CITY OF
BAINBRIDGE ISLAND

UTILITY ADVISORY COMMITTEE
SPECIAL MEETING
WEDNESDAY, SEPTEMBER 2, 2020
5:00-7:00 PM
ZOOM MEETING

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AGENDA

1. CALL TO ORDER / ROLL CALL / ACCEPT OR MODIFY AGENDA / CONFLICT OF INTEREST DISCLOSURE
   5:00 PM
2. APPROVE AUGUST 19, 2020 MINUTES
3. WINSLOW WATER TANK REPLACEMENT DISCUSSION – 60 MIN.
4. SMALL WATER SYSTEMS DISCUSSION (CONTINUED) – 50 MIN.
5. NEXT MEETING AGENDA PLANNING – 5 MIN.
6. ADJOURNMENT
   7:00 PM

For special accommodations, please contact Roz Lassoff
206-780-8624 or at rlassoff@bainbridgewa.gov

UTILITY ADVISORY COMMITTEE
SPECIAL MEETING
WEDNESDAY, AUGUST 19, 2020
ZOOM MEETING

Committee members present: Andy Maron & Ted Jones, co-chairs; Charles Averill, Sheina Hughes, Nancy Nolan, and Emily Sato.
Also present: Council liaison Rasham Nassar; Chris Wierzbicki, COBI Director of Public Works
Absent: Martin Pastucha

CALLED TO ORDER – 5:30 P.M.
The agenda was accepted as written.

Brief discussion of possible new members; no info is available at this time. Current ordinance provides that UAC has 7 members.

Call for Conflicts of Interest disclosures; nobody had anything to disclose.

MINUTES
The February 12, 2020 and July 22, 2020 meeting minutes were unanimously approved as submitted.

PUBLIC WORKS DIRECTOR UPDATE AND PENDING ISSUES
Chris Wierzbicki reported on the following:

- There will be a Special (Zoom) meeting on Sept. 2, meeting with consultant with info about the Winslow reservoir plans.
- An update on the fire code is being worked on. Actual update has been postponed until February. The fire marshal will come talk to this committee on Oct. 14.
- The city is working on a Pre-Treatment policy for light industrial discharges to the sewer system. The Sewer Treatment Plant has capacity limits as to what can be treated, and we're coming close to some of those limits. It may be desirable to regulate those discharges at the source, rather than needing to upgrade the treatment plant. Discharges from wineries and breweries are a particular concern, but more research is needed to understand what kinds of flows come into the system. We hope to receive updates on this in the fall. Andy noted that if pre-treatment changes are required, they may affect not just the business operators, but also potentially the property owners (landlords) who may need to make changes.

SMALL WATER SYSTEMS DISCUSSION
Report from the subcommittee that has been working on this (Andy, Nancy, and Ted).

Andy reported from subcommittee, discussing iterations of memo that was drafted starting 2 years ago. It was last discussed at our meeting in December 2019. The UAC received a draft (not final) from the committee. After discussion, decided to go through memo section by section.

Andy discussed changes from prior drafts. UAC members are asked to review the document further and submit suggestions or questions they may have.

Discussion including the following sections:
The Introduction was expanded, to include a 4th paragraph. Priority focus should be on the systems within and adjacent.

Numbers of water systems (in Section II.B.) have been updated, adding very rough estimates of how many people served in each category. Nancy noted that there may be more Group B's (2-14 connections), e.g. 2-party wells that are not generally known.

Section II.D, Service Areas, information was added since people do not know about the service areas.

Section F not changed dramatically. Summary of rules & regs the apply to water systems

Section III. Analysis & Recommendations. 4 alternatives are discussed:
1. Minimal; 2. Reactive; 3. Active; 4. Active plus acquisition (over time).
Alternatives 3 or 4 will require additional financial support. There was discussion of where funding might come from. Andy noted that it would probably be a mix of utility & general fund – not fair to put all the cost on the city utility, but also not fair to put all on the general fund. So, our recommendation is likely to include a cost-sharing. This will need further discussion.

Section III.B. UAC Recommendation: The recommendation is to move toward a more active role. It was noted that there is no specified timeline. In particular, acquisitions could take a year, or 20 years. But the city needs to identify steps to get there, so it can go forward in a logical manner.

Section III.C. includes nine Suggested Next Steps.
Chris suggested one of the next steps would be to consider how this work plan interfaces with ground-water management plan.

Follow-up to come:

- Andy will make some minor edits (fonts, etc.), and send out the updated version. Members are asked to send further thoughts to Andy, for discussion at our September 2 meeting.
- Sheina offered to write up a sentence or two regarding recommended steps in item 8.
- Ted will distribute the latest version of the matrix. Members should review it and send any comments on the matrix to Ted.
- We may consider adding a shorter version of the report, i.e. an executive summary.

AGENDA FOR THE NEXT MEETING

- Discussion with consultant about the Winslow Reservoir plans. Chris explained that Dept of Health regulations have changed; we will need more standby storage, so the reservoir needs to be bigger than earlier planned.

MEETING ADJOURNED – 6:41 P.M.

_____

Co-Chair Date

CITY OF
BAINBRIDGE ISLAND

Department of Public Works Memorandum

Date: September 1, 2020

To: Utility Advisory Committee (UAC)

From: Christopher Wierzbicki, Public Works Director

Subject: Update on Winslow Water Tank Replacement Project

Purpose

The purpose of this memo is to provide a preview of the information on the status of the Winslow Water Tank Replacement Project that will be shared at the UAC's special meeting on September 2, 2020.

Summary

At the September 2, 2020 UAC meeting, the staff and the City's consultant, Gray and Osborne, will present two emerging issues related to the City's Winslow Water Tank Replacement project resulting from their in-depth evaluation completed for the project Pre-Development Report:

- The total project cost, which is listed in the City's 2019-2024 Capital Improvement Plan (CIP) as $4.2M (tank + high zone improvements), is now estimated to be $10.7M. This cost was identified as early as Spring 2019, but the CIP was not updated to reflect the increase. The current project estimate will now be confirmed in the City's 2021-2016 CIP (see Attachment A).
- Due to changes in state DOH regulations in late 2019 related to the required volume of standby storage, the size of the replacement tank - which was originally planned as 1 million-gallons - is now required to be 2 million-gallons (MG).
- Please note that the increase in the size of the tank is not solely responsible for the increase in the project cost – which was known, but not reflected in the City's CIP. The total cost of the project was determined from an evaluation of all of the project elements (found on PDF page 127 of Attachment B, Pre-Development Report), of which the larger tank only accounts for about 10% of the projected increase in cost (from a previous estimate of $3.3M for the 1MG tank to a new estimate of $4.5M for the 2MG tank.)

The staff look forward to answering questions from the UAC at the September 2nd meeting, and working with the Committee in the coming years to evaluate options and navigate decisions ranging from tank design, to funding and financing options for this critical piece of City water infrastructure.

City of Bainbridge Island
Water CIP (2021 - 2026)
2021-2022 proposed Budget

Project	Location	Grant Eligible	General Comp	Strts Comp	Wtr Comp	Swr Comp	SSWM Comp	Prior Years	2021	2022	2023	2024	2025	2026	2027 - 2040	**Total**
WATER PROJECTS - 6-YEAR CIP (1000s)																
Wyatt Way Reconstruction	Wyatt Way				Y			323	-	-	-	-	-	-	-	323
SCADA Upgrades	Various				Y			208	8	-	-	-	-	-	-	216
Chlorine Generator Upgrades	Various				Y			460	7	-	-	-	-	-	-	467
Winslow Water Tank Replacement	New Brooklyn				Y			1,005	526	32	10,000	-	-	-	-	11,563
Fire Flow Improvements	Winslow				Y			-	89	388	-	-	-	-	-	477
Well Development/Rehab	Pritchard Park				Y			-	54	165	-	-	-	-	-	219
Pipeline Improvements	Shephard Way				Y			-	64	338	-	-	-	-	-	402
Emergency Generator	Head of the Bay				Y			-	-	-	60	135	-	-	-	195
Water Treatment Improvements	Head of the Bay				Y			-	-	-	600	1,600	-	-	-	2,200
Winslow Way West								-	-	-	-	-	60	228	-	288
City Project Funding								1,996	748	923	10,660	1,735	60	228	-	16,349
Grant Totals								-	-	-	-	-	-	-	-	-
TOTALS								1,996	748	923	10,660	1,735	60	228	-	16,349

Amounts in thousands

CITY OF BAINBRIDGE ISLAND

KITSAP COUNTY **WASHINGTON**

CITY OF
BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS
PREDESIGN REPORT

G&O #19648
JUNE 2020

Gray & Osborne, Inc.
CONSULTING ENGINEERS

CITY OF BAINBRIDGE ISLAND

KITSAP COUNTY **WASHINGTON**

CITY OF
BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS
PREDESIGN REPORT

KEITH P. STEWART
STATE OF WASHINGTON
53790
REGISTERED
PROFESSIONAL ENGINEER

G&O #19648
JUNE 2020

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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Appendix D – Preliminary Subsurface Exploration and Geotechnical Engineering Design
Appendix E – Supplementary High School Reservoir Geotechnical Report

CHAPTER 1

INTRODUCTION

INTRODUCTION AND PURPOSE

The City of Bainbridge Island (City) contracted with Gray & Osborne, Inc. in 2017 to provide project assistance and consulting services for a water system assessment to address water quality issues for treated groundwater, investigate existing storage tank seismic and coating issues, address water system pressure at select locations within the service area, and identify standby power requirements for select facilities.

The City utilizes groundwater from three unique sources: Sands Wellfield, Head of the Bay Wellfield, and Fletcher Bay Well. Groundwater from these sources is pumped to the distribution system and any water not directly utilized by distribution system customers is pumped to one of two reservoirs located adjacent to Bainbridge Island High School.

In recent years, the City has noted low water system pressures at high elevations along New Brooklyn Road as well as west of the existing High School Reservoirs. Additionally, the existing reservoirs lack adequate storage capacity for the current and projected demands and in 2018 were found to be seismically deficient. The City is interested in modifying the existing facilities to address susceptibility to low water pressure, to address the noted seismic deficiencies, and to provide sufficient storage capacity for the 20-year planning period.

This report is intended to fulfill the requirements of Washington Administrative Code (WAC) 246-290-110 for a Project Report. The report will provide information on proposed modifications to the existing High School Reservoirs (Reservoir 1 and Reservoir 2) and is generally outlined as follows:

- Chapter 2 includes a description of background information, a summary of previous work and analysis, and a description of the existing facilities.
- Chapter 3 provides a system storage analysis, a description of the low-pressure issues in the system, and a summary of water quality parameters from the three groundwater facilities under consideration.
- Chapter 4 includes a description and analysis of the proposed alternatives.
- Chapter 5 provides recommendations for modifications to the system, a summary of the critical design considerations, and a description of the proposed facilities.

- Finally, Chapter 6 provides recommendations for startup, operation, and maintenance of the proposed facilities.

CHAPTER 2

BACKGROUND INFORMATION

INTRODUCTION

The City of Bainbridge Island (City) is located on Bainbridge Island in Kitsap County, approximately 11 miles west of Seattle across Puget Sound. The City operates a Group A public water system with Water System ID #97650T and serves approximately 7,100 full-time residents via approximately 3,160 service connections. The system also serves a large non-permanent population on weekends and during the summer tourism season. A majority of the City's service connections are residential (~83 percent), however, the City does serve many commercial customers (~ 9 percent). The remaining service connections are for multifamily (~ 4 percent), government (~ 1 percent), and other. These connections are distributed between two pressure zones: the High Zone which covers a majority of the system and the Low Zone which covers roughly the portion of the service area south of Wyatt Way.

The City recently completed an evaluation of several water system facilities with regards to seismic resiliency, physical and structural condition, water quality, electrical redundancy, and performance. The project included seven technical memoranda authored by Gray & Osborne in 2018 and early 2019. One of these memoranda (*Technical Memorandum 17614-4, Gray & Osborne, August 15, 2018*) investigated the physical and structural condition of the City's existing Reservoir 1. The memorandum recommended that the interior and exterior of Reservoir 1 be fully blasted to bare metal and recoated within 5 years to address the existing delaminating coating and protect the integrity of the welded steel materials. Furthermore, Reservoir 1 was found to be seismically deficient in four of six critical parameters. The estimated cost to coat the interior and exterior of Reservoir 1, address all identified seismic deficiencies, and add recommended safety and operational improvement appurtenances was approximately $2,100,000 (including contingency, taxes, design and project administration, and special inspections). The memorandum also noted that the interior and exterior coating systems for Reservoir 2, although showing some signs of damage and fatigue, were in fair condition and have usable service life remaining. The memo recommended that the existing coatings be re-evaluated in 5 years. As with Reservoir 1, Reservoir 2 was also found to be seismically deficient in four of six critical parameters. The estimated cost to address all of the seismic deficiencies and add recommended safety and operational improvement appurtenances was $1,500,000 (including contingency, taxes, design and project administration, and special inspections).

Because of the high cost for these modifications, the susceptibility to low water pressures along New Brooklyn Road, and the fact that the current combination of reservoirs does not provide suitable storage volume for the 20-year planning period, the City is interested in installing a new reservoir in lieu of modifying the existing reservoir. The new

reservoir would be designed in accordance with current building and seismic codes, would provide additional operational and safety appurtenances for City staff, and could address other storage and distribution system deficiencies within the City's water system. The design of the new reservoir would also be intended to improve the pressure issues noted in the High Zone. The report that follows will provide additional information about the existing facilities, provide an analysis of alternatives and their impacts to the City's water system, provide recommendations and preliminary cost estimates, address impacts to the City's distribution system and water quality, and will also provide critical design criteria for the recommended alternative.

PREVIOUS DOCUMENTATION

Several documents have been completed previously and will be referenced throughout this report. These documents are summarized below.

CITY OF BAINBRIDGE ISLAND WATER SYSTEM PLAN

The City's current Water System Plan (*Carollo Engineers, March 2016*) was adopted in 2020 and will be used throughout this report for historical and projected demands, connections, and other planning information.

TECHNICAL MEMORANDUM 17614-1 – WATER QUALITY

This memorandum was produced by Gray & Osborne in 2018 and analyzed water quality data for source water from the Fletcher Bay, Head of the Bay, and Sands well sites. Gray & Osborne compiled and tabulated historical data in an attempt to identify trends or values that exceed current water quality standards. As a result of this memorandum, additional testing was performed from various water sources and the findings from this additional testing helped guide water quality decision making.

TECHNICAL MEMORANDUM 17614-4 – EXISTING RESERVOIR EVALUATION

This memorandum was produced by Gray & Osborne in 2018 and investigated the two existing reservoirs, Reservoirs 1 and 2, which are located just west of Bainbridge Island High School and east of Commodore Lane. The physical condition, seismic design parameters, and coating system were assessed and this memorandum summarizes the findings of this investigation. The memorandum also described the modifications and associated costs for coating recommendations as well as recommendations required to bring each reservoir up to current seismic code.

TECHNICAL MEMORANDUM 17614-5 – PRESSURE ZONE EVALUATION

This memorandum was produced by Gray & Osborne in 2019 and evaluated the existing pressure zones for the Winslow Water System and provided alternatives to address specific areas of low pressure within the service area. Alternatives discussed include the addition of booster stations, interties with neighboring water systems, construction of a new storage reservoir, or a combination of these components.

TECHNICAL MEMORANDUM 17614-8 – SELECTED PRESSURE ZONE ALTERNATIVES ANALYSIS

This memorandum was produced by Gray & Osborne in 2019 and more closely evaluated three of the eight alternatives identified in Technical Memorandum 17615-5. Design criteria and cost estimates for these three alternatives were provided and recommendations were provided for the most cost effective solution to address storage and seismic deficiencies, as well as water quality and low system pressure concerns.

WATER RIGHTS

A full and complete discussion of current water rights was provided in the WSP as well as the City of Bainbridge Island Water Rights Analysis (*Robinson, Noble, & Saltbush, 2002*). A summary of water right information from the WSP is provided in Table 2-1.

TABLE 2-1

City of Bainbridge Island Water Rights Summary

Source	Number	Status	Certificate	Priority Date	Q_i (gpm)	Q_a (acre-ft/yr)
Primary Sources						
Fletcher Bay	01	Operational	G1-20706C	6/14/73	730	1,168
Commodore	01	Not Used	C-6025-A	4/8/68	20	32
	02	Operational	G1-23678C	9/15/80	120	32(1)
Sands	01	Operational	G1-25264C	6/29/88	300	336(1)
	02	Operational	G1-25614P	2/1/90	500	564(1)
Head of the Bay	01/02(2)	Not Used	C-5597-A	3/21/66	55	88(1)
	01/02	Operational	C7410-A	8/18/67	300	336
	03	Operational	G1-22248C	6/28/74	75	160
	04/05	Operational	G1-24349C	7/8/83	200	224
Primary Source Total					**2,300**	**1,920**
Secondary Sources						
Lower Weaver	01	Not Used	C-3170-A	2/20/58	100	160
	02/03	Not Used	C-3171-A	2/20/58	50	80
Wing Point	01	Not Used	C-1011-D	8/1/30	7	11
	02	Not Used	C-3786-A	7/16/57	13	21
Fox	01	Not Used	C-4786-A	5/21/62	30	48 (1)
Surface	-	Not Used	C-7943	6/25/58	157	253
Secondary Source Total					**357**	**525**
Combined Source Total					**2,657**	**2,445(3)**

(1) Supplemental to existing water rights.
(2) Original well(s).
(3) Does not include supplemental water rights.

WATER SOURCES AND BOOSTER FACILITIES

The City primarily uses three sources of water to provide service to the Winslow Water System. These wells, named Head of the Bay, Sands, and Fletcher Bay, are described below.

HEAD OF THE BAY (HOB)

The HOB well site currently provides about 25 percent of the supply for the Winslow Water System. The site has seven active wells (DOH SO1, SO2, SO3, SO8, S09, S10, and S11) that extract ground water from the Sea Level aquifer. Wells 1, 2, and 3 were drilled in 1967, 1971, and 1974, respectively. The control building was expanded, new booster pumps installed, and the 10,000-gallon intermediate reservoir constructed in 1973. Wells 4 and 5 were drilled and a gas chlorinator installed at the well site in 1983. Well 6 was added in 1985 and Well 1A was drilled in 1988. Existing treatment includes

disinfection with sodium hypochlorite generated onsite, and fluoride addition via a fluoride saturator.

The existing facilities include seven wells, each of which is contained within a wood framed or plastic building, a 10,000-gallon intermediate storage tank, and a CMU treatment building. The facility is equipped with connections for a portable generator if backup power is required. The intermediate reservoir is a 10,000-gallon concrete structure located adjacent to the existing treatment building and accepts flow from all seven wells. It is important to note that the wells can flow freely under artesian pressure or be pumped by the well pumps. The artesian flow depends on the aquifer levels and groundwater table within the area, but is significantly less than the pumping capacity of the existing booster pumps. The artesian pressure is also significantly less than the existing booster pumps and is not sufficient to flow to the distribution system without additional pressure boosting.

Water is pumped from the intermediate tank to the City's distribution system via booster pumps. Booster Pump 1 is a variable frequency drive (VFD) controlled pump with a capacity of 600 gpm while Booster Pump 2 is a fixed rate pump with a capacity of 300 gpm.

Under normal operating conditions, the wells are signaled to run by the water level within Reservoirs 1 and 2. Once any of the well pumps are energized, the site operates through a local PLC based on the level of the intermediate reservoir. When Booster Pump 1 is part of the control scheme and the system is called to operate, the selected well pump(s) start and pump into the intermediate reservoir. When the level within this intermediate reservoir rises to the "pump on" setpoint, the booster pump starts and pumps into the distribution system (High Zone). The booster pump modulates motor speed (flow) to match the output of the well pumps and maintain a stable level within the intermediate reservoir. VFD controlled chemical feed pumps provide chemical treatment by injecting chlorine and fluoride into the pump discharge. When the system is signaled to stop, the well pumps shut off, and the booster pump draws down the intermediate reservoir, and then turns off.

Booster pump 2 is a fixed rate pump and is very rarely used to pump from the intermediate storage tank to the distribution system.

SANDS

The Sands Avenue well site currently provides about 50 percent of the supply for the Winslow Water System and includes two wells, DOH Sources S12 and S13. The Sands wells were drilled in 1989 and 1990 and were both fully operational by 1995. A fluoride chemical metering system was installed in 1993 and the chlorination system was most recently replaced in 2002. Between 2002 – 2005 both well pumps were replaced.

The Sands Wellfield is located in the central region of the water system. Each well includes a vertical turbine pump that pumps water from the well to a clearwell located below the treatment building. The treatment building is a wood framed building and includes two booster pumps, chemical storage and metering equipment, onsite hypchlorite generation equipment, and electrical components.

Sands Well Pump 1 and Sands Well Pump 2 have capacities of 350 gpm and 380 gpm, respectively. Under normal conditions, the operation of the facility is controlled by the water level within Reservoir 1 and/or distribution system pressure. When the reservoir water level reaches a lower setpoint, a booster pump energizes and begins pumping water from the clearwell to the distribution system and reservoir. The booster pumps alternate on a typical lead/lag schedule. When the water level within the clearwell reaches its lower setpoint, the well pumps energize and pump water from the well to the clearwell. Over time, the output of the well pumps overcome the output of the booster pump. Under normal conditions, the well pump delivers between 700 – 720 gpm to the clearwell while the booster pump delivers between 615 – 670 gpm from the clearwell – depending on which booster pump is operating. When the water level in the clearwell reaches its upper setpoint, the well pump will shut off until the water level in the clearwell reaches the lower setpoint, at which point the well pump will energize. This process will repeat until the pump call from the reservoir is removed. Both chlorine and fluoride are added to the well and are flow paced based on flow from the well pump.

FLETCHER BAY (FB)

The Fletcher Bay well site currently provides about 25 percent of the supply for the Winslow Water System. The facility is located at the west end of the service area and includes a single well pump, single booster pump, chlorine addition facilities, and fluoride addition facilities, all of which are housed inside a wood framed building. The facility also includes auxiliary power via a diesel powered mobile generator that is normally stationed adjacent to the well building. The well discharge is treated with chlorine and fluoride before entering the 15,000-gallon clearwell located beneath the control building. From the clearwell a single booster pump pumps to the distribution system (High Zone).

Under normal conditions, the operation of the facility is controlled by the water level within the High School Reservoirs and/or distribution system pressure. When the reservoir water level reaches a lower setpoint, the booster pump energizes and begins pumping water from the clearwell to the distribution system and reservoir. When the water level within the clearwell reaches its lower setpoint, the well pump energizes and pumps water to the clearwell. Over time, the output of the well pump overcomes the output of the booster pump. Under normal conditions, the well pump delivers approximately 600 gpm to the clearwell while the booster pump delivers approximately 580 – 590 gpm from the clearwell. When the water level in the clearwell reaches its upper setpoint, the well pump will shut off until the water level in the clearwell reaches the lower setpoint, at which point the well pump will energize. This process will repeat

until the pump call from the reservoir is removed. Both chlorine and fluoride are added to the well and are flow paced based on flow from the booster pump.

Table 2-2 provides a summary of the well pumps installed at each facility. The table provides information on both the design flows as well as the flows measured during a flow test conducted in 2014. The flows measured in 2014 are assumed to be representative of current flows from the well pumps.

Table 2-3 provides information on the existing booster pump operational setpoints. Table 2-4 provides information on the existing booster pumps, including data from two flow tests conducted in 2014 and 2019.

TABLE 2-2

Winslow Water System Existing Well Pump Summary

Pump No.	Year	DOH Source No.	Power	VFD?	Design		2014 Test	
					Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)
HOB 1A	2005	S11	7.5	No	138	156	116	156
HOB 1	-	S01	-	No	37	-	34	-
HOB 2	2011	S02	15	No	206	192	183	192
HOB 3	2005	S03	15	No	250	150	264	150
HOB 4	2005	S08	7.5	No	138	156	83	156
HOB 5	2011	S09	7.5	No	110	175	125	175
HOB 6	2005	S10	5	No	75	100	86	100
Sands 1	2005	S12	40	No	350	-	351	-
Sands 2	1989	S13	40	No	360	-	377	-
FB 1	2009	S07	60	Yes	600	260	591	260
Total					**2,264**	**-**	**2,210**	**-**

TABLE 2-3

Well Site Clearwell and Intermediate Tank Setpoint Summary

Location	Low Alarm (ft)(1)	Booster Pump Stop (ft)	Well Pump Start (ft)	Booster Pump Start (ft)	Well Pump Stop (ft)	High Alarm (ft)	Overflow (ft)
HOB	6.0	7.5	10.0	16.0	17.0	17.5	18.0
Sands	2.2	3.0	3.5	3.5	8.0	8.3	10.0
FB	2.2	3.0	3.5	3.5	8.0	8.3	9.5

(1) Values listed are feet of elevation within each respective tank or clearwell.

TABLE 2-4

Well Site Booster Pump Summary

Pump No.	Year	Power (hp)	VFD?	Design		2014 Test		2019 Test	
				Flow (gpm)	TDH (feet)	Flow (gpm)	TDH (feet)	Flow (gpm)	TDH (feet)
HOB 1	1997	100	Yes	630	380	600	380	630	381
HOB 2	2008	40	No	250	375	300	375	-	-
Sands 1	1990	50	No	450	307	616	307	583	300
Sands 2	2004	50	No	450	307	670	307	633	300
FB 1	2006	60	No	600	265	586	265	580	243

The flows from the Sands booster pumps are beyond the design curve both in 2014 and 2019, which is unusual. Discussions with WTP staff indicate that the flow meters are old, have not been recently calibrated, and may not be providing accurate flow measurements. To be realistically conservative, all subsequent calculations will assume that flows from the HOB, Sands, and Fletcher Bay booster pumps are 620 gpm, 650 gpm, and 580 gpm, respectively. This equates to a total installed source capacity of 1,850 gpm.

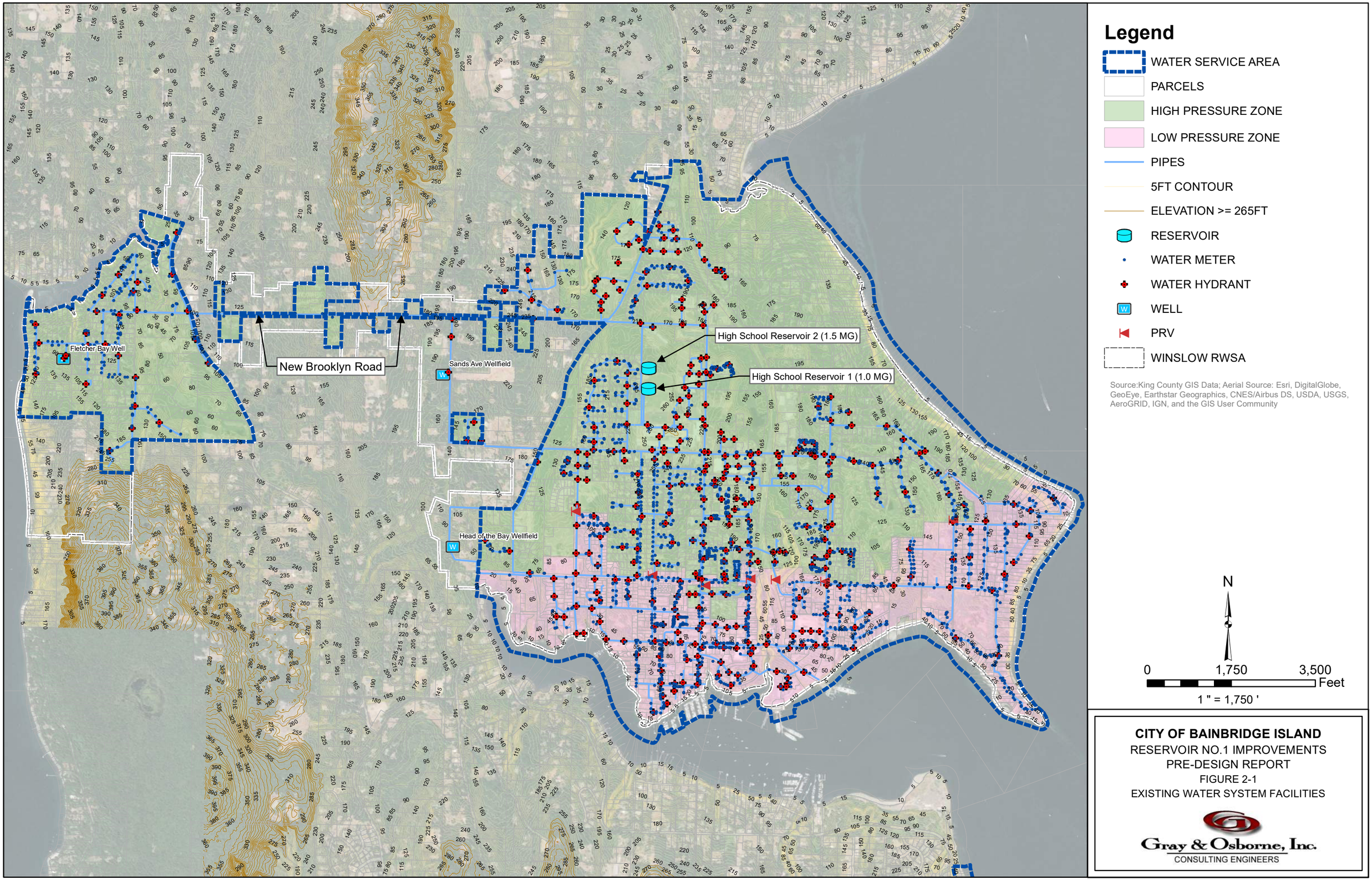
DISTRIBUTION SYSTEM

PRESSURE ZONES

The Winslow Water System is divided into two pressure zones, High and Low. The High Zone serves a majority of the system north of Wyatt Way and has a hydraulic grade line of approximately 334 feet. The Low Zone is served from the High Zone by six active pressure reducing valve (PRV) stations and has a hydraulic grade line of approximately 230 feet. Figure 2-1 highlights the existing pressure zones for the City.

In recent years, the City has received complaints of low water pressure in the area immediately west of Reservoirs 1 and 2 (Commodore Lane and Capstan Drive) as well as select areas along New Brooklyn Road (Mandus Olson Road and Grizdale Lane). These low pressures are largely due to the fact that these residences are located at higher elevations and when the water level within Reservoirs 1 and 2 drops, the pressure available to these locations is reduced.

All alternatives evaluated in this report to address seismic deficiencies of Reservoir 1 or 2 will also consider addressing low system pressure at various locations within the High Zone. Additional analysis on these issues is provided in subsequent sections of this report.



CITY OF BAINBRIDGE ISLAND
RESERVOIR NO.1 IMPROVEMENTS
PRE-DESIGN REPORT
FIGURE 2-1
EXISTING WATER SYSTEM FACILITIES

Gray & Osborne, Inc.
CONSULTING ENGINEERS

L:\Bainbridge Island\19648 - Reservoir 1 Improvements Pre-Design Report\GIS\FIG.2-1 EX FACILITIES.mxd

PRESSURE REDUCING STATIONS

The City maintains six active pressure reducing valve (PRV) stations which control flow between the High and Low pressure zones described above and shown in Figure 2-1.

With the exception of the HOB PRV, each of the six existing PRV stations are equipped with a 2-inch low flow pressure reducing valve and a 6-inch or 8-inch high flow pressure reducing/sustaining valve. In addition, each station has a 1-inch PRV set 1 to 2 psi higher than the low flow PRV to maintain a constant flow to the low zone and help maintain chlorine residuals. These were shut off in December 2016 and are no longer being used. The downstream PRV pressure settings are adjusted to maintain a minimum pressure of 30 psi at the highest elevation in the Low Zone. The HOB PRV was installed in 2003 and includes a single 1-inch PRV whose primary function is to improve water circulation and maintain chlorine residual in the southwest quadrant of the Low Zone.

Table 2-5 provides a summary of the existing PRV stations within the Winslow Water System, with the exception of the 1-inch PRVs currently not in use at each facility.

TABLE 2-5

Winslow Water System PRV Summary

Location	Installed	Type	Size (in)	Pressure Settings (psi)			
				Up	Down	Sustaining	Relief
HOB	2003	Reducing	1	128	83	-	-
Grow and Wyatt	1993	Reducing	6	76	26	40	-
			2	76	31	-	-
Madison and Knechtel(1)	2003	Reducing	4	-	-	-	-
			1.5	-	-	-	-
Ericksen and Wyatt	2003	Reducing	6	78	33	40	-
		Reducing	2	78	38	-	-
		Relief	3	33	-	-	53
Madison and Madrona	2000	Reducing	8	90	40	45	-
		Reducing	2	90	45	-	-
		Relief	2	40	-	-	60
Ferncliff Avenue	1998	Reducing	8	84	33	40	-
			2	84	38	-	-
Cherry Avenue	1993	Reducing	6	75	21	41	-
		Reducing	2	75	26	-	-
		Relief	3	26	-	-	40

(1) This PRV station is currently closed.

Weaver and Cave

DISTRIBUTION SYSTEM PIPING

The distribution system consists of approximately 47 miles of pipe, ranging in size from 2-inch to 12-inch in diameter. The majority of the system is well looped and consists of pipe greater than 8-inches in diameter. Table 2-6 provides a summary of the approximate lengths of pipes in the system by diameter. The piping between each of the three wells that supply the High School Reservoirs is all 12-inch diameter ductile iron pipe and is believed to be in good condition.

TABLE 2-6

Winslow Water System Distribution Pipe Summary

Pipe Size (in)	Ductile Iron	Asbestos Cement	PVC	Galvanized Iron	HDPE	Total (ft)	Percent (%)
< 4-inch (ft)	506	-	1,864	2,851	532	**5,753**	2
4-inch (ft)	7,631	16,877	398	-	778	**25,683**	10
6-inch (ft)	7,385	19,636	8,460	-	-	**35,482**	14
8-inch (ft)	127,207	3,599	4,465	-	-	**135,270**	54
10-inch (ft)	4,388	1,606	2,464	-	-	**8,458**	3
12-inch (ft)	38,177	1,693	-	-	-	**39,870**	16
TOTAL (ft)	**185,294**	**43,411**	**17,651**	**2,851**	**1,309**	**250,516**	**-**
Percent (%)	74	17	7	1	1	-	-

STORAGE FACILITIES

The City owns three storage reservoirs within the Winslow Water System. The City utilizes only the two largest tanks, High School Reservoirs 1 and 2, to provide active storage.

GRAND RESERVOIR

The Grand Reservoir is located near the intersection of Grand Ave and Park Avenue on the eastern side of the City. The reservoir was constructed in 1983 with a capacity of 300,000 gallons; however, the current HGL of the tank does not provide useful storage for the Winslow Water System. The Grand reservoir was taken offline in 2003 and will not be considered further.

RESERVOIRS 1 AND 2

Reservoirs 1 and 2 are located northwest of Bainbridge Island High School and are accessed from a private access road near the intersection of NE New Brooklyn Road and Northtown Drive. Critical design information for these reservoirs is provided in Table 2-7.

TABLE 2-7

Winslow Water System Storage Reservoir Summary

Parameter	Reservoir 1	Reservoir 2
Year Constructed	1973	1989
Type	Above ground	Above ground
Composition	Welded Steel	Welded Steel
Height (ft)	80.7	89.3
Diameter (ft)	46	53
Volume (gallons)	1,003,180	1,473,650
Gallons per Foot	12,430	16,520
Base Elevation (ft)	253.3	246.3
Well Pump On/Off Elevation (ft)	324.5/332.5	324.5/332.5(1)
Overflow Elevation (ft)	334.0	335.6
Inlet/Outlet(2)	12-inch diameter	12-inch diameter

(1) Set points are controlled based on the elevation of water within Reservoir 1.
(2) The reservoir is served by a single pipe, which serves as both the inlet and outlet depending on system demand, system pressure, and operation of the well booster pumps.

Reservoir 1

Reservoir 1 was constructed in 1973 and is an above ground, welded steel tank with a capacity of approximately 1.0 million gallons (MG). The tank rests on an octagonal, thickened edge concrete foundation and is located directly adjacent to single family residences as well as the Bainbridge Island High School playfields. The property is owned by Bainbridge Island High School but is leased by the City. The reservoir is located on a 0.25-acre, flat, grass-covered site and is secured via security chain link fencing. Electrical service is available at this location and the site is accessed by a 12-foot wide double swing gate.

The Reservoir contains a single 30-inch manway at the base of the tank and a roof vent located at the top of the tank. The roof is accessed via a ladder equipped with two intermediate platforms and a safety cage. The ladder does not contain a harnessed ascent system and the base of the ladder is restricted with a padlock security gate.

Reservoir 1 floats on the distribution system and is served by a single 12-inch diameter inlet/outlet pipe. When the water level in Reservoir 1 reaches an elevation of 324.5 feet (water height of 71.5 feet), as measured by a pressure transducer, the well and associated booster pumps located at either HOB, Sands, or Fletcher Bay will energize and supply water to the distribution system. Any water not utilized within the distribution system will flow to Reservoir 1 and 2, and will fill both tanks until the water within Reservoir 1 reaches an elevation of 332.5 feet (water height of 79.5 feet), at which point the well/booster pumps will de-energize.

As highlighted in *Technical Memorandum 17614-4* (Appendix B), Reservoir 1 has several significant seismic deficiencies, and the existing coating system shows signs of delamination. This memorandum recommended that the existing interior and exterior coating system be completely replaced within 5 years, and that seismic deficiencies be addressed within 3 years.

