathodic protection system is comprised of an autopotential rectifier and 60 high silicon cast iron anodes supported from the roof. The anodes are arranged in three rings with diameters of 40, 120, and 180-ft. Each ring contains 20 anodes which are connected to a common header cable.

Whatcom Falls #2 Tank

The Whatcom Falls #2 Tank, constructed in 1995, is 22-ft tall, 350-ft diameter and consists of spiral reinforced concrete. An impressed current cathodic protection system is installed for corrosion control of submerged and concrete embedded metallic tank components. The configuration and type of anode material is unknown.

TEST PROCEDURES AND ANALYSIS

Current Output

System current output is recorded by measuring the voltage drop across the calibrated current measuring shunt installed on the rectifier panel board. As an additional check, a portable clamp-on ammeter is used to measure the current flow in the anode header cable.

Structure-to-Electrolyte Potentials

Structure-to-Electrolyte potentials were recorded between the tank structures and a portable copper-copper/sulfate (CSE) reference electrode. Data was collected along the interior of each tank from the water surface level to the bottom of the tank at two-foot intervals using the roof access hatches. Potentials were measured using a high impedance digital voltmeter. Potentials recorded during this survey included:

ON – The ON potential is recorded between the structure and a reference electrode while the cathodic protection system is in operation. This value gives an indication as to whether the structure is receiving current from the anodes. However, this measurement includes an error introduced into the circuit as a result of current flow. Because of this error, these values are only used when it is not possible to completely disconnect all current flow sources or when comparing previously established ON and Instant Off readings.

Instant Off – This measurement is the actual electrical potential between the structure and reference electrode. To measure this value, the cathodic protection system is momentarily switched off, resulting in a polarized potential, free of error that establishes if adequate cathodic protection is being provided.

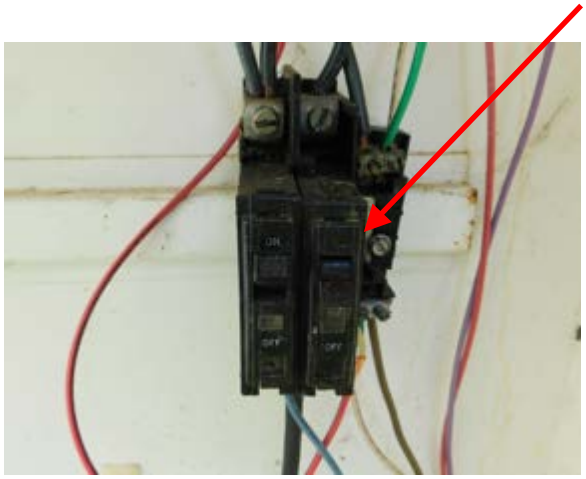
Rectifier and Structure-to-Electrolyte data collected during this survey is presented in Tables I and II at the end of this report.

RESULTS

Whatcom Falls #1 Tank

Using a portable reference electrode, all potential measurements meet NACE criteria for effective corrosion control[1]. The stationary reference electrode used to operate the rectifiers autopotential circuitry appears to be failing as its readings are significantly more positive than normal.

Upon our arrival, it was noted that the rectifier was not functioning. Troubleshooting resulted in determining that the AC breaker for the unit was defective. As a temporary measure, the AC conductors for the rectifier were landed on an adjacent breaker in order to test the cathodic protection equipment. After testing, the rectifier was left terminated to the operating breaker, however, a dedicated breaker needs to be provided for the unit.



Defective rectifier breaker

Rectifier panelboard

[1] SP0388 Impressed Current Cathodic Protection of Internal Submerged Surfaces of Carbon Steel Water Storage Tanks

As part of the internal inspection of the Whatcom Falls #1 tank, it was noted that one of the anode support hangers had become detached from the roof. This should be replaced in the near future to ensure that the anode does not electrically short to the bottom of the tank.

Broken anode hanger, should be suspended from the roof

Whatcom Falls #2 Tank

The Whatcom Falls #2 tank is constructed of concrete and as such, the electrical potential requirements for corrosion control are less than the typical (-)850 instant off requirement for carbon steel. The application of excessive current to metallic materials embedded in concrete can result in damage to the reinforcing steel and spiral-wound cables. Cathodic protection current output should be limited to that which allows for approximately 100 millivolts of polarization (typically in the -500 to -600 millivolt range versus a CSE).

The Whatcom Falls #2 tank does not have readily accessible areas to make metallic connection for testing purposes. Normally, a structure connection is made to the interior ladder, roof hatch lid, or other electrically continuous component of the tank. In this case, the installed stainless steel ladder in the #1 hatch is not continuous with the reinforcing steel. When performing the potential profile survey, the negative lead connection was made to the negative terminal of the rectifier. This does not pose a problem so long as Instant Off potentials are measured as recording On readings will result in additional measurement error due to the negative terminal being a current carrying conductor.

The Whatcom Falls #2 tank does not include provisions for autopotential circuitry. The rectifier is adjusted to a maintain a constant voltage output regardless of the level of water in the tank. This can become problematic if the water level fluctuates on a regular basis as the embedded steel will be subjected to higher voltage gradients which could result in damage to these tank structural components.

RECOMMENDATIONS

1. Replace defective AC breaker for the Whatcom Falls #1 tank rectifier.
2. Replace aging stationary reference electrode used to operate the autopotential circuitry of Whatcom Falls #1 tank.
3. Repair defective anode hanger on Whatcom Falls #1 tank.
4. Install a new autopotential rectifier, stationary reference electrode, and sensing ground for the Whatcom Falls #2 tank.
5. All cathodic protection systems should be inspected by a qualified Corrosion Engineer on an annual basis.

We appreciate the opportunity of assisting you with this project. Please feel free to contact our office if you have any questions or would like additional information.

Sincerely,
Northwest Corrosion Engineering

Jeremy A. Hailey, P.E

TABLE 1A: Rectifier Data – Whatcom Falls #1

Manufacturer: Goodall
Model: CRAYSA 60-22 FNPRSZ-0
Serial No: 8601683

Name Plate Data: AC Volts 115 DC Volts 60
AC Amps 17.7 DC Amps 22
60 Hertz, Single Phase, Amb. Temp 45ºC
Max Tap Setting – Coarse E, Fine 5
Shunt Rating – 50mV/30A

As-Found

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	5.0	3.78
Amps DC	1.0	0.320
Set Potential	-1000	-1207 ON
Read Potential	-1250	-502 Instant Off
Tap Setting	Course A Fine 3	-320 Depol. In 30 seconds

TABLE 1B: Structure-To-Electrolyte Potential Data

Ref. Cell Depth, ft	ON, mV	Instant Off, mV	Ref. Cell Depth, ft	ON, mV	Instant Off, mV
Top of Water	-1918	-1028	8	-1911	-1007
2	-1933	-1024	10	-1839	-1008
4	-1932	-1026	12	-1771	-999
6	-1931	-1007	14	-1753	-998

NOTES:

1. The AC breaker for the rectifier is defective and needs to be replaced.
2. The instant off potential of the stationary reference electrode is more positive than normal, however, using a portable reference electrode, all potentials are within normal range.
3. No adjustments to the rectifier output were made.
4. The rectifier was left operating in autopotential mode.
5. All potentials meet criteria for effective corrosion control.

TABLE 2A: Rectifier Data – Whatcom Falls #2

Manufacturer: RTS
Model: CSAYSA 60-8 Z
Serial No: C-991376

Name Plate Data: AC Volts 115 DC Volts 60
AC Amps 6.35 DC Amps 8
60 Hertz, Single Phase, Amb. Temp 45ºC
Max Tap Setting – Coarse E, Fine 5
Shunt Rating – 50mV/10A

As-Found

Rectifier Output	Rectifier Meter	Portable Meter
Volts DC	34	34.2
Amps DC	3.0	3.02
Tap Setting	Course C Fine 2	

TABLE 2B: Structure-To-Electrolyte Potential Data

Ref. Cell Depth, ft	ON, mV	Instant Off, mV	Ref. Cell Depth, ft	ON, mV	Instant Off, mV
Top of Water	-910	-596	10	-934	-607
2	-913	-587	12	-939	-613
4	-917	-591	14	-944	-622
6	-922	-596	16	-941	-618
8	-928	-601			

NOTES:

1. Potential data was collected at hatch #1.
2. The rectifier is a constant voltage model and is not capable of automatic current output adjustment.
3. The stainless steel ladder at hatch #1 is not continuous with the embedded steel within the tank. For testing purposes, the structure lead was connected to the rectifiers negative terminal.
4. All potentials meet criteria for effective corrosion control.

Appendix F-3 Whatcom Falls II Structural Report

CITY OF BELLINGHAM

CH 8: WHATCOM FALLS 2 PRESTRESS RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

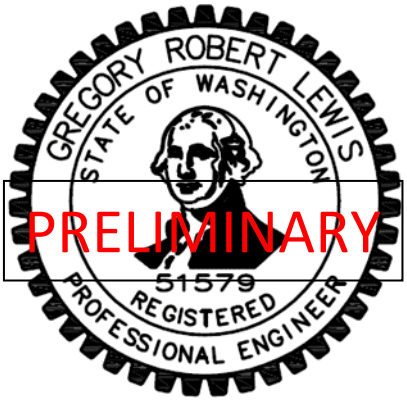
Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Whatcom Falls 2, 15.6 Million Gallon (MG) prestress reservoir. The reservoir is located near 3504 Arbor Ct, Bellingham, WA (Lat. 48.7507, Long. -122.4355), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to inspect and visually evaluate the reservoir on June 12th, 2019 and again on November 7th, 2019 by Peterson Structural Engineers (PSE) and Murraysmith, Inc. The reservoir was filled during both inspections which included a floating evaluation of the interior roof and baffle system and an evaluation of the exterior wall and roof elements. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Whatcom Falls 2 Prestress Reservoir – 15.6 MG

2.1 Description & Background

The reservoir design was prepared by Barrett Consulting Group and the original construction drawings provided are dated May 1993. The reservoir is a 350-foot inside diameter by 23-foot high, strand-wrapped, prestressed concrete water reservoir with interior columns. The roof is 10-inches thick and is supported by (221) 24-inch diameter columns. Four construction joints lines, dividing the roof into 9 sections, were observed in the roof slab.

Per the original drawings the reservoir contains vertical prestressing bars in the core wall. The roof slab bears on the tank's wall and is connected with shear cans at each vertical tendon. There are 288 total vertical prestressing tendons. Per the drawings, each tendon is 1-1/4-inch in diameter. The wall base connection utilizes 288 seismic cable sets (with 2 cable sets per vertical tendon). Each seismic cable consists of (4) ½-inch diameter, 7-wire galvanized strands. Both the roof and the floor connections are detailed with a continuous elastomeric bearing pad.

The original drawings specify a roof snow load of 25-psf along with a 10-psf incidental live load. This meets the current code requirements of 25 psf for snow load design. Roof dead loads included a 5-psf load to account for the baffle which is hung from the underside of the roof. For the evaluation contained herein an additional 5-psf dead load was included to cover the observed roof-mounted solar array which was installed after the design of the reservoir.

The reservoir has a 6-foot diameter inlet pipe and a 7-foot square opening outlet. The baffle system installed in the reservoir ensures the water entering the reservoir follows a spiral length of approximately 2,600-feet in order to reach the outlet at the center of the reservoir. Drainage of the reservoir is achieved with an in-floor grated drainage system consisting of 26 floor grates. As this grid is throughout the floor of the reservoir, the floor slab was constructed without a slope. The reservoir does not have an overflow pipe or any installed systems allowing the installation of an overflow pipe should one be desired. Per the drawings, the design maximum operating height is listed as 22-feet, which is 2-feet below the roof slab. Per the City, the reservoir's operating level is set by the remaining reservoirs in the service zone, and typically averages between 12 and 21-feet in the winter and 13-21-feet in the summer for a operational maximum storage volume of about 14.9 MG.

2.1.1 Description of Additional Site Structures and Features

The overall site includes the Whatcom Falls 1 steel reservoir, approximately 150-feet to the northwest of Whatcom Falls 2. The structural adequacy of Whatcom Falls 1 is discussed in a separate report. The site also includes a pump station, located about an eighth of a mile to the east. The pump station was not evaluated by PSE as part of this project. Neither of these structures are near enough to the Whatcom Falls 2 reservoir to adversely impact it during a seismic event.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit to observe the as-built current condition of the reservoir's interior roof, interior roof mounted appurtenances, exterior roof, exterior wall, and site conditions. The reservoir was filled

during both site visits. The first site visit involved a float of the reservoir and was performed June 12th, 2019 while a follow-up site visit was performed on November 7th, 2019.

Concrete Roof: The exterior of the reservoir's roof showed no major areas of visible structural failure; both the main slab and joints appeared to be in good visual condition. The main body of the roof slab was noted to be in good condition with some minimal, non-structural, pattern cracking observed. Four sets of joints were observed in the roof slab dividing it into 9 sections. The edge of the roof appeared to be in good condition although the parapet was causing ponding issues. The roof overhang was measured from the ladder and found to be 5.5-inches. An inset drip edge was located on the underside of the parapent curb, about 1.5-inches away from the face of the shotcrete layer. On the roof, a solar array had been added. The weight of this array did not appear to have resulted in any adverse issues around the area of its installation.

The reservoir's roof was constructed with a parapet in order to direct roof run-off to 10 drain locations. It was noted that the parapets were allowing ponding to occur along the edges of the roof due to debris build-up which impeded flow to the drains. At the drain locations, the parapet was observed to have been retrofitted by core-drilling through the side of the parapet in order to create a section flush with the rooftop. This allows water to bypass the drains should they get clogged with debris. However, while this would likely prevent a condition where blocked drains cause multiple inches of water to build up on the roof, localized ponding along the edges was still observed to be occurring. This reservoir was constructed with shear cans that are the full thickness of the roof slab and in many instances appear to extend to the surface of the roof. This configuration can allow standing water to seep between the shear can and concrete as there is no waterstop. Locations were observed on the wall of the reservoir where this seepage appeared to be happening through the parapet, via the shear cans. Figure 2-15 outlines how this leakage can occur due to the roof configuration and when ponding is allowed. If water is seeping into the shear cans, it is possible that the interior of the reservoir would also be susceptible to this type of leakage. While no instances of leakage were identified during PSE's float of the reservoir, our interior inspection was conducted in the summer when the roof surface was dry, and signs of leakage may not have been apparent.

The roof has multiple large and small access hatches located at various locations on the roof roughly corresponding to the baffle's route. These hatches include three 6x6-foot square equipment hatches, four 3x3-foot square access hatches, and an additional five 3x3-foot square vents. For these hatches the drawings call for 8-inch thick curbs around the perimeter of the hatch along with #3 rebar hoops. However, all the hatch curbs appear to have been constructed with two thinner sides and without the required hoop reinforcing. Per Figure 2-17 this appears to have resulted in cracking along the thinner sides and is likely due to freeze-thaw issues. For the appurtenances themselves, the vents appeared to meet current requirements for water quality. The equipment and access hatches, which have an edge inset into the curb, are considered high-maintenance designs per the Washington State Department of Health. The gutter-drains for these hatches should be screened to further protect them.

Roof Columns: The roof itself is supported with 24-inch diameter columns on octagonal footings. The footings could be observed through the water with enough detail to verify they were indeed octagonal in shape, but no additional information was able to be gathered.

Prestress Walls: The exterior walls of the reservoir are covered in shotcrete to protect the strand wrapping around the core wall. Overall the shotcrete was found to be in visibly good condition with only a few minor locations of noted efflorescence or alligator cracking observed. A majority of the wall was observed as the reservoir is unburied. The site is fenced to within about 16-feet on the northeast and southwest sides and further away towards Whatcom Falls 1 in the northwest. Beyond the fence the reservoir site is forested. The forest is in closer proximity along the south side which is likely the reason for the greater amount of debris noted on that side of the roof which appears to be impeding roof drainage.

In additional to our observation, PSE performed "sounding" of the walls around the perimeter of the reservoir. Sounding is a process of tapping the reservoir's exterior surface to listen to the report of the hammer strike. A sharp "ping" sound is generally indicative of a competent shotcrete layer while a dull or hollow "thud" sound can be indicative of delamination within the shotcrete layer. A driving force behind delamination is typically water infiltration between the corewall and shotcrete and subsequent freeze-thaw which not only causes the delamination but also makes these areas more suspectable to future water infiltration and delamination. Due to this, where efflorescence or alligator cracking issues were noted, these areas were specifically targeted for sounding checks. However, no major issues were identified during our sounding. It should be noted that a significant portion of the reservoir walls are out of reach and could not be sounded with a hammer. Therefore, other locations of delamination could be present which were not identified during PSE's evaluation.

The visible portions of the interior walls of the reservoir were observed and no major issue were identified above the waterline. The piping inlet and outlet were observed through the water and appeared in line with the drawings. No overflow pipe was observed which was also consistent with the drawings.

Appurtenances and Baffle System: The interior roof and upper baffle were observed while floating the interior of the reservoir. Overall no major visible areas of structural concern were noted. Minor cracking and efflorescence was noted at a variety of locations along the interior roof slab. Many of these cracks had been previously identified and sealed via epoxy injection and then coated. From what was observed, the crack repairs appeared to remain effective. The interior of the reservoir also includes a support structure, a baffle system, and a chlorine solution diffuser system. While some minor corrosion was noted in these elements, particularly along the baffle anchors, the overall support structure and connections appeared to be in visually good condition.

2.2.1 Visual Condition of Additional Site Structures and Features

As noted above, no additional site structures are discussed in this report.

2.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the reservoir under the current adopted code and standards. Seismic loads were determined using the American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-

10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13, "Wire and Strand Wound Circular Prestressed Concrete Water Tanks" was utilized. Evaluation was based on the provided as-built original construction documents and site visit observations.

2.3.1 Hydrostatic and Gravity Analysis

Roof Slab: The roof slab has a thickness of 10-inches which exceeds the code recommended minimum thickness of 8-inches for the spans provided. The additional thickness helps to resist cracking resulting from potential long-term creep problems. Based on the plans, reinforcing in the roof slab was determined to be sufficient for both flexural strength and crack control for current code requirements.

During the observation some cracking and efflorescence was noted on the underside of the slab and at the edges of some the column drop panels. Overall this cracking appears to be relatively minor but should be monitored as part of routine maintenance to check for further development or associated leaking. Some of these cracks were noted to have already been repaired and a similar repair method can likely be employed for the remaining cracks.

As noted above, the reservoir does not have an overflow pipe installed. Currently, the water level in the reservoir is run to a level consistent with the zone's water level. This level is below the base of the roof in the 21-foot range. However, if the water level exceeded the wall height, then water pressure loads would be exerted on the roof. The roof system is design for downward gravity loads, not upward pressure loads. This water pressure load could cause damage to and/or fail the roof system. Depending on the City's overall system redundancy and the maximum possible operating level of the zone, it is recommended that an overflow system be installed as a redundancy measure and to meet the requirements of AWWA D110-13 Section 3.11.2.1. Should any system issues occur which cause the Whatcom Falls 2 reservoir be filled faster than it can drain, an overflow would help to prevent the resulting water pressure loads on the roof.

Vertical Wall Reinforcement: The reservoir as-built drawings show vertical prestressing within the walls spaced at 3'-9-15/16" on center. While PSE could not observe the vertical prestressing directly, we did observe that the location of some of the visible shear cans in the roof slab roughly matched this spacing. Per analysis, the size and spacing of the prestressing was determined to meet current code requirements.

Columns: Per the original as-built drawings, the columns were found to be suitable for anticipated design loads when analyzed under current code requirements. During the site visit no visible defects or signs that would indicate any deficiencies were observed in the above-water portions of the columns that PSE evaluated. Per analysis the columns meet current code requirements for static load resistance. See Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings were evaluated based on information obtained from the provided as-built drawings. The design was found to be adequate for current codes. Per the Geotechnical report the allowable bearing capacity for the site was given as 3,500-psf. Based on PSE's design checks, this bearing capacity is adequate for a maximum design soil bearing pressure of approximately 2,900-psf, which occurs at an operating level of up to 22-feet.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Joints: Per the drawings there are 288 sets of seismic cables (two sets per vertical prestressing bar). Based upon PSE's analysis it appears that the provided number of cables meet the current code requirements and provides the necessary lateral capacity for an operating level of up to 22-feet. Additionally, the underlaying foundation elements are adequate and soil bearing capacities are not exceeded.

Strand Wrap: The circumferential prestressing requirements and wrapping diagram for the reservoir are detailed on the prestressing load distribution diagram on Drawing S-8 of the original drawings dated May 1993. Based on PSE's evaluation under current code requirements, the circumferential prestressing has sufficient capacity for the anticipated static and seismic loads resulting from an operating level up to 22-feet.

Columns: The exterior roof edge is supported by a continuous bearing pad on top of the wall and is restrained by shear cans which limit the potential seismic deflection of the roof and columns to a maximum of 0.125-inches. The existing columns were determined to be adequate to resist the seismic and eccentric loads induced by this 0.125-inch deflection. Therefore, the columns are adequate to resist both seismic and static loads at the maximum operating level of up to 22-feet, which is based on current code requirements.

Freeboard/Slosh: A slosh wave results from seismic movement of the reservoir which causes an excitation of the stored water. Freeboard is intended to provide space for this wave allowing it to dissipate rather than impact structural systems such as the roof, hatches, and vents. This reservoir has a few unique features that complicate slosh and freeboard considerations. The first is that there is no overflow; technically the reservoir can be operated up to the bottom of the roof resulting in a "no freeboard" condition. Secondly, it has a multitude of ceiling mounted systems associated with the chlorine solution diffuser system. These pipes are not protected or supported to resist a slosh impact wave. Finally, the baffle system, which would also see an impact from a slosh wave, can impede the development of a slosh wave as the baffle reduces the wave development length.

Based on PSE's analysis the maximum slosh wave height was determined to be 34-inches high. Per the City, the maximum listed operating level is 21-feet and with a roof height of 24-feet, this results in freeboard of 36-inches which is sufficient to ensure the slosh wave would not impact the roof. However, the chlorine piping system is hung lower than 24-feet, meaning this system could be impacted by water movement. When accounting for the baffles, the wave development length is reduced from 350-feet (the diameter of the reservoir) to approximately 35-feet (the average spacing between the baffles). This was found to only result in a minimal improvement, reducing the slosh height down to 30-inches. For this height the chlorine piping elements, like those shown in Figure 2-21, would still be susceptible to impact from a slosh wave. Further, even if the piping is located outside the slosh impact zone, it was not observed to have any diagonal bracing which would make it able to resist lateral loading and so may still be susceptible to failure in a seismic event independent of a slosh wave impact. While this system is inadequately braced it is unlikely that the piping systems failure would impact the overall structural performance of the reservoir.

PSE evaluated the baffle system for the differential water loads that would be caused by sloshing, as well as for seismic fluid loads. Per product documentation the tensile strength of the baffles is between 2,300 to 2,900-psi, equating to tear strength of about 1,300-lbs per foot. Analysis found that the baffles would likely be subjected to much higher loads in a code level seismic event and could tear. While failure of the baffles would not result in structural issues, they could potentially block the outlet, should they be sucked in. As some baffles are attached to columns, PSE also evaluated the columns for anticipated loads that might be imparted to the columns prior to the reaching the expected failure load on the baffles. Based on PSE's checks the columns have sufficient capacity to handle the anticipated baffle loads up to their expected failure point.

Outside of the chlorination and baffle system, PSE determined the existing roof has the requisite structural capacity to resist a slosh impact wave resulting from operating at a 22-foot operating level. Further, at a 22-foot operating level, the roof hatches and vents would be out of the impact zone as they are installed in raised curbs.

2.4 Summary

The Whatcom Falls 2 Reservoir was constructed in 1993 and appears to be in substantial conformance to current design codes and standards. The reservoir's maximum operating level is listed as 22-feet in the drawings and the design evaluation determined the reservoir was adequate for loads at this operating level. At the current typical maximum operating level of 21-feet, the reservoir is adequate for the expected seismic loads. Additionally, the overall body of the structure including the roof and walls appeared to be in good condition with a few exceptions.

Primary issues noted during the evaluation included ponding around the edges of the roof due to the parapet and easily blocked drains. This blockage appeared to be resulting in some leakage through the parapet via the shear can locations. Hatches were also noted to be cracking likely due to inadequate thickness and lack of reinforcing. Internally, the observable portions of the reservoir were in generally good condition, including the chlorination and baffle support systems. PSE's evaluation did find the chlorination system lacked lateral support and may be susceptible to damage in a seismic event. Further, the baffle system is not designed for seismic sloshing loads and could be damage in a seismic event.

The reservoir is not currently operated at a level that would utilize an overflow pipe. However, the overflow pipe is also meant to help account for issues that might overfill a reservoir (blocked drain, etc.). An overflow pipe or relief system should be installed for emergency or atypical operating conditions.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

Primary structural systems of the reservoir were observed to be in good condition. Adequacy of these systems is also supported by PSE's analysis and there are no major upgrade recommendations for the structural systems.

<u>Roof Drainage:</u> Primary issues on the exterior include the ponding issues and hatch cracking which are currently minor but could develop into larger issues with time. For the roof drainage issue, PSE would recommend the installation of additional drainage locations through the parapet. A similar core-drilling

process that has already been implement near the drains could be employed. At locations where staining is visible on the walls, the shear cans in these areas should be investigated to verify the concrete around them is still competent and they should be resealed where needed. Where the tops of shear cans can be observed in the roof, these cans should be coated to prevent any potential infiltration of water around the cans.

Hatch Curbs: Cracked hatch curbs around hatch perimeters should be resealed so as to prevent further infiltration of water. Where there are larger gaps (greater than 1/2-inch) between the curb and hatch inset, concrete in these areas should be chipped away and the curb repaired with high strength non-shrink repair mortar/grout. Further, around the roof access hatch gutter drainage points, these drains should be screened to protect them. This will bring them into compliance with the Washington Department of Health requirements for the sanitary protection of reservoirs.

Chlorination System and Baffle Seismic Bracing: On the interior of the reservoir, the chlorination system appears to be in good condition for gravity loads. However, the chlorination piping is not configured to resist lateral or slosh loads. Based on the layout of the system, it should have both vertical supports and horizontal bracing to strengthen the systems against lateral loads. This upgrade could be accomplished with Unistrut-type bracing as well as additional support columns mounted next to vertical members and mounted to the roof and floor for stability against slosh and fluid loads.

The baffle system itself would likely be damaged due to differential water loads and sloshing action. PSE is unaware of a way to effectively retrofit this type of system and would recommend either testing the existing system to see if has additional capacity or replacing with a system that is design for the anticipated slosh loads.

Overflow Piping: PSE recommends the installation of an overflow pipe or relief system to meet the requirements of AWWA D110-13 Section 3.11.2.1. While the zone might generally be run at a level below the base of the roof, each individual reservoir should be able to vent water as necessary per current code requirements. Overfilling a reservoir can result in damage to the roof since they are not designed to withstand hydrostatic or uplift loads. As no overflow pipe is currently in place, PSE recommends running an overflow pipe through the floor to vent to a suitable location away from the reservoir.

2.6 Scans of Select Construction Documents

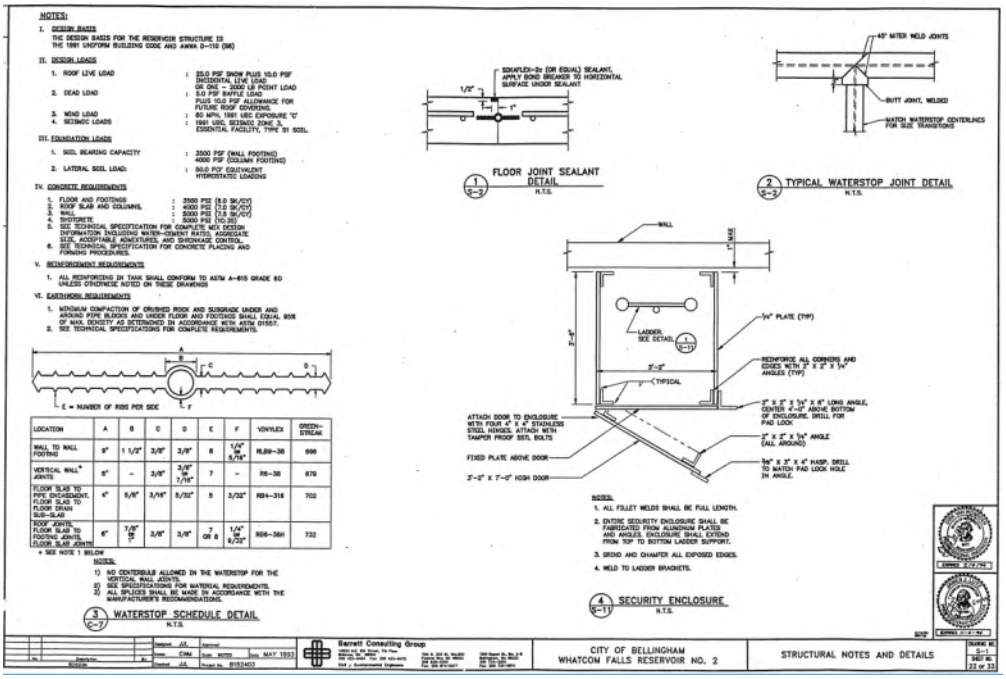


Figure 2-1: Whatcom Falls 2 - Structural Notes and Details

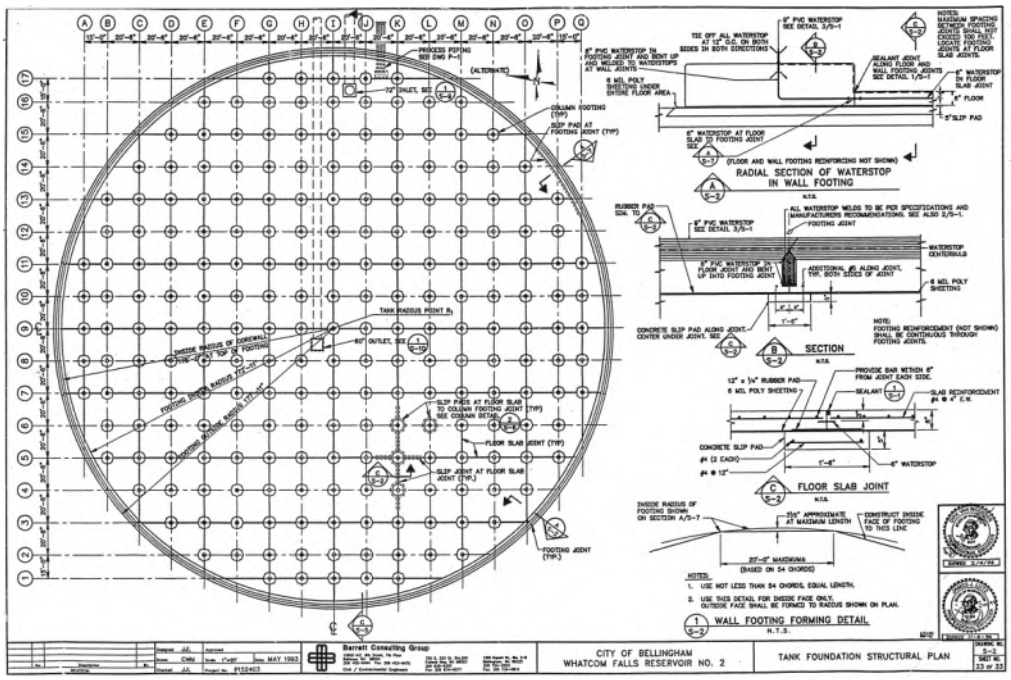


Figure 2-2: Whatcom Falls 2 - Foundation Plan and Details

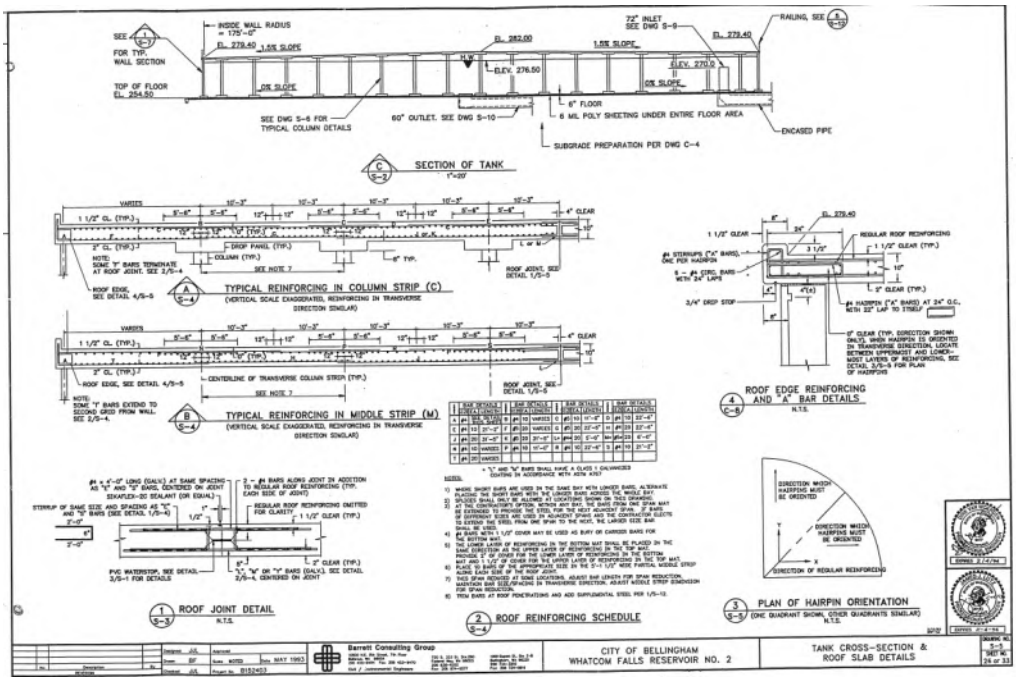


Figure 2-3: Whatcom Falls 2 - Tank and Roof Cross Section and Details

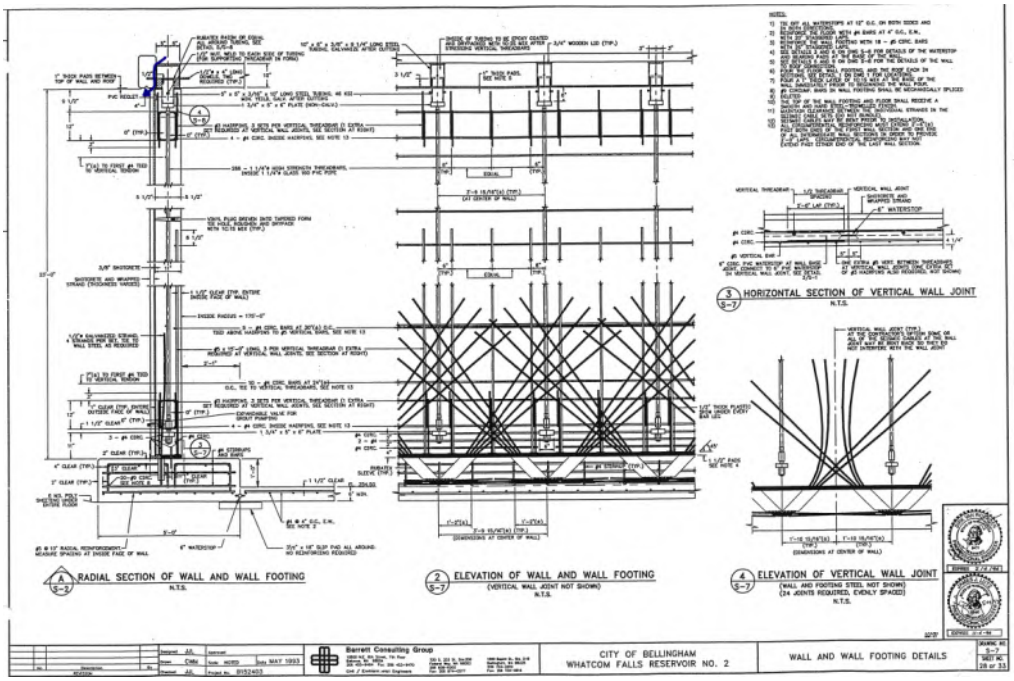


Figure 2-4: Whatcom Falls 2 - Wall Section and Seismic Cable Layout

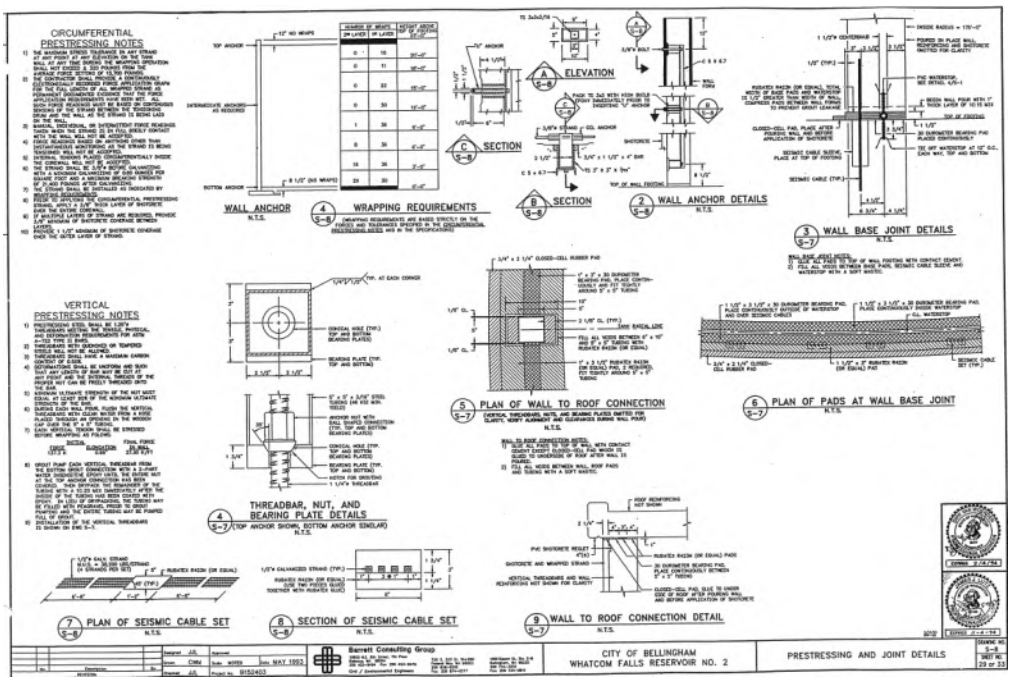


Figure 2-5: Whatcom Falls 2 – Prestressing Information and Joint Details

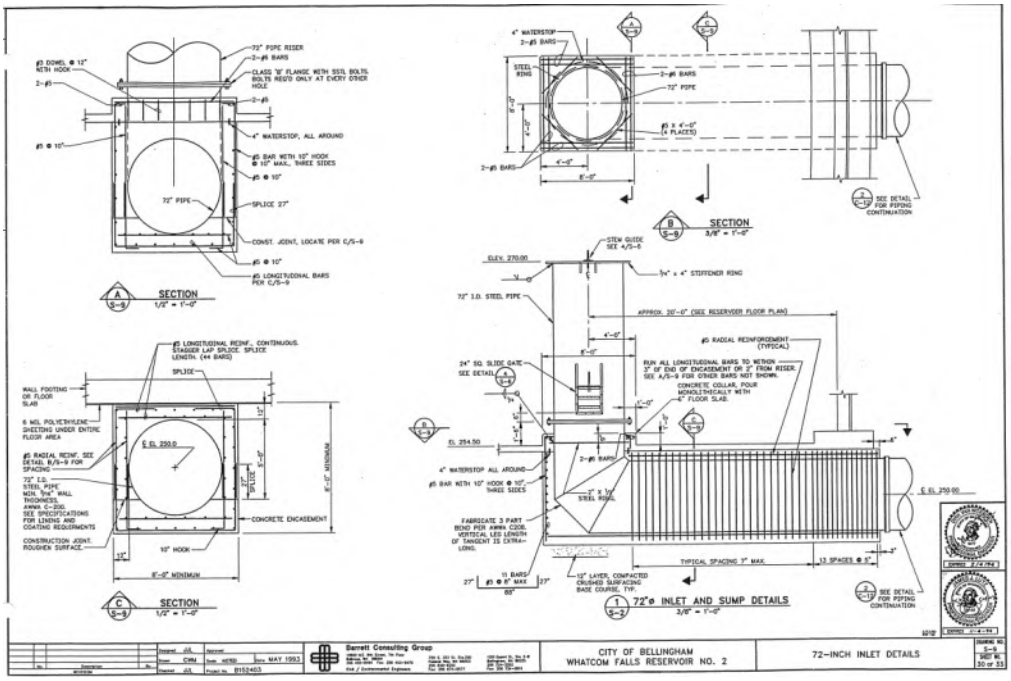


Figure 2-6: Whatcom Falls 2 – Inlet Details

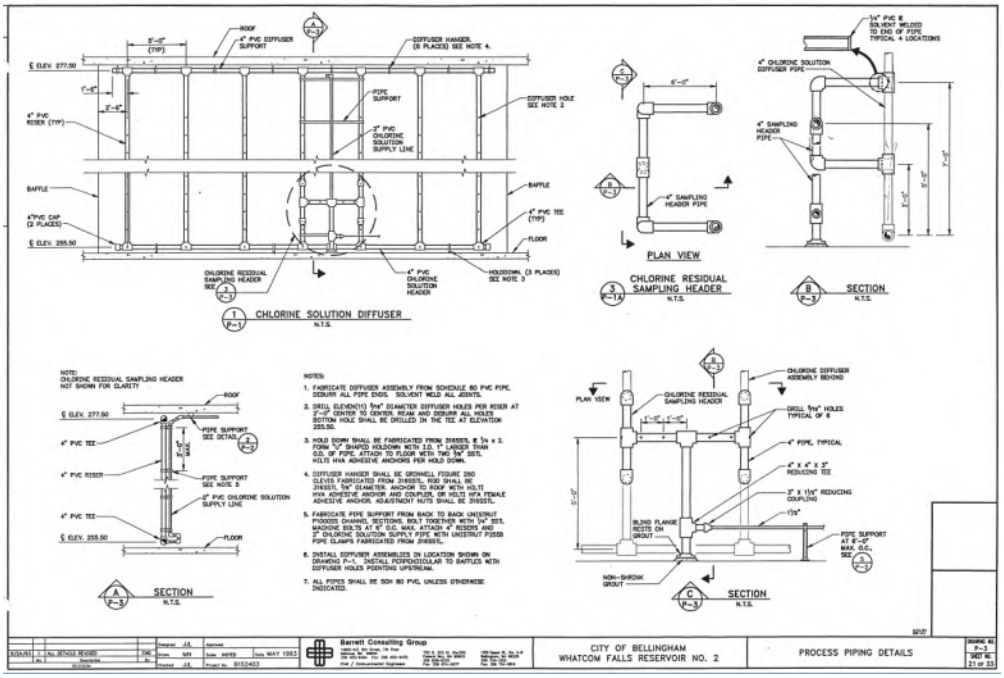


Figure 2-7: Whatcom Falls 2 – Chlorine Solution Diffuser Details

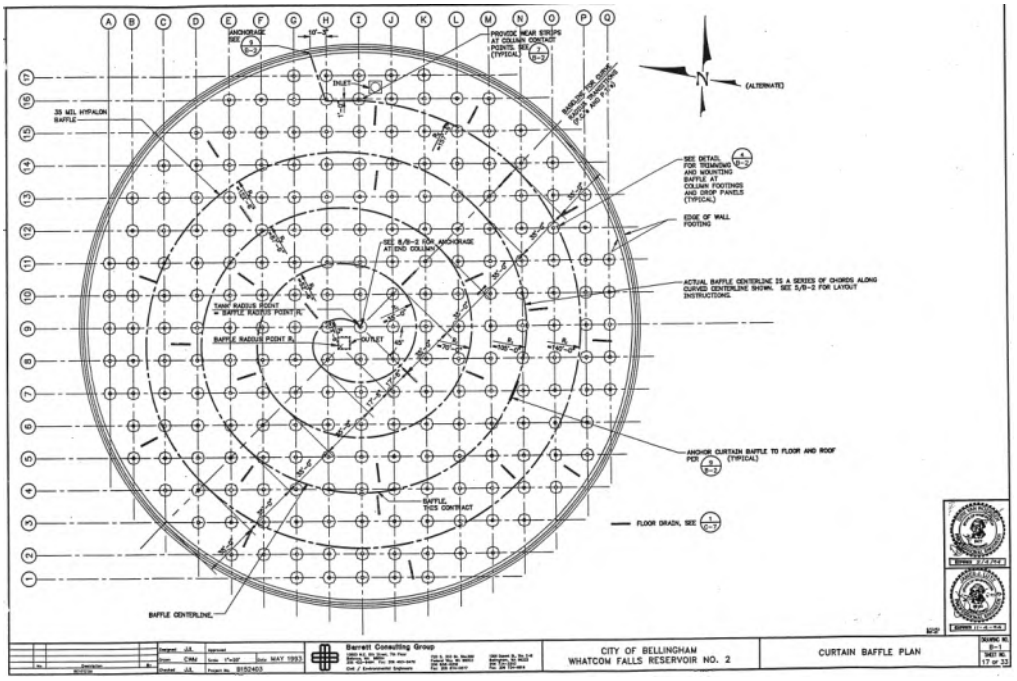


Figure 2-8: Whatcom Falls 2 – Baffle Layout Plan

2.7 Observations Pictures

Figure 2-9: Whatcom Falls 2 - Reservoir South Elevation

Figure 2-10: Whatcom Falls 2 - Reservoir South Elevation & Exterior Ladder

Figure 2-11: Whatcom Falls 2 – Reinforced Roof Drain

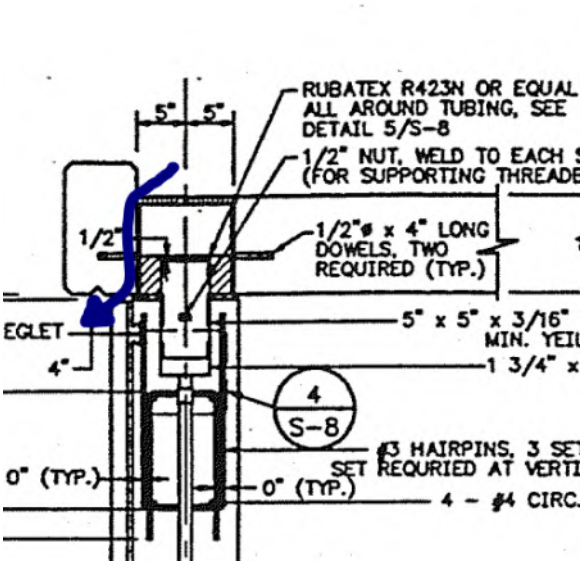
Figure 2-12: Whatcom Falls 2 – Wall Shotcrete (minor alligator cracking observed)

Figure 2-13: Whatcom Falls 2 – Location of Wall Discoloration (likely in line with shear can per Figure 2-15)

Figure 2-14: Whatcom Falls 2 – Blocked Drain and Core Demo'ed Parapet to Facilitate Drainage.

a) Location of Ponding

b) Top of Shear Can, with coating repair



c) Shear can near edge can allow for leakage

d) Water and staining emanating from underside of roof.

Figure 2-15: Whatcom Falls 2 - Potential Leakage Path Resulting from Ponding and Shear Can configuration.

Figure 2-16: Whatcom Falls 2 – 6x6-foot Equipment Hatch.

a) Difference between thicknesses along sides

b) Cracking along thin edge

Figure 2-17: Whatcom Falls 2 - Cracking noted in Hatch Curbs

Figure 2-18: Whatcom Falls 2 - Roof Mounted Solar Panel Array

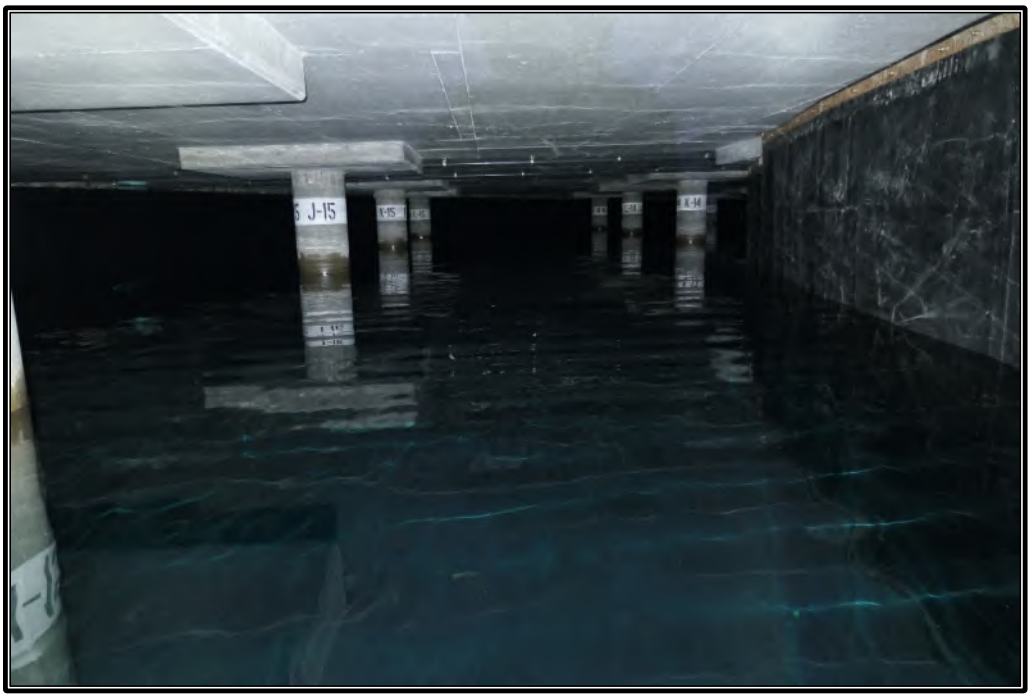


Figure 2-19: Whatcom Falls 2 - Interior Columns

Figure 2-20: Whatcom Falls 2 - Roof Mounted Conduit

Figure 2-21: Whatcom Falls 2 - Components part of Chlorine Diffuser System

Figure 2-22: Whatcom Falls 2 – Baffle Adjacent and Anchored to Column

Figure 2-23: Whatcom Falls 2 – Previous Crack Sealing Work

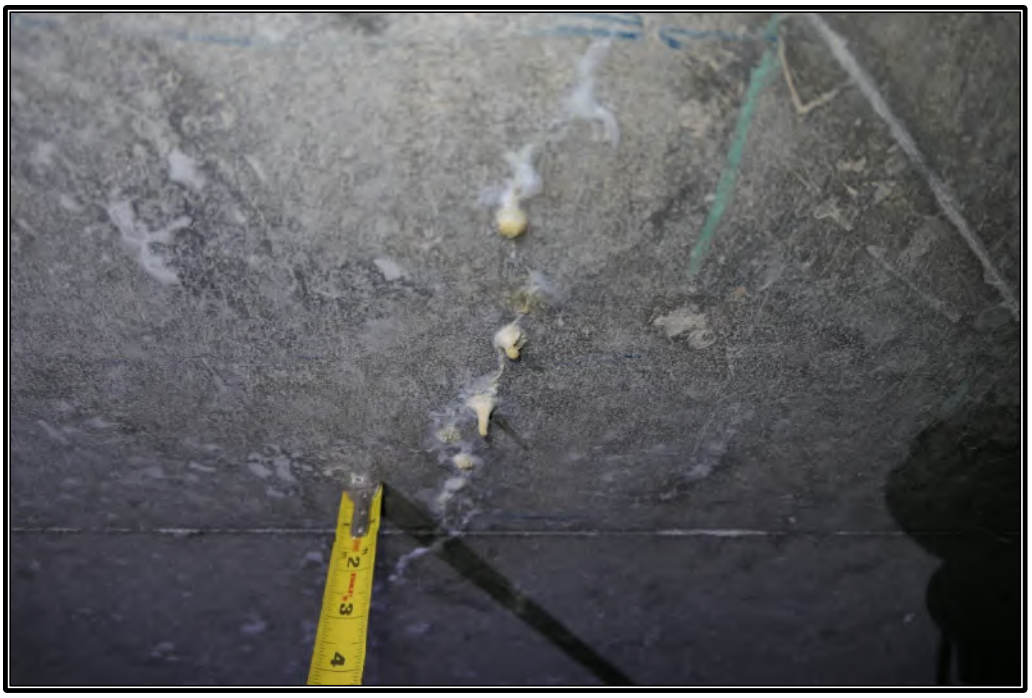


Figure 2-24: Whatcom Falls 2 – Mineralization Seeping through New Crack

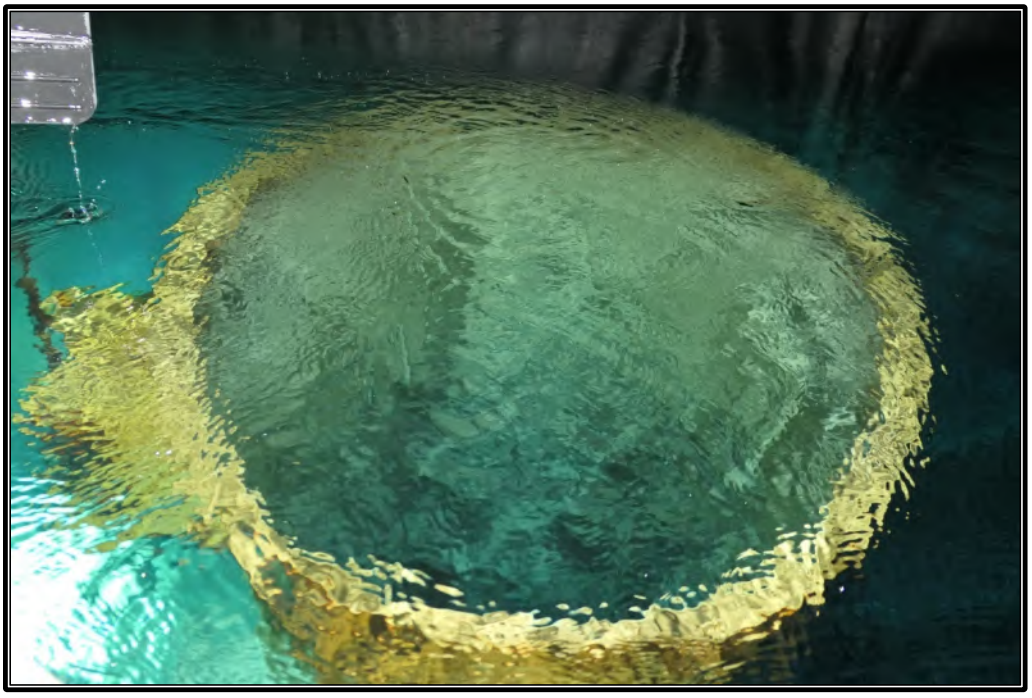


Figure 2-25: Whatcom Falls 2 – Submerged Inlet Pipe

2.8 Field Notes

PRESTRESSED RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Evaluations

PROJECT NUMBER: A1802-0019

PRESTRESSED RESERVOIR SITE INSPECTION (48.7509, -122.4355)

Reservoir Name: Whatcom II - 3201 Aber Ct, Bellingham, WA

Site Visit Date: 1st 6/12/19 2nd 11/7/19 Reservoir Type: Pre-stress Conc. - 15.6 MG

Temperature and weather: 1st Visit Clear Sunny 79°F, 2nd Clear, Sunny 45°F

Site Conditions: Wet, some muddy conditions

PSE Staff: Gray / Nick

Client/Other Staff: Chris, Corey - MSt, City

Exterior Inspection At Ladder

Backfill Dim. to Top of Roof Slab: (N) 20.8' (E) 20.1' (S) 20.2' (W) 21.4'
0 105 210 315

Roof Slab Thickness: / (drawings/measured) Roof Overhang Dimension: / 5.5" with parapet (drawings/measured)

Drip Groove? (Y/N): / Y (drawings/measured) Threadbar Pocket Spacing: / ≅4' (drawings/measured)

Top Surface Roof Slab Condition: General Good. Solar array added.

Has parapet which causes ponding and some other issues w/ cracking a corners but efflorescence minor.

Ladder/Vents/Hatch/Joint Conditions: Ladder - Good, Vent - Good, Joint - Good

Hatches - OK some w/ thin concrete sides have concrete freeze/thaw breaking

Exterior Shotcrete Condition: Gen good with few instances of efflorescence.

Operating Winter 12'-21'
Summer 13'-21'
Avg 16'

PRESTRESSED RESERVOIR SITE INSPECTION

Sound shotcrete at regular intervals and record results: GENERALLY GOOD, A FEW HOLLOW SOUNDING THUDS @ NOTE (5)

Wire/Strand Wrapping Condition: GENERALLY GOOD, NO CRACKING OR EFFLORESCENCE NOTED

Other Comments: ALLIGATOR CRACKING @ A FEW SPOTS, MINOR EFFLORESCENCE, FRESH VERT. STAINS @ NOTE (6) FROM ROOFTOP RUNOFF

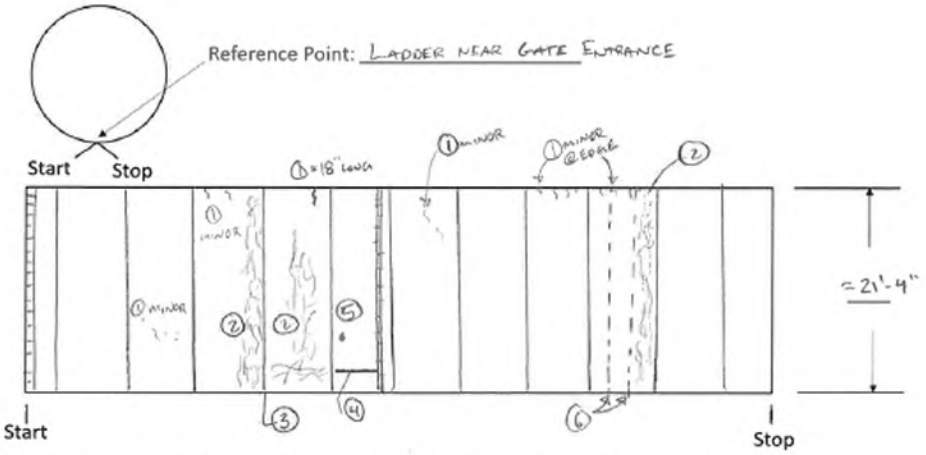
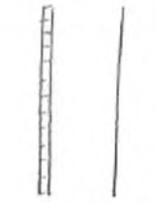


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

KEY:



(1) = EFFLORESCENCE
(2) = ALLIGATOR CRACKING
(3) = DISCOLORATION/EFFLORESCENCE AROUND DRAIN
(4) = HORIZONTAL LIGHT DISCOLORATION ALONG A LINE ROUGHLY 3-4' UP
(5) = DEEP SOUNDING THUD ROUGHLY 6' UP.
(6) = VERTICAL WALL STAIN, APPEARS FRESH & NOT FULL HEIGHT

2

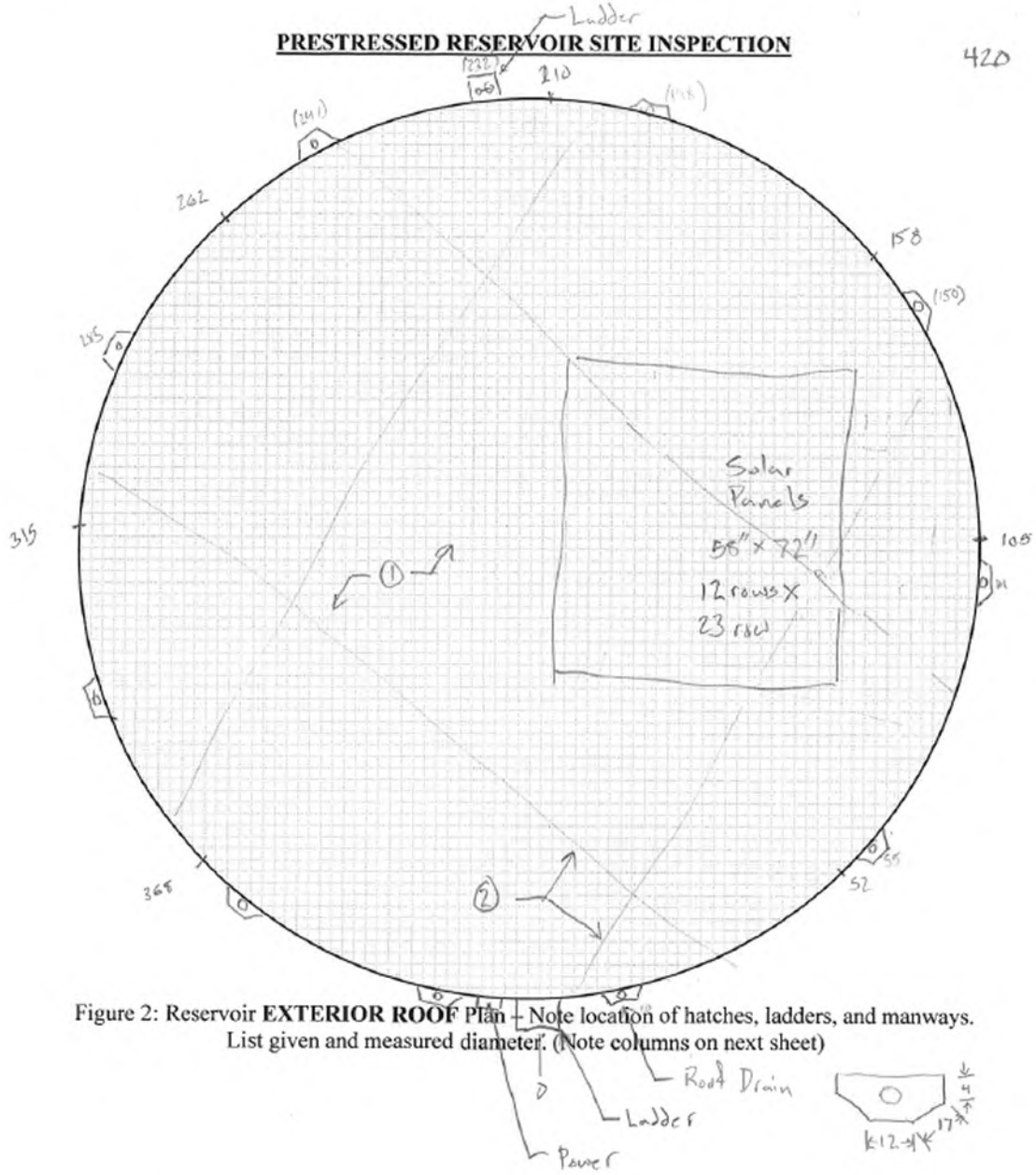


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

① Multiple Vents & Hatches, ref plans for addition loc/size info

② Approx. location of control joints roof is in (9) sections

3

END OF SECTION

Appendix F-4 Whatcom Falls II General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Whatcom Falls II (CT) Reservoir *General Info*

Field Visit Date: 6/12/ & 11/7/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	6/12/ & 11/7/2019
Reservoir Name and Location:	Whatcom Falls II - SSE of 3201 Arbor Court 98229
Inspected by:	Nate Hardy, Corey Poland, Chris Hiatt, Greg Lewis, Nick Welling, Jeremy Hailey
Client Staff Present:	Jenny, Nick
Year Constructed:	1993
Overflow Destination:	N/A
Discharge Destination/Zone:	276 North Zone, Dakin & Yew Pump
Fill Location:	WTP
Reservoir Material:	Pre-Stressed Concrete

Measurement Type	Measurement	Unit
Volume:	16	MG
Diameter (or other dimensions - see notes):	350	ft
Height	22	ft
Overflow Elevation:	276.5	ft AMSL
Bottom Elevation:	254.5	ft AMSL
Level of Overflow	276	ft AMSL
Minimum Normal Operating Level:	13	ft
Maximum Normal Operating Level:	21	ft
Notes: Operating levels are for summer. Winter levels range from 12' to 21'. Overflow via clear well.		

Whatcom Falls II (CT) Reservoir *Exterior Inspection*

Field Visit Date: 6/12/ & 11/7/2019

Exterior Ladder:	
Present on site?	Yes
Material:	Stainless Steel
Condition:	Very Good
Corrosion:	No
Cage:	No
Security Type:	lockable enclosure
Security Condition:	Very Good
Wall Attachment Type:	anchor
Wall Attachment Condition:	Very Good
Rung Shape:	Circular

Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	26	in
Rung Spacing:	12	in
Side Clearance:	4	in
Front Clearance:	12	in
Back Clearance:	31	in
Notes:		

Exterior Fall Prevention System:	
Present at Site:	Yes
Type:	Safe-T-Climb w/ Extension
Fall Protection System Condition:	Very Good
Notes:	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof (Multiple)	
Material:	Aluminum	
Condition:	Fair	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Roof	
Measurement Type	Measurement	Unit
Size:	6x8 (equipment) and 3x3 (access)	ft
Curb Height:	12	in
Notes: High Maintenance design. Cracks with efflorescence on many of the openings.		

Roof Vents and Screen:		
Material:	Galvanized Steel	
Condition:	Very Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	#24	in
Notes: 6 ft total width of upper portion. Bird and bug screens present		

Roof:		
Condition:	Poor	
Roof Sloped:	Yes	
Downspouts:	Yes	
Ponding on Roof:	Yes	
Roof Finish:	smooth	
Slope of roof	0 to 2 degrees	
Measurement Type	Measurement	Unit
Overhang Distance:	6	in
Thickness of roof slab	10	in
Notes: Roof drains do no match design. Missing overflows. No structural issues noted, but drains blocked and significant organic material. Roof slope does not allow roof to drain properly.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Poor	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	4	in
Top Height:	42	in
Notes: Mid-rail height: 28.5 and 15.5". 1.5" ID schedule 40 aluminum pipe. Toe guard is a concrete lip. Rail heights are measured above concrete lip. 8.5in between rails. Some anchors missing nuts.		

Grating:	
Present at Site:	No

Foundation:		
Able to be inspected?	No	
Condition:		
Anchoring Condition:		
Photo of Anchoring System:		
Flexible Couplings at Foundation:	Yes	
Measurement Type	Measurement	Unit
Grade (North)		in
Grade (Other areas)		in
Notes:		

Walls:	
Condition:	Fair
Notes: fence 15 feet from walls. trees 4 ft from fence. Staining on walls. Some minor cracking and efflorescence.	

Exterior Coating	
Exterior Walls:	No Coating
Exterior of Roof:	No Coating
Exterior Piping:	N/A
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A
Notes:	

Whatcom Falls II (CT) Reservoir *Interior Inspection*

Field Visit Date: 6/12/ & 11/7/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Stainless Steel	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	Locked accesses hatches	
Security Condition:	Very Good	
Wall Attachment Type:	Welded	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	26	in
Rung Spacing:	12	in
Side Clearance:	4	in
Front Clearance:	12	in
Back Clearance:	31	in
Notes: Only observed from floating inspection		

Interior Fall Prevention System:	
Present at Site:	Yes
Type:	Safe-T-Climb w/ Extension
Fall Protection System Condition:	Good
Notes:	

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
		ft
Notes: Some corroded elements.		

Columns:		
Present at Site:	Yes	
Material:	Reinforced Concrete	
Condition:	Good	
Measurement Type	Measurement	Unit
Width/Diameter	24	in
Base width	7	ft
Column Spacing/Configuration: 221 evenly spaced spiral-reinforced concrete. Footings are 7' octagon; roof drop panels are 7.5' squares.		

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: Could only view from floating inspection.	

Walls:	
Condition:	Good
Painters Rings Present:	No
Notes: Could only view from floating inspection.	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	Galvanized Steel
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Whatcom Falls II (CT) Reservoir

Miscellaneous

Field Visit Date: 6/12/ & 11/7/2019

Piping		
Inlet Piping:	Size (Inches OD):	72
	Condition:	Good
	Material:	Steel
	Notes: Could only view from floating inspection.	
Outlet Piping:	Size (inches OD):	60
	Condition:	Good
	Material:	Steel
	Lip (Inches)	3
	Notes: Could only view from floating inspection.	
Overflow Piping:	Size (inches OD):	N/A
	Condition:	
	Air Gap:	
	Screened:	
	Material:	
	Outlet Location:	
	Erosion Evident:	
	Screen Condition:	
	Overflow to roof (feet)	
	Notes: Overflow controlled via clear well.	
Drain Piping:	Size (inches OD):	8
	Condition:	N/A
	Outlet Location:	Sanitary Sewer
	Screened:	
	Material:	Ductile Iron
	Silt Stop Type:	
	Air Gap:	Yes
	Screen Condition:	N/A
	Notes: Floor drains are 1.5" Fiberglass grate to 6" DI pipe. Drains go to 8" drain pipe via 90 deg elbow or 8"x6" Tee. Did not observe during floating inspection.	

Piping Facilities		
Exterior Valving:	Type:	GV/BFV
	Condition:	N/A
	Secured:	No
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	NW
	Size (Inches OD):	8
Roof/Wall Piping Penetrations	Sealed:	N/A
	Leaks:	N/A
Notes: Did not observe washdown piping during floating inspection.		

Electrical	
Cathodic Protection:	Yes
Impressed Current:	Yes
Anodes:	Yes
Notes: Constant voltage model	

Other	
Scour Present	N/A
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	Yes
Altitude Valve:	No
Check Valves:	No
Common Inlet/Outlet:	No
Manual Level Indicator:	N/A
Seismic Upgrades:	No
Security Issues:	Yes
Hydraulic Mixing System Type and Mfg.:	High inlet and baffling.
Sediment Build-Up Height Above Floor (in)	Unknown
Water Quality Sample Taps?	Yes
Notes:	

Appendix F-5 Whatcom Falls II Condition Assessment Score Sheet

Whatcom Falls II Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	3	0	No Camera. Evidence of Vandalism
	Vegetation Separation	0	0	0	0	0	0	1	0	
	Site Drainage	0	0	0	0	0	0	3	0	Adjacent soils are wet.
Walls	Exterior Walls	4	4	5	5	5	0	5	0	Staining and minor cracking in shotcrete near roof
	Interior Walls	5	5	5	5	5	0	5	0	
Floor/ Foundation	Foundation	0	0	5	5	0	0	0	0	
	Interior Floor	3	0	5	5	0	0	0	0	Difficult to inspect while floating - appears to be some sediment
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	5	0	0	0	0	
Roof	Exterior Roof	1	4	4	5	4	0	1	0	Significant organic material build up. Ponding/0 degree slope in areas. Exterior spans too long which can exacerbate ponding along the edges
	Interior Roof and Supports	0	0	1	4	0	0	0	0	No overflow which can cause overpressure issues
	Columns	0	0	5	5	0	0	0	0	Roof get hit by slosh wave but can resist loads
Appur-tenances	Exterior Ladders/Fall Protection	0	5	0	0	0	5	5	0	
	Interior Ladders/Fall Protection	0	5	0	0	0	5	5	0	
	Access Hatches	0	3	0	0	3	3	5	0	High maintenance design w/o screens. Curbs not constructed per drawings, causing failure along edges. Need roof hatch railings.
	Railings and Roof Fall Protection	5	4	0	0	0	5	5	0	Missing hardware
	Vents	0	4	0	0	5	0	5	0	Missing hardware and minor corrosion
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	0	0	0	3	0	0	0	0	Drawings do not indicate flexible couplings on these large pipes
	Outlet Piping	0	0	0	3	0	0	0	0	Drawings do not indicate flexible couplings on these large pipes
	Drain Piping	0	0	0	3	0	0	0	0	
	Overflow Piping	0	0	0	0	0	0	0	0	No overflow installed
	Washdown Piping	0	0	0	0	0	0	5	0	
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	0	0	0	0	0	0	5	5	
	Isolation Valving	0	2	0	0	0	0	1	5	Unable to drain reservoir. Corrosion on inlet gate stem.
Misc.	Cathodic Protection System	0	0	0	0	0	0	2	3	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	3	0	0	0	0	Roof supported mixing lines mostly unbraced, may detach in earthquake if slosh impacts
	Categorical Score	3.6	4.0	4.3	4.3	4.4	4.5	3.8	4.3	

Overall Score
4.2

Appendix G 40th Street

Appendix G-1 40th Street Geotechnical Report

GeoEngineers

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs 40th Street Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the 40th Street reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the 40th Street site, located as shown in the Vicinity Map, Figure 1. The 40th Street reservoir is a round reinforced concrete structure with a hopper base.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) maps for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by undifferentiated glacial deposits. Based on our experience in the area, undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift. We have observed Bellingham (glaciomarine) Drift overlying bedrock in this area.

Surface Conditions

The project site is located 400 feet to the east of 40th Street and 700 feet south of Mill Avenue. The reservoir is located on top of a small hill and the site drops off in all directions. Multiple cell towers are located to the northwest. The site is bounded by a wooded area in all directions. A small gravel roadway leads to 40th Street from the northwest.

Subsurface Exploration

Subsurface soil and groundwater conditions for this project were evaluated by completing one new geotechnical boring—B-2 (2019)—on March 22, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 5½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites.

- **Chuckanut Sandstone** – Chuckanut sandstone was encountered at the surface of the exploration. The boring was terminated at 5.5 feet bgs because of refusal on hard bedrock. The fine to coarse grained sandstone was brown with weak cementation and occasional bedding planes.

Groundwater

Groundwater seepage was not observed at the final depth of the boring. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or "pods" at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

The reservoir site is underlain by bedrock from the ground surface. Based on review of the project plans by John W. Cunningham & Associates dated July 1961, the foundation for the reservoir extends into bedrock. Based on the results of our boring and review of the as-builts, the reservoir base is constructed on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped

faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on sandstone which is not at risk of liquefaction.

American Concrete Institute/American Society of Civil Engineers 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (g)	0.96
1-second Period Spectral Response Acceleration, S_1 (g)	0.38
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.399
Seismic design value, S_{DS}	0.652
Seismic design value, S_{D1}	0.359

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_w 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeters per second, g = gravity

M_w 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location.

