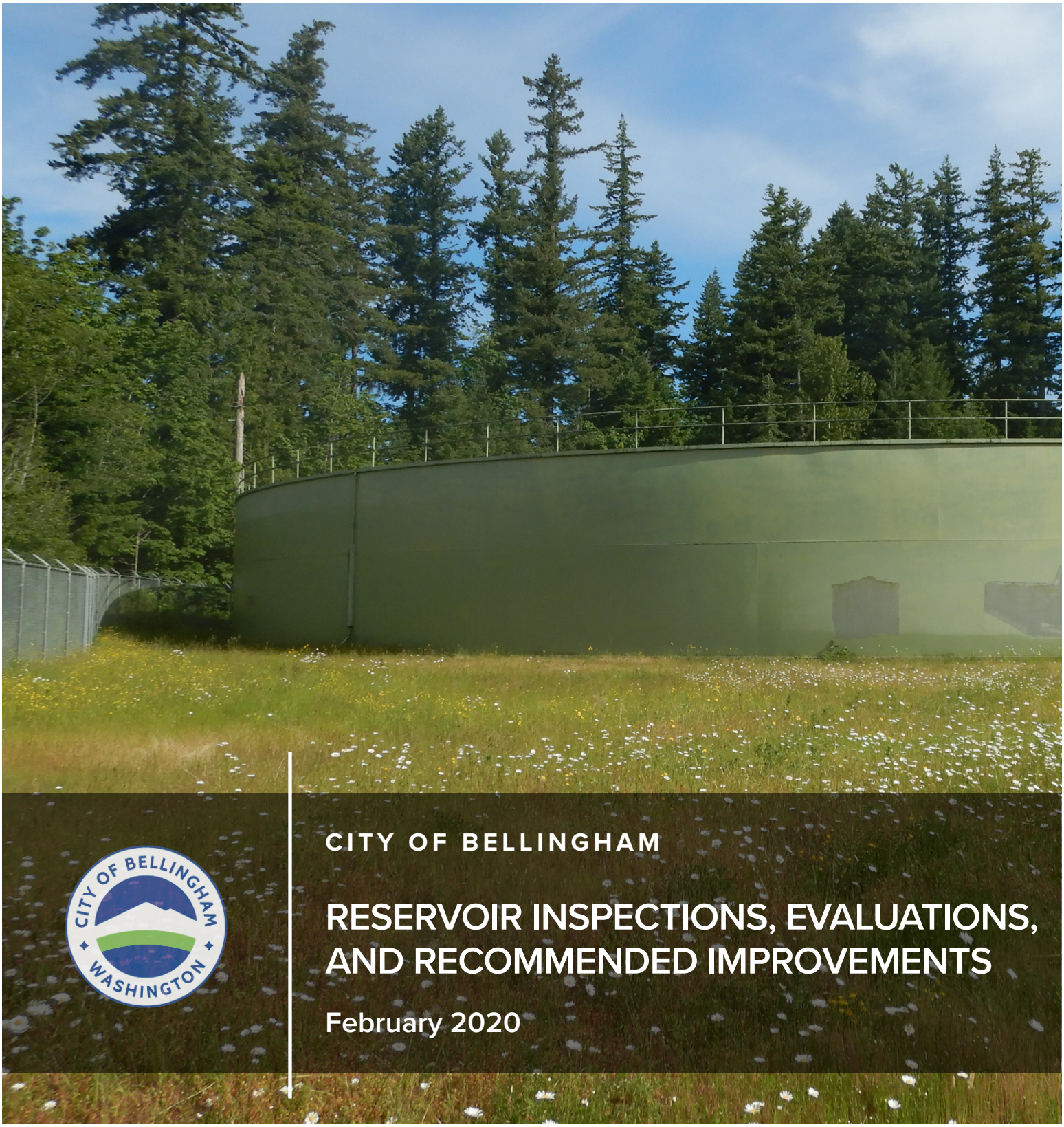

CITY OF BELLINGHAM

RESERVOIR INSPECTIONS, EVALUATIONS, AND RECOMMENDED IMPROVEMENTS

February 2020

Reservoir Assessments

The City of Bellingham

February 2020

PRELIMINARY

Murraysmith

2707 Colby Avenue
Suite 1110
Everett, WA 98201

Preface

This report summarizes the findings of Murraysmith to date. An underwater diver inspection of the Whatcom Falls II Reservoir is planned for April 2020.

Acronyms & Abbreviations

AC	Alternating Current
ACI	American Concrete Institute
AMSL	Above mean sea level
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
bgs	below ground surface
CBD	Central Business District
City	City of Bellingham
CMU	Concrete masonry unit
Code	2015 International Building Code
DFT	dry film thickness
DOH	Washington State Department of Health
Domed RC	reinforced domed concrete tanks
DOSH	Division of Occupational Safety and Health within L&I
EPA	United States Environmental Protection Agency
FRP fabric	Fiberglass reinforced polymer fabric
L&I	Department of Labor and Industries
MCE	maximum considered earthquake
MGD	million gallons per day
MG	million gallons
mils	thousandths-of-an-inch
NWC	Northwest Corrosion
OSHA	Occupational Safety and Health Administration
PAHs	Polycyclic Aromatic Hydrocarbons
PCA	Portland Cement Association
PSE	Peterson Structural Engineers
PSC	Prestressed concrete
psf	pounds per square foot
RC	reinforced concrete
RCC	roller compacted concrete
Score Matrix	Condition Assessment Score Matrix
TCLP	Toxic Characteristic Leaching Procedure
Type 2 gauge	portable electromagnetic dry film thickness gauge
USGS	United States Geological Survey
WAC	Washington Administrative Code
WISHA	Washington Industrial Safety and Health Act
WTP	water treatment plant

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Executive Summary

Introduction

In December 2018, Murraysmith was authorized by the City of Bellingham (City) to provide engineering services for the City's Reservoir Inspection/Development of Bid Specifications for Reservoir Repairs and Repair Oversight Project. Phase 1 of this project is to perform a thorough, comprehensive evaluation of the City's 13 reservoirs including condition assessments and structural/seismic evaluations. This phase generally includes the following steps:

- Field Inspections – perform inspections of drained reservoirs, and conduct floating inspections as needed
- Condition Assessments – prepare condition scoring system and summarize deficiencies
- Recommended Improvements – identify improvements to resolve deficiencies

Future phases of this project include prioritizing recommended improvements and developing bid documents to complete the improvements.

Field Inspection and Assessment Methods

The Murraysmith team performed inspections to determine the overall condition of the facilities. This included completing geotechnical investigations and physical inspections of each reservoir. The geotechnical investigation at the reservoir sites includes a description of the general subsurface conditions, foundation design parameters and seismic design criteria to aid the structural analysis.

The physical inspections of the reservoirs were completed by a multi-disciplinary team including engineers from Murraysmith and subconsultants Peterson Structural Engineers and Northwest Corrosion Engineering. In addition to the overall condition of each reservoir, the team reviewed coatings, appurtenances and completed corrosion and structural inspections.

Murraysmith developed a condition assessment score matrix to uniformly assesses the condition of each reservoir. Twenty-eight reservoir individual system/structures were identified for scoring and grouped into the seven following System Groups:

- Site/Security
- Walls
- Floor/Foundation
- Roof
- Appurtenances
- Piping/Valving
- Miscellaneous

Each System Group was evaluated on seven assessment categories (and subcategories):

- Cleanliness and Coatings
- Material Deterioration (Concrete Deterioration/Metal Corrosion)
- Structural Performance (Static, Seismic)
- Water Quality/Sanitary
- Safety
- Operations and Maintenance (Site and Security, Roof Drainage, Appurtenances, Valving and Piping, Misc.)
- Obsolescence

Scores were assigned to each System Group ranging from one (1), implying that the component is deficient, to five (5), implying that it meets criteria completely. A weighting system was used to determine an overall score for each reservoir. This weight was developed with input from City staff and accounts for the risk associated with each assessment category, shown in Table ES-1. For example, a seismic event-induced issue would likely cause a major disruption to service and be very time-consuming and expensive to rectify.

Table ES-1: The weighting system used in determining reservoir scores.

Category	Weight (out of 100)
Cleanliness and Coatings	5
Material Deterioration	10
Static Structural Performance	20
Seismic Structural Performance	30
WQ/Sanitary	10
Safety	10
O&M	10
Obsolescence	5

Condition Assessments and Recommend Improvements

The City has done a superb job at delivering high quality water to its customers over the years. While areas for improvement were identified on the reservoir systems, they are all completely operational and previous problems have been addressed proactively by the City. The assessments and recommendations are based on current and planned code requirements and standards.

Some common operational-type issues appeared on many or all the reservoirs, such as vegetation clearances, air gaps, and dechlorination systems. A summary of some of the reservoir specific deficiencies are provided below.

Marietta: The currently empty reservoir shell and roof supports are in good condition, likely due to coating materials that prevent corrosion and do not deteriorate. However, the coatings are archaic, and the City can consider a modern coating system. If used, the maximum operating level is recommended to be reduced, anchors removed, and flexible couplings installed on the piping to increase seismic resiliency of the reservoir. Safety, drain, overflow, and vent improvements are also recommended.

Padden: The reservoir is in fair condition given the observed corrosion, coating failure, and potential seismic issues. Significant corrosion was noted the interior roof and supports. The exterior and interior coating systems are at the end of their service life; interior and exterior recoating is recommended. To alleviate seismic event slosh damages, the maximum operating level is recommended to be lowered from 24.5 to 19.75 feet or alternately, retrofit/replacement should be considered. The foundation will also require retrofits if operated higher than 22.25 feet.

Whatcom Falls I: The reservoir is in generally good condition. However, to alleviate seismic event slosh damages, the maximum operating level is recommended to be lowered from 16.5 to 14 feet or alternately, retrofit/replacement should be considered. Depending on the new operating level, the corrosion on the interior roof and its supports needs to be addressed. The exterior coating needs only coating touch ups. The cathodic protection system's breaker was not functioning, which needs immediate attention.

Dakin II: While in good condition, a roof drainage problem is potentially negatively affecting the structure. Organic material accumulates on the roof and blocks drains; the stagnant water's freeze-thaw cycles are likely causing observed cracking. Roof drainage is recommended to be improved and concrete repaired.

Kearney: The 14-year old reservoir is in very good condition. However, the roof appears to be draining onto the exterior walls, which potentially is causing delamination of the outer shotcrete layer in two specific areas and staining in others. These areas should be monitored, and potential roof drainage modifications should be considered to reduce the risk of damage to the exterior shotcrete and underlying wire strand.

Whatcom Falls II: The Whatcom Falls II Reservoir is an almost 30-year old prestressed concrete tank that is still in very good condition. One major issue that the City is unable to draw down the reservoir while meeting chlorine contact time requirements. This will likely be problematic in the future if the City needs to conduct inspections or maintenance. The roof is subjected to heavy organic loads that the reservoir cannot shed that is exacerbated by unsuitable drains and the roof being very flat. The roof drainage deficiencies do not yet appear to be inducing major roof curb, roof slab, or wall shotcrete issues, but the reservoir should continue to be monitored.

40th Street/College Way/Consolidation/Dakin I/Reveille (Reinforced Concrete Domes): The reservoirs are in fair to poor condition. The water-tight seals that allow the reservoirs to hold water above the roof-to-wall interface do not allow differential thermal movement between the roof

and walls; this has caused concrete cracking over time. In order to ensure continued long-term, sustained operation, we recommend conducting a detailed evaluation weighing the option of roof replacement or retrofit compared to potentially complete structure replacement. These reservoirs also have valve vaults that are connected to the reservoirs, which during seismic events, could potentially lead to pipe failure. Seismic valves and flexible piping connections are recommended. Although a similar style reservoir as the others, the 40th Street reservoir's reinforcing was found to be inadequate for static and seismic loads. To alleviate structural risk, we recommend reducing the maximum operating level from 23 to 16 feet if possible.

Sehome: At almost 100 years old, this reservoir does not meet current structural or seismic codes due to minimal/missing wall reinforcement. The adjacent sloping ground surface appears to allow organic material-laden surface runoff to drain to the roof. Operationally, it was noted that the roof drain passes through the reservoir interior and vents are undersized. Rehabilitation is not feasible; therefore, replacement of the reservoir is recommended.

Parkhurst: This reservoir is in good condition. The observed exterior efflorescence can likely be addressed by the addition of an interior coating. A new interior ladder is also recommended.

Conclusion

This comprehensive study of the City of Bellingham's water holding structures shows the City's commitment to providing water to its constituents. The City is recognized as a leader in this field. With the average age of the City's reservoirs being around 50 years, it is time to consider options for rehabilitation and/or replacement.

Recommended improvements were developed to address noted deficiencies on the condition assessments. The preference to rehabilitate or replace the Padden, Whatcom Falls I, and 40th Street Reservoirs is reliant on if the City can lower maximum operating levels. Rehabilitation of the Seahome Reservoir does not appear to be feasible. For the other reservoirs, it appears that rehabilitation options will be the most cost effective. Improvements can be weighed with replacement costs and lifespans and prioritized in the next phase of this work program. Special attention should be given to the site selection and improvements as the City moves into the design phase of this project.

Table ES-2 summarizes the details of the reservoirs assessed, results from the condition assessments, and estimated costs to rehabilitate the structures.

One-sheet summaries that outline the reservoir details, condition scoring, and improvement budgetary cost estimates for each reservoir are provided on pages ES-6 to ES-18.

Table ES-2: Summaries of reservoir Information, condition assessments, and recommend improvements: The first four columns show the names, year of construction, capacity in mega gallons, and construction material. The materials are welded steel (Steel), prestressed concrete (PSC), and reinforced concrete (RC). Condition assessment scores are shown in Column 5 and the primary deficiency areas are shown in columns 6 through 10. Estimated costs to fully rectify reservoirs and written explanations of deficiencies are presented in the last two columns.

Reservoir	Year Constructed	Capacity (MG)	Material	Score[1]	Cleanliness & Coatings, Concrete Deterioration/Corrosion	Static	Seismic	Safety	Operations & Maint.	Estimated Cost to Correct Deficiencies[2]	Deficiency Notes
Marietta	1969	3.0	Steel	3.3			X			$410,000	Seismic slosh damages roof and anchor straps
Padden	1967	0.5	Steel	3.6	X		X			$0.85-1.2M	Seismic slosh damages roof and foundation; coatings failure (interior and exterior) and corrosion (interior)
Whatcom Falls I	1984	4.0	Steel	3.8	X		X			$2.1-3.6M	Seismic slosh damages roof; coatings failure and corrosion (interior)
Dakin II	1990	0.5	PSC	4.5	X				X	$740,000	Roof drains clogged and subsequent cracking
Kearney	2006	2.5	PSC	4.6					X	$210,000	Roof draining onto exterior walls
Whatcom Falls II	1993	15.6	PSC	4.2						$840,000	Unable to draw down reservoir and baffles cannot resist seismic load
40th Street	1958	0.5	RC	3.6		X	X			$0.96-1.7M	Walls under-reinforced; roof and walls cannot independently expand and contract; seismic event induced pipe failure (no flex couplings)
College Way	1968	0.5	RC	3.9		X	X			$1.1M	Roof and walls cannot independently expand and contract; seismic event induced pipe failure (no flex couplings)
Consolidation	1959	0.5	RC	3.3		X	X			$1.1M	Roof and walls cannot independently expand and contract; seismic event induced pipe failure (no flex couplings)
Dakin I	1987	0.5	RC	3.7		X	X			$1.1M	Roof and walls cannot independently expand and contract; seismic event induced pipe failure (no flex couplings)
Reveille	1958	0.3	RC	3.7		X	X			$1.3M	Roof and walls cannot independently expand and contract; seismic event induced pipe failure (no flex couplings)
Sehome	1920	0.7	RC	2.6			X		X	$3.4M	Seismic event structure failure (lack of reinforcing); operational deficiencies (slope drains to roof, vents undersized, poor roof drainage)
Parkhurst	1997	0.2	RC	4.1	X			X		$330,000	Efflorescence on exterior, interior ladder corrosion

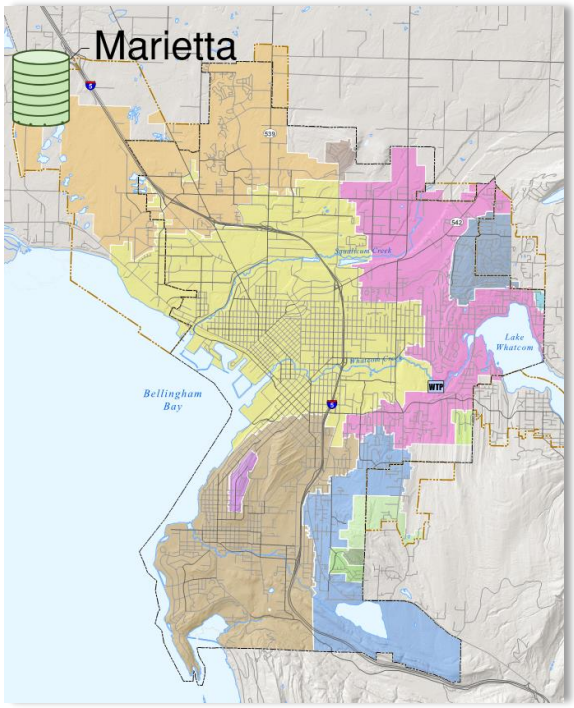
[1] Scoring (1-5, Low to High)

[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

Marietta Reservoir

Date Inspected:
January 24, 2019

Address:
1404 Marietta Avenue
Bellingham, WA 98226



Reservoir Details	
Zone Served	276 North
Storage Volume	3.0 MG
Type	Welded steel
Year Constructed	1969
Dimensions	100’ dia, 55’-3” tall
Inlet	12”
Outlet	16”
Overflow	8” at 276’ (50’ Abv floor)
CP System	Constant voltage

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.4	$ 23,000
Material Deterioration	4.5	$ -
Structural Performance - Static	4.0	$ 86,000
Structural Performance - Seismic	2.2	
Water Quality / Sanitary	4.1	$ 2,000
Safety	2.0	$ 126,000
Operations & Maintenance	3.7	$ 172,000
Obsolescence	4.3	$ -
Overall	**3.3**	**$ 410,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

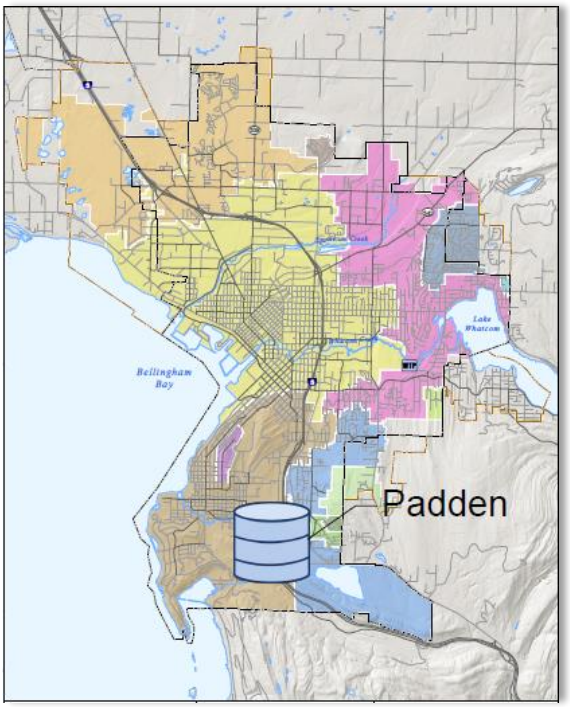
Current max operating level:
50 ft

Updated max operating level, no structural repairs:
44.5 ft

Padden Reservoir

Date Inspected:
April 9, 2019
April 30, 2019

Address:
3820 Broad Street
Bellingham, WA 98229



Reservoir Details	
Zone Served	457 South
Storage Volume	0.5 MG
Type	Welded steel
Year Constructed	1967
Dimensions	59' dia, 22'-3" tall
Inlet	16"
Outlet	Combined w/ inlet/drain
Overflow	8" at 457' (25' Abv floor)
CP System	Constant voltage

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	3.3	$ 399,000
Material Deterioration	3.7	$ 29,000
Structural Performance - Static	5.0	$ 225,000 min –
Structural Performance - Seismic	2.8	$ 572,000 max
Water Quality / Sanitary	4.3	$ 8,000
Safety	2.3	$ 126,000
Operations & Maintenance	4.2	$ 65,000
Obsolescence	3.0	$ -
Overall	**3.6**	**$ 850,000 min – $1,200,000 max**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

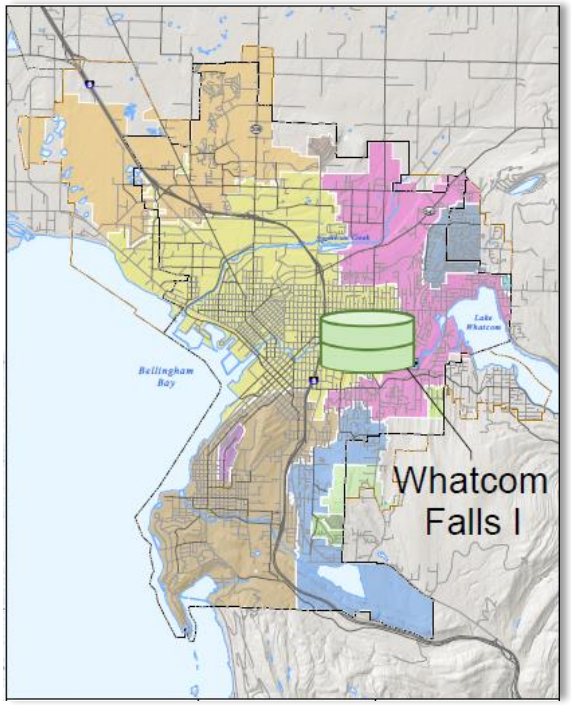
Current max operating level:
24.5 ft

Updated max operating level, no structural repairs:
19.75 ft

Whatcom Falls I Reservoir

Date Inspected:
June 12, 2019
November 30, 2019

Address:
3201 Arbor Street
Bellingham, WA 98229



Reservoir Details	
Zone Served	276 North
Storage Volume	4.0 MG
Type	Welded steel
Year Constructed	1984
Dimensions	201' dia, 17'-6" tall
Inlet	24"
Outlet	Combined w/ inlet/drain
Overflow	N/A
CP System	Autopotential

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	3.5	$ 59,000
Material Deterioration	4.2	$ 34,000
Structural Performance - Static	4.0	$1,328,000 min -
Structural Performance - Seismic	3.5	$2,862,000 max
Water Quality / Sanitary	2.5	$ 443,000
Safety	4.5	$ -
Operations & Maintenance	4.1	$ 195,000
Obsolescence	4.7	$ -
Overall	**3.8**	**$2,100,000 min – $3,600,000 max**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

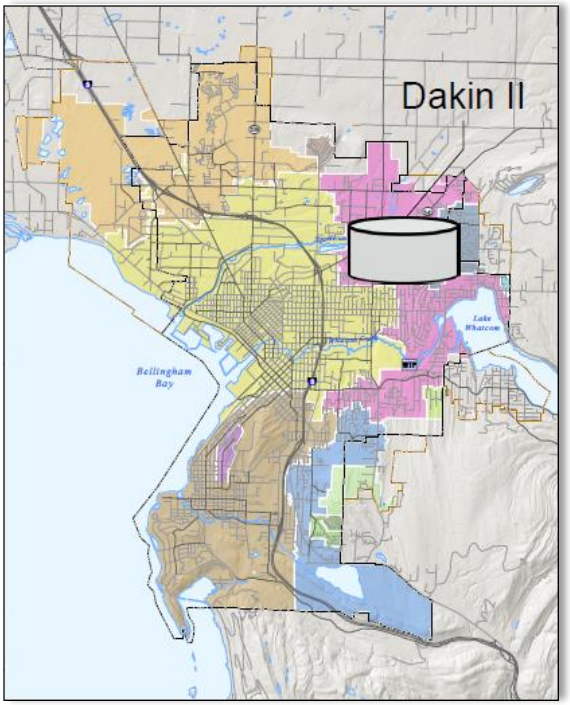
Current max operating level:
16.5 ft

Updated max operating level, no structural repairs:
14 ft

Dakin II Reservoir

Date Inspected:
April 30, 2019

Address:
3819 Balsam Lane
Bellingham, WA 98226



Reservoir Details	
Zone Served	519 Dakin & Yew
Storage Volume	0.5 MG
Type	Prestressed Concrete
Year Constructed	1990
Dimensions	68' dia, 19'-6" tall
Inlet	12"
Outlet	Combined w/ inlet/drain
Overflow	8" at 519' (18'-6" Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	3.8	$ 4,000
Material Deterioration	3.8	$ 221,000
Structural Performance - Static	5.0	$ -
Structural Performance - Seismic	5.0	
Water Quality / Sanitary	3.5	$ 258,000
Safety	3.8	$ 141,000
Operations & Maintenance	4.4	$ 114,000
Obsolescence	5.0	$ -
Overall	**4.5**	**$ 740,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

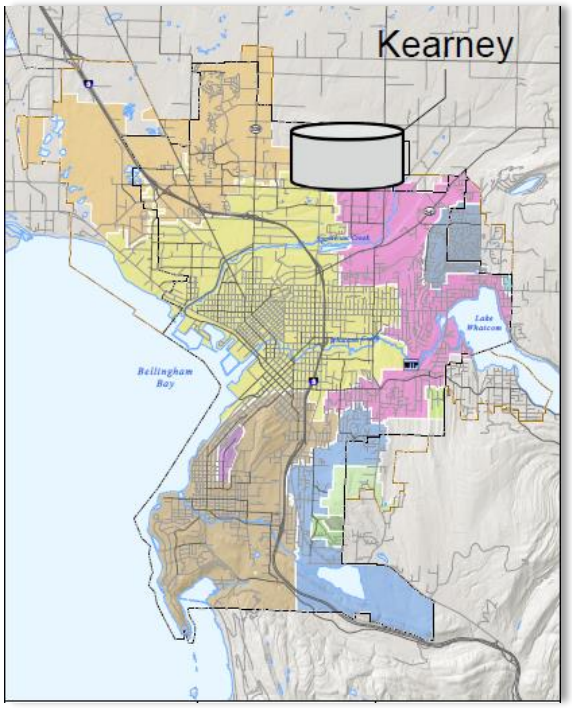
Current max operating level:
15.5 ft

Updated max operating level, no structural repairs:
15.5 ft

Kearney Reservoir

Date Inspected:
March 14, 2019

Address:
465 Kearney Street
Bellingham, WA 98226



Reservoir Details	
Zone Served	276 North
Storage Volume	2.5 MG
Type	Prestressed Concrete
Year Constructed	2006
Dimensions	130' dia, 27' tall
Inlet	20"
Outlet	30"
Overflow	N/A
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.6	$ 6,000
Material Deterioration	4.5	$ 2,000
Structural Performance - Static	4.4	$ -
Structural Performance - Seismic	5.0	
Water Quality / Sanitary	4.4	$ 6,000
Safety	4.5	$ 15,000
Operations & Maintenance	4.3	$ 177,000
Obsolescence	5.0	$ -
Overall	**4.6**	**$ 210,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

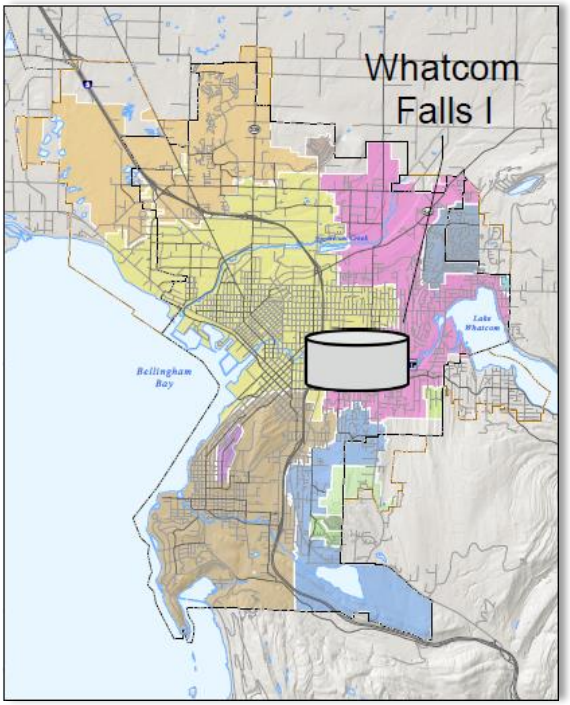
Current max operating level:
22 ft

Updated max operating level, no structural repairs:
22 ft

Whatcom Falls II Reservoir

Date Inspected: June 12, 2019
November 30, 2019

Address:
3201 Arbor Street
Bellingham, WA 98229



Reservoir Details	
Zone Served	276 North
Storage Volume	15.6 MG
Type	Reinforced concrete
Year Constructed	1993
Dimensions	350’ dia, 23’ tall
Inlet	72”
Outlet	60”
Overflow	N/A
CP System	Constant voltage

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	3.6	$ 109,000
Material Deterioration	4.0	$ 31,000
Structural Performance - Static	4.3	$ 286,000
Structural Performance - Seismic	4.3	
Water Quality / Sanitary	4.4	$ 4,000
Safety	4.5	$ 92,000
Operations & Maintenance	3.8	$ 322,000
Obsolescence	4.3	$ -
Overall	**4.2**	**$ 840,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

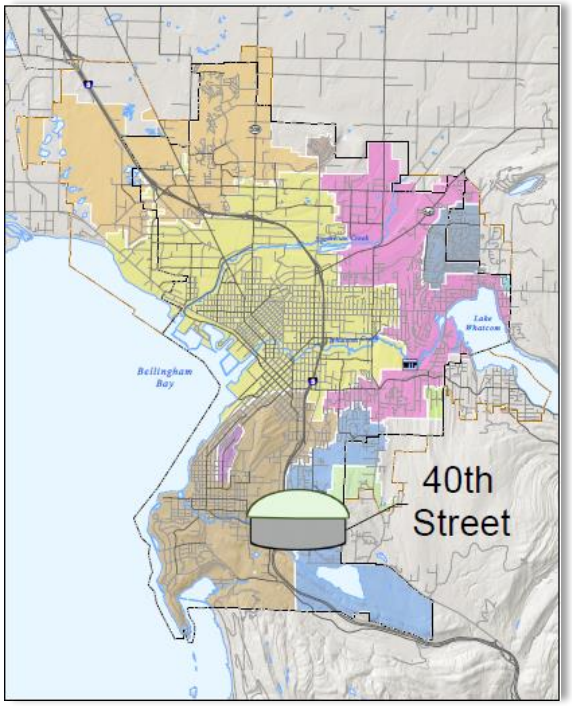
Current max operating level:
21 ft

Updated max operating level, no structural repairs:
21 ft

40th Street Reservoir

Date Inspected:
April 30, 2019

Address:
1399 40th Street
Bellingham, WA 98229



Reservoir Details	
Zone Served	696 Padden Yew
Storage Volume	0.5 MG
Type	Reinforced concrete
Year Constructed	1958
Dimensions	60' dia, 22'-5" tall
Inlet	10"
Outlet	10" combined w/ inlet/drain
Overflow	6" at 696' (26'-8" Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.4	$ 15,000
Material Deterioration	3.9	$ 40,000
Structural Performance - Static	4.0	$ 691,000 min -
Structural Performance - Seismic	3.2	$1,463,000 max
Water Quality / Sanitary	3.9	$ 8,000
Safety	3.0	$ 153,000
Operations & Maintenance	4.1	$ 57,000
Obsolescence	3.0	$ -
Overall	**3.6**	**$ 960,000 min – $1,700,000 max**

Current max operating level:
23 ft

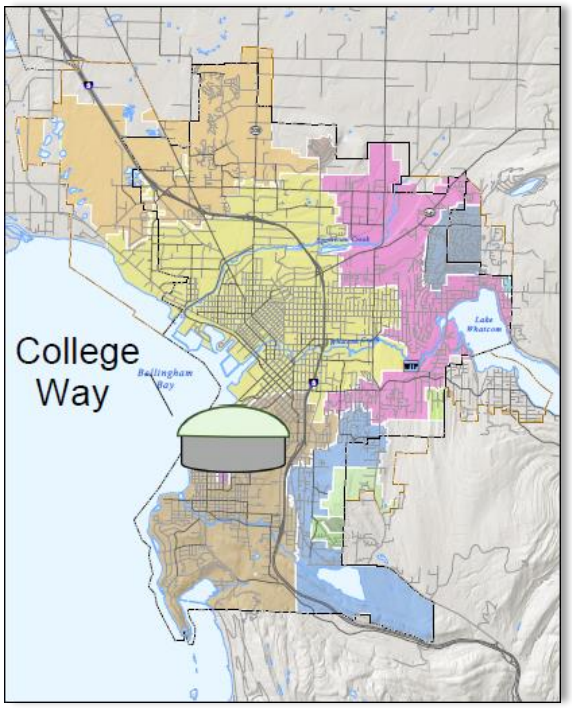
Updated max operating level, no structural repairs:
16 ft

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

College Way Reservoir

Date Inspected:
March 14, 2019

Address:
231 Highland Drive
Bellingham, WA 98225



Reservoir Details	
Zone Served	457 South
Storage Volume	0.5 MG
Type	Reinforced concrete
Year Constructed	1968
Dimensions	64' dia, 19'-6" tall
Inlet	10"
Outlet	16" to College Way Pump Station
Overflow	6" at 457' (23'-6" Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	3.8	$ 15,000
Material Deterioration	3.7	$ 6,000
Structural Performance - Static Structural Performance - Seismic	4.1 3.7	$ 692,000
Water Quality / Sanitary	4.5	$ 183,000
Safety	3.3	$ 156,000
Operations & Maintenance	4.1	$ 71,000
Obsolescence	5.0	$ -
Overall	**3.9**	**$1,100,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

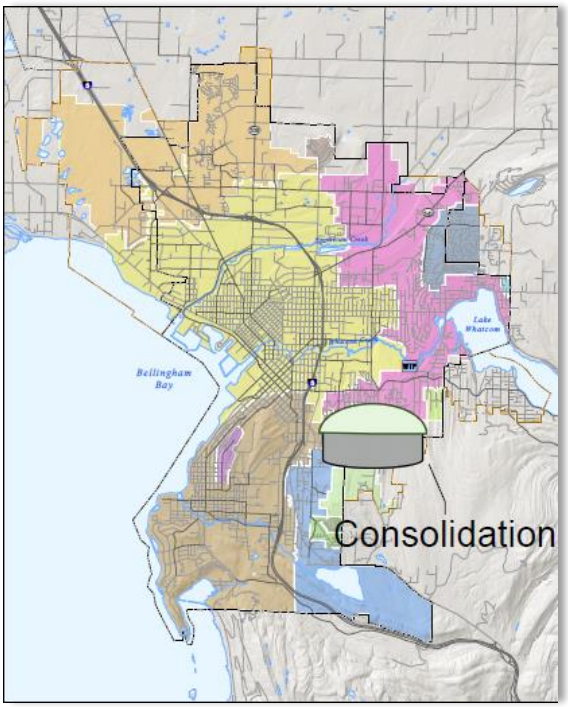
Current max operating level:
19 ft

Updated max operating level, no structural repairs:
19 ft

Consolidation Reservoir

Date Inspected:
November 7, 2019

Address:
2500 Yew Street Road
Bellingham, WA 98229



Reservoir Details	
Zone Served	519 Dakin & Yew
Storage Volume	0.5 MG
Type	Reinforced concrete
Year Constructed	1959
Dimensions	64' dia, 18'-9" tall
Inlet	10"
Outlet	10" combined w/ inlet/drain
Overflow	6" at 519' (23' Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	2.9	$ 10,000
Material Deterioration	2.6	$ 296,000
Structural Performance - Static	4.1	$ 647,000
Structural Performance - Seismic	3.0	
Water Quality / Sanitary	3.6	$ 8,000
Safety	4.0	$ 71,000
Operations & Maintenance	3.6	$ 86,000
Obsolescence	1.0	$ -
Overall	**3.3**	**$1,100,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

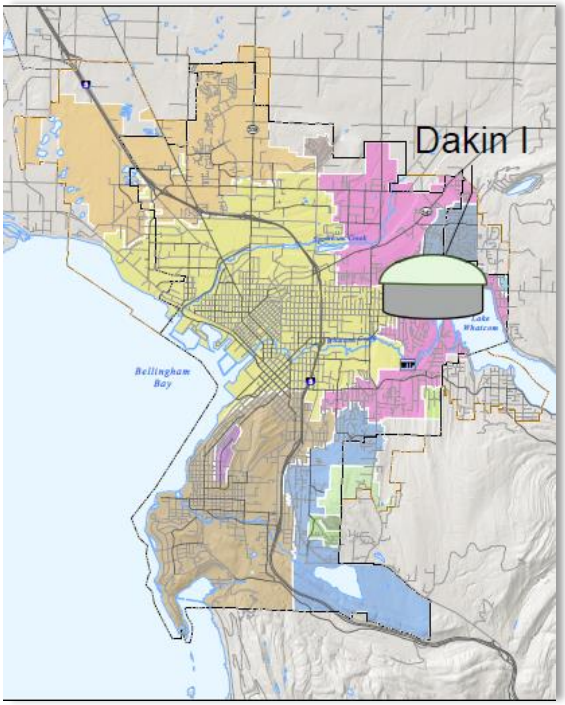
Current max operating level:
19.5 ft

Updated max operating level, no structural repairs:
19.5 ft

Dakin I Reservoir

Date Inspected:
April 8, 2019

Address:
3819 Balsam Lane
Bellingham, WA 98226



Reservoir Details	
Zone Served	519 Dakin & Yew
Storage Volume	0.5 MG
Type	Reinforced concrete
Year Constructed	1987
Dimensions	66'-8" dia, 17'-8" tall
Inlet	10"
Outlet	10" combined w/ inlet/drain
Overflow	6" at 519' (21'-11" Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.1	$ 17,000
Material Deterioration	4.2	$ 2,000
Structural Performance - Static Structural Performance - Seismic	4.1 3.3	$ 692,000
Water Quality / Sanitary	4.1	$ 221,000
Safety	4.0	$ 118,000
Operations & Maintenance	4.2	$ 25,000
Obsolescence	1.0	$ -
Overall	**3.7**	**$1,100,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

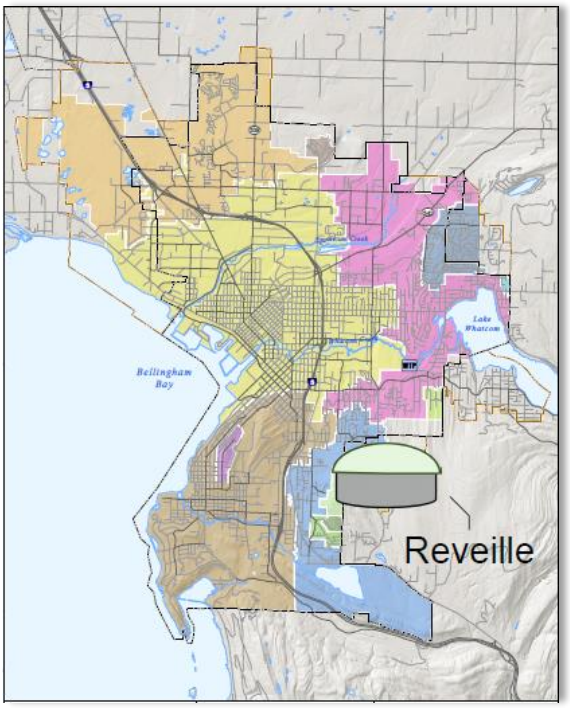
Current max operating level:
17 ft

Updated max operating level, no structural repairs:
17 ft

Reveille Reservoir

Date Inspected:
May 21, 2019

Address:
2400 Yew Street Road
Bellingham, WA 98225



Reservoir Details	
Zone Served	696 Padden Yew
Storage Volume	0.3 MG
Type	Reinforced concrete
Year Constructed	1958
Dimensions	50' dia, 19'-3" tall
Inlet	10"
Outlet	Combined w/ inlet/drain
Overflow	6" at 696' (20' Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.4	$ 4,000
Material Deterioration	3.9	$ 198,000
Structural Performance - Static	4.1	$ 610,000
Structural Performance - Seismic	3.3	
Water Quality / Sanitary	3.8	$ 259,000
Safety	2.3	$ 122,000
Operations & Maintenance	3.8	$ 65,000
Obsolescence	5.0	$ -
Overall	**3.7**	**$1,300,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

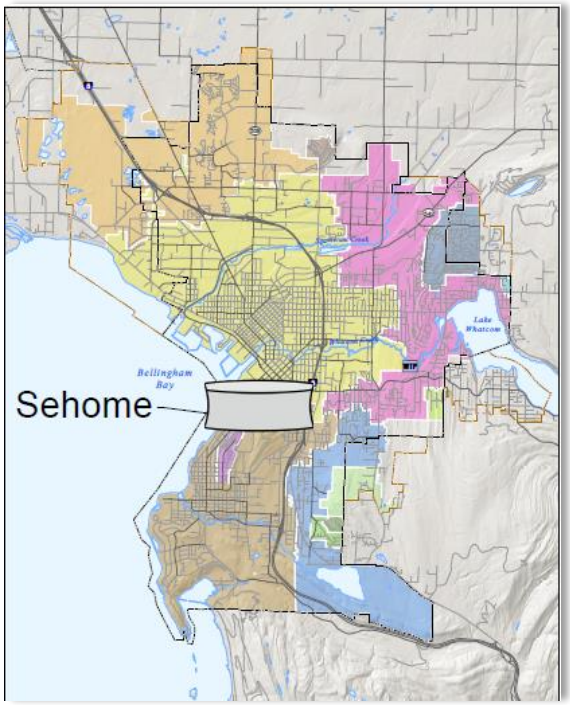
Current max operating level:
19 ft

Updated max operating level, no structural repairs:
19 ft

Sehome Reservoir

Date Inspected:
January 24, 2019

Address:
600 25th Street
Bellingham, WA 98225



Reservoir Details	
Zone Served	457 South
Storage Volume	0.7 MG
Type	Reinforced concrete
Year Constructed	1920
Dimensions	100x73' rectangle, 16' tall
Inlet	12"
Outlet	12" combined w/ inlet
Overflow	6" at 457' (14' Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	2.9	$ -
Material Deterioration	3.4	$ -
Structural Performance - Static	3.3	$ -
Structural Performance - Seismic	1.7	
Water Quality / Sanitary	1.6	$ -
Safety	4.5	$ -
Operations & Maintenance	2.9	$ -
Obsolescence	1.0	$ -
Overall	**2.6**	$3,400,000†

Current max operating level:
13 ft

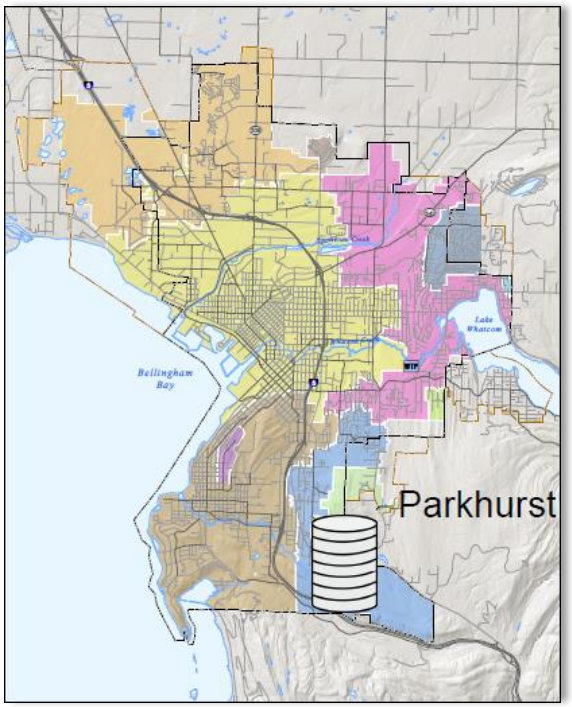
Updated max operating level, no structural repairs:
13 ft

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded
†New reservoir recommended

Parkhurst Standpipe

Date Inspected:
May 21, 2019

Address:
4329 Samish Crest Drive
Bellingham, WA 98229



Reservoir Details	
Zone Served	873 Governor Road
Storage Volume	0.185 MG
Type	Reinforced concrete
Year Constructed	1997
Dimensions	30' dia, 35' tall
Inlet	6"
Outlet	10" w/ anti-vortex device
Overflow	6" at 873' (34'-6" Abv floor)
CP System	N/A

Category	Score[1]	Planning Level Improvement Costs[2]
Cleanliness and Coatings	4.4	$ 120,000
Material Deterioration	3.8	$ 11,000
Structural Performance - Static	4.8	$ -
Structural Performance - Seismic	4.0	
Water Quality / Sanitary	4.4	$ 4,000
Safety	2.5	$ 149,000
Operations & Maintenance	4.0	$ 46,000
Obsolescence	5.0	$ -
Overall	**4.1**	**$ 330,000**

[1] Scoring (1-5, Low to High)
[2] Includes 30% contingency, 8.7% tax, & 35% engineering, rounded

Current max operating level:
24 ft

Updated max operating level, no structural repairs:
24 ft

Section 1

Introduction

1.1 Authorization and Objective

In December 2018, Murraysmith was authorized by The City of Bellingham (City) to conduct a thorough condition assessment of 13 storage reservoirs. The objectives of this condition assessment are to:

- perform site visits to each reservoir,
- establish a detailed condition scoring system to assess the conditions of each reservoir, determining deficiencies
- Outline recommended improvements to resolve deficiencies

1.2 Background

The City's water system is comprised of an 11-million gallons per day (MGD) water treatment plant (WTP), 13 water holding structures, and nine pump stations. Water is sourced from the Middle Fork of the Nooksack River and Lake Whatcom. Total storage capacity is 28.43 million gallons (MG) (CH2MHill 2009). A map of the City's system is provided in Figure 1-1 in the end of this section.

The City wishes to assess the condition of reservoirs, determine necessary repairs/replacements, develop bid specification documents and oversee upgrades. This project has following phases:

- Phase 1: Condition Assessments and Structural Evaluations
- Phase 2: Development of Bid Documents
- Phase 3: Construction Support Services

This report includes the findings and conclusions from Phase 1. A description of the 13 reservoirs that were assessed is provided in Table 1-1.

1.3 Project Team and approach

To evaluate the condition of each reservoir, Murraysmith gathered a team of technical experts to work with City staff. The consultant team included:

- Murraysmith - General interior and exterior reservoir condition; valving, piping, and appurtenance evaluation; and project management
- Geoengineers - Geotechnical investigations, conducted on separate site visits

- Northwest Corrosion Engineering (NWC) - Coating systems, corrosion evaluations, cathodic protection systems
- Peterson Structural Engineers (PSE) - Structural and seismic evaluations and assessments

Table 1-1: City of Bellingham reservoirs Inspected and assessed by Murraysmith

Reservoir Name	Year Built	Capacity (MG)	Construction	Configuration	Dimensions
Marietta	1969	3.0	Welded Steel	Circular; Above-grade	Diameter: 100 ft Spill Height: 50 ft
Padden	1967	0.5	Welded Steel	Circular; Above-grade	Diameter: 59 ft Spill Height: 25 ft
Whatcom Falls I	1984	4.0	Welded Steel	Circular; Above-grade	Diameter: 201 ft Spill Height: N/A
Dakin II	1990	0.5	Prestressed Concrete	Circular; Partially buried	Diameter: 68 ft Spill Height: 18.5 ft
Kearney	2006	2.5	Prestressed Concrete	Circular; Partially buried	Diameter: 130 ft Spill Height: N/A ft
Whatcom Falls II	1993	15.6	Prestressed Concrete	Circular; Partially buried	Diameter: 350 ft Spill Height: N/A
40th Street	1958	0.5	Reinforced Concrete	Circular; Partially buried	Diameter: 60 ft Spill Height: 26.7 ft
College Way	1968	0.5	Reinforced Concrete	Circular; Mostly Buried	Diameter: 64 ft Spill Height: 23.5 ft
Consolidation	1959	0.5	Reinforced Concrete	Circular; Mostly buried	Diameter: 64 ft Spill Height: 23 ft
Dakin I	1987	0.5	Reinforced Concrete	Circular; Mostly buried	Diameter: 66.7 ft Spill Height: 21.9 ft
Reveille	1958	0.3	Reinforced Concrete	Circular; Mostly buried	Diameter: 50 ft Spill Height: 20 ft
Sehome	1920	0.7	Reinforced Concrete	Football-shaped; Partially buried	L*W: 107*73 ft Spill Height: 14 ft
Parkhurst	1997	0.185	Reinforced Concrete	Circular; Partially buried	Diameter: 30 ft Spill Height: 34.5 ft

The project team conducted field inspections of each reservoir, taking detailed notes and photographs to document the condition of each reservoir. Inspections included drained inspection of all reservoirs and floating inspections of the Padden and the Whatcom Falls I and II reservoirs. During these inspections all aspects of the reservoirs were examined in detail with each member of the team focusing on their area of experience. Whatcom Falls could not be drained so will be assessed by LiquiVision through an underwater inspection.

To objectively assess the condition of each reservoir, Murraysmith created a Condition Assessment Score Matrix (Score Matrix). The Score Matrix provides a numerical system and associated criteria to rate the condition of each tank component which can be used in future criticality modeling, condition comparisons over time and other analyses. Score definitions were developed with input from NWC and PSE. The score sheet and definitions were presented to the City for refinement.