Reservoir 2

Reservoir 2 was constructed in 1989 and is an above ground, welded steel tank with a capacity of approximately 1.5 MG. The tank rests on a circular, 3-foot thick concrete foundation and is located directly adjacent to the reservoir access road as well as the Bainbridge Island High School playfields. The property is owned by Bainbridge Island High School but is leased by the City. The reservoir is located on a 0.5-acre, gently sloped, grass and asphalt covered site and is secured via security chain link fencing. The site is surrounded by trees and other facilities used for cellular service are located within the secured area. Electrical service is available at this location and the site is accessed by a 20-foot wide double swing gate.

Reservoir 2 contains two 24-inch manways at the base of the tank and a roof vent located at the top of the tank. The roof is accessed via a ladder equipped with two intermediate platforms and a safety cage. The ladder does not contain a harnessed ascent system and the base of the ladder is restricted with a padlock security gate.

Reservoir 2 floats on the distribution system and is served by a single 12-inch diameter inlet/outlet pipe. The flow to Reservoir 2 is controlled by the setpoints listed above for Reservoir 1, even though Reservoir 2 has a slightly higher overflow value as shown in Table 2-7.

Similar to Reservoir 1, Reservoir 2 has several significant seismic deficiencies, however, the existing coating system has effective service life remaining. *Technical Memorandum 17614-4* recommended that the existing interior and exterior coating system be re-evaluated within 5 years, and that seismic deficiencies be addressed within 3 to 5 years.

CHAPTER 3

SYSTEM ANALYSIS

INTRODUCTION

The requirements for a project report listed in WAC 246-290-110 require that any proposed modifications to a public water system include a water system component analysis to ensure that the modifications are capable of meeting both current and projected future demands. Historical and projected system demands were fully developed in the City's WSP; however, they are summarized below with regards to the proposed reservoir modifications. This chapter also provides analysis with regards to low system pressure at select locations within the distribution system and the potential effects the proposed modifications will have to water quality.

DEMAND

HISTORICAL PRODUCTION

Historical water production and historical water system demand date from the WSP are highlighted in Table 3-1 and 3-2. Water system demand remained fairly consistent over the period with the exception of 2014 in which the production from HOB increased, and in 2015 when the production from Fletcher Bay increased and the production from Sands decreased. This shift in production was to account for high chlorine demand found during an investigation of groundwater from the Sands wells. Production from the Commodore Well has been minimal since 2010. The Winslow water system demand increased slightly in 2015, but has remained relatively consistent since that time.

Additional analysis for system demand and consumption can be found in the WSP.

TABLE 3-1

Bainbridge Island Selected Historical Water Production

Location	2011	2012	2013	2014	2015	2016	2017	2018	2019	Average	2015-2019 Average
HOB (MG)	60.1	54.7	55.3	80.6	87.1	96.0	89.0	84.8	92.9	**77.8**	**90.0**
Sands (MG)	96.1	91.6	101.6	100.2	77.7	80.2	83.1	99.4	104.0	**92.6**	**88.9**
FB (MG)	60.5	76.8	56.9	49.8	81.9	69.4	80.7	85.0	69.1	**70.0**	**77.3**
Commodore (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	**0.0**	**0.0**
Total	**216.7**	**223.1**	**213.8**	**230.6**	**246.7**	**245.6**	**252.8**	**269.2**	**266.1**	**240.5**	**256.1**

TABLE 3-2

Bainbridge Island Historical System Demand

Parameter	2011	2012	2013	2014	2015	2016	2017	2018	2019	Average	2015-2019 Average
ADD (mgd)	0.59	0.61	0.59	0.58	0.64	0.63	0.66	0.67	0.68	**0.62**	**0.66**
MDD (mgd)(1)	1.30	1.37	1.49	-	-	-	-	-	-	**1.33**	**-**
PF (MDD/ADD)(1)	2.18	2.24	2.55	-	-	-	-	-	-	**2.16**	**-**

(1) Data for 2014 through 2019 not available. Average values are for data between 2006 and 2013.

PROJECTED DEMAND

Projected system demand was also highlighted in the WSP, but is summarized below. The projections include a range of growth scenarios, from low growth to high growth.

TABLE 3-3

City of Bainbridge Island Projected Demand

Parameter & Scenario (mgd)	Year		
	2021	2025	2035
ADD – Low	0.83	0.94	1.21
ADD – Med	0.86	0.97	1.24
ADD – High	0.92	1.03	1.31
MDD – Low	1.67	1.88	2.41
MDD – Med	1.86	2.09	2.68
MDD – High	2.20	2.47	3.15

STORAGE

Storage components for the City's water system are fully detailed in the WSP. Additional guidance can be found in the 2019 WSDOH Water System Design Manual (Manual); however, the storage analysis is summarized below.

DEAD STORAGE

Dead storage is the volume of stored water not available to all customers at the minimum design pressure in accordance with WAC 246-290-230(5) and (6). Dead storage is excluded from the volumes provided to meet the other storage requirements. WAC 246-290-230(5) and (6) require that a minimum of 30 psi be maintained system wide under peak hour demand (PHD) conditions (equalization storage depleted) and that 20 psi be maintained system wide under maximum day demand plus fire flow conditions (equalization and fire suppression storage depleted). Because dead storage increases the overall cost for a facility and provides no service benefits, municipalities typically attempt to minimize the volume of dead storage built into any storage project.

Topography within the Winslow Water System varies greatly from a low of 0 feet to a high of approximately 280 feet. Even though the potential for property development within this system includes elevations up to 280 feet along New Brooklyn Road, the City has designated an elevation of 260 feet as the highest elevation at which a meter will be installed. This maximum meter elevation is necessary to avoid costly system modifications required to serve very few residential connections. This elevation is approximately equal to the highest surface elevation of New Brooklyn Road. Any and all development above this elevation will require an individual system analysis to determine whether the minimum pressure requirements set forth by DOH are met, and the City has

determined that any booster stations required to deliver and maintain the required system pressures would be constructed by the property owner/developer.

The City is required to provide the minimum system pressure at the service meter, and if the maximum allowable installation elevation of 260 is utilized, all water stored below an elevation of 306 feet would be considered dead storage. This equates to approximately 659,000 gallons of dead storage currently within Reservoir 1 (66%) and 990,000 gallons of dead storage currently within Reservoir 2 (66%).

OPERATIONAL STORAGE

Operational storage is the volume of the reservoir devoted to supplying the water system while under normal operating conditions. This volume is typically set by operations staff and is dependent upon the reservoir water level sensors and the tank configuration necessary to prevent excessive cycling of well booster pump motors. Operational storage is in addition to other storage components, thus providing a factor of safety for equalizing, standby, and fire suppression components. The City's current operational storage volume is based off of a pump "on" elevation of 324.5 feet, and a pump "off" elevation of 332.5 feet within Reservoir 1. Historically, these setpoints have worked well for the City staff and optimize booster pump stop/start cycles and water quality within the reservoirs and distribution system.

This 8-foot band results in a total operational storage of 231,400 gallons (99,400 gallons for Reservoir 1 and 132,000 gallons for Reservoir 2), which is approximately 9 percent of the total stored volume within both reservoirs.

Given that the City has experienced no issues with the operation, runtime, or performance of the existing well and booster pumps, an operational storage volume of 230,000 gallons will be used for subsequent reservoir sizing.

EQUALIZING STORAGE

Equalizing storage is typically used to meet diurnal demands that exceed the average day and maximum day demands. Water systems must be able to provide PHD at no less than 30 psi at all service connections throughout the distribution system when all equalizing storage is depleted (WAC 246-290-230(5)). The water system must meet this requirement at all existing and proposed service meters.

If the maximum design elevation for providing service pressure of 260 feet is used as described above, the minimum elevation at which water will be available at 30 psi is 329 feet. Given that the elevation of the lower operational storage setpoint for Reservoir 1 is 324.5, and that the equalizing storage component cannot include operational storage, neither Reservoir 1 or Reservoir 2 contain any equalizing storage.

The Manual dictates that equalizing storage be provided according to the equation below:

V_{ES} = (PHD – Q_S) (150 minutes)

V_{ES} = Equalizing storage component (gallons)
PHD = Peak hour demand (gpm)
Q_S = Total source of supply capacity, excluding emergency sources (gpm)

The projected peak hour demand flow for the year 2035 is 3,025 gpm, which was calculated using Equation 3-1 from the Manual, an ERU_{MDD} value of 350 gpd/ERU, the projected number of ERUs in 2035 from the WSP (7,640), a C-factor of 1.6, and an F factor of 225. Given that the total source of supply capacity value for HOB wells 1A, 2, 3, 4, 5, 6, Sands Well 1, and Fletcher Bay Well 1 is estimated to be 1,850 gpm, the resulting required equalizing storage volume is 177,000 gallons. For this analysis, a more conservative combined requirement of 180,000 gallons will be used.

FIRE SUPPRESSION STORAGE

Fire suppression storage is provided to ensure that the volume of water required for fighting fires is available when necessary. The amount of water required for firefighting purposes is specified in terms of flow in gallons per minute (gpm) and duration. Fire flow must be provided while maintaining a residual water system pressure of at least 20 pounds per square inch (psi) throughout the water system. The current available fire suppression / standby storage is calculated as the total volume of water between 20 psi (EL = 306) and the lower operational setpoint (EL = 324.5), and is equal to 543,900 gallons.

The required fire suppression storage is calculated using the following equation:

FSS = (FF)(t_m)

FSS = Required fire suppression storage component (gallons)
FF = Required fire flow rate, as specified by fire protection authority (gpm)
T_m = Duration of FF rate, as specified by fire protection authority (minutes)

Within the Winslow Water System, the highest fire flow requirement exists at Bainbridge Island High School. The fire flow required for the High School as listed in the 2015 International Building Code is 3,000 gpm for 180 minutes, which is equal to 540,000 gallons.

STANDBY STORAGE

Standby storage is provided in order to meet demands in the event of a system failure such as a power outage, an interruption of supply, or a break in a major transmission line. The amount of standby storage is defined in the Manual and is calculated according to the equation below:

$$SB_{TMS} = (N * SB_i * T_d)$$

SB_{TMS} = Standby storage component for a multiple source system (gallons)
N = Number of ERU's based on the ERU_{MDD} method
SB_i = Locally adopted unit SB volume in gallons per day per ERU (number of ERU's based on the ERU_{MDD} value.)
T_d = Number of days selected to meet the water system determined standard of reliability

Although standby storage volumes are intended to satisfy the requirements imposed by system customers for unusual situations and are addressed by WAC 246-290-420, DOH recommends that standby storage volume be at least 200 gallons/ERU.

Additionally, standby storage may be "nested" with fire suppression storage if allowed by the local fire jurisdiction. By nesting the smaller of standby storage and fire suppression storage within the larger of these quantities, municipalities are able to reduce the overall volume of storage required within the system. The City currently nests their standby storage with their fire suppression storage as allowed by the local fire jurisdiction. The City's existing standby storage is calculated as the volume of water available at 20 psi not including operational storage. This equates to all water between elevation 306 and 324.5 feet, or approximately 233,700 gallons for Reservoir 1 and 310,200 gallons for Reservoir 2 (543,900 gallons combined).

To calculate the required standby storage using the equation above, the projected number of ERUs in 2035 (7,640) is multiplied by the maximum day use per ERU (ERU_{MDD} = 350 gpm) and the City's adopted standard of reliability (T_d = 1 day). The City has elected to provide 1 day of service for their standard of reliability (Td). This means that the City is confident that they can restore service to the water system after any interruptions in service within 24 hours. This is reasonable given that the City maintains multiple sources, a complete staff of trained and experienced operations personnel, sufficient equipment to address a significant majority of all potential maintenance issues, and a stable public works department with the infrastructure to swiftly address and remedy all likely disruptions in service.

Using these values from the WSP in the equation above results in a standby storage requirement of 2.674 MG, which is significantly larger than the volume currently provided (540,000 gallons). For comparison, if the minimum DOH recommended

standby storage value of 200 gpd/ERU is used, the recommended standby storage volume is 1.528 MG.

It is noteworthy that the WSP was completed under the guidance of the 2009 Water System Design Manual, which uses a different calculation for Standby Storage. The previous equation is as follows:

$$SB_{TMS} = (2 \text{ days}) * [(ADD * N) - (t_m * (Q_S - Q_L))]$$

SB_{TMS} = Total standby storage component for a multiple source water system (gallons)
ADD = Average day demand for the design year (gpd / ERU)
N = Number of ERUs for the design year
Q_S = Sum of all installed and continuously available supply source capacities, except emergency sources, (gpm)
Q_L = The largest capacity source available to the water system (gpm)
T_m = Time the remaining sources are pumped on the day when the largest source is not available (minutes).

Using the equation above, an ADD value of 152 gpd/ERU, the project number of ERUs in 2035 (7,640), 1,080 minutes (18 hours) per day, a value for Q_S of 1,850 gpm (includes HOB, Fletcher Bay, and Sands sources), and value for Q_L of 650 gpm (Sands source), the calculated required standby storage volume is negative, which indicates that the City is able to meet two times the average day demands using their installed source capacity with the Sands source out of service. This suggests that no standby storage is required for the Winslow Water System.

Providing standby storage per the formulas listed in the 2019 Manual would provide the City with an exceptionally robust and versatile water system; however, providing up to 2.5 MG of additional standby storage is not possible without building an additional storage reservoir, either to augment Reservoir 1 and Reservoir 2, or replace one or both tanks. Replacing Reervoir 1 or 2 with a new, larger tank is likely the most cost effective method to address coating, seismic, and storage deficiencies identified with Reservoir 1.

Because the City has multiple sources, the full standby storage calculated from the 2019 Manual may not be necessary. Since the City has had concerns with stagnation and water quality in the past, the City has elected to design any new storage facility to provide the minimum recommended value of 200 gpd/ERU from the 2019 Manual. This will ensure that sufficient storage is provided for the Winslow Water System and that the City can reliably meet the storage needs of the system in both short- and long-term demand scenarios. Using a value of 200 gpd/ERU and the projected number of ERUs for 2035 (7,640) the standby storage volume utilized for design of the City's storage facilities will be 1,528,000 gallons.

EFFECTIVE STORAGE

The effective storage capacity is the capacity of the reservoir that is reliably available and is capable of being withdrawn from the reservoir at the rates and pressures required for water use purposes. For the purposes of this analysis, effective storage is assumed to be equal to the sum of equalizing, standby, and fire-suppression storage. Effectively, this is equal to all water available at 20 psi below the lower operational elevation of 324.5 feet.

STORAGE ANALYSIS AND CONCLUSIONS

Table 3-4 provides an analysis of the existing and required storage values for both 2019 and 2035. As shown in Table 3-4, the City maintains a total effective storage volume of 535,500 gallons for the Winslow Water System. The system shows a deficit in both 2019 and 2035. These values do not correlate well with the values listed in the WSP; however, the larger deficits reflect the City's desire to include the minimum recommended standby storage volume of 200 gpd/ERU per the 2019 Manual.

The alternatives listed in Chapter 4, and the proposed modifications listed in Chapter 5 will all address this storage deficit. The selected recommendation should also provide sufficient water storage in a manner that helps ensure good water quality (both in the reservoir and distribution system), minimizes dead storage, and provides adequate service pressure to all elevations at or above 260 feet.

TABLE 3-4

Existing Reservoir Storage Summary

Parameter	Equalizing Storage Required (gallons)	Standby Storage Required (gallons)	Fire Suppression Storage Required (gallons)	Total Required Storage (gallons)	Total Effective Storage (gallons)(1)	Surplus / Deficit (gallons)
Year 2019(2)	43,000	1,060,000	540,000	1,103,000	535,500	(567,500)
Year 2035	180,000	1,528,000	540,000	1,708,000	535,500	(1,172,500)

(1) For both Reservoir 1 and Reservoir 2.
(2) Includes an estimated 5,300 ERUs per the current WSP

LOW SYSTEM PRESSURE

In recent years, the City has received complaints of low water pressure in the area immediately west of Reservoirs 1 and 2 (Commodore Lane and Capstan Drive) as well as select areas along New Brooklyn Road (Mandus Olson Road and Grizdale Lane). This is because these residences are located at higher elevations and when the water level within Reservoirs 1 and 2 drops, the pressure available to these locations is reduced. These areas are identified by the orange topographical lines shown in Figure 2-1.

As noted in Table 3-3, the well pump call elevation for Reservoirs 1 and 2 is 324.5 feet (water height of 71.5 feet), which represents the bottom of the operational storage component. The equalizing storage component must also be provided at 30 psi, which is equivalent to 70 feet of head. The highest elevation for new meter installation as adopted by the City is 260 feet as described previously. At a water surface elevation of 324.5 feet, all elevations above 254.5 feet will not be supplied with the required 30 psi. The City would like to address this pressure shortcoming as part of the reservoir improvements.

While increasing system pressure to high elevation services within the High Zone is a primary goal for the project, any modifications will also need to include an analysis on effects to lower elevations within the High Zone. While an increase in system pressure is typically beneficial to its customers, the Manual states that water system pressure should not exceed 100 psi, except under qualifying circumstances. Above 80 psi, the Uniform Plumbing Code (UPC) requires pressure reducing valves and expansion tanks to be installed on individual services.

The alternatives considered in Chapter 4 strive to address the pressure issues described above and include an analysis of the effects on both high and low service elevations within the High Zone.

WATER QUALITY

Water quality for each of the three groundwater sources was discussed in Technical Memorandum 17614-1 (*Gray & Osborne, 2018*). One issue identified in the memorandum that is affected by storage is high chlorine demand identified in groundwater from the Sands Wellfield.

Previous analysis (*Gray & Osborne, 2003*) identified that water from the Sands Wellfield has significantly higher chlorine demand when compared to either Fletcher Bay or Head of the Bay. High chlorine demand is most significant when coupled with long retention times in a distribution system or reservoir. If water with high chlorine demand is stored in a reservoir with long residence times, the water within the reservoir has the potential to develop low chorine residuals. If water system demand suddenly increases this low-residual water can then enter the distribution system and leave the system susceptible to violation of WAC 246-290- 451 (7)(b) under the definition of a minimum detectable residual as listed in WAC 246-290-010 (80).

Each of the alternatives described below attempts to minimize the effects of this large chlorine demand and improve water quality within the reservoir and distribution system. A brief summary of the effects on water quality for the recommended alternative is provided in Chapter 5.

CHAPTER 4

SYSTEM ALTERNATIVES

In order to address storage deficiencies for the Winslow water system, the potential for low system pressures at higher elevations along New Brooklyn Road and at Commodore Lane, and seismic deficiencies with Reservoir 1 identified in *Technical Memorandum 17614-4*, the City is interested in making modifications and/or improvements to the Reservoir 1. Potential alternatives for accomplishing these goals are described below.

WATER SYSTEM ALTERNATIVES

The City has established addressing seismic deficiencies, and water storage deficiencies, and areas susceptible to low water system pressure as primary goals for this project. Several alternatives were considered in Memorandum 17614-5 (*Gray & Osborne, 2018*).

ALTERNATIVE 1: NEW RESERVOIR SERVES NEW 351 ZONE AND NEW BOOSTER STATIONS SERVE NEW BROOKLYN ZONE

This alternative includes the creation of two new pressure zones: the New Brooklyn Zone and the Commodore Zone, and includes the construction of a new welded steel reservoir and two booster stations. A proposed taller welded steel reservoir would replace existing Reservoir 1 to serve a new, higher Commodore Pressure Zone near the reservoir site. The design criteria for this new reservoir are summarized in Table 4-1. Reservoir 2 would continue to serve the remaining High and Low Zones, but some volume of the proposed Reservoir 1 would be available for the High and Low Zones through PRVs to augment Reservoir 2 storage.

TABLE 4-1

Alternative 1 – Proposed New Reservoir Design Criteria

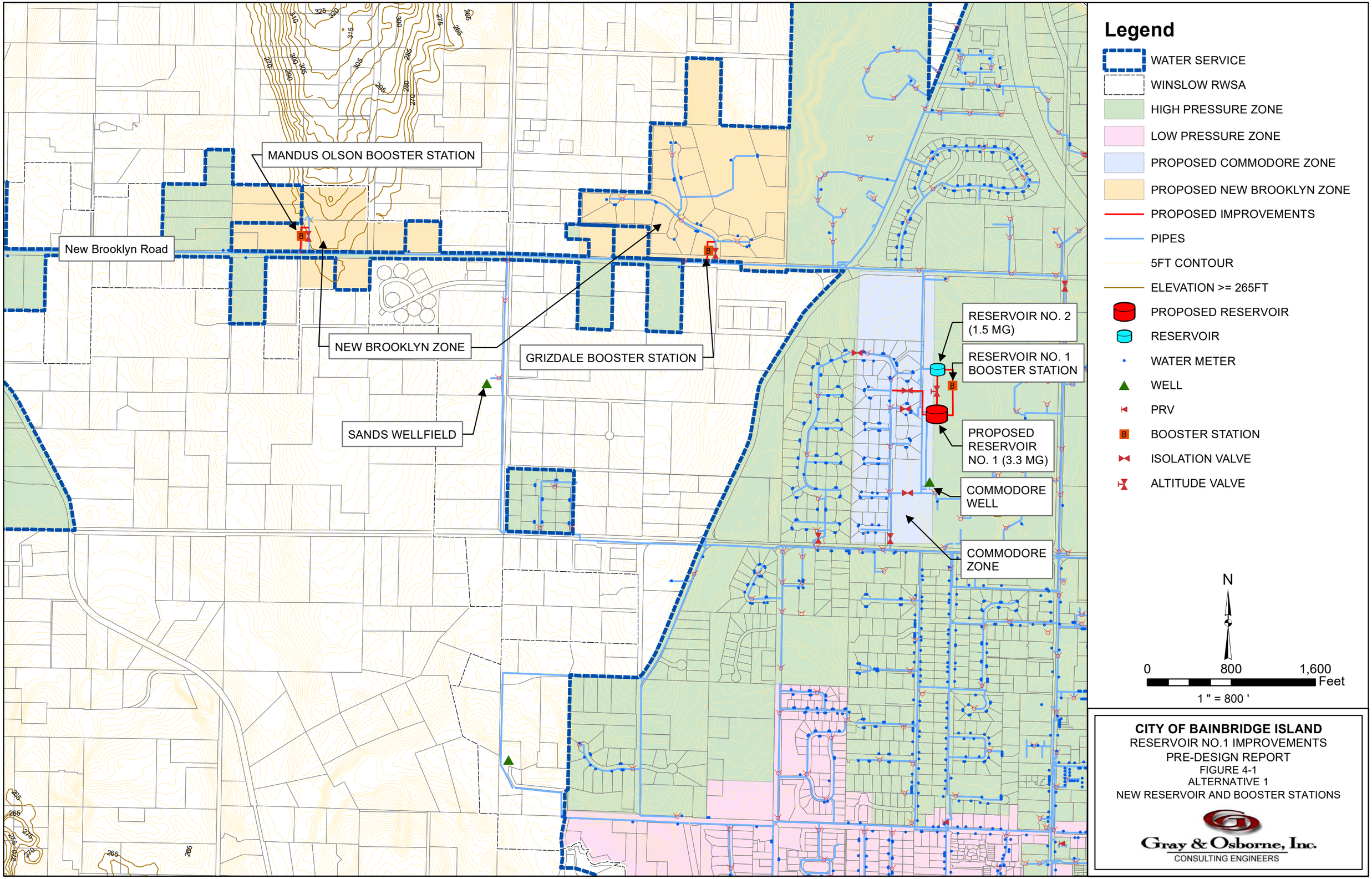
Parameter	Value
Type	Circular, Welded Steel
Location	Existing Reservoir 1 Location
Diameter (ft)	70
Sidewall Elevation (ft)	379.0
Base Elevation (ft)	253.0
Overflow Height (ft)	116.0
Overflow Elevation (ft)	369.0
Nominal Volume (MG)	3,314,000
Volume per Foot (gal/ft)	28,817
Effective Storage (gal)	1,700,200(1)
Inlet	Top, 12-inch; Fed from Reservoir 1 Booster Station
Outlet	Bottom, 12-inch; Connects to Commodore Zone Distribution System Piping

(1) 285,000 gallons is designed for use within the Commodore Zone while the remaining volume is available for the entire Winslow Water System High and Low Zones.

The proposed Commodore Zone, located as shown on Figure 4-1, would be served directly through a connection to a new Reservoir 1 and would have an HGL of 369 feet. A dedicated outlet would connect with existing distribution system piping, and appropriate valves and appurtenances would be added to isolate this zone from the remaining High Zone.

The proposed reservoir would provide adequate fire flow for the proposed Commodore Zone, which includes only single-family residences. The fire flow requirement for this Zone is 180,000 gallons (1,500 gpm for 120 minutes). Standby storage was calculated by multiplying the estimated maximum number of service ERUs in the proposed Commodore Zone (300) by the minimum recommended ERU_{MDD} value of 200 gpd/ERU. This results in a standby storage requirement for the Commodore Zone of 60,000 gallons which is nested with fire suppression storage per the City's standard protocols. Equalizing storage was calculated using a peak hour demand of 8.5 gpm/ERU and a maximum installed source capacity of 1,850 gpm. The estimated equalizing storage requirement is thus 105,000 gallons.

The New Brooklyn Zone would be served by two new booster stations connected to the existing High Zone along New Brooklyn Road and would have an HGL of 360 feet. The proposed booster stations would connect to the existing distribution system piping, would provide additional service pressure to the New Brooklyn Zone, and would be provided with isolation valves to ensure that only the target area is served. The proposed booster stations would likely consist of two small booster pumps (one duty, one standby) that could each meet average demands. Each of these booster stations would also be provided



CITY OF BAINBRIDGE ISLAND
RESERVOIR NO.1 IMPROVEMENTS
PRE-DESIGN REPORT
FIGURE 4-1
ALTERNATIVE 1
NEW RESERVOIR AND BOOSTER STATIONS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

L:\Bainbridge Island\19648 - Reservoir 1 Improvements Pre-Design Report\GIS\FIG.4-1 ALTERNATIVE 1.mxd

with permanent auxiliary power supplies. Fire flow for these residential areas would be equal to 1,500 gpm for 120 minutes at a minimum of 20 psi. This fire flow would be provided by the existing High Zone distribution system piping. The proposed pumps, valves, and controls could be located inside a CMU, wood, or metal building which would include lighting, heating and ventilation, electrical service, alarms, and access doors. These booster stations would be owned and operated by the City. Control for this booster station would be pressure/demand based, and the proposed booster pumps would run on VFD motor starters. If the pressure drops below an operator-selectable set point, then the pumps would energize in order to provide additional pressure. When demand is low, the pump motor speed would decrease to its minimum recommended value and maintain that speed until the zone pressure set point is reached. Once this set point is reached, pump speed would steadily increase until pressure was restored. Pressure in the zone would be measured via redundant pressure transmitters and displayed on the City's central HMI/SCADA system for monitoring purposes.

The proposed Reservoir 1 would be filled by a dedicated booster station that would move water from Reservoir 2 to Reservoir 1. The Reservoir 1 Booster Station will consist of two pumps (one duty/one standby). The pumps would be housed in a CMU building located near Reservoir 2. The enclosure would include lighting, heating and ventilation, electrical service, alarms, access doors, and would be installed on a concrete slab. The operation of these pumps would be controlled by the level in Reservoir 1 – either by an ultrasonic level sensor or submersible pressure transmitter. A permanent diesel generator would also be installed to serve the Reservoir 1 Booster Station. The addition of this booster station will help improve water quality within both Reservoirs 1 and 2 by improving water turnover and mixing within both tanks.

Reservoir 2 would remain in service as part of this alternative and would continue to provide service to the High Zone, as well as water to fill Reservoir 1. The well controls currently installed in Reservoir 1 would be relocated to Reservoir 2. A storage analysis for the Winslow Water System under this alternative is provided in Table 4-2 and shows a storage surplus in 2019 and in 2035. As designed, the new Reservoir 1 will provide adequate storage for the new Commodore Zone as well as augment Reservoir 2 such that the two reservoirs provide adequate storage for the entire Winslow Water System.

TABLE 4-2

Alternative 1 – Storage Analysis

Parameter	Equalizing Storage Required (gallons)	Standby Storage Required (gallons)(1)	Fire Suppression Storage Required (gallons)	Total Required Storage (gallons)	Total Effective Storage (gallons)(2)	Surplus/ Deficit (gallons)
Commodore Zone						
Year 2019	105,000	60,000	180,000	285,000	2,005,820(1)	1,720,820
Year 2035	105,000	60,000	180,000	285,000	2,005,820(1)	1,720,820
High Zone						
Year 2019	43,000(2)	1,060,000(3)	540,000	1,103,000	2,005,820(1)	902,820
Year 2035	180,000(4)	1,528,000(3)	540,000	1,708,000	2,005,820(1)	297,820

(1) Includes both Reservoir 2 and proposed Reservoir 1.
(2) Calculated using an ERU_{MDD} value of 350, an estimate of 5,300 ERUs, a C-factor of 1.6, an F-Factor of 225, and a total source of supply capacity of 1,850 gpm.
(3) Calculated using an estimated value of 5,300 ERUs (2019), 7,640 ERUs (2035), and a value of 200 gpd/ERU.
(4) Calculated using an ERU_{MDD} value of 350, an estimate of 7,640 ERUs, a C-factor of 1.6, an F-Factor of 225, and a total source of supply capacity of 1,850 gpm.

The estimated construction cost for Alternative 1 is $14,175,000. A budgetary construction cost estimate for this alternative is provided in Exhibit C. A summary of project components is as follows:

- 3.3 MG Welded Steel Reservoir;
- All Required Sitework and Appurtenances;
- Connection to the Existing System;
- Reservoir 1 Booster Station;
- New Brooklyn Booster Stations (2x);
  - One duty pump, one standby pump;
  - A 800-square-foot wood enclosure with concrete pad and backup generator.
- Additional Isolation Valves, Blowoff/Pressure Relief Assembly;
- Pressure Monitoring Equipment;
- Electrical, Telemetry, and Integration;
- Contingency (30 percent);
- Washington State Sales Tax (9.0 percent);
- Design and Construction Administration (25 percent).

Notable exclusions from the cost estimate above are acquisition of additional property and/or right of way. This alternative assumes that the proposed pump stations will be constructed within City right-of-way; however, depending on the location for these

booster stations, additional property or right or way may be required, the cost of which is unknown at this point in time. This alternative assumes that the existing electrical service at each proposed booster station location is adequate to support the installation of up to two new pumps. Lastly, the seismic upgrades to Reservoir 2 that were recommended in Technical Memorandum 17614-4 are not included in the costs above or the budgetary cost estimate in Appendix C. The seismic upgrades for Reservoir 2 are estimated to cost approximately $1,000,000, which does not include reservoir coating work, recommended access/inspection improvements, tax, contingency, or project administration. It is important to note that Reservoir 2 would need to be utilized in this alternative in order to meet the overall system water storage requirements; however, the seismic, coating, and access/inspection improvements could be completed as a separate project in the next 3 to 7 years.

Because the proposed welded steel reservoir contains a large volume of dead storage and serves only one small residential zone, it may be susceptible to low turnover and/or higher water age. A mechanical mixer or more frequent operation of Reservoir 1 Booster Station may be effective in circulating the water and providing consistent water quality from the reservoir. Alternatively, the inlet piping could be fitted with a passive mixing manifold which would provide circulation of the reservoir contents when the Reservoir 1 Booster Station is in operation. Furthermore, the top inlet-bottom outlet piping orientation will also help improve water turnover within the reservoir, which should help provide consistent water quality within the distribution system. Additional systems such as supplemental chlorine injection equipment may also be required, and these supplementary systems are not included in the cost estimate at this point in time.

The primary advantage of Alternative 1 is that it provides a simple, localized solution to low distribution system pressures only in the areas of concern and minimizes the impact to the remaining High Zone. It also addresses the seismic and storage deficiencies identified with Reservoir 1. Some disadvantages to this alternative are that it requires three new booster stations which will increase the operational cost and complexity of the water system, increases the volume of dead storage within the water system, and requires additional equipment/operational changes to ensure consistent and high water quality in the distribution system.

ALTERNATIVE 2: NEW WELDED STEEL RESERVOIR SERVES EXISTING HIGH ZONE WITH NEW HGL

Alternative 2 will address storage and pressure issues through the construction of a new, taller, welded steel reservoir to provide service to the existing High Zone while increasing the zone's HGL

Design criteria for the proposed reservoir are provided in Table 4-3.

TABLE 4-3

Alternative 2 – Welded Steel Reservoir Design Criteria Summary

Parameter	Value
Type	Circular, Welded Steel
Location	Existing Reservoir 1 Site
Diameter (ft)	70
Sidewall Height (ft)	131.0
Base Elevation (ft)	253.0
Overflow Height (ft)	121.5
Overflow Elevation (ft)	374.5
Nominal Volume (MG)	3,472,450
Volume per Foot (gal/ft)	28,817
Operational Storage (gal)	230,540
Equalizing Storage (gal)	187,310
Fire Flow/Standby Storage (gal)	1,527,300
Dead Storage (gal)	1,527,300
Effective Storage (gal)	1,714,610
Inlet/Outlet	12 inch; common inlet/outlet with passive mixing manifold
Additional High Zone Pressure (psi)	17.4

A 70-foot diameter was selected due to site constraints and the need to maintain access around the full perimeter of the reservoir. The new reservoir is designed to be taller than the existing Reservoir 1 to raise the HGL of the High Zone to eliminate the existing pressure problems and provide sufficient storage. Consequently, all service connections within the current High Zone will be subject to higher system pressure. While some additional pressure may be acceptable, too much pressure will likely require that additional pressure reducing valves be installed to bring the water pressure below the DOH recommended maximum value of 100 psi. For the purposes of this analysis, it is assumed that two additional pressure reducing valve (PRV) stations will be required to address large areas within the Winslow Water System susceptible to pressures greater than 100 psi. This equipment would be installed on selected distribution system water main piping within buried vaults within the City right-of-way and will include hydraulic control valves that reduce the pressure of water for all downstream connections. In general, services that will be susceptible to these higher pressures are located in the northeast regions of the Winslow system as well as the Fletcher Bay area, are connected to the existing High Zone, and have service meters located at elevations less than approximately 115 feet.

The proposed reservoir contains a large volume of dead storage, which can negatively impact turnover and water quality within a reservoir and, subsequently the distribution system. A mechanical or passive mixing system may help improve water quality within

these taller reservoirs. Several mixing alternatives were identified and analyzed in Technical Memorandum 17614-6 *(Gray & Osborne, 2018).*

Raising the HGL of the existing High Zone will also have an impact on the existing well booster pumps. If a new taller reservoir is constructed, the static head on the existing booster pumps will increase, which would decrease the flow from these pumps to the new reservoir. Table 4-4 summarizes the head and flow conditions from the existing booster pumps and estimates the impact to projected flows.

TABLE 4-4

Alternative 2 – Existing Booster Pump Flow Analysis Summary

Location	Current Max. Water Elevation (ft)	Current Design Head (ft)	Current Flow (gpm)(1)	Proposed Max. Water Elevation (ft)	Estimated Flow (gpm)	Potential Flow Reduction (gpm)
Reservoir A						
Fletcher Bay	332.5	265	580	373.5	420	160
HOB 1	332.5	380	620	373.5	280	340
Sands 2	332.5	307	650	373.5	580	70
Total Estimated Reduction in Flow (gpm)						**570**
Reduction in Flow (Percent)						**30**

(1) Flow and head data were as listed in the WSP. It should be noted that some flow and head measurements significantly exceeded the pump curves provided for this analysis.

Lastly, the analysis above and cost estimates provided below assume that the Winslow Water System would be served solely by the new Reservoir 1 since the hydraulic gradeline of the new zone will be higher than Reservoir 2 could accommodate. Reservoir 2, which also exhibited seismic deficiencies and coating fatigue described in Technical Memorandum 17614-4 could be removed from service. The City may elect to demolish the existing reservoir, decommission the reservoir but leave it empty in the event that Reservoir 1 must be taken out of service for cleaning/maintenance, or may continue to operate the reservoir to provide additional system capacity and fire flow. If Reservoir 2 is demolished, it reduces the level of redundancy available to the City, and accommodations should be made for when Reservoir 1 must be taken offline or drained for recoating. During this period, which can often last between 3 and 9 months, the existing well booster pumps must be able to provide enough flow and pressure to the distribution system during both peak/maximum demands as well as during fire flow situations. If Reservoir 2 remains in service, the inlet/outlet piping should be outfitted with an altitude valve so that it does not overflow from the resulting higher HGL in Reservoir 1. Furthermore, the turnover within Reservoir 2 may decrease due to the higher HGL of Reservoir 1. As such, a small booster station, mechanical mixer, or chlorine booster system may be warranted in order to provide consistent water quality

from Reservoir 2 to the distribution system. Given that the Reservoir exhibits seismic deficiencies, would not be required to provide additional storage volume to serve the Winslow System, and it's use would complicate the operation of the High Zone and require additional pressure or chlorine boosting facilities, we recommend that if Alternative 2 is implemented, Reservoir 2 be removed from service but maintained for use in order to provide system storage redundancy in the event that the proposed Reservoir 1 must be removed from service.

The estimated construction cost for Alternative 2 (Reservoir B) is $11,174,000. A budgetary construction cost estimate is provided in Appendix C. A summary of project components is as follows:

- A 3.47 MG welded steel reservoir;
- All required sitework and appurtenances;
- Two pressure reducing valve stations;
- Connection to the existing system;
- Additional isolation valves, blowoff/pressure relief assembly;
- Pressure monitoring equipment;
- Electrical, telemetry, and integration;
- Contingency (30 percent);
- Washington State sales tax (9.0 percent);
- Design and construction administration (25 percent).

The cost above does not include the acquisition of additional property or right of way as the proposed facilities could be constructed on existing City property. The estimate also does not include seismic upgrades to Reservoir 2 as described in Alternative 1. If the City elects to maintain Reservoir 2 as permanent redundant storage, we recommend that these improvements be completed as a separate project in the next 3 to 7 years.