These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	18	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.40	0.72	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and review of project plans from 1961, the existing reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The existing reservoir includes below grade walls. Our recommendations for evaluating below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section and backfilled with structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the north that is approximately 50 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:tln

Attachments-

Figure 1 – Vicinity Map

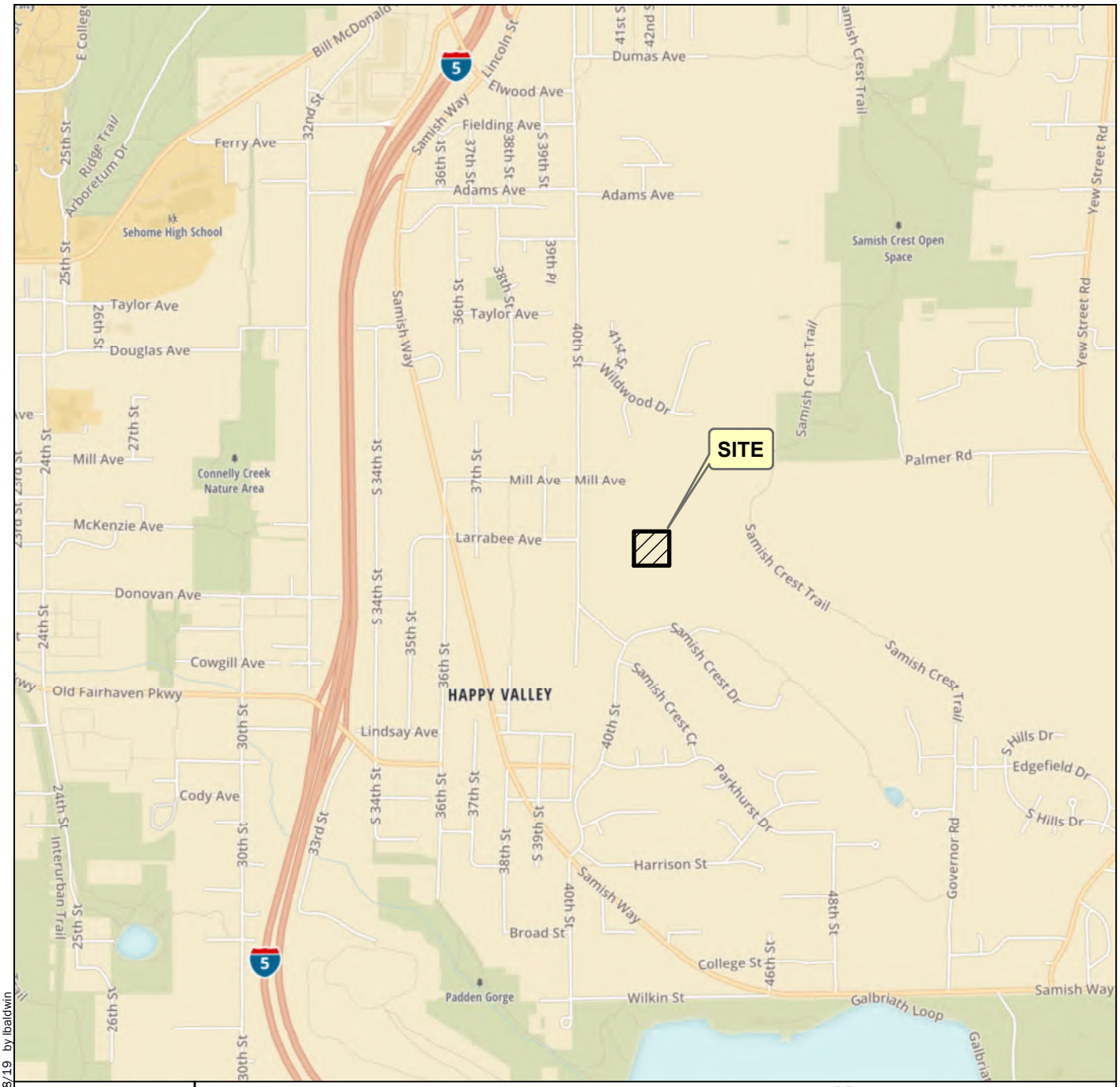
Figure 2 – Site Plan

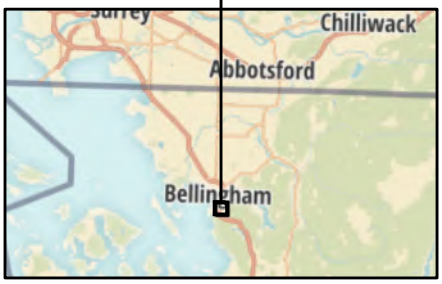
Figure 3 –Key to Exploration Logs

Figure 4 –Log of Boring B-2

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault





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1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

40th Street Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_40thStreet.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

100 0 100

Feet

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W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

40th Street Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

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SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled	Start 3/25/2019	End 3/25/2019	Total Depth (ft) 5.25	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	680 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) Northing (Y)	1247130 630950		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							Sandstone	Brown sandstone (Chuckanut Formation)			
		☒	6	50/4"		1						
675	5	☒	3	50/3"		2						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-2

GeoEngineers

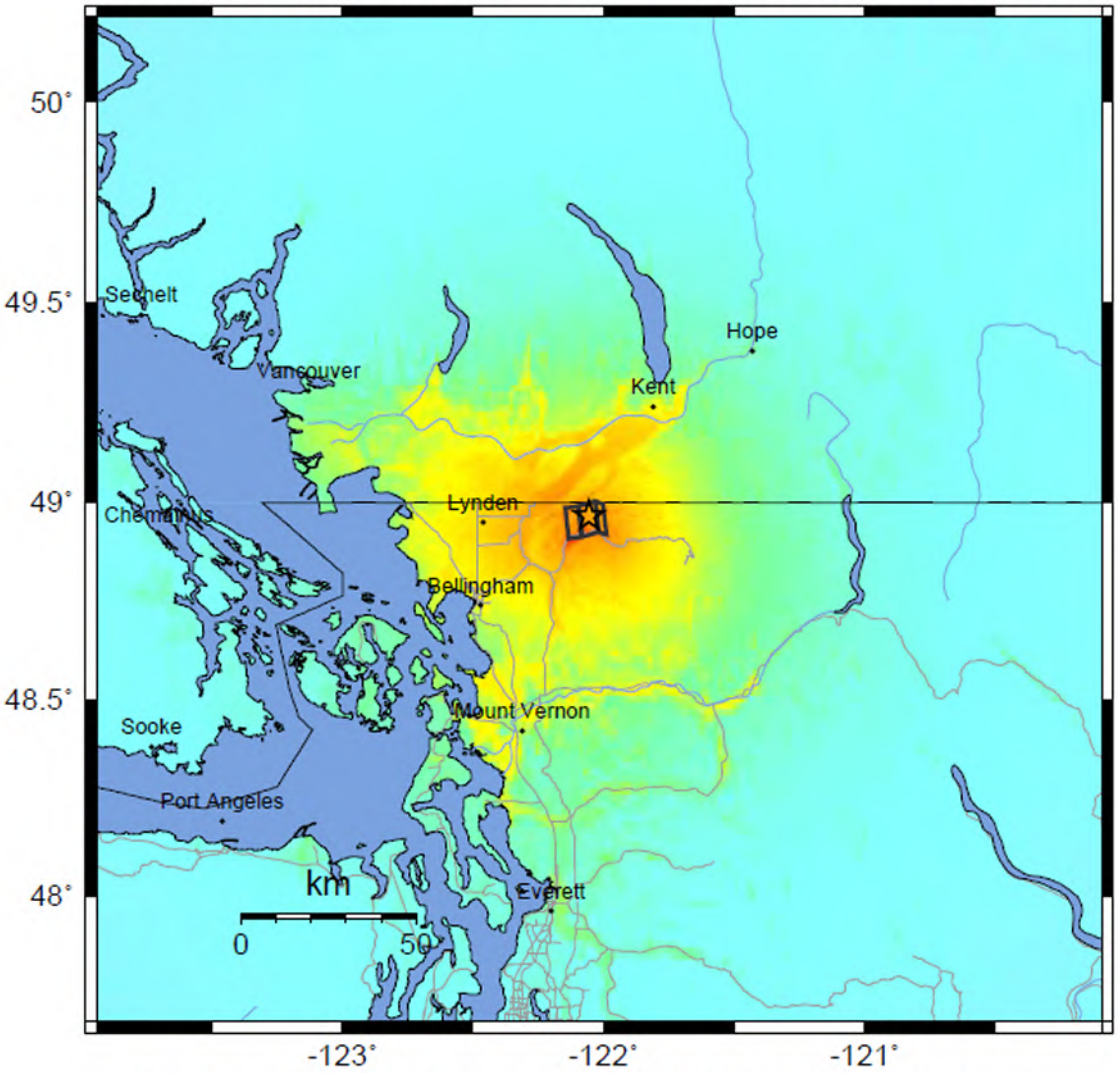
Project: COB Reservoir Inspection and Repair - 40th Street
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

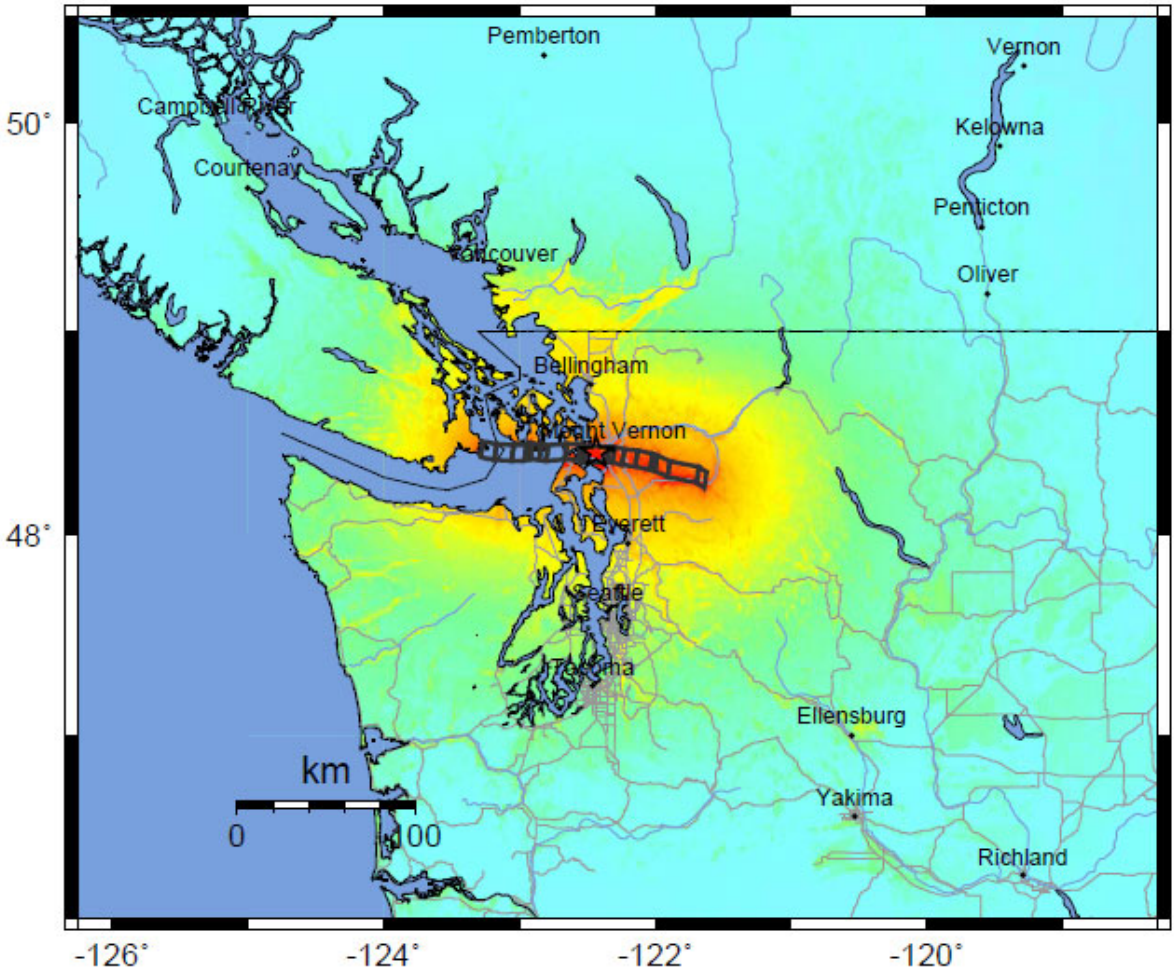
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix G-2 40th Street Corrosion and Coatings Report

June 5, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – 40th Street Concrete Tank Coating Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an internal and external coating evaluation for the City of Bellingham's 40th Street concrete water storage tank. Specific tasks completed during this inspection included:

1. Perform an assessment of both the interior and exterior coatings.
2. Measure interior coating thicknesses at representative locations.
3. Note any corrosion related activity on exposed steel components of the concrete tank.
4. Evaluate coating losses and corrosion on visible surfaces.

BACKGROUND INFORMATION

The 40th Street tank is approximately 60-ft in diameter and 29.5-ft tall (floor to top of roof dome). The exterior roof and interior surfaces including the floor and up to the transition from vertical wall to the domed roof were coated in 2010. The coating material consisted of VersaFlex AquaVers 20 (prime coat) and AquaVers 405 (top coat). This plural component polyurea was applied to a thickness ranging from 80 – 155 mils on the tank interior, as noted during coating application. Broadcast sand was applied to the roof exterior surface during top coat application as a measure of providing a skid-resistant surface.

COATING EVALUATION METHODS

The coating evaluation consisted of an inspection of the visible interior and exterior surfaces and measuring the thickness of the internal lining at multiple locations. As the exterior roof surfaces are rough due to the application of anti-skid, thickness measurements could not be obtained without performing destructive testing. As the coating was in sound condition, destructive testing was not completed.

Dry Film Thickness

The thickness of the interior coating system was measured using a DeFelsko PosiTector Model 6000 electromagnetic dry film thickness gauge (Type 2 gauge) with a model 200 transducer calibrated for polyurea. A dial micrometer was used to measure the thickness of paint chips removed from the exterior tank surface.

INSPECTION RESULTS AND ANALYSIS

Exterior Coating Thickness

Exterior sidewall thickness measurements recorded in sound coating locations ranged from 20 - 40 mils. The reading do fluctuate significantly as the uneven surface makes it difficult to take measurements. Where coating was peeled off, it was possible to measure the chip thickness directly using a micrometer.

Exterior roof thicknesses could not be measured due to the application of anti-skid sand over the whole area. However, thickness measurements during the original coating application were in the 95 – 140 mil range. There was very little coating degradation noted and the existing roof coating thickness is estimated to be in this same range.

Exterior Coating Assessment

The exterior sidewall tank coating has many locations of longitudinal cracking on the surface with efflorescence in the form of calcium carbonate protruding from behind the coating, causing the coating to spall off at these sites. There are also instances of concrete loss at these locations.

The lower 4-inches of the tank has coating disbondment and is missing coating in many locations with moss and other organic material buildup extending from grade up to the first 6-inches of the exposed concrete. The original coating appears to be a three coat system with a beige primer, green intermediate coat and blue overcoat. At a later date, it appears that the sidewalls were covered with a green coating. The top coating is moderately tight and measures 5 – 8 mils thick. A majority of the bug holes have been coated over as opposed to filled during the coating work.

The exterior surfaces, particularly the roof, are very dirty and need to be cleaned in order to discourage continued coating damage.

40th Street – typical exterior surface condition around sidewalls. Cracks in the concrete have led to calcium carbonate accumulation on the surface.

40th Street tank sidewall and eave

Sidewall concrete damage

Green overcoat, blue undercoating has cracking over entire surface, unknown if this is typical

Tank roof surfaces, should be power washed to remove material buildup

In order to help preserve the life of the concrete, the tank exterior sidewalls will need to be cleaned and have all cracks properly sealed. This will extend the life of the existing coating an additional (+/-) 10 years. The roof coating should be cleaned to remove all materials accumulation.

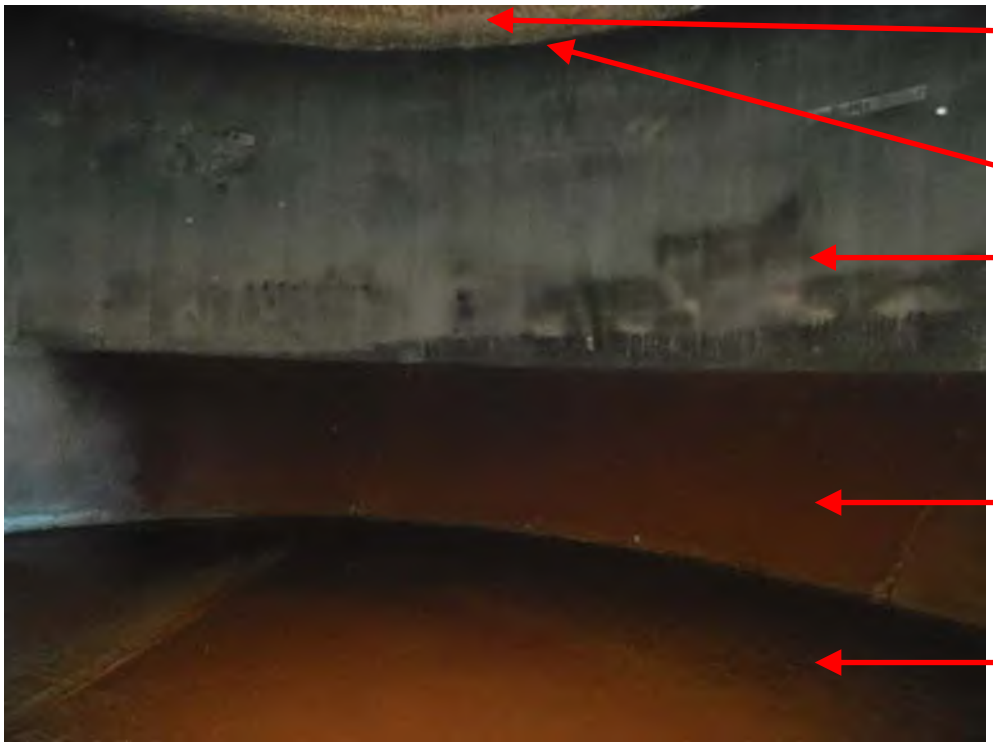
Interior Coating Thickness

Measurements of the interior polyurea coating thickness varied from 80 - 130 mils. This is in agreement with the coating application requirements. The coated surfaces include the floor, sidewall transition, and sidewall up to the domed roof joint.

Interior Coating Assessment

A majority of the floor and wall transition were covered with a thin layer of dirt making it difficult to view much of these surfaces. The entire sidewall area was void of debris and could be inspected from the tank floor.

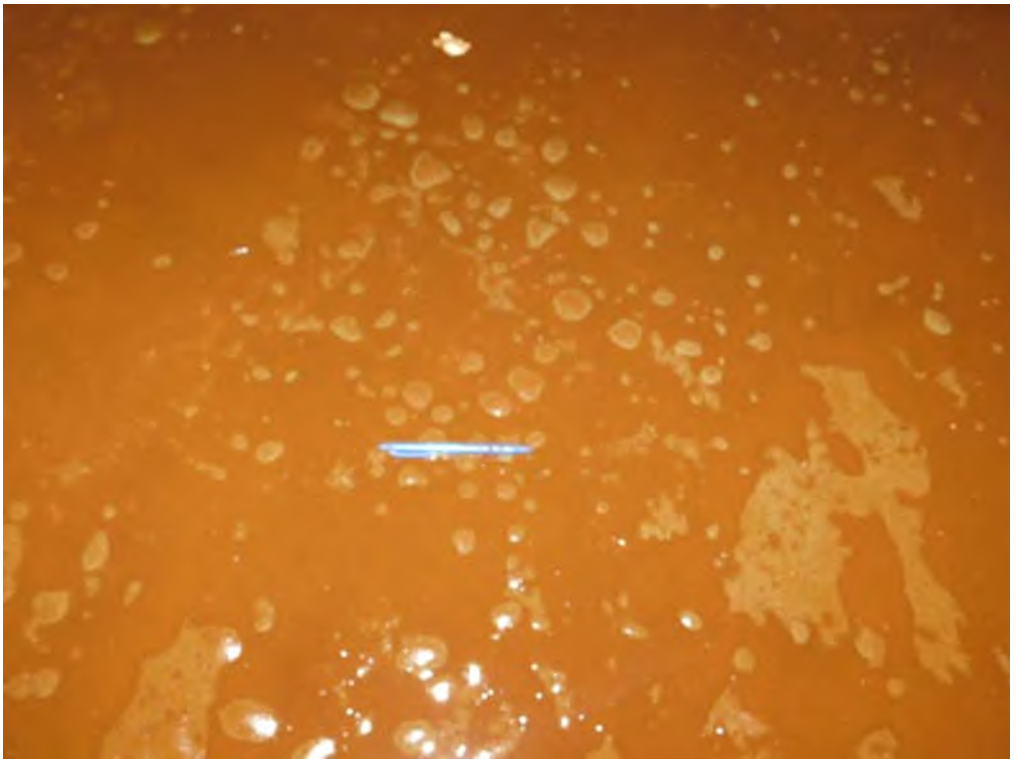
Outlines of the tape seams applied to the floor joints were visible in multiple areas. The tape material is still adhered to the floor surface. There was coating blistering found on the floor and sidewall transition. This may be due to groundwater effects as the tank was properly dry and cooling (to discourage outgassing) when the coating material was applied.



Interior of 40th Street tank

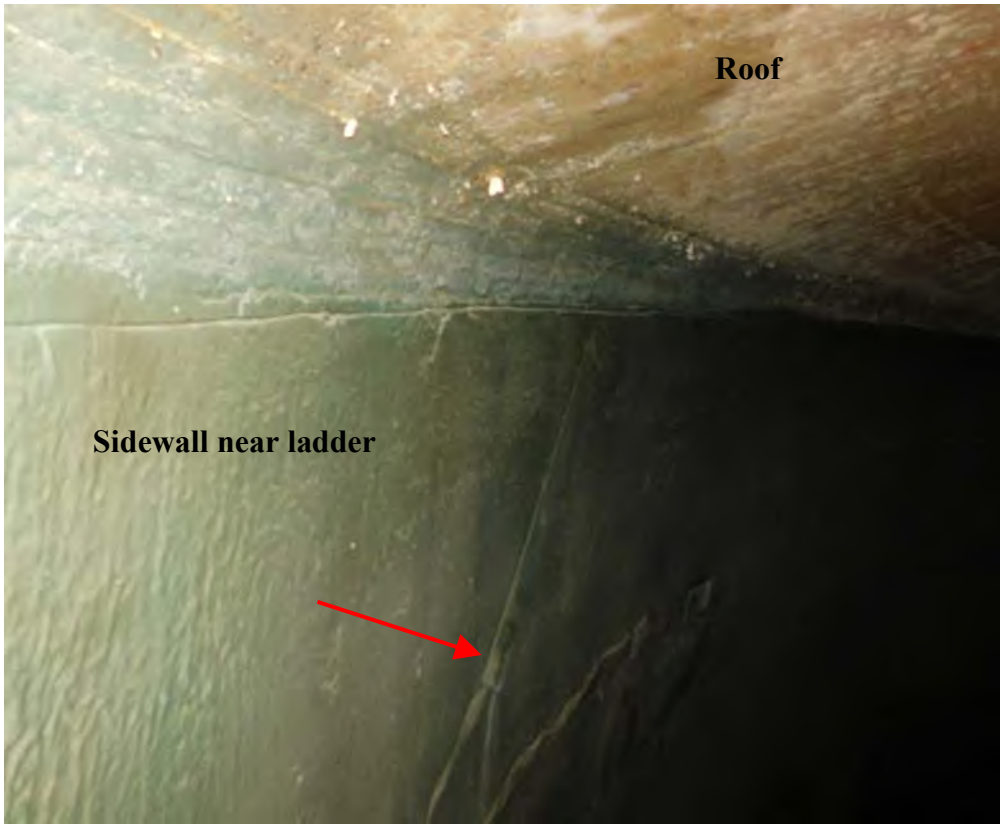
Floor with tape applied over joint seams

Blistering at floor/side wall transition zone



Blisters in floor coating, this site was more heavily blistered than typical

Sidewall coating



Tape repairs on wall, applied during original coating

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the tank.

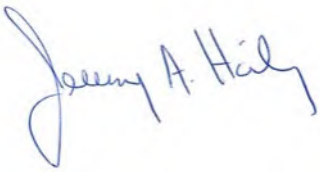
1. There are a large number of cracks on the exterior sidewall surfaces with many areas showing loss of concrete.
2. The bottom 4 – 6 inches around the circumference of the tank exterior has missing coating and moss growth. These areas tend to degrade quicker than other surfaces due to their close proximity to the soil and grass.
3. The exterior sidewall coating has moderate adhesion to the concrete. The top coat is generally adhered well to the undercoating.
4. The roof coating is dirty. There does not appear to be any significant coating damage on the roof.
5. The interior floor and sidewall transition are dirty. Blistering is occurring on approximately 3 – 5% of these locations. The blisters are not cracking.
6. The interior sidewall is in very good condition with no coating delamination.

RECOMMENDATIONS

1. The exterior tank sidewall and roof surfaces need to be pressure washed to remove dirt and other debris.
2. Spot repair areas of exterior sidewall concrete damage and exterior coating loss. This will require the development of appropriate surface preparation and coating application procedures. This should extend the useful life of the coating an additional 10 years.
3. The interior blistering is not a concern at this time. However, increased inspection intervals to every 3 – 5 years should be performed in order to photograph and evaluate if any additional blistering is occurring. This work can be completed by City personnel.

We appreciate the opportunity to work with you on this project. If you have any questions or would like any additional information, please feel free to contact our office.

Sincerely,
Northwest Corrosion Engineering



Jeremy A. Hailey, P.E.

Appendix G-3 40th Street Structural Report

CITY OF BELLINGHAM

CH 9: 40TH ST RESERVOIR

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the 40th St, 0.5 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located near 1318 40th St, Bellingham, WA (Lat. 48.718, Long. -122.463), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed on April 30th, 2019 by Peterson Structural Engineers (PSE), Northwest Corrosion, and Murraysmith, Inc to visually inspect and evaluate the reservoir. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 40th St Round Reinforced Concrete (RC) Reservoir – 0.5 MG

2.1 Description & Background

Per information provided by the City, the 40th St Reservoir was designed by John W. Cunningham & Associates Consulting Engineers and built around 1961. The City provided design drawings for the reservoir. The reservoir has a storage volume of around 0.5MG, a dome roof, and a hopper base. The reservoir is a round reinforced concrete reservoir with a measured interior diameter of 60-feet. The interior wall was measured to be 16.5-feet high and the hopper base 6-feet deep. The overflow weir is located approximately 26.75-feet above the bottom of the reservoir and is located above the top of the wall within the access hatch box. The reservoir uses a portion of the dome roof for its storage volume at full capacity.

Where details or sections could not be directly observed or measured, the original design drawings have been used as a reference. Per these drawings the wall is 10-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. The roof is a reinforced 4.5-inch thick dome with a thickened edge with circumferential hoop reinforcing. The floor is a reinforced 5-inch thick hopper-sided slab that transitions into a 12-inch thick by 10-inch wide footing. Where piping is run under the footing to the valve vault, the drawings show that the piping is encased in an unreinforced concrete block for protection. The reservoir section drawing is shown in Figure 2-1 while a schematic reference drawing denoting the variables used for the analysis is provided in Figure 2-2.

2.1.1 Description of Additional Site Structures and Features

The site includes an attached valve vault which contains equipment associated with the reservoir's operation. This valve vault was constructed as part of the reservoir and the rear of the vault shares a wall and footing with the reservoir. This vault is located on the southwest side of the reservoir and is two levels, with one level located below grade. The vault is 9.5-feet deep by 9.5-feet wide and the main level has an internal height of 8-feet while the lower level is 10-feet high. The lower level is accessed via a 24 by 30-inch opening in the main level's 6-inch thick slab floor. The reservoir's drain, outlet, and overflow are all run through the valve vault. The roof of the vault has a parapet and the air gap for the reservoir overflow is located on the roof. If debris was to clog the overflow, the layout of the top of the valve vault is such that it would fill with water up to the parapet height. While the roof had a secondary drain, it is insufficiently sized to handle any overflow volume. Additionally, as the overflow and drain pipes are joined below the roof line, any material or issue that blocks the overflow would also be likely to clog the drain as well.

2.1 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit to observe the as-built current condition of the reservoir's interior and exterior as well as the general site conditions. The reservoir was drained for our inspection and the site visit was performed April 30th, 2019.

<u>Dome Roof:</u> The reservoir has a self-supporting concrete dome roof with a thickened edge. The surface of the roof is coated and is in generally good visual condition. The southeast side of the roof is over-hung by trees located to the south of the site and the adjacent roof location appears to be collecting tree-litter

and growing moss. The roof coating has a textured non-slip finish, which likely results in an increased retention of tree litter and facilitates the moss growing on the roof. Visually, the dome roof appears to be in good condition with no major cracks or concrete failures observed. Minor cracking may be present but not observed due to the applied coating.

The roof has one 3 by 3-foot access hatch that is part of a larger access box. This box is formed to include the reservoir's overflow weir. The location of the overflow is above the top of the wall necessitating a waterstop in both the roof-to-wall joint and access hatch-to-roof joint per the plans. Overall the hatch appeared to be in good condition with minimal instances of cracking or other structural defects noted.

At the center of the roof, the reservoir has a 36-inch diameter vent. Observable components and the surrounding roof do not appear to have any visual structural issues associated with the vent with the exception of some coating loss. Per the available reference drawings and site observation of the interior of the vent, the vent includes a series of (12) 4-inch by 11-inch openings around its exterior. However, these openings are covered by a sheet-metal cover. Please note, the tightness of the metal cover could pose some issues with the venting requirements and Murraysmith should be consulted to determine if airflow is restricted and how best to mitigate any associated issues. Inadequate venting can create significant structural loads when the reservoir is filled or drained if the vent cannot keep up with the change in storage volume.

Reservoir Walls and Interior: As a majority of the reservoir's main wall is above grade, PSE was able to visually assess a majority of the wall surface. Cracking with efflorescence was noted below the dome roof and at various locations around the wall. Primarily this cracking was noted to occur at about mid-height along the wall in the form of a series of non-continuous cracks banding the reservoir. Additional cracks were noted around the reservoir which appeared to have resulted from various defects or inclusions as a result of the concrete forming. The exterior of the reservoir is coated with three coat system which might obscure any additional minor cracking.

Per the referenced drawings, the walls are 10-inches thick with vertical and horizontal (hoop) reinforcing. At the top of the wall, there is a keyed joint and two (2) vertical bars of reinforcing are located at 12-inches on center around the top of the wall to anchor the dome roof to the wall. This connection is required as the design of the reservoir intends for water to be stored above the top of the wall and within the roofline of the dome. This rigid attachment results in a restraint that that does not accommodate any thermal expansion of the roof. Thermal expansion occurs due to a change in temperature and will cause a material to expand or contract. In this case the roof would be expected to radially expand outwards or contract inwards. For the walls this expansion/contraction results in a thrust load perpendicular to the wall face. This load can cause deflection and cracking in the main body of the wall. There were two key locations of observed cracking: 1) below the dome roof at the top of the wall and 2) circumferentially around the reservoir at the approximate mid-height of the wall. At the first location, the keyed joint at the top of the wall is likely insufficient to resist the thermal thrust force . At the second location, the mid-height cracks were noted to coincide with maximum bending stress resulting from combined thermal contraction and hydrostatic loads. This cracking likely results from both insufficient reinforcing and a lack of backfill which would help restrain the wall against outward deflection. Near the vault, cracking was noted to coincide

with the top of the vault. As the vault itself acts as a brace again wall radial deflection and movement cracking it localized along the stiffness transition associated with the vault roof/wall interface.

The interior of the reservoir was visually found to be in fair condition, although the main walls were covered with a polyurea coating which likely obscured some potential structural defects. Observed near the access hatch, the wall coating was built-up to cover a crack-line. This crack-line runs horizontally approximately 18-feet in length. This patch appeared to coincide with the observed exterior cracking near the vault. Outside of this crack, the remaining exterior cracks did not appear to have corresponding interior observable cracking. The coating terminated above the wall and the dome roof is uncoated. Most of the dome appeared to be in good visual condition with minor cracking and efflorescing. An instance of spalling was observed around the vent opening which had exposed some of the underlying reinforcing.

Appurtenances: The inlet/outlet pipe visually appeared to be in generally good condition, although some small corrosion carbuncles were noted to be developing on the interior of the pipe. The overflow weir is constructed of concrete and located in the access hatch. It was observed to be in good condition along with the associated steel parts including the shear gate and handle.

2.1.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appears to be in generally good visual condition. No visual signs of major cracking or settlement issues were noted. Although it shares a wall-line with the reservoir, visual indications of issues with water infiltration along wall edges were noted to be minimal. Based upon the reference drawings the vault was constructed using a keyed joint along the wall interface and has reinforcing which connects it to the reservoir wall. This joint does not include a waterstop; rather the vault roof-to-reservoir wall joint is built up and sloped away from the joint. For this reservoir, this detail appears to be adequate to prevent water infiltration.

2.2 Structural Analysis

The following design analysis is based on the provided reservoir drawings and field measurements. For elements which could not be observed, such as reinforcing, the drawings were used for reference. Where elements could be observed and found to vary from the design, the actual dimensions were used in PSE's analysis. Based on the results of PSE's analysis, potential issues and retrofit options are discussed.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Circular Concrete Tanks without Prestressing", published 1993 were also utilized.

2.2.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof was evaluated per concrete design criteria as covered in AWWA D110-13 and was found to meet minimum requirements for rise-to-span, thickness versus buckling, and edge reinforcing assuming the reinforcing is consistent with the reference drawings. The dome roof was evaluated at a 23-foot operating level. At this level the water is stored above the top of the wall and there are additional hydrostatic loads for which the roof must be checked. Based on our analysis, the circumferential bars in the roof are sufficient to handle this hydrostatic load.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will allow for differential thermal movement between the reservoir roof and the wall as both components deform differently as a result of temperature variations due to temperature or solar gain. At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the shear resisting structural elements of the reservoir in order to resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement. This is a result of its original design that allows water to be stored above the wall line. To adequately account for thermal movement the operating level of the water would need to be reduced and the existing roof attachment would need to be modified or replaced with a roof system able to accommodate thermal movement. Based on storage needs, this may not be practical or economically feasible.

Wall Reinforcement: Per the design drawings the wall is reinforced with #4 vertical bars at 12-inches on center on the exterior face, #4 bars at 24-inches on center along the interior face. There are additional #5 vertical dowel bars at 12-inches on center at the interior base. The wall also contains horizontal (hoop) reinforcing consisting of #5 bars at 3.5-inches on center near the base decreasing to #5 bars at 6.5-inches on center towards the top of the wall. At the very top of the wall there are two (2) #6 circumferential hoops. This variation in the hoop reinforcing is based on the variable pressure distribution resulting from the hydrostatic fluid load.

Per PSE's analysis, it was determined that the vertical wall reinforcing appears to be under-reinforced for the design loads under current code requirements for flexure. Backchecking at the current operating level, the reservoir was found to have insufficient reinforcing needed for the factored hydrostatic load. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3 sanitary factor. This sanitary factor helps to minimize the potential for cracking and leaks. Per our analysis the outward force exerted on the walls, due to the stored water, results in a hoop tensile force of approximately 32-kips-per-foot at the base. The reservoir's maximum operating level would need to be reduced to 18-feet (8.75-feet below overflow and 5-feet below the current operating level) to meet current structural design criteria and reduce the hoop tensile to acceptable levels. This reduction in height is discussed further in the seismic section as hydrodynamic loads also effect the wall design requirements as they occur concurrently with the hydrostatic loads.

Non-flexural checks for wall at the 23-foot operating level determined the remaining wall reinforcing to be adequate. This included checks for shear loads, for hoop tension forces (when accounting for the larger 1.65 sanitary factor required by code when checking reinforcing in tension), and when considering the compressive wall loads resulting from soil backfill. Finally, per ACI 350.3, the maximum spacing for wall

reinforcing was checked to verify it meets the minimum 12-inches on center spacing. The wall meets this criterion along its exterior face but not along the interior face. In the upper section of the interior wall face, the spacing exceeds this limit with reinforcing located at 24-inches on center.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the geotechnical evaluation, the site's bearing capacity was determined to be 6,000-psf. Using this bearing capacity and checking for the 23-foot operating level up to overflow, the bearing pressure was determined to be within acceptable ranges.

2.2.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: Per PSE's analysis, the addition of seismic loads results in additional forces on the wall of the structure. This is a result of the water slosh wave as well as the mass movement of the structure itself. The wall flexural stresses were increased by about 20% when compared to the static loads at a 23-foot operating level. As the reservoir's vertical reinforcing was already determined to be inadequate for hydrostatic loads, the increased loads resulting from the additional hydrodynamic forces further exceeded the wall's flexural capacity. Per PSE's analysis the reservoir's operating level would need to be reduced 2 additional feet to 16-feet (10.75-feet below overflow and 7-feet below the current operating level) to reduce wall loads to within acceptable limits for seismic loads.

Evaluating the wall for shear, hoop tensile stresses resulting from seismic loads, and compressive soil loads due to seismic forces, the reservoir was found to have adequate seismic capacity when evaluated at the 23-foot operating level.

In addition to the wall flexural and tensile checks listed above, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing to resist seismic lateral loads up to the overflow operating level. No additional reinforcing or connectivity is needed between the walls and the foundation based upon the current configuration.

Freeboard/Slosh: This reservoir is designed to store water above the wall within the reservoir's dome, up to 4.25-feet above the wall. At the current operating level, water is only stored 6-inches above the top of the wall. During a seismic event the resulting from seismic movement would result in a 2.7-foot high slosh wave which would be constrained by the roof. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. At the overflow operating level of 26.75-feet, the force of this wave could potentially damage or even cause a failure of the roof at the roof-to-wall interface and/or of the dome itself. At the current operating level of 23-feet, while there is insufficient freeboard, the roof was still determined to have sufficient capacity to resist the expected slosh load.

Valve Vault: Per the reference drawings the valve vault is connected to the reservoir with #5 rebar dowels at 12-inches on center. This attachment should limit differential movement between the vault and reservoir, or "pounding", that occurs in a seismic event. Additionally, where the hopper base and the lower level of the vault are adjacent, this zone is shown to be backfilled with plain concrete or "trench backfill". In the event of an earthquake, this is expected to provide support to the hopper base to limit its potential to fail the lower level wall of the vault and collapse onto the piping. Of primary

concern is the vault's wall which is cast as part of the reservoir footing. It is recommended that the pipes be retrofitted to have flexible coupling to address potential differential movement between the two structures occur during a seismic event.

2.3 Summary

Based on the available drawings and site visit it appears that some of the structural elements in the reservoir are adequate for the expected loads at the current operating level. However, the wall flexural capacity was found to be inadequate for the reservoir's current operating level. Further, when considering lateral and hydrodynamic loads, the elements are further stressed beyond their code capacity. It was noted that there are a variety of cracks and efflorescence ringing the reservoir that could be a result of the combined thermal and operational loading condition. Based on PSE's analysis, the reservoir's current maximum operating level would need to be drop by 7-feet to meet current code requirements for performance in a seismic event.

Elements outside of the wall, such as the dome roof and footing were determined to be adequate when operated at a 23-foot operating level. However, while reinforcing in these areas were found to be adequate, the dome-to-roof reinforced connection is rigid and unable to resist expansion/contraction loads resulting from thermal effects. This has resulted in some damage to this interface and the actual capacity at this connection is likely lower than our analysis has determined. This could be a potential concern in a seismic event.

Remaining observable components of the reservoir appear to be in fair condition although cracking adjacent to the vault appears to have affected the entire cross-section of the wall with the effects of the cracking observable on the interior and exterior wall faces. Overall, the reservoir's connection to the valve vault still appears to be structurally sound and in good condition. As the vault and pipe support is rigid, the vault may also be a candidate to be upgraded to accommodate for potential vertical and horizontal differential movement between the vault and the reservoir, as might occur during a seismic event.

2.4 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 23-feet, which is 6-inches above the top of the wall. Due to the types of issues noted, these retrofits might not be cost effective or easy to implement.

<u>Wall Flexural Capacity</u>

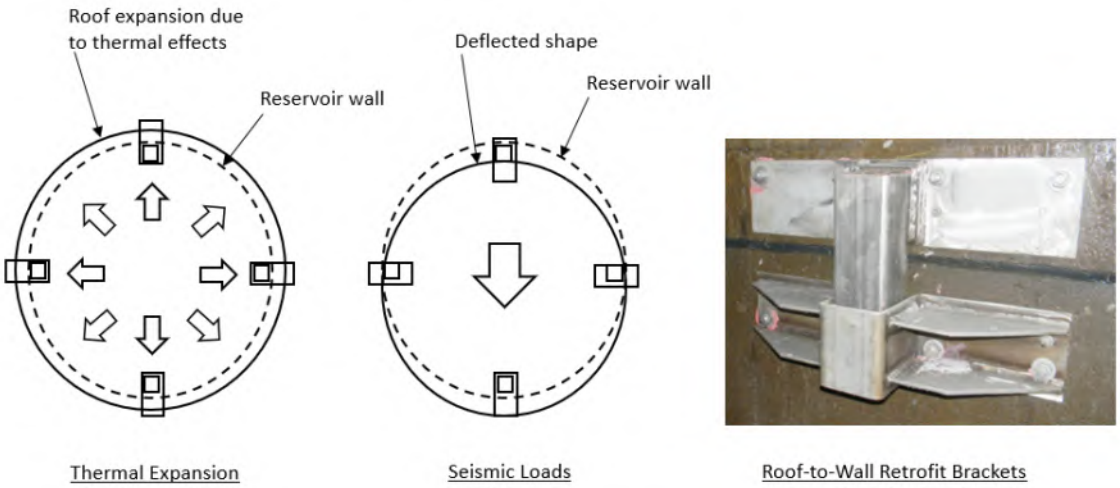
Based on a maximum operating level of 23-feet, PSE determined the wall flexural capacity is inadequate. To accommodate both hydrostatic and hydrodynamic loads, PSE would recommend dropping the water level 7-feet to a maximum operating level of 16-feet. If this operating level is not acceptable, PSE would recommend a testing firm be employed to map the existing reinforcing and test the concrete compressive strength and reinforcing's yield strength to verify the actual values. PSE's current evaluation uses typical design values appropriate for the current era of construction; however, these values could be overly conservative. Based on the gathered information, the reservoir can be re-evaluated to confirm the actual as-built design capacity. Should this updated capacity still not be sufficient, a retrofit using fiberglass

reinforced polymer (FRP) fabric could be implemented to improve the wall's flexural strength in areas determined to be weak.

Roof-to-Wall Retrofit

The effects of thermal movement appear to have damaged the roof-to-wall interface and have likely affected its structural shear capacity and water-tightness. To alleviate the thermal issue, PSE would recommend either the retrofit or replacement of the roof. Note that this may not be economically feasible or practical.

The first option would be to remove the existing roof and replace it with a new roof. In this case the water level would need to be reduced by at least 1.5-feet so that the final water level has a minimum freeboard of 1-foot. Once done, there are a variety of options available. For example, aluminum geodesic dome roofs have been used to either add new roofs or retrofit existing roofs to many different types of reservoirs. As an aluminum dome roof is relatively light, strengthening of the existing walls and foundation would be limited if required at all. Alternately, a new concrete dome or flat roof could be designed and installed.



If the existing dome roof cannot be removed, a retrofit bracket could be installed. An example of this is shown above. This type of bracket is configured to allow for thermal expansion of the roof while restricting lateral movement due to a seismic event. This type of connection would not impart additional operating or thermal loads on the walls. In a seismic event the brackets would "catch" the roof limiting is movement and transferring its lateral load into the walls. This option could potentially be more difficult to implement (versus an aluminum dome roof) as it requires an elastomeric bearing pad to be placed between the roof and the top of the wall. Lifting the roof to install such a pad might not be practical. However, this type of retrofit would allow the current roof to remain without it being demolished. Alternately, retrofit brackets could be installed and the bearing pad omitted with the knowledge that in a seismic event the top of the wall connection could be significantly damaged, but the brackets would retain the dome and help prevent a complete failure of the roof.

General Recommendations

PSE recommends the exterior wall be cleaned and all efflorescence or loose concrete be removed. For damaged concrete in the main body of the wall, these areas can be cleaned, and a non-shrink grout used to fill and patch these areas. For damage due to the thermal expansion effects, these areas can be cleaned but left un-grouted. As thermal movement is likely to continue to occur, PSE does not recommend stiffening or reinforcing this area. By constraining the roof, the failure zone could be moved and potentially cause issues within the dome roof itself if it is constrained against expansion. Rather, the cracking around the base of the dome should be cleaned and coated to protect any reinforcing against water infiltration and to prevent further damage to the concrete. Coatings and any repair medium should be flexible to prevent further cracking during any future thermal movement. This is not a long-term fix but intended to limit the impact of water and corrosion on this area until a new roof or roof-to-wall retrofit solution is selected. Once the area is cleaned and any damaged concrete removed it is recommended that it be observed by a Structural Engineer to review the extent of damaged concrete and to determine if any additional or alternate repairs are advisable at that time.

On the interior of the reservoir, spalled concrete around the vent opening should be removed back to competent material and any corrosion removed. Once cleaned any exposed rebar should be coated to prevent further corrosion and then appropriately patched.

Around the reservoir there are a few trees that are in close proximity to the reservoir walls with branches that overhang the reservoir. These trees should be monitored and trimmed back to limit debris which may collect on the roof and to prevent falling trees or branches that might damage vents or hatch elements.

Finally, the valve vault piping should be retrofitted to ensure the piping has flexible fittings which allow for differential horizontal and lateral movement to occur between the vault and reservoir in a seismic event. As the structure is near the reservoir's foundation, which is cast as part of the vault's wall, there is a potential for settlement or movement at this interface. Notches or overflow scuppers should be installed in the parapet to prevent the roof from overflowing if the drain backs-up.

2.5 Scans of Select Construction Documents

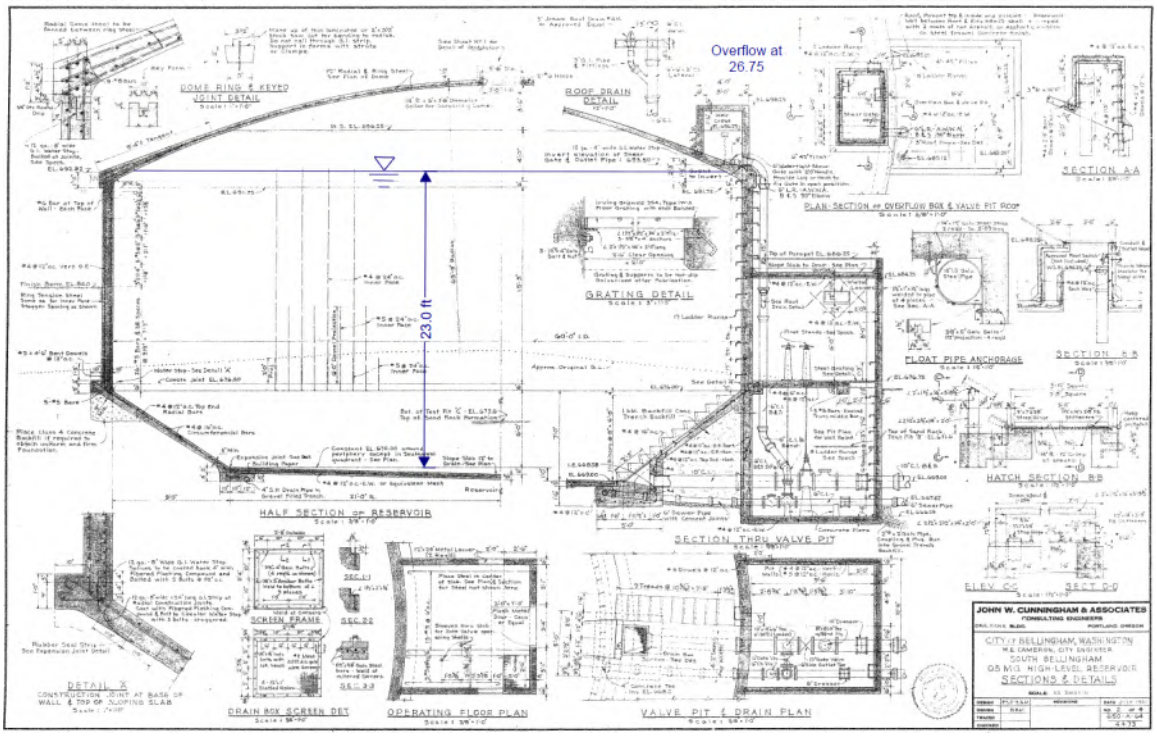


Figure 2-1: 40th St Reservoir Sections and Details

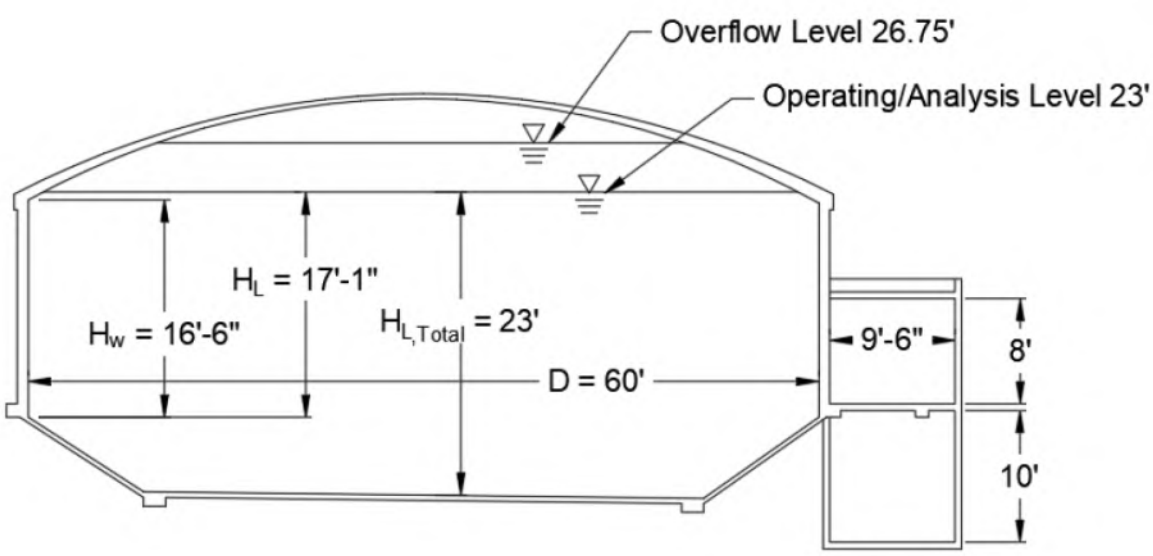


Figure 2-2: 40th St Reservoir Elevations Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, $H_{L,Total}$ = Total Operating Water Height relative to Base)

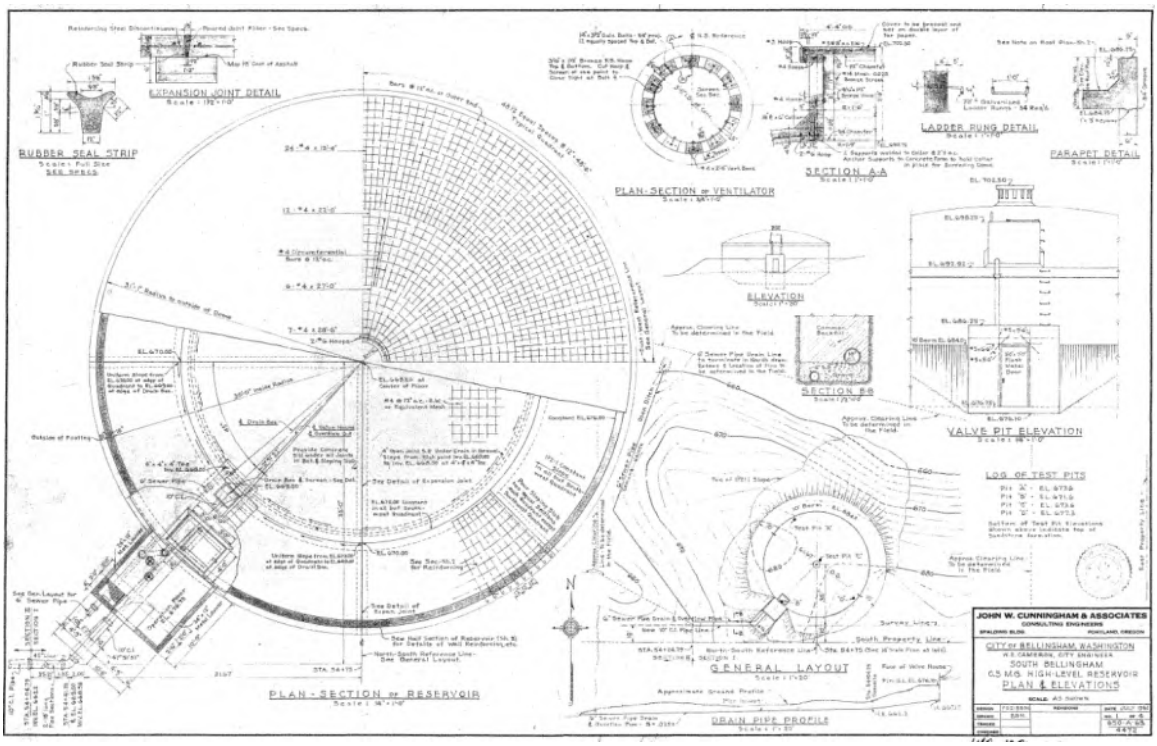


Figure 2-3: 40th St Reservoir Plan and Elevations

2.6 Observations Pictures

Figure 2-4: 40th St Reservoir – Elevation

Figure 2-5: 40th St Reservoir – Entry to Valve Vault

Figure 2-6: 40th St Reservoir – Circumferential Cracking around Reservoir below Roof Line and along Wall

Figure 2-7: 40th St Reservoir – Cracking adjacent to Valve Vault

Figure 2-8: 40th St Reservoir – Rock Pocket and Associated Failure and Cracking

Figure 2-9: 40th St Reservoir – Dome Roof Vent

Figure 2-10: 40th St Reservoir – Moss on Roof in southwest side of roof overhung by trees

Figure 2-11: 40th St Reservoir – Steps Cast into Hopper Base and Inlet/Outlet

Figure 2-12: 40th St Reservoir – Coating Patchwork Over Interior Crack adjacent to Valve Vault

Figure 2-13: 40th St Reservoir – Dome Roof Interior Side of Vent – Concrete Spall noted around perimeter.

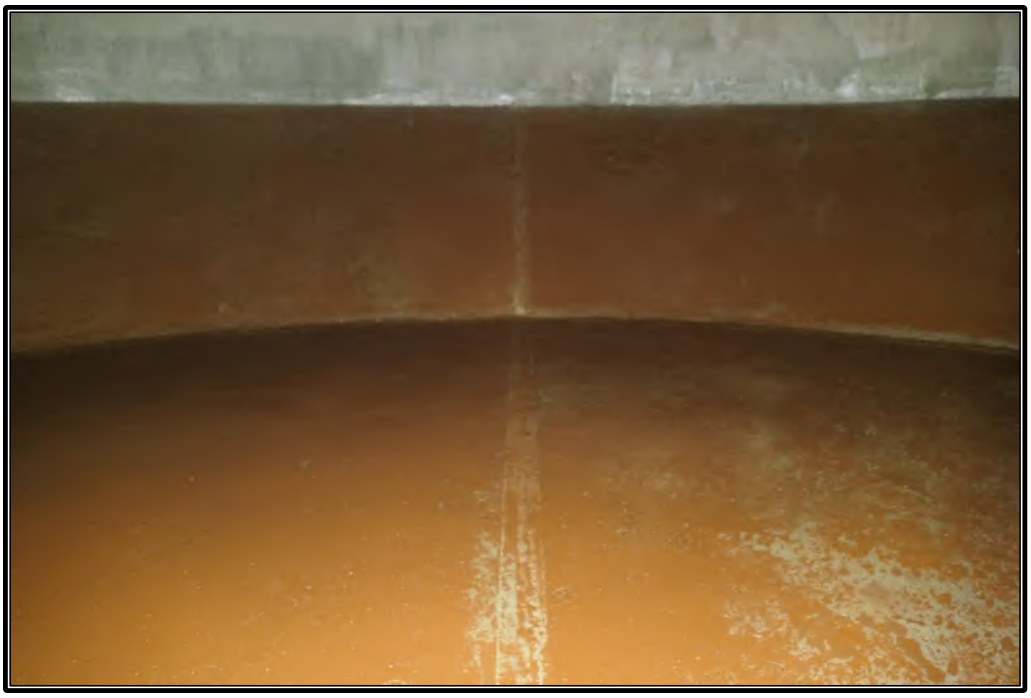


Figure 2-14: 40th St Reservoir – Reservoir Floor and Hopper Base

Figure 2-15: 40th St Reservoir – Reservoir Floor Coating Joint

Figure 2-16: 40th St Reservoir – Access Hatch

Figure 2-17: 40th St Reservoir – Overflow Weir Gate and Shear Gate Handle - Located in Access Hatch

2.7 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Evaluation

PROJECT NUMBER: A1802-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: 40th St Reservoir Access 1318 40th St, Bellingham 98229 Lat 48.7182, Long -122.4625

Site Visit Date: 4/30 Reservoir Type: Round RC w/ Dome Roof & Hopper Base

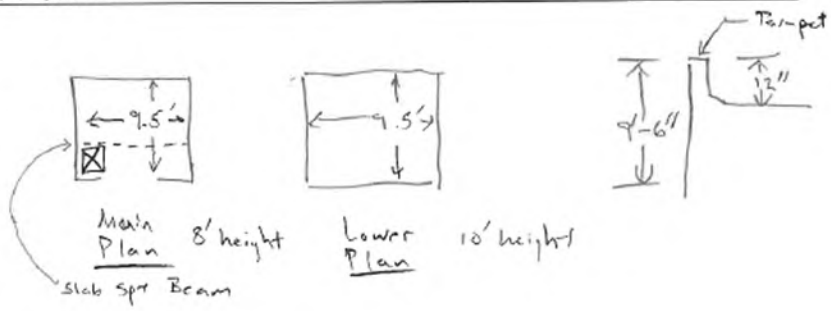
Temperature and weather: Clear, Sunny, 41°F

Site Conditions: Dry, hilly, slope South west

PSE Staff: Greg Lewis

Client/Other Staff: Nate, Coroy (Murraysmith); Jeremy (NW Corrosion) City Personel

Vault



2 vent ea. side 22"x10"
1 door 3'x7'
1 hatch in floor 24"x30"
Slab thickness 6" & beam 12"x12"
Overall clean and in good condition, Some efflorescence in base along wall

1

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to ~~Top~~ Base of Roof Slab: (~~N~~) 1 14.5' (~~E~~) 2 14' (~~S~~) 3 11.8' (~~W~~) 4 9.7'

Roof Slab Thickness: 4.5" / 18" @ edge (drawings/measured) Roof Overhang Dimension: 3" / 3" (drawings/measured)

Drip Groove? (Y/N): Y / Y (1" wide) (drawings/measured)

Top Surface Roof Slab Condition: Good, some moss growth on E side, no cracks but could be because coating covers

Ladder/Vents/Hatch/Joint Conditions: New ladder: 1½" ø rung, 12" o.c., 20" wide

Other Comments:

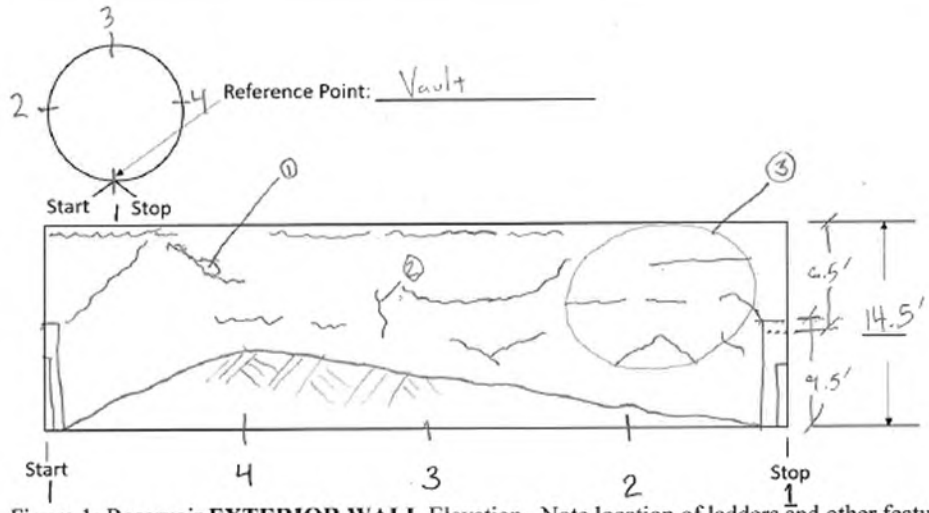


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

3. 12' from fence, 4. 12' from fence, 1. & 2. open towards gate

① Rock Pocket
② Vert. Efflorescence along joint line
③ Cracks in this zone have large amount of efflorescence relative to other cracks

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2

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

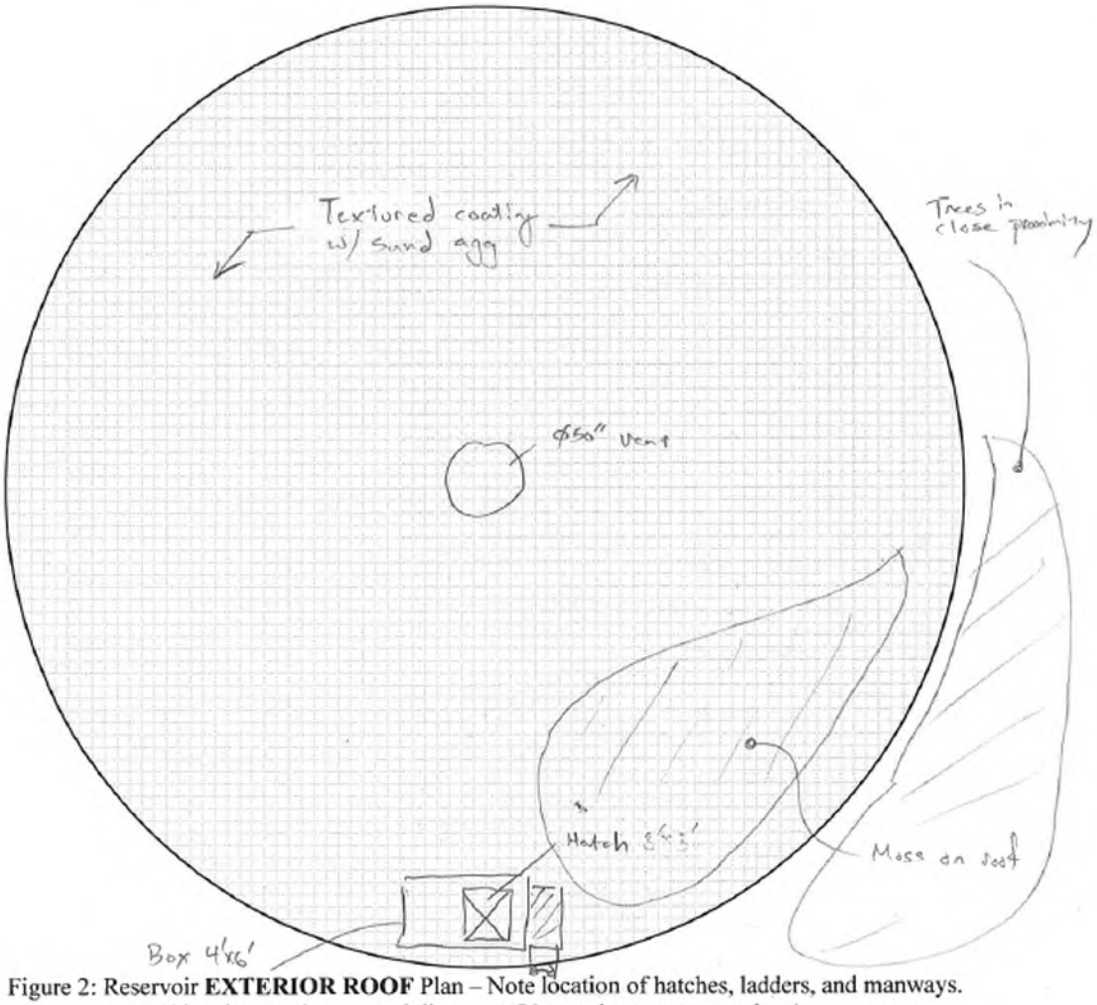

Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

No cracking noted, likely due to coating on roof.

3

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Good, spall noted around vent opening w/ corroded bar. Some pieces of conc. on floor (very small)

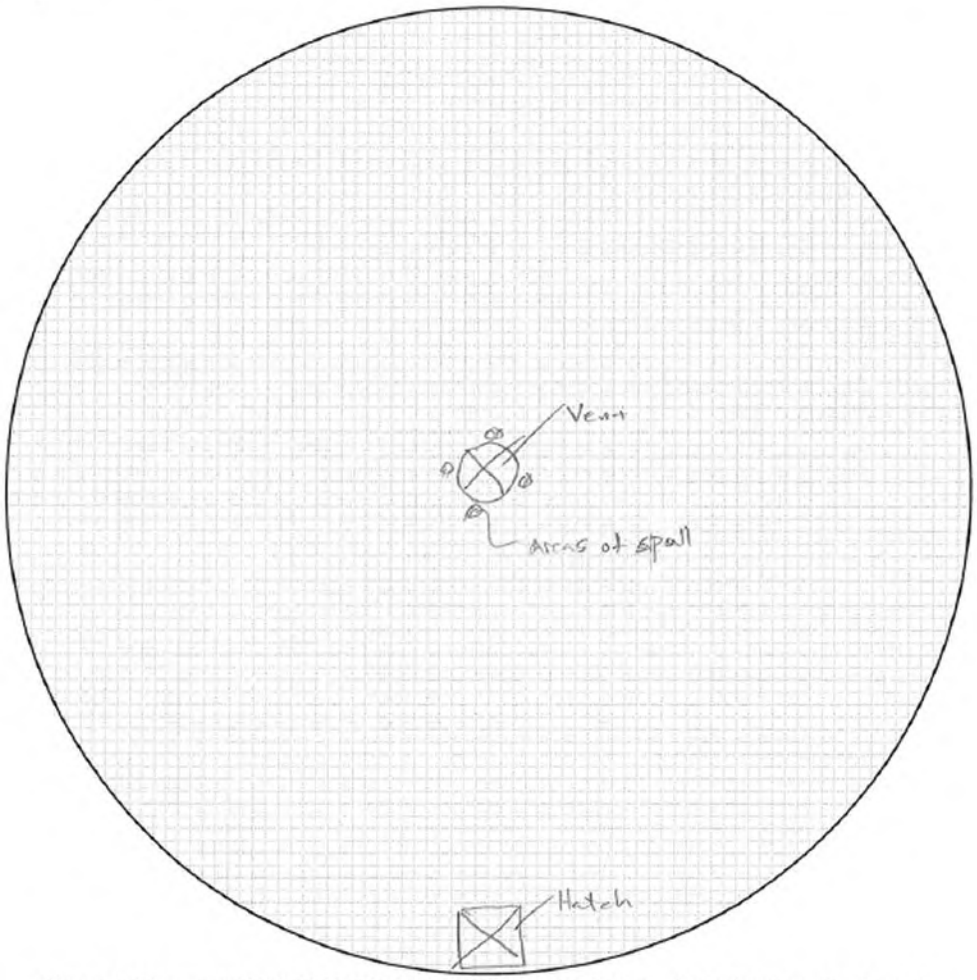

Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Column Diameter: N/A / None (drawings/measured) Footing Size/Thickness: 10"×12" / Not seen (drawings/measured)

Column Spacing: N/A / N/A (drawings/measured) Wall Curb Dimensions: N/A / None (drawings/measured)

Floor Slab Condition: Coated floor. No cracking visible. Coating is bubbling in some areas

Floor Slab Joints Spacing/Condition: Joint divide slab into quads. Joints coated and filled.

Column/Footing Conditions: N/A

5

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

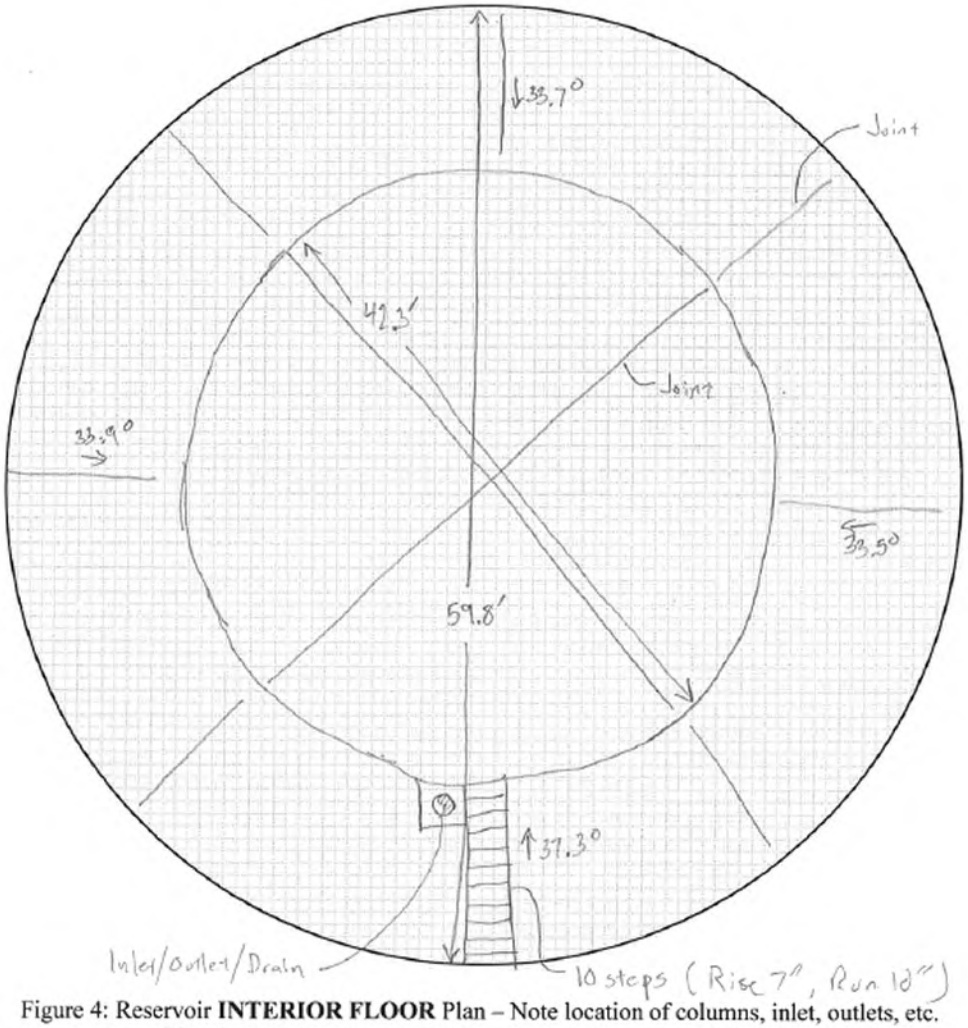
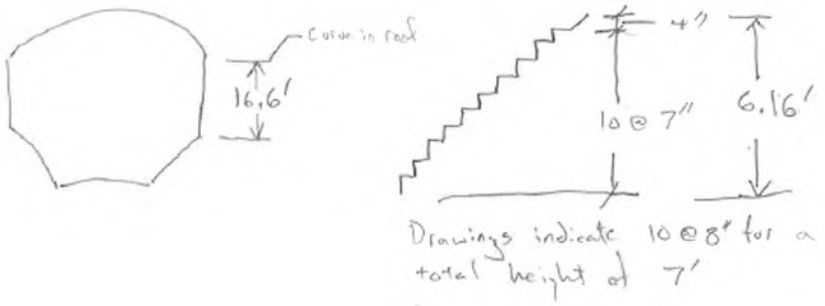


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)



6

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Base is coated some pockmarking noticable through coating. Cracks were not visible

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): Interior coating but areas of cracking still visible through coating

# of wall sections: N/A

Ladder/Pipes/Overflow Conditions:

Overflow Height: 26.75', 26.75' (drawings/measured) see below Operating Height: 20'-23' (per City/PUD/other)

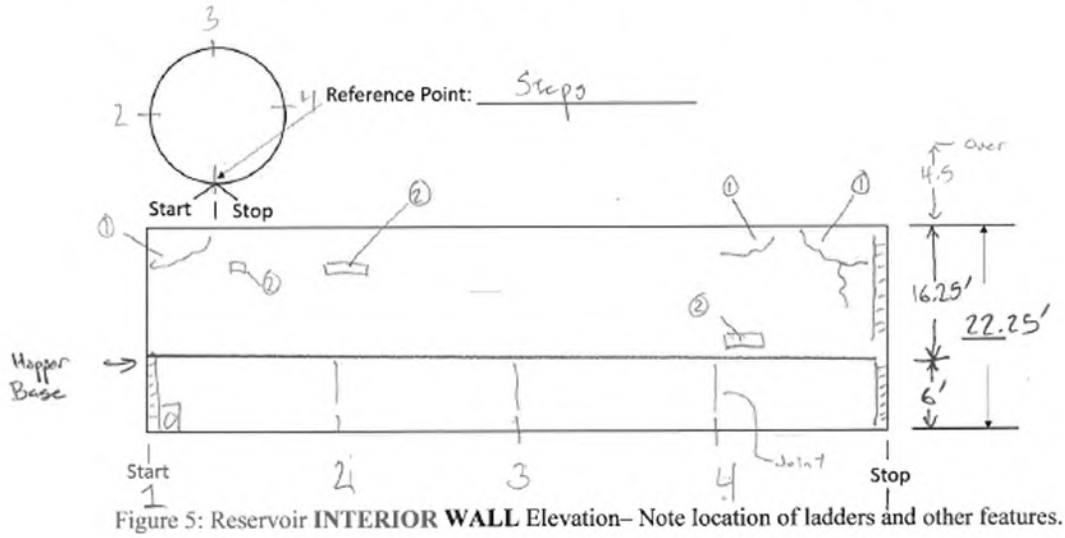

Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

① Crack, covered by coating but visible corresponds to exterior crack

② Rectangular coating patch

Overflow = 6+16.25+4.5 = 26.75'

7

END OF SECTION

Appendix G-4 40th Street General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

40th Street Reservoir *General Info*

Field Visit Date: 4/30/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	4/30/2019
Reservoir Name and Location:	40th St - 500 ft East of 1318 40th St 98229
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis, Jeremy Hailey
Client Staff Present:	Shayla Francis, Steve Bradshaw, Nick Leininger, Jenny Eakins
Year Constructed:	1958
Overflow Destination:	Storm drain to SW
Discharge Destination/Zone:	Main to Padden-Yew 696 Zone
Fill Location:	SW side, From 38th St Pump Station
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	60	ft
Height	26.75	ft
Overflow Elevation:	696	ft AMSL
Bottom Elevation:	676.1	ft AMSL
Level of Overflow	26.7	ft
Minimum Normal Operating Level:	20	ft
Maximum Normal Operating Level:	23	ft
Notes:		

40th Street Reservoir *Exterior Inspection*

Field Visit Date: 4/30/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	Locked enclosure	
Security Condition:	Good	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1.5	in
Ladder Width	20	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	7.5	in
Back Clearance:	N/A	in
Notes: (Security) May be possible to scale vault - no locked upper ladder.		

Exterior Fall Prevention System:	
Present at Site:	No

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof	
Material:	Aluminum	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Toward center of reservoir	
Measurement Type	Measurement	Unit
Size:	3x3	ft
Curb Height:	4	in
Notes: 3 ft 2in from roof to top of concrete		

Roof Vents and Screen:		
Material:	Concrete	
Condition:	Poor	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	Unknown	in
Notes: 50" approximate diameter. The screen has become disconnected from the reservoir concrete.		

Roof:		
Condition:	Good	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	Textured coating w/ sand	
Slope of roof	3 to 20 degrees	
Measurement Type	Measurement	Unit
Overhang Distance:	3	in
Thickness of roof slab	4.5, 18 at edge	in
Notes: Some moss growth on E side. No cracks noted, but may be result of coating over cracks.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Fair	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	N/A	in
Top Height:	41	in
Notes: Mid rail at 20 in. Railing only near access vault, not around entire perimeter of reservoir.		

Grating:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Very Good	
Corrosion present?	No	
Clips:	No	
Removable Panels:	No	
Measurement Type	Measurement	**Unit**
Approximate Panel Dimensions:	2x5	ft
Notes:		

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Fair
Notes: Cracked with efflorescence. Rock pocketing noted.	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	AquaVers 20 and 405
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	Walls: 20 to 40 mils
Exterior Coating Adhesion Testing Results:	Poor in efflorescing areas
Notes:	

40th Street Reservoir *Interior Inspection*

Field Visit Date: 4/30/2019

Interior Ladder:	
Present at Site:	No
Notes: Used removable ladder.	

Interior Fall Prevention System:	
Present at Site:	No
Notes: Used tripod and winch	

Interior Roof:		
Condition:	Fair	
Measurement Type	Measurement	Unit
N/A	N/A	ft
Notes: Concrete spalling near roof vent		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: No visible cracks. Coating bubbling in some areas. Four slabs w/ joints	

Walls:	
Condition:	Good
Painters Rings Present:	No
Notes: Cracks have been sealed. Pockmarking visible under coating.	

Interior Coating	
Interior Walls:	AquaVers 20 and 405
Interior Floor:	AquaVers 20 and 405
Interior of Roof:	No Coating
Interior Ladder:	N/A
Interior Piping:	Unknown
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	80 - 130 mils
Interior Coating Adhesion Testing Results:	Excellent
Notes:	

40th Street Reservoir *Miscellaneous*

Field Visit Date: 4/30/2019

Piping		
Inlet Piping:	Size (Inches OD):	10
	Condition:	Fair
	Material:	Cast Iron
	Notes: Common inlet/outlet/drain. Corrosion on interior of pipe	
Outlet Piping:	Size (inches OD):	10
	Condition:	Fair
	Material:	Cast Iron
	Lip (Inches)	0
	Notes:	
Overflow Piping:	Size (inches OD):	6
	Condition:	Good
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	Wooded area NW of reservoir
	Erosion Evident:	Yes
	Screen Condition:	Good
	Overflow to roof (feet)	0
	Notes: Overflow weir is sharp-crested above bottom of dome. Air gap screen in good condition.	
Drain Piping:	Size (inches OD):	10
	Condition:	Fair
	Outlet Location:	Wooded area NW of reservoir
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	None
	Air Gap:	No
	Screen Condition:	N/A
	Notes: Drains w/ overflow via GV. No screen on outlet pipe. Dechlorination via bag.	

