Memorandum

Date: January 23, 2019

Project: Task Order No. 34, Facility Structural Assessments

To: Mr. Jason Canady
City of Grants Pass

From: Mr. Matthew L. Hickey, PE
Mr. Michael L McKillip, PE
Mr. Thomas W. Bloomer, PE (CA)
Murraysmith

Re: Water Infrastructure Capital Maintenance Plan

Section 1 Introduction

This memorandum presents a Water Infrastructure Capital Maintenance Plan to support planning associated with capital improvements and maintenance of the City of Grants Pass (City) water system reservoirs and pump stations. This memorandum also summarizes the Structural Assessment of Reservoirs & Pump Stations conducted by Murraysmith's team member Peterson Structural Engineering (PSE) and presents a prioritized schedule of improvements that considers facility criticality, operational relationships between facilities, and system improvements planned in the City's Water Distribution System Master Plan (WDSMP). The WDSMP focused on identifying existing and future capacity related improvements while assessment and prioritization work conducted under this study focuses on identifying and resolving structural and seismic facility deficiencies not typically identified in a master planning effort.

Project Goals and Approach

The project goals developed with City staff are outlined below:

1. Inspect and analyze the water system reservoirs and pump stations to identify structural condition, evaluate seismic performance, estimate remaining service life and develop improvement recommendations.

2. Prepare capital costs for identified facility improvements (rehabilitation or replacement) to support subsequent financial planning.

3. Prioritize the identified capital improvements considering criteria developed with the City and develop a schedule of improvements. The schedule will outline when dollars need to

be spent to complete improvements, and present average annual costs estimated in present worth dollars to allow the City to determine annual budgeting to rehabilitate or replace existing facilities.

4. Provide a plan to better inform facility capital improvement needs and define which type of projects are associated with maintenance and which projects are capital upgrades.

5. Provide structural analysis for reservoirs and pump stations to support a subsequent resiliency study and planning efforts.

Relationship to Other Planning Documents

Water system improvements planning for the City comes from multiple planning studies. This Water Infrastructure Improvements Plan is intended to work in conjunction with the WDSMP, Water Treatment Plant Facilities Plan (WTP FP), and future seismic resiliency study to help identify and coordinate a comprehensive list of long-range capital improvements to assist the City in managing its water system assets. To avoid double-counting projects, those in the existing Capital Improvements Plan (CIP) will be discussed relative to how these relate to the projects developed under this study but maintained in a separate list from new projects identified in this plan. An overview of the various plans is provided below:

Water Infrastructure Capital Maintenance Plan:

- Seismic focused improvements.
- Concerned with facility seismic performance and service life.
- Planning to provide adequate budgeting for long range facility maintenance and capital improvements and replacements.
- Projects involve pump station and reservoir structures over the service life of each facility.

Water Distribution System Master Plan & Water Treatment Plant Facility Plan:

- Growth focused improvements.
- Concerned with existing and future capacity issues.
- Planning to provide day to day water service and fire protection.
- Projects are based on deficiencies for all water facilities over a 20-year planning cycle.

(Future) Resiliency Plan:

- Water system backbone focused improvements.
- Concerned with resiliency, redundancy issues.
- Planning to provide water to critical facilities (hospitals, fire stations, etc) and reduced water service under emergency conditions.
- Projects are focused over 50-year window.

Plan Outline

This report is structured as follows:

1. Introduction
2. Facility structural inspection and assessment and identification of deficiencies
3. Resiliency
4. Seismic Improvements
5. Recommendations and Improvements
6. Project prioritization
7. Cost estimating information
8. Appendices (Improvement project tables; project plates; structural assessment)

Section 2 Facility Structural Inspection and Assessment and Identification of Deficiencies

Facilities Considered

The existing and planned future reservoir facilities, along with the status relative to facility evaluation, are listed in **Table 1**. The existing and planned future pump station facilities, along with the status relative to facility evaluation, are listed in **Table 2**. Planned facilities identified in the 2015 City's WDSMP Capital Improvements Plan (CIP) are included in the improvements schedule in this plan.

Table 1
Reservoir Facilities Considered

Facility	Level of Status Relative to Evaluation
Reservoir No.3	New, meets current codes; no structural analysis or visual inspection.
Reservoir No.4	Replacement assumed; no structural analysis or visual inspection
Reservoir No.5	Assessed
Reservoir No.6	Assessed
Reservoir No.8	Assessed
Reservoir No.11	Assessed
Reservoir No.13	Planned to be replaced; no structural analysis or visual inspection
Reservoir No.14	Planned reservoir in CIP
Reservoir No.15	Assessed
Reservoir No.16	Planned reservoir in CIP
Reservoir No.17	Planned reservoir in CIP
Reservoir No.18	Planned reservoir (not in CIP)
Reservoir No.19	Planned reservoir in CIP

Table 2
Pump Station Facilities Considered

Facility	Level of Status Relative to Evaluation
Lawnridge Pump Station	Assessed
Madrone Pump Station	Assessed
Harbeck Pump Station	Visual inspection. No structural analysis
Hilltop Pump Station	Visual inspection. No structural analysis
New Hope Pump Station	Assessed
Meadow Wood	Visual inspection. No structural analysis; includes planned capacity upgrades in CIP
Champion Pump Station	Visual inspection. No structural analysis
Starlite Pump Station	Visual inspection. No structural analysis
Laurel Ridge Pump Station	Visual inspection. No structural analysis
Williams Crossing Pump Station	Visual inspection. No structural analysis
Panoramic Loop Pump Station	Visual inspection. No structural analysis, includes planned capacity upgrades in CIP
Hefley Pump Station	Visual inspection. No structural analysis
North Valley Pump Station	Visual inspection. No structural analysis, includes planned pump station replacement in CIP
Ausland Pump Station	Planned facility in CIP
Zone 4N Pump Station	Planned facility in CIP

Results of Facility Assessments

The results of the facility assessments, in support of the project goals, are as follows:

- List of deficiencies associated with capital improvement projects or replacement recommendations.
- Identification of facilities needing capacity expansion or abandonment as identified in other planning documents.
- Project costs for improvement and replacement projects.
- Estimated remaining service life of pump station and reservoir facilities (for budgeting purposes).

Capital Improvements, Capital Maintenance and Routine Maintenance

The City has historically approached non-capacity related facility improvements such as pump replacement and electrical upgrades as routine maintenance. As part of this study, the team worked with the City to better define projects considered routine maintenance related and those projects considered capital maintenance or capital improvements. Table 3 shows upgrades and improvements organized into routine maintenance or capital project categories. The facility

evaluations included in this study focused on identifying capital maintenance and capital improvement needs. As such, routine maintenance improvements are addressed outside of the improvements plan in this effort.

Table 3
Routine Maintenance vs. Capital Projects Criteria

Criteria for Routine Maintenance	Criteria for Capital Projects
RESERVOIRS	
Facility appurtenances (ladders, vents, hatches, lights, etc.) replacement	Reservoir foundation modifications such as base restraint seismic cables and interior wall-curbs
Repair spalled shotcrete on reservoirs	Additional reservoir circumferential wrapping and shotcreting
	Piping and flexible pipe connections upgrades, mixing improvements
PUMP STATIONS	
Pump station roofing repair or replacement	Pump station vault or structure replacement
Pump station piecewise MCC, soft-start, VFD replacement	Pump station electrical overhaul (would coincide with other improvements)
Pump Replacement due to age and use	Pump replacement due to capacity or operational needs
Mechanical or electrical equipment seismic anchorage improvements.	
Mechanical piping and valving piecewise replacement	Comprehensive valve & piping replacement due to poor condition and/or age

Facility Assessments and Findings

Murraysmith and our team's structural engineer, PSE, visited the facilities in February 2018, to conduct visual and structural inspections of the selected facilities. Based on initial findings, destructive investigation of the shotcrete on Reservoirs 5, 6, and 11 were also conducted. A summary of the findings for the facilities structurally evaluated is presented in Table 4 and a summary of the findings for the facilities for which visual inspection without a structural evaluation was conducted is presented in Table 5. For further detail, please refer to the report by PSE included in the appendices.

Table 4
Summary of Structural Analysis Findings

Reservoirs		
Description	**Key Findings**	**Improvements**
Reservoir No. 5 Date constructed: 1982 3.5 Million Gallons Circular, prestressed, concrete tank 144 ft dia. x 30 ft wall Serves Zone 1 Water Delivered from WTP	▪ Insufficient freeboard at maximum level. ▪ Deficient base restraint cables for base shear. ▪ Additional circumferential prestressing required. ▪ Shotcrete repair required. ▪ Hatch has signs of corrosion and does not meet current Oregon OHA requirements.	▪ Add base restraint cables in conjunction with additional circumferential prestressing. ▪ Analyze roof for slosh wave impact. ▪ Replace hatch.
Reservoir No. 6 Date constructed: 1980 3.5 Million Gallons Circular, prestressed, concrete tank 144 ft dia x 30 ft wall Serves Zone 2 Water Delivered from Lawnridge PS	▪ Insufficient freeboard at maximum level. ▪ Deficient base restraint cables for base shear. ▪ Additional circumferential prestressing required. ▪ Shotcrete repair required. ▪ Hatch has signs of corrosion and does not meet current Oregon OHA requirements.	▪ Add base restraint cables in conjunction with additional circumferential prestressing. ▪ Analyze roof for slosh wave impact. ▪ Replace hatch.
Reservoir No. 8 Date constructed: 1982 2.0 Million Gallons Circular, prestressed, concrete tank 108 ft dia x 30 ft wall Serves Zone 3 Water Delivered from Champion PS	▪ Insufficient freeboard (at maximum operating level). ▪ Deficient base restraint cables for base shear. ▪ Additional circumferential prestressing required. ▪ Shotcrete repair required. ▪ Hatch has signs of corrosion and does not meet current Oregon OHA requirements.	▪ Add base restraint cables in conjunction with additional circumferential prestressing. ▪ Replace hatch.
Reservoir No. 11 Date constructed: 2000 4.5 Million Gallons Circular, prestressed, concrete tank 162 ft dia x 30 ft wall Serves Zone 1 Water Delivered from WTP	▪ Seismic shear cans in the roof are exposed with signs of corrosion. ▪ Hatch has signs of corrosion and does not meet current Oregon OHA requirements.	▪ Remove remaining roof seismic shear can mortar and replace with new non-shrink mortar. ▪ Replace hatch.

Reservoir No. 15 Date constructed: 1985 1.3 Million Gallons Circular, prestressed, concrete tank 85 ft dia x 31 ft wall Serves Zone 4N Water Delivered from North Valley PS	▪ Insufficient freeboard at maximum level. ▪ Deficient base restraint cables for base shear. ▪ Additional circumferential prestressing required. ▪ Shotcrete repair required. ▪ Hatch has signs of corrosion and does not meet current Oregon OHA requirements.	▪ In order to operate the tank at the maximum level, the following are recommended: ➢ Add base restraint cables in conjunction with additional circumferential prestressing. ➢ Analyze roof for slosh wave impact. ▪ Replace hatch.
Pump Stations		
Description	**Key Findings**	**Improvements**
Lawnridge Pump Station Date constructed: 1969 8" CMU grouted and reinforced above grade wall. Timber roof. Concrete slab on grade with perimeter footing. Serves Zone 2, and Reservoir 6	▪ Overall in Fair Condition, structurally. ▪ Electrical equipment may not be anchored. ▪ Roof decking and beams meet current code, for gravity loads. ▪ Perimeter wall footings are good.	▪ Add connections between the roof and the top of the in-plane and out-of-plane walls. Without connections the structure is at risk of collapse during an earthquake. ▪ Trim tree to reduce impact to roof.
Madrone Pump Station Date constructed: 1954 Cast-in-place concrete floor, wall and roof. Serves Zone 2, and Reservoir 4	▪ Soil settlement, differential movement. ▪ Diagonal cracks, from settlement. ▪ Limited drawings available, keeping a detailed analysis from being completed.	▪ Extensive improvements required to bring structure up to current code. Due to age, condition, and extent of needed improvements, recommend facility replacement.
New Hope Pump Station Date constructed: 2000 Reinforced CMU walls, partially grouted with vermiculite insulation and a light framed wood premanufactured truss roof. Serves Zone 2NH, constant pressure	▪ Need bracing for out-of-plane gable end walls. Without bracing there can be separation and potentially collapse during an earthquake. ▪ Verify number of connections at the gamble end walls.	▪ Check connections. If found to be less than specified on the project documents, add clips.

Table 5
Summary of Visual Inspection Findings

Reservoirs		
Description	**Deficiencies Noted**	**Key Findings**
Reservoir 4 Date constructed: 1953 0.75 Million Gallons Circular concrete tank with domed concrete roof	Not inspected. ▪ Aging facility. ▪ Valve vault has flooding issues. ▪ By age and type of construction, anticipated to not meet current structural codes.	No structural analysis ▪ Replacement assumed at end of service life. (Reservoir No. 4 has exceeded its service life.)
Pump Stations		
Description	**Deficiencies Noted**	**Key Findings**
Harbeck Pump Station Date constructed: 1999 Package station with enclosure	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Planned abandonment once growth connects zones 2HT and 2HK. ▪ Review generator anchorage adequacy.
Hilltop Pump Station Date constructed: 2009 Two stations, enclosed and unenclosed	▪ No preliminary visual deficiencies noted.	No structural analysis
Meadow Wood Pump Station Date constructed: 2002 Stick frame	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Minor adjustment to grade to remove dirt against the siding.
Champion Pump Station Date constructed: 1982 Concrete vault	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ By age and construction, does not meet seismic standards.
Starlite Pump Station Date constructed: 1982 Concrete vault	▪ Leaking roof hatch.	No structural analysis ▪ Service area could be served from upper zone. Future growth may allow abandonment of facility. ▪ By age and construction, does not meet seismic standards.
Laurel Ridge Pump Station Unknown construction date Two fiberglass clam shells	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Review generator anchorage adequacy.

Williams Crossing Pump Station Date constructed: 2005 Stick frame over/around fiberglass clam shell	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Future growth of pressure zone may allow for replacement of facility. ▪ Minor adjustment to grade to protect foundation and remove dirt against the siding
Panoramic Loop Pump Station Date constructed: 2006 Trailer at grade	▪ No preliminary visual deficiencies noted.	No structural analysis
Hefley Pump Station Date constructed: 1996 CMU/concrete vault	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Planned replacement with Reservoir No. 13 replacement and new Ausland pump station.
North Valley Pump Station Date constructed: 1983 Steel can with cathodic protection	▪ No preliminary visual deficiencies noted.	No structural analysis ▪ Recommend maintaining cathodic protection system.

Approaches to Reservoir Seismic Deficiencies

Seismic Deficiencies and Remedies

The seismic assessment of the water reservoirs included consideration of the effects of water movement in response to an earthquake. Two critical considerations include the calculated height of the slosh wave and the force against the wall. The slosh wave height is compared against the available freeboard, the distance between the roof and the water level at the wall. Deficiencies can be resolved by strengthening the wall to roof joint or lowering the maximum operating water level. The force against the wall and the wall to floor joint can be addressed through strengthening the wall to floor connection and/or applying additional circumferential wrappings and shotcrete. These improvements are the basis for the recommended seismic improvements.

Over the years, the values calculated for the slosh wave height have steadily increased based on prescribed factors. This has resulted in some of the reservoirs assessed were found to have deficiencies relative to current structural codes.

As noted in PSE's report, reducing the water depth in the reservoir may not only solve insufficient freeboard issues but it could also eliminate the need for additional circumferential wrapping and base joint restraint to address base shear. Table 6 presents the current seasonal operating reservoir depths, the recommended maximum operating level based on the structural analysis, and the resulting reduction in recommended water depth for select reservoirs. The next subsection discusses this operational approach to mitigating the seismic deficiencies.

Table 6
Seasonal Reservoir Operating Water Depths and Recommended Depths to Address Slosh

Reservoir (volume – mg)	Overflow Depth (ft)	Freeboard (ft)	Operating Depths (ft)			
			Summer Low/High	Winter Low/High	Seismic Code Rec'd	Rec'd Reduction
Zone 1						
No. 3 (5.0)	30	1	15/27	20/28	30.00	--
No. 5 (3.5)	29[1]	1	15/27	20/28	25.42	2.58
No. 11 (4.5)	30	1	15/27	20/28	26.33	1.67
Zone 2						
No. 4 (0.75)	24		15/20		Not evaluated	Not evaluated
No. 6 (3.5)	29	1	21/26		25.56[2]	0.44
Zone 3[3]						
No. 8 (2.0)	29	1	19/24		25.75	--
No. 15 (1.3)	29	3	4.5/7		25.00[4]	--

Notes:
1. Reservoir No. 5 is shorter than Nos. 3 and 11 but matches overflow elevations.
2. For Reservoir No. 6, allowable depth relative to slosh is 25.56 ft, current wall wrappings limit depth to 26 ft.
3. Reservoir No. 13 in Zone 4 was not evaluated.
4. For Reservoir No. 15, allowable depth relative to slosh is 27 ft, current wall wrappings limit depth to 25 ft.

Reduced Reservoir Water Level Approach

A straightforward, low capital cost approach to addressing slosh height deficiencies is to operate the reservoir at a lower maximum water level. The disadvantage of this approach is the lost storage volume. This is particularly of concern when more than one reservoir operates in the same pressure zone as operationally, they need to "float" at approximately the same water level. This requires a reservoir to operate at a lower level than it would otherwise need to operate. Consequently, if all the City's reservoirs were operated at the recommended slosh height based on the existing structural capabilities, and the reservoirs in the main pressure zone were operated at the lowest slosh height of the three reservoirs, then approximately 2.4 million gallons of storage, or 12 percent of the existing total city's storage would not be available for use. This reduces the water available for emergencies and effectively means the city is operating without one of its reservoirs, a significant financial investment.

For the main pressure zone, this approach results in a significant loss of storage and may not be a desirable approach. However, in the short term, a reduced maximum operating water level does resolve structural/seismic concerns at Reservoir No. 15, which is already operated at a lower level to address water age and water quality issues. The zone 2 reservoirs are also currently operated at a lower water level. A volume of 67,000 gallons of further storage reduction in the zone would address structural/seismic issues in Reservoir No. 6.

Damage Mitigation Approach

An alternative to extensive structural improvements or reduced operating water levels to meet the slosh height requirements would be to minimize the time the water is above the recommended slosh height and make lower cost improvements to minimize resulting damage from inadequate freeboard. For certain reservoirs, the wintertime operating level could be set to meet the freeboard requirement. During summer or shoulder season periods, when additional storage is needed, an increased water depth could be used. Minimizing the time the water depth exceeds the recommended maximum operating level reduces the risk of damage and the potential magnitude of the damage.

Reviews of damage to modern AWWA D110 and conventionally reinforced reservoirs during large seismic events in North America have found that spalling of concrete from the reservoir roof and some leakage at the wall to floor joint occurred when the seismic event exceeded the event for which the structure was designed. The spalled concrete can present risks to reservoir and distribution system piping and especially valving. To prevent sizable blocks of concrete from entering the piping system, stainless steel screening covering the pipe penetrations can be installed.

This approach is not intended to provide a resilient remedy, but the approach acknowledges the financial challenges that can limit improvement plans. The City's future resiliency plan will identify the critical water system backbone and help inform which facilities should be prioritized and which can be allowed greater risk of being taken out of service as a result of a seismic event.

Best Value Approach

Four reservoir structures (Nos. 5, 6, 8 and 15) have identified deficiencies that can be resolved with wall base improvements and additional circumferential wrapping. The cost of implementing these improvements depends upon how much excavation is required to expose the footing and base of wall and the number of additional wrapping required. It may not be financially feasible to make all these improvements. Other options can be used to provide the best value:

- When possible, reduced water operating level can be used to provide a no-cost remedy, although storage volume is sacrificed.
    - This approach is suitable for Reservoir No. 15.
    - As all the main zone reservoirs need to operate at the same maximum water level, and because of the relatively large volume that would be lost to operate at the lowest recommended level, this approach is not suitable for Reservoir Nos. 5 and 11.
- When full improvements are costly, a combination of seasonal water depth reduction and addition of piping screens as discussed previously can be considered. As the current reservoir operations include reservoir water levels near full for a small portion of each

day, the risk of experiencing a seismic event while the water level is above the recommended level is reduced.

  - This approach could be considered for Reservoir Nos. 5 and 11.

- A no-action approach can be used when improvements have a high marginal cost, or when the facility is near its design life. It may be more effective to plan on replacing a facility in the long-term than making costly near-term improvements.

  - This approach is suitable for Reservoir Nos. 4 and 13.

Recommended improvements are discussed in **Section 5**, Recommendations and Improvements.

Section 3 Seismic Improvements

Overview of Requirements

On January 10, 2018, new Oregon Administrative Rules (OARs) were adopted for the State of Oregon – Oregon Health Authority (OHA), Drinking Water Services. One key component of the revisions to OAR 033-061 addressed seismic resiliency of water systems: OAR 333-061-0060(5)(J). This revision added an element to Water System Master Plans, requiring a seismic risk assessment and mitigation plan for water systems. The specific components required include:

1. Identifying critical facilities capable of supplying key community needs, including fire suppression, health and emergency response and community drinking water supply points.

2. Identifying and evaluating the likelihood and consequences of seismic failures for each critical facility.

3. A mitigation plan that may encompass a 50-year planning horizon and include recommendations to minimize water loss from each critical facility, capital improvements or recommendations for further study or analysis.

Resiliency plans require proper asset characterization as one of the first steps in the planning effort. Asset characterization includes identification of risks to operations, structures, and life/safety concerns. Risks may include structure failure from wind or seismic loadings, flooding, facility security, ground stability, power disruption or facility access disruption.

It is recommended that the City address seismic resiliency of the water system during the next Water Distribution System Master Plan update. While this work supports future resiliency planning, there is substantial further effort to meet the OAR requirements, to include:

1. Establish level of service goals for during and after an emergency event.

2. Identification of the supply/transmission, distribution system, storage and pumping "backbone" that connects the essential water system facilities to each other and other non-water system critical facilities, such as hospitals, critical medical facilities, and emergency services

3. Using the findings of this study and geotechnical data regarding the seismic response of local geology, evaluate the likelihood and consequences of failure.

4. Develop a mitigation plan that will prioritize improvements to meet level of service goals.

Reservoir Resiliency

To support subsequent resiliency planning efforts planned by the City, facility evaluations identified opportunities to improve seismic performance including assessing critical piping connections to pump stations and reservoirs, and determine if new flexible piping connections are required facilities. Evaluations also identified on-site or nearby risks to structures and utilities such as utility crossings, slope stability risks, stream or culvert crossings. These facility risks and criticality are presented in Table 7.

It is assumed that future reservoir structures will be designed to meet seismic design standards consistent with current Structural and Mechanical Specialty codes. In addition to improving the seismic performance of the reservoir structures, flexible expansion joints and seismically-activated isolation valves can be used to improve the storage system resiliency.

Pipe to Reservoir Connections

At each distribution or transmission piping connection to a reservoir, significant stress can be placed on the pipe as a result of differential movement between the buried pipe and the reservoir structure. To minimize the risk of pipe breakage at this location, it is recommended that a flexible expansion joint be installed at this interface. Flexible expansion joints must be capable of allowing axial expansion/contraction and transfers differential movement.

Automated Isolation Valves

Automated isolation valving with seismic valve actuators should be considered on reservoir outlet piping. There are several considerations to determine whether use of an automatic shut-off valve at a reservoir is recommended:

<u>Pros:</u>

- Reservoir water volume is preserved for use during recovery from an emergency.

<u>Cons:</u>

- Water may not immediately be available for fire suppression use. This drawback can be mitigated by use of remotely actuated valves or only partially closing the valve.

- Requires maintenance of batteries for backup power.
- Potential for accidental closures due to false alarms. This can be mitigated by using multiple sensors to actuate the valve. For example, a significant seismic event followed by a measured large flow rate within a few minutes would trigger valve closure, but each independently would not.

The City should consider the specific performance objectives of each reservoir associated with a seismic event and the anticipated response and recovery period to determine whether the installation of seismically actuated valves is warranted. For example, if two reservoirs serve a pressure zone, one may be equipped with seismic valves to preserve the water volume for use during recovery while the other will remain connected to the system to provide fire suppression and emergency water with the risk that this volume may be lost through main breaks.

As part of participation in Shake Alert, the City has installed a seismic event alarm signal at the City's Reservoir No. 3 site. Staff are currently developing a strategy for early warning response.

Reservoir Seismic Assessment Summary

The non-structural seismic related risks are summarized in Table 7 which also identifies the criticality for each reservoir. Seismic valves are recommended for the higher criticality reservoirs.

Table 7
Reservoir Seismic Related Risks

Reservoir Facility	Flexible Pipe Connections Present	On-site Risks to Structures	Criticality
Reservoir No. 3	Yes	Inlet/Outlet crosses under irrigation culvert	**Highest** (newest, most robust reservoir in main zone)
Reservoir No. 4	No		**Low** (small volume, other reservoir in zone)
Reservoir No. 5	No	Trees present maintenance issue. Inlet/Outlet crosses under irrigation culvert	**Moderate** (Multiple reservoirs in main zone)
Reservoir No. 6	No		**Highest** (Main reservoir in large zone 2; supports several facilities)
Reservoir No. 8	No		**High** (several facilities and zones rely on Res 8)
Reservoir No. 11	No		**Moderate** (Multiple reservoirs in main zone)
Reservoir No. 13	No		**Low** (small volume, can be supplied by pumping)
Reservoir No. 15	No		**High** (provides fire protection for North Valley area)

Pump Station Resiliency

Seismic risks and improvements are reported in **Table 4**. In addition to the structural risks, electrical equipment anchorage should be reviewed for all pump stations. Especially at the older stations, electrical panels lacked anchorage to the facility walls and/or floor that could result in further damage during a seismic event.

Similar to reservoir structures, pipe connections at the pump station buildings present specific vulnerability as a result of differential movement and settlement. To minimize the risk of pipe breakage at this location, it is recommended a flexible expansion joint be installed at this interface. Flexible expansion joints should be capable of allowing axial expansion/contraction and vertical or horizontal offset.

Standby power should also be provided, by standby generators, at all critical pump station facilities. The standby generators should be equipped with on-site fuel storage for at least 24 hours of operation. While a significantly greater volume of fuel will likely be required to sustain operation of the generator through the recovery period following a seismic event, storage of greater volumes of fuel present complications and are likely not economically feasible. Standby generator improvements are identified in the current WDSMP.

Section 4 Project Prioritization

Projects associated with major improvements for each facility were prioritized to assist the City with assessing and scheduling upgrades. For each facility, the next recommended major improvement, which may include an overhaul, structural improvements, or a replacement project, is identified. These improvements will be prioritized for incorporation into the improvements schedule considering the prioritization factors discussed below.

Prioritization Factors

Several prioritization factors will be considered in determining the best order in which to make improvements. Draft prioritization will be developed and reviewed with the City as part of the Evaluation Workshop to be conducted at the City offices.

- Facility criticality (service area size, redundancy)
- Facility condition (failing, poor, good, new)
- Associated planned improvements
- Planned facility retirement
- Facility interdependencies (for example, pump station with adjacent reservoir)
- Others:
    - Distribution of improvements over time to help manage budgets
    - Service life based on City's experience with maintaining similar facilities

Design Life and Budget-level Improvement Schedule

Facility life can be difficult to estimate for reservoirs and pump stations. While there are numerous examples of 100-year old plus facilities in the pacific northwest, there are also facilities that are replaced after only a couple decades of service. Typically, capacity issues drive water system pump station and reservoir replacement although seismic performance can be a key consideration in reservoir replacement and improvement.

For planning purposes, it can be difficult to anticipate when a facility will need to be replaced or require major improvements; however, in budgeting for capital improvements, some assumptions are required to establish a design life of the facilities. Selecting a design life is critical as the design life assumptions can have a significant impact on water system rates.

For budgeting purposes, major improvements and facility replacement should be planned at an established frequency. The proposed frequencies shown in Table 8 establish a baseline for estimating facility life expectancy.

Table 8
Baseline Assumed Project Frequency

Facility	Major Improvements	Facility Replacement
Pump Station	25 & 50 years	75 years
Concrete Reservoir	50 years	100 years

Periodic Improvements Schedule Review and Update

This report establishes the framework for prioritizing upgrades, capital maintenance, and replacement of pump station and reservoir facilities for long-term budgeting. The resulting improvements schedule is intended to be reviewed and updated periodically. The final timing of major improvements should also consider fiscal, political, system growth and up to date facility conditions when determining capital expenditures. Mechanical failures may trigger capital improvement projects, and some facilities may perform better than anticipated. Furthermore, future conditions may change the project impetus or provide for cost saving opportunities when coordinated with other public or private improvement projects.

Prioritized Improvements Schedule

The selected improvement cost, project prioritization, and major improvement frequencies were used to generate the capital improvements schedule. The first improvement timing were determined by the project prioritization effort. Subsequent projects for the same facility will be scheduled through the budget-level design life of the facility to prepare a complete improvements schedule picture. The following figure presents a conceptual improvement schedule.

A summary of project prioritization factors is presented in **Table 9** (on page 19)for reservoir projects and **Table 10** (on page 20) for pump station projects. Facilities with more orange- and yellow-colored features shown in the tables should be given a higher prioritization. In general, reservoirs No. 8, 11, and 15 are most critical due to their importance in supplying their respective service areas and the number of pump stations interacting with Reservoir No. 11.

The timing for new facilities, major improvements/maintenance and replacements projects are illustrated in **Figure 1** (page 21). **Figure 2** (page 22) shows the estimated project costs for 5-year blocks for the next 50 years. **Figure 3** and **Figure 4** in **Appendix E** present a 100-year window to show how project costs can be anticipated. Many of the City's facilities are of similar age, as such, the replacement projects are clustered together.

In addition to the project prioritization factors presented in **Table 9** and **Table 10**, certain facilities have site-specific factors that affect prioritization and are discussed below:

- Reservoir No. 4: Located in pressure zone 2, this reservoir has maintenance issues associated with its age has a relatively small storage volume. Planned Reservoir No. 19

will serve the nearby Foothills Boulevard area industrial development in pressure zone 2 and can be sized to accommodate replacement of Reservoir No. 4 and thereby provide a cost-effective replacement of the existing storage. Ideally, Reservoir No. 19 should be constructed prior to Reservoir No. 4 being abandoned.

- Reservoir No. 13: This reservoir is undersized and has maintenance issues. The CIP includes replacement with a larger structure in the short-term.

- Harbeck Pump Station: It is anticipated that the pressure zone 2 service areas for the Harbeck and Hilltop pump stations will be connected. Once connected the smaller Harbeck pump station can be abandoned.

- Hefley & Ausland Pump Stations: Zone 4 in the northeast of the city is served by Hefley pump station and Reservoir No. 13. To resolve storage and current and future pumping capacity deficits, Reservoir No. 13 is proposed to be replaced and a new Ausland Pump Station will replace the Hefley Pump Station.

- New Zone 4N Pump Station: Per the 2016 WDSMP, a new constant pressure pump station adjacent to North Valley Reservoir No. 15 is recommended to serve potential future development along Highland Avenue between Morewood Lane and Pony Lane. This area is too high in elevation to be adequately served from existing adjacent North Valley transmission mains.

The City's current WDSMP also identifies four proposed reservoirs in the water system CIP, Reservoirs No. 14, 16, 17 and 19. The timing for construction for each of these reservoirs is dependent on its service area development and growth. The WDSMP also discussed a potential reservoir that may be required, Reservoir No. 18, that is not in the current CIP. Reservoir No. 18 is also dependent on its service area development and growth and is further dependent on finalization of the potential pressure zone boundary changes, which are also growth dependent.

The proposed reservoirs and pump stations identified in the CIP are included on the list of projects on the project schedule, **Figure 1**, for context.