The primary advantage of Alternative 2 is that it addresses the seismic and storage deficiencies identified with Reservoir 1 without the requirement for additional booster pump stations. The primary disadvantages to this alternative are that it increases the pressure of the existing High Zone and will require additional pressure reducing stations, it may require modifications to the existing well booster pumps, and it does not reduce the overall volume of dead storage.

ALTERNATIVE 3: NEW ELEVATED STORAGE RESERVOIR SERVES EXISTING HIGH ZONE WITH NEW HGL

This alternative is analogous to Alternative 2, but would utilize an elevated storage tank instead of a welded steel reservoir to minimize dead storage. The proposed elevated storage tank would be connected to the existing inlet/outlet piping, and as a result would raise the HGL of the existing High Zone similar to Alternative 2.

A Hydropillar® is one common example of an elevated storage tank. Hydropillars or spheroid tanks are common in the central United States where topography is typically very flat; however, there are elevated storage tanks in use within the Puget Sound region (Poulsbo, Lacey, Seattle, etc.). These structures have a slender central column which supports the main storage body at the desired elevation and have very little dead storage. Additionally, the space immediately around the central support column can be available for storage, administrative, pumping, or other uses.

Table 4-5 highlights the design criteria for a Hydropillar that would meet the City's current and projected storage needs.

TABLE 4-5

Alternative 3 – Elevated Storage Tank Design Criteria Summary

Parameter	Value
Type	Elevated Storage; Hydropillar
Location	Existing Reservoir 1 Location
Base Column Diameter (ft)	66
Top Tank Diameter (ft)	75
Top Tank Sidewall Elevation (ft)	347.0
Base Elevation (ft)	253.0
Overflow Height (ft)	94.0
Overflow Elevation (ft)	347.0
Nominal Volume (MG)	2.0
Volume per Foot (gal/ft)	49,970
Operational Storage (gal)	249,850
Equalizing Storage (gal)	199,900
Fire Flow/Standby Storage (gal)	1,549,070
Effective Storage (gal)	1,998,800
Dead Storage (gal)	<1,000
Inlet/Outlet	12-inch
Additional High Zone Pressure (psi)	5.6

The new reservoir is designed to be taller than the existing Reservoir 1 to raise the HGL of the High Zone to eliminate the existing pressure problems and provide sufficient storage. All service connections within the current High Zone will be subject to higher system pressures. Similar to Alternative 2, PRV stations will be required to reduce the pressure within certain portions of the High Zone down below 100 psi. For the purposes of this report we will assume that two additional PRV stations are required and will be installed as described in Alternative 2.

As seen in Table 4-5, because the water is stored at elevation, the volume of dead storage within the proposed Reservoir 1 is minimal, which should help ensure consistent water quality both within the tank and within the distribution system.

Similar to Alternative 2, the proposed reservoir may have an impact on the flow from each of the existing well booster pumps. Table 4-6 summarizes this potential impact and shows that the new reservoir has the potential to reduce flows by up to 7 percent.

TABLE 4-6

Alternative 3 – Existing Booster Pump Flow Analysis Summary

Location	Maximum Water Elevation (feet)	Current Design Head (feet)	Current Flow (gpm)(1)	Proposed Maximum Water Elevation (feet)	Estimated Flow (gpm)(2)	Potential Flow Reduction (gpm)
Fletcher Bay	332.5	265	580	346.0	565	15
HOB 1	332.5	380	620	346.0	530	90
Sands 2	332.5	307	650	346.0	627	23
Total Estimated Reduction in Flow (gpm)						**128**
Reduction in Flow (Percent)						**7**

This potential reduction in flow is acceptable given the storage surplus provided by the proposed reservoir as shown in Table 4-7.

TABLE 4-7

Alternative 3 – Winslow Water System Reservoir Storage Analysis

Parameter	Equalizing Storage Required (gallons)	Standby Storage Required (gallons)	Fire Suppression Storage Required (gallons)	Total Effective Storage Required (gallons)	Total Effective Storage (gallons)	Surplus / Deficit (gallons)
Year 2019	43,000	1,060,000	540,000	1,103,000	1,998,800	895,800
Year 2035	180,000	1,528,000	540,000	1,708,000	1,998,800	290,800

Table 4-7 shows that the proposed reservoir provides suitable effective storage to provide storage capacity through the year 2035 for the Winslow Water System.

Lastly, the analysis above and cost estimates provided below assume that the Winslow Water System would be served solely by the new Reservoir 1. Reservoir 2, which also exhibited seismic deficiencies and coating fatigue described in Technical Memorandum 17614-4 could be removed from service, could be demolished, or could be utilized to provide additional storage capacity. Given that the Reservoir exhibits seismic deficiencies, would not be required to provide additional storage volume to serve the

Winslow System, and it's use would complicate the operation of the High Zone and require additional pressure or chlorine boosting facilities, we recommend that if Alternative 3 is implemented, Reservoir 2 be removed from service but maintained for use in order to provide system storage redundancy in the event that the proposed Reservoir 1 must be removed from service.

The estimated construction cost for Alternative 3 is $10,667,000. A budgetary construction cost estimate is provided in Exhibit C. A summary of project components is as follows:

- A 2.0 MG hydropillar;
- All required sitework and appurtenances;
- Two pressure reducing valve stations;
- Connection to the existing system;
- Additional isolation valves, blowoff/pressure relief assembly;
- Pressure monitoring equipment;
- Electrical, telemetry, and integration;
- Contingency (30 percent);
- Washington State sales tax (9.0 percent);
- Design and administration (25 percent).

The cost above does not include the acquisition of additional property or right of way as the proposed facilities could be constructed on existing City property. The cost also does not include the seismic upgrades to Reservoir 2 as described in Alternative 1. If the City elects to maintain Reservoir 2 as redundant storage, we recommend that these improvements be completed as a separate project in the next 3 to 7 years.

The primary advantage of Alternative 3 is that it addresses the seismic and storage deficiencies identified with Reservoir 1 without the need for additional booster pump stations while minimizing dead storage. The primary disadvantage to this alternative is that it increases the pressure of the existing High Zone and will require additional pressure reducing stations.

ALTERNATIVES ANALYSIS

Three unique alternatives were presented above, each with distinct advantages and disadvantages, and are summarized in Table 4-8.

TABLE 4-8

Reservoir 1 Improvements Alternative Summary

Alternative No.	Description	Capital Cost	Advantages	Disadvantages
1	New Pressure Zone with Reservoir and Booster Station	$14,175,000	• Small and localized • Increases in pressure are targeted toward problem areas only • Meets long-term storage requirements	• Multiple booster stations add complexity • May require acquisition of property • Low reservoir turnover may negatively impact Commodore Zone • Requires continued use of Reservoir 2, which requires seismic upgrades
2	New HGL for High Zone – Welded Steel Tank	$11,174,000	• Meets long-term storage requirements • Minimizes dead storage	• Large volume of dead storage • Potential impacts to booster pumps • Requires PRV stations
3	New HGL for High Zone – Elevated Storage Tank	$10,667,000	• Meets long-term storage requirements • Minimizes dead storage	• Potential impacts to booster pumps • Requires PRV stations

To determine which alternative the City may wish to pursue, a decision matrix is a useful tool that will weigh the City's critical factors and rank each alternative according to these factors. The decision matrix shown in Table 4-9 below rates each alternative according to the following factors:

- Capital Cost
  - Includes full construction cost, contingency, tax, and project design and administration.
- Operational Cost
  - Includes costs for operation as well as coating for reservoirs.
- Constructability
  - Includes factors such as space, property acquisition, and location.
- Complexity
  - Includes factors such as operator knowledge, operator level of effort, reliance on programming/SCADA, and dependence on specific valves or operational equipment
- Environmental Impact
  - Includes land disturbance and resurfacing requirements
- Water Quality Impact
  - Addresses water quality by minimizing dead storage and promoting water turnover within the tank and the distribution system

The matrix includes a weighting factor to allow some criteria to be given more weight than others. Each alternative has been rated on each criterion based on a rating system from 1 (least favorable) to 10 (most favorable). The rating for each criterion is multiplied by the weighting factor to develop a score for each criterion. The scores for each criterion are then summed for each alternative.

TABLE 4-9

City of Bainbridge Island Alternatives Decision Matrix

Criteria	Weighting Factor	Alternative 1 New Pressure Zones with Reservoir and Booster Station		Alternative 2 New HGL for High Zone with Welded Steel Tank		Alternative 3 New HGL for High Zone with Elevated Tank	
		Rating	Score	Rating	Score	Rating	Score
Capital Cost	35	8	280	6	210	5	175
Operational Cost	20	3	60	7	140	7	140
Constructability	10	4	40	6	60	6	60
Complexity	15	4	60	9	135	9	135
Environmental Impact	10	6	60	8	80	8	80
Water Quality Impact	10	5	50	4	40	9	90
Total	**100**	**-**	**550**	**-**	**665**	**-**	**680**
Rank	**-**	**-**	**3**	**-**	**2**	**-**	**1**

The decision matrix suggests that Alternative 3, serving the existing High Zone with a higher HGL using an elevated storage tank most successfully addresses the City's selected criteria. As such, we recommend that the City proceed with Alternative 3 and construct a new elevated storage tank that will address both seismic and storage deficiencies, will minimize the impacts to the existing High Zone service pressure, and will provide additional water quality benefits by minimizing dead storage. Furthermore, we recommend that the City remove Reservoir 2 from service, but maintain the reservoir as part of the water system in order to provide redundancy in the event that the proposed elevated tank must be removed from service for maintenance, cleaning, or coating.

WATER QUALITY ANALYSIS

Historically, the City has had isolated and sporadic issues with low chlorine residual in specific areas of the distribution system; however, they have always maintained a residual of 0.2 mg/L. Per WAC 246-290-662, this is now defined as the minimum required chlorine residual in order to maintain compliance. The City has addressed this issue in recent years through appropriate blending of the water sources and modifications to the system's operation. We do not anticipate any significant impacts to water quality as a result of this project, but accommodations will be made to the proposed modifications that will allow for easy installation of additional mixing systems or chlorine addition facilities that will address potentially low chlorine residuals within the tank and distribution system.

Although the completion of this project should improve water quality and limit the potential for low chlorine residual water held within the Reservoir to enter the distribution system, the City may wish to design the system to accommodate a future chlorine booster system and/or reservoir mixing system. In any case, the addition of this chlorine equipment should not affect the pH, temperature, or other raw water characteristics, and as such, we do not anticipate any negative affects to corrosivity or reactivity within the distribution system.

CHAPTER 5

RECOMMENDATIONS AND DESIGN CRITERIA

RESERVOIR 1 MODIFICATIONS

A description of the proposed improvements was provided in Chapter 4 (Alternative 3), but is summarized below.

Proposed improvements will include the following items:

- Removal and wastehauling of existing Reservoir 1
    - During execution of this work, Reservoir 2 will provide service to the entire Winslow Water System
    - During this period, the operational levels of Reservoir 2 will be increased to utilize all of the available storage volume (additional 1.5 feet).
- Modifications to, and relocation of, to existing piping at the Reservoir 1 site
    - This will include relocating existing piping to provide space and accommodations for the new tank foundation and appurtenances.
- Replace existing site security fencing
- Construction of a new Hydropillar elevated storage tank
    - A site plan, along with a tank plan and section are shown in Figures 5-1 and 5-2, respectively.
    - Design criteria for the storage tank is summarized in Table 5-1
- Installation of 2 pressure reducing valve stations
    - Final location of stations will be determined from complete Winslow Water system map. Stations will include isolation and pressure reducing valves and will be located within a vault in City right of way if possible.

- Remove Reservoir 2 from service, but maintain the tank so that it can provide redundancy in the event that the proposed Reservoir 1 must be removed from service for maintenance.

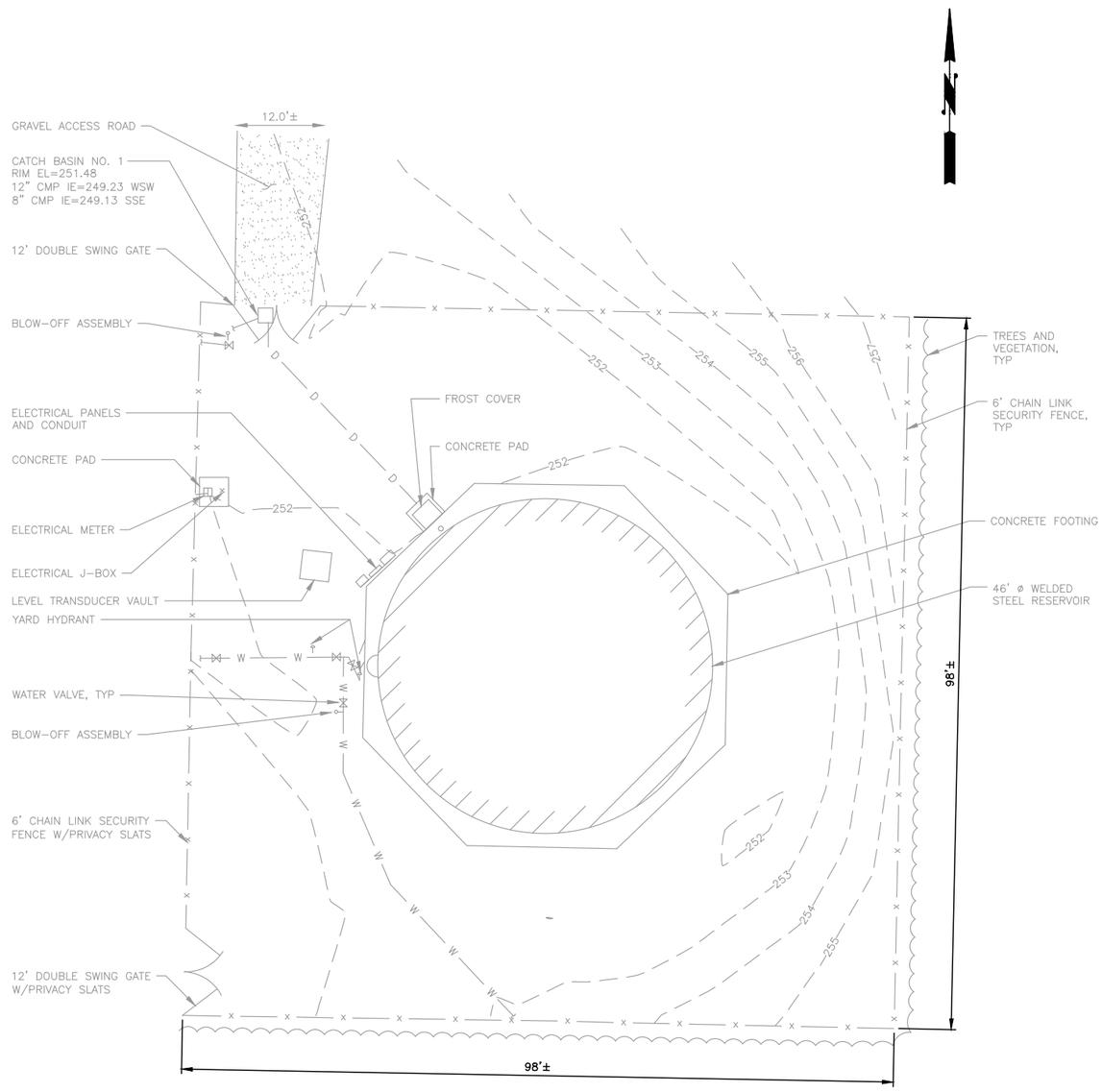
DESIGN CRITERIA

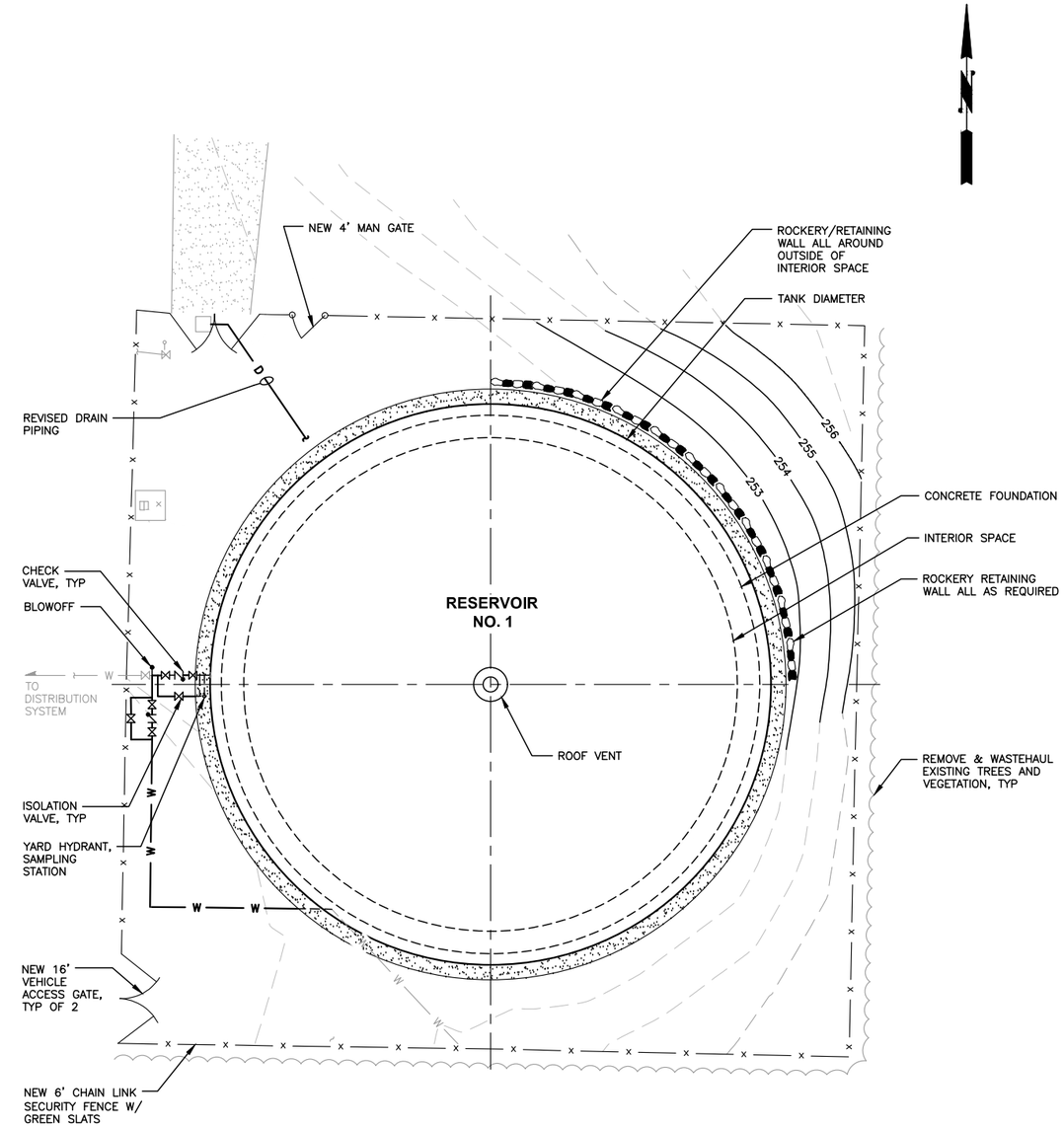
Design criteria for the proposed elevated storage tank and associated booster station are provided in Table 5-1.

TABLE 5-1

Proposed Storage Tank Design Criteria Summary

Parameter	Value
Proposed Reservoir 1	
Type	Elevated, Hydropillar
Location	Existing Reservoir 1 Location
Material	A36 Carbon Steel
Base Column Diameter (ft)	66
Top Tank Diameter (ft)	75
Head Range (ft)	40.0
Top Tank Sidewall Elevation (ft)	347.0
Base Elevation (ft)	253.0
Overflow Height (ft)	94.0
Overflow Elevation (ft)	347.0
Nominal Volume (MG)	2.0
Volume per Foot (gal/ft)	49,970
Operational Storage (gal)	249,850
Equalizing Storage (gal)	199,900
Fire Flow/Standby Storage (gal)	1,549,070
Dead Storage (gal)	<1,000
Effective Storage (gal)	1,998,800
Inlet	12-inch
Level control	Pressure transducers
Alarm control	Float switch(es)
Alarms	Low Low-low High High-high





0 1" 2"

TWO INCHES AT FULL SCALE.
IF NOT, SCALE ACCORDINGLY

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS
PRE-DESIGN REPORT

FIGURE 5-1

EXISTING AND PROPOSED RESERVOIR 1 SITE PLAN

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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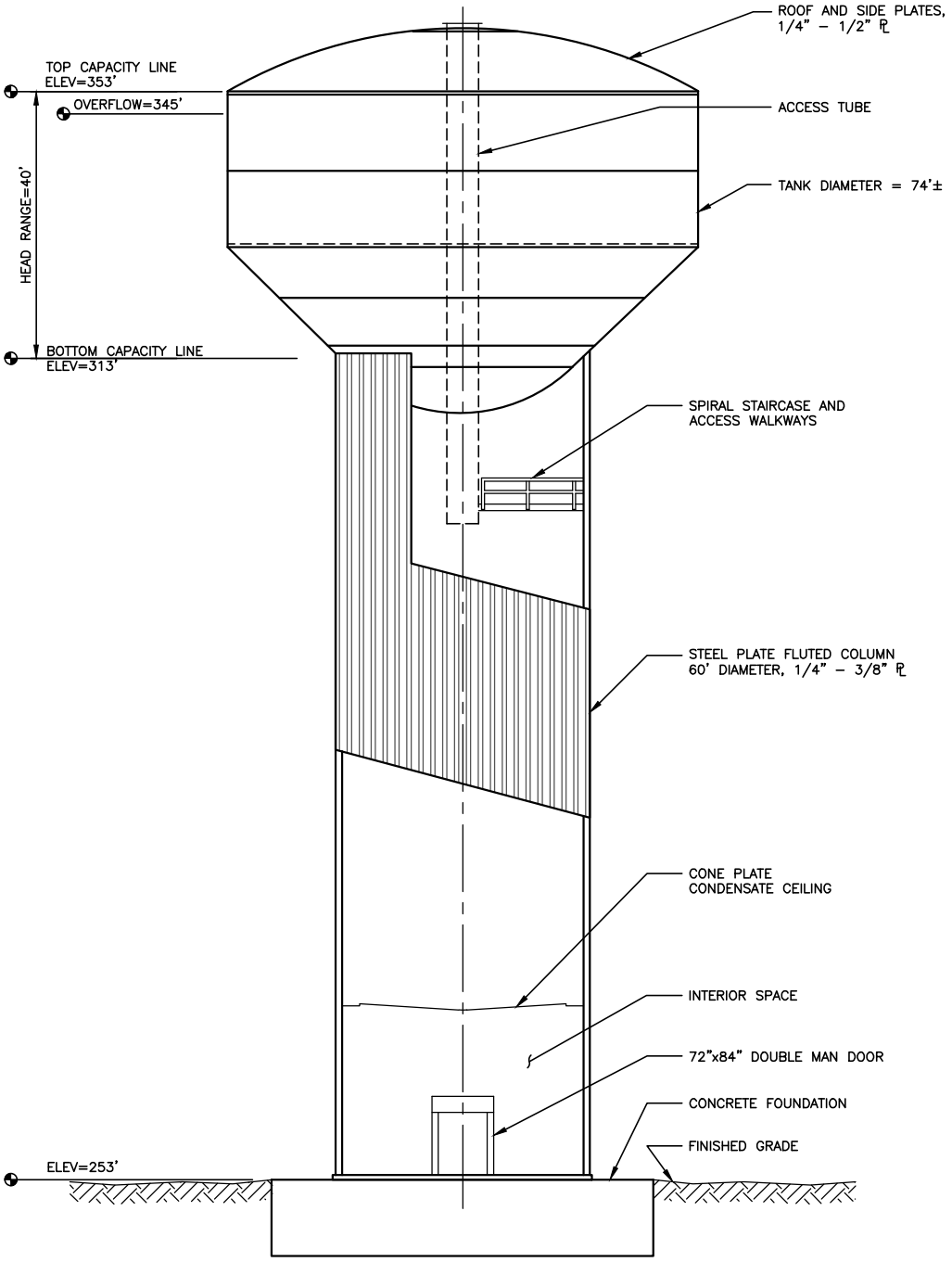

ELEVATED TANK SECTION

NOT TO SCALE

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS
PRE-DESIGN REPORT

FIGURE 5-2

PROPOSED ELEVATED STORAGE TANK

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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PROPOSED RESERVOIR COMPONENTS

SITE DESIGN

The proposed reservoir will be constructed at the site of the existing Reservoir 1. The site is covered with grass, grass/dirt mix, and grass/gravel mix, is generally flat, but does contain a slightly higher elevation in the northeast corner of the property.

For construction, the existing Reservoir 1 will be demolished, removed from the site, and the new reservoir constructed at the same location. Figure 5-1 shows both the existing and proposed site plans and Figure 5-2 shows an elevation of a typical Hydropillar elevated storage tank.

It is important to note that significant additional area will be required for staging and construction. Construction of the proposed reservoir will require heavy equipment and will need a wide footprint for bracing and rigging. This will have two significant impacts for the project. The first is that the existing trees and vegetation on the east and south sides of the existing site will likely need to be removed to provide access during construction. This vegetation provides good screening and cover from neighboring properties – especially the High School playfields. The second impact is that because this property is owned by the Bainbridge Island High School and not the City, the City will likely need to procure a temporary construction easement from the High School to allow use of the property during construction. This temporary easement will allow use and access for construction vehicles and personnel during construction. After construction is completed, the easement area must be restored to its original condition with grass, seeding, and/or vegetation. Each of the alternatives described in Chapter 4 would require a temporary construction easement, and once the City selects their desired alternative, they should initiate discussions with the High School Ownership group soon to ensure that the project can be completed on time and according to the City's desired schedule.

A subsurface geotechnical investigation was completed by Associated Earth Sciences in 2000. The investigation (Appendix D) confirmed the depth of foundations for Reservoir 1 and noted the soil types, depths, and depth to groundwater for the tank. Generally, soils below Reservoir 1 are suitable for construction, and soil liquefaction was not a concern at this location.

Additional geotechnical investigation was completed in December 2017 by PanGEO of Seattle, Washington. This investigation confirmed the soil type identified by AES in 2000, confirmed the depth of the foundations for both reservoirs, and provided recommendations on the soil bearing pressure for any retrofit work. Additional geotechnical investigations or recommendations will take place based on the specific requirements for the design of a foundation suitable to support the proposed reservoir. The foundation design will be provided by the tank manufacturer as part of the equipment design package.

STRUCTURAL DESIGN CONSIDERATIONS

The reservoir will be designed, erected, and tested in accordance with AWWA D100, Section 14 and the 2018 International Building Code (IBC). The foundation design will be completed as part of the tank design package, and will be stamped and signed by a professional structural engineer licensed in the State of Washington.

INLET AND OUTLET PIPING

To limit the potential for stagnation of water in the reservoir, the reservoir will be equipped with separate inlet and outlet pipes. The reservoir inlet will extend part of the way up the reservoir wall and discharge into the body of the tank. A nozzle may be outfitted to the discharge end of the pipe to increase fluid velocities and promote mixing. Check valves will be provided on the reservoir inlet and outlet piping to force water to enter the reservoir through the inlet pipe and exit out the outlet pipe. Inlet piping will be 12-inches in diameter. Outlet piping will also be 12 inches in diameter to match the existing piping.

Reservoir 1 will be filled via the existing well booster pumps. The existing pumps appear to be capable of delivering flow to the proposed tank – although at a rate slightly less than currently delivered to the tank as shown in Table 4-6. The reduction in flow is minimal and does not affect the City's ability to support the requirements of the Winslow Water System.

MIXING

Because the proposed reservoir will contain very little dead storage, we do not anticipate issues with water stagnation or high water age. The separate inlet and outlet piping mentioned above should adequately mix the water within the tank.

SEISMIC PIPING CONNECTIONS

While the reservoir will be anchored to the foundation, there is the possibility that either the reservoir or the ground under the reservoir could shift in the event of a major earthquake. If the reservoir itself or the ground under the reservoir should shift, the piping or the connection to the base of the reservoir could break or be damaged and the contents of the reservoir could be released. Such a release could cause significant damage as well as result in the loss of the stored water in the reservoir that would likely be needed in an emergency situation. For this reason, the inlet and outlet piping connections to the bottom of the reservoir will include flexible expansion couplings which will allow for some shifting and settling of the reservoir without placing stress on the piping or on the connection to the base of the reservoir. The flexible couplings will be forced balanced double-ball expansion joints.

SEISMIC VALVE

In the event of a major earthquake, there is also the possibility of major and multiple water main breaks within the water distribution system. Water main breaks in the distribution system can cause significant physical damage as well as deplete the stored water in the reservoir. To protect against this scenario, motor operated seismic valves will be installed on the inlet and outlet piping. A seismic sensor will be provided in the reservoir column annular space to register an alarm condition during a seismic event. The PLC will then be programmed to close the inlet and outlet control valves if a seismic event is sensed by the seismic controller and abnormally high flows from the reservoir are detected.

OVERFLOW

The reservoir overflow will be sized to accommodate an inflow rate of up to 3,000 gpm with no more than 6 inches of rise above the overflow. A 12-inch diameter pipe with a 36-inch cone will be provided. The overflow will discharge through a duckbill-type valve into a catch basin to provide an air gap to prevent entry of insects, birds or animals and the possibility of backflow into the reservoir via the overflow pipe. Overflow will be directed into a catch basin, which will drain to the existing storm system.

DRAIN

The reservoir will be equipped with a separate drain, controlled by a gate valve. The drain will connect to the overflow pipe and discharge to a proposed catch basin, which will drain to the existing storm system.

MUD RING

The reservoir will be equipped with a 6-inch tall mud ring on the reservoir outlet pipe to prevent silt and sediment that may accumulate on the reservoir floor from going out the reservoir outlet pipe.

WATER QUALITY SAMPLING

A water quality sampling station will be provided within the tank's annular space to allow operators to withdraw a sample from various elevations in the reservoir.

VENT

To prevent reservoir damage due to vacuum created as water flows from the reservoir, the reservoir will include a screened air vent. The vent must be sized to allow air into and out of the reservoir at not only the maximum projected system demand, but at the maximum plausible flow rate in the event of a major water main break near the reservoir. The vent will be sized to allow air to flow in at a rate of up 3,600 cfm with a maximum

pressure differential of 0.25 inches of water (1.3 psf) with the screen 30 percent blocked. In addition, freezing weather can cause ice buildup on a reservoir vent screen to the point of restricting air flow. Therefore, the reservoir vent will include a safety device that will allow air flow into the reservoir if air flow through the screen is obstructed.

ACCESS

Reservoir roof access needs to be provided for maintenance purposes. The proposed reservoir can be provided with either ladder or stair access to the elevated portions of the tank. The proposed reservoir will include a staircase welded/mounted to the inside wall of the tank column. This will limit the need for safety harnessing equipment associated with ladders and will also provide for easy access while carrying tools and other equipment. At the bottom of the tank, a larger platform/walkway will be provided for staging. A ladder will then be provided from this landing up to the roof access hatch. All stairs will be provided with extended landings approximately every 50 vertical feet.

The roof access area will be provided with a medium sized, flat access platform. The platform will include stainless steel or aluminum grating and will also include safety guardrail and electrical connections for tools, lights, intrusion alarms, etc.

HATCHES

The reservoir will be equipped with hatches to allow entry from the access ladder to both the tank as well as the roof; and entry to the tank from the roof. Hatches will be approximately 3-foot square and will be provided with a hinged lid. Additional hatches will be provided based on the desires of the City

GUTTERS AND DOWNSPOUTS

Gutter and / or downspouts are not proposed for this tank. Rainwater will drip down the sides of the tank and fall to finished grade.

EXTERIOR LEVEL GAUGE

No exterior sight gauge will be provided for this tank. The tank will be equipped with two sets of electronic level sensing equipment. Controllers for this equipment will be located in the tank's annular space, will have a local display, and will relay the measured information to the City's SCADA system.

PAINTING

The interior and exterior of the reservoir will be coated using materials that meet ANSI/NSF Standard 61 certification. Steel sheets for the reservoir will be shop-primed. Weld seams will be ground smooth, sand blasted and field primed. After all priming is completed, two coats of low emission epoxy paint will be applied to the inside of the

reservoir. The exterior will be coated with one coat of low emission epoxy paint over the primer, and one coat of polyurethane finish coat. The polyurethane finish is UV resistant, resistant to vandalism, and easily cleaned with readily available solvents.

CATHODIC PROTECTION

Cathodic protection can be used on reservoirs to extend the expected life of reservoir coating systems.

Cathodic protection can be installed as a passive system using galvanic anodes, or an active system using impressed current. The advantages of a passive system include low maintenance, easy installation, and a uniform distribution of protective current. Furthermore, no additional power source is required to provide galvanic protection. Disadvantages of a passive system include the requirement for additional anodes, a low driving voltage/current that is not adjustable, low effectiveness in high-resistivity environments, and a requirement for mechanical connections that may cause output limitations.

Advantages for an active galvanic system (impressed current) are that it requires fewer anodes, satisfies high current requirements with a single installation, effectively protects uncoated and poorly coated structures, and has a longer service life than passive galvanic protection. Disadvantages to an active system include higher and more frequent maintenance requirements, a requirement for an external power source, and the risk for overprotection resulting in coating damage if the system is not properly maintained.

Because modern coating systems, if properly applied, provide a high level of protection against corrosion, a galvanic protection system will not be installed as part of this project. However accommodations for a passive galvanic protection system will be provided so that it may be installed easily in the future.

LEVEL CONTROL

Level control will be provided via connection of the new equipment to the City's existing SCADA system. Water level will be determined by two pressure transducers. This will provide level redundancy in the event that one measuring device fails. Redundant measuring devices are especially critical as an external sight gauge will not be provided for this tank. The pressure transducers and controllers will be located within the column annular space. Analog (4-20 mA) signals from both of these instruments will be relayed to the City's SCADA system, where it will be displayed. The PLC/SCADA system will be programmed to provide low-low, low, high, and high-high level alarms to warn City staff that the system requires attention.

FLOW CONTROL

Magnetic flow meters will be provided on the reservoir inlet and outlet piping. A valve will be installed on the reservoir inlet to allow the City's operators to control the flow into the reservoir.

ELECTRICAL AND TELEMETRY

Electrical service is present at the existing Reservoir 1 site. The existing service will be modified and used to provide power for the proposed equipment. Wherever possible, electrical equipment will be located within the tank column to protect against exposure to the weather and UV radiation. Although no yard lighting is proposed for this project, the access doors to the tank column will be provided with LED lighting operated on a photocell. This lighting will help prevent vandalism and will provide access to the tank column. The column access door lighting will be provided with a 24-hour battery backup system to provide lighting during a disruption in electrical service.

The new equipment will be controlled via a programmable logic controller (PLC) that will communicate with the City's existing SCADA system. The PLC will be used to relay level information, operational status, and alarms, and will provide some level of pump control. The existing phone communication cables will be reused and will provide for communication between the tank and monitoring facility.

TELECOMMUNICATIONS EQUIPMENT

At Reservoir 2, telecommunications companies have asked to lease space on the reservoirs for installation of telecommunications equipment. It is anticipated that telecommunications companies will wish to mount their equipment on the proposed reservoir. For this project, provisions will be made to allow future installation of telecommunications equipment at the site without disrupting access and operations of the reservoir. The following provisions will be made:

- Identify space on the site for ground support equipment;
- Size the primary power service to support additional power loads from telecommunications equipment;
- Provide a conduit chase through the reservoir foundation and up the side of the reservoir for installation of cables and conduits; and
- Design the reservoir perimeter handrail to accommodate antennas.

SECURITY

The existing site is surrounded by galvanized chain link security fencing with three-strand barbed wire. It is unlikely that the existing fence may remain in place during construction. As such, we recommend that the City remove the existing fence, install temporary construction fencing for site security, and install new, green vinyl coated

6-foot tall chain link fence with 3-strand barbed wire prior to project completion.

The site is accessed by a single manual 12-foot double swing gate in the northwest corner of the site. There is also a 12-foot double swing gate in the southwest corner of the property that leads to a private property. A 3-foot single swing pedestrian gate will be added to the existing site fencing.

No additional video surveillance will be included at this time, but the project will include accommodations for future video surveillance equipment to be installed by the City.

Access doors to the tank column will include door intrusion alarms. These alarms will be connected to the SCADA system and so that staff can be notified if the alarm is activated.

AUXILIARY GENERATOR

No auxiliary generator is planned for this project. Connections for a small, portable generator will be provided in the instance that primary electrical service to the site is interrupted.

PRESSURE ZONE AND DISTRIBUTION SYSTEM MODIFICATIONS

Per the description in Chapter 4 and the description above, the recommended modifications will include construction of a new elevated tank. As a result of the higher HGL for the proposed tank, the HGL of the existing High Zone will increase by approximately 5 to 6 psi.

need to do

To account for this increase in pressure, we recommend that the City install pressure reducing valve stations at select locations within the distribution system to limit the pressure in the system to 90 psi, which is below the maximum recommended value of 100 psi.

Although the exact number of PRV stations will be determined during additional investigation by both the City and Gray and Osborne, it is anticipated at this time that two individual stations are required. These stations should be installed at the locations shown on Figure 5-3. Figure 5-4 shows a typical, buried vault PRV station and is representative of the type, number, and size of components that will be installed at these locations.

EXISTING WELL/BOOSTER PUMP MODIFICATIONS

The proposed reservoir will be filled from the existing well booster pumps. As previously described, there may be a small reduction in flow due to the increase in HGL for the High Zone. To quantify the actual pump output, the existing booster pumps will be flow tested after construction of the proposed reservoir. To conduct this flow testing, new magnetic flowmeters should be installed at all three groundwater facilities (Fletcher

Bay, HOB, and Sands). City staff have indicated the existing meters to be inaccurate, and providing new, factory calibrated, instantaneous read flow meters will help ensure that any reduction in flow from the booster pumps can be accurately quantified.