Piping Facilities		
Exterior Valving:	Type:	Gate valves
	Condition:	Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	In valve vault
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	N/A
Notes: No flex couplings		

Electrical	
Cathodic Protection:	N/A
Impressed Current:	N/A
Anodes:	N/A
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	Yes
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	No
Sediment Build-Up Height Above Floor (in)	<0.1
Water Quality Sample Taps?	No
Notes: Hydroranger miltronics level indicator. Altitude valve serves as check valve	

Appendix G-5 40th Street Condition Assessment Score Sheet

40th St Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No camera
	Vegetation Separation	0	0	0	0	0	0	1	0	Under the dripline of trees. Organic debris on roof.
	Site Drainage	0	0	0	0	0	0	3	0	Bubbles in coating
Walls	Exterior Walls	4	3	2	2	0	0	4	0	Roof-wall interface not designed for thermal movement - cracks walls
	Interior Walls	5	4	2	2	0	0	5	0	Wall is overstressed - lower op level
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	4	4	5	5	4	0	5	0	Sediment covered floors. Bubbles in coating - groundwater
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	4	4	5	3	5	0	5	0	Textured coating collects debris - needs cleaning
	Interior Roof and Supports	0	4	5	3	0	0	0	0	
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	5	5	0	No fall protection required
	Interior Ladders/Fall Protection	0	0	0	0	0	3	3	0	No fall protection required, but would be good to have. Hard to use interior ladder/winch
	Access Hatches	5	4	0	0	4	0	3	0	Difficult to get into hatch. High maintenance design.
	Railings and Roof Fall Protection	5	5	0	0	0	1	0	0	Very high drop from dome and steep.
	Vents	5	2	0	0	3	0	5	0	Concrete spall noted around interior vent lip. Passed design checks. Screen likely too coarse.
	Balconies/Landings/Grating	5	5	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	2	3	0	3	4	0	5	0	Comb in/out/drain. Coating and corrosion on interior of pipe. Needs flex coupling
	Outlet Piping	0	0	0	3	0	0	5	0	Comb in/out/drain.
	Drain Piping	0	0	0	3	3	0	2	0	Comb in/out/drain. No silt stop. No screen or energy dissipation. Poor dechlorination
	Overflow Piping	5	4	0	0	3	0	3	0	Connects with drain and outlets w/o screen or energy dissipation.
	Washdown Piping	0	0	0	0	0	0	5	0	Hose bib in valve vault
	Attached Valve Vault Structure	5	4	4	3	0	0	4	0	Roof drainage needs improvement
	Control Valving	4	4	0	0	5	0	5	3	Altitude valve shows corrosion
	Isolation Valving	4	3	0	0	0	0	5	3	Gate valve shows corrosion
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	No mixing system
	Categorical Score	4.4	3.9	4.0	3.2	3.9	3.0	4.1	3.0	

Overall Score
3.6

Appendix H College Way

Appendix H-1 College Way Geotechnical Report

GeoEngineers

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs College Way Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the College Way reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at College Way site, located as shown in the Vicinity Map, Figure 1. The College Way reservoir is a round reinforced concrete structure with a hopper base.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. Undifferentiated glacial deposits are mapped nearby.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone. The character of the bedrock at the site is known to vary considerably over short distances.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift. Based on previous experience in the area, Bellingham (glaciomarine) Drift overlies the bedrock in this area. The Bellingham Drift is a glaciomarine drift deposit which consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper 5 to 15 feet of this unit in upland areas is typically stiff. The stiff layer possesses relatively high shear strength and low compressibility characteristics. The stiff layer oftentimes grades to medium stiff or even soft, gray, clayey silt or clay with depth. The entire profile can stiff, likely from being partially glacially

overridden, when it is a shallow profile over bedrock. The soft to medium stiff glaciomarine drift possesses relatively low shear strength and moderate to high compressibility characteristics.

Surface Conditions

The project site is located approximately 75 feet east of Highland Drive and 75 feet south of West College Way. The reservoir is located on top of a ridge that drops off toward the east. A cell tower is located on the eastern edge of the site. The site is bounded by a wooded area to the east, West College Way to the north, Highland Drive to the west, and residential houses to the south.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-8 (2019)—on March 26, 2019 using a track-mounted drill rig subcontracted to GeoEngineers, Inc. (GeoEngineers). The boring was completed to a depth of 5½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan Figure 2. A key to the boring log symbols is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites.

- **Chuckanut sandstone** – Chuckanut sandstone was encountered at the surface of the exploration. The boring was completed at 5½ feet. The upper approximately 3 feet of the sandstone was weathered and consisted of very dense fine to coarse sand. Refusal was encountered in the sandstone at about 5½ feet bgs.

Groundwater

Groundwater seepage was not observed at final depth of the boring. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

The reservoir site is underlain by a very shallow mantle of weathered bedrock (3 feet of very dense sand) overlying unweathered bedrock. No project plans were available for our review to determine how the foundation was constructed. However, for purposes of this project, it is our opinion that it is reasonable to assume that the reservoir base is constructed on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on sandstone which is not at risk of liquefaction.

ACI/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted

maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	96.5
1-Second Period Spectral Response Acceleration, S_1 (percent g)	38.0
Seismic Coefficient, F_a	1.01
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.400
Seismic design value, S_{DS}	0.652
Seismic design value, S_{D1}	0.359

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	8	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.20	0.36	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.08	0.14	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic,

geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and our observations, we anticipate that the existing reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic foot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The existing reservoir includes below grade walls. Our recommendations for evaluating below grade concrete walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the "Shallow Foundations" section and the wall backfill consists of structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf
Allowable Passive Earth Pressure Coefficient (K_p)	3.5
Allowable Passive Earth Pressure[1]	350 pcf
Allowable Concrete Wall Foundation Coefficient of Sliding	0.45

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the south that is approximately 25 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on

bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

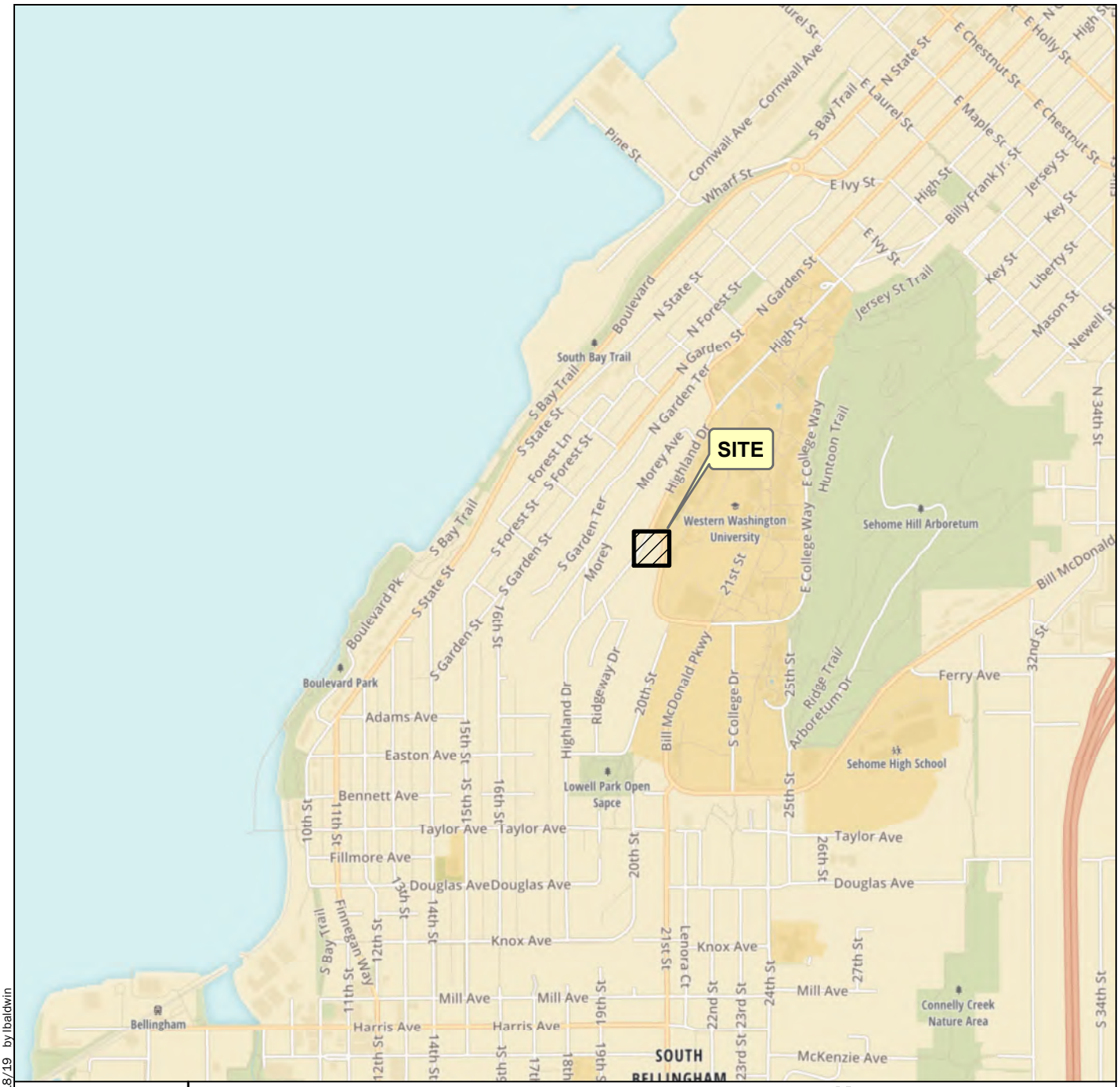
Figure 2 – Site Plan

Figure 3 –Key to Exploration Logs

Figure 4 –Log of Boring B-8

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault



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Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

College Way Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

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Legend

Boring by GeoEngineers (2019)

75 0 75

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Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

College Way Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

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SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled Start 3/26/2019 End 3/26/2019	Total Depth (ft) 5.25	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) 450 Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment EC-95	
Easting (X) 1240400 Northing (Y) 636510		System Datum WA State Plane North NAD83 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	2 inches topsoil			
								SP	Brown fine to coarse sand (very dense, moist) (weathered sandstone)			
		X		85		1						
								Sandstone	Brown sandstone (Chuckanut Formation)			
445	5	X	3	50/3"		2						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-8

GeoEngineers

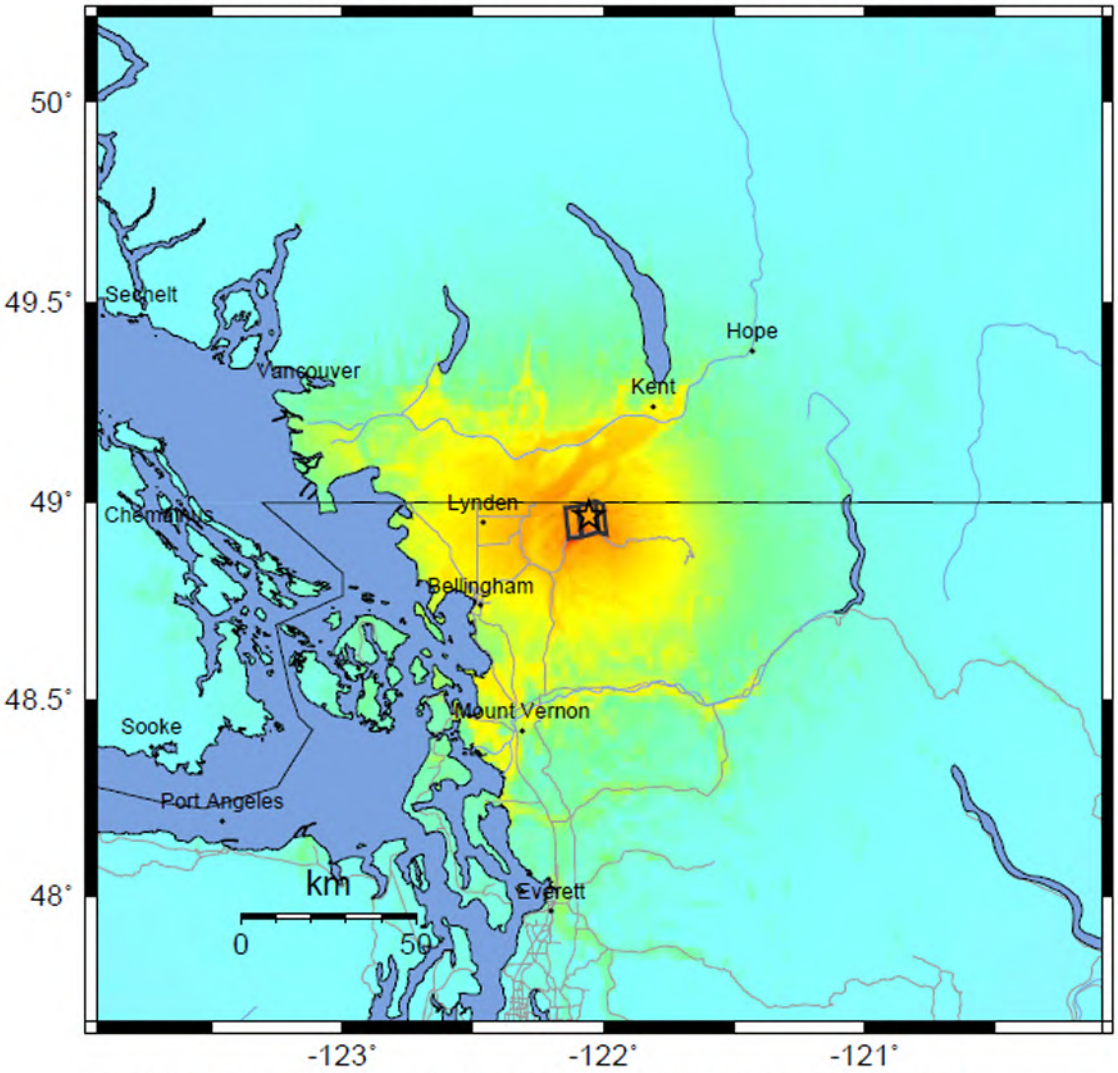
Project: COB Reservoir Inspection and Repair - College Way
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

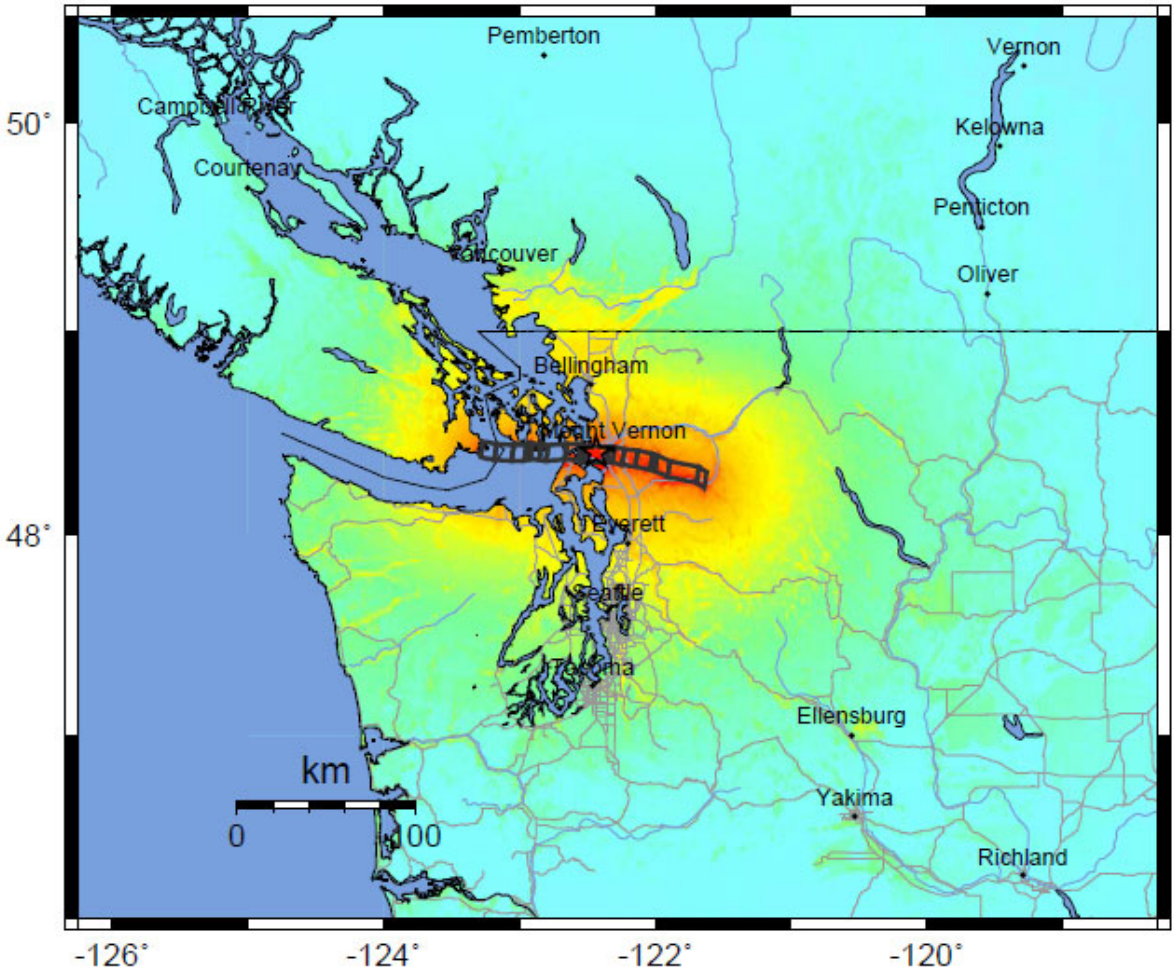
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix H-2 College Way Corrosion and Coatings Report

March 18, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – College Way Concrete Tank Coating Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an internal and external coatings evaluation for the City of Bellingham's College Way concrete water storage tank. Specific tasks completed during this assessment included:

1. Complete an assessment of both the interior and exterior coating.
2. Measure interior coating thickness at representative locations.
3. Note any corrosion related activity on exposed interior steel tank components.
4. Evaluate coating losses and corrosion on visible surfaces.

BACKGROUND INFORMATION

The College Way tank is approximately 65-ft in diameter and 20-ft tall (floor to bottom of roof dome) with a majority of its volume buried belowground. The tank is constructed of concrete and was internally lined in January 2013. The internal lining consists of a prime coat of VersaFlex AquaVers 20 followed by an application of VersaFlex AquaVers 405, a high build polyurea. The lining was applied to the floor and wall surfaces up to the bottom of the domed roof. The coating material was applied to a general thickness of 85 – 100 mils (1,000 mils = 1 inch).

Information was not available as to the type and installation date of the exterior coating. It is known that the exterior coating was applied well before the interior lining was installed.

COATING EVALUATION METHODS

The coating evaluation consisted of a visual inspection of the accessible interior and exterior surfaces and measuring the thickness of the internal lining at multiple locations. Destructive testing of the coating materials was not performed.

Dry Film Thickness

The thickness of the interior coating system was measured using a DeFelsko PosiTector Model 6000 electromagnetic dry film thickness gauge (Type 2 gauge) with a model 200 transducer used for measuring polyurea.

INSPECTION RESULTS AND ANALYSIS

Exterior Coating Assessment

The exterior roof surfaces are covered with organic matter and debris. The top coat material is peeling off in several locations and ~ 15% of the top coating has delaminated exposing the underlying coating. The underlying coating is tightly adhered to the concrete surface, however the top coat can be easily peeled off using a razor knife at the defect areas. Otherwise, the top coat material appears to be well adhered.

The sidewall surfaces also have significant coating damage, mostly near cracks in the concrete where pressure exerted on the coating has peeled it from the surface. This is readily evident along most of the upper sidewall perimeter. A small area was also noted where the concrete at the soil interface had not been coated and aggregate material is exposed to the atmosphere. There is also a large crack in what appears to be a mortar type lining near the exterior roof access ladder. This may have been a previous concrete repair area that is now failing.

College Way Tank – Roof and sidewall, looking to the northeast

Heavy debris accumulation and top coat loss

Top coat loss and delamination on sidewalls

Sidewall staining

Missing coating at base of tank

Roof vent

Additional staining on sidewall

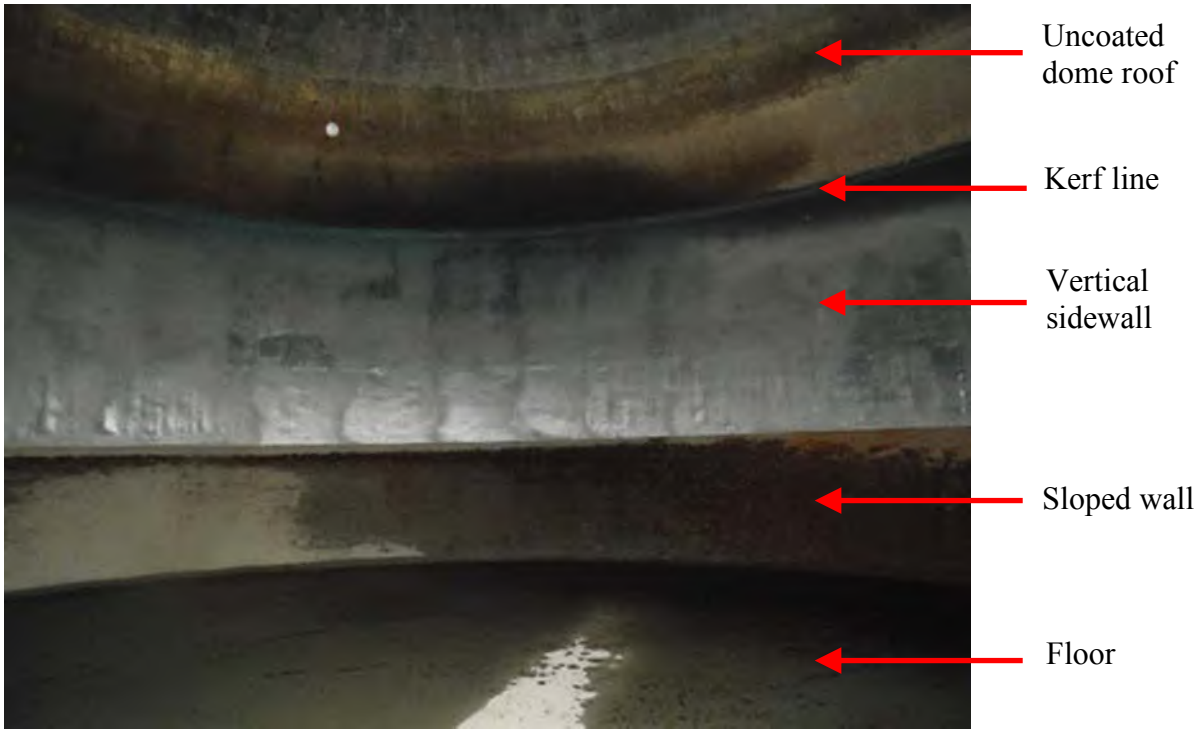
In order to help preserve the life of the concrete, the tank will need to be cleaned, have all cracks properly sealed, and a repair overcoating applied. This will extend the life of the existing coating an additional (+/-) 10 years. Otherwise, removal of all existing coatings, repair of concrete and sealing of cracks, plus the application of a new exterior protective coating can be applied that will provide a protective service life of more than 20 years with appropriate minor maintenance.

Interior Coating Assessment

A thin layer of leftover residue in the water is covering approximately 35% of the floor and sidewall surfaces. The vertical walls appear to be generally clean.

The interior lining of the tank is in very good condition. The kerf line (1/4" cut in concrete made between coated and non-coated transition used as an anchor point for the coating) did not show any instances of coating delamination. There was no noted coating disbondment or significant blistering. In particular, the discharge area adjacent to the stairs and the stair steps

themselves had intact coatings all around. Normally, these sharp-edged surfaces tend to be the first areas of coating adhesion loss.



Interior of College Way reservoir

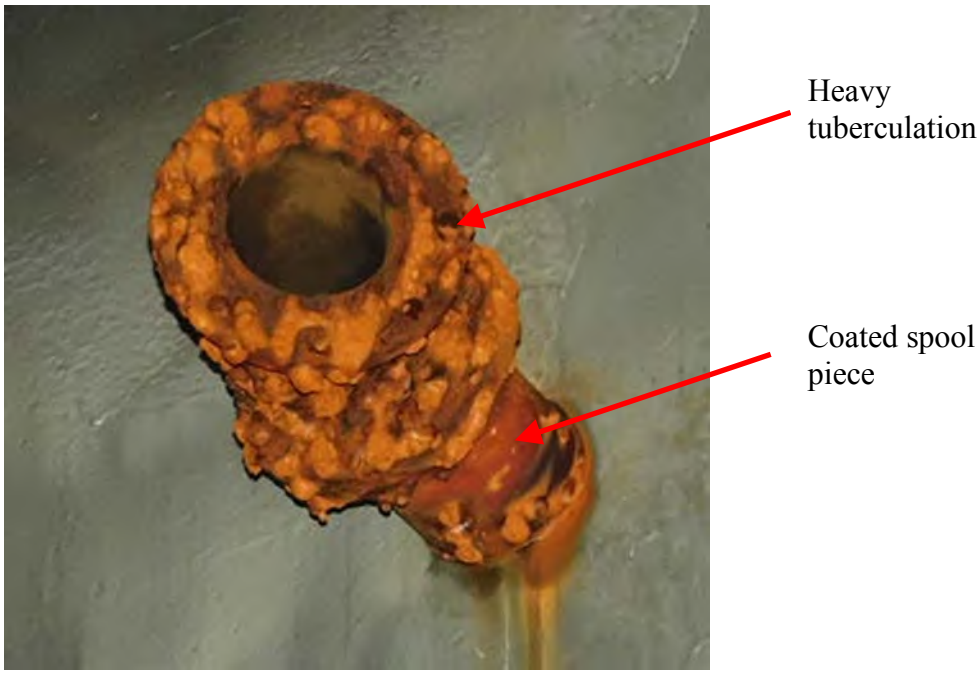
Original coating being applied (2013)

Interior coating

Sidewall coating and uncoated roof

Interior coating

The hydraulic mixing pipe has developed a thick layer of external corrosion and tuberculation. This is the result of a metallic couple whereby the ductile iron surfaces in contact with the water become anodic relative to the ductile iron in contact with (transitioning through) the concrete wall. If the piping transitioning through the wall is in contact with prestressing cable or rebar, the surface area ratio between the anode (pipe in contact with water) and the cathode (embedded pipe and cable) is very large resulting in the noted corrosion. As can be seen in the picture below, the center of the short spool piece exiting the concrete is in relatively good condition because of the its external coating. The outlet pipe to the pump station also has a lesser degree of tuberculation development on the exposed metallic edges.



Hydraulic mixing pipe

Outlet pipe to pump station

Dry Film Thickness

The exterior concrete surface texture is rough and measuring coating thickness using a dry film thickness gauge is not practical. However, we were able to remove delaminated sections of top coat material and using a micrometer, the thickness of the topcoat ranged from 8 – 14 mils. This is typical for the exterior surfaces of this type of tank construction.

Measurement of the interior coating thickness using a polyurea gauge showed coatings in the range of 80 – 110 mils. Approximately thirty readings were recorded on floor, angled wall transition, and on the vertical sidewalls adjacent to the ladder. These values are consistent with data collected after coating application and prior to filling the tank.

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the tank.

1. The interior of the tank was lined in 2013. It is unknown when the external surfaces were coated.
2. The exterior sidewall and roof surfaces of the tank are dirty.
3. Approximately 15% of the exterior roof top coat is gone. The undercoating appears to be fairly tightly adhered to the concrete surface.
4. Multiple locations around the exterior sidewall have missing top coat and there are a few locations of exposed bare concrete.
5. The interior coating is in very good condition with coating thickness measurements consistent with those recorded as part of the recoating project in 2013.
6. The exposed metallic piping components within the tank have active corrosion. In particular, the hydraulic mixing pipe has heavy tuberculation.

RECOMMENDATIONS

1. The exterior tank surfaces should be pressure washed to remove dirt and other debris.
2. Spot repair areas of coating loss. This will require the development of appropriate surface preparation and coating application procedures. This should extend the useful life of the coating an additional 10 years.
3. If a longer coating life is desired, the existing coating will need to be completely removed and a new coating system installed.
4. The next time the tank is taken down for maintenance, the hydraulic mixing and outlet pipes should be cleaned and coated to prevent further corrosion. It may be more cost effective to replace the nozzle on the mixing pipe as opposed to cleaning and coating the existing piece.

We appreciate the opportunity to work with you on this project. If you have any questions or would like assistance with the implementation of the report recommendations, please do not hesitate to contact our office.

Sincerely,
Northwest Corrosion Engineering

Jeremy A. Hailey, P.E.

Appendix H-3 College Way Structural Report

CITY OF BELLINGHAM

CH 10: COLLEGE WAY RESERVOIR

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessment & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the College Way, 0.5 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located at 231 Highland Dr, Bellingham, WA (Lat. 48.733, Long. -122.491), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on March 14th, 2019 by Peterson Structural Engineers (PSE), and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 College Way Round Reinforced Concrete (RC) Reservoir – 0.5 MG

2.1 Description & Background

Per information provided by the City, the College Way reservoir was likely designed by John W. Cunningham & Associates Consulting Engineers and built in 1968. Drawings were not available for the reservoir, but it is similar in design to other reservoirs in the City's inventory with a storage volume of around 0.5MG, a dome roof, and a hopper base. For similar reservoirs types, which do have available drawings, Cunningham & Associates is the designer of record. The reservoir itself is a round reinforced concrete reservoir with a measured interior diameter of 64-feet. The interior wall was measured to be 13.5-feet high while the hopper base was 6-feet deep. The overflow weir is located approximately 23.5-feet above the bottom of the reservoir and is located above the top of the wall within the access hatch box. The reservoir, at full capacity, uses a portion of the dome roof for its storage volume.

Where details or sections could not be directly observed or measured, they have been assumed to be comparable to other similarly sized reservoirs designed by Cunningham & Associates built in the same era. Per those reference drawings it is assumed that the wall is around 10-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. The roof is likely a reinforced 4.5-inch thick dome with an edge that thickens to 1'-2". The floor is likely a reinforced 5-inch thick slab. The footing is likely a reinforced 12-inch thick by 10-inch wide footing that transitions into the hopper base. Finally, where piping is run under the footing, it is assumed to be encased in an unreinforced concrete block for protection. Drawings of a similar reservoir configuration designed by Cunningham & Associates and built in the same era have been included in Figure 2-1 & Figure 2-2 for reference while a schematic drawing of the reservoir can be found in Figure 2-3.

2.1.1 Description of Additional Site Structures and Features

The site includes two additional structures related to the reservoir's operation. The first is a valve vault which was constructed as part of the reservoir. The rear of the vault shares a wall and footing with the reservoir. This vault is located on the west side of the reservoir and is two levels, with one level located below grade. The main and lower vault levels are 8-feet square and the main level has an 8-foot clearance while the lower level has a 9-foot clearance. The lower level is accessed via a 30 by 36-inch opening in the main level's 5.5-inch thick slab floor. The reservoir's drain, outlet, and overflow are all run through the valve vault. The roof of the vault has a parapet and the air gap for the reservoir overflow is located above the roof. If debris were to clog the overflow, the layout of the top of the valve vault is such that it would fill with water up to the height of the parapet on the roof. While the roof had a secondary drain, it is insufficiently sized to handle any overflow volume. Additionally, as the overflow and drain pipes are joined below the roof line, any material or issue that blocks the overflow would also be likely to clog the drain as well.

To the north of the reservoir is a new booster pump station which was built in 2007 and designed by RH2. While this pump station is outside of PSE's scope, a cursory review was performed. Based on this review, no issues were visually apparent in our limited visual inspection.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit on March 14th, 2019 to observe the as-built current condition of the reservoir's interior and exterior as well as the site conditions. The reservoir was drained for our inspection.

Dome Roof: The reservoir has a self-supporting concrete dome roof with a thickened edge. The surface of the roof is coated. While this coating is generally competent on the west side of the reservoir, the coating is compromised toward the east side, where debris from trees are prone to collect. In this zone, larger sections of the coating are missing or damaged. Structurally the roof appears to be in generally good visual condition. The most significant item noted in the roof was a circumferential crack that ran around the entire circumference of the roof approximately half-way between the center and edge. Additional cracking may be present that is not visible due to the applied coating. Per the reference drawings, it is assumed that the radial roof dome reinforcing fans out along the circumference with additional reinforcing added as the width between the fanned bars increases. This cracking appears to be occurring roughly along the line where the additional reinforcing at the fanning begins and may be a result of a stiffness differential in the roof.

The roof has one 3 by 3-foot access hatch that is part of a larger access box. This box is formed to include an overflow weir. The location of the overflow is above the top of the wall and necessitates a waterstop in both the roof-to-wall joint and access hatch-to-roof joint. Few issues were noted within or around the hatch, which seems in good condition and working order. Only minor cracking was noted on the front of the box, where the overflow pipe is located.

At the center of the roof the reservoir has a 36-inch diameter vent. Per the available assumed reference drawings and pictures, the vent should include a series of (12) 4-inch by 11-inch openings around its exterior. However, these openings and the structure itself were obscured by a sheet-metal cover. Observable components and the surrounding roof do not appear to have any visual structural issues. The tightness of the metal cover could pose some issues with the venting and Murraysmith should be consulted to determine if a problem exists due to restricted airflow. Inadequate venting can create a structural overload condition when the reservoir is filled or drained if the vent can't keep up with the change in storage volume.

Reservoir Walls and Interior: Per the assumed referenced drawings, the walls are likely 10-inches thick with vertical and horizontal (hoop) reinforcing. At the top of the wall, it is assumed there is a keyway and (2) bars of reinforcing connect the roof to the wall. This connection is required as the design of the reservoir allows for water to be stored above the top of the wall and within the dome. This results in a restraint that that does not accommodate thermal expansion of the roof. As a temperature change occur the roof will expand or contract radially while the wall expands or contracts vertically. As the reservoir is partially buried, this creates resistance to the wall from outward movement.

The effect of this differential movement has resulted in a crack which runs around the exterior of the reservoir's wall, just below the base of the dome roof edge. This crack is likely a result of the thermal expansion pushing outward on the top of wall and failing it at the thinner section of its joint key. Along the front of the reservoir, the backfill height is reduced to accommodate the valve vault. The vault itself

creates a stiff-point relative to the adjacent backfill, while the access hatch in the roof acts as a break in the roof line, creating a less-stiff zone. This limits the amount of direct shear on the top of the wall and results in more flexure on the overall wall which is then "bent" around the top of the valve vault. This variation in the load likely caused the crack moving along the 45-degree lines from the top of the wall to the to the top of vault, noted on either side.

As the reservoir is partially buried, a limited inspection of the exterior surface was obtained. Further, the exterior of the reservoir is coated, and it is possible that this coating is covering additional unforeseen conditions. However, additional significant problems such as those resulting from the aforementioned thermal cracking would likely be visible through the coating. Reinforced, non-prestressed concrete reservoirs, can develop issues with creep resulting in gradual failure of the reservoir walls. However, as this reservoir is partially buried, the confining pressure of the soil backfill helps to counter these hydrostatic tensile forces. Looking for these types of issues, no major crack issues or failures outside of the thermal cracking were noted in our visual inspection.

The interior of the reservoir was found to be in good visual condition. The floor slab, hopper sides of the floor, walls, and interior dome roof all appeared to be in good visual condition. A coating had been applied to protect the concrete and appears to be intact. The coating was applied up onto the lower portion of the dome. Above the coating, no visible issues or significant cracks were noted. The coating appeared competent and no underlying structural issues or corrosion were visible that might compromise or effect the coating.

Appurtenances: The inlet, outlet, and overflow weir all appeared to be in generally good visible condition although some small corrosion carbuncles were noted. A recirculation pipe located on the wall on the northwest side of the reservoir did appear to have a greater amount of corrosion and requires cleaning and/or maintenance to remove corrosion.

2.2.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appears to be in generally good visual condition with signs of general wear and tear noted along its exterior. No visual signs of major cracking or settlement were noted. However, as it shares a wall-line with the reservoir, some issues with water infiltration were noted along edges. Based upon the reference drawings it is assumed that the vault was poured using a keyed joint and reinforcing to connect it to the reservoir wall. However, there is no indication of the presence of a waterstop or more watertight joint type in the drawings. As a result, some leakage was noted along the main level roof joint. The bottom of the vault was filled with approximately 1-foot of water. While it is unclear what was causing the water a sump pump should either be installed or if there is an existing pump, it should be repaired. Pipes should be checked to determine if there are any leakage issues which are contributing to the observed water and upgraded to meet current code requirements.

2.3 Structural Analysis

The following design analysis is based on the reservoir drawings associated with the 40th St Reservoir which is a similar type of reservoir with a dome roof and hopper base. The City has a few of these types of reservoirs in its inventory and other similar reservoirs include Dakin I (built 1987), Reveille (built 1958),

Consolidation (built 1959), and 40th St (built 1959). Only the Reveille and 40th St reservoirs have drawings available and both sets of drawings were prepared by John W. Cunningham & Associates, Consulting Engineers. As the 40th St reservoir is of a similar size to College Way (0.5MG versus Reveille's 0.3MG size), the 40th St drawings have been used to provide a baseline for PSE's analysis.

Where field measurements could be made, the actual dimensions were used in PSE's analysis. In this case the size and diameter of the reference reservoir varies somewhat with 40th St having a 60-foot diameter and a 15.5-foot wall height while College Way was found to have a 64-foot diameter and a 13.5-foot wall height. Where dimensions or information could not be obtained during the site visit, such as the reinforcing size, layout, and spacing, those values were based on the 40th St reservoir drawings. Based on the similarity of multiple versions of this reservoir, this assumption is a reasonable and cost-effective method to determine a presumed level of the basic adequacy of the reservoir. Based on the results of the analysis, issues and retrofit options are discussed. However, as these calculations are an approximation, any assumptions made should be verified though additional destructive and non-destructing testing prior to undertaking any retrofit or repair work.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Circular Concrete Tanks without Prestressing", published 1993 were also utilized.

2.3.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof was evaluated per concrete design criteria as covered in AWWA D110-13 and found to meet minimum requirements for rise-to-span, thickness versus buckling, and edge reinforcing assuming the reinforcing is consistent with the reference drawings. The dome roof was evaluated at a 19-foot operating level. At this level the water is stored below the top of the wall and there are no additional hydrostatic loads on the roof. Should the reservoir be operated at overflow, analysis found that the assumed circumferential dome edge ring reinforcing was adequate for the additional hydrostatic load.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will allow for differential thermal movement between the reservoir roof and the wall as both components deform differently as a result of temperature variations due to temperature or solar gain. At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the components of the structure able to resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement. This is a result of its original design in which water is stored above the wall line. To adequately account for thermal movement the existing roof attachment would need to be modified or replaced with a roof system able to accommodate thermal movement. This type of retrofit may not be practical or economically feasible.

Wall Reinforcement: Per the assumed reference drawings the wall is likely reinforced with #4 vertical bars at 12-inches on center on the exterior face and at #5 vertical bars at 24-inches on center on the interior face. It is assumed that the horizontal (hoop) reinforcing starts out with #5 bars at 3.5-inches on center towards the base and the reinforcing density decreases to #5 bars at 6.5-inches on center towards the top of the wall. At the very top of the wall there are assumed to be (2) #6 circumferential hoops. This variation in the hoop reinforcing is based on the variable pressure distribution resulting from the storage of liquids.

Per PSE's analysis, it was determined that the vertical wall reinforcing appears to be sufficient for code strength requirements for the current operating level. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3 sanitary factor. This sanitary factor is intended to minimize the potential for cracking and leaks. While the reservoir is adequate for the current operating level, the wall reinforcing would be exceeded if the operating level were to be increased to the overflow level.

Additional checks were performed for the wall at the 19-foot operating level and determined the remaining wall reinforcing to be adequate. This included checks for shear loads, for hoop tension forces (when accounting for the larger 1.65 sanitary factor required by code when checking reinforcing in tension), and when considering the compressive wall loads resulting from soil backfill.

Finally, per ACI 350.3, the maximum spacing for wall reinforcing was checked. The maximum allowable spacing for bar is limited to 12-inches on center. In the upper sections of the wall, the spacing of the interior and exterior reinforcing is at 24-inches on center, exceeding the maximum limit. While this is unlikely to be the sole issue causing the observed cracking, this larger spacing could be one factor (especially the wide cracks noted adjacent to the vault).

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the geotechnical evaluation, the site's bearing capacity was determined to be 6,000-psf. Using this bearing capacity and checking for the 19-foot operating level up to overflow, the bearing pressure was determined to be within acceptable ranges.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: The addition of seismic loads results in additional forces on the wall of the structure. This is a result of the water slosh wave as well as forces resulting from mass movement of the structure itself. PSE found that the walls flexural stresses were increased by about 20% when compared to the static loads at a 19-foot operating level. As the reservoir was designed for a higher static operating level, there is reserve capacity and the reservoir was found to be able to resist the increased seismic loads. Should the reservoir be operated at overflow, the seismic load would exceed the wall's flexural capacity and the reservoir would need to be retrofitted.

The wall's hoop tensile stresses were found to be within acceptable limits when the reservoir was evaluated at the 19-foot operating level. However, limited capacity remains to meet design requirements when operated at higher levels. For lateral seismic loads, the horizontal hoop reinforcing was determined to be adequate for the 19-foot operating level.

In addition to the wall flexure and tensile checks, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing at the overflow level. No additional reinforcing or connectivity is needed between the walls and the foundation based upon the assumed construction.

Freeboard/Slosh: At overflow, the reservoir stores water above the top of the wall. For such an operating level, during a seismic event, the roof would constrain the slosh wave. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. The force of this wave in a code level event would be sufficient to damage and potentially cause failure of the roof at the roof-to-wall interface, the dome itself, as well as hatches and other appurtenances. At the current operating level there is minimal freeboard which is not sufficient to prevent a 2.5-foot slosh wave from impacting the roof. However, the current operating level is such that it results in a lower slosh wave impact force. Therefore, at an operating level of 19-feet, the available roof reinforcing, along with the roof's thickness and weight, appears to be sufficient to resist the maximum slosh load.

Valve Vault: Per the reference drawings it is assumed that the valve vault is connected to the reservoir with #5 rebar dowels at 12-inches on center. This attachment should limit differential movement or "pounding" that occurs in a seismic event. Additionally, where the hopper base and the lower level of the vault are adjacent, this zone is assumed to be backfilled with plain concrete or "trench backfill". In the event of an earthquake, this will provide support to the hopper based to limit its potential to fail the lower level wall and collapse onto the piping. Of primary concern is the vault's wall which is cast as part of the reservoir footing. Depending on the direction of ground motion, pipes should be retrofitted to have flexible coupling should differential movement between the two structures occur during a seismic event.

2.4 Summary

Based on the available drawings and site visit it appears that a majority of the structural elements in the reservoir are adequate for the expected loads at the current operating level. It was noted that ringing the reservoir there is cracking along the exterior of the wall that could be a result of the combined thermal and operational loading conditions. Further, wall reinforcing was found to exceed maximum spacing allowances, and these combined issues could be contributing to some of the cracking issues noted.

Elements outside of the wall, such as the dome roof and footing were determined to be adequate when operated at a 19-foot operating level. However, while reinforcing in these areas was found to be adequate, the dome-to-roof reinforced connection is rigid and unable to resist expansion/contraction loads resulting from thermal effects. This has likely been the cause of damage noted around the reservoir at the roof-to-wall interface. As a result, the actual capacity at this connection is expected to be lower than our analysis has determined. This could be a concern in a seismic event.

Remaining observable components of the reservoir appear to be in visual good condition with few interior defects noted. The area of primary concern was cracking and wall failure due to thermal expansion and contraction of the dome roof; it appears that this connection was not designed to accommodate thermal effects. The reservoir is also attached to a valve vault which appeared to be structurally sound. However, the vault was observed to have a fairly substantial amount of water in the lower level which should be

removed, and the source of the water identified so the leak can be stopped. Piping in the vault may also be a candidate to be upgraded to accommodate vertical and horizontal differential movement between the vault and the reservoir during a seismic event.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 19-feet, which is 0.5-feet below the top of the wall. Due to the types of issues noted, these retrofits might not be cost effective or easy to implement.

Wall Flexural Capacity

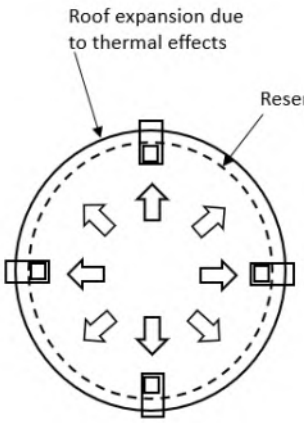
Based on a maximum operating level of 19-feet, PSE determined the wall flexure was adequate. However, these design checks are based on an assumed reinforcing layout. Should the reservoir be operated at a higher level or additional confirmation be required, PSE would recommend a testing firm be employed to map the existing reinforcing. Additionally, the testing firm could perform testing to verify the concrete compressive strength and reinforcing's yield strength. Based on the gathered information, the reservoir can be re-evaluated to confirm the actual as-built design capacity.

Roof-to-Wall Retrofit

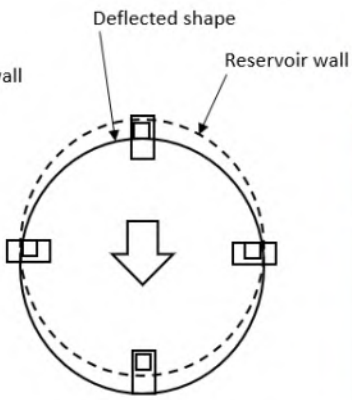
The effects of thermal movement appear to have damaged the roof-to-wall interface and has likely affected its structural shear capacity and water-tightness. To alleviate this issue, PSE would recommend either the retrofit or replacement of the roof. Note, that this may not be economically feasible or practical.

The first option would be to remove the existing roof and replace it with a new roof. As the operating height is now below the wall height, there are a variety of options available. For example, aluminum geodesic dome roofs have been used to either add new roofs or retrofit existing roofs to many types of reservoirs. As an aluminum dome roof is relatively light, strengthening of the existing walls and foundation would be limited if required at all. Alternately, a new concrete dome or flat roof could be designed and installed.

If the existing dome roof cannot be removed, a retrofit bracket similar to as shown bellow could be installed. This type of bracket is configured to allow for thermal expansion of the roof while restricting lateral movement due to a seismic event. This type of connection would not impart additional operating or thermal loads on the walls. In a seismic event the brackets would "catch" the roof limiting its movement and transferring its lateral load into the walls. This option could be more difficult to implement (versus an aluminum dome roof) as it requires an elastomeric bearing pad to be placed between the roof and the top of the wall. Lifting the roof to install such a pad might not be practical. However, this type of retrofit would allow the current roof to remain without it being demolished. Alternately, retrofit brackets could be installed and the bearing pad omitted with the knowledge that in a seismic event the top of the wall connection could be significantly damaged, but the brackets would retain the dome and help prevent a complete failure of the roof.



Thermal Expansion



Seismic Loads

Roof-to-Wall Retrofit Brackets

General Recommendations

PSE recommends the exterior wall be cleaned to remove all concrete which has been damaged due to the thermal expansion effects. As thermal movement is likely to continue, PSE does not recommend stiffening or reinforcing this area. By constraining the roof, the failure zone could be moved and potentially cause issues within the dome roof itself, if it is constrained against expansion. Rather, the cracking around the exterior should be cleaned and coated to protect any reinforcing against water infiltration and to prevent further damage to the concrete. Coatings and any repair medium should be flexible to prevent cracking during thermal movement. This is not a long-term fix but intended to limit the impact of water and corrosion on this area until a new roof or roof-to-wall retrofit solution is selected. Once the area is cleaned and any damaged concrete removed it is recommended that it be observed by a Structural Engineer to review the extent of damaged concrete and to determine if any additional or alternate repairs are advisable at that time.

The roof itself should be cleaned and all debris and dirt removed. Where the roof coating is damaged, it should be repaired. Where trees are in close proximity to the reservoir, it is recommended that they be cut back to limit leaf litter build-up on the roof. Ideally trees should be trimmed and/or cut so that they are no closer than their height (i.e. a 10-foot tall tree should be no closer than 10-feet to the reservoir). This will help limit any impact of the tree's root system and the potential for falling trees or branches to damage the vents and hatches or impacting the roof dome in a large storm event.

Finally, the valve vault piping should be retrofitted to ensure the piping has flexible fittings which can allow for differential horizontal and lateral movement between the vault and reservoir in a seismic event. Without appropriate flexible fitting piping could be damaged during a seismic event. As the structure is in close proximity to the reservoir's foundation, which is cast as part of the vault's wall, there is a potential for differential settlement or movement at this interface. As part of general maintenance, the vault should be cleaned and where it is attached to the reservoir, those joints should be sealed so as to prevent leakage. The lower level of the vault should be cleaned, and the leak location identified and sealed. Notches or overflow scuppers should be installed in the parapet to prevent the roof from overflowing if the drain backs-up.

2.6 Scans of Select Construction Documents

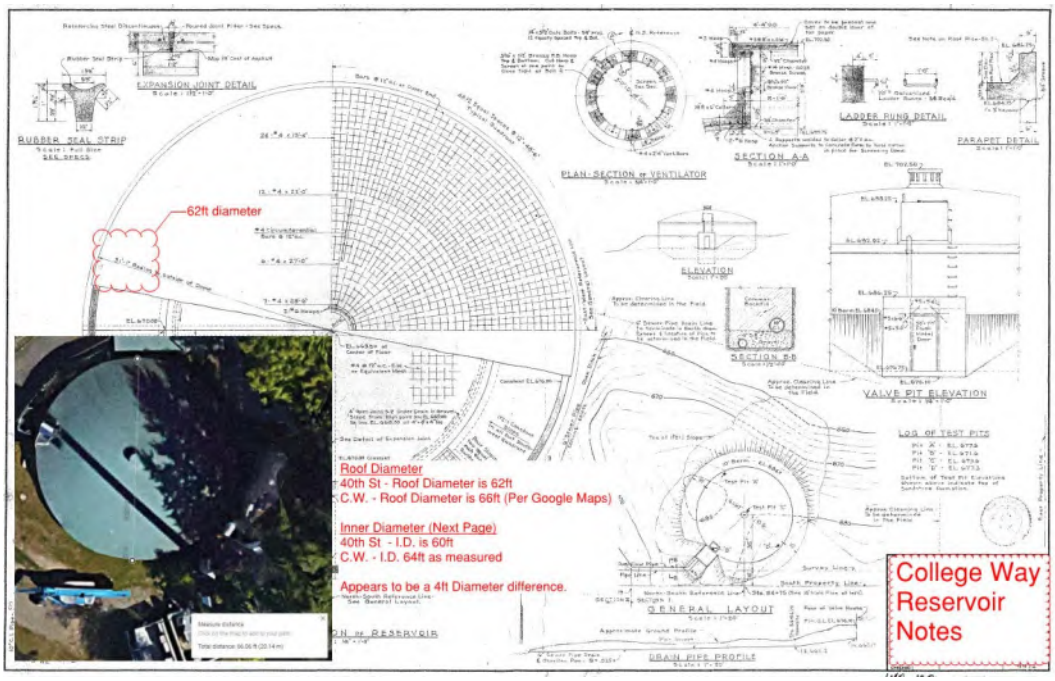


Figure 2-1: College Way Reservoir Plans and Elevations (Drawings are based on 40th St)

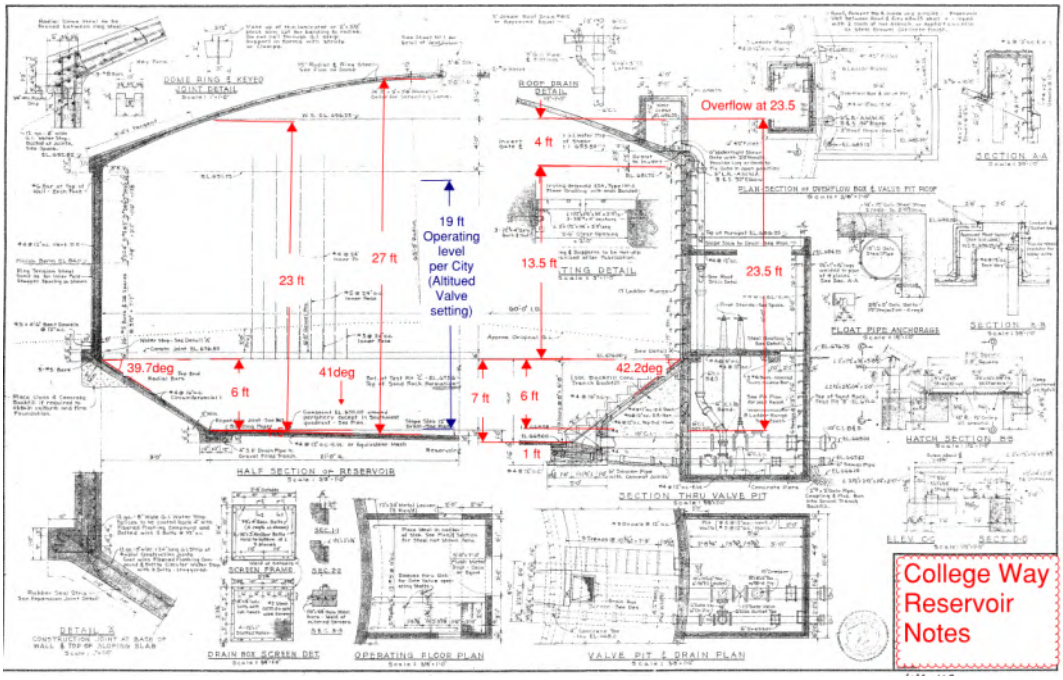


Figure 2-2: College Way Reservoir Sections and Details (Drawings are based on 40th St)

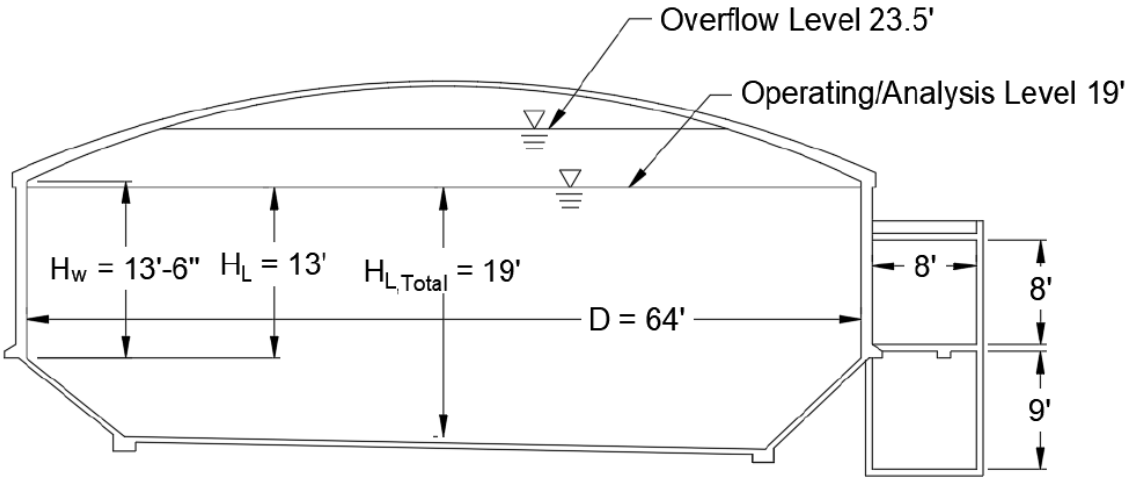


Figure 2-3: College Way Reservoir Elevations Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, $H_{L,Total}$ = Total Operating Water Height relative to Base)

2.7 Observations Pictures

Figure 2-4: College Way Reservoir – Elevation

Figure 2-5: College Way Reservoir – Entry to Valve Vault

Figure 2-6: College Way Reservoir – Circumferential Cracking around Reservoir below Roof Line

Figure 2-7: College Way Reservoir – Circumferential Cracking at Valve Vault

Figure 2-8: College Way Reservoir – Dome Roof Vent

Figure 2-9: College Way Reservoir – Circumferential Cracking around Dome Roof Edge

Figure 2-10: College Way Reservoir – Steps Cast into Hopper Base and Outlet

Figure 2-11: College Way Reservoir – Dome Roof Interior Side of Vent

Figure 2-12: College Way Reservoir – Corrosion on Recirculation Pipe

Figure 2-13: College Way Reservoir – Inlet Pipe in Hopper Base

Figure 2-14: College Way Reservoir – Access Hatch

Figure 2-15: College Way Reservoir – Overflow Weir Located in Access Hatch

2.8 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Inspection

PROJECT NUMBER: A1802-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: College Way - 231 Highland Dr.

Site Visit Date: 3/14/19 Reservoir Type: 0.5 MG

Temperature and weather: 40°F Dry & Clear

Site Conditions: Dry

PSE Staff: Greg

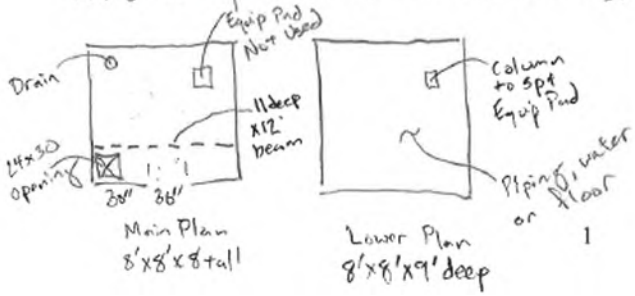
Client/Other Staff: Nathan, Corey - Murraysmith; Jeremy NW Corrosion

Pump Station Notes: (N.I.S.)

Newer construction, partially below grade. Conc. walls w/ metal sheet roof. Block pattern on walls. Over all condition looks very good. No noticeable cracks or settlement issues. All equipment look well-founded, dry, with no issues at anchorage.

Vault Notes:

Concrete vault, cast with reservoir creation a stiff point. Some cracking notes at top of vault at roof-wall-res. Inter face. Floor 6" slab.



Leak appears to be from piping rather than slab.

Roof Has 1' parapet

Res. overflow drains onto roof

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to Top of Roof Slab: 1 ~~(N)~~ 12.5' 2 ~~(E)~~ 8' 3 ~~(S)~~ 2.5' 4 ~~(W)~~ 3'

Roof Slab Thickness: 4½" / (drawings/measured) Roof Overhang Dimension: 3" / 3½" (drawings/measured)

Drip Groove? (Y/N): Y / N (drawings/measured)

Top Surface Roof Slab Condition: Gen. Good. Areas of coating delam. esp. along back where trees located. Some circumferential cracking adjacent to hatch

Ladder/Vents/Hatch/Joint Conditions: Ext. ladder no longer used, new ladder (Alum.) w/ landing used.

Other Comments: Wall appears to be cracking at dome-wall interface due to (pot.) temp. shrinkage. Backend, where wall const. below roof, above vault (a stiff pt) cracking at a diag.

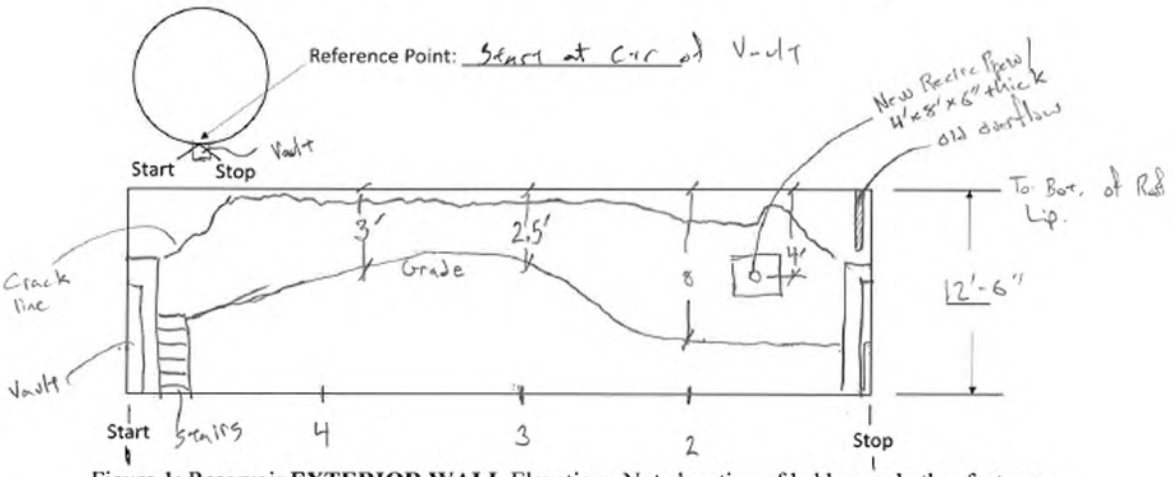


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

Crack run around exterior below roof line, only deviating at entry vault

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

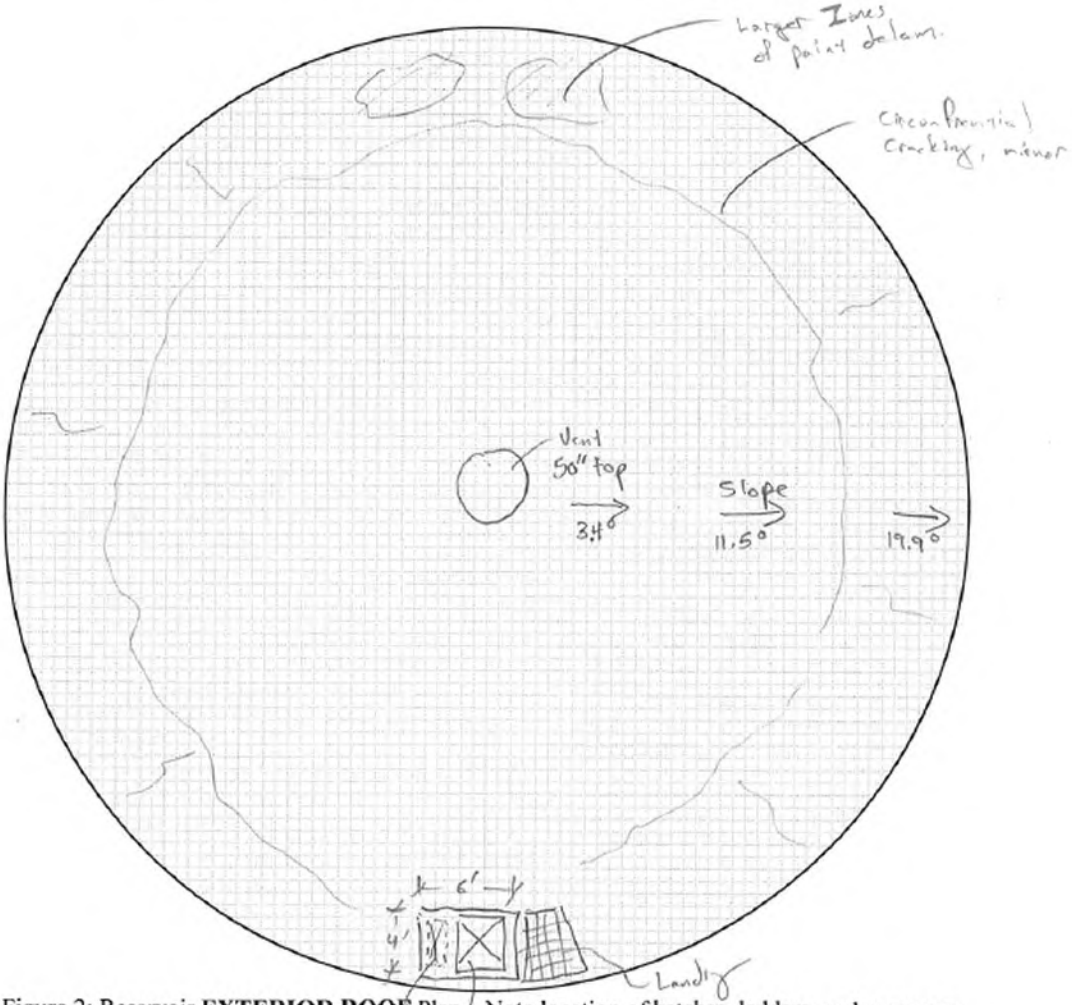


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

Entry 3'x3'

overflow weir at 20'

3

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: No real cracks noted from ground

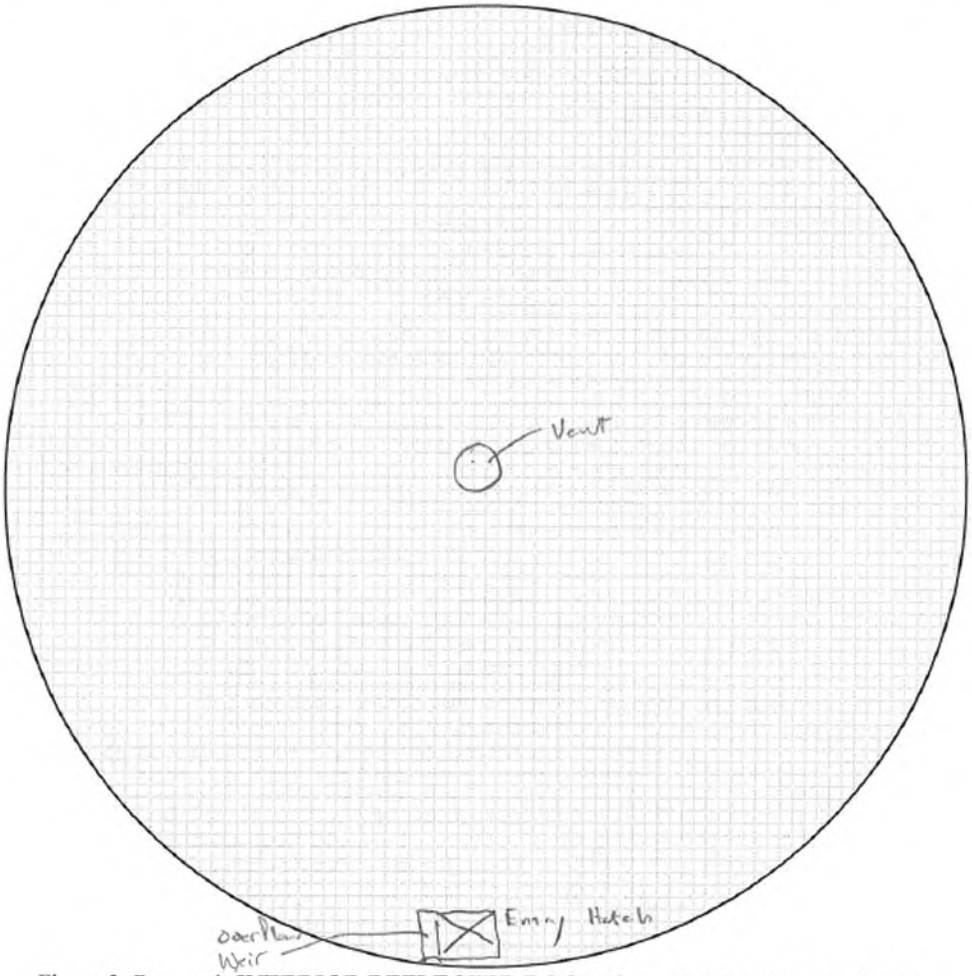


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Column Diameter: N/A (drawings/measured) Footing Size/Thickness: N/A (drawings/measured)

Column Spacing: N/A (drawings/measured) Wall Curb Dimensions: N/A (drawings/measured)

Floor Slab Condition: Good, coating and condition in good shape no cracks noted

Floor Slab Joints Spacing/Condition: No joints

Column/Footing Conditions: N/A

5

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PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

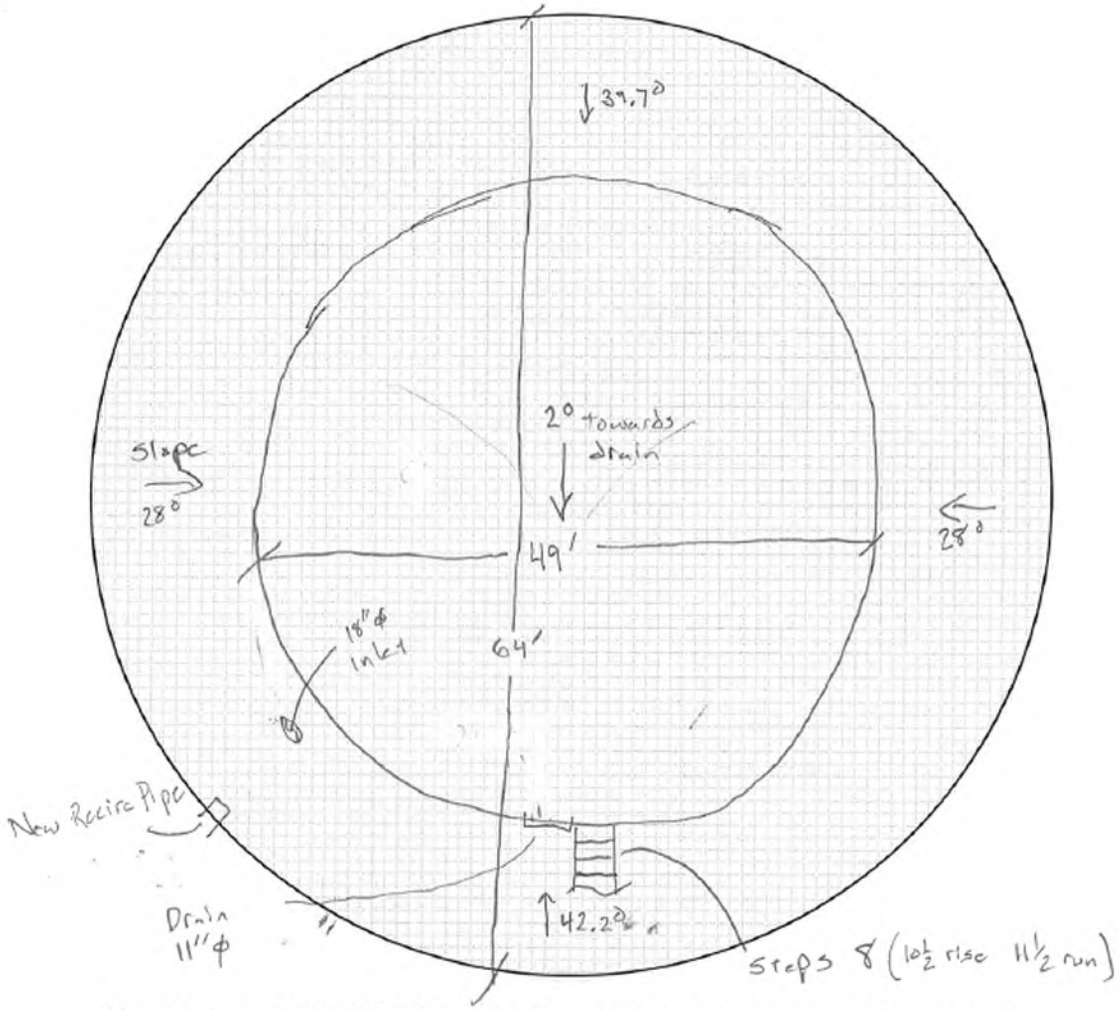


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Overall good, no major issue

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth):

Minor noted, coat probably cover and issues no

# of wall sections: N/A - no joints noted

Ladder/Pipes/Overflow Conditions:

Overflow Height: / (drawings/measured) Operating Height: 14' min (per City/PUD/other)

18.5' stnd

19' Max.

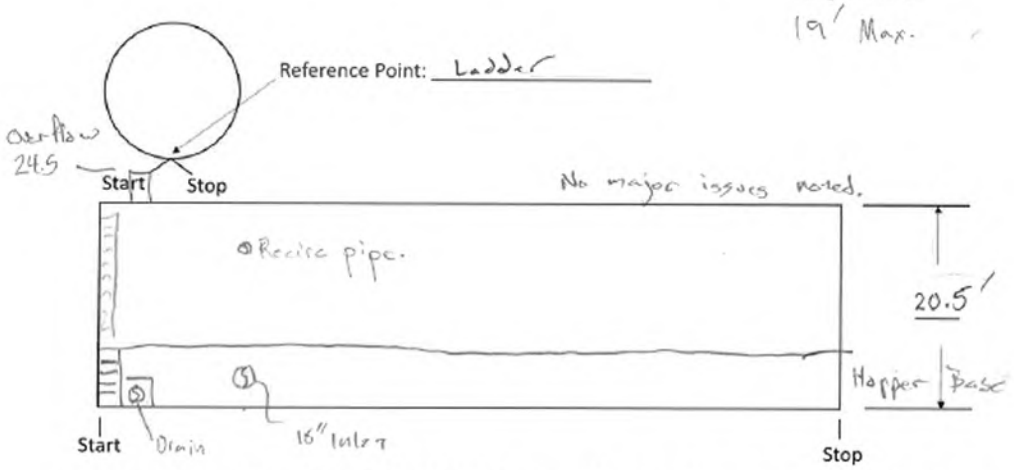
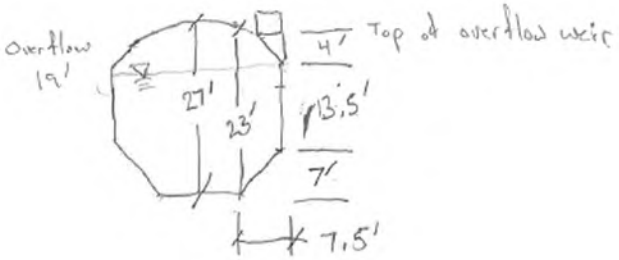


Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.



7

END OF SECTION

Appendix H-4 College Way General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

College Way Reservoir *General Info*

Field Visit Date: 3/14/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	3/14/2019
Reservoir Name and Location:	College Way - 231 Highland Drive (near W College Way)
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis, Jeremy Hailey
Client Staff Present:	Shayla Francis, Steve Bradshaw, Alex
Year Constructed:	1968
Overflow Destination:	Storm sewer in Highland Drive
Discharge Destination/Zone:	457 South Zone (College Way Pump Station to 541 Zone)
Fill Location:	457 South Zone
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	64	ft
Height	19.5	ft
Overflow Elevation:	541	ft AMSL
Bottom Elevation:	517.5	ft AMSL
Level of Overflow	23.5	ft
Minimum Normal Operating Level:	14	ft
Maximum Normal Operating Level:	19	ft

Notes: No specific as-built plans for reservoir, so used 40th St reservoir plans as guide for assessment (similar construction, but different overall dimensions). Heights are 19.5 ft to the knuckle and 27 ft to the top of the dome.

College Way Reservoir *Exterior Inspection*

Field Visit Date: 3/14/2019

Exterior Ladder:	
Present on site?	Yes
Material:	Aluminum
Condition:	Very Good
Corrosion:	No
Cage:	No
Security Type:	None
Security Condition:	Good
Wall Attachment Type:	Set into roof
Wall Attachment Condition:	Very Good
Rung Shape:	Circular

Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	12	in
Front Clearance:	24	in
Back Clearance:	N/A	in
Notes: Side clearance is to vault.		

Exterior Fall Prevention System:	
Present at Site:	No

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:	
Hatch Location:	Roof
Material:	Aluminum
Condition:	Good
Gasketed:	Yes
Intrusion Alarm:	Yes
Lock:	Yes
Frame Drain Location:	N/A

Measurement Type	Measurement	Unit
Size:	3x3	ft
Curb Height:	4	in
Notes:		

Roof Vents and Screen:		
Material:	Concrete	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	Unknown	in
Notes: Unable to access roof vent screen; sheet metal placed around it.		

Roof:		
Condition:	Fair	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	None	
Slope of roof	Varies (3.4, 11.5, 19.9 degrees)	
Measurement Type	Measurement	Unit
Overhang Distance:	4	in
Thickness of roof slab	Varies (4.5 to 12)	in
Notes: Slab widens near the walls, so unable determine exact thickness. Some circumferential and radial cracking.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Fair	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	N/A	in
Top Height:	41	in
Notes: Mid rail at 20 in. Railing only near access vault, not around entire perimeter of reservoir.		

Grating:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Clips:	No	
Removable Panels:	No	
Measurement Type	Measurement	**Unit**
Approximate Panel Dimensions:	2X5	ft
Notes: Grating located by the entry hatch.		

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Fair
Notes: Wall cracks run exterior below roof line.	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	Unknown
Exterior Piping:	No Coating
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	8-14 mils
Exterior Coating Adhesion Testing Results:	See notes.
Notes: Exterior coating is peeling off in several areas, ~15% of top coating has delaminated exposing underling coating which appears tightly adhered to the concrete surface. Roof's coating in poor condition.	

College Way Reservoir *Interior Inspection*

Field Visit Date: 3/14/2019

Interior Ladder:	
Present at Site:	No
Notes: Used removable ladder	

Interior Fall Prevention System:	
Present at Site:	No

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
N/A	N/A	
Notes:		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Notes:	

Walls:	
Condition:	Good
Painters Rings Present:	No
Notes: Some chipping near inlet/outlet.	

Interior Coating	
Interior Walls:	Polyurea
Interior Floor:	Polyurea
Interior of Roof:	No Coating
Interior Ladder:	N/A
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	80 - 110 mils
Interior Coating Adhesion Testing Results:	Well-adhered
Notes: Interior coating applied in January 2013 is in very good condition.	