Table 9
Reservoir Project Prioritization Factors Summary

Reservoir	Redundancy	Condition	Associated facilities	Comments
No. 3	1 of 3 reservoir in zone	Good		
No. 4	1 of 2 reservoir in zone	Poor	Res. 19	Aged, maintenance issues
No. 5	1 of 3 reservoir in zone	Good		
No. 6	1 of 2 reservoir in zone	Good		
No. 8	Sole reservoir in zone	Good	Champion PS, North Valley PS, Laurel Ridge PS	
No. 11	1 of 3 reservoir in zone	Good		
No. 13	Sole reservoir in zone	Fair	Hefley PS, Res. No. 13	Planned replacement, undersized
No. 14 (future)	Sole reservoir in zone	(future)	Laurel Ridge PS	Growth dependent
No. 15	Sole reservoir in zone	Good	North Valley PS	
No. 16 (future)	Sole reservoir in zone	(future)	Meadow Wood PS	Growth dependent
No. 17 (future)	Sole reservoir in zone	(future)	New Hope PS	Growth dependent
No. 18 (future)	1 of 2 reservoir in zone	(future)		Growth dependent
No. 19 (future)	Sole reservoir in zone o	(future)	Res. 4	Growth dependent

Table 10
Pump Station Project Prioritization Factors Summary

Pump Station	Redundancy	Condition	Associated facilities	Comments
Lawnridge	1 of 2 stations supplying zone	Good		
Madrone	1 of 2 stations supplying zone	Poor		Replacement recommended
Harbeck	Sole supply to service area	Good		Planned abandonment
Hilltop	Sole supply to service area	Good		
New Hope	Sole supply to service area	Good	Res. 17, Williams Crossing PS	
Meadow Wood	Sole supply to service area	Good	Res. 16, Panoramic Loop PS	
Champion	Sole supply to service area	Poor	Res. 8, Laurel Ridge PS	
Starlite	Partial supply at failure	Fair		Future growth may allow replacement; vault and/or hatches may be leaking and should be repaired.
Laurel Ridge	Sole supply to service area	Good	Res. 14, Champion PS	
Williams Crossing	Sole supply to service area	Good		Future growth may allow replacement
Panoramic Loop	Sole supply to service area	Good	Meadow Wood PS	
Hefley	Sole supply to service area	Good	Res. 13, Ausland PS	Planned abandonment
North Valley	Sole supply to service area	Good	Res. 15	Anticipate replacement with Paradise Ranch development due to inadequate build-out capacity.
Ausland (future)		(future)		Growth dependent
PS P-4 (future)	Sole supply to service area	(future)		Growth dependent

Figure 1
50-Year Schedule of Projects

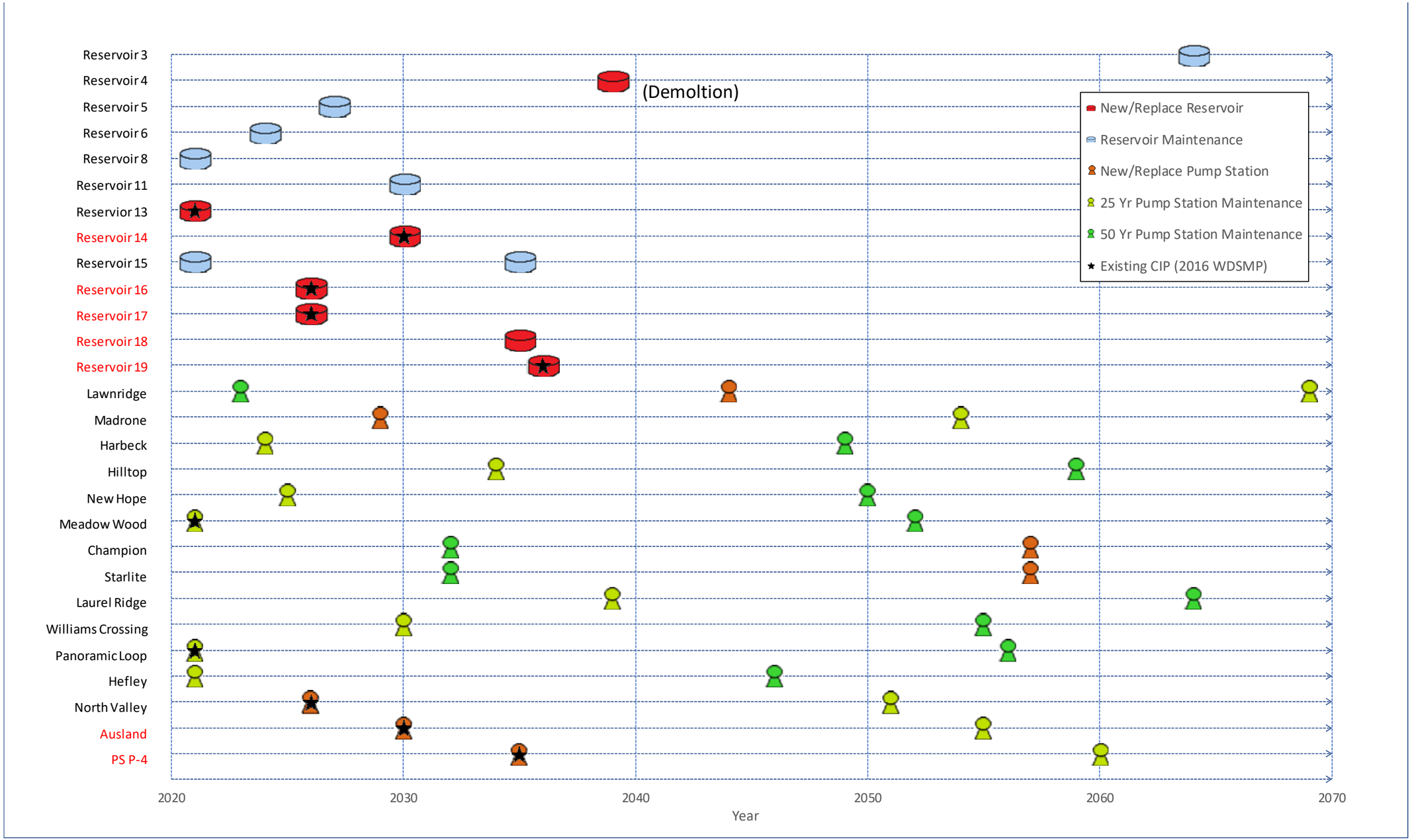
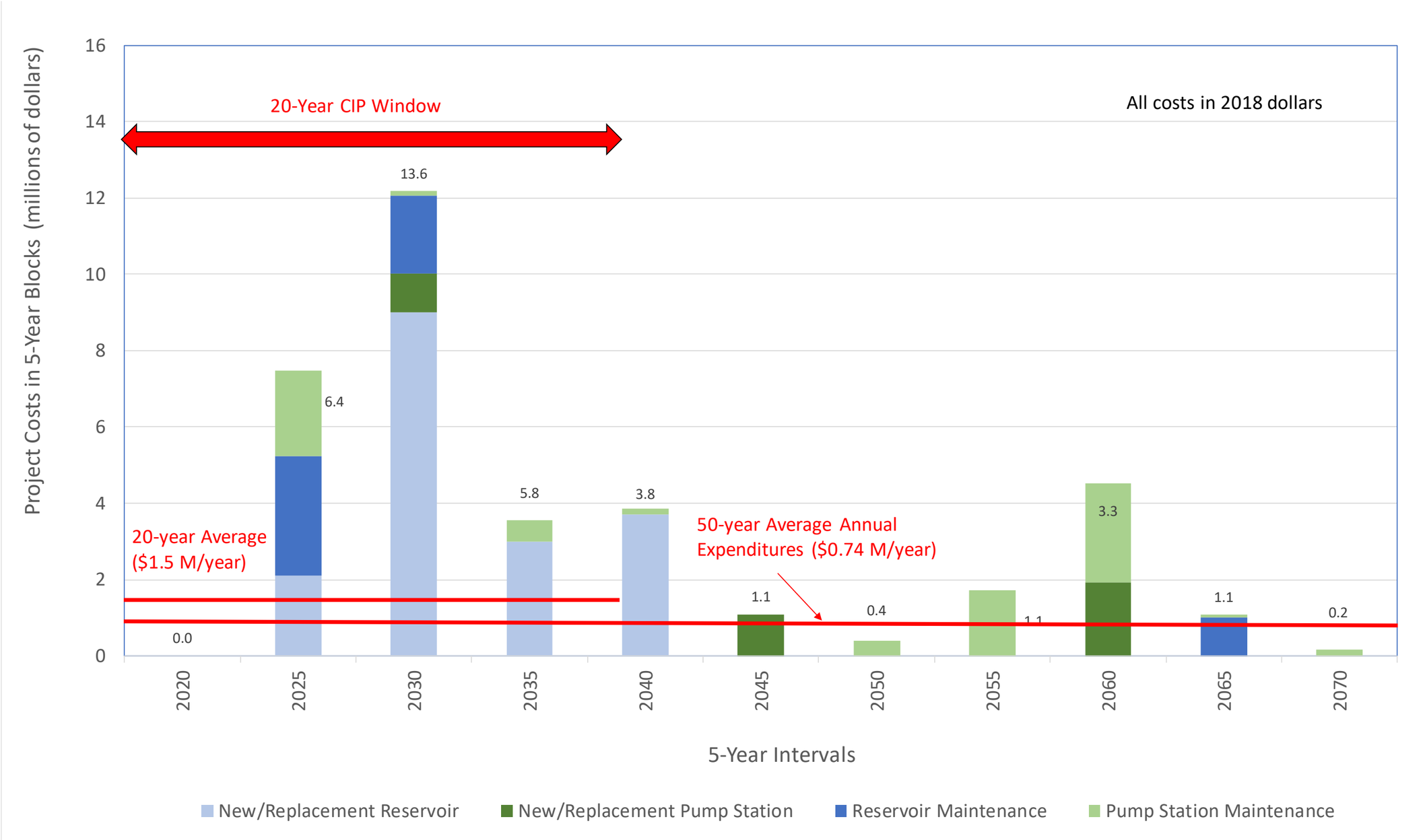


Figure 2
Scheduled Project Costs (5-Year Block Intervals), 50-Year Window



Section 5 Recommendations and Improvements

This section discussed the recommended improvements for all pump station and reservoir facilities. Costs are reported in the project plates in **Appendix C**, the capital improvement/maintenance project table in **Appendix B** and used in **Figure 1** and **Figure 2**.

Several minor maintenance improvements are recommended and described for certain facilities. These improvements are minor and are recommended to be completed within the next 2 years (2021). It is anticipated that the cost of these improvements can be addressed through existing maintenance budgets and performed by City staff or solicitation for quotes from third-party contractors.

Reservoirs

The existing reservoir structures and their recommended improvements are discussed below, and include the improvements noted in **Table 4**. Specific improvements will be determined during future design. For budgeting purposes, reservoir improvements are assumed to include any noted appurtenances, flexible connection improvements, and exterior foundation, circumferential wrapping, and shotcreting to improve the seismic performance of the structure.

Reservoir No.3

- Constructed in 2014. Meets current structural codes. No recommended improvements.
- Schedule: Maintenance improvements are anticipated when the structure is 50-years old in 2064.

Reservoir No.4

- Constructed in 1953. Structure has reached its service life and should be replaced. Planned Reservoir No. 19 will serve the Foothills Boulevard area industrial development in pressure zone 2. Reservoir No. 19 can be sized to accommodate future replacement of Reservoir No. 4.
- Schedule: It is assumed Reservoir No. 19 will be sized to allow for the demolition of the existing Reservoir No. 4. A project for Reservoir No. 4 demolition is included within 3 year of Reservoir No. 19 construction.

Reservoir No.5

- The base of wall connection is inadequate and requires additional restraint. More seismic cables or a new internal curb at the base of wall is required.
- The freeboard to accommodate the slosh height appears inadequate and requires more detailed analysis to determine the needed improvements.

- Roof access hatches have corroded hinges and do not meet current OHA standards. The hatches should be replaced.

- Flexible expansion joints should be added to the reservoir common inlet/outlet and drain piping to improve seismic performance.

- Schedule: Reservoirs 5, 6 and 8 have recommended seismic performance improvements. The city would like to perform one of these improvements every 3 years. Reservoir No. 5 has a backup reservoir in the same zone that meets code and is older than Reservoir No. 11 making it third in priority. Improvements are recommended to be made in 2027.

Reservoir No.6

- Review of the WDSMP storage analysis shows that the Zone 2 storage needs are adequate to operate Reservoir No. 6 at a reduced maximum operating level, approximately 6 inches below the current operating level. This approach resolves both the base of wall connection inadequacy and the apparent freeboard above overflow elevation inadequacy. Future Reservoir No. 19, which will replace Reservoir No. 4 in Zone 2 can be sized to compensate for any lost storage volume or future storage needs. Increasing the size of a future reservoir will be comparable to or more economical than the seismic improvements on a per gallon basis.

- Roof access hatches have corroded hinges and do not meet current OHA standards. The hatches should be replaced.

- Flexible expansion joints can be added to the reservoir common inlet/outlet and drain piping to improve seismic performance.

- Schedule: Reservoirs 5, 6 and 8 have recommended seismic performance improvements. The city would like to perform one of these improvements every 3 years. Reservoir No. 5 has a backup reservoir in the same zone that meets code, reducing it in priority. Reservoir No. 6 improvements are recommended in 2024.

Reservoir No.8

- The base of wall connection is inadequate and requires additional restraint. More seismic cables or a new curb is required.

- The freeboard appears inadequate and requires more detailed analysis to determine the needed improvements.

- Roof access hatches have corroded hinges and do not meet current OHA standards. The hatches should be replaced.

- Flexible expansion joints can be added to the reservoir common inlet/outlet and drain piping to improve seismic performance.

- Schedule: Reservoirs 5, 6 and 8 have recommended seismic performance improvements. The city would like to perform one of these improvements every 3 years. Reservoir No. 8 is recommended to receive the first improvement of the three structures:
    - Reservoir No. 8 is the sole storage reservoir in its service area, unlike Reservoirs No. 5 which has a backup reservoir in the same zone that meets code.
    - Reservoir No. 6 has a second reservoir in its service area.
    - The loss of Reservoir No. 8 would require the North Valley and Laurel Ridge pump stations to operate in series with the Champion pump station, which is operationally challenging.

  Reservoir No. 8 improvements are recommended in 2021.

Reservoir No.11

- Roof access hatches have corroded hinges and do not meet current OHA standards.
- Flexible expansion joints can be added to the reservoir common inlet/outlet and drain piping to improve seismic performance.
- Schedule: Reservoir No. 11 does not have recommended seismic performance improvements for the structure.
    - The shear can grout should be repaired as a near-term project to avoid any further deterioration or corrosion. Repairs could be performed by city staff or by a third-party contractor.
    - The flexible expansion and roof access hatch repairs are scheduled for 2030. If budget permits, there may some economy to performing improvements along with Reservoir No. 5 improvements in approximately 2027.
- Near-term Minor Maintenance: Shear can grout infill should be repaired or replaced.

Reservoir No.13

- Structure is undersized and planned to be replaced with a new, larger structure under the existing WDSMP improvements plan.
- Schedule: The current CIP includes replacement of the reservoir by 2021.

Reservoir No.15

- The reservoir has identified free board, circumferential wrapping, and base restraint deficiencies at the reservoir overflow water level depth of 29 feet.

- In the short-term, the recommendation is to operate the water level at or below the maximum safe operating level of 25 feet, which reduces the volume to 1.1 MG. This meets the storage requirements identified in the 2016 WDSMP and allows improvement costs to be safely deferred.
- When the North Valley service area begins to grow, the need for improvements, replacements, or a new operating strategy should be reviewed. For budgeting purposes, seismic improvements are planned when the structure is 50-years old in 2035.

- Near-term Minor Maintenance: The following minor maintenance improvements are recommended:
  - Shotcrete repair to prevent damage to circumferential wrapping.
  - Roof access hatches have corroded hinges and do not meet current OHA standards.
  - Clean/unclog roof drain scuppers.

Pump Stations

All existing pump station structures and their improvements, if any, are discussed below and include the improvements noted in **Table 4** and **Table 5.** Future planned major maintenance improvements will be determined during future assessment and design.

Lawnridge Pump Station

- Add connections between the roof and the top of the in-plane and out-of-plane walls. Without connections the structure is at risk of collapse during an earthquake.
- Roof is old and needs replacement soon.
- Wood trim needs some repair.
- Unclear if electrical equipment is anchored.
- Schedule: The station is 50-years old in 2019. Maintenance improvements are recommended within the next 5 years (2023).
- Near-term Minor Maintenance: Trim tree to reduce impact to roof.

Madrone Pump Station

- Replacement is recommended. Extensive improvements are required to bring the structural up to current code. Building also has a settlement issue. It appears to be more cost effective to replace the station compared to retrofitting.

- Schedule: Station replacement is anticipated when the station is 75-years old in 2029.

Harbeck Pump Station

- Review generator anchorage.
- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2024.

Hilltop Pump Station

- No deficiencies noted.
- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2034.

New Hope Pump Station

- Wall anchorage at the gable end walls is out-of-plane and noncompliant. Minor improvements are required to adequately brace and support the roof trusses to prevent separation and potential roof collapse during a seismic event.
- Sealant at construction joint in CMU wall is cracked and may need repair soon.
- Unclear if electrical equipment is anchored.
- Hydro-pneumatic tank anchorage and flexible connection improvements needed to meet current codes.
- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2025.

Meadow Wood

- Minor adjustment to grade to remove dirt against the siding.
- Schedule: The current CIP includes capacity improvements for fire flow of the pump station by 2021.

Champion Pump Station

- By age and construction, does not meet seismic standards.
- Schedule: Maintenance improvements are anticipated when the station is 50-years old in 2032.

Starlite Pump Station

- By age and construction, does not meet seismic standards. Abandonment assumed when service area expands.
- Apparent hatch leakage should be resolved to protect electrical and mechanical equipment.
- Schedule: Maintenance improvements are anticipated when the station is 50-years old in 2032.
- Near-term Minor Maintenance: Repair of the hatch leakage is recommended as a near-term minor maintenance task.

Laurel Ridge Pump Station

- Review generator anchorage adequacy.
- Unable to determine if the older, skid-mounted clam shell was anchored to meet current seismic code requirements.
- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2039.

Williams Crossing Pump Station

- No structural issues noted.
- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2030.
- Near-term Minor Maintenance: Make minor adjustment to grade to protect foundation and remove dirt against the siding.

Panoramic Loop Pump Station

- No structural issues noted.
- Schedule: The current CIP includes capacity improvements for fire flow of the pump station by 2021.

Hefley Pump Station

- There is a substantial amount of efflorescence built up on the CMU walls. Recommend cleaning the wall to see if the efflorescence returns. If so, this would be indicative of

water transmission through the walls which should be addressed, such as with a sealant, to prevent long term weakening of the CMU walls.

- Schedule: Maintenance improvements are anticipated when the station is 25-years old in 2021.

North Valley Pump Station

- Active cathodic protection system should be maintained as required to protect the steel structure.
- The 2016 WDSMP identified a capacity limitation of the existing steel can pump station.
- Schedule: The current CIP includes replacement of the pump station by 2026.

Section 6 Cost Estimating Data

An estimated project cost has been developed for each improvement project recommended in this section. Cost estimates represent opinions of preliminary costs only, acknowledging that final costs of individual projects will vary depending on actual labor and material costs, market conditions for construction, regulatory factors, final project scope, project schedule and other factors. The Association for the Advancement of Cost Engineering International (AACE) classifies cost estimates depending on project definition, end usage and other factors. The cost estimates presented here are considered Class 4 with an end use being a study or feasibility evaluation and an expected accuracy range of -30 percent to +50 percent. As the project is better defined, the accuracy level of the estimates can be narrowed.

Estimated project costs are based upon recent experience with construction costs for similar work in Oregon and southwest Washington and assume improvements will be accomplished by private contractors. Estimated project costs include approximate construction costs and an aggregate 45 percent allowance for administrative, engineering and other project related costs. Estimates do not include the cost of property acquisition. Since construction costs change periodically, an indexing method to adjust present estimates in the future is useful. The Engineering News-Record (ENR) Construction Cost Index (CCI) is a commonly used index for this purpose. For purposes of future cost estimate updating; the current ENR CCI for Seattle, Washington is 11480.25 (July 2018).

Reservoir replacement cost estimates were developed assuming a reservoir structure cost and costs for mass excavation, foundation improvements, yard piping, and associated project work. Specific estimates are included in the project plates included in Appendix C. Estimated project costs include approximate construction costs and allow for contingency, permitting, administration, and engineering fees. Costs do not include any land or right-of-way acquisition or demolition and do not include any ongoing maintenance or operation expenses.

Pump station replacement cost estimates were developed using a methodology that consisted of summarizing and assigning a replacement cost to major equipment components and the structure. Major components included pumps, control valves, isolation valves, chemical feed pumps, instrumentation and control panels, and backup power. These costs were summed and multiplied by a factor of two or three to account for ancillary costs like procurement, supplier profit, and installation. The factor was determined based on the perceived complexity of the pump station. Finally, a structure cost was calculated by multiplying the existing building area in square feet by a unit cost per square foot. Similar to the pump station factor, the unit structure cost was chosen based on the perceived complexity of the existing building. The sum of the equipment and structure cost is the estimated replacement cost for the pump station. Estimated project costs include approximate construction costs and allow for contingency, permitting, administration, and engineering fees. Costs do not include any land or right-of-way acquisition or demolition and do not include any ongoing maintenance or operation expenses.

Maintenance project costs, for budgeting purposes, were taken to be 15 percent of the pump station replacement cost and 15 percent of the reservoir replacement cost. Actual maintenance project timing and costs should be evaluated on a case-by-case basis to better refine short-term (< 5 years) budget needs as the need for timing of these projects is better defined.

All costs include a 45 percent allowance for engineering, permitting, administration, and construction contingency.

Replacement project costs for all pump stations and reservoirs are included in **Table 11** and **Table 12**.

Table 11
Reservoir Replacement Project Costs

Facility	Project Replacement Cost ($M)
Reservoir No.3	10.00
Reservoir No.4 (demolition)	0.10
Reservoir No.5	7.00
Reservoir No.6	7.00
Reservoir No.8	4.00
Reservoir No.11	9.00
Reservoir No.13	2.10
Reservoir No.14	1.50
Reservoir No.15	2.60
Reservoir No.16	3.90
Reservoir No.17	3.60
Reservoir No.18	3.00
Reservoir No.19	3.60

Table 12
Pump Station Replacement Project Costs

Facility	Project Replacement Cost ($M)
Lawnridge Pump Station	1.09
Madrone Pump Station	1.03
Harbeck Pump Station	0.36
Hilltop Pump Station	0.90
New Hope Pump Station	1.33
Meadow Wood	1.36
Champion Pump Station	1.18
Starlite Pump Station	0.75
Laurel Ridge Pump Station	0.51
Williams Crossing Pump Station	0.38
Panoramic Loop Pump Station	0.57
Hefley Pump Station	0.92
North Valley Pump Station	1.00
Ausland Pump Station	0.50
Zone 4N Pump Station	1.20

Section 7 Appendices:

A. Current (2016 WDSMP) Capital Improvement Projects

B. Improvements Schedule for Capital Improvements, Resiliency, and Capital Maintenance Projects

C. Selected Project Plates

D. Reservoir Slosh Wave Height Background

E. 100-year project schedule

F. Report, "Structural Assessment of Reservoirs & Pump Stations," Peterson Structural Engineers, December 2018.

MLH:mlm:twb

APPENDIX A
CURRENT (2016 WDSMP)
CAPITAL IMPROVEMENT PROJECTS

Table 5-5
Capital Improvement Program (CIP) Summary

Improvement Category	CIP No.	Project Description	CIP Schedule and Project Cost Summary				Preliminary Cost % to Growth
			5-year thru 2021	10-year 2022-2026	20-year 2027-2036	Estimated Project Cost	
Storage Reservoirs	R-13	0.7 MG Ausland Reservoir - Zone 4 Reservoir No. 13 replacement	$ 2,100,000			$ 2,100,000	40%
	R-14	0.5 MG Laurel Ridge Reservoir			$ 1,500,000	$ 1,500,000	40%
	R-16	1.3 MG Meadow Wood Reservoir		$ 3,900,000		$ 3,900,000	69%
	R-17	1.2 MG New Hope (Cathedral Hills) Reservoir		$ 3,600,000		$ 3,600,000	42%
	R-19	1.2 MG Pearce Park Reservoir - Zone 2 Spalding Industrial Park			$ 3,600,000	$ 3,600,000	100%
		Capital Maintenance	$ 75,000			$ 75,000	52%
		Subtotal	$ 2,175,000	$ 7,500,000	$ 5,100,000	$ 14,775,000	$ 9,282,000
Pump Stations	P-1	Meadow Wood P.S. high (Zone 3MW) - fire flow capacity upgrade	$ 250,000			$ 250,000	52%
	P-2	Panoramic P.S. - fire flow capcity upgrade	$ 400,000			$ 400,000	52%
	P-3	Ausland P.S. supplying proposed Ausland Reservoir (R-13)	$ 500,000			$ 500,000	52%
	P-4	Zone 4N P.S. - constant pressure			$ 1,200,000	$ 1,200,000	100%
	P-5	North Valley P.S. replacement		$ 1,000,000		$ 1,000,000	79%
		Capital Maintenance	$ 125,000			$ 125,000	52%
		Subtotal	$ 1,275,000	$ 1,000,000	$ 1,200,000	$ 3,475,000	$ 2,654,145
PRVs	V-1	Spalding Industrial Area - Ament Rd PRV			$ 150,000	$ 150,000	100%
	V-2	Zone 4N Highland Ave PRV			$ 150,000	$ 150,000	100%
	V-3	Blue Gulch PRV			$ 150,000	$ 150,000	100%
	V-4	Overland PRV			$ 150,000	$ 150,000	100%
	V-5	10th Street PRV	$ 150,000			$ 150,000	52%
	V-6	NW B Street PRV			$ 150,000	$ 150,000	100%
	V-7	Zone 2A PRV replacements (Capital Maintenance)	$ 250,000			$ 250,000	52%
		Subtotal	$ 400,000	$ -	$ 750,000	$ 1,150,000	$ 958,000
Distribution Mains	M-1, 2, 3, 9, 10	Piping improvements for fire flow	$ 683,000			$ 683,000	52%
	M-4 to 8	Zone 2A - Hwy 99, Savage, Manzanita Loop	$ 758,000			$ 758,000	52%
	M-11, 12	Proposed Zone 2H - connect Harbeck and Hilltop			$ 532,000	$ 532,000	100%
	M-13 to 22	Spalding Industrial Area - Zone 2 expansion			$ 3,181,000	$ 3,181,000	100%
	M-24, 25, 26	Zone 3 Granite Hill to Scoville Loop			$ 1,415,000	$ 1,415,000	100%
	M-27 to 30	Zone 3 Scoville to Spring Moutain Loop			$ 1,107,000	$ 1,107,000	100%
	M-31 to 33, 42	Zone 3 I-5 crossing at Cedar Loop, Spring Moutain to Hillcrest Loop			$ 1,396,000	$ 1,396,000	100%
	M-34 to 41, 52	Proposed Ausland P.S. (P-3) and Reservoir (R-13) mains	$ 2,897,000			$ 2,897,000	52%
	M-43, 44	Zone 3 I-5 crossing at Humane Society			$ 570,000	$ 570,000	100%
	M-45, 46	Zone 3 Vine Street Loop - Highland to Hawthorne			$ 996,000	$ 996,000	52%
	M-47 to 51	Zone 4N mains			$ 1,996,000	$ 1,996,000	100%
	M-53 to M-57	Zone 1 Spalding Industrial Area loop			$ 1,362,000	$ 1,362,000	100%
	M-58 to 62	Meadow Wood future mains		$ 1,173,000		$ 1,173,000	100%
	M-63 to 68	New Hope future mains		$ 2,532,000		$ 2,532,000	100%
	M-69 to 75	Laurel Ridge and Blue Gulch future mains			$ 1,870,000	$ 1,870,000	100%
	M-76, 77, 81, 82, 83	Zone 1 Fruitdale future mains			$ 2,087,000	$ 2,087,000	100%
	M-78, 79, 80	Zone 1 Looping- Cloverlawn & Grandview		$ 639,000		$ 639,000	52%
	M-84 to 87	Existing system looping		$ 955,000		$ 955,000	52%
	M-88 to M-102	2-inch main replacement for fire flow	$ 696,000	$ 770,000	$ 420,000	$ 1,886,000	52%
		Routine Main Replacement Program (Capital Maint.)	$ 7,800,000	$ 7,800,000	$ 15,600,000	$ 31,200,000	52%
		Subtotal	$12,834,000	$13,869,000	$ 32,532,000	$ 59,235,000	$ 40,028,280
Planning		Seismic Resilience Study	$ 100,000			$ 100,000	52%
		Water Management & Conservation Plan update		$ 50,000		$ 50,000	52%
		Water Distribution System Master Plan update			$ 150,000	$ 150,000	52%
		Unidirectional Flushing (UDF) Program Development	$ 80,000			$ 80,000	52%
		Distribution Piping Corrosion Study	$ 100,000			$ 100,000	52%
		Subtotal	$ 280,000	$ 50,000	$ 150,000	$ 480,000	$ 249,600
		Capital Improvement Program (CIP) Total	**$16,964,000**	**$22,419,000**	**$ 39,732,000**	**$ 79,115,000**	**$ 53,172,025**
			Annual Average CIP Cost				
			$3,392,800 5-year	**$3,938,300 10-year**	**$3,955,750 20-year**		

APPENDIX B
IMPROVEMENTS SCHEDULE FOR CAPITAL IMPROVEMENTS, AND CAPITAL MAINTENANCE PROJECTS

murraysmith

Table A-1
Capital Maintenance and Capital Improvement Program Project List

Improvement Category	CIP No.	Project Description	CIP Schedule and Project Cost Summary			
			5-year thru 2023	10-year 2024-2029	20-year 2030-2039	50-year 2040-2070
Reservoirs	R-3M	Reservoir No. 3 50-Year Maintenance				$ 1,000,000
	R-4D	Reservoir No. 4 Demolition			$ 100,000	
	R-5M	Reservoir No. 5 Seismic Improvements		$ 1,855,000		
	R-6M	Reservoir No. 6 Seismic Improvements		$ 1,737,000		
	R-8M	Reservoir No. 8 Seismic Improvements	$ 1,375,000			
	R-11M	Reservoir No. 11 Seismic Improvements			$ 192,000	
	R-13R	Reservoir No. 13 Replacement	$ 2,100,000			
	R-14	New 0.5 MG Laurel Ridge Reservoir			$ 1,500,000	
	R-15Ma	Reservoir No. 15 Minor Maintenance	$ 26,000			
	R-15Mb	Reservoir No. 15 Seismic Improvements			$ 1,215,000	
	R-16	New 1.3 MG Meadow Wood Reservoir		$ 3,900,000		
	R-17	New 1.2 MG New Hope (Cathedral Hills) Reservoir		$ 3,600,000		
	R-18	New Zone 3 Granite Hill Reservoir			$ 3,000,000	
	R-19	New 1.2 MG Pearce Park Reservoir - Zone 2 Spalding Industrial Park			$ 3,600,000	
		Subtotal	$ 3,501,000	$ 11,092,000	$ 9,607,000	$ 1,000,000
Pump Stations	P-1	Meadow Wood P.S. high (Zone 3MW) - fire flow capacity upgrade	$ 250,000			
	P-1M	50-Year Maintenance				$ 200,000
	P-2	Panoramic P.S. - fire flow capcity upgrade	$ 400,000			
	P-2M	50-Year Maintenance				$ 90,000
	P-3	Ausland P.S. supplying proposed Ausland Reservoir (R-13)			$ 500,000	
	P-3Ma	25-Year Maintenance				$ 80,000
	P-4	Zone 4N P.S. - constant pressure			$ 1,200,000	
	P-4Ma	25-Year Maintenance				$ 180,000
	P-5	North Valley P.S. Replacement		$ 1,000,000		
	P-5Ma	25-Year Maintenance				$ 150,000
	P-6Mb	Lawnridge 50-Year Maintenance	$ 160,000			
	P-6R	Lawnridge Replacement				$ 1,080,000
	P-6Ma	50-Year Maintenance				$ 160,000
	P-7R	Madrone Replacement		$ 1,325,000		
	P-7Ma	25-Year Maintenance				$ 150,000
	P-8Ma	Harbeck, 25-Year Maintenance		$ 50,000		
	P-8Mb	50-Year Maintenance				$ 50,000
	P-9Ma	Hilltop			$ 130,000	
	P-9Mb	50-Year Maintenance				$ 130,000
	P-10Ma	New Hope 25-Year Maintenance		$ 200,000		
	P-10Mb	50-Year Maintenance				$ 200,000
	P-11Mb	Champion, 50-Year Maintenance			$ 180,000	
	P-11R	Champion Replacement				$ 1,170,000
	P-12Mb	Starlite			$ 110,000	
	P-12R	Starlite Replacement				$ 740,000
	P-13Ma	Laurel Ridge, 25-Year Maintenance			$ 80,000	
	P-12Mb	50-Year Maintenance				$ 80,000
	P-14Ma	Williams Crossing, 25-Year Maintenance			$ 60,000	
	P-14Mb	50-Year Maintenance				$ 60,000
	P-15Ma	Hefley, 25-year Maintenance	$ 140,000			
	P-15Mb	50-Year Maintenance				$ 140,000
		Subtotal	$ 950,000	$ 2,575,000	$ 2,260,000	$ 4,660,000
		Total	**$ 4,451,000**	**$ 13,667,000**	**$ 11,867,000**	
			Annual Average CIP Cost			
			$890,200	**$1,811,800**	**$1,499,250**	**$599,700**
			5-year	**10-year**	**20-year**	**20-year**

APPENDIX C
SELECTED PROJECT PLATES

City of Grants Pass

Water System Seismic Improvement Project: **S-1, Reservoir No. 5 Improvements**

Water Facility: Reservoir No. 5

Estimated Project Cost: $**1,855,000**, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

- The base of wall connection is inadequate and requires additional restraint. More seismic cables or a new curb is required.
- The slosh height appears inadequate and requires more detailed analysis to determine the needed improvements.
- Roof access hatches have corroded hinges and do not meet current OHA standards. The hatches should be replaced.
- Flexible expansion joints can be added to the reservoir common inlet/outlet and drain piping to improve seismic performance.

Background:

Reservoir No. 5 was constructed in 1983. It is one of three reservoirs in the main service level. Due to the redundancy of facilities in Reservoir No. 5's service area, improvements to Reservoir No. 5 can be deferred to more critical infrastructure improvements.

Related Facilities and Projects:

- Reservoir No. 11 in the same service level also has identified structural improvements.

Project Cost Summary:

Hatch Replacement	$ 10,000
Flexible Pipe Connections	$ 193,000
Structural Improvements	$ 920,000
Mobilization & Misc	$ 113,000
Construction Subtotal	*$ 1,236,000*
Engineering, Permitting, and Administration (30%)	$ 371,000
Contingency (20%)	$ 248,000
Total	**$ 1,855,000**

City of Grants Pass

Water System Seismic Improvement Project: **S-2, Reservoir No. 6 Improvements**

Water Facility: Reservoir No. 6

Estimated Project Cost: $**1,737,000**, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

- Seismic improvements and maintenance improvements to include analysis and selection of seismic improvements to the concrete structure.
- Addition of flexible expansion joints at the piping connections to the reservoir.
- Replacement of access hatches due to condition and compliance with current Oregon Health Authority standards.

Background:

Reservoir No. 6 was constructed in 1982. It is one of two reservoirs in its service area.

Related Projects: None.

Project Cost Summary:

Hatch Replacement	$ 10,000
Flexible Pipe Connections	$ 81,000
Structural Improvements	$ 960,000
Mobilization & Misc	$ 106,000
Construction Subtotal	*$ 1,157,000*
Engineering, Permitting, and Administration (30%)	$ 348,000
Contingency (20%)	$ 232,000
Total	**$ 1,737,000**

City of Grants Pass

Water System Seismic Improvement Project: **S-3, Reservoir No. 8 Improvements**

Water Facility: Reservoir No. 8

Estimated Project Cost: $**1,375,000**, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

- Seismic improvements and maintenance improvements to include analysis and selection of seismic improvements to the concrete structure.
- Addition of flexible expansion joints at the piping connections to the reservoir.
- Replacement of access hatches due to condition and compliance with current Oregon Health Authority standards.

Background:

Reservoir No. 8 was constructed in 1983. It is the sole reservoir in its service area. It provides suction supply for the North Valley pump station.

Related Projects:

None.