If the reduction in flow to the proposed reservoir is in fact significant, select well booster pumps will be modified or replaced to accommodate the new hydraulic conditions, and this replacement/modification effort may also involve modifying the current control/operational scheme for the pumps.

PERMITTING AND OTHER REGULATORY REQUIREMENTS

As with any project that affects the equipment used to provide potable water to a municipal water system, regulatory and permitting concerns must be addressed. The section below summarizes the regulatory efforts needed to complete the proposed project.

DOH PROJECT REPORT

Per WAC 246-290-110, a Project Report must be submitted to the Washington State Department of Health (DOH) for any modification or addition to a water system. This report is intended to fulfill the requirements of WAC 246-290-110.

CONSTRUCTION DOCUMENTS

Per WAC 246-290-120, Construction Documents must be submitted to DOH for review and approval prior to constructing modifications or additions to a water system. Plans and specifications will be submitted prior to beginning construction of the project.

When approved by DOH, construction documents (Plans, Specifications, Forms, etc.) will be provided by the City via a public forum for bidding by responsive, responsible contractors. If awarded, the project will then be constructed as shown on these Plans and as defined by the Contract Specifications.

SEPA

Per RCW 43.21C and WAC 197-111, all government agencies must consider the environmental impacts of a proposed project. A SEPA checklist and supporting documentation have been prepared for this project and are included in Appendix A.

CITY PERMITTING

A city building permit is required for any project to construct, enlarge, repair, alter, move, demolish, or change the occupancy of any building within the City limits. City building permit applications and fee schedules are available online at https://ci-bainbridgeisland-wa.smartgovcommunity.com/Public/Home. The building permit application will require two sets of construction plans, two site plans, structural

different tank
types, $, etc.
hybrid
Fletcher Bay?

Ind PRVs!
New Main?

testing of
booster pump
data and more
discussion

Fletcher Bay Well

New Brooklyn Road

Sands Ave Wellfield

High School Reservoir 2 (1.5 MG)

High School Elevated Storage Tank (2.0 MG)

Head of the Bay Wellfield

Legend

- WATER SERVICE
- PRV
- PROPOSED PRV
- RESERVOIR
- WELL
- PIPES
- PARCELS
- WINSLOW RWSA
- ELEVATION >= 265FT
- 5FT CONTOUR
- HIGH PRESSURE ZONE
- LOW PRESSURE ZONE

Source:King County GIS Data; Aerial Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

N

0 1,750 3,500 Feet

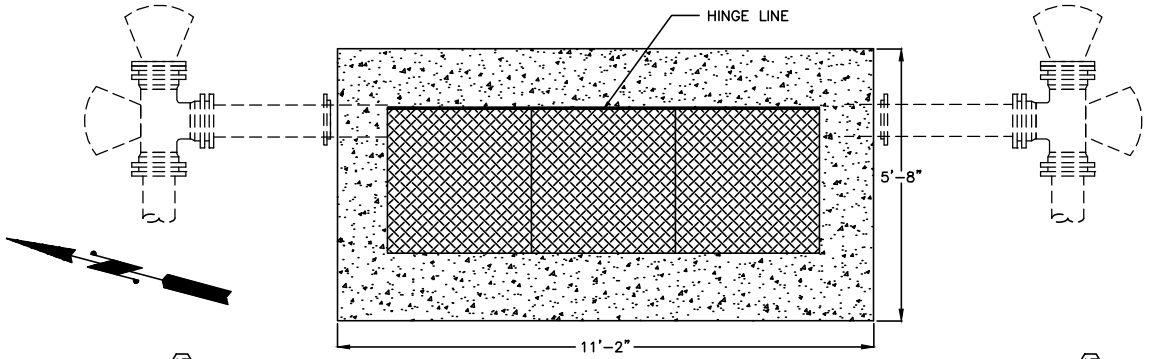
1 " = 1,760 '

CITY OF BAINBRIDGE ISLAND

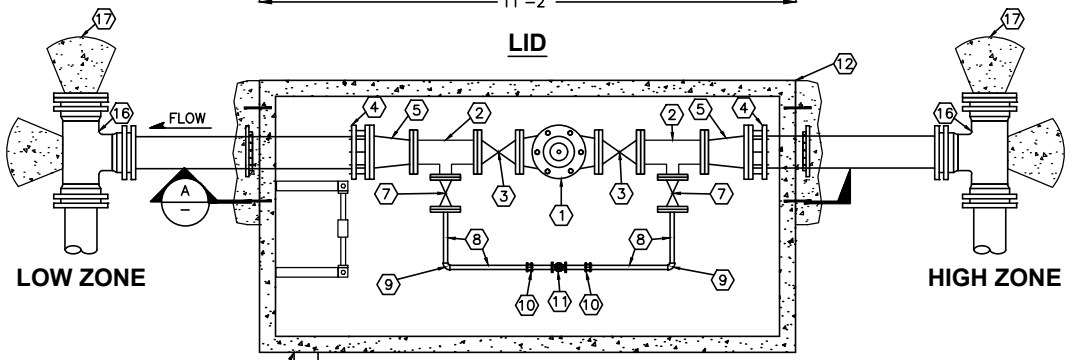
RESERVOIR NO.1 IMPROVEMENTS
PRE-DESIGN REPORT
FIGURE 5-3
PROPOSED PRV LOCATIONS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

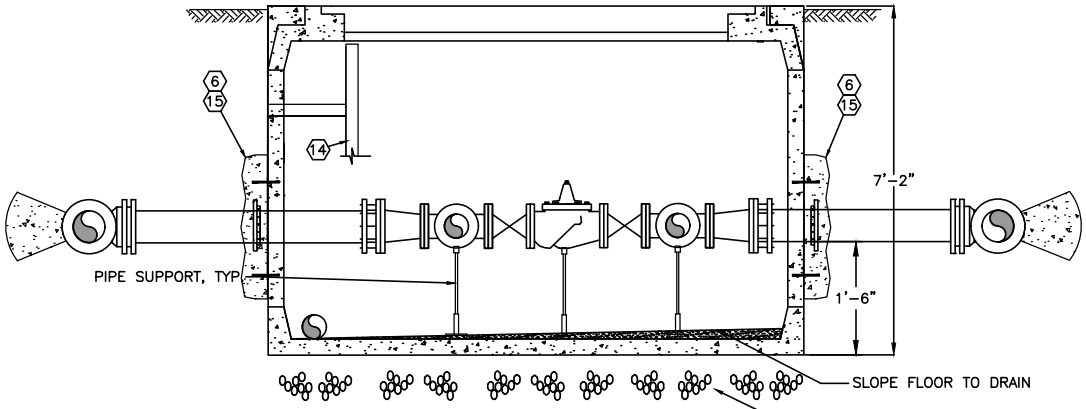
L:\Bainbridge Island\19648 - Reservoir 1 Improvements Pre-Design Report\GIS\FIG.5-3 PROPOSED PRV.mxd



LID



PLAN



SECTION A-A

TYPICAL PRV VAULT

SCALE: 1/4"=1'-0"

0 1" 2"

TWO INCHES AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

LEGEND:

1. CLA-VAL 92G-01BPVY PRESSURE REDUCING AND SUSTAINING VALVE
2. TEE (FL)
3. GATE VALVE (FL)
4. FLANGE COUPLING ADAPTER W/ RESTRAINTS
5. REDUCER (FL)
6. UNI-FLANGE ADAPTER FLANGE
7. GATE VALVE (FL) WITH THREADED TAP
8. BRASS PIPE
9. 90° BEND
10. UNION FITTING
11. CLA-VAL CRD-L PRESSURE REDUCING CONTROL
12. 5106-LA OLDCASTLE PRECAST VAULT
13. ACCESS HATCH, H-30 RATED, 36"x96" TRIPLE LEAF DOOR
14. LADDER AND LADDER-UP
15. 3'x3'x1' CONCRETE BLOCK, CONNECT TO VAULT WITH ANCHOR, 4 EACH PER BLOCK.
16. TEE (RJ) W/ THRUST BLOCK
17. PLUG (RJ) W/ THRUST BLOCK

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS PRE-DESIGN REPORT

FIGURE 5-4

TYPICAL PRV BURIED VAULT INSTALLATION

Gray & Osborne, Inc.
CONSULTING ENGINEERS

calculations, project specifications, and energy code forms.

In addition to a new commercial building permit application, it is anticipated that a grading permit may also be required. If required, the grading permit application will require four sets of grading plans.

Because the use of the site is not changing as a result of this project, it is not anticipated that a conditional use permit will not be required for this project.

As previously mentioned, a temporary construction easement will be required because the property is owned by Bainbridge Island High School and is leased by the City.

In total, the permitting process is anticipated to take 6 to 8 weeks from initial submittal to final permit approval.

ELECTRICAL PERMITTING

Department of Labor & Industry electrical permits and inspections will be required, but these applications and coordination will be provided by the general contractor awarded the project and their chosen electrical sub-contractors as part of the construction.

STORMWATER

The proposed project will increase the area of impervious surfacing on the site and additional stormwater facilities will be constructed to accommodate this increased area. All stormwater facilities will be designed and implemented in accordance with the 2019 City of Bainbridge Island Stormwater Management Program Plan.

OTHER PERMITS AND CONSIDERATIONS

Because the existing reservoir site is currently developed and is commercial in nature, and the proposed modifications will not alter the use of the site, additional land use, critical area, or other site permits are not anticipated.

The existing site is large enough to accommodate all of the recommended facilities and additional land acquisition will not be required.

PROJECT FUNDING

Funding for this project will be provided by the City out of their existing resources. No additional funding assistance is anticipated in order to complete this project.

CHAPTER 6

COMMISSIONING AND USE

STARTUP AND TESTING

Startup and testing for the new Reservoir will occur as individual systems are brought online and at the conclusion of the project. Startup services will be provided by the individual component or equipment representatives who are specially trained in the startup and operation of their equipment. Inspection and startup reports will be completed by these individuals and provided to the City as part of the final project Operations and Maintenance Manual. Once the reservoir is substantially complete, all of the equipment and supporting components will be tested together as a complete system.

Testing will include individual booster pump flow testing, level sensor and pressure transducer measure down confirmation, thorough inspections by the City and the design engineer, and water quality testing. Water quality testing will, at a minimum, include the following analysis:

- Bacteriological (E. coli);
- Organics and volatiles analysis (EPA 524.2).

Results will be provided to DOH for review. A construction completion report will only be issued to DOH upon full project completion and acceptance by the City.

OPERATIONS AND MAINTENANCE

The tank and supporting systems will be operated and maintained similar to the existing control scheme. Well pumps will continue to operate on their current scheme as described in Chapter 2 using new setpoints from the level transducer in the new reservoir.

The City maintains a full staff that operates and maintains the water system facilities. It is not anticipated that additional staff will be required to manage and maintain the proposed facilities. Regular water sampling, daily inspection, and other maintenance activities will continue to occur as they do with the existing reservoir. The additional fittings and level sensing equipment proposed for Reservoir 1 will be added to the City's equipment inventory and regular maintenance schedule to ensure successful operation of the system.

APPENDIX A

SEPA CHECKLIST AND SUPPORTING INFORMATION

SEPA CHECKLIST

OWNER:	City of Bainbridge Island
PROJECT:	Reservoir 1 Improvements Pre-Design Report
G&O#:	19648.00
PROJECT TEAM:	M. Basden, M. Pentzke, R. Porter, K. Stewart
TITLE:	SEPA Checklist
DATE:	4/2/2020
CREATED BY:	M. Pentzke
LAST EDITED:	4/2/2020
EDITED BY:	K. Stewart

A. Background

1. Name of proposed project, if applicable:

Reservoir 1 Improvements Pre-Design Report

2. Name of applicant:

City of Bainbridge Island

3. Address and phone number of applicant and contact person:

Christian D. Munter, PE, PMP, ENV SP
280 Madison Avenue North
Bainbridge Island, WA 98110
City of Bainbridge Island
206-780-3720

4. Date checklist prepared: 3/12/2020

5. Agency requesting checklist: Washington State Department of Health **(**WSDOH)

6. Proposed timing or schedule (including phasing, if applicable):

Bellow is described the proposed schedule:

2021

AUG	-Submit contract documents for DOH approval
SEPT 1	- Advertise for Bids
OCT 1	- Open Bids

NOV 1 - Award Contract
DEC 1 - NTP

2022
JAN 1 - Tank Submittals Received
MAY 1 - Tank Erection begins
SEPT 1 - Tank Coating work
DEC 1 - Project Completion

7. Do you have any plans for future additions, expansion, or further activity related to or connected with this proposal? If yes, explain.

There are no future plans or activities related to this proposal

8. List any environmental information you know about that has been prepared, or will be prepared, directly related to this proposal.

The only environmental concern regarding this project is related to the noise produced on a short-term basis during construction

9. Do you know whether applications are pending for governmental approvals of other proposals directly affecting the property covered by your proposal? If yes, explain.

There are no applications pending that are directly affecting the property covered by our proposal

10. List any government approvals or permits that will be needed for your proposal, if known.

- DOH Drinking Water Operating Permit
- DOH Water System Construction and Operation Approval
- City of Bainbridge Island Demolition Permit
- City of Bainbridge Island Business License
- City of Bainbridge Island Building & Grading Permits

11. Give brief, complete description of your proposal, including the proposed uses and the size of the project and site. There are several questions later in this checklist that ask you to describe certain aspects of your proposal. You do not need to repeat those answers on this page. (Lead agencies may modify this form to include additional specific information on project description.)

The City of Bainbridge Island recently completed a Water System Improvements Project which analyzed selected existing facilities with regards to seismic resiliency, physical

and structural condition, water quality, electrical redundancy, and performance. During this Project Reservoir 1 was found to be seismically deficient in four of six critical parameters.

Rather than seismically retrofit and recoat Reservoir 1, the City has elected to replace Reservoir 1 with a new Water Storage Tank. Furthermore, in order to reduce the volume of dead storage within the Reservoir, which will positively affect overall water quality within the reservoir and distribution system, the City has elected to install an elevated storage tank.

12. Location of the proposal. Give sufficient information for a person to understand the precise location of your proposed project, including a street address, if any, and section, township, and range, if known. If a proposal would occur over a range of area, provide the range or boundaries of the site(s). Provide a legal description, site plan, vicinity map, and topographic map, if reasonably available. While you should submit any plans required by the agency, you are not required to duplicate maps or detailed plans submitted with any permit applications related to this checklist.

The project site is occupied by one existing reservoir located approximately 1,000 feet west of Bainbridge High School in City of Bainbridge Island, Washington and is accessed from a private access road near the intersection of NE New Brooklyn Road and Northtown Drive.

The coordinates are: 47°38'21.9"N 122°31'34.0"W and parcel #222502-4-003-2008.

B. Environmental Elements

1. Earth

a. General description of the site:(circle one): Flat, rolling, hilly, steep slopes, mountainous, other _____

The project site is considered flat, its highest slope is 11% but only covers one corner of the location, the remainder of the site is essentially flat. The highest elevation is 257 ft and its lowest is 252 ft.

b. What is the steepest slope on the site (approximate percent slope)?

Given that the rise between 257 ft and 252 ft is 5 ft and distance is 44 ft the percent slope is 11%

c. What general types of soils are found on the site (for example, clay, sand, gravel,peat, muck)? If you know the classification of agricultural soils, specify them and note any agricultural land of long-term commercial significance and whether the proposal results in removing any of these soils.

The Preliminary Geologic Map of Bainbridge Island (Haugerud, 2005) indicates that the

surficial geologic units in the vicinity of the project are Vashon glacial till (Map Unit Qvt) and ice-contact deposits (Qvi). Glacial till is a very dense heterogeneous mixture of silt, sand, and gravel laid down at the base of an advancing glacial ice sheet. Glacial till typically exhibits low compressibility and high strength characteristics. Ice-contact deposits are described as gravel, sand, and diamict deposited against stationary ice. Ice contact deposits may or may not have been consolidated by glacial advance.

d. Are there surface indications or history of unstable soils in the immediate vicinity? If so, describe.

There are no surface indications or history of unstable soils in the immediate vicinity

e. Describe the purpose, type, total area, and approximate quantities and total affected area of any filling, excavation, and grading proposed. Indicate source of fill.

Total area of project is 9557 ft^2, diameter of foundation will be approximately 70 ft. Where new material is required, virgin material will be provided and will meet all applicable WSDOT standards for aggregates and gravel materials.

f. Could erosion occur as a result of clearing, construction, or use? If so, generally describe.

No erosion is anticipated as a result from the construction of the project and no clearing is needed. Erosion protection measures will be provided during construction and wilil include sediment fences and filter socks for existing catch basins.

g. About what percent of the site will be covered with impervious surfaces after project construction (for example, asphalt or buildings)?

Given that the total area of the site is 9,557 ft^2 and the impervious surfaces area is 2,827 ft^2 the percentage of covered impervious surfaces is approximately 29.6%

h. Proposed measures to reduce or control erosion, or other impacts to the earth, if any:

Disturbed areas not occupied by the proposed reservoir and associated appurtenances will be replanted with grass or other landscaping materials consistent with Bainbridge Island Code requirements. Silt fences will be used during construction

2. Air

a. What types of emissions to the air would result from the proposal during construction,operation, and maintenance when the project is completed? If any, generally describe and give approximate quantities if known.

Air quality will only be impacted during construction due to construction equipment

b. Are there any off-site sources of emissions or odor that may affect your proposal? If so, generally describe.

No emissions or odors may affect our proposal

c. Proposed measures to reduce or control emissions or other impacts to air, if any:

Idling of construction vehicles and equipment will be minimized and emissions equipment will be properly operated and maintained.

3. Water

a. Surface Water:

1) Is there any surface water body on or in the immediate vicinity of the site(includingyear-round and seasonal streams, saltwater, lakes, ponds, wetlands)? If yes, describe type and provide names. If appropriate, state what stream or river it flows into.

There is no surface water body in the immediate vicinity of the site

2) Will the project require any work over, in, or adjacent to (within 200 feet) the describedwaters? If yes, please describe and attach available plans.

The project is not close to any surface water body therefore, this question is not applicable

3) Estimate the amount of fill and dredge material that would be placed in orremovedfrom surface water or wetlands and indicate the area of the site that would be affected. Indicate the source of fill material.

No amount will be filled or dredged

4) Will the proposal require surface water withdrawals or diversions? Give general description, purpose, and approximate quantities if known.

The project does not require any surface withdrawals or diversions, hence this question is not applicable

5) Does the proposal lie within a 100-year floodplain? If so, note location on the site plan.

No

6) Does the proposal involve any discharges of waste materials to surface waters? If so,describe the type of waste and anticipated volume of discharge.

The proposal does not involve any discharges of waste materials to surface waters

b. Ground Water:

1) Will groundwater be withdrawn from a well for drinking water or other purposes? If so, give a general description of the well, proposed uses and approximate quantities withdrawn from the well. Will water be discharged to groundwater? Give general description, purpose, and approximate quantities if known.

No. The new tank will be filled with water extracted from three existing groundwater sources. No new groundwater extraction is proposed as part of this project.

2) Describe waste material that will be discharged into the ground from septic tanksor other sources, if any (for example: Domestic sewage; industrial, containing thefollowing chemicals. . . ; agricultural; etc.). Describe the general size of thesystem, thenumber of such systems, the number of houses to be served (ifapplicable), or the number of animals or humans the system(s) are expected to serve.

No waste material will be discharged into the ground from septic tanks as part of this project.

c. Water runoff (including stormwater):

1) Describe the source of runoff (including storm water) and method ofcollectionand disposal, if any (include quantities, if known). Where will this water flow? Will this water flow into other waters? If so, describe.

Roof runoff will be collected and disposed using splash blocks and other small scale dispersion equipment.

2) Could waste materials enter ground or surface waters? If so, generally describe.

No Waste materials could enter the ground or surface waters.

3) Does the proposal alter or otherwise affect drainage patterns in the vicinity of the site? If so, describe.

The proposal will not alter or affect drainage patterns.

d. Proposed measures to reduce or control surface, ground, and runoff water, and drainage pattern impacts, if any:

None proposed measures are required given that existing storm will be utilized.

4. Plants

a. Check the types of vegetation found on the site:

____deciduous tree: alder, maple, aspen, other
____evergreen tree: fir, cedar, pine, other
____shrubs
_X__grass
____pasture
____crop or grain
____ Orchards, vineyards or other permanent crops.
____ wet soil plants: cattail, buttercup, bullrush, skunk cabbage, other
____water plants: water lily, eelgrass, milfoil, other
____other types of vegetation

b. What kind and amount of vegetation will be removed or altered?

Minor areas of grass need to be removed for reservoir construction; nohard surfaces will be replaced with grass as part of the project

c. List threatened and endangered species known to be on or near the site.

The USFWS Information Planning and Consultation Species List for the project area did not include any threatened or endangered plant species.

d. Proposed landscaping, use of native plants, or other measures to preserve or enhance vegetation on the site, if any:

Disturbed areas not covered by the new water reservoir and appurtenant structures (foundation, access road etc.) will be replanted with grass or other landscaping in accordance with City of Bainbridge Island Landscaping Requirements.

e. List all noxious weeds and invasive species known to be on or near the site.

None known noxious weeds or invasive species are near the site

5. Animals

a. List any birds and other animals which have been observed on or near the site or are known to be on or near the site.

Examples include:

birds: hawk, *heron, eagle, songbirds*, other:
mammals: *deer*, bear, elk, beaver, other:
fish: bass, salmon, trout, herring, shellfish, other _____

b. List any threatened and endangered species known to be on or near the site.

The USFWS IPaC Species List for the project area included the following species: Marbled Murrelet, Yellow-billed Cuckoo, Streaked Horned Lark and Bull Trout. Further, the species list indicated that there are no critical habitats for any of these species in the immediate vicinity. Salmonids and marine mammals under the jurisdiction of the National Marine Fisheries Service/NOAA Fisheries are present off-shore in Puget Sound, but they will not be affected by the proposed reservoir replacement project. NO GOPHERS OR VOLES ON THE ENDANGERED SPECIES LIST in Washington…though they could be present on the site.

c. Is the site part of a migration route? If so, explain.

Bainbridge Island lies within the Pacific Flyway for waterfowl. However, there is no open-water habitat for waterfowl nearby.

c. Proposed measures to preserve or enhance wildlife, if any:

Construction of the new reservoir on the same site as the existing tank will minimize any potential impacts to wildlife habitat

e. List any invasive animal species known to be on or near the site.

None known invasive animal species are known to be on or near the site

6. Energy and Natural Resources

a. What kinds of energy (electric, natural gas, oil, wood stove, solar) will be used to meetthe completed project's energy needs? Describe whether it will be used for heating,manufacturing, etc.

Electrical power will be used for equipment operation

b. Would your project affect the potential use of solar energy by adjacent properties? If so, generally describe.

The project will not affect the potential use of solar energy in adjacent properties

c. What kinds of energy conservation features are included in the plans of this proposal? List other proposed measures to reduce or control energy impacts, if any:

The project includes modern Energy efficient pumps and the distribution system operates by gravity.

7. Environmental Health

a. Are there any environmental health hazards, including exposure to toxic chemicals, riskof fire and explosion, spill, or hazardous waste, that could occur as a result of this proposal?
If so, describe.

1) Describe any known or possible contamination at the site from present or past uses. '

There is no known or possible contamination, as the project site has been dedicated to water storage and distribution since the existing reservoirs were constructed 47 years ago.

2) Describe existing hazardous chemicals/conditions that might affect project development and design. This includes underground hazardous liquid and gas transmission pipelines located within the project area and in the vicinity.

There is no existing hazardous chemical that might affect the project development and design

3) Describe any toxic or hazardous chemicals that might be stored, used, or produced during the project's development or construction, or at any time during the operating life of the project.

None toxic or hazardous chemicals, other than fuels, lubricants and coolants in construction vehicles and equipment.

4) **Describe special emergency services that might be required.**

None likely to be required

5) **Proposed measures to reduce or control environmental health hazards, if any:**

Good construction practices. Construction vehicles and equipment will be equipped with hazardous materials spill clean-up kits and operators shall be trained in their use.

b. Noise

1) What types of noise exist in the area which may affect your project (for example:traffic, equipment, operation, other)?

Noise will be generated by the demolition of the existing reservoir, the construction of the new tank and the booster pump station

Construction equipment in pump stations to is expected to consist of a forklift, excavator, compactor, backhoe,crane, water truck and two concrete trucks **.**

2) **What types and levels of noise would be created by or associated with the project on a short-term or a long-term basis (for example: traffic, construction, operation, other)? Indicate what hours noise would come from the site.**

Short-term basis due construction will be created with the project, the long-term noises will be created by the monthly generator testing consisting of 20 minutes.

3) Proposed measures to reduce or control noise impacts, if any:

Construction will take place during business hours only and the standby generator will be installed in a sound-proof enclosure

8. Land and Shoreline Use

a. What is the current use of the site and adjacent properties? Will the proposal affect current land uses on nearby or adjacent properties? If so, describe.

The existing Reservoir 1 is surrounded by a residential area and Bainbridge High School. This reservoir will be demolished and replaced by the new tank, therefore no adjacent properties will be affected.

b. Has the project site been used as working farmlands or working forest lands? If so, describe. How much agricultural or forest land of long-term commercial significance will be converted to other uses as a result of the proposal, if any? If resource lands have not been designated, how many acres in farmland or forest land tax status will be converted to nonfarm or nonforest use?

The project site has been dedicated to the water Bainbridge Island system since 1973. It has not been used as working farmlands.

1) **Will the proposal affect or be affected by surrounding working farm or forest land normal business operations, such as oversize equipment access, the application of pesticides, tilling, and harvesting? If so, how:**

The proposal will not affect, or be affected by, any surrounding working farm or forest land activities.

c. Describe any structures on the site.

Reservoir 1 was constructed in 1973 and is an above ground, welded steel tank with a capacity of approximately 1.0 million gallons (MG) and a diameter of approximately 46-feet. The tank rests on an octagonal, thickened edge concrete foundation and is located directly adjacent to single family residences as well as the Bainbridge Island High School playfields.

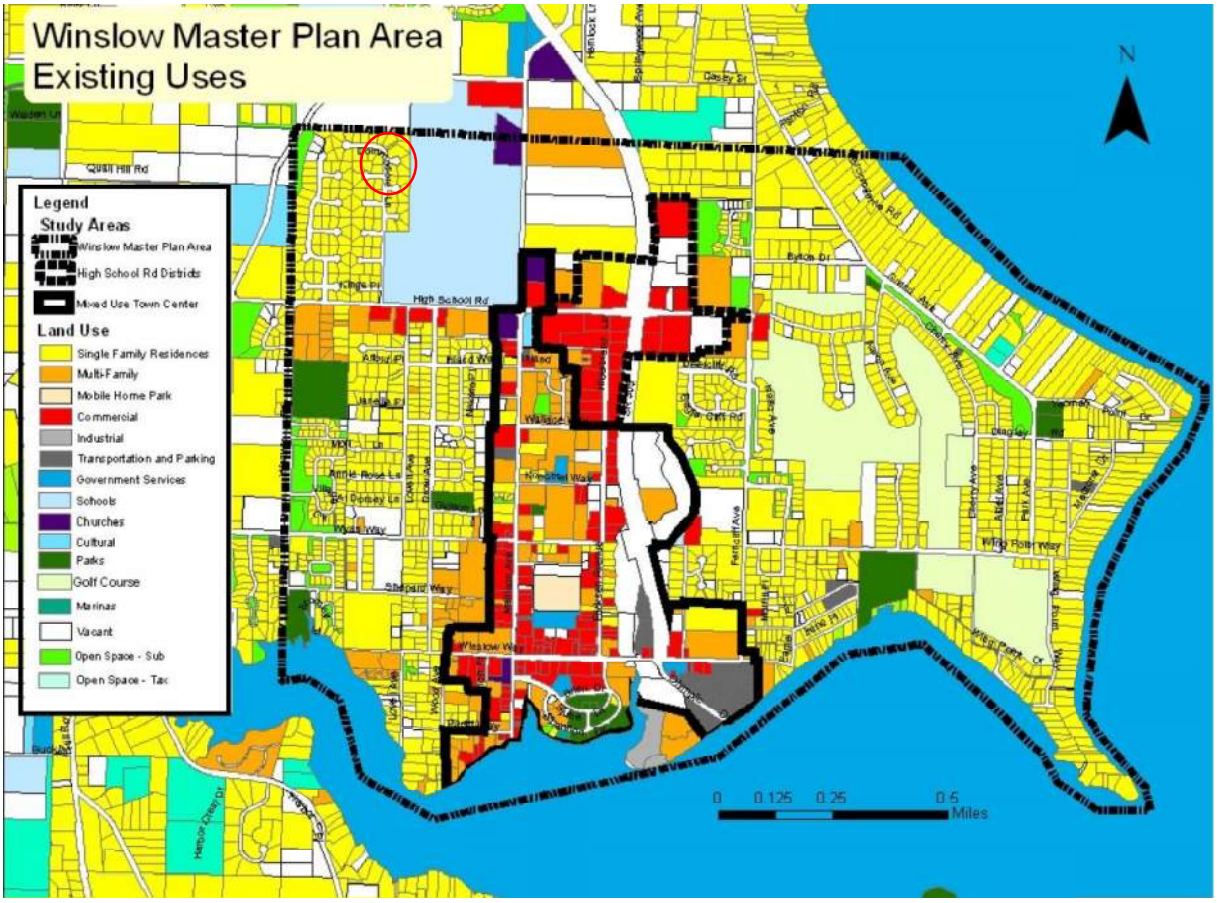
d. Will any structures be demolished? If so, what?

The existing Reservoir 1 will be demolished

e. What is the current zoning classification of the site?

According to City of Bainbridge Island Official Zoning Map in the most recent WSP the classification zone is both R-2.9 (2.9 units per Acre Zone) and R-3.5 (3.5 Units per Acre Zone).

f. What is the current comprehensive plan designation of the site?



Winslow Master Plan (2006) states that the designation of the site is classified.

g. If applicable, what is the current shoreline master program designation of the site?

Not applicable, as the project area is located more than 200 feet from Puget Sound.

h. Has any part of the site been classified as a critical area by the city or county? If so, specify.

The Kitsap County GIS system shows how there is no critical area in any part of the project site.

i. Approximately how many people would reside or work in the completeproject?

There will be no people residing or working in the complete project.

j. Approximately how many people would the completed project displace?

No people will be displaced by the project.

k. Proposed measures to avoid or reduce displacement impacts, if any:

The measures are not needed.

L. Proposed measures to ensure the proposal is compatible with existing and projected land uses and plans, if any:

The new tank will be placed on the same site as the existing reservoir.

m. Proposed measures to reduce or control impacts to agricultural and forest lands of long-term commercial significance, if any:

The measures are not needed

9. Housing

a. Approximately how many units would be provided, if any? Indicate whether high, middle, or low-income housing.

No housing will be provided nor eliminated.

b. Approximately how many units, if any, would be eliminated? Indicate whether high,middle, or low-income housing.

No units will be eliminated.

c. Proposed measures to reduce or control housing impacts, if any:

Not applicable.

10. Aesthetics

a. What is the tallest height of any proposed structure(s), not including antennas; what isthe principal exterior building material(s) proposed?

The proposed structure is a hydropillar elevated storage welded steel tank with a total height of approximately 105 ft.

b. What views in the immediate vicinity would be altered or obstructed?

No views will be altered or obstructed in the immediate vicinity.

d. Proposed measures to reduce or control aesthetic impacts, if any:

Reservoir will be coated in an aesthetically pleasing color, or painted to blend into the surrounding landscape.

11. Light and Glare

a. What type of light or glare will the proposal produce? What time of day would it mainlyoccur?

No glare or light will be produced.

b. Could light or glare from the finished project be a safety hazard or interfere with views?

Not applicable as no glare or light will be produced.

c. What existing off-site sources of light or glare may affect your proposal?

No existing off-site sources of light or glare will affect the proposal.

d. Proposed measures to reduce or control light and glare impacts, if any:

Site lighting will be directed toward the center of the reservoir site/compound to avoid interfering with surrounding views etc.

12. Recreation

a. What designated and informal recreational opportunities are in the immediate vicinity?

There are two baseball fields used by Bainbridge Island High School east of the proposed project location.

b. Would the proposed project displace any existing recreational uses? If so, describe.

The proposed project will not displace these fields.

c. Proposed measures to reduce or control impacts on recreation, including recreation opportunities to be provided by the project or applicant, if any:

Not applicable.

13. Historic and cultural preservation

a. Are there any buildings, structures, or sites, located on or near the site that are over 45 years old listed in or eligible for listing in national, state, or local preservation registers? If so, specifically describe.

The nearest buildings are residential and they were built between 1988-1989. The Bainbridge Island Historic Resources Survey and Inventory (2017) suggests that there is no preservation site on the project location (or eligible National Register of Historic Places). (See Figure3)

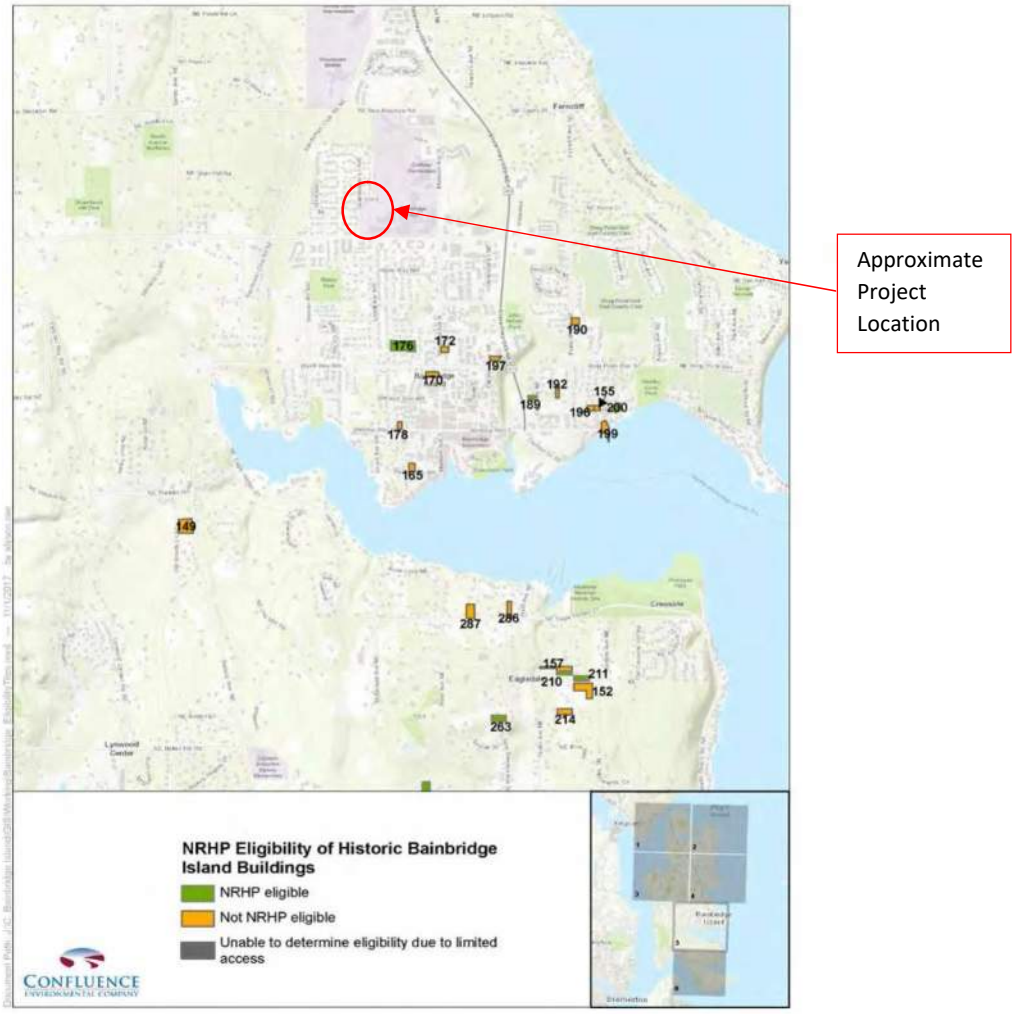


Figure 3. Eligibility of Historic Buildings

b. Are there any landmarks, features, or other evidence of Indian or historic use or occupation? This may include human burials or old cemeteries. Are there any material evidence, artifacts, or areas of cultural importance on or near the site? Please list any professional studies conducted at the site to identify such resources.

According tothe Bainbridge Island Historic Resources Survey & Inventory (2017) there are no landmarks, features or other evidence of Indian or historic site on the project location

c. Describe the methods used to assess the potential impacts to cultural and historic resources on or near the project site. Examples include consultation with tribes and the department of archeology and historic preservation, archaeological surveys, historic maps, GIS data, etc.

In order to assess the potential impacts to cultural and historic resources the Bainbridge Island Historic Resources Survey & Inventory and, the Kitsap GIS County data was consulted. We will follow the city's protocol for construction & cultural resources response as necessary.

d. Proposed measures to avoid, minimize, or compensate for loss, changes to, and disturbance to resources. Please include plans for the above and any permits that may be required.

There will be no disturbance or loss due to the project activities.

14. Transportation

a. Identify public streets and highways serving the site or affected geographic area and describe proposed access to the existing street system. Show on site plans, if any.

Reservoir 1 has limited access; it can only be accessed by NE New Brooklyn Road as pointed in Figure 4.

Figure 4. Project site street access

b. Is the site or affected geographic area currently served by public transit? If so, generally describe. If not, what is the approximate distance to the nearest transit stop?

The site is not currently served by public transit. As Figure 5 shows, the nearest transit stop is 1509 ft away from the site.