College Way Reservoir *Miscellaneous*

Field Visit Date: 3/14/2019

Piping		
Inlet Piping:	Size (Inches OD):	10
	Condition:	Good
	Material:	Cast Iron
	Notes: Shared inlet and drain	
Outlet Piping:	Size (inches OD):	18
	Condition:	Good
	Material:	Ductile Iron
	Lip (Inches)	45
	Notes: Retrofitted outlet to pump station	
Overflow/Circulation Piping:	Size (inches OD):	8
	Condition:	Poor
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	Storm sewer
	Erosion Evident:	No
	Screen Condition:	Good
	Overflow to roof (feet)	0
	Notes: The exterior overflow pipe and interior circulation pipe have substantial corrosion.	
Drain Piping:	Size (inches OD):	12
	Condition:	Good
	Outlet Location:	Storm sewer
	Screened:	Yes
	Material:	Cast Iron
	Silt Stop Type:	None
	Air Gap:	Yes
	Screen Condition:	Good
	Notes: Did not note dechlorination system during inspection	

Piping Facilities		
Exterior Valving:	Type:	Gate Valves
	Condition:	Fair
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	N/A
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	No
Notes: No washdown facilities noted during inspection		

Electrical	
Cathodic Protection:	No
Impressed Current:	No
Anodes:	No
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	Yes
Common Inlet/Outlet:	No
Manual Level Indicator:	N/A
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	0.1
Water Quality Sample Taps?	Yes
Notes: Sediment removed by City personnel during inspection.	

Appendix H-5 College Way Condition Assessment Score Sheet

College Way Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No Camera
	Vegetation Separation	0	0	0	0	0	0	1	0	Under the dripline of trees. Organic debris on roof.
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	1	2	2	2	0	0	3	0	Roof not designed for thermal movement; larger cracks adjacent to vault. Coating failing. Water could seep
	Interior Walls	5	4	3	3	5	0	5	0	
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	4	4	5	5	5	0	5	0	Some sediment accumulation
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	3	4	5	3	5	0	5	0	Textured coating loss and OM accumulation
	Interior Roof and Supports	0	4	5	3	0	0	0	0	Roof uncoated
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	5	5	0	No fall protection required
	Interior Ladders/Fall Protection	0	0	0	0	0	3	3	0	No fall protection required, but would be good to have. Hard to use interior ladder
	Access Hatches	5	4	0	0	4	0	3	0	Difficult to get into hatch. High maintenance design.
	Railings and Roof Fall Protection	5	5	0	0	0	2	0	0	10-foot drop and highly sloped
	Vents	4	4	0	0	3	0	2	0	16-inch inlet pipe causes failure. Screen likely too coarse.
	Balconies/Landings/Grating	5	5	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	4	4	0	5	5	0	5	0	Need to check which pipe is inlet. And if new pipe from PS has flex coupling
	Outlet Piping	0	0	0	5	5	0	5	0	Need to check which pipe is inlet. And if new pipe from PS has flex coupling
	Drain Piping	4	4	0	3	3	0	3	0	No silt stop (May not be needed on retrofit) No flex coupling. Dechlorination system unknown.
	Overflow Piping	3	2	0	0	4	0	5	0	Major corrosion an metal part of weir gate and exterior pipe above air gap
	Washdown Piping	0	0	0	0	0	0	0	0	Unknown washdown piping
	Attached Valve Vault Structure	4	3	4	3	0	0	4	0	Bottom of vault has 1ft of water. Roof drainage needs improvement
	Control Valving	0	0	0	0	0	0	5	5	Need to check which pipe is inlet and what is in use
	Isolation Valving	0	0	0	0	0	0	5	5	Need to check which pipe is inlet and what is in use
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	5	0	5	0	
	Hydraulic Mixing System	1	1	0	0	5	0	4	0	Very corroded
	Categorical Score	3.8	3.7	4.1	3.7	4.5	3.3	4.1	5.0	

Overall Score
3.9

Appendix I Consolidation

Appendix I-1 Consolidation Geotechnical Report

GeoEngineers

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

Memorandum

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Consolidation Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Consolidation reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the Consolidation site, located as shown in the Vicinity Map, Figure 1. The Consolidation reservoir is a 496,000 gallon round reinforced concrete structure with a hopper base, built in 1959.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by undifferentiated glacial deposits. Sandstone of the Chuckanut Formation is mapped nearby.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift. Based on previous experience in the area, Bellingham (glaciomarine) Drift overlies bedrock in this area. The Bellingham Drift is a glaciomarine drift deposit which consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper 5 to 15 feet of this unit in upland areas is typically stiff. The stiff layer possesses relatively high shear strength and low compressibility characteristics. The stiff layer oftentimes grades to medium stiff or even soft, gray, clayey silt or clay with depth. The entire profile can stiff, likely from being partially glacially overridden, when it is a shallow profile over bedrock. The soft to medium stiff glaciomarine drift possesses relatively low shear strength and moderate to high compressibility characteristics.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone. The character of the bedrock at the site is known to vary considerably over short distances.

Surface Conditions

The project site is located northwest of Yew street and San Juan Boulevard. The reservoir is in a small field that dips slightly downward to the northeast. The site is bounded by a wooded area to the north and west and roads to the south and east. A small gravel roadway leads to the reservoir from the east. The reservoir is buried to the lid on the south side along San Juan Boulevard and then more exposed on the northern side with driveway access to the pump station.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-6 (2019)—on March 26, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 15½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan, Figure 2. The boring was located approximately 30 feet east of the reservoir based on utilities and the slope adjacent to the reservoir. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites.

- **Fill** – Below approximately ½ foot of topsoil, fill was encountered to approximately 4½ feet bgs. The fill consisted of stiff brown sandy silt with gravel.
- **Glaciomarine Drift** –Glaciomarine drift extended from 5 to 12.5½ feet bgs. The glaciomarine drift consists of stiff brown silt with sand and occasional gravel grading to dense silty sand.
- **Chuckanut Sandstone** – Chuckanut sandstone was encountered at 12½ feet bgs. The boring was completed due to refusal at 15½ feet. The bedrock consists of gray sandstone.

Groundwater

Groundwater seepage was not observed at final depth of the boring. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Our boring was drilled approximately 30 feet east of the buried reservoir and encountered fill and overburden soils. Based on review of the as-built drawings, the reservoir is 22 feet tall; we estimate the boring was completed approximately 7 feet below the top of the reservoir and bedrock was encountered at 12½ feet bgs in B-6. Therefore, we anticipate that the reservoir is founded directly on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our boring and review of the as-built drawings, the existing structure bears on bedrock which is not at risk of liquefaction.

American Concrete Institute/American Society of Civil Engineers 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	95.6
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.5
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.43
MCE_G peak ground acceleration, PGA	0.395
Seismic design value, S_{DS}	0.648
Seismic design value, S_{D1}	0.356

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the M 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	12	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

M_W 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the

North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	18	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.36	0.65	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring, and review of as-built sketches that indicate the reservoir is 22 to 24 feet tall, we anticipate that the existing reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic foot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The reservoir includes below grade walls. Our recommendations for concrete below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the "Shallow Foundations" section the wall backfill consists of structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

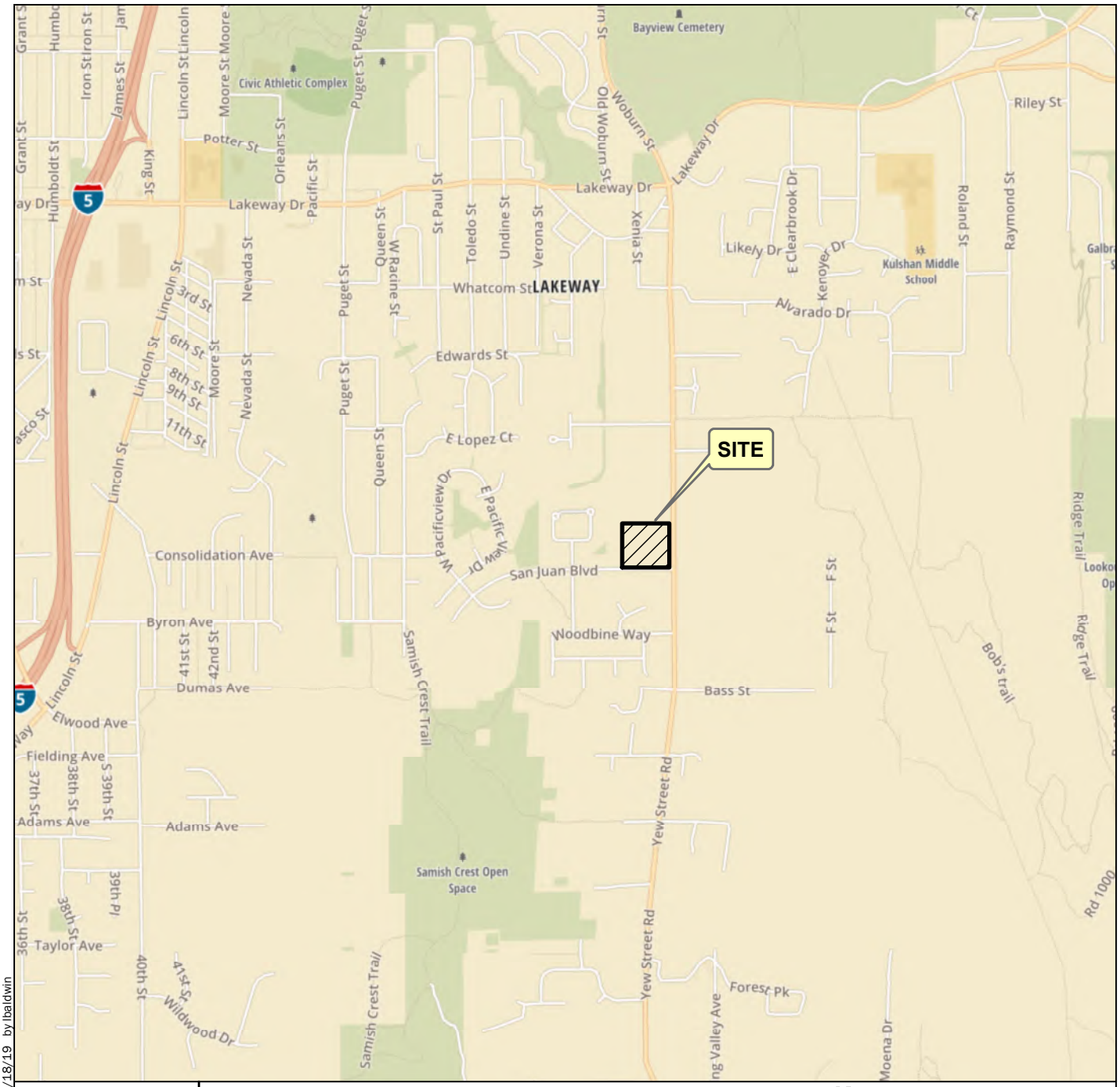
Figure 2 – Site Plan

Figure 3 – Key to Exploration Logs

Figure 4 – Log of boring B-6

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault



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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Consolidation Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_Consolidation.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

75 0 75

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Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Consolidation Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled Start 3/26/2019 End 3/26/2019	Total Depth (ft) 15.25	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) 510 Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment EC-95	
Easting (X) 1252040 Northing (Y) 637000		System Datum WA State Plane North NAD83 (feet)		Groundwater not observed at time of exploration	
Notes:					

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	6 inches topsoil			
								ML	Brown sandy silt with occasional gravel (stiff, moist to wet) (fill)			
		X	18	11		1 MC				28		
505	5	X	15	12		2 MC		ML	Brown silt with sand and occasional gravel; iron staining (stiff, moist) (glaciomarine drift)	23		
		X	18	16		3 %F		SM	Brown silty fine to coarse sand (dense, moist)	20	29	
500	10	X	18	25		4						
								Sandstone	Gray sandstone (Chuckanut Formation)			
495	15	X	3	50/3"		5						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-6

GeoEngineers

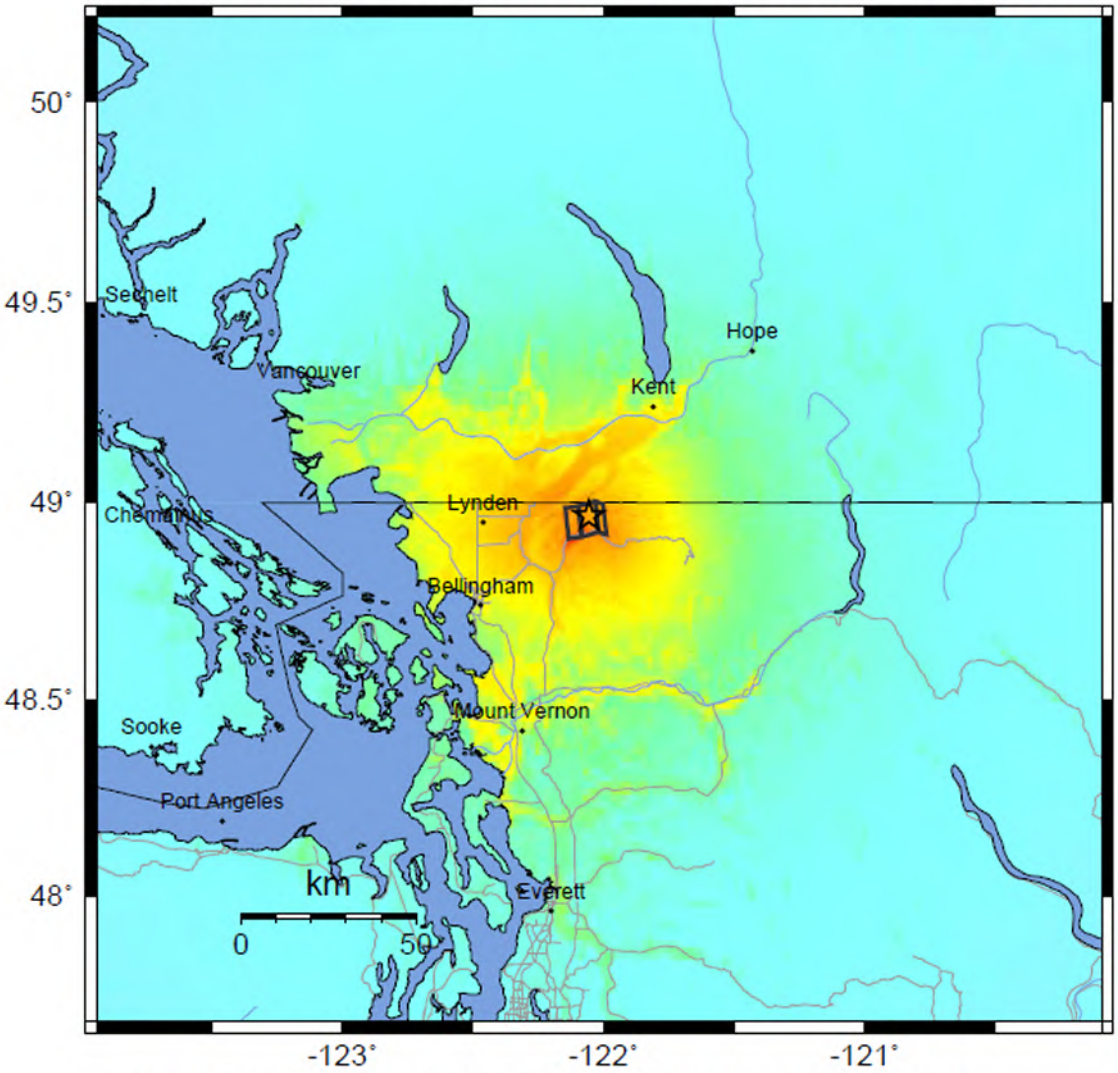
Project: COB Reservoir Inspection and Repair - Consolidation Avenue
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

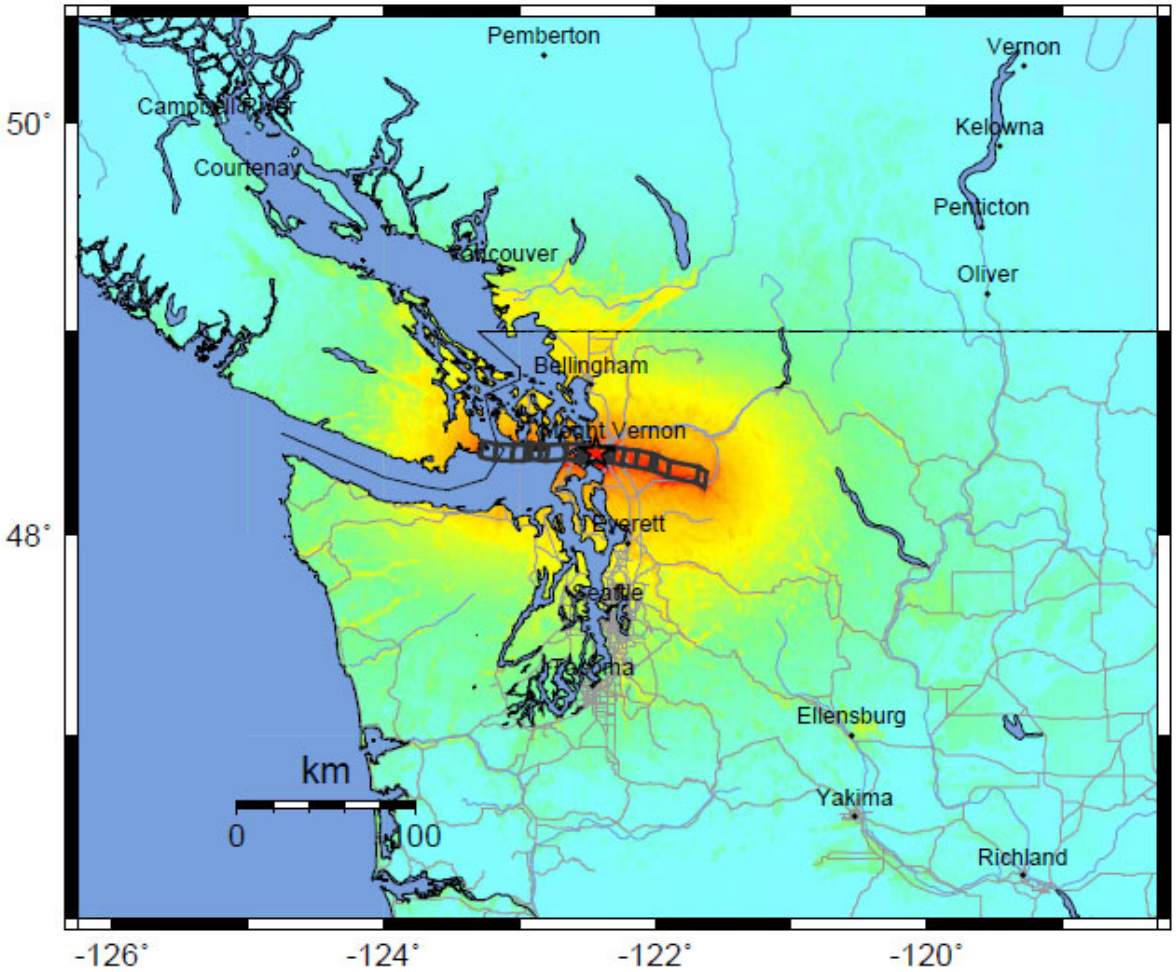
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix I-2 Consolidation Structural Report

CITY OF BELLINGHAM

CH 11: CONSOLIDATION RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessment & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Consolidation, 0.5 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located at 2594 Yew St, Bellingham, WA (Lat. 48.735, Long. -122.443), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on November 7th, 2019 by Peterson Structural Engineers (PSE), and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 Consolidation Round Reinforced Concrete (RC) Reservoir – 0.5 MG

2.1 Description & Background

Per information provided by the City, the Consolidation reservoir was likely designed by John W. Cunningham & Associates Consulting Engineers and built in 1959. Drawings were not available for the reservoir, but it is similar in design to other reservoirs in the City's inventory with a storage volume of around 0.5MG, a dome roof, and a hopper base. For similar reservoirs types, which do have available drawings, Cunningham & Associates is the designer of record. The reservoir itself is a round reinforced concrete reservoir with a measured interior diameter of 64-feet. The interior wall was measured to be 11.25-feet high while the hopper base is 7-feet deep. The overflow weir is located approximately 23.0-feet above the bottom of the reservoir and is located above the top of the wall within the access hatch box. The reservoir, at full capacity, uses a portion of the dome roof for its storage volume. Per measurements taken, the overflow volume was determined to be 476,000 gallons while the operating volume was estimated to be 421,000 gallons.

Where details or sections could not be directly observed or measured, they have been assumed to be comparable to other similarly sized reservoirs designed by Cunningham & Associates built in the same era. Per those reference drawings it is assumed that the wall is approximately 10-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. The roof is likely a reinforced 4.5-inch thick dome with an edge that thickens to 1'-2". The floor is likely a reinforced 5-inch thick slab. The footing is likely a reinforced 12-inch thick by 10-inch wide footing that transitions into the hopper base. Finally, where piping is run under the footing, it is assumed to be encased in an unreinforced concrete block for protection. Drawings of a similar reservoir configuration designed by Cunningham & Associates and built in the same era have been included in Figure 2-1 & Figure 2-2 for reference while a schematic drawing of the reservoir can be found in Figure 2-3.

2.1.1 Description of Additional Site Structures and Features

The site includes a valve vault which was constructed as part of the reservoir. The rear of the vault shares a wall and footing with the reservoir. This vault is located on the north side of the reservoir and is two levels, with one level located below grade. The main and lower vault levels are 8-feet square and the main level has an 8-foot clearance while the lower level has an 11-foot clearance. The lower level is accessed via a 24 by 30-inch opening in the main level's 6-inch thick slab floor. The reservoir's drain, outlet, and overflow are all run through the valve vault. The roof of the vault has a parapet and the air gap for the reservoir overflow is located above the roof. If debris were to clog the overflow, the layout of the roof of the valve vault is such that it would fill with water up to the height of the parapet. While the roof had a secondary drain, it is insufficiently sized to handle any overflow volume. Additionally, as the overflow and drain pipes are joined below the roof line, any material or issue that blocks the overflow would also be likely to clog the drain as well.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit on November 7th, 2019 to observe the as-built current condition of the reservoir's interior and exterior as well as the site conditions. The reservoir was drained for our inspection.

Dome Roof: The reservoir has a self-supporting concrete dome roof with a thickened edge. The surface of the roof is coated. The roof coating was in poor condition with instances of heavy wear and multiple locations of failure of the coating on the roof surface and its sides. Additional coating failure zones were found along radial and circumferential crack lines. Additional cracking may be present that is not visible due to the applied coating. Structurally the roof appears to be currently adequate despite the extensive cracking. The radial cracking extends up the roof side to intersect a series circumferential cracks that run around the entire reservoir roof. The circumferential cracks are located approximately 6-feet up the side of the roof. Per the assumed reference drawings, it is assumed that the roof dome radial reinforcing fans out along the circumference of the roof with additional reinforcing added as the width between the fanned bars increases. This cracking appears to be occurring roughly along the line where the fanning of the reinforcing begins and may be occurring at the observed location due a stiffness differential in the roof.

The roof has one 3 by 3-foot access hatch that is part of a larger access box. This box is formed to include an overflow weir. The location of the overflow is above the top of the wall, necessitating a waterstop in both the roof-to-wall joint and access hatch-to-roof joint. No major issues were noted within or around the hatch, which appears to be in good condition and working order. The box's coating was found to be in generally good condition, although the roof coating adjacent to the overflow pipe was failing.

At the center of the roof is a 36-inch diameter vent. Per the assumed reference drawings and photos, the vent should include a series of (12) 4-inch by 11-inch openings around its exterior. However, these openings and the structure itself were obscured by a sheet-metal cover. Minor radial racking was noted around the vent. The vent supports, which consist of short metal legs, where found to be bearing on and damaging the roof coating. Additionally, the tightness of the metal cover itself could pose issues with the venting of the reservoir and Murraysmith should be consulted to determine if a problem exists due to restricted airflow. Inadequate venting can create a structural overload condition when the reservoir is filled or drained if the venting cannot keep up with a negative pressure load resulting from a rapid change in storage volume.

Reservoir Walls and Interior: Per the assumed referenced drawings, the walls are likely 10-inches thick with vertical and horizontal (hoop) reinforcing. At the top of the wall, it is assumed there is a keyway and vertical reinforcing bars each face of the wall connecting the roof to the wall. This connection is required as the design of the reservoir allows for water to be stored above the top of the wall and within the dome resulting in a restraint that that does not accommodate thermal expansion of the roof. As a temperature change occurs, the roof will expand or contract radially while the wall expands or contracts vertically

A likely result of this constrained differential movement is the observed cracking which runs around the exterior of the reservoir's wall, just below the base of the dome roof edge. This cracking is likely a result of thermal expansion pushing outward on the top of wall and failing the joint along the thinner section of its joint key. Unlike the other reservoirs of a similar type being evaluating for this project, this reservoir is almost fully backfilled. This backfill condition further restrains the wall against movement. In this case, as the roof tries to expand outwards, it is constrained by the backfill. This constraint likely forces the roof to deflect upwards as it is limit by how far it an expand radially outwards. This could be why the radial

cracking observed in this reservoir's roof is much more pronounces than in ones with less backfill constraint.

The interior of the reservoir was found to be in acceptable visual condition for the structural elements. The floor slab and hopper sides of the reservoir appeared competent with no observed issues indicative of differential settlement. Overall the walls were found to be competent with only one large crack noted. The observed crack is a 10 to 15-foot long horizontal wall crack and was observed to be opposite the entry hatch (located south, near the street). The cracking on the interior of the dome roof mirrored the radial and circumferential cracking observed on the exterior. However, no major issues with interior effloresce or other modes of failure were noted in either the roof or along the wall crack.

Appurtenances: The inlet/outlet and overflow weir all appeared to be in serviceable condition with the inlet/outlet pipe noted to have a numerous amount of small corrosion carbuncles. This pipe requires cleaning and/or maintenance to remove said corrosion.

2.2.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appeared to be in generally good visual condition with instances of coating loss around the exterior. Diagonal cracking was noted along the east sidewall of the vault but did not appear to be a major structural concern. The overflow and roof drainpipes were noted to be covered in mineralization and the joint around them likely needs to be re-sealed. Based on the reference drawings it is assumed that the vault was poured using a keyed joint and reinforcing to connect it to the reservoir wall. However, there is no indication of the presence of a waterstop or more watertight joint type in the drawings. The bottom of the vault was wet, but an existing sump pump appeared to be effective in keeping the lower level from filling with water. Pipes should be checked to determine if there are any leakage issues which are contributing to the observed water.

2.3 Structural Analysis

The following design analysis is based on the reservoir drawings associated with the 40th St Reservoir which is similar type of reservoir with a dome roof and hopper base. The City has a few of these types of reservoirs in its inventory including Dakin I (built in 1987), Reveille (built in 1958), College Way (built in 1968), and 40th St (built in 1959). Only the Reveille and 40th St reservoirs have drawings available and both sets of drawings were prepared by John W. Cunningham & Associates, Consulting Engineers. As the 40th St reservoir is of a similar size to Consolidation (0.5MG versus Reveille's 0.3MG), the 40th St drawings have been used to provide a baseline for PSE's analysis.

Where field measurements could be made, the actual dimensions were used in PSE's analysis. In this case the size and diameter of the reference reservoir varies somewhat with 40th St having a 60-foot diameter and a 15.5-foot wall height while Consolidation was found to have a 64-foot diameter and an 11.25-foot wall height. Where dimensions or information could not be obtained during the site visit, such as the reinforcing size, layout, and spacing, those values were based on the 40th St reservoir drawings. Based on the similarity of multiple versions of this reservoir, this assumption appears to be a reasonable and cost-effective method to determine a presumed level of adequacy of the reservoir. Based on the results of the analysis, issues and retrofit options are discussed. However, as these calculations are an approximation,

any assumptions made should be verified though additional destructive and non-destructing testing prior to undertaking any retrofit or repair work.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published in 2002, and "Circular Concrete Tanks without Prestressing", published in 1993 were also utilized.

2.3.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof was evaluated per concrete design criteria as covered in AWWA D110-13 and found to meet minimum requirements for rise-to-span, thickness versus buckling, and edge reinforcing assuming the reinforcing is consistent with the reference drawings. The dome roof was evaluated at a 19.5-foot operating level. At this level the water is stored above the top of the wall which results in some hydrostatic loads on the roof. This loading was within an acceptable range and should the reservoir be operated at overflow, analysis found that the assumed circumferential dome edge ring reinforcing was adequate for any additional hydrostatic loads.

As the reservoir site is unfenced, the local residents have access to the roof and, per discussion with the City, apparently use the roof as a sledding hill in the winter. Due to this usage, the reservoir roof was also checked for a concurrent snow and 40-psf live load. Based on PSE's checks, the roof should be adequate for these types of light incidental loadings. The radial and circumferential cracking observed in the roof is likely a result of thermal effects rather than any usage loads. However, this usage could be a reason the roof coating itself was in poor condition observed as minor cracking and coating failures can be exacerbated by residents utilizing the roof.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will allow for differential thermal movement between the reservoir roof and the wall as both components deform differently as a result of temperature variations due to temperature or solar gain. At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the components of the structure able to resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement. This is a result of its original design in which water is stored above the wall line. To adequately account for thermal movement the existing roof attachment would need to be modified or replaced with a roof system able to accommodate thermal movement. This type of retrofit may not be practical or economically feasible.

Wall Reinforcement: Per the assumed reference drawings the wall is likely reinforced with #4 vertical bars at 12-inches on center on the exterior face and #4 vertical bars at 24-inches on center on the interior face with #5 dowel bars at 12-inches on center along the base.. It is assumed that the horizontal (hoop) reinforcing starts out with #5 bars at 3.5-inches on center towards the base of the wall and the reinforcing

density decreases to #5 bars at 6.5-inches on center towards the top of the wall. At the very top of the wall there are assumed to be (2) #6 circumferential hoops. This variation in the hoop reinforcing is based on the variable pressure distribution resulting from the storage of liquids.

Per PSE's analysis, it was determined that the assumed vertical wall reinforcing appears to be sufficient for code-level strength requirements for the current operating level. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3 sanitary factor. This sanitary factor is intended to minimize the potential for cracking and leaks. The reservoir was found to be adequate for the current operating level the overflow level based on the assumed flexural (vertical) reinforcing.

Additional checks were performed for the wall at the 19.5-foot operating level. These checks included shear, tensile hoop forces, and compressive soil loads. Per PSE's analysis, the reservoir components have adequate capacity for the loads determined based on the current operating level. However, for higher operating levels the hoop tensile (horizontal) reinforcing capacity would be exceeded.

Finally, per ACI 350.3, the maximum spacing for wall reinforcing was checked. The maximum allowable spacing for bars is limited to 12-inches on center. In the lower section of the walls, the spacing of the interior and exterior reinforcing is assumed to be at 12-inches on center. Above the 6-feet the assumed interior spacing changes to 24-inches on center, exceeding the maximum limit. Cracking was noted in the upper section interior rear surface of the wall (opposite the entry hatch). Bar spacing might play a part in this observed cracking, however, the remainder of the interior wall appearing to be in good visual condition.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the geotechnical evaluation, the site's bearing capacity was determined to be 6,000-psf. Using this bearing capacity and checking for the 19.5-foot operating level up to overflow, the bearing pressure was determined to be within acceptable ranges.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: The addition of seismic loads results in additional loads on the wall of the structure. This is a result of the water slosh wave as well as forces resulting from mass movement of the structure itself. PSE found that the wall's flexural stresses were increased by about 20% when compared to the static loads at a 19.5-foot operating level. As the reservoir was designed for a higher static operating level, there is reserve capacity and the reservoir was found to be able to resist the increased seismic loads. Should the reservoir be operated at overflow, the seismic loads would exceed the wall's flexural capacity and the reservoir would need to be retrofitted.

The wall's hoop tensile stresses were found to be within acceptable limits when the reservoir was evaluated at the 19.5-foot operating level. However, there is limited reserve capacity to meet design requirements when operated at higher levels. For lateral seismic loads, the horizontal hoop reinforcing was determined to be exceeded by 2% for the 19.5-foot operating level. While this load is unlikely to warrant a retrofit at the current operating level, it is not recommended to operate the reservoir at higher levels.

In addition to the wall flexure and tensile checks, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing at the overflow level. No additional reinforcing or connectivity is needed between the walls and the foundation based on the assumed construction.

Freeboard/Slosh: At overflow, the reservoir stores water above the top of the wall. During a seismic event, if water were stored at the overflow operating level the roof would be required to constrain the slosh wave. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. This wave would have sufficient force to damage and potentially cause failure of the roof at the roof-to-wall interface, the dome itself, the hatches, and other appurtenances.

At the current operating level there is insufficient freeboard to accommodate the anticipated 2.5-foot slosh wave. However, the current operating level is such that it results in a lower slosh wave impact force than if it were operated at overflow. Therefore, at an operating level of 19.5-feet, the available roof reinforcing along with the roof's thickness and weight appears to be sufficient to resist the anticipated slosh load.

Valve Vault: Per the reference drawings it is assumed that the valve vault is connected to the reservoir with #5 rebar dowels at 12-inches on center. This attachment should limit differential movement or "pounding" that occurs in a seismic event. Additionally, where the hopper base and the lower level of the vault are adjacent, this zone is assumed to be backfilled with plain concrete or "trench backfill". In the event of an earthquake, this will provide support to the hopper base and limit its potential to fail the lower level wall and collapse onto the piping. Of primary concern is the vault's wall which is cast as part of the reservoir footing. Depending on the direction of ground motion, pipes should be retrofitted to have flexible coupling should differential movement between the two structures occur during a seismic event.

2.4 Summary

Based on the available assumed reference drawings and site visit it appears that a majority of the structural elements in the reservoir are either adequate or slightly overstressed for the expected loads at the current operating level. It was noted that ringing the reservoir there is cracking along the exterior of the wall and radial and circumferential cracking in the dome roof that could be a result of the combined thermal and operational loading conditions. Further, interior wall reinforcing was assumed to exceed maximum spacing allowances.

Elements outside of the wall, such as the dome roof and footing were determined to be adequate when operated at a 19.5-foot operating level. However, while the assumed reinforcing in these areas was determined to be adequate, the dome-to-roof reinforced connection is rigid and unable to resist expansion/contraction loads resulting from thermal effects. This has likely been the cause of damage noted around the reservoir at the roof-to-wall interface and the radial and circumferential cracking in the dome, observable on the interior and exterior. As a result of the circumferential wall cracking, the actual capacity at the dome/wall interface connection is expected to be lower than our analysis has determined. This could be a concern in a seismic event.

Remaining observable components of the reservoir appear to be in visually acceptable structural condition with primary defects being cracking and coating failure. The reservoir is also attached to a valve vault which appeared to be structurally sound, though some cracking was noted along the vault wall. Piping in the vault may also be a candidate to be upgraded to accommodate vertical and horizontal differential movement between the vault and the reservoir during a seismic event.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 19.5-feet, which is 1.25-feet above the top of the wall. Due to the types of issues noted, these retrofits might not be cost effective or easy to implement.

Wall Flexural Capacity

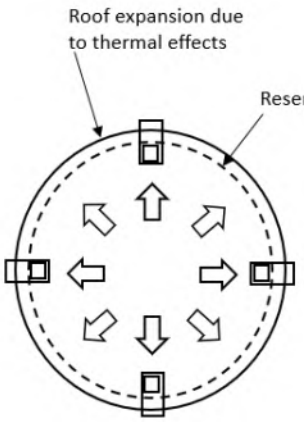
Based on a maximum operating level of 19.5-feet, PSE determined the wall's flexure capacity was adequate. However, these design checks are based on an assumed reinforcing layout. Should the reservoir be operated at a higher level or additional confirmation be required, PSE would recommend a testing firm be employed to map the existing reinforcing. Additionally, the testing firm could verify the concrete compressive strength and reinforcing's yield strength. Based on the gathered information, the reservoir can be re-evaluated to confirm the actual as-built design capacity.

Roof-to-Wall Retrofit

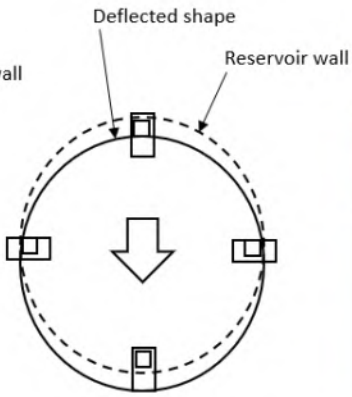
The effects of thermal movement appear to have damaged the roof-to-wall interface and has likely affected its structural shear capacity and water-tightness. To alleviate this issue, PSE would recommend either a retrofit or replacement of the roof. Note that this may not be economically feasible or practical.

The first option would be to remove the existing roof and replace it with a new roof. As the operating height is now above the wall height, this would require either lowering the operating level or increasing the wall height through the addition of a curb. By adding a curb, a new aluminum geodesic dome roof could be installed outside of the slosh impact zone. As an aluminum dome roof is relatively light, strengthening of the existing walls and foundation would be limited if required at all. Alternately, if the water level is reduced, a new concrete dome or flat roof could be designed and installed.

If the existing dome roof cannot be removed, then the water level could be reduced, and a retrofit bracket could be installed. See the photo and diagrams as follows. This type of bracket is configured to allow for thermal expansion of the roof while restricting lateral movement due to a seismic event. This type of connection would not impart additional operating or thermal loads on the walls. In a seismic event the brackets would "catch" the roof limiting its movement and transferring its lateral load into the walls. This option could be more difficult to implement (versus an aluminum dome roof) as it requires an elastomeric bearing pad to be placed between the roof and the top of the wall. Lifting the roof to install such a pad might not be practical. However, this type of retrofit would allow the current roof to remain without it being demolished. Alternately, retrofit brackets could be installed and the bearing pad omitted with the knowledge that in a seismic event the top of the wall connection could be significantly damaged, but the brackets would retain the dome and help prevent a complete failure of the roof.



Thermal Expansion



Seismic Loads

Roof-to-Wall Retrofit Brackets

General Recommendations

PSE recommends the exterior visible portion of the wall be cleaned to remove all concrete which has been damaged due to the thermal expansion effects. As thermal movement is likely to continue, PSE does not recommend stiffening or reinforcing this area as further damage could occur to the dome. By constraining the roof, the failure zone could be moved, potentially causing issues within the dome roof itself if it is constrained against expansion. Rather, the cracking around the exterior should be cleaned and coated to protect any reinforcing against water infiltration and to prevent further damage to the concrete. Coatings and any repair medium should be flexible to prevent cracking during thermal movement. This is not a long-term fix but intended to limit the impact of water and corrosion on this area until a new roof or roof-to-wall retrofit solution is selected. Once the area is cleaned and any damaged concrete removed it is recommended that it be observed by a Structural Engineer to review the extent of damaged concrete and to determine if any additional or alternate repairs are advisable at that time.

The roof itself should be cleaned and where the roof coating is damaged, it should be repaired. Cracking in the roof should be epoxy injected to seal the cracks and limit any potential for infiltration and further damage. Additionally, cracking noted on the upper section of the interior wall should be epoxy injected and the outside should be evaluated to confirm the extent of cracking. The site itself should be fenced to prevent wear and tear on the new roof coating.

Finally, the valve vault piping should be retrofitted to ensure the piping has flexible fittings which can allow for differential horizontal and lateral movement between the vault and reservoir in a seismic event. As the structure is in close proximity to the reservoir's foundation which is cast as part of the vault's wall, there is a potential for settlement or movement at this interface. Notches or overflow scuppers should be installed in the parapet to prevent the roof from overflowing if the drain backs-up.

2.6 Scans of Select Construction Documents

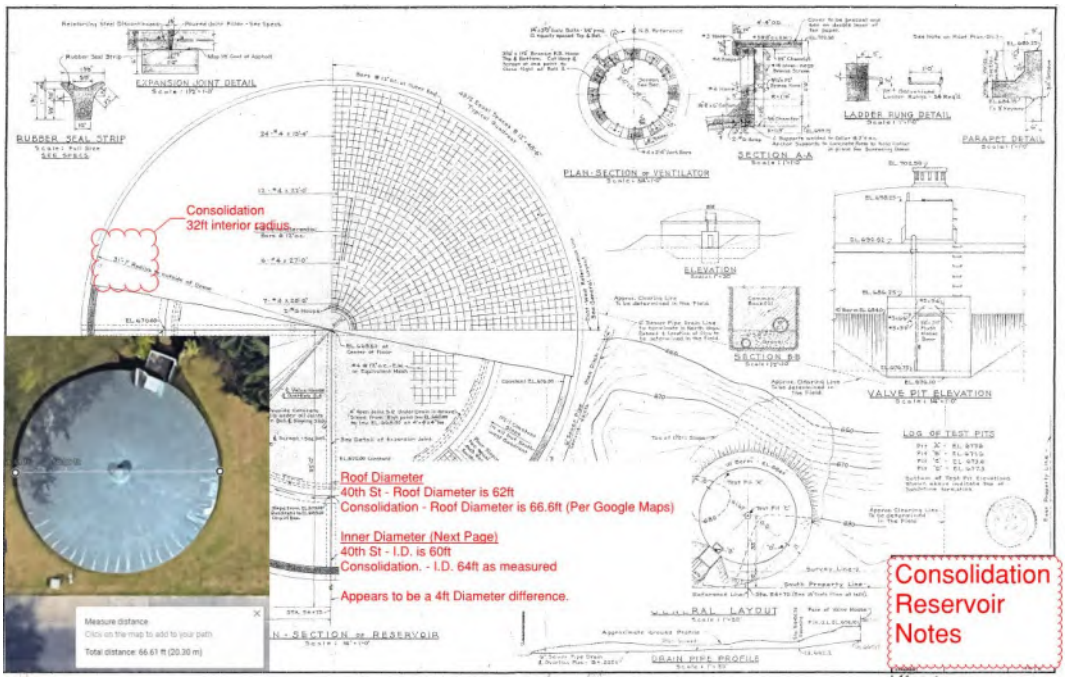


Figure 2-1: Consolidation Reservoir Plans and Elevations (Drawings are based on 40th St)

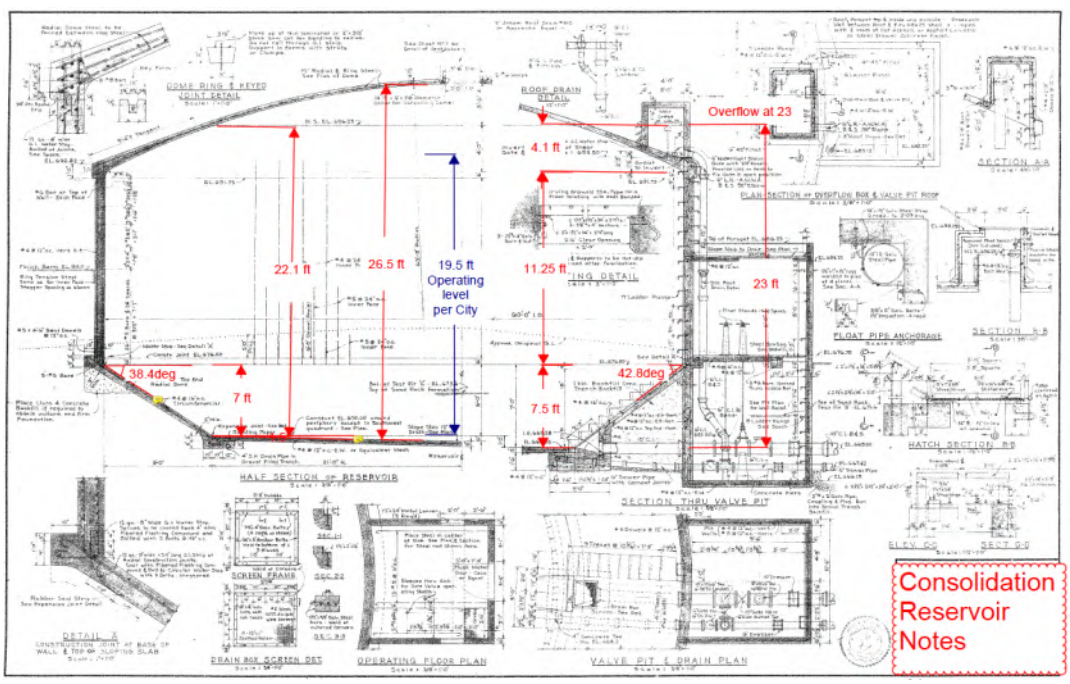


Figure 2-2: Consolidation Reservoir Sections and Details (Drawings are based on 40th St)

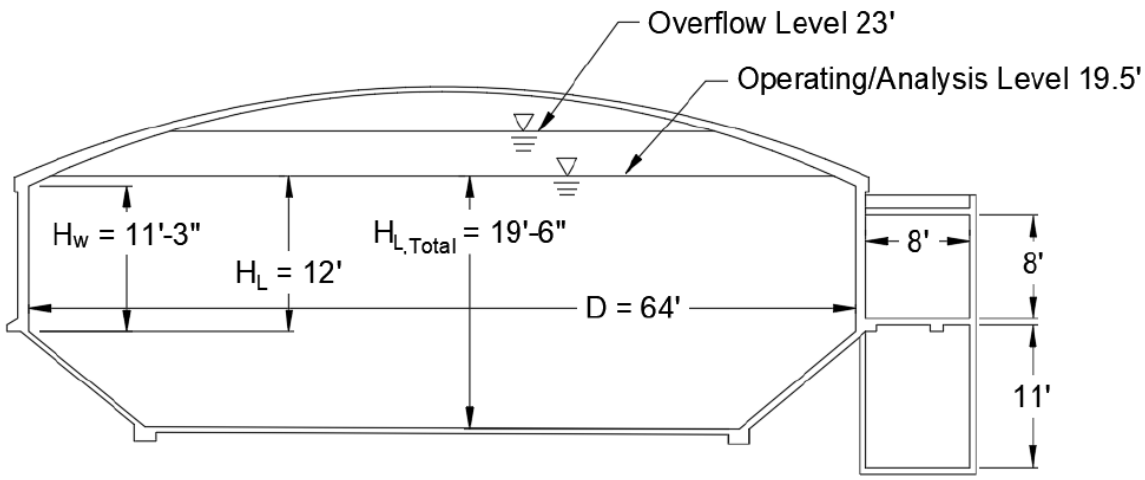


Figure 2-3: Consolidation Reservoir Elevation Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, $H_{L,Total}$ = Total Operating Water Height relative to Base)

2.7 Observations Pictures

Figure 2-4: Consolidation Reservoir –Elevation

Figure 2-5: Consolidation Reservoir – Entry to Valve Vault

Figure 2-6: Consolidation Reservoir – Circumferential Cracking around Reservoir below Roof Line

Figure 2-7: Consolidation Reservoir – Circumferential Cracking Close-up

Figure 2-8: Consolidation Reservoir – Dome Roof Vent

Figure 2-9: Consolidation Reservoir – Radial and Circumferential Cracking around Dome Roof Edge

Figure 2-10: Consolidation Reservoir – Steps Cast into Hopper Base and Outlet

Figure 2-11: Consolidation Reservoir – Dome Roof Interior Side of Vent

Figure 2-12: Consolidation Reservoir – Rear Wall (opposite entry hatch) with Cracking in Upper Wall Section

Figure 2-13: Consolidation Reservoir – Interior Roof-to-Wall Joint

Figure 2-14: Consolidation Reservoir – Interior Hopper-to-Floor Slab Joint

Figure 2-15: Consolidation Reservoir – Interior of Overflow Weir, Located in Access Hatch

2.8 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Eval

PROJECT NUMBER: A1502-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: Consolidation @ 2594 Yew St. 98229
48.7351, -122.4431

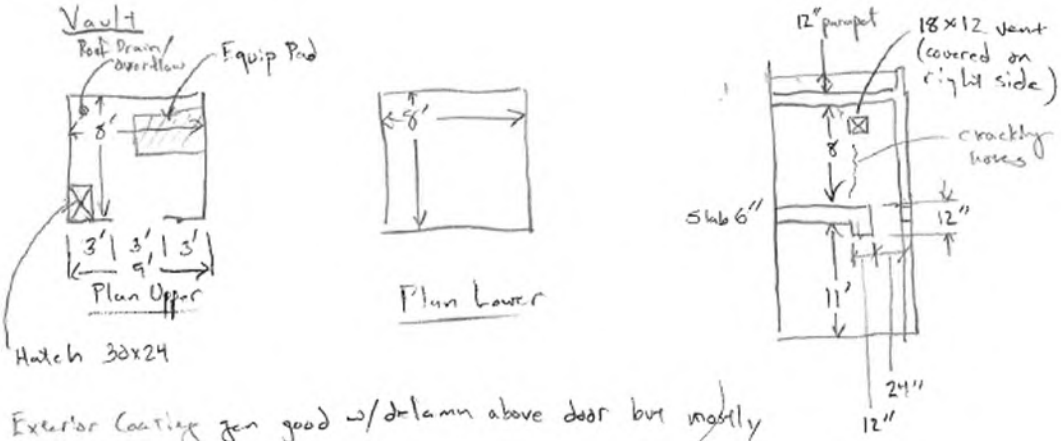
Site Visit Date: 11/7/19 Reservoir Type: Round RC

Temperature and weather: Clear, sunny 50°F

Site Conditions: Dry. Site is sloped up to base of roof, gradual slope w/ space for vault room

PSE Staff: Greg Lewis / Nick

Client/Other Staff: Corey / Chris - MSA, City staff



Exterior Coating gen good w/ delamn above door but mostly good around exterior.

Interior in good condition but efflorescence around roof drain & overflow pipes.

Paint adhesion worse in lower level which is wetter, with some bubbling in coating

Overall wall appears structurally sound & corrosion is minimal

some cracking around vents

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to Top of Roof Slab: (N) 42" (E) 6" (S) 5" (W) 7"

Roof Slab Thickness: / 16.5" edge (drawings/measured) Roof Overhang Dimension: / 3" (drawings/measured)

Drip Groove? (Y/N): / N (drawings/measured)

Top Surface Roof Slab Condition: Coating in poor condition w/ growth. Radial pattern cracking around roof exterior leading into circumferential crack

Ladder/Vents/Hatch/Joint Conditions: Vent - coating deteriorating, shield has some corrosion, Hatch - AL shoebox in gen. good condition w/ some corroded bolts, hatch concrete body in good shape, Ladder/landing - good cond.

Other Comments:

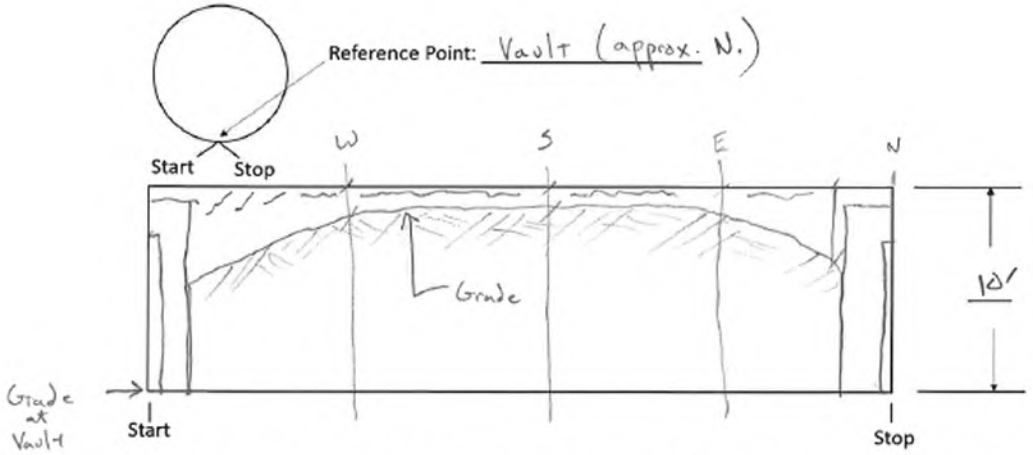


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

Cracking around exterior roof/wall interface w/ efflorescence & coating failure.

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

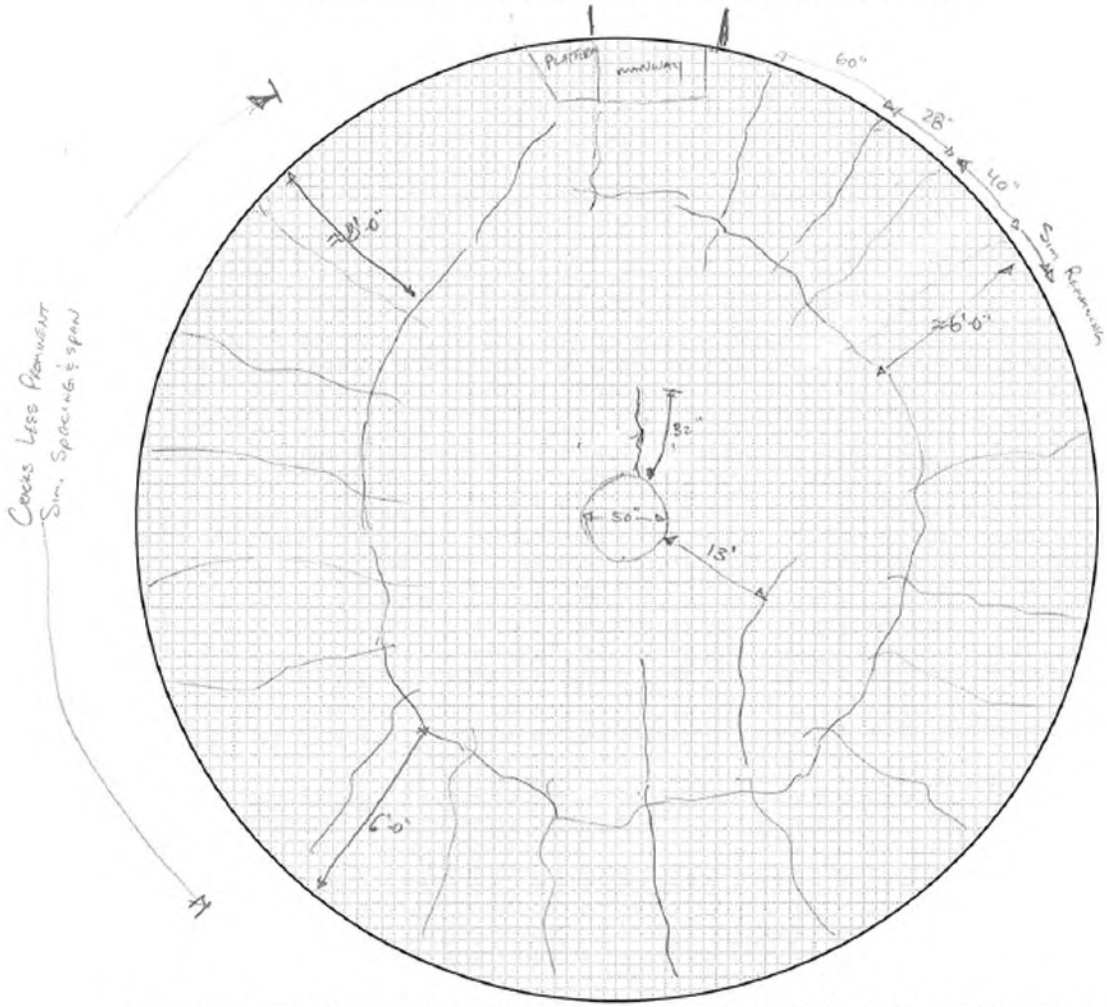


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: MANY CRACKS VISIBLE ON EXTERIOR REFLECTED ON INTERIOR

EFFLORESCENCE NOTED

Internal Diameter: _____

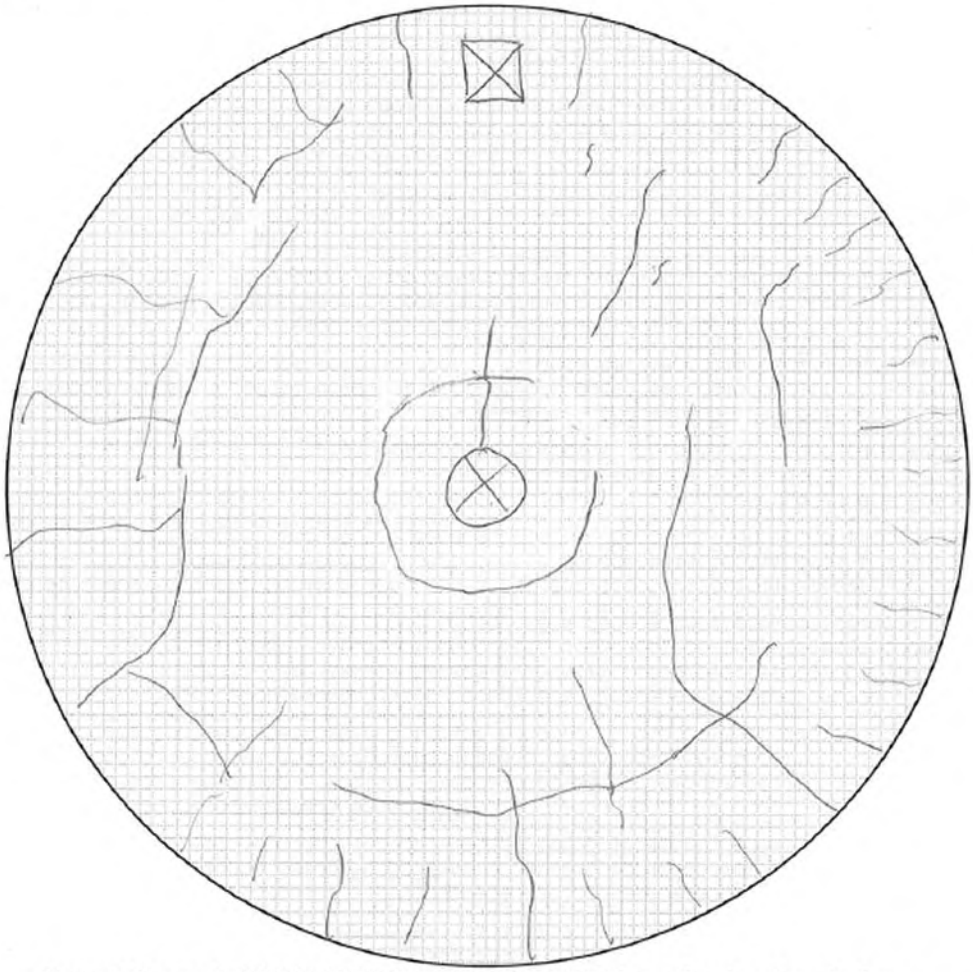


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

4

PSE
PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Column Diameter: N/A / N/A (drawings/measured) Footing Size/Thickness: / Not observed (drawings/measured)

Column Spacing: N/A (drawings/measured) Wall Curb Dimensions: / Not observed (drawings/measured)

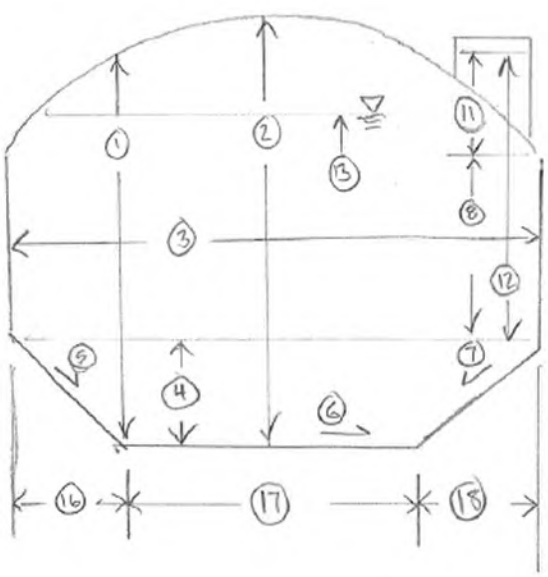
Floor Slab Condition: Good, uncoated, 4-quad pour. Cracking in field or edges not noted

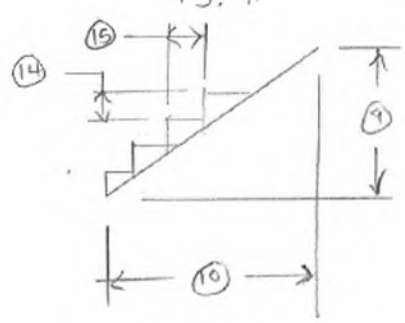
Floor Slab Joints Spacing/Condition: Slab 4-quad, joints cross at center. Hopper base has 8 sections w/ joint inline & between slabs. Some minor crack noted but otherwise appeared in good condition.

Column/Footing Conditions: N/A or not observed

Operates between 15'-19' (varity)

1. 22.1'
2. 26.5'
3. 64'
4. 7' approx.
5. 38.4°
6. 1.2°
7. 42.8°
8. 11.3'
9. 7.5'
10. 9.1
11.
12.
13. 19' per City
14. 9"
15. 11"
16. 9' approx
17. 46.5'
18. 9' approx





10 risers

5

PSE
PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

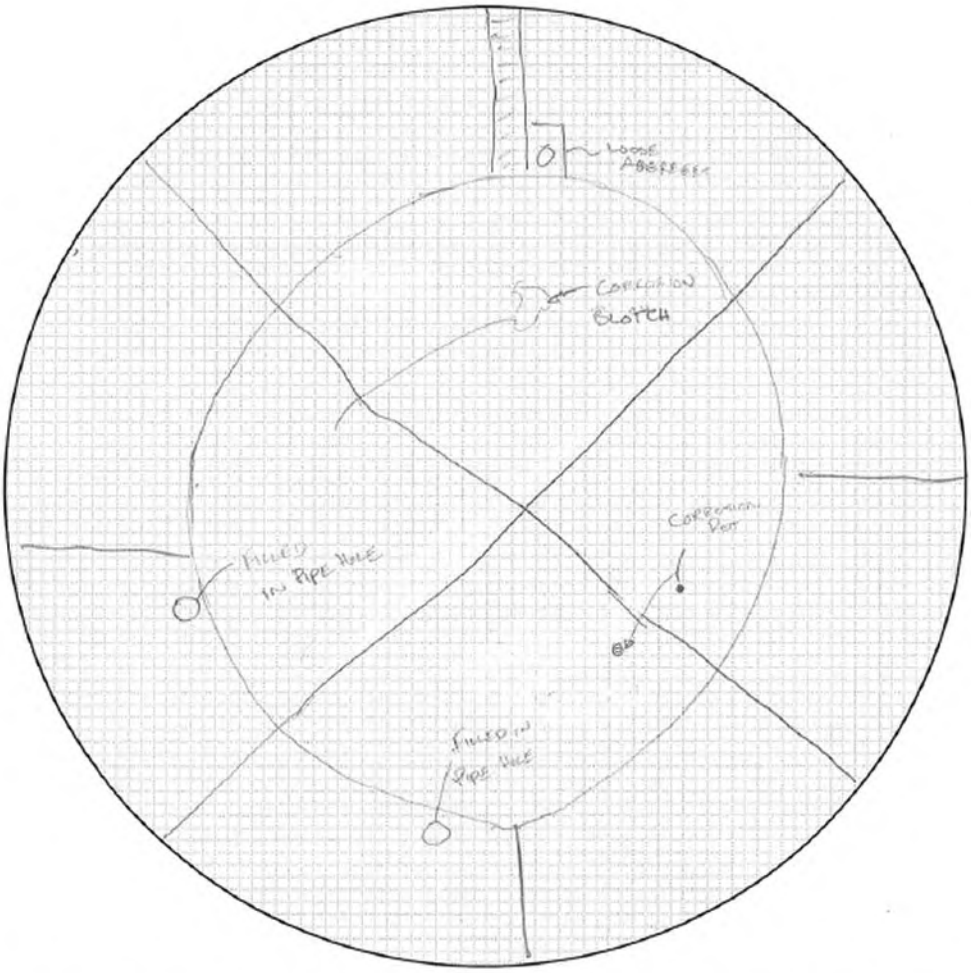


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

- MINOR CHIPS & LOOSE AGGREGATE IN SOME SPOTS
- LIGHT DISCOLORATION @ JOINT W/WALL
- NO MAJOR CRACKING OBSERVED IN SLAB
- LIGHT SCRATCHES/SCRAPING
- ≈ 64' DIAMETER

6

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Gen good, some cracking high on wall on S side, also corroded embeds around wall base coinciding w/ doorling

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): Surface rough but pitting is minor. No gross construction issues noted

# of wall sections: Likely 22 coinciding w/ "Box shape"

Ladder/Pipes/Overflow Conditions:

Overflow Height: / N/A (drawings/measured) Operating Height: 19' (per City/PUD/other)

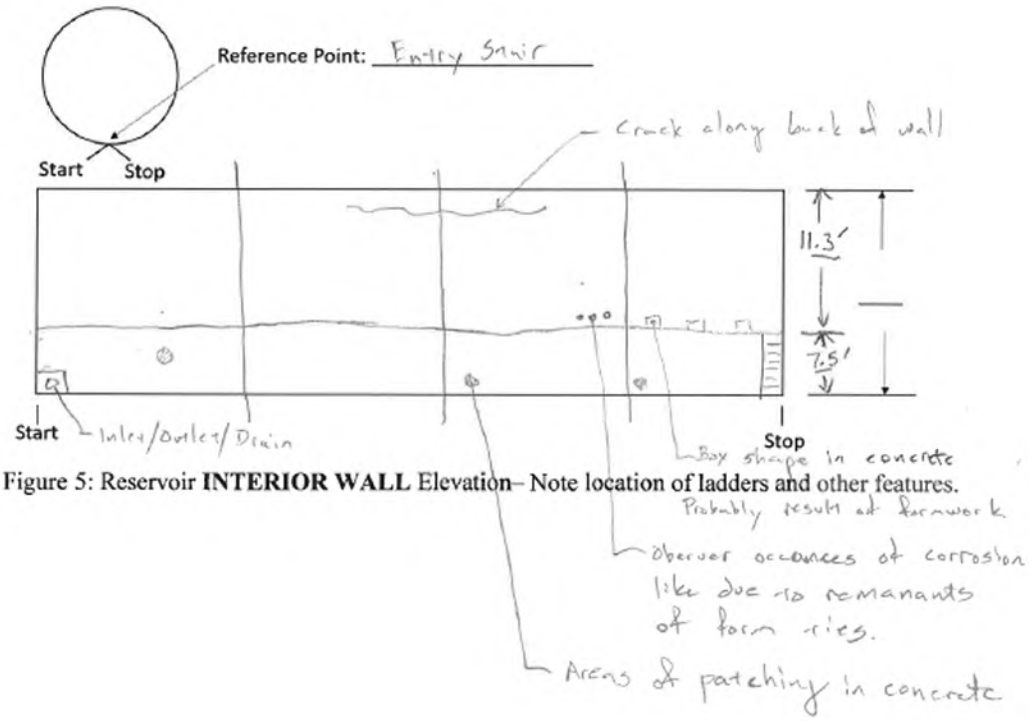

Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

7

END OF SECTION

Appendix I-3 Consolidation General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Consolidation Reservoir *General Info*

Field Visit Date: 11/7/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	11/7/2019
Reservoir Name and Location:	Consolidation - San Juan Blvd and Yew St; across from 881 San Juan Blvd
Inspected by:	Chris Hiatt, Corey Poland, Greg Lewis, Nick
Client Staff Present:	Jenny and Steve
Year Constructed:	1959
Overflow Destination:	North to woods
Discharge Destination/Zone:	519 Dakin and Yew Zone
Fill Location:	North
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	64	ft
Height	22	ft
Overflow Elevation:	519	ft AMSL
Bottom Elevation:	496	ft AMSL
Level of Overflow	23	ft
Minimum Normal Operating Level:	15.5	ft
Maximum Normal Operating Level:	19.5	ft
Notes:		

Consolidation Reservoir *Exterior Inspection*

Field Visit Date: 11/7/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	N/A	
Security Condition:	N/A	
Wall Attachment Type:	N/A	
Wall Attachment Condition:	N/A	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1.25	in
Ladder Width	20	in
Rung Spacing:	12	in
Side Clearance:	5.5	in
Front Clearance:	6.5	in
Back Clearance:	N/A	in
Notes: 88 in tall		

Exterior Fall Prevention System:	
Present at Site:	No

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof	
Material:	Aluminum	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	N/A	
Measurement Type	Measurement	Unit
Size:	36*36	in
Curb Height:	3	in
Notes: Gasket loose.		

Roof Vents and Screen:		
Material:	Concrete	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	1/4	in
Notes: Sheet metal covering screen.		

Roof:		
Condition:	Fair	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	Anti-slip	
Slope of roof	2-20 degrees	
Measurement Type	Measurement	Unit
Overhang Distance:	3	in
Thickness of roof slab	18	in
Notes: Radial Cracking and graffiti.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Very Good	
Corrosion present?	Yes	
Mid-rail:	Yes	
Attachment Condition:	Very Good	
Attachment Type:	anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	40	in
Top Height:	20	in
Notes: corrosion on nut.		

Grating:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Clips:	No	
Removable Panels:	No	
Measurement Type	Measurement	Unit
Approximate Panel Dimensions:	5x3	ft
Notes:		

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Very Poor
Notes: Mostly buried. cracks with efflorescence. wet as water leaks. Bugs crawling in concrete holes (!)	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	Unknown
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	poorly adhered
Notes:	

Consolidation Reservoir *Interior Inspection*

Field Visit Date: 11/7/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Cast Iron	
Condition:	Very Poor	
Corrosion:	Yes	
Cage:	No	
Security Type:	Locked access hatch	
Security Condition:	Good	
Wall Attachment Type:	Set into wall	
Wall Attachment Condition:	Very Poor	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	14	in
Rung Spacing:	16	in
Side Clearance:	N/A	in
Front Clearance:	5	in
Back Clearance:	N/A	in
Notes: Very corroded and missing rungs. Not safe to use. Removable ladder used for inspection.		

Interior Fall Prevention System:	
Present at Site:	No

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
		ft
Notes: Circumferential and radial cracks		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	
Severity:	
Notes:	

Walls:	
Condition:	Fair
Painters Rings Present:	
Notes: cracks including vertical cracks from roof-to-tall interface. uncoated	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	N/A
Interior Piping:	N/A
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Consolidation Reservoir *Miscellaneous*

Field Visit Date: 11/7/2019

Piping		
Inlet Piping:	Size (Inches OD):	10
	Condition:	Poor
	Material:	Cast Iron
	Notes: Combined inlet/outlet/drain. Standing water very turbid. Hear flowing water in pipe. Corrosion around pipe frame. Interior of pipe very corroded.	
Outlet Piping:	Size (inches OD):	10
	Condition:	Poor
	Material:	Cast Iron
	Lip (Inches)	0
	Notes:	
Overflow Piping:	Size (inches OD):	6
	Condition:	Fair
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	Pond
	Erosion Evident:	Yes
	Screen Condition:	Good
	Overflow to roof (feet)	N/A
	Notes: Air gap approx. 10 inches. Overflow above top of wall	
Drain Piping:	Size (inches OD):	10
	Condition:	Poor
	Outlet Location:	Pond
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	None
	Air Gap:	No
	Screen Condition:	N/A
	Notes: Dechlorination system (bag) not working. Outlets to culvert @ ground level. Water very turbid.	

Piping Facilities		
Exterior Valving:	Type:	GVs
	Condition:	Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	In valve vault
	Size (Inches OD):	Small copper
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	No
Notes: Valve vault roof drain is blocked.		

Electrical	
Cathodic Protection:	No
Impressed Current:	No
Anodes:	No
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	Yes
Common Inlet/Outlet:	Yes
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	Yes
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	0
Water Quality Sample Taps?	Yes
Notes: No fence	

Appendix I-4 Consolidation Condition Assessment Score Sheet

Consolidation Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	1	0	No fence or Camera
	Vegetation Separation	0	0	0	0	0	0	2	0	Under the dripline of trees. Organic debris on roof.
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	1	1	2	2	3	0	2	0	Roof not designed for thermal movement. Bugs in wall. cracks at base of roof and by vault
	Interior Walls	0	3	3	3	3	0	2	0	crack along back wall but no major issues
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	0	4	5	5	0	0	4	0	
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	2	2	5	3	4	0	4	0	Ext roof needs cleaning. Cracking significant. many radial crack along the edge and coat is thin or missing in many places
	Interior Roof and Supports	0	3	5	3	0	0	0	0	Int roof uncoated. Water does not appear to be infiltrating.
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	0	0	0	0	5	5	0	No fall protection required
	Interior Ladders/Fall Protection	1	1	0	0	0	3	1	0	Existing interior ladder not functional. Fixed ladder less than 24 feet.
	Access Hatches	4	3	0	0	4	0	3	0	Difficult to get into hatch. High maintenance design. Corroded hardware
	Railings and Roof Fall Protection	5	0	0	0	0	4	0	0	Could use railing around valve vault roof.
	Vents	0	0	0	0	4	0	5	0	Screen likely too coarse. Passed design checks.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	2	3	0	2	4	0	5	0	Comb in/out/drain. Corrosion on frame. does not appear to have flex joints
	Outlet Piping	0	0	0	2	0	0	5	0	Comb in/out/drain.
	Drain Piping	0	0	0	2	2	0	2	0	Comb in/out/drain. No Silt stop. No screen. Poor dechlorination. Reservoir cannot drain w/ water in pipe.
	Overflow Piping	3	3	0	0	5	0	5	0	
	Washdown Piping	0	0	0	0	0	0	5	0	washdown hose bib in valve vault.
	Attached Valve Vault Structure	0	3	4	3	0	0	3	0	Roof drainage needs imp. Drain very clogged.
	Control Valving	0	3	0	0	0	0	5	1	
	Isolation Valving	0	0	0	0	0	0	1	1	Water leaking in from pump station indicates isolation valve not functioning.
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	2.9	2.6	4.1	3.0	3.6	4.0	3.6	1.0	

Overall Score
3.3

Appendix J Dakin I

Appendix J-1 Dakin I Geotechnical Report

GEOENGINEERS

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Dakin I and Dakin II Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Dakin I and Dakin II reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the Dakin I and Dakin II site, located as shown in the Vicinity Map, Figure 1. The Dakin I reservoir is a round reinforced concrete structure with a hopper base and the Dakin II reservoir is a prestressed concrete reservoir.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone.

Surface Conditions

The project site is located approximately 550 feet to the east of Sylvan Street and 50 feet north of Balsam Lane. The reservoir is located on top of a small hill and the site drops off in all directions. The site is bounded by a wooded area, associated with Big Rock Park, in all directions. A small gravel roadway leads to the site from the southwest.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing two new geotechnical borings B-4 and B-5 (2019)—on March 25, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The borings were completed to a depth of 5½ to 17½ feet below the existing ground surface (bgs). The location of the explorations are shown in the Site Plan Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration logs are presented in Figures 4 and 5.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites. We also reviewed as-built drawings for this site.

- **Fill** – Fill was encountered at the surface of both explorations. Fill extended to 4 feet in B-4. The fill was significantly thicker in B-5 extending to 15 feet bgs. The fill generally consisted of medium stiff to stiff blue-brown silt with variable amounts of sand, gravel, and organic matter. Boring B-5 encountered a thin layer of wood at the interface between the fill and sandstone. As discussed subsequently, we expect that the fill is representative of backfill around the reservoir foundation and does not extend below the foundation.

- **Chuckanut Sandstone** – Chuckanut sandstone was encountered below the fill at 4 feet bgs in B-4 and 15 feet bgs in B-5. The fine to coarse grained sandstone was brown with weak cementation and occasional bedding planes.

Groundwater

Groundwater seepage was not observed at the final depth of borings. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Dakin I

Based on the conditions encountered in our boring, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. Available drawings and sketches do not provide any additional foundation details.

Dakin II

Based on review of drawings for the project by PEI Consulting Engineers & Surveyors dated 1991 and our boring, the existing Dakin II reservoir has a mixed bearing profile. A portion of the reservoir is supported on bedrock, and the southern portion is supported on lean concrete extending to bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a

Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest know faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structures bear on sandstone which is not at risk of liquefaction.

American Concrete Institute/ASCE 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI), publication D110-13 of the American Water Works Association (AWWA) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI, AWWA and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group (Dakin I)	II
AWWA Seismic Use Group (Dakin II)	III
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	94.4
1-Second Period Spectral Response Acceleration, S_1 (percent g)	36.9
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.43
MCE_G peak ground acceleration, PGA	0.390
Seismic design value, S_{DS}	0.643
Seismic design value, S_{D1}	0.352

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_u 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the

two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 6 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	12	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeters per second, g = gravity

M_w 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends more than 78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 7 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	14	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Dakin I

Based on the conditions encountered in our boring, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure

of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Dakin II

Based on review of project drawings referenced previously and our boring, the existing Dakin II reservoir has a mixed bearing profile. The reservoir is supported on bedrock, although the southern portion is supported on lean concrete extending to bedrock. According to the as-built drawing, a design allowable bearing pressure of 4,000 psf was used for design. This structure could be evaluated based on the design bearing pressure of 4,000 psf; in our opinion, an allowable bearing pressure of 6,000 psf would also be appropriate for this tank. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

We understand that the existing reservoirs include below grade walls. Our recommendations for evaluating below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section and backfilled with structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Dakin I

As previously mentioned, we anticipate that the existing Dakin I reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

Dakin II

Based on review of publicly available LiDAR for the site, there are slopes inclined at 30 to 40 percent to the southeast of Dakin II that is approximately 25 feet tall. The existing Dakin II reservoir has a mixed bearing profile between bedrock and lean concrete extending to bedrock. Based on the structure support extending to bedrock, it is our opinion that there is a low risk of slope instability that would impact the existing reservoir. Based on the soil conditions encountered in boring B-5 and site topography which includes slopes at 2H:1V or flatter, it is our opinion that there is a low risk of slope instability for the slope adjacent to the tank.