Project Cost Summary:

Hatch Replacement	$ 10,000
Flexible Pipe Connections	$ 52,000
Structural Improvements	$ 770,000
Mobilization & Misc	$ 84,000
Construction Subtotal	*$ 916,000*
Engineering, Permitting, and Administration (30%)	$ 275,000
Contingency (20%)	$ 184,000
Total	**$ 1,375,000**

City of Grants Pass

Water System Seismic Improvement Project: **S-4, Reservoir No. 11 Improvements**

Water Facility: Reservoir No. 11

Estimated Project Cost: $192,000, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

- Seismic improvements and maintenance improvements to include analysis and selection of seismic improvements to the concrete structure.
- Addition of flexible expansion joints at the piping connections to the reservoir.
- Replacement of access hatches due to condition and compliance with current Oregon Health Authority standards.

Background:

Reservoir No. 11 was constructed in 1999. It is one of three reservoirs in the main service level. Due to the redundancy of facilities in Reservoir No. 11's service area, improvements to Reservoir No. 11 can be deferred to more critical infrastructure improvements.

Related Projects:

Reservoir No. 5 in the same service level also has identified structural improvements.

Project Cost Summary:

Hatch Replacement	$ 10,000
Flexible Pipe Connections	$ 86,000
Structural Improvements	$ 19,000
Mobilization & Misc	$ 12,000
Construction Subtotal	*$ 127,000*
Engineering, Permitting, and Administration (30%)	$ 39,000
Contingency (20%)	$ 26,000
Total	**$ 192,000**

City of Grants Pass

Water System Seismic Improvement Project: **S-5, Reservoir No. 15 Improvements**

Water Facility: Reservoir No. 15

Estimated Project Cost: $**1,215,000**, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

- Seismic improvements and maintenance improvements to include analysis and selection of seismic improvements to the concrete structure.
- Addition of flexible expansion joints at the piping connections to the reservoir.
- Replacement of access hatches due to condition and compliance with current Oregon Health Authority standards.

Background:

Reservoir No. 15 was constructed in 1985. The reservoir is the sole storage facility for the North Valley service area. The North Valley pump station can provide some supply for the service area should Reservoir No. 15 be off-line.

Related Projects:

None.

Project Cost Summary:

Hatch Replacement	$ 10,000
Flexible Pipe Connections	$ 36,000
Structural Improvements	$ 690,000
Mobilization & Misc	$ 74,000
Construction Subtotal	*$ 810,000*
Engineering, Permitting, and Administration (30%)	$ 243,000
Contingency (20%)	$ 162,000
Total	**$ 1,215,000**

City of Grants Pass

Water System Seismic Improvement Project: **PS-1, Lawnridge Pump Station Improvements**

Water Facility: Lawnridge Pump Station

Estimated Project Cost: $160,000, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

Seismic and maintenance improvements:

- Add connections between the roof and the top of the in-plane and out-of-plane walls. Without connections the structure is at risk of collapse during an earthquake.
- Further investigate electrical equipment anchoring to determine if anchorage is needed. Some anchorage is not readily visible.
- Tree adjacent to building needs to be trimmed to reduce impact to roof.
- Roof is fairly old and needs replacement soon.
- Building wood trim needs some repair.

Background:

Joint pump station and park restroom facilities constructed in 1969. Major improvements or facility replacements may benefit from coordination of park facility improvements in terms of contracting, budgeting and construction efficiency.

Related Projects:

The Lawnridge and Madrone pump stations serve the same large service area. Madrone pump station improvement/replacement should consider long term capacity and reliability of the Lawnridge Pump Station.

After Lawnridge pump station improvements are performed, it is recommended to anticipate a pump station replacement project in another 25 years. Future assessment may determine that the station's condition is adequate to continue operation with some major maintenance to pumping, electrical, SCADA, roofing and coating features as appropriate.

City of Grants Pass

Water System Seismic Improvement Project: **PS-2, Madrone Pump Station Replacement**

Water Facility: Madrone Pump Station

Estimated Project Cost: $1,325,000, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

Seismic and Maintenance Improvement:

- Replacement is recommended. Extensive improvements are required to bring the structure up to current structural specialty code. Building also has a settlement issue. It is cost competitive to replace the station compared to retrofitting.

Background:

The Madrone Pump Station was constructed in 1954. The station has had several improvements to the pumping, roof, and outside vault. The foundation has experienced settlement. Due to the age of the building, upgrades are not anticipated to be cost effective. Relocating the pump station may also put the station is a better location both within the developed neighborhood and a hydraulically favorable location.

Related Projects:

The Lawnridge and Madrone pump stations serve the same large service area. Madrone pump station improvement/replacement should consider long term capacity and reliability of the Lawnridge Pump Station.

After pump station replacement, a 25-year station maintenance project should be anticipated to overhaul major pumping, electrical, SCADA, roofing and coating conditions as appropriate.

Project Cost Summary:

Station Replacement Construction Cost	$	705,000
Engineering, Permitting, Administration, and Contingency	$	320,000
Project Total	**$**	**1,325,000**

City of Grants Pass

Water System Seismic Improvement Project: **PS-3, New Hope Pump Station Improvements**

Water Facility: New Hope Pump Station

Estimated Project Cost: $200,000, (in 2018 dollars)

ENR CCI for Seattle, Washington is 11480.25 (July 2018)

Project Description:

Seismic and maintenance improvements:

- Wall anchorage at the gable end walls is out-of-plane and noncompliant. Minor improvements are required to adequately brace and support the roof trusses to prevent separation and potential roof collapse during a seismic event.
- Sealant at construction joint in CMU wall is cracked and may need repair soon.
- Unclear if electrical equipment is anchored.
- Hydro-pneumatic tank anchorage and flexible connection improvements needed to meet current codes.

Background:

The New Hope pump station was constructed in 2000.

Related Projects:

Planned Reservoir No. 17 (Cathedral Hills Reservoir) is intended to expand storage capacity in the New Hope Pump Station service area.

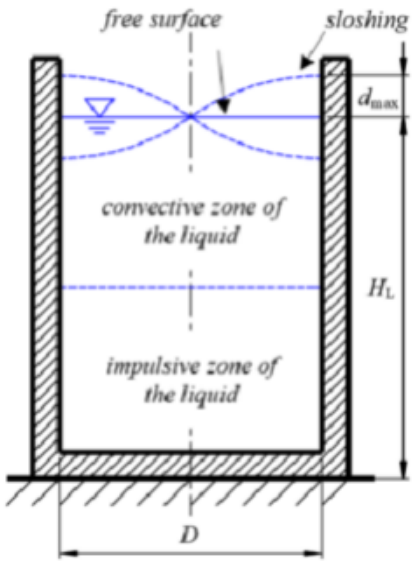
After pump station improvements are performed, a 25-year station maintenance project should be anticipated to overhaul major pumping, electrical, SCADA, roofing and coating conditions as appropriate.

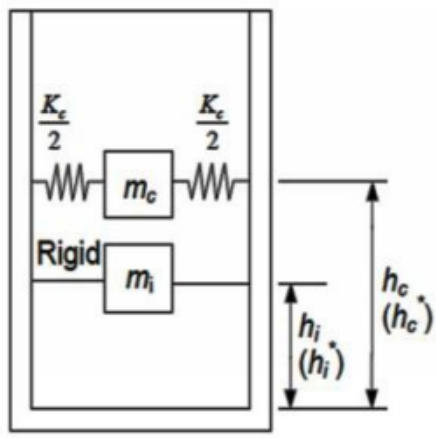
APPENDIX D
RESERVOIR SLOSH WAVE HEIGHT BACKGROUND

Appendix D: Reservoir Slosh Wave Height Background

This attachment provides technical background regarding the design of concrete reservoirs and the changes in design approaches used in the industry.

Part of the structural assessment of a water storage tank covered by Peterson Structural Engineers includes the calculation of the slosh wave height. During an earthquake, the fluid within a tank is excited and begins to move as impulsive and convective masses. The impulsive mass is the lower portion of water that acts with the tank while the convective mass is the free oscillating portion of water near the top. The convective mass is where we find the slosh wave. The slosh wave is dependent on the following parameters: diameter, water depth (which provide a convective period), site specific seismic accelerations (Sds and Sd1) and the mapped long period transition period from ASCE 7/IBC).





Over the years, the values calculated for the slosh wave height have steadily increased. When the City's tanks were constructed it was very common to see calculated values of between 1 and 2 feet, which is what the City's tanks currently have. These calculated values were based on George Housner's study of fluid inside nuclear reactors and is very much the basis of today's equations. And while the foundation of today's equations is rooted in Housner's philosophy, there have been some new ideas introduced. American Water Works Association Standard *D110 – Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks* and American Society of Civil Engineers (ASCE) *7-10 – Minimum Design Loads for Buildings and Other Structures* provide two different approaches to calculating the slosh wave height, as shown below. At face value, the two equations look identical, but they produce two different results because of how each arrives at a convective coefficient (Cc and Sac). The difference is because of how the convective coefficient is determined, AWWA correlates to Ts which is the ratio of Sd1/Sds and ASCE

correlates the convective coefficient, Sac, to T_L which is the mapped long period transition period found in ASCE 7-10.

AWWA D110 – 13: Slosh Wave Eqn.	ASCE 7-10: Slosh Wave Eqn.
• Displacement = 0.42 x Diameter x C_c Where, C_c = Convective Coefficient $T_C \leq \frac{1.6}{Ts}\ Sec$: $C_c = \frac{1.5*Sd1}{Tc} \leq 1.5\ Sds$ $T_C > \frac{1.6}{Ts}\ Sec$: $Cc = \frac{2.4*Sds}{(Tc)^2}$ Where, Ts = Sd1/Sds But need not exceed ASCE 7 or the following: • Displacement = $\frac{3*r*\coth(\sqrt{3.375}*\frac{H}{r})}{\left(\frac{6*(Tc)^2}{Cc*r}\right)-\sqrt{54}}$ Where, r = Radius C_c = Convective Coefficient H = Height of Water T_c = Convective Period	• Displacement = 0.42 x Diameter x S_{ac} x I_e Where, S_{ac} = Spectral acceleration of the sloshing liquid (convective component) based on the sloshing period T_c and 0.5% damping I_e = Importance Factor, taken as 1.0 when structure is Risk Category V • $T_C \leq T_L$: $S_{ac} = \frac{1.5*Sd1}{Tc} \leq 1.5\ Sds$ • $T_C > T_L$: $S_{ac} = \frac{1.5*Sd1*TL}{(Tc)^2}$

Since the slosh wave height is influenced by the water depth in the tank and it appears based on information provided by the City and shown in the table below, that the water depth within each tank varies from the original design level. The slosh wave heights provided in the structural assessment report are based on the overflow water levels, unless noted otherwise, and the larger of the two slosh waves calculated using AWWA and ASCE. As noted in Peterson Structural Engineers' report, making a reduction in the water depth may not only solve insufficient freeboard but it could also mean additional circumferential wrapping and base restraint for base shear are solved.

APPENDIX E
100-YEAR PROJECT SCHEDULE

Figure 3: Schedule of Projects – 100-Year Window

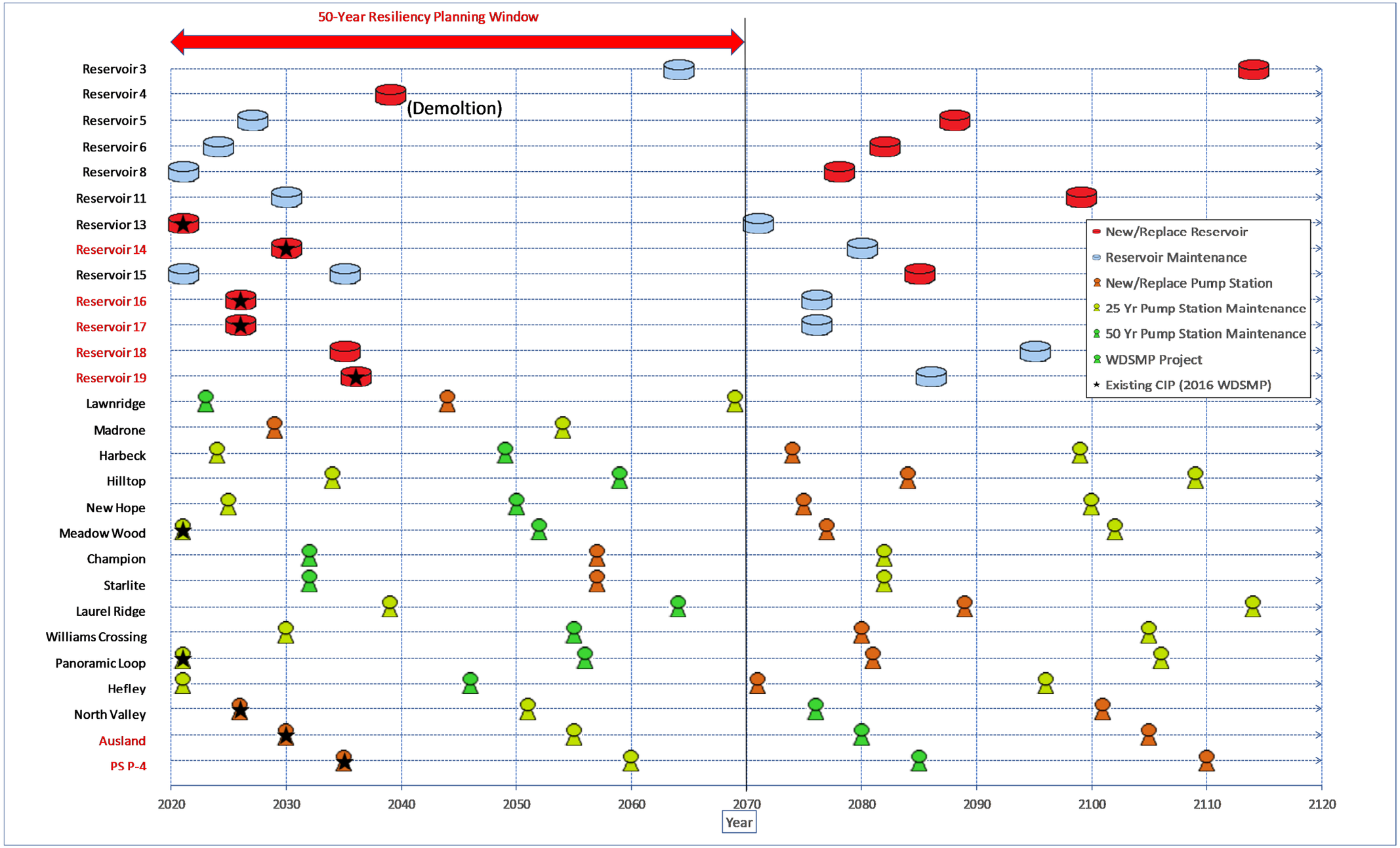
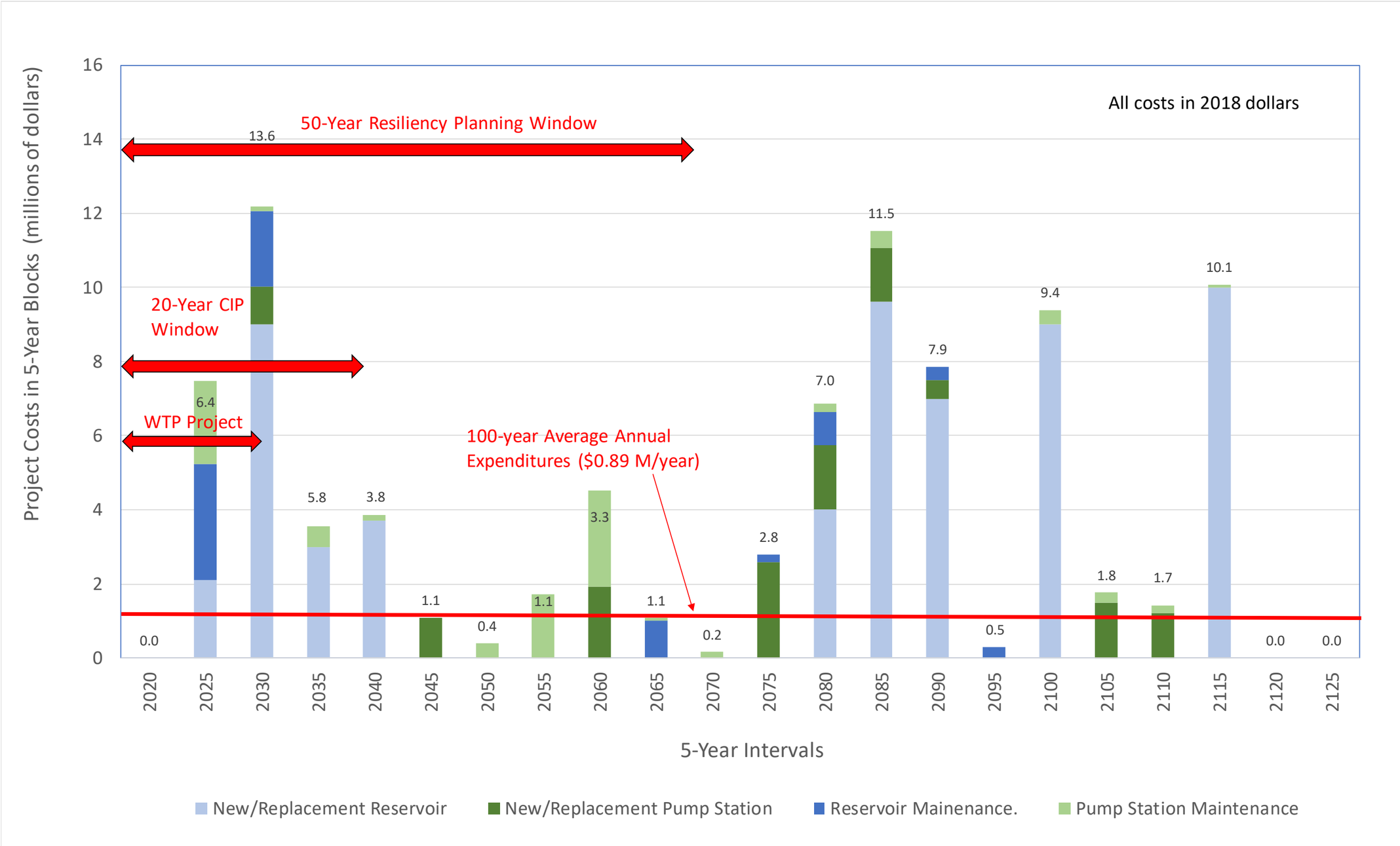


Figure 4: Scheduled Project Costs (5-Year Block Intervals) – 100-Year Window



APPENDIX F
REPORT, "STRUCTURAL ASSESSMENT OF RESERVOIRS & PUMP STATIONS," PETERSON STRUCTURAL ENGINEERS, DECEMBER 2018

murraysmith

CITY OF GRANTS PASS WATER SYSTEMS EVAULATIONS

FINAL REPORT

Structural Assessment of Reservoirs & Pump Stations

January 17, 2019

murraysmith

PSE
PETERSON STRUCTURAL ENGINEERS

City of Grants Pass – Water Systems Evaluation
Structural Assessment

Reservoir & Pump Stations

January 17, 2019

Portland Office

9400 SW Barnes Road, Suite 100

Portland, OR 97225

Table of Contents

1 Introduction

1.1 Executive Summary

Peterson Structural Engineers (PSE) performed various levels of structural assessment of five Reservoirs and thirteen pump stations located in the City of Grants Pass, Oregon. The assessments consisted of a visual structural condition assessment of each structure as well as additional structural analysis and evaluation of selected structures. Based upon the initial visual structural assessment additional follow-up destructive testing was performed on three Reservoirs.

For the visual structural condition assessment, PSE considered:

I. Review of provided original construction documents
II. Site visits to review the existing as-built condition of each structure
    a. For the Reservoirs, only the exterior of the Reservoir was observed as they were undrained
    b. For the pump stations, interior and exterior areas were observed, except as noted
III. Auxiliary/ancillary structures and building envelope concerns (as they relate to the structure)
IV. Visual evidence of any deferred maintenance
V. Additional destructive testing was performed at three Reservoirs where noted

For the structural analysis, PSE considered

I. Review of the original construction documents
II. The gravity and lateral structural systems including both wind and seismic loads
III. Current code requirements for each structure

For the structural evaluation, PSE performed a current code analysis of each structure using the current Oregon Structural Specialty Code (OSSC) and the referenced standards, ASCE-7, 'Minimum Design Loads and Associated Criteria for Buildings and Other Structures' as well as American Water Works Association (AWWA) D110, 'Wire and Strand-Wound, Circular, Prestressed Concrete Water Tanks' and other associated referenced codes. While on site, PSE also looked for discrepancies between the as-built structures and the construction documents, when available.

1.2 Project Summary

PSE performed a visual structural assessment of five Reservoirs and thirteen pump stations located in the City of Grants Pass. The structures are part of the City's current water system and are in active use.

PSE performed the inspections over the course of several days on January 30-31, 2018 and February 13-14, 2018. The weather was generally cold, cloudy, with some light rain on the last day of inspections. At the time of our on-site inspections Mike McKillip, PE, PhD from Murraysmith was present as well as various City Staff depending on the day.

1.2.1 Purpose of Evaluations

The purpose of the evaluations was to identify any structural deficiencies or deterioration of the structures for assisting in future planning. In addition, the field inspections were used to verify the information contained in the original permit documents.

1.2.2 Structures Assessed

PSE visually assessed and performed structural evaluations of the of the following structures:

Reservoirs:	**Name**	**Size**	**Date Constructed**
	• Reservoir No. 5	3.5 MG	1983
	• Reservoir No. 6	3.5 MG	1982
	• Reservoir No. 8	2.0 MG	1983
	• Reservoir No. 11	4.5 MG	1999
	• Reservoir No. 15	1.3 MG	1985

Pump Stations:	**Name**	**Date Constructed**
	• Lawnridge Pump Station	1969
	• Madrone Pump Station	1954
	• New Hope Pump Station	2000

In addition, PSE performed limited visual structural assessments of the following pump stations:

Pump Stations – Limited Assessment

Name	**Date**
• Williams Crossing	2005
• Meadow Wood	2002
• Panoramic Loop	2006
• Harbek	1999
• Hilltop	2009
• Heffley	1996
• Champion	1982
• North Valley	1983
• Starlite	1982
• Laurel Ridge	2014

The limited visual structural assessments of the ten pump stations were limited in scope to a quick visual assessment of each structure. No additional structural analysis or evaluation of these structures was performed. The estimated dates for all of the structures are based upon the date shown on the original construction drawings and/or information provided by the City so should be considered circa dates.

1.3 Approach and Assumptions

PSE assessed the following aspects of the structures:

1) Vertical loads (self-weight, hydrostatic, occupants where applicable, snow etc.)
2) Horizontal loads (wind, seismic, hydrostatic, hydrodynamic, sloshing)
3) Ancillary Structural elements and Building Envelope (hatches, vents, ladders, penetrations, etc.)
4) Other Serviceability Concerns (deferred maintenance and miscellaneous observations)

1.3.1 Current Code Seismic Assumptions

For the seismic assessment, PSE performed a current code evaluation according to the procedures outlined in ASCE 7-10 and/or AWWA D110-13

To perform the analysis, some basic assumptions about seismic loads needed to be established. Since no site-specific geotechnical investigation was performed, assumed seismic values have been used in accordance with current code guidelines and seismic values from the USGS database. For this assessment the following assumptions have been made for each structure:

- Soil Site Class – D
- Seismic Design Values – Values from USGS database for each site as shown in the table below

Seismic Design Values (g's) Source: USGS				
Reservoir Site	Ss	S1	Sds	Sd1
No. 5	0.776	0.415	0.615	0.439
No. 6	0.794	0.423	0.626	0.445
No. 8	0.797	0.424	0.628	0.445
No. 11	0.805	0.428	0.632	0.449
No. 15	0.802	0.427	0.63	0.448

PSE assumed that the structures were built in accordance with the original construction documents and have used the values for concrete strengths and steel strengths provided therein for our analysis. Where additional assumptions have been made they are noted in the analysis and evaluation sections.

1.4 Detailed Structure Report Contents

Each subsequent section of this report is meant to be a standalone document which covers the specific structure investigated. Information provided in this opening section provides background, approach and assumption information for the investigation but is supplementary to the individual reports as well as an executive summary of the results.

The sections cover:

- Description & Background
- Visual Condition Assessment
- Structural Analysis
- Summary
- Recommendations
- Copies of PSE field notes
- Scans of Select Construction Documents
- Observation Pictures

1.5 Summary of Findings

The following bulleted list is a brief summary of the noncompliance issues for each of the Reservoirs and pump stations that underwent full assessment. More detailed information can be found in each structure's respective section.

Since the scope of the current work was to perform an evaluation of the existing reservoirs an in-depth analysis of the elements that were shown to be non-code compliant has not been performed at this time. For instance, the evaluation of the required free board on several of the reservoirs at the operating levels and/or the original design operating levels is insufficient for current code requirements. Based upon this the slosh wave is expected to impact the roof. Additional analysis of the roof for the slosh impact is recommended for the reservoirs where the proposed operating levels will induce potential impacts from seismic sloshing. Where this occurs, additional analysis has been recommended as noted.

- Reservoir No. 5: At its current operating level (28'), Reservoir No. 5 has inadequate freeboard height and seismic restraint at the base. There were also some areas of delamination of the outer layer of shotcrete. Additionally, the roof hatch has a corroded hinge and the hatch design does not comply with current water quality standards. In addition, the Reservoir has inadequate circumferential prestressing when evaluated at the maximum operating level. The Reservoir roof does not meet the current code requirements for minimum reinforcing requirements for flexural load and crack control and long term creep/ponding may be a concern.

- Reservoir No. 6: At its current operating level (26'), Reservoir No. 6 has inadequate freeboard height and seismic restraint at the base. There were also some areas of delamination of the outer layer of shotcrete. Additionally, the roof hatch has corroded hinges and the hatch design does not comply with current water quality standards. In addition, the Reservoir has inadequate

circumferential prestressing when evaluated at the maximum operating level. The Reservoir roof does not meet the current code requirements for minimum reinforcing requirements for flexural load and crack control and long-term creep/ponding may be a concern.

- Reservoir No. 8: At its current operating level (24'), Reservoir No. 8 has inadequate seismic restraint at the base. Additionally, the roof hatch has corroded hinges and the hatch design does not comply with current water quality standards. In addition, the Reservoir has inadequate freeboard and circumferential prestressing when evaluated at the maximum operating level. The Reservoir roof does not meet the current code requirements for minimum reinforcing requirements for flexural load and crack control and long-term creep/ponding may be a concern.

- Reservoir No. 11: At its current operating level (28'), Reservoir No. 11 complies with current structural code. The infill for the shear cans has deteriorated, which has left the top of the cans exposed and corroding. There were also some areas of delamination of the outer layer of shotcrete. Additionally, the roof hatch design does not comply with current water quality standards.

- Reservoir No. 15: At its current operating level (7'), Reservoir No. 15 complies with current structural code. However, the roof hatch has corroded hinges and the hatch design does not comply with current water quality standards. When operating at maximum operating level, the Reservoir has inadequate freeboard, seismic restraint at the base, and circumferential prestressing.

- Lawnridge Pump Station: This pump station is nominally code-compliant, with the exception of appropriate connections at the top of the walls to transfer in-plane and out-of-plane lateral forces from the roof diaphragm to the walls below. Additionally, there is a neighboring tree whose growth could eventually impact the roof.

- Madrone Pump Station: This pump station would require significant upgrades in order to be brought up to code. Additionally, the building appears to have undergone significant differential settlement.

- New Hope Pump Station: This pump station is nominally code-compliant, with the exception of required connections at the top of the walls to transfer in-plane and out-of-plane lateral forces from the roof diaphragm to the walls below.

1.6 Summary of Recommendations

The following bulleted list is a brief summary of PSE's recommendations for each of the Reservoirs and pump stations that underwent full assessment. These recommendations only apply to the current operating conditions of the structures indicated. More detailed information can be found in each structure's respective section if higher operating levels are desired.

- Reservoir No. 5: Upgrade connection at the base of the wall and perform analysis of the roof to determine if upgrades are required to resist slosh wave forces. Repair/replace roof hatch.

- Reservoir No. 6: Upgrade connection at the base of the wall and perform analysis of the roof to determine if upgrades are required to resist slosh wave forces. Repair/replace roof hatch.

- Reservoir No. 8: Upgrade connection at the base of the wall. Repair/replace roof hatch.

- Reservoir No. 11: Repair/replace shear can infill. Repair/replace roof hatch.

- Reservoir No. 15: Repair/replace roof hatch.

- Lawnridge Pump Station: Install connections at the top of the walls to transfer in-plane and out-of-plane lateral forces. Cut back and monitor growth of adjacent tree.

- Madrone Pump Station: Determine the cause of the settlement issues and either perform a full retrofit upgrade or replace the existing building with a new pump station

- New Hope Pump Station: Install connections at the top of the walls to transfer in-plane and out-of-plane lateral forces.

1.7 Areas of Further Study

Since the Reservoirs were in operation during our visual observations, we recommend that during the next down time for each Reservoir an interior inspection of each Reservoir be performed to observe the existing conditions of each structure. Ideally the interior of the Reservoirs would be cleaned prior to inspection.

Based upon the results of the evaluations contained herein, the operating levels and long-term planning for the Reservoirs should be evaluated to determine if potential upgrades are desired. For the Reservoirs that have delaminated shotcrete, repairs to the shotcrete would ideally be made if/when potential upgrades are being performed.

1.8 Limitations

This evaluation is limited to information obtained during a visual structural assessment over the course of several days and limited destructive testing. While PSE made every attempt to be as thorough as possible in the assessment, not every member, connection or component can be visually assessed or evaluated. PSE has attempted to capture as many items as possible, however, it is likely that there are areas that are not addressed in this assessment.

1.9 Disclaimer

The information presented within this report represents the opinion of a Structural Engineer registered in the State of Oregon. Following any major earthquakes, other damage, modifications or upgrades, change of use/operation, and/or other substantial changes the results herein should be reviewed, and the structures reassessed following any of the above aforementioned events.

1.10 Endorsement

This report was prepared by Travis McFeron, PE, SE (OR #63186) or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Oregon.

2 Reservoir No. 5 – 3.5 MG

2.1 Description & Background

The original Reservoir was designed by CH2M Hill and the original construction drawings provided are dated May 1982. It is reported by the City that the original construction was circa 1983. The Reservoir is a ground-supported partially buried, 144' inside diameter x 30' high, strand-wrapped, pre-stressed concrete water Reservoir with interior columns. The roof is 7.5" thick and is supported by (32) columns. The drawings show an option for square or round columns, and we were able to verify that the columns are square during our site inspection by observation through the hatch. The original roof was poured in nine sections and appears to match the layout in the original drawings.

Per the original drawings the Reservoir does contain vertical pre-stressing bars in the core wall. The roof slab bears on the tank shell wall with shear cans at every other vertical tendon. There are 168 total vertical tendons each 1.25" in diameter. The wall base connection utilizes 144 double seismic cable sets each with (2) 0.375" diameter, 7-wire galvanized strands. Both the roof and the floor connection are detailed with an elastomeric bearing pad.

2.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the Reservoirs on the exterior. The Reservoir was not able to be drained during our inspection, so we were unable to observe the interior except as noted through the hatch. The site visit was performed January 30th, 2018. A second site visit was performed as noted below to perform destructive testing to observe the existing strand wrapping by chipping away the shotcrete layer at three locations. Following these observations, the chipped areas were patched with non-shrink high strength concrete.

The existing condition of the Reservoir roof showed small areas of ponding that correspond with the larger roof edge spans. The roof has a moderate amount of small cracking throughout the roof that appear to correspond with the negative moment column strips as well as larger cracking over columns. The edge of the roof appears to be in good condition. The roof overhang is 6" and there is a drip edge located 2" from the exterior edge.

The roof hatch is 4'x6' and appears to be in fair condition, however it appears to be an older style hatch which is no longer considered code-compliant for water quality. One of the hatch hinge links is very corroded and may have not been galvanized. The roof vent appears to be in good condition with no apparent structural deficiencies noted.

The interior of Reservoir was not able to be observed as the Reservoir was undrained. No visual issues were noted from observation through the hatch. The columns were noted to be square columns. The interior ladder appeared to be in good condition.

The exterior walls of Reservoir are covered in shotcrete to protect the strand wrapping around the core wall. The Reservoir is partially buried, and the exposed height of the wall varies from just over 8' at the Northern elevation to upwards of 14' at the Southwestern elevation. The grade varies gradually around

the perimeter of the Reservoir. A row of trees is present around approximately 120 degrees of the circumference on the North to Southwestern side that are located approximately 10-15' away from the Reservoir wall. The grade slopes generally from North towards the South and an irrigation drainage ditch/swale is located towards the South side of the Reservoir a distance away. There were no visual indications of any slope movement or impact of the adjacent ditch/swale. It is noted that the trees may eventually be a potential hazard to the roof edge and/or the root systems may eventually reach the tank wall and could cause damage to the areas below grade. In addition, the roof has deleterious organic debris from the adjacent trees in the form of pine needles, see Figure 2-11. This isn't a structural concern, but could increase the potential for stains down the side of the Reservoir.

We performed sounding of the walls around the perimeter of the Reservoir using a standard framing hammer to listen for hollow sounds indicative of delaminated shotcrete. These were performed every few feet around the full circumference from ground level (i.e. walking around). An area of clearly delaminated shotcrete was observed that appears to extend the full perimeter of the Reservoir from approximately the top of the wall to a location 3ft down. A line of efflorescence is observable at the line demarcating this change. Below this level there are very few areas of 'hollow' sounds.

2.2.1 Destructive Testing

Based upon the aforementioned area of potential delaminated shotcrete it was recommended that destructive testing be performed to observe the existing wrapping to look for potential corrosion or other issues related to delaminated shotcrete. A separate site visit was performed on February 14th, 2018 to perform the additional investigation. At this time three areas were selected, and the shotcrete was chipped away to expose the existing strand wrapping.

The observation of the strands showed no areas of corrosion where the strands were exposed. At these locations the strand appears to be in good condition. The locations chosen were based upon hammer soundings to locate areas with potential delamination/hollow spots that would be more likely to experience corrosion damage. The shotcrete in each of these locations was determined to be delaminated during the chipping process and in poor condition and was removed easier than competent shotcrete would be until the delaminated portion was removed. The area that appears to be delaminated appears to be limited to the shotcrete layer covering the strands. The layer of delaminated shotcrete appeared to be approximately two layers each 0.5" in thickness. Beyond that a layer of approximately 1" of competent shotcrete was observed before the strands were observed. The shotcrete behind the strands appears to be in good condition from the limited exploration. The depth of shotcrete was approximately 2" to the strands at all three locations.