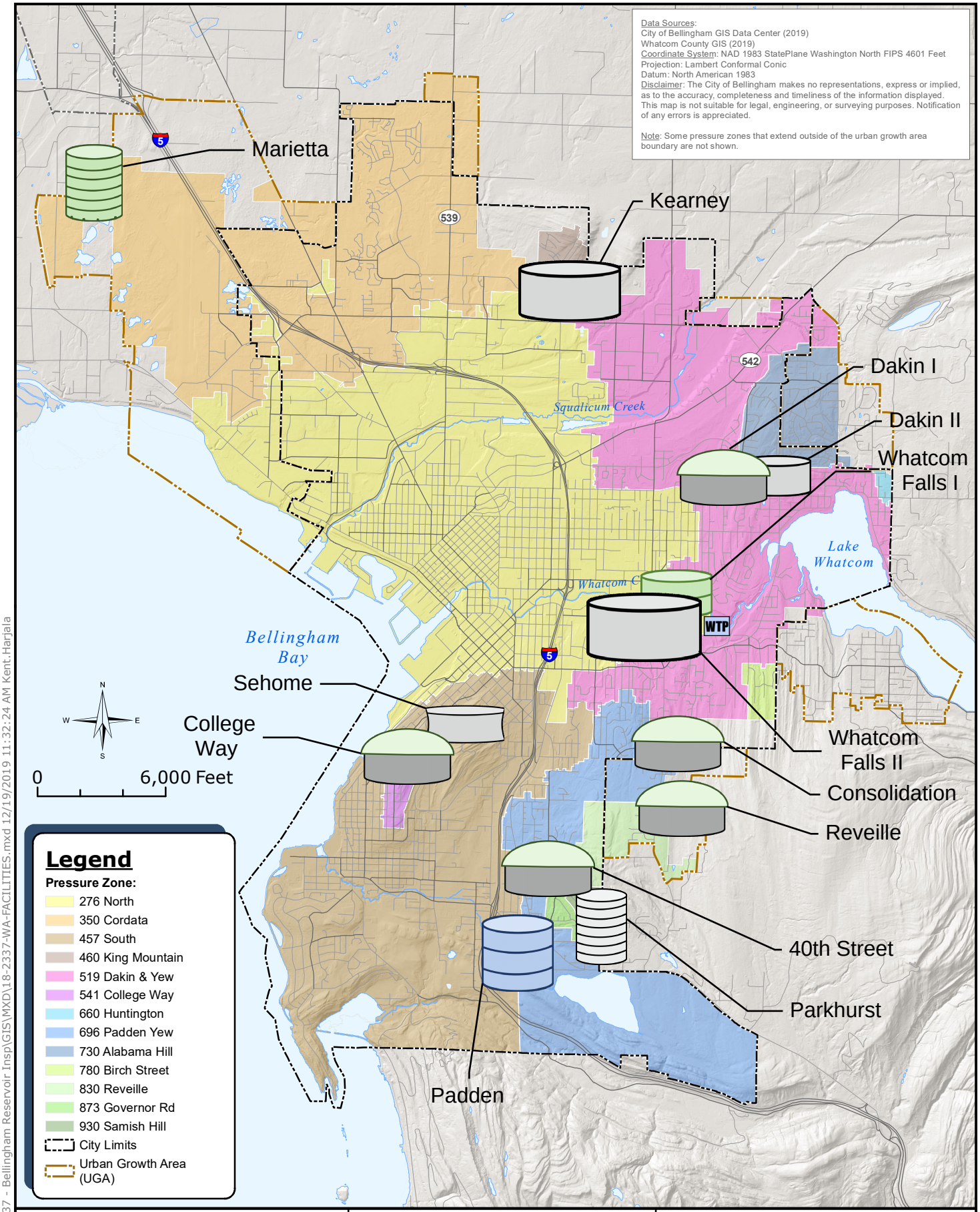
The finalized score sheet was then used to score and establish a baseline condition of each reservoir.

1.4 Technical Report Format

This report is organized into 16 sections. The Introduction, Section 1, provides background information for the report and a general introduction to the project team, project approach, and report format. Section 2 outlines the procedure for inspection and assessment of the reservoirs. Sections 3 to 15 are reports for each tank, containing general overviews of the reservoirs, observations from field observations, condition assessments, and lists of recommended improvements with planning level cost estimates. Specifically, Sections 3 to 5 are the steel tanks, Sections 6 to 8 are the prestressed concrete tanks (PSC), Sections 9 to 13 are the Bellingham typical reinforced domed reinforced concrete tanks (Domed RC), and Sections 14 and 15 are the other reinforced concrete tanks. Section 16 is the conclusion.

- Section 1 – Introduction
- Section 2 – Inspection and Assessment Methods
- Section 3 – (Steel) Marietta
- Section 4 – (Steel) Padden
- Section 5 – (Steel) Whatcom Falls I (Forthcoming)
- Section 6 – (PSC) Dakin II
- Section 7 – (PSC) Kearney
- Section 8 – (PSC) Whatcom Falls II (Forthcoming)
- Section 9 – (Domed RC) 40th Street
- Section 10 – (Domed RC) College Way
- Section 11 – (Domed RC) Consolidation (Forthcoming)
- Section 12 – (Domed RC) Dakin I
- Section 13 – (Domed RC) Reveille
- Section 14 – (RC) Sehome
- Section 15 – (RC) Parkhurst
- Section 16 – Conclusion
- Works Cited

Each reservoir has its own appendix, which includes the geotechnical report, coatings/corrosion report and cathodic protection checkout report (if applicable), structural report with inspection notes, general inspection notes, and the full Score Matrix. The general inspection notes do not include photos, as these photos are included in the body of this report and subconsultant reports.



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murraysmith

CITY OF BELLINGHAM WASHINGTON

City of Bellingham Reservoir Inspections

Figure 1-1 Pressure Zones and Reservoirs

December 2019

18-2337.408

Section 2

Inspection and Assessment Methods

2.1 Introduction

A standard methodology was developed to inspect and assess each reservoir. The team analyzed provided drawings and materials and then conducted field investigations. Assessments of observations and additional coating, structural, and vent assessments were performed. The Score Matrix was developed to uniformly assess each reservoir and make comparisons. Based on the assessments, codes, and best practices, recommended improvements with associated costs were developed for each reservoir.

2.2 Field Inspection Methods

2.2.1 Geotechnical Investigations

After preliminary site visits were conducted to determine boring locations at each of the reservoir sites, a geotechnical investigation was conducted. Subsurface soil and groundwater explorations were performed at each site by drilling one boring to refusal on bedrock or 100 feet, whichever was shallower. The material encountered was systematically logged and samples were taken for laboratory testing to obtain physical and engineering characteristics.

Based on the geotechnical investigations, the following were attributed to each site:

1. Foundation Conditions - the general type of soils the structure is founded on

2. General Seismicity – All reservoirs sites are located within the Puget Sound area, which is known to be seismically active.

3. Surface Fault Rupture – an offset in the ground resulting from a fault rupture that extends to the earth's surface. This is based on United States Geological Survey (USGS) fault line mapping.

4. Liquefaction Potential – a phenomenon where soils experience a rapid loss of internal strength because of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. The estimated liquefaction induced subsidence is determined using the soil type and groundwater level.

5. **Seismic design criteria** – information about the maximum considered earthquake (MCE) that is used in the design and evaluation of structures. The MCE is a seismic event that has a 2 percent probability of being exceeded in a 50-year timeframe. Another way to think about this event is that, on average, an event of at least this magnitude occurs once every 2,500 years. This estimate is based on seismic data, mapped faults, and geologists' projections. The parameters are based on American Water Works Association (AWWA) and American Society of Civil Engineers (ASCE) 7-10 design criteria. The seismic parameters for each reservoir are used in the structural evaluations.

6. **Allowable bearing pressure** – a measurement of the amount of force per area, in pounds per square foot (psf), that the soil is expected to be able to support. This should be greater than the pressure exerted by the reservoir structure's foundation and water held within it. A value is given for the long term but can be increased by up to one-third for wind or seismic loads.

7. **Lateral Resistance and Design Parameters** – (concrete tanks) values used to determine the effect of soils on the buried foundation and/or walls of the reservoir. These values are not discussed in this report but are included in the appendices.

8. **Global Stability** – risk of slope failure or landslides in the vicinity of the reservoir.

2.2.2 Reservoir Inspections

Except for Whatcom Falls II, each reservoir was inspected in a drained condition. Steel reservoirs were also inspected in a partially full condition to investigate roofs and their supports. The visual conditions of structures, components, appurtenances, valving, and piping were noted in the field. Where relevant, the material thicknesses, coating thicknesses, and cracking or corrosion extents were carefully measured. Photographs were taken and conversations were also conducted with City staff to aid in identifying operational and water quality issues. The inspection summaries are broken into exterior, interior, and piping and valving sections.

The exterior inspection summaries include the site and security, exterior walls, foundation, roof, and appurtenances. The inspection team walked around the site, examining nearby vegetation and site drainage conditions. The walls, foundation, and associated appurtenances were inspected from the ground. The roof and associated appurtenances were examined from the roofs. Exterior appurtenances include side hatches, ladders, railings, roof hatches, vents, and vaults. Reservoir vents and access hatches were visually inspected to identify potential water quality concerns such as sizes and condition of screens and gaskets, condition and location of locks and alarms, and any evidence of ponding which can cause contamination or other reservoir deficiencies.

The interior inspection summaries include the walls, floor, roof, and appurtenances. The team accessed the interior, inspected the condition, and took measurements. The piping and valving inspection summaries includes the inlet, outlet, overflow, and drain piping. The valving was also

inspected, as was any washdown piping on site. The piping and valving were assessed from the interior, within the valve vaults, and from the exteriors where relevant/possible.

To inspect the coating systems on structures and appurtenances, a visual inspection was first conducted. Then measurements were taken of the diameter/size of blister or areas of missing coating. Measurements of dry film thickness (DFT), which indicates the amount of coating, were taken at multiple locations and carefully noted using a portable electromagnetic dry film thickness gauge (Type 2 gauge). This gauge measures the thickness of the coating between the gauge probe and the metallic substrate. A model 200 transducer was used to measure polyurea coatings on concrete structures.

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

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

Corrosion was inspected by observing and measuring the corrosion extent. Pitting depth (surface depressions that form in corroded metal and indicate the extent of corrosion damage) was measured with a surface gauge. The conditions of steel plates, rafters, girders, joists, columns, and appurtenances were also noted in the field. For concrete reservoirs, extent and size of cracking and general condition of concrete were noted.

2.3 Condition Assessment Score Matrix Development

The Condition Assessment Score Matrix (Score Matrix) is a tool that uniformly assesses the condition of each reservoir in the City's system. The Score Matrix provides a numerical system to rate the condition of multiple components of each reservoir. These ratings can be used in future criticality modeling, condition comparisons over time, and other analyses. The complete Score Matrices for all the City's reservoirs are included in each reservoir's appendix.

2.3.1 System/Structure Groups

Individual components of the reservoir were identified for scoring. In order to provide a comprehensive, detailed evaluation of all reservoirs, 28 reservoir system/structures were used. Not all components are applicable for each reservoir. In order to provide a higher-level understanding of the reservoir condition, individual components were grouped into System/Structure Groups. These Groups allow for efficient assessment of the condition of areas or sections of the reservoirs. The seven Groups are:

- **Site/Security** – includes reservoir components that keep the site secure such as fencing, intrusion alarms, and locks; site drainage and vegetation separation are also assessed.

- **Walls** – includes reservoir wall surface condition and structural components such as joints, rivets or welds.

- **Floor/Foundation** – includes reservoir foundation, interior floor, and anchoring systems for steel reservoirs or seismic cables for prestressed reservoirs.

- **Roof** – includes roof structure and support system, specifically roof slope, plates/concrete slabs, plate/section joints, rafters, joists, beams, columns, and knuckles.

- **Appurtenances** – includes non-pipe related appurtenances such as ladders, hatches, railings, vents, and landings/grating.

- **Piping/Valving** – includes all pipes and related appurtenances on the site, including the inlet, outlet, drain, and overflow, and washdown piping, valving and valve vaults. Checks for the presence of flexible couplings or seismic valves are conducted in this section.

- **Misc. –** includes cathodic protection system, level sensors, and hydraulic mixing system.

2.3.2 Assessment Categories

Each component of the reservoir is assessed and scored on the following seven categories, with associated subcategories in parentheses:

- Cleanliness and Coatings
- Material Deterioration (Concrete Deterioration and Metal Corrosion)
- Structural Performance (Static and Seismic)
- Water Quality/Sanitary
- Safety
- Operations and Maintenance (Site and Security, Roof Drainage, Appurtenances, Valving and Piping, Misc.)
- Obsolescence

Scores range from one (1) to five (5) and are assigned based on how well each reservoir component meets the stated criteria for that scoring category. A score of 5 implies that the component meets the criteria completely; 1 implies that the component is deficient. Not all scoring categories or subcategories are relevant for each reservoir component. If this is the case, a zero (0) is assigned to the Score Matrix and does not factor into the numerical scoring. The written explanation of numerical scores for each category is shown in Table 2-1.

Table 2-1: Condition Assessment Scoring Definitions

Criteria	5	4	3	2	1
Cleanliness and Coatings	Good condition (<1% exposed); Recently Cleaned / No growth or debris	5% loss of any coating layer or visual blistering. Spot repair defects. OR: Has organic growth or debris and will need cleaning soon	10% loss of any coating layer. Repair coating damage within 5 years. Or: Needs cleaning	20% loss of any coating layer, most likely full recoat within 5 years. May also need cleaning	> 20% coating loss, Poor Condition / Full Recoat within 3 years. May also need cleaning
Concrete Deterioration/ Corrosion	Good condition. Steel has isolated rust staining, no pitting (<1% exposed). Concrete has no visible signs of wear and cracks are limited to crazing.	Steel has rust staining and general surface corrosion between 1 and 5% of surface area; pitting depth < 5% of nominal wall thickness. Concrete has cracking is limited to a few locations and jointing is still tight. Large scale cracking is limited to crazing. Point instances of efflorescence observed.	Steel has widespread rust staining and general surface corrosion between 5 and 20% of surface area; pitting depth between 5 and 10% of nominal wall thickness. Concrete has pattern cracking at joints or along boundary indictive of operational loads in exceedance of capacity or constrained thermal movement. Efflorescence is occurring along cracks.	Steel has heavy rust staining and widespread general surface corrosion over 20% of surface area; pitting between 10 and 20% of nominal wall thickness. Concrete has significant cracking in which separation of the joint has occurred (< 1/16"), efflorescence build-up is occurring along joints and cracks, and/or delamination of concrete (incident size < 0.5ft^2) is occurring due to underlying reinforcing expansion	Steel has rust staining and general surface corrosion over 20% of surface area; pitting > 20% of nominal thickness. Concrete has major widespread cracking (> 1/16") with significant efflorescence build-up along cracks and joints, delamination of concrete surfaces (incident size >0.5 ft^2), subsidence cracks, slab and /or curling
Structural Performance	Major structural components meet the current design codes and standards, minor deficiencies noted that can be repaired during standard maintenance. No operational changes are needed to improve performance.	Design meets code but moderate deficiencies are noted repairs are more complex than can be handled by standard maintenance. Non-code compliant issues might be present but have already been corrected through operational changes (e.g. the reservoir is overstressed at the max operating level but based on the actual operating level the problems noted have been alleviated).	Design does not meet current code but no structural failure appears imminent and no structural systems require repairs (due to factors like age, damage, or corrosion). Deficiencies noted can be corrected thru minimal operational changes (e.g. an acceptable lowering of the operating level). Generally the tank is in good condition but of an older design.	Structural issues noted and 1) a failure of a structural system is anticipated or 2) any deficient members that were identified will require extensive retrofit when repair is initiated and operational changes cannot easily be implemented. Issues noted here are not likely to require the reservoir to stop operation while waiting for repair.	Failure imminent due to issues like corrosion (with significant section loss) or the anticipated design loads greatly exceed allowable structural capacity based on analysis. Operational changes cannot be implemented and retrofit is necessary. Issues noted here could require taking the reservoir offline until repair is conducted.
Water Quality/ Sanitary	Meets DOH best practices.	Meets DOH best practices, but minor maintenance or modifications recommended.	Doesn't meet DOH best practices, minor maintenance or modifications required.	Doesn't meet current DOH requirements, potential health risk.	Imminent health risks.
Safety	Meets current OSHA / WISHA requirements	Meets current OSHA / WISHA requirements but minor maintenance or modifications recommended	Doesn't meet current OSHA/WISHA requirements, but in compliance	Doesn't meet current OSHA/WISHA requirements; upgrades recommended	Doesn't Meet Current OSHA / WISHA requirements, not in compliance
Operations and Maintenance	System/Structure functions as designed. No maintenance required.	System/Structure exhibits some wear, but is fully functional. No maintenance required.	System/Structure is operational, but does not function optimally. Maintenance/ replacement recommended in the near future.	System/Structure minimally functions. Maintenance/replacement required in the near future.	System/Structure is not functioning and needs immediate maintenance/replacement.
Obsolescence	Still manufactured; parts available	(Not used)	No longer manufactured; parts available	(Not used)	No longer manufactured; no parts available

2.4 Reservoir Condition Assessment Methods

To arrive at a condition score for each component, operating information provided by the City, design and construction drawings, field measurements, observed conditions, and additional assessments were compiled. The score with a definition that best matched the component's condition was assigned. Deficiencies are compiled in this section.

2.4.1 Cleanliness and Coatings

To assess cleanliness, the timeline for cleaning is approximated based on the level of organic growth noted. The need for cleaning portion of the score can only cause the score to drop to 3 and no lower.

Coatings provide corrosion and/or leak prevention system to reservoirs or appurtenances. Coatings on steel tanks are assessed according to AWWA D102-17 "Coating Steel Water-Storage Tanks." To assess the coatings, the measured thickness was compared to specified thicknesses or common thicknesses of other high-performance coatings on similar structures. The estimated remaining life of the coating systems are provided.

2.4.2 Material Deterioration

On concrete tanks, the overall patterns of concrete cracking are amalgamated and a likely explanation of why concrete is cracking is presented. The thickness of concrete and associated deterioration was compared against design drawings.

To assess corrosion on the steel tanks, the results of the inspection were compiled. The percent of the material that exhibited corrosion was estimated, using ASTM D610 Standard Practice for Evaluating Degree of Rusting on Painted Steel Surfaces (2019). Any material loss from corrosion is found in variations in steel thicknesses in the course or deviations from the as-built drawings. The observed corrosion's impact on the structure is estimated. A timeline for when the corrosion needs to be addressed is estimated.

2.4.3 Structural Performance

The Structural Performance scoring category includes two subcategories. The Static Score is the structural condition under day-to-day loads (Section 2.4.3.1). The Seismic Score is structural condition during earthquake loads (Section 2.4.3.2). These analyses differ between the various reservoir types (welded steel, prestressed concrete, and reinforced concrete).

Collected measurements were used in the development of structural calculations and to evaluate conformance to applicable codes, standards and best practices. While on site, discrepancies between the observed structures and the construction documents were identified. As-built (or

plan) details were incorporated into the analysis. Relevant seismic and soils data were taken from the geotechnical reports.

For the structural evaluation, a code analysis was conducted for each structure using the current 2015 International Building Code (Code) as amended by the State of Washington and ASCE 7-10 'Minimum Design Loads and Associated Criteria for Buildings and Other Structures'. The AWWA standards D110-13 'Wire and Strand-Wound, Circular, Prestressed Concrete Water Tanks' and AWWA D100-11 'Welded Carbon Steel Tanks for Water Storage', along with other associated reference standards were used to further improve the accuracy of the evaluation.

For the reinforced concrete tanks, the American Concrete Institute (ACI) Standards ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures and Commentary" and ACI 318-14 "Building Code Requirements for Structural Concrete" were used. Also, The Portland Cement Association (PCA) references "Design of Liquid-Containing Structures for Earthquake Forces", published 2002, "Circular Concrete Tanks without Prestressing", published 1993, and "Rectangular Concrete Tanks", Revised 5th Edition, published 1998, were also utilized. Because AWWA does not have a current standard for non-prestressed reinforced tanks, modern standards have been adopted from AWWA D110-13.

To assess the structural performance, material thickness, reinforcement and supports (including pre-stressing bars and seismic cables for pre-stressed reservoirs), and foundations are assessed related to the governing codes. Please note that the overflow height is recommended to be used as the maximum operating level Per AWWA D100-11 and D110-13 as maximum reported operating heights can change throughout time. If the reservoir assessed at the overflow level does not pass requirements, the City's reported maximum operating level is also assessed for reference.

2.4.3.1 Static Analysis

The static analysis consists of evaluating non-seismic related forces on the reservoir structure and foundation. For all tanks, total weight of the reservoir, roof loads, and water held within is determined and applied to the foundation footprint. This pressure is compared to the allowable bearing pressure that was determined in the geotechnical reports.

2.4.3.1.1 Steel Tanks

For steel tanks in the system, the roof framing is assessed to determine if it can withstand self-weight, roof live load (from people or equipment during construction or renovation), and snow load. Structures include the columns, girders (girders support the rafter beams on the foundation), rafters, and roof plates. Based on the operating or overflow levels, the reservoir shell is assessed to determine if the steel is thick enough to withstand the forces of the water held within it.

2.4.3.1.2 Prestressed Concrete

First, the roof slab is assessed for thickness over the span length and reinforcement. If a dome, the thickness and reinforcing are assessed. During temperature fluctuations and solar gain, the roof expands and contracts radially which results in stress on the roof and/or walls. In modern prestressed reservoirs, a feature called a shear can allows this differential movement to occur at the roof-to-wall interface without damaging the reservoir. These features are checked to ensure they meet design requirements.

Based on provided plans and field measurements, the size and spacing of pre-stressing wire and strand wrap are assessed to determine if they are adequate to support the structure, roof with loads, and water held within it. Columns are assessed to determine if they can support forces induced from the roof. A roof with a curb must be able to support the live load and any water or materials that could collect on it.

2.4.3.1.3 Reinforced Concrete Dome Roof

The domed roofs are assessed on modern requirements for prestressed concrete dome roofs, outlined in D110-13. For reinforced concrete reservoirs with clear span concrete domes, the roof is assessed to determine if it meets design criteria for rise-to-span, thickness versus buckling, and edge reinforcing. The roof-to-wall interface is assessed to see if it can account for any hydrostatic forces acting on it and if it can handle differential thermal movement between the roof and wall. This can be accomplished with bearing pads or brackets. Wall reinforcement is also assessed to find if it can handle the loads from water held within the reservoir.

2.4.3.1.4 Other Concrete Structures

The Sehome Reservoir and Parkhurst Standpipe are assessed in a similar method to Section 2.4.3.1.2. Without any known reinforcing on Sehome, the walls are assessed assuming no reinforcement. The columns at Sehome are assessed in a manner similar to Sections 2.4.3.1.2.

2.4.3.2 Seismic Analysis

A seismic load analysis was also conducted. The Code currently requires structures be able to withstand the seismic event that has a 2 percent chance of occurring in 50 years at the site. For each reservoir, based on the maximum operating level (and overflow height), diameter/shape, and seismic design parameters, a different slosh wave will be induced. The slosh and its effects potential on reservoir roofs and foundations are much better understood today than when many of the tanks were designed and built.

The freeboard height is the distance between the overflow and base of the rafters or bottom of the roof (where rafters are not present). If adequate freeboard is not present, the slosh wave will be constrained by roof, which would stress the roof and support system. Significant damage may occur. The analysis determines if the current freeboard is suitable, or if the slosh wave will cause

the roof to exceed the load capacity, overstressing it. As stated above, if the reservoir fails requirements at the overflow level, the reported maximum operating level is also assessed. The slosh wave may also cause the foundation to be overburdened, resulting in possible tipping.

The columns, if present, are assessed based on if they can withstand the deflection induced by the design seismic event.

Many of the reservoirs have valve vaults the pipes run though. Pipes must also be able withstand movement and differential settlement between the reservoir structure and vaults that may occur in the design seismic event. Reservoirs with an attached vault are also assessed to determine if the attachment is suitable to limit differential movement and impact during aseismic event. The pipes should be protected from being crushed during the event as well.

2.4.3.2.1 Steel Tanks

For the steel tanks, the reservoir shell was assessed to determine if it can withstand forces acting upon them induced by the design seismic event. The impact of the slosh wave may cause damage the roof and rafters and dislodge roof plates. Overturning and anchorage were also assessed. The slosh wave may induce a soil bearing pressure that exceeds the temporarily allowable bearing pressure, which indicates the structure would overturn.

Steel reservoirs may be anchored with straps, but current ACI standards require them to be ductile relative to their connection. In other words, the anchor straps should be somewhat flexible with a system such as anchor chairs. During a seismic event, improper anchors can negatively impact the foundation by being pushed into concrete (breakout) or being pulled out. Alternatively, the welded attachment may rupture causing damage to the steel shell.

2.4.3.2.2 Prestressed Concrete

For prestressed concrete reservoirs, the seismic joints and strand wrap are assessed as to if they meet code requirements. These reservoirs are also checked to see if the roof-to-wall interface accounts for the different movement of the roof and walls that is projected to occur during the seismic event. Anchorage of the concrete reservoirs is accomplished through the seismic joint connection between the wall and floor and is assessed as part of the wall system.

2.4.3.2.3 Reinforced Concrete Dome Roof

The reservoirs are assessed to determine if the wall reinforcement is suitable for the design seismic event. The roof-to-wall connection is assessed to determine if it can withstand the seismic event.

2.4.3.2.4 Other Concrete Structures

These reservoirs are assessed to determine if the walls are adequately reinforced against seismic events. The roof to wall connection is assessed to determine if it can withstand the seismic event.

2.4.4 Water Quality/Sanitary

The Water Quality/Sanitary scoring category relates to conditions of the reservoir that may affect hygiene and health, and therefore, its scores are based on how well the component meets Washington State Department of Health (DOH) best practice recommendations. The structure itself is also assessed for if it likely to cause water quality issues, such as a leaky roof that has organic material accumulation.

The ability of the facility to promote high water quality is based on current recommendations from the DOH, the AWWA, and the United States Environmental Protection Agency (EPA). Problems that can affect this section are inlet/outlet/drain piping configurations, and screens or hatches that can be conduits for contamination.

The piping should be configured to facilitate adequate mixing within the reservoir. Inlet pipes can be higher in the reservoir or on opposite sides or hydraulic mixing systems can be implemented to facilitate adequate mixing.

Screens need to be present on openings to the reservoir, such as the vent(s). Screens must be capable of preventing bird and insect intrusion into the reservoirs. A #24-mesh noncorrodible screen is recommended by DOH (2016) and EPA (2014). The DOH also recommends backing the #24-mesh with a 4-mesh screen for added protection.

Drain piping must follow recommendations from The AWWA Water System Design Manual (2019), which states "Reservoir designs must include drain facilities that drain to daylight or an approved alternative that is adequate to prevent cross-connection contamination," per Washington Administrative Code (WAC) 246-290-235(1) The must be an "air gap or other feature to prevent cross contamination." A drywell may hold the drained reservoir water if it has backflow prevention. If the reservoir cannot be drained by gravity, a system such a sump pump can be used. Also, "the reservoir drain should be separate from the outlet pipe to minimize the risk of a cross connection and prevent sediment from entering the distribution system." Daylight drains should be properly screened with screens that can be removed during cleaning events (EPA 2014).

Overflow piping must follow recommendations from The AWWA Water System Design Manual (2019) as well. "Every reservoir design must include an overflow pipe with atmospheric discharge and suitable means to prevent cross-connection contamination per WAC 246-290-235(1)." Overflows should be:

1. properly screened or otherwise secured (with a screen or duckbill valve),
2. easy to observe and maintain… extending "to an elevation of 12 to 24 inches above ground level," and
3. protected against cross connections

Air gaps are also assessed in this section, which prevent backups from sanitary and storm sewers from entering the reservoirs. To be an approved air gap, the DOH (2011) requires a minimum

vertical gap between the drain or overflow pipe outlet and the overflow rim of the receiving storm or sanitary sewer. For free standing pipes, this vertical distance must be at least twice the diameter of the overflow/drain pipe. The vertical gap must at least three times the diameter of the overflow/drain pipe if there is a wall within a distance equal to three times the diameter of the pipe. This gap minimum is also required if a corner is within four times the diameter of the pipe.

Hatches present a possible conduit for reservoir contamination. The DOH outlines good and poor roof hatch design, as well as high maintenance designs (2017). A good hatch design is "A cast-in-place hatch frame is framed at least 4 [inches] above the roof and has a continuous neoprene seal along all four sides." These hatches have watertight covers that overlap the frame. A good hatch design for steel tanks is a bolted access hatch lid with rubber seals or gaskets. One example of a high maintenance design is an access hatch where the drainage from the lid collects in a gutter. Another example is a thin steel frame that frequently caused seal issues with the lid.

Vent design is also considered to reduce risk of water quality issues (DOH 2019). Other than having proper screens, vents should have downturned openings that have a venting surface at least 24 inches from the horizontal surface below (EPA 2016 And DOH 2016). Mushroom or hooded vents on taller reservoirs should be made from noncorrodible steel, have downward or sideways openings, and have a cover that extends at least down to the mesh. The EPA also requires the bottom of the mesh be at least 8 inches from the horizontal surface below it. Vents should be easy to observe, access, and maintain as well (DOH 2019).

2.4.5 Safety

The Safety Score is based on how well the reservoir component meets current occupational safety and health requirements. While the Occupational Safety and Health Administration (OSHA) governs workplace safety in the United States, Washington is one of twenty-two states that have a state plan in place that cover both private sector and state and locate government workers. The Washington Industrial Safety and Health Act (WISHA) gives power to the Division of Occupational Safety and Health (DOSH) in the Department of Labor and Industries (L&I) to govern occupation safety and health within the State. This assessment follows L&I requirements, but also checks to make sure requirements are met in OSHA standards, if they are stricter.

2.4.5.1 Ladder Safety

Per WAC 296-876-60065, fixed or permanent ladders that are 24 feet or taller are required to have ladder safety devices. Ladder safety devices are defined "as any device, other than a cage or well, designed to arrest the fall of a person using a fixed ladder." They are commonly referred to fall protection. Per WAC 296-876-60065, the maximum ladder length is 50 feet, or landing platforms need to be provided at a maximum interval of 50 feet.

Per WAC 296-876-60030, the maximum spacing between ladder rungs is 12 inches. These rungs are to be at least 1-inch wide on reservoir interior ladders, as they are "subject to unusually corrosive exposures," while ladders on the exterior must be at least 3/4-inch in diameter. Rungs

need to be 16 inches wide. Per WAC 296-876-60050, the ladder side railing must extend at least 42 inches above the roof.

Per the definition of a ladder safety device in the WAC, cages do not serve as fall protection for ladders higher than 24 feet. However, it is unclear in the WAC when cages need to be upgraded. Instead, we default to OSHA requirements; 29 CFR 1910.28(b)(9)(i)(D) states that ladders installed after November 19, 2018 cannot use cage systems for fall protection. Further, it states that for ladders installed before this date, the cage systems alone as fall protection are being phased out. The fall protection on all ladders will need to be upgraded by 2036 at the latest. If ladder-related rehabilitation is occurring, the fall protection must be upgraded at that time.

2.4.5.2 Roof Safety

Per WAC 296-155-246, falls of 10 feet or more require a Fall Protection Work Plan. Per WAC 296-155-24609, Fall protection is required at 4 feet or more. "You must ensure that the appropriate fall protection system is provided, installed, and implemented according to the requirements in this part when employees are exposed to fall hazards of 4 feet or more to the ground or lower level when on a walking/working surface." A standard guardrail system that meets requirements of WAC 296-155-24615(2) has a height of 42 inches (+/- 3 inches), has a mid-rail, and has a toe board that is 3.5 inches high.

Fall protection requirements are loosened for some professions. On concrete and masonry work, fall protection is required for falls above 6 feet. For roofing work, it is required at 10 feet. Per WAC 296-155-24603, "Roofing work means the hoisting, storage, application, and removal of roofing materials and equipment, including related insulation, sheet metal, and vapor barrier work, but not including the construction of the roof deck."

A "steep pitched roof" is one "with a pitch greater than 4 in 12." A low-pitched roof has a slope less than or equal to 4 in 12. This ratio corresponds to a slope of 18.4 degrees.

For low sloped roofs with falls of 4 feet or more, several options are available:

- Fall restraint system;
- Fall arrest system;
- Positioning device system;
- Safety monitor and warning line system; or
- Safety watch system.

For steep pitched roofs with falls of 4 feet or more, the following options are available:

- Fall restraint system;
- Fall arrest system;
- Positioning device system;

Additionally, "Employees exposed to falls of 4 feet or more while working on a hazardous slope must use personal fall restraint systems or positioning device systems." Per WAC 296-155-24603,

a hazardous slope is defined as "a slope where normal footing cannot be maintained without the use of devices due to the pitch of the surface, weather conditions, or surface material." For our analysis, we assume the definition of a steep roof applies reservoir roofs.

For reference, the following definitions are provided, per WAC 296-155-24603:

- Fall restraint system: A system in which all necessary components function together to restrain/prevent an employee from falling to a lower level. Types of fall restraint systems include standard guardrail systems, personal fall restraint systems, warning line systems, or a warning line system and safety monitor.
- Fall arrest system: A fall protection system that will arrest a fall from elevation. Fall arrest systems include personal fall arrest systems that are worn by the user, catch platforms, and safety nets.
- Positioning device system: A full body harness or positioning harness that is worn by an employee, and is rigged to allow an employee to be supported on an elevated vertical or inclined surface, such as a wall, pole or column and work with both hands free from the body support.
- Safety monitoring system: A type of fall restraint system in which a competent person whose only job responsibility is to recognize and warn employees of their proximity to fall hazards when working between the warning line and the unprotected sides and edges, including the leading edge of a low pitch roof or other walking/working surface.
- Safety watch system: A fall protection system as described in WAC 296-155-24615(6), in which a competent person monitors one worker who is engaged in repair work or servicing equipment on low pitch roofs only.

2.4.5.3 Hatch Safety

Per WAC 296-155-24609, "you must guard floor openings by one of the following fall restraint systems:

- A standard guardrail system…
- A warning line system…
- If it becomes necessary to remove the cover, the guardrail system, or the warning line system, then an employee must remain at the opening until the cover, guardrail system, or warning line system is replaced. The only duty the employee must perform is to prevent exposure to the fall hazard by warning persons entering the area of the fall hazard."

Also, per WAC 296-809-60004, for entrances into confined spaces (i.e. reservoirs), "when entrance covers are removed, promptly guard the opening with a railing, temporary cover, or other temporary barrier to prevent accidental falls through the opening and protect entrants from objects falling into the space." (L&I, n.d.).

Also, per WAC 296-876-60055, roof hatches where ladders are accessed should be at least 30 inches wide.

For steel tanks, AWWA D-100 Section 7.4.4 requires two access hatches in the lower level for emergency ingress and egress. “At least one manhole shall be circular with a minimum diameter of 30 inches. Other manholes may be circular, 24 inches in diameter… minimum size.”

2.4.6 Operations and Maintenance

In the Operations and Maintenance Section, the reservoir structure and appurtenances are assessed to see if they function as designed or affect the other systems. Any operational difficulties noted by City staff is noted in this section, as well as issues noted during the inspection.

2.4.6.1 Site and Security

Site and security components are assessed in the Operations and Maintenance section since if standards are not met, the reservoir may not function optimally. Are the components operating in a fashion similar to current design standards? The fence and security devices such as intrusion alarms, motion detectors, and cameras are assessed. Graffiti is indicative of poor site security.

Vegetation separation is assessed. Generally, trees should be no closer to the reservoir than their height (i.e. a 10-foot tall tree should be no closer than 10 feet to the reservoir). This will limit damage when these trees fall, increase soil water evaporation near the structure, and protect the foundation from root damage. Also, it will reduce the amount of organic material buildup on the exterior. For partially buried reservoirs, DOH (2019) recommends a 50-foot minimum spacing between the reservoir and trees to prevent root penetration. The area that is recommended to be cleared around each reservoir was approximated for the cost estimate.

Site drainage must also facilitate drainage away from the reservoir. Generally, on modern reservoirs, a 2 percent slope away from the structure is called out to limit rainwater draining to the reservoir or its foundation, which present water quality risks. The soils should also be drained near the structure, not pooling or waterlogged during the inspection.

2.4.6.2 Roof Drainage

The roof is assessed to determine if can drain properly. Adjacent structures, such as valve vaults, are examined to check if they have drainage or design issues that could cause water to enter the reservoir.

2.4.6.3 Appurtenances

Exterior appurtenances (ladders, hatches, and manways) are assessed if they can be used by staff or function as designed. In the event of a failure in the largest pipe (inlet, outlet, or drain) or accounting for the design inflow rate, the vents are checked to ensure they allow for adequate air movement to alleviate vacuum or positive per DOH (2019). First, the roof penetration area is assessed to determine if it is large enough. Then the screened area is assessed, as air can only flow through the open areas of the screens. Both the existing and required #24 mesh are assessed.

Finally, if there are any retrofitted collars, airflow through the existing available area is also evaluated.

2.4.6.4 Valving and Piping

The inlet, outlet, drain, and overflow piping are assessed to if they function properly. City staff reports on if there are problems operating the reservoir. Then any daylight drains are assessed if they have adequate energy dissipation and dechlorination. Dechlorination systems should stay in place and have the capacity to treat all water before being drained to a storm sewer or the environment.

2.4.6.5 Misc.

Assessed in this category are the cathodic protection system, level sensors, and hydraulic mixing systems, if applicable. Cathodic protection systems are assessed to determine if they can control corrosion without negatively impacting the coating below the waterline. As the water level changes, the cathodic protection system output should change, known as auto-potential systems.

2.4.7 Obsolescence

The Obsolescence Score is based on how current the component is and how easy it would be to replace it if it. Murraysmith's knowledge of current products are utilized in this section.

2.4.8 Overall Scores

To determine the overall score of each reservoir, first a Category score is found by averaging the assessed, relevant components. Then a weight system, shown in Table 2-2, is implemented to determine the overall scores. Each of the categories provide a weighted score, which add up to 100. These are applied to the mean system scores to determine the overall score. This weighting system was implemented based on the associated risk from the categories. A structural issue would likely take the reservoir out of service for a considerable time and would induce a high risk for the City's supply.

Table 2-2: Overall score weighting system

Category	Weight
Cleanliness and Coatings	5
Material Deterioration	10
Static Structural Performance	20
Seismic Structural Performance	30
WQ/Sanitary	10
Safety	10
Operational	10
Obsolescence	5

2.5 Recommended Improvements

An improvement is recommended to rectify each deficiency noted in the Condition Assessments. In some cases, there are multiple options to select from. Because the static and seismic recommendations often involve similar retrofits, they are combined in the recommended improvement section. Although coatings and corrosion issues are often addressed simultaneously on steel tanks though abrasive blasting and coating, these categories are kept separate. Recoating is addressed in the coating section on steel reservoirs.

Most planning level cost estimates are based on unit costs, derived from previous projects Murraysmith or subconsultants have received contractor bids on. As some of these recommended improvements are unique to the City, there is estimation involved with some of the recommended improvements. Once summed, a 30 percent contingency is applied to the subtotal. Then the current Bellingham tax rate, 8.7 percent is applied to the subtotal and contingency. Then a 35 percent engineering rate is applied to the total cost of the project (subtotal, contingency, and tax), which accounts for design services, administration, and construction management. We believe that these planning-level estimates are appropriate for the City to begin preparing for rehabilitation and/or replacement.

2.6 Conclusions

Each reservoir's conclusions are wrapped up. Major deficiencies and recommended improvements are summarized.

The next inspection time is also proposed. For steel tanks AWWA D100-11 states "inspection of the interior and exterior of the entire tank with corrective maintenance at three-year intervals is recommended." However, AWWA recommends "where water supplies have sediment problems, annual washouts are recommended (2013). "Annual inspection and maintenance of the exposed side of the tank shell-to-bottom connection" is also recommended. For prestressed concrete structures, AWWA D110 states routine inspections should take place every 5 to 10 years. Concrete structure inspections are recommended to begin with a watertightness test.

Section 3

Marietta Reservoir

3.1 Tank and Site Overview

The Marietta Reservoir is a 100-foot diameter above-ground welded steel cylinder with a 2.5-MG capacity, shown in Figure 3-1. It was constructed in 1969. It is 50 feet, 3 inches tall, consisting of five 8-foot plates, one 10-foot plate, and a 3-inch angle to the roof. The overflow is located at the base of the angle at a height of 50 feet which is 3 inches below the roof plate. The roof is supported by one central and eight additional 10-inch diameter columns. The 8-foot by 20-foot roof plates are approximately 0.204 inches thick and sloped at an angle of 0.75:12. According to the provided plans, a 12-inch wide by 4-foot deep footing supports the reservoir to which the shell is anchored with 33 6-inch wide anchor straps.

A 10-foot by 10-foot concrete masonry unit (CMU) block building with a wood-frame roof and a concrete slab is situated next to the reservoir. It houses the electrical systems associated with the reservoir's corrosion protection system. A 12-foot by 7-foot below-grade concrete equipment vault houses the outlet and water main lines.

Figure 3-1: The Marietta Reservoir, viewed looking northeast

The reservoir is not currently in operation, but would be operated between 45 and 50 feet, as noted by the City. It is situated in a rural neighborhood that is forested, about 5 miles northwest

of the Bellingham Central Business District (CBD). If operating, the reservoir serves the 276 North Zone.

The geotechnical investigation indicated that the reservoir is founded on very dense glacially consolidated soils. These soils were encountered from the surface to the extent of the boring, 40.5 feet below the existing ground surface (bgs). Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Soils encountered at this site have an allowable bearing pressure of 5,000 psf and are not at risk of liquification. This site is not expected to have issues with slope instability.

3.2 Inspection Summary

A drained inspection was conducted January 24, 2019.

3.2.1 Exterior Inspection Summary

3.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the property that is 10 to 15 feet from the walls. The site is generally well-graded, directing runoff away from the reservoir. However, standing water was noted in the northwest side of the reservoir (Figure 3-2). The reservoir was clear of the drip line of trees, but a few trees were found just beyond the fence (Figure 3-1).

Figure 3-2: Standing water along foundation edge

3.2.1.2 Exterior Walls

The reservoir exterior was overcoated in 2013 according to documents provided by the City. Lead is noted in the original specification's prime coat. Coating DFT was measured to be 8–16

thousands-of-an-inch (mils) thick. Typical high-performance coatings for the exterior surfaces of water storage tanks are on the order of 12 to 16 mils. Overall, steel pitting and coating chips were minimal (Figure 3-3 Left). Heavy organic growth was noted where water drained from the roof (Figure 3-3 Right).

Figure 3-3: Exterior ladder and wall staining/scratching

3.2.1.3 Foundation

The reservoir is supported on a concrete ring foundation, which did not exhibit settlement or major cracking. No grout layer was noted between the shell and top of concrete ring foundation. While the provided plans indicate a 12-inch wide by 4-foot deep footing, the exposed portion of this footing was measured to be 15.5 inches wide on site. This footing is likely to be closer to 30 inches wide since anchors are generally centered on footings.

3.2.1.4 Exterior Roof

The roof's plates did not appear to be lapped correctly as water was ponding at the margins of many of the plates. The coating was failing at the locations and near the perimeter (Figure 3-4).

Figure 3-4: The exterior roof's ponding and damage

3.2.1.5 Exterior Appurtenances

This tank has one circular side manway that had a 24-inch diameter (Figure 3-5). Corrosion was noted on the hatch.

Figure 3-5: The singular 24-inch side entry hatch.

The exterior ladder is caged and had a cable fall protection system. This system did not appear to be operational during our inspection (Figure 3-3 Left and Figure 3-6 Left). The roof entry hatch is a 2-foot by 2-foot clam style and had an intrusion alarm (Figure 3-6). Beehives were noted on the lid and an encompassing gasket as missing. Some corrosion was noted throughout the hatch lid.

Figure 3-6: The roof entry hatch and caged ladder

The roof vent did not exhibit corrosion (Figure 3-7). The penetration and hood diameters are approximately 16 inches and 30 inches, respectively. The meshed areas are approximately 4 inches x 6 inches, and the screen has approximately 0.25-inch openings with 0.04-inch wire.

Figure 3-7: The roof vent and screen

3.2.2 Interior Inspection Summary

3.2.2.1 Interior Walls

The interior coal tar enamel coating, ranging in thickness from 100–300 mils, was tightly adhered to the walls (Figure 3-8). It is likely that this material is the original coating applied during construction. No corrosion was noted on the walls. To protect the walls from corrosion, a constant voltage model cathodic protection system is installed. Measurements of plate thicknesses are shown in Table 3-1.

Figure 3-8: The interior roof and walls

Table 3-1: Steel plate thicknesses

Location	Design Thickness (in)	Measured Thickness (in)
Roof	0.188	0.200-0.208
Sixth Course	0.250	0.249
Fifth Course	0.250	0.240
Fourth Course	0.375	0.378
Third Course	0.469	0.443
Second Course	0.568	0.558
Bottom Course	0.688	0.663-0.667

3.2.2.2 Floor

The floor coating is brittle and cracks in places when walked on. At sound coating locations, the material continues to be tightly adhered to the floor. No corrosion was noted on the floor, but dirt has accumulated (Figure 3-9).

Figure 3-9: Interior floor

3.2.2.3 Interior Roof and Columns

The roof and roof support members show minor areas of rust staining, particularly on the edges of the beams and rafters (Figure 3-8 Right). However, when viewed from the roof access hatch, there was no observed significant metal deterioration. The columns still have good coating adhesion and are not corroding.

3.2.2.4 Interior Appurtenances

The Marietta Reservoir does not have an interior ladder.

3.2.3 Piping and Valving Inspection Summary

The inlet piping to the Marietta Reservoir has been upgraded to facilitate mixing within the reservoir (Figure 3-10 Left). This system is braced against a column, which appears to have impacted the column coating at this location (Figure 3-10 Right). The outlet (Figure 3-11 Left) and drain (Figure 3-11 Right) piping exhibited little to no coating loss and/or corrosion. The overflow pipe was very close to the roof plate (Figure 3-12 Left). The outlet of the overflow pipe did not extend to the ground surface and did not have energy dissipation (Figure 3-12 Right). The drain piping connects to a drainage ditch, but the outlet was unable to be found during our inspection.

Figure 3-10: Retrofitted inlet pipe

Figure 3-11: Outlet (Left) and drain (Right) pipes.

Figure 3-12: Overflow piping inside (Left) and outside (Right) the reservoir

The piping and valving were updated in 2012 and still looked new (Figure 3-13). This configuration includes check valves and isolation valves. A level transmitter is in the adjacent pump building.

Figure 3-13: Retrofitted piping and valving in the Marietta Reservoir

Washdown piping was not observed during our inspection.

3.3 Condition Assessments

3.3.1 Cleanliness and Coatings

The exterior surface of the tank is dirty and has several areas of organic material accumulation on the roof and sidewalls. If cleaned and spot treated, the exterior coating will remain viable for many years to come. Although extensive organic growth was observed where water drains from the roof, the coating itself on the exterior walls was still found to be in generally good condition. However, the exterior roof's coating was in poor condition in a few areas due to ponding affecting the topcoat. Lead is present in the prime coat of the exterior, which is not approved under AWWA D102-17.

Overall, the interior coating is in very good visual condition and when coupled with the cathodic protection system, would provide a minimum of 10 years of additional service life. Coal Tar Enamel was once widely used due to its exceptional corrosion performance and resistance to organic growth. However, this coating is known to leach alkyl benzenes and polycyclic aromatic hydrocarbons (PAHs) into drinking water supplies (EPA 2002, National Research Council Safe Drinking Water Committee 1982). Although the leaching of these chemicals reduces overtime, this coating is currently not approved for use as an interior reservoir coating under AWWA D102-17.

3.3.2 Material Deterioration

The visible portion of the concrete foundation is in good condition. Also, there was no observed significant corrosion on the structure or appurtenances, On the interior, the rust staining and coating loss of the roof, rafters, and girders are considered minor. Marietta is good condition from a corrosion perspective.

3.3.3 Structural Performance

3.3.3.1 Static Analysis

The roof framing, the roof plate thicknesses, rafters, columns, and girders (which support the rafters from the column) were all sized appropriately. The reservoir shell was measured to be slightly thinner than the design. Because of this, the walls are overstressed by 1 to 3 percent at the overflow operating level. Based on the field measurements of the foundation, the bearing pressure is likely 3,100 psf. This is acceptable based on the allowable soil bearing capacity of 5,000 psf noted in the geotechnical report.

3.3.3.2 Seismic Analysis

Shell plates can temporarily exceed their capacity during seismic events. If operated at the overflow level of 50 feet, all shell courses other than the fifth from the bottom (second from the top) meet this design requirement. The failing course exceeded its capacity by 1 percent, which is relatively small. Operating at 50 feet, the design seismic event is estimated to induce a slosh wave that is 4.5 feet high which would significantly damage the roof. The maximum operating level to save the roof from slosh impact is 44.5 feet. At the 45.5-foot operating level, only the outer part of the roof would be impacted. The temporary maximum allowable bearing pressure would be exceeded if operated above 45.5 feet.

Based on current code, the existing anchor straps are not adequate to resist overturning at the 50-foot operating level. The current code calls for anchor straps to be ductile, relative to their connections, which the installed straps are not. If the maximum operating level is lowered to 45.5 feet or lower, the anchor straps are not needed. However, with the anchor straps removed, the horizontal displacement of the pipes increases from 0.5 inches to 2 inches. With this displacement, flexible pipe couplings are required on the reservoir's piping that run through the vault.

3.3.4 Water Quality/Sanitary

The retrofitted inlet appears to be suitable to facilitate mixing within the reservoir. The outlet, drain, and overflow piping also appeared to facilitate high quality water within the reservoir. Possible contamination sources include the roof hatch and vent system, which do not prevent insect intrusion and were found to be deficient. The roof hatch would be classified as a "high-maintenance design" because the gaskets often fail to provide a tight seal against the relatively small surface area of the frame. Overall, the hatch is in poor condition and the vent is in fair condition related to water quality.

3.3.5 Safety

The exterior ladder fall protection system was not operating during the inspection, which needs further investigation. If the City is relying on the cage system alone as fall protection, it will not

meet code in 2036, when cage systems are being phased out as fall protection. On the low-slope roof, the lack of any fall protection, warning system, or safety railing was noted, which presents a safety risk for workers on the roof. The roof hatch requires a roof hatch railing or temporary barrier. Another safety consideration is that the reservoir only has one side entry hatch, that does not meet best practices in AWWA D100 Sect 7.4, which requires two entry manways, one of which is at least 30 inches in diameter. Although the reservoir is currently not in use, the safety condition of the reservoir is poor.

3.3.6 Operations and Maintenance

3.3.6.1 Site and Security

Based on the inspection, the fence appeared suitable to deter vandalism. An intrusion alarm is installed on the roof's access hatch. However, tall trees were observed to be too close, particularly on the north side of the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, and can cause organic material accumulation problems. This can be identified on the Google aerial image (Figure 3-14). Standing water was observed on the surface near the foundation. The site/security is in fair condition.

Figure 3-14: An aerial image of the Marietta Reservoir

3.3.6.2 Roof Drainage

The roof's steel plates were lapped in a fashion the allows water to pond and has damaged coating.

3.3.6.3 Appurtenances

The side hatch has a 24-inch diameter, which made it difficult to get in and out of the reservoir. The vent was found to be undersized, not meeting requirements in the DOH Water Systems Design Manual (2019). Assuming a rapid draining during failure of the 16-inch outlet pipe at a 50-foot reservoir operating level, the existing roof vent failed to provide adequate venting. The roof penetration would need be increased to approximately 21 inches in diameter with a properly sized

screened area to allow for the rapid draining scenario. The appurtenances' operational condition is poor because of the difficulty using the hatch and the undersized vent.

3.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. However, the DOH Water System Design Manual recommends the overflow to "extend down to an elevation of 12 to 24 inches above ground level and discharge into a splash plate or rocked area." Also, these pipes must "be located where they can be inspected as part of routine maintenance (WAC 246-290-235(1)(c))." The outlet of the drain could not be located. It is possible the screen, dechlorination, and energy dissipation systems do not meet recommendations. While the inlet and outlet pipes are operationally in good condition, the overflow and drain pipes are in poor operational condition.

3.3.6.5 Misc.

The existing cathodic protection system transformer rectifier is a constant voltage model. Rectifier units for water tanks should be auto-potential in order to account for fluctuations in water level.