Figure 5. Nearest transit stop to project site

c. How many additional parking spaces would the completed project or non-project proposal have? How many would the project or proposal eliminate?

The project will not have any additional parking nor eliminate any existing parking.

d. Will the proposal require any new or improvements to existing roads, streets, pedestrian, bicycle or state transportation facilities, not including driveways? If so, generally describe (indicate whether public or private).

The project will not require any new or existing road improvements.

e. Will the project or proposal use (or occur in the immediate vicinity of) water, rail, or air transportation? If so, generally describe.

The project will not use any water, rail, or air transportation.

e. **How many vehicular trips per day would be generated by the completed project or proposal? If known, indicate when peak volumes would occur and what percentage of the volume would be trucks (such as commercial and nonpassenger vehicles). What data or transportation models were used to make these estimates?**

Up to 2 personal vehicle trips per day.

g. Will the proposal interfere with, affect or be affected by the movement of agricultural and forest products on roads or streets in the area? If so, generally describe.

The project doesn't affect the movement of agricultural or forest products.

h. Proposed measures to reduce or control transportation impacts, if any:

None, delivery of oversized vehicles and construction materials and equipment will be routed and timed to avoid heavy traffic.

15. Public Services

a. Would the project result in an increased need for public services (for example: fire protection, police protection, public transit, health care, schools, other)? If so, generally describe.

The project doesn't require an increase in public services.

b. Proposed measures to reduce or control direct impacts on public services, if any.

Measures are not needed. Construction and operation of the new reservoir will ensure adequate water storage for the Bainbridge Island service area through the 20-year planning period (2035).

16. Utilities

a. Circle utilities currently available at the site: electricity, natural gas, water, refuse service, telephone, sanitary sewer, septic system, other _____

f. Describe the utilities that are proposed for the project, the utility providing the service,and the general construction activities on the site or in the immediate vicinity which mightbe needed.

Electricity provided by KPUD and water by COBI, existing services will be modified to accommodate new equipment.

C. Signature

The above answers are true and complete to the best of my knowledge. I understand that the lead agency is relying on them to make its decision.

Signature: _____

Name of signee _____

Position and Agency/Organization _____

Date Submitted: _____

D. Supplemental sheet for nonproject actions

(**IT IS NOT NECESSARY** to use this sheet for project actions)

Because these questions are very general, it may be helpful to read them in conjunction with the list of the elements of the environment.

When answering these questions, be aware of the extent the proposal, or the types of activities likely to result from the proposal, would affect the item at a greater intensity or at a faster rate than if the proposal were not implemented. Respond briefly and in general terms.

1. How would the proposal be likely to increase discharge to water; emissions to air; production, storage, or release of toxic or hazardous substances; or production of noise?

   Proposed measures to avoid or reduce such increases are:

2. How would the proposal be likely to affect plants, animals, fish, or marine life?

   Proposed measures to protect or conserve plants, animals, fish, or marine life are:

3. How would the proposal be likely to deplete energy or natural resources?

   Proposed measures to protect or conserve energy and natural resources are:

4. How would the proposal be likely to use or affect environmentally sensitive areas or areas designated (or eligible or under study) for governmental protection; such as parks, wilderness, wild and scenic rivers, threatened or endangered species habitat, historic or cultural sites, wetlands, floodplains, or prime farmlands?

Proposed measures to protect such resources or to avoid or reduce impacts are:

5. How would the proposal be likely to affect land and shoreline use, including whether it would allow or encourage land or shoreline uses incompatible with existing plans?

   Proposed measures to avoid or reduce shoreline and land use impacts are:

6. How would the proposal be likely to increase demands on transportation or public services and utilities?

   Proposed measures to reduce or respond to such demand(s) are:

7. Identify, if possible, whether the proposal may conflict with local, state, or federal laws or requirements for the protection of the environment.

APPENDIX B

TECHNICAL MEMORANDUM 17614-4, HIGH SCHOOL RESERVOIRS EVALUATION

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TECHNICAL MEMORANDUM 17614-4

TO:	CHARLES KRUMHEUER, PUBLIC WORKS MANAGER
FROM:	MYRON BASDEN, P.E., S.E. RYAN HALE, E.I.T., NACE LEVEL 1 COATING INSPECTOR ALEX QUINN, E.I.T.
DATE:	AUGUST 15, 2018
SUBJECT:	HIGH SCHOOL RESERVOIRS EVALUATION, WATER SYSTEM IMPROVEMENTS CITY OF BAINBRIDGE ISLAND, KITSAP COUNTY, WASHINGTON G&O #17614.00

EXECUTIVE SUMMARY

The City of Bainbridge Island (City) contracted with Gray & Osborne to assess the system resiliency and determine the exterior and interior coating condition for the two High School Reservoirs. This memorandum contains the general condition assessment, geotechnical investigation results, coating evaluations, structural seismic analysis, and recommendations for the 1.0-million-gallon and 1.5-million-gallon reservoirs.

Gray & Osborne has completed the general, coating, and seismic analyses of both tanks. Both tanks are seismically deficient with regard to their foundations and soil bearing capacities, anchorage to their foundations, and shell wall steel to resist compression buckling. In the event of a design level earthquake near them, the tanks would likely be rendered unusable and perhaps fail catastrophically. The proximity of the tanks to residential housing represents a significant danger during a design level earthquake. The catastrophic failure of one, or both, of the tanks could result in significant property damage and potential loss of life.

In addition to the damage to property and potential for loss of life, the City's ability to provide potable water storage and fire protection would be jeopardized. The time required to reconstruct the facilities in the aftermath would range between 18 months and 3 years, depending on design, contracting/bidding, permitting, and funding. Repairs to a damaged tank would likewise require engineering evaluation, design, would depend on the availability of contractors, and the amount of time required before putting the tanks back in service would depend upon the nature and extent of the damage.

1130 Rainier Avenue South, Suite 300 Seattle, Washington 98144 (206) 284-0860 Fax (206) 283-3206

If the City chooses to not seismically upgrade the tanks, the tanks will remain seismically deficient with the risk of structural failure. Contingency plans should be developed for how to operate the water system with these tanks rendered inoperable for up to 3 years in the event of an emergency. Additionally, the 1.0-million-gallon (MG) tank should be recoated within 3 to 5 years while the 1.5 MG tank should be re-evaluated in 5 years because the reservoir coating systems are nearing the end of their service lives. A delay in recoating the reservoirs beyond their recommended coating lives may result in higher steel repair costs which are not be quantifiable until the tanks are blasted and the damage from corrosion made available for inspection. Cost estimates have been prepared for reservoir recoating, accessories, and seismic retrofits.

1.0 MG RESERVOIR

Background

The City's 1.0 MG High School Reservoir is an 81-foot-tall, 46-foot-diameter welded steel standpipe constructed in 1973 located on the northwest side of the playfields of Bainbridge Island High School. The reservoir sits atop an octagonal foundation footing that extends between 2 and 4 feet from the reservoir shell and is 10 feet deep. The reservoir is secured to the foundation with 14 straps spaced an average of approximately 10 feet apart. The original straps consist of 0.75-inch-thick and 2.5- to 3.5-inch-wide straps embedded in the foundation, extending up at a 45-degree angle to the reservoir shell, and then continuing up the shell for 6 inches where they are seal welded to the reservoir. The straps previously broke during the Nisqually earthquake in 2001 and were repaired in 2002 with two vertical 8-inch-tall and 0.75-inch-thick plates on either side of each of the original straps, 4 inches apart, and seal welded to the reservoir and original strap (see Figure 1B). The reservoir has one 30-inch inside diameter manway, located on the west side of the reservoir, as shown on Figure 1A.

A site visit to the reservoir was conducted on November 2, 2017, for interior and exterior coating evaluation and setting the adhesion test dollies. A follow-up site visit was conducted on December 4 to pull the adhesion test dollies, expose the reservoir foundation, conduct the geotechnical investigation, and collect shell steel thickness readings.

Ryan Hale, Myron Basden, and Alex Quinn of Gray & Osborne performed the reservoir assessments and inspection during the site visits. Adhesion tests were conducted utilizing a DeFelsko® PosiTest® Adhesion Tester Model AT-M Serial Number AT10353, in accordance with ASTM D4541 Test Method E. Steel shell thicknesses were collected

A: EXISTING 30-INCH MANWAY

B: SEISMIC STRAPS

C: RESERVOIR ROOF VENT

D: DELAMINATING ROOF COATING SYSTEM

CITY OF BAINBRIDGE ISLAND

2018 WATER SYSTEM IMPROVEMENTS PROJECT

1.0 MG HIGH SCHOOL RESERVOIR COATING CONDITION EVALUATION

FIGURE 1

Gray & Osborne, Inc.

L:\Bainbridge Island\17614 2017 Water System Improvements\RESERVOIR\1.0 Figure 1.mxd

utilizing a DeFelsko PosiTector® Serial Number 774455 with a UTG-C Probe with Serial Number 256944.

1.0 MG Exterior Coating Evaluation

The original coating system of the 1.0 MG reservoir is unknown. The exterior appears to have been topcoated previously, and a sample of the exterior coating system was collected and analyzed for Resource Conservation and Recovery Act (RCRA) 8 metals. The coating does contain RCRA 8 metals including barium and lead, but not in sufficient concentrations to cause concern or impact costs, although additional coating samples should be taken and analyzed for RCRA 8 metals prior to removal and disposal. The exterior topcoat was observed to be delaminating from the previous coating system, most notably on the bottom of the reservoir near the sill and anchor chairs. The delamination is also most notably present on the roof of the reservoir. In other places, the topcoat and previous coating systems were found to be well adhered to the reservoir. Photographs of the reservoir exterior coating condition are included as Figure 1.

Pull-off adhesion tests were performed in compliance with ASTM D4541 Test Method E utilizing a DeFelsko PosiTest adhesion tester. Three dollies were set on the reservoir sidewall with their location, pull strength in psi, and failure location/type reported in Table 1. The coating layers are as follows:

- A – Substrate: steel reservoir
- B – Primer: first coat applied to the reservoir
- C – Intermediate Coat: second coat applied to the reservoir
- D – Finish Coat: third coat applied to the reservoir
- E – Tie Coat: coating applied over the original coating system to allow the topcoat to adhere to the original coating system
- F – Topcoat: final coat applied to the reservoir
- Y – Adhesive: used to adhere the testing dolly to the reservoir coatings
- Z – Dolly: metal testing implement adhered to reservoir coatings

Failures listed with just one location, for example “A,” are cohesive failures that resulted from the internal cohesion of a single layer of the coating failing. Failures listed equally between two layers, for example “A/B,” are adhesive failures between coating layers that resulted in one coating being pulled from the other. An adhesion test report is contained in Attachment A.

TABLE 1

1.0 MG Reservoir Pull-Off Adhesion Test Results

Dolly ID	Location	Pull Strength (psi)	Failure Location/Type
7	Side of Reservoir	1,037	100% B
8	Side of Reservoir	248	100% D/E
9	Side of Reservoir	1,399	95% B, 5% B/C

The intermediate coat and finish coat are failing, in particular around the base of the reservoir, and this is reflected in the pull strength of Dolly 8. An effective coating system will fail at or above approximately 1,500 psi. Failures that occur below this value signal that the reservoir coating is nearing the end of its effective service life. The prime coat is not failing but shows signs of weakened adhesion to the reservoir based on Dollies 7 and 9. Based on the pull strength results presented above, the reservoir exterior coating system is near the end of its effective service life. The low pull strength of Dolly 8 is consistent with visual observations of the reservoir coating delaminating around the bottom of the reservoir near the anchor straps.

Dollies were not placed on the roof of the reservoir due to the lack of working space and poor weather conditions. The reservoir roof coating system was visually observed to be delaminating, especially the final coat from the intermediate coat, on a majority of the reservoir roof surface. It is expected that results similar to Dolly 8 would be observed for the roof coating system.

1.0 MG Interior Coating Evaluation

The interior coating system is unknown. It is unlikely that the coating system contains any significant amount of RCRA 8 metals of concern due to the age of the reservoir, although coating samples should be taken and analyzed for RCRA 8 metals prior to removal and disposal. The interior of the reservoir was not inspected by Gray & Osborne because the reservoir was in service and the water level was not lowered for access. Based on the video footage and subsequent report from the recent cleaning of the reservoir interior in April 2017 by LiquiVision Technology Diving Services, the self-supporting dome roof is in good condition with little to no corrosion evident.

1.0 MG Coating Recommendations

Due to the mild corrosion on the interior, the critical coating is the exterior and a new interior coating system can be applied when the exterior coating system is applied. Due

to the poor coating adhesion visually observed near the anchor straps and the roof and the issues noted with coating adhesion, it is recommended that the exterior of the reservoir be blasted to bare metal and recoated. Corrosion or metal loss of the reservoir structure was not observed at this time and the reservoir should be reinspected in 3 years and a possible blast and recoat should be schedule in 3 to 5 years. Following an inspection in 3 years, it is likely that the recommended inspection interval will change to annually.

In the event of seismic upgrades that require welding to the reservoir structure, the entire reservoir should be blasted to bare metal and recoated as part of the project as the welding will burn off the existing coatings.

1.0 MG Seismic Evaluation – Introduction

Table 2 is a summary of geometry data gathered from the field and from original drawings for the 1.0 MG tank.

TABLE 2

1.0 MG Reservoir Geometry

Item	Description
Diameter	46 ft
Height	81 ft
Foundation Type	Octagonal mat foundation with thickened edge
Foundation Diameter	50 ft (at minimum)
Foundation Thickness	2.5 ft at interior and 10 ft at thickened edge
Anchor Quantity and Type	14 steel straps

The thickness of the steel shell ring plates was measured using an ultrasonic thickness gauge. The gauge utilizes a non-destructive ultrasonic pulse-echo principle to measure the wall thickness. The gauge will accurately measure the thickness of an uncoated steel plate; however, the reservoirs have a coating over the steel which has a much lower density than the steel plate. The lower density causes the sound pulse to travel slower both as it pulses through the coating material and on the echo return to the gauge. Since the thickness is calculated using the time required for the echo to return to the gauge, the thickness will appear to be slightly greater than the actual thickness. Since coating thickness around the tank can be expected to vary, the amount of extra thickness shown on an individual reading due to the effects of sound travelling through the coating can be expected to vary as well, although this variance is assumed to be negligible. The results of the testing are provided in Table 3. All readings were taken from within reach of the access ladder so the entire circumference of each ring was not examined.

TABLE 3

1.0 MG Reservoir Measured Thickness of Shell Plate

Shell Course ID	Height to Top of Shell Course (ft)	Measured Shell Thickness (in)
1	81	0.25
2	75	0.25
3	69	0.25
4	63	0.26
5	55	0.26
6	47	0.32
7	39	0.38
8	31	0.41
9	23	0.44
10	15	0.51
11	7	0.60

1.0 MG Seismic Analysis Parameters

The reservoir has been analyzed for the seismic requirements of AWWA D100-11 "Welded Carbon Steel Tanks for Water Storage." Table 4 provides a summary of the seismic design parameters used for the analysis.

TABLE 4

1.0 MG Reservoir Seismic Design Parameters

Description	Value	Notes
Seismic Use Group	III	Required for facilities deemed essential for post-earthquake recovery, e.g., post-earthquake fire suppression.
Seismic Importance Factor, I_e	1.5	Determined by Seismic Use Group.
S_S	1.40 g	Design earthquake spectral response acceleration at 0.2-second period (per Geotechnical Report).
S_1	0.55 g	Design earthquake spectral response acceleration at 1.0-second period (per Geotechnical Report).
Allowable Soil Bearing Capacity for Seismic Loading	1.0 MG: 8,000 psf	Per Geotechnical Report.

The spectral response acceleration parameters shown in Table 4 above are based on an event that has a 2 percent chance of exceedance in the next 50 years, as required by the current building code. This is equivalent to a recurrence period of 2,500 years.

The seismic analysis includes calculation of the design level earthquake forces required to be applied to the reservoir in accordance with AWWA D100-11. The resulting stresses in the structural elements of the reservoir are calculated and compared to their calculated capacities in accordance with AWWA D100-11. Based on these calculations, a number of deficiencies were found and are summarized in Table 5.

TABLE 5

Summary of Results for 1.0 MG Reservoir

Item Evaluated	Result	Notes
Vertical Compression of Shell Due to Overturning Forces	**FAIL**	Overstressed by factor of 1.8
Hoop Stress on Shell Plate	**PASS**	
Tension Force in Anchorage	**FAIL**	Overstressed by factor of 3.6
Freeboard for Sloshing Wave	**FAIL**	Required height: 4.9 ft
Horizontal Sliding	**PASS**	
Soil Bearing Pressure for Case of Seismic Overturning Forces	**FAIL**	Actual 15,400 psf versus allowable of 8,000 psf

1.0 MG Seismic Retrofits

Following is a discussion of the retrofit options for each of the seismic deficiencies noted in the table above and the associated cost estimates.

Vertical Compression of Shell Plate Due to Overturning Forces

Failure of the shell plate under vertical compression results in buckling of the shell plate. Such a failure is also known as "Elephant's Foot" because of the characteristic bulging and folding of the base of the shell often observed on reservoirs after a significant seismic event. Such a failure may cause extreme deformation of the shell and associated damage at manways, piping, and other accessories. Leaks may develop in the shell that would cause loss of the contents of the tank.

An efficient way to address this deficiency is to retrofit the shell of the reservoir with regularly spaced vertical steel "ribs" that start at the base of the shell and extend up to a height as determined by an in-depth seismic analysis and design. According to our analysis, the calculated stresses in the shell plates of the 1.0 MG reservoir are up to 1.8 times larger than the code acceptable stresses.

Table 6 below presents detailed results for the shell courses of the reservoir and show that the retrofit is required for the lower 55 feet of the 1.0 MG reservoir. The retrofit could be accomplished with solid strips of steel plate, tube, or channel running vertically up the tank. These vertical strips would be regularly spaced along the shell and could be installed on the interior or exterior of the shell.

The estimated project cost for this retrofit option is $170,000 to $260,000 for the 1.0 MG reservoir.

TABLE 6

1.0 MG Reservoir Detailed Results for Vertical Compression of Shell Plate

Shell Course ID	Height to Top of Shell Course (ft)	Shell Thickness (in)	Capacity/ Demand	Result
11	81	0.2500	8.58	PASS
10	75	0.2500	4.04	PASS
9	69	0.2500	1.91	PASS
8	63	0.2600	1.12	PASS
7	55	0.2600	0.67	FAIL
6	47	0.3170	0.65	FAIL
5	39	0.3760	0.66	FAIL
4	31	0.4110	0.59	FAIL
3	23	0.4370	0.55	FAIL
2	15	0.5140	0.66	FAIL
1	7	0.5980	0.79	FAIL

Tension Force in Anchor Straps

The anchor straps of the reservoirs are intended to resist uplift forces in the shell that occur during an earthquake. Without adequate anchorage, portions of the reservoir shell and floor could lift off of the foundation repeatedly during an earthquake and cause significant damage.

The existing anchor straps of the 1.0 MG reservoir would be significantly overstressed in a seismic event. According to our analysis, the calculated stress is 3.6 times greater than the code acceptable stress. In addition to the anchor straps being overstressed, the geometry and spacing of the straps do not comply with the requirements of AWWA D100-11. The code states the maximum allowable slope of an anchor strap is 5 degrees from vertical; the actual slope of the anchor straps is approximately 45 degrees. The code also states that the anchors be uniformly spaced at a spacing not to exceed 10 feet; the actual spacing is not uniform, and in some locations, the maximum spacing observed is greater than 11 feet. Because the anchor straps are severely overstressed and not code compliant, the retrofit options assume that the existing anchor straps do not contribute to the anchorage of the tank. Note that many of these straps experienced a rupture failure during the 2001 Nisqually earthquake.

The recommended retrofit to address this item is to remove sections of concrete around the perimeter of the foundation, place anchor bolts, and then fill with concrete. The anchorage would consist of the anchor bolts and welded anchor chairs on the outside face of the reservoir shell. In order to reduce the labor cost of removing sections of existing concrete foundation, the anchor spacing would be maximized and large-diameter anchor bolts would be used. Anchors embedded in epoxy are not an option in this case because the required strength is beyond the practical capacity of any adhesive anchors. The estimated project cost for the anchor bolt retrofit is $210,000 to $300,000.

Freeboard for Sloshing Wave

Based on the existing geometry and location of the overflow at the top of the reservoir shell, the reservoir does not have existing freeboard and the existing freeboard is less than the height of the sloshing wave required by AWWA D100-11. However, this condition should not pose a significant threat to the operation of the reservoir after an earthquake. At worst, the impact of the sloshing wave on the underside of the roof could cause local warping and/or tearing at the roof and would not lead to a catastrophic failure or an immediate loss of a significant volume of storage. Options for achieving adequate freeboard include lowering the typical level of operation or raising the roof of the reservoir. Raising the roof would be costly and would not provide as much benefit as other retrofit items. At this time, it is assumed that no retrofit work will be performed for this item.

Soil Bearing Pressure for Case of Seismic Overturning Forces

There are several risks associated with insufficient foundation bearing area. First, a permanent settlement could occur if an earthquake causes the actual soil bearing pressures to exceed the allowable soil bearing pressure. The settlement could be nonuniform under the foundation, resulting in the reservoir becoming out of plumb. Second, edges of the foundation could lift off the supporting soil momentarily during a design level earthquake, causing additional stress on the reservoir and piping connections to the reservoir.

The concrete foundation for the 1.0 MG reservoir is a 2′-6″ thick mat slab under the entire reservoir with a thickened edge that extends down to 10′-0″ below the top of the foundation. The foundation is in the shape of an octagon with sides that measure about 20.72 feet and an approximate equivalent circle diameter of 50 feet. Unfortunately, this diameter is not adequate to limit the soil bearing pressure – the calculated soil bearing pressure for the seismic load case is 15,400 psf and the allowable soil bearing pressure is 8,000 psf.

Our analysis has found that the diameter of the concrete foundation would need to be increased from 50 feet to 63 feet in order for the calculated soil bearing pressure to not exceed to the allowable soil bearing pressure. Therefore, a new circular ring of concrete foundation that is a maximum of 6.5 feet in width from the existing octagonal foundation and 4 feet deep would need to be added around the existing octagonal-shaped foundation. The new concrete would be anchored to the existing concrete with rebar dowels embedded in holes with epoxy adhesive.

The estimated project cost for the concrete foundation retrofit option is $380,000 to $470,000 for the 1.0 MG reservoir.

Seismic Fittings

The reservoir piping does not currently have flexible seismic fittings between the reservoir inlet/outlet and the distribution system. FLEX-TEND® fittings allow the reservoir and foundation to move independently of the distribution system to reduce the likelihood of the piping breaking at the base of the reservoir and the tank draining. It is recommended that if the reservoir foundation is modified to meet seismic codes, the reservoir existing inlet/outlet and drain and overflow piping be modified with FLEX-TEND fittings installed between the reservoir and vault. If the clearance between the foundation and existing vault is not enough to accommodate the installation of seismic fittings, a new vault should be installed further from the reservoir foundation. A new vault may also include a seismically actuated valve on the inlet/outlet to ensure that the reservoir is not drained by a distribution system main break during a seismic event.

1.0 MG Appurtenances

During site visits, the existing reservoir appurtenances were observed and modifications that would benefit operations and maintenance and meet DOH requirements are proposed herein. Any welding needed on the reservoir should occur prior to installation of the coating system.

The existing reservoir roof vent does not appear to be adequately sized and appears to have been modified in the past to reduce the potential for tampering. It is recommended that the reservoir roof vent including the pressure pallet be replaced with a Newlin's Welding & Tank Maintenance vandal-proof roof vent, sized for the expected flows into and out of the reservoir.

The reservoir roof does not currently have a landing or full-circumference roof railing. Due to the self-supporting dome structure of the roof, the slope of the roof makes roof

access hatch and roof vent access difficult. The addition of a ladder landing, full-circumference railing, and adjustment of the existing access hatch to meet the level of the ladder landing would improve access to the reservoir and safety for operations and maintenance activities. A 36-inch access manway should be added on the opposite side of the reservoir base from the existing 30-inch access manway per AWWA D100-11 standards.

There is evidence of unauthorized access to both the site and reservoir roof. It is recommended that the ladder cage gate at the bottom of the reservoir be replaced to better prevent unauthorized climbing and the reservoir level gauge be replaced with a half-travel gauge that is not accessible from the ground level.

The overflow should be fitted with a flapper or duckbill valve and air gap to meet DOH requirements.

The existing pressure transducer vault on site is known to flood, causing operations and maintenance staff to spend excessive time draining the vault. The vault fills with water from both the ground surface and through holes in the vault floor meant to drain the vault to the subgrade. Were the tank to be taken offline for modifications and recoating, the vault could be replaced with a sealed vault containing a sump and pump that could be piped away from the vault and reservoir.

1.0 MG Summary and Cost Estimates

The 1 MG reservoir exterior coating system is nearing the end of it service life and is not a good candidate for topcoating. The interior coating system shows little visible corrosion. The interior and exterior coating systems are still protecting the steel of the reservoir from significant steel loss. It is recommended that the 1 MG reservoir coating systems be re-evaluated in 3 years and replaced in 3 to 5 years.

The reservoir recoating project costs below include design engineering, construction, construction management, contingency, specialty inspection of the coating application, and taxes. Preliminary cost estimates for recoating and appurtenances are provided in Table 7.

The seismic retrofit of the 1 MG reservoir would require additional foundation concrete as well as new anchor bolts and vertical reinforcing of the tank shell. The reservoir is significantly deficient in these areas and the proposed retrofit options would be required in order for the reservoir to meet current seismic codes. Table 8 summarizes seismic retrofit items and the estimated cost to address the deficiencies.

TABLE 7

1.0 MG Reservoir Preliminary Cost Estimates

Recoating Items	1.0 MG Cost Range
Interior Reservoir Preparation and Recoating	$340,000–$430,000
Exterior Reservoir Preparation and Recoating	$380,000–$500,000
Total Recoating Range	**$720,000–$930,000**
Appurtenances	**1.0 MG Cost Range**
Roof Vent	$20,000
Circumference Roof Railing	$90,000–$100,000
Roof Access Hatch	$30,000
Ladder Cage and Landing	$90,000
36-inch Manway	$45,000
Half-Travel Level Gauge	$30,000
New Vault	$90,000
Inlet/Outlet and Drain/Overflow Modifications	$90,000–$130,000

TABLE 8

1.0 MG Reservoir Summary of Seismic Retrofit Items with Estimated Construction Costs

Item	1.0 MG Cost Range
Vertical Ribs to Address Buckling Failure of Shell Plate	$170,000–$260,000
Added Anchor Bolts at Connection of Shell to Foundation	$210,000–$300,000
Foundation Retrofit to Address Soil Bearing Pressure for Case of Seismic Overturning Forces	$380,000–$470,000
Total Range	**$760,000–$1,030,000**

The estimated cost range if the recoating, all appurtenances, and seismic retrofit are done is $1,965,000 to $2,485,000. The cost of constructing a new welded steel tank with similar dimensions, including design, construction management, and tax, is estimated to be $2,430,000.

1.5 MG RESERVOIR

Background

The City's 1.5 MG High School Reservoir is an 89-foot-tall, 53-foot-diameter welded steel standpipe constructed in 1989 located on the northwest side of the playfields of

Bainbridge Island High School. The reservoir sits atop a circular foundation footing that extends 6′-6″ from the reservoir shell and is 3 feet deep. The reservoir is secured to the foundation with 64 anchor bolts and chairs. The anchor chairs consist of two vertical 1-inch-thick plates with a 6″ x 7″ x 2″ thick plate on top. Each 2.5-inch-diameter anchor bolt is cast in the foundation and is aligned vertically through the 2-inch-thick top plate, 3 inches from the reservoir shell, where it is then fastened with a threaded nut.

The reservoir sill plate is 0.25-inch thick and is continuous for the entire reservoir circumference. A space of approximately 0.75 inch between the sill plate and the reservoir foundation is filled with grout, finished at a 45-degree angle. The reservoir has two 24-inch-diameter reservoir manways.

A site visit to the reservoir was conducted on November 2, 2017, for interior and exterior coating evaluation and to set the adhesion test dollies. A follow-up site visit was conducted on December 4 to pull the adhesion test dollies, expose the reservoir foundation, conduct the geotechnical investigation, and collect shell steel thickness readings.

Ryan Hale, Myron Basden, and Alex Quinn of Gray & Osborne performed the reservoir assessments and inspection during the site visits. Adhesion tests were conducted utilizing a DeFelsko PosiTest Adhesion Tester Model AT-M Serial Number AT10353, in accordance with ASTM D4541 Test Method E. Steel shell thicknesses were collected utilizing a DeFelsko PosiTector Serial Number 774455 with a UTG-C Probe with Serial Number 256944.

1.5 MG Exterior Coating Evaluation

The original coating system of the 1.5 MG reservoir is unknown. The exterior appears to have been topcoated up to six times based on the coating samples collected and Figure 2B. A sample of the exterior coating system was collected and analyzed for RCRA 8 metals. The coating does contain RCRA 8 metals including barium and lead, but not at concentrations significant enough to cause concern or impact costs. Additional coating samples should be taken and analyzed for RCRA 8 metals prior to removal and disposal. The exterior topcoat was observed to be delaminating from the previous coating system, most notably on the bottom of the reservoir near the sill and anchor chairs. In other places, the topcoat and previous coating systems were found to be well adhered to the reservoir. On the roof, the coating system was observed to have been vandalized including paint spills and graffiti. Photographs of the reservoir exterior coating condition are included as Figure 2.

A: RESERVOIR ANCHOR BOLTS

B: COATING SAMPLE SHOWING MULITPLE RECOATS

C: RESERVOIR ROOF INCLUDING ROOF ACCESS HATCH

D: RESERVOIR ROOF VENT AND GRAFFITI

CITY OF BAINBRIDGE ISLAND

2018 WATER SYSTEM IMPROVEMENTS PROJECT

1.5 MG HIGH SCHOOL RESERVOIR COATING CONDITION EVALUATION

FIGURE 2

Gray & Osborne, Inc.

L:\Bainbridge Island\17614 2017 Water System Improvements\RESERVOIR\1.5 Figure 2.mxd

Pull-off adhesion tests were performed in compliance with ASTM D4541 Test Method E and were performed utilizing a DeFelsko PosiTest adhesion tester. Three dollies were set on the reservoir sides with their location, pull strength in psi, and failure location/type reported in Table 9. The failures were a majority of adhesion failures and one dolly failed to adhere to the reservoir properly. Due to the number of past topcoats of the reservoir, only pertinent coating layers are listed. The coating layers are as follows:

- A – Tie Coat: coating applied over the previous topcoat system to allow the next topcoat to adhere to the previous topcoat system
- B – Previous Topcoat: topcoat applied to the reservoir
- C – Tie Coat: coating applied over the previous topcoat system to allow the next topcoat to adhere to the previous topcoat system
- D – Topcoat: final coat applied to the reservoir
- Y – Adhesive: used to adhere the testing dolly to the reservoir coatings
- Z – Dolly: metal testing implement adhered to reservoir coatings

Failures listed with just one location, for example "A," are cohesive failures that resulted from the internal cohesion of a single layer of coating failing. Failures listed equally between two layers, for example "A/B," are adhesive failures between coating layers that resulted in one coating being pulled from the other. An adhesion test report is contained in Attachment A.

TABLE 9

1.5 MG Reservoir Pull-Off Adhesion Test Results

Dolly ID	Location	Pull Strength (psi)	Failure Location/Type
1	Side of Reservoir	1,516	85% D, 15% B
2	Side of Reservoir	1,443	100% D
3	Side of Reservoir	1,215	100% D
4	Top of Reservoir	953	50% D, 50% C/B
5	Top of Reservoir	1,640	75% C, 25% C/B
6	Top of Reservoir	—	100% Y

The exterior coating failures occurred mostly in the most recent two topcoat layers. An effective coating system will fail at or above approximately 1,500 psi. Failures that occur below this pressure signal that the reservoir coating is nearing the end of its effective service life. Based on the pull strength results presented above, the reservoir exterior coating system generally has effective service life left and should be re-evaluated in 5 years to track the degradation of the coating system.

1.5 MG Interior Coating Evaluation

A sample of the interior coating system was collected and analyzed for RCRA 8 metals. The coating does contain RCRA 8 metals including barium and lead, but not at concentrations significant enough to cause concern or impact costs. Additional coating samples should be taken and analyzed for RCRA 8 metals prior to removal and disposal. The interior of the reservoir was not examined by a floating inspection because the roof is a self-supporting dome and had little to no apparent corrosion upon visual inspection from the roof hatch. Corrosion was evident on the ladder and roof hatch; however, the corrosion is not extensive.

1.5 MG Coating Recommendations

The pull-off adhesion tests show the exterior coating system to have effective service life left and we recommend that the reservoir condition be evaluated again in 5 years. While the majority of failures occurred at a non-critical pressure, poor adhesion in select locations of previous topcoats was exhibited, requiring our recommendation that when recoating is performed the exterior of the reservoir be blasted to bare metal and recoated. Corrosion or metal loss of the reservoir structure was not observed at this time and the reservoir should be re-evaluated in 5 years, and the subsequent inspection interval adjusted according to the condition present at that time.

Due to the mild corrosion on the interior, the critical coating is the exterior and a new interior coating system can be applied when the exterior coating system is applied. The interior coating should be re-evaluated in 5 years along with the exterior.

In the event of seismic upgrades that require welding to the reservoir structure, the reservoir should be blasted to bare metal and recoated as part of the project as the welding will burn off the existing coatings.

1.5 MG Seismic Evaluation – Introduction

Table 10 is a summary of geometry data gathered from the field and from original drawings.

TABLE 10

1.5 MG Reservoir Geometry

Item	Description
Diameter	53 ft
Height	90 ft
Foundation Type	Circular mat foundation
Foundation Diameter	66 ft
Foundation Thickness	3 ft
Anchor Type	Bolts
Anchor Quantity and Size	64 – 2-1/2-inch diameter

The thickness of the steel shell ring plates was measured using an ultrasonic thickness gauge. The gauge utilizes a non-destructive ultrasonic pulse-echo principle to measure the wall thickness. The gauge will accurately measure the thickness of an uncoated steel plate; however, the reservoirs have a coating over the steel which has a much lower density than the steel plate. The lower density causes the sound pulse to travel slower both as it pulses through the coating material and on the echo return to the gauge. Since the thickness is calculated using the time required for the echo to return to the gauge, the thickness will appear to be slightly greater than the actual thickness. Since coating thickness around the tank can be expected to vary, the amount of extra thickness shown on an individual reading due to the effects of sound travelling through the coating can be expected to vary as well, although this is assumed to be a negligible amount. The results of the testing are provided in Table 11. All readings were taken from within reach of the access ladder so the entire circumference of each ring was not examined. Steel shell courses are 7 feet tall.

TABLE 11

1.5 MG Reservoir Measured Thickness of Shell Plate

Shell Course ID	Height to Top of Shell Course (ft)	Shell Thickness (in)
1	90	0.25
2	82	0.25
3	74	0.25
4	66	0.25
5	58	0.27
6	50	0.34
7	42	0.35
8	35	0.44
9	28	0.49
10	21	0.54
11	14	0.58
12	7	0.72

1.5 MG Seismic Analysis Parameters

The reservoir has been analyzed for the seismic requirements of AWWA D100-11 "Welded Carbon Steel Tanks for Water Storage." Table 12 provides a summary of the seismic design parameters used for the analysis.

TABLE 12

1.5 MG Reservoir Seismic Design Parameters

Description	Value	Notes
Seismic Use Group	III	Required for facilities deemed essential for post-earthquake recovery, e.g., post-earthquake fire suppression.
Seismic Importance Factor, I_e	1.5	Determined by Seismic Use Group.
S_S	1.40 g	Design earthquake spectral response acceleration at 0.2-second period (per Geotechnical Report).
S_1	0.55 g	Design earthquake spectral response acceleration at 1.0-second period (per Geotechnical Report).
Allowable Soil Bearing Capacity for Seismic Loading	1.5 MG: 5,333 psf	Per Geotechnical Report.

The spectral response acceleration parameters shown in Table 12 above are based on an event that has a 2 percent chance of exceedance in the next 50 years, as required by the current building code. This is equivalent to a recurrence period of 2,500 years.

The seismic analysis includes calculation of the design level earthquake forces required to be applied to the reservoir in accordance with AWWA D100-11. Then the resulting stresses in the structural elements of the reservoir are calculated and compared to their calculated capacities in accordance with AWWA D100-11. Based on these calculations, a number of deficiencies were found and are summarized in Table 13.

TABLE 13

Summary of Results for 1.5 MG Reservoir

Item Evaluated	Result	Notes
Vertical Compression of Shell Due to Overturning Forces	**FAIL**	Overstressed by factor of 1.7
Hoop Stress on Shell Plate	**PASS**	
Tension Force in Anchor Bolts	**FAIL**	Overstressed by factor of 1.15
Freeboard for Sloshing Wave	**FAIL**	Required height: 5.2 ft
Horizontal Sliding	**PASS**	
Soil Bearing Pressure for Case of Seismic Overturning Forces	**FAIL**	Actual 11,000 psf versus allowable of 5,333 psf

1.5 MG Seismic Retrofits

Following is a discussion of the retrofit options for each of the seismic deficiencies noted in the table above and the associated cost estimates.

Vertical Compression of Shell Plate Due to Overturning Forces

Failure of the shell plate under vertical compression results in buckling of the shell plate. Such a failure is also known as "Elephant's Foot" because of the characteristic bulging and folding of the base of the shell often observed on reservoirs after a significant seismic event. Such a failure may cause extreme deformation of the shell and associated damage at manways, piping, and other accessories. Leaks may develop in the shell that would cause loss of the contents of the tank.