2.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13 was utilized. Evaluation was based on original construction documents and site visit observations.

2.3.1 Gravity Analysis

Roof Slab: The roof slab thickness of 7.5" is above the code recommended 7.17" minimum for the spans provided. Reinforcing in the roof slab is insufficient in most regions for flexural load and crack control requirements, when evaluated to current code requirements. It may be noted that current code requirements have increased safety factors than those mandated during the period of the original construction. Eventually the concrete creep will likely make the observed ponding increase but does not appear to be a significant concern at this time. Overall the roof appears to be in good condition presently and appears to have performed well under static loads.

Vertical Wall Reinforcement: Reservoir No. 5 does have vertical pre-stressing within the walls. The existing vertical pre-stressing appears to meet the current code requirements.

Columns: The columns in Reservoir No. 5 are square. Note that the original drawings provide options for both square and circular columns. The columns meet code requirements for static load resistance, see Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings appear to meet current code strength requirements. No allowable bearing pressure was provided for the site, but the maximum design soil bearing demand determined under the wall footings is about 4,200 psf, assuming a maximum operating water height of 28'.

2.3.2 Lateral Analysis

Seismic Joints: There are 144 sets of seismic cables (two cables per set), which provides just under half the capacity required to meet code requirements. Additional restraint at the wall base is required. More seismic cables or a new curb would be required at the base of the tank, as the existing cabling is inadequate to resist the base shear loads at the wall base.

Based on the dimensions and material properties given in the original construction drawings, the bearing pad at the base of the wall appears to be lightly overstressed. Upgrading the pad does not appear feasible as this would require increasing the pad width which is not accessible. This does not appear to be a significant concern as the amount of overstress is relatively small, less than 5%.

Strand Wrap: The circumferential prestressing requirements for the Reservoir are detailed on the prestressing load distribution diagram on sheet 7 of the original drawings dated May, 1982. The wrapping schedule was not shown on the original drawings, however, calculating the area of the diagram will provide the total prestressing force that was required during the original design and construction. Dividing the total prestressing force by the final force of each machine applied wrap will provide the total number of wraps required. Based on the shop drawings provided by DN Tanks (known as DYK Prestressed Concrete Tanks when the Reservoir was constructed), dated October 8, 1982, the methodology described above was followed. It is important to note, that when the Reservoir was designed and constructed AWWA D110 had not been published and it is likely that designer followed ACI 344 – *Design and Construction of Circular Wire and Strand Wrapped Concrete Structures*, which was originally published in 1970.

Based on the prestressing load diagram shown CH2M Hill's drawings, DYK provided 185 wraps on their submitted shop drawing for the project. Following the current edition of AWWA D110-13 requirements, we have considered the hoop loads generated for two load cases as follows:

1. Static Load at Overflow water elevation plus 200 psi residual compression for differential temperature and dryness effects on the wall.
2. Hydrodynamic seismic loads, at the service water level, based on the USGS values for the site.

Applying this approach to the Reservoir at the current operating elevation of 28', results in the following required number of wraps:

1. 183 Wraps Load Case 1: Static Loads
2. 160 Wraps Load Case 2: Hydrodynamic Seismic Loads

Please note, since the Reservoir has some backfill around the circumference, we have followed the provisions in AWWA D110, Section 3.5.2.1, and tapered the 200 psi linearly to 50 psi over a below grade depth of 6 ft. Since the Reservoir requires 183 wraps, and it has 185 wraps, it meets the current code requirements under current operating levels.

If the Reservoir is evaluated for the maximum operating level at the overflow of 29' elevation the number of wraps per current code is 194 wraps for load case 1 and 172 wraps for load case 2, which exceeds the number of wraps provided. Columns: The roof is supported by bearing pads on top of the wall and restrained by shear cans to limit the potential deflection to 0.125". This allowable movement allows the roof to move and lateral resistance within this 0.125" range for the roof is created by the moment transfer into the existing columns. The existing columns are able to resist the seismic loads induced and are not overstressed under current design seismic loads. As noted above, they meet code requirements for static loads, and can easily accommodate roughly 0.125" of lateral translation for seismic loads, which is also the deflection limit of the shear cans, so the columns have limited reserve capacity.

Freeboard/Slosh: The current freeboard height (the distance between the top of the wall and operating level) provided does not meet the height required by ASCE 7-10, even with the operating level reduced to the 28'. The maximum slosh wave height was calculated as 55" at the full operating height, which exceeds the actual freeboard height by 31". A quick check of the roof shows that it appears to have insufficient dead load to resist the slosh impact loads. We recommend additional analysis of the slosh wave impact on the roof may be performed if the current water level is to be maintained or if the water level needs to be increased.

2.4 Summary

The Reservoir roof slab appears to be under-designed per current code requirements, but the roof appears to still be in decent condition. The ponding noticed at the end spans will likely increase with age. The Reservoir has adequate wrapping for seismic loads but are inadequate for static loads for the operating water level plus shrinkage and temperature conditions when evaluated by current code which requires the wrapping to provide a minimum level of circumferential compression in the walls. However,

that is based upon a minimum wrapping diagram, the actual wrapping may exceed that. The Reservoir needs upgrades at the wall to foundation to provide adequate load transfer for seismic loads and the roof slosh should be further evaluated. Also, additional maintenance to the hatch hinges should be considered.

Based upon the evaluation and aforementioned conditions, Reservoir No. 5 could reasonably be brought up to current code requirements.

2.5 Recommendations

There are several options available to bring Reservoir 5 into reasonable compliance with current code. The first option is to perform no upgrades and lower the operating limit so that the roof slosh height and seismic cable capacity meet the required demand. However, PSE understands that lowering the operating level of this Reservoir would require lowering the operating levels of the other Reservoirs in Zone 1, which could cause supply issues.

In order to maintain (or increase) the current operating level and nominally comply with current structural code, several upgrades would be required to the wall and roof. The inadequate capacity of the seismic cables could be addressed by either adding new cables at the base with new wrapping and shotcrete, or by placing a new concrete curb around the perimeter of the wall base at the foundation to prevent sliding that would engage the existing cables in the case of a seismic event. A finite element analysis would be required to determine whether or not the roof is overstressed by a slosh wave induced by the desired operating level. If the roof is shown to be overstressed it could potentially be upgraded or potential damage to the roof may be accepted as part of a risk management program.

Possible upgrades could include reinforcing the appropriate areas with a fiber-reinforced polymer (FRP) wrap or other alternate methods. Another option would be to accept potential damage to the roof in a significant enough seismic event and provide protection for the inlet/outlet pipe from being blocked should portions of the roof collapse into the tank

From a water quality standpoint, the roof hatch does not meet current code requirements. Given this and the hatch's corroded hinges. We recommend that the City replace the hatch with a model that meets current Oregon DEQ water quality standards.

2.6 Field Notes

Reservoir Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	January 30, 2018	**Observation Time**	11:45am (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Clear 50 degrees, rained .75" +/- 2-days earlier		

General Information

Structure	Reservoir No. 5
Address	Sherman Lane
Date Constructed (if known)	1983, per City
Date Retrofitted (if any)	n/a
Original Design Code	UBC 1979
Reservoir Type	D110, Type I - Concrete
Size	3.5 MG
Inside Diameter	144’
Wall Height	30’
Adjacent Structures (?)	None, see notes regarding trees
General Notes – An arc of trees is present from North to Southwest approximately 10-15’ from the Reservoir wall, which may eventually be an issue. Operating levels: 28’ in Winter, 27’ Summer, 29’ overflow. Water approx. 8’ below top of wall during observation.	

Exterior Inspection Information

Backfill (Height to top of wall)	N: 8’-3”, E: 11’-9”, SE: 13’-3”, SW: 14’-0”
Site Slope	North to South, drainage ditch/swale to the South
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), no obvious signs of any movement
Foundation Condition	NA - Buried
Roof Condition	(9) roof pours, matches original drawings 7.5” thickness 6” overhang, w/ drip edge 2” from edge Moderate number of small cracks, consistent w/ negative moment at column strip and over columns Ponding areas at larger edge spans
Roof Hatch(s)	SSE Location 4’x6’, East Edge, good condition Old style – non-compliant for water quality Hinge link rusted (not galvanized)
Roof Vent(s)	Northwest and Southeast Edges, no structural concerns appear in good condition
Wall	General minor cracking in shotcrete Delaminated band top 3’ of Reservoir Efflorescence at edge of band (3ft below top of wall)

Interior Inspection Information

NA – Reservoir not drained

Destructive Testing Information

General Date: February 13th Time: 8.00 am, Approximately Number of Locations: 3
Location No. 1 Site: Southeast, approximately 32” down from top of wall Strand: (1) exposed in 6” vertical expanse, no corrosion present Shotcrete: (2) 0.5” layers of delaminated, then 1” of competent before strand, appears competent past
Location No. 2 Site: Northeast, approximately 32” down from top of wall Strand: (1) exposed in 7” vertical expanse, no corrosion present Shotcrete: (2) 0.5” layers of delaminated, then 1” of competent before strand, appears competent past
Location No. 3 Site: Northwest, approximately 36” down from top of wall Strand: (1) exposed in 7” vertical expanse, no corrosion present Shotcrete: (2) 0.5” layers of delaminated, then 1” of competent before strand, appears competent past

2.7 Scans of Select Construction Documents

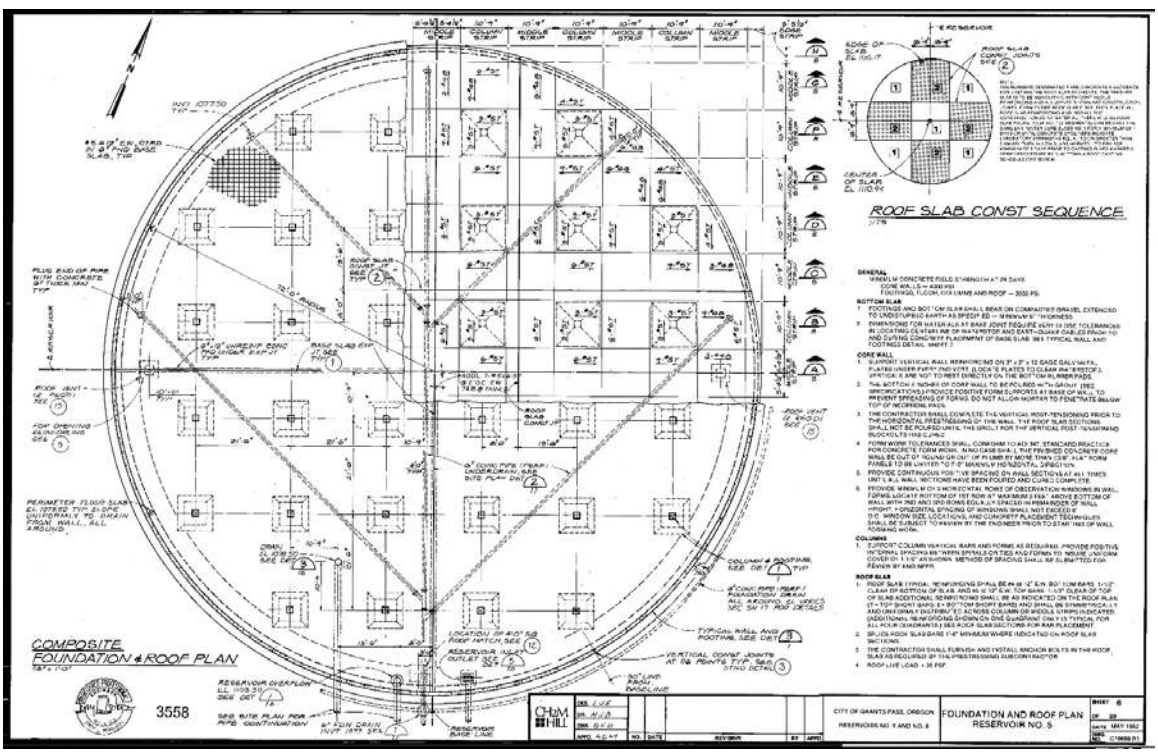


Figure 2-1: Composite Foundation & Roof Plan

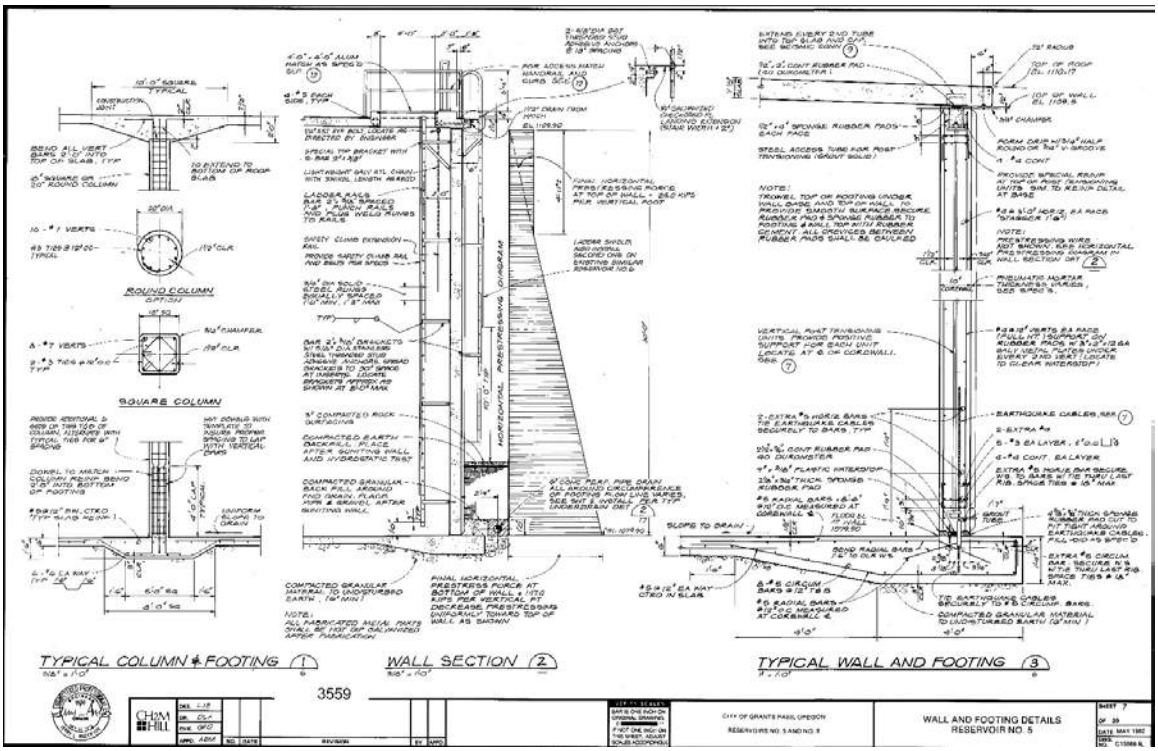


Figure 2-2: Wall & Footing Details

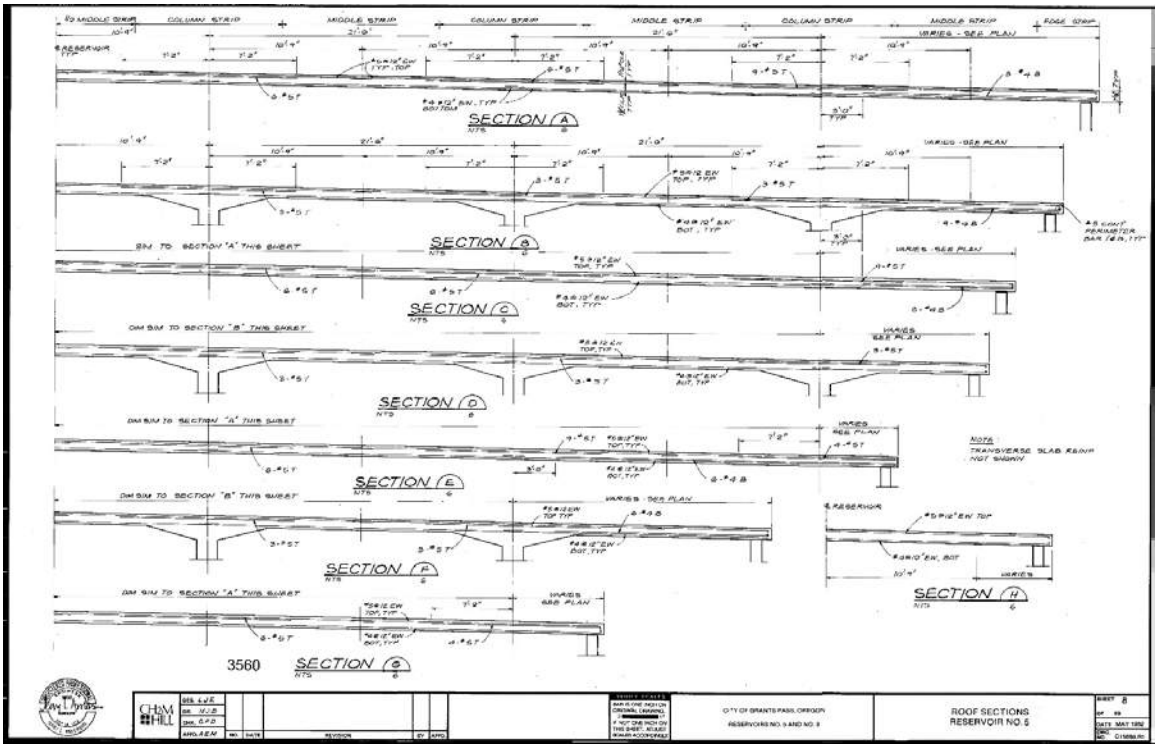


Figure 2-3: Roof Sections

2.8 Observations Pictures

Figure 2-4: Reservoir No. 5 - Elevation

Figure 2-5: Reservoir No. 5 – Adjacent Tree Line

Figure 2-6: Reservoir No. 5 – Exterior Ladder

Figure 2-7: Reservoir No. 5 – Roof

Figure 2-8: Reservoir No. 5 – Roof Hatch

Figure 2-9: Reservoir No. 5 – Roof Joint

Figure 2-10: Reservoir No. 5 – Roof Hatch

Figure 2-11: Reservoir No. 5 – Roof – Organic Debris

Figure 2-12: Reservoir No. 5 – Roof Ponding

Figure 2-13: Reservoir No. 5 – Destructive Testing – Strand Wrapping

Figure 2-14: Reservoir No. 5 – Destructive Testing – Strand Wrapping

Figure 2-15: Reservoir No. 5 – Destructive Testing – Strand Wrapping

END OF SECTION

3 Reservoir No. 6 – 3.5 MG

3.1 Description & Background

The original Reservoir was designed by CH2M Hill and the original construction drawings provided are dated April 1980. It is reported by the City that the original construction was circa 1982. The Reservoir is a ground-supported, partially buried, 144' inside diameter x 30' high, strand-wrapped, pre-stressed concrete water Reservoir with interior columns. The roof is 7.0" thick and is supported by (32) columns. The drawings show an option for square and round columns; we were able to verify that the columns are square during our site inspection by observation through the hatch. The original roof was poured in nine sections and appears to match the layout in the original drawings.

Per the original drawings the Reservoir does contain vertical pre-stressing bars in the core wall. The roof slab bears on the tank shell wall with shear cans at every other vertical tendon. There are 168 total vertical tendons each 1.25" in diameter. The wall base connection utilizes 144 seismic cable sets each with (2) 0.375" diameter, 7-wire galvanized strands. Both the roof and the floor connection are detailed with an elastomeric bearing pad.

The original drawings specify a roof live load of 35 psf for design. This exceeds the current code requirements of 25 psf for snow load design for current code. For the evaluation contained herein the original specified roof live load of 35 psf was conservatively used. Note, this increase of live/snow load over current code requirements does not change the results contained herein; items identified as non-code compliant do not change if the live/snow load is reduced to 25 psf.

3.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the Reservoir on the exterior. The Reservoir was not able to be drained during our inspection, so we were unable to observe the interior except as noted through the hatch. The site visit was performed January 30th, 2018. A second site visit was performed as noted below to perform destructive testing to observe the existing strand wrapping by chipping away the shotcrete layer at three locations. Following these observations, the chipped areas were patched with non-shrink high strength concrete.

The existing condition of the Reservoir roof showed several areas of ponding that correspond with the exterior larger roof edge spans as well as some interior spans. The roof has a lot of hairline cracks throughout the roof area as well as a moderate amount of small cracking throughout the roof that appear to correspond with the negative moment column strips as well as larger cracking over columns. The edge of the roof appears to be in good condition. The roof overhang is 6" and there is a drip edge located 2" from the exterior edge. There are a couple of areas in the roof that appear to have small cracks and spalled sections at the edges or corners of the original construction joints. These appear to be fairly old and may date back to shortly after the original construction.

There appears to be some areas of the shotcrete spalling from the wall at the roof joint, presumably from thermal expansion where the wall meets the roof. There are several areas of small spalled off corners as well as several pieces of loose concrete around the perimeter on the ground.

The roof hatch is 4'x4' and appears to be in fair condition, however it appears to be an older style hatch which is no longer considered code compliant for water quality. Both hatch hinge links are very corroded and do not appear to be galvanized. The roof vent appears to be in good condition with no apparent structural deficiencies noted.

The interior of the Reservoir was not able to be observed as the Reservoir was undrained. No visual issues were noted from observation through the hatch. The columns were noted to be square columns. The interior ladder appeared to be in good condition.

The exterior walls of Reservoir are covered in shotcrete to protect the strand wrapping around the core wall. The Reservoir is partially buried, and the exposed height of the wall varies from approximately 20-25'. The grade varies gradually around the perimeter of the Reservoir and slopes roughly North to South. The Reservoir site appears to have been cut into a shallow hillside and the surrounding grade slopes upwards on the Northern half of the site. An arc of trees is present around approximately 200 degrees of the circumference on the Southeast counterclockwise to the East that vary in distance from the Reservoir. There were no visual indications of any slope movement or settlement. It is noted that the trees may eventually be a potential hazard to the roof edge and/or the root systems may eventually reach the tank wall that could cause damage to the areas below grade.

We performed sounding of the walls around the perimeter of the Reservoir using a standard framing hammer to listen for hollow sounds indicative of delaminated shotcrete. These were performed every few feet around the full circumference from ground level (i.e. walking around). There was an arc length from approximately the Southwest to the East face counter clockwise that had a considerable amount of areas that appeared may be delaminated. The remaining circumference showed only a few minor areas of hollow sounding areas.

3.2.1 Destructive Testing

Based upon the aforementioned area of potential delaminated shotcrete it was recommended that destructive testing be performed to observe the existing strand wrapping to look for potential corrosion or other issues related to delaminated shotcrete. A separate site visit was performed on February 13th, 2018 to perform the additional investigation. At this time three areas were selected, and the shotcrete was chipped away to expose the existing wrapping.

The observation of the strands showed no areas of corrosion where the strands were exposed. At these locations the strands appear to be in good condition. The locations chosen were based upon hammer soundings to locate areas with potential delamination/hollow spots that would be more likely to experience corrosion damage. The shotcrete in each of these locations was determined to be delaminated during the chipping process and in poor condition and was removed easier than competent shotcrete would be until the delaminated portion was removed. The area that appears to be delaminated appears to be limited to the shotcrete layer covering the strands. The shotcrete behind the strands appears to be in good condition from the limited exploration. The depth of shotcrete was approximately 1.5" to the strands at all three locations.

3.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13 was utilized. Evaluation was based on original construction documents and site visit observations.

3.3.1 Gravity Analysis

Roof Slab: The roof slab thickness of 7" is above the code recommended 6.79" minimum for the spans provided. Reinforcing in the roof slab is insufficient in most regions for flexural load and crack control requirements, when evaluated to current code requirements. It may be noted that current code requirements have increased safety factors than those mandated during the period of the original construction. Eventually the concrete creep will likely make the observed ponding increase but does not appear to be a significant concern at this time. Overall the roof appears to be in good condition presently and appears to have performed well under static loads.

Vertical Wall Reinforcement: Reservoir No. 6 does have vertical pre-stressing within the walls. The existing vertical pre-stressing appears to meet the current code requirements.

Columns: The columns in Reservoir No. 6 are square. Note that the original drawings provide options for both square and circular columns. They meet code requirements for static load resistance, see Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings appear to meet current code strength requirements. No allowable bearing pressure was provided for the site, but the maximum design soil bearing demand determined under the wall footings is about 3,700 psf, assuming an operating limit of 26'.

3.3.2 Lateral Analysis

Seismic Joints: There are 144 sets of seismic cables (with two cables per set) which provides roughly half the capacity required to meet code requirements. Additional restraint at the wall base is required. More seismic cables or a new curb would be required at the base of the tank, as the existing cabling is inadequate to resist the base shear loads at the wall base.

Strand Wrap: The circumferential prestressing requirements for the Reservoir are detailed on the prestressing load distribution diagram on sheet 7 of the original drawings dated April, 1980. The wrapping schedule was not shown on the original drawings, however, calculating the area of the diagram will provide the total prestressing force that was required during the original design and construction. Dividing the total prestressing force by the final force of each machine applied wrap will provide the total number of wraps required. Based on the shop drawings provided by DN Tanks (known as DYK Prestressed Concrete Tanks when the Reservoir was constructed), dated August 12, 1980, the methodology described above was followed. It is important to note, that when the Reservoir was

designed and constructed AWWA D110 had not been published and it is likely that designer followed ACI 344 – *Design and Construction of Circular Wire and Strand Wrapped Concrete Structures*, which was originally published in 1970.

Based on the prestressing load diagram shown CH2M Hill's drawings, DYK provided 185 wraps on their submitted shop drawing for the project. Following the current edition of AWWA D110-13 requirements, we have considered the hoop loads generated for two load cases as follows:

3. Static Load at Overflow water elevation plus 200 psi residual compression for differential temperature and dryness effects on the wall.
4. Hydrodynamic seismic loads, at the service water level, based on the USGS values for the site.

Applying this approach to the Reservoir at the current operating elevation of 26', results in the following required number of wraps:

3. 180 Wraps Load Case 1: Static Loads
4. 138 Wraps Load Case 2: Hydrodynamic Seismic Loads

Please note, since the Reservoir has some backfill around the circumference, we have followed the provisions in AWWA D110, Section 3.5.2.1, and tapered the 200 psi linearly to 50 psi over a below grade depth of 6 ft. Since the Reservoir requires 180 wraps, and it has 185 wraps, it meets the current code requirements under current operating levels.

If the Reservoir is evaluated for the maximum operating level at the overflow of 29' elevation the number of wraps per current code is 212 wraps for load case 1 and 172 wraps for load case 2, which exceeds the number of wraps provided.

Columns: The roof is supported by bearing pads on top of the wall and restrained by shear cans to limit the potential deflection to 0.125". This allowable movement allows the roof to move and lateral resistance within this 0.125" range for the roof is created by the moment transfer into the existing columns. The existing columns are able to resist the seismic loads induced by a 0.125" deflection. As noted above, the columns meet code requirements for both static and seismic loads.

Freeboard/Slosh: The current freeboard provided is less than the minimum amount required by ASCE 7-10, even with the operating level of 26'; the required freeboard is 53.3" at the current operating level. Therefore, uplift forces from slosh waves must be reviewed and accounted for. However, given the minimum amount of encroachment on the roof the lack of freeboard does not appear to be a concern at the current operating level. A quick check of the roof shows that it appears to have sufficient dead load to resist the slosh impact loads. The maximum slosh wave height was calculated as 55.5" at the full operating height, which significantly exceeds the available freeboard. A more thorough analysis of the slosh wave impact on the roof would be recommended for higher operating levels.

3.4 Summary

The Reservoir roof slab appears to be under-designed per current code requirements, but the roof appears to still be in decent condition. The ponding noticed at the end spans will likely increase with age. At the current operating level, the Reservoir has adequate wrapping for seismic loads and for static loads plus shrinkage and temperature conditions when evaluated by current code, which requires the wrapping to provide a minimum level of circumferential compression in the walls. The Reservoir needs upgrades at the wall-to-roof and wall-to-foundation to provide adequate load transfer for seismic loads. Additional maintenance to the hatch hinges should also be considered.

Based upon the evaluation and aforementioned conditions, Reservoir No. 6 can be brought up to current code requirements.

3.5 Recommendations

In order to maintain (or increase) the current operating level and nominally comply with current structural code, several upgrades would be required to the wall. The inadequate capacity of the seismic cables could be addressed by either adding new cables at the base with new wrapping and shotcrete, or by placing a new concrete curb around the perimeter of the wall base at the foundation to prevent sliding that would engage the existing cables in the case of a seismic event.

Possible upgrades could include reinforcing the appropriate areas with a fiber-reinforced polymer (FRP) wrap or other alternate methods. Another option would be to accept potential damage to the roof in a significant enough seismic event and provide protection for the inlet/outlet pipe from being blocked should portions of the roof collapse into the tank

From a water quality standpoint, the roof hatch does not meet current code requirements. Given this and the hatch's corroded hinges, we recommend that the City replace the hatch with a model that meets current Oregon DEQ water quality standards.

3.6 Field Notes

Reservoir Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	January 30, 2018	**Observation Time**	5:00 pm (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Clear 50 degrees, rained 0.75" +/- 2-days earlier		

General Information

Structure	Reservoir No. 6
Address	NW Crown Street
Date Constructed (if known)	1982, per City
Date Retrofitted (if any)	n/a
Original Design Code	UBC 1979
Reservoir Type	D110, Type I - Concrete

Size	3.5 MG
Inside Diameter	144’
Wall Height	30’
Adjacent Structures (?)	None
General Notes –	

Exterior Inspection Information

Backfill (Height to top of wall)	Estimated, >20’ all around
Site Slope	North to South, site cut from hill side, site slopes up on NW to E sides where hillside was cut
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), no obvious signs of any movement
Foundation Condition	NA - Buried
Roof Condition	(9) roof pours, matches original drawings 7” thickness 6” overhang, w/ drip edge 2” from edge Moderate to fair amount of small hairline cracks Pattern of slightly larger cracks, consistent w/ negative moment at column strip at each middle column line quadrants as well as centerline each direction of Reservoir in the midpoint of the middles strip down Reservoir center lines Ponding evident areas at larger edge spans
Roof Hatch(s)	South Location 4’x4’, good condition Old style – non-compliant for water quality Hinge link rusted (not galvanized) Drain appears to be clogged
Roof Vent(s)	Northeast and Southwest Edges, no structural concerns appear in good condition
Wall	General minor cracking in shotcrete East to Southwest arc found a few isolated areas of potential delamination Quite a few areas of potential delaminations found in arc from SW to East based upon sounding

Interior Inspection Information

NA – Reservoir not drained

Destructive Testing Information

General Date: February 13th Time: 10.00 am, Approximately Number of Locations: 3
Location No. 1 Site: South-Southeast, approximately 26' down from top of wall Strand: (3) exposed @ 2.25" o.c. in 7" vertical expanse, no corrosion present Shotcrete: 1.5" of shotcrete cover, delamination appears local
Location No. 2 Site: South-Southwest, approximately 26.5' down from top of wall Strand: (3) exposed @ 2.5" o.c. in 7" vertical expanse, no corrosion present Shotcrete: 1.5" of shotcrete cover, delamination appears local
Location No. 3 Site: East-Northeast approximately 20' down from top of wall Strand: (3) exposed @3.5" o.c. in 7" vertical expanse, no corrosion present Shotcrete: 1.75" of shotcrete cover, delamination appears local