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AJH:HP:JRG:tln

Attachments-

Figure 1 – Vicinity Map

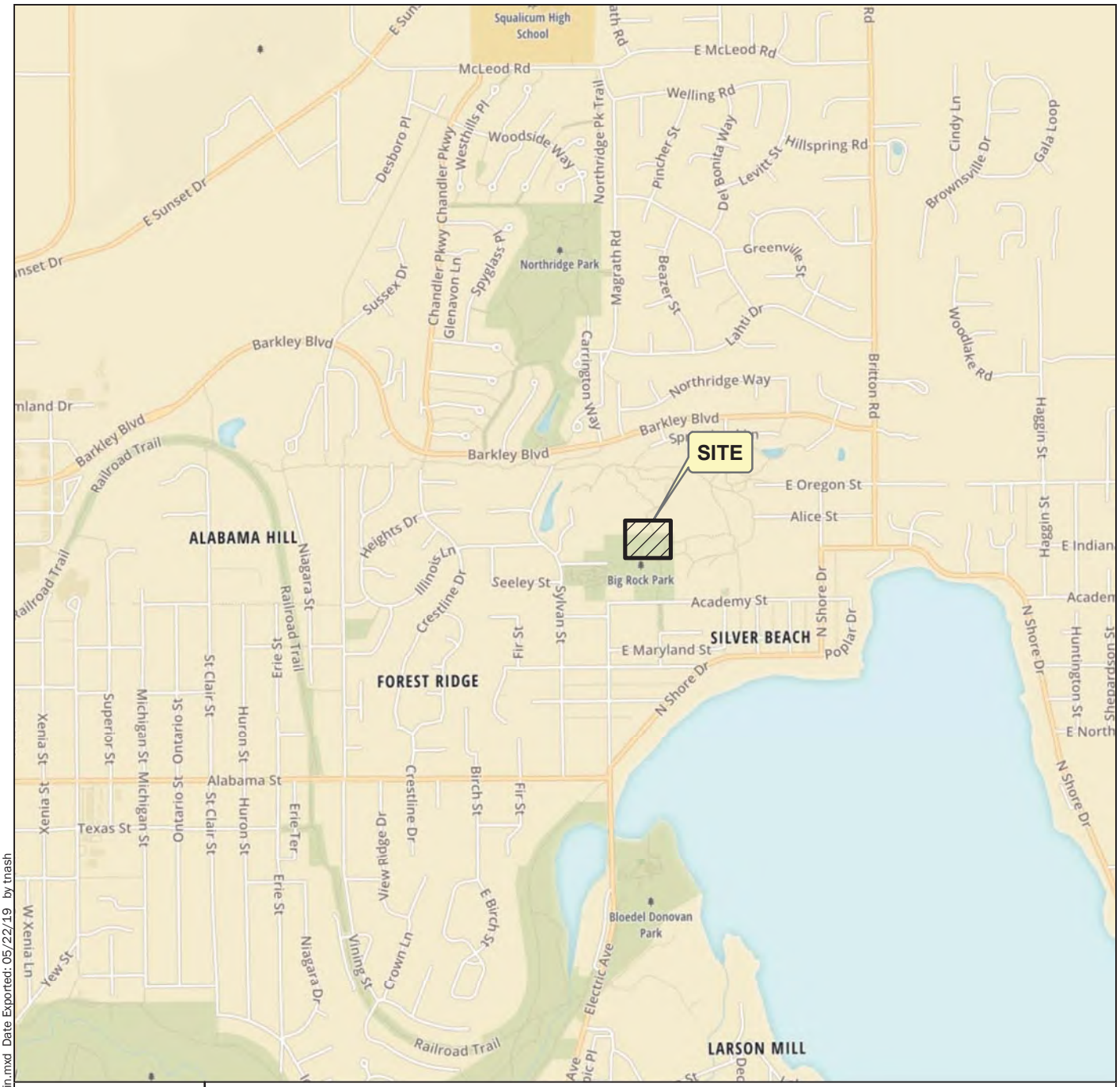
Figure 2 – Site Plan

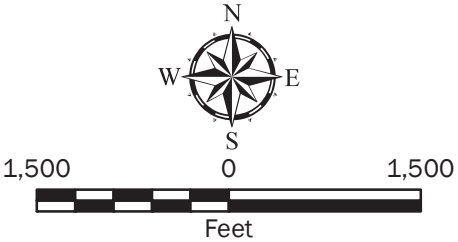
Figure 3 – Key to Exploration Logs

Figures 4 and 5 –Log of Borings B-4 and B-5

Figure 6 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 7 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault





Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Dakin I and II Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 1

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Legend

Boring by GeoEngineers (2019)

75 0 75

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Dakin I and II Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled Start 3/25/2019 End 3/25/2019	Total Depth (ft) 5.5	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) 510 Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment EC-95	
Easting (X) 1257680 Northing (Y) 649290		System Datum WA State Plane North NAD83 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown silt with sand, gravel and organic matter (soft to medium stiff, moist) (fill)			
		X	12	53/11"		1						
505								Sandstone	Brown sandstone (Chuckanut Formation)			
	5	X	4	50/4"		2						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-4

GeoEngineers

Project: COB Reservoir Inspection and Repair - Dakin I
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

Drilled	Start 3/25/2019	End 3/25/2019	Total Depth (ft) 17.75	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	500 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) Northing (Y)	1257840 649280		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown silt with sand, gravel and occasional organic matter (stiff, moist) (fill)			
		☒	5	50/5"		1 MC				13		
495	5	☒	18	23		2 MC		ML	Blue-brown silt with sand, gravel and wood fibers (stiff, moist)	13		
		☒	18	7		3 MC			Becomes medium stiff	18		
490	10	☒	6	4		4 MC				20		
485	15	☒	5	50/5"		5		Wood	Wood fibers			
								Sandstone	Brown sandstone (Chuckanut Formation)			
		⊠	2	50/2"		6						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-5

GeoEngineers

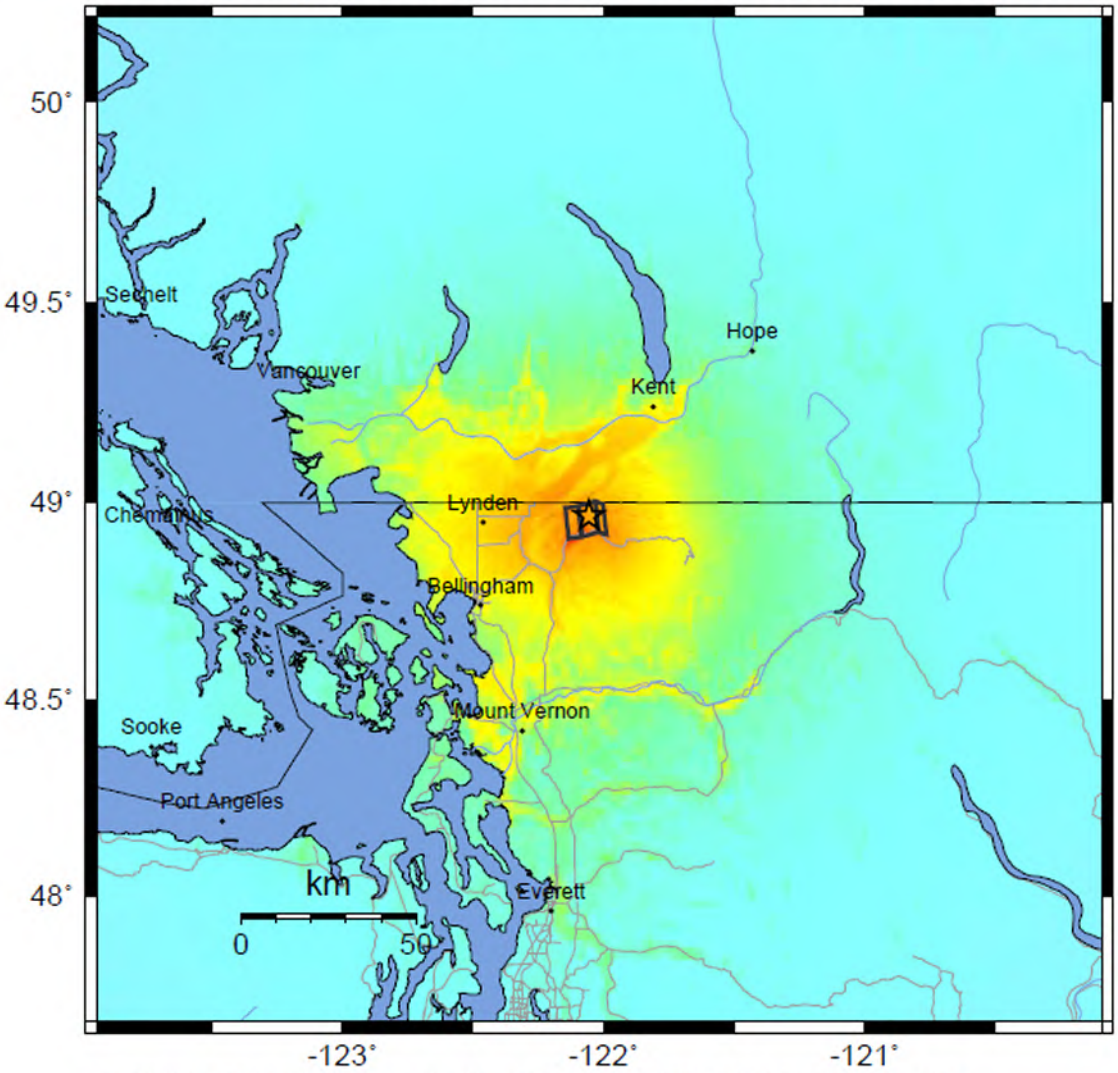
Project: COB Reservoir Inspection and Repair - Dakin II
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 5
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 6

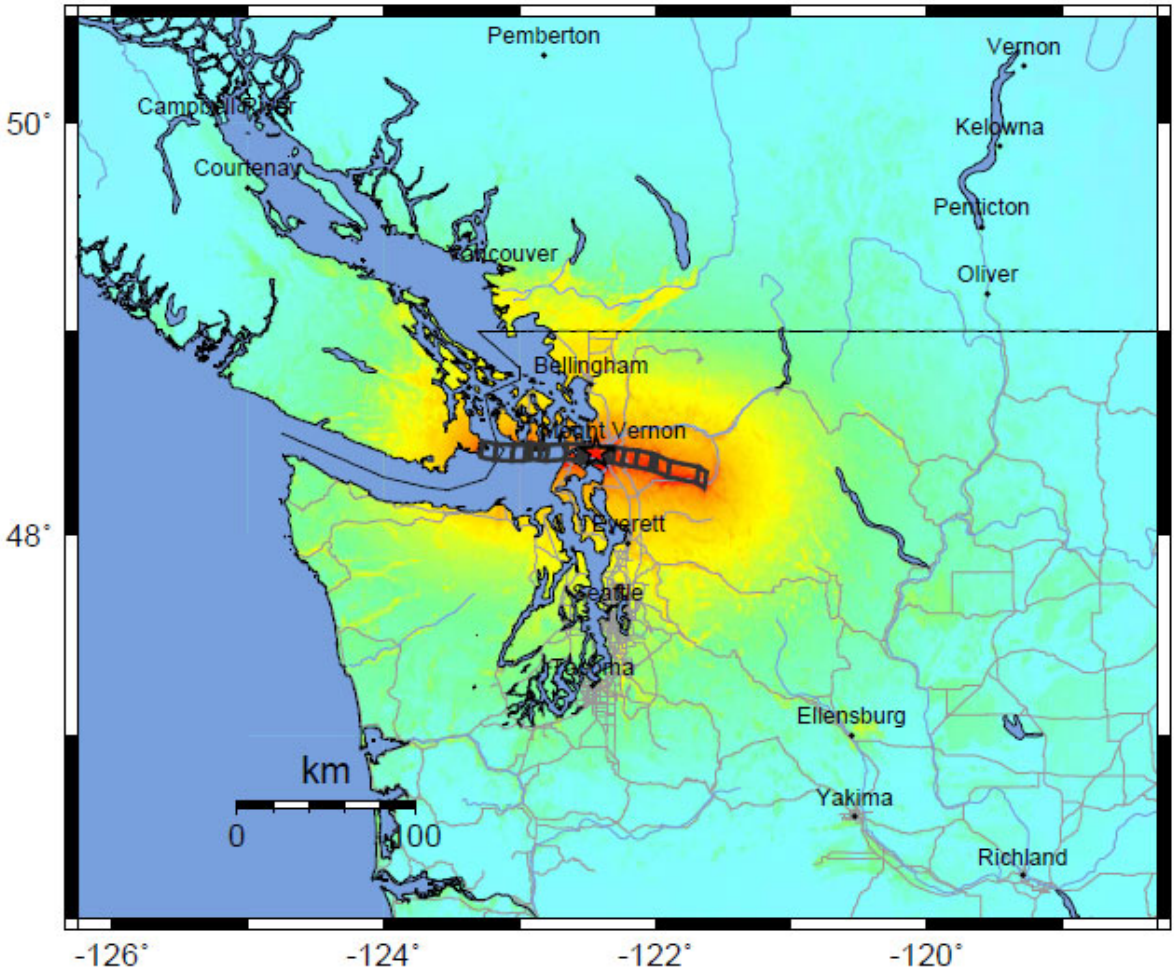
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 7

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix J-2 Dakin I Corrosion and Coatings Report

April 26, 2019

Northwest Corrosion Engineering

P.O. Box 905 Burlington, WA 98233
Phone: (360) 391-1041 Cell: (360) 391-0822

Mr. Nathan Hardy, P.E.
Murraysmith
2707 Colby Avenue, Suite 1110
Everett, WA 98201

SUBJECT: City of Bellingham – Dakin 1 Concrete Tank Coating Evaluation

Mr. Hardy,

Northwest Corrosion Engineering completed an internal and external coating evaluation for the City of Bellingham's Dakin 1 concrete water storage tank. Specific tasks completed during this inspection included:

1. Complete an assessment of both the interior and exterior coating.
2. Measure interior and exterior coating thicknesses at representative locations.
3. Note any corrosion related activity on exposed steel components of the concrete tank.
4. Evaluate coating losses and corrosion on visible surfaces.

BACKGROUND INFORMATION

The Dakin 1 tank is approximately 64-ft in diameter and 22.17-ft tall (floor to bottom of roof dome) with a majority of its volume buried belowground. Based upon previous inspection reports, the tank interior was coated in 2009 using a VersaFlex primer and AquaBurst polyurea and was to have been coated to a minimum dry film thickness of 80 mils. Coated was applied to the entire floor, floor/wall transition and on the all vertical surfaces. The interior tank roof was not coated. The type of exterior coating is not known.

COATING EVALUATION METHODS

The coating evaluation consisted of a visual inspection of the available interior and exterior surfaces and measuring the thickness of the internal lining and exterior coating at multiple locations. Destructive testing of the coating materials was not performed.

Dry Film Thickness

The thickness of the interior coating system was measured using a DeFelsko PosiTector Model 6000 electromagnetic dry film thickness gauge (Type 2 gauge) with a model 200 transducer calibrated for polyurea. A dial micrometer was also used to measure the thickness of paint chips removed from the exterior tank surface.

INSPECTION RESULTS AND ANALYSIS

Exterior Coating Thickness

The exterior coating of the tank appears to be a three coat system with a mint green primer, white intermediate coating, and dark green topcoat. The measured total thickness of the external coating on the sidewalls ranged from 24 – 33 mils. We were unable to obtain dry film thickness measurements on the external roof surface, due to the uneven surface texture, as well as debris and organic growth accumulation. However, it is reasonable to expect the same level of coating effort on the roof as was noted on the sidewalls.

Exterior Coating Assessment

There is a significant amount of calcium carbonate (lime) buildup on the sidewalls. Approximately 5-10% of the top coat is missing on the sidewall surfaces, mostly at the ground interface. From grade to 6-inches above grade, top coat loss is approximately 40%, with 10-15% of the concrete exposed. There are multiple locations (approximately 1-3%) of blistering in the coating, however the prime coat appears to be in good condition underneath the blisters. On the attached shed and roof hatch column, about 40% of the top coat is missing. A razor knife test showed the coating to be tightly adhered at sound locations on the tank sidewalls.

The exterior roof surfaces are covered with organic matter and debris. A razor knife test showed the coating on the roof to be well adhered. There are four locations of exposed concrete around the vent most likely due to mechanical damage from the vent cover legs. There are a few axial and circumferential cracks, but the coating is intact around the cracks.

Dakin 1 Tank – Roof and sidewall

Heavy debris accumulation and calcium carbonate leaching

Exposed concrete at grade to 6-inches above grade

Heavy debris accumulation on roof

Blister in coating on sidewall
(filled with water)

Missing coating on attached shed
and roof hatch column

Roof vent

Area of missing coating at base of roof vent

In order to help preserve the life of the concrete, the tank will need to be cleaned, and have all cracks properly sealed. This will extend the life of the existing coating an additional (+/-) 10 years. Otherwise, removal of all existing coatings, repair of concrete and sealing of cracks, plus the application of a new exterior protective coating can be applied that will provide a protective service life of more than 20 years with appropriate minor maintenance.

Interior Coating Thickness

Approximately 80% of the floor and sloped wall transition were covered with a thin layer of debris making it difficult to assess these locations. All sidewall surfaces were visible. The interior roof is not coated.

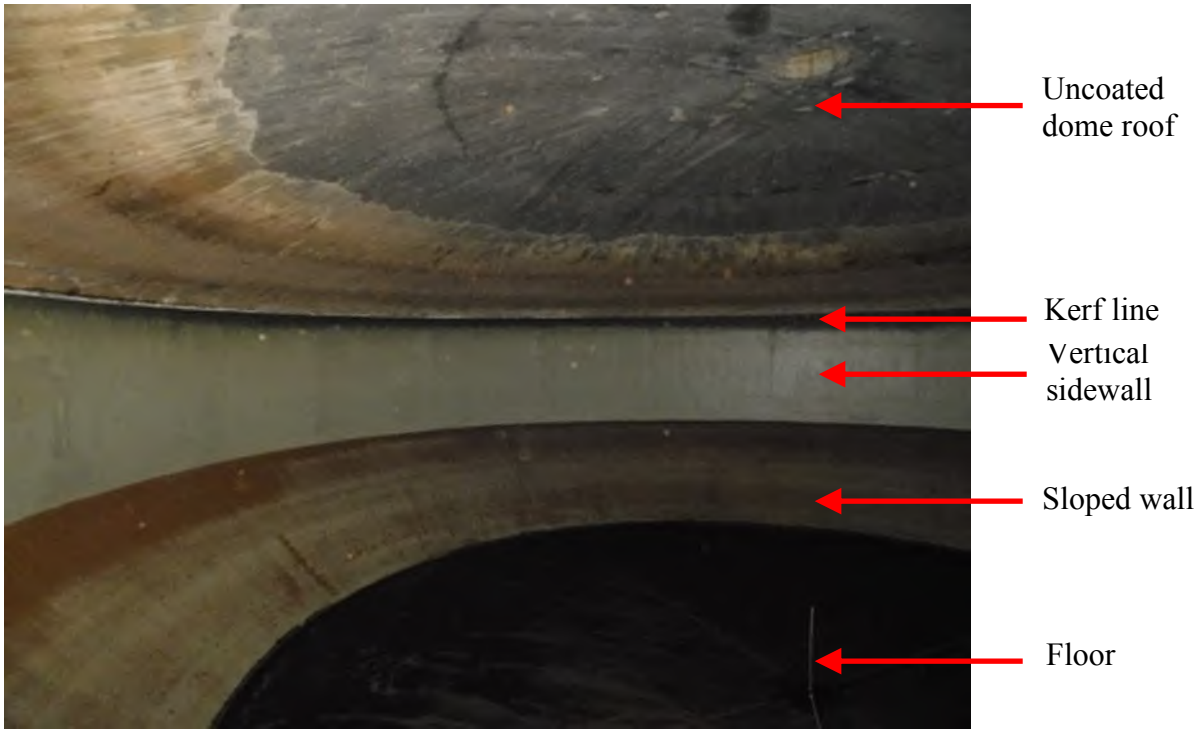
The interior coating on the floors and sidewalls of the tank is a two coat polyurea system with a blue primer and gray topcoat. The measured total thickness of the sidewall coating generally ranged from 80 – 100 mils. The specified minimum dry film thickness was to be 80 mils.

Interior Coating Assessment

The kerf line (1/4" cut in concrete made between coated and non-coated transition used as an anchor point for the coating) did not show any instances of coating delamination. The vertical sidewalls were 100% visible and there are multiple areas of coating runs and sags, but no failures are evident on these surfaces.

At the 60° transition on the sidewalls, 40% of the coating is blistering. The top coat is easily peeled off, exposing the blue prime coat underneath. The top coat is very elastic and water is present under the blisters. The blue prime undercoating is well adhered.

On the visible areas of the floor, no blistering was noted. The water basin at the foot of the stairs is well coated with no defects. These areas tend to be more difficult to coat given the amount of sharp edges and transitions.



Interior of Dakin 1 tank

Interior floor covered with dirt

Blistering on 60° wall transition, noted on over 40% of the area

Intact blue prime coat underneath coating blister

Runs and sags in coating, not affecting coating integrity

Piping transition is well coated with water staining on the surface

Well coated stair steps

Uncoated metallic piping with surface corrosion and extensive tuberculation

CONCLUSIONS

The following conclusions are based upon results of the field testing and visual inspection of the tank.

1. The interior of tank was lined in 2009. It is unknown when the external surfaces were coated.
2. The exterior sidewall and roof surfaces of the tank are dirty.
3. Approximately 5-10% of the exterior sidewall top coat is gone. The undercoating appears to be fairly tightly adhered to the concrete surface.
4. Between grade and 6-inches above grade on the exterior sidewalls there is approximately 40% top coat loss and blistering, however the prime coat underneath appears intact.
5. 40% of the interior coating at the 60° wall transition is blistering. The prime coat underneath the blisters in this area is well adhered.
6. There are sags and runs in the interior wall coatings. These conditions are not affecting the performance of the coating material.
7. The interior floor was dirty but the visible surfaces showed only a few instances of blistering.

RECOMMENDATIONS

1. The exterior tank surfaces need to be pressure washed to remove dirt and other debris.
2. Spot repair areas of exterior coating loss. This will require the development of appropriate surface preparation and coating application procedures. This should extend the useful life of the coating an additional 10 years.
3. The interior blistering is not a concern at this time. However, increased inspection intervals to every 3 – 5 years should be performed in order to photograph and evaluate if any additional blistering is occurring or if the top coat is separating and exposing the underlying coating material. This work can be completed by City personnel.
4. The metallic piping extending along the access hatch should be internally and externally coated to prevent additional corrosion loss.

We appreciate the opportunity to work with you on this project. If you have any questions or would like any additional information, please feel free to contact our office.

Sincerely,
Northwest Corrosion Engineering

Jeremy A. Hailey, P.E.

Appendix J-3 Dakin I Structural Report

CITY OF BELLINGHAM

CH 12: DAKIN 1 RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

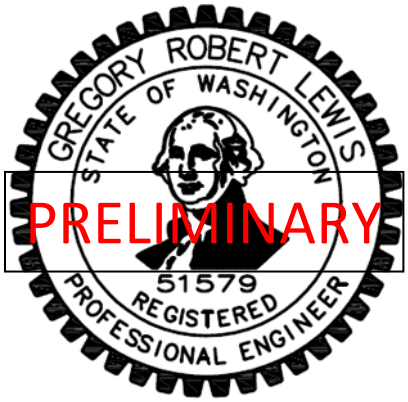
Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Dakin 1, 0.5 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located at 2918 Sylvan St, Bellingham, WA (Lat. 48.769, Long. -122.420), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on April 9th, 2019 by Peterson Structural Engineers (PSE), Northwest Corrosion, and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.



2 Dakin 1 Round Reinforce Concrete (RC) Reservoir – 0.5 MG

2.1 Description & Background

Per information provided by the City, the Dakin 1 reservoir was likely designed by John W. Cunningham & Associates Consulting Engineers and built in 1987. Drawings were not available for the reservoir, but it is similar in design to other reservoirs in the City's inventory with a storage volume of around 0.5MG, a dome roof, and a hopper base. For similar reservoirs types within the City's inventory for which drawings are available, Cunningham & Associates is the designer of record. The reservoir itself is a round reinforced concrete reservoir with a measured interior diameter of 66.8-feet. The interior wall was measured to be 10.3-feet high while the hopper base was approximately 7.3-feet deep. The overflow weir is located approximately 21.9-feet above the bottom of the reservoir and is located above the top of the wall within the access hatch box. The reservoir uses a portion of the dome roof for its storage volume at full capacity.

Where details or sections could not be directly observed or measured, they have been assumed to be comparable to other similarly sized reservoirs designed by Cunningham & Associates built in the same era for the City. Per these drawings it is assumed that the wall is 10-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. Similarly, it is assumed that the roof is likely a reinforced 4.5-inch thick dome with an edge that thickens to 1'-2". The floor is assumed to be a reinforced 5-inch thick slab. The footing is assumed to be a reinforced 12-inch thick by 10-inch wide footing that transitions into the hopper base. Finally, where piping is run under the footing, it is assumed to be encased in an unreinforced concrete block for protection. Drawing of a similar reservoir configuration designed by Cunningham & Associates and built in the same era have been included in Figure 2-1 along with the in-situ measured size Figure 2-2 for reference.

2.1.1 Description of Additional Site Structures and Features

The site includes an additional valve vault related to the reservoir's operation. This valve vault was constructed as part of the reservoir and the rear of the vault shares a wall and footing with the reservoir. This vault is located on the southwest side of the reservoir and is two levels, with one level located below grade. The vault is 8-feet deep by 9-feet wide and the main level has an internal height of 8-feet while the lower level is 11-feet high. The lower level is accessed via a 26 by 30-inch opening in the main level's 6-inch thick slab floor. The reservoir's drain, outlet, and overflow are all run through the valve vault. The roof of the vault has a parapet and the air gap for the reservoir overflow is located on the roof. If something were to clog the overflow, the layout of the top of the valve vault is such that it would fill with water up to the height of the parapet on the roof. While the roof had a secondary drain, it is insufficiently sized to handle any overflow volume. Additionally, as the overflow and drain pipes are joined below the roof line, any material or issue that blocks the overflow would also be likely to clog the drain as well.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit to observe the as-built current condition of the reservoir's interior and exterior as well as the site conditions. The reservoir was drained for our inspection. The site visit was performed April 9th, 2019.

Dome Roof: The reservoir has a self-supporting concrete dome roof with a thickened edge. The surface of the roof is coated, and the coating appears to be in good visual condition only having incidental issues localized around the hatch and roof vent. Structurally the roof appears to be in generally good visual condition. The only structural item visually noted in the roof was a circumferential crack that runs around the entire circumference of the roof approximately half-way up the dome. Additional cracking may be present but is not visible due to the applied coating. Per the reference drawings, it is assumed that the radial roof dome reinforcing is fanned out along the circumferential edge reinforcing with additional reinforcing added as the width between the fanned area increases. This cracking looks to be occurring roughly along the line where the additional reinforcing at the fanning begins and could be a result of a stiffness differential in the roof.

The roof has one 3 by 3-foot access hatch that is part of a larger access box. This box is formed to include the reservoir's overflow weir. The location of the overflow is above the top of the wall and per the reference drawings it is assumed that this necessitates a waterstop in both the roof-to-wall joint and access hatch-to-roof joint. Coating loss was noted around the base of the hatch, but no major structural issues were noted beyond some minor cracking and chipping. Overall the hatch appears to be in good condition and working order.

At the center of the roof the reservoir has a 36-inch diameter vent. Per the available reference drawings and pictures, the vent would typically include a series of (12) 4-inch by 11-inch openings around its exterior. However, these openings and the structure itself were obscured by a sheet-metal cover. Observable components and the surrounding roof do not appear to have any visual structural issues associated with the vent beyond some coating loss. Please note, the tightness of the metal cover could pose some issues with the venting requirements and Murraysmith should be consulted with to determine if a problem exists due to the potential restricted airflow and how best to mitigate any associated issues. Inadequate venting can create significant structural loads when the reservoir is filled or drained if the vent cannot keep up with the change in storage volume.

Reservoir Walls and Interior: Per the referenced drawings, the walls are likely 10-inches thick with vertical and horizontal (hoop) reinforcing. At the top of the wall, there is likely a keyed joint and two (2) bars of reinforcing at 12-inches on center are assumed to connect the roof to the wall. This connection is required as the design of the reservoir intends for water to be stored above the top of the wall and within the dome. This results in a restraint that that does not accommodate any thermal expansion for the roof. When temperature changes occur, the roof will expand or contract radially while the wall expands or contracts primarily in the vertical direction. As the reservoir is partially buried, the soil restricts the wall from moving outward. The effect of this differential movement appears to have resulted in a crack which runs around the exterior of the reservoir's wall, just below the base of the dome roof edge. This crack is likely a result of the thermal expansion pushing on and failing the top of the wall at a thinner section of the keyed joint. Unlike other similar reservoirs within the City's system, this reservoir's valve vault is located just below the reservoir's roof. As a result, the location of the crack does not vary in location around the top of the wall, staying at a relatively consistent height.

As the reservoir is partially buried, a limited view of the exterior surface was obtained. Further, the exterior of the reservoir is coated with a three-coat system. It is possible that this coating might be obscuring additional conditions. However, additional problems such as those resulting from the aforementioned thermal cracking would likely be visible through the coating as noted above. Reinforced non-prestressed concrete reservoirs can develop issues with creep which can result in gradual failure of the reservoir walls over time. However, as this reservoir is partially buried, the confining pressure of the backfill soil helps to counter these tensile forces. Looking for these types of issues, no major crack issues or failures, outside of the thermal cracking, were noted in our visual inspection.

The interior of the reservoir was visually found to be in good condition. The floor slab, hopper sides of the floor, and walls were not observed to have any major visual defects or issues that might compromise the coating. A coating had been applied to protect the concrete and this coating appeared to be intact. The coating was applied along the floor and up to the base of the dome. Above the coating, in the dome, no visible issues or visually significant cracks were noted.

Appurtenances: The inlet/outlet pipe visually appeared to be in generally good condition, although some small corrosion carbuncles were noted to be developing. The overflow weir is concrete and observed to be in good condition although the associated steel parts were noted to be heavily corroded with section loss beginning to occur in the shear gate handle and guide. Corrosion is such that these members will like need to be replaced if they are intended to still be used.

2.2.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appears to be in generally good visual condition, although all sides were observed to be experiencing issues with coating loss. No visual signs of major cracking or settlement issues were noted. However, as it shares a wall-line with the reservoir, some issues with water infiltration were noted along edges. Based upon the reference drawings it is assumed that the vault was poured using a keyed joint and has reinforcing to connect it to the reservoir wall. However, there is no indication of the presence of a waterstop or a watertight joint type in the drawings. As a result, the minor leakage noted along the main level roof joint is likely a result of this detailing omission.

2.3 Structural Analysis

The following design analysis is based on the reservoir drawings associated with the 40th St reservoir which is a similar type of reservoir with a dome roof and hopper base. The City has a few of these types of reservoirs in its inventory and other similar reservoirs include College Way (built 1968), Reveille (built 1958), Consolidation (built 1959), and 40th St (built 1959). Only the Reveille and 40th St reservoirs have drawings available and both sets of drawings were prepared by John W. Cunningham & Associates, Consulting Engineers. As the 40th St reservoir is of a similar size to Dakin 1 (0.5MG versus Reveille's 0.3MG size), the 40th St drawings have been used to provide a baseline for PSE's analysis.

Where field measurements could be made, the actual dimensions were used in PSE's analysis. In this case the size and diameter of the reference reservoir varies somewhat with 40th St having a 60-foot diameter and a 15.5-foot wall height while Dakin 1 was found to have a 66.8-foot diameter and a 10.3-foot wall height. Where dimensions or information could not be obtained during the site visit, such as the

reinforcing size, layout, and spacing, those values were based on the 40th St reservoir drawings. Based on the similarity of multiple versions of this reservoir, this assumption is seen as a reasonable and cost-effective method to determine a presumed level of the basic adequacy of the reservoir. Based on the results of PSE's analysis, potential issues and retrofit options are discussed. However, as these calculations are an approximation, any assumptions made should be verified though additional destructive and non-destructing testing prior to undertaking any retrofit or repair work.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Circular Concrete Tanks without Prestressing", published 1993 were also utilized.

2.3.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof was evaluated per concrete design criteria as covered in AWWA D110-13 and found to meet minimum requirements for rise-to-span, thickness versus buckling, and edge reinforcing assuming the reinforcing is consistent with the reference drawings. The dome roof was evaluated at a 17-foot operating level. At this level the water is stored below the top of the wall and there are no additional hydrostatic loads on the roof.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will simultaneously allow for differential thermal movement between the reservoir roof and the wall as both components deform differently as a result of temperature variations due to temperature or solar gain. At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the components of the structure able to resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement and has resulted in cracking along the top of the wall as noted above. This is a result of its original design that allows water to be stored above the wall line. To adequately account for thermal movement the existing roof attachment would need to be modified or replaced with a roof system able to accommodate thermal movement. This may not be practical or economically feasible.

Wall Reinforcement: Per the reference drawings the wall is likely reinforced with #4 vertical bars at 12-inches on center on the exterior face and #5 vertical bars at 12-inches on center near the base on the interior face. It is assumed that the horizontal (hoop) reinforcing starts out with #5 bars at 3.5-inches on center at the base and the spacing increases to #5 bars at 6.5-inches on center towards the top of the wall. At the very top of the wall there are assumed to be two (2) #6 circumferential hoops. This variation in the hoop reinforcing is based on the variable pressure distribution resulting from the storage of liquids.

Per PSE's analysis, it was determined that the vertical wall reinforcing appears to be acceptable for the design loads under current code requirements. Backchecking at the overflow level, which the original

reservoir should have been designed for, the reservoir appears to have the reinforcing necessary for the ACI factored hydrostatic load. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3 sanitary factor. This sanitary factor helps to minimize the potential for cracking and leaks. As this reservoir is operated at 17-feet, which is 4.9-feet below overflow, load demands are further reduced, and the existing reinforcing is determined to be adequate per current code requirements for static loads.

Additional checks for wall shear and cracking at the 17-foot operating level found the wall system to be adequate assuming it was built similarly to the reference drawings. The interior face vertical reinforcing was checked for flexure and determined to be adequate when using the required sanitary load factors. The interior face's vertical rebar consists of #4 bars in the main wall and #5 bars near the base, with a maximum spacing of 12-inches on center. This meets ACI 350.3 spacing requirements. For the hoop tension forces, the required reinforcing was found to be acceptable for ranges of 17-feet up to overflow for current code. This check includes the larger 1.65 sanitary factor used when checking reinforcing in tension as required by current code.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon information obtained from the comparable reservoir drawings. Per the geotechnical evaluation the site's bearing capacity was determined to be 6,000-psf. PSE evaluated the reservoir using this bearing capacity and for the 17-foot operating level up to overflow, the bearing pressure determined was within acceptable ranges.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: Per PSE's analysis, the addition of seismic loads results in additional forces on the wall of the structure. This is due to the water slosh wave as well as from forces resulting from movement of the structure itself. PSE found that the wall flexure stresses were increased by about 30% when operating at a 17-foot water level. As the reservoir was designed for a higher static operating level, there is reserve capacity and the reservoir can resist the increased seismic loads when evaluation is done based on current design codes and the assumed construction values for the current 17-foot operating level. Should the reservoir be operated at overflow, the seismic load would exceed the wall's flexural capacity and the reservoir would need to be retrofitted.

The wall's hoop tensile stresses were found to be within acceptable limits when the reservoir was evaluated at its overflow design level for hydrostatic loads. When the reservoir is evaluated at the current maximum operating level of 17-feet (5-feet below the original design level), sufficient capacity is available to meet design requirements from current seismic load and using modern loads factors. For lateral loads, the horizontal hoop reinforcing was determined to be adequate.

In addition to the wall flexure and tensile checks, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing to resist seismic lateral loads at the overflow level. No additional reinforcing or connectivity is needed between the walls and the foundation based upon the assumed construction.

Freeboard/Slosh: At overflow, the water is stored within the roof-line of the reservoir. In this case, during a seismic event, the slosh wave would be constrained by the roof. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. The force of this wave would be sufficient to damage and potentially cause failure of the roof at the roof-to-wall interface, the dome itself, as well has hatches and appurtenances. The current reduced operating level results in a freeboard of 0.6-feet. While this is not an adequate amount of freeboard to handle the projected 3.1-foot projected slosh wave, it is sufficient to lower the slosh wave's impact force on the roof. At an operating level of 17-feet the assumed available roof reinforcing, along with its thickness and weight of the roof appears to be adequate to resist the code required maximum slosh load. However, incidental damage to the hatches and appurtenances may still occur during a seismic event from a slosh wave impact.

Valve Vault: Per the reference drawings it is assumed that the valve vault is connected to the reservoir with #5 rebar dowels at 12-inches on center. This attachment should limit differential movement or "pounding" that occurs in a seismic event. Additionally, where the hopper base and the lower level of the vault are adjacent, this zone is assumed to be backfilled with plain concrete or "trench backfill". In the event of an earthquake, this will provide support to the hopper base so as to limit its potential to fail the lower level wall and collapse onto the piping. Of primary concern is the vault's wall which is cast as part of the reservoir footing. It is recommended that the pipes be retrofitted to have flexible coupling to address potential differential movement between the two structures occur during a seismic event.

2.4 Summary

As noted at the start of the analysis section, PSE has used a comparable reservoir as a basis for the design checks of this reservoir. Assuming the reservoirs were built similarly, it appears that this reservoir's flexural, shear, and hoop tensile capacity are all sufficient to meet current code requirements.

When considering lateral and hydrodynamic loads, it appears that the reservoir is under-designed for current seismic codes when operated at its overflow capacity. When operated at a 17-foot operating level, all reinforcing was found to be acceptable. The dome-to-roof reinforcing is sufficient to handle lateral loads from an analysis standpoint; however, thermal effects have caused damage to this interface and the actual capacity is likely much lower and could be a potential concern in a seismic event.

Remaining observable components of the reservoir appear to be in good condition with few interior defects noted. The area of primary concern was cracking and wall failure due to thermal expansion and contraction of the dome roof; it appears that this connection was not designed to accommodate thermal effects. The reservoir is also attached to a valve vault which appears to be structurally sound based upon the assumed construction. Piping in the vault may also be a candidate to be upgraded to accommodate potential vertical and horizontal differential movement between the vault and the reservoir, as might occur during a seismic event.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 17-feet, which is 0.6-feet below the top of the wall. Due to the types of issues noted, these retrofits might not be cost effective or easy to implement.

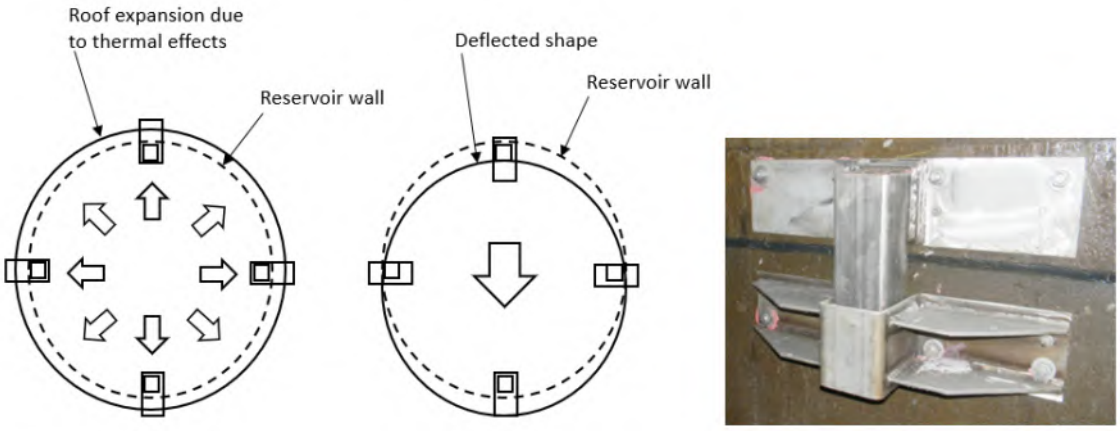
Wall Flexural Capacity

Based on a maximum operating level of 17-feet, PSE determined the wall flexure was adequate. However, these design checks are based on an assumed reservoir reinforcing layout. Should the reservoir be operated at a higher level or additional confirmation be required, PSE would recommend a testing firm be employed to map the existing reinforcing. Additionally, the testing firm could perform testing to verify the concrete compressive strength and reinforcing's yield strength. Based on the gathered information, the reservoir can be re-evaluated to confirm the actual as-built design capacity.

Roof-to-Wall Retrofit

The effects of thermal movement appear to have damaged the roof-to-wall interface and have likely affected its structural shear capacity and water-tightness. To alleviate this issue, PSE would recommend either the retrofit or replacement of the roof. Note that this may not be economically feasible or practical.

The first option would be to remove the existing roof and replace it with a new roof. As the operating height is now below the wall height, there are a variety of options available. For example, aluminum geodesic dome roofs have been used to either add new roofs or retrofit existing roofs to many types of reservoirs. As an aluminum dome roof is relatively light, strengthening of the existing walls and foundation would be limited if required at all. Alternately, a new concrete dome or flat roof could be designed and installed.



Thermal Expansion | Seismic Loads | Roof-to-Wall Retrofit Brackets

If the existing dome roof cannot be removed, a retrofit bracket similar to as shown above could be installed. This type of bracket is configured to allow for thermal expansion of the roof while restricting lateral movement due to a seismic event. This type of connection would not impart additional operating or thermal loads on the walls. In a seismic event the brackets would "catch" the roof limiting is movement and transferring its lateral load into the walls. This option could potentially be more difficult to implement (versus an aluminum dome roof) as it requires an elastomeric bearing pad to be placed between the roof and the top of the wall. Lifting the roof to install such a pad might not be practical. However, this type of retrofit would allow the current roof to remain without it being demolished.

Alternately, retrofit brackets could be installed and the bearing pad omitted with the knowledge that in a seismic event the top of the wall connection could be significantly damaged, but the brackets would retain the dome and help prevent a complete failure of the roof.

General Recommendations

PSE recommends the exterior wall be cleaned to remove all concrete which has been damaged due to the thermal expansion effects. As thermal movement is likely to continue to occur, PSE does not recommend stiffening or reinforcing this area. By constraining the roof, the failure zone could be moved and potentially cause issues within the dome roof itself if it is constrained against expansion. Rather, the cracking around the exterior should be cleaned and coated to protect any reinforcing against water infiltration and to prevent further damage to the concrete. Coatings and any repair medium should be flexible to prevent cracking during thermal movement. This is not a long-term fix but intended to limit the impact of water and corrosion on this area until a new roof or roof-to-wall retrofit solution is selected. Once the area is cleaned and any damaged concrete removed it is recommended that it be observed by a Structural Engineer to review the extent of damaged concrete and to determine if any additional or alternate repairs are advisable at that time.

Around the reservoir there are a few trees that are in close proximity to the reservoir walls. These trees should be monitored and trimmed and/or cut it they begin to overgrow the reservoir or their root system looks to be damaging the reservoir body. As a rule of thumb, trees should ideally be no closer to the reservoir than their height (i.e. a 10-foot tall tree should be no closer than 10-feet to the reservoir). This will help limit any impact of the tree's root system and the potential for falling trees or branches to damage the vents and hatches or impacting the dome roof in a large storm event.

Finally, the valve vault piping should be retrofitted so as to ensure the piping has flexible fittings which allow for differential horizontal and lateral movement to occur between the vault and reservoir in a seismic event. As the structure is in close proximity to the reservoir's foundation, which is cast as part of the vault's wall, there is a potential for settlement or movement at this interface. Notches or overflow scuppers should be installed in the parapet to prevent the roof from overflowing if the drain backs-up.

2.6 Scans of Select Construction Documents

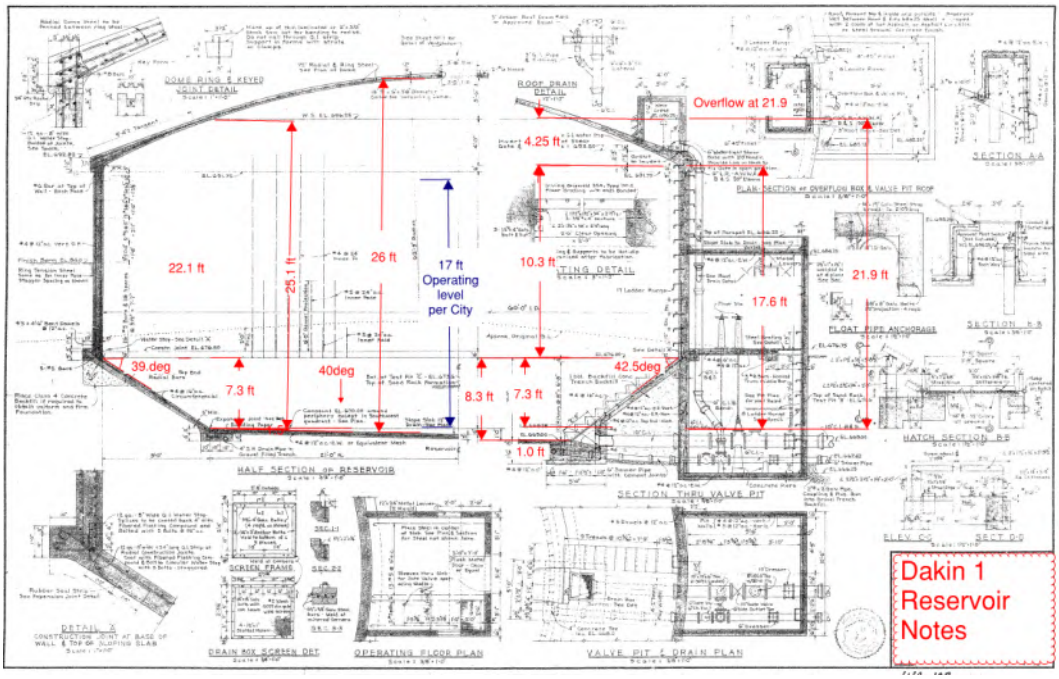


Figure 2-1: Dakin 1 Reservoir Sections and Details – Assumed Reference Drawings

(Drawings are based on 40th St Reservoir w/ Field Annotations added from the Dakin 1 Site Observation)

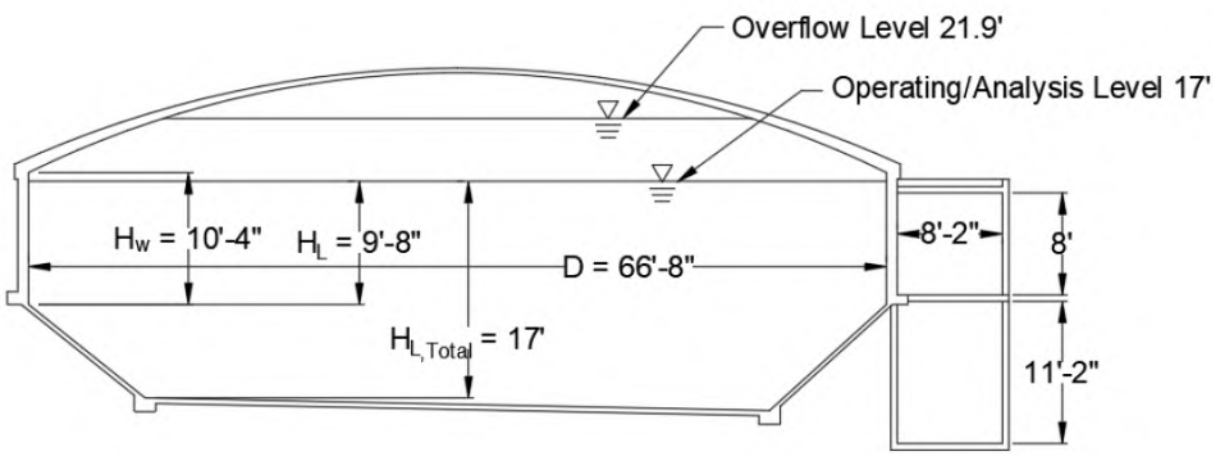


Figure 2-2: Dakin 1 Reservoir Elevations Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, $H_{L,Total}$ = Total Operating Water Height relative to Base)

2.7 Observations Pictures

Figure 2-3: Dakin 1 Reservoir – Elevation

Figure 2-4: Dakin 1 Reservoir – Entry to Valve Vault

Figure 2-5: Dakin 1 Reservoir – Circumferential Cracking around Reservoir below Roof Line

Figure 2-6: Dakin 1 Reservoir – Circumferential Cracking at Valve Vault

Figure 2-7: Dakin 1 Reservoir – Dome Roof Vent

Figure 2-8: Dakin 1 Reservoir – Circumferential Cracking around Dome Roof Edge

Figure 2-9: Dakin 1 Reservoir – Steps Cast into Hopper Base and Outlet

Figure 2-10: Dakin 1 Reservoir – Dome Roof Interior Side of Vent

Figure 2-11: Dakin 1 Reservoir – Reservoir Wall

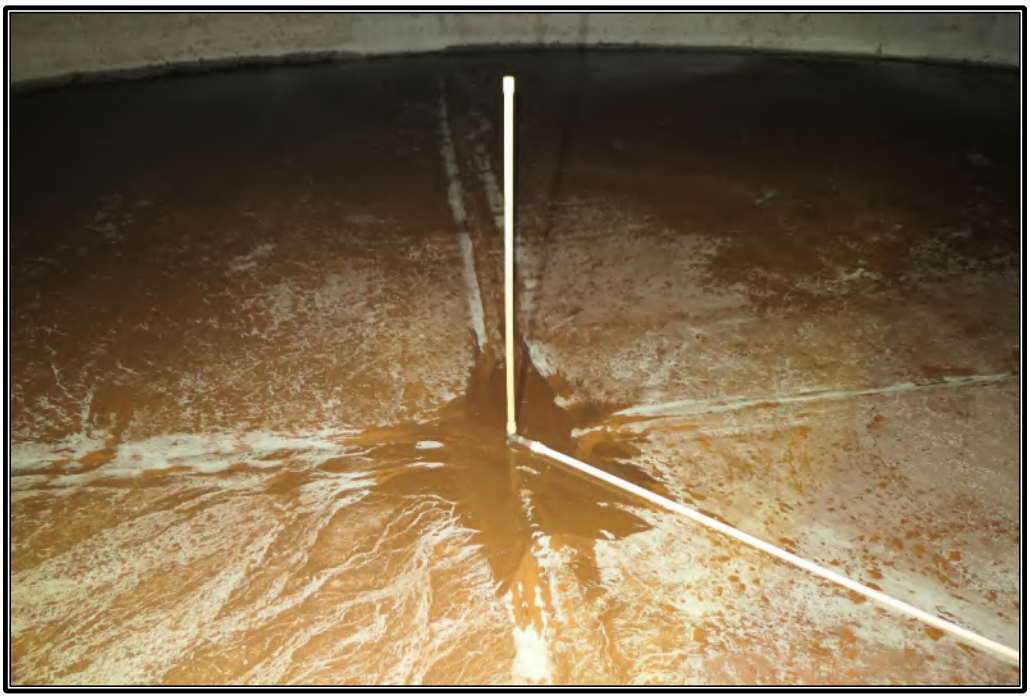


Figure 2-12: Dakin 1 Reservoir – Reservoir Floor

Figure 2-13: Dakin 1 Reservoir – Access Hatch

Figure 2-14: Dakin 1 Reservoir – Overflow Weir Gate, Shear Gate Handle, and Guide - Located in Access Hatch

2.8 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Inspection

PROJECT NUMBER: A1802-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: Dakin I 2918 Sylvan St, Bellingham, WA
48.7690, -122.4203

Site Visit Date: 4/9/19 Reservoir Type: RC Round - Dome Roof/Hopper Base

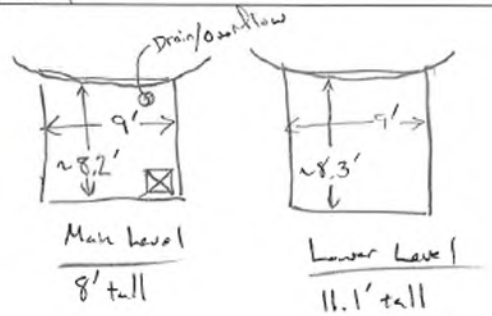
Temperature and weather: 46°F, overcast & light rain

Site Conditions: Wet. Water on ground on SE side. Also water running in front of vault is from pipe leak by hydrant in front of vault.

PSE Staff: Greg Lewis

Client/Other Staff: Jeremy, NWC; Nathan/Cory, Murraysmith; Cory of Bellingham

Valve Vault:



7" parapet on roof over flow drains into pipe on roof

Vault, min. crack or issues on interior. Wall @ reservoir joint has some seepage. Side walls 17½ x 11½ vents, 3' x 7' door opening opening to lower level 25½ x 30. Slab floor 6" thick with area of blow-out around embedded metal (old hangers or form ties?), 4 locations but otherwise slab is competant. Appox. 6" x 16" bench (part of res. footing?) locate on under side of slab.

1

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to ~~Top~~ Bot. of Roof Slab: ~~(N)~~ 1 9' ~~(E)~~ 2 2.7' ~~(S)~~ 3 2.8' ~~(W)~~ 4 3.6'

Roof Slab Thickness: N/A / 16" @ edge (drawings/measured) Roof Overhang Dimension: N/A / 3" w/ chamfer (drawings/measured)

Drip Groove? (Y/N): N/A / No (drawings/measured)

Top Surface Roof Slab Condition: No major cracks or loss. Primarily circumferential cracking w/ radial cracks - (see next pg)

Ladder/Vents/Hatch/Joint Conditions: Vent: has SS jacket, top has coating delam.; Ladder: rebar rungs in good condition, new aluminum ladder and landing in good condition; Hatch: gen good condition w/ cracking at base

Other Comments:

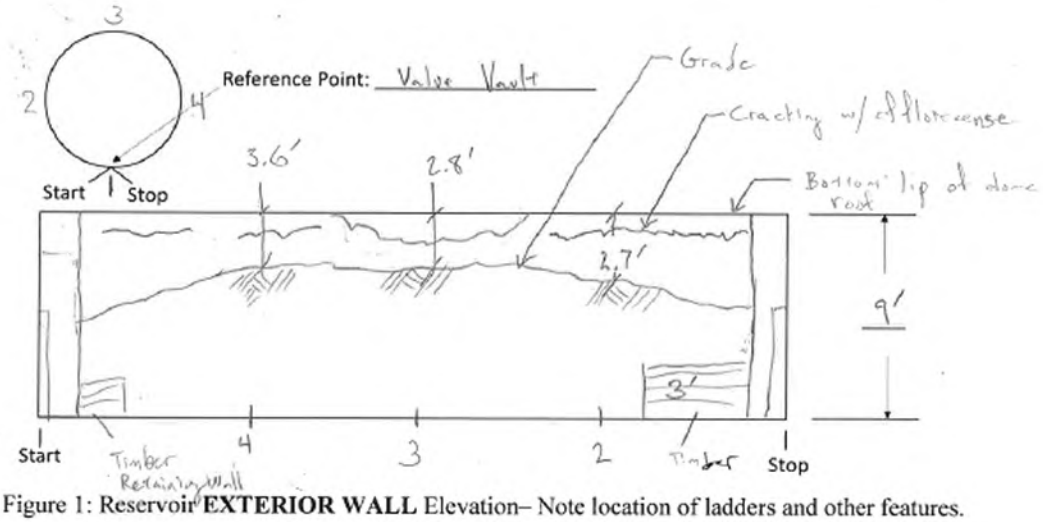


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

Trees located on either side of vault. Roots likely in contact w/ tank and could cause issues. Larger trees outside of fenced area.

PSE PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

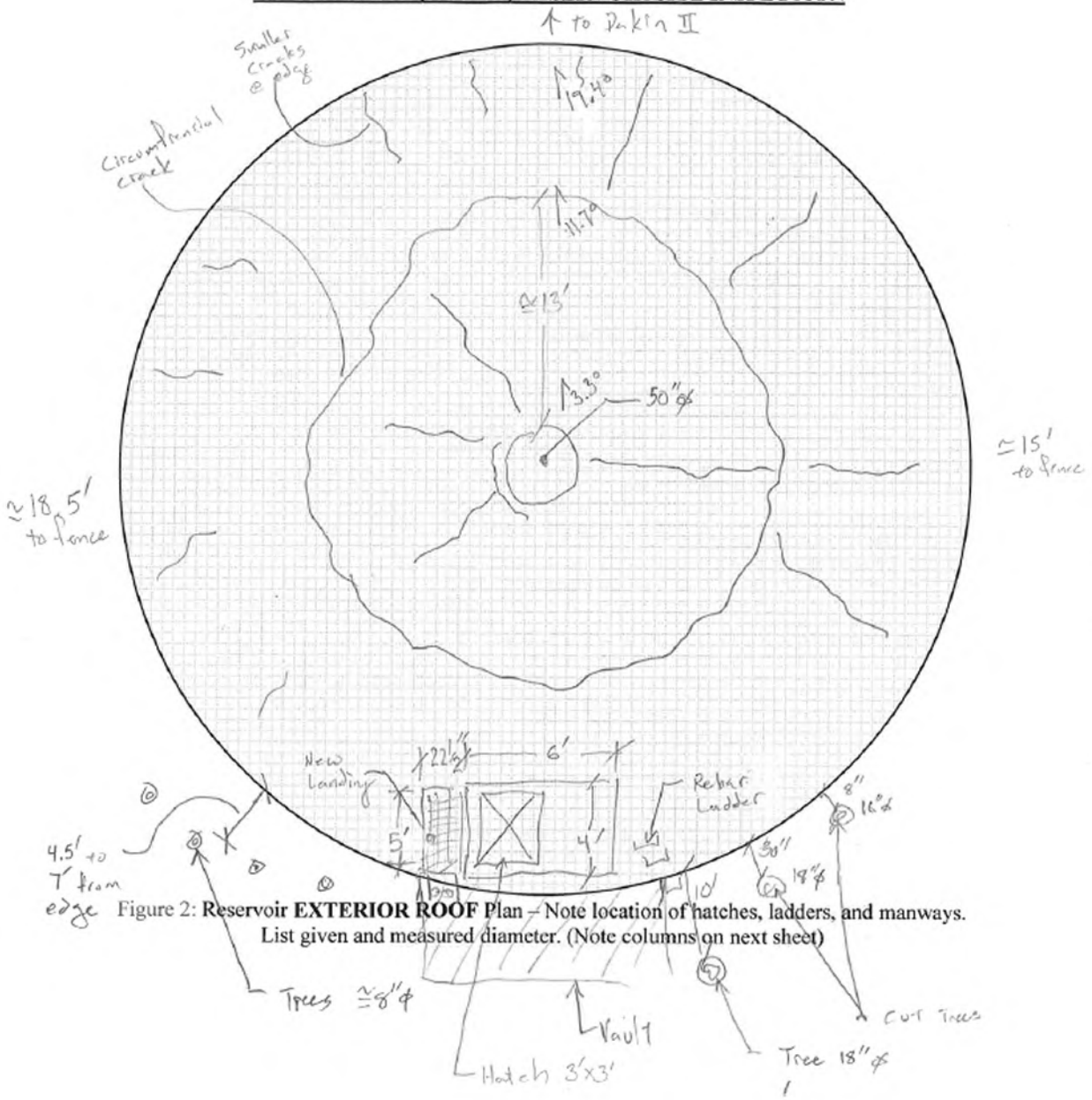


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: (Gen. good), incidental cracking as noted, other smaller cracks throughout

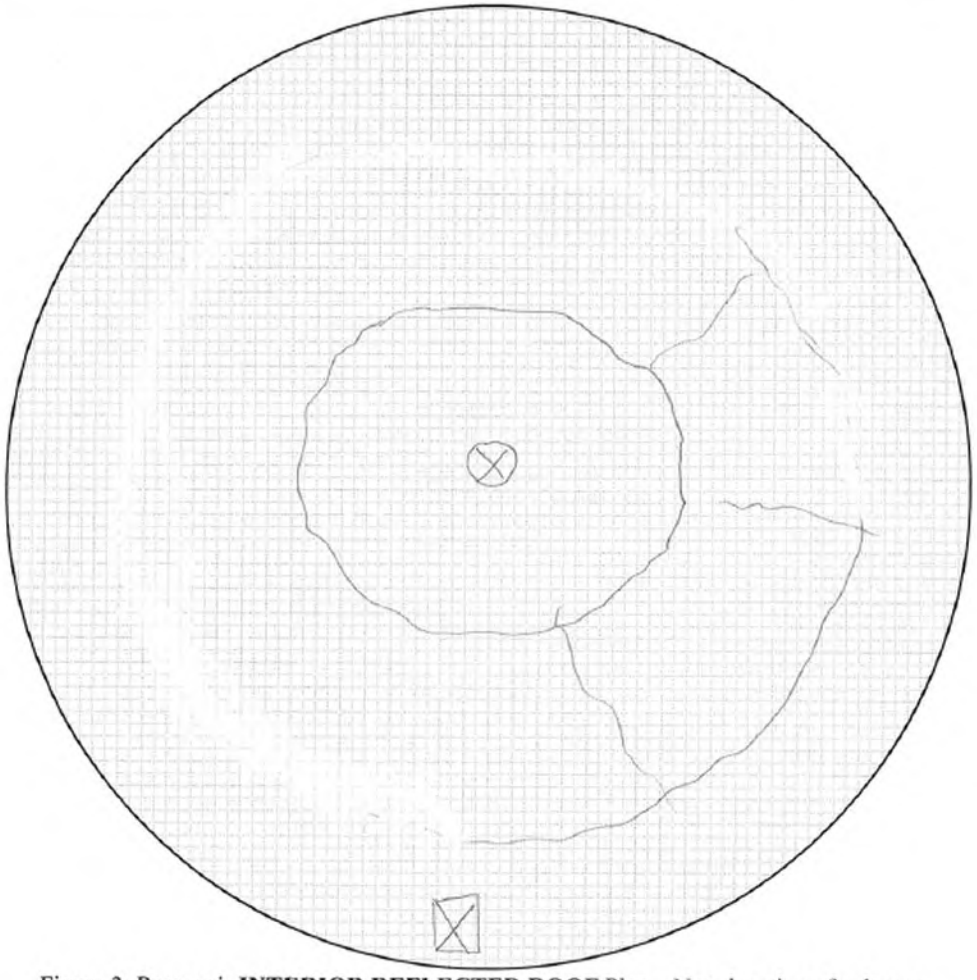


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

4

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

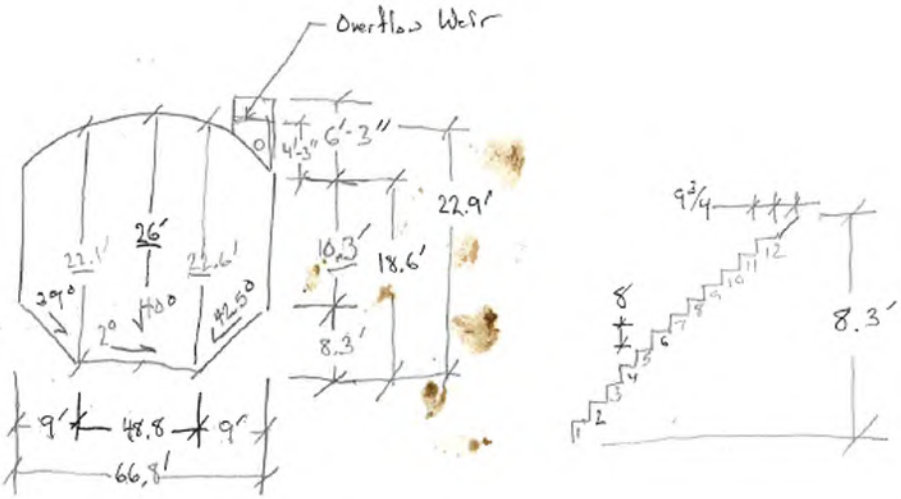
Column Diameter: N/A / N/A (drawings/measured) Footing Size/Thickness: N/A / Not Observed (drawings/measured)

Column Spacing: N/A / N/A (drawings/measured) Wall Curb Dimensions: N/A / N/A (drawings/measured)

Floor Slab Condition: Good, any defects covered by coating

Floor Slab Joints Spacing/Condition: 2 joints (divide slab into 4 quads) coating over joints intact.

Column/Footing Conditions: N/A



5

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

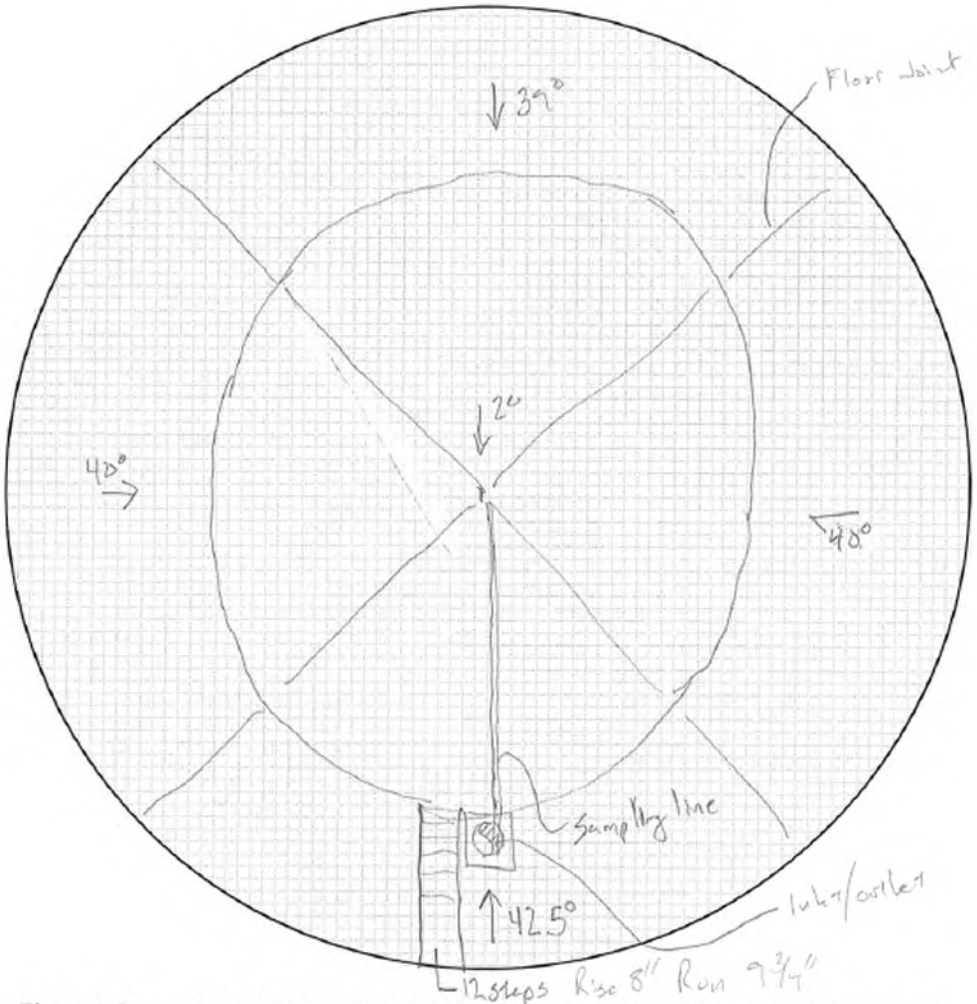


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

See Dim. Note other page.
No cracking/coating good

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Good

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): None

# of wall sections: N/A

Ladder/Pipes/Overflow Conditions:

Overflow Height: 20' / 22.9' (drawings/measured) City Per Site Visit Operating Height: 13.5'-17' (per City/PUD/other)

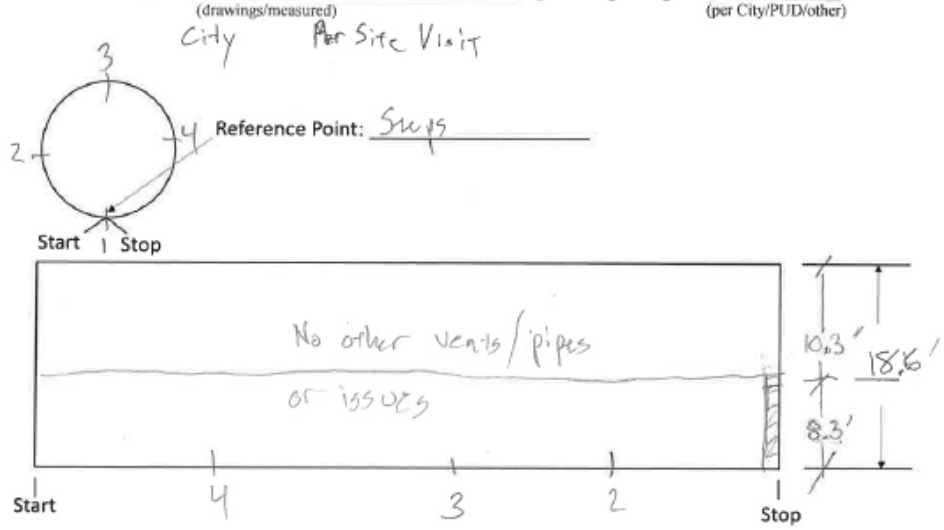


Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

7

END OF SECTION

Appendix J-4 Dakin I General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Dakin I Reservoir *General Info*

Field Visit Date: 4/8/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	4/8/2019
Reservoir Name and Location:	Dakin I - Balsam Ln, Parking ~540 ft E of Sylvan St; SW of Dakin II
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis
Client Staff Present:	Shayla Francis, Nick Leininger
Year Constructed:	1987
Overflow Destination:	SW - Storm drain MH
Discharge Destination/Zone:	SW side of reservoir; To Dakin-Yew 519 Zone
Fill Location:	SW
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.5	MG
Diameter (or other dimensions - see notes):	64	ft
Height	22.1	ft
Overflow Elevation:	519	ft AMSL
Bottom Elevation:	497.1	ft AMSL
Level of Overflow	21.9	ft
Minimum Normal Operating Level:	13.5	ft
Maximum Normal Operating Level:	17	ft

Notes: No specific as-built plans for reservoir, so 40th St reservoir plans were used as a guide for assessment (similar construction, but different overall dimensions). Heights are 22.1 ft to the knuckle and 26 ft to the top of the dome.

Dakin I Reservoir *Exterior Inspection*

Field Visit Date: 4/8/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	none	
Security Condition:	N/A	
Wall Attachment Type:	Set into roof	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	7	in
Back Clearance:	N/A	in
Notes: Smaller ladder to pumphouse roof: 13 inch width, 13 inch spacing, 5.5 inch front clearance; ladder condition is fair - coating failing near anchor points.		

Exterior Fall Prevention System:	
Present at Site:	No

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof west	
Material:	Aluminum	
Condition:	Very Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	N/A	
Measurement Type	Measurement	Unit
Size:	3x3	ft
Curb Height:	3.6 from base	in
Notes: Minor cracking at base. Water from hatch appears to drip onto reservoir roof and toward SE into crack.		

Roof Vents and Screen:		
Material:	Concrete	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	Unknown	in
Notes: 4 ft 2 in diameter of top. Screen appears to be in good condition based on picture taken form under sheet metal. Top coating failing.		

Roof:		
Condition:	Fair	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	Paint	
Slope of roof	Varies (3.3, 11.7, 19.4 degrees)	
Measurement Type	Measurement	Unit
Overhang Distance:	2.5	in
Thickness of roof slab	Varies (4.5 to 12)	in
Notes: Slab widens near the walls, so unable determine exact thickness. Minor primarily circumferential cracking w/radial cracks. Roof is within drip line of trees.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Very Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Very Good	
Attachment Type:	Set into roof	
Measurement Type	Measurement	Unit
Toe Guard Height:	N/A	in
Top Height:	3.5	ft
Notes: Railing present on grating only		

Grating:	
Present at Site:	Yes
Material:	Aluminum
Condition:	Good
Corrosion present?	No
Clips:	No
Removable Panels:	No

Measurement Type	Measurement	Unit
Approximate Panel Dimensions:	5x2	ft
Notes:		

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Fair
Notes: Cracking with efflorescence running throughout.	

Exterior Coating	
Exterior Walls:	Paint
Exterior of Roof:	Paint
Exterior Piping:	Paint
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	24-33 mils
Exterior Coating Adhesion Testing Results:	Poorly adhered
Notes: Exterior coating is peeling off near cracks and on valve vault.	

Dakin I Reservoir *Interior Inspection*

Field Visit Date: 4/8/2019

Interior Ladder:	
Present at Site:	No
Notes: Used Removable ladder	

Interior Fall Prevention System:	
Present at Site:	No

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
N/A	N/A	ft
Notes: Incidental cracking noted with other smaller cracks		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: Any defects are covered by coating. Floor grade is 2 degrees.	

Walls:	
Condition:	Good
Painters Rings Present:	No
Notes: interior diameter 66.8 ft. blistering coating.	

Interior Coating	
Interior Walls:	Polyurea
Interior Floor:	Polyurea
Interior of Roof:	No Coating
Interior Ladder:	N/A
Interior Piping:	N/A
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	80-100 mils
Interior Coating Adhesion Testing Results:	Well adhered
Notes: Interior coating has blistering.	

Dakin I Reservoir *Miscellaneous*

Field Visit Date: 4/8/2019

Piping		
Inlet Piping:	Size (Inches OD):	10
	Condition:	Fair
	Material:	Cast Iron
	Notes: Inside diameter is 11 inches at wall, then tapered. Inlet, outlet and drain single pipe in reservoir.	
Outlet Piping:	Size (inches OD):	10
	Condition:	Fair
	Material:	Cast Iron
	Lip (Inches)	0
	Notes:	
Overflow Piping:	Size (inches OD):	6
	Condition:	Good
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	SE to storm drain
	Erosion Evident:	No
	Screen Condition:	Good
	Overflow to roof (feet)	4.1
	Notes: Combines with drain and vault roof drain in vault	
Drain Piping:	Size (inches OD):	6
	Condition:	Fair
	Outlet Location:	SE to storm drain
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	N/A
	Air Gap:	No
	Screen Condition:	N/A
	Notes: Combines with overflow in vault if valve is opened.	

Piping Facilities		
Exterior Valving:	Type:	Gate valves
	Condition:	Fair
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	Outside
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	N/A
	Leaks:	N/A
Notes: Fire hydrant was used for washdown.		

Electrical	
Cathodic Protection:	No
Impressed Current:	No
Anodes:	No
Notes: Concrete reservoir	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	Yes
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	0.1 to 0.4
Water Quality Sample Taps?	Yes
Notes: Sample pipe is 1/2 inch PVC located within inlet/outlet/drain. Height is 44in above floor.	