3.3.7 Obsolescence

Overall, the equipment on site appeared to be modern and easily replicable other than the cathodic protection system. The reservoir has a favorable obsolescence score.

3.3.8 Condition Scoring

Overall, the condition score of the Marietta Reservoir was fair, at 3.3. It was primarily affected by the seismic score. Operated at 50 feet, the roof would be affected by the slosh wave, the reservoir is susceptible to overturning, and the anchors may induce structural damage. Should the reservoir be operated at the recommended height, anchors removed, and flexible couplings installed, the score would increase considerably. Other issues include the safety of exterior ladder and roof, the undersized vent with too coarse mesh, evidence of insects in the roof hatch, and cathodic protection system being of a constant voltage model. A summary of the scoring is shown in Table 3-2 and the full Score Matrix is included in Appendix A.

Table 3-2: Condition Scoring of the Marietta Reservoir

Categorical Scores	
Cleanliness and Coatings	4.4
Material Deterioration	4.5
Structural Performance - Static	4.0
Structural Performance - Seismic	2.2
WQ/ Sanitary	4.1
Safety	2.0
Operations & Maintenance	3.7
Obsolescence	4.3
Overall	**3.3**

3.4 Recommended Improvements

3.4.1 Cleanliness and Coatings

First, the exterior surfaces need to be pressure washed to remove dirt and other debris which will help extend the life of the coating. Then repair of the areas of topcoat damage, particularly near the roof access hatch where water ponds are recommended. This will preserve the life of the prime coat material and keep the underlying steel from corroding. The City should consider removing the exterior lead-containing coating and applying an AWWA D102-17-compliant coating.

Up-to-date requirements on coal tar enamel coatings should be consulted prior to this reservoir returning to service. The City should consider removing the coal tar enamel coating and applying an AWWA D102-17-compliant coating to the interior in advance of the reservoir returning to service.

3.4.2 Material Deterioration

There are no recommended improvements for concrete deterioration and corrosion at the Marietta Reservoir.

3.4.3 Structural

Because the Marietta reservoir is currently not needed for storage, the option to perform no structural upgrades and lower the maximum operating level to 44.5 feet is recommended. At this level, slosh effects, foundation loads, and shell issues are all addressed. Other options are discussed in the structural analysis in Appendix A. The anchor straps will need to be removed from the reservoir to prevent damage to either the reservoir shell or foundation during the design seismic event. The City can expect to spot repair coating at these locations. Inlet and outlet pipes will then need flexible couplings installed to account for the increased horizontal movement of the pipes. The foundation width also needs to be verified in the design phase of reservoir retrofits.

3.4.4 Water Quality/Sanitary

To rectify deficiencies in water quality/sanitary, the beehives in the roof hatch should be removed and new neoprene gaskets applied to both the hatch frame and lid. If the reservoir comes back into normal service, a better option would be replacing the hatch. Additionally, the vent screen is too coarse. It should be replaced with a #24-mesh and a 4-mesh screen backing. However, the vent should be replaced, as outlined in Section 3.4.6.

3.4.5 Safety

The exterior ladder's fall protection needs to be repaired or replaced. If the fall protection is not functioning, either it or the entire ladder will need to be replaced by the 2036 deadline. Furthermore, the roof needs suitable fall protection, such as a railing. While the reservoir does not have an interior ladder, and one is not required, the City can consider the installation of an interior ladder to facilitate future floating inspections or other access needs. The roof hatch would likely need to be upsized to 30 inches if used for access. At any rate, a safety railing is recommended around the roof hatch. The installation of a 30-inch hatch in the lower course, opposite the existing hatch, is also recommended.

3.4.6 Operations and Maintenance

3.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view. Some of the trees may be on the adjacent property, 4585 Wynn Rd, so coordination with the property owner will be required. Installation of a French drain system is also recommended where water is ponding adjacent to the foundation. Adjusting the grading in this area to direct water away from the reservoir may be a possibility as well.

3.4.6.2 Roof Drainage

The most appropriate means for lessening the effect of the poor roof plate lapping is to have City personnel perform an annual inspection on the roof and complete any minor coating repair work at that time, as outlined in Section 3.4.1.

3.4.6.3 Appurtenances

The vent penetration and screened area should be increased to allow adequate airflow, bringing the reservoir into compliance with the DOH requirements (2019). The penetration diameter should be increased from 16 inches to approximately 21 inches. Alternatively, a new vent can be installed on the reservoir with a penetration area of at least 1-square foot. In such a case, the existing mesh should be replaced with #24 mesh with 4-mesh backing. Before being brought back

online, the vent improvements should be implemented so air exchange is not further reduced with a finer mesh. Recommendations for the side entry hatch were discussed in Section 3.4.5.

3.4.6.4 Valving and Piping

The City should carry out an inspection to locate the outlet of the drain piping. This location should be permanently marked, and the trail maintained so it can be easily found in the future. Any issues with the screen, dechlorination, and/or energy dissipation of the outlet should be investigated and corrected. Because the overflow is too close to the ceiling and does not extend near the ground level, installation of a new overflow is recommended. The level of the overflow should keep the maximum operating level below 44.5 feet.

3.4.6.5 Misc.

To extend the life of the coating below the waterline in the interior, the existing constant voltage rectifier should be replaced with an auto-potential unit.

3.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Marietta Reservoir are shown in Table 3-3.

Table 3-3: Marietta deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 2,000
	Intermittent exterior coating failure and roof plates lapping causing ponding	Spot Repair Coating	$ 10,000
	Exterior coating contains lead and interior coating is coal tar enamel. Both are not AWWA-approved.	Consider removal of coating and application of AWWA-approved coating systems	$ -
STRUCT	Wall overstressed & roof impacted by slosh; Reservoir overturns	Lower max operating level to 44.5 ft; remove anchors.	$ 13,000
	Without anchors, pipe horizontal displacement increased from 0.5 inches to 2 inches	Install flexible coupling(s)	$ 32,000
WQ	Roof hatch has evidence of insects and is a high-maintenance design (small contact area)	Remove beehives & replace gasket w/ neoprene seals; consider upgrades	$ 1,000
	Vent screen non-compliant	Replace vent screen (addressed with new vent)	$ -
SAFETY	Exterior ladder fall protection is not functioning, but has cage	Repair exterior ladder fall protection or replace cage by 2036	$ 6,000
	Lacks roof fall protection and roof hatch railing	Install roof fall protection and roof hatch railing	$ 44,000
	Missing second side access hatch	Install new 30" side access hatch	$ 16,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 12,000
	Site not draining	Install French drain	$ 3,000
	Vent undersized	Replace roof vent	$ 15,000
	Unable to locate drain pipe	Locate; replace screen, retrofit energy dissipation & dechlorination system if needed	$ 6,000
	Overflow non-compliant	Replace overflow	$ 44,000
	Cathodic protection system is constant voltage model and lacks stationary reference electrode	Install automatically controlled potential rectifier and stationary reference electrode	$ 10,000
		Subtotal	$ 214,000
		30% Contingency	$ 64,200
		8.7% Tax	$ 24,203
		Engineering, Administration, and Construction Management	$ 105,841
		Total Project Costs Estimate	$ 408,245
		Total Project Costs Estimate (rounded)	**$ 410,000**

3.5 Conclusion

The Marietta Reservoir is currently out of service and the City is unsure if the Reservoir's storage will be required in the future. The City should begin by assessing how this reservoir's lower maximum operating level would affect the 276 North Zone. If this level is not feasible, other options outlined in the structural report in Appendix A should be implemented. If the City wishes to keep the reservoir up to code, the anchor straps should be removed, and flexible couplings installed on the piping. Regardless of timing of these structural retrofits, pressure washing and spot repairing the exterior coating is recommended in the near future to avoid degradation of the steel shell. The City may also wish to promptly address safety issues associated with the reservoir to facilitate future inspections.

While the reservoir remains empty, the roof should be inspected every 3 years and coating reapplied to areas that are failing. Cathodic protection inspections by a qualified engineer should occur annually if the reservoir is brought back into service. General inspections should take place every 3 years regardless the operating state of the reservoir.

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Section 4

Padden Reservoir

4.1 Tank and Site Overview

The Padden Reservoir is a 59-foot diameter above-ground welded steel cylinder with a 0.5-MG capacity, shown in Figure 4-1. It was constructed in 1967. The reservoir shell plates consist of three courses and are a total height of 22 feet, 3 inches. The overflow is located at the base of the angle and within the rafter line. The overflow is at a height of 25 feet which is 3 inches below the roof plate. The roof is supported by one centrally located 6-inch diameter column. The 6-foot by 18-foot roof plates are approximately 0.189 inches thick and sloped at an angle of 1:12. According to the provided plans, a 12-inch wide by 3.5-foot deep footing supports the reservoir without any anchors. Outlet piping is shown to run approximately 2 feet below the base of the footing rather than through the footing. The reservoir is operated between 16.5 and 24.5 feet, as noted by the City.

Figure 4-1: The Padden Reservoir, viewed looking northwest

It is situated at the southern part of the distribution area. It is in a forested area about three miles southeast of the Bellingham CBD. The two nearby structures are a 10-foot by 12-foot concrete altitude valve vault and a 513-square foot pump station. The reservoir serves the 457 South Zone. The geotechnical investigation indicated that the reservoir is founded on fill to 6.5 feet and undifferentiated glacial drift soil below 6.5 feet to the boring termination depth of 71.5 feet. The glacial drift soil was quite variable, comprised of sand with silt and gravel and silty sand. A layer of stiff silt was found from 14 to 19 feet bgs. Groundwater seepage was found at 19 feet.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Soils encountered at this site have an allowable bearing pressure of 3,000 psf and are not at risk of liquification. This site is not expected to have issues with slope instability.

4.2 Inspection Summary

Drained and floating inspections were conducted on April 9 and April 30, 2019, respectively.

4.2.1 Exterior Inspection Summary

4.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed is located about 10 feet from the shell. The site is generally well-graded, directing runoff away from the reservoir. Tall trees were found growing just beyond the fence and were contacting the shell (Figure 4-2).

Figure 4-2: Exterior walls organic material accumulation

4.2.1.2 Exterior Walls

Heavy organic buildup was noted on the exterior walls throughout the reservoir (Figure 4-1 and Figure 4-2), and the topcoat was chipped and peeling in many areas (Figure 4-3). The reservoir exterior was overcoated at some point after construction, according to conversations with the City. Coating DFT was measured to be 6.6–14.3 mils. Typical high-performance coatings for the exterior surfaces of water storage tanks are on the order of 12–16 mils.

Figure 4-3: Exterior wall coating is peeling has been damaged

4.2.1.3 Foundation

The reservoir is supported by a concrete ring foundation, which did not exhibit settlement or major cracking. However, the grout layer was failing between the floor plate and top of footing, especially on the north side (Figure 4-4).

Figure 4-4: Failure of grout at base (multiple locations noted)

4.2.1.4 Exterior Roof

Like the walls, significant buildup of organic material was noted on the roof. There was very little ponding (Figure 4-5); it appears roof lapping was done correctly. Roof coating DFT measurements ranged from 3.5 to 9.6 mils.

Figure 4-5: Exterior roof

4.2.1.5 Exterior Appurtenances

This tank has one circular side manway that had a 24-inch diameter opening (Figure 4-6). The exterior ladder is caged above the lockable enclosure (Figure 4-6 and Figure 4-7 Left). Corrosion was noted on the hand rail (Figure 4-7 Left) and rungs of the ladder (Figure 4-7 Right). No railing or fall protection is present on the roof.

Figure 4-6: The singular 24-inch side entry hatch and exterior ladder

Figure 4-7: Exterior ladder roof hatch lid

The roof entry hatch is a 2-foot by 2-foot clam style and had an intrusion alarm (Figure 4-7 Left and Figure 4-8). Corrosion was noted on the lid and interior of the hatch.

Figure 4-8: Interior of roof hatch corrosion

The roof vent, located near the perimeter of the reservoir, had organic debris on the overhang (Figure 4-9). The screen system installed appeared to prevent both bird and insect intrusion. The roof penetration is approximately 20 inches in diameter and the vent screen is approximately 15 inches tall. A 28-inch collar with an overhang was also installed.

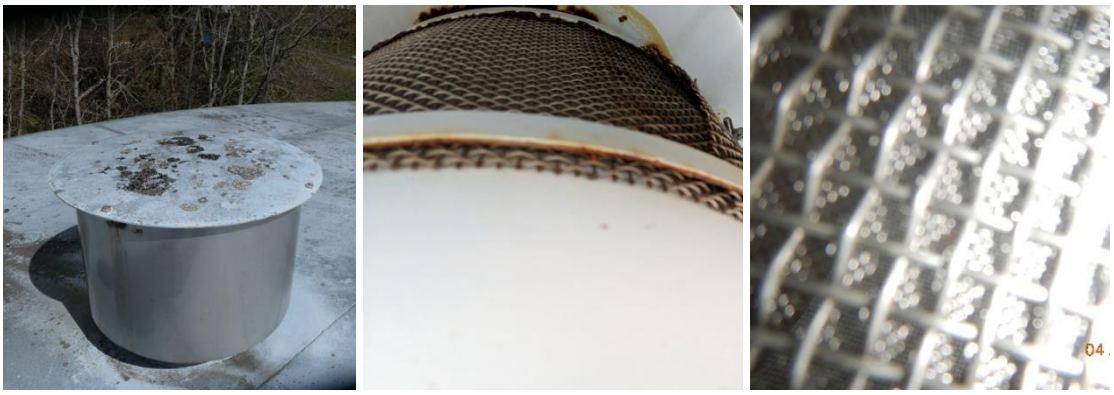

Figure 4-9: Roof vent and screen.

4.2.2 Interior Inspection Summary

4.2.2.1 Interior Walls

On the interior (Figure 4-10 Left), blistering was noted on the bottom 4 inches of the walls (Figure 4-10 Right). To protect the walls from corrosion, a constant voltage model cathodic protection system is installed. On-site testing indicated that the corrosion protection criteria was not being met at all testing locations. Therefore, the rectifier output was increased to bring it into compliance. A full report of the rectifier and structure-to-electrolyte potential data is included in

the Appendix B. Exposed steel locations below the waterline did not exhibited pitting. This is likely due to the operation of the cathodic protection system. The interior coating DFT measurements ranged from 5.5 to 10 mils; typical high-performance coating DFT ranges are 12 to 20 mils. Measurements of plate thickness are shown in Table 4-1.

Figure 4-10: Interior walls, roof, column, and overflow

Table 4-1: Steel Plate Thicknesses

Location	Design Thickness (in)	Measured Thickness (in)
Roof	N/A	0.189
Knuckle	N/A	0.276
Third Course	N/A	0.258
Second Course	N/A	0.262
Bottom Course	N/A	0.307-0.311
Interior Floor	N/A	0.250

4.2.2.2 Floor

The coating on the floor and weld seams was found to be bubbling under the anodes, which is likely due to running the cathodic protection system at a higher-than-required voltage (Figure 4-11). Measurements of coating DFT ranged from 5.5 to 14.5 mils on the floor. Some sediment was accumulating near the base of the column. There was topcoat wear near the inlet/outlet/drain pipe.

Figure 4-11: Interior floor coating

4.2.2.3 Interior Roof and Column

On the ceiling panels at roof plate overlaps, coating was failing, and significant corrosion was observed (Figure 4-12). Areas of sound coating, such as the middle of roof plates, are moderately adhered. The roof knuckle and support members also had significant coating loss and corrosion (Figure 4-13). Near the access hatch, heavy corrosion was also found, and a nut was found to be missing (Figure 4-14).

The column still has good coating adhesion and is not corroding.

Figure 4-12: Interior roof seam corrosion

Figure 4-13: Roof knuckle (Left) and support members (Right) corrosion.

Figure 4-14: Roof hatch corrosion (Left) and missing nut (Right).

4.2.2.4 Interior Appurtenances

The interior ladder did not have any fall protection devices. It also was found to have corrosion near the roof connection point and bubbling in the lower extent (Figure 4-15).

Figure 4-15: Interior ladder corrosion and bubbling

The vent penetration, as viewed from the interior, was missing coating and was severely corroded (Figure 4-16).

Figure 4-16: Roof vent penetration corrosion

4.2.3 Piping and Valving Inspection Summary

The inlet, outlet, and drainpipes are combined in a single 16-inch diameter pipe (Figure 4-17). This pipe is equipped with a silt stop, which was removed during the inspection and replaced after completion. It did not appear to sit flush with the floor. The interior of the pipe appeared to be very corroded. The overflow pipe is only a few inches from the roof plate (Figure 4-18 Left), is braced to the wall, and has a wall penetration in the first course (Figure 4-19 Left). It drains to a storm drain structure without a compliant screen (Figure 4-19 Right), and eventually discharges to Padden Creek with the drain (Figure 4-18 Left). The outlet of the drainpipe did not have a screen or energy dissipation. Dechlorination is reported to take place within a storm drain manhole on site.

The reservoir has an altitude valve and isolation valves, which exhibited minor exterior corrosion (Figure 4-20).

Figure 4-17: The common inlet/outlet/drain with retrofitted silt stop

Figure 4-18: The overflow inlet (Left) and drain/outlet pipe outlet without a screen or energy dissipation (Right).

Figure 4-19: Overflow piping (Left) and screen (Right)

Figure 4-20: The valving includes an altitude valve and isolation valves.

Washdown operation can occur via a fire hydrant near the exterior ladder (Figure 4-1).

4.3 Condition Assessments

4.3.1 Cleanliness and Coatings

The exterior surface of the tank is dirty and has several areas of organic material accumulation on the roof and sidewalls. Based on field observations, the exterior coating is expected to be viable only until 2025 if pressure washed. It is in poor condition.

The interior blistering below the waterline does not appear to be impacting the structure. However, the coating DFT measurements and blistering indicate the interior needs recoating. Above the waterline, the coating is experiencing major failure, especially at the panel seams and on the structural support members. Overall, the interior coating is in poor condition.

4.3.2 Material Deterioration

The seal between the foundation and tank was found to be in poor condition, at the end of its useful life. Even without an intact coating system, the exterior of the structure is free of significant corrosion.

The interior roof is in poor condition; its corrosion problems are significant, particularly in between roof plates and in overlap areas between the beams and plates. The corrosion (and related coating loss) should be addressed within the next 2 years. The pipe's condition related corrosion in assessed to be fair.

4.3.3 Structural Performance

4.3.3.1 Static Analysis

Based on drawings and measurements, the roof framing, the roof plate thicknesses, rafters, and column were all sized appropriately. However, major corrosion issues can jeopardize roofs. All plates are acceptable for the anticipated hydrostatic stresses. Based on the 12-inch footing, the bearing pressure is estimated to be 2,700 psf. This is acceptable based on the allowable soil bearing capacity of 3,000 psf determined in the geotechnical report.

4.3.3.2 Seismic Analysis

The reservoir's shell is adequate for the site's anticipated seismic loads. The slosh wave in this reservoir is estimated to be 4.0 feet high based on the design seismic event. Since damage to the roof would occur below the 25-foot overflow level at 23.75 feet, operating level would need to be lowered to 19.75 feet to reduce confined slosh risk to an acceptable level. A 20.5-foot operating level's slosh wave would only affect the roof knuckle, not the rafters and main plates. This would incur less substantial retrofit requirements than maintaining the 24.5-foot operating level.

This reservoir structure itself was determined to be stable against overturning, even without anchors. However, based on the design seismic event's constrained slosh wave, the seismic bearing on the relatively narrow foundation footing would increase to 6,600 psf. This is higher than the 4,000 psf allowable bearing pressure during a seismic event. A maximum operating height of 22.25 feet would bring the increased bearing pressure to acceptable level. It should be noted that if operated at this level without retrofits, the roof problems would still exist.

4.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A functioning silt stop is also a current DOH best practice; this reservoir's silt stop appears to need some modification. No drain air gap is needed since this reservoir drains to Padden Creek. However, the screen on the overflow and drainpipes were found to be out of compliance as they were not #24 mesh. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gaskets to provide a tight seal against the relatively small surface area of the frame. Overall, the condition of the reservoir related to water quality/sanitary is fair.

4.3.5 Safety

The exterior ladder fall protection is a cage system, which alone as fall protection will not meet code in 2036, when cage systems are being phased out as fall protection. On the roof, the lack of any fall protection or safety railing was noted, which also presents a safety risk. The roof hatch requires a roof hatch railing or temporary barrier. The hatch is undersized for entry if it is to be used. There is also no fall protection on the interior ladder, which would need to be added if workers are to enter from the roof hatch. Due to the heavy corrosion on the ladder, it is not recommended to be used. Another safety consideration is that the reservoir only has one side entry hatch, that does not meet best practices in AWWA D100 Sect 7.4, which requires two entry manways, one of which is at least 30 inches in diameter. The safety of the reservoir is in fair to poor condition.

4.3.6 Operations and Maintenance

4.3.6.1 Site and Security

Based on the inspection, the fence was suitable to deter vandalism and the soils adjacent to the reservoir were draining properly. An intrusion alarm is installed on the roof's access hatch. However, tall trees were found to be too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, and can cause organic material accumulation problems. The site/security is in fair condition.

4.3.6.2 Roof Drainage

There did not appear to be issues with roof drainage at the Padden Reservoir.

4.3.6.3 Appurtenances

The side hatch has a 24-inch diameter, which made it difficult to get in and out of the reservoir. No difficulties were found using the exterior ladder. Padden's vent passed design requirements for roof penetration and free area. However, the vent is not centrally located on the reservoir, which does not meet the requirements of D100-11 Section 5.5, which requires one tank vent to be located near the center of the roof. Other than difficulty using the hatch, the reservoir's appurtenances are in good operational condition.

4.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. However, the dechlorination system was a bag that could be easily pushed out of the way with the force of drained water. The energy dissipation system on the drain/overflow outlet may not prevent erosion in Padden Creek.

4.3.6.5 Misc.

The existing transformer rectifier is a constant voltage model. Rectifier units for water tanks should be auto-potential in order to account for fluctuations in water level.

4.3.7 Obsolescence

Overall, the equipment on site appeared to be older but could be replaced in the event of failure.

4.3.8 Condition Scoring

Overall, the condition score of the Padden Reservoir was fair to good, at 3.6. It was primarily affected by the seismic score. Operated at 24.5 feet, the roof would be affected by the slosh wave and the allowable bearing pressure would be exceeded. Other issues include the failing coating throughout the exterior and above the waterline on the interior, the corrosion above the waterline, and cathodic protection system being of a constant voltage model. A summary of the scoring is shown in Table 4-2 and the full Score Matrix is included in Appendix B.

Table 4-2: Condition Scoring of the Padden Reservoir

Categorical Scores	
Cleanliness and Coatings	3.3
Material Deterioration	3.7
Structural Performance - Static	5.0
Structural Performance - Seismic	2.8
WQ/ Sanitary	4.3
Safety	2.3
Operations & Maintenance	4.2
Obsolescence	3.0
Overall	3.6

4.4 Recommended Improvements

4.4.1 Cleanliness and Coatings

First, the exterior surfaces need to be pressure washed to remove dirt and other debris which will help extend the life of the coating approximately five more years. In some locations, the topcoat is expected to be removed during pressure washing operations due to low adhesion qualities. A full exterior recoat should be completed by 2025. The walls will need to have existing coating removed, proper surface preparation, and application of a new coating. Exterior roof coating recommendations are dependent on the structural retrofits, outlined in Section 4.4.3. The instances of coating failure on the exterior ladder can be dealt with by spot treating or addressed at the time of recoating.

On the interior, numerous instances of coating failure have been noted. Removal of all existing coating with abrasive blasting, proper surface preparation, and application of a new protective coating is recommended within the next 2 years. The improvement for coating issues on the interior roof is dependent on the selected maximum operating level, discussed in Section 4.4.3.

4.4.2 Material Deterioration

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

Shell wall corrosion issues will be addressed with during recoating process, outlined in Section 4.4.1. During this process, repairs of corrosion-induced pits and weld failures throughout the reservoir are also expected. We noted jagged welds near the hatch which will be ground down and repaired during this process. The improvement for corrosion issues on the interior and exterior roof is dependent on the selected maximum operating level, discussed in Section 4.4.3.

As the inlet/outlet/drain pipe has a bend, pigging and relining options are limited. Instead, the City can consider removing the corrosion with mechanical agitation to the extent possible from the floor of the reservoir and the recoating interior of the pipe. Although it was not possible to view deep into the pipe during the inspection, the extent of the corrosion appears to be limited, so this improvement is a consideration rather than a recommendation.

4.4.3 Structural

To address the roof and foundation issues resulting from the design seismic event, the City has four options. The scale of the required retrofits varies depending on if/how far the maximum operating level of 24.5 feet is reduced. Options 1a and 1b involve leaving the existing roof in place. Option 1b should only be considered if no other options are possible. For each of the following maximum operating levels, the following improvements are needed:

Option 1a (19.75 feet): Options 1a and 1b would first involve an inspection to ensure the roof is able to be retrofitted as corrosion may render retrofits unfeasible. If repairs are feasible, Option 1a would involve removing the existing interior and exterior roof coatings; seal welding the roof plates; repair of roof support joint connections, pits, and weld failures; proper surface preparation; and recoating the interior and exterior roof along with the rest of the reservoir. This work is recommended to be conducted within the next 2 years if the roof is to remain.

Option 1b (20.5 feet): The code-level seismic slosh would impact the knuckle only. Under this maximum operating level, retrofit roof knuckle is required in addition to the requirements of Option 1a. Only the rafter ends would need retrofitting. It should be noted this will not meet current code since there will not be enough freeboard. However, the potential for damage would be reduced.

Option 2 (22.25 feet): This option involves replacement of the existing roof. As the reservoir is relatively narrow, a new self-supported dome roof is recommended. Although roof retrofits may be possible, the corrosion extent on the interior has likely rendered the roof unfit for any such retrofit. Should full roof retrofits warrant further investigation, an in-depth structural analysis of the roof would first need to be conducted. Reinforcing may be possible, or the City could add another shell course and raise the existing roof. These options are projected to be more expensive and have less longevity, compared to a new roof. Installation of new roof eliminates the need to seal weld, blast, and recoat the exiting roof.

Option 3 (24.5 feet): Above the 22.25-foot operating level, foundation retrofits are also required. In addition to the requirements of Option 2, if the City retains the 24.5-foot operating level, the foundation will need to be expanded to handle to increased bearing loads from the slosh induced by the design seismic event. Further geotechnical investigation is needed to confirm the additional foundation area. For budgetary purposes, a foundation expansion and micropiles are explored to meet these requirements.

To be commensurate with the 65 percent (4,000 to 6,600 psf) exceedance of the bearing pressure, the foundation footprint can be expected to be increased by about this percentage. The 12-inch wide ring footing centered on the tank has an area of approximately 185 ft^2. A 65 percent increase in the area would equate to an additional 120 ft^2 of footing area, totaling 306 ft^2. Similar seismic retrofit projects have used micropiles at 5 feet on center throughout the reservoir's circumference.

During any retrofits, the City may consider lowering the overflow to ensure the water level in the reservoir does not surpass the maximum allowable water level.

4.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. Only a minor adjustment, such as a gasket, is needed to remedy the silt stop on the Padden Reservoir. Water quality risks can also be reduced by updating the drain and overflow pipes screens to a #24-mesh with a 4-mesh screen backing. The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area.

4.4.5 Safety

To address the safety concerns of the Padden Reservoir, the exterior fall protection should be upgraded before the 2036 deadline. The City may consider upgrading the exterior ladder concurrently with other retrofits. Furthermore, the roof perimeter and roof entry hatch need suitable fall protection. If the interior ladder is to be used, an OSHA-complaint fall protection system will need to be installed and the entry hatch increased in size to 30 inches. Given the condition of the ladder, replacement would likely be necessary. It does not appear the ladder is currently in use. Finally, a new 30-inch manway should be installed on the reservoir's first course to meet AWWA best practices.

4.4.6 Operations and Maintenance

4.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

4.4.6.2 Roof Drainage

There are no recommendations on roof drainage on this reservoir.

4.4.6.3 Appurtenances

The air exchange through the vent should continue to be monitored as the vent is not centered. Updating the location can be considered during future upgrades of the reservoir to follow current AWWA recommendations.

4.4.6.4 Valving and Piping

The energy dissipation and dechlorination systems should be updated before the next time the reservoir is drained. Proper dechlorination at the Padden site could consist of a weighted system that is set in the manhole on site or a sturdy system that could be installed on the outlet pipe at Padden Creek. An example of a basket system is shown in Figure 4-21.

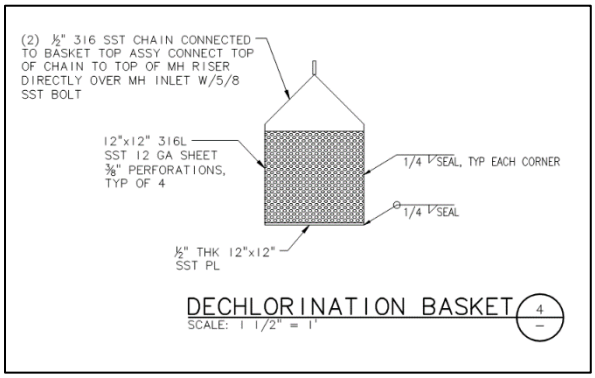

Figure 4-21: An example of a dechlorination system

4.4.6.5 Misc.

To extend the life of the coating below the waterline in the interior, the existing constant voltage rectifier should be replaced with an auto-potential unit.

4.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Padden Reservoir are show in Table 4-3. This table is organized such that any roof repairs coincide with structural repairs, dependent on the updated maximum operating level. Wall and floor costs are assumed to be independent of the roof retrofits, so are included in the Cleanliness and Coating and Material Deterioration Sections.

Table 4-3: Padden deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 2,000
	Exterior coating failure (walls)	Abrasively blast exterior and recoat	$ 89,000
	Interior coating thin (walls and floor)	Abrasively blast interior and recoat	$ 118,000
DETERIOR-ATION	Grout layer failure	Remove, fill gaps w/ non-shrink grout. Non-grout-able areas: fill w/ backer & caulking	$ 6,000
	Unforeseen corrosion-related pits and weld failures (walls and floor)	Unforeseen corrosion-induced pit and weld failure repairs (walls and floor)	$ 9,000
	Corrosion on interior of piping	Monitor. Consider cleaning and recoating	$ -
STRUCT & ROOF	Interior and exterior roof coating is failing and structural members are corroding. Roof impacted by seismic slosh and soil bearing capacity is exceeded	OPTION 1: Retain Existing Roof	
		a. Roof structural inspection	$ 3,000
		b. Abrasively blast exterior roof and recoat after repairs	$ 52,000
		c. Abrasively blast interior roof and recoat after repairs	$ 44,000
		d. Seal weld roof plates	$ 9,000
		e. Unforeseen roof joint connection repairs	$ 6,000
		f. Unforeseen roof pit and weld failure repairs	$ 4,000
		(1a) Lower max operating level from 24.5 to 19.75 ft	
		Subtotal for Option 1a	**$ 118,000**
		(1b) Lower max operating level from 24.5 to 20.5 ft	
		g. Retrofit roof knuckle.	$ 60,000
		Subtotal for Option 1b	**$ 178,000**
		OPTION 2: Lower max operating level from 24.5 to 22.25 ft	
		Demo/remove existing roof. Install new self supported dome roof	$ 140,000
		Subtotal for Option 2	**$ 140,000**
		OPTION 3: Maintain 24.5 ft operating level	
		a. Demo/remove existing roof. Install new self supported dome roof	$ 140,000
		b. Foundation improvements	$ 160,000
		Subtotal for Option 3	**$ 300,000**
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	Silt-stop not functioning	Retrofit silt-stop	$ 1,000
	Drain pipe screen non-compliant	Replace drain pipe screen	$ 1,000
	Overflow pipe screen non-compliant	Replace overflow pipe screen	$ 1,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
SAFETY	Exterior ladder fall protection is cage	Replace exterior ladder fall protection by 2036	$ 6,000
	Lacks roof fall protection and roof hatch railing	Install roof fall protection and roof hatch railing	$ 44,000
	Lacks interior ladder fall protection. Corrosion on interior ladder renders it unsafe.	If used, replace ladder	$ -
	Missing second side access hatch	Install new 30" side access hatch	$ 16,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 20,000
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000
	Cathodic protection system is constant voltage model and lacks stationary reference electrode	Install automatically controlled potential rectifier and stationary reference electrode	$ 10,000

	Max operating Level (ft)			
	19.75	20.5	22.25	24.5
Subtotal	$ 446,000	$ 506,000	$ 468,000	$ 628,000
30% Contingency	$ 133,800	$ 151,800	$ 140,400	$ 188,400
8.7% Tax	$ 50,443	$ 57,229	$ 52,931	$ 71,027
Engineering, Administration, and Construction Management	$ 220,585	$ 250,260	$ 231,466	$ 310,599
Total Project Costs Estimate	$ 850,828	$ 965,289	$ 892,797	$ 1,198,026
Total Project Costs Estimate (rounded)	**$ 850,000**	**$ 970,000**	**$ 890,000**	**$ 1,200,000**

4.5 Conclusion

The Padden Reservoir has major issues that include coating failure on both the interior and exterior, heavy interior corrosion above the waterline, and a foundation and roof that do need meet seismic code at the current operating level. With the reservoir being built in 1958, it is likely nearing the end of its service life. The City should determine what maximum level the reservoir can be operated at while meeting requirements and needs of the 457 South Zone. If the maximum operating level the reservoir cannot be reduced, the estimated costs of retrofits appear to approach the cost of building a new reservoir.

As the corrosion is recommended to be dealt with within two years, we recommend this tank be inspected next after this work is complete. It is recommended the cathodic protection system be inspected by a corrosion engineer on a yearly basis. The cathodic protection system inspections entail checking the rectifier output and measuring the structure-to-electrolyte potential in the water at 2-foot increments from the roof hatch while the reservoir is in operation.

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Section 5

Whatcom Falls I Reservoir

5.1 Tank and Site Overview

The Whatcom Falls I Reservoir is a 200-foot diameter above-ground welded steel cylinder with a 4.0-MG capacity, shown in Figure 5-1. Construction was completed in 1982 and as-built plans were published in 1984. The reservoir shell plates consist of two courses and are a total height of 17 feet, 6 inches. There is no overflow on the reservoir, and it runs on gravity with the Whatcom Falls II Reservoir. The roof's rafters are supported by two rows of girders. The girders are supported by one centrally located 8-inch diameter column and two rings of (16) 5-inch diameter columns at radii of 38 and 71 feet. The 6-foot by 30-foot roof plates are approximately 0.188 inches thick and are sloped at an angle of 0.75:12. According to the provided drawings, a 24-inch wide by 3.5-foot deep ring wall footing supports the reservoir without any anchors. The piping is shown to run below the base of the footing and encased in lean concrete fill. The reservoir is operated between 7 and 16.5 feet the summer and between 6.5 and 15.5 feet in the winter, as noted by the City.

Figure 5-1: The Whatcom Falls I Reservoir, viewed looking northwest.

The site is centrally located within the distribution area, about two miles east of the Bellingham CBD. The site also includes the Whatcom Falls II reservoir and a pump station, which are not close enough to impact the structure during a seismic event. The reservoir serves the 276 South Zone.

The geotechnical investigation indicated that the site has 1.5 to 3 feet of forest duff/topsoil, glacial till, and Chuckanut sandstone. The foundation extends to the sandstone layer. Perched groundwater seepage was found above the till layer.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. The bedrock the reservoir is founded on has an allowable bearing pressure of 6,000 psf and bedrock is not at risk of liquification. This site has a low risk of slope instability.

5.2 Inspection Summary

Floating and drained inspections were conducted on June 12 and November 30, 2019, respectively.

5.2.1 Exterior Inspection Summary

5.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the Whatcom Falls I and II Reservoirs. The fence is 15 to 16 feet from the shell on the east, west, and north sides of the reservoir. The site is generally well-graded, directing runoff away from the reservoir. Tall trees were found growing just beyond the fence.

5.2.1.2 Exterior Walls

Heavy organic buildup was noted on the exterior walls throughout the reservoir (Figure 5-2), especially on the northeast side. There were areas of graffiti that had been covered up. Some of these areas are peeling. There were scratches (Figure 5-3 Left) and small areas of missing coating (Figure 5-3 Center). Heavier organic staining was noted near the roof drains (Figure 5-3 Right). The coating system consists of a red primer and green topcoat. Coating DFT was measured to be 5.8–11.5 mils, averaging 7.6 mils. Typical high-performance coatings for the exterior surfaces of water storage tanks are on the order of 12–16 mils. A field lead check swab tested negative for the presence of lead on the exterior walls.

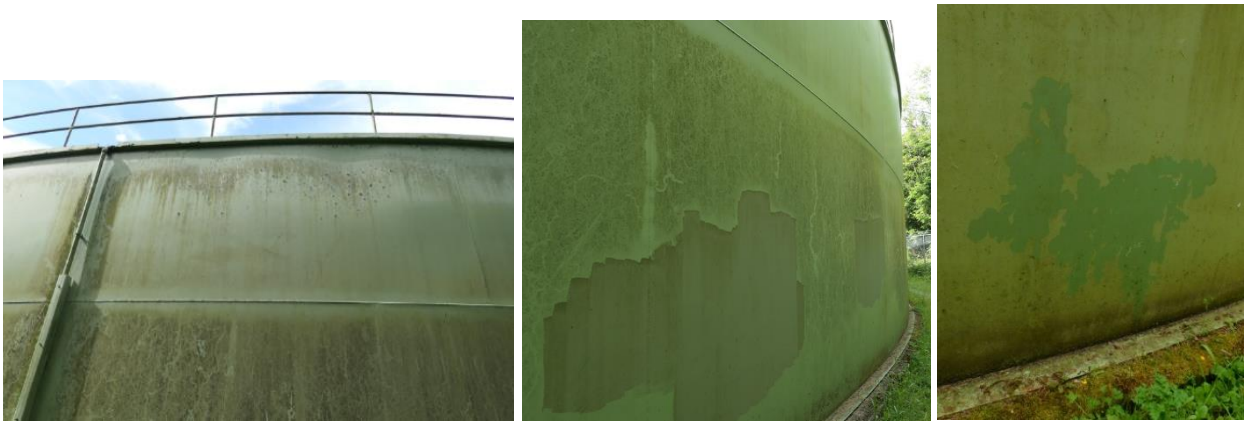

Figure 5-2: Exterior wall topcoat is dirty. Areas of graffiti cover are peeling. Underlying coating is sound.

Figure 5-3: The exterior walls has scratches (Left), areas of missing coating (Center), and heavier organic staining near the roof drains (Right).

5.2.1.3 Foundation

The reservoir's concrete ring foundation exhibited no visible settlement issues or major cracking. However, the grout layer was missing between the floor plate and top of footing, allowing for plants to grow between the structure and the foundation (Figure 5-4). There was some coating loss on the chime.

Figure 5-4: Missing grout at base and plants growing

5.2.1.4 Exterior Roof

Like the walls, significant buildup of organic material was noted on the roof. Larger tree debris was building up especially near the toeboard (Figure 5-5). There were areas of ponding noted where roof plates lapping inhibited drainage. Roof coating DFT measurements ranged from 5.4 to 11.1 mils, averaging 8.0 mils.

Figure 5-5: The roof of the reservoir's minor instances of ponding at the perimeter.

Figure 5-6: Areas of ponding were noted at the margins of roof plates.

5.2.1.5 Exterior Appurtenances

The roof drainage holes are approximately 2 inches in diameter. These drains have accumulated debris (Figure 5-7 Left). The pipes of the downspouts were corroding in a few instances (Figure 5-7 Right).

Figure 5-7: Some roof drains are clogged (Left) and the downspout is corroding (Right).

This tank has two circular side manways with 30-inch diameter openings (Figure 5-8). The exterior ladder has a lockable enclosure (Figure 5-9 Left). The roof railing is 42 inches tall and has a mid-rail 24 inches from the bottom and 18 inches from the top railing. Corrosion and peeling coating were noted on the handrail (Figure 5-10)

Figure 5-8: The 30-inch diameter manways

Figure 5-9: The exterior ladder's lockable enclosure's coating was peeling

Figure 5-10: Coating failure and rusting on the roof handrail

The roof entry hatch is a 30-inch by 36-inch clam style and had an intrusion alarm (Figure 5-11). This lid has an opening that is used for reservoir ventilation. The vented area is approximately 24 inches by 20 inches, and 6 inches tall. The screen openings are about 1/4-inch. The coating was failing on the exterior of the lid.

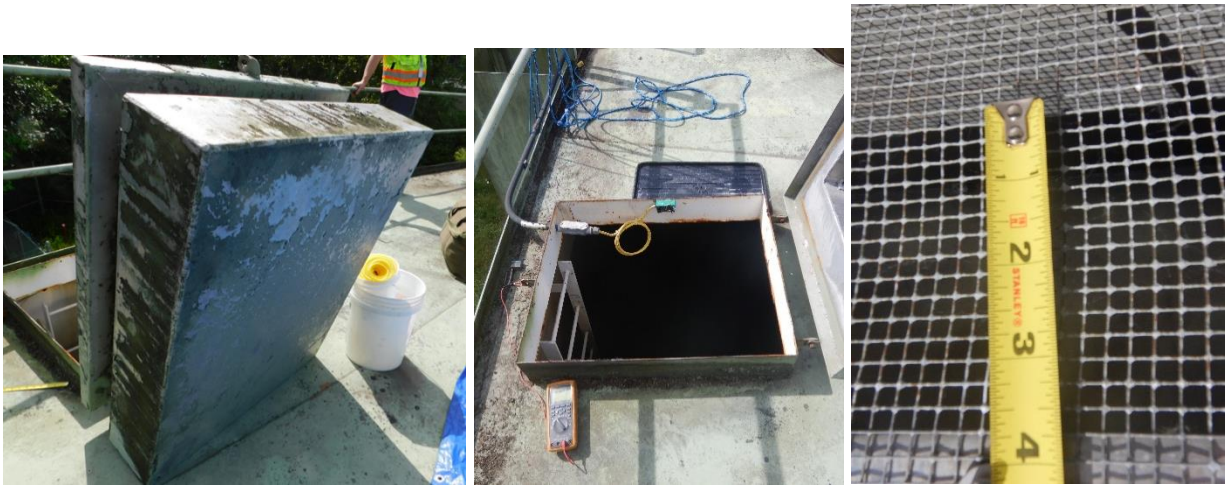

Figure 5-11: The roof hatch coating is peeling.

The roof vent is centrally located and has a roof penetration has a diameter of 8 inches (Figure 5-12). The hood is approximately 16 inches above the roof surface and is 24 inches in diameter. A frame holds the screen in place, reducing the free area air can travel through. The coating was failing on the exterior side of the hood. Heavy corrosion was noted on the underside of the hood and light corrosion was found on the roof below the vent. The screen has 1/4-inch openings.

Figure 5-12: The roof vent has corrosion on the underside and a coarse screen.

5.2.2 Interior Inspection Summary

5.2.2.1 Interior Walls

The interior walls had water staining below the waterline (Figure 5-13). There were minor, isolated instances of blistering (Figure 5-13 and Figure 5-14). The interior coating DFT measurements ranged from 7.3 to 20 mils with only a few small areas of low coating; typical high-performance coating DFT ranges are 12 to 20 mils. Measurements of plate thickness are shown in Table 5-1. Exposed steel locations below the waterline are in very good condition with no observed pitting.

To protect the walls from corrosion, a constant voltage model cathodic protection system is installed. On-site testing indicated that the corrosion protection criteria was being met at all testing locations. However, the stationary reference electrode used to operate the rectifiers auto potential circuitry appears to be failing as its readings are significantly more positive than normal.

It was noted that the rectifier was not functioning. Troubleshooting resulted in noting that the alternating current (AC) breaker for the unit was defective. As a temporary measure, the AC conductors for the rectifier were landed on an adjacent breaker in order to test the cathodic protection equipment. After testing, the rectifier was left terminated to the operating breaker.

A full corrosion report and report of the rectifier and structure-to-electrolyte potential data is included in Appendix C.

Above the waterline, the walls appeared to be impacted by general surface corrosion and crevice corrosion.

Figure 5-13: Interior walls exhibited water discoloration (Left) and minor blistering (Right).

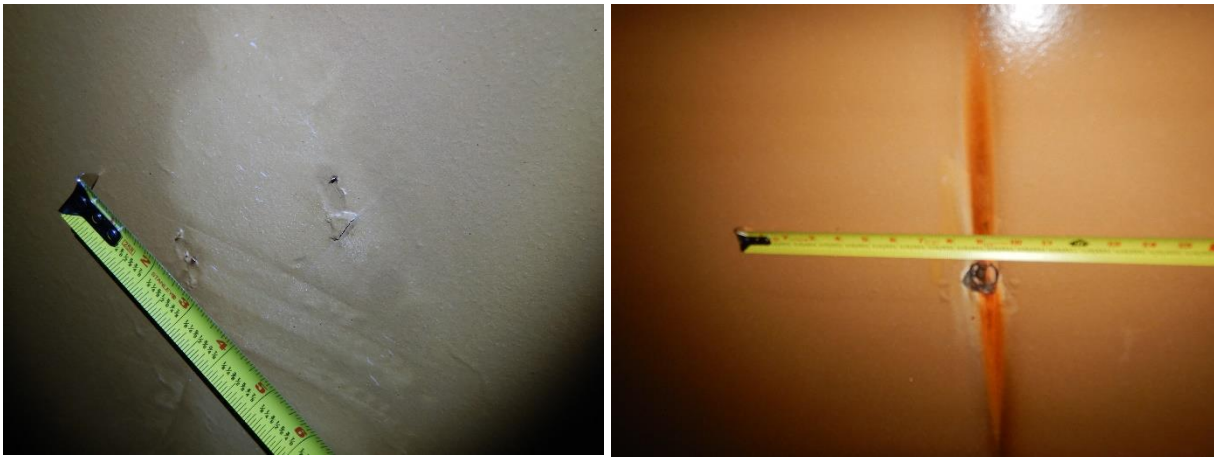

Figure 5-14: Minor isolated incidents of coating failure below the waterline

Table 5-1: Steel Plate Thicknesses

Location	Design Thickness (in)	Measured Thickness (in)
Roof	0.188	0.188
Top Course	0.375	0.397
Bottom Course	0.438	0.465
Chime plate	0.250	0.252

5.2.2.2 Floor

The coating on the floor (Figure 5-13 Left) was sound other than a few areas of bubbling (Figure 5-15). Measurements of coating DFT ranged from 5.5 to 14.5 mils on the floor. Some sediment was found near the base of the column.

Figure 5-15: Coating blisters on the floor.

5.2.2.3 Interior Roof and Columns

The interior roof exhibited light rust staining on the panels (Figure 5-16). The panels were deflecting at the edges and heavy staining correlated with areas of ponding on the exterior roof (Figure 5-17). The roof shows rust staining at all roof plate overlaps, typical for these non-seal welded areas. The structural components surrounding the access hatch and areas above the water level are experiencing general surface corrosion and crevice corrosion at steel overlap areas. Cathodic protection hangers were also rusting and in one instance, had fallen (Figure 5-18).

The columns exhibit similar rust staining as the walls (Figure 5-19). The bases of these columns have angle guides, which is atypical of water tank construction. These areas exhibited corrosion.

Figure 5-16: The roof panel light rust staining (Left and Center) and fasteners corrosion (Right).

Figure 5-17: Panels rusting, deflecting, (Left) and staining (Center). Support members also exhibited corrosion (Right)

Figure 5-18: The cathodic protection system holders are rusting (Left) and one has completely failed causing the anode to be supported only by its wires (Right).

Figure 5-19: Columns and column bases

5.2.2.4 Interior Appurtenances

The interior ladder did not have any fall protection devices. Like the walls, the ladder exhibited water staining below the waterline (Figure 5-20).

Figure 5-20: The interior ladder water staining below the waterline

5.2.3 Piping and Valving Inspection Summary

The inlet, outlet, and drain pipes are combined in a single 24-inch diameter pipe (Figure 5-21). This pipe is equipped with a silt stop, which was removed during the inspection and replaced after completion. It did not appear to sit flush with the floor. The interior of the pipe exhibited minor corrosion nodules. The drain piping was observed to have insect and bird screens (Figure 5-22). While the provided cover sheet indicates that the reservoir drains to sanitary sewer, staff reported that the reservoir drains to Whatcom Creek (Figure 5-23). The dechlorination system was not observed. This reservoir has no overflow.

Figure 5-21: The common inlet/outlet/drain with retrofitted silt stop

Figure 5-22: Drain piping (Left) and screen (Right).

Figure 5-23: Drain pipe outlet

Washdown operations can occur via a fire hydrant near the exterior ladder (Figure 5-9).

5.3 Condition Assessments

5.3.1 Cleanliness and Coatings

The exterior surface of the tank is dirty and has several areas of organic material accumulation on the roof and sidewalls. Specifically, the roof appeared to be hold significant organic debris, which is blocking a few the drains. After cleaning, the walls and roof will be in good condition. While the exterior coating is less thick than the it would be new, it is still performing. The instances of scratches and nicks are relatively minor. The coating loss on the exterior aluminum roof access hatch and ladder compartment are likely due to improper surface preparation and/or coating type. Typically, these appurtenances are left uncoated as they are non-corrosive if they are isolated from different metals. The railing and vent, which exhibited coating failure were assessed to be in fair and poor condition, respectfully.

The interior of the reservoir was clean. However, the coating on the interior roof is failing on most of the support beam edges, edges of steel plating, and areas of roof plate overlap. The coating appears to be generally intact below the waterline.

Overall, the cleanliness and coatings are assessed to be in fair condition.

5.3.2 Material Deterioration

The seal between the foundation and tank was found to be at the end of its useful life and is in poor condition.

The exterior of the shell is not corroding and is still in good condition. The railing and vent, which exhibited coating failure are experiencing corrosion and were assessed to be in fair and poor condition, respectfully. The vent's corrosion is occurring as the bare carbon steel is reacting with the bare stainless steel mesh resulting in galvanic corrosion.

On the interior, the rust colored water staining appears to be a cosmetic issue currently as steel thickness requirements are still being met. The areas of wall and floor corrosion are minor, so are overall in good condition. However, on the roof, corrosion is evident on most of the support beam edges, edges of steel plating, areas of plate overlap. These sites are prone to corrosion if a stripe coating is not applied during the coating process. As wet coatings cure, they tend to pull back from the corners and edges exposing the underlying steel. The application of a stripe coat provides and additional layer of coating thickness and protection against these losses. The roof's rust staining at all roof plate overlaps is due to the plates not being seal welded. Overall, the roof is in poor condition related to corrosion. To maintain the reservoir's structural integrity, the coating loss and corrosion should be addressed within the next five years.