3.7 Scans of Select Construction Documents

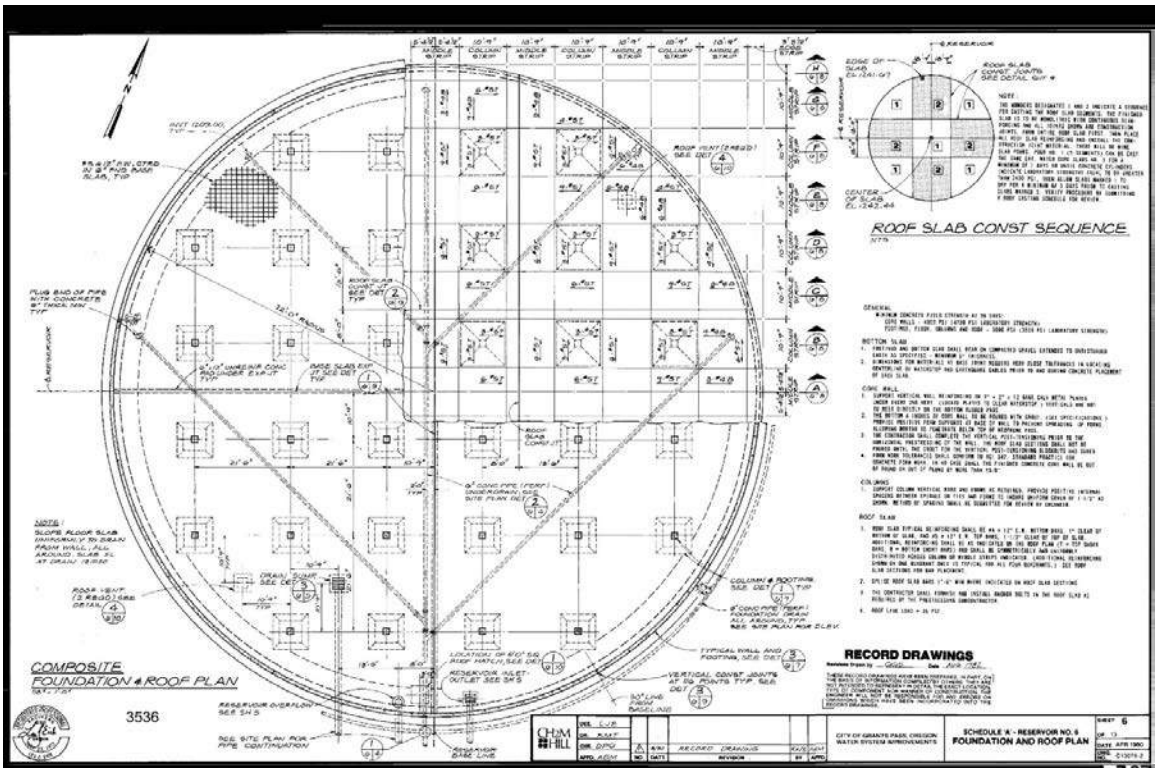


Figure 3-1: Composite Foundation & Roof Plan

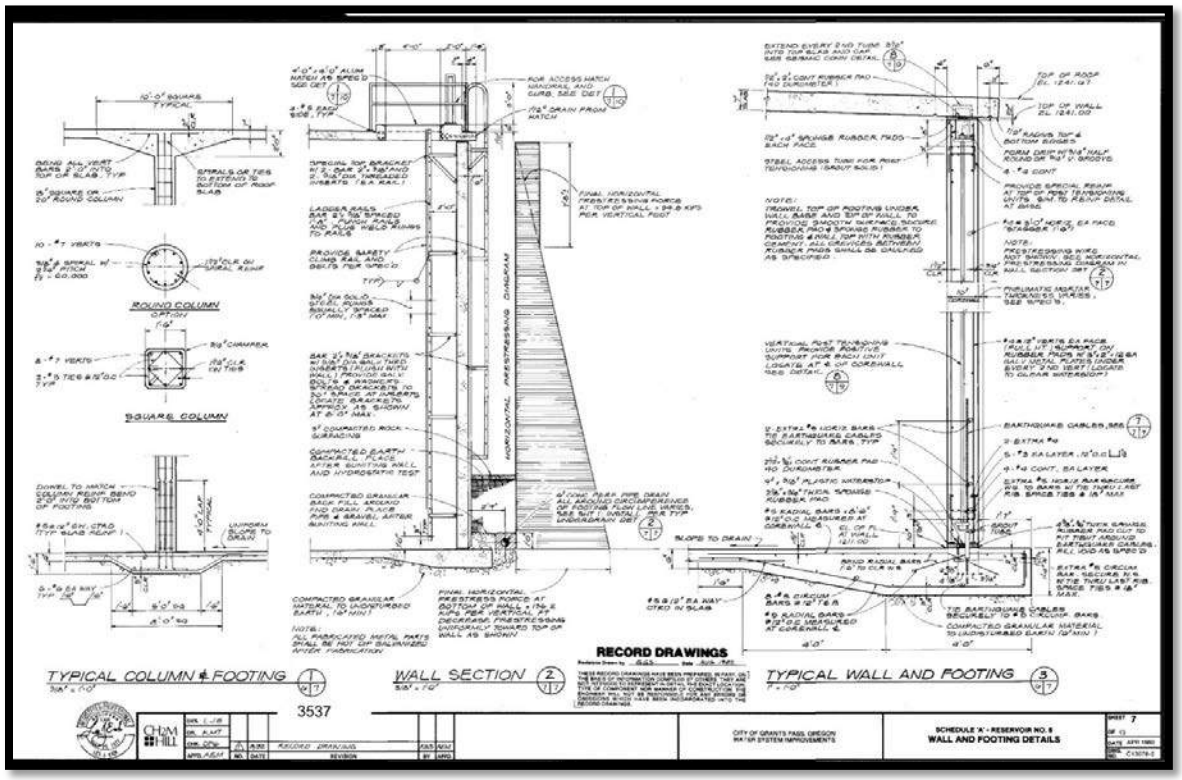


Figure 3-2: Wall & Footing Details

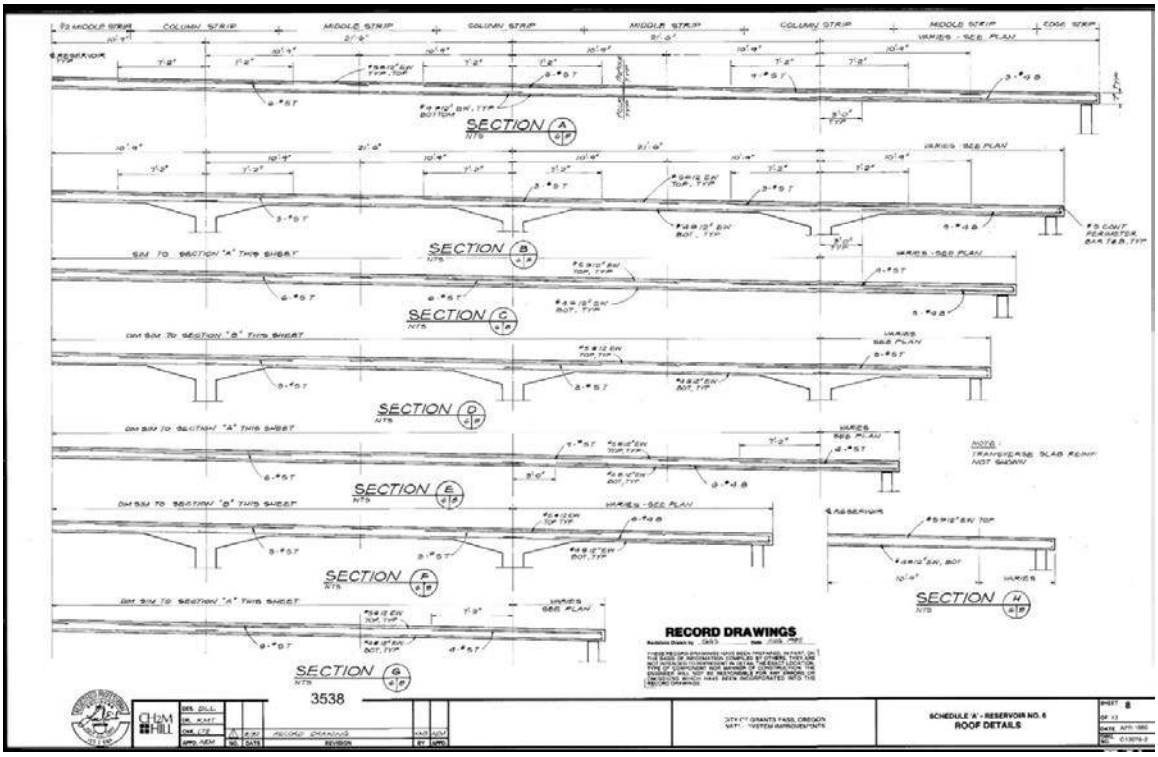


Figure 3-3: Roof Details

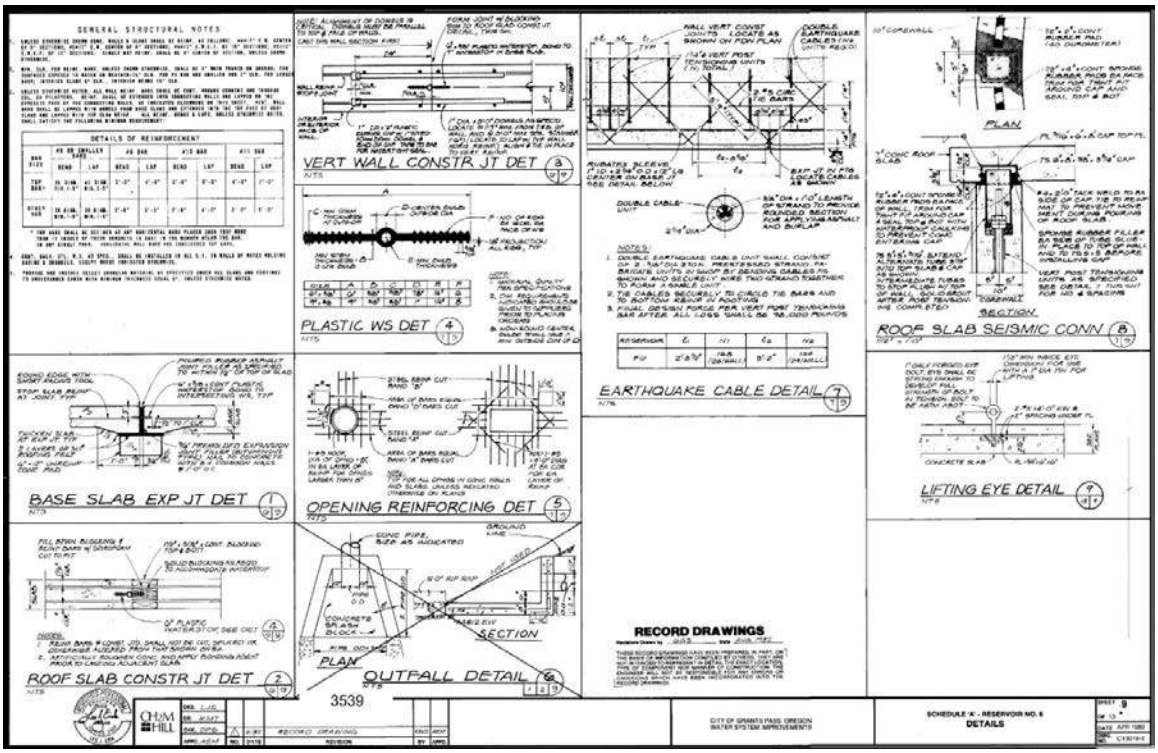


Figure 3-4: Details

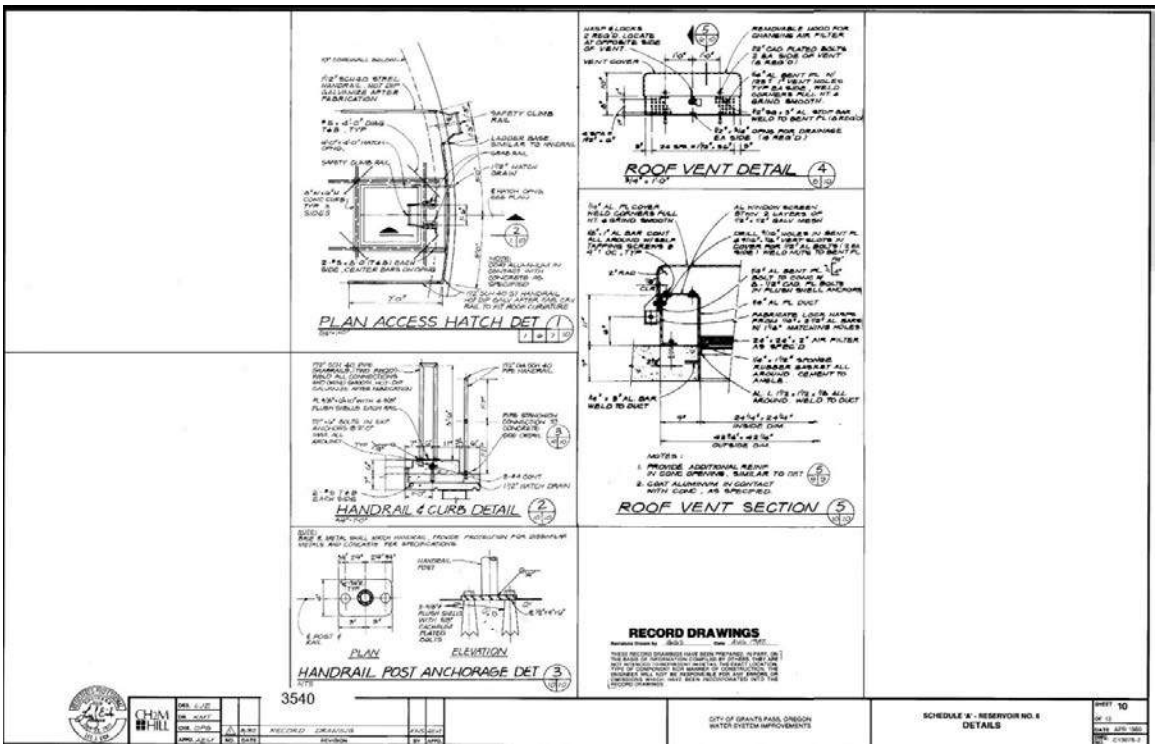


Figure 3-5: Details

3.8 Observations Pictures

Figure 3-6: Reservoir Elevation

Figure 3-7: Ladder

Figure 3-8: Hatch

Figure 3-9: Roof Joint & Typical Cracking

Figure 3-10: Crack at Roof Joint Intersection

Figure 3-11: Cracking/Spall at Roof Joint

Figure 3-12: Typical Cracking in Roof Slab

Figure 3-13: Roof Vent & Ponding

Figure 3-14: Spalled Concrete at Top of Wall to Roof Joint

END OF SECTION

4 Reservoir No. 8 – 2.0 MG

4.1 Description & Background

The original Reservoir was designed by CH2M Hill and the original construction drawings provided are dated May 1982. It is reported by the City that the original construction was circa 1983. The Reservoir is a ground-supported, partially buried, 108' inside diameter x 30' high, strand-wrapped, pre-stressed concrete water Reservoir with interior columns. The roof is 7.5" thick and is supported by (16) columns. The drawings show an option for square and round columns, and we were able to verify that the columns are square during our site inspection by observation through the hatch. The original roof was poured in four sections and appears to match the layout in the original drawings.

Per the original drawings the Reservoir does contain vertical pre-stressing bars in the core wall. The roof slab bears on the tank shell wall with shear cans at every other vertical tendon. There are 132 total vertical tendons—each 1.25" in diameter. The wall base connection utilizes 88 seismic cable sets each with (2) 0.375" diameter, 7-wire galvanized strands. Both the roof and the floor connection are detailed with an elastomeric bearing pad.

The original drawings specify a roof live load of 35 psf for design. This exceeds the current code requirements of 25 psf for snow load design for current code. For the evaluation contained herein the original specified roof live load of 35 psf was conservatively used. Note, this increase of live/snow load over current code requirements does not change the results contained herein; items identified as non-code compliant do not change if the live/snow load is reduced to 25 psf.

4.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the Reservoir on the exterior. The Reservoir was not able to be drained during our inspection, so we were unable to observe the interior except as noted through the hatch. The site visit was performed January 30th, 2018.

The existing condition of the Reservoir roof showed several areas of ponding that correspond with the exterior larger roof edge spans. The roof has a lot of hairline cracks throughout the roof area as well as a moderate amount of small cracking throughout the roof that appear to correspond with the negative moment column strips over the outermost column lines. There is some additional diagonal cracking at roughly each quadrant. The edge of the roof appears to be in good condition. The roof overhang is 6" and there is a drip edge located 2" from the exterior edge. There are a couple of areas in the roof that have voids from rock pockets.

The roof hatch is 4'x6' and appears to be in fair condition, however it appears to be an older style hatch which is no longer considered code compliant for water quality. Three of the four hatch hinge links are very corroded and do not appear to have been galvanized. The roof vent appears to be in good condition with no apparent structural deficiencies noted.

The interior of Reservoir was not able to be observed as the Reservoir was undrained. No visual issues were noted from observation through the hatch. The interior ladder appeared to be in good condition.

The exterior walls of Reservoir are covered in shotcrete to protect the strand wrapping around the core wall. The Reservoir is mostly above grade with what appears to be only a small amount of the wall buried, presumably for frost protection, and the exposed height of the wall is approximately 28'. The surrounding grade varies gradually around the perimeter of the Reservoir and slopes roughly North to South. The Reservoir site appears to have been cut into a shallow hillside at the top of a slope. The Reservoir site is located in a mostly forested area. There were no visual indications of any slope movement or settlement. It is noted that the trees may eventually be a potential hazard to the roof edge and/or the root systems may eventually reach the tank wall that could cause damage to the areas below grade.

We performed sounding of the walls around the perimeter of the Reservoir using a standard framing hammer to listen for hollow sounds indicative of delaminated shotcrete. There were very few areas of hollow sounding and appear to be very few areas of potential delamination. There are a number of small cracks in the shotcrete, but these appear to be cosmetic.

4.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13 was utilized. Evaluation was based on original construction documents and site visit observations.

4.3.1 Gravity Analysis

Roof Slab: The roof slab thickness of 7.5" is above the code recommended 7.48" minimum for the spans provided. Reinforcing in the roof slab is insufficient in most regions for flexural load and crack control requirements, when evaluated to current code requirements. It may be noted that current code requirements have increased safety factors than those mandated during the period of the original construction. Eventually the concrete creep will likely make the observed ponding increase but does not appear to be a significant concern at this time. Overall the roof appears to be in good condition and appears to have performed well under static loads.

Vertical Wall Reinforcement: Reservoir No. 8 does have vertical pre-stressing within the walls. The existing vertical pre-stressing appears to meet the current code requirements.

Columns: The columns in Reservoir No. 8 are square. Note that the original drawings provide options for both square and circular columns. The columns meet code requirements for static load resistance, see Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings appear to meet current code strength requirements. No allowable bearing pressure was provided for the site, but the maximum design soil bearing demand determined under the wall footings is about 3,900 psf, assuming an operating level of 24'.

4.3.2 Lateral Analysis

Seismic Joints: There are 88 sets of seismic cables (with two cables per set), which provides roughly 80% of the capacity required to meet code requirements at the current operating level. Additional restraint at the wall base is required. More seismic cables would be required at the base of the tank, as the existing cabling is inadequate to resist the base shear loads at the wall base.

Strand Wrap: The circumferential prestressing requirements for the Reservoir are detailed on the prestressing load distribution diagram on sheet 13 of the original drawings dated May, 1982. The wrapping schedule was not shown on the original drawings, however, calculating the area of the diagram will provide the total prestressing force that was required during the original design and construction. Dividing the total prestressing force by the final force of each machine applied wrap will provide the total number of wraps required. Based on the shop drawings provided by DN Tanks (known as DYK Prestressed Concrete Tanks when the Reservoir was constructed), dated October 8, 1982, the methodology described above was followed. It is important to note, that when the Reservoir was designed and constructed AWWA D110 had not been published and it is likely that designer followed ACI 344 – *Design and Construction of Circular Wire and Strand Wrapped Concrete Structures*, which was originally published in 1970.

Based on the prestressing load diagram shown CH2M Hill's drawings, DYK provided 146 wraps on their submitted shop drawing for the project. Following the current edition of AWWA D110-13 requirements, we have considered the hoop loads generated for two load cases as follows:

5. Static Load at Overflow water elevation plus 200 psi residual compression for differential temperature and dryness effects on the wall.
6. Hydrodynamic seismic loads, at the service water level, based on the USGS values for the site.

Applying this approach to the Reservoir at the current operating elevation of 24', results in the following required number of wraps:

5. 131 Wraps Load Case 1: Static Loads
6. 88 Wraps Load Case 2: Hydrodynamic Seismic Loads

Please note, since the Reservoir has some backfill around the circumference, we have followed the provisions in AWWA D110, Section 3.5.2.1, and tapered the 200 psi linearly to 50 psi over a below grade depth of 6 ft. Since the Reservoir requires 131 wraps, and it has 146 wraps, it meets the current code requirements under current operating levels.

If the Reservoir is evaluated for the maximum operating level at the overflow of 29' elevation the number of wraps per current code is 171 wraps for load case 1 and 129 wraps for load case 2, which exceeds the number of wraps provided.

Columns: The roof is supported by bearing pads on top of the wall and restrained by shear cans to limit the potential deflection to 0.125". This allowable movement allows the roof to move and lateral resistance within this range for the roof is created by the moment transfer into the existing columns. The existing

columns are able to resist the seismic loads induced by a 0.125" deflection. Therefore, the columns are adequate to resist both seismic and static loads at the currently used operating level of 24'.

Freeboard/Slosh: The current freeboard provided at the operating level of 24' exceeds the minimum amount required by ASCE 7-10. However, the slosh wave height is 53" if operated at the full 29' operating level. If the operating level is increased above 25'-9", uplift forces from slosh waves must be reviewed and accounted for. The roof appears to have inadequate dead load to resist the slosh impact loads at the operating, level so a more thorough analysis of the roof under slosh impact would be required.

4.4 Summary

The Reservoir roof slab appears to be under-designed per current code requirements, but the roof appears to still be in decent condition. The ponding noticed at the end spans will likely increase with age. The Reservoir has adequate wrapping for the current operating level. For the original water level, the Reservoir has adequate wrapping for seismic loads and static loads for the operating water level plus shrinkage and temperature conditions when evaluated by current code, which requires the wrapping to provide a minimum level of circumferential compression in the walls. The Reservoir needs upgrades at the wall-to-foundation to provide adequate load transfer for seismic loads. Also, additional maintenance to the hatches should be considered.

Based upon the evaluation and aforementioned conditions, Reservoir No. 8 can be brought up to current code requirements, with the exception of the roof slab.

4.5 Recommendations

In order to maintain (or increase) the current operating level and nominally comply with current structural code, several upgrades would be required to the wall and roof. The inadequate capacity of the seismic cables could be addressed by either adding new cables at the base with new wrapping and shotcrete, or by placing a new concrete curb around the perimeter of the wall base at the foundation to prevent sliding that would engage the existing cables in the case of a seismic event. A finite element analysis would be required to determine whether or not the roof is overstressed by a slosh wave induced by the desired operating level. If the roof is shown to be overstressed, it could potentially be upgraded or potential damage to the roof may be accepted as part of a risk management program.

Possible upgrades could include reinforcing the appropriate areas with a fiber-reinforced polymer (FRP) wrap or other alternate methods. Another option would be to accept potential damage to the roof in a significant enough seismic event and provide protection for the inlet/outlet pipe from being blocked should portions of the roof collapse into the tank. In addition, if the operating level were to increase, additional wrapping may be required depending on the final operating level.

From a water quality standpoint, the roof hatch does not meet current code requirements. Given this and the hatch's corroded hinges, we recommend that the City replace the hatch with a model that meets current Oregon DEQ water quality standards.

4.6 Field Notes

Reservoir Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	January 30, 2018	**Observation Time**	1:30 pm (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Clear 53 degrees, rained .75” +/- 2-days earlier		

General Information

Structure	Reservoir No. 8
Address	NW Hieglen Loop
Date Constructed (if known)	1983, per City
Date Retrofitted (if any)	n/a
Original Design Code	UBC 1979
Reservoir Type	D110, Type I - Concrete
Size	2.0 MG
Inside Diameter	108’
Wall Height	30’
Adjacent Structures (?)	None
General Notes –	

Exterior Inspection Information

Backfill (Height to top of wall)	Estimated, 28’ all around
Site Slope	North to South, site cut from top of shallow hill side, site slopes generally down North to South
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), no obvious signs of any movement
Foundation Condition	NA - Buried
Roof Condition	(4) roof pours, matches original drawings 7.5” thickness 6” overhang, w/ drip edge 2” from edge Moderate to fair amount of small hairline cracks Pattern of slightly larger cracks, consistent w/ negative moment at exterior most column strip with additional diagonal cracking at each quadrant. Ponding evident areas at larger edge spans in four quadrant locations.
Roof Hatch(s)	East Location 4’x6’, good condition Old style – non-compliant for water quality

	Hinge links rusted (not galvanized)
Roof Vent(s)	Northeast and Southwest Edges quadrants, middle of each quadrant, no structural concerns appear in good condition
Wall	General minor cracking in shotcrete, appears cosmetic Very few areas of potential delamination found, appears generally to be sound.

Interior Inspection Information

NA – Reservoir not drained

4.7 Scans of Select Construction Documents

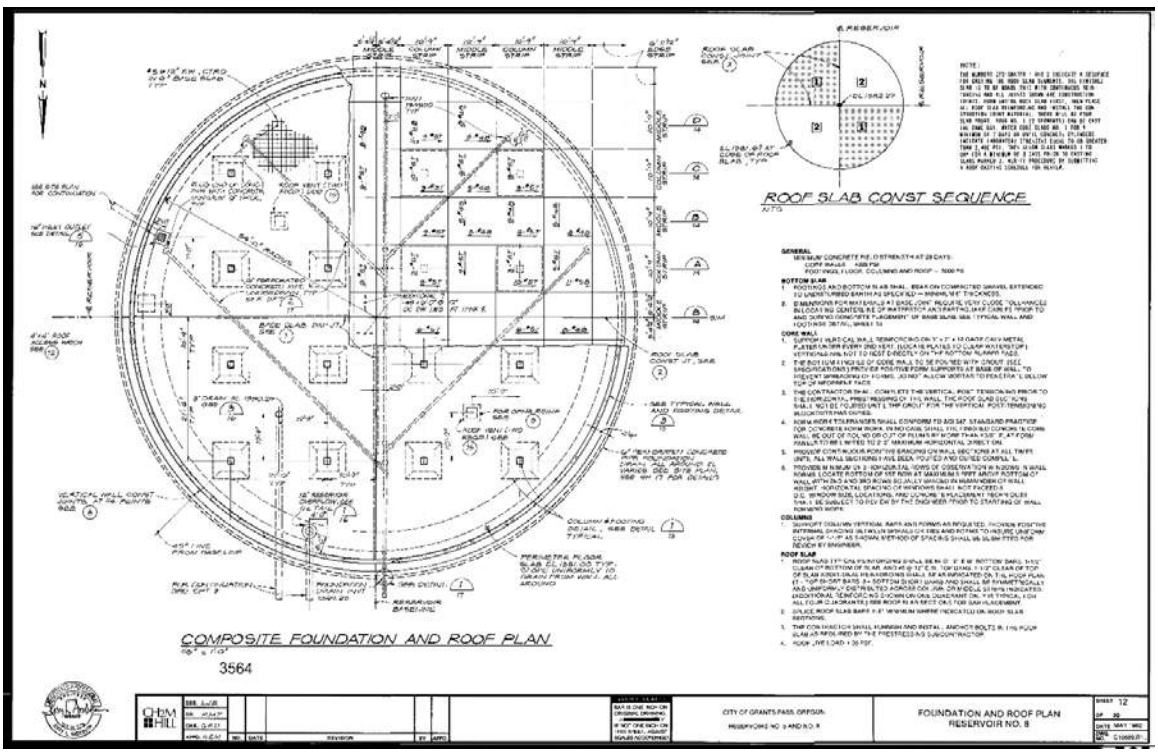


Figure 4-1: Foundation & Roof Plan

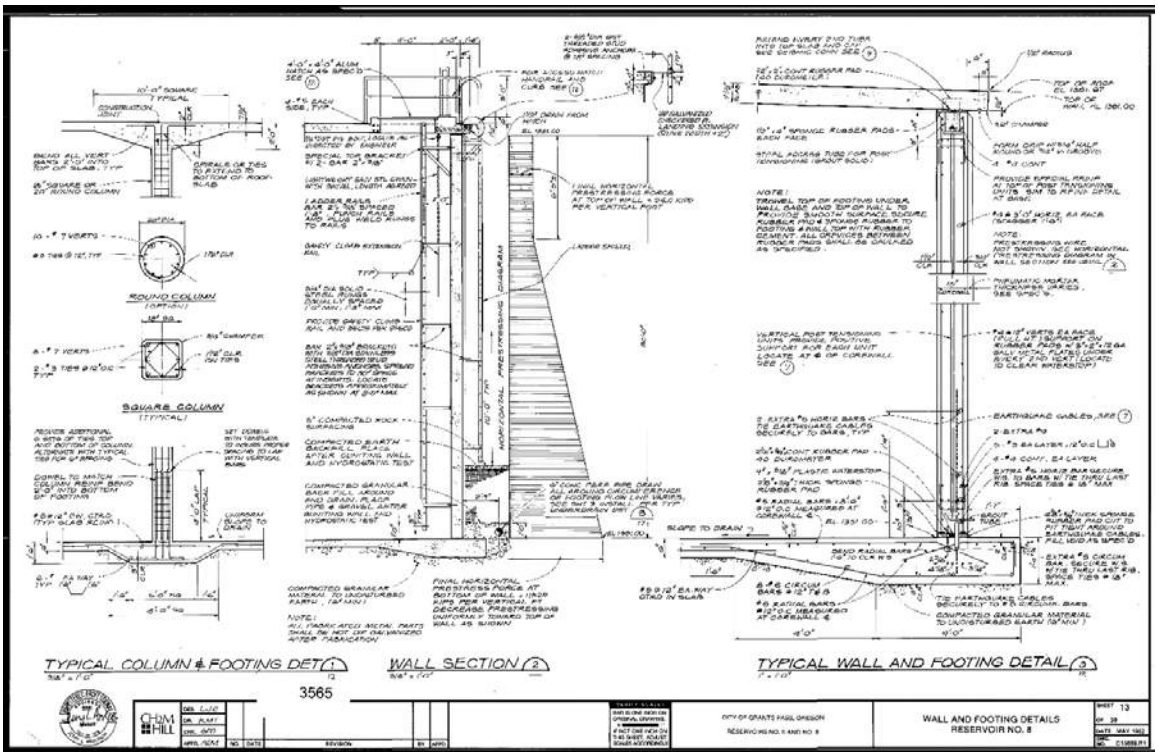


Figure 4-1: Wall & Footing Details

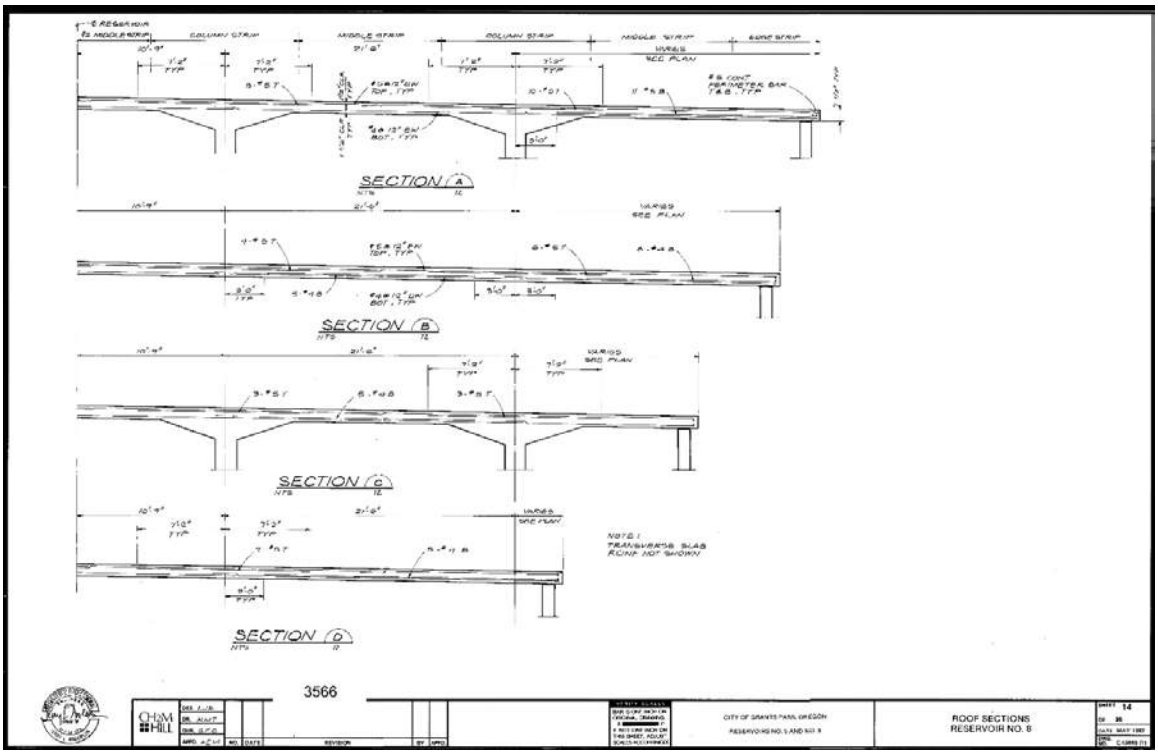


Figure 4-2: Roof Sections

4.8 Observations Pictures

Figure 4-3: Reservoir Elevation & Exterior Ladder

Figure 4-5: Reservoir Elevation

Figure 4-6: Typical Cracking in Shotcrete

Figure 4-7: Typical Roof Cracking

Figure 4-8: Hatch

Figure 4-9: Large Rock Pocket (Dirt & Debris removed by hand)

Figure 4-10: Typical Roof Cracking & Small Rock Pocket (Filled with Dirt & Moss)

Figure 4-11: Typical Ponding at Quadrants

Figure 4-12: Roof Joint at Center

END OF SECTION

5 Reservoir No. 11 – 4.5 MG

5.1 Description & Background

The original Reservoir was designed by OBEC and the original construction drawings provided are dated July, 2000 and are noted 'As Constructed'. It is reported by the City that the original construction was circa 1999. It appears that the drawings provided are the final as-built drawings following construction. The Reservoir is a ground-supported partially buried, 162' inside diameter x 30' high, wire-wrapped, pre-stressed concrete water Reservoir with interior columns. The walls sit on top of the exterior 12" thick footing which sits on top of the floor slab. The roof is 9.0" in thickness and is supported by (52) 18" diameter circular columns. The original roof was poured in four sections and appears to match the layout in the original drawings.

Per the original drawings the Reservoir contains vertical pre-stressing bars in the core wall. The roof slab bears on the tank shell wall with shear cans at every vertical tendon. There are 132 total vertical tendons each 1.25" in diameter. The wall base connection utilizes a pair of 0.5" diameter seismic cables at each vertical tendon in each direction. Both the roof and the floor connection are detailed with an elastomeric bearing pad.

Based upon our historical knowledge of the original construction we believe that the original tank contractor was Holm II out of Stayton, Oregon. It is our understanding that they built a few tanks in the region but have not built a significant number of tanks. In addition, we believe that the original Reservoir pre-stressing subconsultant was Crom Tanks. Crom Tanks specializes in D110 Type II tanks, whereas Reservoir No. 11 is a D110 Type I tank. It is our understanding that Crom built and used an electro-servo controlled mechanical wire wrapping machine for a few tanks in this era. This corresponds with our observations below that note that the pre-stressing consists of wire as opposed to strands for the pre-stressing wraps.

The original drawings specify a roof snow load of 25 psf for design. This is consistent with current code requirements and was used for the evaluation contained herein.

5.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the Reservoir on the exterior. The Reservoir was not able to be drained during our inspection, so we were unable to observe the interior except as noted through the hatch. The site visit was performed January 31th, 2018. A second site visit was performed as noted below to perform destructive testing to observe the existing wire wrapping by chipping away the shotcrete layer at three locations. Following these observations, the chipped areas were patched with non-shrink high strength concrete.

The existing condition of the Reservoir roof appears to be in good condition. No obvious signs of ponding were observed, but there may be some fairly minor areas at the larger exterior spans. The roof has some minor hairline cracks throughout the roof area that appear to be minor and typical. The edge of the roof appears to be in good condition. The roof overhang is 6" and there is a drip edge located 2" from the exterior edge.

The tops of the roof shear cans at each vertical tendon appear to extend to within 1” of the top of the roof slab per the original drawings and were dry packed with a cement sand (1C:2S) mix. The roof slab does not cover the shear cans and the edges of the roof were chamfered to the top of the shear cans and then infilled with the cement sand slurry. This appears to have performed very poorly and the top of the shear can is exposed in some areas where the cement sand slurry has eroded away and easily removed in other areas. Where it has eroded away, this area was filled with dirt and other growth.