Appendix J-5 Dakin I Condition Assessment Score Sheet

Dakin I Reservoir Condition Assessment										
System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance		WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
				Static	Seismic					
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	4	0	No Camera
	Vegetation Separation	0	0	0	0	0	0	1	0	Under the dripline of trees. Organic debris on roof.
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	3	3	2	2	0	0	3	0	Roof not designed for thermal movement
	Interior Walls	3	4	3	3	5	0	5	0	Some blistering in coating
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	3	4	5	5	4	0	5	0	Lots of sediment - needs cleaning
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	4	4	5	3	5	0	5	0	Ext roof needs cleaning
	Interior Roof and Supports	0	4	5	3	0	0	0	0	Int roof uncoated
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	5	5	0	No fall protection required
	Interior Ladders/Fall Protection	0	0	0	0	0	3	1	0	Large pipe prevents makes egress very difficult
	Access Hatches	5	4	0	0	3	0	3	0	Difficult to get into hatch. High maintenance design.
	Railings and Roof Fall Protection	5	5	0	0	0	4	0	0	Roof to ground is five feet.
	Vents	4	5	0	0	4	0	5	0	Screen likely too coarse. Passed design checks.
	Balconies/Landings/Grating	5	5	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	5	5	0	3	4	0	5	0	Comb in/out/drain.
	Outlet Piping	0	0	0	3	0	0	5	0	Comb in/out/drain.
	Drain Piping	0	0	0	3	3	0	3	0	Comb in/out/drain. Connects to storm sewer without an air gap. No Silt stop. Unknown Dechlorination
	Overflow Piping	5	4	0	0	4	0	0	0	
	Washdown Piping	0	0	0	0	0	0	5	0	Fire hydrant for washdown operations
	Attached Valve Vault Structure	2	3	4	3	0	0	4	0	Minor spalled concrete (under vault slab), leaking/staining at joints. Roof drainage needs imp.
	Control Valving	4	4	0	0	5	0	5	1	
	Isolation Valving	4	4	0	0	0	0	5	1	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	4.1	4.2	4.1	3.3	4.1	4.0	4.2	1.0	

Overall Score
3.7

Appendix K Reveille

Appendix K-1 Reveille Geotechnical Report

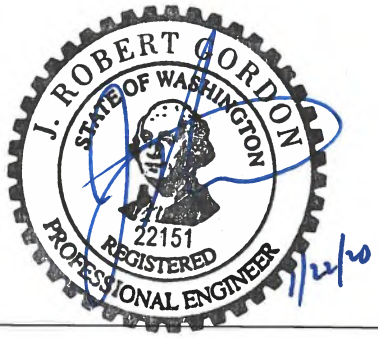
GEOENGINEERS

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Reveille Site



INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Reveille reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at the Reveille site, located as shown in the Vicinity Map, Figure 1. The Reveille reservoir is a round reinforced concrete structure with a hopper base.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by undifferentiated glacial deposits. The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift.

Surface Conditions

The project site is located approximately 100 feet north of Reveille Street and 50 feet west of Yew Street. The reservoir is located in a small clearing that slopes gently downward to the north. The site is bounded by a wooded area to the north and west, south by Reveille Road and east by Yew Street. A small gravel roadway leads to the site from Yew Street.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-7 (2019)—on March 26, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 8 feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan, Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites..

- **Fill** – Fill was encountered at the surface of the exploration and extended to 4½ feet bgs. The fill consists of red-brown silt with sand, gravel, and occasional organic matter.
- **Glaciomarine Drift** – Glaciomarine drift was encountered between 4½ to 7 feet bgs. The glaciomarine drift was comprised of very stiff brown silt with sand and gravel.
- **Chuckanut Sandstone** – Chuckanut sandstone was encountered at 7 feet bgs. The boring was completed at 8 feet bgs within the Chuckanut sandstone. The fine to coarse grained sandstone was brown with weak cementation and occasional bedding planes.

Groundwater

Groundwater seepage was not observed at final depth of the boring. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or "pods" at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

The reservoir site is underlain by bedrock from the ground surface. Based on review of the project plans by John W. Cunningham & Associates dated August 1958, the foundation for the reservoir extends approximately 10 to 12 feet below existing grade. Therefore, based on the results of our boring and review of the as-builts, the reservoir base is constructed on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago.

Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on sandstone which is not at risk of liquefaction.

American Concrete Institute/American Society of Civil Engineers 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	95.7
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.6
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.396
Seismic design value, S_{DS}	0.649
Seismic design value, S_{D1}	0.357

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_w 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash

deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	12	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.32	0.58	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeters per second, g = gravity

M_w 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends more than 78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on

Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	18	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.40	0.72	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and review of as-builts referenced previously, the existing reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The reservoir includes below grade walls. Our recommendations for concrete below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the "Shallow Foundations" section and backfilled with structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Global Stability

As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock. No significant slopes are located close to the reservoir. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:tln

Attachments-

Figure 1 – Vicinity Map

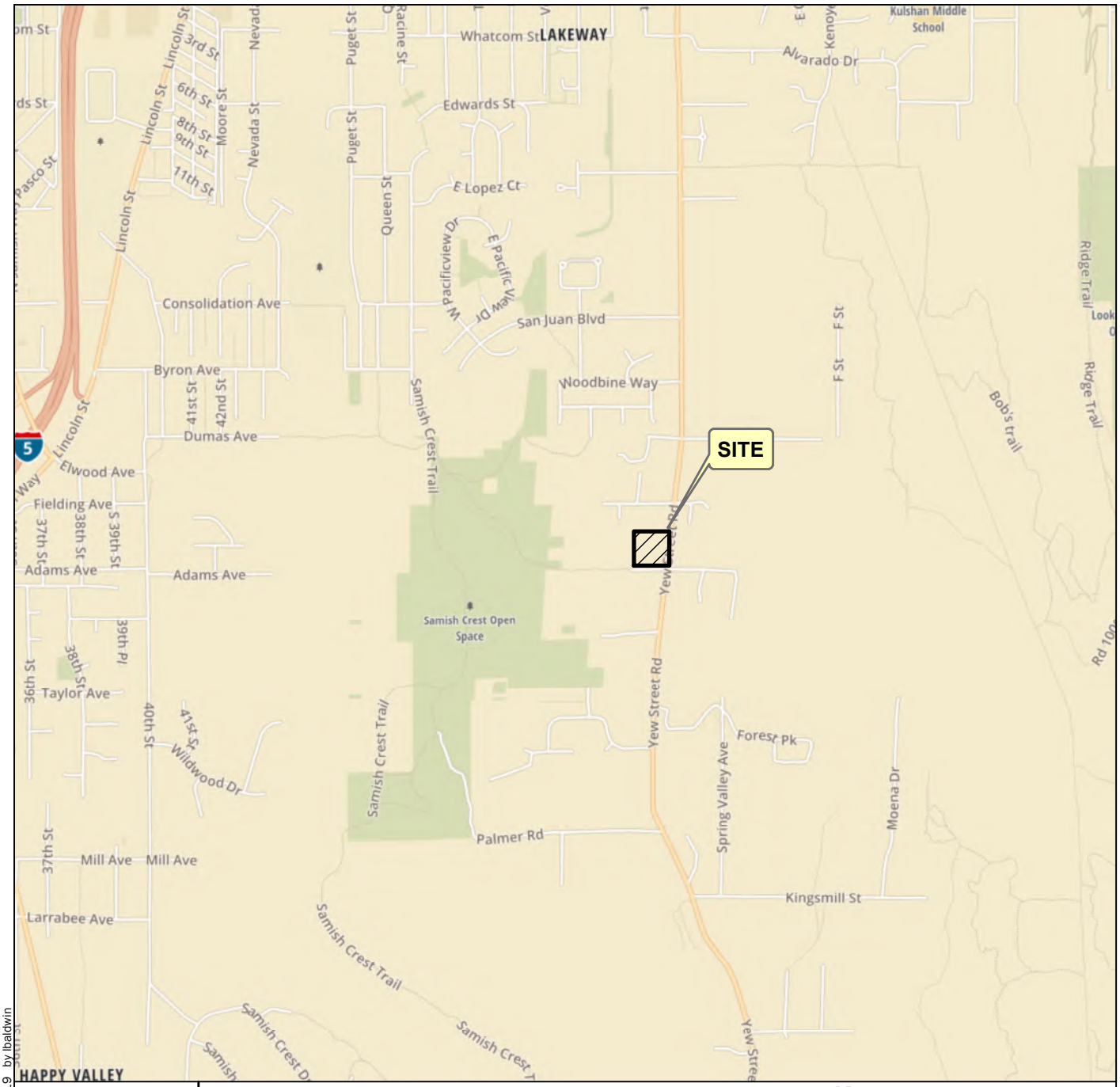
Figure 2 – Site Plan

Figure 3 – Key to Exploration Logs

Figure 4 – Log of boring B-7

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault



N
W E
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1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Reveille Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_Reveille.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

75 0 75

Feet

N W E S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Reveille Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

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Figure 3

Drilled	Start 3/26/2019	End 3/26/2019	Total Depth (ft) 8	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	690 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) Northing (Y)	1251830 634560		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	6 inches topsoil			
								ML	Red-brown silt with sand, gravel, roots and occasional organic matter (medium stiff, moist) (fill)			
		☒	18	16		1 MC				17		
685								ML	Brown silt with sand and gravel (very stiff, moist) (glaciomarine drift)			
	5	☒	18	29		2 MC				15		
		☒	4	50/4"		3		Sandstone	Brown sandstone with gray sandstone fragments (Chuckanut Formation)			

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-7

GeoEngineers

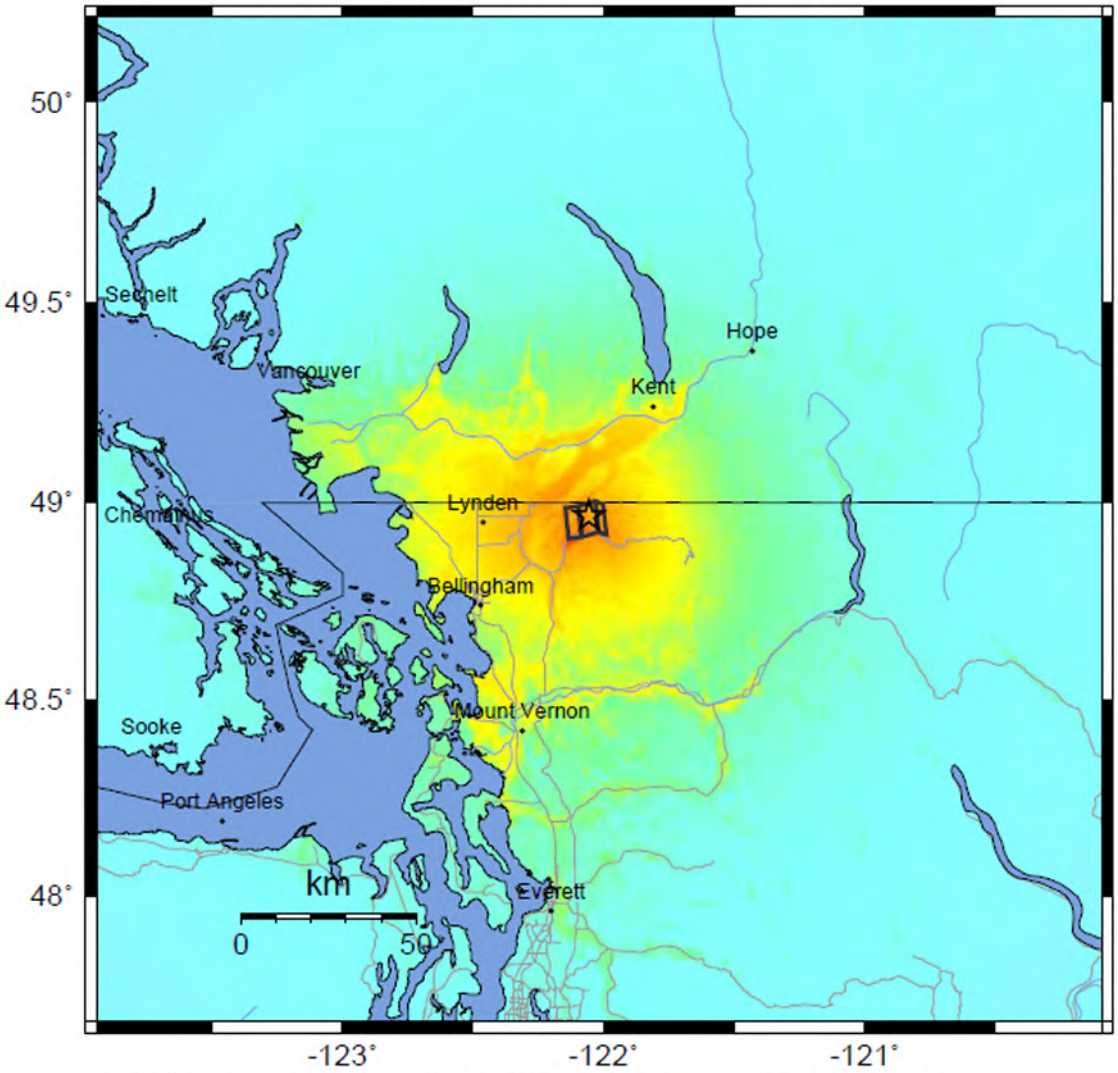
Project: COB Reservoir Inspection and Repair - Reveille Street
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

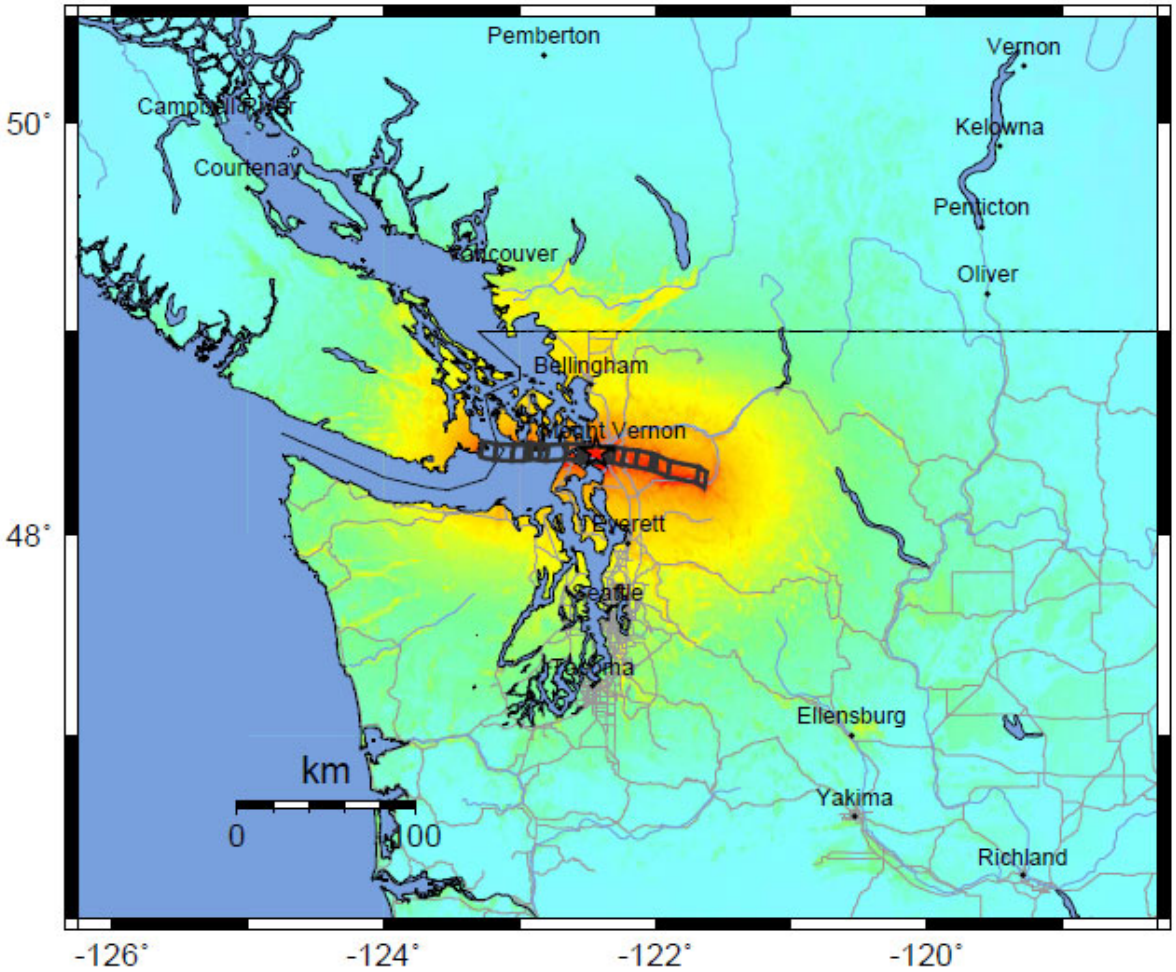
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix K-2 Reveille Structural Report

CITY OF BELLINGHAM

CH 13: REVEILLE RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Reveille, 0.3 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located near 2400 Yew St Rd, Bellingham, WA (Lat. 48.729, Long. -122.443), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on May 21st, 2019 by Peterson Structural Engineers (PSE) and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 Reveille Round Reinforced Concrete (RC) Reservoir – 0.3 MG

2.1 Description & Background

Per information provided by the City, the Reveille Reservoir was designed by John W. Cunningham & Associates Consulting Engineers and built around 1958. The City was able to provide design drawings for the reservoir. The reservoir has a storage volume of around 0.3MG, a dome roof, and a hopper base. The reservoir is a round reinforced concrete reservoir with a measured interior diameter of 50-feet. The interior wall was measured to be 13-feet high and the hopper base 6-feet deep. The overflow weir is located approximately 23.25-feet above the bottom of the reservoir and is located above the top of the wall within the access hatch box. The reservoir uses a portion of the dome roof for its storage volume at full capacity.

Where details or sections could not be directly observed or measured, the original design drawings have been used as a reference. Per these drawings the wall is 10-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. The roof is a reinforced 4.5-inch thick dome with a thickened edge with circumferential hoop reinforcing. The floor is a reinforced 5-inch thick hopper-sided slab that transitions into a 12-inch thick by 10-inch wide footing. Where piping is run under the footing to the valve vault, the drawings show that the piping is encased in an unreinforced concrete block for protection. The reservoir section drawings are shown in Figure 2-1 and Figure 2-2 while a schematic reference drawing denoting the variables used for the analysis is provided in Figure 2-3.

2.1.1 Description of Additional Site Structures and Features

The site includes an attached valve vault which contains equipment associated with the reservoir's operation. This valve vault was constructed as part of the reservoir and the rear of the vault shares a wall and footing with the reservoir. This vault is located on the northeast side of the reservoir and is two levels, with one level located below grade. The vault is 8-feet deep by 8-feet wide and the main level has an internal height of 8-feet while the lower level is 9.5-feet high. The lower level is accessed via a 24 by 30-inch opening in the main level's 6-inch thick slab floor. The reservoir's drain, outlet, and overflow are all run through the valve vault. The roof of the vault has a parapet and the air gap for the reservoir overflow is located on the roof. If debris were to clog the overflow, the layout of the top of the valve vault is such that it would fill with water up to the height of the parapet on the roof. While the roof had a secondary drain, it is insufficiently sized to handle any overflow volume. Additionally, as the overflow and drain pipes are joined below the roof line, any material or issue that blocks the overflow would also be likely to clog the drain as well.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit to observe the as-built current condition of the reservoir's interior and exterior as well as the general site conditions. The reservoir was drained for our inspection and the site visit was performed May 21st, 2019.

<u>Dome Roof:</u> The reservoir has a self-supporting concrete dome roof with a thickened edge. The surface of the roof is coated, and this coating appeared to be largely competent. Structurally the roof appears be in fairly good visual condition with some zones of cracking noted. This cracking included a circumferential

crack that runs around the entire circumference of the roof approximately 7-feet up the dome. Additional radial cracking was observed to run perpendicular to the circumferential crack, towards the roof edge. The radial cracks are approximately at 4 to 5-feet on center. Per the reference drawings, it is assumed that the radial roof dome reinforcing is fanned out along the circumference with additional reinforcing added as the width between the fanned area increases. This cracking looks to be occurring roughly along the line where the additional reinforcing at the fanning zone begins and could be a result of a stiffness differential in the roof.

The roof has one 3 by 3-foot access hatch which is part of a larger access box. This box is formed to include the reservoir's overflow weir. The location of the overflow is above the top of the wall, which necessitates a waterstop in both the roof-to-wall joint and access hatch-to-roof joint. While the anchors attaching the access hatch to the concrete appear to be corrosion resistant steel the nuts and washers do not. The nuts and washers should be replaced to match the anchor bolt steel. Overall the hatch appeared to be in good condition with minimal instances of cracking or other structural defects noted.

At the center of the roof, the reservoir has a 36-inch diameter vent. Observable components and the surrounding roof do not appear to have any visual structural issues associated with the vent. Per the available reference drawings and site observation of the interior of the vent there are a series of (12) 4-inch by 11-inch openings around its exterior. However, these openings are obscured by a sheet-metal cover. Please note, the tightness of the metal cover could pose issues with the venting requirements and Murraysmith should be consulted to determine if a problem exists due to potential restricted airflow and how to mitigate this potential issue. Inadequate venting can create significant structural loads when the reservoir is filled or drained if the vent cannot keep up with the change in storage volume.

<u>Reservoir Walls and Interior:</u> The reservoir is constrained by a soil berm which is backfilled to about 1 to 2-feet below the dome roof edge, the backfill extends around the majority of the reservoir's circumference. As a result, PSE was only able to assess a limited section of the exterior wall. Cracking with efflorescence was noted below the dome roof around most of the structure constrained by the berm. Where the berm height is reduced to make room for the vault, the crack-line slopes along a diagonal to intersect with the roof of the vault.

Per the referenced drawings, the walls are 10-inches thick with vertical and horizontal (hoop) reinforcing. At the top of the wall, there is a keyed joint and two (2) vertical bars of reinforcing are located at 12-inches on center around the top of the wall to anchor the dome roof to the wall. This connection is required as the design of the reservoir intends for water to be stored above the top of the wall and within the roofline of the dome. This attachment results in a restraint that does not accommodate any thermal expansion of the roof. Thermal expansion occurs due to a change in temperature and will cause a material to expand or contract. In this case the roof would be expected to expand radially outwards or contract inwards. For the walls this expansion/contraction results in thrust loads perpendicular to the wall face. For this wall, which is constrained by a berm, this load can result in the shear cracking observed around much of the reservoir below the roof edge. Where the berm constraint is not present, such as near the vault, the cracking migrates until the wall is again restrained by the vault. This results in the observed diagonal cracking.

The interior of the reservoir was visually observed to be in fair to poor condition. Unlike other similar types of reservoirs in the City's inventory this reservoir did not have an interior coating on the walls. This absence of coating allowed for a better view of the cracking in the walls and along the hopper base. Interior cracking was observed around the circumference along the upper portion of the wall. The wall crack appears to generally follow the observed exterior cracking around the circumference at the top of the wall; they also follow the diagonal cracking associated with the vault. Near the ladder, where the circumferential wall crack could be observed up close, the crack width was measured to be in the 1/8 to 1/4-inch range. In addition to circumferential wall cracking, circumferential cracking was also noted along the upper section of the hopper base.

The hopper base was also found to have instances of radial cracking. These radial cracks were more pronounced towards the side and rear of the reservoir opposite the side restrained by the vault. The hopper cracks were found to be smaller than the wall cracks and in the 1/16-inch range. For the dome, a majority of the surface appeared to be in fairly good visual condition, but a couple larger cracks were noted and had associated efflorescing. These cracks appeared to correspond with the exterior circumferential crack. While cracking was prevalent, no instances of concrete spalling or exposed rebar were observed.

Appurtenances: The inlet/outlet pipe, itself, visually appeared to be in generally good condition. However, the mouth of the pipe had a rough-concrete interface. The grating of the pipe was removed but the remaining anchor points were noted to be heavily corroded. This reservoir ladder was also noted to have a high amount of corrosion along with section loss and should be considered unsafe to use. Located in the access hatch are the concrete overflow weir and steel shear gate. The concrete weir box was observed to be in good condition while the steel gate components were observed to have some surface corrosion. This corrosion did not appear to have resulted in any section loss and the shear gate and handle were in fair condition.

2.2.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appears to be in generally good visual condition with no major structural defects observed. No visual signs of major cracking or settlement issues were found. It was noted that one of the pipe penetrations through the slab floor was poorly executed but based on the redundancy of the floor slab reinforcing, this penetration is not likely to have a major impact on the overall structural capacity of the floor. This vault shares a wall-line with the reservoir, and some minor issues with water infiltration were observed along the wall edges in the upper vault. Based upon the reference drawings the vault was constructed using a keyed joint along the wall interface and has reinforcing which connects it to the reservoir wall. This joint does not include a waterstop, rather the vault roof-to-reservoir wall joint is built up and sloped away from the joint. In the lower vault, water infiltration was more significant with the likely origin being the reservoir. Three locations of rust-tinged water were observed, as shown in Figure 2-10. This water could potentially be infiltrating through the circumferential crack observed on the interior of the reservoir in the hopper-base and which shares the wall-line with the vault.

2.3 Structural Analysis

The following structural analysis is based on the provided reservoir drawings and field measurements. For elements which could not be observed, such as reinforcing, the drawings were used for reference. Where elements could be observed and were found to vary from the design, the actual dimensions were used in PSE's analysis. Based on the results of PSE's analysis, potential issues and retrofit options are discussed.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Circular Concrete Tanks without Prestressing", published 1993 were also utilized.

2.3.1 Hydrostatic and Gravity Analysis

Dome Roof: The dome roof was evaluated per concrete design criteria as covered in AWWA D110-13 and found to meet minimum requirements for rise-to-span, thickness versus buckling, and edge reinforcing assuming the reinforcing is consistent with the reference drawings. The dome roof was evaluated at a 19-foot operating level. At this level the water is stored at the top of the wall and there are no additional hydrostatic loads on the roof.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will allow for differential thermal movement between the reservoir roof and the wall as both components deform differently as a result of temperature variations due to temperature or solar gain. At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the shear resisting structural elements of the reservoir and resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement. This is a result of its original design that allows water to be stored above the wall line. To adequately account for thermal movement the existing roof attachment would need to be modified or replaced with a roof system able to accommodate thermal movement. This may not be practical or economically feasible.

Wall Reinforcement: Per the design drawings the wall is reinforced with #5 vertical bars at 18-inches on center on the exterior and interior face and additional #5 intermediate vertical dowel bars at 18-inches on center at the base. This layout results in #5 bars effectively placed at 9-inches on center along the interior base. Horizontal (hoop) reinforcing density starts out with #5 bars at 5-inches on center towards the base and decreasing to #5 bars at 10-inches on center towards the top of the wall. At the very top of the wall there are two (2) #8 circumferential hoops. This variation in the hoop reinforcing is based on the variable pressure distribution resulting from the hydrostatic fluid load.

Per PSE's analysis, it was determined that the vertical wall reinforcing appears to be sufficient for current code strength requirements for the current operating level. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3

sanitary factor. This sanitary factor is intended to minimize the potential for cracking and leaks. While the reservoir is adequate for the current operating level, the wall reinforcing would be exceeded if the operating level were to be increased to the overflow level.

Additional checks were performed for the wall at the 19-foot operating level and determined the remaining wall reinforcing to be adequate. This included checks for shear loads, hoop tension forces (when accounting for the larger 1.65 sanitary factor required by code when checking reinforcing in tension), and compressive wall loads resulting from soil backfill.

Finally, per ACI 350.3, the maximum spacing for wall reinforcing was checked. The maximum allowable spacing for bars is limited to 12-inches on center. In the upper sections of the wall, the spacing of the interior and exterior reinforcing is at 18-inches on center, exceeding the maximum limit. While this is unlikely to be the sole issue causing the observed cracking, this larger spacing could be one of the reasons the cracking (especially the wide cracks noted adjacent to the vault) have been able to develop.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the geotechnical evaluation, the site's bearing capacity was determined to be 6,000-psf. Using this bearing capacity and checking for the 19-foot operating level up to overflow, the bearing pressure was determined to be within acceptable ranges.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: Per PSE's analysis, the addition of seismic loads results in additional forces on the wall of the structure. This is a result of the water slosh wave as well as forces resulting from the mass movement of the structure itself. PSE found that the wall flexural stresses were increased by about 23% when compared to the static loads at a 19-foot operating level. As the reservoir was designed for a higher static operating level, there is reserve capacity and the reservoir was found to be able to resist the increased seismic loads based on current design codes and the assumed construction values. Should the reservoir be operated at overflow, the seismic load would exceed the wall's flexural capacity and the reservoir would need to be retrofitted.

The wall's hoop tensile stresses were found to be within acceptable limits when the reservoir was evaluated at its overflow design level for hydrostatic loads. When the reservoir is evaluated at a reduced 19-foot operating level, sufficient capacity is available to meet design requirements from current seismic load and using modern loads factors. For lateral loads, the horizontal hoop reinforcing was determined to be adequate.

In addition to the wall flexure and tensile checks, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing to resist seismic lateral loads at the overflow level. No additional reinforcing or connectivity is needed between the walls and the foundation based upon the assumed construction.

Freeboard/Slosh: At overflow, this reservoir stores water above the top of the wall. For such an operating level, during a seismic event the roof would constrain the slosh wave. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. The force of this wave would be

sufficient to damage and potentially cause failure of the roof at the roof-to-wall interface, the dome itself, as well has hatches and other appurtenances. At the current operating level there is minimal freeboard which is insufficient to prevent a 2.4-foot slosh wave from impacting the roof. However, while there is inadequate freeboard, the current operating level is such that it results in a lower slosh wave impact force then would be experienced at the overflow level. Based on the available roof reinforcing, roof thickness, and weight, the roof appears to have sufficient capacity to resist the code determined maximum slosh load at the 19-foot operating level.

Valve Vault: Per the reference drawings the valve vault is connected to the reservoir with #5 rebar dowels at 12-inches on center. This attachment should limit differential movement or "pounding" that occurs in a seismic event. Additionally, where the hopper base and the lower level of the vault are adjacent, this zone is shown to be backfilled with plain concrete or "trench backfill". In the event of an earthquake, this will provide support to the hopper base so as to limit its potential to fail the lower level wall of the vault and collapse onto the piping. Of primary concern is the vault's wall which is cast as part of the reservoir footing. Depending on the direction of ground motion, pipes should be retrofitted to have flexible coupling should differential movement between the two structures occur during a seismic event.

2.4 Summary

Based on the available drawings and site visit it appears that a majority of the structural elements in the reservoir are adequate for the expected loads at the current operating level. It was noted that ringing the reservoir are a variety of cracks on both the interior and exterior of the wall that could potentially be a result of the combined thermal and operational loading conditions. Further, wall reinforcing was found to exceed maximum spacing allowances and these combined issues could be contributing to some of the cracking issues noted.

Elements outside of the wall, such as the dome roof and footing were determined to be adequate when operated at a 19-foot operating level. However, while reinforcing in these areas were found to be adequate, the dome-to-roof reinforced connection is a rigid connection and unable to resist expansion/contraction loads resulting from thermal effects. This is likely the cause of the damage noted around the reservoir at the roof-to-wall interface. As a result, the actual capacity at this connection is expected to be lower than our analysis has determined. This could be a potential concern in a seismic event.

Remaining observable components of the reservoir appear to be in generally fair condition although some areas are in poor condition. Cracking adjacent to the vault appears to have affected the entire cross-section of the wall with the effects of the cracking observable on the interior and exterior wall faces. Within the vault's lower level there was observed to be rust-tinged water infiltrating this area and the likely source is the reservoir itself. Additionally, the vault's pipe supports were noted to be non-flexible and the pipes should be upgraded to accommodate for potential vertical and horizontal differential movement between the vault and the reservoir, as might occur during a seismic event.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 19-feet, which is near the top of the wall. Due to the types of issues noted, these retrofits might not be cost effective or easy to implement.

Wall and Hopper Cracking

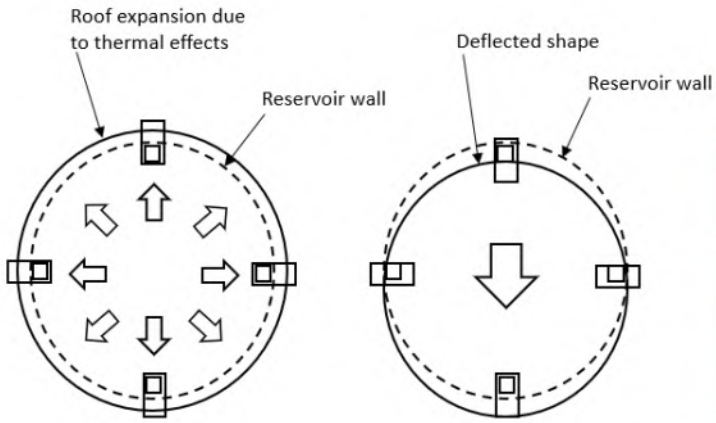
Based on a maximum operating level of 19-feet, PSE determined the wall flexural capacity had adequate strength for the current operating level. However, the reservoir's reinforcing does not meet requirements for the maximum allowable bar spacing. A potential retrofit would be to coat the interior wall and foundation of the reservoir to epoxy-inject and then use and exterior coating to seal any of the noted cracks. This coating would likely be a similar type to the other coating systems being used at reservoirs in the City's inventory (i.e. College Way, 40th St, etc.). Recoating would be beneficial in protecting underlying reinforcing against corrosion and to limit deleterious impacts to the reservoir's overall structural capacity. Please note coatings are outside of PSE's expertise and we would recommend consulting with Murraysmith and NW Corrosion to determine the best ways to seal observed cracks and coat the interior.

Roof-to-Wall Retrofit

The effects of thermal movement appear to have damaged the roof-to-wall interface and have likely affected its structural shear capacity and water-tightness. To alleviate the thermal issues, PSE would recommend either the retrofit or replacement of the roof. Note that this may not be economically feasible or practical.

The first option would be to remove the existing roof and replace it with a new roof of which there are a variety of options available. For example, aluminum geodesic dome roofs have been used to either add new roofs or retrofit existing roofs to many different types of reservoirs. As an aluminum dome roof is relatively light, strengthening of the existing walls and foundation would be limited if required at all. Alternately, a new concrete dome or flat roof could be designed and installed.

If the existing dome roof cannot be removed, a retrofit bracket similar to as shown above could be installed. This type of bracket is configured to allow for thermal expansion of the roof while restricting lateral movement due to a seismic event. This type of connection would not impart additional operating or thermal loads on the walls. In a seismic event the brackets would "catch" the roof limiting is movement and transferring its lateral load into the walls. This option could potentially be more difficult to implement (versus an aluminum dome roof) as it requires an elastomeric bearing pad to be placed between the roof and the top of the wall. Lifting the roof to install such a pad might not be practical. However, this type of retrofit would allow the current roof to remain without it being demolished. Alternately, retrofit brackets could be installed and the bearing pad omitted with the knowledge that in a seismic event the top of the wall connection could be significantly damaged, but the brackets would retain the dome and help prevent a complete failure of the roof.



Thermal Expansion | Seismic Loads | Roof-to-Wall Retrofit Brackets

General Recommendations

PSE recommends the exterior wall be cleaned and all efflorescence or loose concrete be removed. For damage due to the thermal expansion effects, these areas can be cleaned but left un-grouted. As thermal movement is likely to continue to occur, PSE does not recommend stiffening or reinforcing this area. By constraining the roof, the failure zone could be moved and potentially cause issues within the dome roof itself if it is constrained against expansion. Rather, the cracking around the base of the dome should be cleaned and coated to protect any reinforcing against water infiltration and to prevent further damage to the concrete. Coatings and any repair medium should be flexible to prevent further cracking during any future thermal movement. This is not a long-term fix but intended to limit the impact of water and corrosion on this area until a new roof or roof-to-wall retrofit solution is selected. Once the area is cleaned and any damaged concrete removed it is recommended that it be observed by a Structural Engineer to review the extent of damaged concrete and to determine if any additional or alternate repairs are advisable at that time.

On the interior of the reservoir, heavily corroded areas, such as the ladder rungs and around the piping should be removed, replaced, or cleaned. These elements should be stripped back to competent material so that no corrosion remains. Once cleaned any exposed rebar or anchors should be coated to prevent further corrosion. Along where the hatch is anchored to the concrete the corroded nuts and washers should be removed and replaced with new nuts and washers of either a similar material or using a separating material to prevent contact and corrosion between dis-similar metals.

Around the reservoir there are a few trees that are in close proximity and one tree in particular that is growing on the berm surrounding the reservoir. These trees should be monitored and trimmed back to limit debris which may collect on the roof and to prevent falling trees or branches that might damage vents or hatch elements. For the tree growing in the berm it should be cut as it's situated in such a way that it could dislodge during a seismic event and potentially impact the reservoir.

Finally, the valve vault piping should be retrofitted so as to ensure the piping has flexible fittings which allow for differential horizontal and lateral movement to occur between the vault and reservoir in a

seismic event. As the structure is in close proximity to the reservoir's foundation, which is cast as part of the vault's wall, there is a potential for settlement or movement at this interface. Notches or overflow scuppers should be installed in the parapet to prevent the roof from overflowing if the drain backs-up.

2.6 Scans of Select Construction Documents

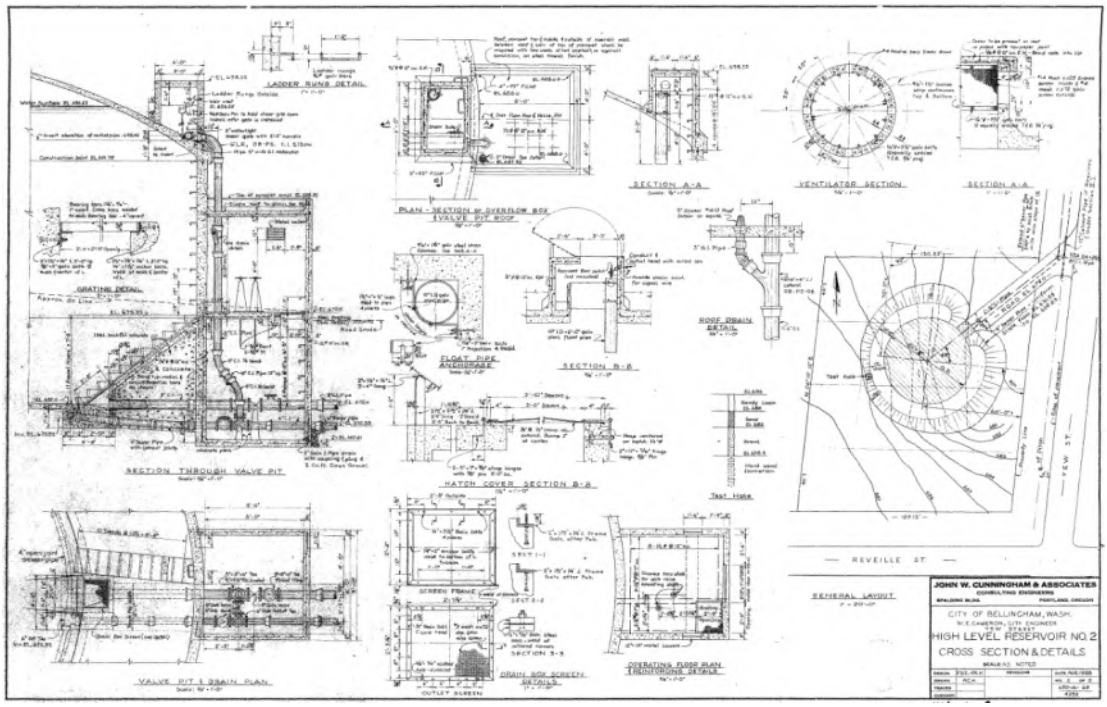


Figure 2-1: Reveille Reservoir Cross Section and Details

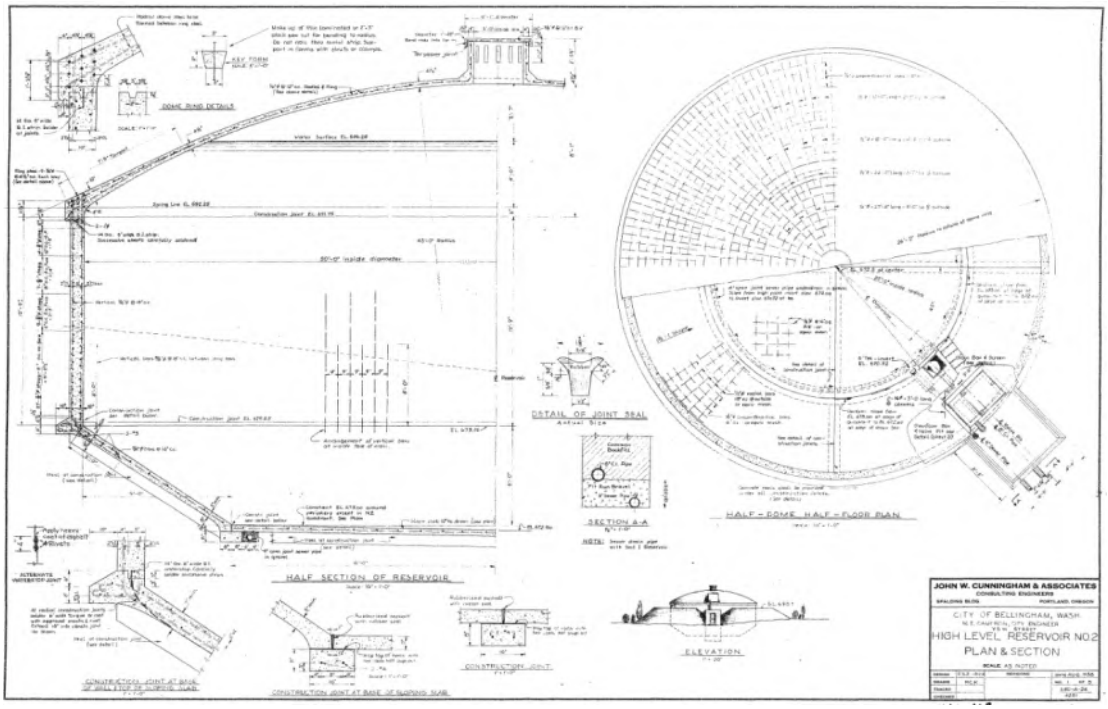


Figure 2-2: Reveille Reservoir Plan and Section

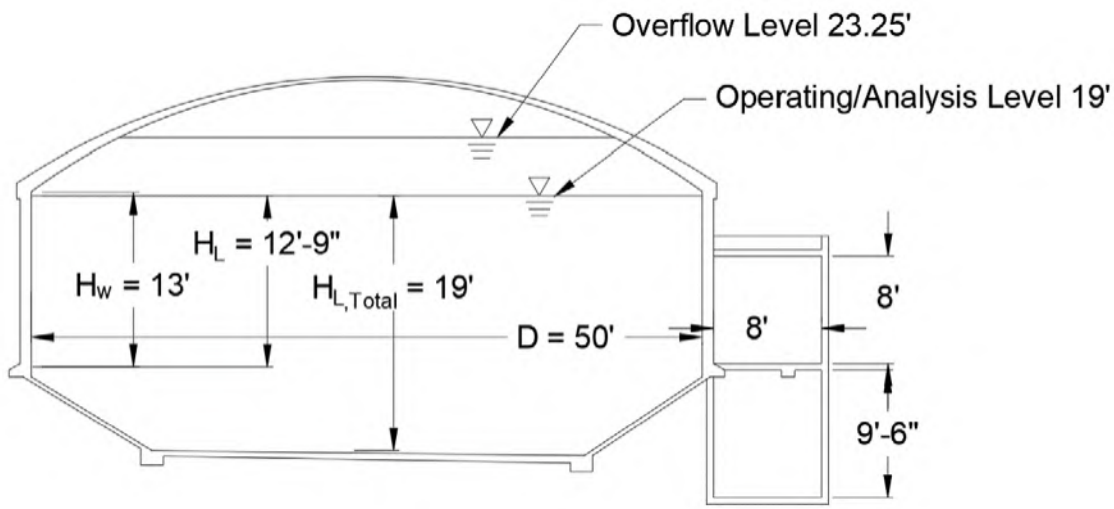


Figure 2-3: Reveille Reservoir Elevations Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, $H_{L,Total}$ = Total Operating Water Height relative to Base)

2.7 Observations Pictures

Figure 2-4: Reveille Reservoir – Elevation

Figure 2-5: Reveille Reservoir – Entry to Valve Vault

Figure 2-6: Reveille Reservoir – Adjacent Tree Growing from Reservoir Berm

Figure 2-7: Reveille Reservoir – Circumferential Cracking Below Roof and Diagonal Cracking near Vault

Figure 2-8: Reveille Reservoir – Close-up of Cracking and Efflorescence under lip of Roof

Figure 2-9: Reveille Reservoir – Valve Vault Floor Slab Penetration

Figure 2-10: Reveille Reservoir –Rust-tinged Leaks Entering Lower Level of Valve Vault

Figure 2-11: Reveille Reservoir – Dome Roof Vent

Figure 2-12: Reveille Reservoir – Circumferential and Radial Roof Cracking

Figure 2-13: Reveille Reservoir – Roof Access Hatch

Figure 2-14: Reveille Reservoir – Steps Cast into Hopper Base and Inlet/Outlet

Figure 2-15: Reveille Reservoir – Reservoir Interior

Figure 2-16: Reveille Reservoir – Circumferential Cracking in Hopper Base

Figure 2-17: Reveille Reservoir – Radial Cracking in Hopper Base

Figure 2-18: Reveille Reservoir – Circumferential Cracking in Reservoir Wall (This location adjacent to Valve Vault)

Figure 2-19: Reveille Reservoir – Close-up of Wall Cracking from Ladder

Figure 2-20: Reveille Reservoir – Dome Roof Interior Side of Vent

Figure 2-21: Reveille Reservoir – Reservoir Floor and Hopper Base

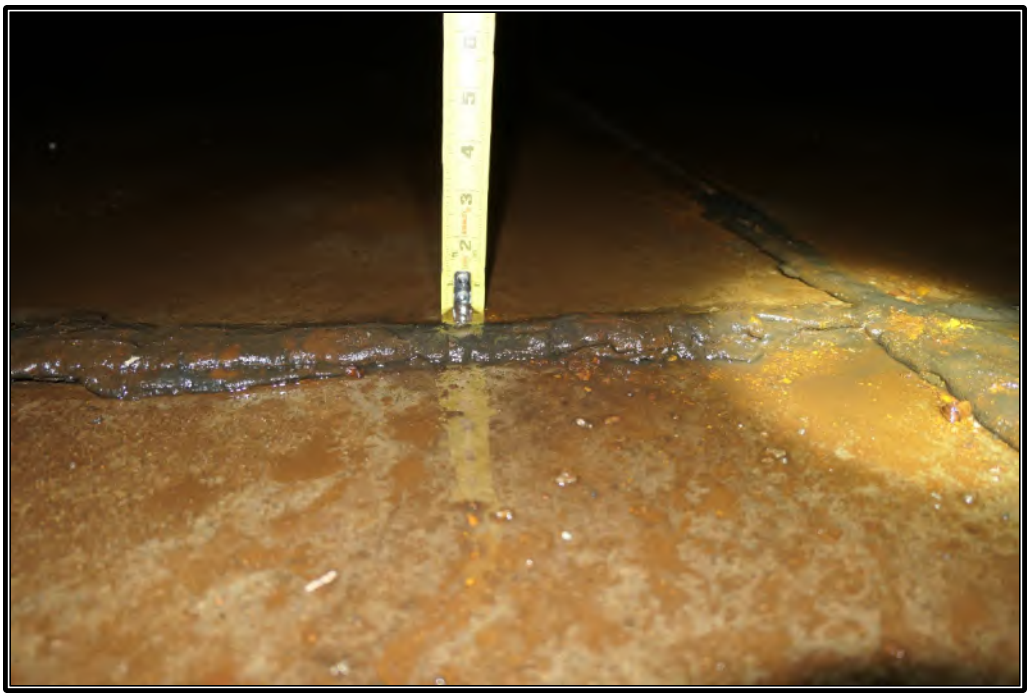


Figure 2-22: Reveille Reservoir – Reservoir Joint Sealant

Figure 2-23: Reveille Reservoir – Ladder Rung Cross-Section Loss

Figure 2-24: Reveille Reservoir – Inlet/Outlet Drain and Corrosion Around Frame and Rough Concrete around pipe.

2.8 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Eval.

PROJECT NUMBER: A1802-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: Reveille Res 2400 Yew St Rd, Bellingham (48.7285, -122.4431)

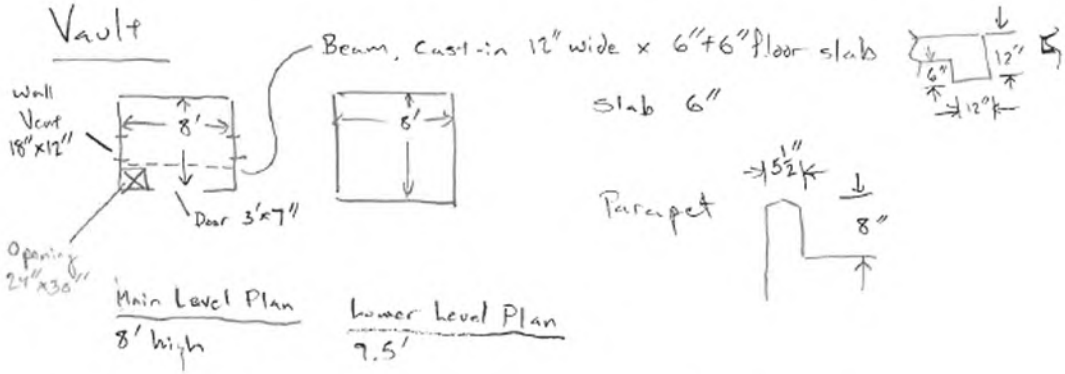
Site Visit Date: 5/21/19 8 am Reservoir Type: Round RC Res. w/ Hopper Base & Dome

Temperature and weather: Overcast, Rained Previous night, 50°F

Site Conditions: Grass field, trees to N & NW. Berms built up around reservoir to about 2' below roof edge

PSE Staff: Grey

Client/Other Staff: Corey Poland Murraysmith, Danny Baba Murraysmith City staff



Main - Good condition, some delam of paint and failure of grout around windows. Leakage & mineralization around roof/tank interface but currently dry

Lower - wet but sump pump appears to be working. 3 location where corrosion occuring along slab/wall edge w/ heavy run-off. New slab pen. rough 16" dia, has left rebar exposed but no corrosion noted.

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to ~~Top~~ Bot. of Roof Slab: Approx. Dir (N) 17" (E) 13' (S) 10" (W) 12"

Roof Slab Thickness: 4.5" / 15" edge (drawings/measured) Roof Overhang Dimension: 4" / 4" (drawings/measured)

Drip Groove? (Y/N): Y / Y (drawings/measured)

Top Surface Roof Slab Condition: Mostly clear of debris, small moss spot. Gen. competent w/ radial cracks at edge about 2' o.c. Circumferential crack around dome about 7' up from edge

Ladder/Vents/Hatch/Joint Conditions: Ex. Ladder Good, Vent Covered w/ screen but exterior good.

Other Comments:

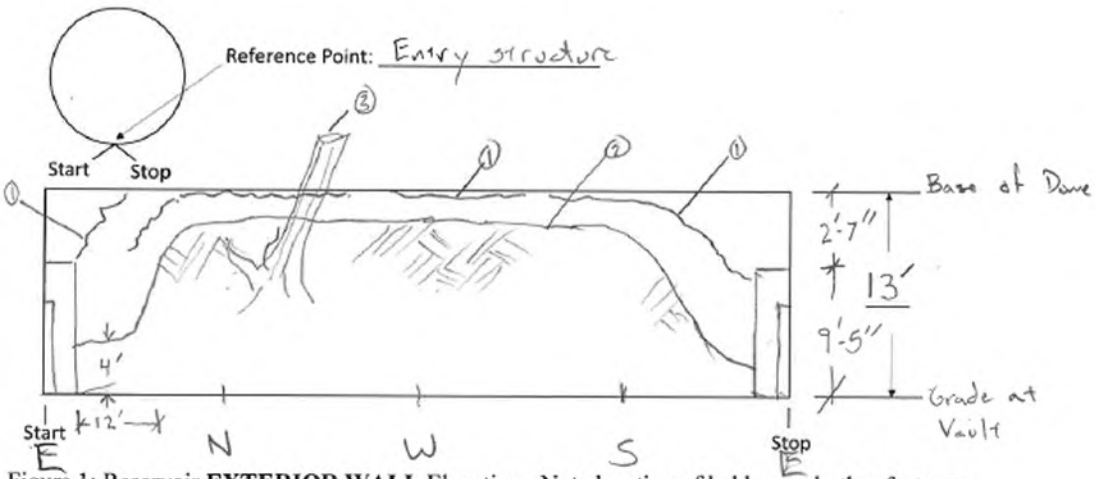


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

① Crack around perimeter ② Berm approx. 12' wide at top

③ Tree to NW 20' from tank, 4' dia.

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

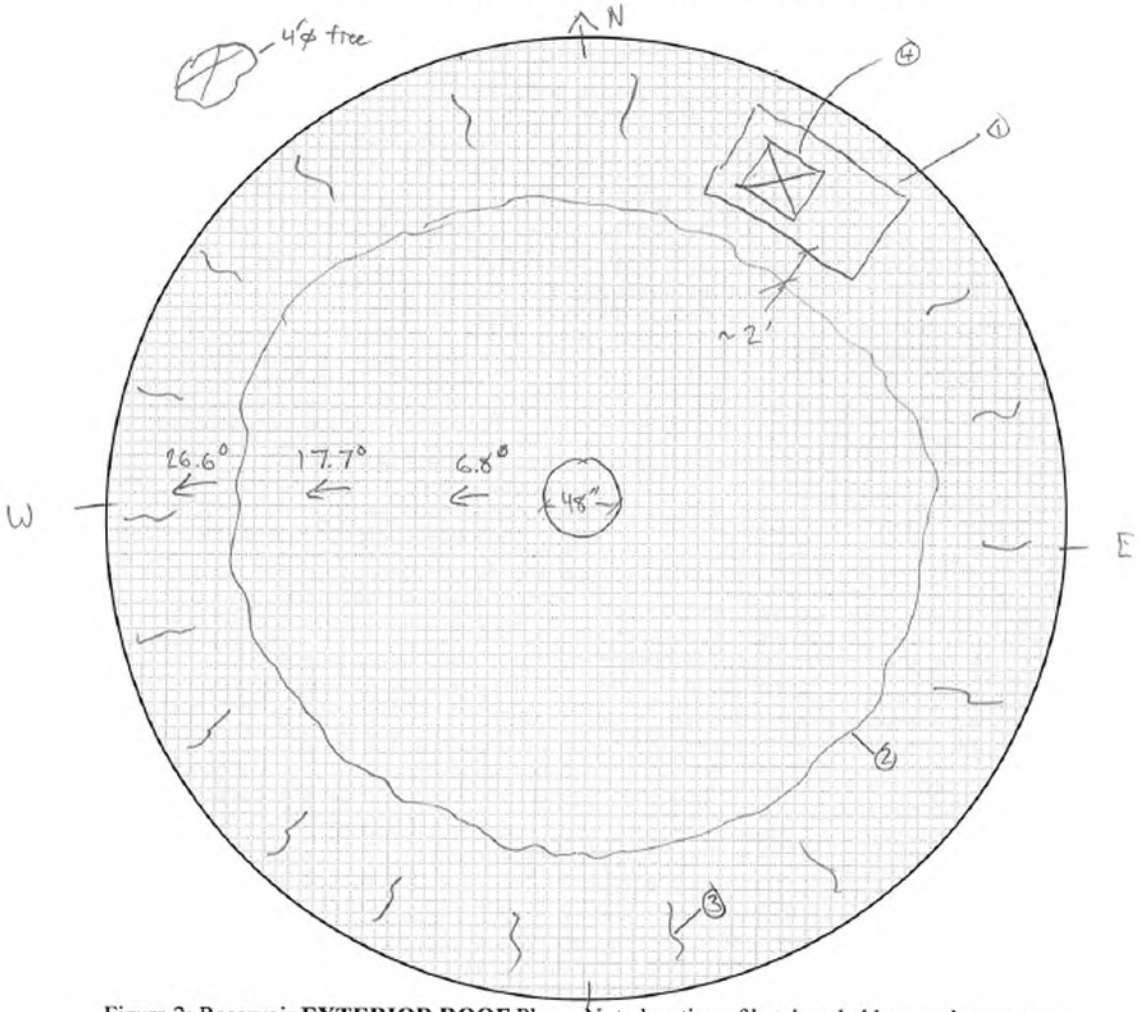


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

① Note Hatch loc. used as approx. north in ref.
② Circumferential crack, about 7' up
③ Radial cracks @ about 4' to 5' on center
④ Entry 4'x6' opening 3'x3'

3

PSE
PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Gen Compacted but w/ radial cracking

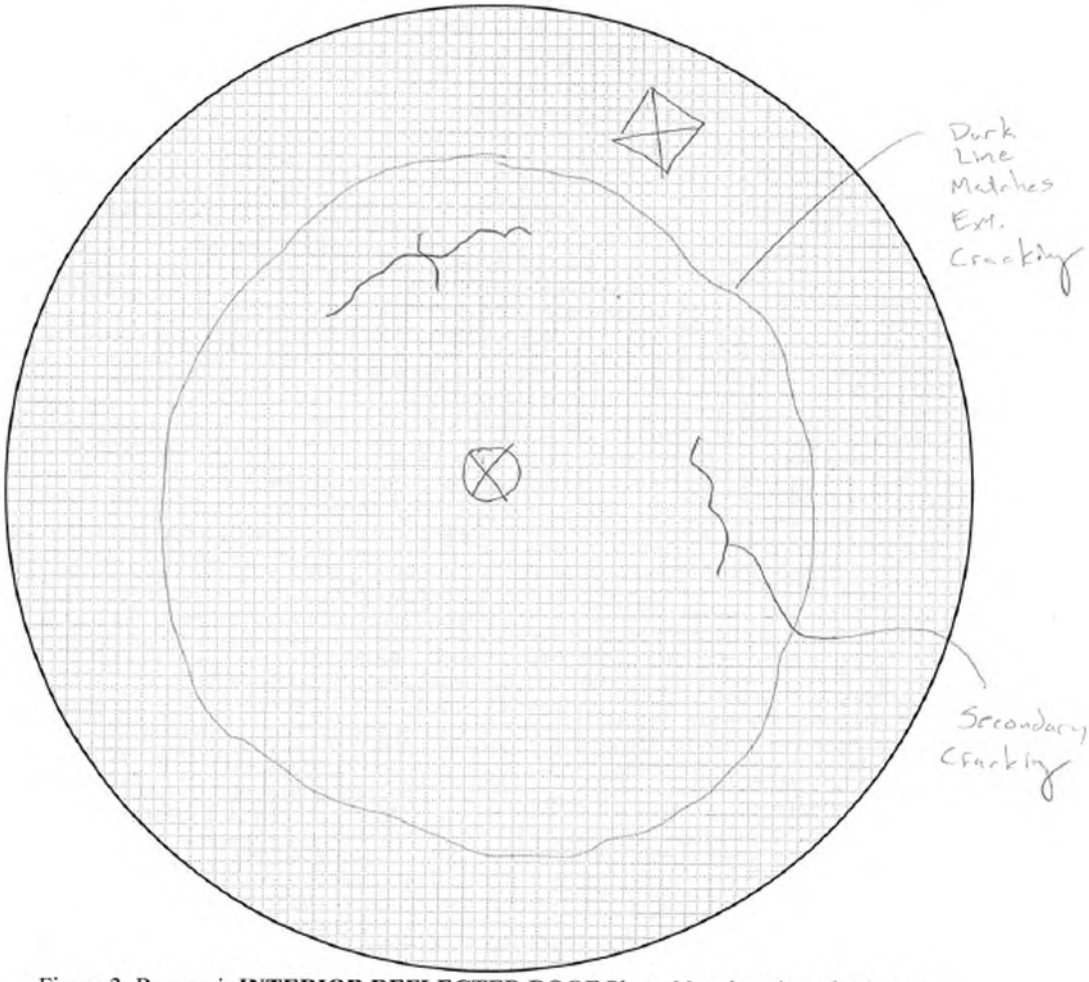


Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

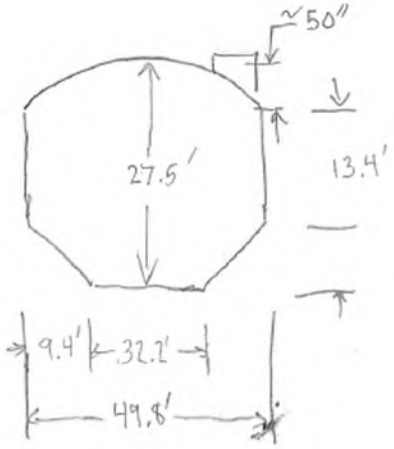
Column Diameter: N/A / None (drawings/measured) Footing Size/Thickness: / None (drawings/measured)

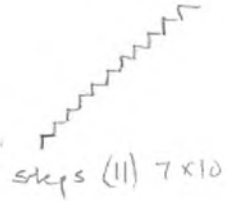
Column Spacing: N/A / None (drawings/measured) Wall Curb Dimensions: / Not observed (drawings/measured)

Floor Slab Condition: No cracking. Poured in 4 quads. Joint material is in poor condition

Floor Slab Joints Spacing/Condition: 4 quads through center

Column/Footing Conditions: N/A





5

PSE
PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

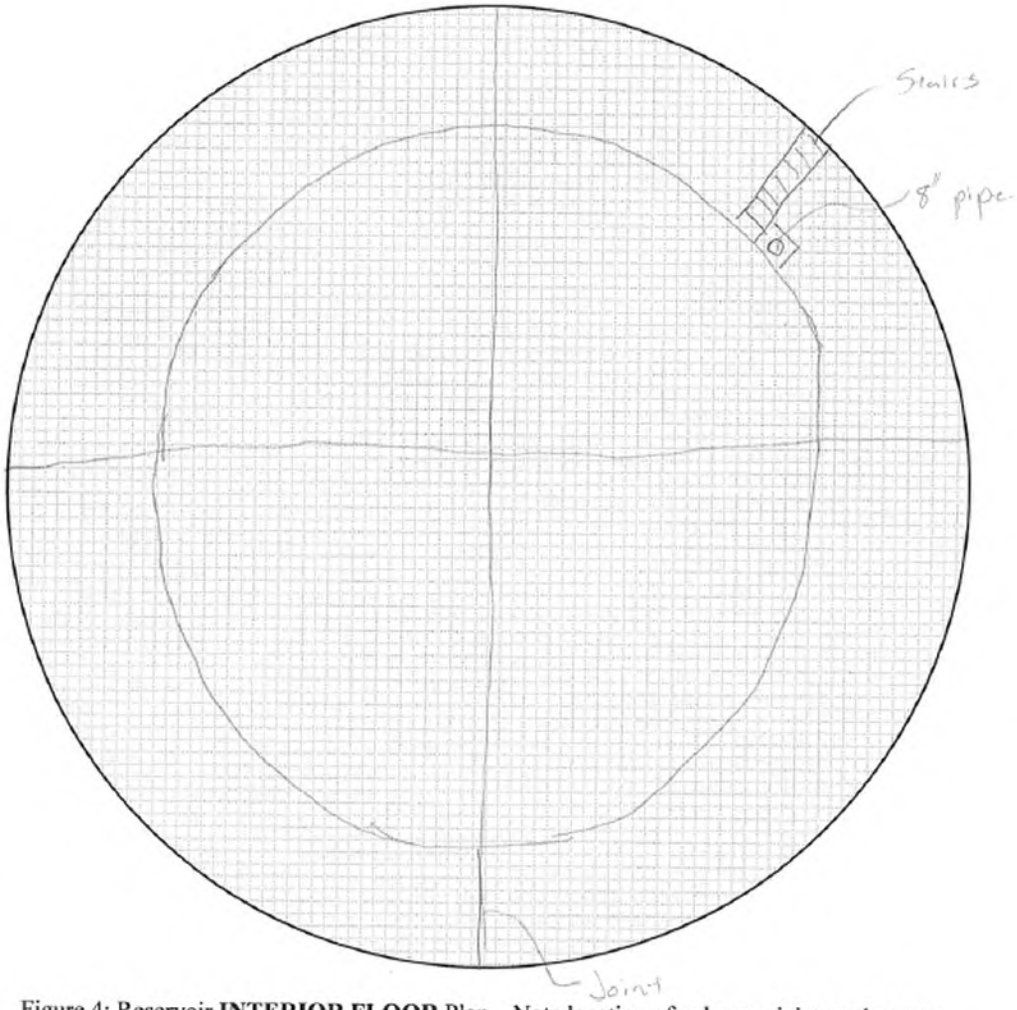


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Gen condition good w/ few bug holes. Multiple large crack notes (this res. is uncoated unlike others) cracking follow exterior issues

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): Few but concrete fail, some debris noted by outer access

# of wall sections: N/A

Ladder/Pipes/Overflow Conditions:

Overflow Height: 23.25' (drawings/measured) Operating Height: 15' - 19' (per City/PUD/other)

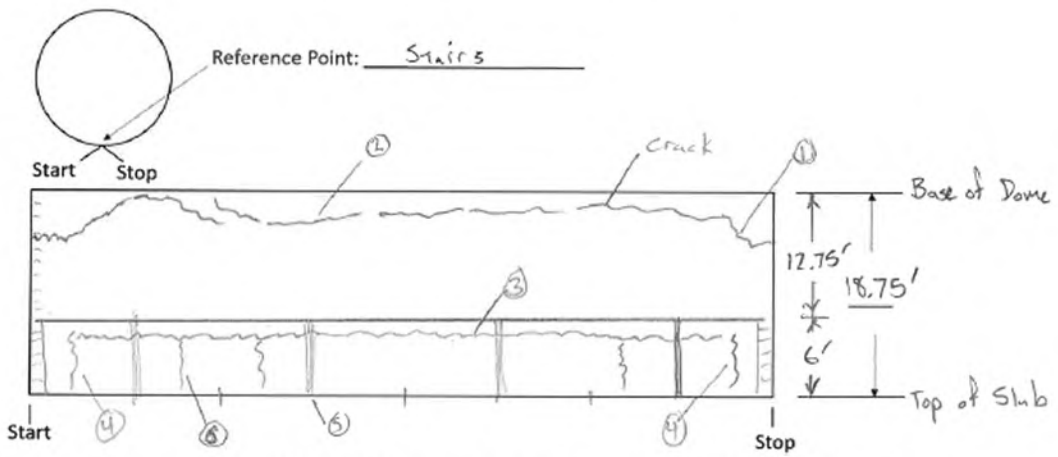


Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

① Wall crack located behind vault
② Circumfrential Crack located about 2' down from top of wall
③ Circumfrential Crack located about 1~1.5' foot down on hopper
④ Radial crack adjacent to stairs
⑤ Joint
⑥ Other Radial cracks in hopper

7

END OF SECTION

Appendix K-3 Reveille General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Reveille Reservoir *General Info*

Field Visit Date: 5/21/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	5/21/2019
Reservoir Name and Location:	Reveille - 2400 Reveille St
Inspected by:	Corey Poland, Danny Baba, Greg Lewis
Client Staff Present:	Shayla Francis, Nick Leininger, Jenny Eakins
Year Constructed:	1958
Overflow Destination:	NE of reservoir into storm drain
Discharge Destination/Zone:	NE of reservoir to Padden-Yew 696 Zone OR 830 Zone via Reveille St. Pump Station
Fill Location:	NE of reservoir
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.3	MG
Diameter (or other dimensions - see notes):	52	ft
Height	25	ft
Overflow Elevation:	696	ft AMSL
Bottom Elevation:	672.5	ft AMSL
Level of Overflow	20	ft
Minimum Normal Operating Level:	15	ft
Maximum Normal Operating Level:	19	ft
Notes:		

Reveille Reservoir *Exterior Inspection*

Field Visit Date: 5/21/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion:	No	
Cage:	No	
Security Type:	None	
Security Condition:	Poor	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	1.5	in
Ladder Width	24	in
Rung Spacing:	12	in
Side Clearance:	5	in
Front Clearance:	7.5	in
Back Clearance:	N/A	in
Notes: Ladder is not secured. Width is 20 in without vertical rail. Distance from paved step to first rung is 18.5 in.		

Exterior Fall Prevention System:	
Present at Site:	No

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof - NE	
Material:	Aluminum	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Perimeter	
Measurement Type	Measurement	Unit
Size:	36	in
Curb Height:	4	in
Notes: The metal lid is 46 in square. Distance from roof to top of frame is 41in. Gasket is not secured		

Roof Vents and Screen:		
Material:	Aluminum	
Condition:	Fair	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	Unknown	in
Notes: 36" roof vent. Exterior vent roof measured 49 in diameter and 26" high. Vent screen is blocked by a welded sheet metal enclosure. Vent not secure due to no fencing.		

Roof:		
Condition:	Fair	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	paint	
Slope of roof	6.8°, 17.7°, 26.6°	
Measurement Type	Measurement	Unit
Overhang Distance:	4	in
Thickness of roof slab	15	in
Notes: Some organic growth. Radial cracks near perimeter. Circumferential crack 7 ft up from perimeter.		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Good	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	N/A	in
Top Height:	39	in
Notes: 19" to mid-rail. Railing extends around entry hatch and grating.		

Grating:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Clips:	No	
Removable Panels:	No	
Measurement Type	Measurement	**Unit**
Approximate Panel Dimensions:	30x60	in
Notes: Maximum width is 48 in		

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Fair
Notes: Cracks with efflorescence	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	Unknown
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A
Notes:	

Reveille Reservoir *Interior Inspection*

Field Visit Date: 5/21/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Mild Steel A36	
Condition:	Fair	
Corrosion:	Yes	
Cage:	No	
Security Type:	Locked hatch	
Security Condition:	Fair	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Fair	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	15	in
Rung Spacing:	15.5	in
Side Clearance:	N/A	in
Front Clearance:	5	in
Back Clearance:	N/A	in
Notes: Corrosion noted on ladder rungs		

Interior Fall Prevention System:	
Present at Site:	No
Type:	N/A
Fall Protection System Condition:	N/A
Notes: Used tripod and winch	

Interior Roof:		
Condition:	Good	
Measurement Type	Measurement	Unit
N/A	N/A	ft
Notes: Circumferential cracking, secondary cracks, and cracks radiating from vent. Efflorescence also present.		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes:	

Walls:	
Condition:	Fair
Painters Rings Present:	No
Notes: Some bug holes present. One major crack throughout, following pattern of exterior cracking. Sloped lower walls have a circumferential crack.	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	No Coating
Interior Piping:	N/A
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Reveille Reservoir *Miscellaneous*

Field Visit Date: 5/21/2019

Piping		
Inlet Piping:	Size (Inches ID):	8
	Condition:	Good
	Material:	Cast Iron
	Notes: Combined inlet/outlet/drain	
Outlet Piping:	Size (inches ID):	8
	Condition:	Good
	Material:	Cast Iron
	Lip (Inches)	0
	Notes: Common w/ inlet	
Overflow Piping:	Size (inches ID):	6
	Condition:	Good
	Air Gap:	Yes
	Screened:	Yes
	Material:	Cast Iron
	Outlet Location:	Storm
	Erosion Evident:	No
	Screen Condition:	Good
	Overflow to roof (feet)	4.5 above
	Notes:	
Drain Piping:	Size (inches OD):	6
	Condition:	Good
	Outlet Location:	storm
	Screened:	No
	Material:	Cast Iron
	Silt Stop Type:	N/A
	Air Gap:	No
	Screen Condition:	N/A
	Notes: 6in tie into storm. Dechlorination in catch basin.	

Piping Facilities		
Exterior Valving:	Type:	Gate Valves
	Condition:	Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	N/A
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	No
Notes: One stub with corrosion noted. Inlet/outlet/drain pipe penetration looks poor. Leaks in lower vault. Valves look new.		

Electrical	
Cathodic Protection:	N/A
Impressed Current:	N/A
Anodes:	N/A
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	0
Chlorine Injection:	No
Altitude Valve:	No
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	Yes
Seismic Upgrades:	No
Security Issues:	Yes
Hydraulic Mixing System Type and Mfg.:	N/A
Sediment Build-Up Height Above Floor (in)	0.1
Water Quality Sample Taps?	Yes
Notes: sample pipe in box adjacent to vault. No fence	

Appendix K-4 Reveille Condition Assessment Score Sheet

Reveille Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance		WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
				Static	Seismic					
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	1	0	No fence or camera, Evidence of use as sledding/snowboard hill
	Vegetation Separation	0	0	0	0	0	0	3	0	Two trees close - A little accumulation in valve vault roof
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	2	3	2	2	0	0	2	0	Roof not designed for thermal movement
	Interior Walls	0	2	3	3	2	0	2	0	Wall by ladder has the larger cracks that look full depth and >1/16" NOT coated
Floor/ Foundation	Foundation	0	0	5	5	0	0	3	0	Inside vault rusty water is infiltrating likely through foundation cracks.
	Interior Floor	4	3	5	5	3	0	3	0	Foundation likely leaking
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	5	4	5	3	5	0	5	0	
	Interior Roof and Supports	0	4	5	3	0	0	0	0	
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	3	4	0	Gap too far on first rung. Fall protection not needed.
	Interior Ladders/Fall Protection	1	1	0	0	0	1	1	0	This vault still has it's ladder and it's in bad shape - not safe for use.
	Access Hatches	5	4	0	0	4	0	3	0	Difficult to get into hatch.
	Railings and Roof Fall Protection	5	5	0	0	0	3	0	0	3-foot distance from roof to ground in unprotected areas.
	Vents	5	4	0	0	4	0	5	0	Screen likely too coarse. Passed design checks.
	Balconies/Landings/Grating	5	5	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	5	4	0	3	4	0	5	0	Comb in/out/drain. Corrosion on frame.
	Outlet Piping	0	0	0	3	0	0	5	0	Comb in/out/drain.
	Drain Piping	0	0	0	3	2	0	5	0	Comb in/out/drain. No air gap on drain, No silt stop
	Overflow Piping	5	4	0	0	5	0	5	0	
	Washdown Piping	0	0	0	0	0	0	5	0	Hose bib in valve vault
	Attached Valve Vault Structure	5	4	4	3	0	0	4	0	Roof drainage needs improvement
	Control Valving	5	5	0	0	5	0	0	5	
	Isolation Valving	5	5	0	0	0	0	5	5	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	4.4	3.9	4.1	3.3	3.8	2.3	3.8	5.0	

Overall Score
3.7

Appendix L Sehome

Appendix L-1 Sehome Geotechnical Report

GeoEngineers

Memorandum

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

To:	Nathan Hardy, PE (Murraysmith, Inc.)
From:	Aaron Hartvigsen, PE Hamilton Puangnak, PE J. Robert Gordon, PE
Date:	January 22, 2020
File:	0356-159-00
Subject:	City of Bellingham Reservoirs Inspection and Repairs Sehome Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Sehome reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memorandum provides a summary of the conditions encountered at Sehome site, located as shown in the Vicinity Map, Figure 1. The Sehome reservoir is an irregularly shaped reinforced concrete reservoir with a hopper base built in 1920.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) map for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by the Chuckanut Formation. Undifferentiated glacial deposits are mapped nearby.

The Chuckanut Formation consists of sandstone, conglomerate, shale and coal deposits. The bedrock typically encountered in the study area consists of sandstone or siltstone. The character of the bedrock at the site is known to vary considerably over short distances.

The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift. Based on previous experience in the area, Bellingham (glaciomarine) Drift overlies the bedrock in this area. The Bellingham Drift is a glaciomarine drift deposit which consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper 5 to 15 feet of this unit in upland areas is typically stiff. The stiff layer possesses relatively high shear strength and low compressibility characteristics. The stiff layer oftentimes grades to medium stiff or even soft, gray, clayey silt or clay with depth. The entire profile can stiff, likely from being partially glacially

overridden, when it is a shallow profile over bedrock. The soft to medium stiff glaciomarine drift possesses relatively low shear strength and moderate to high compressibility characteristics.

Surface Conditions

The project site is located approximately 800 feet to the north of the parking lot on Arboretum Drive, along Huntoon Trail. The reservoir is located in a small valley within the Sehome Hill Arboretum. The site rises upward quickly to the north and south, drops quickly to the east and west. The site is bounded by wooded area in all directions. A small gravel roadway leads to the site from the southeast. The reservoir likely has a mixed bearing profile bearing on both rock and glaciomarine drift.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-9 (2019)—on March 26, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 17½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan, Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites.

- **Fill** – Below approximately 1 foot of forest duff, fill was encountered to approximately 6 feet bgs. The fill consisted of stiff brown sandy silt with gravel.
- **Relict Topsoil/Weathered Horizon** - Relict topsoil was encountered underlying the fill at a depth of 6 feet bgs. This relict topsoil/weathered soil horizon transitions into native glaciomarine drift at 9 feet. The relict topsoil/weathered horizon was comprised of medium stiff brown silt with sand and organic matter.
- **Glaciomarine Drift** – Glaciomarine drift extended from 9 to 15 feet bgs in the boring. The glaciomarine drift was found to be stiff brown sandy silt with gravel.
- **Chuckanut Sandstone** – Chuckanut sandstone was encountered at 15 feet bgs. The boring was completed due to refusal at 17½ feet.

Groundwater

Groundwater seepage was not observed at final depth of the boring. The Chuckanut sandstone and glaciomarine drift units commonly have isolated saturated sandier zones or “pods” at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

Our boring was drilled adjacent to the partially buried reservoir which encountered fill and overburden soils overlying bedrock. Based on review of the as-built drawings, the reservoir is likely founded on a mixed bearing condition consisting of both bedrock and stiff glaciomarine drift.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site, respectively. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our boring and review of the as-built drawings, the existing structure likely bears on bedrock and stiff clay which are not at risk of liquefaction.