5.3.3 Structural Performance

5.3.3.1 Static Analysis

Based on drawings and measurements, the roof framing, the roof plate thicknesses, rafters, and columns were all sized appropriately. However, corrosion issues can jeopardize roofs. Depending on the system hydraulics, without an internal overflow, this reservoir may be operated up to the roof level or even higher, causing roof damage. At the roof level of 17.5 feet, the shell was overstressed by only 2.5 percent. However, at the current maximum operating of 16.5 feet, the stresses were found to be within acceptable levels. The foundation is sized appropriately based on the findings of the geotechnical report. Other than the missing overflow, from a static structural perspective, the reservoir is in good condition.

5.3.3.2 Seismic Analysis

The reservoir's shell is adequate for the site's anticipated seismic loads. The slosh wave in this reservoir is estimated to be 3.6 feet high, based on the design seismic event. As a result, the operating height would need to be reduced to 14 feet to alleviate all slosh related loads on the roof components and roof rafter supports. The roof plates are not able to resist slosh wave-induced forces and would need to be welded to the rafters if impacted. Operating at a 16.5-foot

level would exceed the rafter's capacity. This reservoir was determined to be stable against overturning. At the current operating level, the reservoir is in poor seismic condition.

5.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality, none have been reported. The diameter of this reservoir coupled with the combined inlet/outlet configuration are very likely causing improper mixing and stagnation within the Whatcom Falls I Reservoir. Often, diagnosing these issues requires robust sampling; no records of such sampling were supplied. To determine if adequate mixing is occurring, the City following measurements can be used:

- typical operational cycling time,
- time taken to fill and draw from minimum to maximum depths,
- time to turn over tank.

A functioning silt stop is also a current DOH best practice; this reservoir's silt stop appears to need a slight modification. Additionally, the screen on the vent and hatch were found to be out of compliance as they were not #24 mesh. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gaskets to provide a tight seal against the relatively small surface area of the frame. The possibility of the roof leaking presents a minor risk to water quality as well. Due to these issues, the reservoir is in poor water quality condition.

5.3.5 Safety

Both the interior and exterior ladders are less than 24 feet high. Although no fall protection systems are required on these ladders, many shorter ladders have fall protection on them as an added safety measure. The perimeter roof railing is compliant. The roof hatch requires a roof hatch railing or temporary barrier. From a safety perspective, the reservoir is in good condition.

5.3.6 Operations and Maintenance

5.3.6.1 Site and Security

Based on the visible signs of vandalism, the fence at this site is not fully suitable to deter illicit access. However, an intrusion alarm is installed on the roof's access hatch. The soils adjacent to the reservoir were draining properly. Tall trees were found to be too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, and are causing organic material accumulation problems. The site/security is in fair condition.

5.3.6.2 Roof Drainage

The roof's steel plates were lapped in a fashion the allows water to pond. Also, the roof drains appear to be undersized for the amount of organic material that accumulates on the roof. While the blocked drains do not appear to have major negative impacts the structure yet, the condition appears to be affecting the shell coating. Based on our observation of ponding on the exterior roof surface and staining on the interior, a small amount of water may be seeping into the reservoir. Despite this minor evidence, the roof appears to be generally functioning as designed and in fair condition.

5.3.6.3 Appurtenances

No difficulties were found using the exterior ladder or hatches. Whatcom Falls I's vent met design requirements for roof penetration area and screened area with the existing 1/4-inch screen. However, the further constrained free area of the recommended #24 mesh would cause the vent to be undersized.

5.3.6.4 Valving and Piping

The staff did not note any operational difficulties associated with this reservoir. This reservoir drains to Whatcom Creek, so dechlorination and energy dissipation are required. Without an internal overflow, this reservoir would not meet the requirements of AWWA D100-11. In the event of an unforeseen problem, the water level within the zone may exceed the roof level and cause structural damage to the roof. Alternatively, system hydraulics can be further investigated if the City has preference on omitting the overflow.

5.3.6.5 Misc.

The currently inoperable cathodic protection system is in poor condition and needs to be serviced. While the rectifier unit is auto-potential, which accounts for fluctuations in water level, the defective AC breaker meant that the unit has not been turned on/functioning. The anode that is hanging by its wire could cause a short circuit if it touches the floor and needs attention immediately, ideally prior to or concurrent with replacing the AC breaker.

5.3.7 Obsolescence

Overall, the equipment on site appeared current and could be replaced in the event of failure.

5.3.8 Condition Scoring

Overall, the condition score of the Whatcom Falls I Reservoir was fair to good, at 3.8. It was primarily affected by the seismic score. Operated at above 14 feet, the roof would be affected by the slosh wave. Other issues include the failing coating and corrosion above the waterline in the

interior, the lack of an overflow pipe, and the cathodic protection system being in disrepair. A summary of the scoring is shown in Table 5-2 and the full Score Matrix is included in Appendix C.

Table 5-2: Condition Scoring of the Whatcom Falls I Reservoir

Categorical Scores	
Cleanliness and Coatings	3.5
Material Deterioration	4.2
Structural Performance - Static	4.0
Structural Performance - Seismic	3.5
WQ/ Sanitary	2.5
Safety	4.5
Operations & Maintenance	4.0
Obsolescence	4.7
Overall	3.8

5.4 Recommended Improvements

5.4.1 Cleanliness and Coatings

First, the exterior surfaces need to be pressure washed to remove dirt and other debris. On the exterior roof, the accumulated debris should be removed, and drains unclogged. The maintenance frequency of clearing debris from the roof is recommended to be increased until the tree spacing recommendation is met. The coating failures on the exterior walls and chime can be addressed with spot coating. The City can consider a full surface preparation and recoat of the chime as well. Exterior roof coating recommendations are dependent on the structural retrofits, outlined in Section 5.4.3.

For the aluminum exterior appurtenances, such as the ladder and roof entry hatch, the areas in contact with the roof should be checked to ensure they are not creating a metallic couple. If they are not, the coating can be removed from the surface and left as is. The steel appurtenances, such as the railing, roof vent, and drain downspouts can be addressed by removing the existing coating, preparing the surface with abrasive blasting, and recoating.

The ideal method of dealing with the intermittent corrosion below the waterline on the interior is to remove the loosely adhered coating material, exposing the underlying steel. The steel should then be abraded to provide a sufficient surface anchor profile. One to two inches of sound coating surrounding the defect should be scuffed to provide a rough surface per the coating manufacturer's recommendation. A repair coating, NSF-61 rated, should then be applied at a thickness of 8 – 10 mils in one or two coats. The discoloration on the walls can be addressed by agitation and/or detergent if the City wishes. The improvement for coating issues on the interior roof is dependent on the selected maximum operating level, discussed in Section 5.4.3.

5.4.2 Material Deterioration

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

Any corrosion on the walls and floor can be addressed through spot treatment, outlined in Section 5.4.1. The improvement for corrosion issues on the interior and exterior roof is dependent on the selected maximum operating level, discussed in Section 5.4.3.

5.4.3 Structural

To address the roof issues resulting from the design seismic event, the City needs to lower the operating level to 14.0 feet or consider major retrofits. No effective intermediate storage volume exists as the roof is has a large diameter and does not have knuckle. The City should consider Options 1a and 2, with Options 1b and 1c only being considered if the City cannot employ Options 1a or 2. This work is recommended to be conducted within the next 5 years. For each of the following maximum operating levels, the following improvements are needed:

Option 1a (14.0 feet): Options 1a, 1b, and 1c would first involve an evaluation to ensure the roof is able to be retrofitted as corrosion may render this unfeasible. If repairs are feasible, Option 1a would involve removing the existing interior roof coating; seal welding the roof plates; repair of roof support joint connections, pits, and weld failures; proper surface preparation; recoating the interior roof; and spot treating the exterior roof coating.

Option 1b (14.5 feet): The code-level seismic slosh would impact only the exterior 10 feet of roof. The structure must first pass the retrofit evaluation to check the roof can be seal welded and allow the plates to be welded to the rafters. In addition to the requirements of Option 1a, the outer 10 feet of roof would need to be retrofitted by welding the roof plates to the rafters. It should be noted this will not meet current code since there will not be enough freeboard. However, the potential for damage would be reduced.

Option 1c (15.5 feet): The code-level seismic slosh would impact only the exterior 20 feet of roof. The structure must first pass the retrofit evaluation requirements of Option 1b, and the outer columns need to be found suitable for retrofitting. In addition to the requirements of Option 1a, the outer 20 feet of roof can be retrofitted by welding the roof plates to the rafters. The outer columns would also need to be retrofitted. It should be noted this will not meet current code since there will not be enough freeboard. However, the potential for damage would be reduced.

Option 2 (16.5 feet): This option would at minimum require the exterior 30 feet of the reservoir roof to be retrofitted. Our cost estimate indicates that this level of retrofitting the roof may be

uneconomical compared to installing a new roof. For planning purposes, a new shell course and new conventional roof can be considered. This reservoir is too wide to have a self-supported roof, so a conventional roof is required. This option eliminates the need to seal weld, abrasively blast, and recoat the exiting roof.

Should the reservoir continue operating without an overflow, a maximum water elevation of 17.5 feet would impact the outer 40 feet of the roof. During any retrofits, the City may consider lowering the overflow to ensure water level in the reservoir is not brought higher than the maximum allowable water level, depending on how this will impact other reservoirs in the 276 North Zone.

5.4.4 Water Quality/Sanitary

Robust Water quality monitoring is recommended in the reservoir as the common inlet/outlet/drain configuration and reservoir diameter are likely causing water stagnation. Separation of the inlet and outlet is likely, but other options can be explored during the design phase. Retrofits would likely resemble those implemented at the Marietta Reservoir where the inlet pipe extended to the other side of the reservoir and elevated. For budgetary purposes new piping is assumed.

Replacing the gasket will remedy the silt stop on the Whatcom Falls I Reservoir. Water quality risks can also be reduced by updating the screens to a #24-mesh with a 4-mesh screen backing on the roof vents, in accordance with DOH best practices. This will likely be addressed with a new vent, per Section 5.4.6.3. The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area.

5.4.5 Safety

The City can consider installing fall protection on the interior and exterior ladders, although this improvement is not mandated by L&I. As the access hatch is blocked on one side and has a railing on the other, a temporary railing would be a good option when work is conducted on the roof for this reservoir.

5.4.6 Operations and Maintenance

5.4.6.1 Site and Security

The security can be improved with fence retrofits, motion detectors, flood lights, and/or cameras. Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view. Because the reservoir is located in a park, removal of trees may not be feasible.

5.4.6.2 Roof Drainage

The roof drainage should continue to be monitored and maintenance frequency increased until the tree spacing requirement is met, or indefinitely if it cannot be met. The City may wish to upsize and retrofit these drains to account for the organic material load they are currently subjected to.

5.4.6.3 Appurtenances

A new vent system is recommended for the Whatcom Falls I reservoir. The penetration area needs to be approximately 3.0 square feet with a screened area of approximately 6.9 square feet to account allow for pressure relief. This could be achieved with a new centrally located, upsized vent or additional vent(s). Additional vents may also allow this wide reservoir to vent water vapor more efficiently. "Vent openings should never be… installed as an integral part of the roof hatch access structure." (DOH 2019), so the hatch vent should be removed.

5.4.6.4 Valving and Piping

The energy dissipation and dechlorination systems should be updated before the next time the reservoir is drained. Proper dechlorination at the Whatcom Falls I site could consist of a weighted system that is set in the manhole on site or a sturdy system that could be installed on the outlet pipe at Whatcom Creek. An example of a basket system is shown in Figure 4-21. The addition of an overflow is also recommended to meet AWWA requirements or system hydraulics confirmed.

5.4.6.5 Misc.

To extend the life of the coating below the waterline in the interior, the AC breaker should be repaired, a reference electrode installed, and the defective anode hanger replaced.

5.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Whatcom Falls I Reservoir are shown Table 5-3. This table is organized such that any roof repairs coincide with structural repairs, dependent on the updated maximum operating level. Wall and floor costs are assumed to be independent of the roof retrofits, so are included in the Cleanliness and Coating Section.

Table 5-3: Whatcom Falls I deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 2,000
	Organic material blocking roof drains	Remove organic material, clear drains, and increase maintenance frequency until tree spacing requirement is met	$ 2,000
	Intermittent exterior coating failure (walls and chime)	Spot Repair Coating	$ 7,000
	Exterior appurtenance coating failure	Remove coating on appurtenances. Prepare surface and recoat steel appurtenances	$ 10,000
	Intermittent interior wall and floor coating failure	Spot Repair Coating	$ 10,000
DETERIOR-ATION	Grout layer failure	Remove, fill gaps w/ non-shrink grout. Non-grout-able areas: fill w/ backer & caulking	$ 18,000
STRUCT & ROOF	Interior roof coating is failing and structural members are corroding. Roof impacted by seismic slosh. Intermittent exterior roof coating failure.	OPTION 1: Retain Existing Roof	
		a. Roof structural inspection	$ 3,000
		b. Abrasively blast interior roof and recoat after repairs	$ 503,000
		c. Seal weld roof plates.	$ 95,000
		d. Unforeseen roof joint connection repairs	$ 61,000
		e. Unforeseen roof pit and weld failure repairs	$ 24,000
		f. Spot repair exterior roof coating	$ 10,000
		(1a) Lower max operating level from 16.5 to 14.0 ft	
		Subtotal for Option 1a	**$ 696,000**
		(1b) Lower max operating level from 16.5 to 14.5 ft	
		g. Retrofit exterior 10 feet of roof	$ 230,000
		Subtotal for Option 1b	**$ 926,000**
		(1c) Lower max operating level from 16.5 to 15.5 ft	
		g. Retrofit exterior 20 feet of roof	$ 440,000
		h. Retrofit outer columns	$ 64,000
		Subtotal for Option 1c	**$ 1,200,000**
		OPTION 2: Maintain 16.5 ft operating level	
		a. Demo/remove old roof	$ 270,000
		b. New shell course	$ 130,000
		c. New Conventional roof	$ 1,100,000
		Subtotal for Option 2	**$ 1,500,000**
WQ	Inlet and outlet are combined and reservoir has large diameter leading to probable water stagnation	Improve inlet and outlet configuration, install hydrodynamic mixing system, associated piping improvements	$ 230,000
	Silt-stop not functioning	Retrofit silt-stop	$ 1,000
	Roof access hatch screen and vent screen non-compliant	Replace screens (addressed with new vent)	$ -
	Roof hatch is a high-maintenance design (small contact area). Hatch has screen.	Replace gasket w/ neoprene seals; removed screen portion and consider replacement	$ 1,000
SAFETY	Lacks roof hatch railing(s) or temporary barrier	Install roof hatch railing(s) or use temporary barrier	$ 1,000
O&M	Evidence of vandalism	Improve site security (w/ Whatcom Falls II)	$ 4,000
	Tree spacing recommendation not met	Tree removal and restoration	$ 33,000
	Poor roof drainage	Consider upgrades to downspouts	$ -
	Vent undersized	Replace roof vent	$ 15,000
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000
	Lacks internal overflow	Install overflow	$ 44,000
	Anode hanger defective	Replace anode hanger	$ 1,000
	Stationary reference electrode failing and rectifier's AC breaker defective	Replace reference electrode and AC breaker	$ 1,000

	Max operating Level (ft)			
	14	14.5	15.5	16.5
Subtotal	$ 1,080,000	$ 1,310,000	$ 1,584,000	$ 1,884,000
30% Contingency	$ 324,000	$ 393,000	$ 475,200	$ 565,200
8.7% Tax	$ 122,148	$ 148,161	$ 179,150	$ 213,080
Engineering, Administration, and Construction Management	$ 534,152	$ 647,906	$ 783,423	$ 931,798
Total Project Costs Estimate	$ 2,060,300	$ 2,499,067	$ 3,021,773	$ 3,594,079
Total Project Costs Estimate (rounded)	**$ 2,100,000**	**$ 2,500,000**	**$ 3,000,000**	**$ 3,600,000**

5.5 Conclusion

The most pressing issue with this reservoir it does not meet current seismic code at the current maximum operating level due to slosh impacting the roof. Retrofitting this wide reservoir would be a significant cost, so the City should evaluate if the maximum operating level can be reduced and how this will affect the 276 North Zone.

We recommend a full washout inspection of this tank next in 3 years. The roof should be inspected, and drains cleared frequently, especially in the fall, until the tree spacing recommendation is met. It is recommended the cathodic protection system be inspected by a corrosion engineer on a yearly basis. The cathodic protection system inspections entail checking the rectifier output and measuring the structure-to-electrolyte potential in the water at 2-foot increments from the roof hatch while the reservoir is in operation.

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Section 6

Dakin II Reservoir

6.1 Tank and Site Overview

The Dakin II Reservoir is a 68-foot diameter, partially buried, strand wrapped prestressed concrete reservoir (Figure 6-1). It has a listed storage capacity of 0.5 MG and was built in 1990. From floor to bottom of roof, the height is 19.5 feet. The dome roof is 4 inches thick at the center and has a thickened edge. Prestressing wire information can be found in the City-provided drawings and in the structural report in Appendix D. The overflow height is listed at 18.5 feet, which is 1-foot below the roof-to-wall interface. The operating range is 12 to 15.5 feet. This equates to an operational storage volume of 0.42 MG.

Figure 6-1: The Dakin II Reservoir, viewed looking northeast

The site where the Dakin I and II Reservoirs are located is within a forested area about 3 miles northwest of the Bellingham CBD. It is located within the 519 Dakin and Yew Zone, which is fed from the gravity zone via the Dakin and Yew Booster Pump Station. The Dakin Reservoirs also supply the 730 Alabama Hill Zone via the Balsam Lane Pump Station.

The geotechnical investigation found 15 feet of fill material around the reservoir, which consisted of medium stiff to stiff blue-brown silt with variable amounts of sand, gravel, and organic matter. This layer likely did not extend below the foundation of the reservoir. Chuckanut Sandstone was encountered at 15 feet bgs, which the reservoir is founded on. A thin layer of wood was found at

the interface between the fill and sandstone. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Dakin II Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. This contrasts the as-built design allowable bearing pressure of 4,000 psf. Bedrock is not at risk of liquification. This site has low risk of issues with slope instability.

6.2 Inspection Summary

A drained inspection of the Dakin II Reservoir took place on April 30, 2019.

6.2.1 Exterior Inspection Summary

6.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire is about 12.5 feet from the walls (Figure 6-2 Left). The site is generally well-graded, directing runoff away from the reservoir. Tall trees were growing just outside the fence, contributing organic material accumulation to the roof (Figure 6-2 Right).

Figure 6-2: Perimeter fence and removed graffiti (Left) and leaves accumulating in the curbed roof from the nearby trees (Right)

6.2.1.2 Exterior Walls

The above-grade, visible portion of the exterior walls did not have any major issues. One 3-foot long area of efflorescence on the east side (Figure 6-3 Left). Alligator cracking was noted to encompass large areas, but the individual cracks appeared very minor (Figure 6-3 Right). On-site hammer sounding tests of the outer shotcrete layer indicate it is competent throughout the inspected areas. Graffiti and coverup areas were also noted (Figure 6-2 Left).

Figure 6-3: The exterior walls exhibit minor efflorescence and alligator cracking.

6.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

6.2.1.4 Roof and Curb

The exterior roof of the reservoir exhibited only minor circumferential cracking in the middle of the dome (Figure 6-4). The total slope of the roof ranges from nearly flat at the center to approximately 17 degrees where the roof flattens out. The distance from the roof to ground level ranges from 14 to 21 feet.

Figure 6-4: Reservoir roof and vent

The reservoir wall extends above the roof resulting in a curb. Four roof drains are located along the perimeter but appear to be inadequate to handle the amount of debris which collects along the roof edge (Figure 6-5 Left). When debris were cleared away from drain openings the drainpipes were observed to be clogged. Access ports on the roof curb allow for the walls to be prestressed with steel threadbars (Figure 6-5 Right). The ports are generally filled with high-strength, non-shrink concrete to protect the threadbars from the elements. The seals were found to be missing and stress cracks with efflorescence were forming near some of the ports. Steel threadbars were exposed in eight of the 52 pockets.

Figure 6-5: Roof debris accumulation and blocked drains (Left). Some tensioning access ports on the curb were missing required fill and exhibited cracking with efflorescence (Right).

6.2.1.5 Exterior Appurtenances

A 14-foot exterior aluminum ladder provides access to the reservoir's roof (Figure 6-6 Left). This ladder was found to be missing an anchor point (Figure 6-6 Right).

Figure 6-6: The exterior ladder has a lockable enclosure but is missing one of the anchors.

The access hatch has a gutter inset into the curb (Figure 6-7 Left). A spider was found living inside of the roof hatch (Figure 6-7 Center). A 3-foot square penetration provides air exchange with the 2-foot diameter vent (Figure 6-4 Right). The roof vent's screen appeared to be coarser than #24 mesh (Figure 6-7 Right).

Figure 6-7: Roof access hatch (Left) with insect evidence (Center), and roof vent screen (Right)

6.2.2 Interior Inspection Summary

6.2.2.1 Interior Walls

The lower portions of interior walls were generally did not have observable issues, but mineralization was occurring along the upper portions throughout (Figure 6-8 Left). Many instances of cracking and staining were observed near the roof-to-wall interface (Figure 6-8 Right). There appeared to be organic staining as well.

Figure 6-8: Mineralization/cracking at roof-to-wall interface and overflow

6.2.2.2 Floor

The interior floor slab had no observed cracking or signs of failure. While mineralization observed, it did not appear to affect the structural integrity of the floor (Figure 6-9).

Figure 6-9: Mineralization on the floor

6.2.2.3 Interior Roof

The ceiling had a small area with rust staining (Figure 6-10 Left) and one crack with efflorescence (Figure 6-10 Right).

Figure 6-10: Minor roof rust staining (Left) and efflorescence (Right)

6.2.2.4 Interior Appurtenances

The interior aluminum ladder is anchored near the roof (Figure 6-7 Left) and at the base of the reservoir (Figure 6-11). Similar to the floor, there was mineral buildup (Figure 6-11) on the ladder. The ladder's galvanized cable style fall protection system was severely corroded and inoperable.

Figure 6-11: Mineralization on the aluminum ladder and corrosion on the fall protection system

6.2.3 Piping and Valving Inspection Summary

The combined inlet/outlet/drain piping (Figure 6-12 Left) had sediment accumulating at the bottom of the pipe (Figure 6-12 Center). The silt stop appeared to allow finer particles through; about half an inch of clearance was noted (Figure 6-12 Right). The overflow pipe is corroding at the floor coupling (Figure 6-12 Left), but otherwise did have signs of corrosion (Figure 6-8 Left). The overflow and drain pipes connect to the storm sewer without an air gap. The dechlorination is reported to occur in a storm drain manhole on site.

Figure 6-12: The common inlet/outlet/drain pipe with silt stop

A fire hydrant is located outside of the reservoir and is used in washdown operations. The hydrant had a black bag over it during our inspection.

6.3 Condition Assessments

6.3.1 Cleanliness and Coatings

The interior and exterior are uncoated. The exterior walls are clean. However, the roof has significant debris accumulation, clogging the drains. On the interior, there is minor aggregate accumulation and mineralization. While the interior was generally in good cleanliness condition, but the exterior was in poor condition due to the roof.

6.3.2 Material Deterioration

The exterior walls' efflorescence and alligator cracking instances are minor. Concrete deterioration is occurring on the curb (Figure 6-5 Right) and on the roof-to-wall interface (Figure 6-8). This may be a result of the roof drains not being able to handle the amount of organic material that has accumulated on the roof. Rainwater and snowmelt likely have a high residence time on the roof as the water has no way to efficiently drain. Freeze-thaw cycles of this retained water are detrimental to the roof-to-wall interface joint, causing or contributing to the observed concrete cracking and mineralization in Figure 6-8 (Right). In addition, the cracks in the curb and missing threadbar caps may be related to this issue. Overall, the curb and upper portion of the exterior walls are in poor condition.

Rainwater is naturally acidic and would become more acidic as it dissolves organic material on the roof. This low-pH solution, given time, is likely acidic enough to absorb the calcium carbonate in the concrete. The calcium carbonate would precipitate out of solution once in contact with the higher-pH water in the reservoir. The mineral depositions shown in Figure 6-9 may be the precipitate.

The corrosion and efflorescence issues on the interior roof shown in Figure 6-10, while minor, indicate that water has found its way through the roof slab in these areas. The rust staining indicates the reinforcing members are likely corroding within the roof's reinforcement.

The interior ladder's fall protection system and the base coupling of the overflow pipe are severely corroded. The ladder is made from aluminum and the fall protection system is made from galvanized steel; neither of which is ideal in an underwater environment. Further, having dissimilar metals in contact with one another induces corrosion. Ideally in underwater environmental, both would be constructed from stainless steel (or heavily/frequently coated) to limit corrosion. The overflow pipe coupling and interior ladder system are in poor corrosion-related condition.

6.3.3 Structural Performance

6.3.3.1 Static Analysis

The dome roof was found to meet requirements under AWWA D110-13. For wall reinforcement, the size and spacing of the prestressing was determined to be adequate. An overflow level of 18.5 feet would induce a soil bearing pressure of 2,500 psf. This is suitable based on the 4,000 psf and 6,000 psf allowable soil bearing pressures identified in the design documentation and geotechnical investigation report, respectively.

6.3.3.2 Seismic Analysis

At the 18.5-foot overflow level, the wall reinforcement was found to be suitable for the design seismic event. The strand wrap was found to be slightly out of code compliance at the overflow

level of 18.5 feet but was suitable at the current 15.5-foot maximum operating level. It is anticipated that a code-level seismic event would induce a 3-foot high slosh wave. While the roof would not be impacted at an operating level of 15 feet, the wave would be constrained at the 18.5-foot operating level. During such a constrained slosh event, the roof reinforcing appears to be suitable to protect the roof from damage. The reservoir hatch or other appurtenances may be damaged though, but no significant structural damage of the roof would be anticipated.

6.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A functioning silt stop is also a current DOH best practice; this reservoir's silt stop appears to need some modification. As this reservoir drains and overflows to a storm sewer, DOH approved air gaps are required on both the overflow and the drainpipes. No air gaps are present on either pipe. The vent screen appeared to be coarser than the #24 mesh DOH recommends. Also, the access hatch is considered a high-maintenance-type design under current DOH guidelines as it can collect materials in the channels. A screen is required on the drain outlet, which was not installed. The spider living within the hatch is further evidence that the hatch needs modification. The roof-drainage issues pose a threat to water quality as well as water is apparently infiltrating into the reservoir. With these issues, the reservoir is in fair to poor condition from a water quality/sanitary perspective.

6.3.5 Safety

With the total fall distance from the exterior ladder being less than 24 feet, no exterior ladder fall protection is required. The City may wish to have a working fall protection system on this exterior ladder for safety reasons, however. The missing anchor was a noted deficiency on the exterior ladder. The roof lacks fall perimeter fall protection and a roof hatch railing or temporary barrier.

The total distance from the roof hatch to the floor of the reservoir being less than 24 feet, no fall protection is technically required on the interior ladder. However, in the interest of safety, such a system is desirable. As stated in Section 6.3.2, the ladder and fall protection system should be constructed from stainless steel. Overall, the safety is in fair condtion.

6.3.6 Operations and Maintenance

6.3.6.1 Site and Security

With the evidence of graffiti on the reservoir, the fence may not be suitable at this site. However, an intrusion alarm is installed on the roof's access hatch. The site drainage was found to be adequate. However, tall trees were found growing too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is in fair condition.

6.3.6.2 Roof Drainage

The roof does not appear to be adequate to keep water from entering the reservoir, so the roof drainage is in poor condition.

6.3.6.3 Appurtenances

No difficulties were found using the exterior or interior ladders. The Dakin II Reservoir passed airflow capacity design requirements for the vent's roof penetration and screened area.

6.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. Dechlorination likely needs improvement in this reservoir.

6.3.7 Obsolescence

Overall, the equipment on site appeared to be of newer design and would be easily replaced if needed.

6.3.8 Condition Scoring

Overall, the condition score of the Dakin II Reservoir was good, 4.5. Problems associated with this reservoir were related to the cracking on the roof-to-wall interface and curb, and apparent water infiltration. A summary of the scoring is shown in Table 6-1 and the full the full Score Matrix is included in Appendix D.

Table 6-1: Condition Scoring of the Dakin II Reservoir

Categorical Scores	
Cleanliness and Coatings	3.8
Material Deterioration	3.8
Structural Performance - Static	5.0
Structural Performance - Seismic	5.0
WQ/ Sanitary	3.5
Safety	3.8
Operations & Maintenance	4.4
Obsolescence	5.0
Overall	4.5

6.4 Recommended Improvements

6.4.1 Cleanliness and Coatings

While the exterior walls are generally clear of organic material buildup, the accumulation of organic material on the roof has likely been detrimental to the reservoir. This material should be removed, and the drains cleared as soon as possible to help prevent further damage to the structure. The maintenance frequency of clearing debris from the roof is recommended to be increased until the tree spacing recommendation is met. Roof-drainage and tree spacing improvements are discussed in Section 6.4.6. These improvements should occur prior to or concurrently with fixing the concrete, so the freeze-thaw cycles do not continue to degrade it.

6.4.2 Material Deterioration

The concrete deterioration issue should be dealt with in a timely fashion to keep this relatively new reservoir from experiencing structural issues. Degradation may be slowed if the roof is able to drain properly. The missing threadbar caps and other associated issues in the curb may be a result of the blocked drains and freeze-thaw cycle-induced strain the roof.

The cracks on the curb can be ground down to competent material. Any exposed metal can be coated and then the areas can be patched. For the missing threadbar caps, all the threadbar block-outs should be cleared of concrete. Block-outs should be safely cleaned out (threadbars are under high load) by a competent contractor familiar with prestressed concrete construction. After removal of the concrete, the top of the threadbars should be safely observed by a qualified structural engineer to determine if any additional issues are present which could adversely impact the performance of the vertical threadbar. Once inspected, any vertical threadbar issues should be corrected or repaired. After repair, all exposed surfaces (such as the concrete and the end of the threadbar) should be treated with a corrosion inhibitor/bonding agent (for example: Armetek EpoCem 100). After the corrosion inhibitor/ bonding agent has set, the block-outs should be filled with a high-strength non-shrink grout that does not contain chlorides.

Along the interior roof-to-wall interface, any cracked concrete should be removed, and the areas observed for additional corrosion. Where efflorescence and corrosion are noted, the areas should be removed back to component material. These areas should be coated with a bonding agent and patched. The exterior top and interior bottom of the roof-to-wall interface should be coated to prevent water infiltration through this joint. The areas of corrosion and efflorescence on the interior roof should be ground back, have metal treated, and patched.

The accumulated minerals on the floor may be removed during renovation activities. The corrosion on the overflow pipe coupling can be addressed by cleaning and recoating. Recommended improvements to the interior ladder are discussed in Section 6.4.5.

6.4.3 Structural

There are no structural improvements recommended on the Dakin II Reservoir.

6.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. Only a minor adjustment, such as a gasket, is needed to remedy the silt stop on the Dakin II Reservoir. Air gaps should be installed on the overflow and drain pipe for this reservoir. It may be possible to combine improvements with the improvements to Dakin I, which also requires an air gap on the drain pipe. As the combined drain/overflow pipe is underground, draining the entire reservoir via an air gap may not be possible. Should the site configuration not allow for the reservoir to be fully drained via an air gap, possible options are draining to daylight, an appropriately sized dedicated dry well with backflow prevention, or an air gap with a sump and sump pump that can pump water than cannot be drained by gravity.

The roof access hatch gutter drainage points should be screened to protect them. This will bring them into compliance with the DOH requirements for the sanitary protection or reservoirs. Finally, the vent screen should be replaced with a #24-mesh and a 4-mesh screen backing.

6.4.5 Safety

To address the safety concerns of the Dakin II Reservoir, the anchor should be replaced on the exterior ladder. Care should be taken not to impact the strand wrap of the reservoir during this process. Furthermore, the roof needs suitable fall protection, such as a railing. As the access hatch is blocked on one side and has a railing on the other, a temporary railing would be a good option when work is conducted on the roof for this reservoir.

The interior ladder's fall protection needs to be replaced. As the existing ladder is aluminum, replacement with a stainless-steel system would likely still cause corrosion. The best option would be to replace the entire ladder and fall protection with a stainless steel one. A Safety-Climb System or one of similar design is recommended. Alternatively, the ladder could be coated and re-coated routinely.

6.4.6 Operations and Maintenance

6.4.6.1 Site and Security

The City may wish to update the fence on site to further prevent vandalism. Other options include motion detectors, flood lights, and/or cameras. Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

6.4.6.2 Roof Drainage

Improvement of the roof drainage is essential, especially if the nearby trees cannot be removed to the recommended distance. The number of drainage ports can be increased from four to eight. The pipe size should be increased, and overflows installed should the pipes become blocked. An example of one such design is shown in Figure 6-13.

Figure 6-13: One retrofit option for the roof drainage to combat clogged drains

6.4.6.3 Appurtenances

There are no recommendations for the appurtenances at the Dakin II Reservoir.

6.4.6.4 Valving and Piping

Proper dechlorination of the Dakin II reservoir will need to consist of a weighted system that is set in the manhole on site. An example of a basket system is shown in Figure 4-21.

6.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Dakin II Reservoir are show in Table 6-2 .

Table 6-2: Dakin II deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material blocking roof drains	Remove organic material, clear drains, and increase maintenance frequency until tree spacing requirement is met	$ 2,000
DETERIOR-ATION	Parapet is has cracks and is missing threadbar caps	Clear all threadbars block-outs, inspect threadbars, treat w/ corrosion inhibitor, fill w/non-shrink grout. Clean and patch parapet cracks	$ 55,000
	Cracks & efflorescence at roof-to-wall-interface. Corrosion and efflorescence on underside of roof	Remove cracked concrete, clean & coat reinforcement material, patch concrete, and coat exterior top and interior bottom of the roof-wall-interface	$ 60,000
	Corrosion on overflow pipe coupling	Clean and (re)coat	$ 1,000
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	Silt-stop not functioning	Retrofit silt-stop	$ 1,000
	No overflow or drain air gap	Replace overflow and drain pipe	$ 132,000
	Roof hatch is a high-maintenance design (gutter type)	Add screen to access hatch gutter	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Missing anchor on ext. ladder	Replace anchor	$ 1,000
	Lacks roof fall protection and roof hatch railing	Install roof fall protection and roof hatch railing	$ 43,000
	Interior ladder fall protection unusable	Replace interior aluminum ladder/ galvanized steel fall protection with stainless steel ones	$ 30,000
O&M	Evidence of vandalism	Improve site security	$ 4,000
	Tree spacing recommendation not met	Tree removal and restoration	$ 22,000
	Poor roof drainage	Add four drains, increase drain size, and add overflows	$ 32,000
	Dechlorination non-compliant	Update dechlorination system	$ 2,000
		Subtotal	$ 387,000
		30% Contingency	$ 116,100
		8.7% Tax	$ 43,770
		Engineering, Administration, and Construction Management	$ 191,404
		Total Project Costs Estimate	$ 738,274
		Total Project Costs Estimate (rounded)	**$ 740,000**

6.5 Conclusion

The most pressing issue with this reservoir is the inhibited roof drainage which has likely led to concrete issues at the roof-to-wall interface. Without correction, this issue will likely continue to degrade the reservoir. Removal of the organic material should occur immediately. Before structural retrofits and tree removal are complete, we recommend the roof and drains be inspected and cleared of organic material every 3 months and more frequently than that during the fall. After the roof drainage is improved and concrete repaired, the interior roof-to-wall interface should be inspected within 1 year to ensure the patch is functioning. The next washdown inspection should take place 5 years after the patch inspection. The reservoir should be leak tested as a first step of the next inspection.

Section 7

Kearney Reservoir

7.1 Tank and Site Overview

The Kearney Reservoir is a 130-foot diameter, partially buried, strand wrapped prestressed concrete reservoir (Figure 7-1). It has a listed storage capacity of 2.5 MG and is the newest reservoir in the City's system, built in 2006. From the floor to the bottom of roof, the height is 27 feet. The roof has an approximate 1.8-degree slope and is supported by (32) 18-inch diameter columns. Prestressing information derived from the City-provided drawings is included in the structural report in Appendix E. The minimum and maximum operating levels are 14.5 and 22 feet, respectively, which equates to an operational storage of 2.1 MG.

Figure 7-1: The Kearney Reservoir, viewed looking northwest

Kearny is situated between a new housing development and forested area in the north-central portion of the City water service area, about 3 miles north-northeast of the Bellingham CBD.

The below-grade, pre-cast piping valve vault for the reservoir is located 24 feet to the southeast. It is located within the 276 North Zone, which is fed via gravity from Whatcom Falls II. This configuration is why an overflow pipe was omitted. Per the City, the zone overflows at 276 feet City, which is 25 feet above the floor elevation of 251 feet City shown in the provided drawings.

The geotechnical investigation at the Kearney Reservoir consisted of seven test pits and two borings, performed prior to the Reservoir's construction in 2001. The borings were completed to depths of 18 to 28.3 feet. No fill was encountered, other than a very thin, 1-foot layer at one test pit in the 60 feet northeast of the reservoir. Glaciomarine drift was encountered in all explorations, ranging from 0 to 5 feet bgs. This soil consisted of gray to brown stiff clay with variable sand and

gravel/brown loose clayey sand with gravel and cobblestones. The borings indicate that Huntington Formation Bedrock (similar to the Chuckanut Formation) is found beneath the drift. The bedrock varied from siltstone, fine to coarse grained sandstone, to pebble conglomerate. During construction, the drift layer was excavated and replaced with a prism of roller compacted concrete (RCC), placed on the bedrock. Groundwater seepage was observed, ranging from 2 to 7.5 feet bgs.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Kearney Reservoir is bearing directly on RCC and bedrock, an allowable bearing pressure of 8,000 psf can be used for structural analysis. Bedrock and RCC are not at risk of liquification. This site has low risk of issues with slope instability.

7.2 Inspection Summary

A drained inspection of the Kearney Reservoir took place on March 14, 2019.

7.2.1 Exterior Inspection Summary

7.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the property. A tree has fallen on the fence on the south side, damaging the fence (Figure 7-2 Left). Along the east side of the reservoir, a row of tress was planted about 18 feet away from the reservoir walls (Figure 7-2 Right). The site is set up such that water drains from the roof onto the ring road. A series of five catch basins are on the south side of the reservoir that are charged with collecting water. The access road is mostly flat, draining toward to reservoir. In many areas on the ring road, significant organic material is building up along the reservoir's perimeter (Figure 7-3).

Figure 7-2: Tree-induced fence damage by (Left) and east side trees

Figure 7-3: Organic material buildup on the ring road adjacent to the reservoir and wall staining

7.2.1.2 Exterior Walls

Between 1.5 feet and 8 feet of the 24.5-foot wall is buried. On either side of the ladder on the southeast side, areas of efflorescence were noted approximately 8 feet above grade (Figure 7-4 Left). Staining on the walls from roof drainage was found throughout the reservoir (Figure 7-4 Right). Shotcrete was crazing in some of these area (Figure 7-5 Left).

Figure 7-4: South wall efflorescence and Indications of roof drainage onto the walls.

"Sounding" of the walls around the perimeter of the reservoir was performed. Sounding is a process of tapping the reservoir's exterior surface to listen to the report of the hammer strike. A sharp "ping" sound is generally indicative of a competent shotcrete layer while a dull or hollow "thud" sound can be indicative of delamination within the shotcrete layer. While the shotcrete layer was stained in many instances, only two of the areas on the north side of the reservoir were found to be potentially delaminating. These two areas are shown on the right side of Figure 7-4 (Right) and close up is shown in Figure 7-5 (Right). This indicates that the shotcrete is no longer adequately adhered to the core wall in these areas.

Figure 7-5: Organic material on the ring road and heavy staining of exterior walls

7.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

7.2.1.4 Exterior Roof

The exterior roof of the reservoir was mostly clean with minor organic growth only found upgradient of vents (Figure 7-6). It is sloped at approximately 1.8 degrees from the center to the perimeter. The roof is designed to shed water from the exterior walls with the slab overhanging and drip stop (Figure 7-7). The distance from the roof to ground level ranges from approximately 18 to 26 feet.

Figure 7-6: The exterior roof

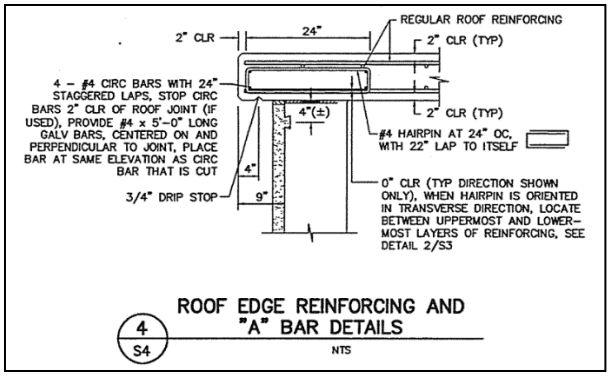

Figure 7-7: The roof overhang and drip stop design

7.2.1.5 Exterior Appurtenances

The exterior ladder (aluminum) has a lockable door were fully operational (Figure 7-4 Left). The railing stretched around the perimeter of the reservoir with a toeboard height of 6 inches and a top height of 43 inches (Figure 7-8 Left). The two 8-foot by 10-foot access hatches have a gutter inset into the curb (Figure 7-8 Left).

Two 46-inch square penetrations on the northwest and southeast quadrants with vent structures provide air exchange for the reservoir (Figure 7-8 Center). Although the mesh could not be inspected, plans and a previously-taken photo (Figure 7-8 Right) indicate that the mesh is 0.125-inch square openings with 14-gauge stainless steel wire cloth.

Figure 7-8: Roof access hatch (Left), roof vent (Center), and a supplied photo of the screen

7.2.2 Interior Inspection Summary

7.2.2.1 Interior walls

The interior walls did not exhibit cracking (Figure 7-9 Left). Formwork holes and epoxy injection ports, while overfilled in some instances (Figure 7-9 Right), which is only a cosmetic concern.

Figure 7-9: Interior walls did not have any observable issues (Left) other than. cosmetically, a few of the injection ports were overfilled (Right).

7.2.2.2 Floor

The interior floor slab had no observed issues of cracking or signs of failure. However, a significant amount of sediment covered the floor (Figure 7-10).

Figure 7-10: Sediment covered the floor.

7.2.2.3 Interior Roof and Columns

The ceiling was in was free of issues, other than incidental cracking around the exterior edge of the roof (Figure 7-11). Cracking was more pronounced near the column drop panels.

Figure 7-11: Minor roof cracking on the perimeter of the ceiling, localized near roof drop panels

7.2.2.4 Interior Appurtenances

The interior stainless steel ladder is anchored near the roof and at the base of the reservoir (Figure 7-9). The ladder's fall protection system was fully operational during our inspection.

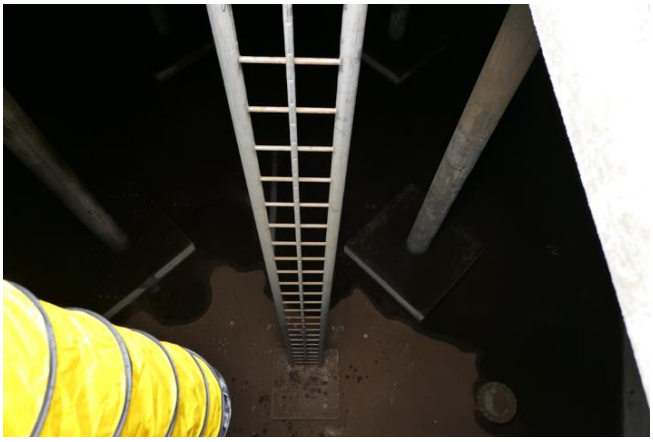

Figure 7-12: The interior ladder and fall protection were fully functional.

7.2.3 Piping and Valving Inspection Summary

The 20-inch inlet pipe had a metal flow director that exhibited corrosion (Figure 7-13 Left). The 30-inch outlet pipe is located on the opposite side of the reservoir (Figure 7-13 Right). The drains appeared to be functional (Figure 7-10 Left), but not have an observable mesh on the outlet (Figure 7-14). For dechlorination, the system appeared to treat water that overflowed, but did not treat water that drained directly to the stormwater detention basin on site. The overflow has not yet been installed (Figure 7-15 Left). The hose bibs used for washdown operations on the interior of the reservoir are corroding (Figure 7-15 Left). The control valve was operational (Figure 7-16).

Figure 7-13: The inlet (Left) and outlet (Right)

Figure 7-14: Exterior drain pipe (Left) and screen (Right).

Figure 7-15: The overflow penetration (Left) and one of two interior hose bibbs that exhibit corrosion (Right)

Figure 7-16: The control valve

7.3 Condition Assessments

7.3.1 Cleanliness and Coatings

The interior and exterior of the structure are uncoated. The exterior walls and roof have minor organic material, but the ring road has significant organic material accumulation.

The sediment accumulating on the floor indicates that the reservoir is due for a full washout inspection. There was missing or inadequate coatings on inlet flow director and hose bibs. Overall, the coatings are cleanliness are assessed to be in good condition.

7.3.2 Material Deterioration

The efflorescence on the south side of the reservoir may be the result of the differential backfill and "water-in-transit" which comes to the surface and evaporates on the south (sunward) face of the reservoir. Efflorescence is typically more of a cosmetic issue rather than a durability concern. Evaluation of the existing efflorescence did not note any additional associated issues such as delamination, cracking, or other failure of the shotcrete layer.

In contrast, the areas of staining and possible delamination noted on the northern extent of the reservoir are of higher concern. A driving force behind delamination is typically water infiltration between the core wall and shotcrete and subsequent freeze-thaw which not only causes the delamination but also makes these areas more suspectable to future water infiltration and delamination. Since the shotcrete was applied in multiple layers, three or four, some of the delamination can occur at these layers as well. Staining on these areas are indicative of roof drainage issues, which are discussed in Section 7.3.6. If these problems are not addressed, the delamination is likely to increase, and additional areas of delamination can be expected. The walls are, overall, in fair condition.

On the interior, corrosion is taking a toll on the hose bibbs and inlet flow director, which are in fair condition. These appurtenances will continue to degrade unless they are cleaned and (re)coated.

7.3.3 Structural Performance

There is no overflow installed on the structure. The City reports the reservoir overflows at 25 feet while, the design drawings indicate an anticipated overflow of 26 feet. The current maximum operating level is reported to be 22 feet.

7.3.3.1 Static Analysis

The roof slab was found to be sufficiently thick and reinforcing was found to be suitable based on current code requirements. Cracks in the roof along the perimeter are likely due to differences in stiffness between the drop panel and roof slab. These cracks appear relatively minor.

For vertical wall reinforcement, the size and spacing of the pre-stressing members were determined to be adequate. Columns were found to be suitable for anticipated design loads. An overflow level of 26 feet would induce a soil bearing pressure of 3,900 psf. This is acceptable based on the allowable soil bearing capacity of 8,000 psf noted in the geotechnical report.

7.3.3.2 Seismic Analysis

The seismic joints, strand wrap, and columns were all found to meet seismic code when operated up to the overflow level of 26 feet. The design seismic event would induce a 3-foot slosh wave at the operating level of 22 feet. This would not impact the roof, which is 5 feet above the overflow. The roof does have sufficient structural capacity to resist slosh impact at the 26-foot overflow level. The roof hatches/vents may blow out or be damaged from slosh during the design seismic event. This would likely be a localized failure or isolated damage to the hatch and isn't expected to impact the overall structural performance of the reservoir.

7.3.4 Water Quality/Sanitary

The inlet/outlet configuration appears to be suitable to facilitate adequate mixing within the reservoir. The drain pipe's air gap meets DOH requirements but may have been missing a proper screen. Based on provided drawings, the vent screens likely do not follow DOH recommendations with a #24 mesh. They are also configured in a way that makes it difficult to inspect the condition of the vent screens. Also, the access hatch is considered a high-maintenance-type design under current DOH guidelines as it can collect materials in the channels. Despite these minor issues, the reservoir is in good condition related to water quality/sanitary.

7.3.5 Safety

Fall protection on the interior and exterior ladders, and on the perimeter of the low slope roof were found to be compliant. The two access hatches did not have a railing or safety barrier, however, which are required. The reservoir is in good condition related to safety.

7.3.6 Operations and Maintenance

7.3.6.1 Site and Security

The fallen tree that has impacted the fence, which compromises the security of the site. Otherwise, the fence appeared to be suitable to prevent illicit access. Intrusion alarms are installed on the roof's access hatches. The trees along the east side are too close to the reservoir. There appeared to water held on the ring road adjacent to the reservoir, which likely is from the reservoir roof. Overall, the site/security is in fair condition.

7.3.6.2 Roof Drainage

The site and roof drainage may be a combined issue at the Kearney Reservoir. Rather than routing stormwater from the roof to the detention pond via roof drains and stormwater pipes, the structure is designed such that water drains from the roof overhang to the access road and then relies on the road's grade to convey it to storm drain catch basins on the south side of the site. For reference, a 1-inch rain event on the roof would cause to 2,300 gallons of water to be discharged onto the access road. The organic growth on the access road near the walls is indicative of excess water that is not draining from the site properly. Further, it appears that water is splashing on the ground and contributing to further staining of the reservoir (Figure 7-5).

Major drainage points from the roof can be random, dictated by minor variations in roof slope. The points of observed dark-colored staining on the sidewalls may be areas where flow is concentrated. Drainage of the roof appeared to not function properly throughout the reservoir. The drip stop shown in Figure 7-7 is possibly not sufficiently far from the walls. Water from the roof is apparently contacting the walls on this reservoir, which will continue to degrade the shotcrete if not corrected.

7.3.6.3 Appurtenances

No difficulties were found using the exterior or interior ladders. The Kearney Reservoir met design requirements for the roof vent penetration and screened areas.

7.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. The current dechlorination system allows chlorinated water to enter to stormwater pond. A best practice

would be a dechlorination basket attached to the outlet of the drain pipe. Without an overflow, the reservoir would not meet the requirements of AWWA D110-13. In the event of an unforeseen problem, the water level within the zone may exceed the roof level and cause structural damage to the roof.

7.3.7 Obsolescence

Overall, the equipment on site appeared to be of newer design and would be easily replicable if needed.

7.3.8 Condition Scoring

Overall, the condition score of the Kearney Reservoir was very good, 4.6. Problems associated with this reservoir were related to the roof drainage and site drainage. A summary of the scoring is shown in Table 7-1 and the full Score Matrix is included in Appendix E.

Table 7-1: Condition Scoring of the Kearney Reservoir

Categorical Scores	
Cleanliness and Coatings	4.6
Material Deterioration	4.5
Structural Performance - Static	4.4
Structural Performance - Seismic	5.0
WQ/ Sanitary	4.4
Safety	4.5
Operations & Maintenance	4.3
Obsolescence	5.0
Overall	4.6

7.4 Recommended Improvements

7.4.1 Cleanliness and Coatings

The exterior, walls, roof, and ring road can be pressure washed to remove the efflorescence and organic material that has accumulated. The interior should also be washed out to remove the sediment accumulation on the floor.