The roof hatch is 4’x6’ and appears to be in good condition, however it appears to be an older style hatch which is no longer considered code compliant for water quality. The roof vent appears to be in good condition with no apparent structural deficiencies noted.

The interior of the Reservoir was not able to be observed as the Reservoir was undrained. No visual issues were noted from observation through the hatch.

The exterior walls of Reservoir are covered in shotcrete to protect the wire wrapping around the core wall. The Reservoir is partially buried; the exposed height of the wall varies from approximately 14’ at the Southeast to approximately 22’ at the Northwest. The surrounding grade varies gradually around the perimeter of the Reservoir and slopes roughly East to West. The Reservoir site appears to have been cut from the top of a hillside. The Reservoir site is located in a mostly forested area. There were no visual indications of any slope movement or settlement. It is noted that the trees may eventually be a potential hazard to the roof edge and/or the root systems may eventually reach the tank wall could cause damage to the areas below grade.

The finished grade around the perimeter of the Reservoir was paved for access around the full circumference. It was observed that there appears to have been roughly 0.75” of settlement of the subgrade around the Reservoir as evidenced by the curb.

We performed sounding of the walls around the perimeter of the Reservoir using a standard framing hammer to listen for hollow sounds indicative of delaminated shotcrete. These were performed every few feet around the full circumference from ground level (i.e. walking around). There were a considerable number of areas that had hollow sounding areas that were indicative of potential delamination. The delamination was generally worst around the Northern half but was significantly more than would be expected around the full perimeter. In general, the finish on the shotcrete also appeared to be below what we would expect for the age of construction.

Additional investigation of the shotcrete was performed as noted below.

5.2.1 Destructive Testing

Based upon the aforementioned area of potential delaminated shotcrete it was recommended that destructive testing be performed to observe the existing wire wrapping to look for potential corrosion or other issues related to delaminated shotcrete. A separate site visit was performed on February 14th, 2018 to perform the additional investigation. At this time three areas were selected, and the shotcrete was chipped away to expose the existing wire wrapping.

The observation of the wire showed no areas of corrosion where they are exposed. At these locations the wire appears to be in good condition. The locations chosen were based upon hammer soundings to locate areas with potential delamination/hollow spots that would be more likely to experience corrosion damage. The shotcrete in each of these locations was determined to be delaminated during the chipping process and in poor condition and was removed easier than competent shotcrete would be until the delaminated portion was removed. The delaminated area appears to be limited to the shotcrete layer covering the wire. The shotcrete behind the wires appears to be in good condition from the limited exploration. The depth of shotcrete was approximately 1.5”-1.75” to the inner (first) wrap of wire at all three locations. We were able to find the transition between inner (first) and outer (second) wrap layers as well and found that the depth of shotcrete between wraps was approximately 0.75” which is consistent with a 0.375” wire and 0.375” of shotcrete cover between wraps.

5.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, “Minimum Design Loads for Buildings and Other Structures” (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13 was utilized. Evaluation was based on original construction documents and site visit observations.

5.3.1 Gravity Analysis

Roof Slab: The roof slab thickness of 9” is above the code recommended 6.5” minimum for the spans provided. Reinforcing in the roof slab is sufficient in for flexural load and crack control requirements, but the reinforcement in some regions is insufficient to meet current code minimum requirements. Since the other strength and serviceability requirements are met, the reinforcing should be adequate based on code requirements for existing structures (specifically the American Society for Civil Engineers “Seismic Evaluation and Retrofit for Existing Buildings” or ASCE 41-13). Overall the roof appears to be in good condition and appears to have performed well under static loads.

Vertical Wall Reinforcement: Reservoir No. 11 does have vertical pre-stressing within the walls. The existing vertical pre-stressing appears to meet the current code requirements.

Columns: The columns in Reservoir No. 11 are circular. The columns meet code requirements for static load resistance, see Lateral Analysis for additional information.

Foundations: The foundations for the columns and wall footings appear to meet current code strength requirements. No allowable bearing pressure was provided for the site, but the maximum design soil bearing demand determined under the wall footings is about 3,700 psf, assuming an operating limit of 30’. The soil bearing capacity was given in the drawings as 4,000 psf.

5.3.2 Lateral Analysis

Seismic Joints: Based on our analysis, the existing cabling is adequate to resist the base shear loads at the wall base. There is one set of cables for every vertical tendon, which provides 33% more than the capacity required. Additional restraint at the wall base is not required.

Wire Wrap: The circumferential prestressing requirements for the Reservoir are detailed on the prestressing load distribution diagram on sheet S303 of the original drawings dated July, 2000. The wrapping schedule was shown on the original drawings and specified 313 total wraps. The original drawings show 3/8" diameter pre-stressing strands, whereas, the actual construction used wire pre-stressing. We were unable to locate any as-built drawings of the actual pre-stressing wire that was placed on the Reservoir.

Following the current edition of AWWA D110-13 requirements, we have considered the hoop loads generated for two load cases as follows:

7. Static Load at Overflow water elevation plus 200 psi residual compression for differential temperature and dryness effects on the wall.
8. Hydrodynamic seismic loads, at the service water level, based on the USGS values for the site.

Applying this approach to the Reservoir at the overflow elevation, results in the following required number of wraps:

7. 228 Wraps Load Case 1: Static Loads
8. 198 Wraps Load Case 2: Hydrodynamic Seismic Loads

Please note, since the Reservoir has some backfill around the circumference, we have followed the provisions in AWWA D110, Section 3.5.2.1, and tapered the 200 psi linearly to 50 psi over a below grade depth of 6 ft.

Assuming the final pre-stressing force was the same as that shown in the original drawings then the prestressing would meet current code requirements.

Columns: The roof is supported by bearing pads on top of the wall and restrained by shear cans to limit the potential deflection to 0.3125". This allowable movement allows the roof to move and lateral resistance within this 0.3125" range for the roof is created by the moment transfer into the existing columns. The existing columns are adequate to resist the seismic loads induced by a 0.3125" deflection. As noted above, the columns also meet code requirements for static loads.

Freeboard/Slosh: The current freeboard of 3' is less than the minimum amount required by ASCE 7-10 (4'-8"), even with the reduced operating level (28' versus the intended 30'). Therefore, uplift forces from slosh waves must be reviewed and accounted for. The roof appears to have adequate dead load to resist the slosh impact loads for both the maximum and actual levels.

5.4 Summary

The Reservoir roof slab meets the current code strength requirements, but the reinforcing appears to fall below the current code-minimum requirements in some areas. The roof appears to be in good condition and currently shows minimal signs of ponding. The Reservoir has adequate wire wrapping for seismic loads and for static loads for the operating water level plus shrinkage and temperature conditions when evaluated by current code, which requires the wrapping to provide a minimum level of circumferential compression in the walls.

Based upon the evaluation and aforementioned conditions, Reservoir No. 11 appears to generally meet or exceeds current code requirements, except as noted. However, as indicated above, some items will require additional maintenance in order to ensure that the tank continues to perform well.

5.5 Recommendations

In order to maintain (or increase) the current operating level and nominally comply with current structural code, no structural upgrades would be required. The shear cans' infill and cover should be repaired in order to mitigate any future corrosion.

From a water quality standpoint, the roof hatch does not meet current code requirements. We recommend that the City replace the hatch with a model that meets current Oregon DEQ water quality standards.

5.6 Field Notes

Reservoir Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	January 31, 2018	**Observation Time**	10:30 am (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Clear 45 degrees, rained 0.75" +/- 3-days earlier		

General Information

Structure	Reservoir No. 11
Address	Denton Trail
Date Constructed (if known)	1999, per City
Date Retrofitted (if any)	n/a
Original Design Code	UBC 1994, per original drawings
Reservoir Type	D110, Type I - Concrete
Size	4.5 MG
Inside Diameter	162'
Wall Height	30'
Adjacent Structures (?)	None
General Notes –	

Exterior Inspection Information

Backfill (Height to top of wall)	Estimated: 14' (SE), 22' (NW), 18'-9" (NE), 16'-9" (SW)
Site Slope	Cut into bluff/hill top extending from general slope that runs East to West
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), no obvious signs of any movement
Foundation Condition	NA - Buried
Roof Condition	(4) roof pours, matches original drawings 9" thickness 6" overhang, w/ drip edge 2" from edge Minor small hairline cracks Potential ponding areas at larger edge spans; not visually clear Shear cans in very poor condition
Roof Hatch(s)	East Location 4'x6', good condition Old style – non-compliant for water quality
Roof Vent(s)	Southeast quadrant near center, no structural concerns appear in good condition
Wall	Visually poor finish, fair amount of cracking in shotcrete A lot of potential delaminations around full perimeter

Interior Inspection Information

NA – Reservoir not drained

Destructive Testing Information

General Date: February 13th Time: 3.00 pm, Approximately Number of Locations: 3
Location No. 1 Site: Southeast side, approximately 20' down from top of wall Wire: (4) exposed @ 0.5" o.c. in 4" vertical expanse, no corrosion present Shotcrete: 1.5" of shotcrete cover, delamination appears to be full outer layer
Location No. 2 Site: North side, approximately 15'-9" down from top of wall Wire: (7) exposed @ 0.5" o.c. in 5" vertical expanse, no corrosion present Shotcrete: 1.75"" of shotcrete cover, delamination appears full outer layer
Location No. 3 Site: West 10.5' down from top of wall Wire: (4) exposed in 7" vertical expanse, no corrosion present Shotcrete: 1.75" of shotcrete cover to (3) wires and 2.25" to (1) wire [Uppermost], delamination appears full outer layer as well as are between differing depth wires. Unclear why the depth between wires varies at this location as it doesn't appear to correspond to the original drawings wrap schedule for 1st & 2nd layers.

5.7 Scans of Select Construction Documents

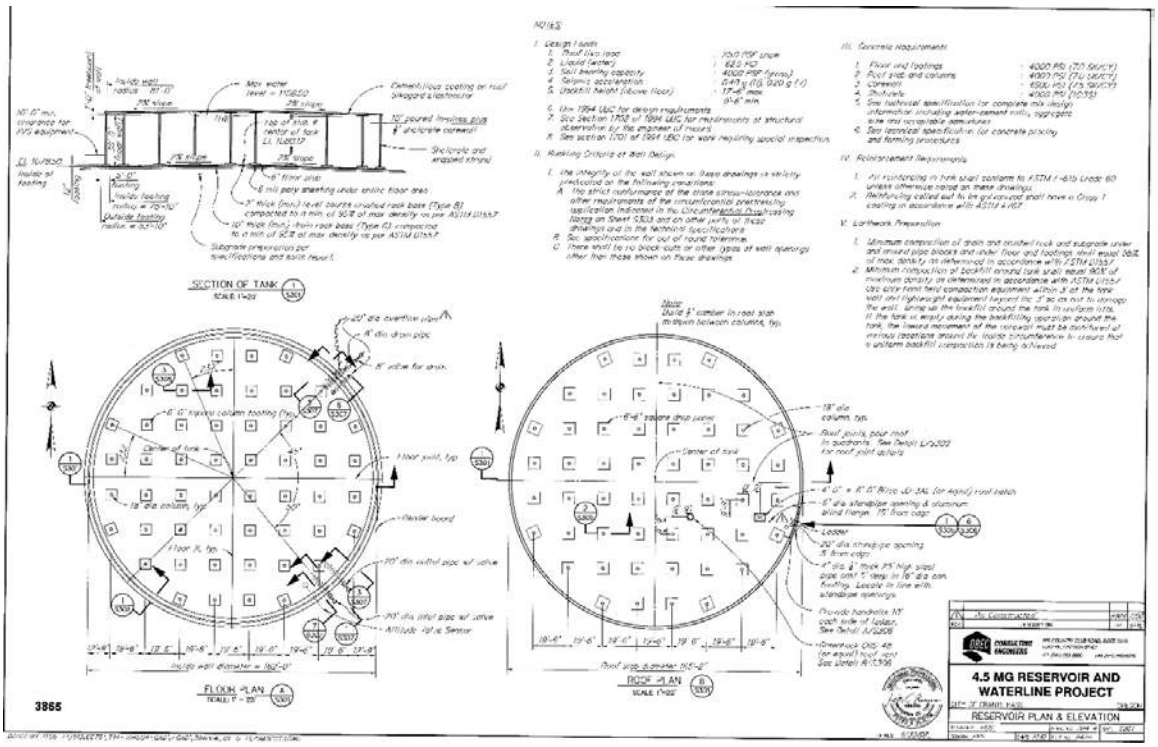


Figure 5-1: Reservoir Plans & Elevation

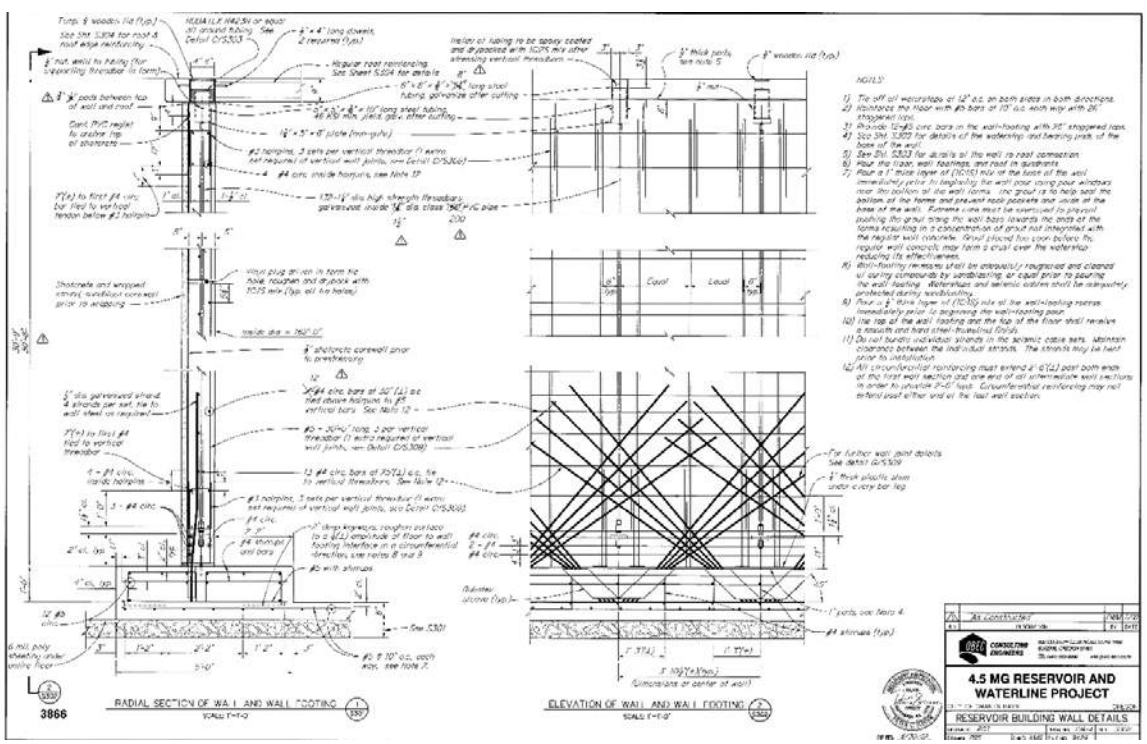


Figure 5-1: Reservoir Building Wall Details

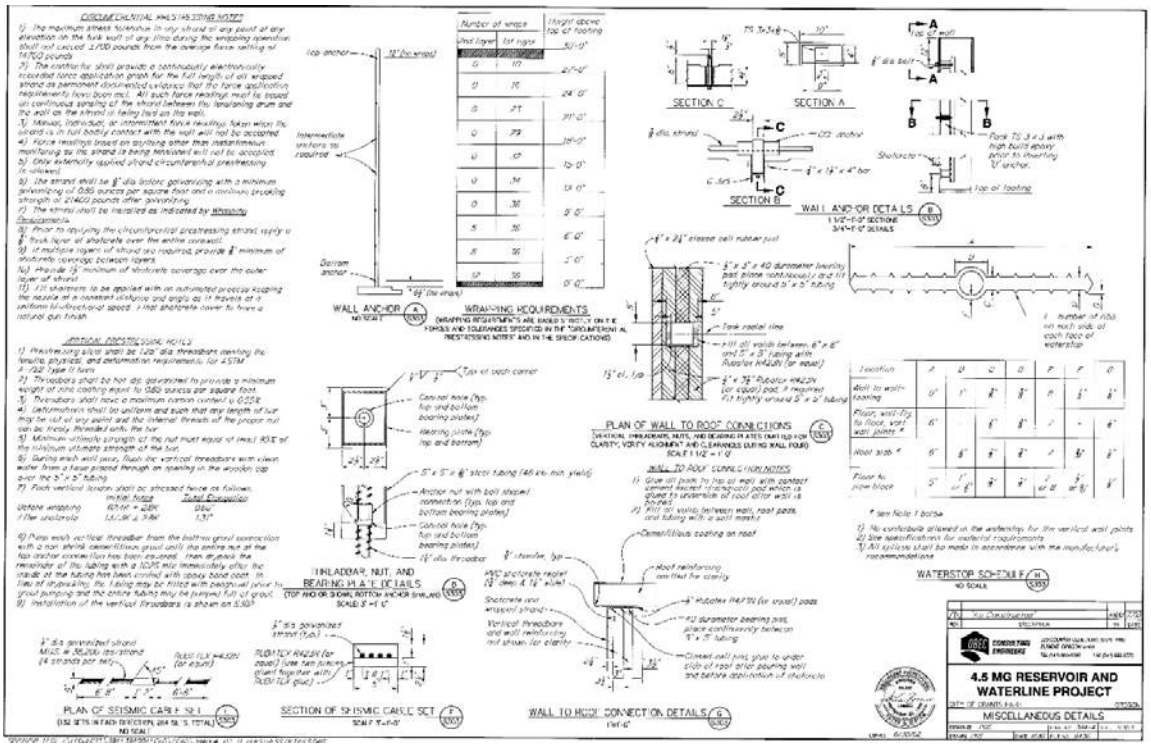


Figure 5-3: Miscellaneous Details

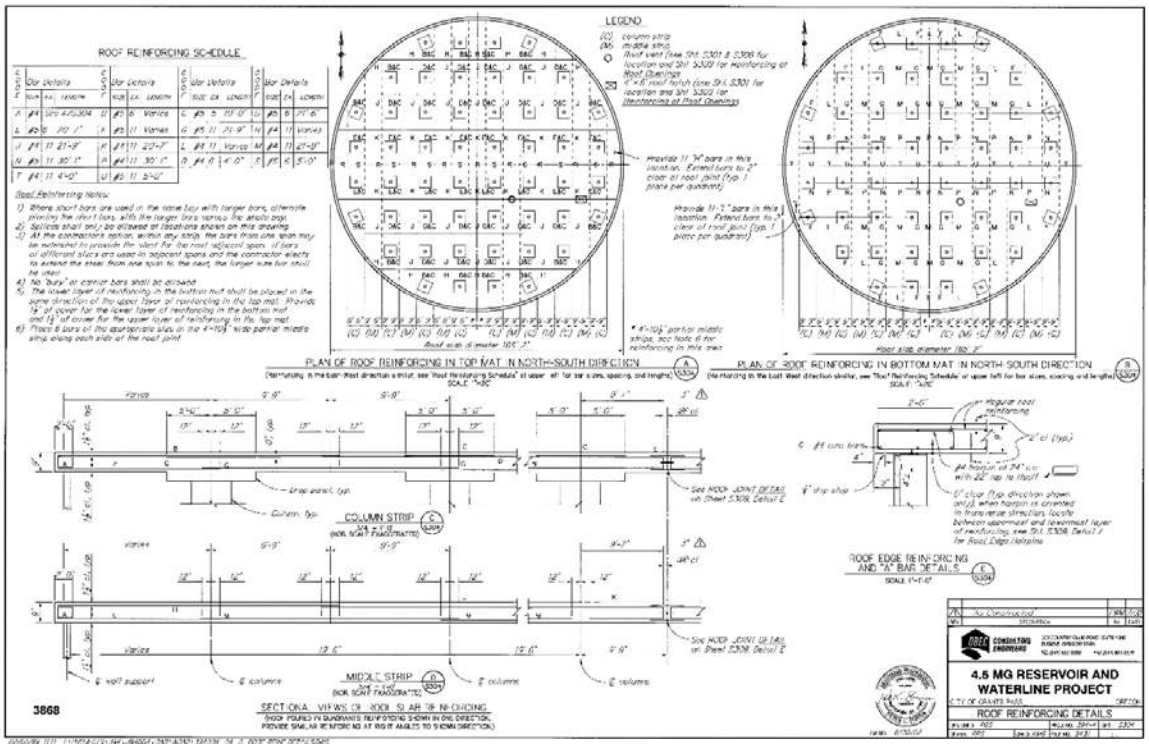


Figure 5-4: Roof Reinforcing Details

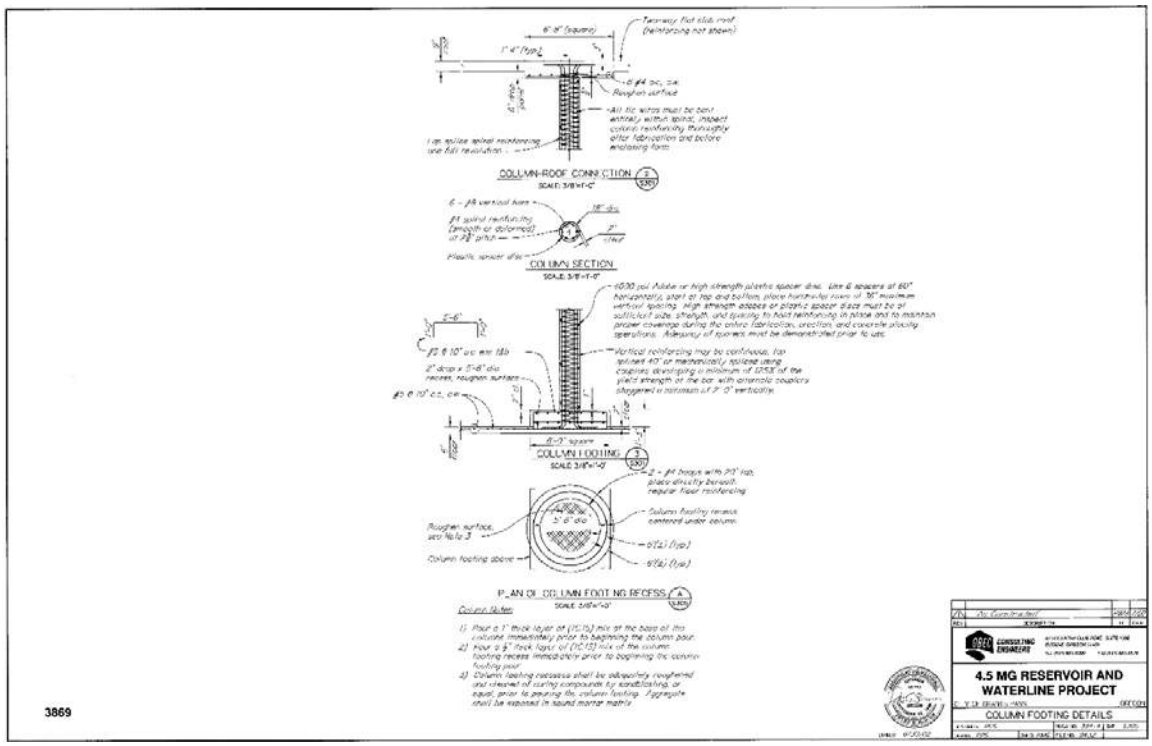


Figure 5-5: Column Footing Details

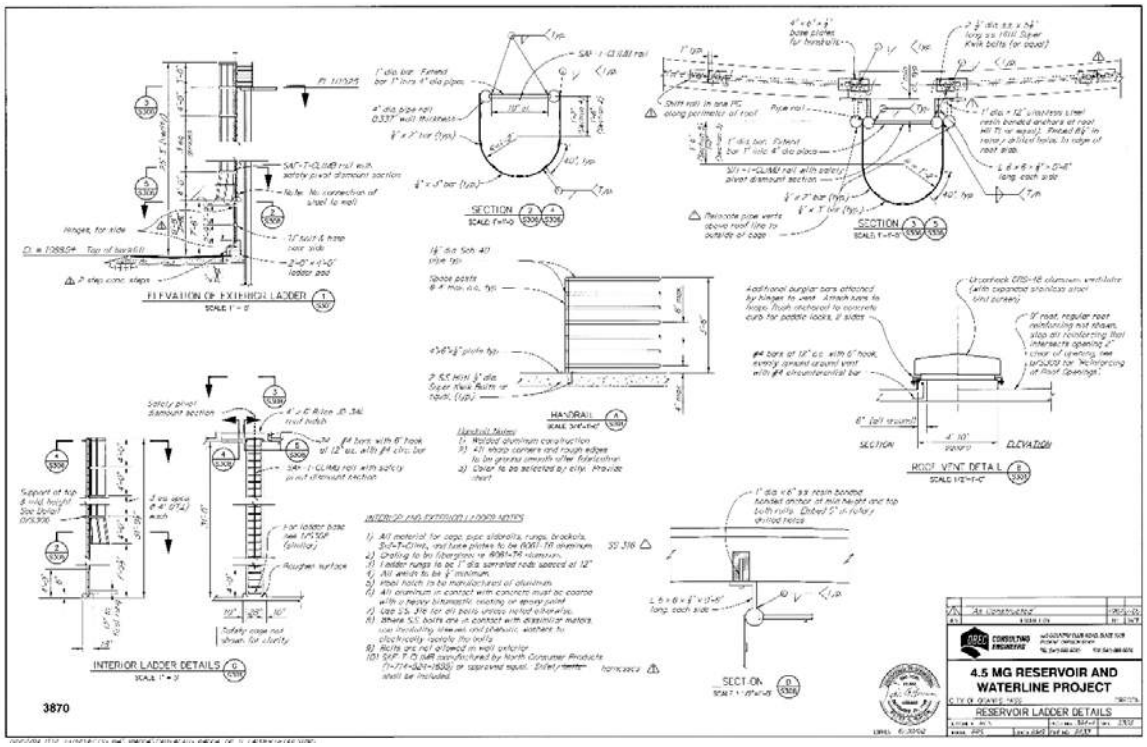


Figure 5-6: Reservoir Ladder Details

5.8 Observations Pictures

Figure 5-7: Reservoir Elevation

Figure 5-8: Settlement at Curb

Figure 5-9: Typical Cracks in Shotcrete

Figure 5-10: Exterior Ladder

Figure 5-11: Shear Can at Vertical Tendons w/ Exposed Steel

Figure 5-12: Shear Can w/ Deteriorated Infill

Figure 5-13: Hatch

Figure 5-14: Roof Joint at Center

Figure 5-15: Typical Roof Cracking

Figure 5-16: Exposed Wire Pre-stressing – Location 1

Figure 5-17: Exposed Wire Pre-stressing – Location 1

Figure 5-18: Exposed Wire Pre-stressing – Location 3

END OF SECTION

6 Reservoir No. 15 – 1.3 MG

6.1 Description & Background

Based upon the documents provided by the City it is our understanding that the Civil Engineer for the Reservoir was Robert E. Myer. However, no drawings of the Reservoir construction itself, other than Civil site drawings, were in the documents provided by the City. Based upon the Civil drawings it appears that the Reservoir was bidder designed by the Contractor. We were able to locate copies of the original Structural Drawings for the Reservoir from the pre-stressing specialty subcontractor from the original construction, DN Tanks (DYK at the time of construction). However, there is no information on those drawings about who designed the Reservoir or who the Structural Engineer of Record is. The drawings appear likely to be Contractor-generated shop drawings. A copy of those drawings was provided to the City for reference.

The Reservoir drawings obtained from DN Tanks are dated September 1st, 1983. It is reported by the City that the original construction was circa 1985. The Reservoir is a ground-supported, 85' inside diameter by 31' high (measured from top of the 12" footing, which sits on top of the floor slab, to top of wall), strand-wrapped, pre-stressed concrete water Reservoir with a domed roof. The roof dome is generally 4.0" in thickness with increased thickness at the heel and crown. According to the provided drawings, the water service level is 31' above the floor elevation at the wall; however, the Reservoir was constructed with a future industrial development in mind, but the development never came to fruition. Therefore, the operating limit maxes out at about 7' in order to avoid water quality issues.

Per the original drawings, the Reservoir does contain vertical pre-stressing bars in the core wall. The roof dome bears on the tank shell wall and is restrained with additional circumferential stressing. There are 135 total vertical tendons, the size of the vertical tendons does not appear to be shown. We have assumed that they are each 1.25" in diameter. The wall base connection utilizes 68 triple seismic cables each 0.375" in diameter. The wall to floor connection is detailed with an elastomeric bearing pad.

The original drawings do not specify the required roof snow or live load. The original roof design was performed by the Contractor and while we were able to locate those drawings they do not list the design criteria. As such we have assumed a roof snow load of 25 psf, which is consistent with current code requirements and was used for the evaluation contained herein.

6.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the Reservoir on the exterior. The Reservoir was not able to be drained during our inspection, so we were unable to observe the interior except as noted through the hatch. The site visit was performed January 30th, 2018.

The roof has only minor hairline cracks throughout the roof area. There is no visual indication of any movement or settling of the dome. The edge of the roof appears to be in good condition. The roof is surrounded by a 9" wide by approximately 6" tall parapet. The fillet area was infilled and sloped to allow drainage to drain/scupper openings around the perimeter. The drain openings are nearly clogged in several locations.

The roof hatch is 4'x6' and appears to be in fair condition, however it appears to be an older style hatch which is no longer considered code-compliant for water quality. One of the hinge links was very corroded and broken and a second appears to have some significant corrosion. The roof vent appears to be in good condition with no apparent structural deficiencies noted.

The interior of the Reservoir was not able to be observed as the Reservoir was undrained. No visual issues were noted from observation through the hatch. The interior ladder appeared to be in good condition.

The exterior walls of the Reservoir are covered in shotcrete to protect the strand wrapping around the core wall. The Reservoir appears to be at or near grade with either the top of the footing exposed or a small slab poured around the perimeter. There is a sealant joint at the wall to top of slab joint that is failing in some locations. It is unclear what occurs below this joint as there is a void behind the sealant that extends downwards. We were unable to observe below the joint and there is no detailing in any of the original drawings that corresponds with the as-built observed condition (See Figure 6-12).

The surrounding grade slopes generally from Northeast to Southwest. The Reservoir site is located in a mostly forested area. There were no visual indications of any slope movement or settlement. It is noted that the trees may eventually be a potential hazard to the roof edge and/or the root systems may eventually reach the tank wall could cause damage to the areas below grade.

We performed sounding of the walls around the perimeter of the Reservoir using a standard framing hammer to listen for hollow sounds indicative of delaminated shotcrete. There were very few areas of hollow sounding and appear to be very few areas of potential delamination. There are a number of small cracks in the shotcrete, but these appear to be cosmetic.

6.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the Reservoirs under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Water Works Associate (AWWA) code AWWA D110-13 was utilized. Evaluation was based on original construction documents and site visit observations.

6.3.1 Gravity Analysis

Roof Slab: The roof slab thickness of 4" is above the (AWWA) code minimum value of 2". Reinforcing in the roof slab is sufficient for flexural load and crack control requirements, and the reinforcement also meets current code minimum requirements. Overall the roof appears to be in good condition and appears to have performed well under static loads.

Vertical Wall Reinforcement: Reservoir No. 15 does have vertical pre-stressing within the walls. The existing vertical pre-stressing could meet the current code requirements if the bars were tensioned to achieve a wall compression stress of 200 psi, per code requirements, however, we don't have records to detail the actual tendon stressing loads.

Foundations: The foundations for the wall footings appear to meet current code strength requirements. No allowable bearing pressure was provided for the site, but the maximum design soil bearing demand determined under the wall footings is about 4,250 psf, assuming an operating limit of 31'. This demand drops down to 2,500 psf at an operating level of 7'. The soil bearing capacity was given in the shop drawings as 5,000 psf.

6.3.2 Lateral Analysis

Seismic Joints: If the full capacity of the Reservoir is ever to be utilized, then more seismic cables would be required at the base of the tank, as the assumed existing cabling is inadequate to resist the base shear loads at the wall base. There are 68 sets of seismic cables, which provides roughly half of the capacity required when considering the full capacity.

If the operating levels for this Reservoir are maintained as they are currently, then the seismic joints appear to be adequate.

Strand Wrap: The circumferential prestressing requirements for the Reservoir were not detailed on the original drawings dated March, 1983. We were able to obtain copies of the shop drawings provided by DN Tanks (known as DYK Prestressed Concrete Tanks when the Reservoir was constructed), dated September 1, 1983, where the prestressing wrapping schedule was shown. It is important to note, that when the Reservoir was designed and constructed AWWA D110 had not been published and it is likely that designer followed ACI 344 – *Design and Construction of Circular Wire and Strand Wrapped Concrete Structures*, which was originally published in 1970.

Based on the prestressing schedule shown on DYK's submitted shop drawings 122 wraps were provided for the project. Following the current edition of AWWA D110-13 requirements, we have considered the hoop loads generated for two load cases as follows:

9. Static Load at Overflow water elevation plus 240 psi residual compression for differential temperature and dryness effects on the wall. Note, since the Reservoir has a domed roof that can restrain the top of the wall AWWA requires a prestress of 240 psi.
10. Hydrodynamic seismic loads, at the service water level, based on the USGS values for the site.

Applying this approach to the Reservoir at the maximum operating elevation of 31', results in the following required number of wraps:

9. 168 Wraps Load Case 1: Static Loads
10. 115 Wraps Load Case 2: Hydrodynamic Seismic Loads

Please note, since the Reservoir has some backfill around the circumference, we have followed the provisions in AWWA D110, Section 3.5.2.1, and tapered the 240 psi linearly to 50 psi over a below grade depth of 6 ft. Since the Reservoir requires 168 wraps, and it has 115 wraps, it is inadequate for current code requirements under the maximum operating levels.

However, as with the seismic joints, if the current operating levels are maintained, then the current wrapping is more than adequate to meet code requirements. Based upon the 122 strand wraps provided it appears that the pre-stressing would meet code requirements for an operating level up to 25'. Note this is well in excess of the current operating level of 7'.