An efficient way to address this deficiency is to retrofit the shell of the reservoir with regularly spaced vertical steel "ribs" that start at the base of the shell and extend up to a height as determined by an in-depth seismic analysis and design. According to our analysis, the calculated stresses in the shell plates of the 1.5 MG reservoir are up to 1.7 times greater than code acceptable stresses.

Table 14 below presents detailed results for the shell courses of the reservoir and show that a retrofit is required for the lower 58 feet of the 1.5 MG reservoir. The retrofit could be accomplished with solid strips of steel plate, tube, or channel running vertically up the tank. These vertical strips would be regularly spaced along the shell and could be installed on the interior or exterior of the shell.

The estimated project cost for this retrofit option is $260,000 to $340,000 for the 1.5 MG reservoir.

TABLE 14

1.5 MG Reservoir Detailed Results for Vertical Compression of Shell Plate

Shell Course ID	Height to Top of Shell Course (ft)	Shell Thickness (in)	Capacity/ Demand	Result
12	90	0.2500	6.86	PASS
11	82	0.2500	3.85	PASS
10	74	0.2500	2.04	PASS
9	66	0.2500	1.16	PASS
8	58	0.2730	0.86	FAIL
7	50	0.3350	0.87	FAIL
6	42	0.3460	0.62	FAIL
5	35	0.4410	0.73	FAIL
4	28	0.4860	0.67	FAIL
3	21	0.5400	0.66	FAIL
2	14	0.5770	0.60	FAIL
1	7	0.7190	0.82	FAIL

Tension Force in Anchor Bolts

The anchor straps and anchor bolts of the reservoir are intended to resist uplift forces in the shell that occur during an earthquake. Without adequate anchorage, portions of the reservoir shell and floor could lift off of the foundation repeatedly during an earthquake and cause significant damage.

The existing anchor bolts for the 1.5 MG reservoir would be slightly overstressed in a seismic event. The code required stress is only 1.15 times greater than the reservoir's calculated acceptable stress capacity. The required stress is based on the forces that are expected to develop during a seismic event with a return period of 2,500 years. The analysis shows that the anchor bolts have sufficient strength to resist the forces that would develop during an earthquake with a return period of approximately 1,500 years, but not 2,500 years.

If desired by the City, additional anchor bolts could be added in order to resist the 2,500-year earthquake in accordance with AWWA D100-11. The additional anchorage would consist of anchor bolts and welded anchor chairs on the outside face of the

reservoir. The anchor bolts could be drilled and epoxied into the existing concrete foundation. This option would require an anchor bolt between every pair of existing anchor bolts, with a diameter of approximately 1 inch. The estimated project cost for this option is $170,000 to $260,000.

Freeboard for Sloshing Wave

Based on the existing geometry and location of the overflow at the top of the reservoir shell, the reservoir does not have existing freeboard and the existing freeboard is less than the height of the sloshing wave required by AWWA D100-11. However, this condition should not pose a significant threat to the operation of the reservoir after an earthquake. At worst, the impact of the sloshing wave on the underside of the roof could cause local warping and/or tearing at the roof and would not lead to a catastrophic failure or an immediate loss of a significant volume of storage. Options for achieving adequate freeboard include lowering the level of operation or raising the roof of the reservoir. Raising the roof would be costly and would not provide as much benefit as other retrofit items. At this time, it is assumed that no retrofit work will be performed for this item.

Soil Bearing Pressure for Case of Seismic Overturning Forces

There are several risks associated with insufficient foundation bearing area. First, a permanent settlement could occur if an earthquake causes the actual soil bearing pressures to exceed the allowable soil bearing pressure. The settlement could be nonuniform under the foundation, resulting in the reservoir becoming out of plumb. Second, edges of the foundation could lift off the supporting soil momentarily during a design level earthquake, causing additional stress on the reservoir and piping connections to the reservoir.

The concrete foundation for the 1.5 MG reservoir is a 3-foot-thick mat slab under the entire reservoir. The circular foundation has a diameter of 66 feet. This diameter is not adequate to limit the soil bearing pressure – the calculated bearing pressure for the seismic load case is 11,000 psf and the allowable bearing pressure is 5,300 psf.

Our analysis has found that the diameter of the concrete foundation would need to be increased from 66 feet to 87 feet in order for the calculated soil bearing pressure to not exceed to the allowable soil bearing pressure. Therefore, a new ring of concrete foundation that is 10.5 feet in width and 3 feet deep would need to be added around the existing foundation. The layout of the retrofitted concrete foundation would need to be coordinated with existing piping, electrical, and other utilities in the ground surrounding the reservoir. The new concrete would be anchored to the existing concrete with rebar dowels embedded in holes with epoxy adhesive.

The estimated project cost for the concrete foundation retrofit option is $550,000 to $640,000 for the 1.5 MG reservoir.

The extent of added concrete could be reduced by using pile foundations as part of the foundation retrofit. However, our preliminary review of a pile-supported option indicates it would not reduce the construction cost as compared to the option described above.

Seismic Fittings

The reservoir piping does not currently have flexible seismic fittings between the reservoir inlet/outlet and the distribution system. FLEX-TEND fittings allow the reservoir and foundation to move independently of the distribution system to reduce the likelihood of the piping breaking at the base of the reservoir and the tank draining. It is recommended that if the reservoir foundation is modified to meet seismic codes, the reservoir inlet/outlet and drain/overflow piping be upgraded, a new vault installed, and FLEX-TEND fittings be installed between the reservoir and new vault. A new vault may also include a seismically actuated valve on the inlet/outlet to ensure that the reservoir is not drained by a distribution system main break during a seismic event.

1.5 MG Appurtenances

During site visits, the existing reservoir appurtenances were observed and modifications that would benefit operations and maintenance and meet DOH requirements are proposed herein. Any welding needed on the reservoir should occur prior to installation of the coating system.

The existing reservoir roof vent does not meet current regulations and appears to have been modified in the past to reduce the potential for tampering. It is recommended that the reservoir roof vent including the pressure pallet be replaced with a Newlin's Welding & Tank Maintenance vandal-proof roof vent, sized according to DOH requirements.

The reservoir roof does not currently have a full-circumference roof railing and it is recommended that a full-circumference roof railing be installed. Additionally, the reservoir roof access hatch was observed to be secured with zip ties and it is recommended that the hatch be replaced. The existing 24-inch reservoir access manways should be replaced with 36-inch access manways to improve accessibility per AWWA D100-11 standards. The overflow should be fitted with a flapper or duckbill valve and air gap to meet DOH requirements.

During the initial site visit, it was observed that the City's Saf-T-Climb™ fall protection system carriers, LAD-SAF Model 6116500, have been recalled due to reported incidents and potential misuse scenarios. The recall notice issued by 3M Fall Protection has been included in Attachment B.

1.5 MG Summary and Cost Estimates

The 1.5 MG reservoir exterior coating system has effective service life left and is not a good candidate for topcoating. The interior coating system shows mild corrosion. The interior and exterior coating systems are still protecting the steel of the reservoir from significant steel loss. It is recommended that the 1.5 MG reservoir be re-evaluated in 5 years.

The project costs below include design engineering, construction, construction management, specialty inspection of the coating application, and taxes. Preliminary cost estimates for recoating and appurtenances are provided in Table 15.

The seismic retrofit of the 1.5 MG reservoir would require additional foundation concrete as well as vertical reinforcing of the tank shell. The reservoir is significantly deficient in these areas and the proposed retrofit options would be required in order for the reservoir to meet current seismic codes. At the option of the City, additional anchor bolts may be installed in order to address the slight deficiency of the existing anchorage. Table 16 summarizes seismic retrofit items and the estimated cost to address the deficiencies.

TABLE 15

1.5 MG Reservoir Preliminary Cost Estimates

Recoating Items	1.5 MG Cost Range
Interior Reservoir Preparation and Recoating	$430,000–$530,000
Exterior Reservoir Preparation and Recoating	$500,000–$610,000
Total Recoating Range	**$930,000–$1,140,000**
Appurtenances	**1.5 MG Cost Range**
Roof Vent	$20,000
Circumference Roof Railing	$100,000–$120,000
Roof Access Hatch	$30,000
36-inch Manways (2)	$90,000
New Vault	$90,000
Inlet/Outlet and Drain/Overflow Modifications	$90,000–$130,000

TABLE 16

1.5 MG Reservoir Summary of Seismic Retrofit Items with Estimated Construction Costs

Item	1.5 MG Cost Range
Vertical Ribs to Address Buckling Failure of Shell Plate	$260,000–$340,000
Added Anchor Bolts at Connection of Shell to Foundation	$170,000–$260,000
Foundation Retrofit to Address Soil Bearing Pressure for Case of Seismic Overturning Forces	$550,000–$640,000
Total Range	**$980,000–$1,240,000**

The approximate cost range for recoating, all appurtenances, and seismic retrofit is $2,330,000 to $2,860,000. The project cost of constructing a new tank with similar dimensions, including design, construction management, and tax, is estimated to be $3,170,000.

ATTACHMENT A

ADHESION TEST REPORTS

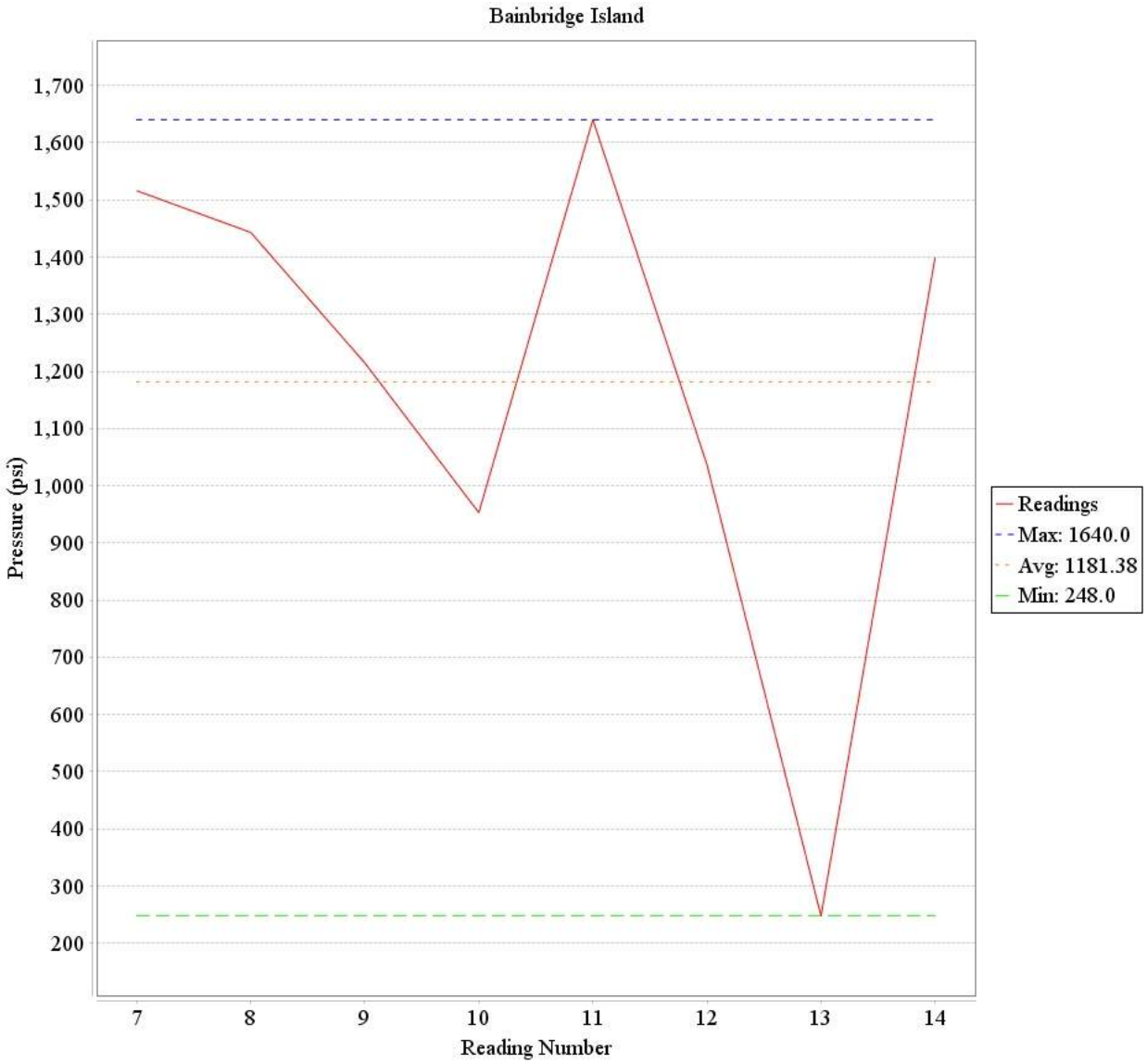
Bainbridge Island

Created:	2017-12-07 09:12:57
PosiTest AT-M S/N:	10353

7-9: 1.5 MG Base
10-11: 1.5 MG Roof
12-14: 1.0 MG Base

Readings

#	Pressure (psi)	
7	1516.0	Rate 126.9 psi/sec, Dur. 11.15 sec, Dolly 20 mm.
8	1443.0	Rate 123.5 psi/sec, Dur. 10.87 sec, Dolly 20 mm.
9	1215.0	Rate 112.3 psi/sec, Dur. 9.92 sec, Dolly 20 mm.
10	953.0	Rate 118.8 psi/sec, Dur. 7.18 sec, Dolly 20 mm.
11	1640.0	Rate 122.5 psi/sec, Dur. 12.57 sec, Dolly 20 mm.
12	1037.0	Rate 132.1 psi/sec, Dur. 7.09 sec, Dolly 20 mm.
13	248.0	Rate 130.9 psi/sec, Dur. 1.13 sec, Dolly 20 mm.
14	1399.0	Rate 132.1 psi/sec, Dur. 9.83 sec, Dolly 20 mm.


Bainbridge Island
Pressure (psi)
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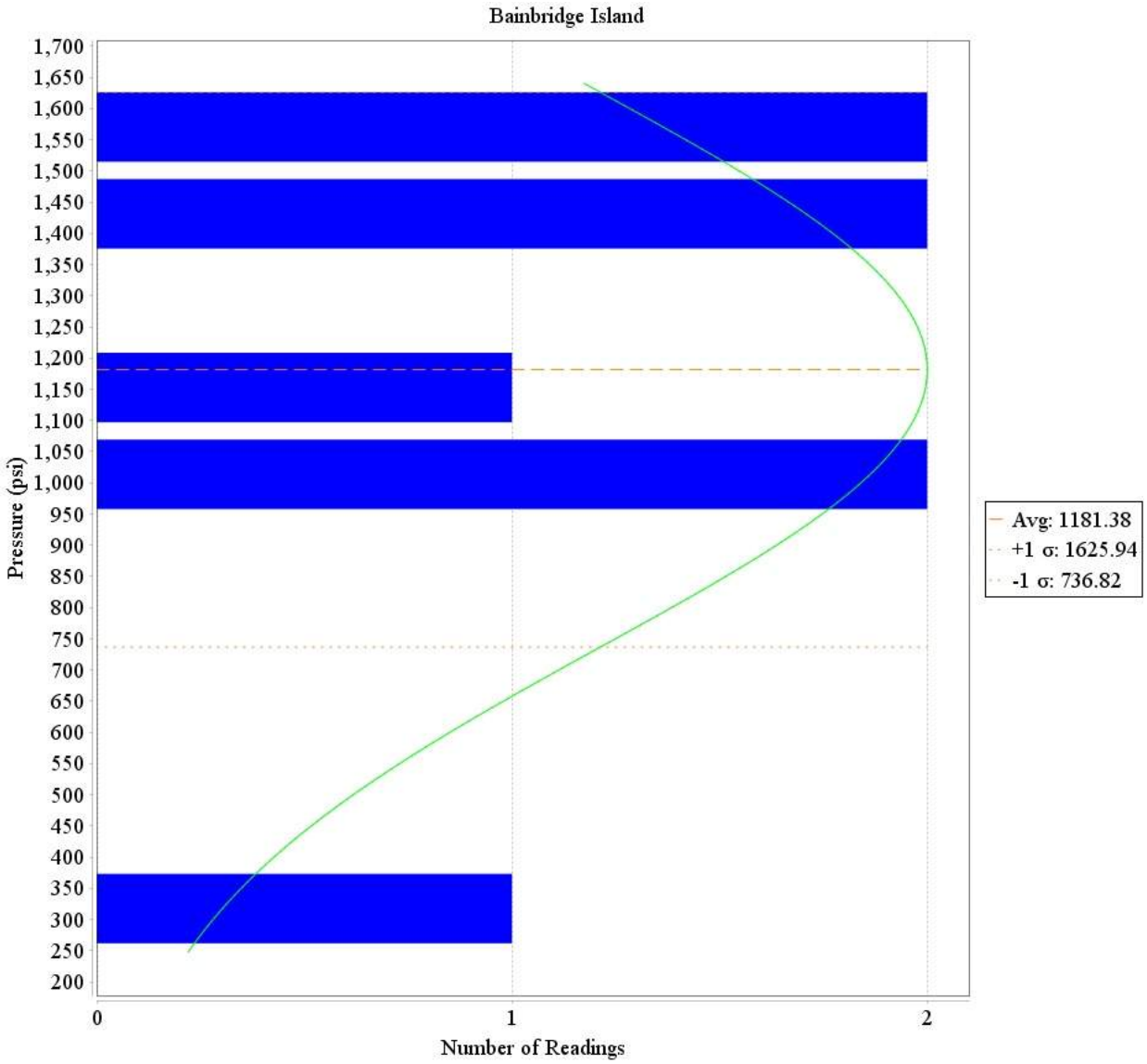
1,600
1,500
1,400
1,300
1,200
1,100
1,000
900
800
700
600
500
400
300
200
7
8
9
10
11
12
13
14
Reading Number
Readings
Max: 1640.0
Avg: 1181.38
Min: 248.0


Bainbridge Island
Pressure (psi)
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1,050
1,000
950
900
850
800
750
700
650
600
550
500
450
400
350
300
250
200
0
1
2
Number of Readings
Avg: 1181.38
+1 σ: 1625.94
-1 σ: 736.82

Bainbridge Island

Created:	2017-12-07 09:12:57
PosiTest AT-M S/N:	10353

7-9: 1.5 MG Base
10-11: 1.5 MG Roof
12-14: 1.0 MG Base

Readings

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9	1215.0	Rate 112.3 psi/sec, Dur. 9.92 sec, Dolly 20 mm.
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11	1640.0	Rate 122.5 psi/sec, Dur. 12.57 sec, Dolly 20 mm.
12	1037.0	Rate 132.1 psi/sec, Dur. 7.09 sec, Dolly 20 mm.
13	248.0	Rate 130.9 psi/sec, Dur. 1.13 sec, Dolly 20 mm.
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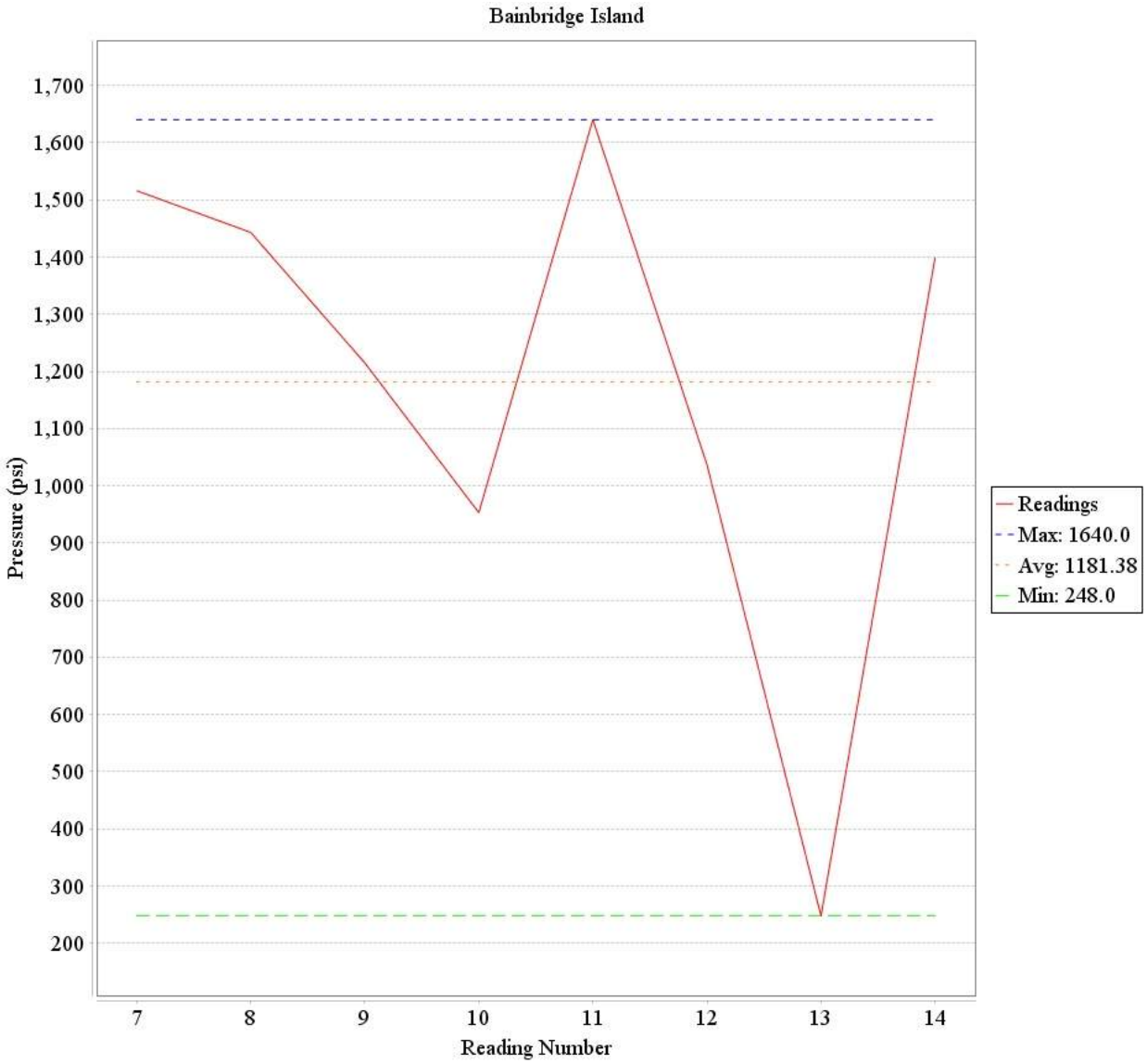
Bainbridge Island
Pressure (psi)
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1,600
1,500
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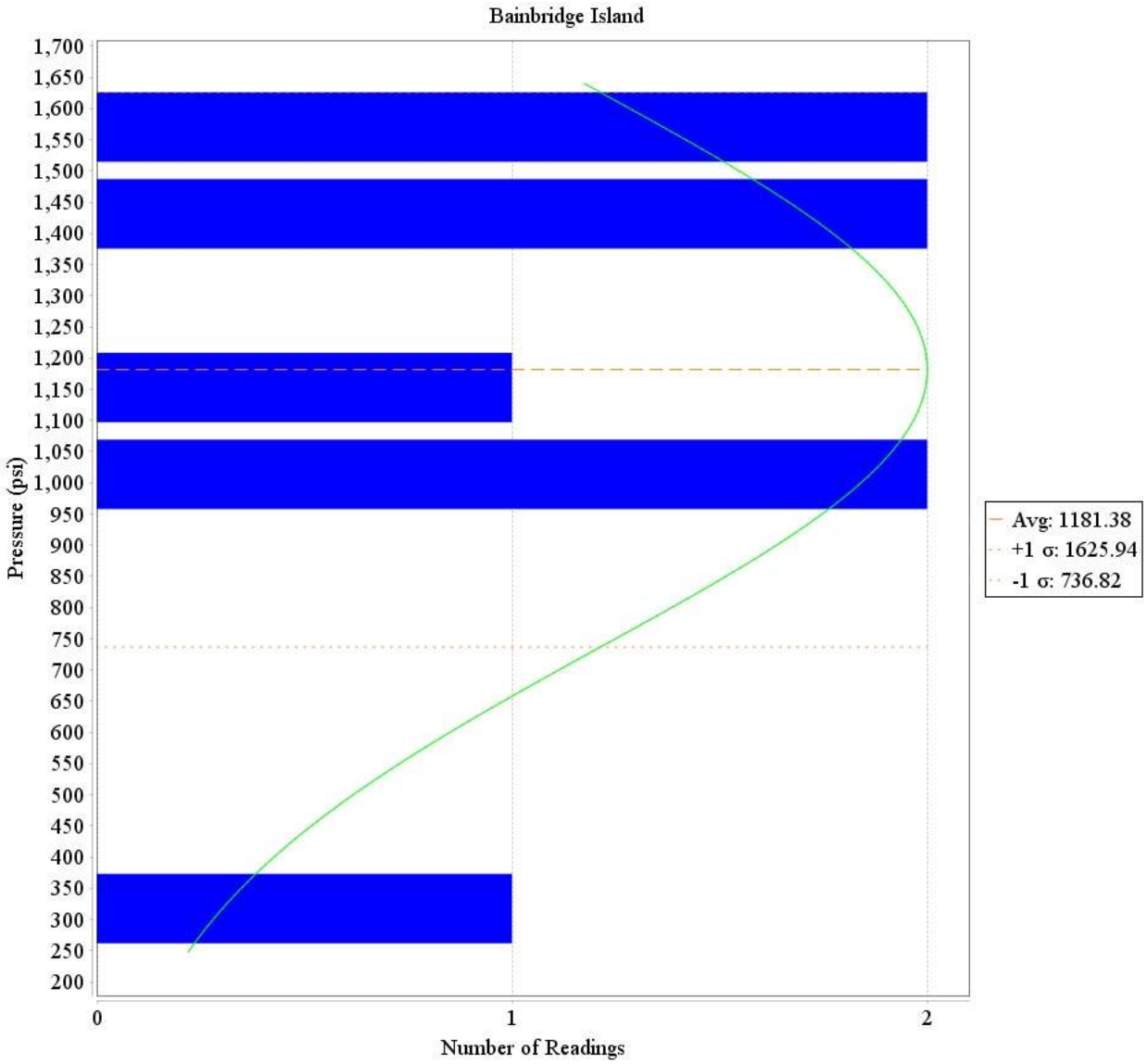
14
Reading Number
Readings
Max: 1640.0
Avg: 1181.38
Min: 248.0


Bainbridge Island
Pressure (psi)
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1,600
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1,250
1,200
1,150
1,100
1,050
1,000
950
900
850
800
750
700
650
600
550
500
450
400
350
300
250
200
0
1
2
Number of Readings
Avg: 1181.38
+1 σ: 1625.94
-1 σ: 736.82

ATTACHMENT B

SAF-T-CLIMB RECALL NOTICE

3M Fall Protection

3833 SALA Way
Red Wing, MN 55066
800 328 6146

now proudly part of 3M:

PRODUCT STOP USE AND RECALL / REPLACEMENT NOTICE
IMMEDIATE ACTION REQUIRED

DBI-SALA Lad-Saf™ Sleeve

Dear 3M Customer:

After more than 30 years of use in the fall protection industry, the original Lad-Saf™ sleeve has been replaced by a completely redesigned next generation Lad-Saf sleeve.

Capital Safety/3M recently reviewed the performance of the original Lad-Saf sleeve in the field, including a limited number of incidents involving a serious injury or death in the United States while using the sleeve. Although our review did not reveal product hazard or risk scenarios that would arise in the ordinary and proper use of the product, it did reveal potential misuse scenarios that could result in serious injury or death. The potential misuse scenarios include interference with the braking mechanism (such as entanglement with cords, lanyards, clothing or other materials, or grasping the sleeve prior to or during a fall), or result from the user attaching the sleeve upside down (user inversion). No safety regulator has made a finding that the design of the original Lad-Saf sleeve is defective.

At 3M, customer safety and confidence are high priorities. In light of the reported incidents and potential misuse scenarios, we have discontinued sale of the original Lad-Saf sleeve, and are voluntarily initiating a full recall of all original Lad-Saf sleeves. Owners / Users of original Lad-Saf sleeves must:

1. **Immediately stop using and quarantine all original Lad-Saf sleeves.** Affected part numbers are**:** 6100016, 6116500, 6116501, 6116502, 6116503, 6116504, 6116505, 6116506, 6116507, 6116509, 6116512, 6116535, 6116540, 6116541, 6116542, 6116500C, 6116500SM, 6116507/A, 6116540b

2. Contact 3M Customer Services at 1-800-328-6146 (ext. 2012), or email us at LADSAFNA@mmm.com to discuss the replacement of your returned units with an X2 or X3 sleeve, depending on your needs, at no cost to you.

3M remains committed to providing quality products and services to our customers. We apologize for any inconvenience that this situation may cause you, but we are confident that you will be very pleased with the latest generation X2 and X3 Lad-Saf sleeves to keep your workers safe at height. We appreciate your continued support of 3M Fall Protection products and services.

3M Fall Protection - August 30, 2016

APPENDIX C

BUDGETARY CONSTRUCTION COST ESTIMATES

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS PROJECT REPORT
PRELIMINARY PROJECT COST ESTIMATE

Alternative 1: Service for New Zone w/Satellite Booster Station (Welded Steel Tank)
June 9, 2020
G&O# 19648.00

NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	AMOUNT
1	Mobilization and Demobilization	1	LS	$ 727,500	$ 727,500
2	Sitework / Site Restoration	1	LS	$ 350,000	$ 350,000
3	Reservoir 1 Booster Station	1	LS	$ 400,000	$ 400,000
4	New Brooklyn Booster Station	2	EA	$ 450,000	$ 900,000
5	Connection to Existing System	3	EA	$ 10,000	$ 30,000
6	Piping, Valves, and Appurtenances	1	LS	$ 300,000	$ 300,000
7	Electrical	1	LS	$ 600,000	$ 600,000
8	Programming and Integration	1	LS	$ 75,000	$ 75,000
9	3.3MG Welded Steel Reservoir	1	LS	$ 4,620,000	$ 4,620,000
				Subtotal	**$ 8,002,500**
				Contingency (30%)	$ 2,400,800
				Subtotal	**$ 10,403,300**
				Washington State Sales Tax (9.0%)	$ 936,300
				Subtotal	**$ 11,339,600**
				Design and Project Administration (25.0%)	$ 2,834,900
				TOTAL CONSTRUCTION COST	**$ 14,175,000**

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS PRE-DESIGN REPORT
PRELIMINARY PROJECT COST ESTIMATE

Alternative 2: Raise HGL of Existing High Zone via new Welded Steel Reservoir
June 9, 2020
G&O# 19648.00

NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	AMOUNT
1	Mobilization and Demobilization	1	LS	$ 573,500	$ 573,500
2	Sitework / Site Restoration	1	LS	$ 250,000	$ 250,000
3	Pressure Reducing Valve Station	2	EA	$ 100,000	$ 200,000
4	Connection to Existing System	1	EA	$ 10,000	$ 10,000
5	Piping, Valves, and Appurtenances	1	LS	$ 200,000	$ 200,000
6	Electrical	1	LS	$ 250,000	$ 250,000
7	Programming and Integration	1	LS	$ 65,000	$ 65,000
8	3.4MG Welded Steel Reservoir	1	LS	$ 4,760,000	$ 4,760,000
				Subtotal	**$ 6,308,500**
				Contingency (30%)	$ 1,892,600
				Subtotal	**$ 8,201,100**
				Washington State Sales Tax (9.0%)	$ 738,100
				Subtotal	**$ 8,939,200**
				Design and Project Administration (25.0%)	$ 2,234,800
				TOTAL CONSTRUCTION COST	**$ 11,174,000**

CITY OF BAINBRIDGE ISLAND

RESERVOIR 1 IMPROVEMENTS PRE-DESIGN REPORT
PRELIMINARY PROJECT COST ESTIMATE

Alternative 3: Raise HGL of Existing High Zone via Elevated Tank (Hydropillar)
June 9, 2020
G&O# 19648.00

NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	AMOUNT
1	Mobilization and Demobilization	1	LS	$ 547,500	$ 547,500
2	Sitework / Site Restoration	1	LS	$ 250,000	$ 250,000
3	Pressure Reducing Valve Station	2	EA	$ 100,000	$ 200,000
4	Connection to Existing System	1	EA	$ 10,000	$ 10,000
5	Piping, Valves, and Appurtenances	1	LS	$ 200,000	$ 200,000
6	Electrical	1	LS	$ 250,000	$ 250,000
7	Programming and Integration	1	LS	$ 65,000	$ 65,000
8	2.0MG Hydropillar	1	LS	$ 4,500,000	$ 4,500,000
				Subtotal	**$ 6,022,500**
				Contingency (30%)	$ 1,806,800
				Subtotal	**$ 7,829,300**
				Washington State Sales Tax (9.0%)	$ 704,600
				Subtotal	**$ 8,533,900**
				Design and Project Administration (25.0%)	$ 2,133,500
				TOTAL CONSTRUCTION COST	**$ 10,667,000**

APPENDIX D

PRELIMINARY SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING DESIGN

ASSOCIATED EARTH SCIENCES, INC

BAINBRIDGE ISLAND OFFICE
179 Madrone Lane North
Bainbridge Island, WA 98110
(206) 780-9370
FAX (206) 780-9438

CORPORATE OFFICE
911 Fifth Avenue, Suite 100
Kirkland, Washington 98033
(425) 827-7701
FAX (425) 827-5424

May 8, 2000
AESI Project No. BE99103A

City of Bainbridge Island
Public Works Department
Engineering Division
280 Madison Avenue North
Bainbridge Island, Washington 98110

RECEIVED
MAY 10 2000
PW/ENG
cc: MG

Attention: Mr. Jeffrey M. Jensen, P.E.
City Engineer

Subject: **PRELIMINARY SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING DESIGN**
Existing High School Water Tank Modification
Bainbridge Island, Washington

CORR LOG
#2965

Dear Mr. Jensen:

This letter presents the results of our preliminary geotechnical investigation of the footings for the City of Bainbridge Island water tank, located north and west of the Bainbridge High School. We received written authorization to proceed with the project from you on July 22, 1999.

INTRODUCTION

The City operates two steel aboveground water storage tanks on a parcel of City property located west of Bainbridge High School, between High School Road and New Brooklyn Road. The top of one tank is currently about 7 feet higher than the other tank. Since the two tanks are interconnected, the capacity of the combined system is currently limited to the maximum height of the lower tank.

The City is investigating the possibility of increasing the storage capacity of the system by raising the height of the lower tank to match the height of the higher tank. Several alternatives have been discussed. One alternative is to simply add a standpipe to the lower tank to accommodate the additional height of the water column that would result from fully filling the higher tank. This option will only add storage capacity to the higher tank by allowing it to be filled completely. Another alternative is to increase the height of the lower tank by about 7 feet, thereby increasing the usable capacity of both tanks.

The purpose of this project is to determine if the footings of the existing lower tank are capable of supporting the additional load, under static and seismic conditions, presented by an additional 7 feet of tank and water. Currently, AESI has completed a series of subsurface explorations and soil classifications, and formulated a preliminary assessment of the existing conditions. We understand that additional information may be forwarded from the project structural engineer, if necessary. Due to current project requirements, AESI has been requested to compile our existing data and evaluations completed to date and prepare this summary report.

EXISTING CONDITIONS

The existing water tanks occupy a slight topographic rise located north of the high school. The lower of the two steel tanks (the tank) is the subject of this investigation. The tank was estimated to be about 80 feet high and about 46 feet in diameter. The tank rests on an octagonal, cast concrete footing with a total overall diameter of about 50 feet. Each octagonal footing face was measured at about 21½ feet across. The tank is constructed of welded steel plates. The tank appears to be held onto the concrete footing with steel bands that are cast into the concrete and welded to the steel tank.

The City has no records of the tank construction. City personnel believe the tank was built in the 1970's. One City employee remembered that the backfill around the tank footing settled somewhat after construction.

SUBSURFACE INVESTIGATIONS

Field investigations were conducted in two phases. The first phase consisted of backhoe explorations. The backhoe was used first to examine the existing footings and then to explore soil conditions in the event that undisturbed natural bearing soils were present at a relatively shallow depth. After completing the backhoe exploration program, it was determined that deeper subsurface information was required and a drill rig was used to collect deep soil and ground water level information. Results of the subsurface exploration program are described below. The locations of the explorations are presented on Figure 1, the *Site and Exploration Plan.*

A rubber tire backhoe operated by the City of Bainbridge Island excavated three exploration pits around the perimeter of the tank to investigate the nature of the face of the footing, the fill soils around the footing, and the soils below the bottom of the footing. McDonald Drilling of Milton, Washington, using a truck-mounted hollow-stem-auger-drilling rig, assisted with collection of deeper subsurface information. Two soil borings totaling 80 linear feet were drilled near the perimeter of the tank footing. Soil samples were generally collected at 5-foot depth intervals

using a 2-inch outside diameter Standard Penetration Test (SPT) split-spoon sampler driven with a 140-pound hammer falling from a height of 30 inches. A summary of the soil conditions encountered in the explorations is presented on the exploration logs at the end of this report.

Soils

Surficial geologic units in the project area were mapped as Vashon glacial till (Yount, Minard, and Glen, U.S. Geological Survey Open File Report OFR 93-233; and Deeter, 1979, unpublished geologic map of Kitsap County). Glacial till commonly occurs as a relatively thin unit that mantles older glacial and non-glacial units, and may not be present everywhere. Our analysis suggests that the soils in the project vicinity consist of glacio-lacustrine transition beds, a soil unit that stratigraphically lies beneath the Vashon till. Transition beds consist of fine-grained sediments deposited in a glacial lake that formed in front of an advancing continental glacier. Transition beds may contain lenses of sand and gravel as well as poorly sorted glacial till-like sediments.