7.4.2 Material Deterioration

While efflorescence is primarily cosmetic, it does consist of soluble salt-forming components. If these are allowed to build up for extended periods of time, these salts can result in a form of salt attack which can deteriorate and weaken the shotcrete. As efflorescence can be harder to remove over time, it is recommended that regular cleaning of the reservoir take place to removed existing efflorescence to both ensure the long-term adequacy of the shotcrete and to help remove any

issues which might obscure any underlying cracking which would otherwise be identified for repair. The efflorescence is expected to be dealt with by pressure washing. Any areas of efflorescence and crazing can continue to be monitored for growth and/or severity.

The concrete delamination on the exterior is likely a problem resulting from poor roof drainage; improvements are discussed in Section 7.4.6. While it may be possible to remove the shotcrete and assess the strand, we do not recommend this work be conducted at this time. If there are indicators of corrosion of the strand wrapping in the future, destructive testing should take place. Instead, sounding tests should be conducted at regular intervals by City staff to check if this issue is getting worse.

The corrosion on the interior pipe bend and hose bibb can be addressed by cleaning and recoating. If the hose bibbs are beyond repair, they can be abandoned and washdown water sourced from the exterior of the reservoir.

7.4.3 Structural

There are no structural improvements recommended on the Kearney Reservoir.

7.4.4 Water Quality/Sanitary

Because a compliant screen was not found on the drain pipe, an inspection should be carried out of this pipe and a DOH-compliant mesh installed on the pipe if it is found to be missing. The roof access hatch gutter drainage points should be screened to protect them. This will bring them into compliance with the DOH requirements for the sanitary protection or reservoirs. Finally, the vent screen should be replaced with a #24-mesh and 4-mesh screen backing. The City can also devise ways to make these screens observable for inspection.

7.4.5 Safety

Safety railings around the roof access hatches are recommended. Alternatively, temporary barriers can be used during inspections and maintenance.

7.4.6 Operations and Maintenance

7.4.6.1 Site and Security

The tree that has fallen should be removed and the fence repaired. The trees along the east side of the reservoir should be removed or transplanted. This area can be replanted with bushes that are not as tall, although impacts from root penetration are still possible at this distance. Site drainage improvements may be dependent on the selected improvement for roof drainage deficiencies, discussed in Section 7.4.6.2. Because the access road does not appear to be draining water away from the reservoir, the road should be regraded to increase the slope from the

reservoir. Generally, this is called out at a 2 percent minimum grade. Slopes less than this typically are difficult to install. The City may also consider improved stormwater conveyance system at this site if conditions do not improve.

7.4.6.2 Roof Drainage

Many options are possible to deal with the roof drainage issue. These improvements are essential to reduce the damage to the shotcrete and potential impacts to the strand wrapping underneath. A first step can be hosing down the roof during dry weather and observing where the water is draining. At minimum, the roof drainage should clear the walls in areas where dark discoloration is occurring but could be expanded to include the entire circumference. One such a retrofit is conceptually depicted in Figure 7-17. Care should be taken no to impact the underlying strand in the core wall or hairpins during these repairs.

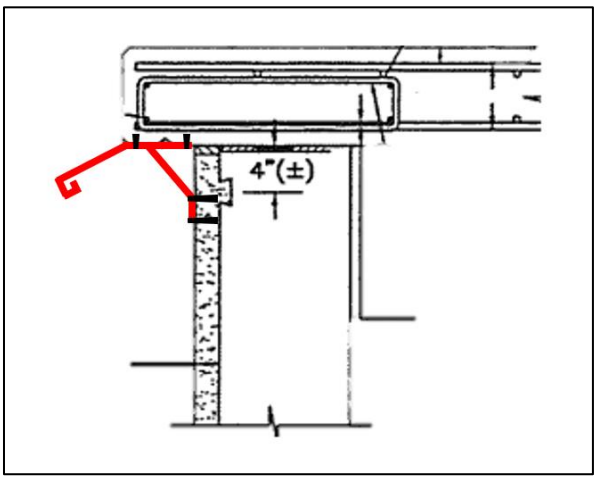

Figure 7-17: One retrofit option for the roof drainage to combat water contacting sidewalls

Alternatively, the entire roof can be improved with flow directors and a more robust drainage system with downspouts. The City can also consider piping water from new roof downspouts to catch basins. Fortunately, this reservoirs roof does not appear to be subjected to the high degree of organic material deposition as many of the City's other reservoirs. Piping this water may elevate the observed draining issues on the ring road.

7.4.6.3 Appurtenances

There are no appurtenance-related operations and maintenance improvements recommended on the Kearney Reservoir.

7.4.6.4 Valving and Piping

The dechlorination at the Kearney site could consist of a system that attaches to the drain outlet or a basket a weighted system that is set in the manhole on site. An example of a basket system is shown in Figure 4-21. A proper dechlorination system should be installed before the next time the

reservoir is drained. An overflow should also be installed to meet requirements of AWWA D110-13 or system hydraulics confirmed.

7.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Dakin II Reservoir are shown in Table 7-2.

Table 7-2: Kearney deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Efflorescence noted and organic material buildup on exterior and on ring road	Pressure wash exterior and ring road	$ 2,000
	Sediment accumulation on floor	Perform regular washdown inspections	$ 1,000
DETERIOR-ATION	Delaminating shotcrete on north sidewall	Monitor and track staining and delamination	$ -
	Corrosion on inlet pipe bend & hose bibbs	Clean and (re)coat	$ 1,000
WQ	Drain pipe screen non-compliant	Replace drain pipe screen	$ 1,000
	Roof hatch is a high-maintenance design (gutter type)	Add screen to access hatch gutter	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Lacks roof hatch railing(s) or temporary barrier	Install roof hatch railing(s) or use temporary barrier	$ 8,000
O&M	Fence broken with tree	Remove tree and fix fence	$ 2,000
	Trees along east side too close	Tree removal and restoration	$ 4,000
	Ring road moist and has organic growth - does not appear to shed water	Upgrade site drainage. Dependent on roof drain improvements.	$ 50,000
	Roof drainage is not clearing exterior walls	Improve roof drainage	$ 20,000
	Dechlorination non-compliant	Update dechlorination system	$ 2,000
	Lacks internal overflow	Install overflow	$ 15,000
		Subtotal	$ 108,000
		30% Contingency	$ 32,400
		8.7% Tax	$ 12,215
		Engineering, Administration, and Construction Management	$ 53,415
		Total Project Costs Estimate	$ 206,030
		Total Project Costs Estimate (rounded)	**$ 210,000**

7.5 Conclusion

The most pressing issue with this relatively new reservoir is the shotcrete that is delaminating. Other areas stained with a similar pattern are likely to delaminate in the future. The City may wish to inquire with DN tanks to determine if this problem is covered by warranty. If not, the roof drainage should be improved.

The maximum water level in this reservoir could be increased from 22 feet to 26 feet and still meet structural code. If water can be hydraulically brought into the reservoir at this level, it may reduce pressure related issues in the Kearney Reservoir's service area.

Because the interior has sediment accumulation, we recommend this reservoir be prioritized in its next washout inspection. This should take place within the next 2 years. The reservoir should be

leak tested as a first step of the next inspection. We recommend City staff conduct sounding tests and map shotcrete discoloration and delamination extents every 6 months to start with. This will help determine if the delaminated areas are expanding or if additional areas of delamination are occurring. This frequency can be lessened if the roof drainage issue is resolved or if multiple inspections indicate no further deterioration or expansion of delaminating areas.

Section 8

Whatcom Falls II Reservoir

8.1 Tank and Site Overview

The Whatcom Falls II Reservoir is a 350-foot diameter, partially buried, strand wrapped prestressed reservoir (Figure 8-1). It has a listed storage capacity of 16 MG and was built in 1993. From the floor to the bottom of roof, the height is 23 feet. and is supported by (221) 24-inch diameter columns. Prestressing information derived from the City-provided drawings is included in the structural report in Appendix F. The minimum and maximum operating levels are 13 and 21 feet in the summer and 12 to 21 feet in the winter, respectively.

Figure 8-1: The Whatcom Falls II Reservoir, viewed from atop the Whatcom Falls I Reservoir, looking southeast

The site is centrally located within the distribution area, about 2 miles east of the Bellingham CBD. The site also includes the Whatcom Falls I reservoir and a pump station, which are not close enough to impact the structure during seismic event. The reservoir serves the 276 South Zone.

The geotechnical investigation at the Whatcom Falls II Reservoir consisted of six test pits, performed in 1992, prior to the Reservoir's construction. The pits were dug to depths of 1 to 15 feet. Forest duff was encountered in the first 0.5 or 1 feet of each test pit. Recent alluvium (dense silty sand), advance outwash (medium dense to very dense sand and gravel with occasional

cobbles), and glacial till (very dense silty sand with gravel) were encountered in the various test pits. Chuckanut Sandstone- Siltstone was also encountered at depths of 1-foot to 14 feet bgs. Groundwater was encountered in two test pits at 5.5 and 9.5 feet bgs. The as-built drawings indicate the alluvial soils were removed and replaced with structural fill. Thus, the reservoir is founded on dense glacial soils and structural fill.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Whatcom Falls II Reservoir is bearing on dense glacial soils and structural fill, an allowable bearing pressure of 3,500 psf for wall footings and 4,000 psf for column footings can be used for structural analysis. The soils are not at risk of liquification. This site has low risk of issues with slope instability.

8.2 Inspection Summary

Floating and exterior inspections took place on June 12 and November 30, 2019, respectively. An underwater inspection is yet to be conducted.

8.2.1 Exterior Inspection Summary

8.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed encompasses the Whatcom Falls I and II Reservoirs. The fence is 15 to 16 feet from the walls on all but the northwest quadrant where the site extends to the Whatcom Falls I Reservoir. The soils near the reservoir appeared to be wet. Tall trees were found growing just beyond the fence. (Figure 8-2).

Figure 8-2: Fence, adjacent trees, and access road (Left). Tire splashing (Right).

8.2.1.2 Exterior Walls

The exterior walls have a layer of shotcrete which protect the strand wrapping around the core wall. Many instances of graffiti-cover up were noted (Figure 8-3 Left). Staining on the walls from roof drainage was found throughout the reservoir including some areas that appeared to have concentrated flow with heavier staining (Figure 8-3 Right). There was a very minor crack with efflorescence along the southeast wall (Figure 8-4 Left) and minor cracking noted on the roof slab (Figure 8-4 Right). Sounding tests indicated the shotcrete is still adhered throughout the reservoir.

Figure 8-3: Graffiti covered and evidence of drainage from the roof contacting the sidewall

Figure 8-4: Minor cracks in the upper wall (Left) and roof slab (Right).

8.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

8.2.1.4 Exterior Roof

The roof has a perimeter curb, ten drain locations, and retrofitted scuppers to allow the roof to shed water should the drains clog with debris. While the exterior roof (Figure 8-5) had minimal organic material building up during the summer inspection, there were minor instances of water ponding (Figure 8-6). During the fall inspection, the roof was covered with organic debris (Figure 8-7 and Figure 8-8). Many of the roof drains were clogged and water was pooling heavily. Many roof drains had the silt stops removed, although drains with and without silt stops were clogged with debris (Figure 8-9 Left). The downspout pipes appeared corroded (Figure 8-9 Right). The roof is designed to shed water from the exterior walls with scuppers in the event of drain clogs. The slab overhanging has a drip stop if the scupper clogs.

Inspecting the roof for structural issues, no major areas of visible structural failure. On the slab, only minimal, non-structural, pattern cracking observed. No additional cracking or signs of stress were noted around the solar array. The joints of the roof slabs did not exhibit structural cracking. The roof's slope was measured to be about 0 degrees near the edges and hatches. The distance from the roof to ground level is 21 feet, 4 inches.

Figure 8-5: The exterior roof

Figure 8-6: Roof in summer clear of debris with ponding and staining

Figure 8-7: Heavy organic material and ponding in the fall, roof railing

Figure 8-8: Clogged roof drains

Figure 8-9: Clogged drain with sediment (Left) and downspout pipes had corrosion (Right).

8.2.1.5 Exterior Appurtenances

The exterior stainless steel ladder has a lockable door and had sandpaper covering the rungs, except for the rungs where this sandpaper had fallen off (Figure 8-10). The roof railing stretches around the perimeter of the reservoir with a toe guard height of 4 inches and a top height of 42 inches (Figure 8-7 Right). Some of the nuts holding the railing in place were coming off or missing (Figure 8-11).

Figure 8-10: The exterior ladder has a lockable enclosure and has a safety climb rail.

Figure 8-11: The railing's bolts are at various angles and are missing nuts in a few instances.

The roof has seven 6-foot by 6-foot equipment hatches and six 3-foot by 3-foot access hatches. These hatches have a gutter inset into the curb (Figure 8-12 Left). Cracks were found in the frames of curbs of the hatches, some with significant efflorescence.

The reservoir has eight roof vents that open downward and have insect and bird screens (Figure 8-13). Water was found to be condensing and dripping allowing minor organic growth. There are

also 20 access ports on the reservoir (Figure 8-14 Right). The vents and ports had missing nuts on anchoring bolts.

Figure 8-12: The roof access hatch curbs are cracking and have gutter systems.

Figure 8-13: The roof vents and screen

Figure 8-14: Hardware missing on various roof appurtenances

8.2.2 Interior Inspection Summary

8.2.2.1 Interior walls

The interior walls did not exhibit cracking above the waterline (Figure 8-15). The interior walls were unable to observed below the waterline.

Figure 8-15: The reservoir's interior walls above the waterline

While cathodic protection systems are normally in place to protect steel tanks from corrosion, Whatcom Falls II has one installed to protect interior appurtenances. To measure output, a continuous metal circuit is generally made, which was not possible since the ladder at the #1 hatch was not in contact with the reinforcing steel. For testing purposes, the structure lead was connected to the rectifier's negative terminal, which are not as accurate as if a sensing ground were present.

8.2.2.2 Floor

The interior floor slab was unable to be fully inspected from the floating inspection but appeared to be in good condition (Figure 8-16). There did appear to be a minor amount of sediment buildup on the floor.

Figure 8-16: The floor appears to have minor sediment accumulation

8.2.2.3 Interior Roof and Columns

The interior roof (Figure 8-17 Left) did not exhibit structural cracking but was noted to have some minor cracking with efflorescent on the slab. Many of these areas had been sealed with epoxy injecting and coated. (Figure 8-17 Right), while others appear to be untreated (Figure 8-18). This cracking was primarily noted at the edges of some of the column drop panels.

Figure 8-17: Some cracking was noted on the perimeter of the ceiling, localized near roof drop panels.

Figure 8-18: Minor cracks were noted in the ceiling

8.2.2.4 Interior Appurtenances

The interior stainless steel ladders are anchored near the roof and at the base of the reservoir (Figure 8-19). The ladders' fall protection systems appeared to be fully operational during the inspections. Light rust staining was noted on the ladder rail and rungs. Heavy corrosion and material loss were noted on the inlet slide gate shaft. Minor rust staining was also noted on the chlorine injection system and curtain hangers (Figure 8-20 and Figure 8-21).

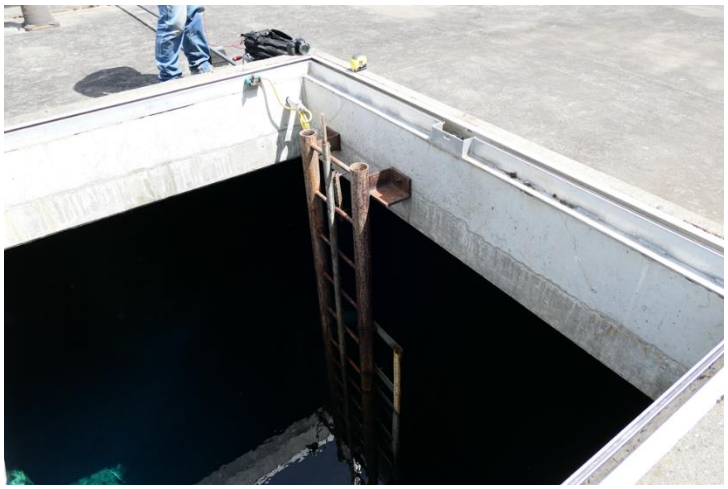

Figure 8-19: The interior ladder and fall protection minor rust discoloration

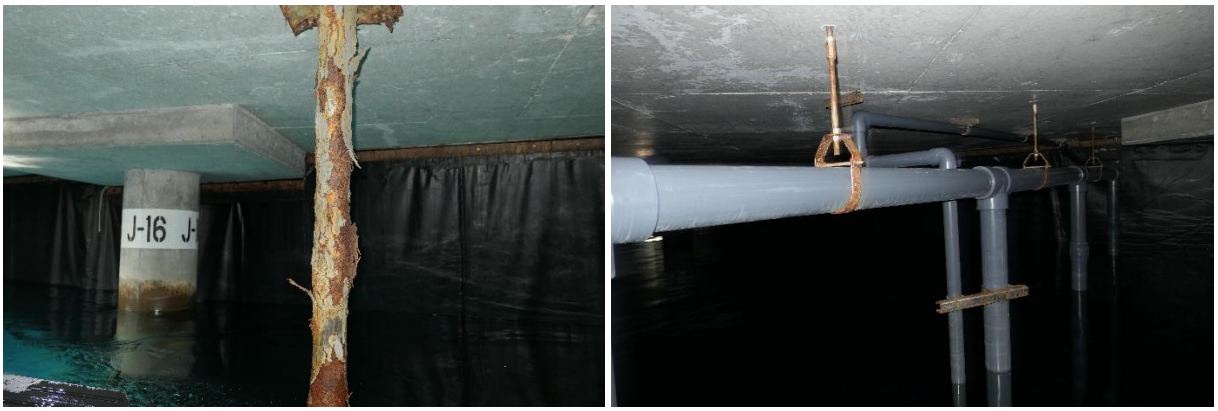

Figure 8-20: Heavy corrosion on the inlet slide gate stem (Left) and minor corrosion on the chlorine injection system hangers (Right)

Figure 8-21: The chlorine injection ceiling straps and baffle curtain mounts minor rust staining

8.2.3 Piping and Valving Inspection Summary

The 72-inch inlet pipe (Figure 8-22 Left) and the 60-inch outlet pipe (Figure 8-22 Right) did not have observable issues. The floor drains were not observed from the floating inspection. The drain piping was observed to have insect and bird screens (Figure 8-23). While the provided cover sheet indicates that the reservoir drains to sanitary sewer, staff reported that the reservoir drains to Whatcom Creek. The dechlorination system and outfall were not observed. No overflow is was noted on this reservoir.

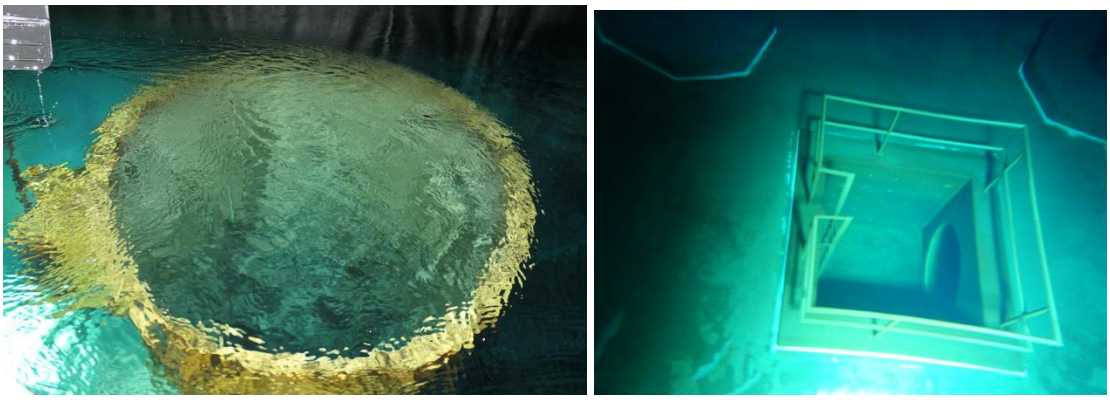

Figure 8-22: The inlet (Left and outlet (Right) pipes

Figure 8-23: Drain piping (Left) and screen (Right).

Figure 8-24: Drain pipe outlet to Whatcom Falls Creek

8.3 Condition Assessments

8.3.1 Coatings and Cleanliness

The roof was noted to have a large amount of accumulated organic debris, especially during the fall inspection. This debris originated from the adjacent vegetation and was in various stages of decomposition. This material was not present during our summer inspection. Thus, it is evident that the City clears the roof of debris as part of routine maintenance activities. The roof cleanliness was found to be in fair and poor condition during the summer and fall inspections, respectfully.

The staining on the exterior walls is discussed in Section 8.3.6. This discoloration appears to be cosmetic at this time as the shotcrete is still tightly adhered per sounding tests. This staining, if not corrected, may degrade the shotcrete, resembling what is occurring at the Kearny Reservoir (Figure 7-5 Right).

On the interior, the sediment accumulating on the floor indicates that the reservoir is overdue for a washout.

8.3.2 Material Deterioration

The cracking on the roof slab and alligator cracking in the structure appear to be minor. Some of the cracking may be due to roof drainage issues, discussed in Section 8.3.6.

According to the provided as-built drawings, the hatch curbs appear to have been constructed with two thinner sides and without the required hoop reinforcing. This is likely the reason the cracking is occurring at these areas. The missing hardware on the various external components may be occurring as the structure expands and contracts. Overall, the observed concrete of the structure is in good condition, but the concrete on the hatches is in fair condition.

On the interior, corrosion on the inlet slide gate stem was severe. Coating of this element may have been missed during construction. The rust discoloration of the other interior elements can be considered minor and are to be expected in such an environment. Overall, observed concrete deterioration and metal corrosion issues are minor on the interior, so it is in good condition.

8.3.3 Structural Performance

8.3.3.1 Static Analysis

The roof slab was found to be sufficiently thick and reinforcing was found to be suitable based on current code requirements, including the excess weight of the solar panels. Cracks in the interior roof slab near drop panels do present a structural concern currently.

For vertical wall reinforcement, the size and spacing of the pre-stressing members were determined to be adequate. Columns were found to be suitable for anticipated design loads. While the foundation design was found to be adequate for current codes, the soil's maximum bearing pressure would be exceeded at an operating level of 22 feet. This level is 1-foot higher than the current maximum operating level of 21 feet.

8.3.3.2 Seismic Analysis

The seismic joints, strand wrap, and columns were all found to meet seismic code when operated at a level of 22 feet. The design seismic event would induce a 34-inch slosh wave at the operating level of 22 feet. The roof structure was found to have requisite structural capacity to resist a slosh impact wave resulting from operating at a 22-foot operating level. While the hatches and vents would be clear of such a wave because they are in a raised curb, the chlorine injection system would be impacted by the wave. Additionally, the injection system was not observed to have diagonal bracing, protecting it from seismic event-induced lateral loading. Even if it were not affected by the wave, the piping is still susceptible to failure. The baffle system curtains tear strength would likely be greatly exceeded due to a code-level seismic event, meaning they would tear. Although this would likely not induce structural failure, the curtains could block the outlet.

8.3.4 Water Quality/Sanitary

The inlet/outlet configuration appears to be suitable to facilitate adequate mixing within the reservoir. The access hatches follow a design which is considered high maintenance under current DOH guidelines as these hatches can collect materials in the channels and are susceptible to pests without a screened outlet.

8.3.5 Safety

Because the fixed ladders are less than 24 feet tall, L&I does not mandate fall protection. However, the fall protection on these ladders were in working order. The perimeter railing on the low slope roof was also found to be compliant. The six access hatches and six equipment hatches did not have a railing or safety barrier, however, which are required. The safety systems on the reservoir are in good condition.

8.3.6 Operations and Maintenance

8.3.6.1 Site and Security

While the fence did not have any noted issues, the large amount of graffiti found on the reservoir indicates that this site's security is not functioning adequately. However, intrusion alarms are installed on the roof's access hatches. The wet access road around the reservoir indicates the soil adjacent to the reservoir may not be draining water away suitably. This may be caused or exacerbated by the roof drainage overflowing onto the area. Tall trees were found to be too close

to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. The site and security are in fair to poor condition.

8.3.6.2 Roof Drainage

The roof drainage is not operational with many of the drains fully blocked with debris. Only one of the ten roof drains had its cover in place while the other covers were strewn about the roof in various places. The existing drain downspout covers (Figure 8-25 Left) do not match the design (Figure 8-25 right).

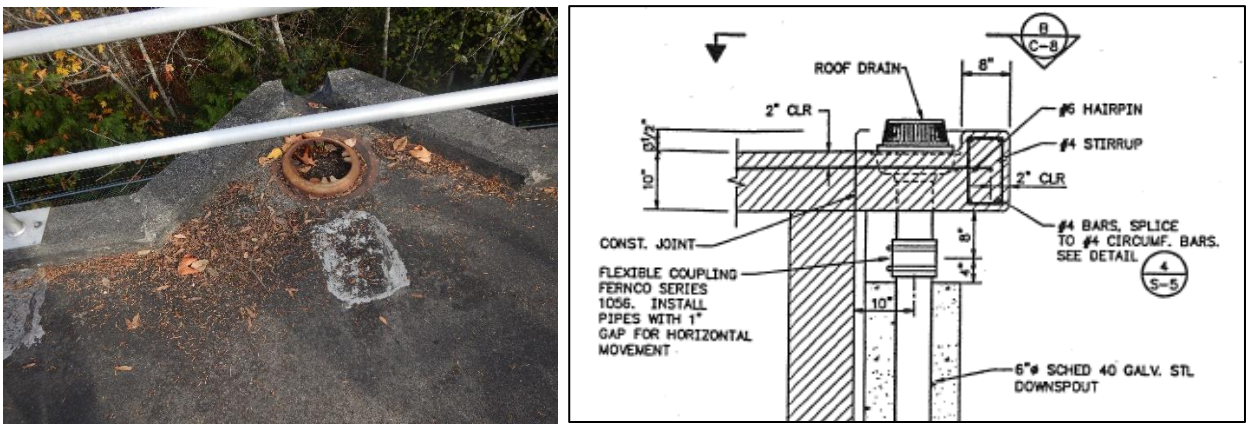

Figure 8-25: Comparison of the existing roof drain covers (Left) and the original design for the roof drains (Right)

It appears that neither these unspecified covers nor having the drains uncovered is suitable to facilitate adequate roof drainage with the organic material loads the roof is subjected to. If the covers are left in place, they appear to block water from flowing into the downspouts and instead route water to the scupper notch. This water would flow onto the adjacent access ring road. This is likely the reason these covers were removed from most of the drains. However, without any cover in place, many of the screens have become clogged with decomposing organic matter (Figure 8-8).

The ponding on the roof has caused the staining on the exterior sidewall. This is occurring either as a result of water draining through the shear cans or as water overtops the curb. Standing water migrating though the shear cans has been known to occur on reservoirs of this type of construction; the flow route for this is shown in Figure 8-26.

The other possibility for the sidewall staining is water overflowing the perimeter curb and draining onto the sidewalls. Should the scupper become so debris-laden water cannot drain, the curb would likely be overtopped. To check if water overtopping the curb would contact the sidewalls, water was poured onto the top of the curb during our inspection. This water was found to be contacting the sidewalls near where the staining has been occurring. This indicates that the drip

edge design is not suitable to keep water from contacting the sidewalls should the curb be overtopped. Water was noted to be near the top of the curb during our fall inspection.

Either mechanism for roof water contacting the sidewall presents an issue insomuch that water is retained on the roof when it should not be. The staining would likely cease when the roof drainage be improved. Should the roof retain standing water in the winter, the concrete may be stressed by freeze/thaw cycles. Cracking observed on the perimeter of the roof slab and upper portion of the walls may be due this process. Further, water held on the roof presents a concern for water quality as it may permeate through the roof.

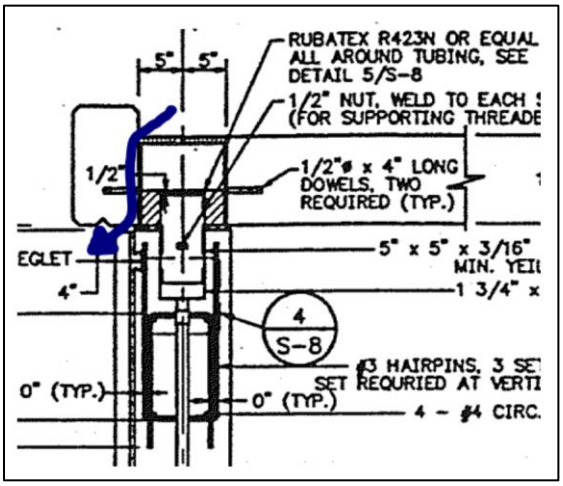

Figure 8-26: A possible route for standing roof water to contact the sidewall

With many of the drains being clogged, water is likely pouring off the scuppers (or overtopping the curb) during precipitation and snowmelt events. This may be inducing the muddy conditions encountered on the adjacent access road. For example, during a 1-inch rain event, each of the ten downspouts would be charged with handling approximately 6,000 gallons of water. A clogged drain would likely cause most of this water to be shed onto the access road.

The roof drainage problem is exacerbated by the roof being so gradually sloped that the flowing water may not force accumulated debris to the scuppers. Roofs on prestressed concrete tanks should be sloped at least 1.5 percent per AWWA D110-13. While the as-built drawing references this slope (Figure 8-27), a difference in elevation is 2.6 feet over the 175-foot radius is also indicated. This equates to a slope of only 0.85 degrees. The roof having a measured slope of 0 degrees and observed ponding at the perimeter indicates the roof slope is less than 1.5 degrees. If concrete reservoirs have a slope that is less than 1.5 percent, a “suitable coating or membrane… to prevent liquids from leaking into the tank,” is required per AWWA D 110-13.

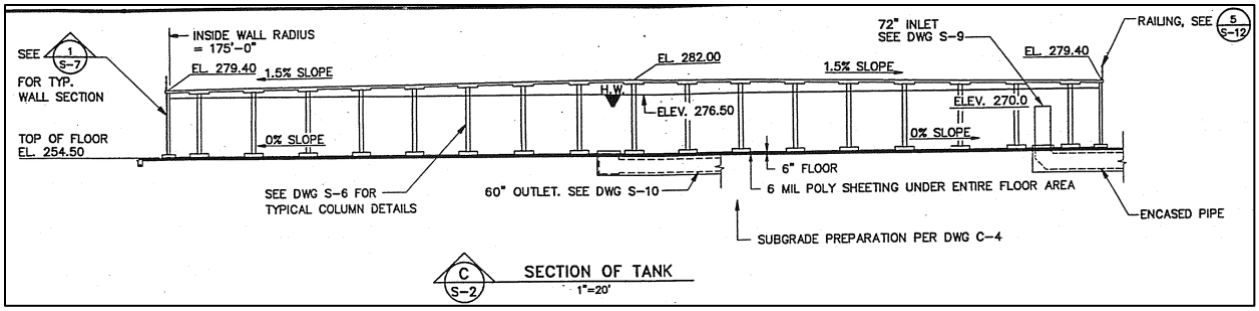

Figure 8-27: The roof's slope is 0.85 degrees, calculated with provided elevations.

Overall, the roof drainage is in poor condition with the current organic material loads.

8.3.6.3 Appurtenances

The appurtenances were, in general in good condition. No difficulties were found using the exterior or interior ladders. The Whatcom Falls II Reservoir met design requirements for the roof vent penetration and screened areas.

8.3.6.4 Piping and Valving

The operation of the piping and valving was not able to be assessed. It was not possible to drain this reservoir while meeting requirements for chlorine residual contact time. Thus, it was not possible to perform a full washdown inspection of the reservoir. This is a problem as there will be difficulties in performing future maintenance of the reservoir.

The drain piping appears to lack proper dechlorination and energy dissipation. Without an overflow, the reservoir would not meet the requirements of AWWA D110-13. In the event of an unforeseen problem, the water level within the zone may exceed the roof level and cause structural damage to the roof.

8.3.6.5 Misc.

The existing cathodic protection system transformer rectifier is a constant voltage model. Rectifier units for water tanks should be auto-potential in order to account for fluctuations in water level. Additionally, there is no continuous testing terminal to check out the cathodic protection system while it is on.

8.3.7 Obsolescence

Overall, the equipment on site appeared to be of newer design and would be easily replicable if needed.

8.3.8 Condition Scoring

Overall, the condition score of the Whatcom Falls II Reservoir was very good, 4.2. Problems associated with this reservoir were related to the roof drainage and inability to draw down the reservoir. A summary of the scoring is shown in Table 8-1 and the full Score Matrix is included in Appendix F.

Table 8-1: Condition Scoring of the Whatcom Falls II Reservoir

Categorical Scores	
Cleanliness and Coatings	3.6
Material Deterioration	4.0
Structural Performance - Static	4.3
Structural Performance - Seismic	4.3
WQ/ Sanitary	4.4
Safety	4.5
Operations & Maintenance	3.8
Obsolescence	4.3
Overall	4.2

8.4 Recommended Improvements

8.4.1 Cleanliness and Coatings

The accumulation of organic material on the roof may induce structural issues with the reservoir. Organic material should be removed, and the drains cleared as soon as possible to help prevent damage to the structure. Tree spacing and roof-drainage improvements are discussed in Section 8.4.6. Until at least one of these items is complete, the City should conduct more frequent maintenance of the roof, especially in the fall when organic material loading to the roof is heaviest.

The staining on the sidewalls should monitored for shotcrete delamination by the City. This would entail regular sounding tests and mapping of the staining to see if the area is expanding. In areas where staining is visible on the walls, the shear cans should be investigated to verify the concrete around them is still competent and they should be resealed where needed. Where the tops of shear cans can be observed in the roof, these cans should be coated to prevent any potential infiltration of water around the cans.

The interior should also be washed to remove the sediment accumulation on the floor. In lieu of a washdown, the City can consider having divers remove any accumulated sediment. A quote from LiquiVision is used for planning purposes.

8.4.2 Material Deterioration

To address the hatch curbs that are cracking, smaller cracks (<1/2-inch) can be resealed. For larger cracks, concrete can be chipped away and then the areas can be repaired with a high strength

non-shrink mortar/grout. For the missing and loose hardware on roof appurtenances, the hardware can be replaced with a Loctite to prevent them from moving.

The inlet slide gate stem appears too corroded to repair, and for planning purposes, it is recommended to be replaced. The City may also wish to investigate the gate as well which was not observable during the floating inspection.

8.4.3 Structural

To address the chlorination piping not being configured to resist lateral or slosh loads, vertical supports and horizontal bracing can be installed. Addressing the deficiencies in the baffle system needs to be fully addressed during the design phase of the project. One possible option would be to add seismic bracing to the baffling system which would likely consist of additional vertical supports that the curtain could be anchored to. Another option would be to prevent the torn curtain for the blocking the outlet of the reservoir. This could be an outlet structure or additional blocking and reinforcing of the baffling area closest to the outlet. Alternatively, contact time could occur at another site and the baffling removed completely.

8.4.4 Water Quality/Sanitary

The roof access hatch gutter drainage points should be screened to protect them. This will bring them into compliance with the DOH requirements for the sanitary protection or reservoirs.

8.4.5 Safety

Safety railings around the roof access hatches are recommended. Alternatively, temporary barriers can be used during inspections and maintenance if only a few of the hatches are opened at a time.

8.4.6 Operations and Maintenance

8.4.6.1 Site and Security

The security concerns of the site can be addressed by adding motion sensors, cameras, and/or floodlights. Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

8.4.6.2 Roof Drainage

Improvement of the roof drainage is essential to prevent damage to the core wall, shotcrete, tensioning members within the walls, and roof slab. It will also help the reservoir be resilient against environmental contamination through roof water infiltration. One way the roof drainage

can be resolved is by following the tree spacing recommendation. As the reservoir is situated in a park, we recognized removing many trees may not be feasible. As an alternative, the roof drainage should be able to handle the organic material loads the roof is subjected to. This could possibly be achieved through more regular cleaning, but retrofits are recommended. These retrofits could include one or more of following:

- Increasing the number of drainage downspouts and scuppers from ten to twenty.
- Widening of the overflow scuppers drainage point to allow material to more easily be flushed from the roof.
- Installation downspout caps, resembling those shown in the design drawings.
- Increasing the size of the downspout pipes.
- The addition of crickets to facilitate drainage to downspouts and scuppers. This is essentially a sloped surface which directs water away from the walls and to the downspouts. An example of such a system on a residential roof is shown in Figure 8-28.

Figure 8-28: Crickets facilitating roof drainage on a residential roof

To address the roof slope likely being less than 1.5 degrees, a roof lining would be the best practice. However, coating the 96,000-square foot roof would be costly. Also, the solar panels, and piping and wiring would need to be moved to allow for the coating process. Fortunately, there does not appear to be large scale infiltration into the reservoir currently. While the roof does not meet DOH recommendations, the interior of the roof and water quality should continue to be monitored. The City should plan to apply a coating to this essentially flat roof during its lifetime.

8.4.6.3 Appurtenances

There are no operational improvements recommended for the Whatcom Falls II Reservoir appurtenances.

8.4.6.4 Valving and Piping

The inability to draw down this reservoir is a major identified deficiency in the City's system. The City should consider redundancies in contact time to facilitate draw down. Drawdown of this reservoir would likely need to occur for periodic maintenance as the structure ages.

The dechlorination system on the drain pipe can be improved with a structure that attaches to the pipe or a basket that is set into the manhole, similar to that shown in Figure 4-21. A proper dechlorination system should be installed if/when the reservoir is drained. The energy dissipation can also be improved at the outlet. An overflow should also be installed to meet requirements of AWWA D110-13 or system hydraulics confirmed.

8.4.6.5 Misc.

To extend the life of the any metal below the waterline in the interior, the existing constant voltage rectifier should be replaced with an auto-potential unit. A sensing ground should also be installed on the Whatcom Falls II tank to facilitate future cathodic protection inspections.

8.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Dakin II Reservoir are show in Table 8-2.

Table 8-2: Whatcom Falls II deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material blocking roof drains	Remove organic material, clear drains, and increase maintenance frequency until tree spacing requirement is met	$ 2,000
	Staining of exterior sidewalls	Monitor for shotcrete delamination. Investigate shear cans and reseal if necessary. Coat tops of shear cans visible from the roof slab surface.	$ 10,000
	Sediment accumulation on floor	Underwater cleaning (or washdown inspection)	$ 45,000
DETERIOR-ATION	Hatch curbs are cracked	Reseal smaller cracks (<1/2"). For larger cracks, chip away concrete and repair with high strength non-shrink mortar/grout	$ 7,000
	Missing and loose hardware on roof appurtenances	Tighten and replace hardware. Use Loctite	$ 1,000
	Inlet slide gate stem is corroded	Replace inlet slide gate stem	$ 8,000
STRUCT	Chlorination piping is not configured to resist lateral or slosh loads	Install vertical supports and horizontal bracing	$ 50,000
	Baffle system not designed to resist seismic loads	Seismic upgrades of baffle system, outlet protection, and/or contact time	$ 100,000
WQ	Roof hatch is a high-maintenance design (gutter type)	Add screen to access hatch gutter	$ 2,000
SAFETY	Lacks roof hatch railing(s) or temporary barrier	Install roof hatch railing(s) or use temporary barrier	$ 48,000
O&M	Evidence of vandalism	Improve site security (w/ Whatcom Falls I)	$ 4,000
	Tree spacing recommendation not met	Tree removal and restoration	$ 83,000
	Poor roof drainage. Ring road soils are saturated w/ moss growing.	Improve roof drainage or follow tree spacing requirement	$ 20,000
	Roof slope is less than 1.5 degrees	Monitor roofs for leaks and water quality. Plan for future membrane application	$ -
	Unable to draw down reservoir	Consider redundancies in contact time to facilitate draw down	$ -
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000
	Lacks internal overflow	Install overflow	$ 44,000
	Cathodic protection system is constant voltage model and lacks stationary reference electrode	Install automatically controlled potential rectifier and stationary reference electrode	$ 10,000
	Cathodic protection system lacks sensing ground	Install CP system sensing ground	$ 4,000
		Subtotal	$ 442,000
		30% Contingency	$ 132,600
		8.7% Tax	$ 49,990
		Engineering, Administration, and Construction Management	$ 218,607
		Total Project Costs Estimate	$ 843,197
		Total Project Costs Estimate (rounded)	**$ 840,000**

8.5 Conclusion

In conclusion, the Whatcom Falls II Reservoir's structure is in major compliance with the structural code. As this reservoir is one of the most important in the City's system, careful attention should be given to it. This reservoir should stay in service for many years to come, especially if the roof drainage issues can be addressed. The City should begin planning a way to meet contact time requirements, as at some point in the future, this reservoir will likely need to be brought offline. This is the largest system-wide deficiency we noted. If another way of providing contact time is implemented, it may address both the inability to draw down the reservoir and lack of seismic integrity of the baffling system.

This reservoir has an underwater inspection scheduled. The next inspection time recommendation will be made following the underwater inspection. The reservoir should be leak tested if possible.

Section 9

40th Street Reservoir

9.1 Tank and Site Overview

The 40th Street Reservoir, shown in Figure 9-1, is one of five similar dome reservoirs that were built between 1958 and 1987. Drawings were prepared by John W. Cunningham & Associates for both the 40th Street and Reveille Reservoirs. With slight differences in dimensions, the five reservoirs are 0.3- or 0.5-MG reinforced concrete, self-supporting, domed roof hoppers that were originally built with common inlet/outlets/drains. The reservoirs were designed to hold water above the roof-to-wall interface. This reservoir is almost completely above-ground, and the roof of the attached valve vault is located about halfway up the reservoir's wall. It was built in 1961 and has a capacity of 0.5 MG.

Figure 9-1: The 40th Street Reservoir, viewed looking southeast

According to measurements taken in the field (Figure 9-2), the 40th Street Reservoir has an interior diameter of 60 feet. Dimensionally, the hopper base is 6 feet deep and the interior wall is 16.5 feet high. The overflow weir was measured to be 26.75 feet above the reservoir floor, which according to City-supplied documentation, corresponds to 696 feet. This height is higher than the top of the vertical wall. The ranges of operation are 20 to 23 feet. Reinforcement and concrete thicknesses of the walls and roof were shown in the provided drawings. The exterior roof and interior walls and floor of the 40th Street Reservoir were coated in 2010 using VersaFlex AquaVers primer and polyurea. The roof has a skid-resistant coating.

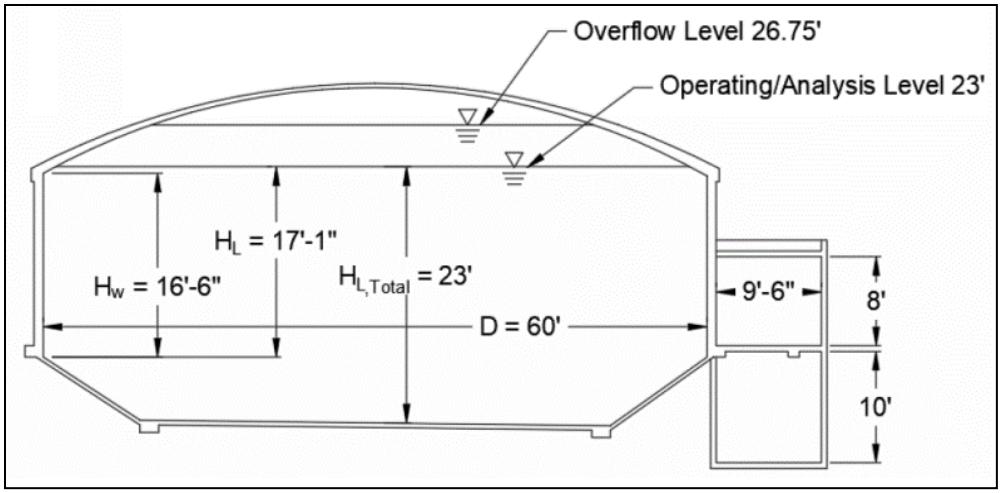

Figure 9-2: The measured interior dimensions of the 40th Street Reservoir

The 40th Street Reservoir is in the southern part of the distribution area, in a forested area about 2 miles south-southeast of the Bellingham CBD. The valve vault shares a wall with the reservoir itself and is 9.5 feet wide by 9.5 feet deep. Piping is run to the valve vault, encased in an unreinforced concrete block for protection under the footing. It serves the 699 Padden Yew Zone.

The geotechnical investigation indicated Chuckanut Sandstone from the surface to 5.5 feet bgs, where the boring could not be continued due to hard bedrock. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the 40th Street Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. Bedrock is not at risk of liquification and the site is not expected to have issues with slope instability.

9.2 Inspection Summary

A drained inspection of the 40th Street reservoir took place on April 30, 2019.

9.2.1 Exterior Inspection Summary

9.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire circles the reservoir about 12 feet beyond the walls (Figure 9-1). The site is generally well-graded, directing runoff away from the reservoir. During the site visit, it was noted that the roof of the reservoir is under the dripline of the trees on the southeast side (Figure 9-3).

Figure 9-3: The valve vault, exterior ladder, and hatch

9.2.1.2 Exterior Walls

In contrast to the City's other reservoirs of this design, this reservoir has much more of the exterior walls exposed, up to 14 feet. This is likely due to the bedrock at grade. As can be seen in Figure 9-1, there is cracking with efflorescence on the walls. Cracking was most prominent at mid-height and below the dome roof at the top of the wall. Isolated areas of topcoat failure were also noted (Figure 9-4 Right). On the bottom of the exterior, coating was missing, and moss was building up (Figure 9-5). The topcoat was found to be moderately tightly adhered, measured 5-8 mils thick, exhibited concrete loss at cracks, and was quite dirty.

Figure 9-4: The exterior walls cracking with efflorescence and coating failure of the topcoat

Figure 9-5: Missing coating and moss accumulation on bottom of tank

9.2.1.3 Foundation

The foundation was unable to be observed as it is just below the surface.

9.2.1.4 Exterior Roof

The roof of the reservoir had a textured non-slip finish, and was free of major cracking (Figure 9-6). However, leaves and organic material were found on the roof. No DFT measurements were able to be taken on the roof due to the textured coating. The roof had a grade that ranged from approximately 3 degrees near the vent to approximately 20 degrees near the reservoir roof's perimeter. The distance from the roof edge to the ground level is around 16 feet.

Figure 9-6: Organic material buildup on the roof (Left) and roof vent (Right).

9.2.1.5 Exterior Appurtenances

Two retrofitted ladders provide access to the reservoir roof (Figure 9-7 Left). The distance from the top of the roof curb of the ground level is approximately 9.5 feet. The distance from the grating near the access hatch to the roof of the valve vault is approximately 10 feet. As is typical with the

City's reservoirs of this construction, the retrofitted grating and safety railing extend only around the entry hatch and is missing a toeboard (Figure 9-7 Right). The entry hatch has a thin contact area with the lid and the gasket was not securely attached.

Figure 9-7: The exterior ladders and roof entry hatch

Because the sheet metal collar could not be removed, the 40th Street Reservoir's vent (Figure 9-6 Left) was assessed with the provided drawings and photos of the College Way Reservoir vent retrofit (Figure 9-8 Left). A 3-foot diameter penetration in the roof has a concrete vent structure above it. The concrete overhang is 4 feet, 2 inches. Approximately 2 inches of opening was exposed under the sheet metal collar. The screen was assumed to be similar to the Dakin I screen, woven with approximately 12 strands per inch. As viewed from the interior, the screen was found to be peeling off from the base of the roof with concrete spalling (Figure 9-8 Right). It is likely that the outer screen is still in place.

Figure 9-8: Provided photo of the College Way Roof Vent retrofit from January 2013 (Left) and photo taken of the vent from the interior of the 40th Street reservoir (Right).

The attached valve vault shares a wall and footing with the reservoir. The inlet, outlet, and drain pipes all run through the vault. The overflow runs through via an air gap, connecting with the drain pipe. The vault's roof has a curb that extends all the way around the perimeter of the roof (Figure 9-9).

Figure 9-9: The valve vault curb extends completely around without overflow drainage. The overflow pipe has an air gap. Organic material was building up on the vault roof.

9.2.2 Interior Inspection Summary

9.2.2.1 Interior walls

The coating on the interior walls made it difficult to determine the extent, if any, of the internal cracking of reservoir walls (Figure 9-10). A few areas of repair were visible and looked to be holding well. The measured DFT ranged from 80-130 mils; the specified DFT was 80 mils as per the supplied documents from the City.

Figure 9-10: The reservoir's vertical walls (Left) and hopper base (Right)

9.2.2.2 Floor

The floor was coated and covered by a thin layer of sediment, which made it difficult to inspect the condition (Figure 9-11). Some coating blistering of the floor was found; no blisters had burst.

Figure 9-11: The reservoir's floor was covered in sediment; visible coating bubbling but appeared to be intact.

9.2.2.3 Interior Roof

Concrete spalling was noted near the vent penetration (Figure 9-8 Right) but the roof only exhibited minor cracking and efflorescence in other areas (Figure 9-12).

Figure 9-12: Other than the concrete spalling near the vent, the reservoir roof did not exhibit major issues.

9.2.2.4 Interior Appurtenances

The interior ladder was rungs set into the wall. While the upper rungs within the access vault are still present, the rungs on the wall were removed during the coating process and have not been replaced. A removable ladder, tripod, and winch are used for inspections and maintenance.

Figure 9-13: Removable ladder and common inlet/outlet/drainpipe

9.2.3 Piping and Valving Inspection Summary

The combined inlet/outlet/drain piping exhibited corrosion nodules on the interior of the pipe (Figure 9-14 Left). The valve vault contained isolation valves and an altitude valve (Figure 9-14 Right). The overflow weir gate components (Figure 9-15 Left) exhibited corrosion. The overflow has an air gap (Figure 9-9). The drainpipe connects with the overflow and daylights to a vegetated area to the west of reservoir (Figure 9-15 Right).

Figure 9-14: The common inlet/outlet/drainpipe corrosion nodules (Left) and valving (Right).

Figure 9-15: Overflow weir (Left) and drain/overflow outlet without energy dissipation or a screen and poor dechlorination (Right).

A hose bibb for washdown operations is located within the valve vault.

9.3 Condition Assessments

9.3.1 Cleanliness and Coatings

Based on the inspection results, the exterior coating needs cleaning to extend its life 10 years (+/-) into the future. The areas of missing coating near the base also indicate a fair condition of the exterior coating system.

There is a moderate layer of sediment on the floor of reservoir. The internal coating blisters on the reservoir floor are due to groundwater effects or improper coating application. If caused by groundwater, infiltrated rainwater would likely be transmitted laterally on the relatively impermeable bedrock which the reservoir is founded on. Water beneath the reservoir would exert an upward pressure on the its base inducing bubbling in the coating. If coating application is the culprit, the maximum time between primer and polyurea may have been exceeded. At any rate, the blisters have not yet burst and do not appear to be an issue at this point. As these bubbles are not cracking and haven't burst, the coating is still functioning, and is in good condition.

9.3.2 Material Deterioration

Significant concrete deterioration was noted throughout the exterior, which needs attention. Because this cracking is structural in nature, it is discussed in 9.3.3.1. The interior coating has likely significantly decreased the rate of efflorescence accumulation around the exterior cracks as water is inhibited from passing through the concrete.