American Concrete Institute/American Society of Civil Engineers 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	N_{ave}>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	96.1
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.8
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.398
Seismic design value, S_{DS}	0.651
Seismic design value, S_{D1}	0.358

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_W 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the M 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.24	0.43	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.08	0.14	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm = centimeter, g = gravity

Mw 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends >78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	16	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.36	0.65	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and review of as-builts, the height of the reservoir is approximately 15 feet and we anticipate that the existing reservoir has a mixed bearing condition consisting of both bedrock and stiff glaciomarine drift. We recommend that the structure be evaluated based on an allowable bearing pressure of 3,000 pounds per square foot (psf). The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic foot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The reservoir includes below grade walls. Our recommendations for concrete below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the “Shallow Foundations” section the wall backfill consists of structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope [1]	35 pcf
Active Earth Pressure – 2H:1V Back Slope [1]	55 pcf
At-Rest Earth Pressure – Level Back Slope [1]	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Notes:
[1] Equivalent Fluid Density – triangular pressure distribution

Global Stability

Based on review of publicly available LiDAR for the site, there are slopes inclined at 40 percent or steeper in all directions that rises to approximately 30 feet above reservoir and fall to the east and west. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock and stiff glaciomarine drift clay. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:leh:tln

Attachments-

Figure 1 – Vicinity Map

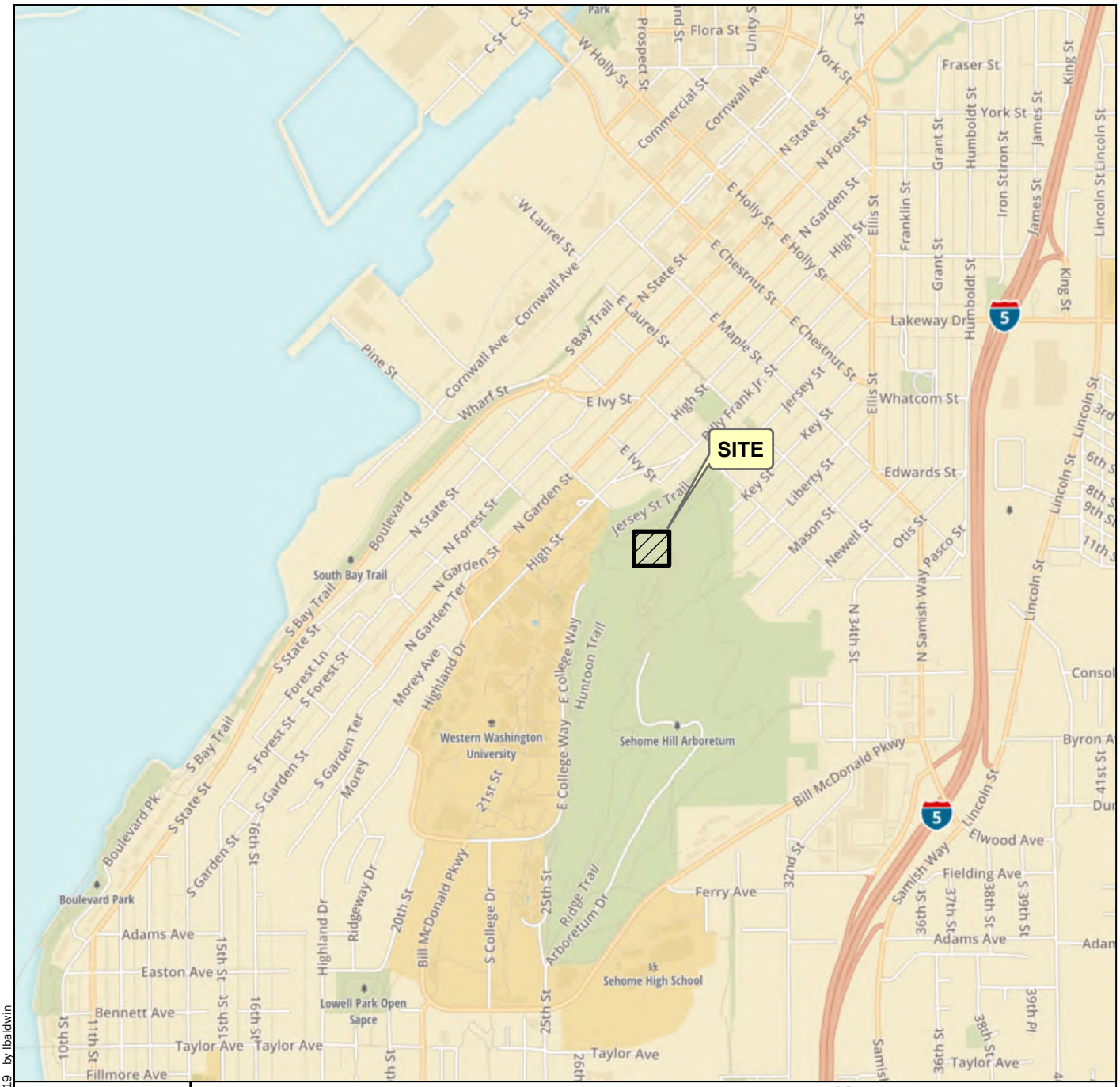
Figure 2 – Site Plan

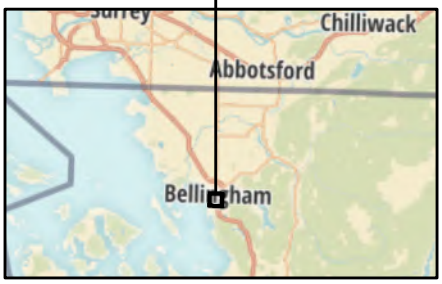
Figure 3 – Key to Exploration Logs

Figure 4 – Log of boring B-9

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault





N
W E
S

1,500 0 1,500

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Sehome Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

P:\0\0356159\GIS\035615900_F01_VM_Sehome.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

Reservoir

150 0 150

Feet

N W E S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Sehome Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GEOENGINEERS

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled	Start 3/26/2019	End 3/26/2019	Total Depth (ft) 17.75	Logged By BWS / Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum	460 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) / Northing (Y)	1242880 638700		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							Duff	12 inches organic forest duff/topsoil			
								ML	Brown sandy silt with gravel (stiff, moist) (fill)			
		X	15	12		1 MC				20		
455	5	X	18	6		2 MC				20		
								ML	Dark brown silt with sand and organic matter (medium stiff, moist) (relict topsoil) (weathered horizon)			
		X	18	6		3 MC				29		
450	10	X	18	9		4 MC		ML	Brown sandy silt with occasional gravel (stiff, moist) (glaciomarine drift)	26		
445	15	X	12	50/6"		5		Sandstone	Brown sandstone (Chuckanut Formation)			
		X	3	50/3"		6			Grades to gray			

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-9

GeoEngineers

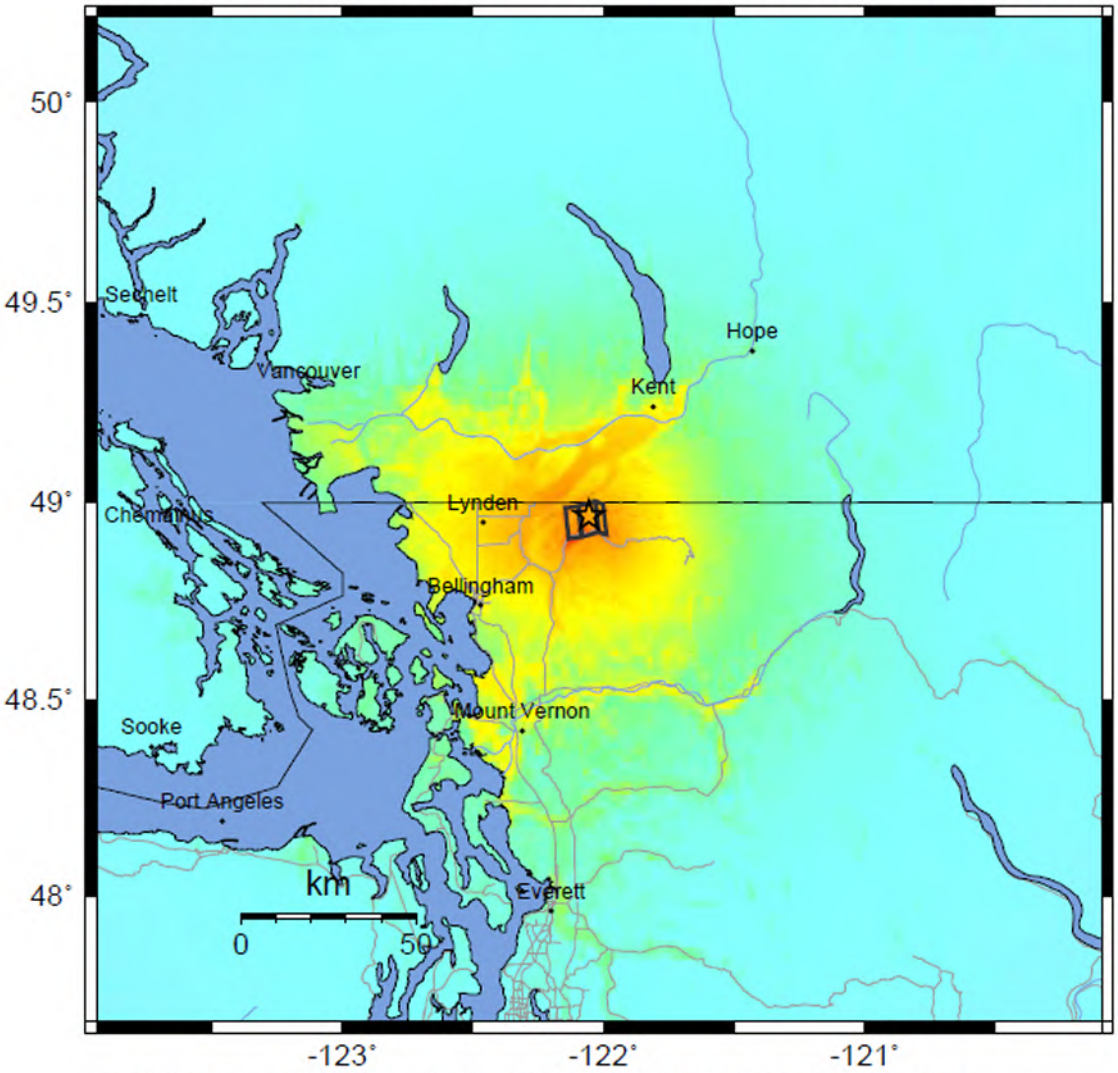
Project: COB Reservoir Inspection and Repair - Sehome Avenue
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

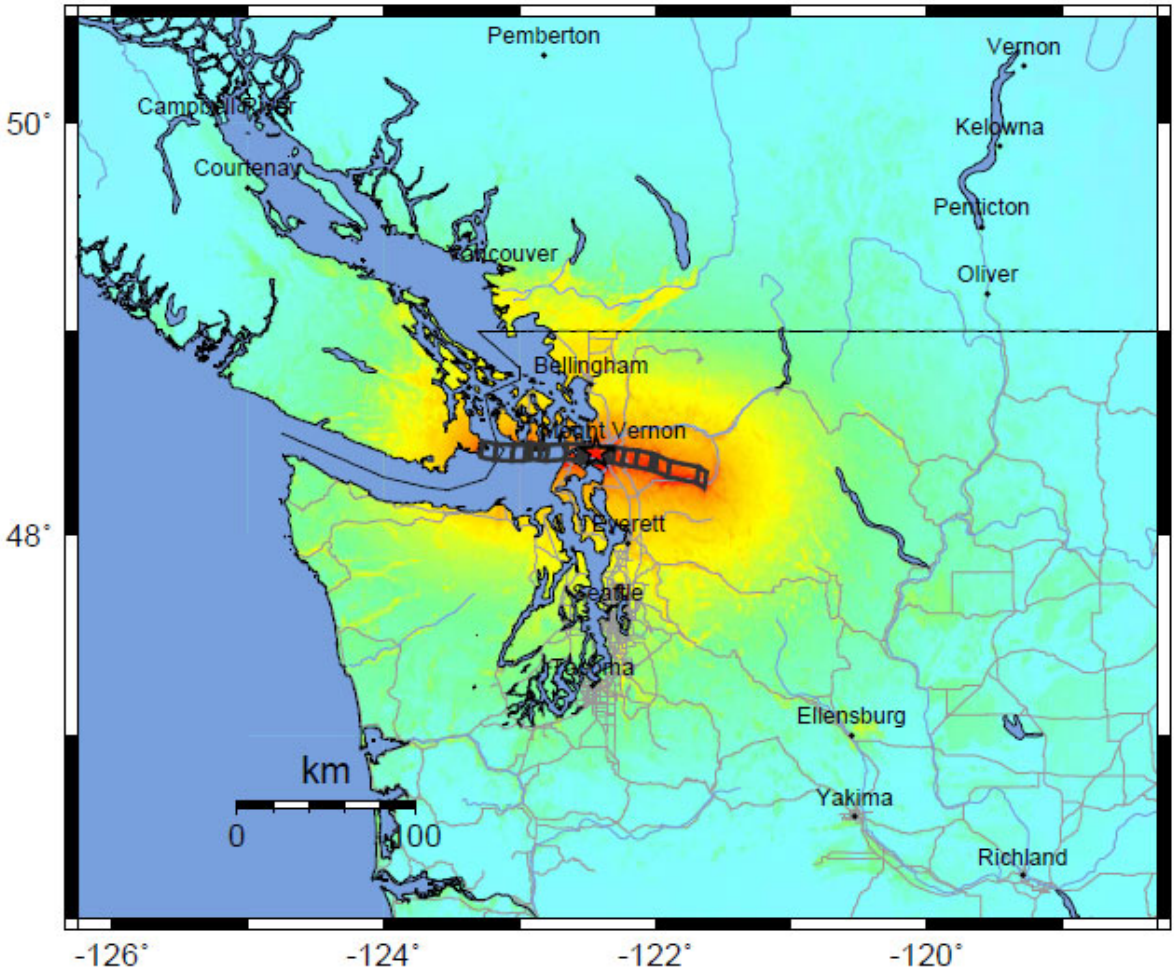
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix L-2 Sehome Structural Report

CITY OF BELLINGHAM

CH 14: SEHOME RESERVOIR

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Sehome, 0.7 Million Gallon (MG) reinforced concrete reservoir. The reservoir is located within the Sehome Hill Arboretum, Bellingham, WA (Lat. 48.739, Long. -122.481), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to inspect and visually evaluate the reservoir on January 24th, 2017 by Peterson Structural Engineers (PSE), and Murraysmith, Inc. The reservoir has been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 Sehome Reinforced Concrete (RC) Reservoir – 0.7 MG

2.1 Description & Background

Based on available information, PSE understands that the original reservoir was built around the 1920's in what is now part of the Sehome Hill Arboretum. The reservoir is supported on the existing grade and located between two hills with the ends walls above grade. The reservoir's concrete base is angled to accommodate the slope of these hills, which results in a hopper bottom running in the long direction of the reservoir. As shown in Figure 2-3 the reservoir has a somewhat football-shaped layout and is approximately 100-feet long (running northeast to southwest) and is 73-feet wide at the center which narrows to 66-feet at each end. The end walls (measured at the center of the reservoir's basin) have a maximum height of 13'-6" on the northeast end and 16'-0" on the southwest end as the floor slopes towards the southwest. No information is available regarding the use of reinforcing in either the slab-base or wall of the original structure.

Sometime in the 1950's the reservoir was expanded and the side walls were modified. This modification added 5'-6" of new wall on top of the existing wall (the original maximum wall height was 10'-6" while it is now 16'-0" at the southwest end). As part of this expansion a column supported slab-and-beam concrete roof was added to the reservoir. Currently, the only available drawings appear to be general site plans along with a single drawing of the original layout and wall thicknesses (Figure 2-1). No drawings were made available covering the retrofit or roof addition. Based on PSE's measurements taken during the site visit the roof is approximately 4-inches thick at the center and appears to increase in thickness to 10-inches along the sides. The roof is supported by (15) 1-foot square columns. The column footings are 42-inches by 42-inches and they are a minimum of 1-foot thick and supported on the original slab floor. The columns are not on an exact grid and follow the geometry of the curved sides of the reservoir. The concrete roof beams that are perpendicular to the long direction of the reservoir (i.e. running northeast to southwest) are 14-inches deep by 12-inches wide while the beams that run parallel are 16-inches deep and 14-inches wide.

2.1.1 Description of Additional Site Structures and Features

The site includes two additional structures adjacent to the reservoir related to its operation. Southwest of the reservoir is a below-grade piping vault which houses the 12-inch main for the reservoir and 10-inch main for Jersey street. The Jersey street main is not part of the reservoir but is routed through the interior of the reservoir, just above the floor.

About 20ft further southwest is the overflow point for the reservoir. The overflow piping is somewhat protected by a rough concrete box enclosure. Overflow water is discharged to the ground surface, downhill of the reservoir.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit on January 24th, 2019 to observe the as-built current condition of the reservoir's interior and exterior. While the sloped sides of the reservoir were partially buried, the end walls were primarily above grade allowing them to be observed. It is our understanding that the reservoir has been

empty and out of service for approximately five years prior to our investigation. The empty condition allowed for a detailed interior inspection.

Concrete Roof: The existing reservoir roof is configured so that the roof slopes towards the center along the length of the reservoir and towards the southwest for drainage. Any water collected on the roof is routed through a single 6-inch diameter drainpipe which is run through the southwest access hatch box assembly. The roof was measured at the hatch to have a thickness of 4-inches. The underside was observed to be flat and so the roof appears to have a variable thickness since it slopes towards the center of the reservoir. Measured from the top of the parapet running along the end walls, which are approximately level, the height above the roof surface varies between 13-inches at the edge and 19-inches at the center. Therefore, PSE assumes it is likely that the roof slab thickness varies from 10-inches at the edge to 4-inches at the center.

The reservoir roof was observed to be covered with leaves and other organic material to about a ¼-inch layer. Based on pictures provided by the City, as recently as 2017, this layer was about 12-inches thick at the center. In 2017 the roof was cleaned (Figure 2-18), and the bulk of the soil and debris was removed. The extent of the previous soil and debris layer can be seen as a discolored ring around the vertical surface of the parapet where the smooth surface concrete has degraded to expose some of the underlying aggregate. Above the ring, the roof parapet is coated in a thick moss layer.

By clearing away some of the debris, PSE could better observe the roof surface. Possible remnants of a surface coating were observed, but the coating is so compromised that it was unclear if it was a damaged coating or deposits left over from the previous debris build-up. At the center of the reservoir roof slab where PSE would expect higher loading stresses to occur, the roof debris was cleared to evaluate for cracks. Based on this cursory observation, no cracks were noted in the areas observed. As will be discussed later, it would not be unexpected to observe cracks in the roof slab, considering the roof thickness measured and the significant loads associated with a saturated layer of debris and soil. It is possible that additional surface cracking is present, but those cracks were not identified during our site visit due to the extent of the debris currently covering the surface. The remaining roof components appeared to be visually structurally sound with minimal chipping or weathering damage.

There are roof hatches located at each end wall along the centerline of the roof. The northeast hatch has a 30-inch by 30-inch opening with an aluminum cover. The southwest roof hatch has a 30-inch by 24-inch opening, also with an aluminum cover. The southwest hatch is part of a larger access area which includes the roof drain, access to the overflow, and a vent pipe. The hatch also accommodates the opening for the side access hatch. The hatches appear to be in fair condition with some degradation of the concrete surface layer in the zone where the previous debris/soil layer was located. Neither hatch includes any hinges but instead are fitted over the concrete body of the hatch and secured in place with eyebolts and padlocks. The roof vent located adjacent to the southwest roof hatch is a 3-inch diameter pipe and is one of five total pipe vents. The remaining four vents are located at the corners of the reservoir and appeared to be in generally good condition.

The interior surface of the roof slab was noted to have a variety of minor cracks where efflorescence was noted to have occurred. Efflorescence is a whiteish crystalline deposit of salts or minerals resulting from water movement through cracks in the concrete. A few of these efflorescence areas were noted to be red tinged which is an indicator that the water flowing through cracks in the slab are corroding embedded rebar and this can weaken the slab. The roof slab's cast-in-place beams were, in general, visually crack-free with the exception of the beams running northeast-to-southwest near the center of the reservoir. These beams had some notable crack-associated efflorescing observed at their mid-section. While cracking was noted throughout, cracking did not visually appear to be due to structural failure or other major issues. Additionally, pattern cracking associated with creep was not visually observed.

Roof Columns: The roof slab is supported by columns which are located along the beam lines running northwest to southeast. The location of the column bases follows along the curved sides of the reservoir floor. While the center row of columns is aligned with the beam line running northeast to southwest, the columns to either side are placed relative to the edge of the sloping floor and are not aligned with the adjacent beams running northeast to southwest. The columns have square footings founded on top of the original slab floor. In general, the columns and footings appear to be in good condition with no visually observed issues.

Reservoir Walls: The exterior of the northeast and southwest end walls (which are covered in painted murals) were observed to be in good condition and without notable instances of cracking and minimal occurrences of bug holes or voids in the concrete. The southwest end wall includes a 4-foot by 3-foot tall side access hatch which is adjacent to and above the overflow weir. The side hatch extends up into the roof parapet zone above the overflow weir. The northwest and southeast side walls run the length of the reservoir. The northwest side wall is partially buried with the exposed portion completely covered in moss. This moss coverage made observation of the underlying wall difficult. The southeast side wall is below grade and an access road is located along this side of the reservoir. As the southeast side wall is below grade, a detailed exterior visual observation was not able to be performed.

Per the available plan drawing of the reservoir, the tops of the original northeast and southwest end walls are 2'-10" thick while the side walls are 1'-8" thick. Based on our visual observation, the original and new wall sections appear to have a smooth transition where they are joined. From the joint location, where the walls are attached, the wall thickness appears to linearly decrease to the 5-inch thickness observed at the top of the wall parapet.

The wall appears to consist of three main sections. The first section comprises the lower portions of the northeast and southwest end walls which each having two buttresses and were built to the top of the sloped floor on either side. A control joint, filled with 1-inch thick black-tar mastic, runs the length of the end walls, and delineates the top of the first section. The second section is the original 3-foot tall wall surrounding the basin of the original open-air reservoir. The final section is the 1950's wall located on top of the original wall which added an additional 5'-6" of wall height along with the roof. Overall the walls appeared to be visually in good condition largely free of bug holes or cracks with two exceptions as noted subsequently.

The first exception noted was found along the northwest side wall. At the center of the wall, the wall is showing signs of heavy efflorescence with a red tinge. Based on the historic section drawings, this area is located approximately at the point where a rock outcropping occurs. In the drawing titled *Sehome Reservoir X-Section*, dated December 1950, and shown in Figure 2-2, it appears that the original reservoir had an undisturbed sandstone wall on the northern side that the reservoir was built around. The reconstruction associated with the roof addition appears to have covered this sandstone feature with concrete. The rock formation likely results in a point of stiffness relative to the remaining foundation and differential settlement around the rock could be the likely proximate cause of cracking in this area and the heavy efflorescence noted along a zone of high stress. The second exception was noted along the center of the northeast wall below the newer wall addition. In this area a few sections of concrete surface were observed to be failing and exposing the underlying wall aggregate. This appeared to be a localized occurrence and while it should be repaired, it did not appear to be part of a larger pattern of failure.

Reservoir Floor: The slab floor appears to be in good visual condition with no issues observed. Control joints were found between the flat and sloping basin sides as well as the basin sides and the walls running the length of the floor. No visible defects in the joints were observed. It is unknown if the joints are leaking and/or if any leak tests have been performed on the reservoir. Also noted were 2x4 solid sawn wood members attached to the floor. Their purpose is unclear, but they appeared to be growing fungus and should be removed if the reservoir is put back into service.

Appurtenances: The inlet, outlet, and overflow weir are all located at the southwest side of the reservoir. The floor of the reservoir slopes from the northeast to the southwest at an approximate 3% slope to these pipes. These items seem to be in generally good condition although some small corrosion carbuncles were noted. The concrete around the 12-inch inlet and 8-inch drainpipes was observed to be generally uncracked and competent and no visible issues were noted around the overflow weir. An additional 10-inch diameter pipe, unassociated with this reservoir's operation, is run through the center of the reservoir. This pipe was recently cleaned and the concrete block, located at its entry point into the reservoir, has been reconstructed. The block appeared to be in good condition although corrosion on the pipe was noted to be pronounced.

2.2.1 Visual Condition of Additional Site Structures and Features

The vault structure located southwest of the reservoir was visually found to be in generally good condition with some instances of water seepage notes at cracks and bug holes in the concrete surface. Generally, there were no observed signs of structural failure or settlement issues that might be an immediate concern.

The concrete box enclosure over the overflow appears to have experienced some section loss due to scour when the reservoir is being drained. While this box does have noticeable damage and does not appear to have a foundation, the box is not necessarily required around the overflow. From a functional perspective, it appears to provide the necessary support and coverage needed for the overflow pipe and appears to remain adequate for its intended role.

2.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the reservoir under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Rectangular Concrete Tanks", Revised 5th Edition, published 1998 were utilized. Evaluation was based on original construction documents and site visit observations.

2.3.1 Hydrostatic and Gravity Analysis

Roof Slab: The roof slab was measured from the hatches to have a thickness of approximately 4-inches at the center. Based on the observed layout, PSE believes this is the thinnest portion of the roof as the roof slopes from the side-wall towards the center. Due to this the roof edges are assumed to be about 10-inches thick based upon field measurements of the slope. At the locations where the slab is the thinnest, it is supported by a beam running down the reservoir's center. Beams parallel to the center beam area are spaced at 10-feet on center. For this spacing the Code recommends a minimum thickness of 4.2-inches for deflection. This is met for the majority of the slab. Although it is unclear if reinforcing is present, PSE back checked the minimum required reinforcing necessary and determined that reinforcing density equivalent to #4 bars at 12-inches on center would be sufficient for a majority of the roof. Areas adjacent to columns and beams were found to require an increased amount of reinforcing with a spacing of 6 inches on center for the positive moment between columns and 3-inches on center above columns. This analysis assumed the roof self-weight and a 25-psf ground snow load were the primary loads.

While the exact amount of reinforcing within the reservoir roof is unknown, it has supported a 12-inch layer of debris/soil load (which was likely saturated) for a significant amount of time prior to our site visit. Figure 2-18 shows the work crew clearing this soil load during the summer of 2017. A soil load of this magnitude would likely weigh 80 to 100-psf, higher than PSE accounted for. Because roof failure has not occurred, the present reinforcing is likely at a spacing either tighter than max spacing PSE determined or the actual material strengths are higher than PSE has assumed. In either case the configuration empirically appears likely to be adequate for the roof dead and ground snow load expected.

Two additional loads were considered for the reservoir. The first is a rain load which is required for roofs with parapets. The Code requires designing for a roof water load assuming a blocked drain which then allows the parapet-enclosed-zone to fill with water. For the max parapet height of 19-inches, this results in a rain load which is 98.8-psf, roughly equivalent to the weight PSE estimated for the soil loads. The second load is a vehicle load. Due to this reservoir's location and ease of roof access by a vehicle, this loading is a possibility. While the reservoir has supported smaller equipment loads in the past, the lack of information on actual roof reinforcing means it cannot be determined if the roof is suitable for all vehicle and equipment load conditions. Due to this, it is recommended that vehicles should be kept off the roof.

As with the roof slab, the roof support beams were also supporting the roof soil load for prolonged time without failure. While the beams' actual reinforcing is unclear, they appear empirically adequate for the potential site loads. Backchecks by PSE determined the beams require a minimum reinforcing area of around 2-square-inches, or the equivalent of (10) #4 bars, to resist the design loads. This reinforcing amount seems reasonable for this era of construction.

It should be noted that current code requirements have increased safety factors than those mandated during the period of the original construction for both short term strength and long-term serviceability/creep considerations. However, considering the soil loads the roof has been subjected to, the roof appears to be in good condition and appears to have performed well under static loads. Therefore, as long as the roof surface is maintained, the roof drainage kept clear, and vehicles and equipment kept off of the roof, the reservoir roof will likely continue to be adequate for gravity loads as the code mandated design loads appear to be less than those already experienced. Please note, if the drains become clogged and/or the roof is allowed to be covered in debris it is still possible that a saturated or flooded condition could exceed previously seen loadings and be a potential cause of failure.

While no gravity load upgrade recommendations are proposed for the roof, some lateral/seismic load upgrades (discussed later) could impact the roof. It is recommended that prior to any major reservoir retrofits that the roof reinforcing be mapped, and samples taken to verify the reinforcing size, spacing, and tensile strength. This would allow for a more thorough structural analysis of the as-built condition of the roof slab. Additionally, as the location of cracking noted on the underside of the roof was observed to have areas of red-tinged efflorescing, a new surface coating should be applied to further protect the roof and prevent water infiltration. Any cracks already noted should be evaluated to determine their depth and then injection sealed and coated on both the top and bottom surface to prevent further potential damage.

<u>Wall Adequacy</u>: Per the available drawings and period of construction, it does not appear as if reinforcing was likely used in the original walls of the reservoir. Rather, those walls were likely poured to be sufficiently thick, ranging from 4'-6" thick at the base to 2'-10" at the top, so as to have the necessary capacity to resist the original assumed static loads without reinforcing. A plain concrete wall section was determined to be adequate to resist the hydrostatic load resulting from the original 10.5-foot storage height as well as the current 13-foot storage height. For these hydrostatic loads PSE evaluated the walls based on the allowable cracking stress versus the applied stress. PSE estimates that the wall has an approximate Factor of Safety of 18 for the anticipated static loads.

While no drawings are available for the newer upper section, built around the 1950's (the same time as the roof), this newer wall section is likely reinforced. As this section is mostly above the waterline, the hydrostatic loads it experiences are much lower than for the original wall. While hydrostatic loads are lower in this zone, it was noted that this area is where a crack was observed along the center of the northwest side wall. As covered previously, we believe that this cracking is likely a result of settlement rather than operating loads. The crack should be evaluated to determine its depth and injection sealed and coated to stop water infiltration.

The hydrostatic loads also result in shear and tensile reaction forces at the base of the structure and the wall ends. Based on the original wall drawings the original northeast and southwest walls are extended past the side walls. It is assumed that these end-wall extensions were intended to buttress the wall ends in lieu of tension reinforcing. In a modern reinforced concrete reservoir, reinforcing would be provided to handle the tensile load between wall corners. In the upper new wall section, which are likely reinforced, the walls do not extend out to the sides.

For the original wall section, lacking reinforcing, the wall extension appears to rely on soil bearing to resist loads that would otherwise cause the walls to separate at the corners due to outward forces. Further, each end wall has two counterforts to further reinforce and brace these walls. Based on performance, the configuration appears to have been sufficient for hydrostatic loads experienced to date. See Lateral Analysis for additional notes on tension induced forces at the wall corners.

Columns: PSE performed column checks based on the lower bound assumed column reinforcing density equivalent to (4) #4 bars. For this level of reinforcing the columns are likely adequate for the worst-case assumed code level static roof loads. Further, based on their observed condition and the previous roof debris/soil loads, the columns all appear to be empirically adequate for gravity loads. See Lateral Analysis for additional notes on column adequacy.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the drawing the reservoir does not appear to have a footing, but rather the wall thickness increases towards the base resulting in a bearing surface which is between 43 to 54-inches wide. Per the geotechnical evaluation, the site's bearing capacity was determined to be 3,000-psf. Using this bearing capacity and checking for the 13-foot operating level up to overflow, the wall bearing pressure was determined to be within acceptable ranges for static loads.

Interior column footings were measured and found to have a 42-inch by 42-inch base. Although the footing reinforcing is unknown the general size of the footing is insufficient for the site's allowable bearing capacity for static loads. As previously noted, the reservoir roof has had been supporting large soil loads and major cracking was not observed in foundation nor was settlement observed. As a result, there are likely additional factors, such a thicker than assumed floor slab, that increase the bearing capacity of the footings as indicated by the observed acceptable performance of the column footings.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: As previously noted, it appears the original reservoir was constructed either without or with minimal reinforcing. The 1950's addition to the reservoir likely did utilize reinforcing; however, no documentation is available to indicate how much reinforcing is present. Current concrete design requires reinforcing to be used in order to prevent a brittle or cataphoric failure and to accommodate tensile and cyclical loading as well as to provide ductile behavior in a seismic event.

At the initiation of a seismic event, PSE's analysis determined the walls have a Factor of Safety of approximately 12 for the anticipated dynamic slosh-wave load. For the side and end walls, which resist in-plain seismic shear, the Factor of Safety was determined to be 5.6 near the base and decreasing to near 1.8 towards the top, as the wall section narrows. While the wall's size and layout appear to be sufficient

to handle the initial hydrodynamic loads, they lack of reinforcing, and therefore ductility. This makes them susceptible to load cycling. For any cracks that are already present or that begin to develop during an earthquake, these faults can lead to progressive cracking (known as zippering). This progressive cracking will substantially decrease the wall's strength over the duration of the seismic event and could result in failure. Additionally, any site faulting or permanent deformation in a seismic event could cause failures since the unreinforced concrete doesn't have any capacity to resist any induced tension loads.

Tension loads at the end wall are relatively low due to the reservoir layout but are still present. Unlike the loads listed above, which are shear and flexure loads, a direct tension load cannot be resisted by plain, unreinforced concrete. The end walls appear to be extended past the side wall and utilize passive soil pressure to brace against tensile loads resulting from hydrostatic loads. In order to resist hydrodynamic loads, the walls require a positive connection at the wall corners to resist any separating forces. As a result, these wall appear to be inadequate for seismic loads and should be retrofitted with connections capable of resisted the anticipated tensile forces. Connections can be mechanical in nature, such as anchors, or they can use FRP wrapping in order to constrain the sides. As noted previously, testing should be conducted first to verify the presence or absence of any reinforcing in order to determine the extent of any retrofit needed.

Columns: The roof is rigidly supported along its sides, which does not allow for thermal expansion or seismic movement. While PSE typically recommends a roof-to-wall connection which allows for flexibility and thermal expansion, the site visit did not identify any thermal cracking issues and the existing rigid wall connection doesn't appear to be causing any operational issues.

By having a rigid wall connection, the columns are not required to translate during a seismic event as the roof moves. With no movement the columns do not need to be reinforced to account for displacement-induced flexure loads. To ensure flexural load transfer does not occur the wall and roof-to-wall connection should have the necessary capacity to handle the anticipated lateral loads. Where the new (1950's) wall was built over the existing (1920's) walls, this interface should be investigated to determine if the reinforcing in this zone is adequate to transfer flexural loads across this interface. If there is insufficient reinforcing this zone can fail and behave as a hinge under lateral loading. Additionally, the roof-to-wall connection should be evaluated to ensure it is adequate to resist applied out-of-plane shear loads. Along the reservoir's northwest and southeast side walls, a thickened corbel runs the length of the wall and increases the connection capacity at the roof-to-wall connection. However, the northeast and southwest end walls do not have a corbel and instead rely on wall reinforcing for the necessary strength. PSE recommends additional investigation into the adequacy of the connections between the new and old portions of the reservoir in order to determine the presence, size, and spacing of any reinforcing. This investigation will help to determine the wall's capacity and allow for a better assessment to determine if the columns and/or wall connections would need to be upgraded.

Freeboard/Slosh: The current freeboard provided for the 13-foot operating level is 3-feet. Per PSE's analysis, the expected slosh is 2.6-feet and there is no code calculated risk of a slosh wave impact on the roof. Should the reservoir be operated at a higher level than typical, a slosh wave impact could occur. As noted previously, information should be gathered on the actual roof geometry and reinforcing layout.

Depending to the roof thickness and reinforcing there could be sufficient strength to resist potential slosh loads without upgrades.

Roof to wall connection: Based on the estimated wall thickness, there is likely insufficient thickness at the roof to wall interface at the end walls to be able to fully resist shear resulting from roof loads occurring during a seismic event. As noted previously, information should be gathered on the actual wall geometry and reinforcing layout. Based on information gathered on the wall, it is likely that additional shear brackets or other upgrades will be needed along the end walls in order to ensure adequate fixity between the roof and wall. This is required to limit column movement and prevent a potential column failure and roof collapse. Based on the gathered information, new support braces could be installed between the existing roof and new and existing wall elements to better transmit the anticipated seismic loads.

2.4 Summary

Based on current design codes and available information on the reservoir's design and past performance, the Sehome reservoir appears to be adequate for the anticipated hydrostatic loads at its current or overflow operating level. For the determined seismic loads, while the reservoir had the size and strength to the resist the initial seismic shock loads, it appears to lack the necessary reinforcing to resist the cyclic loading expected due to seismic shaking. While the reservoir does not appear to be at risk of a catastrophic failure, it is likely susceptible to failures at connections and boundary elements which could compromise the reservoir storage capability after a seismic event.

The roof slab, based on previous performance, is likely sufficiently reinforced for required static loads based on PSE's estimates. The many years the roof has spent supporting a thick debris/soil layer has not resulted in any significant observed issues. While areas of cracking were noted, the cracking observed did not seem excessive given the circumstances. Overall the roof and support structure appeared to remain in good condition.

The remaining elements of the reservoir including columns, their footings, and the slab floors all appeared visually to be in relatively good condition. Generally, disregarding the potential seismic loads, the reservoir appeared to be in good condition with primary deficiencies related to some instances of settlement and cracking.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

Based on PSE's evaluation it appears that a significant amount of retrofit work would be required to bring the reservoir up to current code seismic performance requirements. As the reservoir is currently offline, PSE would recommend that City consider only using the reservoir for non-critical storage, as the reservoir has sufficient capacity for static loads. In this case the City would not rely on the reservoir for water storage critical to emergency response. If the City does require an additional 0.7 MG in critical water storage, PSE would recommend examining the cost of a new reservoir versus the costs of upgrading the Sehome reservoir. This recommendation is based on the likely level of required retrofit that would be necessary to bring the reservoir up to Code and the difficulty of accessing the site within the Sehome Arboretum.

Should the City desire to keep this reservoir in service at the 13-foot operating level and nominally compliant with the current structural code, upgrades would be required to the roof/wall interface connection as well as to the reservoir's interior. While existing reservoir elements might have some reinforcing, the amount of reinforcing is unknown. The first step for retrofit would to be to map the existing reinforcing to determine its extents and spacing. Additionally, samples of the concrete and reinforcing should also be taken to determine the strength of each. Key areas to evaluate could be the slab floor, wall base, new-to-original wall interface, roof-to-wall interface and edge and middle roof slab locations. Based on the information gathered, an updated analysis should be performed, and the design of potential retrofits developed based on the City's operational requirements.

For the roof, in order to limit structural upgrades, PSE would recommend re-installing the gate around the reservoir to prevent vehicle access. Additional recommended upgrades would include recoating the roof. Observed cracks should be epoxy injected and sealed to limit further issues with corrosion. Around the exterior of the roof, additional scuppers should be installed in the roof parapet to increase the number of drainage locations. New roof drains would help limit the impacts of a block drain and the potential for material to build-up on the roof.

Along the northwest sidewall, the identified crack and high-efflorescent zone should be injection sealed and coated to prevent further water infiltration. After the completion of additional investigation and structural sampling any necessary wall retrofit design should be developed, as required. This includes the stabilization of the wall corner connections through the use of mechanical connectors or FRP, the potential design of a shear connector to increase the capacity of the roof-to-wall connection, and the potential retrofit of new-to-original wall connection to prevent any hinging at this connection point.

Regarding ancillary structures observed on the site, the vault appeared to be in generally good condition and its proximity to the reservoir was not such that they could damage or weaken the structure should it fail. No repairs are proposed for these structures beyond typical maintenance. The concrete box surrounding the overflow is in poor condition but does not appear to be at risk of failure. The concrete box should eventually be replaced if the overflow outlet discharge point is ever upgraded.

2.6 Scans of Select Construction Documents

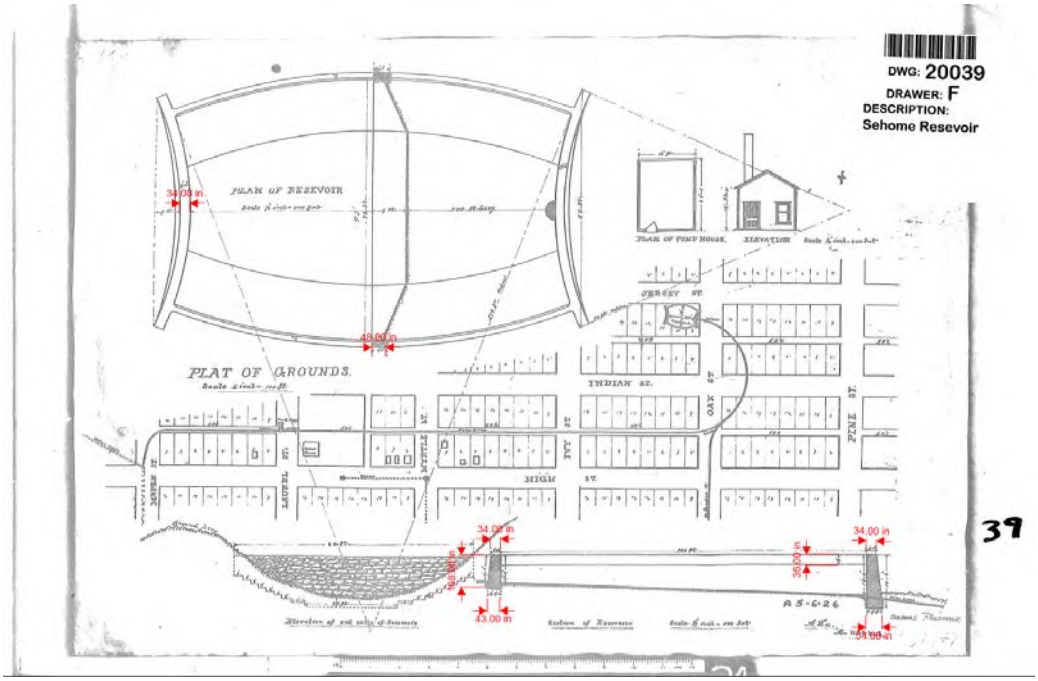


Figure 2-1: Sehome Original Layout Plan

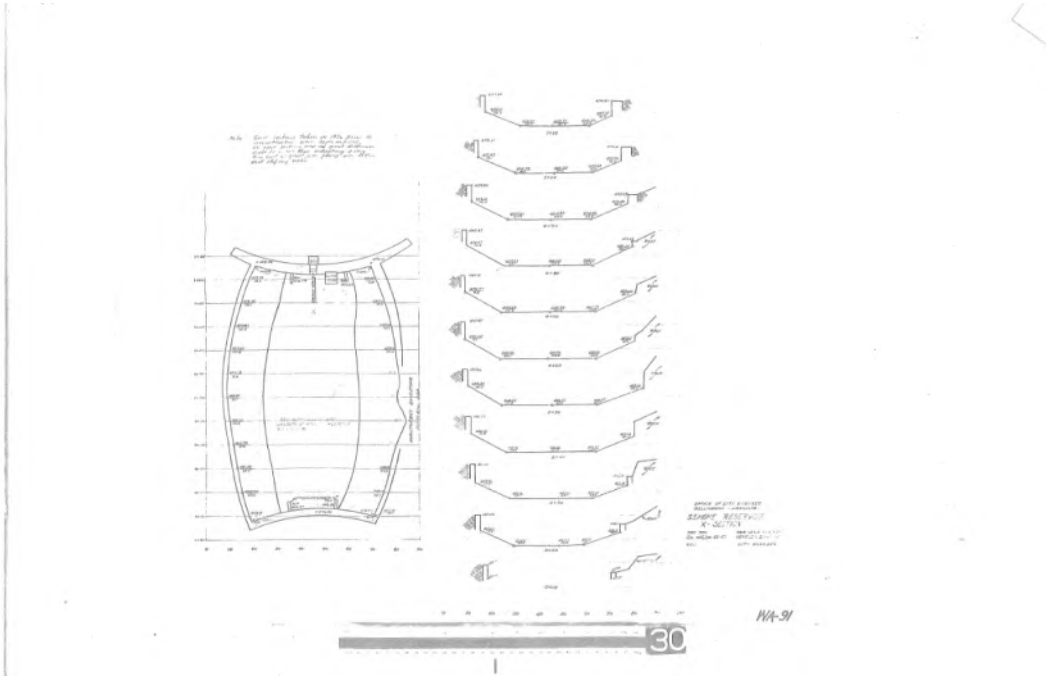


Figure 2-2: Sehome Reservoir original cross sections

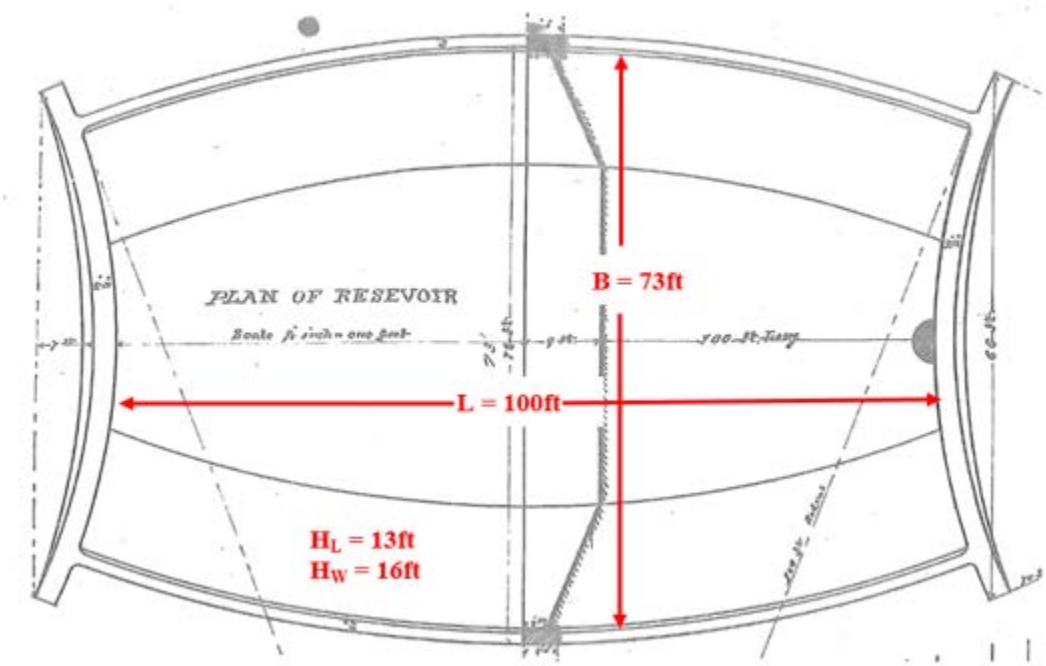


Figure 2-3: Sehome Reservoir Elevations Schematic and Dimensions based on Field Measurements (H_W = Wall Height, H_L = Operating Water Height relative to Wall, B = Width, L = Length)

2.7 Observations Pictures

Figure 2-4: Sehome Reservoir - SW Elevation & Exterior Wall Hatch

Figure 2-5: Sehome Reservoir - NE Elevation & View of Roof Slab

Figure 2-6: Sehome Reservoir - NW Elevation along sandstone outcropping

Figure 2-7: Sehome Reservoir - Piping Vault SW of Reservoir

Figure 2-8: Sehome Reservoir - SW Hatch, Vent Pipe, and Roof Drainpipe

Figure 2-9: Sehome Reservoir - NE Hatch

Figure 2-10: Sehome Reservoir - Roof Slab

Figure 2-11: Sehome Reservoir - Interior North View, showing sloped floor, roof slab with cast-in beams, and support columns

Figure 2-12: Sehome Reservoir - Roof Slab near center, cracking visible at beam centers

Figure 2-13: Sehome Reservoir - NW Wall, Cracking located in wall above sandstone outcropping

Figure 2-14: Sehome Reservoir - Overflow Weir

Figure 2-15: Sehome Reservoir - Inlet, Outlet, and Drain

Figure 2-16: Sehome Reservoir - Location of roof concrete failure around vent located by SW hatch

Figure 2-17: Sehome Reservoir - overflow outlets, SW of reservoir, protected by damaged concrete enclosure

Figure 2-18: Sehome Reservoir – Work crews clearing the soil load from the roof in 2017 (picture provided by the City of Bellingham)

2.8 Field Notes

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Res. Eval

PROJECT NUMBER: 1802-0019

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

Project Name: Sehome, 0.7 MG

Site Visit Date: 1/24/19 Reservoir Type: Concrete w/buttress walls

Temperature and weather: Overcast, wet, 46°F

Site Conditions: Muddy, heavily forested, lots of material (leaves, etc.) on top of res. Before cleaning at least a foot of build-up material on top of res.

PSE Staff: Greg L.

Client/Other Staff: Neal Harding MSA

Overflow: 14'
Operating: 10'-13'

Drawing show original tank. Addition of new roof add columns & ±3.5 ft of new wall height to sides of reservoir.

2

PSE
PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to Top of ~~Roof Slab:~~ Ctr Parapet (N) 7.3' (E) 1-2 9.7' / 2-3 2.9' 5.4' (S) 7.6' (W) 3-4 6.0' / 4-1 0' (break in pan) 1.3'

Roof Slab Thickness: 4" (drawings/measured)

Roof Overhang Dimension: N/A / N/A (drawings/measured)

Drip Groove? (Y/N): N/A / N/A (drawings/measured)

Top Surface Roof Slab Condition: Lots of materials, slope toward center no failures noted

Ladder/Vents/Hatch/Joint Conditions: Hatch (N.) - 30"x30" opening w/ 4" thickness Overflow hatch 30"x24" w/ 4" thickness

Other Comments:

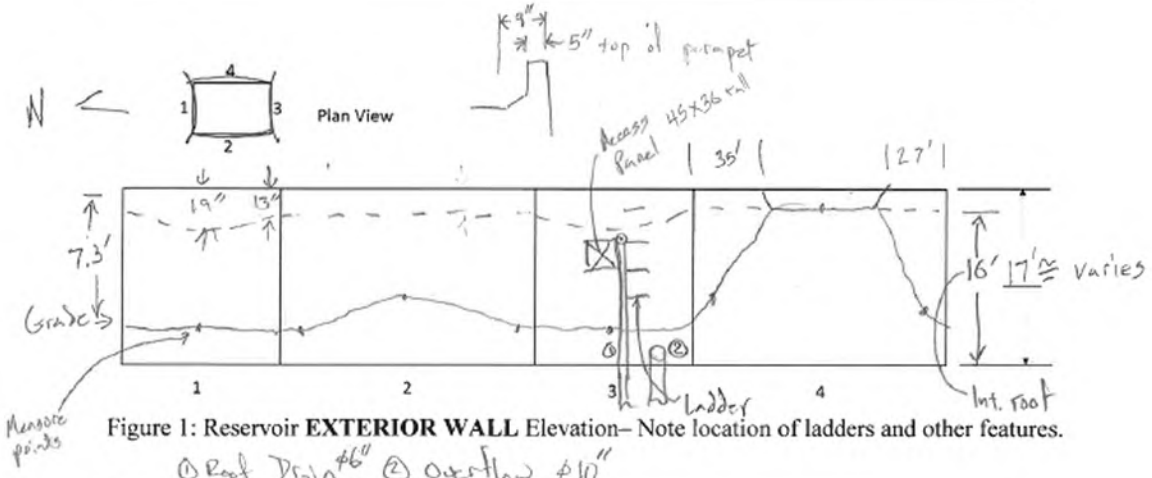


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

① Roof Drain φ6" ② Overflow φ10"

3

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

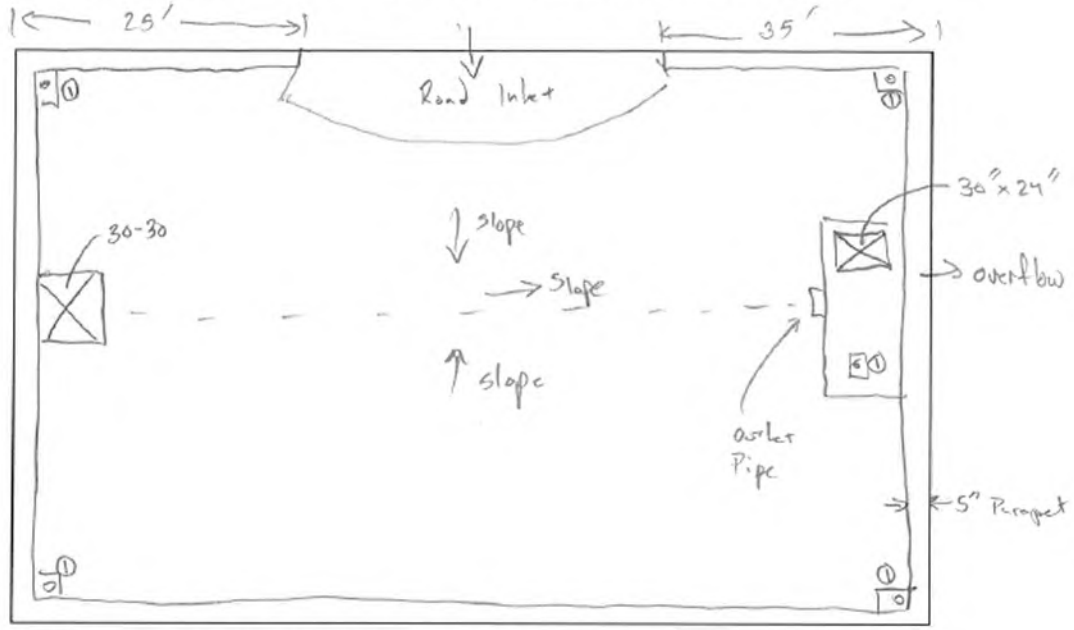


Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

① 3" vent pipe

N

4

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Efflorescing, crack in beam at center between B/C & C/D (see dash line)

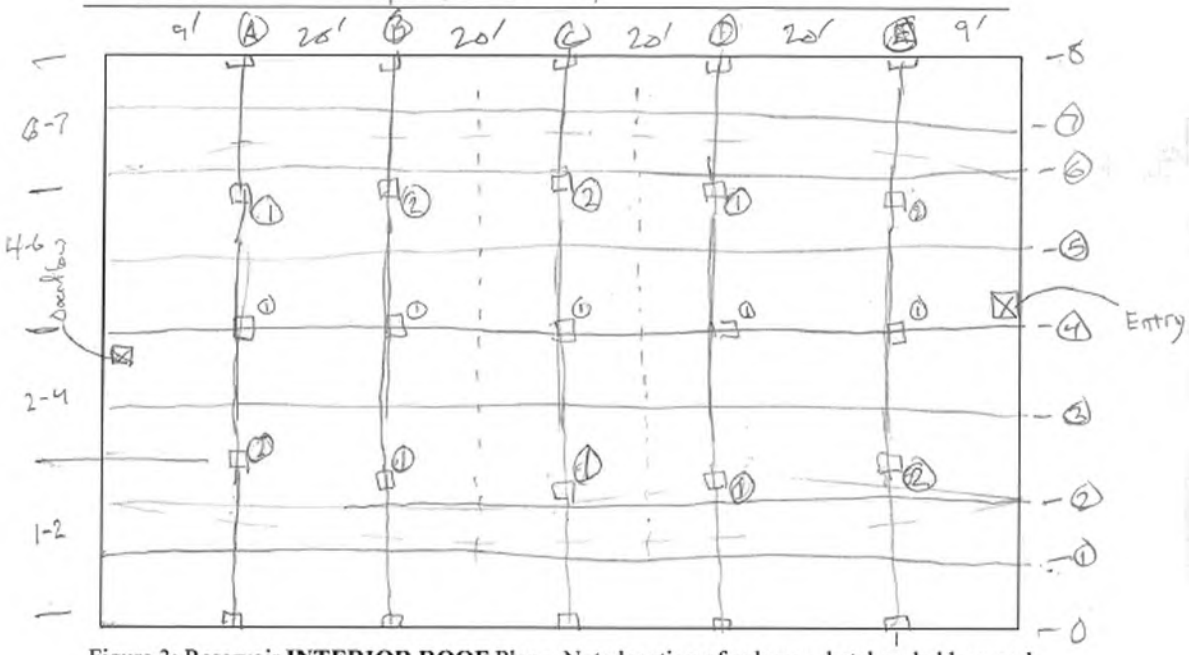


Figure 3: Reservoir **INTERIOR ROOF** Plan – Note location of columns, hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

N

Spacing	A	B	C	D	E
1-2/6-7	14'	14'	15'	16.5'	13'
2-4/4-6	15.5'	17'	18'	17'	15.5'

① 1'x1' column, Base 42"x42"x10" deep

② 1'x1' column, Base 42"x42"x16" deep

Beams A-E 16 deep x 14 wide

Beams 1-7 14 deep x 12 wide

5

PSE
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RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

Column ~~Diameter~~ Square: 1'x1' (drawings/measured) Footing Size/Thickness: 42"x42"x 10"to16" deep (drawings/measured)

Column Spacing: 20' see notes (drawings/measured) Wall Curb Dimensions: sloped (drawings/measured)

Floor Slab Condition: Good min cracking or failure

Floor Slab Joints Spacing/Condition: Joint at top & base of slope

Column/Footing Conditions: Good well consolidated not rust bloom or rock pockets

6

PSE PETERSON STRUCTURAL ENGINEERS

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

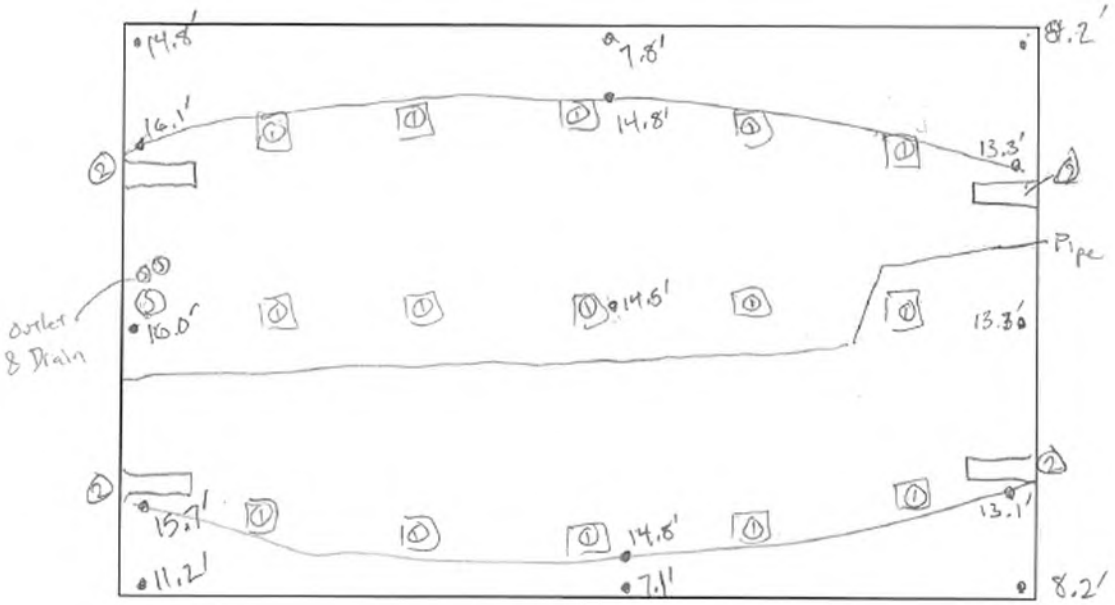
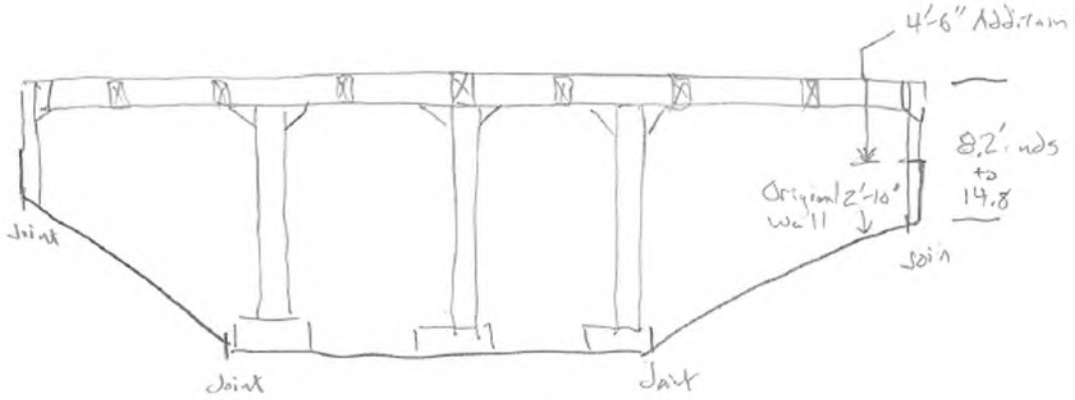


Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

N

① Footing
② Buttress 1'wide, 44" long 5' tall
• Floor to ceiling measurement



Center Section

7

RC CONCRETE (RETANGULAR) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Gen good w/ some loss & rock pockets in wall at entry

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): localize at entry

# of wall sections: N/A

Ladder/Pipes/Overflow Conditions:

Overflow Height: 14' (drawings/measured) Operating Height: 10 - 13' (per City/PUD/other)

Plan View

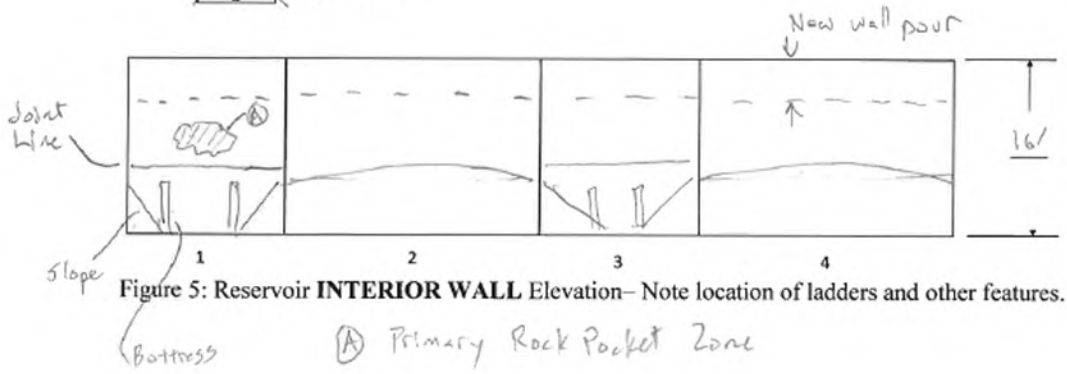


Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

(A) Primary Rock Pocket Zone

8

END OF SECTION

Appendix L-3 Sehome General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Sehome Reservoir *General Info*

Field Visit Date: 1/24/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	1/24/2019
Reservoir Name and Location:	Sehome - Sehome Hill Arboretum off Bill McDonald Parkway
Inspected by:	Nate Hardy, Corey Poland, Greg Lewis
Client Staff Present:	Shayla Francis, Aaron, Kevin
Year Constructed:	1920
Overflow Destination:	Drainage ditch ~30' west of reservoir
Discharge Destination/Zone:	457 South Zone
Fill Location:	457 South Zone (via Otis St PS)
Reservoir Material:	Concrete

Measurement Type	Measurement	Unit
Volume:	0.7	MG
Dimensions:	107 x 73	ft
Height	varies 13.5' (NE) to 16' (SW)	ft
Overflow Elevation:	457	ft AMSL
Bottom Elevation:	443	ft AMSL
Level of Overflow	14	ft
Minimum Normal Operating Level:	10	ft
Maximum Normal Operating Level:	13	ft
Notes: Original reservoir did not have a roof. Addition of roof, columns and ~5.5 ft of new wall height to sides of reservoir have been completed.		

Sehome Reservoir *Exterior Inspection*

Field Visit Date: 1/24/2019

Ladder:	
Present on site?	No
Notes: No exterior ladder present or needed, able to access from ground level.	

Fall Prevention System:	
Present at Site:	No
Notes: Exterior fall protection not needed	

Side Vents and Screens:	
Present at Site:	No
Notes:	

Entry Hatch:		
Hatch Location:	NE roof	
Material:	Concrete	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	No	
Lock:	Yes	
Frame Drain Location:	Unknown	
Measurement Type	Measurement	Unit
Size:	30x30	in
Curb Height:	15	in
Notes: Gasket appears in fair/poor condition.		

Entry Hatch:	
Hatch Location:	SW roof
Material:	Concrete
Condition:	Good
Gasketed:	Yes
Intrusion Alarm:	No
Lock:	Yes
Frame Drain Location:	SW

Measurement Type	Measurement	Unit
Size:	24x30	in
Curb Height:	22	in
Notes: Gasket appears in fair/poor condition.		

Entry Hatch:		
Hatch Location (e.g. side or roof):	SW Side	
Material:	Steel	
Condition:	Good	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Downward	
Measurement Type	Measurement	Unit
Size:	N/A	in
Curb Height:	N/A	in
Notes:		

Roof Vents and Screen:		
Material:	Welded Steel	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	3	in
Notes: Five roof vents, 3-inch, U-shaped pipe. South corner vent has 1\8 in drilled hole screen, others have 1\8 in mesh screens.		

Roof:		
Roof Sloped:	Yes	
Downspouts:	Yes	
Ponding on Roof:	Yes	
Roof Finish:	Concrete	
Slope of roof	Varies, slopes to SW	
Measurement Type	Measurement	Unit
Overhang Distance:	0	in
Thickness of roof slab	4	in
Notes: Relatively flat roof leads to leaves/debris collecting on roof. Slopes from sides to center and then toward roof 6" drain (SW).		

Railing:	
Material:	Steel chain link fence
Condition:	Good
Corrosion:	No
Mid-rail:	No
Attachment Condition:	Good
Attachment Type:	Concrete anchors - side of reservoir

Measurement Type	Measurement	Unit
Parapet Height:	14	in
Top Height:	104	in

Notes: Chain link fence around reservoir roof. Parapet except near access road. Top height = fence + concrete parapet. Currently missing fence segment adjacent to access road invites unauthorized access.

Grating:	
Present at Site:	No
Notes: No grating present	

Other:	
Photo of Anchoring System:	N/A
Flexible Couplings at Foundation:	N/A
Notes:	

Exterior Coating	
Exterior Walls:	No Coating
Exterior of Roof:	No Coating
Exterior Piping:	No Coating
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A

Notes: Roof may have been coated in the past, but has since degraded. Sides are decoratively painted to reduce potential for vandalism.

Sehome Reservoir *Interior Inspection*

Field Visit Date: 1/24/2019

Ladder:	
Present at Site:	No
Notes: No interior ladder present, access provided with extension ladder.	

Fall Prevention System:	
Present at Site:	No
Notes:	

Interior Roof:		
Material:	Concrete	
Condition:	Fair/Good	
Measurement Type	Measurement	Unit
Wall to Mid-span rafter support	Varies	ft
Notes: Concrete roof ~4-inches thick. Cracks observed in beam between B/C and C/D, indicated by a dashed line on diagram below.		

Columns:		
Material:	Concrete	
Condition:	Good	
Measurement Type	Measurement	Unit
Width	12	in
Base width	44	in
Column spacing/configuration: 15 columns in rectangular formation, variable spacing, square columns and bases.		

Floor	
Material:	Concrete
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes: Efflorescent sheen observed on floor of reservoir. Untreated lumber present and mounted to floor, some rotting w/organic growth. Minor cracks observed.	

Walls:	
Painters Rings Present:	No
Condition:	Good
Interior Wall Material:	Concrete
Notes: Generally good w/ localized areas of concrete spalling near north entry.	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	No Coating
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Sehome Reservoir *Miscellaneous*

Field Visit Date: 1/24/2019

Piping		
Inlet/Outlet Piping:	Size (Inches OD):	12
	Condition:	Fair
	Material:	Cast Iron
	Notes: Common inlet and outlet piping, exhibiting signs of moderate/heavy corrosion.	
Overflow Piping:	Size (Inches OD):	8
	Condition:	Good
	Air Gap:	Yes
	Screened:	Yes
	Material:	PVC
	Outlet Location:	Drainage ditch ~30' west of reservoir
	Erosion Evident:	No
	Screen Condition:	Good
	Overflow to roof (Feet)	2
	Notes:	
Drain Piping:	Size (Inches OD):	8
	Condition:	Fair
	Outlet Location:	MH at intersection of High & Oak
	Screened:	Unknown
	Material:	Cast Iron
	Silt Stop Type:	N/A
	Air Gap:	N/A
	Screen Condition:	N/A
	Notes: Unable to access drain outlet at locked manhole.	

Piping Facilities		
Exterior Valving:	Type:	Altitude valve
	Condition:	Good
	Secured:	Yes
Exterior Taps/Hose Bibs	Condition:	N/A
	Secured:	N/A
Washdown Piping	Location:	N/A
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	No
Notes: Water main running through reservoir has major corrosion - suggest maintenance		

Electrical	
Cathodic Protection:	N/A
Impressed Current:	N/A
Anodes:	N/A
Notes: Concrete reservoir	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	Yes
Check Valves:	No
Common Inlet/Outlet:	Yes
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	None
Sediment Build-Up Height Above Floor (ft)	0
Water Quality Sample Taps?	Yes
Notes: As-builts indicate sample station, but did not locate on site visit	

Appendix L-4 Sehome Condition Assessment Score Sheet

Sehome Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance: Static	Structural Performance: Seismic	WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	1	0	Fence missing section. Very suspectable to vandalism
	Vegetation Separation	0	0	0	0	0	0	1	0	Many trees overhang
	Site Drainage	0	0	0	0	1	0	1	0	Roof receives drainage from adjacent hillslope and access road
Walls	Exterior Walls	2	4	3	2	0	0	0	0	Organic growth on sidewalls. Painted ok. Reinforcing unknown; seismic performance likely poor.
	Interior Walls	4	3	3	2	0	0	0	0	Spalled concrete & crack on NW sidewall. Reinforcing unknown; seismic
Floor/ Foundation	Foundation	0	0	5	2	0	0	4	0	Reinforcing unknown; seismic performance likely poor.
	Interior Floor	2	5	3	2	2	0	4	0	Foundation (thickness ?) load exceeded. Reinforcing unknown; seismic performance likely poor. Fungus and sheen
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	1	3	3	2	1	0	1	0	Needs cleaning & better drainage. Roof drains to interior. Reinforcing unknown; seismic performance likely poor.
	Interior Roof and Supports	4	3	3	2	0	0	0	0	Crack and efflorescence @ beam centerlines. Reinforcing unknown; seismic performance likely poor.
	Columns	4	5	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	0	0	0	0	0	5	5	0	Ext Ladder not necessary w/ current grading
	Interior Ladders/Fall Protection	0	0	0	0	0	4	0	0	No interior ladder or fall protection; none required. Some difficulties using removable ladder.
	Access Hatches	3	3	0	0	2	0	3	0	Gasket in poor condition.
	Railings and Roof Fall Protection	0	0	0	0	0	0	5	0	
	Vents	3	2	0	0	1	0	1	0	Failed design checks, non-compliant screen. Spalled concrete @ penetration.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	2	2	0	1	1	0	3	0	Common in/out in bad location. No flexible couplings
	Outlet Piping	0	0	0	1	1	0	3	0	Common in/out in bad location. No flexible couplings
	Drain Piping	2	3	0	1	2	0	3	0	No flexible couplings. Connects to storm sewer without air gap. Unknown dechlorination
	Overflow Piping	5	5	0	0	3	0	0	0	Non compliant screen
	Washdown Piping	0	0	0	0	0	0	2	0	Unknown washdown piping
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	3	3	0	0	0	0	5	1	
	Isolation Valving	3	3	0	0	0	0	5	1	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	1	0	
	Categorical Score	2.9	3.4	3.3	1.7	1.6	4.5	2.9	1.0	

Overall Score
2.6

Appendix M Parkhurst

Appendix M-1 Parkhurst Geotechnical Report

GeoEngineers

600 Dupont Street, Bellingham, Washington 98225, Telephone: 360.647.1510, Fax: 360.647.5044

www.geoengineers.com

Memorandum

To: Nathan Hardy, PE (Murraysmith Inc.)

From: Aaron Hartvigsen, PE
Hamilton Puangnak, PE
J. Robert Gordon, PE

Date: January 22, 2020

File: 0356-159-00

Subject: City of Bellingham Reservoirs Inspection and Repairs
Parkhurst Site

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
1/22/20

INTRODUCTION

The purpose of this memorandum is to provide the results of our geotechnical investigation and evaluation of the Parkhurst reservoir site as part of the evaluation of all the water reservoirs for the City of Bellingham. Our services have been performed in accordance with our Task Order with Murraysmith, Inc. authorized by Nathan Hardy on December 6, 2018.

This memo provides a summary of the conditions encountered at Parkhurst site, located as shown in the Vicinity Map, Figure 1. The Parkhurst reservoir is a round reinforced concrete structure.

SITE CONDITIONS

Geology

We reviewed two United States Geological Survey (USGS) maps for the project area, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976) and "Geologic Map of the Bellingham 1:100,000 quadrangle, Washington" by Lapen (2000). These maps indicate that the site is underlain by undifferentiated glacial deposits. The undifferentiated glacial deposits can consist of a variety of soil types deposited in various glacial environments including glacial till, outwash and glaciomarine drift.

Surface Conditions

The project site is located at the end of Samish Crest Drive, just north of the cul-de-sac. The reservoir is located on top of a hill and the site drops off in all directions, also located next to multiple residential houses to the southeast and west. The site is bounded by a wooded area to the north.

Subsurface Exploration

Subsurface soil and groundwater conditions were evaluated by completing one new geotechnical boring—B-3 (2019)—on March 25, 2019 using a track-mounted drill rig subcontracted to GeoEngineers. The boring was completed to a depth of 5½ feet below the existing ground surface (bgs). The location of the exploration is shown in the Site Plan, Figure 2. A key to the boring log symbol is presented as Figure 3. The exploration log is presented in Figure 4.

Subsurface Conditions

A general description of each of the soil/bedrock units encountered at the project site is provided below. Our interpreted subsurface conditions are based on conditions encountered during our project specific geotechnical boring(s), and review of any previously completed explorations (none available for this site) and our experience at nearby project sites.

- **Chuckanut Sandstone** – Chuckanut sandstone was encountered at the surface of the exploration. The boring was completed at 5½ feet bgs. Weathered sandstone consisting of very dense silty fine to coarse sand with gravel was encountered to a depth of 4 feet. The fine to coarse grained sandstone was brown with weak cementation and occasional bedding planes.

Groundwater

Groundwater seepage was not observed at final depth of the boring. The Chuckanut sandstone unit commonly has isolated saturated sandier zones or "pods" at variable depths and locations. Moderate to rapid seepage can occur in these zones. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Conditions

The reservoir site is underlain by bedrock from the ground surface. Based on review of the project as-builts by Reichardt & Ebe Engineering Inc. dated May 1999, the foundation elevation is approximately 3 feet below grade. Based on the results of our boring and review of the as-builts, the reservoir base is constructed on bedrock.

General Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, three of these earthquakes were large events: (1) in 1949, a moment magnitude (Mw) 6.8 occurred in the Olympia area (2) in 1965, a Mw 6.6 occurred between Seattle and Tacoma, and (3) in 2001, a Mw 6.8 in the Nisqually area (Ichinose et al. 2004; Ichinose et al. 2006) which originated on the Wadati Benioff zone (Wong 2005; Kao et al. 2008).

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Mw 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. Local design practice in Puget Sound and local building codes consider the local seismic conditions including local known faults in the design of structures.

Surface Fault Rupture

No known faults are located in the site vicinity. The nearest known faults shown on maps by the Washington State Department of Natural Resources (DNR) and USGS are the Birch Bay and Sandy Point faults located approximately 5 and 10 miles northwest of the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site. The relevant local deterministic scenarios provided in the Building Seismic Safety County 2014 (BSSC 2014) event set are the Boulder Creek fault (approximately 17 miles northeast) and Devils Mountain fault (approximately 30 miles south), which are used in our analysis.

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at this site we anticipate the existing structure bears on sandstone which is not at risk of liquefaction.

American Concrete Institute/American Society of Civil Engineers 7-10 Seismic Design Information

The following table presents information and seismic design criteria based on ACI 350.3-06 of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) 7-10. The seismic design parameters (S_s, S_1, F_a, and F_v) defined by ASCE 7-10 are assigned to the reservoir based on its intended use and expected performance.

The peak ground acceleration (PGA) value provided below is the maximum considered earthquake geometric mean PGA (MCE_G). The MCE_G PGA corresponds to 2 percent in 50-year probability of exceedance. The short-period and 1-second period spectral response acceleration values provided below are risk-targeted maximum considered earthquake (MCE_R) ground motions. MCE_R ground motions are ground motions in the maximum direction of horizontal spectral response (in contrast to geometric mean) and correspond to a 1 percent in 50-year risk of collapse.