Fill soils, consisting of man-modified or man-placed soils, were encountered adjacent to the tank footing to depths of 8 to 8.5 feet in the three exploration pits, and in the upper 2 feet of boring EB-2. Fill consisted of medium-stiff to stiff, moist, brown silt.

Natural soils (those not modified by man) were encountered in the two exploration borings. These glacio-lacustrine soils consisted of stiff silt with minor amounts of sand and trace gravel, and medium dense to dense silty fine sand. This soil unit was generally brown and graded gray-brown with depth. Soil moisture increased from moist at the surface to wet below the water table at about 9 feet depth. The silty soils were generally of low plasticity. Some heaving sand conditions were noted at a depth of about 43 feet in boring, suggesting the presence of sandy interbeds within the silt.

Ground Water

A ground water measuring piezometer was installed in boring EB-2 to permit periodic measurements of the static ground water levels. The piezometer consists of 1-inch-diameter, polyvinyl chloride (PVC) pipe with screened interval from about 10 to 20 feet depth. At the time of drilling, saturated soils were encountered at the 9-foot depth in both borings.

Tank Footing

The tank footing was investigated by excavating the exploration pits adjacent to the footing. The footing was observed to extend about 1 foot above general ground surface. The base of the footing was encountered at 7.2 feet below ground surface in the three exploration pits. The total height of the footing was 8.2 feet. Gravel encountered at the base of the footing suggests that the footing was placed on a layer of ¾-inch minus gravel and a layer of plastic sheeting. An 8-inch-long section of plastic pipe was encountered in exploration EP-3 near the base of the footing. The recovered pipe section was partially filled with silt. Gravel in the excavation spoils suggested that the pipe might have been surrounded by ¾-inch gravel drain material. The pipe may have been used to drain the topographic depression that was likely created during construction of the footing.

It is not known whether the footing is a thin ring with a concrete slab through the center or a ring footing with a sand or gravel filled center.

CONCLUSIONS

The tank footing extends to about 7.2 feet below ground surface and bears on native medium-stiff to stiff silt and medium-dense to dense silty fine sand. Ground water is present at a depth of about 9 feet below existing site grade. Moderate soil strengths encountered at footing depth suggest that the water tank is capable of withstanding the additional loads created by increasing the height of the tank by 7 feet.

Additional settlements may be incurred due to the increase in load by the 7 feet of water storage. This would be a combination of elastic and re-compression settlement of the soil. This additional movement would be on the order of 1 inch or less and would occur along with the movement of any reloading of the tank after it was drained for alteration. AESI has not received any information from the structural engineer on the seismic loading of the foundation and has not reviewed the seismic response of the foundation, except that liquefaction is highly unlikely due to the dense and silty nature of the sediments.

RECOMMENDATIONS

We recommend completing a non-destructive testing program consisting of the following:

- Determine the thickness of the wall of the steel tank using acoustic or other methods. The thickness of the steel may be observable at the access/cleanout port near the base of the tank.

- Determine if the footing is constructed as a ring and, if so, determine the thickness of the ring and width of the footing bearing zone. This can be accomplished by re-excavating to the base of the footing and either drilling small diameter holes into or through the footing to probe for thickness, or using an acoustic or radar method to determine the approximate thickness of the ring and/or footing.

- If and when the tank is drained, tests should be conducted to determine if the interior of the tank rests on a concrete slab or on compacted fill soils.

- During any unloading and reloading of the tank, vertical changes in elevation should be monitored to provide a true calibration of the actual response of the foundation.

With the above data, a more detailed evaluation of the soil-structure interaction can be prepared.

It has been our pleasure to assist you in this project. If you have any questions, please call us at (206) 780-9370.

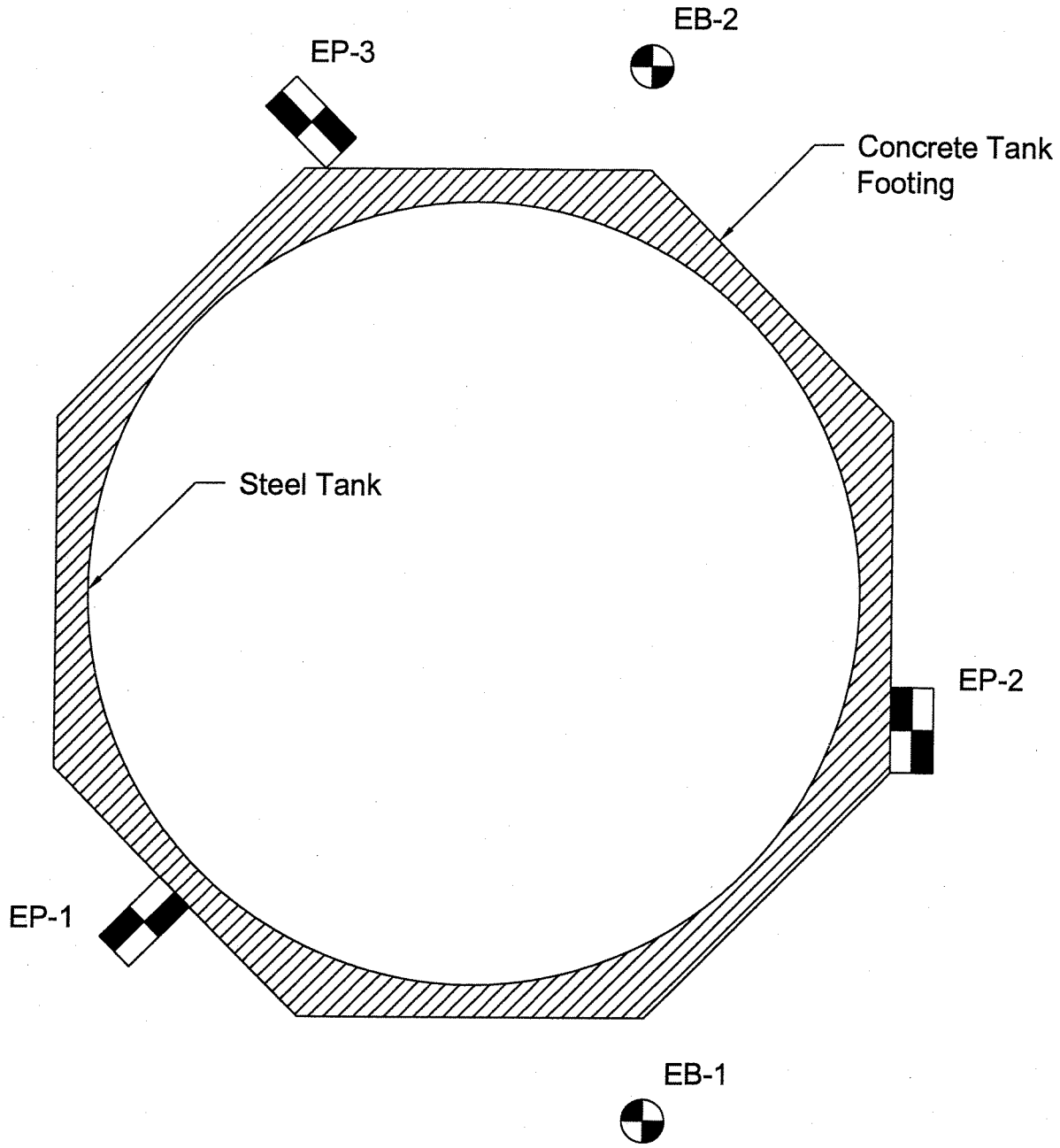
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
West Sound Office

JOHN L. PETERSON
STATE OF WASHINGTON
18802
REGISTERED
PROFESSIONAL ENGINEER
5/5/00
EXPIRES 6/22/01

David H. McCormack
Senior Project Geologist

John L. Peterson, P.E.
Senior Geotechnical Engineer

I:\KPP\INBOX\TOBEFILED\COBI WATER TANK RPT



LEGEND

EB-1 Soil Boring

EP-1 Exploration Pit

Approximate Scale in Feet

0 5 10

BE99103A.dwg 10/07/99 1:1

ASSOCIATED EARTH SCIENCES, INC	DATE: 10/07/99	**Water Tank**	PROJECT NO. BE99103A
	DESIGNED/DWN: DHM/HXT	City of Bainbridge Island Bainbridge Island, Washington	FIGURE NO. 1

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT[2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch

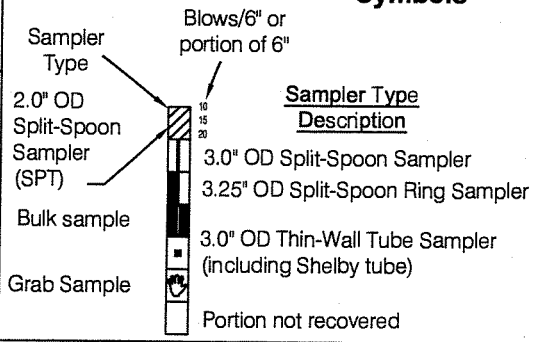
Slightly Moist - Perceptible moisture

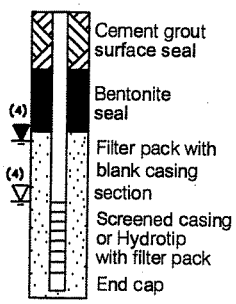
Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols





(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of groundwater
▼ ATD = At time of drilling
▽ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

biserver/odmin/forms/Exploration Log Key.dwg 10/12/99 1:1

ASSOCIATED EARTH SCIENCES, INC

Exploration Log Key

FIGURE
A-1

LOG OF EXPLORATION PIT NO. EP-1

Location: SW Side of Tank

Elevation (approx. ft msl):

Depth, ft	Elev., ft	Graphic Symbol	Description
			This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.
			TOPSOIL
			Sod over medium dense, moist, dark brown SAND with SILT; with organics
1			FILL
			Stiff, moist, brown SILT; few fine sand, trace gravels, silt predominantly coarse; massive - no visible bedding (ML)
2			
3			
4			
5			
6			
7			- slightly mottled, trace iron staining - base of footing at 7.2 ft depth
8			Bottom of hole at 8'
9			

Project No.

City of Bainbridge Island Water Tank

Bainbridge Island, WA

BE99103A

ASSOCIATED EARTH SCIENCES, INC

Date: 8/12/99
Logged by: RRH
Checked by: DHM

Figure No.
A-2

AESTP2 COBI.GPJ October 7, 1999

LOG OF EXPLORATION PIT NO. EP-2

Location: East Side of Tank

Elevation (approx. ft msl):

Depth, ft	Elev., ft	Graphic Symbol	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **Description**
			Sod
			FILL
1			Medium stiff, moist, brown SILT; predominantly coarse, few fine sand, massive - no visible bedding
2			
3			
4			
5			
6			
7			- base of footing at 7.2 ft depth, with a thin layer 3/4"-minus gravel and plastic film below 7.2 ft depth
8			
			Bottom of hole at 8.4'
9			

Project No.

City of Bainbridge Island Water Tank

Bainbridge Island, WA

BE99103A

ASSOCIATED EARTH SCIENCES, INC

Date: 8/12/99
Logged by: RRH
Checked by: DHM

Figure No.
A-3

AESTP2 COBI.GPJ October 7, 1999

LOG OF EXPLORATION PIT NO. EP-3

Location: NW Side of Tank

Elevation (approx. ft msl):

Depth, ft	Elev., ft	Graphic Symbol	Description
			This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.
			Sod
			FILL
1			Medium stiff, moist, brown SILT; predominantly coarse, few fine sand, trace gravel
2			
3			
4			
5			
6			
7			- base of footing at 7.2 ft depth
8			8" plastic pipe encountered, partially-filled with silt
			Bottom of hole at 8.5'
9			

City of Bainbridge Island Water Tank

Bainbridge Island, WA

Project No.

BE99103A

ASSOCIATED EARTH SCIENCES, INC

Date: 8/12/99

Logged by: RRH

Checked by: DHM

Figure No.

A-4

AESTP2 COBI.GPJ October 7, 1999

ASSOCIATED EARTH SCIENCES, INC

Exploration Log

Project Number	Exploration Number	Sheet
BE99103A	EB-1	1 of 2

Project Name: City of Bainbridge Island Water Tank
Location: Bainbridge Island, WA
Driller/Equipment: McDonald Drilling/4" Hollow Stem Auger
Hammer Weight/Drop: 140 lbs/30"

Ground Surface Elevation (ft):
Datum:
Date Start/Finish: 8/20/99-8/20/99
Hole Diameter (in): 9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)	Other tests
				TRANSITION BEDS				
				Stiff, moist, brown SILT; coarse, few sand, trace gravel (ML)				
		1				6 7 8		
5								
		2		- moist to wet at 9', grades trace sand		2 5 7		
10								
		3		- hard, wet, grades few sand, trace organics		7 16 21		
15								
		4		- small 1/4" blue-gray silt clasts in shoe		7 20 33	50+	
20								
		5		- very stiff, grades trace sand		5 10 14		
25								
		6		- hard, gray with brown and iron-stained interbeds, few sand, visible laminations		9 16 21		
30								
		7		- gray, trace gravel		7 14 22		

Sampler Type (ST):
- No Recovery
- 2" OD Split Spoon Sampler
- 3" OD Split Spoon Sampler
- Grab Sample

Lab tests:
- C - Chemical Properties
- P - Permeability
- M - Moisture
- Water Level (Date)
- Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A-5

AESIBOR COBI.GPJ October 7, 1999

Exploration Log

ASSOCIATED EARTH SCIENCES, INC

Project Number	Exploration Number	Sheet
BE99103A	EB-1	2 of 2

Project Name: City of Bainbridge Island Water Tank
Location: Bainbridge Island, WA
Driller/Equipment: McDonald Drilling/4" Hollow Stem Auger
Hammer Weight/Drop: 140 lbs/30"

Ground Surface Elevation (ft):
Datum:
Date Start/Finish: 8/20/99-8/20/99
Hole Diameter (in): 9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)	Other tests
				- harder drilling				
		8		- trace sand		10 12 22		
40								
		9		- sampler sank 18" over approx. 5 minutes under weight of rod; 5' of heave noted in auger, no recovery				
45				- with fine sand, visible laminations				
		10				14 24 37	50+	
		11		- possible laminations		8 24 39	50+	
50								
		12		- moist, with clay, no sand; visible laminations, varved; low plasticity		15 28 47	50+	
55								
				- harder drilling at 56', easier at 57'				
		13		- wet, silt coarse, no clay		14 28 50/6"	50+	
60				Bottom of hole at 59.5 feet				
65								

Sampler Type (ST):
- No Recovery
- 2" OD Split Spoon Sampler
- 3" OD Split Spoon Sampler
- Grab Sample

Lab tests:
- C - Chemical Properties
- P - Permeability
- M - Moisture
- Water Level (Date)
- Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A-5

AESIBOR COBI.GPJ October 7, 1999

ASSOCIATED EARTH SCIENCES, INC

Exploration Log

Project Number	Exploration Number	Sheet
BE99103A	EB-2	1 of 1

Project Name: City of Bainbridge Island Water Tank
Location: Bainbridge Island, WA;
Driller/Equipment: McDonald Drilling/4" Hollow Stem Auger
Hammer Weight/Drop: 140 lbs / 30"

Ground Surface Elevation (ft):
Datum:
Date Start/Finish: 8/20/1999-8/20/1999
Hole Diameter (in): 9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)
				FILL SILT; with gravel and cobbles (ML)			
				TRANSITION BEDS Medium dense, moist, brown SAND with SILT; fine sand; trace organics (SM)		3 6 7	
5							
				- dense, moist to wet at 9', coarse silt		6 10 22	
10							
				- medium dense, wet		9 10 15	
15							
				- dense		9 19 30	
20				Bottom of hole at 20 feet 1-inch ID PVC piezometer installed in borehole			

Sampler Type (ST):
- No Recovery
- 2" Split Spoon Sampler
- Grab Sample

Lab tests:
- C - Chemical Properties
- P - Permeability
- M - Moisture
- Static Water Level
- Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A - 6

AESIBOR COBI.GPJ May 8, 2000

APPENDIX E

SUPPLEMENTARY HIGH SCHOOL RESERVOIR GEOTECHNICAL REPORT

PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

January 4, 2018
File No. 17-398

Mr. Russ Porter, P.E.
Gray & Osborne, Inc.
701 Dexter Avenue North, Suite 200
Seattle, WA 98109

Subject: **Geotechnical Report**
High School Reservoirs Seismic Upgrade
94XX NE New Brooklyn Road, Bainbridge Island, Washington
Gray & Osborne IPN #17614

Dear Mr. Porter,

As requested, PanGEO observed the excavation of four test pits to evaluate subsurface conditions in the vicinity of the existing 1.5-million gallon (MG) and 1.0 MG reservoirs located at 94XX Northeast New Brooklyn Road, in Bainbridge Island, Washington. The following documents the subsurface conditions observed at the test pits and, based on the conditions encountered, geotechnical design parameters to assist you with the proposed seismic upgrade of the existing reservoirs.

SITE DESCRIPTION AND PROJECT UNDERSTANDING

The project site is occupied by two existing reservoirs located approximately 1,000 feet and 1,500 feet north of Bainbridge High School in City of Bainbridge Island, Washington. The approximate site location is shown on Figure 1, Vicinity Map. Based on information from Gray & Osborne, the northern reservoir is a 1.5 MG tank constructed in 1989 that is approximately 89 feet high and 53 feet in diameter. We understand the southern reservoir is a 1.0 MG tank constructed in 1973 that is approximately 81 feet high and 46 feet in diameter. Both reservoirs are of welded steel construction.

3213 Eastlake Avenue East, Ste B
Seattle, WA 98102
Tel:(206) 262-0370
Fax:(206) 262-0374

We understand the City is considering a seismic upgrade for the existing reservoirs. The purpose of this letter is to provide geotechnical parameters to assist with the seismic retrofit.

Plate 1 – Existing 1.5 MG reservoir.

SUBSURFACE EXPLORATIONS

CURRENT EXPLORATIONS

Four test pits (TP-1 through TP-4) were excavated on December 4, 2017, to explore subsurface conditions at the site. The approximate test pit locations were measured from existing structures and site features and are indicated on Figure 2. Test pits TP-1 and TP-2 were located near the 1.5 MG reservoir and were excavated to 3 and 9 feet below grade, respectively. Test pits TP-3 and TP-4 were located near the 1.0 MG reservoir and were excavated to 7 and 10 feet below grade, respectively. The test pits were excavated using

a Deere 310SG rubber-tired backhoe owned and operated by the City of Bainbridge Island Public Works Department.

An engineering geologist from PanGEO was present during the field explorations to observe the test pit excavations, obtain representative samples, and to describe and document the soils encountered in the explorations. Test pit logs are included in Appendix A of this report and provide descriptions of the materials encountered, depths to soil contacts, and depths of seepage or caving, if present, observed in the test pit sidewalls. The relative in-situ density of cohesionless soils, or the relative consistency of fine-grained soils, was estimated from the excavating action of the backhoe, probing the sidewalls with a ½-inch diameter steel rod, and the stability of the test pit sidewalls. Where soil contacts were gradual or undulating, the average depth of the contact was recorded in the log. After each test pit was logged, the excavation was backfilled with the excavated soils and the surface was tamped and re-graded smooth.

PREVIOUS EXPLORATIONS

In addition to our test pits completed for the current study, we also reviewed readily available subsurface data for the project site. Specifically, we reviewed logs of two previous test borings advanced at the 1.0 MG reservoir site (EB-1 and EB-2, Associated Earth Sciences, 2000). Test boring EB-1 was advanced to 59½ feet below grade and EB-2 was advanced to 20 feet below grade. The approximate locations of the previous test borings at the 1.0 MG reservoir are shown on Figure 2 and the summary test boring logs are included in Appendix B. The results of the previous test borings are summarized in the *Subsurface Conditions* section of this report.

The previous test boring logs indicate that soil samples were obtained from the borings in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler is driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

EXISTING RESERVOIR FOUNDATIONS

Photos of the test pits exposing the 1.5 MG and 1.0 MG reservoir foundations are provided below in Plates 2 and 3, respectively. Based on our observations, it is not clear whether the existing foundations are ring footings or mat foundations.

1.5 MG Reservoir **-** Test pit TP-2 was excavated adjacent to the circular concrete foundation of the 1.5 MG reservoir. The bottom of the concrete foundation at TP-2 was 3 feet below grade.

1.0 MG Reservoir **-** Test pit TP-4 was excavated adjacent to the north side of the octagonal concrete foundation of the 1.0 MG reservoir. The bottom of the concrete foundation at TP-4 was about 10 feet below grade. The Associated Earth Sciences report (2000) indicates the bottom of the 1.0 MG reservoir foundation was encountered approximately 7.2 feet below grade at their exploration pits located on the northwest, southwest and east sides of the reservoir.

Plate 2 – 1.5 MG reservoir foundation approx. 3 feet thick at TP-2.

Plate 3 – 1.0 MG reservoir foundation approx. 10 feet thick at TP-4.

SUBSURFACE CONDITIONS

GEOLOGY AND SOIL

The *Preliminary Geologic Map of Bainbridge* Island (Haugerud, 2005) indicates that the surficial geologic units in the vicinity of the project are Vashon glacial till (Map Unit Qvt) and ice-contact deposits (Qvi). Glacial till is a very dense heterogeneous mixture of silt, sand, and gravel laid down at the base of an advancing glacial ice sheet. Glacial till typically exhibits low compressibility and high strength characteristics. Ice-contact deposits are described as gravel, sand, and diamict deposited against stationary ice. Ice contact deposits may or may not have been consolidated by glacial advance.

The soils observed in the test pits were classified and described in the field using the system outlined in Figure A-1 and summary test pit logs are included as Figures A-2 through A-5. The results from our test pits generally confirmed the mapped geology. The subsurface conditions encountered at each reservoir location follow:

1.5 MG Reservoir – Reservoir foundation backfill consisting of medium dense silty fine sand was encountered to 3 feet below grade at TP-2, which was excavated adjacent to the existing reservoir foundation. Underlying the existing fill at TP-2, silty fine sand to sandy silt that we interpret to be an ice-contact deposit was encountered. Probing the foundation bearing soils for the existing reservoir at 3 feet below grade in TP-2 yielded penetrations of 6 to 8 inches indicating that the ice contact deposits were generally in a medium dense condition. Subsurface conditions at TP-1, which was located about 20 feet south/southwest of the reservoir, consisted of medium dense silty fine sand that transitioned to medium dense to dense silty sand to sandy silt around 5½ feet below grade.

1.0 MG Reservoir- Reservoir foundation backfill consisting of medium dense silty fine sand was encountered to the maximum exploration depth of 10 feet below grade at TP-4, which was excavated adjacent to the existing reservoir foundation. At test pit TP-3, medium dense silty sand with gravel that we interpret to be an ice-contact deposit was encountered to the maximum exploration depth of 7 feet below grade.

The Associated Earth Sciences test borings previously drilled near the 1.0 MG reservoir indicate that the reservoir foundation bearing soils (i.e. 7 to 10 feet below grade) were generally in a medium dense to dense condition with SPT N-values ranging from 12 to 32 blows per foot. In general, an increase in relative density with depth was encountered below the reservoir foundation at their boring locations and the soils were classified as glacially overridden silt and sand with silt.

GROUNDWATER

Groundwater was not encountered in our test pits at the time of excavation. However, the Associated Earth Sciences test borings indicate that groundwater was first encountered around 9 feet below grade in hard/dense deposits at both EB-1 and EB-2, which were drilled near the 1.0 MG reservoir in August 1999.

SEISMIC DESIGN PARAMETERS

Seismic retrofit design may be accomplished using the ASCE 7-10 and the 2015 edition of the International Building Code (IBC). Both specify a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). The seismic design of the reservoir should also follow the procedures contained in the American Water Works Association's (AWWA) Standard for Welded Carbon Steel Tanks for Water Storage (AWWA D100-11). Table 1 presents the seismic design parameters in accordance with the 2015 IBC, which are consistent with the 2008 USGS seismic hazard maps for the existing reservoirs as well as the proposed reservoir site:

Table 1 – Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters		Control Periods (sec.)	
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}	T_O	T_S
D	1.40	0.55	1.00	1.50	0.93	0.55	0.12	0.59

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

LIQUEFACTION

Soil liquefaction is a condition where saturated cohesionless soils undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. Soils most susceptible to liquefaction are loose, uniformly graded sands and loose silts with little cohesion. Due to the generally dense soils below the groundwater table in the Associated Earth Sciences test borings, the potential for liquefaction at the site is considered to be low, and special design considerations associated with soil liquefaction are not necessary for this project.

RECOMMENDED BEARING CAPACITY FOR RETROFIT

Based on the subsurface conditions encountered at the test pits and the previous test borings, it appears that the existing reservoir foundations are supported on competent ice-contact deposits. It is our opinion that the following allowable bearing capacities may be used to evaluate the existing reservoir foundations and to site the retrofit foundation:

- **1.5 MG Reservoir:** 4,000 psf
- **1.0 MG Reservoir:** 6,000 psf

For allowable stress design, the allowable bearing pressure may be increased by 1/3 for transient conditions such as wind and seismic loading.

Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundation, and by friction acting on the base of the foundation. The passive resistance surrounding the existing foundation may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5. A friction coefficient of 0.40 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor safety of approximately 1.5. For seismic design, a 1/3 increase of the allowable passive pressure and the friction coefficient is permitted.

If new footings will be needed, we recommend excavating the new foundation at least 1-foot below the bottom of the footing and backfilling with CSBC compacted to the project requirements for structural fill. The bottom of the foundation excavation should be observed and verified by PanGEO to confirm that the exposed subgrade is consistent with the anticipated conditions and adequate to support the reservoir. All foundation excavations should be trimmed neat and the subgrade should be carefully prepared. If soft/loose subgrade soil is encountered, it should be overexcavated to competent native soils.

TEMPORARY EXCAVATIONS

If new footings will be needed, we envision foundation excavations may extend up to approximately 10 feet below grade. Soils anticipated to be encountered in temporary excavations consist of medium dense existing fill and medium dense silty sand to sandy silt (ice-contact deposits).

It is the contractor's responsibility to maintain safe working conditions, including temporary excavation stability. All excavations in excess of 4 feet in depth should be sloped in accordance with Washington Administrative Code (WAC) 296-155, or be shored. Temporary excavation slopes in existing fill and medium dense ice-contact deposits should not be graded steeper than 1H:1V, but should be re-evaluated in the field during construction based on actual observed soil conditions. The cut slopes may need to be flattened if groundwater seepage is encountered in site excavations.

UNCERTAINTY AND LIMITATIONS

We have prepared this report for use by Gray & Osborne, Inc. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent geologic publications, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to

review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

Our scope of services does not include those related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our services specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances or other environmental considerations.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use of this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We trust that the information outlined in this letter meets your need at this time. Please call if you have any questions. We are available to provide a more detailed geotechnical report during the final design phase of the project.

Sincerely,

Steven T. Swenson, L.G.
Project Geologist

Siew L. Tan, P.E.
Principal Geotechnical Engineer

Attachments:

Figure 1 Vicinity Map
Figure 2 Site and Exploration Plan

Appendix A – Summary Test Pit Logs

Figure A-1 Terms and Symbols for Boring and Test Pit Logs
Figure A-2 Log of Test Pit TP-1
Figure A-3 Log of Test Pit TP-2
Figure A-4 Log of Test Pit TP-3
Figure A-5 Log of Test Pit TP-4

Appendix B – Previous Explorations

Logs of Test Borings EB-1 and EB-2 (Associated Earth Sciences, 2000)

REFERENCES

American Water Works Association (AWWA), 2011, *AWWA Standard for Welded Carbon Steel Tanks for Water Storage*, ANSI/AWWA D100-11.

Associated Earth Sciences, Inc., 2000, Preliminary Subsurface Exploration and Geotechnical Engineering Design, Existing High School Water Tank Modification, Bainbridge Island, Washington. Consultant report prepared for City of Bainbridge Island, dated May 8, 2000.

Haugerud, R. A., 2005, *Preliminary geologic map of Bainbridge Island* [part of Bremerton East, Duwamish Head, Shilshole Bay, Suquamish 7½′ quadrangles], Washington: U.S. Geological Survey Open-File Report 2005-1387, version 1.0, 1 sheet, scale 1:24,000.

International Building Code (IBC), 2015, International Code Council.

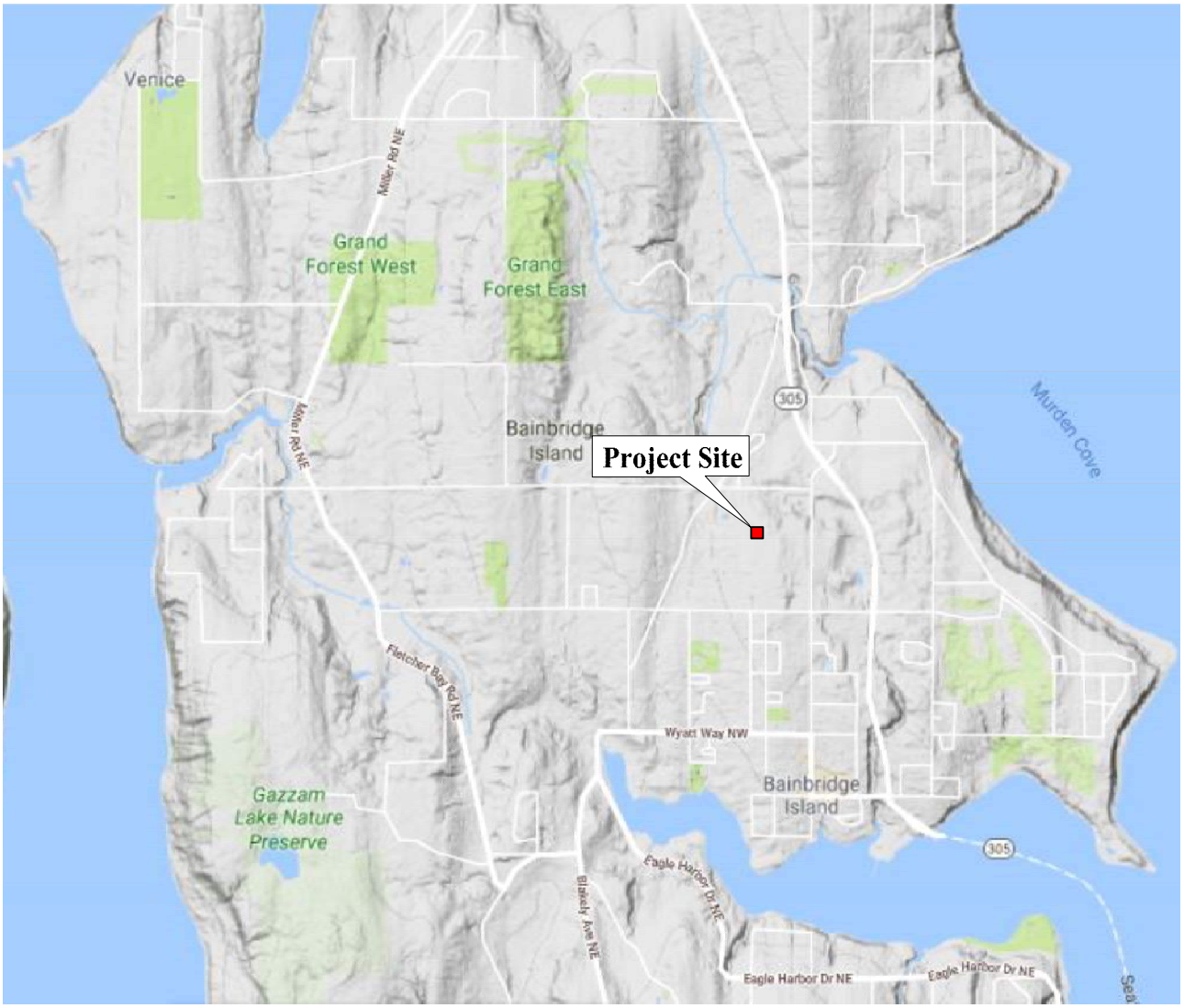


Image Source: Google Maps

Not to Scale

PanGEO INCORPORATED	**High School Reservoirs Seismic Upgrade Bainbridge Island, WA**	**VICINITY MAP**	
		Project No. **17-398**	Figure No. **1**

Legend:

Approx. Test Pit Location (PanGEO 2017)

Approx. Test Boring Location (AESI, 2000)

Note: Imagery obtained from Google Earth.

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High School Reservoirs Seismic Upgrade Bainbridge Island, WA

SITE AND EXPLORATION PLAN

Project No.	Figure No.
17-398	2

17-398 Fig 2 Site Plan.grf 1/4/18 (13:59) STS

APPENDIX A

SUMMARY TEST PIT LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes: 1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered: Units of material distinguished by color and/or composition from material units above and below

Laminated: Layers of soil typically 0.05 to 1mm thick, max. 1 cm

Lens: Layer of soil that pinches out laterally

Interlayered: Alternating layers of differing soil material

Pocket: Erratic, discontinuous deposit of limited extent

Homogeneous: Soil with uniform color and composition throughout

Fissured: Breaks along defined planes

Slickensided: Fracture planes that are polished or glossy

Blocky: Angular soil lumps that resist breakdown

Disrupted: Soil that is broken and mixed

Scattered: Less than one per foot

Numerous: More than one per foot

BCN: Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

TEST PIT LOGS

Test Pit No. TP-1

Location: Latitude 47.640438, Longitude -122.525999
Approximate Ground Surface Elevation: 250 feet (Google Earth)
Date: December 4, 2017
Surface Vegetation: Leaf Litter

Depth (ft)	USCS	Soil Description
0 – 0.5	TOPSOIL	Loose, dark brown silty SAND with organics and forest duff, moist [TOPSOIL]
0.5 – 5.5	SM	Medium dense, light brown, silty fine to medium SAND with well-rounded medium to coarse gravel, moist [ICE CONTACT DEPOSITS] • Tree roots to 2½ feet • Probes 6 inches at 3 feet below grade
5.5 – 9	SM/ML	Medium dense to dense, gray, silty fine SAND/sandy SILT, trace gravel, moist to wet [ICE CONTACT DEPOSITS] • Iron oxide staining • Contains cobbles

Photo TP-1A: View of fine SAND/silty SAND from about 9 feet below grade.

Photo TP-1B: Excavation spoils in backhoe bucket from 9 feet below grade.

Test Pit TP-1 terminated at 9 feet below grade. No groundwater seepage encountered during excavation

Figure A-2

Test Pit No. TP-2

Location: Latitude 47.640437, Longitude -122.52587
Approximate Ground Surface Elevation: 250 feet (Google Earth)
Date: December 4, 2017
Surface Condition: Asphalt Pavement

Depth (ft)	USCS	Soil Description
0 – 0.25	ASPHALT PAVEMENT	Three inches of Asphalt Pavement
0.25 – 3	FILL	Medium dense, light brown, silty fine SAND, moist [FILL] • Tree roots to 2½ feet
3 – 4	SM/ML	Medium dense, gray, silty fine SAND/sandy SILT, moist to wet [ICE CONTACT DEPOSITS] • Probes 6 to 8 inches at 3 feet below grade

Photo TP-2A: View of Test Pit TP-2 at 3 feet deep. Reservoir foundation is exposed.

Photo TP-2B: View of Test Pit TP-2 excavation spoils.

Test Pit TP-2 terminated at 4 feet below grade.
No groundwater seepage encountered during excavation.

Figure A-3

Test Pit No. TP-3

Location: Latitude 47.639342, Longitude -122.526062
Approximate Ground Surface Elevation: 250 feet (Google Earth)
Date: December 4, 2017
Surface Vegetation: Grass

Depth (ft)	USCS	Soil Description
0 – 0.5	TOPSOIL	Loose, dark brown silty SAND with organics and forest duff, moist [TOPSOIL]
0.5 – 7.0	SM	Medium dense, grayish brown, silty fine to medium SAND with well-rounded medium to coarse gravel, moist [ICE CONTACT DEPOSITS] • Tree roots to 2 feet • Probes 6 inches at 4 feet below grade • Silt content decreases with depth • Horizontally bedded, manganese staining along bedding planes

Photo TP-3A: View of Test Pit TP-3 excavation spoils

Photo TP-3B: Manganese staining along bedding

Test Pit TP-3 terminated at 7 feet below grade.
No groundwater seepage encountered during excavation.

Figure A-4

Test Pit No. TP-4

Location: Latitude 47.639494, Longitude -122.526118
Approximate Ground Surface Elevation: 250 feet (Google Earth)
Date: December 4, 2017
Surface Vegetation: Grass

Depth (ft)	USCS	Soil Description
0 – 0.25	TOPSOIL	Loose, dark brown silty SAND with organics and grass, moist [TOPSOIL]
0.25 – 10	FILL	Medium dense, grayish brown, silty fine SAND, moist [FILL] • Probes 6 inches at 4 feet below grade • Decrease in silt below 3 feet, possibly SAND with silt • 6-inch diameter plastic flexible perforated, corrugated pipe bedded in gravel encountered at 9 feet • Encountered concrete (possible footing extension) at 10 feet

Photo TP-4A: View of Test Pit TP-4 at 10 feet deep. Reservoir foundation is exposed on the side of test pit.

Photo TP-4B: View of Test Pit TP-4 excavation spoils

Test Pit excavated adjacent to footing. Footing is approximately 10 feet thick.
Test Pit TP-4 terminated at 10.0 feet below grade.
No groundwater seepage encountered during excavation.