On the interior, the concrete spalling near the roof vent is likely a result of the reinforcing placed with inadequate thickness of concrete. Water can corrode the reinforcing members, causing the rust to expand and the concrete layer to cleave. Observing at the surface, the breaks are flat and

shallow which seems to support this as a possible failure method. The vent is in poor condition due to this concrete failure and screen peeling.

The combined inlet/outlet/drain piping was in poor visual condition with the corrosion nodules. Corrosion noted on the overflow weir gate components is moderate.

9.3.3 Structural Performance

9.3.3.1 Static Analysis

The dome roof was found to meet modern requirements of AWWA D110-13. However, the roof-to-wall interface is not designed to allow for roof expansion and contraction, which has resulted in cracks. Based on the provided design drawings, the wall reinforcing is adequate to handle the 23-foot maximum operating level. Cracks along the midline of the reservoir are likely caused by the walls being under reinforced. The maximum level the reservoir could be operated while still meeting static requirements is 18 feet. Another reinforcing deficiency is the inside face of the upper walls have reinforcing that is spaced 24 inches on center, which exceeds ACI 350.3's maximum allowable spacing of 12 inches on center. The foundation was found to be sized appropriately, based on the geotechnical report's soil maximum bearing capacity of 6,000 psf.

9.3.3.2 Seismic Analysis

With the walls' reinforcing already failing under static loads, they further fail design requirements for seismic forces. The operating level would need to be further reduced to 16 feet to meet design requirements. At the 23-foot operating level, the design seismic event is estimated to induce a constrained slosh wave. However, the roof reinforcing is suitable to handle the additional load. The valve vault attachment is suitable to prevent differential movement and impact during the seismic event. Based on assumed backfill conditions, the hopper base is not likely to fail and collapse onto the piping. However, the pipes do not have flexible couplings which would address potential differential settlement between the two structures during a seismic event.

9.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A removable silt stop is required on combined drains/outlets but was not installed. Since the reservoir drains and overflows to daylight, DOH approved air gaps are not required. The missing screen on the drain/overflow outlet and damaged screen on the vent could lead to water quality issues in the reservoir. While the screen on the vent could not be carefully measured due to the sheet metal collar, it does not appear to be 24-mesh screen backed with 4-mesh screen. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gasket to provide a tight seal against the relatively small surface area of the frame. The gasket not being fully adhered could potentially lead to water quality issues within the reservoir. Overall, the water quality/sanitary condition of the reservoir is fair.

9.3.5 Safety

With the distance being about 10 feet on the exterior ladders, no ladder fall protection system is required. This concrete dome reservoir has a higher distance from the roof to the ground than any of the other dome reservoirs, about 16 feet. Additionally, this roof would be considered steep, as the greatest slope is greater than 4:12. Thus, fall protection on the reservoir and valve vault roofs is required. A toeboard is also required on the existing grating and around the reservoir to protect workers below from dropped items.

With the fall protection and ladder on the interior of the reservoir being non-existent, a removable ladder, tripod, and winch were required for our inspection. While this configuration appears to meet L&I requirements, there is inherent risk of using a tripod mounted on a vault and a removable ladder. While the exterior ladder safety condition is good, the roof and interior ladder safety condition is poor.

9.3.6 Operations and Maintenance

9.3.6.1 Site and Security

The fence was found to be suitable for this reservoir, the site graded appropriately, and the soils drained adequately. An intrusion alarm is installed on the roof's access hatch. Trees were too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is in fair condition.

9.3.6.2 Roof Drainage

The reservoir roof sheds water adequately. However, drainage of the valve vault roof appeared to not function in the event of a drain clog. If the drain were to be clogged, the roof would overflow the curb; no notches or overflow scuppers are present that would send water away from the reservoir. There are apparently no waterstops installed between the reservoir and valve vault.

9.3.6.3 Appurtenances

No difficulties were found using the exterior ladder. However, accessing the interior of the reservoir is more difficult than other style reservoirs as the tripod, winch, and removable ladder must be lifted to the roof. However, the access vault did have a ladder on the side, which made it easier to access the hatch than the other domed reservoirs. The 40th Street Reservoir met design requirements for the vent on the roof penetration, screened area, and free area on the retrofitted collar.

9.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. However, the dechlorination and energy dissipation from the drain/overflow pipe did not appear to function properly.

9.3.7 Obsolescence

The equipment on site appeared to be of older design, but the components could be replaced if needed.

9.3.8 Condition Scoring

Overall, the condition score of the 40th Street Reservoir was fair, 3.6. Problems associated with this reservoir were related to the roof-to-wall interface and underenforcement of the walls. A summary of the scoring is shown in Table 9-1 and the full Score Matrix is included in Appendix G.

Table 9-1: Condition Scoring of the 40th Street Reservoir

Categorical Scores	
Cleanliness and Coatings	4.4
Material Deterioration	3.9
Structural Performance - Static	4.0
Structural Performance - Seismic	3.2
WQ/ Sanitary	3.9
Safety	3.0
Operations & Maintenance	4.1
Obsolescence	3.0
Overall	**3.6**

9.4 Recommended Improvements

9.4.1 Cleanliness and Coatings

First, the exterior should be pressure washed to remove dirt and other debris. Areas of coating failure on the reservoir exterior that are not related to thermal deformation, such as the roof and on the lower reservoir walls can be spot coated.

The interior should be washed out to remove the sediment that has accumulated on the floor. This should be done at a low pressure so as not to burst any of the blisters on the floor. The blisters should continue to be monitored and photographed during regular inspections to determine if they are getting larger, increasing in number, cracking, or bursting.

9.4.2 Material Deterioration

Discussed in Section 9.4.3 are possible solutions to deal with issues associated with the thermal expansion-related to the roof-to-wall interface and under-reinforcement.

At the roof vent penetration, the spalling concrete is recommended to be removed. Then this area can be cleaned, and the reinforcing members can be recoated. Finally, the area can be patched and concrete coated. The interior roof near the vent is expected to be difficult to access.

The corrosion on the interior of the inlet/outlet/drain pipe is recommended to be removed by mechanical agitation or by pigging. The interior of the pipe near the reservoir can be recoated or a cured in place pipe liner can be installed if corrosion jeopardizes the pipe's integrity. Weir gate component corrosion can be addressed with cleaning and recoating.

9.4.3 Structural

The recommended improvement to address the issues related to thermal deformation occurs in stages:

1. A temporary improvement should be employed soon to halt further deterioration of any reinforcing material until a permanent solution is implemented. This would entail removing damaged concrete from the cracking areas, properly cleaning cracks, and assessment by a structural engineer. Likely, the areas will need to be spot coated to prevent water damage to the reinforcing. It is imperative that any repair media be flexible so as not to accelerate structural damage as thermal movement is likely to continue to occur until the problem is addressed. Stiffening or reinforcing the cracking areas is not recommended.

2. To more permanently address the thermal deformation-related issues, several options are available. A new aluminum geodesic roof is recommended. The City can also consider a new concrete roof that is domed or flat. Another option would be lifting the entire roof and retrofitting it with a bracketing system and elastomeric bearing pad between the roof and walls. These options would likely cost much more than a new aluminum geodesic roof.

3. The cracks should be investigated by a structural engineer after the thermal deformation issue is resolved.

To address the under-reinforcement, the maximum operating level needs to be reduced to 16 feet above the floor. Retrofits may be possible on the reservoir if a higher maximum operating level is needed. This would likely first entail employing a testing firm to map the existing reinforcing and test the concrete compressive strength and reinforcing's yield strength to verify the actual values. Then the reservoir can be reevaluated, and an updated maximum level recommended. If this level is still insufficient, fiberglass reinforced polymer (FRP) fabric can be applied to areas determined

to be too weak. The City should also consider lowering the overflow to ensure it is not operated above the maximum allowable operating level in the future.

Because the valve vault shares a footing with the reservoir, there isn't space available to install flexible connections on the pipes. Instead, a new vault should be constructed with sufficient space between it and the reservoir to allow for the installation and proper operation of flexible connections.

9.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. A removable silt stop should be installed on the combined drain/outlet pipe. The overflow/drain pipe outlet can be retrofitted with a #24-mesh and 4-mesh screen backing.

The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area. The vent screen, if in fact is not compliant, it can be replaced with a #24-mesh and 4-mesh screen backing.

9.4.5 Safety

On the exterior, an encompassing L&I-compliant railing is recommended around the perimeter of the reservoir and valve vault roofs. Additionally, on the reservoir roof, a fall restraint system or positioning device system, or possibly a fall arrest system if the work is considered roofing work, is recommended. A toe board should be added to the existing grating.

On the interior, steps should be taken to ensure ingress and egress to the reservoir is as safe as possible. A new interior ladder with fall protection system is recommended. This could be similar to the ladder installed at the Kearney Reservoir, which has a removable extension (Figure 9-16).

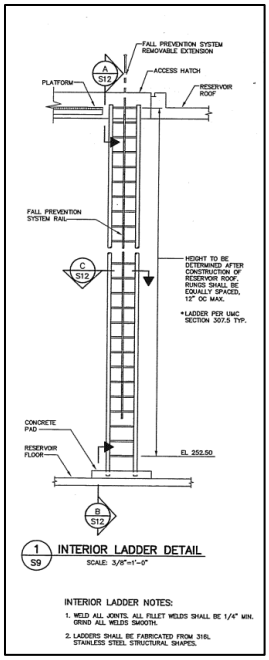

Figure 9-16: The Kearney Reservoir's ladder design, of which a similar design could be implemented the dome Reservoirs.

9.4.6 Operations and Maintenance

9.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

9.4.6.2 Roof Drainage

On the valve vault roof curb, notches or overflow scuppers should be installed to prevent the roof from overflowing the curb if the drain backs-up.

9.4.6.3 Appurtenances

The are no recommended improvements related to the operations and maintenance for the appurtenances on the 40th Street Reservoir.

9.4.6.4 Valving and Piping

Proper dechlorination of the 40th Street Reservoir will need to consist of a weighted system that is set in the manhole on site, similar to that shown in Figure 4-21 or attached to the outlet pipe. The dechlorination system should be updated before the next time the reservoir is drained. The energy dissipation could consist of rip rap applied to the outlet.

9.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the 40th Street Reservoir are show in Table 9-2.

Table 9-2: 40th Street deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 2,000
	Intermittent exterior coating failure	Spot Repair Coating	$ 5,000
	Sediment accumulation on floor	Perform regular washdown inspections	$ 1,000
	Blisters on interior floor coating	Monitor for blister growth and bursting. Draw down cautiously and avoid rupturing blisters.	$ -
DETERIOR-ATION	Interior concrete spalling around vent penetration	Remove spalled concrete, clean, patch and coat	$ 10,000
	Corrosion overflow weir gate components	Clean and (re)coat	$ 1,000
	Corrosion on interior of piping	Remove corrosion with agitation or pig pipe. Recoat where possible or consider cured in place pipe liner	$ 10,000
STRUCT	Roof-wall-interface does not account for thermal deformation	Resolve roof-wall-interface issues	
		a. Clean & coat cracks related to thermal deform.	$ 10,000
		b. Replace roof with new geodesic dome	$ 140,000
		c. Structural inspection to determine next steps	$ 2,000
	Walls under-reinforced, likely resulting in wall cracking on lower portions of walls	Remove cracked concrete, clean & coat reinforcement material, patch and coat concrete. Select Option 1 or 2.	$ 10,000
		OPTION 1: Lower max operating level from 23 to 16 ft	
		Subtotal for Option 1	**$ 10,000**
		OPTION 2: Maximum operating level above 16 ft	
		a. Map existing reinforcing and test concrete strength	$ 5,000
		b. Apply fiberglass reinforced polymer (FRP) fabric to areas under-reinforced.	$ 400,000
		Subtotal for Option 2	**$ 415,000**
	Operated at the overflow elevation, seismic slosh impacts roof and structure is under-reinforced	Consider lowering overflow	$ -
	Missing seismic valves & vault adjacent to reservoir	New seismic valve vault and associated piping	$ 200,000
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	No silt-stop on combined drain/outlet	Install removable silt-stop	$ 1,000
	Drain pipe screen non-compliant	Replace drain pipe screen	$ 1,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Lacks steep roof fall protection, railings, and grating railing toeboard	Install roof fall protection, railing on heights greater than 4 feet, and toeboard on grating railing	$ 50,000
	Missing interior ladder	Interior ladder and fall protection replacement	$ 30,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 24,000
	Valve vault roof lacks overflow drainage	Install notches or overflow scuppers on parapet	$ 2,000
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000

	Max operating Level (ft)	
	16	>16
Subtotal	$ 505,000	$ 910,000
30% Contingency	$ 151,500	$ 273,000
8.7% Tax	$ 57,116	$ 102,921
Engineering, Administration, and Construction Management	$ 249,765	$ 450,072
Total Project Costs Estimate	$ 963,381	$ 1,735,993
Total Project Costs Estimate (rounded)	**$ 960,000**	**$ 1,700,000**

9.5 Conclusion

The 40th Street Reservoir, built in 1958, has major structural issues. The City should determine if the 16-foot maximum operating level is feasible. This 7-foot reduction in maximum operating level is the highest recommended drop of any of the reservoirs. Coupled with the lack of ability of the roof-wall-interface to deal with roof thermal movement and the missing flexible pipe couplings, the under-reinforcement issue will likely tip this reservoir in the side of replacement over rehabilitation. The City should weigh the cost/benefit of reservoir rehabilitation vs. replacement.

Because the interior has sediment accumulation, this reservoir should be prioritized in its next washout inspection. This should take place within the next 2 years. The reservoir should be leak tested as a first step of the next inspection. The internal blistering, while not a concern at this time, should be inspected at 3- to 5- year intervals to determine if the problem is progressing.

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Section 10

College Way Reservoir

10.1 Tank and Site Overview

The College Way Reservoir, shown in Figure 10-1, is one of five similar dome reservoirs that were built between 1958 and 1987. While as-built drawings for it are unavailable, drawings of the 40th Street and Reveille Reservoirs, which were built in the same period, designed by the same consulting engineering group, and are of similar construction are available. With slight differences in dimensions, the five reservoirs are 0.3- or 0.5-MG reinforced concrete, self-supporting, domed roof hoppers that were originally built with common inlet/outlets/drains. The reservoirs were designed to hold water above the roof-to-wall interface. This reservoir is partially buried, and the attached valve vault is located below roof. It was built in 1968 and has a capacity of 0.5 MG.

Figure 10-1: The College Way Reservoir, viewed looking east

According to measurements taken in the field (Figure 10-2), the College Way Reservoir has an interior diameter of 64 feet. The hopper base is 5.5 feet deep and the interior wall is 13.5 feet high. The overflow weir was measured to be 23.5 feet above the reservoir floor, which according to City-supplied documentation, corresponds to 541 feet City. This height is higher than the top of the vertical wall. The operating range is 14 to 19 feet. Wall and roof reinforcement were assumed to be similar to the 40th Street and Reveille Reservoirs. The interior walls and floor of the College Way Reservoir were coated in 2013 using a VersaFlex AquaVers primer and polyurea.

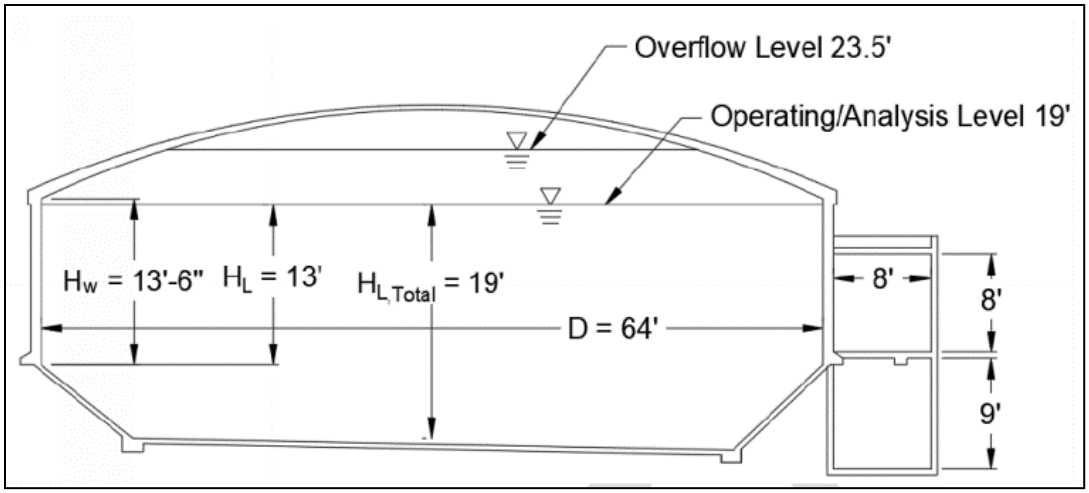

Figure 10-2: The measured interior dimensions of the College Way Reservoir

The site where the College Way Reservoir is situated in a suburban residential neighborhood, adjacent to Western Washington University. It is about one mile south-southwest of the Bellingham CBD. It is located within the 457 South Zone and provides service to the 541 College Way Zone via the College Way Booster Pump Station. The valve vault shares a wall with the reservoir itself and is 8 feet wide by 8 feet deep. The College Way Pump Station was built near the reservoir in 2006. A new hydraulic mixing pipe and outlet to the pump station were constructed at this time.

The geotechnical investigation indicated Chuckanut Sandstone from the surface to the extent of the boring to refusal, 5.5 feet. Weathered Chuckanut Sandstone was found in the top 3 feet of the boring, which consisted of very dense fine to coarse sand. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the College Way Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. Bedrock is not at risk of liquification and the site is not expected to have issues with slope instability.

10.2 Inspection Summary

A drained inspection of the College Way Reservoir took place on March 14, 2019.

10.2.1 Exterior Inspection Summary

10.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the property (Figure 10-1). The site is generally well-graded, directing runoff away from the reservoir.

During the site visit, it was noted that the roof of the reservoir is under the dripline of the trees and other vegetation (Figure 10-3).

Figure 10-3: Trees overhang the reservoir roof and coating failure

10.2.1.2 Exterior Walls

Of the visible portion of the exterior walls above grade, cracking with efflorescence was noted throughout (Figure 10-4). This cracking was found just below the roof overhang and spread diagonally to the attached vault. Multiple locations were noted where the exterior sidewall was missing the topcoat and there were locations where bare concrete was exposed (Figure 10-5). The DFT of the sidewalls ranged from 8-14 mils.

Figure 10-4: Areas of cracking with efflorescence and coating failure on the exterior walls

Figure 10-5: Topcoat loss (Left) and area of missing coating near the base of the reservoir (Right)

10.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

10.2.1.4 Exterior Roof

Approximately 15 percent of the exterior textured roof topcoat is gone, localized in the east quadrant where vegetative debris is allowed to accumulate (Figure 10-6 Left). The undercoating appears to be fairly tightly adhered to the concrete surface. The most significant item noted in the roof was a circumferential crack that ran around the entire circumference of the roof approximately half-way between the center and edge (Figure 10-6 Right). No DFT measurements were able to be taken on the roof due to the textured non-slip coating. The roof had a grade that ranged from 3.4 degrees near the vent to 19.9 degrees near the reservoir roof's perimeter. The distance from the roof to ground level ranges from 4 feet to 10 feet.

Figure 10-6: Debris buildup and coating failure and a minor circumferential crack

10.2.1.5 Exterior Appurtenances

To access the reservoir and vault roof, a single retrofitted ladder is used (Figure 10-7). The total distance from the top of the stairs to the grating is approximately 10 feet. The stair adds an additional 6 feet to the ground level surface. As is typical with the City's reservoirs of this

construction, the retrofitted grating and safety railing extend only around the entry hatch and is missing a toeboard. The entry hatch has a thin contact area with the lid and the gasket was not securely attached.

Figure 10-7: Retrofitted ladder and roof entry hatch

Because the sheet metal collar could not be removed, the College Way Reservoir's vent (Figure 10-8 Left) was assessed via the provided drawings of similar reservoir and photos of its vent retrofit (Figure 10-8 Right). A 3-foot diameter penetration in the roof has a concrete vent structure above it. The concrete overhang is 4 feet, 2 inches. Approximately 2 inches of opening was exposed under the sheet metal collar. The screen was assumed to be similar to the Dakin I screen, woven with approximately 12 strands per inch.

Figure 10-8: The roof vent (Left) and a provided photo of the College Way Roof Vent retrofit from January 2013 (Right)

The attached valve vault shares a wall and footing with the reservoir. The inlet and drain pipes run through the vault. The overflow runs through via an air gap, connecting with the drain pipe. The vault roof has curb that extends all the way around (Figure 10-7 Left). No waterstop has been installed on the join between the valve vault and the reservoir and water was found to be pooling up within the lower level of the valve vault (Figure 10-9).

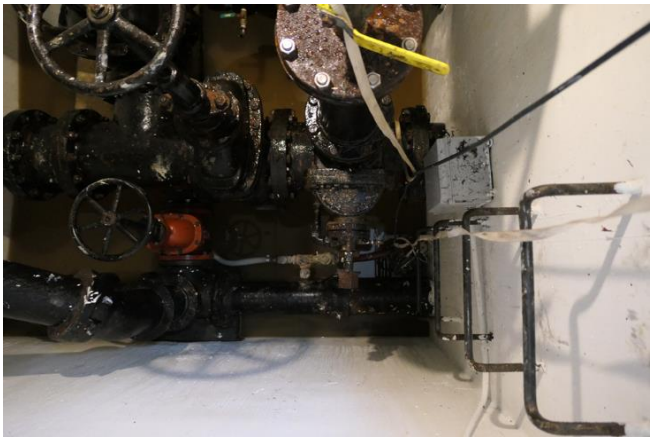

Figure 10-9: Water accumulating in the bottom of the valve vault

10.2.2 Interior Inspection Summary

10.2.2.1 Interior walls

The coating on the interior walls made it difficult to determine the extent, if any, of the internal cracking of reservoir walls. The coating was intact (Figure 10-10). The measured DFT ranged from 80-110 mils; the specified DFT was 80 mils as per the supplied documents from the City.

Figure 10-10: Interior walls and removable ladder (Left). A retrofitted outlet and hydraulic mixing system are installed on the reservoir (Right).

10.2.2.2 Floor

The floor was coated and covered by a thin layer of residue and leftover water (Figure 10-11). From what could be seen, the coating was intact and there was no cracking.

Figure 10-11: The reservoir floor's thin layer of residue

10.2.2.3 Interior Roof

The interior roof did not exhibit cracking (Figure 10-12).

Figure 10-12: The reservoir interior roof was free of observable issues.

10.2.2.4 Interior Appurtenances

The interior ladder was rungs set into the wall. While the upper rungs within the access vault are still present, the rungs on the wall were removed during the coating process and have not been replaced. A removable ladder, tripod, and winch are used for inspections and maintenance. A pipe for the level sensor was near the ladder but did not inhibit entry.

10.2.3 Piping and Valving Inspection Summary

The combined inlet/drain piping exhibited only minor corrosion on the interior of the pipe (Figure 10-13 Left). The attached valve vault contained isolation valves and an altitude valve. Based on the updated College Way Pump Station, the reservoir outlets to the pump station. Minor turbuculation

was noted on the outlet pipe. The drainpipe connects with the overflow and extends to the storm sewer without an air gap (Figure 10-9). Standing water was noted at the bottom of the valve vault.

Figure 10-13: The common inlet/drain (Left) and the outlet (Right) exhibit minor tuberculation.

The overflow weir gate components, the exterior overflow pipe, and interior hydraulic mixing pipe exhibited corrosion. (Figure 10-14) The overflow pipe has an approximate 12-inch air gap. It connects with the drain line in the vault, before going to the storm sewer.

Figure 10-14: Overflow weir gate (Left), exterior overflow pipe (Center), and interior hydraulic mixing pipe (Right) corrosion. Organic material was building up on the vault roof (Center).

No washdown piping was observed during our inspection.

10.3 Condition Assessments

10.3.1 Cleanliness and Coatings

The exterior topcoat on the sidewalls and roof is no longer functional in many places. The organic material accumulation, topcoat deterioration, and exposed concrete indicate the walls are roof are in poor condition. However, the coating on the valve vault was in good condition. If cleaned and spot repaired, the life of the exterior coating can be extended another 10 (+/-) years. There was significant organic debris accumulating within the valve vault, however.

The interior floor has minor sediment on the floor, but the interior coating is in good condition.

10.3.2 Material Deterioration

Significant concrete deterioration was noted throughout the exterior walls, which needs attention. Because this cracking is structural in nature, it is discussed in Section 10.3.3.1. The interior coating has likely significantly decreased the rate of efflorescence accumulation around the exterior cracks as water is inhibited from passing through the concrete.

Corrosion on the ductile iron hydraulic mixing pipe was significant, caused by a metallic couple with the rebar within the walls. The spool piece is coated, which has saved it from heavy tuberculation while the flanged end is uncoated and is exhibiting heavy tuberculation. A similar phenomenon is causing the minor tuberculation in the pump station outlet pipe. Corrosion on the exterior overflow pipe and overflow weir components is likely due to the pipe not being properly coated recently.

10.3.3 Structural Performance

10.3.3.1 Static Analysis

The dome roof was found to meet modern requirements of AWWA D110-13. However, the roof-to-wall interface is not designed to allow for roof expansion and contraction, which has resulted in cracks. The structure is braced at the valve vault, which is why the cracking extends diagonally near to it from the roof. Assuming the walls are reinforced similar to the 40th Street and Reveille Reservoirs' plans, the wall reinforcing appears to be acceptable for static loads. However, the upper sections of the interior wall have reinforcing at 24 inches on center, which exceeds ACI 350.3's maximum allowable spacing of 12 inches on center. The foundation was found to be sized appropriately, based on the geotechnical report's soil bearing capacity of 6,000 psf.

10.3.3.2 Seismic Analysis

At the 17-foot operating level, the walls' reinforcement was found to be suitable for the design seismic event. This was not the case if operated at the overflow level. Similarly, the design seismic

event is estimated to cause a constrained slosh wave, causing damage to the roof if operated at the overflow level. However, at the 19-foot operating level, the roof is adequately reinforced. Incidental damage may still occur to hatches and appurtenances. The valve vault attachment is suitable to prevent differential movement and impact during the seismic event. Also, based on assumed backfill conditions, the hopper base is not likely to fail and collapse onto the piping. However, the pipes do not have flexible couplings which would address potential differential settlement between the two structures during a seismic event.

10.3.4 Water Quality/Sanitary

The retrofitted inlet/outlet pipe configuration with hydraulic mixing system is likely suitable to promote high water quality within the reservoir. As this reservoir drains to a storm drain, a DOH approved air gap is required on both the overflow and the drainpipe. No air gap is installed on the drain pipe. The air gap between the overflow and receiving pipe was found to be too narrow. While the screen on the vent could not be carefully measured due to the sheet metal collar, it does not appear to be 24-mesh screen backed with 4-mesh screen. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gasket to provide a tight seal against the relatively small surface area of the frame. The gasket not being fully adhered could potentially lead to water quality issues within the reservoir. Overall, the water quality/sanitary condition is good.

10.3.5 Safety

With the distance being about 10 feet on the exterior ladder, a ladder fall protection system is not required. This reservoir roof has some areas where a fall would be greater than 4 feet. A fall would be greater than 4 feet from the valve vault as well. Thus, an L&I-compliant railing is required on these areas. The reservoir roof is considered steep as the greatest slope is greater than 4:12. Thus, additional fall prevention is required.

With the fall protection and ladder on the interior of the reservoir being non-existent, a removable ladder, tripod, and winch were required for our inspection. While this configuration appears to meet L&I requirements, there is inherent risk of using a tripod mounted on a vault and a removable ladder. While the exterior ladder safety condition is good, the roof and interior ladder safety condition is fair to poor.

10.3.6 Operations and Maintenance

10.3.6.1 Site and Security

The fence and drainage were found to be suitable for this reservoir. An intrusion alarm is installed on the roof's access hatch. However, trees were too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is in fair condition.

10.3.6.2 Roof Drainage

The reservoir roof sheds water adequately. However, drainage of the valve vault roof appeared to not function in the event of a drain clog. If the drain were to be clogged, the roof would overflow the curb; no notches or overflow scuppers are present that would send water away from the reservoir. There are apparently no waterstops installed between the reservoir and valve vault. The water in the bottom of the valve vault may be a result of this or a leaking pipe.

10.3.6.3 Appurtenances

No difficulties were found using the exterior ladder to access the roof. However, accessing the interior of the reservoir is more difficult than other style reservoirs as the tripod, winch, and removable ladder must be lifted to the roof. College Way does not have a ladder on the exterior of the roof access hatch vault, so climbing into the hatch can also be challenging.

Due to the pump station upgrade, the College Way Reservoir has a larger outlet pipe than the other similar reservoirs. Even with the pipe being higher in the reservoir, water would drain at a higher flow rate than a failure of the existing drain pipe. Using screen and vent information from the Reveille as-built drawings indicates the penetration, screened, and retrofitted collar free area are sufficient for the max operating height. The vent is sized for operations up to 22.5 feet, 1-foot below the overflow level.

10.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. No dechlorination system was observed onsite for drained water.

10.3.6.5 Misc.

Although the hydraulic mixing system could not be observed in operation, it appeared to be suitable for promoting good water quality within the reservoir.

10.3.7 Obsolescence

With the new equipment installed in the College Way Pump Station, the obsolescence favorable.

10.3.8 Condition Scoring

Overall, the condition score of the College Way Reservoir was fair, 3.9. Problems associated with this reservoir were related to the structural problems with the roof-wall-interface. A summary of the scoring is shown in Table 10-1 and the full Score Matrix is included in Appendix H.

Table 10-1: Condition Scoring of the College Way Reservoir

Categorical Scores	
Cleanliness and Coatings	3.8
Material Deterioration	3.7
Structural Performance - Static	4.1
Structural Performance - Seismic	3.7
WQ/ Sanitary	4.5
Safety	3.3
Operations & Maintenance	4.1
Obsolescence	5.0
Overall	**3.9**

10.4 Recommended Improvements

10.4.1 Cleanliness and Coatings

First, the exterior should be pressure washed to remove dirt and other debris. Areas of coating failure on the reservoir that are not related to thermal deformation, such as the roof can be spot coated.

The interior should also be washed out to remove the sediment that has accumulated on the floor.

10.4.2 Material Deterioration

Discussed in Section 10.4.3 are possible solutions to deal with the thermal expansion-related issues of the roof-to-wall interface.

The corrosion on the exterior overflow, outlet pipe, mixing pipe, and overflow weir gate components can be addressed with cleaning and recoating.

10.4.3 Structural

To confirm the assumptions made in the analysis, as-built drawings may be obtained, or a testing firm could be employed to map the existing reinforcing.

The recommended improvement to address the issues related to thermal deformation occurs in stages:

1. A temporary improvement should be employed soon to halt further deterioration of any reinforcing material until a permanent solution is implemented. This would entail removing damaged concrete from the cracking areas, properly cleaning cracks, and assessment by a structural engineer. Likely, the areas will need to be spot coated to prevent water damage to the reinforcing. It is imperative that any repair media be flexible so as not to accelerate

structural damage as thermal movement is likely to continue to occur until the problem is addressed. Stiffening or reinforcing the cracking areas is not recommended.

2. To more permanently address the thermal deformation-related issues, several options are available. A new aluminum geodesic roof is recommended. The City can also consider a new concrete roof that is domed or flat. Another option would be lifting the entire roof and retrofitting it with a bracketing system and elastomeric bearing pad between the roof and walls. These options would likely cost much more than a new aluminum geodesic roof.

3. The cracks should be investigated by a structural engineer after the thermal deformation issue is resolved.

The best approach to deal with the wall reinforcement being spaced 24 inches on center rather than 12 inches on center is to monitor the structure. The City should also consider lowering the overflow to ensure it is not operated above the maximum allowable operating level in the future.

Because the valve vault shares a footing with the reservoir, there isn't space available to install flexible connections on the pipes. Instead, a new vault should be constructed with sufficient space between it and the reservoir to allow for the installation and proper operation of flexible connections.

10.4.4 Water Quality/Sanitary

A removable silt stop should be installed on the combined drain/outlet pipe. A DOH-approved air gap should also be installed on the drain pipe. With the drain pipe being 9 feet underground, this would likely be logistically challenging at the College Way site. Possible options are draining to daylight, an appropriately sized dedicated dry well with backflow prevention, or an air gap with a sump and sump pump that can pump water than cannot be drained by gravity.

The overflow air gap can be widened on the reservoir. A trash rack is recommended on the receiving pipe. With the wall of the reservoir close to the overflow, the gap needs to be three times the pipe diameter.

The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area. The vent screen, if in fact is not compliant, it can be replaced with a #24-mesh and 4-mesh screen backing.

10.4.5 Safety

On the exterior, an L&I-compliant railing is recommended around valve vault roof. The railing will also need to extend around the perimeter of the reservoir at all areas where the fall distance is greater than 4 feet. Additionally, on the reservoir roof, a fall restraint system or positioning device

system, or possibly a fall arrest system if the work is considered roofing work, is recommended. A toe board should be added to the existing grating.

On the interior, steps should be taken to ensure ingress and egress to the reservoir is as safe as possible. A new interior ladder with fall protection system is recommended. This could be similar to the ladder installed at the Kearney Reservoir, which has a removable extension and is shown in Figure 9-16. A new exterior ladder should be installed on the exterior of the access vault that is similar to the ladder installed on the 40th Street Reservoir, shown in Figure 9-7 Right.

10.4.6 Operations and Maintenance

10.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

10.4.6.2 Roof Drainage

To address the standing water in the bottom of the valve vault, the water should be pumped out. The joints between the valve vault and reservoir should be sealed to prevent leakage and pipes checked for leakage. If water returns, the source of the leak should be identified and sealed. A working sump pump can also be installed in the vault. On the valve vault roof curb, notches or overflow scuppers should be installed to prevent the roof from overflowing the curb if the drain backs-up.

10.4.6.3 Appurtenances

The vent is undersized for the overflow elevation but is sized appropriately for the maximum reported operating level of 19 feet and up to 22.5 feet. Thus, the recommended improvement is to continue operating within the reported range. Other than inspecting the vent to verify the assumptions made in the analysis, there are no recommended improvements for the appurtenances, related to operations and maintenance at the College Way Reservoir.

10.4.6.4 Valving and Piping

Proper dechlorination of the College Way Reservoir will need to consist of a weighted system that is set in the manhole. An example of a basket system is shown in Figure 4-21.

10.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the College Way Reservoir are shown in Table 10-2.

Table 10-2: College Way deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 2,000
	Intermittent exterior coating failure	Spot Repair Coating	$ 6,000
DETERIOR-ATION	Corrosion on outlet pipe, exterior overflow, mixing pipe, & overflow weir gate components	Clean and (re)coat	$ 3,000
STRUCT	Roof-wall-interface does not account for thermal deformation	Resolve roof-wall-interface issues a. Clean & coat cracks related to thermal deform. b. Replace roof with new geodesic dome c. Structural inspection to determine next steps	 $ 11,000 $ 150,000 $ 2,000
	Maximum reinforcement spacing exceeded	Continue to monitor for structural issues	$ -
	Operated at the overflow elevation, seismic slosh impacts roof and structure is under-reinforced	Consider lowering overflow	$ -
	Missing seismic valves & vault adjacent to reservoir	New seismic valve vault and associated piping	$ 200,000
WQ	No silt-stop on combined drain/outlet	Install removable silt-stop	$ 1,000
	No drain air gap	Upgrade drain pipe w/ air gap	$ 88,000
	Overflow air gap too narrow	Retrofit overflow pipe air gap	$ 5,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Lacks steep roof fall protection, railings, and grating railing toeboard	Install roof fall protection, railing on heights greater than 4 feet, and toeboard on grating railing	$ 50,000
	Ingress and egress difficult and missing interior ladder	Install exterior access vault ladder and install interior ladder with fall protection	$ 32,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 28,000
	Valve vault roof lacks overflow drainage	Install notches or overflow scuppers on parapet	$ 2,000
	Water in bottom of valve vault	Seal vault-to-reservoir interfaces and check for leaking pipes. Install sump pump if water is still present	$ 5,000
	Dechlorination non-compliant	Update dechlorination system	$ 2,000
		Subtotal	$ 589,000
		30% Contingency	$ 176,700
		8.7% Tax	$ 66,616
		Engineering, Administration, and Construction Management	$ 291,311
		Total Project Costs Estimate	$ 1,123,626
		Total Project Costs Estimate (rounded)	**$ 1,100,000**

10.5 Conclusion

The two biggest items to be dealt with on the College Way Reservoir are the thermal deformation issues and the lack of flexible couplings on the pipes passing through the valve vault. The new seismic valve vault will likely reduce or eliminate costs associated with valve vault drainage, water in the bottom of the vault, and narrow overflow air gap. A new roof would likely reduce or eliminate costs associated the roof hatch and roof fall protection.

The next washdown inspection should take place within the next 3 to 5 years. The reservoir should be leak tested as a first step of the next inspection.

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Section 11

Consolidation Reservoir

11.1 Tank and Site Overview

The Consolidation Reservoir, shown in Figure 11-1, is one of five similar dome reservoirs that were built between 1958 and 1987. While as-built drawings for it are unavailable, drawings of the 40th Street and Reveille Reservoirs, which were built in the same period, designed by the same consulting engineering group, and are of similar construction are available. With slight differences in dimensions, the five reservoirs are 0.3 or 0.5 MG reinforced concrete, self-supporting, domed roof hoppers that were originally built with common inlet/outlet/drain pipes. The reservoirs were designed to hold water above the roof-to-wall interface. This reservoir is mostly buried, and the attached valve vault is located at the roof level. It was built in 1959 and has a capacity of 0.5 MG.

Figure 11-1: The Consolidation Reservoir, viewed looking northwest

According to measurements taken in the field (Figure 11-2), the Consolidation Reservoir has an interior diameter of 64 feet. The hopper base measured 7 feet deep and the interior wall measured 11.25 feet high. The overflow weir was measured to be 23 feet above the reservoir floor, which according to City-supplied documentation, corresponds to 519 feet above mean sea level (AMSL). This height is higher than the top of the vertical wall. The operating range is 15.5 to 19.5 feet. Wall and roof reinforcement were assumed to be similar to the 40th Street and Reveille Reservoirs. The interior of this reservoir has not been coated.

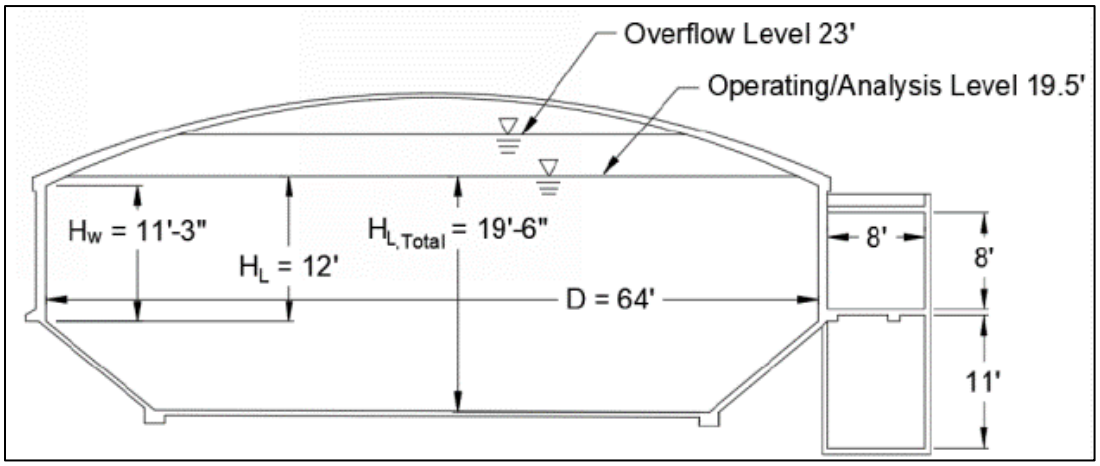

Figure 11-2: The measured interior dimensions of the Consolidation Reservoir

The Consolidation Reservoir is located on the outskirts of town, about 2 miles southeast of the Bellingham CBD. The valve vault shares a wall with the reservoir itself and is 8 feet wide by 8 feet deep. It serves the 519 Dakin and Yew Zone, which is fed from the gravity zone via the Dakin and Yew Booster Pump Station.

The geotechnical investigation indicated 0.5 feet of topsoil with 4.5 feet of fill material around the reservoir. The fill consisted of stiff brown sandy silt with gravel. Glaciomarine drift, which consists of stiff brown silt with sand and occasional gravel grading to dense silty sand, was encountered between 5 and 12.5 feet. Chuckanut Sandstone was encountered from 12.5 feet bgs to boring refusal, 15.5 feet bgs. Based on the height of the reservoir, the soil layers likely did not extend below the foundation of the reservoir. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Consolidation Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. Bedrock is not at risk of liquification and the site is not expected to have issues with slope instability.

11.2 Inspection Summary

A drained inspection of the Consolidation Reservoir took place on November 7, 2019.

11.2.1 Exterior Inspection Summary

11.2.1.1 Site and Security

Unlike many other reservoirs, Consolidation's site does not have a perimeter fence (Figure 11-1). The site is generally well-graded, directing runoff away from the reservoir. During the site visit, it

was noted that the roof of the reservoir and valve box are under the dripline of the trees (Figure 11-3). The consolidated layer of leaves around the perimeter of the reservoir is evidence of leaf accumulation on the roof that fall or are washed off during rain.

Figure 11-3: Fence around site perimeter (Left) and trees overhanging reservoir roof (Right)

11.2.1.2 Exterior Walls

Of the visible portion of the exterior walls above grade, cracking with efflorescence was noted throughout, especially below the roof-to-wall interface (Figure 11-4). Many of these areas were found to be wet to touch (Figure 11-5 Left). Insects were found at some of the locations as well (Figure 11-5 Right). The exterior coating was found to be peeling, blistering, and exhibiting efflorescence. Approximately 50 percent of the topcoat is missing, and 20 percent is blistering on the sidewall surfaces. The valve vault was missing 5 percent of the topcoat (Figure 11-1).

Figure 11-4: The exterior walls exhibit cracking with efflorescence.

Figure 11-5: The exterior walls are blistering and efflorescing (Left) and have insects.

11.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is buried.

11.2.1.4 Exterior Roof

The exterior roof of the reservoir exhibited circumferential and radial cracking with efflorescence, which can be seen in Figure 11-3 and Figure 11-6.The radial cracking extends up the roof side to intersect a series circumferential cracks that run around the entire reservoir roof. The circumferential cracks are located approximately 6-feet up the side of the roof. Coating was also missing in some areas (Figure 11-6 Right). The roof had a grade that ranged from 3 degrees near the vent to 20 degrees near the reservoir roof's perimeter. The distance from the roof to ground level is around 2 feet.

Figure 11-6: The roof circumferential and radial cracking and coating loss

11.2.1.5 Exterior Appurtenances

To access the roof, a single retrofitted ladder is used (Figure 11-1) The total distance from the 1-foot high concrete pad to the grating is approximately 7.3 feet. As is typical with the City's reservoirs of this construction, the retrofitted grating and safety railing extend only around the entry hatch and is missing a toeboard (Figure 11-1). The entry hatch has a thin contact area with the lid and the gasket was not securely attached (Figure 11-7 Left). Corrosion was found on the nuts (Figure 11-7 Right).

Figure 11-7: The roof entry hatch gasket is not securely attached and corrosion is affecting nuts.

The Consolidation Reservoir's vent has a sheet metal collar that was not removed during our inspection (Figure 11-8). The gap between the collar and roof was large enough to photograph the screen (Figure 11-8 Left), which appeared to be woven with approximately 4 strands per inch. It was also assessed via the provided drawings of similar reservoir and photos of the College Way Reservoir vent retrofit (Figure 11-8 Right). A 3-foot diameter penetration in the roof has a concrete vent structure above it. The concrete overhang is 4 feet, 2 inches. Approximately 1.25 inches of opening was exposed under the sheet metal collar.

Figure 11-8: The roof vent screen (Left) and a provided photo of the College Way Roof Vent retrofit from January 2013 (Right)

The attached valve vault shares a wall and footing with the reservoir. The inlet, outlet, and drain pipes all run through the vault and the Consolidation Booster Pump Station is housed in the upper section. The overflow runs through via an air gap, connecting with the drain pipe. The vault roof has curb that extends all the way around (Figure 11-9 Left). There was evidence of organic material buildup and the drain was clogged (Figure 11-9 Right). Within the upper section, the pipes penetrating the roof were efflorescing (Figure 11-10). A minor diagonal crack was noted near the window.

Figure 11-9: The attached valve vault has a curb that collects debris. The drain is blocked.

Figure 11-10: Efflorescence on the overflow and vault roof drain pipe (Left) and pump station with washdown piping (Right)

11.2.2 Interior Inspection Summary

11.2.2.1 Interior walls

The walls, being uncoated, were noted to have a similar cracking pattern as the exterior walls (Figure 11-11).

Figure 11-11: The uncoated interior walls exhibited minor cracking.

11.2.2.2 Floor

The floor had no observed issues (Figure 11-12).

Figure 11-12: The reservoir's floor

11.2.2.3 Interior Roof

The ceiling exhibited a similar cracking as the exterior roof. However, no major issues with interior efflorescence or other modes of failure were noted in the roof (Figure 11-13).

Figure 11-13: The reservoir roof minor cracking without efflorescence.

11.2.2.4 Interior Appurtenances

The interior ladder is rungs set into the wall with rungs spaced 15 inches apart. The ladder was very corroded and exhibited loss (Figure 11-14 Left); it was deemed unsafe for use. A removable ladder, tripod, and winch are used for inspections and maintenance. A large corroded pipe was located adjacent to the removable ladder which made ingress and egress difficult (Figure 11-14 Right).

Figure 11-14: The corroded interior ladder (Left) and remnant pipe (Right)

11.2.3 Piping and Valving Inspection Summary

The combined inlet/outlet/drain pipe had standing water within it (Figure 11-15 Left). This was likely because the pipe leading to the pump station in the upper vault was leaking. Around the pipe frame, corrosion was noted. Corrosion nodules were noted on the interior of the pipe (Figure 11-15 Right). The overflow weir, shown in Figure 11-16 (Left), is in good condition, but rust was noted on the overflow weir gate components. The air gap on the overflow pipe is approximately 11 inches (Figure 11-9).

The valve vault contained an altitude valve and isolation valves (Figure 11-9 Right). The drain pipe connects with the overflow and extends to a swale via a culvert that sits flush with the ground (Figure 11-17 Left). Dechlorination was with a dechlorination bag that was found to be out of the main flow path during our inspection (Figure 11-17 Right). As water was still flowing from the pipe, it was moved more in line with the center of flow. The swale leads to a pond on a neighboring property (Figure 11-18).

Figure 11-15: The common inlet/outlet/drainpipe frame corrosion and standing water (Left), interior pipe corrosion nodules and water flowing from pump station pipe (Right).

Figure 11-16: *Weir gate (Left) and vault piping, altitude valve, and isolation valves (Right).*

Figure 11-17: *Drain/overflow outlet (Left) and its dechlorination bag (Right).*

Figure 11-18: The overflow/drain destination is a pond on an adjacent property

Washdown piping is located within the valve vault.

11.3 Condition Assessments

11.3.1 Cleanliness and Coatings

The exterior is very dirty, and the coating is no longer functional on the roof and walls of the reservoir structure. The debris accumulation, coating condition, exposed concrete, efflorescence, and insects affected the overall condition of the exterior cleanliness and coating, which is poor.

While the interior is uncoated, the staff had just finished a washdown of the reservoir. The efflorescence on overflow and roof drain pipes within valve vault is more of a cosmetic concern but is indicative of poor drainage of the valve vault.

11.3.2 Material Deterioration

The concrete is heavily deteriorating and efflorescing on the exterior; the concrete is in poor condition. Significant concrete deterioration was noted throughout the exterior, which needs attention. The cracks were noted to be wet, which is likely due to the water migrating through the cracks. Because this cracking is structural in nature, it is discussed in Section 11.3.3.1.

The cracks noted on the interior of the structure are related to those noted on the exterior, so is assessed to be in poor condition. Corrosion on the inlet/outlet/drain pipe, ancillary pipe, weir gate components, pipe frame, and vault lid was significant, and were in poor condition.

11.3.3 Structural Performance

11.3.3.1 Static Analysis

The dome roof was found to meet modern requirements of AWWA D110-13. However, the roof-to-wall interface is not designed to allow for roof expansion and contraction, which has resulted in cracks. The cracking in the roof is likely a result of this issue and the walls be further constrained by being almost completely buried.

Assuming the walls are reinforced similar to the 40th Street and Reveille Reservoirs' plans, the wall reinforcing appears to be acceptable for static loads. The horizontal reinforcing was found to be sufficient for the current operating level, but not if operated any higher. However, the upper sections of the interior wall have reinforcing at 24 inches on center, which exceeds ACI 350.3's maximum allowable spacing of 12 inches on center. The foundation was found to be sized appropriately, based on the geotechnical report's soil bearing capacity of 6,000 psf.

11.3.3.2 Seismic Analysis

At the 19.5-foot operating level, the walls' reinforcement was found to be suitable for the design seismic event. This was not the case if operated at the overflow level. Only the horizontal hoop

reinforcing was exceeded by 2 percent, which would likely not warrant lower operation. The design seismic event would cause a constrained slosh wave, causing damage to the roof if operated at the overflow level. However, at the 19.5-foot operating level, the roof is adequately reinforced. Incidental damage may still occur to hatches and appurtenances. The valve vault attachment is suitable to prevent differential movement and impact during the seismic event. Also, based on assumed backfill conditions, the hopper base is not likely to fail and collapse onto the piping. However, the pipes do not have flexible couplings which would address potential differential settlement between the two structures during a seismic event.

11.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A removable silt stop is required on combined drains/outlets but was not installed. Since the reservoir drains to daylight, a DOH approved air gap is not required. The missing screen on the drain/overflow outlet could lead to water quality issues in the reservoir. It did not appear that the screen on the vent is a 24-mesh screen backed with 4-mesh screen. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gasket to provide a tight seal against the relatively small surface area of the frame. The gasket not being fully adhered could potentially lead to water quality issues within the reservoir. The cracks in the concrete could also be a way for water quality issues to develop. Overall, the water quality/sanitary condition is fair.

11.3.5 Safety

With the total fall distance from the exterior ladder to ground level being about 7.3 feet, no ladder fall protection is required. This roof is considered steep, as the greatest slope is greater than 4:12 (18.4) degrees. Even with this slope, the total fall distance from the roof's edge to the ground is less than 4 feet, so L&I does not mandate roof fall protection. The fall from the valve vault roof would be more than 4 feet, so a worker would need fall protection. A toeboard is also required on the existing grating.

With the fall protection and ladder on the interior of the reservoir being non-existent, a removable ladder, tripod, and winch were required for our inspection. While this configuration appears to meet OSHA requirements, there is possible risk of failure in this setup. Overall, the safety condition of the reservoir is fair.