Per the shop drawings, additional wraps were placed around the perimeter of the wall to resist roof thrust forces. The number of wraps for this dome ring were not provided, but a recommended final prestressing force of 191.6 kips was provided. Based on the requirements of AWWA D110, this prestressing force appears adequate to resist the design roof thrust forces.

Freeboard/Slosh: The current freeboard height provided is significantly less than the minimum amount required by ASCE 7-10 for the original service water depth of 31'. At a full 31' operating depth the required free board is 50.5". Therefore, uplift forces from slosh waves must be reviewed and accounted for if the Reservoir is ever to be used at the full capacity. The roof appears to have inadequate dead load to resist the slosh impact loads at the maximum operating level. However, if the service level is kept to a level below 27' then the slosh wave calculated will not impact the roof. Based upon the current operating level of 7' (as it currently is), then there should be no issues with freeboard height or slosh wave effects.

6.4 Summary

The Reservoir roof slab and reinforcing meet the current code strength and minimum requirements. The roof appears to be in good condition and currently shows no signs of ponding or excessive cracking. If the original water level is considered, then the Reservoir has potential seismic deficiencies with the wrapping as well as the seismic base joint. However, at the current operating level, the Reservoir has adequate wrapping for seismic loads and for static loads for the operating water level plus shrinkage and temperature conditions when evaluated by current code, which requires the wrapping to provide a minimum level of circumferential compression in the walls. Maintenance to the hatches should be performed. If required, the operating level of the tank could be raised to 19' and still meet the required seismic joint, wrapping, and freeboard/slosh requirements.

Based upon the evaluation and aforementioned conditions, Reservoir No. 15 meets or exceeds current code requirements for its current operating levels. However, as indicated above, the current hatch does not meet current water quality standards.

6.5 Recommendations

At the current operating level of 7', the Reservoir is in compliance with current structural codes. The operating level could be increased to 19' and still be code-compliant, with the limiting factor being the seismic cables. For higher operating levels, additional seismic base restraint and roof analysis/support may be required.

From a water quality standpoint, the roof hatch does not meet current code requirements. Given this and the hatch's corroded hinges, it may be prudent to replace the hatch with a model that meets current water quality standards.

6.6 Field Notes

Reservoir Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	January 30, 2018	**Observation Time**	4:00 pm (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Overcast 50 degrees, rained 0.75" +/- 2-days earlier		

General Information

Structure	Reservoir No. 15
Address	Highland Avenue
Date Constructed (if known)	1985, per City
Date Retrofitted (if any)	n/a
Original Design Code	UBC 1982 (Assumed)
Reservoir Type	D110, Type I - Concrete
Size	1.3 MG
Inside Diameter	85'
Wall Height	31' (To top of footing), 32' to Floor Slab
Adjacent Structures (?)	None
General Notes –	

Exterior Inspection Information

Backfill (Height to top of wall)	Estimated, 31' all around (i.e. top of Footing exposed)
Site Slope	Northeast to Southwest, site is generally flat, immediately adjacent to I-5 & Highland Ave.
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), no obvious signs of any movement
Foundation Condition	Top of footing appears to be in fair condition w/ minor cracking
Roof Condition	Domed Roof appears to match original drawings Thickness not verified (see drawings) 6" +/- parapet, varies with slope to drains Parapet curb 9" wide Drains nearly clogged with debris/dirt Very little cracking in roof, some small hairline cracks Threadbar pocket spacing at 24" o.c. roughly

Roof Hatch(s)	Northwest Edge of roof 4'x6', good condition (wasp nests) Old style – non-compliant for water quality One broken/Corroded link arm and one arm starting to corrode. Drain appeared to be clogged.
Roof Vent(s)	Center of Dome, no structural concerns, appears to be in good condition
Wall	General minor cracking in shotcrete, appears cosmetic Very few areas of potential delamination found, appears generally to be sound and in good condition.

Interior Inspection Information

NA – Reservoir not drained

6.7 Scans of Select Construction Documents

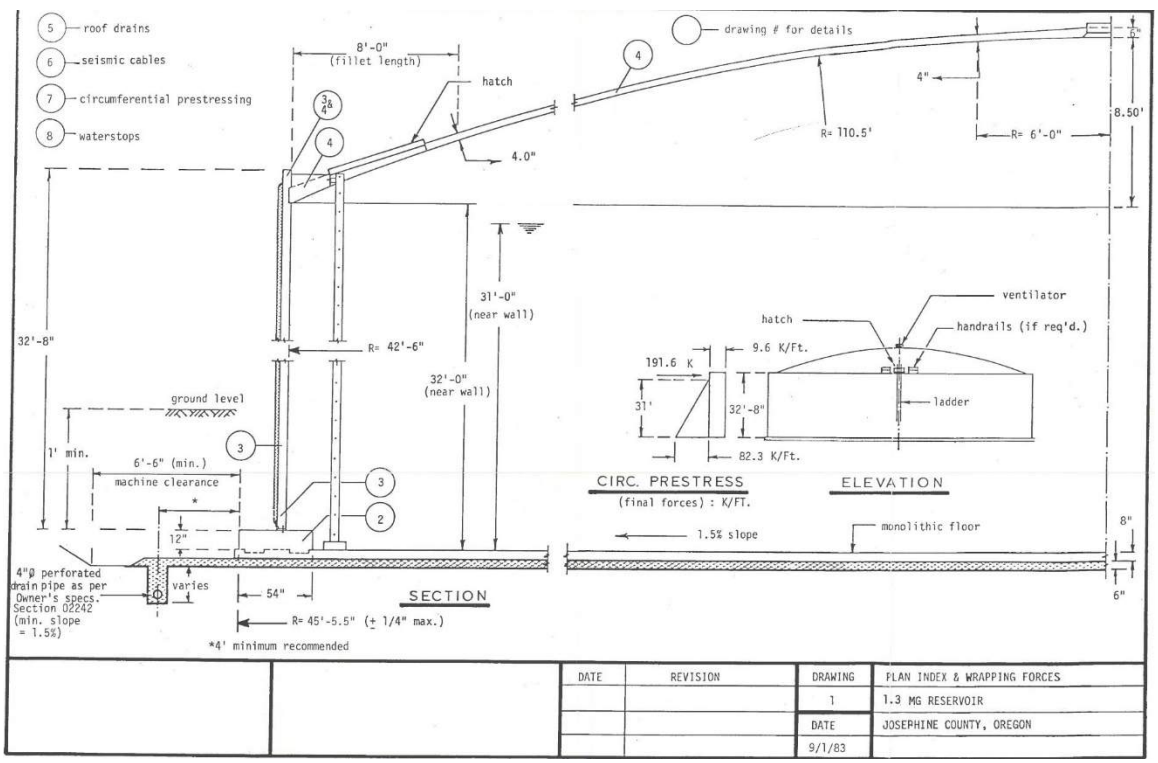


Figure 6-1: Reservoir Plan Index & Wrapping Forces

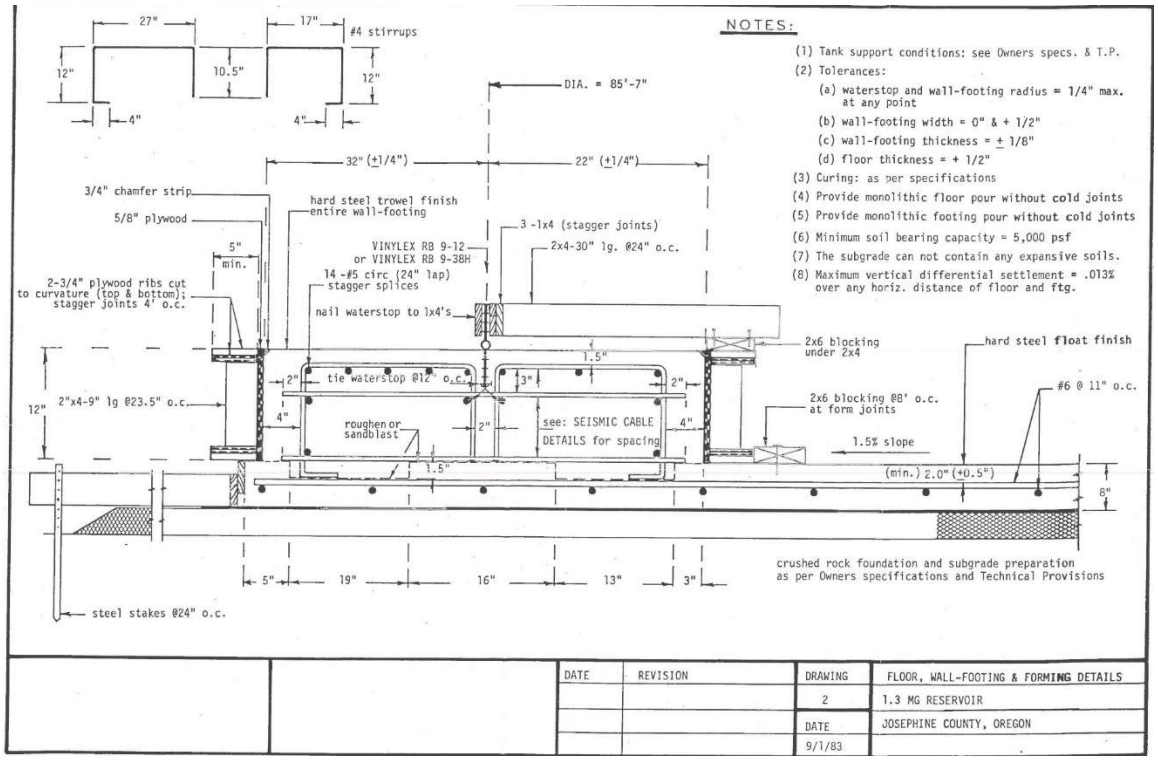


Figure 6-2: Floor, Wall-Footing & Forming Details

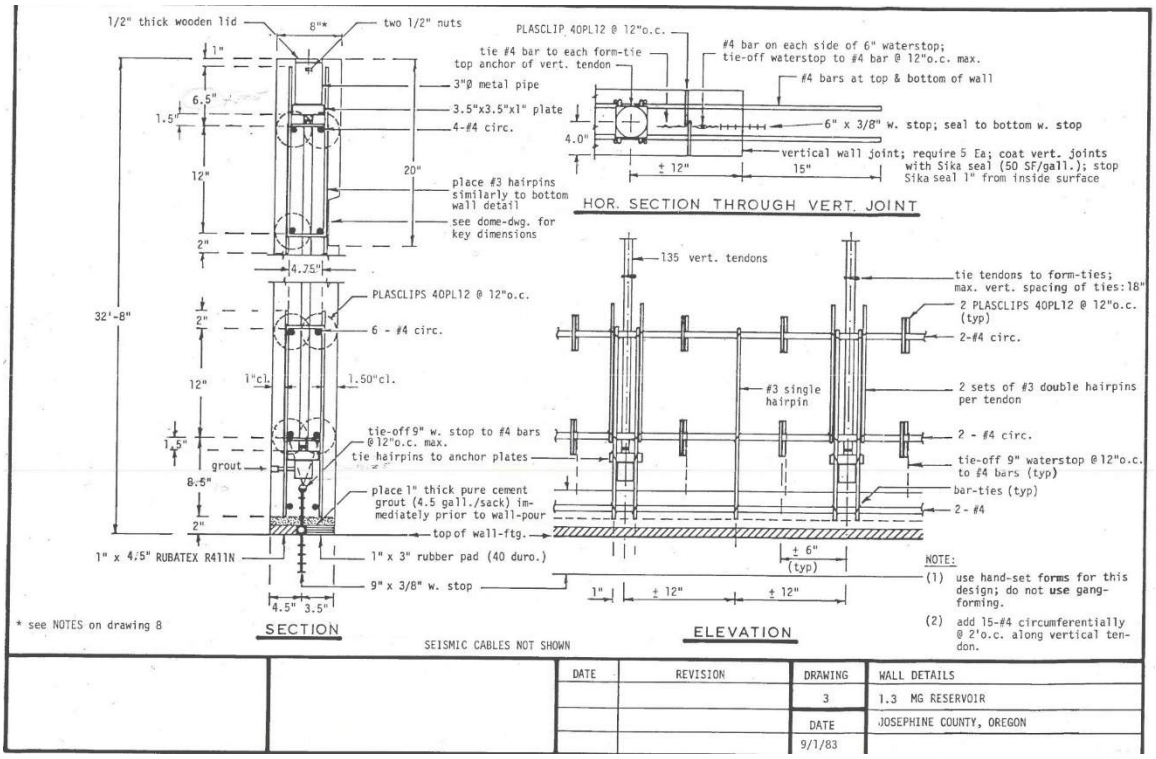


Figure 6-3: Wall Details

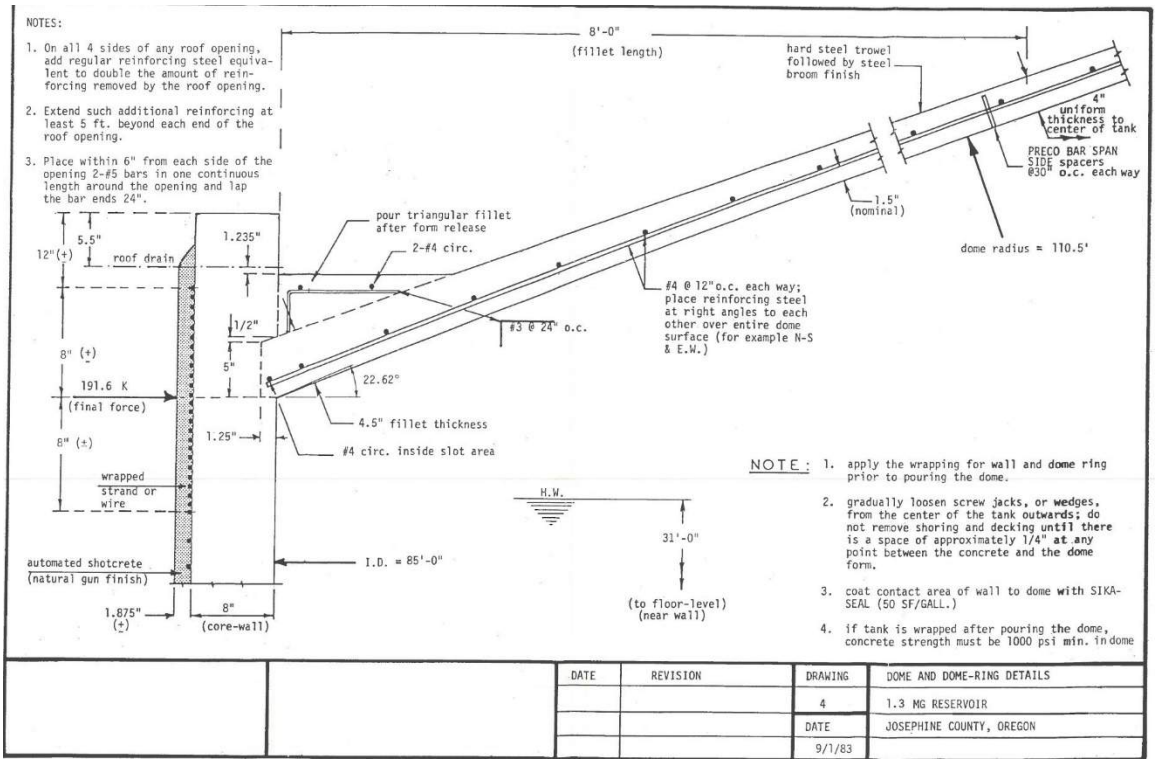


Figure 6-4: Dome & Dome Ring Details

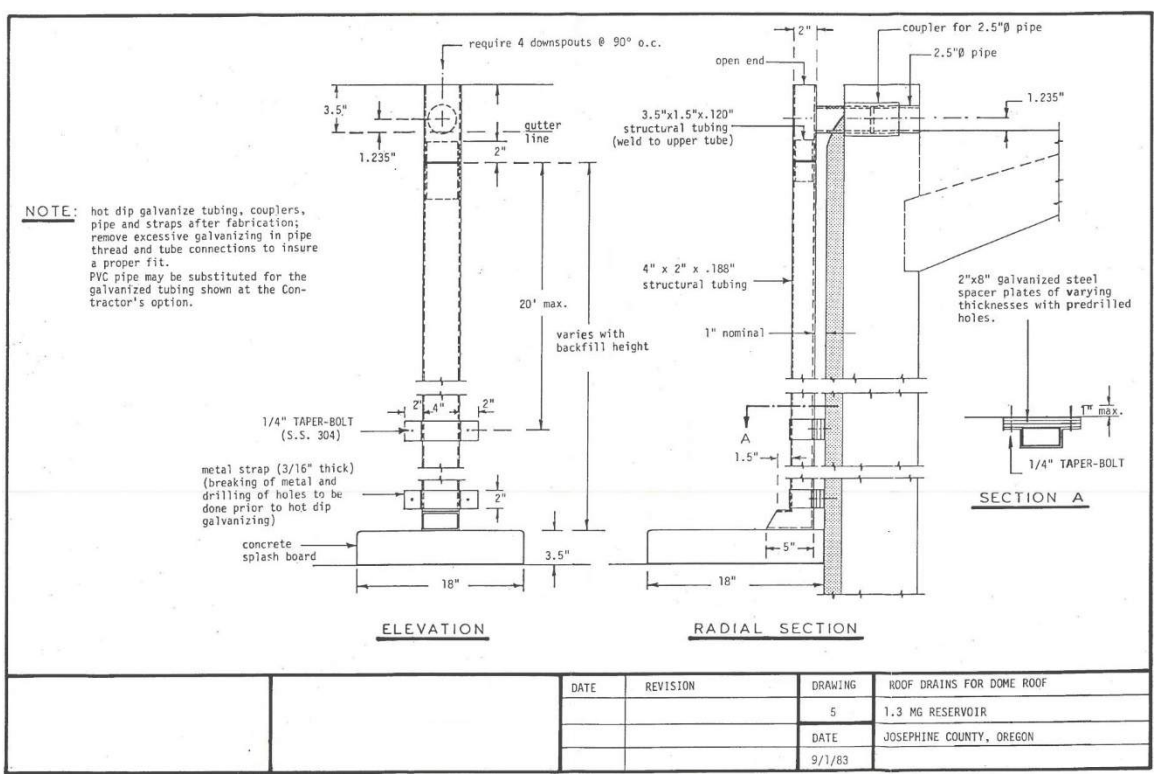


Figure 6-5: Roof Drains for Dome Roof

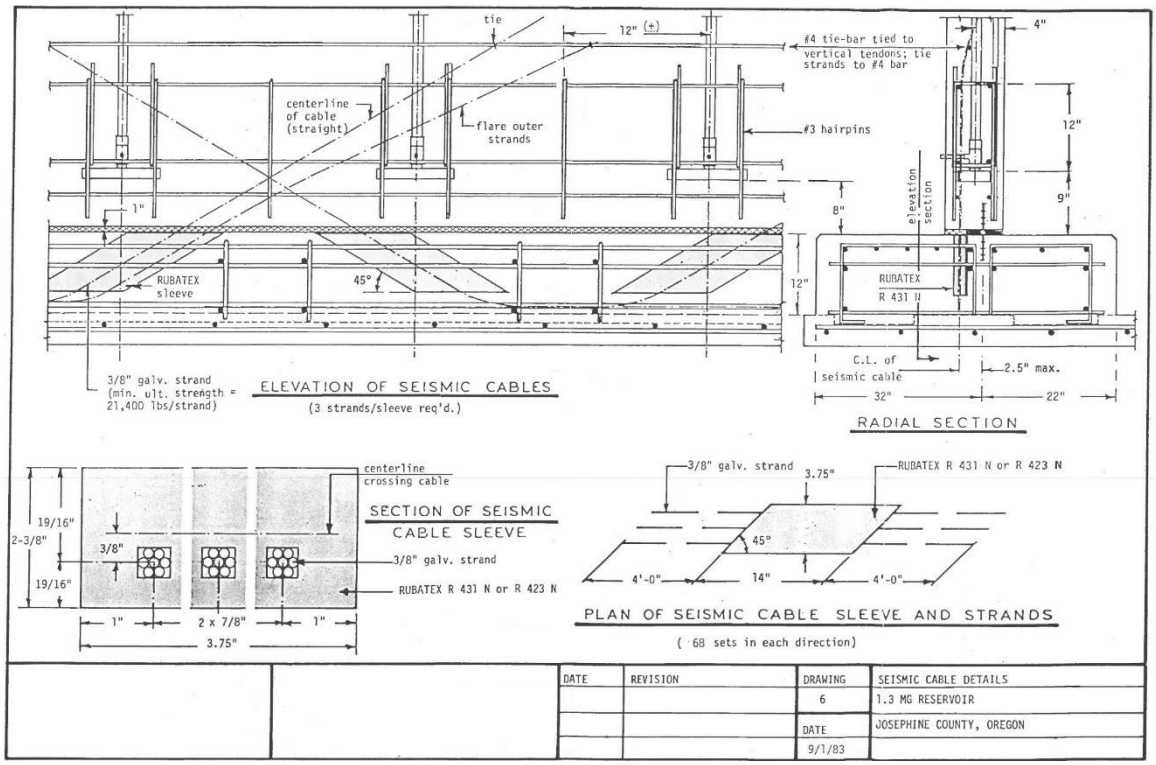


Figure 6-6: Seismic Cable Details

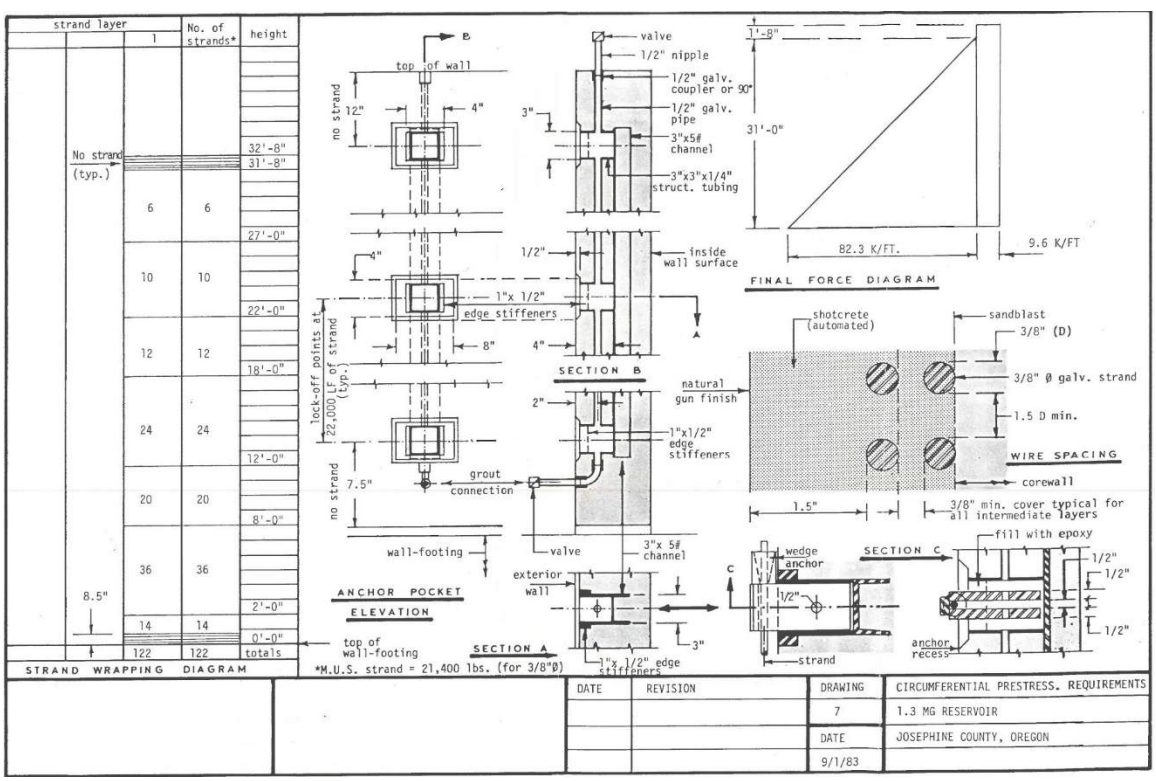
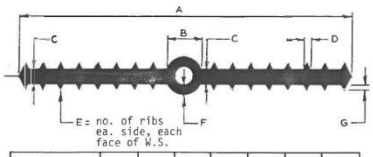


Figure 6-7: Circumferential Prestress Requirements



Location	A	B	C	D	E	F	G
Wall to wall-footing	9"	1"	3/8"	5/32"	8	1/4"	1/8"
Vertical wall joints*	6"	–	3/8"	5/32"	6	–	1/8"

* no centerbulb allowed for vertical wall joints

WATERSTOP SCHEDULE

WATERSTOPS (PVC).

a. Material requirements: see specifications.

b. Min. dimension and rib requirements: see "Waterstop Schedule" on this sheet.

c. Connect the ends of the radial waterstop in the wall footing joints to the circumferential waterstops in the wall-wall footing joints and to the waterstops in the floor-wall footing joints.

NOTES:

The integrity of the wall shown on these drawings is strictly predicated on the following conditions:

1) the strict conformance to the close stress-tolerance and other requirements in the circumferential pre-stressing application indicated in the Technical Provisions.

2) the following out of round tolerances from the true curvature specified at any point of the wall: 3/4" in 50'; 3/8" in 10' and 3/16" in 24"

3) there shall be no block-outs or other types of wall-openings other than those shown in these drawings.

DATE	REVISION	DRAWING	WATERSTOP SCHEDULE & MISC.
		8	1.3 MG RESERVOIR
		DATE	JOSEPHINE COUNTY, OREGON
		9/1/83	

Figure 6-8: Waterstop Schedule & Misc.

6.8 Observations Pictures

Figure 6-9: Reservoir Elevation

Figure 6-10: Top of Footing & Reservoir Wall Base w/ Typical Cracking

Figure 6-11: Wall Base w/ Sealant Hole

Figure 6-12: Close-up of Hole in Wall Base Sealant

Figure 6-13: Gutter at Roof Drain (Typical each Quadrant)

Figure 6-14: Exterior Ladder

Figure 6-15: Hatch

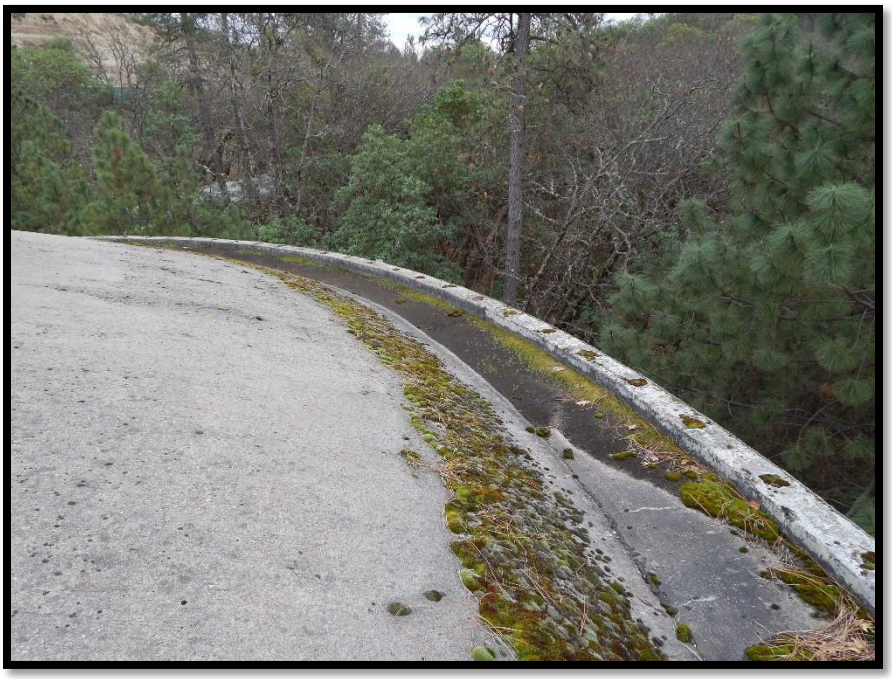


Figure 6-16: Dome Heel & Parapet w/ Fillet

Figure 6-17: Drain Scupper (nearly clogged) & Top of Wall

Figure 6-18: Vent at Center of Dome

END OF SECTION

7 Lawnridge Pump Station

7.1 Description & Background

Based upon the documents provided by the City the Lawnridge pump station was constructed circa 1969. The pump station is approximately rectangular, roughly 40'x16' in plan with an additional bump out for the bathroom entrances that serve the adjacent park. More than half of the building is dedicated to the pump station area with an additional room that provides storage for chlorine. The remaining portion of the building consists of bathrooms for the adjacent park and a storage closet for the Parks Department. The storage closet could not be accessed during our observation.

The construction is 8" CMU that is partially grouted and lightly reinforced per the original drawings. The walls are 8'-1.75" tall with a gable roof supported by glulam beams that are supported on posts above the CMU walls. Between the posts, the walls are enclosed by light frame wood framing. The foundation is a standard concrete strip footing located below all the CMU walls. An additional concrete sump runs down the length of the pump station portion of the building.

7.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the pump station on February 13th, 2018. At this time both the interior and exterior of the structure were observed, except the locked Parks Department storage closet, which was inaccessible.

The roofing is a built-up membrane and appears to be fairly old. There were no signs of leaks or damage to the roof, but it was in fair condition at best. There was a significant amount of organic debris from the adjacent trees that has fallen on the roof. In addition, an adjacent tree is growing against the roof edge is likely going to cause roof damage if it isn't removed or trimmed back. The roof decking appears to be redwood and appears to be in good condition. There were some trim boards that appeared loose and minor maintenance may be required.

The exterior of the pump station is clad in brick over the smooth faced CMU walls. The brick cladding appeared to be in good condition. Above the CMU/Brick the wood framed walls also appeared to be in good condition with no signs of deterioration. Sounding of the CMU walls shows that it does appear to be partially grouted in agreement with the original drawings provided. We were able to verify that anchor bolts are located at the top of the CMU wall in the exposed wall areas in the bathrooms in two locations. The spacing in these two locations were 40" & 48" o.c. which appears to be larger than shown on the original drawings. Other than these two isolated locations we were unable to observe the interior of the wood frame walls above the CMU as they are enclosed on both sides by finishes.

The interior of the bathrooms and chlorine room did not show any notable structural concerns and appears to be in fair condition. The interior of pump station area is the largest room and contains the pump station equipment. It is unclear if the electrical equipment is anchored. The pumps and other equipment appear to be anchored. It was noted that areas of the concrete pump pedestal/bases have some areas of cracking and spalling. In addition, the base of the CMU walls showed the signs of damage from chlorine exposure in some locations.

7.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the pump station under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Society for Civil Engineers, "Seismic Evaluation and Retrofit of Existing Buildings" code ASCE 41-13 was utilized as it provides a framework for performing structural evaluation of existing buildings that do not conform to current code requirements. ASCE41 provides an alternate procedure to analyze nonconforming buildings that is considered equivalent to current code.

7.3.1 Gravity Analysis

The roof decking and roof beams were analyzed using current codes. The roof live load of 40 psf given in the general notes of the construction documents was used in lieu of the now-required 25 psf snow load. The roof decking was called out in the drawings as 3" (thick) redwood tongue and groove (T&G), which was verified during the visual condition assessment. The roof beams were called out as 5"x11.375" DF/DF glulam beams. Both the decking and the roof beams meet the strength requirements of the current code.

They should also meet current deflection criteria, assuming that they are continuous across their supports. If, however, either element is simply supported between each support, then the deflection criteria may be exceeded and there could be potential deflection issues in the future, especially if the neighboring tree is continued to allow depositing debris on top of the roof. No signs of deflection issues were observed during our site visit.

The footings were verified using the construction documents as visual verification was not possible. Assuming 3,000 psi concrete and 40 ksi reinforcing steel, the wall strip footings appear to be adequate to resist gravity loads under current code conditions. Additionally, the soil bearing is under the current code-allowable pressure of 1,500 psf.

7.3.2 Lateral Analysis

Lateral checks were performed in accordance to the Tier 1 provisions of ASCE 41. Based on the information available, the building type was assumed to be a reinforced masonry building with flexible diaphragms. The most stringent performance level for the structural components of a building under a Tier 1 investigation is Immediate Occupancy (IO), which is applicable for pump stations. The level of seismicity was determined to be high. These three criteria dictate which checklists are required for the Tier 1 Evaluation. The checklists are broken out into their own sections below, but the only items that were listed are the ones that are either noncompliant or unknown. For reference, each item on each checklist must be deemed either compliant, noncompliant, not applicable, or unknown.

<u>Immediate Occupancy Basic Configuration Checklist:</u> Most items on this checklist were not applicable to the pump station being evaluated. The only noncompliant items are the lack of continuous load path, and the vertical irregularities that result from this. The issues with both arise from the fact that the roof diaphragm does not connect directly to the masonry walls and, instead, sits on top of a wood stud wall

that does not have adequate anchorage to transfer the shear and uplift forces from the diaphragm to the masonry walls. Note, that these present a potential collapse hazard in a seismic event.

Immediate Occupancy Structural Checklist: The items on this list correspond with the items mentioned in the Basic Configuration Checklist. The first noncompliant item is the ability of the diaphragm to transfer shear to the masonry shear walls, which is addressed in the previous section. The next noncompliant item is the required wall anchorage to transfer out-of-plane (OOP) wall forces to the diaphragm. A positive connection (anchor, strap, tie, etc.) is required between the OOP walls and the diaphragm. The purpose of this connection is for the diaphragm to provide lateral support to these walls. Without these connections, separation and even roof collapse may occur in a seismic event.

7.4 Summary

The building is in fair condition overall and requires several upgrades to extend its service life and to make it compliant with current code seismic requirements. From a structural standpoint, the items that need to be upgraded are the in-plane and out-of-plane connections at the top of the walls. The lack of adequate load path from the roof to shear walls provides a potential collapse hazard for the structure in a seismic event. Additionally, the tree adjacent to the building needs to be cut back and periodically monitored to ensure that it does not adversely impact the roof.

7.5 Recommendations

If it is intended to extend the service life of this pump station, then several upgrades would be necessary to make this building code-compliant. The issues indicated in the summary should be addressed in order to bring this pump station up to code and to extend its service life.