TABLE 1. ACI AND ASCE 7-10 PARAMETERS

ACI and ASCE 7-10 Parameters	Recommended Value
Site Class	C
Soil Profile Type	Very Dense Soil and Soft Rock
Average Field Standard Penetration Resistance	>50
ACI Seismic Use Group	II
Risk Category	IV
Seismic Importance Factor, I_E	1.50
Short Period Spectral Response Acceleration, S_S (percent g)	96.3
1-Second Period Spectral Response Acceleration, S_1 (percent g)	37.9
Seismic Coefficient, F_a	1.02
Seismic Coefficient, F_V	1.42
MCE_G peak ground acceleration, PGA	0.399
Seismic design value, S_{DS}	0.652
Seismic design value, S_{D1}	0.359

Building Seismic Safety Council (BSSC) 2014 Event Set

Seismic design parameters, such as those from ASCE 7-10 summarized above are developed based on the results of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA). A PSHA considers all possible relevant earthquake scenarios and ground motion levels, and computes the rate of occurrence for each scenario. A DSHA is completed based on the same model of earthquake sources as the PSHA; however, only a representative subset of ruptures and specific ground motion levels are considered. Importantly, the rates of occurrence for these ruptures are not computed in the DSHA. The subset of deterministic ruptures considered are for largest characteristic magnitude on each fault. The computed ground motions are median ground motions and correspond to the maximum direction of horizontal spectral response.

The ASCE 7 MCE_R ground motions provided in the previous section were developed by taking the lesser of the probabilistic MCE_R ground motions and the deterministic 84th percentile ground motions, where the 84th percentile ground motions are approximated as 1.8 times the median values, but not less than 0.5 g for PGA, 1.5 g for S_s and 0.6 g for S_1.

We reviewed the relevant local deterministic scenarios provided in the BSSC 2014 event set for the Boulder Creek fault and Devils Mountain fault. The BSSC 2014 event set includes the deterministic ruptures that were used in the 2014 version of the USGS national seismic hazard maps. A scenario represents one realization of a potential future earthquake by assuming a particular magnitude, location, and fault-rupture geometry and estimating the resulting shaking.

M_w 6.8 Scenario Earthquake – Boulder Creek Fault, Kendall Scarp

The Boulder Creek fault is a south-dipping Tertiary normal fault located within the Nooksack River and Boulder Creek valleys east of Deming, Washington. Two Holocene fault scarps, Kendall and Canyon Creek, are in close proximity to the Boulder Creek fault (Sherrod et al. 2013). Trenches excavated across the two

scarps exposed late Quaternary (dated at approximately 12 to 13 thousand years ago) glacial outwash deposits that are faulted and folded (Barnett et al. 2017; Sherrod et al. 2013). The cumulative data obtained from the trenches indicate evidence for an early Holocene and two late Holocene earthquakes dating to 8,050 to 7,250 calendar years before present (cy B.P.), 3,190 to 2,980 cy B.P., and 910 to 740 cy B.P. These three earthquakes are interpreted to have caused surface folding and/or displacement on both scarps with estimated per event vertical separation of the ground surface ranging from 1+0.4 meter (m) to 2.1+0.7 m per event (Sherrod et al. 2013). Empirical relationships between maximum displacement and magnitude (Wells and Coppersmith 1994) indicate the Boulder Creek fault is capable of earthquakes up to Mw 6.8.

Figure 5 presents the ShakeMap for the Mw 6.8 Boulder Creek fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 6.8 Boulder Creek fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking being felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 6.8 Boulder Creek fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 2. DETERMINISTIC GROUND MOTIONS- M_W 6.8 BOULDER CREEK FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.12	0.22	**0.5**
Peak Ground Velocity, PGV (cm/sec)	10	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.28	0.50	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.12	0.22	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec
cm/sec = centimeters per second, g = gravity

M_w 7.5 Scenario Earthquake – Devils Mountain Fault

The Devils Mountain fault is part of the Darrington-Devils Mountain fault zone (DDMFZ) that extends more than 78 miles from the northern Puget Sound region near Victoria, British Columbia eastward to the foothills of the North Cascades south of Darrington, Washington. The fault zone has multiple sub-parallel faults and exhibits south-side-up, left-lateral oblique slip inferred from geologic mapping of Eocene and older rocks along the fault zone (Dragovich and DeOme 2006; Dragovich and Stanton 2007). However, there is seismologic, geologic and paleoseismologic data that indicate the fault is seismically active and there is evidence for postglacial surface displacement on the DDMFZ.

Dragovich and DeOme (2006) and Dragovich and Stanton (2007) present geologic evidence for postglacial displacement on the lowland between Puget Sound and the Cascades foothills. Geologic studies on Whidbey Island and surrounding marine waters provide evidence of Quaternary deformation along the western portion of the DDMFZ and related faults (Johnson et al. 1996, 2001, 2004; Hayward et al. 2006). Three-dimensional trenching across the fault by Personius et al. (2014) provided evidence for 2.2 ± 1.1 m of right-lateral offset of a glacial outwash channel margin, and 45 to 70 cm of north-side-up vertical separation across the fault zone. These offsets indicate a net slip vector of 2.3 ± 1.1 m on a vertical fault plane from one or two paleo-earthquake surface rupturing events. The most recent surface rupture was radiocarbon dated to approximately 2,000 years ago, and stratigraphy in an adjacent wetland to the trench site indicates a possible older earthquake at approximately 8,000 years ago. Personius et al. (2014) estimate the likely magnitude of the paleo-earthquakes to be between Mw 6.7 and Mw 7.0. However, the DDMFZ likely connects with postglacial faults recently identified offshore (Barrie and Greene 2015, 2018) and the Leech River fault on southern Vancouver Island, British Columbia (Morell et al. 2017, 2018). A rupture of these faults in conjunction with the Devils Mountain could generate an earthquake up to Mw 7.5.

Figure 6 presents the ShakeMap for the Mw 7.5 Devils Mountain fault scenario. The ShakeMap provides the location of the epicenter (star) and rupture area (gray box) for the scenario considered and provides geographic representation of the ground shaking intensity. Shaking (or instrumental) intensity is a qualitative measure ranging from I to X+ that portrays the effects of an earthquake scenario at a particular location. These effects include potential damage, perception of shaking and permanent changes in topography. The instrumental intensity at the site from a Mw 7.5 Devils Mountain fault scenario is VI, which corresponds to strong ground shaking with light potential damage. Some observations may include ground shaking be felt by all, weak plaster and masonry crack, and slight damage in poorly constructed buildings.

The table below presents the deterministic ground motions at the site for the Mw 7.5 Devils Mountain fault scenario earthquake. Median, approximate 84th percentile values, and the ASCE 7 minimum values are presented.

TABLE 3. DETERMINISTIC GROUND MOTIONS- M_W 7.5 DEVILS MOUNTAIN FAULT SCENARIO

Parameter	Median	Approximate 84th Percentile	ASCE 7 Minimum
Peak Ground Acceleration, PGA (g)	0.16	0.29	**0.5**
Peak Ground Velocity, PGV (cm/sec)	18	n/a	n/a
Peak Spectral Acceleration for 0.3 sec (g)	0.40	0.72	1.5[1]
Peak Spectral Acceleration for 1.0 sec (g)	0.16	0.29	**0.6**
Peak Spectral Acceleration for 3.0 sec (g)	0.04	0.07	n/a
Instrumental Intensity (I to X+)	VI	n/a	n/a

Notes:
[1] Minimum for T = 0.2 sec

Shallow Foundations

Based on the conditions encountered in our boring and review the project as-builts referenced previously, the existing reservoir is bearing directly on bedrock. We recommend that the structure be evaluated based on an allowable bearing pressure of 6,000 pounds per square foot (psf), which is also consistent with the original

design bearing pressure provided on the as-built. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Lateral Resistance

Lateral foundation loads can be resisted by a combination of friction between the footing and/or slab and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.45 and a passive lateral resistance of 350 pounds per cubic coot (pcf, triangular distribution) may be used. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of 1.5 or greater.

Below Grade Walls

The existing reservoir includes below grade walls. Our recommendations for evaluating below grade walls are presented in the following sections.

Design Parameters

Our recommended design parameters for evaluating below grade walls are presented in Table 4 below. These parameters assume that adequate drainage is provided behind the walls. Additionally, these parameters assume the walls are supported on shallow foundations designed as discussed in the "Shallow Foundations" section and backfilled with structural fill.

TABLE 4. LATERAL EARTH PRESSURES AND SOIL PARAMETERS

Parameter	Value
Backfill and Foundation Soil Unit Weight (γ)	125 pcf
Backfill and Foundation Soil Friction Angle (ϕ)	34°
Active Earth Pressure Coefficient (K_a) – Level Back Slope	0.28
Active Earth Pressure – Level Back Slope	35 pcf
Active Earth Pressure – 2H:1V Back Slope	55 pcf
At-Rest Earth Pressure – Level Back Slope	55 pcf
Seismic Earth Pressure (horizontal)	9H psf

Global Stability

Based on review of publicly available LiDAR for the site, there is a slope inclined at 40 percent or steeper to the north that is approximately 35 feet tall. We did not observe any evidence of slope instability at the time of our explorations. As previously mentioned, we anticipate that the existing reservoir is bearing directly on bedrock. Therefore, it is our opinion that there is a low risk of deep-seated slope instability that would impact the existing reservoir.

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AJH:HP:JRG:tln

Attachments

Figure 1 – Vicinity Map

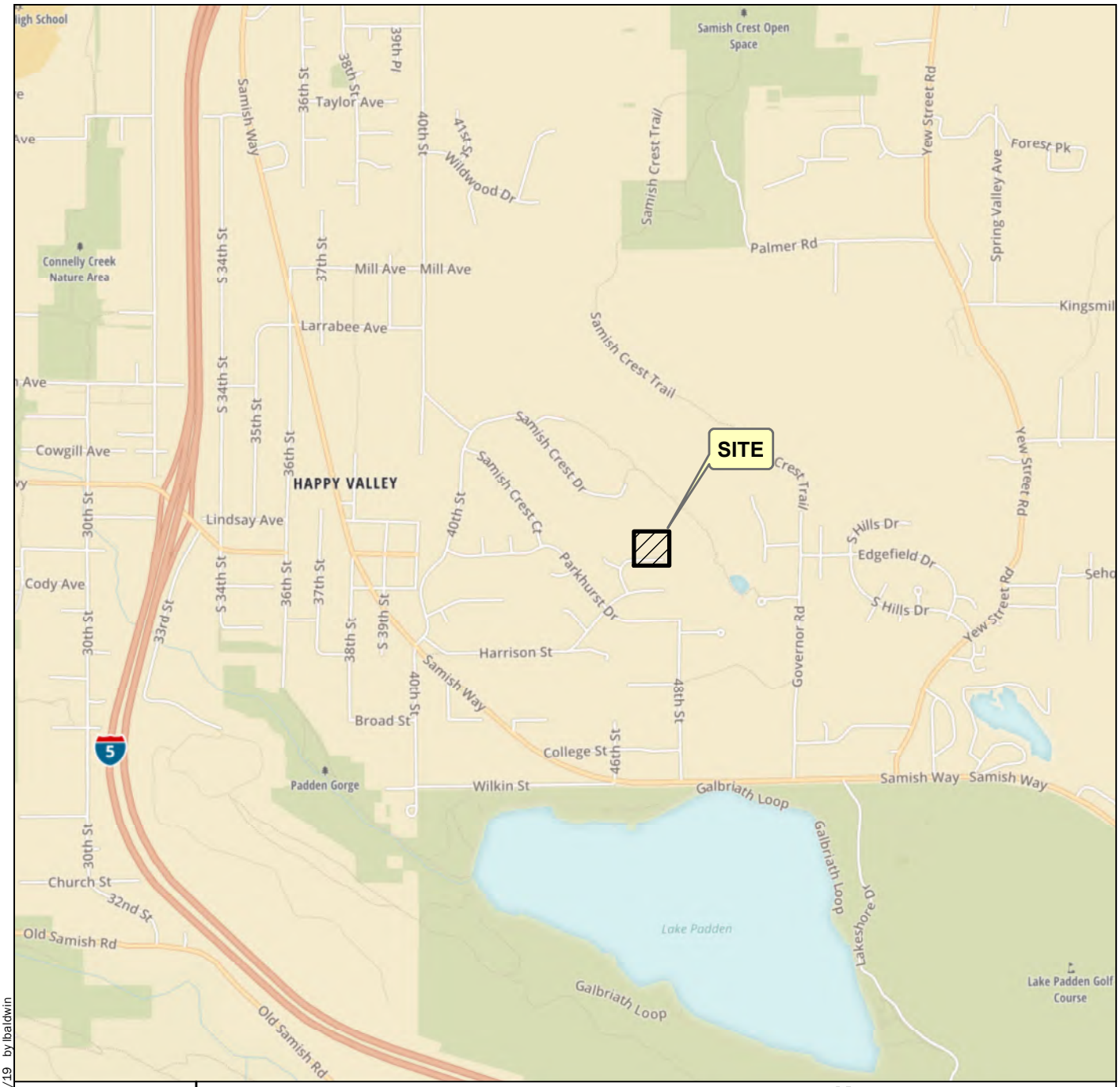
Figure 2 – Site Plan

Figure 3 – Key to Exploration Logs

Figure 4 – Log of boring B-3

Figure 5 – BSSC2014 Scenario Catalog – M 6.8 Boulder Creek Fault, Kendall Scarp

Figure 6 – BSSC2014 Scenario Catalog – M 7.5 Devils Mountain Fault



N
W E
S

1,500 0 1,500
Feet

Parkhurst Vicinity Map

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 1

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

P:\0\0356159\GIS\035615900_F01_VM_Parkhurst.mxd Date Exported: 04/18/19 by lbaldwin

Legend

Boring by GeoEngineers (2019)

75 0 75

Feet

N
W E
S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Parkhurst Site Plan

Reservoir Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure 3

Drilled	Start 3/25/2019	End 3/25/2019	Total Depth (ft) 5.25	Logged By BWS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	840 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	EC-95
Easting (X) Northing (Y)	1248660 629540		System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Gray-brown silty fine to coarse sand with gravel (very dense, moist) (weathered sandstone)			
		X	9	76		1						
835								Sandstone	Brown sandstone (Chuckanut Formation)			
	5	X	3	50/3"		2						

Note: See Figure 5 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on Google Earth.

Log of Boring B-3

GeoEngineers

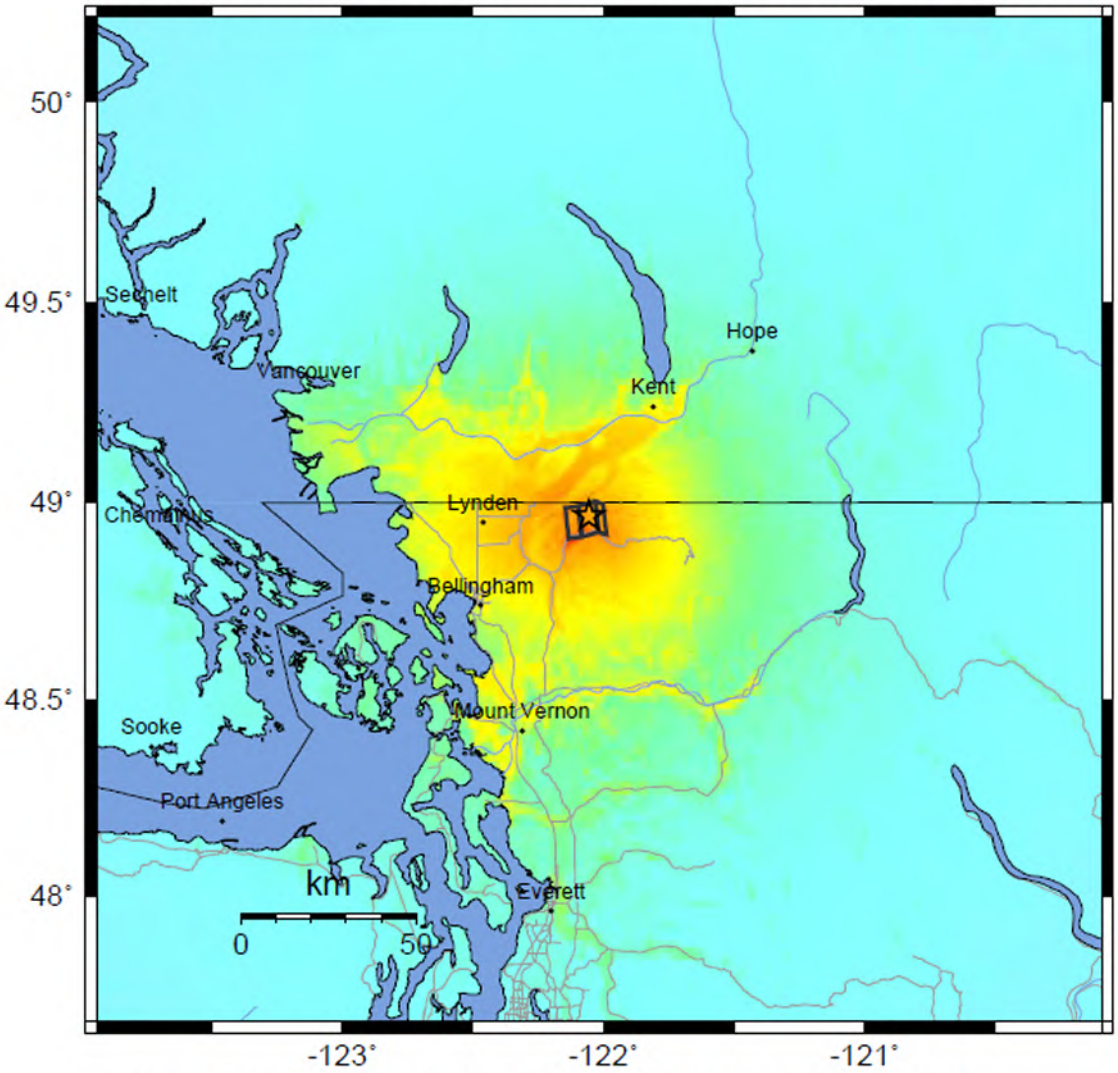
Project: COB Reservoir Inspection and Repair - Parkhurst Drive
Project Location: Bellingham, Washington
Project Number: 0356-159-00

Figure 4
Sheet 1 of 1

-- Earthquake Planning Scenario --

ShakeMap for Boulder Creek fault, Kendall scarp - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 6.8 N48.97 W122.06 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-16 03:09:31 AM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 6.8 Boulder Creek Fault, Kendall Scarp

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 5

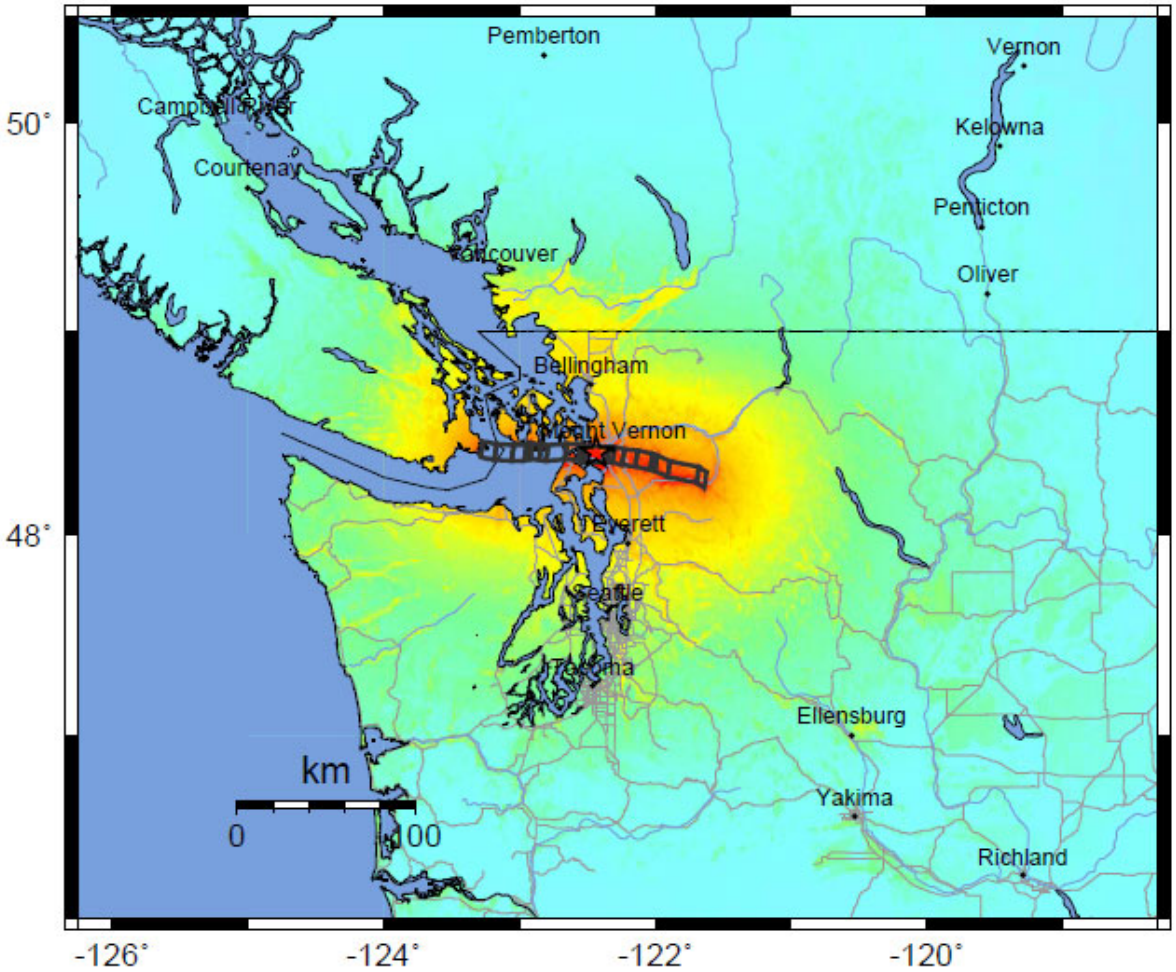
Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

-- Earthquake Planning Scenario --

ShakeMap for Devils Mountain fault - Median ground motions Scenario

Scenario Date: May 12, 2017 02:14:08 PM MDT M 7.5 N48.41 W122.43 Depth: 9.0km



PLANNING SCENARIO ONLY -- Map Version 3 Processed 2017-05-15 10:21:44 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

BSSC2014 Scenario Catalog
M 7.5 Devils Mountain Fault

Reservoirs Inspection and Repair Project
Bellingham, Washington

GeoEngineers

Figure 6

Data Source: https://earthquake.usgs.gov/scenarios/catalog/bssc2014/

0356-159-00 Date Exported: 04/09/15

Appendix M-2 Parkhurst Structural Report

CITY OF BELLINGHAM

CH 15: PARKHURST RESERVOIR

January 24, 2020

Structural Assessment of Steel, Prestressed, and Reinforced Concrete Reservoirs

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Bellingham: Reservoir Inspection - Condition Assessments & Structural Evaluation

Steel, Prestressed, and Reinforced Concrete Reservoirs

January 24, 2020

PSE Tacoma Office

708 Broadway, Ste 110

Tacoma, WA 98402

Table of Contents

1 Introduction

The following memorandum has been generated in order to convey the results from our structural condition evaluation as well as to define potential problem areas that may be candidates for repair or upgrade at the Parkhurst, 0.18 Million Gallon (MG), Mt Baker Silo-type, reinforced concrete reservoir. The reservoir is located near Samish Crest Dr, Bellingham, WA (Lat. 48.745, Long. -122.4561), and is owned and operated by the City of Bellingham in Whatcom County. A site visit was performed to visually inspect and evaluate the reservoir on May 21st, 2019 by Peterson Structural Engineers (PSE) and Murraysmith, Inc. The reservoir had been drained which facilitated the interior inspection. Representatives from the City of Bellingham were present onsite throughout the evaluations.

1.1 Endorsement

This report was prepared by Greg Lewis, PE (WA #51579) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Washington.

2 Parkhurst (Mt Baker Silo-Type) Reservoir – 0.18 MG

2.1 Description & Background

Per information provided by the City, the Parkhurst Reservoir is a Mt Baker Silo, Inc. design. A template design drawing was provided for the project which is based on a typical 30-foot diameter Mt Baker Silo layout and which was originally drawn in 1988. The design was reviewed by Dibble Engineering and updated per the 1997 Uniform Building Code (UBC). The annotated drawing updates are dated August 1998. The reservoir has a monolithic roof and base. The roof slope is 0.8:12 from the center to the roof edge while the floor slab slopes towards a center drain. The structure is a round reinforced concrete reservoir with a measured interior diameter of 30-feet. The interior wall was measured to be 35-feet high and consists of seven (7) 5-foot sections. The overflow pipe is located approximately 34.5-feet above the top of the floor slab. The reservoir is listed as having a nominal storage volume of 185,000 gallons, which occurs when the reservoir is filled to the bottom of the roof (at 35-feet). However, the actual maximum calculated storage volume, as dictated by overflow level of 34.5-feet, is 182,500 gallons.

Where details or sections could not be directly observed or measured, the original design drawings have been used as a reference. Per these drawings the wall is 8-inches thick with variable reinforcing corresponding to the hydrostatic stresses in the walls. The roof is a reinforced 7-inch thick conical slab and is reinforced using an orthogonal grid layout. The floor is a reinforced 12-inch thick slab with two mats of reinforcing on an orthogonal grid. Per the as-built site drawings, by Reichhardt & Ebe (R&E) Engineering, Inc., the piping running under the slab is not shown to be encased in concrete that is commonly used to protect the piping. The annotated reservoir drawing and site plan drawing are shown in Figure 2-1 and Figure 2-2.

2.1.1 Description of Additional Site Structures and Features

The site includes a 5-foot deep by 6.5-foot by 4-foot valve vault which contains piping associated with the reservoir. The vault is located to the southwest, approximately 25-feet from the reservoir's foundation. The inlet and outlet pipes run through the vault while the overflow and drainpipes run to the northeast which daylight immediately off the site.

2.2 Visual Condition Assessment and Associated Recommendations

PSE performed a site visit to observe the as-built current condition of the reservoir's interior and exterior as well as the general site conditions. The reservoir was drained for our inspection and the site visit was performed May 21st, 2019.

<u>Concrete Roof:</u> The reservoir has a self-supporting conical slab roof. The surface of the roof has a broom finish and appears to be generally crack-free and the roof visually appears be in fairly good structural condition. The roof has one 30-inch by 48-inch access opening with a clamshell style hatch. The access zone is surrounded by a railing system attached to the roof slab with wedge anchor bolts. Conduit is run along the top of the roof to an antenna mast adjacent to the roof vent. Incidental corrosion was noted on the hatch and on the hatch's anchor bolts securing it to the roof slab. Observed corrosion should be cleaned and the galvanized parts touched-up with a new paint-on galvanized coating. Generally, the hatch itself appeared in good working order and no section loss or functional defects were observed.

At the center of the roof, the reservoir has a 12-inch diameter vent. Observable components and the surrounding roof do not appear to have any visual structural issues associated with the vent. The vent cover was attached with corroded anchor-bolts which should be repaired as previously described. Please note that the vent could be undersized and Murraysmith should be consulted to determine if a problem exists due to restricted airflow and how to mitigate this issue. Inadequate venting can create significant structural loads if the reservoir is filled or drained faster than the vent's airflow capacity allows.

Reservoir Walls: The exterior of the reservoir was noted to be in fair-to-poor condition and was observed to have multiple instance of vertical and horizontal cracking with efflorescence along course joints, bug holes (small defects in the surface layer of the concrete), and localized point-efflorescent issues. Based on our observation, it appears that the efflorescence build-up had, at one point, been removed by mechanical means (i.e. an angle grinder or equivalent). This was likely done before the painting of the bottom two courses of the concrete shell in an effort to seal these courses. The numerous efflorescent locations indicate this effort was not successful in fully eliminating issues. Cracking and efflorescence issues were noted up to about the fifth shell course before tapering off. Per the referenced drawings, the walls are 8-inches thick with a single layer of vertical and horizontal (hoop) reinforcing. Additionally, the drawings do not show waterstops at any of the reservoir's shell-course joints. A lack of waterstops is likely a contributing factor in the multitude of joint efflorescence issues noted.

The interior of the reservoir was visually observed to be in fair condition. Since the hydrostatic pressure load is oriented outwards, efflorescence issues were not observed on the interior. Additionally, the interior wall face had a smoother finish and while there were fewer instances of bug-holes, a fair number were still observed. Cracking was observed in the walls and the vertical panel edges (a remnant of the formwork) were clearly defined by jagged concrete edge-lines. A coating had been applied to the interior horizontal course joints but based on the amount of exterior efflorescence at joints, this coating does not appear to be wholly effectively.

Reservoir Floor and Appurtenances: The inlet and outlet pipes are located adjacent to the ladder. The inlet pipe extends upwards towards the roof so that water is discharged about 32-feet above the floor. The pipes appeared to be in generally good condition with some corrosion noted to be developing. The outlet is screened, and corrosion was observed to be developing on the screen. This corrosion, left unchecked, could eventually "clog" the mesh and it should be monitored and cleaned as required. The outlet is located at the center of the floor and is embedded in the concrete. No corrosion issues were noted at the outlet. The overflow is comprised of a circular opening in the concrete at the top of the wall. The overflow pipe was observed from the outside and it is made from PVC. As a result, the overflow had no parts prone to corrosion and it appeared to be in good condition.

2.2.1 Visual Condition of Additional Site Structures and Features

The valve vault structure appeared to be in generally good visual condition with no major structural defects observed. No visual signs of major cracking or settlement issues were found. Some moisture was present on the interior floor but did not appear to be sufficient to be indicative of a leak or failure of the vault.

2.3 Structural Analysis

The following structural analysis is based on the provided reservoir drawings and field measurements. For elements which could not be observed, such as reinforcing, the drawings were used for reference. Where elements could be observed and were found to vary from the design, the actual dimensions were used in PSE's analysis. Based on the results of PSE's analysis, potential issues and retrofit options are discussed.

The structural analysis consisted of a seismic and gravity load analysis of the structural elements under the most current edition of the applicable code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary", ACI 318-14 "Building Code Requirements for Structural Concrete", the Portland Cement Association (PCA) References "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, and "Circular Concrete Tanks without Prestressing", published 1993 were also utilized.

2.3.1 Hydrostatic and Gravity Analysis

Conical Roof: The conical roof was evaluated as a circular slab in SAP2000 per criteria as covered in ACI 318-14. The roof was determined to have sufficient capacity for self-weight, roof live, and snow loads. The roof also meets minimum requirements for thickness and reinforcing.

Roof-to-Wall Connection: A concrete reservoir requires a roof-to-wall configuration that will allow for differential thermal movement between the reservoir roof and the wall (as both components deform differently due to temperature variation). At the same time, the roof must be able to engage the walls in order to transmit seismic loads into the reservoir's shear resisting structural elements and resist lateral loads. Currently the roof is supported in a manner that does not allow for thermal movement. However, no issues were observed resulting from this connection types. This is likely owing to the smaller diameter of the reservoir (which results in smaller thermal movement), the relative robustness of the roof-to-wall connection, and the hoop reinforcing in the upper levels of the wall. Evaluation of roof-to-wall connection found that the capacity exceeds the calculated code level expected loads by a factor of ten.

Wall Reinforcement: Per the design drawings the wall is reinforced as noted in Table 1. This variation in the hoop reinforcing is based on the variable pressure distribution of the hydrostatic fluid load.

Per PSE's analysis, it was determined that the vertical wall reinforcing appears to be sufficient for current code strength requirements for operation at the overflow level of 34.5-feet. This design requirement includes an increased design factor for hydraulic loads (1.7 rather than 1.6 as outlined in ASCE) as well as an additional 1.3 sanitary factor. This sanitary factor is intended to minimize the potential for cracking and leaks. While this sanitary factor does help by increasing the total amount reinforcing, the factor alone does not ensure adequate performance. Good construction practices and detailing also play a key role in the reservoir's overall water-tightness and resistance to leaks.

Table 1. Reservoir Reinforcing by Shell Course

Shell Course (8in thick)	Horizontal (Hoop) Reinforcing	Vertical Reinforcing
1 bottom	#8 @ 3.5-inches o.c.	#6 @ 12-inches o.c.
2	#8 @ 4-inches o.c.	#6 @ 14.5-inches o.c.
3	#8 @ 5-inches o.c.	#5 @ 12-inches o.c.
4	#6 @ 3.5-inches o.c.	#5 @ 16-inches o.c.
5	#6 @ 4.5-inches o.c.	#5 @ 18-inches o.c.
6	#5 @ 5-inches o.c.	#5 @ 18-inches o.c.
7 top	#5 @ 7.5-inches o.c.	#5 @ 18-inches o.c.

*o.c. – on center

Additional checks were performed for the wall at the overflow operating level and determined the remaining wall reinforcing to be structurally adequate. This included checks for shear loads and hoop tension forces (when accounting for the larger 1.65 sanitary factor required by code when checking reinforcing in tension). Walls were checked for hoop tension in the concrete itself; the utilization ratio was determined to be 14% whereas typical convention limits this ratio to 10% or less. This exceedance may be a contributing factor for the cracking noted on the interior wall and could have been prevented by using a higher strength concrete in the design.

Finally, per ACI 350.3, the maximum spacing for wall reinforcing was checked. The maximum allowable spacing for bars is limited to 12-inches on center. In the upper sections of the wall, the spacing of the interior and exterior reinforcing is at 18-inches on center which exceeds the current code maximum spacing limit. While this could cause issues, a majority of the cracks and efflorescence were noted in the lower portion of the reservoir, where reinforcing is at a tighter spacing but the hydrostatic forces are at their greatest. Cracking and efflorescence more likely a result of poor detailing and construction practices which also contributed to bug-holes in the concrete and potential issues at construction joints.

Foundations: As the reservoir foundation is buried, the wall footings were evaluated based upon the drawing details. Per the geotechnical evaluation, the site's bearing capacity was determined to be 6,000-psf. Using this bearing capacity and checking for the overflow operating level, the bearing pressure was determined to be within acceptable ranges.

2.3.2 Hydrodynamic and Seismic Analysis

Seismic Wall Reinforcing: Per PSE's analysis, the addition of seismic loads results in additional forces on the wall of the structure. This is a result of the water slosh wave as well as forces resulting from the mass movement of the structure itself. PSE found that the wall flexural stresses were increased by about 45% when compared to the static loads at the overflow operating level. However, even with the increased loading, the reservoir was found to have the flexural and hoop reinforcing needed to resist the increased seismic loads based on current design codes.

Shear at the wall-to-foundation interface, along the leading side of the reservoir experiencing overturning loads, was found to increase by 35%. This resulted in a shear force which exceeds the wall's design capacity by 10%. This load is a result of an evaluation based on the overflow operating level. At the actual 24-foot

maximum operating level (10.5-feet below the overflow), the shear capacity of the wall is not exceeded, and the wall is adequate for shear.

In addition to the wall flexure and tensile checks, PSE also evaluated the reservoir's overall capacity to resist lateral seismic loads. For the in-plane seismic shear forces, PSE determined the reservoir had sufficient reinforcing to resist seismic lateral loads at the overflow level based upon current code requirements. No additional reinforcing or connectivity is needed between the walls and the foundation based upon the assumed construction.

Freeboard/Slosh: At overflow, this reservoir has a freeboard of 6-inches. The available freeboard is insufficient to accommodate the calculated slosh height of 2.9-feet. Due to the location of the roof reinforcing which is on the bottom face of the roof slab, the roof is poorly detailed to resist an uplift load, as would result from a slosh wave. At the overflow operating level, during a seismic event, the roof would constrain the slosh wave. For a constrained slosh wave the force of the wave would act laterally as well as upwards on the roof. The force of this wave would be sufficient to damage and potentially cause failure of the roof at the roof-to-wall interface and could damage hatches and other appurtenances located along the edge of the roof.

However, at the current operating level of 24-feet (10.5-feet below overflow) there is sufficient freeboard to accommodate the anticipated slosh wave and it would not impact the roof. No roof upgrades are required for the current operating level.

2.4 Summary

Based on the available drawings and site visit it appears that a majority of the structural elements in the reservoir are adequate for the expected loads at the current operating level. However, it was noted that the exterior and interior of the reservoir were rife with a variety of vertical and horizontal cracks in conjunction with widespread efflorescence. While the reservoir appears to meet structural requirements for strength, it lacks secondary features such as waterstops and detailing that are designed to limit leakage. Based on the number of bug holes, surface waviness, and joint roughness there are likely some issues with the general level of construction that will continue to result in efflorescence and cracking issues. Note, that continued migration of water through the walls can also initiate and/or increase corrosion of the reinforcing.

Elements outside of the wall, such as the roof and foundation were determined to be adequate when operated at a 24-foot operating level. However, at overflow, the roof is under-designed for the anticipated slosh load. The remaining observable components of the reservoir appear to be in generally fair condition with primary issues being corrosion on metallic appurtenances.

2.5 Compilation of Visual and Structural Analysis-Based Recommendations

There are several options that could be evaluated to bring the reservoir into partial or substantial compliance with current code for an operating level of 24-feet, which is 10.5-feet below the overflow level of 34.5-feet.

Wall and Hopper Cracking and Defect Repair

Based on a maximum operating level of 24-feet, PSE determined the wall had adequate strength for the expected loads under current code. However, the reservoir was still noted to have a significant number of issues associated with cracking and other concrete defects. A potential retrofit would be to fully coat the interior wall and bottom slab of the reservoir. This coating would likely be of a similar type to the other coating systems being used at reservoirs in the City's inventory (i.e. College Way, 40th St, etc.). Recoating would be beneficial in protecting underlying reinforcing against corrosion and in limiting deleterious impacts to the reservoir's overall structural capacity. Please note, coatings are outside of PSE's expertise and we would recommend consulting with Murraysmith and NW Corrosion to determine the best ways to seal observed cracks and coat the interior.

General Recommendations

Along the roof, corroded hatch components and anchors around the railing and vent should be evaluated and replaced or cleaned as necessary. After cleaning, parts should be re-coated with a galvanized paint as recommended by a coating expert such as NW Corrosion.

On the interior of the reservoir, a heavily corroded lug was noted in the wall (see Figure 2-24). The corrosion on this lug is such that the purpose of this item is no longer clear. This element should be removed back to competent material so that no corrosion remains. Once cleaned any steel should be epoxy-coated to prevent further corrosion.

2.6 Scans of Select Construction Documents

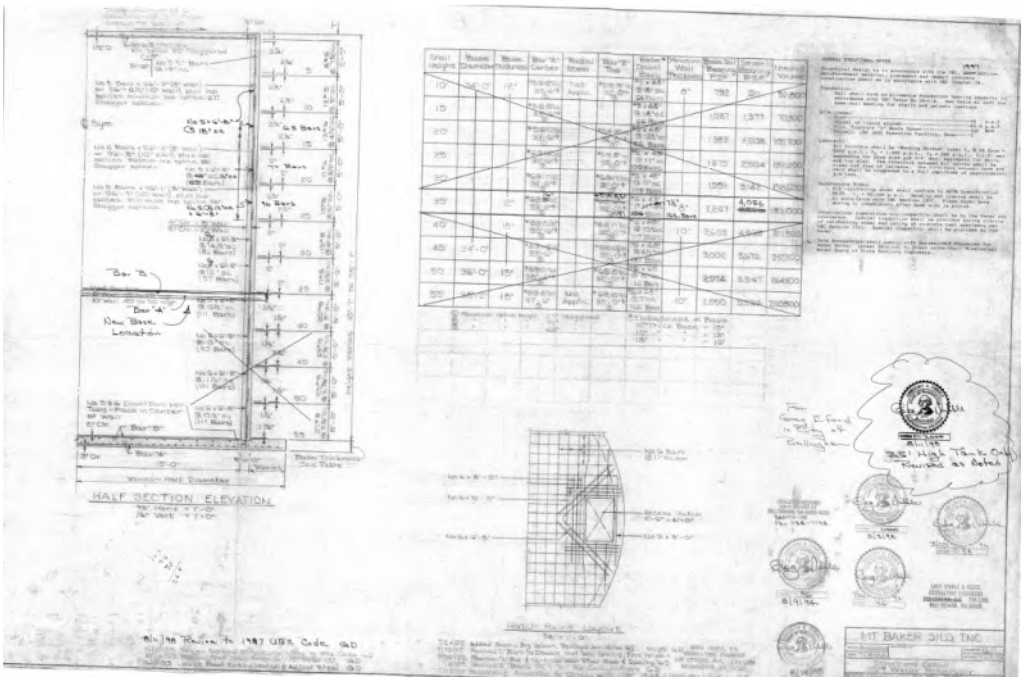


Figure 2-1: Parkhurst Reservoir – Annotated Mt Baker Silo Structural Details, Stamped by Dibble Engineering

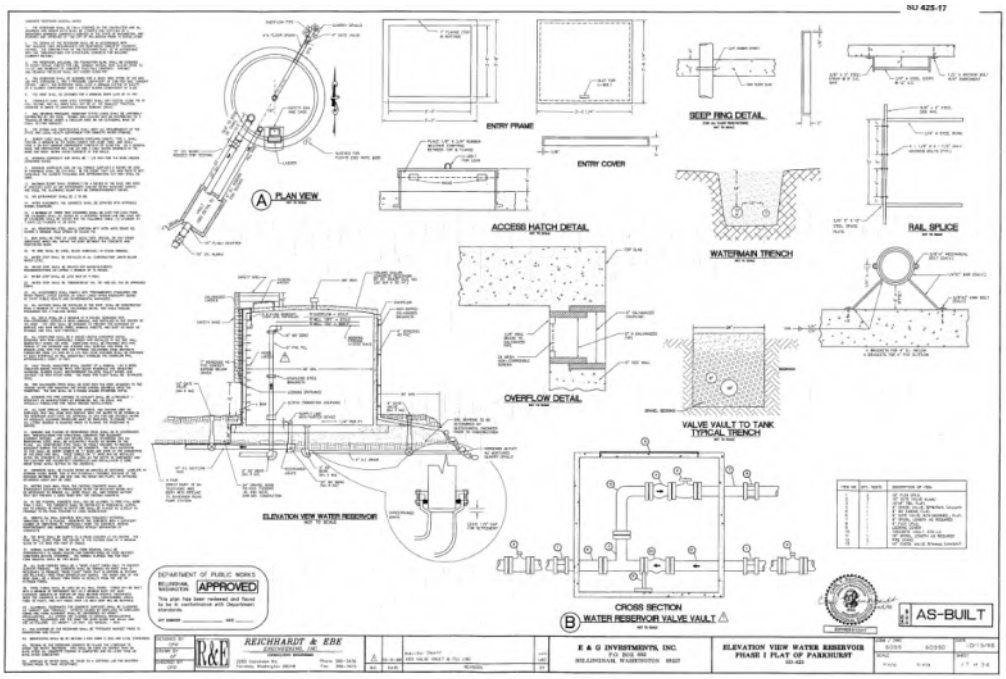


Figure 2-2: Parkhurst Reservoir – Elevation Drawings by Reichhardt & Ebe Engineering, Inc.

2.7 Observations Pictures

Figure 2-3: Parkhurst Reservoir – Elevation

Figure 2-4: Parkhurst Reservoir – Overflow Pipe

Figure 2-5: Parkhurst Reservoir – Entry to Valve Vault

Figure 2-6: Parkhurst Reservoir – Lower Courses and Access Ladder

Figure 2-7: Parkhurst Reservoir – Cracking and Efflorescence (3rd Course)

Figure 2-8: Parkhurst Reservoir – Vertical and Horizontal Cracking and Efflorescence (1st and 2nd Course)

Figure 2-9: Parkhurst Reservoir – Close-up of Vertical Crack (Efflorescence Removed)

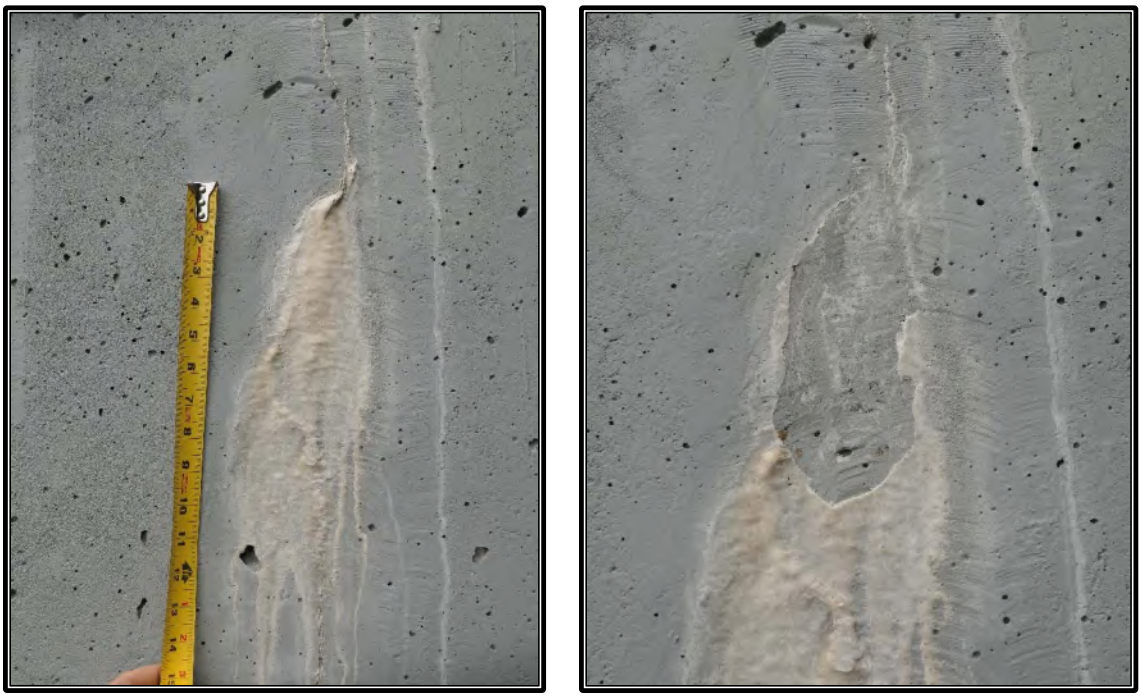


Figure 2-10: Parkhurst Reservoir – Close-up of Efflorescence, Before and After Removal by Rock Hammer

Figure 2-11: Parkhurst Reservoir – Vertical and Horizontal Cracking and Efflorescence (1st and 2nd Course)

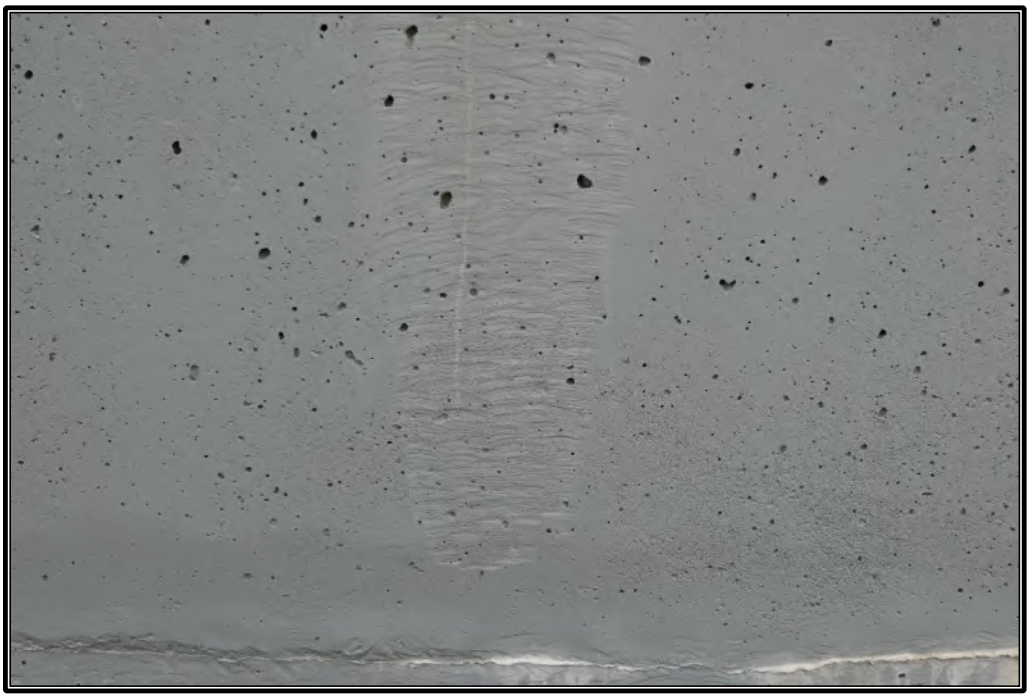


Figure 2-12: Parkhurst Reservoir –Marking on Concrete Indicative of Mechanical Abrasion used to Removed Efflorescence

Figure 2-13: Parkhurst Reservoir – Bug-hole and Defect Density

Figure 2-14: Parkhurst Reservoir – Roof Access Hatch

Figure 2-15: Parkhurst Reservoir – Roof Access Vent

Figure 2-16: Parkhurst Reservoir – Interior Wall

Figure 2-17: Parkhurst Reservoir – Bug-hole and Defect Density

Figure 2-18: Parkhurst Reservoir – Close-up of Coating Used along Joints (Note Bubbling in Coating)

Figure 2-19: Parkhurst Reservoir – Reservoir Roof and Vent Opening

Figure 2-20: Parkhurst Reservoir – Outlet and Inlet Pipes

Figure 2-21: Parkhurst Reservoir – Top of Inlet Pipe

Figure 2-22: Parkhurst Reservoir – Floor with Drain at Center

Figure 2-23: Parkhurst Reservoir – Outlet Pipe (Adjacent to Wall Recess on Left)

Figure 2-24: Parkhurst Reservoir – Corroded Lug in Wall

2.8 Field Notes

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

PROJECT NAME: Bellingham Reservoir Eval.

PROJECT NUMBER: A1802-0019

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Reservoir Name: Parkhurst 0.185 MG 4317 Samish Crest Dr (48.7145, -122.4561)

Site Visit Date: 5/21/19 11am Reservoir Type: Mt Baker Silo Precast Conc.

Temperature and weather: Overcast, rained previous night, 52°F

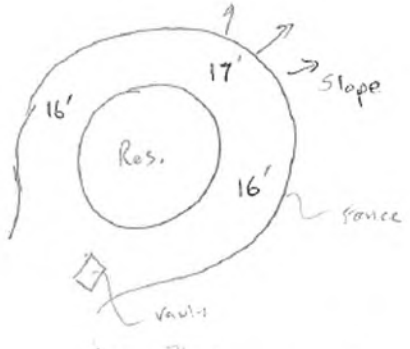
Site Conditions: Cleared w/ fence on top of hill. Hill slopes to east, less so to west which is residential

PSE Staff: Greg Lewis

Client/Other Staff: Corey Poland Murraysmith, Danny Baba Murraysmith City Personnel

185,000 gal Mt Baker Silo 30' Dia × 35' Ht (to top of wall)

Vault near entry, 5' deep, 6.5' × 4.1'
some leakage at wall joint but otherwise in fair condition



Site Plan

1

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Exterior Inspection

Backfill Dim. to Top of Roof Slab: (N) Not Backfilled 1' (E) 1' (S) 1' (W) 1'

Roof Slab Thickness: Unk / 7.5" (drawings/measured) Roof Overhang Dimension: N/A / None (drawings/measured)

Drip Groove? (Y/N): N/A / None (drawings/measured)

Top Surface Roof Slab Condition: Dome, low slope. Thickness measured from hatch

Ladder/Vents/Hatch/Joint Conditions: Ladder - Good, Galv. w/ cage. Vent - 12"ø w/ 16" hood small, galv. sealant on side coming loose. Hatch 30"x48" some corrosion, galv.

Other Comments: Efflorescence on tank appears to have been previously removed w/ a grinder and lower courses painted. Efflorescense is back in some areas. Exterior has high density of pitting & bug holes.

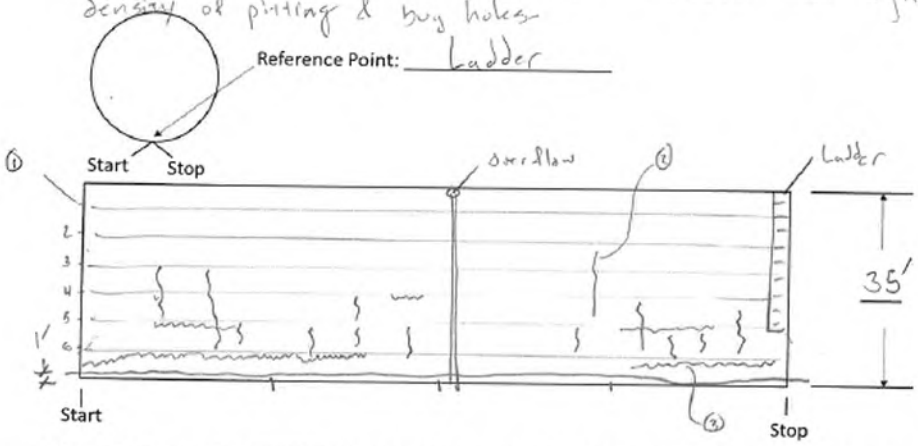


Figure 1: Reservoir **EXTERIOR WALL** Elevation– Note location of ladders and other features.

Shell is composed of 7 precast sections, ea. 5' wide approx. 1' soil over footing. Issues worse on side near ladder

① Joint line, efflorescence worse at join line 6 than 5
② crack w/ efflorescence
③ Horizontal joint efflorescence

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

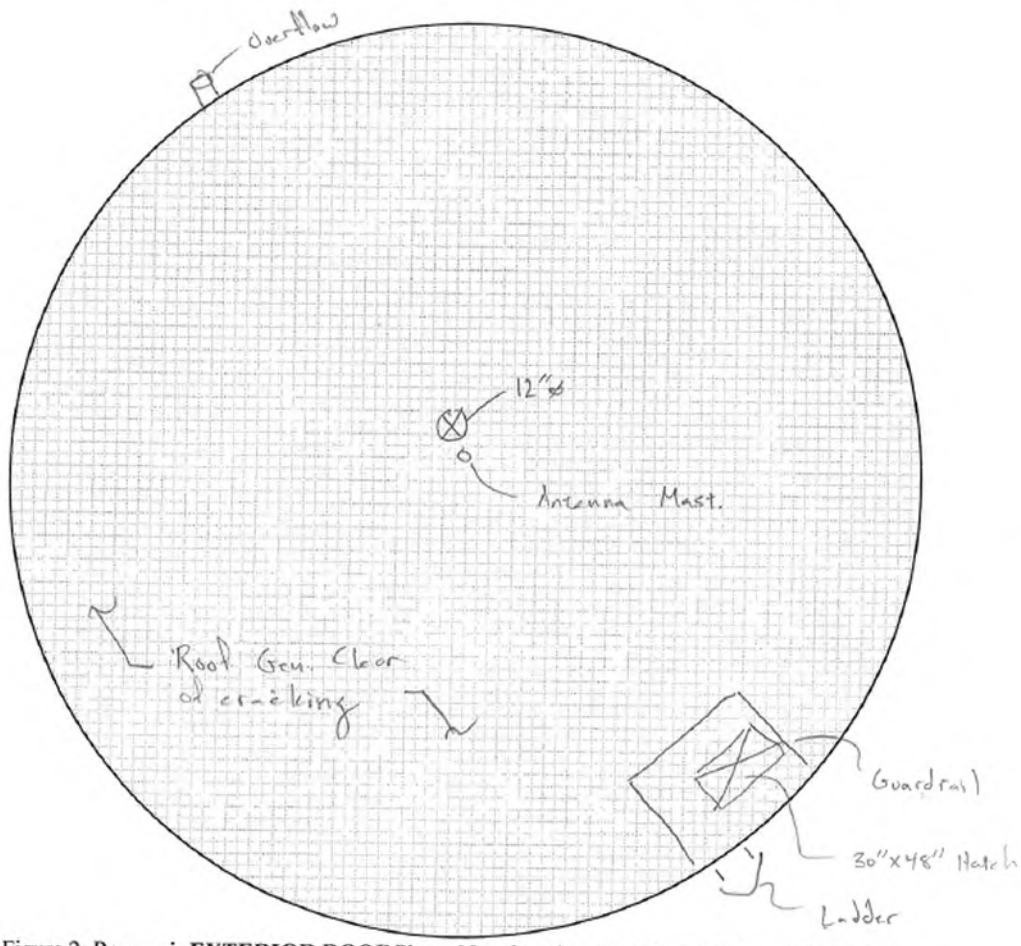

Figure 2: Reservoir **EXTERIOR ROOF** Plan – Note location of hatches, ladders, and manways. List given and measured diameter. (Note columns on next sheet)

RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Inspection

Bottom Surface Roof Slab Condition: Gen. Good. Cracking/discontinuity at form panel edge

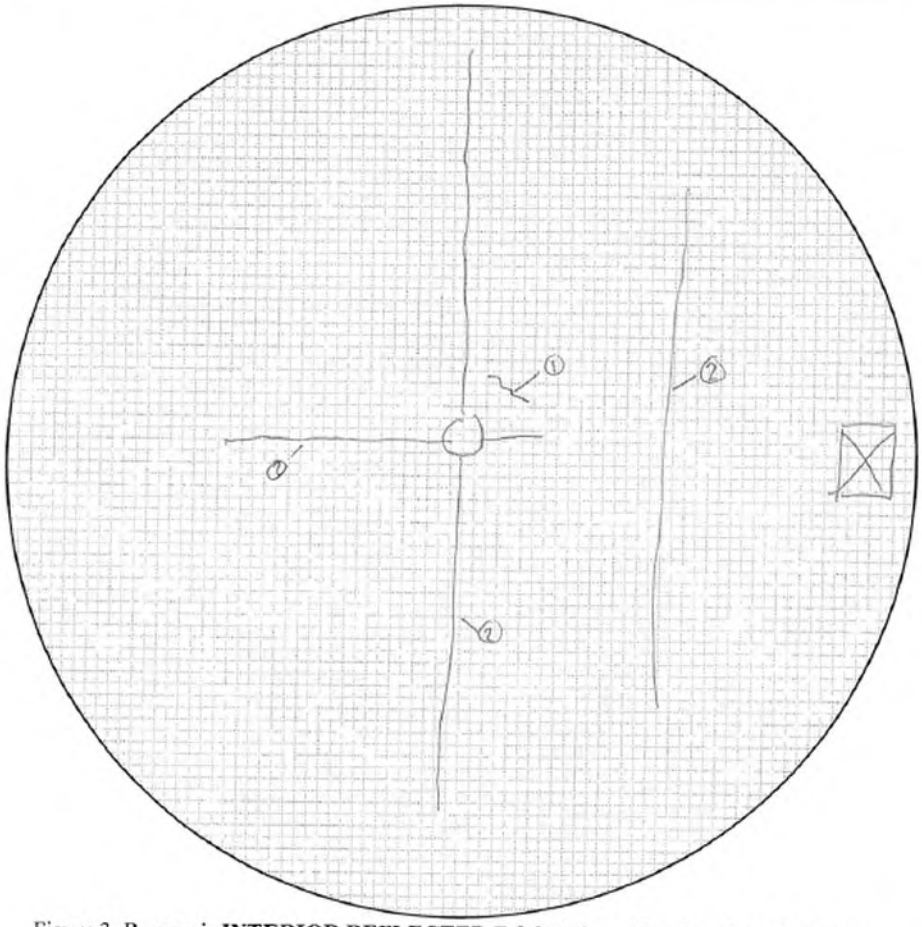

Figure 3: Reservoir **INTERIOR REFLECTED ROOF** Plan – Note location of columns, hatches, ladders, and manways (if present). List given and measured diameter. (Note columns on next sheet)

① crack
② Panel discontinuity

4

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Column Diameter: N/A / None (drawings/measured) Footing Size/Thickness: / Not obs. (drawings/measured)

Column Spacing: N/A / None (drawings/measured) Wall Curb Dimensions: / None (drawings/measured)

Floor Slab Condition: Slab mono-pour, broom finish, no cracks

Floor Slab Joints Spacing/Condition: None

Column/Footing Conditions: N/A

Drain in center of floor, floor slopes to drain. Feature next to drain is covered w/ epoxy, unclear what it is.

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

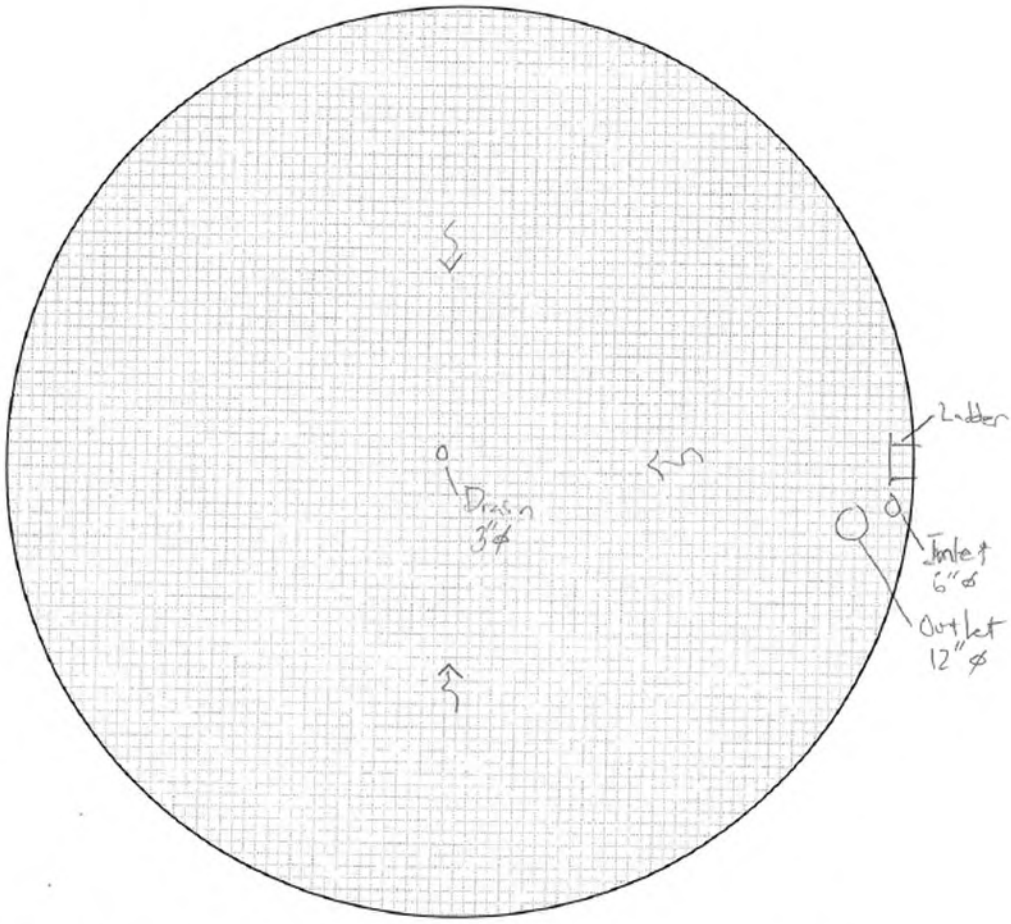

Figure 4: Reservoir **INTERIOR FLOOR** Plan – Note location of columns, inlet, outlets, etc. List given and measured diameter. (Note columns on next sheet)

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RC CONCRETE (ROUND) RESERVOIR SITE INSPECTION

Interior Wall Surface/Base Condition: Flush Joint, no sealant noted

Interior Wall Surface – Pitting and Bugholes (Concentration and Average Depth): Fewer instances than on exter, wall surface still has numerous pit issues

# of wall sections: 14

Ladder/Pipes/Overflow Conditions: Ladder - good minor corrosion; Outlet - screen and pipe corroded

Overflow Height: 34.5', 34.5' (drawings/measured) Operating Height: _____ (per City/PUD/other)

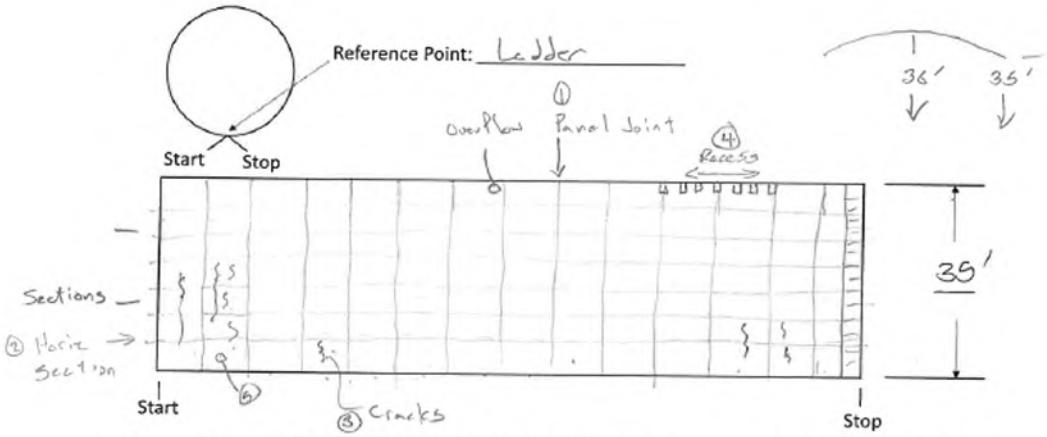

Figure 5: Reservoir **INTERIOR WALL** Elevation– Note location of ladders and other features.

① Vert panel joints approx 1" thick
② Horiz sections, coated w/ Epoxy. Bubbling noted but edges appear competant
③ Various cracks noted, other small cracks present, highlited ones are larger
④ Top panel has recessed point (2 in the body, 2 shared)
⑤ Corroded lug

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END OF SECTION

Appendix M-3 Parkhurst General Inspection Notes

Project Name: Bellingham Reservoir Inspections
Project Number: 18-2337

Parkhurst Reservoir *General Info*

Field Visit Date: 5/21/2019

Project Name:	Bellingham Reservoir Inspections
Project Number:	18-2337
Date:	5/21/2019
Reservoir Name and Location:	Parkhurst - at the end of Samish Crest Drive; 48.7144, -122.4563
Inspected by:	Corey Poland, Danny Baba, Greg Lewis
Client Staff Present:	Shayla Francis, Nick Leininger, Jenny Eakins
Year Constructed:	1997
Overflow Destination:	NE side of reservoir to Quarry Spalls
Discharge Destination/Zone:	Governor Rd Pump Station and 830 Zone
Fill Location:	SW
Reservoir Material:	Reinforced Concrete

Measurement Type	Measurement	Unit
Volume:	0.185	MG
Diameter (or other dimensions - see notes):	30	ft
Height	35	ft
Overflow Elevation:	872.5	ft AMSL
Bottom Elevation:	838	ft AMSL
Level of Overflow	34.5	ft
Minimum Normal Operating Level:	16	ft
Maximum Normal Operating Level:	24	ft
Notes:		

Parkhurst Reservoir *Exterior Inspection*

Field Visit Date: 5/21/2019

Exterior Ladder:		
Present on site?	Yes	
Material:	Galvanized Steel	
Condition:	Good	
Corrosion:	No	
Cage:	Yes	
Security Type:	Locked gate	
Security Condition:	Good	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	16	in
Rung Spacing:	12	in
Side Clearance:	4	in
Front Clearance:	8	in
Back Clearance:	28	in
Notes: Width is 16.75in with side rails.		

Exterior Fall Prevention System:	
Present at Site:	Yes
Type:	Cage
Fall Protection System Condition:	Fair
Notes:	

Side Vents and Screens:	
Present at Site:	No

Entry Hatch:		
Hatch Location:	Roof - SW	
Material:	Aluminum	
Condition:	Fair	
Gasketed:	Yes	
Intrusion Alarm:	Yes	
Lock:	Yes	
Frame Drain Location:	Perimeter	
Measurement Type	Measurement	Unit
Size:	30x36	in
Curb Height:	7	in

Notes: Hatch lid measures 32x38in. Gasket is on lid. Corrosion on upper side of lid.

Roof Vents and Screen:		
Material:	Aluminum	
Condition:	Good	
Prevents Water Intrusion:	Yes	
Measurement Type	Measurement	Unit
Screen Size:	Unknown	in
Notes: Roof penetration is 12in diameter w/ 16.5in diameter outside. 8.5in tall. Possibly undersized.		

Roof:		
Condition:	Good	
Roof Sloped:	Yes	
Downspouts:	No	
Ponding on Roof:	No	
Roof Finish:	Smooth	
Slope of roof	<5 degrees	
Measurement Type	Measurement	Unit
Overhang Distance:	0	in
Thickness of roof slab	7.5	in
Notes:		

Railing:		
Present at Site:	Yes	
Material:	Aluminum	
Condition:	Good	
Corrosion present?	No	
Mid-rail:	Yes	
Attachment Condition:	Good	
Attachment Type:	Anchored	
Measurement Type	Measurement	Unit
Toe Guard Height:	N/A	in
Top Height:	41	in
Notes: Mid-rail height is 21 inches high. Railing is only located around entry hatch.		

Grating:	
Present at Site:	No

Foundation:	
Able to be inspected?	No

Walls:	
Condition:	Poor
Notes: Many cracks are present w/ efflorescence. For the lower two courses, efflorescence was previously ground off and coated.	

Exterior Coating	
Exterior Walls:	Unknown
Exterior of Roof:	No Coating
Exterior Piping:	Unknown
Exterior Coating System Lead Concerns:	No
Paint Samples Collected:	No
Soil Samples Collected:	No
Exterior Coating DFT Testing Results:	N/A
Exterior Coating Adhesion Testing Results:	N/A
Notes:	

Parkhurst Reservoir *Interior Inspection*

Field Visit Date: 5/21/2019

Interior Ladder:		
Present at Site:	Yes	
Material:	Stainless Steel	
Condition:	Fair	
Corrosion:	Yes	
Cage:	No	
Security Type:	Locked hatch	
Security Condition:	Good	
Wall Attachment Type:	Anchored	
Wall Attachment Condition:	Good	
Rung Shape:	Circular	
Measurement Type	Measurement	Unit
Rung Diameter:	0.75	in
Ladder Width	16.5	in
Rung Spacing:	12	in
Side Clearance:	N/A	in
Front Clearance:	8.5	in
Back Clearance:	N/A	in
Notes: Ladder, both rails and anchor points have corrosion.		

Interior Fall Prevention System:	
Present at Site:	Yes
Type:	Cable/slider type
Fall Protection System Condition:	Poor
Notes: Existing system exhibits significant corrosion. Used Dual carabiner ladder straps for inspection.	

Interior Roof:		
Condition:	Fair	
Measurement Type	Measurement	Unit
N/A	N/A	ft
Notes: 8 panels at centerline. Panels are discontinuous		

Columns:	
Present at Site:	No

Floor	
Condition:	Good
Leaks:	No
Approximate Location:	N/A
Severity:	N/A
Notes:	

Walls:	
Condition:	Fair
Painters Rings Present:	No
Notes: Horizontal joints have been sealed but vertical have not. Residual appurtenance sticking out to the right of ladder has corrosion. Wall surface exhibits pitting.	

Interior Coating	
Interior Walls:	No Coating
Interior Floor:	No Coating
Interior of Roof:	No Coating
Interior Ladder:	No Coating
Interior Piping:	No Coating
Interior Coating System Lead/Coal Tar Concerns:	No
Interior Coating DFT Testing Results:	N/A
Interior Coating Adhesion Testing Results:	N/A
Notes:	

Parkhurst Reservoir *Miscellaneous*

Field Visit Date: 5/21/2019

Piping		
Inlet Piping:	Size (Inches OD):	6
	Condition:	Good
	Material:	PVC & Ductile Iron
	Notes: PVC connected to ductile iron pipe at base. Pipe is 6.5in from wall. Inlet 34 feet above floor. Corrosion bracing anchoring.	
Outlet Piping:	Size (inches OD):	10
	Condition:	Good
	Material:	Ductile Iron
	Lip (Inches)	20
	Notes: Includes supply-line anti-vortex device. DI section and device exhibit corrosion. Outlet gate valve in located in grass.	
Overflow Piping:	Size (inches OD):	6
	Condition:	Good
	Air Gap:	No
	Screened:	Yes
	Material:	PVC
	Outlet Location:	Quarry Spalls
	Erosion Evident:	Yes
	Screen Condition:	Good
	Overflow to roof (feet)	0.5
	Notes: On exterior. It is 11in from reservoir. 6ft spacing of braces. Screen too coarse.	
Drain Piping:	Size (inches OD):	4
	Condition:	Poor
	Outlet Location:	Quarry spalls
	Screened:	No
	Material:	Ductile Iron
	Silt Stop Type:	N/A
	Air Gap:	No
	Screen Condition:	Poor
	Notes: Gate valve in ground in reservoir. Measured exterior drain as 5in inside. 3/8in by 3/4in screen. Dechlorination system appears to be ineffective. Erosion exists near drain. Large wire mesh	

Piping Facilities		
Exterior Valving:	Type:	Gate valves
	Condition:	Fair
	Secured:	No
Exterior Taps/Hose Bibs	Condition:	Fair
	Secured:	Yes
Washdown Piping	Location:	N/A
	Size (Inches OD):	N/A
Roof/Wall Piping Penetrations	Sealed:	Yes
	Leaks:	No
Notes: Valving can be accessed from grass.		

Electrical	
Cathodic Protection:	N/A
Impressed Current:	N/A
Anodes:	N/A
Notes:	

Other	
Scour Present	No
Volume of Dead Storage (MG):	TBD
Chlorine Injection:	No
Altitude Valve:	No
Check Valves:	Yes
Common Inlet/Outlet:	No
Manual Level Indicator:	No
Seismic Upgrades:	No
Security Issues:	No
Hydraulic Mixing System Type and Mfg.:	Elevated inlet, anti-vortex
Sediment Build-Up Height Above Floor (in)	0
Water Quality Sample Taps?	Yes
Notes: Inlet and outlet combine after valve vault.	

Appendix M-4 Parkhurst Condition Assessment Score Sheet

Parkhurst Reservoir Condition Assessment

System/ Structure	Assessment Category / Component	Cleanli-ness and Coatings	Material Deterior-ation	Structural Performance		WQ/ Sanitary	Safety	Operations and Maintenance	Obsolescence	Notes
				Static	Seismic					
Site/ Security	Fences and Gate, Security	0	0	0	0	0	0	5	0	Has Camera
	Vegetation Separation	0	0	0	0	0	0	3	0	
	Site Drainage	0	0	0	0	0	0	5	0	
Walls	Exterior Walls	3	2	5	5	0	0	3	0	Lots of efflorescence and cracking <1/16"
	Interior Walls	4	2	4	3	5	0	4	0	Less cracking but location with crack with in 1/16" ranges
Floor/ Foundation	Foundation	0	0	5	5	0	0	5	0	
	Interior Floor	5	5	5	5	5	0	5	0	
	Anchors (Steel) / Seismic Cables (PSC)	0	0	0	0	0	0	0	0	
Roof	Exterior Roof	5	5	5	3	5	0	5	0	
	Interior Roof and Supports	0	4	5	3	0	0	0	0	
	Columns	0	0	0	0	0	0	0	0	
Appur-tenances	Exterior Ladders/Fall Protection	5	5	0	0	0	3	4	0	Compliant through 2036.
	Interior Ladders/Fall Protection	2	1	0	0	0	1	4	0	Fall protection required on 35-foot ladder. Ex fall protection is too corroded to use. Anchors corroding.
	Access Hatches	4	3	0	0	0	4	5	0	Corroded roof hatch lid.
	Railings and Roof Fall Protection	5	3	0	0	0	2	0	0	No major corrosion on rails but in anchors, a critical part.
	Vents	5	4	0	0	0	0	1	0	No major corrosion on vent but in anchors. Penetration and structure undersized.
	Balconies/Landings/Grating	0	0	0	0	0	0	0	0	
Piping/ Valving	Inlet Piping	4	4	0	0	5	0	5	0	Corrosion on bracing and anchors
	Outlet Piping	4	3	0	0	5	0	4	0	Screen is being clogged as the mesh corrodes
	Drain Piping	5	4	0	0	2	0	1	0	Screen on outlet too big. Dechlorination system is very poor.
	Overflow Piping	5	5	0	0	3	0	3	0	Overflow is PVC. Screen out of compliance and no energy dissipation
	Washdown Piping	0	0	0	0	0	0	5	0	Hose in valve vault
	Attached Valve Vault Structure	0	0	0	0	0	0	0	0	
	Control Valving	5	5	0	0	5	0	0	5	Check valves
	Isolation Valving	5	5	0	0	0	0	0	5	
Misc.	Cathodic Protection System	0	0	0	0	0	0	0	0	
	Level Sensors	0	0	0	0	0	0	5	0	
	Hydraulic Mixing System	0	0	0	0	0	0	0	0	
	Categorical Score	4.4	3.8	4.8	4.0	4.4	2.5	4.0	5.0	

Overall Score
4.1

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