11.3.6 Operations and Maintenance

11.3.6.1 Site and Security

The lack of a perimeter fence was a noted deficiency with the reservoir. However, an intrusion alarm is installed on the roof's access hatch. Soil drainage was found to be suitable for this reservoir. Trees were too close to the reservoir. Nearby trees endanger the structure, slow

evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security condition is fair to poor.

11.3.6.2 Roof Drainage

The reservoir roof sheds water adequately. However, drainage of the valve vault roof appeared to not function in the event of a drain clog. If the drain were to be clogged, the roof would overflow the curb; no notches or overflow scuppers are present that would send water away from the reservoir. There are apparently no waterstops installed between the reservoir and valve vault.

11.3.6.3 Appurtenances

No difficulties were found using the exterior ladder to access the roof. However, accessing the interior of the reservoir is more difficult than other style reservoirs as the tripod, winch, and removable ladder must be lifted to the roof. Consolidation does not have a ladder on the exterior of the roof access hatch vault, so climbing into the hatch can also be challenging. The Consolidation Reservoir met design requirements for the vent on the roof penetration, screened area, and free area on the retrofitted collar.

11.3.6.4 Valving and Piping

The staff noted that draining reservoir of this reservoir is difficult and time consuming as the neighboring property can be flooded during the process. The pipe was leaking a lot of water during our inspection indicating poor condition of the isolation valving. The dechlorination was not adequately treating all flow from the drain pipe. Overall, the condition of the valving and piping operation was poor.

11.3.7 Obsolescence

Overall, the equipment on site appeared to be of older design, and difficulty would be expected replacing components.

11.3.8 Condition Scoring

Overall, the condition score of the Consolidation Reservoir was fair, 3.3. Problems associated with this reservoir were related to the structural problems with the roof-wall-interface. A summary of the scoring is shown in Table 11-1 and the full Score Matrix can be found in Appendix I.

Table 11-1: Condition Scoring of the Consolidation Reservoir

Categorical Scores	
Cleanliness and Coatings	2.9
Material Deterioration	2.6
Structural Performance - Static	4.1
Structural Performance - Seismic	3.0
WQ/ Sanitary	3.6
Safety	4.0
Operations & Maintenance	3.6
Obsolescence	1.0
Overall	**3.3**

11.4 Recommended Improvements

11.4.1 Cleanliness and Coatings

First, the exterior should be pressure washed to remove dirt and other debris. The valve vault drain should also be cleared. All the exposed exterior areas that are experiencing coating failure are due to thermal effects, discussed in Section 11.4.3.

For the efflorescence on the overflow and valve vault roof drain, cleaning and recoating is recommended. The valve vault roof is also recommended to be resealed and drain unclogged.

11.4.2 Material Deterioration

One approach to deal with the apparent leaking concrete is to line the interior, like what has been done in the other dome reservoirs. Discussed in Section 11.4.3 are possible solutions to deal with the thermal expansion-related issues of the roof-to-wall interface.

The corrosion on the access hatch vault lid nuts, overflow weir gate components, ancillary pipe, and pipe frame can be addressed with cleaning and recoating The corrosion on the interior of the inlet/outlet/drain pipe is recommended to be removed by mechanical agitation or by pigging. The interior of the pipe near the reservoir can be recoated or a cured in place pipe liner can be installed if corrosion jeopardizes the pipe's integrity.

11.4.3 Structural

To confirm the assumptions made in the analysis, as-built drawings may be obtained, or a testing firm could be employed to map the existing reinforcing.

The recommended improvement to address the issues related to thermal deformation occurs in stages:

1. A temporary improvement should be employed soon to halt further deterioration of any reinforcing material until a permanent solution is implemented. This would entail removing damaged concrete from the cracking areas, properly cleaning cracks, and assessment by a structural engineer. Likely, the areas will need to be spot coated to prevent water damage to the reinforcing. It is imperative that any repair media be flexible so as not to accelerate structural damage as thermal movement is likely to continue to occur until the problem is addressed. Stiffening or reinforcing the cracking areas is not recommended.

2. To more permanently address the thermal deformation-related issues, several options are available. A new aluminum geodesic roof is recommended. The City can also consider a new concrete roof that is domed or flat. Another option would be lifting the entire roof and retrofitting it with a bracketing system and elastomeric bearing pad between the roof and walls. These options would likely cost much more than a new aluminum geodesic roof.

3. The cracks should be investigated by a structural engineer after the thermal deformation issue is resolved.

Since the cracks in the roof are probably due to the thermal issue, a similar procedure should be followed as part 1, ensuring the structure is not restrained by non-flexible repairs.

The best approach to deal with the wall reinforcement being spaced 24 inches on center rather than 12 inches on center is to monitor the structure. The City should also consider lowering the overflow to ensure it is not operated above the maximum allowable operating level in the future.

Because the valve vault shares a footing with the reservoir, there isn't space available to install flexible connections on the pipes. Instead, a new vault should be constructed with sufficient space between it and the reservoir to allow for the installation and proper operation of flexible connections.

11.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. A removable silt stop should be installed on the combined drain/outlet pipe. The overflow/drain pipe outlet can be retrofitted with a #24-mesh and 4-mesh screen backing.

The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area. The vent screen, if in fact is not compliant, it can be replaced with a #24-mesh and 4-mesh screen backing.

11.4.5 Safety

On the exterior, an L&I-compliant railing can be installed around the valve vault roof. A toeboard can also be installed on the grating.

On the interior, steps should be taken to ensure ingress and egress to the reservoir is as safe as possible. A new interior ladder with fall protection system is recommended. This could be similar to the ladder installed at the Kearney Reservoir, which has a removable extension and is shown in Figure 9-16. A new exterior ladder should be installed on the exterior of the access vault that is similar to the ladder installed on the 40th Street Reservoir, shown in Figure 9-7 right.

11.4.6 Operations and Maintenance

11.4.6.1 Site and Security

A fence should be installed around the reservoir for security reasons. Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

11.4.6.2 Roof Drainage

On the valve vault roof curb, notches or overflow scuppers should be installed to prevent the roof from overflowing the curb if the drain backs-up.

11.4.6.3 Appurtenances

The are no recommended improvements related to the operations and maintenance for the appurtenances for the Consolidation Reservoir.

11.4.6.4 Valving and Piping

Proper dechlorination of the Consolidation Reservoir will need to consist of a weighted system that is set in the manhole or affixed to the culvert at the outlet. An example of a basket system is shown in Figure 4-21. The City was informed of the leaking valve, but this should be repaired or replaced to facilitate draining of the reservoir and minimize contamination issues.

11.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Consolidation Reservoir are shown in Table 11-2

Table 11-2: Consolidation deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior. Clear valve vault drain.	$ 1,000
	Valve vault roof drain is blocked.	Clear clogged drain.	$ 1,000
	Efflorescence on overflow and roof drain pipes within valve vault	Clean and recoat. Seal valve vault roof.	$ 3,000
DETERIOR-ATION	Cracks in concrete and leaks	Coat interior	$ 143,000
	Corrosion on access hatch vault lid nuts, overflow weir gate components, ancillary pipe, and pipe frame	Clean and (re)coat. Remove ancillary pipe if not needed.	$ 2,000
	Corrosion on interior of piping	Remove corrosion with agitation or pig pipe. Recoat where possible or consider cured in place pipe liner	$ 10,000
STRUCT	Roof-wall-interface does not account for thermal deformation	Resolve roof-wall-interface issues a. Clean & coat cracks related to thermal deform. b. Replace roof with new geodesic dome c. Structural inspection to determine next steps	 $ 17,000 $ 120,000 $ 2,000
	Maximum reinforcement spacing exceeded	Continue to monitor for structural issues	$ -
	Operated at the overflow elevation, seismic slosh impacts roof and structure is under-reinforced	Consider lowering overflow	$ -
	Missing seismic valves & vault adjacent to reservoir	New seismic valve vault and associated piping	$ 200,000
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	No silt-stop on combined drain/outlet	Install removable silt-stop	$ 1,000
	Drain pipe screen non-compliant	Replace drain pipe screen	$ 1,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Lacks valve vault roof fall protection and grating railing toeboard	Install toeboard on grating railing and railing around valve vault roof	$ 4,000
	Ingress and egress difficult; interior ladder corroded and non-compliant	Install exterior vault access ladder and replace interior ladder with one with fall protection	$ 33,000
O&M	Fence does not encompass reservoir	Install/replace fence	$ 18,000
	Tree spacing recommendation not met	Tree removal and restoration	$ 11,000
	Valve vault roof lacks overflow drainage	Install notches or overflow scuppers on parapet	$ 2,000
	Isolation valve for pump station not functioning	Replace isolation valve	$ 10,000
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000
		Subtotal	$ 585,000
		30% Contingency	$ 175,500
		8.7% Tax	$ 66,164
		Engineering, Administration, and Construction Management	$ 289,332
		Total Project Costs Estimate	$ 1,115,996
		Total Project Costs Estimate (rounded)	**$ 1,100,000**

11.5 Conclusion

The two biggest items to be dealt with on the Consolidation Reservoir are the thermal deformation issues and the lack of flexible couplings on the pipes passing through the valve vault. If the City wishes to update to a modern seismic valve vault soon, it may not be necessary address issues with the existing vault.

The next washdown inspection should take place within the next 3 to 5 years. The reservoir should be leak tested as a first step of the next inspection.

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Section 12

Dakin I Reservoir

12.1 Tank and Site Overview

The Dakin I Reservoir, shown in Figure 12-1, is one of five similar dome reservoirs that were built between 1958 and 1987. While as-built drawings for it are unavailable, drawings of the 40th Street and Reveille Reservoirs, which were built in the same period, designed by the same consulting engineering group, and are of similar construction are available. With slight differences in dimensions, the five reservoirs are 0.3 or 0.5 MG reinforced concrete, self-supporting, domed roof hoppers that were originally built with common inlet/outlets/drainpipes. The reservoirs were designed to hold water above the roof-to-wall interface. This reservoir is partially buried, and the attached valve vault is located at the roof level. It was built in 1987 and has a capacity of 0.5 MG.

Figure 12-1: The Dakin I Reservoir, viewed looking northeast

According to measurements taken in the field (Figure 12-2), the Dakin I Reservoir has an interior diameter of 66 feet, 8 inches. The hopper base measured 7.3 feet deep and the interior wall measured 10.3 feet high. The overflow weir was measured to be 21.9 feet above the reservoir floor, which according to City-supplied documentation, corresponds to 519 feet City. This height is higher than the top of the vertical wall. The operating range is 13.5 to 17 feet. Wall and roof reinforcement were assumed to be similar to the 40th Street and Reveille Reservoirs. The interior walls and floor of the Dakin I Reservoir were coated in 2009 using VersaFlex AquaVers primer and polyurea.

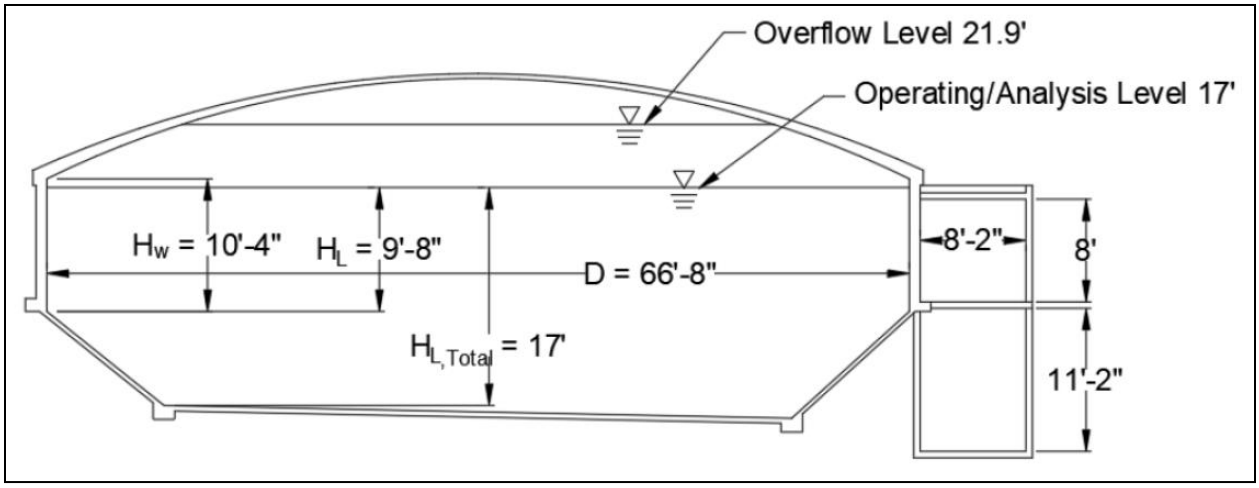

Figure 12-2: The measured interior dimensions of the Dakin I Reservoir

The site where the Dakin I and II Reservoirs are located is within a forested area about 3 miles northwest of the Bellingham CBD. The valve vault shares a wall with the reservoir itself and is 8 feet wide by 9 feet deep. It serves the 519 Dakin and Yew Zone, which is fed from the gravity zone via the Dakin and Yew Booster Pump Station. The Dakin Reservoirs also supply the 730 Alabama Hill Zone via the Balsam Lane Pump Station.

The geotechnical investigation indicated 4 feet of fill material around the reservoir, which consisted of medium stiff to stiff blue-brown silt with variable amounts of sand, gravel, and organic matter. This layer likely did not extend below the foundation of the reservoir. Chuckanut Sandstone was encountered at 4 feet bgs. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Dakin I Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. Bedrock is not at risk of liquification and the site is not expected to have issues with slope instability.

12.2 Inspection Summary

A drained inspection of the Dakin I Reservoir took place on April 8, 2019.

12.2.1 Exterior Inspection Summary

12.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the property (Figure 12-3 Left). The site is generally well-graded, directing runoff away from the reservoir. During the site visit, it was noted that the roof of the reservoir and valve box are under the dripline of the trees (Figure 12-3 Right).

Figure 12-3: Fence around site perimeter (Left) and trees overhanging reservoir roof (Right)

12.2.1.2 Exterior Walls

Of the visible portion of the exterior walls above grade, cracking with efflorescence was noted throughout, especially below the roof-to-wall interface (Figure 12-4). The exterior coating was found to be peeling, blistering, and exhibiting efflorescence (Figure 12-5 Left). Areas of missing coating were also found near ground level (Figure 12-5 Right) The DFT of the sidewalls ranged from 24-33 mils. Approximately 5-10 percent of the topcoat is missing, and 1-3 percent is blistering on the sidewall surfaces, mostly at the ground interface.

Figure 12-4: The exterior walls cracking with efflorescence

Figure 12-5: Exterior wall blistering and efflorescing (Left) and missing coating near ground level (Right)

12.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is buried.

12.2.1.4 Exterior Roof

The exterior roof of the reservoir exhibited minor circumferential and radial cracking (Figure 12-6) and organic matter/debris accumulation. No DFT measurements were able to be taken on the roof due to the textured non-slip coating. The roof had a grade that ranged from 3.3 degrees near the vent to 19.4 degrees near the reservoir roof's perimeter. The distance from the roof to ground level ranges from 4 to 5 feet.

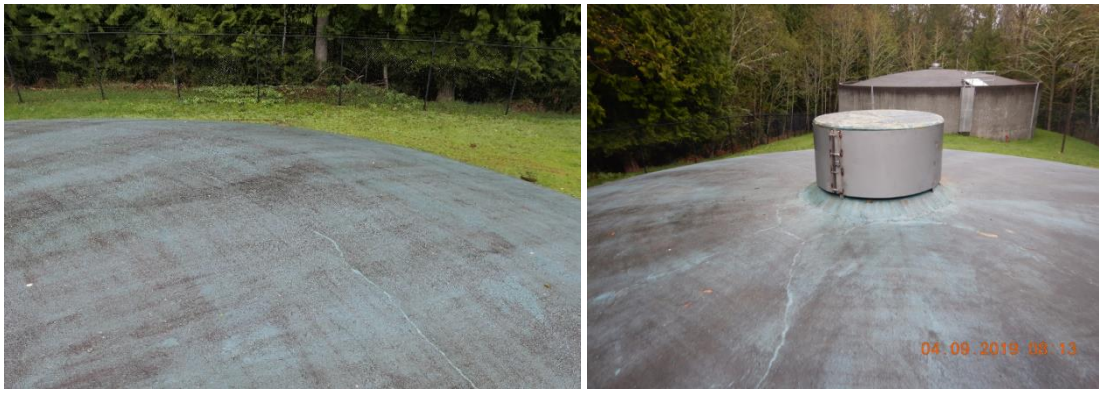

Figure 12-6: Roof minor circumferential and radial cracking and roof vent

12.2.1.5 Exterior Appurtenances

The original ladder that is set into the walls (Figure 12-1) is used to access the valve vault roof and even the roof in general. A retrofitted ladder with grading similar to the other reservoirs also provides access to the roof. As is typical with the City's reservoirs of this construction, the

retrofitted grating and safety railing extend only around the entry hatch and is missing a toeboard (Figure 12-7 Left). The entry hatch has a thin contact area with the lid and the gasket was not securely attached (Figure 12-7 Right).

Figure 12-7: The roof entry hatch (Left and Right) and overflow weir (Right)

Dakin I Reservoir's vent has a sheet metal collar that was not removed during our inspection (Figure 12-6). The gap was large enough the photograph the screen (Figure 12-8 Left) which appeared to be woven with approximately 12 strands per inch. It was also assessed via the provided drawings of similar reservoir and photos of the College Way Reservoir vent retrofit (Figure 12-8 Right). A 3-foot diameter penetration in the roof has a concrete vent structure above it. The concrete overhang is 4 feet, 2 inches. Approximately 2 inches of opening was exposed under the sheet metal collar.

Figure 12-8: The roof vent screen (Left) and a provided photo of the College Way Roof Vent retrofit from January 2013 (Right)

The attached valve vault shares a wall and footing with the reservoir. The inlet, outlet, and drain pipes all run through the vault. The overflow runs through via an air gap, connecting with the drain pipe. The vault roof has curb that extends all the way around (Figure 12-7 Left). The coating on the attached valve vault and entry vault was failing (Figure 12-1 and Figure 12-3 Left).

12.2.2 Interior Inspection Summary

12.2.2.1 Interior walls

The coating on the interior walls made it difficult to determine the extent, if any, of the internal cracking of reservoir walls. Small blistering on the hopper base and coating sags in the upper walls were noted in the interior coating (Figure 12-9). The blue prime coat is still intact beneath blisters, but water was present under the blisters. The measured DFT ranged from 80-100 mils; the specified DFT was 80 mils as per the supplied documents from the City.

Figure 12-9: The reservoir's walls (Left) minor bubbling on the hopper base(Right)

12.2.2.2 Floor

The floor was also coated and covered by a thin layer of sediment, which made it difficult to inspect the condition (Figure 12-10). From what could be seen, the floor's coating was intact. No blistering was observed.

Figure 12-10: The reservoir's floor covered in sediment

12.2.2.3 Interior Roof

The ceiling exhibited a similar minor crack as the exterior roof and is uncoated.

Figure 12-11: The reservoir roof exhibited circumferential cracking

12.2.2.4 Interior Appurtenances

The interior ladder was rungs set into the wall. While the upper rungs within the access vault are still present, the rungs on the wall were removed during the coating process and have not been replaced (Figure 12-12 Left). A removable ladder, tripod, and winch are used for inspections and maintenance. A large corroded pipe was located adjacent to the removable ladder which made ingress and egress difficult (Figure 12-12 Right).

Figure 12-12: Removable interior ladder (Left) and a large uncoated corroded pipe

12.2.3 Piping and Valving Inspection Summary

The combined inlet/outlet/drain piping exhibited minor discoloration on the interior of the pipe (Figure 12-13). The valve vault contained an altitude valve and isolation valves (Figure 12-14 Left). The drain pipe connects with the overflow and extends to the storm sewer without an air gap (Figure 12-14 Right).

Figure 12-13: The common inlet/outlet/drainpipe with a sample line running though

Figure 12-14: Piping, an altitude valve, and isolation valves within the vault (Left). The drainpipe connects to the storm sewer without air gap (Right).

The overflow weir, shown in Figure 12-7 (Right) had rust on the gate components (Figure 12-15 Left). The total air gap distance is approximately 6 inches (Figure 12-15 Right). It connects with the drain line in the vault, draining to the storm sewer.

Figure 12-15: Corrosion on the overflow gate (Left) and exterior overflow piping with very narrow air gap (Right)

A fire hydrant is located outside of the reservoir and is used in washdown operations.

12.3 Condition Assessments

12.3.1 Cleanliness and Coatings

The exterior roof walls and roof have organic material buildup and efflorescence. The exterior topcoat is no longer functional in many places on the reservoir structure, valve vault, and access vault. If spot treated, the exterior coating will be viable for up to 10 years (+/-) to come. The debris accumulation, topcoat condition, and exposed concrete caused the exterior walls and roof to be in fair to poor condition.

On the interior, the amount of accumulated sediment indicates that this reservoir is due for a washout inspection. The bubbling and sags on the interior coating should continue to be monitored, but do not appear to be affecting the integrity of the coating at this time. As the prime coat is intact under the blisters on the hopper base, they are likely not due to groundwater effects. Instead, these blisters are more likely due to air entrapment during the polyurea's application.

12.3.2 Material Deterioration

Significant concrete deterioration was noted throughout the exterior, which needs attention. Because this cracking is structural in nature, it is discussed in Section 12.3.3.1. The interior coating has likely significantly decreased the rate of efflorescence accumulation around the exterior cracks as water is inhibited from passing through the concrete.

The combined inlet/outlet/drain piping was in good visual condition, but the corrosion on the ancillary pipe and weir gate components were significant, causing it to be in poor condition.

12.3.3 Structural Performance

12.3.3.1 Static Analysis

The dome roof was found to meet modern requirements of AWWA D110-13. However, the roof-to-wall interface is not designed to allow for roof expansion and contraction, which has resulted in cracks. Assuming the walls are reinforced similar to the 40th Street and Reveille Reservoirs' plans, the wall reinforcing appears to be acceptable for static loads. However, the upper sections of the interior wall have reinforcing at 24 inches on center, which exceeds ACI 350.3's maximum allowable spacing of 12 inches on center. The foundation was found to be sized appropriately, based on the geotechnical report's soil bearing capacity of 6,000 psf.

12.3.3.2 Seismic Analysis

At the 17-foot operating level, the walls' reinforcement was found to be suitable for the design seismic event. This was not the case if operated at the overflow level. Similarly, the design seismic event would cause a constrained slosh wave, causing damage to the roof if operated at the

overflow level. However, at the 17-foot operating level, the roof is adequately reinforced. Incidental damage may still occur to hatches and appurtenances. The valve vault attachment is suitable to prevent differential movement and impact during the seismic event. Also, based on assumed backfill conditions, the hopper base is not likely to fail and collapse onto the piping. However, the pipes do not have flexible couplings which would address potential differential settlement between the two structures during a seismic event.

12.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A removable silt stop is required on combined drains/outlets but was not installed. As this reservoir drains to a storm drain, a DOH approved air gap is required on both the overflow and the drain pipe. No air gap is installed on the drain pipe. The air gap between the overflow and receiving pipe was found to be too narrow. While the screen on the vent could not be carefully measured due to the sheet metal collar, it does not appear to be 24-mesh screen backed with 4-mesh screen The roof hatch would be classified as a "high-maintenance design" because of issues getting the gasket to provide a tight seal against the relatively small surface area of the frame. The gasket not being fully adhered could potentially lead to water quality issues within the reservoir. Overall, the water quality/sanitary condition is fair to good.

12.3.5 Safety

With the exterior ladders being short, no ladder fall protection is required. This reservoir roof has some areas where a fall would be greater than 4 feet. A fall would be greater than 4 feet from the valve vault as well. Thus, an L&I-compliant railing is required on these areas. The reservoir roof is considered steep as the greatest slope is greater than 4:12. Thus, additional fall prevention is required.

With the fall protection and ladder on the interior of the reservoir being non-existent, a removable ladder, tripod, and winch were required for our inspection. While this configuration appears to meet OSHA requirements, there is possible risk of failure in this setup. While the exterior ladder safety condition is good, the valve vault roof condition is fair, and the interior ladder safety condition is fair to poor.

12.3.6 Operations and Maintenance

12.3.6.1 Site and Security

The fence and drainage were found to be suitable for this reservoir. An intrusion alarm is installed on the roof's access hatch. However, trees were too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is fair condition.

12.3.6.2 Roof Drainage

The reservoir roof sheds water adequately. However, drainage of the valve vault roof appeared to not function in the event of a drain clog. If the drain were to be clogged, the roof would overflow the curb; no notches or overflow scuppers are present that would send water away from the reservoir. There are apparently no waterstops installed between the reservoir and valve vault.

12.3.6.3 Appurtenances

No difficulties were found using the exterior ladder to access the roof. However, accessing the interior of the reservoir is more difficult than other style reservoirs as the tripod, winch, and removable ladder must be lifted to the roof. Dakin I does not have a ladder on the exterior of the roof access hatch vault, so climbing into the hatch can also be challenging. The Dakin I Reservoir met design requirements for the vent on the roof penetration, screened area, and free area on the retrofitted collar.

12.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. No dechlorination system was observed onsite.

12.3.7 Obsolescence

Overall, the equipment on site appeared to be of older design, and difficulty would be expected replacing components.

12.3.8 Condition Scoring

Overall, the condition score of the Dakin I Reservoir was fair, 3.7. Problems associated with this reservoir were related to the structural problems with the roof-wall-interface. A summary of the scoring is shown in Table 12-1 and the full Score Matrix can be found in Appendix J.

12.4 Recommended Improvements

12.4.1 Cleanliness and Coatings

First, the exterior should be pressure washed to remove dirt and other debris. Areas of coating failure on the exterior that are not thermal related such as the valve vault and access vault can be spot coated.

The interior should be washed out to remove the sediment that has accumulated on the floor. This should be done at a low pressure so as not to burst any of the blisters on the hopper base.

The blisters should continue to be monitored and photographed during regular inspections to determine if they are getting larger, increasing in number, cracking, or bursting. The prime coat should also be monitored.

Table 12-1: Condition Scoring of the Dakin I Reservoir

Categorical Scores	
Cleanliness and Coatings	4.1
Material Deterioration	4.2
Structural Performance - Static	4.1
Structural Performance - Seismic	3.3
WQ/ Sanitary	4.1
Safety	4.0
Operations & Maintenance	4.2
Obsolescence	1.0
Overall	**3.7**

12.4.2 Material Deterioration

Discussed in Section 12.4.3 are possible solutions to deal with the thermal expansion-related issues of the roof-to-wall interface.

The corrosion on the weir gate components, should the City wish to continue operating them, can be addressed with cleaning and recoating. The corroded pipe within the access vault is recommended to be removed per section 12.4.6.3. If it cannot be removed, it should be cleaned and coated.

12.4.3 Structural

To confirm the assumptions made in the analysis, as-built drawings may be obtained, or a testing firm could be employed to map the existing reinforcing.

The recommended improvement to address the issues related to thermal deformation occurs in stages:

1. A temporary improvement should be employed soon to halt further deterioration of any reinforcing material until a permanent solution is implemented. This would entail removing damaged concrete from the cracking areas, properly cleaning cracks, and assessment by a structural engineer. Likely, the areas will need to be spot coated to prevent water damage to the reinforcing. It is imperative that any repair media be flexible so as not to accelerate structural damage as thermal movement is likely to continue to occur until the problem is addressed. Stiffening or reinforcing the cracking areas is not recommended.

2. To more permanently address the thermal deformation-related issues, several options are available. A new aluminum geodesic roof is recommended. The City can also consider a new concrete roof that is domed or flat. Another option would be lifting the entire roof and retrofitting it with a bracketing system and elastomeric bearing pad between the roof and walls. These options would likely cost much more than a new aluminum geodesic roof.

3. The cracks should be investigated by a structural engineer after the thermal deformation issue is resolved.

The best approach to deal with the wall reinforcement being spaced 24 inches on center rather than 12 inches on center is to monitor the structure. The City should also consider lowering the overflow to ensure it is not operated above the maximum allowable operating level in the future.

Because the valve vault shares a footing with the reservoir, there isn't space available to install flexible connections on the pipes. Instead, a new vault should be constructed with sufficient space between it and the reservoir to allow for the installation and proper operation of flexible connections.

12.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. A removable silt stop should be installed on the combined drain/outlet pipe. A DOH-approved air gap should also be installed on the drain pipe. With the drain pipe being 11 feet underground, this would likely be logistically challenging at the Dakin I site. Possible options are draining to daylight, an appropriately sized dedicated dry well with backflow prevention, or an air gap with a sump and sump pump that can pump water than cannot be drained by gravity.

While the overflow air gap is too narrow, extending it is not possible with the current configuration; the roof of the valve vault is too close to the overflow outlet. To meet current DOH recommendations, the overflow would likely need replacement. The new overflow could outlet to the site or pass around the valve vault with an approved air gap before re-entering the vault or storm sewer.

The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area. The vent screen, if in fact is not compliant, it can be replaced with a #24-mesh with a 4-mesh screen backing.

12.4.5 Safety

On the exterior, an L&I-compliant railing is recommended around valve vault roof. The railing will also need to extend around the perimeter of the reservoir at all areas where the fall distance is greater than 4 feet. Additionally, on the reservoir roof, a fall restraint system or positioning device

system, or possibly a fall arrest system if the work is considered roofing work, is recommended. A toe board should be added to the existing grating.

On the interior, steps should be taken to ensure ingress and egress to the reservoir is as safe as possible. A new interior ladder with fall protection system is recommended. This could be similar to the ladder installed at the Kearney Reservoir, which has a removable extension and is shown in Figure 9-16. A new exterior ladder should be installed on the exterior of the access vault that is similar to the ladder installed on the 40th Street Reservoir, shown in Figure 9-7 Right.

12.4.6 Operations and Maintenance

12.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

12.4.6.2 Roof Drainage

On the valve vault roof curb, notches or overflow scuppers should be installed to prevent the roof from overflowing the curb if the drain backs-up.

12.4.6.3 Appurtenances

To facilitate access to the interior of the reservoir, the corroded ancillary pipe should be removed.

12.4.6.4 Valving and Piping

Proper dechlorination of the Dakin I Reservoir will need to consist of a weighted system that is set in the manhole or affixed to the culvert at the outlet. An example of a basket system is shown in Figure 4-21.

12.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Dakin I Reservoir are shown in Table 12-2

Table 12-2: deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 1,000
	Coating failure on reservoir (not thermal related - small areas) - vaults and roof	Spot Repair Coating	$ 7,000
	Sediment accumulation on floor	Perform regular washdown inspections	$ 1,000
	Blisters on hopper base coating	Monitor for blister growth	$ -
DETERIOR-ATION	Corrosion on overflow weir gate components	Clean and recoat overflow weir gate components	$ 1,000
STRUCT	Roof-wall-interface does not account for thermal deformation	Resolve roof-wall-interface issues a. Clean & coat cracks related to thermal deform. b. Replace roof with new geodesic dome c. Structural inspection to determine next steps	 $ 11,000 $ 150,000 $ 2,000
	Maximum reinforcement spacing exceeded	Continue to monitor for structural issues	$ -
	Operated at the overflow elevation, seismic slosh impacts roof and structure is under-reinforced	Consider lowering overflow	$ -
	Missing seismic valves & vault adjacent to reservoir	New seismic valve vault and associated piping	$ 200,000
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	No silt-stop on combined drain/outlet	Install removable silt-stop	$ 1,000
	No drain air gap	Upgrade drain pipe w/ air gap	$ 88,000
	Overflow air gap too narrow	Replace overflow	$ 25,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Lacks steep roof fall protection, railings, and grating railing toeboard	Install roof fall protection, railing on heights greater than 4 feet, and toeboard on grating railing	$ 30,000
	Ingress and egress difficult and missing interior ladder	Install exterior access vault ladder and install interior ladder with fall protection	$ 32,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 8,000
	Valve vault roof lacks overflow drainage	Install notches or overflow scuppers on parapet	$ 2,000
	Corroded pipe near entry hatch inhibits ingress	Remove pipe. Clean and coat if cannot be removed	$ 1,000
	Dechlorination non-compliant	Update dechlorination system	$ 2,000
		Subtotal	$ 564,000
		30% Contingency	$ 169,200
		8.7% Tax	$ 63,788
		Engineering, Administration, and Construction Management	$ 278,946
		Total Project Costs Estimate	$ 1,075,934
		Total Project Costs Estimate (rounded)	**$ 1,100,000**

12.5 Conclusion

The two biggest items to be dealt with are the thermal deformation issues and the lack of flexible couplings on the pipes passing through the valve vault. The Dakin I reservoir, built in 1987, is considerably newer than the other dome reservoirs which were built between 1958 and 1968. Because the reservoir is still relatively new, the City may be favor rehabilitation of this reservoir in comparison to the other older reinforced dome reservoirs.

Because the interior has sediment accumulation, this reservoir is recommended to be prioritized in its next washout inspection. This should take place within the next 2 years. The reservoir should be leak tested as a first step of the next inspection. The internal blistering, while not a concern at this time, should be inspected at 3- to 5-year intervals to determine if the problem is progressing.

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Section 13

Reveille Reservoir

13.1 Tank and Site Overview

The Reveille Reservoir, shown in Figure 13-1, is one of five reservoirs that were built between 1958 and 1987. Drawings were prepared by John W. Cunningham & Associates for both the 40th Street and Reveille Reservoirs. With slight differences in dimensions, the five reservoirs are approximately 0.5 MG reinforced concrete, self-supporting, domed roof hoppers that were originally built with common inlet/outlets/drainpipes. The reservoirs were designed to hold water above the roof-to-wall interface. This reservoir is partially buried, and the attached valve vault is located below roof. This reservoir is the oldest, built in 1958 and has a capacity of only 0.3 MG.

Figure 13-1: The Reveille Reservoir, viewed looking southwest

According to measurements taken in the field (Figure 13-2), The Reveille Reservoir has an interior diameter of 52 feet. The hopper base is 6 feet deep and the interior wall is 13 feet high. The overflow weir was measured to be 23.25 feet above the reservoir floor, which according to City supplied document, corresponds to 696 feet City. This height is higher than the top of the vertical wall. Operational heights range from 15 to 19 feet. Reinforcement and concrete thicknesses of the walls and roof were shown in the provided drawings. The interior is uncoated.

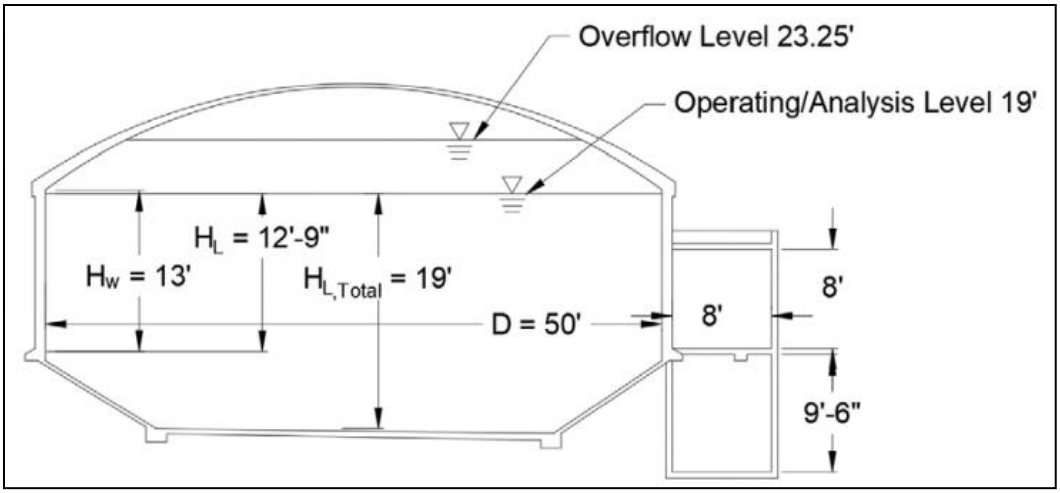

Figure 13-2: The measured interior dimensions of the Reveille Reservoir

Reveille is located in a suburban neighborhood in the southern portion of the service district, about 2 miles southeast of the Bellingham CBD. It serves the 696 Padden Yew Zone. The valve vault shares a wall with the reservoir itself and is 8 feet wide by 8 feet deep. Piping is run to the valve vault, encased in an unreinforced concrete block for protection under the footing. The upper section of vault contains the Reveille Pump Station.

The geotechnical investigation indicated fill consisting of red-brown silt with sand, gravel, and occasional organic matter from the surface to 4.5 feet bgs. Glaciomarine drift, which was comprised of very stiff brown silt with sand and gravel was found between 4.5 to 7 feet bgs. Chuckanut Sandstone was encountered from the 7 to 8 feet bgs, the extent of the boring to refusal. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Because the Reveille Reservoir is bearing directly on bedrock, an allowable bearing pressure of 6,000 psf can be used for structural analysis. Bedrock is not at risk of liquification and the site is not expected to have issues with slope instability.

13.2 Inspection Summary

A drained inspection of the Reveille Reservoir took place on May 21, 2019.

13.2.1 Exterior Inspection Summary

13.2.1.1 Site and Security

Unlike many other reservoirs, Reveille's site does not have a perimeter fence (Figure 13-1). The site is apparently frequented by local skiers and snowboarders, who use the sloped roof and berm to gain speed for a jump. The site is generally well-graded, directing runoff away from the

reservoir. During the site visit, it was noted that the roof of the reservoir is under the dripline of the trees and other vegetation (Figure 13-3).

Figure 13-3: Trees overhang the reservoir roof and a diagonal crack spreads to the valve vault.

13.2.1.2 Exterior Walls

A berm encompasses most of reservoir's walls. Cracking with efflorescence was noted below the roof overhang and spread diagonally to the valve vault (Figure 13-3 and Figure 13-4).

Figure 13-4 A crack with efflorescence located below the roof overhang

13.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

13.2.1.4 Exterior Roof

The exterior roof of the reservoir had coating intact and appeared largely competent with only minor cracking (Figure 13-5). These cracks included a full-circumference circumferential crack approximately 7 feet up the dome and radial cracks at 4 to 5 feet on center. The roof had a grade

that ranged from 6.8 degrees near the vent to 26.6 degrees near the roof's perimeter. The distance from the roof to ground level is about 3 feet but is greater near the valve vault.

Figure 13-5: Minor circumferential and radial cracking of the roof

13.2.1.5 Exterior Appurtenances

To access the roof, a single retrofitted ladder is used (Figure 13-6 Left). The total distance from the 1-foot high concrete pad to the grating is approximately 10.5 feet. The lowest rung is 18.5 inches from above the concrete pad. As is typical with the City's reservoirs of this construction, the retrofitted grating and safety railing extend only around the entry hatch and is missing a toeboard (Figure 13-6 Right). The entry hatch has a thin contact area with the lid and the gasket was not securely attached. Corrosion was noted on the nuts and washers holding the hatch base to the concrete.

Figure 13-6: The retrofitted exterior ladder (Left) and entry hatch (Right)

Because the sheet metal collar could not be removed, the Reveille Reservoir's vent (Figure 13-7 Left) was assessed via the provided drawings of similar reservoirs and photos of the College Way Reservoir vent retrofit (Figure 13-7 Right). A 3-foot diameter penetration in the roof has a concrete vent structure above it. The concrete overhang is 4 feet, 2 inches. Approximately 2 inches of opening was exposed under the sheet metal collar. The screen was assumed to be similar to the Dakin I, woven with approximately 12 strands per inch.

Figure 13-7: Roof vent (Left) and a provided photo of the College Way Roof Vent retrofit from January 2013 (Right)

The attached valve vault shares a wall and footing with the reservoir. The inlet, outlet, and drainpipes all run through the vault. It appears the inlet/outlet/drainpipe penetration through the slab floor was poorly executed, but likely will not affect the structural capacity of the floor (Figure 13-8 Left). Both the upper and lower vault roofs had signs of water infiltration along the wall edges (Figure 13-8 Right). Infiltration in the lower vault was more severe and may be originating from a circumferential crack on the hopper base. The overflow runs through via an air gap, connecting with the drainpipe. The vault's roof had curb that extend all the way around roof (Figure 13-6 Left).

Figure 13-8: The floor penetration into the reservoir is poorly executed (Left). Infiltration was noted in the lower vault (Right).

13.2.2 Interior Inspection Summary

13.2.2.1 Interior walls

The walls, being uncoated, were noted to have a similar cracking pattern as the exterior (Figure 13-9). Observations of the cracking near the ladder showed the crack to be between 1/8-inch and 1/4-inch in width. Circumferential cracks in the hopper base were 1/16-inch in width.

Figure 13-9: The reservoir's walls

Figure 13-10: Cracks in the walls (Left) and hopper base (Right)

13.2.2.2 Floor

The uncoated floor was covered by a thin layer of sediment, which slightly obscured the floor (Figure 13-11). Cracks were present in the floor.

13.2.2.3 Interior Roof

The ceiling exhibited minor cracking with efflorescence. No instances of concrete spalling or exposed rebar were observed (Figure 13-12).

Figure 13-11: Floor's thin layer of residue (Left) and cracks (Right).

Figure 13-12: The reservoir roof minor radial and circumferential cracking

13.2.2.4 Interior Appurtenances

The interior ladder is rungs set into the wall with rungs spaced 15 inches apart. The ladder was very corroded and exhibited loss (Figure 13-13); it was deemed unsafe for use. A removable ladder, tripod, and winch are used for inspections and maintenance.

Figure 13-13: The interior ladder corroding with material loss

13.2.3 Piping and Valving Inspection Summary

The combined inlet/outlet/drain piping exhibited minor corrosion on the interior of the pipe (Figure 13-14). However, the mouth of the pipe had a rough-concrete interface. The frame around the pipe was also heavily corroded. Significant accumulation of sediment was noted near the pipe. The drainpipe connects with the overflow and extends to the storm sewer without an airgap.

Figure 13-14: The common inlet/outlet frame exhibits corrosion (Left) and significant sediment has accumulated in and around the pipe (Right).

The overflow weir, shown in Figure 13-6 (Right) was crack-free, but rust was noted on the overflow weir gate components. The distance between the supply pipe and overflow rim of the drain pipe is approximately 1 foot (Figure 13-15 Left). The overflow connects with the drain line in the vault, draining to the storm sewer. The attached valve vault contained update isolation valves (Figure 13-15 Right).

A hose bibb is in the valve vault, used in washdown operations.

Figure 13-15: The overflow air gap and organic material accumulation on the vault roof (Left). Piping and valving within the vault (Right).

13.3 Condition Assessments

13.3.1 Cleanliness and Coatings

The exterior roof walls and roof have organic material buildup and efflorescence. The exterior topcoat is no longer functional in many places on the reservoir structure, valve vault, and access vault. If spot treated, the exterior coating will be viable for up to 10 years (+/-) to come. The debris accumulation, topcoat condition, and exposed concrete caused the exterior walls and roof to be in fair to poor condition.

The rust-colored sediment accumulation around the inlet/outlet/drain pipe indicates the reservoir is due for a washout inspection. The staining on the lower section of the valve vault may be caused by water leaking from the reservoir.

13.3.2 Material Deterioration

Significant concrete deterioration was noted at the roof-to-wall interface, which needs attention. Because this cracking is structural in nature, it is discussed in Section 6.3.3.1. The interior walls and floor are also in poor condition. Similar in pattern to the exterior, the interior walls, as well as the hopper base and floor are cracking. These cracks may be allowing water to infiltrate into or exfiltrate out of the reservoir.

Corrosion on the interior ladder, hatch anchoring, pipe frame, and overflow weir is significant and needs attention.

13.3.3 Structural Performance

13.3.3.1 Static Analysis

The dome roof was found to meet modern requirements of AWWA D110-13. However, the roof-to-wall interface is not designed to allow for roof expansion and contraction, which has resulted in cracks. Based on the supplied plans, the wall reinforcing appears to be acceptable for static loads. However, the upper sections of the interior wall have reinforcing at 18 inches on center, which exceeds ACI 350.3's maximum allowable spacing of 12 inches on center. The foundation was found to be sized appropriately, based on the geotechnical report's soil bearing capacity of 6,000 psf.

13.3.3.2 Seismic Analysis

At the 17-foot operating level, the walls' reinforcement was found to be suitable for seismic event. This was not the case if operated at the overflow level. Similarly, the design seismic event would cause a constrained slosh wave, causing damage to the roof if operated at the overflow level. However, at the 19-foot operating level, the roof is adequately reinforced. Incidental damage may

still occur to hatches and appurtenances. The valve vault attachment is suitable to prevent "differential movement and impact during the seismic event. Also, based on assumed backfill conditions, the hopper base is not likely to fail and collapse onto the piping. However, the pipes do not have flexible couplings which would address potential differential settlement between the two structures during a seismic event.

13.3.4 Water Quality/Sanitary

Although the common inlet/outlet/drainpipe configuration can cause problems with water quality issues, none have been reported. A removable silt stop is required on combined drains/outlets but was not installed. As this reservoir drains to a storm drain, a DOH approved air gap is required on both the overflow and the drainpipe. No air gap is installed on the drainpipe. The air gap between the overflow and receiving pipe was found to be too narrow. While the screen on the vent could not be carefully measured due to the sheet metal collar, it does not appear to be 24-mesh screen backed with 4-mesh screen. The roof hatch would be classified as a "high-maintenance design" because of issues getting the gasket to provide a tight seal against the relatively small surface area of the frame. The gasket not being fully adhered could potentially lead to water quality issues within the reservoir. Overall, the reservoir is in fair water quality/sanitary condition.

13.3.5 Safety

With the total fall distance from the exterior ladder to ground level being 10.5 feet, no ladder fall protection is required. However, the exterior ladder's first rung is 18.5 inches off the ground, so it is not in current L&I compliance; the maximum rung spacing is 12 inches. This reservoir roof has some areas where a fall would be greater than 4 feet. A fall would be greater than 4 feet from the valve vault as well. Thus, an L&I-compliant railing is required on these areas. The reservoir roof is considered steep as the greatest slope is greater than 4:12. Thus, additional fall prevention is required.

The interior ladder is too corroded to use and the rung spacing is 16 inches, more than the maximum OSHA-allowed 14 inches. It was deemed unsafe and there is no fall protection in the interior. We used a mechanical winch to lower ourselves into the reservoir for the inspection. While this configuration appears to meet OSHA requirements, there is inherent risk of using a tripod mounted on a vault and a removable ladder. Overall, the safety condition is fair.

13.3.6 Operations and Maintenance

13.3.6.1 Site and Security

The lack of a perimeter fence was a noted deficiency with the reservoir. An intrusion alarm is installed on the roof's access hatch. Drainage was found to be suitable for this reservoir. Trees were too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the

soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is in fair to poor condition.

13.3.6.2 Roof Drainage

The reservoir roof sheds water adequately. Drainage of the valve vault roof appeared to not function in the event of a drain clog. If the drain were to be clogged, the roof would overflow the curb; no notches or overflow scuppers are present that would send water away from the reservoir. There are apparently no waterstops installed between the reservoir and valve vault. The water visible at the roof-wall interface also shows that rainwater or reservoir water may be infiltrating into the reservoir.

13.3.6.3 Appurtenances

The first rung of the exterior ladder was difficult to climb. Also, accessing the interior of the reservoir is more difficult than other style reservoirs as the tripod, winch, and removable ladder must be lifted to the roof. Reveille does not have a ladder on the exterior of the roof access hatch vault, so climbing into the hatch can also be challenging. The Reveille Reservoir met design requirements for the vent on the roof penetration, screened area, and free area on the retrofitted collar.

13.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. However, no dechlorination system was observed onsite for drained water.

13.3.7 Obsolescence

Overall, the equipment on site appeared to be of newer design, so a favorable obsolescence score was given.

13.3.8 Condition Scoring

Overall, the condition score of the Reveille Reservoir was fair, 3.7. Problems associated with this reservoir were related to the structural problems with the roof-to-wall interface. A summary of the scoring is shown in Table 13-1 and the full Score Matrix is included in Appendix K.

Table 13-1: Condition Scoring of the Reveille Reservoir

Categorical Scores	
Cleanliness and Coatings	4.4
Material Deterioration	3.9
Structural Performance - Static	4.1
Structural Performance - Seismic	3.3
WQ/ Sanitary	3.8
Safety	2.3
Operations & Maintenance	3.8
Obsolescence	5.0
Overall	**3.7**

13.4 Recommended Improvements

13.4.1 Cleanliness and Coatings

First, the exterior should be pressure washed to remove dirt and other debris. The interior should also be washed out to remove the sediment that has accumulated on the floor, especially near the inlet/outlet/drain pipe.

13.4.2 Material Deterioration

To halt water from migrating through the cracks, the interior is recommended to be coated with a product similar to that used on the other reinforced concrete dome reservoirs (Versaflex AquaVers primer and polyurea). Discussed in Section 13.4.3 are possible solutions to deal with the thermal expansion-related issues of the roof-to-wall interface.

The corrosion on the access hatch vault lid nuts, overflow weir gate components, and inlet/outlet/drain pipe frame can be addressed with cleaning and recoating. The existing corroded interior ladder should be removed and replaced, per Section 13.4.5.

13.4.3 Structural

The recommended improvement to address the issues related to thermal deformation occurs in stages:

1. A temporary improvement should be employed soon to halt further deterioration of any reinforcing material until a permanent solution is implemented. This would entail removing damaged concrete from the cracking areas, properly cleaning cracks, and assessment by a structural engineer. Likely, the areas will need to be spot coated to prevent water damage to the reinforcing. It is imperative that any repair media be flexible so as not to accelerate structural damage as thermal movement is likely to continue to occur until the problem is addressed. Stiffening or reinforcing the cracking areas is not recommended.

2. To more permanently address the thermal deformation-related issues, several options are available. A new aluminum geodesic roof is recommended. The City can also consider a new concrete roof that is domed or flat. Another option would be lifting the entire roof and retrofitting it with a bracketing system and elastomeric bearing pad between the roof and walls. These options would likely cost much more than a new aluminum geodesic roof.

3. The cracks should be investigated by a structural engineer after the thermal deformation issue is resolved.

The best approach to deal with the wall reinforcement being spaced 18 inches on center rather than 12 inches on center is to monitor the structure. The City should also consider lowering the overflow to ensure it is not operated above the maximum allowable operating level in the future.

Because the valve vault shares a footing with the reservoir, there isn't space available to install flexible connections on the pipes. Instead, a new vault should be constructed with sufficient space between it and the reservoir to allow for the installation and proper operation of flexible connections.

13.4.4 Water Quality/Sanitary

Water quality should continue to be monitored as common inlet/outlet/drain configurations can cause water stagnation in the reservoir. A removable silt stop should be installed on the combined drain/outlet pipe. A DOH-approved air gap should also be installed on the drain pipe. With the drain pipe being 10 feet underground, this would likely be logistically challenging at the Reveille site. Possible options are draining to daylight, an appropriately sized dedicated dry well with backflow prevention, or an air gap with a sump and sump pump that can pump water than cannot be drained by gravity.