Figure A-5

APPENDIX B

PREVIOUS EXPLORATIONS

(High School Water Tank, Associated Earth Sciences, 2000)

ASSOCIATED EARTH SCIENCES, INC

Exploration Log

Project Number	Exploration Number	Sheet
BE99103A	EB-1	1 of 2

Project Name	City of Bainbridge Island Water Tank	Ground Surface Elevation (ft)	
Location	Bainbridge Island, WA	Datum	
Driller/Equipment	McDonald Drilling/4" Hollow Stem Auger	Date Start/Finish	8/20/99-8/20/99
Hammer Weight/Drop	140 lbs/30"	Hole Diameter (in)	9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)	Other tests
				TRANSITION BEDS				
				Stiff, moist, brown SILT; coarse, few sand, trace gravel (ML)				
		1				6 7 8		
5								
		2				2 5 7		
				- moist to wet at 9', grades trace sand				
10								
		3		- hard, wet, grades few sand, trace organics		7 16 21		
15								
		4		- small 1/4" blue-gray silt clasts in shoe		7 20 33	50+	
20								
		5		- very stiff, grades trace sand		5 10 14		
25								
		6		- hard, gray with brown and iron-stained interbeds, few sand, visible laminations		9 16 21		
30								
		7		- gray, trace gravel		7 14 22		

Sampler Type (ST): No Recovery; 2" OD Split Spoon Sampler; 3" OD Split Spoon Sampler; Grab Sample

Lab tests: C - Chemical Properties; P - Permeability; M - Moisture; Water Level (Date); Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A-5

AESIBOR COBI.GPJ October 7, 1999

ASSOCIATED EARTH SCIENCES, INC

Exploration Log

Project Number	Exploration Number	Sheet
BE99103A	EB-1	2 of 2

Project Name	City of Bainbridge Island Water Tank	Ground Surface Elevation (ft)	
Location	Bainbridge Island, WA	Datum	
Driller/Equipment	McDonald Drilling/4" Hollow Stem Auger	Date Start/Finish	8/20/99-8/20/99
Hammer Weight/Drop	140 lbs/30"	Hole Diameter (in)	9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)	Other tests
				- harder drilling				
		8		- trace sand		10 12 22		
40								
		9		- sampler sank 18" over approx. 5 minutes under weight of rod; 5' of heave noted in auger, no recovery				
45		10		- with fine sand, visible laminations		14 24 37	50+	
		11		- possible laminations		8 24 39	50+	
50								
		12		- moist, with clay, no sand; visible laminations, varved; low plasticity		15 28 47	50+	
55								
				- harder drilling at 56', easier at 57'				
		13		- wet, silt coarse, no clay		14 28 50/6"	50+	
60				Bottom of hole at 59.5 feet				
65								

Sampler Type (ST):
- No Recovery
- 2" OD Split Spoon Sampler
- 3" OD Split Spoon Sampler
- Grab Sample

Lab tests:
- C - Chemical Properties
- P - Permeability
- M - Moisture
- ▽ Water Level (Date)
- ▼ Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A-5

AESIBOR COBI.GPJ October 7, 1999

Exploration Log

ASSOCIATED EARTH SCIENCES, INC

Project Number	Exploration Number	Sheet
BE99103A	EB-2	1 of 1

Project Name: City of Bainbridge Island Water Tank
Location: Bainbridge Island, WA
Driller/Equipment: McDonald Drilling/4" Hollow Stem Auger
Hammer Weight/Drop: 140 lbs / 30"

Ground Surface Elevation (ft):
Datum:
Date Start/Finish: 8/20/1999-8/20/1999
Hole Diameter (in): 9

This log is part of the report prepared by Associated Earth Sciences (AESI) for the named project and should be read together only with that report for complete interpretation. This summary applies only to the location of this exploration and at the time of exploration. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth, ft	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Blows/6"	Blows/Foot (10 20 30 40)
				FILL SILT; with gravel and cobbles (ML)			
				TRANSITION BEDS Medium dense, moist, brown SAND with SILT; fine sand; trace organics (SM)		3 6 7	
5							
				- dense, moist to wet at 9', coarse silt		6 10 22	
10							
				- medium dense, wet		9 10 15	
15							
				- dense		9 19 30	
20				Bottom of hole at 20 feet 1-inch ID PVC piezometer installed in borehole			

Sampler Type (ST):
- No Recovery
- 2" Split Spoon Sampler
- Grab Sample

Lab tests:
- C - Chemical Properties
- P - Permeability
- M - Moisture
- Static Water Level
- Water Level at time of drilling (ATD)

Logged by: RRH
Approved by: JLP
Figure No. A - 6

AESIBOR COBI.GPJ May 8, 2000

COBI UTILITY ADVISORY COMMITTEE

MEMORANDUM REGARDING COBI'S POLICIES TOWARD SMALL WATER SYSTEMS

(Discussion draft 8/30/20)

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COBI UTILITY ADVISORY COMMITTEE

MEMORANDUM REGARDING COBI'S POLICIES TOWARD SMALL WATER SYSTEMS
(Discussion draft 8/30/20)

I. INTRODUCTION

In 2018, the City Council asked the Utility Advisory Committee (UAC) to "study and recommend a process for facilitating consolidation of small water systems." The council's direction begins implementation of Comprehensive Plan Utilities Element Policy U-11.7, which is reproduced in the next section.

There are 30 Group A (the largest) water systems on Bainbridge Island, and 144 Group B systems (the smallest). COBI currently owns and manages two of the Group A systems (Winslow-Fletcher Bay and Rockaway Beach) and one Group B system (Casey Street).

The UAC gathered data on Island water systems, considered the history of water system management on the Island, and consulted with water system experts within state, county-wide, and city governments, and from the private sector.

The UAC then developed four alternatives for a COBI policy toward small water systems: (1) minimal role; (2) reactive to requests from water systems – which the UAC suggests is an apt description of the City's current policy; (3) active assistance to water systems; and (4) active assistance along with acquisition over time of small water systems. Descriptions of these alternatives are contained in this memo and on an attached matrix.

As discussed in this memorandum, the UAC believes it is time for the City to depart from its laissez faire attitude toward water management on Bainbridge Island, Accordingly, the UAC recommends that the City adopt a policy of providing active assistance, if requested, to the small water systems on Bainbridge Island, and encourage, but not mandate, consolidation of those systems into the City's current water utility. Priority focus should be on the systems within and adjacent to the City's current county-assigned service areas.

The primary reasons for this policy are to help insure all Island residents have adequate, safe drinking water and to protect Island water resources.

II. BACKGROUND

A) Comprehensive Plan

In 2017, the City Council approved an update to the Comprehensive Plan, which included a newly revised Utilities Element. Among the goals and policies in the Utilities Element are the following pertinent to this subject (key clauses are identified by bold highlighting):

- Goal U-10. *Ensure that city-managed and* ***to the extent possible non-city managed utility services*** *are sufficient, cost effective, reliable, and that safe water utility service is provided.*

- Goal U-11: *Require utilities to operate in a manner that* ***preserves and protects the water resources*** *of the Island.*

- Policy U 11.6: ***Encourage and support water utilities to enter into cooperative activities****, such as jointly managed operations, shared storage, and construction of interties, to manage water resources and systems more efficiently, economically, and safely.*

- Policy U 11.7: ***Encourage and facilitate consolidation of water systems, with particular emphasis on mergers of contiguous and small systems****, to manage water resources and systems more efficiently, economically, and safely.*

The Comprehensive Plan also contains Policy. U 11.9 which states as follows: "*Conduct a study of consolidation of water systems owned by the City and Kitsap Public Utility District. Pursue long-term consolidation of larger water systems.*" As is shown below, the City and KPUD own the three largest water systems on the Island, all of which are substantially larger than the remainder of Island water systems.

The UAC was not asked to study and recommend a process for implementing Policy 11.9 – but rather to focus on "small water systems." The size of "small water systems" was not defined, but for purposes of this memo the UAC will consider "small" to be all systems other than the three largest owned by the City and KPUD.

The drinking water policies and goals in the afore-described 2017 Comprehensive Plan build on those already outlined in the Water Resources Element of the City's 2004 Comprehensive Plan, namely:

- WR 3.2: *The City may elect to facilitate small water system management services by applying to the Department of Health to be an approved Satellite Management Area (SMA).*

- WR 3.3: *New development in previously unclaimed water service areas may be required to dedicate public water systems to the City if the system meets City standards and the City determines it is appropriate to accept, own and operate such systems*.

All of these goals and policies were adopted to help ensure that all island residents have safe drinking water.

B) Water Systems on Bainbridge Island

Potable water service is provided to residents of Bainbridge Island from private single or double[1] -connection wells or public water systems classified by the Washington State Department of Health as Groups A or B, depending on the number of connections.

The UAC estimates that roughly 65% of Island residents are served by Group A systems, 7.5% by Group B systems, and the remaining 27.5% have private single-connection wells.[2] ~~*[Later number seems high.]*~~

1) Group A. Those water systems serving 15 or more connections or the general public are labeled Group A. There are 30 Group A systems on Bainbridge Island which are listed below by number of connections, and shown on a map attached as Exhibit A. (These numbers and ownership should be updated.) [3]

Winslow/Fletcher Bay (COBI)	2700
North Bainbridge (KPUD)	1922
South Bainbridge (KPUD)	1386
Meadowmeer (managed by KPUD)	346
Island Utility (KPUD)	317
Port Madison (managed by KPUD)	101
Bucklin (managed by WA Water Co)	95
Bill Point (managed by KPUD)	84
Emerald Heights	81
Rockaway Beach (COBI)	66
Phelps Road (managed by WA Water Co)	22
Harbor Crest (KPUD)	21
Rose Avenue	20

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[1] The 2005 Kitsap Coordinate Water System Plan notes that normally only single-connection wells are considered "private," but the Kitsap Public Health District has an exemption that allows double-connection residential wells meeting certain criteria to be classified as a "private well."

[2] This calculation assumes 7250 Group A and Group B connections contain 2.1 residents per connection, with the remainder of Bainbridge's 24,000 citizens being on private wells.

[3] This data is provided by COBI Public Works Department from information provided by Kitsap Public Health District. The data regarding the Kitsap Public Utility District (KPUD) is from the KPUD Water Service Plan, adopted March 2020.

Place Eighteen (managed by KPUD)	18
Crystal Springs (managed by KPUD)	18
Manzanita	18
Ferncliff (managed by KPUD)	17
Seabold Heights (managed by KPUD)	12
Strawberry Hill Park	10
Strawberry	6
Carden Country School	5
COBI Public Works	4
Port Madison Yacht Club	2
Fort Ward Park	2
Eagle Harbor Marina	1
Bloedel Reserve	1
Messenger House	1
Island Center Community Center	1
Eagledale Park	1
Battle Point Park	1

2) **Group B**. Those water systems serving 2 to 14 connections are classified by the State and Kitsap Health District as Group B. There are 144 Group B systems on Bainbridge Island, many of which are shown on a map attached as Exhibit B.

3) **Private Wells**. Many properties owners obtain potable water for a single residence or business from a private well. These are not considered "public water systems," and therefore are not addressed in detail in this analysis.

C) Ownership and Management of Water Systems

Water systems are owned by a variety of organizations: governments like COBI, for-profit corporations, non-profit organizations such as homeowner associations, and private individuals and businesses.

Many water systems, primarily very small ones, are self-managed. That is, the owners utilize volunteers or contractors to perform needed operational and administrative tasks.

In addition, two large organizations manage water systems owned by others pursuant to negotiated contracts of varying durations. They are: Kitsap Public Utility District (KPUD) and Northwest Water Service, Inc.

KPUD owns and manages 4 Group A systems on Bainbridge Island: North Bainbridge, South Bainbridge, Island Utility, and Harbor Crest; and at least 3 Group B systems. See map at Exhibit A. KPUD also manages Group A water systems on Bainbridge Island owned by other entities, typically homeowner associations. Examples are Port Madison, Seabold Heights, Ferncliff, Place 18, and recently Bill Point and Meadowmeer.

Northwest Water Service, a private for-profit company headquartered in Port Orchard, manages 6 Group A and 16 Group B systems on Bainbridge Island. It manages 450 systems in Washington. Northwest Water generally does not provide complex engineering and construction services, but may refer that work to KPUD or Washington Water.

Washington Water Service, a division of a publicly-traded California corporation with a local office in Gig Harbor, owns and manages three Group A water systems on the Island. It used to manage other water systems as well. However, in 2018, Washington Water changed its business model and will now only manage water systems it owns. The company thus withdrew from managing various Island small water systems unless and until they agreed to sell to Washington Water.

D) Service Areas.

The 2005 Kitsap County Coordinated Water System Plan assigned "service areas" to Group A systems within the County. Most areas covered just the areas currently being served by the water systems. Some larger water systems, like COBI and KPUD, were also assigned areas that were not currently being served by that utility. This assignment essentially established that particular water system as the primary provider for the area. Since 2005, KPUD has taken over ownership or management of several small water systems outside its assigned service areas.

The attached Exhibit B shows the assigned service areas on Bainbridge Island. COBI has three assigned areas: a large swath across the middle of the Island in which the Winslow-Fletcher Bay system is located; the Rockaway Beach area; and the north tip of the Island in which neither COBI or any other Group A system presently operates.

E) Problems Identified with Small Water Systems

Beginning in the 1970's, the WA State Department of Health identified significant deficiencies with many small water systems throughout the state. Generally, many small water systems have some or all of the following problems: (1) lack of professional management; (2) outdated and deteriorating infrastructure; (3) lack of appropriate capital improvement planning and capability; (4) insufficient financial resources; (5) inadequate backup and support; and (6) limited fire suppression facilities.

Commented [1]: Health mentioned fire? I'm surpised.

For example, there are no requirements by either the State Office of Drinking Water or Kitsap Public health that Group Bs have reserves for improvements or repairs. The State Department of Health does require Groups A systems to have a capital reserve at the time they are initially created and approved. There is also a requirement

that the amount of reserve be reviewed every 5 years during a Group A sanitary survey inspection, but this requirement is dependent on the State Department of Health's staffing. (Anecdotally, DOH did fail to review Meadowmeer's reserves at its last inspection due to new staff.) There are no reserve requirements for Group B systems. This situation is becoming more significant because water small water systems were developed in the 1960's and 1970's, and their equipment is reaching the end of their useful lives.

Small water systems also often rely on volunteer staff with little to no professional experience or knowledge of regulatory changes. Volunteers may also be reluctant to raise rates to make necessary safety improvements like backup generators for power outages, fire hydrants and seismic improvements. These safety concerns are becoming more pressing due to the effects of global warming and the drier, longer summers Bainbridge is experiencing.

DOH has encouraged small water systems to develop more "professional" planning, maintenance, financing, and staffing, and to consider consolidation with other water systems in order to jointly possess the resources to develop those attributes of effective water system management. As noted above, the City's comprehensive plans have similarly called for consolidation.

Neither the City's Public Works Department nor the UAC has conducted a survey of the condition of the Group A and B systems on the Island, so we are not in a position to state which, if any, of the above-identified deficiencies are suffered by Island water systems. However, the UAC has ~~taken testimony~~received complaints from customers of numerous small water systems about deficiencies in their systems and the need for assistance. Therefore, more data would be helpful.

Finally, as identified in the 2005 Kitsap County Coordinated Water System Plan, smaller water systems negatively impact the ability to accomplish water resource management made possible by larger interconnected water systems, and Group B water systems, which typically have shallow wells, are more likely to negatively impact stream flows.

F) Legal and Regulatory Authorities Pertinent to Small Water Systems

~~a.~~ 1) Federal: EPA Safe Water Drinking Act (1974). The Act protects public drinking water supplies throughout the United States. It sets regulatory limits for the amounts of certain contaminants in water provided by public water supply systems. The EPA also implements various financial programs to ensure drinking water safety. (See Title XIV of Public Health Service Act.)

~~b.~~ 2) State: Public drinking water is regulated at the State level by the Washington State Department of Health. Group A's (15 or greater connections or

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public access): The Washington State Department of Health (DOH) regulates Group A's under state law and a formal agreement with the US Environmental Protection Agency (EPA) for carrying out the federal Safe Drinking Water Act. DOH's Drinking Water Division's highest priority is responding to actual or potential public health emergencies. DOH's technical staff is available to water systems 24 hours a day. See Ch. 246-290 WAC.

~~c.~~a. Group B's (2-14 connections**)**: Group B systems are regulated under Chapter 246-291 of the Washington Administrative Code. Group B regulations suggest that new Group B owners should design their systems to comply with the more regulated Group A requirements like seismic safety, power outage, back-up systems, and capital reserves in order to allow for future consolidation into Group As. Older Group B systems had no such requirements. Group B rules for ongoing oversight were amended in 2014 to be optional due to State budget cuts at DOH, so DOH does not provide oversight of Group B systems.

~~d.~~b. Revolving Fund**:** The DOH makes funds available to Group A drinking water systems to pay for infrastructure improvements and consolidation of Class A systems or consolidation of Group B's into existing Group A systems through the Drinking Water State Revolving Fund**.** There was $20 million available in the last cycle**.** The loan program is funded through state and federal money and is subject to state laws and additional federal regulations. These loans cover capital improvements that increase public health and compliance with drinking water regulations. They are for Public entities, so COBI and KPUD can apply for these loans, but private management agencies (like Washington Water) can not apply.

Commented [2]: Does that also mean that HOAs may not apply for loans, no matter who is managing the system; volunteers or pros?

~~e.~~c. State Loans: In implementation of the priority the State Department of Health places on the consolidation of Group A water systems, the State has created a loan and grant program. ~~Among the~~One such program~~s~~ ~~i~~offers $25,000 grants to help public entities (like the City) investigate consolidation.

~~f.~~d. DOH Priorities: One of the priorities of the DOH is to consolidate Groups A's (and possibly Group B's with Group A's or other B's) for the following reasons:

- Better economies of scale ~~and~~ ensure~~s~~ better supervision and monitoring
- Large systems are generally more reliable
- Large systems reduce the administrative burden on the Health Department
- Improved well head protection ensures aquifer protection for all
- Improved fire safety

~~4)~~ 3) Kitsap Public Health District: Although often mis-identified as a Kitsap County department, Kitsap Public Health District (KPHD) is a separate district governed by a seven-person Board composed of the three county commissioners and the mayors of the four Kitsap cities, including Bainbridge Island.

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Group A's: KPHD defers to the State DOH to regulate Group As, but assists State DOH with sanitary survey inspections/reports through a joint plan of responsibility agreement.

Groups B's: KPHD initially inspects Group B's when they are installed. In September 2018 the KPHD adopted the proposed ordinance 1999-6, now approved as Ordinance 2018-01 which regulates ongoing operation of Group B's. The new ordinance requires all Group B's to purchase an annual permit, have a sanitary service inspection every 5 years, and obtain a water status report from the Health District at the time of sale to be delivered to the purchaser. Between 2009 and 2018, due to lack of funding, no one was monitoring Group B's for potential acute health problems from bacteria, nitrates and poorly monitored well head protection.

Wells: The KPHD publishes wellhead protection guidelines for owners of wells, but it is not clear whether these rules are well distributed or whether any government entity ensures that the rules are followed.

~~5)~~ 4) WUTC regulation: The Washington Utilities and Transportation Commission regulates the rates charged by private water companies (such as Washington Water Co. and Northwest Water Inc.) operating within Washington State that have 100 or more connections or charges more than $557 a year per customer.

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G) History of COBI Policies and Activities Related to Island Water Systems

Prior to 1990, the City of Winslow owned and operated a water system to serve the Winslow area. The rest of Bainbridge Island was served by many private water

systems, most of which were for-profit corporations or homeowner associations. One of the issues influencing the Home Rule movement in the 1980's was a desire to better protect and manage the Island's water supply.

In 1991, the City of Winslow annexed the remainder of Bainbridge Island creating the City of Bainbridge Island. This had no direct effect on the ownership or management of water systems on the Island, but it did mean that the new COBI could begin to develop plans for water supply and system management. In February 1993, COBI adopted its first such planning document, Resolution 93-3., attached as Exhibit C. This document stated, among other things, that the City shall be prepared to, upon request from a water system, acquire a system, become a contract service provider for a system, or provide support assistance to a system.

In 1994, the Rockaway Beach Homeowner Association approached the City because its water system was failing. In implementation of the above policy, the City acquired the Rockaway Beach water system, and utilized the Local Improvement District (LID) process to construct a new well, reservoir, and distribution lines.

In subsequent years, the City acquired other small water systems in the Fletcher Bay area, relatively near the City's deep well obtained from KPUD in the early 1990's. This well is a major water source for Winslow, and large transmission line extends from Fletcher Bay along New Brooklyn Rd. to Winslow.

Commented [3]: Is main a better term than line?

In the various City comprehensive plans since inception there have been water policies calling for ~~_____~~:

- Ensuring reliable, safe, and cost-effective water for City and non-City managed utility servie;
- Preserving and protecting water resources;
- Encouraging water utilities to enter into cooperatie activities;
- Encouraging and facilitating consolidation with particular emphasis on mergers of contiguous and small systems;
- Applying to the Department of Health to become a satellite management agency (SMA); and
- Requiring new development to build according to City standards (Group A standards) and dedicate water systems to the City when it isa appropriate for the City to accept such system.

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~~-~~Regarding the City serving as ~~and for the City to become~~ a Satellite Management Agency (SMA), it did so~~. In fact,~~ in 1994 during the City's assumption of the Rockaway Beach system~~, the City did act as an SMA. In the late 90~~and apparently provided limited support services in the late 1990's to 19 small water systems. However, d~~, d~~ue to staffing concerns and other more pressing issues, in 2000, the City

stopped allowed the services contracts to expire~~operating as an SMA~~. In 2002~~1 (??)~~, the City revoked that portion of Resolution 93-3 regarding serving as an SMA.[4]

In later comprehensive plans, the City was encouraged to again reapply to the Department of Health to become an SMA in order to facilitate consolidation and ensure better drinking water safety for its citizens. (See Section IA above for reference to WR 3.2 of the Water Resource Element of December 2004's comprehensive plan.)

In 2001, the North Bainbridge Water Company approached the City about purchasing its system. The City staff commenced negotiations, but the owner of the company sold it to KPUD. KPUD Board of Commissioner minutes indicate KPUD believed COBI had decided not to purchase the company. That is not correct; the issue never reached the City Council.

In 2016, KPUD purchased the South Bainbridge Water Company and Island Water Company. These systems were sold to KPUD by their private owners without notice to the City despite an informal agreement between the City and KPUD that KPUD would not purchase any water systems within the City without first obtaining agreement from the City.

In the 2017 Comprehensive Plan, the City adopted the policies listed on the first page of this memorandum. (See section I.A. above.)

In 2017, a small 8-customer Group B water system known as Casey Street approached the City about acquiring it. After consideration, the City chose to do so because: (1) it is within the City's water service area; (2) it is located in the north Ferncliff area relatively close to the City's existing Winslow water system; (3) the City has periodically discussed constructing a new water pipeline from North Ferncliff to North Madision to "loop" the Winslow system, which would be directly adjacent to the Casey Street system; and (4) owning this system would be a "test case" about how COBI might manage small systems in the future.

In 2018, two Group A systems, Bill Point and Place 18, were faced with changing managers when Washington Water Service withdrew. COBI staff had preliminary discussions regarding assuming satellite management of the systems, which would require eventual approval from Washington DOH, but the water systems contracted with others.

In 2019-20, three larger Group A systems, Emerald Heights, Bill Point, and Meadowmeer contacted UAC representatives and some City senior staff about possible assistance from COBI. In February 2020, the UAC recommended to the City Council and senor staff that COBI consult with the three systems regarding their needs and the assistance the City might be able to provide. Apparently, City representatives have not

[4] The Resolution revoking SMA authority, Resolution 2002-25, indicates in its title that the entire Resolution 93-3 has been ~~vacated~~repealed. However, the content of Resolution 2002-25 states it is ~~vacating~~ eliminating the SMA ~~authority~~program only. Thus, there may be an ambiguity as to whether the remaining portion of Resolution 93-3 is still in effect.

had such consultations (COVID limitations no doubt impacted the staff), and Bill Point and Meadowmeer have retained KPUD as manager for the time being.

Lastly, the 2019-2020 budget called for the City to begin a Groundwater Management Plan (GMP). For a variety of reasons, this process did not begin, and we understand that during the 2021-22 budget deliberations the City Council will be re-engaging the discussion around the development of a GMP. We further understand the planned first step in the GMP process was to be the hiring of a hydrogeologist to manage the plan. The UAC supports the development of a GMP, and suggests the work can be expanded to include implementation of many of the recommendations contained in this memorandum.

III. ANALYSIS AND RECOMMENDATIONS

In the three decades since all-island annexation, the City has had only limited focus on Island water systems that it did not own. This is surely because the City was focused on other more pressing issues, such as land use policies and transportation, and because Island water systems were in general not seeking assistance from the City since they were still newer and in better repair.

Historically, the City had not been involved with the smaller water systems, and the City's passage of Resolution 93-3 did not significantly change that position. The City has no direct regulatory authority over Island water systems, other than management of the City's rights-of-way. Therefore, the City has had only minor interactions with Island water systems, and did not "advertise" its availability as a resource for those systems. Accordingly, Island water systems needing assistance had to turn to private water companies and KPUD out of necessity.

In the meantime, interest in preserving water resources and in improving water quality in aging systems has become a larger concern of most island citizens, particularly in light of the steadily increasing development of the island and increasing population. To meet this increasing concern, t~~T~~he City and others conducted studies of Island water resources and the various aquifers which exist beneath the island. Unfortunately, to date, no studies were conducted about how the multitude of water systems, either individually or in the aggregate, were effectively and efficiently managing the resource and whether small water systems were protecting the health of islanders by ensuring good water quality. The import of this issue of local ~~City~~ control of the water consumed by its citizens has also been highlighted by the catastrophic consequences of Flint, Michigan's water supply being taken under control by the State. Similarly, stories across the country of private entities controlling water supplies and charging exorbitant rates while failing to ensure water quality have increased.

As mentioned, for many decades the State encouraged the consolidation and professionalization of water systems. Kitsap County has followed those policies. For example, the 2005 County Plan noted that Group B's with shallow wells negatively impacted stream flows and negatively impacted the ability to accomplish water resource management. The 2005 County Plan thus calls upon the County's water systems to

Commented [4]: I do not believe mentioning Flint actually helps our case. We are staying in general that larger and more governmental is safer, but MI failed them catastrophically. Really the issue there was that the state acted like it was not accountable to its citizens.

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coordinate and consolidate and to stop the proliferation of new small water systems. One of the principal points of the 2005 County Plan is to "provide improved coordination of new development and to restrict the proliferation of small public water systems." (See section 2-3.) The 2005 County Plan also called for "future growth planning" to be "in consideration and coordinated with the water service that it is in." (See Exhibit 2-4)

Most importantly, the conclusion in section 1.2.11 and in section 9 of the 2005 County Plan concerning concentration of Kitsap growth and resultant potential water shortages in other areas of Kitsap County like the Dyes Inlet and Sinclair Inlet (opposite the South end of Bainbridge) and North Kitsap (to the North of Bainbridge) and the conclusion that it will be necessary to develop water sources in the surrounding parts of the County and "establish transmission mains to move the water to where growth is occurring" clarifies that it is important for the City to retain control of its wells to ensure that the City retains a seat at the table in any future discussions concerning use of water from Bainbridge Island wells for future expansion needs of other areas in Kitsap County.

The 2005 County Plan suggests that Bainbridge as a water purveyor should implement land use policies that ensure efficient and effective management of water resources for numerous reasons. The 2005 County Plan also assigned responsibility to Bainbridge Island's water service to plan for water system development within its assigned water service area, to require new development to be built to Group A standards to help facilitate connection at a later time, and to require said new development to record a covenant at the registry of deeds and note the covenant on a title report for subsequent owners. (See Section 5.3.3)

It is meaningful to note that the 2005 County Plan (section 1.2.11 and in section 9) projects population growth concentrated in Central Kitsap and potential resultant water shortages in Dyes Inlet and Sinclair Inlet (opposite the South end of Bainbridge) and North Kitsap (to the North of Bainbridge), and the conclusion that it will be necessary to develop water sources in the surrounding parts of the County and "establish transmission mains to move the water to where growth is occurring." This argues for the City to retain control of Island wells to ensure that the City retains a seat at the table in any future discussions concerning use of water from Bainbridge Island wells in other areas in Kitsap County.

Given this situation, what should be the City's role toward Island water systems? Should the City continue to ignore requests for help and thus consolidation opportunities from smaller water purveyors like Emerald Heights, Bill Point, and Meadowmeer? Should it adopt a somewhat more active policy of providing assistance only when approached by a small water purveyor? Or more proactively still, should the City seek out water systems to assist in the management (and perhaps ownership) of their water systems, so as to encourage more professional management of them and more island-wide planning and cooperation? What should be the City's role in ensuring Island residents have access to safe and adequate water?

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Whatever policy is chosen, the focus of the City should be the long term. How do the citizens of Bainbridge Island want their water resource to be managed in 10, 30, 50, 75, 100 years? Is it through many, decentralized small water systems, or by one or more large organizations? As stated, for many of the reasons outlined in this memo above, the State and County have ~~has~~ made it clear that it believes consolidation over time is the best means for ensuring quality drinking water for all citizens.

A) Policy Alternatives

The UAC suggests there are four policy alternatives for COBI's relationship with the other water systems on Bainbridge Island. They are:

1) Minimal

In this situation, COBI decides it will have nothing to do with water systems on Bainbridge Island other than those it currently owns: Winslow-Fletcher Bay, Rockaway Beach, and Casey Street. COBI will defer entirely to other water purveyors/managers, primarily to KPUD and Northwest Water Company.

The City might do this because it takes the least effort, staff time, and cost, and allows it to focus fully on its current 2700 water customers. On the other hand, this policy abdicates any assistance to the vast majority of Bainbridge Island land and citizens, and likely gives up any long term ability to consider, manage, and protect the entire Island water resource or protect the safety and adequacy of drinking water for Island citizens.

2) Reactive

This Is essentially the policy in Resolution 93-3, and describes what the City has been doing in the past 30 years. That is, on occasion, the City has provided assistance to small water systems or acquired them. This policy is the status quo – the City reacts to requests from other water systems and may respond consistent with staff time and interest.

This policy is slightly more energetic than the Minimal alternative discussed above, in that the City may decide to deal with the needs of other water systems if the city staff or council conclude that it can do so within existing resources. But, this reactive policy could lead to unfair practices, ie., assistance to some water systems and not to others.

3) Active

Under this policy, the City reaches out to the small water systems and encourages assistance and cooperation. It does so by devoting sufficient City staff, time, and resources to determining the needs of the water systems and developing financial, engineering, and management solutions. This policy does not mandate that

small water systems participate with the City in any way, but rather calls for the City to develop the resources that will attract cooperation from small systems.

This policy will enable COBI to become more actively engaged with managing and thus preserving and protecting Island water resources. This policy has the potential to increase revenue to the City's existing water utility, thus utilizing economies of scale to benefit both current City and small water system customers. Also, in the long term, this policy may be the most cost effective means to protect Island water supplies and insure safe and adequate drinking water for Island citizens.

Among those steps that could be taken are to become a Satellite Management Agency (SMA) pursuant to WAC 246-295, to require new development within or adjacent to the City's designated service areas to either connect to the City water system or build to Group A standards to facilitate future connection. The City could also begin implementation of the 2005 County Plan's directive to create a water system plan which includes a program of capital improvements required to provide the anticipated level of service and to ensure that Bainbridge's water utility plan responds to its own land use and comprehensive plan policies.

This active policy will, at least at the outset, require some investment by the City. There should be no doubt about this. A major reason the City has not been more involved with Island water systems and resources is that it has not had the budget to do so. To be effective, the City must hire staff and perhaps consultants, when needed. The UAC has not attempted to determine any details of the costs to implement this policy; the Public Works Department is best suited to do that.

It is possible the funding for this investment could be shared by the City's water utility and general fund accounts so as to not unduly burden the City's existing water customers. That is because there is a general island-wide purpose behind this policy (water resource protection) and also a potential COBI water utility benefit (economies of scale).

4) Active Plus Acquisition (over time)

This policy is essentially the same as the Active policy above, but with one significant change. That is, if a small water system requests assistance from the City with meeting its defined needs, be it management, testing, billing, reconstruction, etc., the City may agree to do so on condition that the water system agree to be consolidated within the COBI water utility at some established time in the future. The UAC does not recommend a set period, leaving that to negotiation between the parties.

This policy is in furtherance of DOH's long-standing objective to consolidate small water systems within larger ones, and encourages long-range planning of water resource management.

To aid in the consideration of these four alternatives, the UAC has prepared a discussion Matrix, attached as Exhibit D to this Memo. The matrix outlines possible levels of city action for water systems within COBI's Current and Future Retail Service

Areas as defined by the Kitsap County Coordinated Water System Plan, and within small water systems that are not within COBI's assigned service areas. Thus, specific long-range action plans are outlined for water systems that border the existing City service areas, and suggest deference to KPUD for areas that are too distant from City service.

The matrix also differentiates between smaller Group B's and the larger Group A systems, and posits that the City may establish different policies for them. Finally, the matrix provides pros and cons of the possible recommendations and provides examples of sample water systems that fit within the matrix sections.

B) UAC Recommendation

For the reasons outlined above, the UAC believes that the City should depart from its laissez faire attitude toward other water systems on Bainbridge Island, and become more actively involved with managing the water resources on the Island.

Accordingly, the UAC recommends that the City adopt alternative #4 for small water systems within and adjacent to the City's assigned service areas. That is, the City should actively seek to provide assistance as needed by those systems, and in doing so there should be a plan for eventual consolidation within the City's water utility.

However, for the small water systems located outside the City's service areas, the UAC recommends adopting policy alternative #3. That is, the City should reach out to small water systems to offer assistance as needed.

When implementing the above recommendations, the UAC suggests that the City start by focusing on the 14 small Group A systems that provide service to water customers (as opposed to the remaining Group A systems which primarily serve parks and facilities for the general public, such as Bloedel Reserve.). Thereafter, the City could expand its outreach to the many Group B systems on the Island. However, due to their large number and likely limited incentive to seek professional assistance, the City should proceed with Group B systems much more slowly.

C) Relationship with KPUD

KPUD is a major provider of water service on the Island, owning four systems (two large and two small) and managing six others. This effort by the UAC was specifically directed toward small water systems. The UAC was not asked to discuss or evaluate the present and future relationship between the City and KPUD. Thus, to a certain extent, this analysis and recommendation has been prepared while "avoiding the elephant in the room."

The City and KPUD both serve approximately the same number of customers (approximately 3000) on the Island, and both have two large County-designated service areas. Yet there is no island-wide water resource and management planning being done by KPUD or the City, separately or together.

At the current time, KPUD and the City are collaborating neighboring water utilities serving their own customers and cooperating in limited ways on planning and projects, while at the same time being competitors toward water systems they don't own or manage.

At some point in the future, the City needs to develop a policy about how it intends to work with KPUD. Comprehensive Plan Policy U 11.9 anticipates that analysis, when it states: "*Conduct a study of consolidation of water systems owned by the City and Kitsap Public Utility District. Pursue long-term consolidation of larger water systems.*"

Such a study *could* result in consolidation of one of more of the large water systems, but it could also result in collaborative activities such as construction of storage facilities and transmission lines, or preparation of island or sub-regional water resource planning. At a minimum, an interlocal agreement should be developed which outlines how and under what circumstances either utility will provide services to or assume ownership of non-owned water systems on the Island. This will prevent the acquisition of water systems by either utility without knowledge of the other.

D) Suggested Next Steps

If the Council is prepared to establish policies toward small water systems, the UAC suggests the following initial steps:

1) Create or assign one or more staff positions within the Public Works Department to focus on water systems and water resource policy.

~~1)~~2) Coordinate – and to the extent practical, include – the recommended next steps on small water systems into the City's development and implementation of upcoming work on a Groundwater Management Plan.

~~2)~~3) Update and fine tune its information about Island water system in a comprehensive inventory and map. There is data in a variety of locations about the water systems, which has been the basis for information provided in this memorandum, but there is no single collection of the information in one usable and up-to-date location.

~~3)~~4) Internally consider how COBI can provide management, assistance, and capital improvement services for small water systems. The idea is to develop a list of services the City may be able to provide to smaller water systems if needed, such as billing and other administrative tasks, equipment rental, emergency assistance, and inspection and testing.

~~4)~~5) Convene a meeting(s) of all small Group A and B systems to discuss problems, potential solutions, cooperation, consolidation, etc., and to advise of the availability of City resources to assist.

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~~5)~~6) Following the meeting(s) described above, consider and propose steps to solve identified problems in the attending small water systems, and consider and propose recommendations for a larger policy for the City toward small water systems. Priority should be given to Group A systems. Among the issues to consider when developing such steps and policies is the financial fees charged to support the City's efforts.

~~6)~~7) An example of City services might be the establishment of a common venue to enable small water systems to do water sampling, and providing an experienced City employee to assist volunteers with their questions about these and other operational tasks.

~~7)~~8) Consider applying to the State Department of Health to become an approved Satellite Management Agency (SMA).

~~8)~~9) Consider establishing ~~a~~ policies that make consolidation of systems less administratively burdensome and costly to the property owners and water systems. For example, consider ~~y that r~~requiring~~es~~ all new Group B systems to be constructed to Group A standards, thus more easily allowing future consolidation with Group A's, and consider requiring new building permits for private single-connection wells to consent to later connection to public water systems.

~~9)~~10) At the appropriate time, develop an established plan or pathway for the City to assume ownership of and provide capital improvements to systems which need assistance. Such planning would include necessary financing mechanisms, such as Local Improvement Districts (LID's) and state loans and grants.

IV CONCLUSION

The UAC believes it is time for the City to depart from its laissez faire attitude toward water management on Bainbridge Island, ~~-~~Accordingly, the UAC recommends that the City adopt a policy of providing active assistance to the small water systems on Bainbridge Island, and encourage voluntary consolidation over time of those systems within the City's current water utility. The UAC recommends the City prioritize its efforts and focus on Group A water systems and the small water systems within or adjacent to the City's current county-assigned service areas.