The overflow air gap can be widened on the reservoir. A trash rack is recommended on the receiving pipe. With the wall of the reservoir close to the overflow, the gap needs to be three times the pipe diameter.

The gasket on the roof access hatch can be replaced with neoprene seals on both the frame and the lid or replaced with a hatch with better contact area. The vent screen, if in fact is not compliant, it can be replaced with a #24-mesh with a 4-mesh screen backing.

13.4.5 Safety

The exterior ladder can have an additional rung installed on the lowest level. An L&I-compliant railing is recommended around valve vault roof. The railing will also need to extend around the perimeter of the reservoir at all areas where the fall distance is greater than 4 feet. Additionally, on the reservoir roof, a fall restraint system or positioning device system, or possibly a fall arrest

system if the work is considered roofing work, is recommended. A toe board should be added to the existing grating.

On the interior, steps should be taken to ensure ingress and egress to the reservoir is as safe as possible. A new interior ladder with fall protection system is recommended. This could be similar to the ladder installed at the Kearney Reservoir, which has a removable extension and is shown in Figure 9-16. A new exterior ladder should be installed on the exterior of the access vault that is similar to the ladder installed on the 40th Street Reservoir, shown in Figure 9-7 Right.

13.4.6 Operations and Maintenance

13.4.6.1 Site and Security

A fence with barbed wire should be installed around the reservoir. Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

13.4.6.2 Roof Drainage

On the valve vault roof curb, notches or overflow scuppers should be installed to prevent the roof from overflowing the curb if the drain backs-up. These improvements would be incorporated into the design of any new valve vault. To address the staining and apparent infiltration in the lower portion of the valve vault, the joints between the valve vault and reservoir should be sealed to prevent leakage. Coating of the interior of the reservoir may halt this infiltration into the vault as well. The staining can then be cleaned, and further action taken if the water reappears.

13.4.6.3 Appurtenances

The are no recommended improvements related to the operations and maintenance for the appurtenances for the Reveille Reservoir.

13.4.6.4 Valving and Piping

Proper dechlorination of the Reveille Reservoir will need to consist of a weighted system that is set in the manhole or affixed to the culvert at the outlet. An example of a basket system is shown in Figure 4-21.

13.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Reveille Reservoir are shown in Table 13-2.

Table 13-2 Reveille deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior	Pressure wash exterior	$ 1,000
	Sediment accumulation on floor	Perform regular washdown inspections	$ 1,000
DETERIOR-ATION	Cracks in concrete and leaks	Coat interior	$ 99,000
	Corrosion on access hatch vault lid nuts, overflow weir gate components, and pipe frame	Clean and (re)coat	$ 5,000
STRUCT	Roof-wall-interface does not account for thermal deformation	Resolve roof-wall-interface issues a. Clean & coat cracks related to thermal deform. b. Replace roof with new geodesic dome c. Structural inspection to determine next steps	 $ 8,000 $ 110,000 $ 2,000
	Maximum reinforcement spacing exceeded	Continue to monitor for structural issues	$ -
	Operated at the overflow elevation, seismic slosh impacts roof and structure is under-reinforced	Consider lowering overflow	$ -
	Missing seismic valves & vault adjacent to reservoir	New seismic valve vault and associated piping	$ 200,000
WQ	Inlet and outlet are combined	Continue to monitor water quality; consider upgrades	$ -
	No silt-stop on combined drain/outlet	Install removable silt-stop	$ 1,000
	No drain air gap	Upgrade drain pipe w/ air gap	$ 88,000
	Overflow air gap too narrow	Replace overflow	$ 44,000
	Roof hatch is a high-maintenance design (small contact area)	Replace gasket w/ neoprene seals; consider replacement	$ 1,000
	Vent screen non-compliant	Replace vent screen	$ 1,000
SAFETY	Missing ladder rung on ext. ladder	Install ladder rung	$ 1,000
	Lacks steep roof fall protection, railings, and grating railing toeboard	Install roof fall protection, railing on heights greater than 4 feet, and toeboard on grating railing	$ 30,000
	Ingress and egress difficult; interior ladder corroded and non-compliant	Install exterior vault access ladder and replace interior ladder with one with fall protection	$ 33,000
O&M	Fence does not encompass reservoir	Install/replace fence	$ 18,000
	Tree spacing recommendation not met	Tree removal and restoration	$ 11,000
	Valve vault roof lacks overflow drainage	Install notches or overflow scuppers on parapet	$ 2,000
	Lower level valve vault leaks	Clean and seal	$ 1,000
	Dechlorination non-compliant	Update dechlorination system	$ 2,000
		Subtotal	$ 659,000
		30% Contingency	$ 197,700
		8.7% Tax	$ 74,533
		Engineering, Administration, and Construction Management	$ 325,932
		Total Project Costs Estimate	$ 1,257,164
		Total Project Costs Estimate (rounded)	**$ 1,300,000**

13.5 Conclusion

The two biggest items to be dealt with on the Reveille Reservoir are the thermal deformation issues and the lack of flexible couplings on the pipes passing through the valve vault. If the City wishes to update to a modern seismic valve vault and coat the interior soon, it may not be necessary to deal with the staining in the valve vault. The interior should be coated in a similar fashion to the other dome reservoirs to prevent leaking or groundwater infiltration into the reservoir.

With the heavy accumulation of debris near the inlet/out/drain pipe, this reservoir is recommended to be prioritized in its next washout inspection. This should take place within the next 2 years. The reservoir should be leak tested as a first step of the next inspection.

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Section 14

Sehome Reservoir

14.1 Tank and Site Overview

The Sehome Reservoir is a partially buried, concrete reservoir that was built around the 1920s (Figure 14-1). It has a listed storage capacity of 0.7 MG, resembling in shape a football with the ends excised. As shown in Figure 14-2, the interior is approximately 100 feet long (running northeast to southwest). The reservoir is 73 feet wide at the center and narrows to 66 feet at each end. The end walls (measured at the center of the reservoir's basin) have a maximum height of 13.5 feet on the northeast end and 16.0 feet on the southwest end. The roof, which was added in the 1950s, has a slight slope to the center then to southwest and is supported by 15 1-foot square columns. While no reinforcement or concrete drawings are available for the reservoir, additional information is available in the structural report in Appendix L.

Figure 14-1: The Sehome Reservoir, viewed looking northeast

Two structures are located on the site. A below-grade piping vault that houses the 12-inch main for the reservoir and 10-inch main for Jersey Street is located just southwest of the reservoir. Twenty feet further to the southwest is the concrete box enclosure that houses the overflow pipe outlet.

Located in the Sehome Hill Arboretum off Bill McDonald Parkway, it is around 1-mile south of the Bellingham CBD. Although the reservoir has been empty for the last 5 years, the minimum and

maximum operating levels would be 10 and 13 feet, respectively. It is located within the 457 South Zone, which is fed via the Otis Street Pump Station.

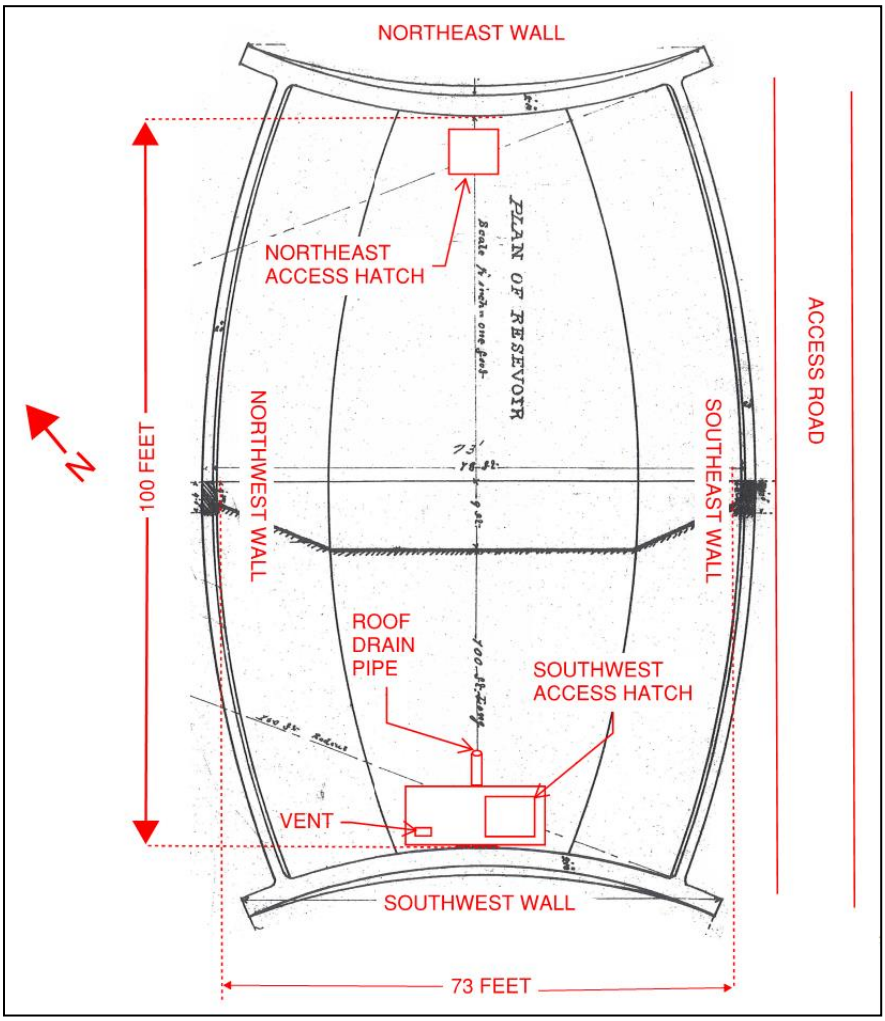

Figure 14-2: Interior dimensions, exterior appurtenances, and site overview of the Sehome Reservoir

A boring drilled east of the reservoir in the geotechnical investigation indicated 1-foot of forest duff and fill to 6 feet bgs. Relict topsoil/weathered horizon was found from 6 to 9 feet bgs, consisting of medium stiff brown silt with sand and organic matter. Glaciomarine drift, which was stiff, brown, sandy silt was found between 9 and 15 feet bgs. Chuckanut sandstone was encountered 15 feet bgs to the boring refusal, 17.5 feet bgs. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. With the reservoir bearing directly on bedrock and glaciomarine drift, an allowable bearing pressure of 3,000 psf can be used. The soils are not at risk of liquification and the site is expected to have low risk of slope instability.

14.2 Inspection Summary

A drained inspection of the Sehome Reservoir took place on January 24, 2019.

14.2.1 Exterior Inspection Summary

14.2.1.1 Site and Security

The reservoir has a barbed wire fence on the northeast, northwest, and southwest walls (Figure 14-1 and Figure 14-3 Left). The fence is missing from the access road on the southeast side of the reservoir (Figure 14-3 Right). It appears the adjacent slope and access road are graded in such a way drainage is routed to the reservoir roof. During the site visit, trees were found overhanging the reservoir.

Figure 14-3: Painted northeast wall, fence, two U-shaped roof vents in the corners (Left). Adjacent slope and access road drains to heavily organic material-laden roof.

14.2.1.2 Exterior Walls

Despite the reservoir's age, the visible portion of the walls were without signs of major cracking and few instances of bugholes or voids in the concrete. The northeast and southwest walls have been painted in murals and were generally free of debris (Figure 14-1 and Figure 14-3 Left). The northwest wall was covered in a thick layer of moss which hampered a complete visual assessment (Figure 14-4). The walls extend higher than the roof, resulting in a curb that ranges from 13 to 19 inches above the roof surface. On the southeast side, the curb is not present where the access road has been graded to the reservoir roof.

Figure 14-4: Moss growth on the northwest wall

14.2.1.3 Foundation

The foundation was unable to be observed as the reservoir is partially buried.

14.2.1.4 Exterior Roof

The roof is accessible from the access road. A significant amount of organic debris accumulation was noted throughout (Figure 14-3 Right) but was noted to be especially heavy along the southeast side near the access road. Clearing debris, it appears the roof may have been coated in the past but has since degraded (Figure 14-5 Left). No cracks were noted in the roof, but much of the roof could not be inspected due to debris. It is slightly sloped, draining to a 6-inch pipe on the centrally located southwest access vault (Figure 14-5 Right). The roof drainpipe extends through the interior of the reservoir and outlets on the near the southeast wall (Figure 14-6). The distance from the roof to ground level ranges from 0 to approximately 7 feet.

Figure 14-5: Exterior roof (Left) and roof drain, southwestern access hatch and one of five U-shaped vents (Right)

Figure 14-6: Roof drain passing through the interior of the reservoir (Left) and outlets to the right of the side access hatch (Right). Jersey Street Water Main penetrates the reservoir near unused exterior ladder (Right)

14.2.1.5 Exterior Appurtenances

A 4-foot by 3-foot side entry hatch on the southwestern wall provides access to the level sensor and overflow weir (Figure 14-6 Right). An exterior ladder was installed to the right of the hatch. This ladder is not currently in use, as the fence is installed at the top of the wall and the roof can be accessed from the adjacent road.

There are two roof hatches installed on the reservoir. The 30-inch square northeast hatch is set on a vault (Figure 14-7) while the 30-inch by 24-inch southwest access hatch shares the vault with a vent and the roof drainpipe (Figure 14-5 Right). While both roof hatch lids appeared to be capable of keeping water out, the applied silicon gasket system was not functional. Some concrete deterioration was noted at the bottom of the structures.

Figure 14-7: The northeast roof access hatch

In each of the four corners is a 3-inch U-shaped roof vent pipe (Figure 14-3 Left and Figure 14-8 Left) that provide air exchange. One additional vent is located on the southwest roof vault (Figure

14-5 Right), for a total of five vents. The south corner vent has 0.125-inch drilled hole screen (Figure 14-8 Center) while the other four vents have 0.125-inch mesh screens (Figure 14-8 Right). The distances from the screens to the surfaces below are approximately 12 inches for all vents.

Figure 14-8: Sehome vents are 3-inch U-shaped pipes (Left). The south corner vent has 0.125-inch drilled holes (Center) while the others have 0.125-inch screens (Right).

14.2.2 Interior Inspection Summary

14.2.2.1 Interior walls

The northeast and southwest interior end walls appeared to consist of a lower portion that extends to just above the slope floor on the northwest and southeast sides (Figure 14-9). A 1-inch thick layer of black-tar mastic can be found at the top of this section. Above the sloped walls and mastic are the original 3-foot high walls that encompassed the open-air reservoir. The 1950s renovation added 5 to 6 feet of wall height to these walls along with the roof.

Figure 14-9: The reservoir's walls did not exhibit major issues (Left). The layer between the old and new walls appeared competent (Right).

Overall, the walls appeared to be largely free of bugholes or cracks other than in two areas. One of those areas is an area of heavy efflorescence on the northwest side wall (Figure 14-10 Left). The

other was the northeast wall below the newer wall addition where a few concrete sections were observed to be failing, exposing the underlying aggregate (Figure 14-10 Right).

Figure 14-10: An area of heavy efflorescence on the northwest side wall (Left) and an area of exposed aggregate on the northeast wall (Right)

14.2.2.2 Floor

The slab floor did not exhibit cracking. An efflorescent sheen was, however, observed on floor. Untreated lumber was mounted to the floor and was found to be rotting with organic growth (Figure 14-11).

Figure 14-11: A sheen and decaying wood on the reservoir floor

14.2.2.3 Interior Roof and Columns

The interior surface of the roof slab was noted to have a variety of minor efflorescing cracks (Figure 14-12 Left). A red hue was noted in some of these areas. The beams appeared to be crack-free, other than the beams running northeast-to-southwest near the center (Figure 14-12 Right). These beams had some notable crack-associated efflorescing observed at their mid-section. The cracking did not appear to be due to structural failure or creep.

Figure 14-12: Cracking was noted on the ceiling panels (Left) and some of the beams (Right)

14.2.2.4 Interior Appurtenances

No interior ladder is installed on the reservoir. The roof penetration for the centrally located roof vent on the southwest hatch was exhibited spalling (Figure 14-13 Left). Heavy corrosion was also noted on the Jersey Street Water Main, especially on the southwest side (Figure 14-13 Right).

Figure 14-13: Concrete failure on one of the roof vent penetrations (Left) and corrosion on the Jersey Street Water Main (Right).

14.2.3 Piping and Valving Inspection Summary

The 12-inch common inlet/outlet and drain piping are all located adjacent to one another on the southwest side (Figure 14-14 Left). All of these pipes appeared to be heavily corroded (Figure 14-14 Center). The drain piping outlet that connects to the storm sewer at Oak Street and High Street was not accessible (Figure 14-14 Right). It did not appear a dechlorination system was in place.

Figure 14-14: The left picture shows the three pipes on the southwest side of the reservoir. From left to right, these pipes are the 12-inch common inlet/outlet, a 6-inch unknown pipe, and the 8-inch drain. The center picture shows a closeup of the combined inlet/outlet pipe, which has corrosion and debris accumulation. The right picture is the drain outlet, which we were unable to access during our inspection due to it being locked.

The overflow weir (Figure 14-15 Left) and overflow outlet (Figure 14-15 Right) were functional. However, the PVC cap on the outlet had 0.125-inch (+/-) holes drilled.

Figure 14-15: The overflow weir and outlet

Within the valve vault, the altitude valve and gate valves appeared to be of older design and were experiencing light corrosion (Figure 14-16).

Washdown piping was not observed during our inspection.

Figure 14-16: The valves and piping within the valve vault are of older design.

14.3 Condition Assessments

14.3.1 Cleanliness and Coatings

The exterior roof, having an exceptional amount of debris, was found to be in poor cleanliness condition. The moss growth on the northwest wall may be deleterious to concrete as the moss produces acidic conditions and the roots can cause or exacerbate cracking in the walls. However, the observable portion of the exterior northeast and southwest walls' decorative coating looks to be holding.

On the interior, the fungus growth and sheen on the interior indicated poor cleanliness condition. The interior pipes were also noted to have corrosion-related material buildup.

14.3.2 Material Deterioration

Overall, the observed concrete deterioration on the exterior is minimal. Minor observed corrosion was noted on the exterior reservoir vents.

Inside the reservoir, the cracking with efflorescence in the interior roof is indicative of water migrating through cracks. Because the red tinge is present on some of these cracks, corrosion could be occurring within the reinforcement. The efflorescence on the northwest wall interior (Figure 14-10 Left) may be due to differential settlement. The cause may be due to during the 1950s roof retrofit, a sandstone outcropping was covered in concrete making this area stiffer compared to the remainder of the foundation. The area of concrete failure on the northeast wall (Figure 14-10 Right) is localized and does not appear to be part of a larger failure pattern. Spalling of concrete near the SW vent may be due to poor reinforcement.

The coating on the observable portion of interior pipes has degraded, leading to corrosion. The Jersey Street Water Main appears to be at end of life due to heavy corrosion.

14.3.3 Structural Performance

14.3.3.1 Static Analysis

Based on field measurements, other than the center, which is supported by a beam, the roof meets requirements for thickness. It is unknown how, or if, the roof is reinforced. Figure 14-17 shows the large amount of debris that was removed from the roof in 2017. Because the roof has not collapsed during previous soil/debris loads, the roof reinforcement likely exceeds static code requirements for spacing. Per code, the roof could likely withstand a load induced by roof drain backup to curb height since the load would be similar to that induced from historic soil/debris accumulation. However, a combination of debris accumulation and water weight from a plugged drain could exceed previous loads, inducing failure. It is unclear if the roof reinforcement is designed to support the weight of a vehicle.

Figure 14-17: A photo taken in 2017 of City staff removing accumulated organic material, soil, and vegetation from the roof

Based on the supplied drawings, it appears the original walls of the Sehome Reservoir are not reinforced, but rather poured thick enough to withstand expected hydrostatic forces. The walls were found to be suitable to withstand the current 13-foot maximum operating level-induced force. The upper section of the roof, mostly located above the waterline, is likely reinforced and does not experience high hydrostatic loads. Based on historical performance, the wall configuration appears to have been sufficient to withstand additional forces on the base of the structure and wall ends.

The columns appear to be suitable to resist design loads applied to them from the roof based on assumed reinforcing density and historical performance. The reservoir does not appear to have a footing, but rather wall thickness varying from 43 to 54 inches wide. Operating at the 13-foot level, the reservoir would induce a bearing pressure less than the maximum allowable pressure of 3,000 psf recommended in the geotechnical report.

14.3.3.2 Seismic Analysis

Current concrete design requires reinforcing to be used to prevent concrete from failure during seismic event. Because the Sehome Reservoir apparently does not have, or has minimal, reinforcing in the lower sections of the walls, the seismic wall reinforcing is deficient. The roof-to-wall interface lacks the ability to deal with thermal expansion or seismic events, but unlike the domed reservoirs, this has not caused significant cracking problems. Based on field estimates, there is likely insufficient thickness at the roof-wall-interface at the end walls to be able to withstand the seismic event. A code-level seismic event at the 13-foot operating level would induce a 2.6-foot slosh wave. At this level, there is currently 3 feet of freeboard, so the roof would not be impacted.

14.3.4 Water Quality/Sanitary

The combined inlet/outlet configuration, coupled with the reservoir's shape (longer than wide), likely means that water is not mixing properly, and stagnation is occurring. The vent and overflow screens do not follow DOH recommendations with a #24 mesh. The undersides of the vents were not high enough to meet DOH recommendations. Also, the access hatch lack of a proper gasket is a noted DOH deficiency. The lack of an air gap on the drainpipe is also an issue. Furthermore, the roof drainpipe passes through the interior of the reservoir. These designs are very risky because if this pipe were to leak, water contamination is expected. The Sehome Reservoir is in poor water quality/sanitary condition.

14.3.5 Safety

The fence encompassing the reservoir is suitable to prevent falls from the roof. No fall protection is needed on the exterior ladder since it is currently not in use and is less than 24 feet. With the total drop on the interior being 16 feet with a removable ladder, fall protection is not required by OSHA. However, fall protection and a permanent ladder would improve safety at this reservoir. The safety condition is good.

14.3.6 Operations and Maintenance

14.3.6.1 Site and Security

The lack of a fence on the southeast wall allows for unpermitted access to the reservoir. There did not appear to be intrusion alarms on the hatches. Trees and vegetation were found to be too close to the reservoir. Nearby trees endanger the structure, slow evaporation from the soil by the structure, are causing organic material accumulation problems, and their roots can penetrate the structure. Drainage from the adjacent slope and access road is directed to the reservoir roof. The lion's share of copious amounts of organic material accumulation on the roof is likely a result of this material being carried with overland flow to the roof. The site/security is in poor condition.

14.3.6.2 Roof Drainage

Water was apparently making its way through the roof cracks. Drainage of the roof appeared to not function as would be required. The pipe is too small and can become easily clogged by the amount of organic material that is on the roof. The roof drainage is in poor condition.

14.3.6.3 Appurtenances

Accessing the interior of the reservoir was manageable, but not ideal with the removable ladder. The Sehome Reservoir did not meet design requirements for the vent on the roof penetration and screened area.

14.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. With the drainpipe manhole being locked, we were unable to assess dechlorination procedures.

14.3.7 Obsolescence

Overall, the equipment on site appeared to be of older design and difficulties in obtaining parts would be expected.

14.3.8 Condition Scoring

Overall, the condition score of the Sehome Reservoir was poor, 2.6. Problems associated with this reservoir were related to the lack of reinforcement on the lower sections, site grading, and litany of DOH deficiencies. A summary of the scoring is shown in Table 14-1 and the full Score Matrix can be found in Appendix L.

Table 14-1: Condition Scoring of the Sehome Reservoir

Categorical Scores	
Cleanliness and Coatings	2.9
Material Deterioration	3.4
Structural Performance - Static	3.3
Structural Performance - Seismic	1.7
WQ/ Sanitary	1.6
Safety	4.5
Operations & Maintenance	2.9
Obsolescence	1.0
Overall	**2.6**

14.4 Recommended Improvements

If the Sehome reservoir is to be salvaged, several major improvements will need to occur. First a testing firm should be employed to map any existing reinforcing in the structure and estimate the concrete's strength. Major structural retrofits would likely be very expensive on this nonstandard, geriatric structure. Should the structure be able to be structurally retrofitted, other improvements will be required. These improvements include regrading the adjacent road; other concrete repairs, removing corrosion from the pipes, improving roof drainage; and installing new vents, a water mixing system, access hatches, interior ladders, and a drain air gap. This work should also include tree removal and restoration around the reservoir. Because the reservoir is located in an arboretum, removal of trees may not be feasible.

While the Sehome Reservoir is historic and unique, it does not appear fiscally advantageous to rehabilitate the structure. Thus, it is recommended to be replaced. Please note that replacing the structure at this site and elevation is not recommended due to the adjacent slopes and abundance of nearby (possibly protected) trees.

While remaining, the City should assume this reservoir will fail during a seismic event, so it should be used only for non-critical storage. The roof should not be driven on, as no information is available about the roof's reinforcing.

It appears the Jersey Street Water Main will need to be replaced.

Planning level costs for a new reservoir are shown in Table 14-2.

Table 14-2: Sehome deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material buildup on exterior		
	Fungus and sheen on interior floor		
	Material buildup in pipes		
DETERIOR-ATION	Roof and wall cracking		
	Concrete spalling near roof vent penetration		
	Corrosion on interior of piping		
	Jersey Street Water Main corroding		
STRUCT	Walls unlikely reinforced		
WQ	Lacks water mixing system		
	Overflow pipe screen non-compliant		
	No drain air gap		
	Roof drain pipe passes through reservoir		
	Access hatches and gaskets non-compliant		
	Vent Screens non-compliant		
SAFETY	Missing interior ladder		
O&M	Fence does not encompass reservoir		
	Lacks hatch intrusion alarms		
	Tree spacing recommendation not met		
	Site is graded so flow is routed to reservoir		
	Poor roof drainage; roof drain blocked	Demolish Ex Reservoir	$ 65,000
	Vents undersized and of non-compliant design	Replace Jersey St Water Main	$ 12,000
	Dechlorination non-compliant	New 0.7 MG prestressed reservoir	$ 1,700,000
		Subtotal	$ 1,777,000
		30% Contingency	$ 533,100
		8.7% Tax	$ 200,979
		Engineering, Administration, and Construction Management	$ 878,878
		Total Project Costs Estimate	$ 3,389,956
		Total Project Costs Estimate (rounded)	**$ 3,400,000**

14.5 Conclusion

Although the Sehome Reservoir may have historic value, it likely represents more of a liability to the City at this time. The City should consult a historic register to ensure the structure is not protected before moving forward with plans to demolish it. It is possible the structure or its material may be able to be reused or repurposed. Should the reservoir remain and be unused, it should be inspected again at 3- to 5-year intervals. The reservoir should be leak tested if it returns to service.

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Section 15

Parkhurst Standpipe

15.1 Tank and Site Overview

A standpipe is a water holding structure that is taller than it is wide. With a height of 35 feet and a diameter of 30 feet, the Parkhurst structure (Figure 15-1) would be considered a standpipe. It was built in 1997 and it has a listed storage capacity of 0.185 MG. There are seven concrete courses, each of which is 5 feet tall. The lowest course had approximately 1-foot buried, and 4 feet exposed. Additional information is included in the structural report in Appendix M.

Figure 15-1: The Parkhurst Standpipe, viewed looking northeast

It is located in the southeastern extent of the service area, about 2.5 miles from the Bellingham CBD. The minimum and maximum operating levels are 16 and 24 feet, respectively, which equates to an operational storage of 0.183 MG. A 6.5-foot by 4-foot valve vault containing associated piping is located 25 feet to the southwest of standpipe.

The Parkhurst Standpipe is the only water holding structure within the highest zone of the distribution system, the 873 Governor Road Zone. The zone is fed via the Governor Road Pump Station. The overflow is located at 34.5 feet, or 872.5 feet City.

The geotechnical investigation indicated Chuckanut Sandstone from the surface to the terminal of the boring at 5.5 feet bgs. From the surface to 4 feet, the sandstone was weathered, consisting of very dense silty fine to coarse sand with gravel. Groundwater seepage was not observed at the final depth of boring.

With the nearest mapped faults being 5 and 10 miles away, there is not a significant risk of ground rupture at the site. Since the standpipe is founded on bedrock, the maximum allowable bearing pressure is 6,000 psf; bedrock is not at risk of liquification. This site has a low risk of issues with slope instability.

15.2 Inspection Summary

A drained inspection of the Parkhurst Standpipe took place on May 21, 2019.

15.2.1 Exterior Inspection Summary

15.2.1.1 Site and Security

To limit unauthorized access to the site, a chain link fence with barbed wire encompasses the property (Figure 15-1). The site is generally well-graded, directing runoff away from the standpipe. Trees were growing just outside of the fence which was 16 to 17 feet away from the standpipe walls. Some of the trees near the fence were taller than the standpipe. This is the only site that had a camera, which was located just below the exterior ladder.

15.2.1.2 Exterior Walls

While the upper five courses have not been coated, the lower two courses have been previously coated. The upper exterior walls appeared darkly discolored where roof drainage contacts the walls (Figure 15-2 Left).

The lower three courses have efflorescence emanating from between the adjacent concrete course joints (Figure 15-2 and Figure 15-3). There is also pinpoint efflorescence originating from the middle of the course (Figure 15-2 Right). Additional efflorescing horizontal and vertical cracking was also observed in these courses (Figure 15-3 and Figure 15-4 Left). This cracking is minor, less than 1/16-inch wide. Bugholes, which are small defects in the surface layer of the concrete, were also noted throughout the first two courses (Figure 15-4).

It appears that areas of efflorescence in the first two courses had at one point been mechanically ground off, possibly before being sealed, to prevent further efflorescence. The problems have since reemerged.

Figure 15-2: Dark staining in the upper courses (Left) and efflorescence at the concrete course joints and pinpoint efflorescence in the middle of the course (Right).

Figure 15-3: Lower two courses exterior wall cracking efflorescence.

Figure 15-4: Vertical cracking, efflorescence, and bug holes (a name for small defects in concrete)

15.2.1.3 Foundation

The foundation was unable to be observed as the standpipe is partially buried.

15.2.1.4 Exterior Roof

The exterior roof of the standpipe was free of issues (Figure 15-5). It is sloped at approximately 08:12 (3.8 degrees) from the center. The distance from the roof to ground level is approximately 35 feet.

Figure 15-5: The exterior roof and access hatch

15.2.1.5 Exterior Appurtenances

The exterior galvanized steel caged ladder has a lockable door (Figure 15-1 and Figure 15-6 Left). A railing was installed around the 30-inch by 48-inch entry hatch, but not around the perimeter of the standpipe (Figure 15-5 Left and Figure 15-6 Right). The gasket is rubber adhered to the lid. The vent roof penetration and the hood are 8 and 12 inches in diameter, respectfully (Figure 15-7 Left). The mesh has 1/8-inch square openings with 14-gauge stainless steel wire cloth (Figure 15-7 Right). Some corrosion was noted on the hatch lid and anchor points of the railing and vent.

Figure 15-6: The exterior ladder (Left) and roof access hatch (Right)

Figure 15-7: The roof vent (Left) and screen (Right)

15.2.2 Interior Inspection Summary

15.2.2.1 Interior walls

The interior walls had a seal applied to the course joints (Figure 15-8 Left). Bugholes were also present (Figure 15-8 Right), but less notable than the exterior. Some efflorescence was noted within the remnant form panel edges (Figure 15-9).

Figure 15-8: Interior walls sealed at the joints (Left) and smaller bugholes (Right)

Figure 15-9: Course joint seal that is blistering (Left) and minor efflorescence in remnant form panel edges and overflow (Right)

15.2.2.2 Floor

The interior floor slab had no observed issues of cracking or signs of failure. The floor was generally clear of sediment (Figure 15-10 Left).

15.2.2.3 Interior Roof

The interior roof had no observed issues (Figure 15-10 Right).

Figure 15-10: Clean floor and outlet and anti-vortex, ladder, and base of inlet pipe (Left). Roof had no observable issues (Right)

15.2.2.4 Interior Appurtenances

The interior ladder is galvanized steel, anchored to the walls (Figure 15-6 Right and Figure 15-11 Left); both the ladder itself and its anchors were heavily corroded (Figure 15-11). The galvanized steel cable fall protection system was heavily corroded and unusable. A remnant lug was also exhibiting corrosion (Figure 15-12).

Figure 15-11: The interior ladder and fall protection corrosion

Figure 15-12: The Remnant lug's heavy corrosion

15.2.3 Piping and Valving Inspection Summary

As shown in Figure 15-13 (Left and Center), the 6-inch inlet pipe coupling and wall brace are corroding. The PVC is attached and runs 32 feet above the standpipe floor with a 90 degree bend (Figure 15-13 Right). The 10-inch outlet pipe and anti-vortex device are exhibiting corrosion as well (Figure 15-14 Left). A 4-inch drain is centrally located on the floor (Figure 15-14 Right).

Figure 15-13: The inlet pipe coupling (Left) and brace (Center) corrosion. The inlet has a 90-degree bend near the roof (Right).

Figure 15-14: The outlet and anti-vortex device corrosion (Left). The drain is centrally located (Right).

The 6-inch overflow pipe extends below the ground level (Figure 15-15). The overflow and drain daylight to the northeast side of standpipe to quarry spalls. The overflow outlet is above, shown in Figure 15-16 (Left) in the upper center of the photo. The drain pipe is located below the overflow outlet, behind and to the left the dechlorination bag in the photo. Water flows downhill from this area, shown in Figure 15-16 (Right). A closeup of the overflow (Left) and drain (Right) pipes are shown in Figure 15-17. The screens have coarser meshes than #24 mesh.

The valve vault (Figure 15-18) contains a check valve and isolation valves on the inlet and outlet. A hose is located in the vault that can be used in washdown operations.

Figure 15-15: The overflow pipe

Figure 15-16: The daylit drain and overflow pipes and dechlorination system (Left) and area where flow is directed (Right)

Figure 15-17: The outlets of the overflow (Left) and drain pipe (Right)

Figure 15-18: The valve vault and washdown hose

15.3 Condition Assessments

15.3.1 Cleanliness and Coatings

Based on the efflorescence on the exterior walls, the coating that has been applied to the exterior lower two courses is not functioning properly. Discoloration from roof drainage onto walls is common with this type of temporary of structure but does not appear to be negatively affecting the it. The interior is generally clean and is uncoated. Overall, the observed cleanliness and coatings issues appear minor, so the standpipe is good condition related to this assessment category.

15.3.2 Material Deterioration

The observed exterior cracking with efflorescence, efflorescing concrete course joints, and bugholes are discussed in Section 15.3.3 as they may be related to structural issues.

The roof's appurtenances including the roof hatch, railing, and vent, and their anchors are constructed from galvanized steel. In modern reservoirs, these are generally made from aluminum, which is more corrosion resistant on the exterior of this type of structure. The anchors exhibiting rust indicates that these items need attention and are in poor condition.

The interior appurtenances, such as the ladder, fall protection system, pipe clamps, and anti-vortex device are also made from galvanized steel. Galvanized steel is generally not specified for underwater environments on the interior of reservoirs unless they are heavily coated. These appurtenances are generally constructed from stainless steel. Of the corrosion noted on the interior, the ladder, its anchor points, and fall protection system are the most concerning. Should the ladder fail while in use, significant personnel injury may occur. The major corrosion on the remnant lug needs to be addressed.

15.3.3 Structural Performance

15.3.3.1 Static Analysis

The conical roof met requirements for thickness and reinforcing, having sufficient capacity for self-weight, roof live, and snow loads. While the roof-wall-interface does not allow for differential thermal deformation between the roof and wall, it appears that this has not (yet) induced cracking. This may be because the high amount of reinforcing at the roof-to-wall interface.

The vertical wall reinforcing, and all other wall reinforcing appear to be sufficient for current code strength requirements for operation up to the overflow level. In the upper sections of the wall, the spacing of interior and exterior reinforcing is 18 inches on center, which exceeds the current maximum allowable spacing of 12 inches on center. This is likely not the reason for observed issues in the lower courses of the structure.

The cracking in the lower three courses may be due to the concrete being overstressed; checking the wall for the tension induced by the circular reinforcing, the concrete was found to be slightly over capacity. The possibility remains that the cracking and efflorescence has been induced by poor detailing and construction practices which also contributed to bug holes in the concrete. The provided as-builts do not indicate that waterstops between courses or other detailing designed to limit leakage from the reservoir have been installed. Instead the joints have attempted to be sealed, which is not suitable per the bubbling and efflorescence at joints.

The reservoir's foundation is sized appropriately, based on the geotechnical findings, provided as-build drawings, and overflow operating level.

15.3.3.2 Seismic Analysis

The seismic wall reinforcing was found to meet requirements based on the design seismic event. However, during the design event and operated at the overflow level, the standpipe would exceed overturning requirements. The highest level the reservoir could be operated at while meeting requirements for overturning is 31.5 feet. Even at the overflow level, the standpipe was found to meet requirements for seismic lateral loads.

The design seismic event would induce a 2.9-foot slosh wave at the overflow level. Because the overflow is only 6 inches below the roof, a constrained slosh wave would occur; the roof would be damaged. However, at the reported maximum operating height, there is sufficient room to accommodate the slosh wave from the design event.

Operating at the overflow level, the design event would induce a soil bearing pressure less than the maximum allowable bearing pressure of 6,000 psf based on the geotechnical report.

15.3.4 Water Quality/Sanitary

The inlet/outlet configuration appears to be suitable to facilitate adequate mixing within the standpipe. The vent screen appears to follow DOH recommendations with a #24 mesh. However, the overflow and drain pipes do not have this type of mesh installed.

15.3.5 Safety

On the exterior, the fall protection system on the ladder is a cage system, which alone as fall protection will not meet code in 2036. On the roof, the railing extends around the access hatch but not around the reservoir entirely.

As discussed in Section 15.3.2, the corrosion on the galvanized steel interior ladder, fall protection, and anchors is significant. The corrosion on the anchors and ladder presents a safety concern.

Since the interior ladder is greater than 24 feet, it requires working fall protection. Overall, the safety condition of the reservoir is poor.

15.3.6 Operations and Maintenance

15.3.6.1 Site and Security

The fence and drainage were found to be suitable for the standpipe. An intrusion alarm is installed on the roof's access hatch. However, tall trees were found to be too close to the standpipe. Nearby trees endanger the structure, slow evaporation from the soil by the structure, can cause organic material accumulation problems, and their roots can penetrate the structure. Overall, the site/security is in fair condition.

15.3.6.2 Roof Drainage

Although there is not a gutter system or other way to convey water from the structure, there does not appear to be significant damage to the sidewalls yet. Overall, the roof drainage is in good to fair condition.

15.3.6.3 Appurtenances

No difficulties were found using the exterior or interior ladder. The Parkhurst Standpipe did not meet design requirements for neither the vent's roof penetration area nor the screened area.

15.3.6.4 Valving and Piping

The staff did not note any difficulties associated with operating of the reservoir. Assessing the dechlorination system, the bag likely would be pushed out of the way from the force of the water being drained from standpipe. Draining the reservoir appears to cause erosion at the outlet without an adequate energy dissipation system.

15.3.7 Obsolescence

Overall, the equipment on site appeared to be of newer design and would be easily replicable if needed.

15.3.8 Condition Scoring

Overall, the condition score of the Parkhurst Standpipe was good, 4.0. Problems associated with this standpipe were related to the poor condition of the concrete in the lower courses, corrosion on interior appurtenances, and lack of a compliant dechlorination system. A summary of the scoring is shown in Table 15-1 and the full Score Matrix is included in Appendix M.

Table 15-1: Condition Scoring of the Parkhurst Standpipe

Categorical Scores	
Cleanliness and Coatings	4.4
Material Deterioration	3.8
Structural Performance - Static	4.8
Structural Performance - Seismic	4.0
WQ/ Sanitary	4.4
Safety	2.0
Operations & Maintenance	4.0
Obsolescence	5.0
Overall	**4.0**

15.4 Recommended Improvements

15.4.1 Cleanliness and Coatings

The exterior should be pressure washed to remove efflorescence and any accumulated organic material. To halt water from migrating through the concrete and associated efflorescence, the interior is recommended to be coated with a product similar to that used on the reinforced concrete dome reservoirs (Versaflex AquaVers primer and polyurea). The City can consider grinding down the exterior efflorescing areas and patching if desired.

15.4.2 Material Deterioration

Issues with exterior concrete deterioration should be rectified through recoating the interior, outlined in Section 15.4.1.

The corrosion on the exterior roof hatch could be addressed with cleaning and touched-up with a new paint-on galvanized coating. The anchors of the railing and vent should be cleaned and then assessed if they are still structurally competent. These anchors can be replaced with non-corrodible anchors as well. Instead of continuously cleaning and recoating, the City may wish to replace these appurtenances with ones that are constructed from aluminum.

The corrosion on the piping requires cleaning and coating to ensure the pipes keep functioning as designed. If corrosion grows unabated on the outlet pipe, it may eventually block flow. The remnant lug, if not serving a purpose, is recommended to be removed. Otherwise, it should be cleaned back to competent material so that no corrosion remains and coated. While the pipe braces and anti-vortex device could be cleaned and recoated, a better solution would be to replace them with ones constructed from stainless steel.

Recommendations for improvements to address corrosion on the interior ladder, ladder anchors, and fall protection are presented in Section 15.4.5.

15.4.3 Structural

While the roof-wall-interface does not account for thermal movement, no issues are yet observable. This is recommended to be monitored. Similarly, the best approach to deal with the wall reinforcement being spaced 18 inches on center rather than 12 inches on center is to monitor the structure. The City should also consider lowering the overflow to ensure it is not operated above the maximum 31.5 feet, reducing overturning and slosh risk to an appropriate level.

15.4.4 Water Quality/Sanitary

The overflow and drain pipe screens are recommended to be replaced with a #24-mesh with a 4-mesh screen backing.

15.4.5 Safety

The exterior fall protection should be upgraded before the 2036 deadline. The City may consider upgrading the exterior ladder concurrently with other retrofits. For the roof, a fully encompassing railing is recommended.

Given the corrosion on the interior ladder, its anchors, and its fall protection, it is recommended to be replaced. It should be replaced with an L&I-compliant stainless steel ladder and fall protection, similar to that found on the Kearney Reservoir, shown in Figure 9-16.

15.4.6 Operations and Maintenance

15.4.6.1 Site and Security

Trees that are closer to the reservoir than they are tall should be removed or transplanted. The cleared areas can be replanted with shorter growing shrubs on the perimeter to shield the reservoir from view.

15.4.6.2 Roof Drainage

The roof drainage is recommended to be monitored. Should the City conduct other retrofits on the structure, better roof drainage can be considered.

15.4.6.3 Appurtenances

The vent size is recommended to be increased, allowing for adequate airflow during filling and draining events. Designing the vent around the overflow level is recommended. The overflow level requires a minimum penetration area of approximately 1.2 square feet with a safety factor

account for flow through a vent structure. This could be achieved by expanding the existing vent or the addition of a new vent.

15.4.6.4 Valving and Piping

The energy dissipation and dechlorination systems should be updated before the next time the reservoir is drained. Proper dechlorination at the Parkhurst site could consist of a weighted system that is set in the manhole on site or a sturdy system that could be installed on the outlet pipe at the back of the site. An example of a basket system is shown in Figure 4-21.

15.4.7 Capital Improvements Plan Table

Planning level costs for the retrofits to the Parkhurst Standpipe are shown in Table 15-2.

Table 15-2: Parkhurst deficiencies, recommended improvements, and estimated costs

Category	Deficiency	Recommended Improvement	Estimated Cost
CLN & COAT	Organic material and efflorescence on exterior	a. Pressure wash exterior b. Coat interior	$ 2,000 $ 61,000
DETERIOR-ATION	Corrosion on exterior hatch/hardware	Clean and (re)coat. Consider replacing exterior appurtenances with aluminum ones	$ 3,000
	Corrosion on and interior appurtenances, anti-vortex device, and remnant lug	Clean and (re)coat. Consider removing remnant lug. Consider replacing with stainless steel	$ 3,000
STRUCT	Roof-to-wall-interface does not account for thermal deformation	Continue to monitor for structural issues	$ -
	Maximum reinforcement spacing exceeded	Continue to monitor for structural issues	$ -
	Operated at the overflow elevation, seismic slosh impacts roof and structure overturns	Consider lowering overflow to 31.5 feet	$ -
WQ	Drain pipe screen non-compliant	Replace drain pipe screen	$ 1,000
	Overflow pipe screen non-compliant	Replace overflow pipe screen	$ 1,000
SAFETY	Exterior ladder fall protection is cage	Replace exterior ladder fall protection by 2036	$ 6,000
	Lacks roof fall protection	Install roof fall protection	$ 42,000
	Interior ladder fall protection unusable	Interior ladder and fall protection replacement	$ 30,000
O&M	Tree spacing recommendation not met	Tree removal and restoration	$ 5,000
	Vent undersized	Replace roof vent	$ 15,000
	Energy dissipation and dechlorination non-compliant	Update energy dissipation and dechlorination system	$ 4,000
		Subtotal	$ 173,000
		30% Contingency	$ 51,900
		8.7% Tax	$ 19,566
		Engineering, Administration, and Construction Management	$ 85,563
		Total Project Costs Estimate	$ 330,030
		Total Project Costs Estimate (rounded)	**$ 330,000**

15.5 Conclusion

While the Parkhurst Standpipe was designed to be a temporary structure, it is still structurally competent and only needs minor maintenance to keep it functioning. The next washdown inspection should take place within the next 3 years. The reservoir should be leak tested as a first step of the next inspection. This inspection could take place during or after the interior coating and ladder replacement.

Section 16

Conclusion

16.1 Major Notes

The City has many small reservoirs that are in tight sites. The average age of the structures is around 50 years; AWWA (2013) estimates the average lifespan of steel and prestressed reservoirs to be around 50 years. However, Murraysmith has observed well-maintained structures to last significantly longer than this lifespan estimate.

A few improvements should be prioritized. These items include the following:

- Clearing the organic debris from the roofs and roof drains as soon as possible, and on a more frequent basis for reservoirs experiencing problems
- Pressure washing the exteriors of the reservoirs.
- Repairing the AC breaker for the Whatcom Falls I cathodic protection system
- Safety related issues - We recommend that when on the ladders and roofs of reservoirs, dedicated and trained staff be present to monitor roof safety until safety improvements can be implemented.
- Deciding on improvements for the 40th Street Reservoir
- Tree removal and restoration planning
- Repairing the leaking pipe/valve at the Consolidation Reservoir

Before being drained again, all the reservoirs should have dechlorination improvements. The City should also make sure washdown piping is available in reservoirs where we did not observe any during our inspections.

Padden, Whatcom Falls I, and 40th Street Reservoirs have recommendations on lowering the maximum operating level or be faced with expensive structural retrofits. Marietta does as well, although it is not in use. The City should consider how lowering the operating ranges of the reservoirs may affect other reservoirs in their zone. A summary of potential updates in each zone is provided in Table 16-1. If the maximum operating ranges cannot be reduced in these zones, reservoir replacement may begin to look more favorable.

Table 16-1: Differences in Maximum Operating Levels for the Reservoirs

Zone Name	Reservoir Name	Maximum Operating Elevation (ft)		Difference (ft)	Max Operating Level Drop in Zone (ft)
		Current	Adjusted w/o structural repairs		
276 North	Kearney	22	22	0	5.5
	Marietta	50	44.5	5.5	
	Whatcom Falls I	16.5	14	2.5	
	Whatcom Falls II	21	21	0	
457 South	College Way	19	19	0	4.75
	Padden	24.5	19.75	4.75	
	Sehome	13	13	0	
519 Dakin & Yew	Consolidation	19.5	19.5	0	0
	Dakin I	17	17	0	
	Dakin II	15.5	15.5	0	
696 Padden Yew	40th Street	23	16	7	7
	Reveille	19	19	0	
873 Governor Rd	Parkhurst	24	24	0	0

For planning purposes, should the City desire new tanks, provided in Table 16-2 are planning level cost estimates for new reservoirs.

Table 16-2: Planning level cost estimates for new reservoirs

Capacity (MG)	Base Cost[1] ($ Million)	Total Cost[2] ($ Million)
0.5	1.5	2.9
0.75	1.7	3.2
1	1.8	3.4
1.5	2.1	4.0
2	2.4	4.6
3.5	3.2	6.1
4	3.5	6.7
5	4	7.6
7.5	5.4	10
10	6.8	13
15	9.6	18
20	12	24

[1] Includes structure cost. Does not include other costs such as site/land preparation, drawings, access roads, electrical, instrumentation and control, testing/ startup, site finishing, and yard piping.
[2] Includes 30% contingency, 8.7% tax, and 35% Engineering

16.2 Site Considerations

As the City continues in the planning process for water storage, the sites should be considered. A suitable site for a reservoir does not have trees closer to the reservoir than the trees are tall. For partially buried reservoirs, the minimum spacings is 50 feet. Sufficiently spaced trees or shrubs can be planted on the perimeter of the site to shield the structure from view. Part of routine maintenance is clearing any encroaching vegetation. Following this guideline will help extend the life of structures.

It is also important to plan for replacement of these structures. One solution is select a suitable site that is large enough to build another reservoir when the existing structure is nearing the end of its useful life. This will facilitate easy yard piping improvements and demolition can take place after the new structure is online. An example of a site that has space for a new reservoir and adequate vegetation spacing is shown in Figure 16-1.

Figure 16-1: An example of a reservoir site is provided. It is clear of trees and has space to build an adjacent replacement reservoir when the existing structure has reached the end of its life.

To deter vandalism, like what was implemented on the Sehome Reservoir, the City may wish to decoratively paint structures. An example of such a practice is shown in Figure 16-2.

While only the Parkhurst site had a camera, the City should also consider increasing security on its reservoir sites. Additional requirements are likely to be mandated from the Department of Homeland Security in addition to DOH and EPA.

Figure 16-2: Decorative painting on the exterior of reservoirs can deter vandalism

16.3 Future Inspections

AWWA M42 states "Tanks should be washed out and inspected at least once every 3 years, and where water supplies have sediment problems, annual washouts are recommended." Prestressed reservoirs should be inspected on 5- to 10- year intervals per AWWA D110-13. Based on this round of inspections, the City is comfortable draining almost all its reservoirs and can plan future inspections. With 13 reservoirs in its inventory, we recommend the City to conduct inspections on around four reservoirs each year.

16.4 Closing Remarks

Overall, the recommended improvements are the items we noted from our as-built or plan reviews, inspections, and additional analyses. These improvements would be already be included if the structure was built today. There is a great deal more understanding about the structures, appurtenances, and piping/valving than when the structures were designed and built. The governing codes have also changed since these times. Many cities and water suppliers in the region have similar issues to the reservoirs in the City's inventory. The City has done an excellent job at proactively addressing issues and getting high quality water to its customers. This report serves as a roadmap to address the reservoirs and aid the City's decision makers in preparing for next steps.

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