7.6 Field Notes

Pump Station Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	February 13, 2018	**Observation Time**	10.30 am (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Overcast 45 degrees		

General Information

Structure	Lawnridge Pump Station
Address	Hawthorne Ave.
Date Constructed (if known)	1969, original drawings
Date Retrofitted (if any)	n/a
Original Design Code	1967 UBC (Assumed), Predates state building code adoption
Building Construction Type	CMU w/ Flexible wood roof diaphragm
Size	1,200 square feet (approx.)
Wall Height	8’-1.75” to top of CMU
Adjacent Structures (?)	None

General Notes – Could not access storage room that is used by Park Department as it is locked from the outside by a separate key.

Exterior Inspection Information

Site Slope	Flat
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), flat site
Foundation Condition	NA, not exposed, no signs of settlement
Roof Condition	Composite/membrane style roof Fair-not quite poor condition (appears fairly old) No signs of any leaks Lots of leaf litter and tree debris, adjacent tree needs to be cut back as it is impacting roof edge Wood trim loose in some areas
Wall Condition	CMU appears partially grouted Some Chlorine damage in areas Anchor bolts appear to be 0.625" diameter and vary 40-48" o.c. in the two locations where it could be observed in the bathrooms Remaining tops of walls are enclosed and not observable
Siding/Cladding	Brick Cladding over CMU appears to be in good condition

Interior Inspection Information

Equipment	Unable to determine if electrical equipment is anchored Pipe support cracked at one location, concrete spalled
General Observation	CMU walls have wood framed walls above, likely seismic deficiency for out-of-plane hinging Roof decking is redwood and appears to be in good condition, no signs of leaks or damage

7.7 Scans of Select Construction Documents

GENERAL

G1 SCOPE
THE NOTES AND DETAILS ON THIS SHEET ARE GENERAL AND APPLY TO THE ENTIRE PROJECT EXCEPT WHERE THERE ARE SPECIFIC INDICATIONS TO THE CONTRARY.

G2 APPLICABLE SPECIFICATIONS AND CODES
CONSTRUCTION SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF THE UNIFORM BUILDING CODE. THE ABOVE SHALL COVERN EXCEPT WHERE OTHER APPLICABLE CODES OR THE FOLLOWING NOTES ARE MORE RESTRICTIVE.

G3 ALTERNATIVE DESIGNS
THE STRUCTURAL SYSTEMS AND DETAILS ON THESE PLANS ARE THE PRIORITY DESIGN. ALTERNATE SYSTEMS AND DETAILS MAY BE USED IF THE CONTRACTOR SUBMITS PLANS WITH SUBSTANTIATING CALCULATIONS AND TEST DATA, AND IF THE ALTERNATIVE PLANS ARE ACCEPTED BY THE ENGINEER.

G4 DIMENSIONS
STRUCTURAL DIMENSIONS CONTROLLED BY OR RELATED TO MECHANICAL OR ELECTRICAL EQUIPMENT SHALL BE VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

G5 PROVISIONS FOR EQUIPMENT
MECHANICAL AND ELECTRICAL EQUIPMENT SUPPORTS, ANCHORAGES OPENINGS, RECESSES AND REVERALS NOT SHOWN ON THE STRUCTURAL DRAWINGS BUT REQUIRED BY OTHER CONTRACT DRAWINGS SHALL BE PROVIDED PRIOR TO CASTING CONCRETE.

G6 CONSTRUCTION LOADS
STRUCTURES HAVE BEEN DESIGNED FOR OPERATIONAL LOADS ON COMPLETED STRUCTURES. DURING CONSTRUCTION, STRUCTURES SHALL BE PROTECTED BY BRACING AND BALANCING WHEREVER EXCESSIVE LOADS MAY OCCUR.

G7 DRAINAGE SURFACES
SLOPE DRAINAGE SURFACES UNIFORMLY TO DRAIN. SLOPE SHALL BE 1-4-1%, PER FOOT EXCEPT WHERE NOTED OTHERWISE ON THE PLANS.

G8 FLOOR DRAINS
SEE ARCHITECTURAL AND MECHANICAL DRAWINGS FOR LOCATIONS AND SIZES.

STRUCTURAL

S1 DESIGN CODE
DESIGN IS IN ACCORDANCE WITH THE LATEST EDITION OF THE UNIFORM BUILDING CODE EXCEPT WHERE OTHER APPLICABLE CODES OR THE FOLLOWING NOTES ARE MORE RESTRICTIVE.

S2 DESIGN LIVE LOADS
A. FLOOR AREAS:
(1) PUMP ROOM - 200 PSF
(2) STORAGE - 250 PSF
(3) OTHER AREAS - 100 PSF
B. GRATINGS - SAME LOADINGS AS ADJACENT FLOOR AREAS
C. ROOF - 40 PSF
D. WIND - 20 PSF
E. SEISMIC LOADING - ZONE III, UNIFORM BUILDING CODE LATEST EDITION

S3 CONSTRUCTION JOINTS
LOCATION OF ALL CONSTRUCTION JOINTS SHALL HAVE THE APPROVAL OF THE ENGINEER. CONSTRUCTION JOINTS SHALL BE AS DETAILED ON THE DRAWINGS.

ALUMINUM

A1 APPLICABLE CODE
DESIGN AND CONSTRUCTION OF ALUMINUM STRUCTURES SHALL CONFORM TO "SUGGESTED SPECIFICATIONS FOR STRUCTURES OF ALUMINUM ALLOYS 6061-T6 AND 6061-T6" AMERICAN SOCIETY OF CIVIL ENGINEERS PROCEEDINGS PAPER NO. 3341, DECEMBER 1962.

A2 MATERIAL
UNLESS OTHERWISE INDICATED, STRUCTURAL ALUMINUM SHALL BE ALLOY 6061-T6 AS SPECIFIED IN ASTM B-308.

A3 ALUMINUM IN CONTACT WITH CONCRETE
WHERE ALUMINUM IS IN CONTACT WITH CONCRETE OR MASONRY SURFACES, CONTACT SURFACES SHALL BE COATED WITH HEAVY ALKALI-RESISTANT BITUMINOUS PAINT.

STEEL

S11 APPLICABLE CODE
STEEL CONSTRUCTION SHALL CONFORM TO SPECIFICATIONS AND STANDARDS PRESENTED IN THE LATEST EDITION OF AISC STEEL CONSTRUCTION MANUAL.

S12 MATERIAL
ALL STRUCTURAL SHAPES, BARS, PLATES AND SHEETS INDICATED ON THE DRAWINGS SHALL BE STEEL MEETING ASTM A-36

S13 WELDING
WELDING SHALL CONFORM TO AWS CODE FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION.

S14 ENCASED STEEL
STEEL COMPLETELY ENCASED IN CONCRETE SHALL NOT BE GALVANIZED OR PAINTED AND SHALL HAVE A CLEAN SURFACE FOR BONDING TO CONCRETE.

S15 PAINTING
STRUCTURAL STEEL SHALL BE PAINTED IN ACCORDANCE WITH SPECIFICATIONS.

CONCRETE

C1 APPLICABLE CODE
CONCRETE CONSTRUCTION SHALL CONFORM TO THE LATEST EDITION OF THE ACI BUILDING CODE (ACI-318).

C2 REINFORCING STEEL DETAILS
ALL DETAILING, FABRICATION, AND ERECTION OF REINFORCING BARS, UNLESS OTHERWISE NOTED SHALL BE IN ACCORDANCE WITH MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI-315), LATEST EDITION.

C3 DESIGN STRESSES
A. CONCRETE, $f_c' = 3,000$ PSI ULTIMATE COMPRESSIVE STRESS AT 28 DAYS, ($f_c' = 1,350$ PSI).
B. REINFORCING STEEL
TENSION DUE TO FLEXURE, INTERMEDIATE GRADE - 20,000 PSI

C4 CONCRETE COVER
CONCRETE COVER FOR REINFORCING BARS SHALL BE AS FOLLOWS WITH MINIMUM COVER OF ONE BAR DIAMETER.
A. FOOTINGS AND FOUNDATION MATS CAST ON GROUND - 3".
B. CONCRETE TO BE IN CONTACT WITH GROUND OR WEATHER.
(1) AT BARS GREATER THAN #5 - 2"
(2) AT BARS #5 OR LESS - 1-1/2"
C. CONCRETE NOT TO BE EXPOSED TO GROUND, WEATHER, OR WATER. SLABS AND WALLS - 1".

C5 DOWELS
DOWELS SHALL BE THE SAME SIZE AND SPACING AS BARS WITH WHICH THEY ARE LAPPED. THE LAP EMBEDMENT SHALL BE 24 DIAMETERS MINIMUM FOR EACH.

C6 BAR SPLICES
VERTICAL REINFORCING BAR SPLICES SHALL HAVE AT LEAST 20 BAR DIAMETER LAP. ALL OTHER BAR SPLICES SHALL BE LAPPED AT LEAST 24 BAR DIAMETERS.

C7 RESTRICTED BAR ANCHORAGE
IN CASES WHERE REINFORCING BARS CANNOT BE EXTENDED AS FAR AS REQUIRED DUE TO THE LIMITED EXTENT OF THE ADJACENT CONCRETE STRUCTURE, THE BARS SHALL EXTEND AS FAR AS POSSIBLE AND END IN STANDARD HOOKS.

C8 STANDARD HOOKS
BARS ENDING IN RIGHT ANGLE BENDS OR HOOKS SHALL CONFORM TO THE REQUIREMENTS OF PART 800, ACI-318.

C9 SLOPING SLABS
MONOLITHIC SLABS WITH TOPS THAT ARE SLOPED SHALL HAVE BOTTOMS SLOPED THE SAME AMOUNT, MAINTAINING A UNIFORM SLAB THICKNESS.

C10 CHAMFERS
EXCEPT AS OTHERWISE REQUIRED, EXPOSED CONCRETE CORNERS AND EDGES SHALL HAVE 3/4" CHAMFERS. RE-ENTRANT CORNERS SHALL NOT HAVE FILLETS.

C11 ANCHOR BOLTS
USE OF ANCHOR BOLTS SHALL BE GOVERNED BY THE FOLLOWING TABLE:

BOLT DIAMETER	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
MIN. EMBEDMENT IN STRUCTURAL CONCRETE	4"	4"	5"	6"	7"	8"	9"
ALLOWABLE SHEAR	750#	1000#	1500#	2000#	2500#	3000#	3500#

ALL BOLTS SHALL BE HOT-DIPPED GALVANIZED UNLESS OTHERWISE NOTED. EMBEDMENT LENGTH FOR BOLTS IN TENSION SHALL BE DETERMINED BY THE ENGINEER.

WOOD

W1 TIMBER. STRUCTURAL TIMER FOR DECKING SHALL BE COMMERCIAL HEART STRUCTURAL GRADE REDWOOD (1300f). ALL OTHER STRUCTURAL TIMBER SHALL BE COMMERCIAL DOUGLAS FIR CONSTRUCTION GRADE (1500f). ALL TIMBER SHALL CONFORM TO NATIONAL FOREST PRODUCTS ASSOCIATION SPECIFICATIONS, LATEST EDITION.

W2 GLUED LAMINATED MEMBERS. GLUED LAMINATED MEMBERS SHALL BE STRESS GRADE COMBINATION 'A' OR BETTER (2400f) DOUGLAS FIR DRIED TO MOISTURE CONTENT OF 8 TO 12% AND SHALL CONFORM TO LATEST EDITION OF W.C.L.A. SPECIFICATIONS, FOR THE ADHESIVES AND FABRICATION. SEE ARTICLE CX.0.0214 OF SPECIFICATIONS. ALL GLU-LAM BEAMS SHALL BE A.I.T.C. CERTIFIED.

W3 CUTS AND BORED HOLES. CUTS AND BORED HOLES IN JOISTS AND BEAMS SHALL NOT EXTEND DEEPER THAN 1/6 OF THE MEMBERS DEPTH FROM ITS TOP AND SHALL BE WITHIN THREE TIMES THE MEMBERS DEPTH FROM EITHER SUPPORT. PIPES OR CONDUITS THAT RUN THROUGH PLATES SHALL PASS THROUGH BORED HOLES THAT HAVE DIAMETER LESS THAN 1/3 THE PLATE WIDTH. WHERE NOTCHES OR HOLES ARE MADE IN OTHER PORTIONS OF THE MEMBER, THEY SHALL BE APPROVED BY THE ENGINEER PRIOR TO CUTTING.

W4 BOLTS, NAILS AND FITTINGS. ALL BOLTS, NAILS AND WOOD FASTENINGS SHALL CONFORM TO THE NATIONAL DESIGN SPECIFICATION FOR STRESS-GRADE LUMBER AND ITS FASTENINGS, LATEST EDITION. BOLT HEADS AND NUTS BEARING AGAINST WOOD SHALL BE PROVIDED WITH CUT WASHERS. NAILS SHALL BE GALVANIZED COMMON WIRE NAILS. THERE SHALL BE AT LEAST 2 NAILS IN EACH STRUCTURAL NAILED CONNECTION. NAILING SHALL BE AS SCHEDULED UNLESS OTHERWISE SHOWN ON THE DRAWINGS.

W5 MINIMUM NAILING SCHEDULE
3" DECKING TO BEARING - 2 - 30d NAILS PER BOARD
2 - 30d NAILS @ 24" O.C. @ PARALLEL SUPPORTS
JOIST TO SILL OR BEAMS - 2 - 16d TOENAIL EACH SIDE
STUD TO PLATES - 2 - 16d END NAIL OR
3 - 16d TOENAIL
TOP PLATES - SPIKED TOGETHER - 16d @ 16" OC
LAPS AND INTERSECTIONS 4 - 16d EACH SIDE
PLYWOOD TO STUDS, PLATES - 8d @ 6" OC @ PANEL EDGES
AND DECKING 8d @ 12" OC ON INTERMEDIATE SUPPORTS
CORNER STUDS AND ANGLES - 16d @ 24" OC
PLATE TO JOIST - 16d @ 24" OC

MASONRY WALLS

ALL CONCRETE BLOCKS SHALL BE CLASS U-1, CONFORMING TO ASTM C-90. BRICK UNITS SHALL BE GRADE MW, CONFORMING TO ASTM C-62. EDGES OF OPENINGS LARGER THAN 2'-0 IN EITHER DIRECTION SHALL HAVE 2-#4 BARS AROUND OPENINGS BENT 1'-6" INTO THE BOND BEAM. PROVIDE AT LEAST ONE VERTICAL BAR AT CORNERS AND INTERSECTIONS. LAP HORIZONTAL BARS 24 DIAMETERS AND VERTICAL BARS 40 DIAMETERS MINIMUM.

BROWN AND CALDWELL CONSULTING ENGINEERS SAN FRANCISCO · ALAMEDA, CALIFORNIA
CITY OF GRANTS PASS WATER SYSTEM IMPROVEMENTS
GENERAL STRUCTURAL NOTES
8 of 20 S1

Figure 7-1: General Structural Notes

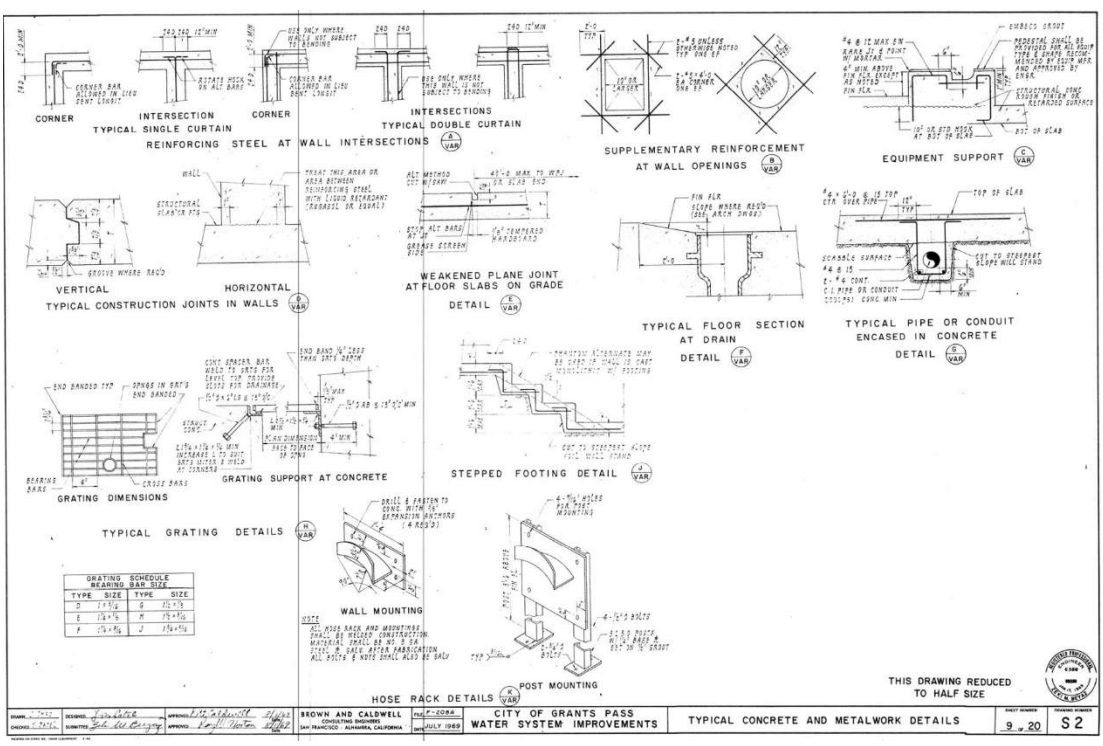


Figure 7-2: Typical Concrete & Metalwork Details

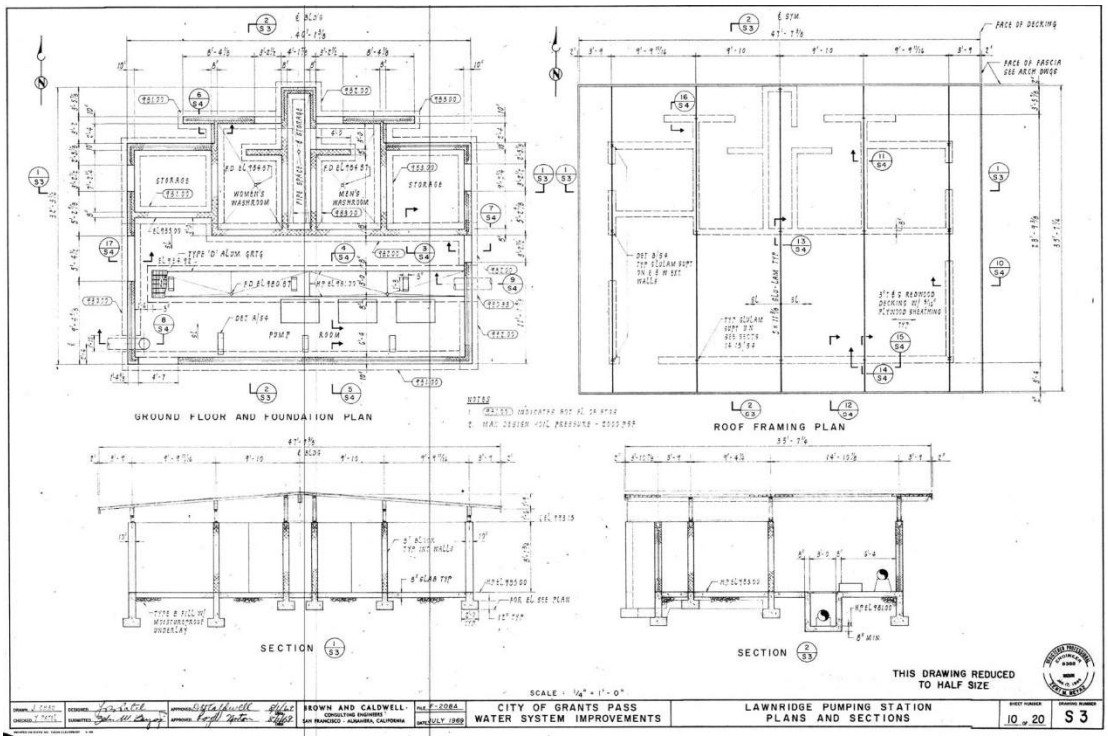


Figure 7-3: Plans & Sections

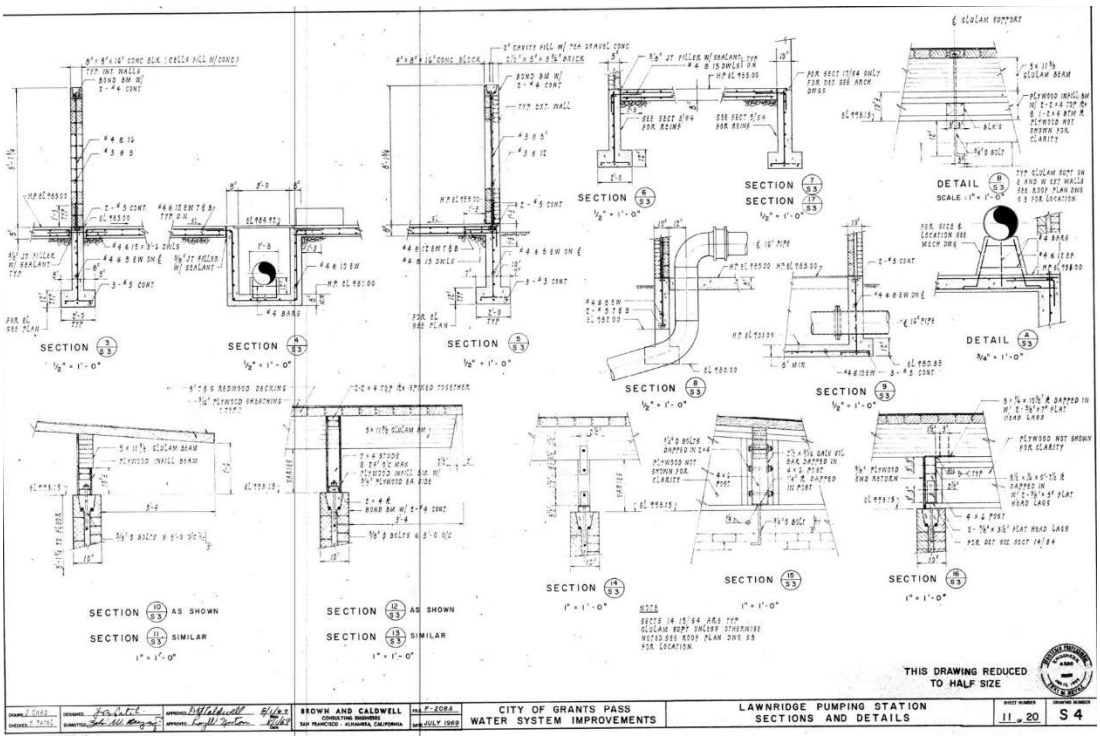


Figure 7-4: Sections & Details

7.8 Observations Pictures

Figure 7-5: Lawnridge Pump Station Exterior Elevation

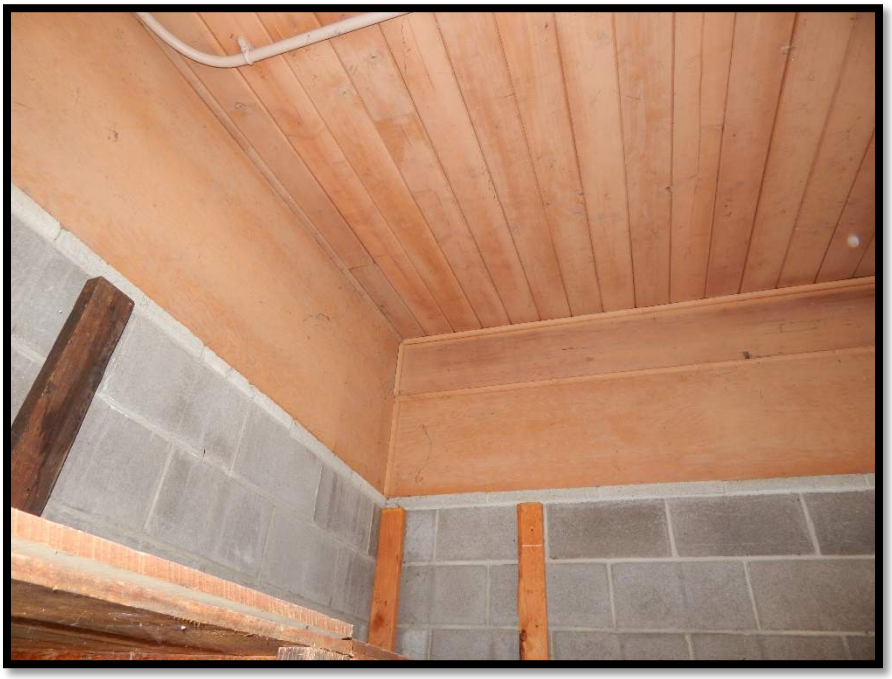


Figure 7-6: Underside of Roof Decking - Interior

Figure 7-7: Underside of Roof Decking - Exterior

Figure 7-8: Pump Station Interior

Figure 7-9: Cracked/Spalled Pump Base

Figure 7-10: Chlorine Damaged CMU & Cracked Pump Base

Figure 7-11: Roof w/ Organic Debris

Figure 7-12: Tree growing against edge of Roof

END OF SECTION

8 Madrone Pump Station

8.1 Description & Background

Based upon the documents provided by the City the Madrone pump station was constructed circa 1954. The pump station is rectangular, 25.5'x11.5' in plan. The construction is entirely cast-in-place reinforced concrete. The walls are 6" thick and the roof slab varies in thickness as it slopes to a central drain. The walls are 8'-6" tall and the roof has a very small parapet of a couple of inches. The foundation is a standard concrete strip footing located around the full perimeter. An additional concrete sump runs down the length of the pump station portion of the building.

8.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the pump station on February 13th, 2018. At this time both the interior and exterior of the structure were observed. The site generally slopes from East to West. The pump station site is generally flat, however, there is a retaining wall at/near the property line to the West face approximately 6' from the edge of the building. The retained height is fairly small, on the order of 1-2', but the grade then slopes downwards at gentle slope (assumed 2:1, Horiz:Vert) for another 3-4' before again flattening out.

It appears that the building has had some significant differential settlement. The West elevation appears to have settled around 1". The entry door on the North face is unable to be completely opened due to the settlement.

The roof is a built-up membrane type roof that we understand was recently added over the existing concrete roof. The City staff on hand indicated that it was installed over insulation added over the existing concrete deck. The membrane roof had some areas of ponding and biological growth in some areas, but appears to be in good condition. The roof drains to a central internal drain that penetrates through the roof slab and into a drainage pipe through the building volume below. The drain appears to be working, but may be susceptible to clogging so should be cleaned as need.

The exterior of the pump station has been painted several times. There is evidence of cracking in several locations, which can also be seen on the interior of the pump station walls in some areas, so they appear to transverse through the full thickness in some areas. Several of these cracks appear to have been repaired and/or painted over a few times. Much of the cracking can likely be attributed to the differential settlement of the structure.

The interior of the pump station shows the same type of cracking as shown on the exterior. The interior has also been painted so some cracking is likely obscured. Settlement on the East side of the building is evident at the wall to floor transition. In addition, the base of the walls show the signs of damage from chlorine exposure in some locations. The floor slab appears to have settled as well in relation to the exterior walls, as evident at the wall to floor joint and the pump base to wall joints that show signs of movement.

8.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the pump station under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Society for Civil Engineers, "Seismic Evaluation and Retrofit of Existing Buildings" code ASCE 41-13 was utilized as it provides a framework for performing structural evaluation of existing buildings that do not conform to current code requirements. ASCE41 provides an alternate procedure to analyze nonconforming buildings that is considered equivalent to current code.

8.3.1 Gravity Analysis

The roof slab was analyzed using current code requirements. Since very little design information was provided on the construction documents, assumptions had to be made regarding concrete strength (3,000 psi), reinforcing strength (40 ksi), and clear cover (2"). The minimum slab thickness (4.8") was used and the slab was assumed to be one-way and simply supported in the short direction. Since the slab thickness varies for drainage purposes, the maximum thickness was used to determine the design dead load. The roof live/snow load was assumed to be 25 psf. With these assumptions, the roof slab appears to be adequate. However, the actual condition of the roof appears to be worse than the analysis implies, which may be directly related to the differential settlement that has taken place below the exterior walls.

The strip footings below the exterior walls appear to satisfy the presumptive soil values provided by current code but cannot be considered code-compliant due to lack of reinforcing in the footings. The available structural drawings show no reinforcing in the footings, but the details do callout the reinforcing in the stem walls, which seems to indicate that this was a conscious omission, which would be fairly common for the period of construction.

8.3.2 Lateral Analysis

Lateral checks were performed in accordance to the Tier 1 provisions of ASCE 41. Based on the information available, the building type was assumed to be a reinforced concrete building with a stiff diaphragm. The most stringent performance level for the structural components of a building under a Tier 1 investigation is Immediate Occupancy (IO), which is applicable for pump stations. The level of seismicity was determined to be high. These three criteria dictate which checklists are required for the Tier 1 Evaluation. The checklists are broken out into their own sections below, but the only items that were listed are the ones that are either noncompliant or unknown. For reference, each item on each checklist must be deemed either compliant, noncompliant, not applicable, or unknown.

<u>Immediate Occupancy Basic Configuration Checklist:</u> Most items on this checklist were not applicable to the pump station being evaluated. There were no noncompliant or unknown items for this checklist

<u>Immediate Occupancy Structural Checklist:</u> There were several items in this checklist that were deemed noncompliant. The first noncompliant item is the lack of secondary gravity system in case the exterior shear walls are incapable of resisting simultaneous gravity and lateral loads. Preliminary analysis of the worst-case wall shows that the walls do not meet this criterion. If the primary load bearing walls fail and

there is no secondary system in place to resist the weight of the roof above, then the chance of roof collapse is greatly increased in the case of a seismic event.

The next noncompliant item is the lack of stirrups in the header above the entrance. Since this portion of the wall acts as coupling beam between the shear walls on either side of the opening. Potential issues associated with this lack of proper reinforcing include falling debris from the crumbling header and decreased stability for the walls connected by the beam.

8.4 Summary

Due to age, settlement, and historic style detailing, there are several items in this building that do not meet current code requirements and would require upgrades to bring it into conformance. If the life of this pump station is to be extended, then the cause of the settlement should be determined and mitigated, if possible.

Because of the construction type, it may be difficult to perform a seismic retrofit that brings this building up to code in a cost-effective manner.

8.5 Recommendations

Given the extent of the issues facing this pump station, significant upgrades would be required to retrofit the building and to make it code-compliant. Due to the stringent upgrade requirements, it may be more economical to replace the existing building with one that is code-compliant. For either case (retrofit or replacement), the settlement issue would need to be addressed in order to avoid additional future problems.

8.6 Field Notes

Pump Station Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	February 13, 2018	**Observation Time**	11.30 am (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Overcast 45 degrees		

General Information

Structure	Madrone Pump Station
Address	Madrone Street & Beacon Drive
Date Constructed (if known)	1954, original drawings
Date Retrofitted (if any)	n/a
Original Design Code	1952 UBC (Assumed), Predates state building code adoption
Building Construction Type	Reinforced Concrete (Shear walls & Diaphragm)
Size	300 square feet (approx.)
Wall Height	8’-6”, slab to underside of roof slab
Adjacent Structures (?)	None

General Notes –

Exterior Inspection Information

Site Slope	Generally flat, gentle slope East to West, retaining wall 6' from West wall (presumed Property line), approx. 1.5' tall then 2:1 slope for 3-4' before gentle slope again
Liquefaction Potential (?)	Unknown (assumed no)
Slope Stability (?)	Unknown (assumed no), flat site
Foundation Condition	NA, not exposed, significant differential settlement at West side of approximately 1", door won't fully open, cracking.
Roof Condition	Composite/membrane style roof Good condition No signs of any leaks Some moss and other debris Some minor ponding Center drain (likely prone to clogging)
Wall Condition	Lots of cracking, some repaired, and painted over so likely can't see full extent Some Chlorine damage in areas
Siding/Cladding	NA, exposed concrete, painted

Interior Inspection Information

Equipment	Unable to determine if electrical equipment is anchored
General Observation	Floor slab appears has settled as well in relation to the exterior walls, evident at pump base to wall locations and floor to wall joints

8.7 Scans of Select Construction Documents

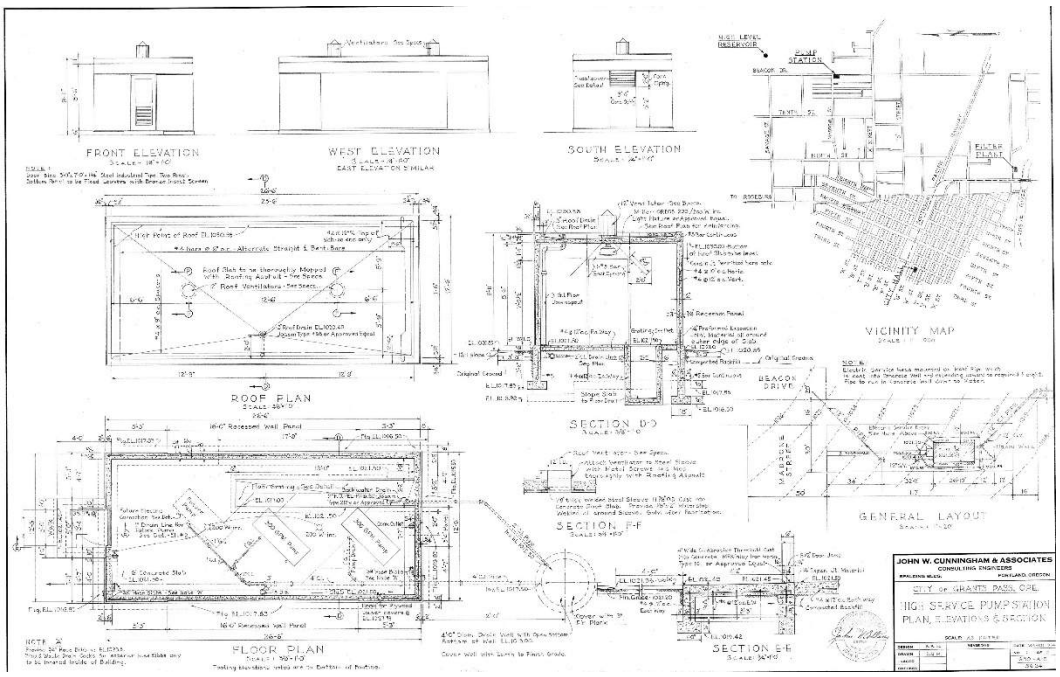


Figure 8-1: Plans, Elevations & Sections

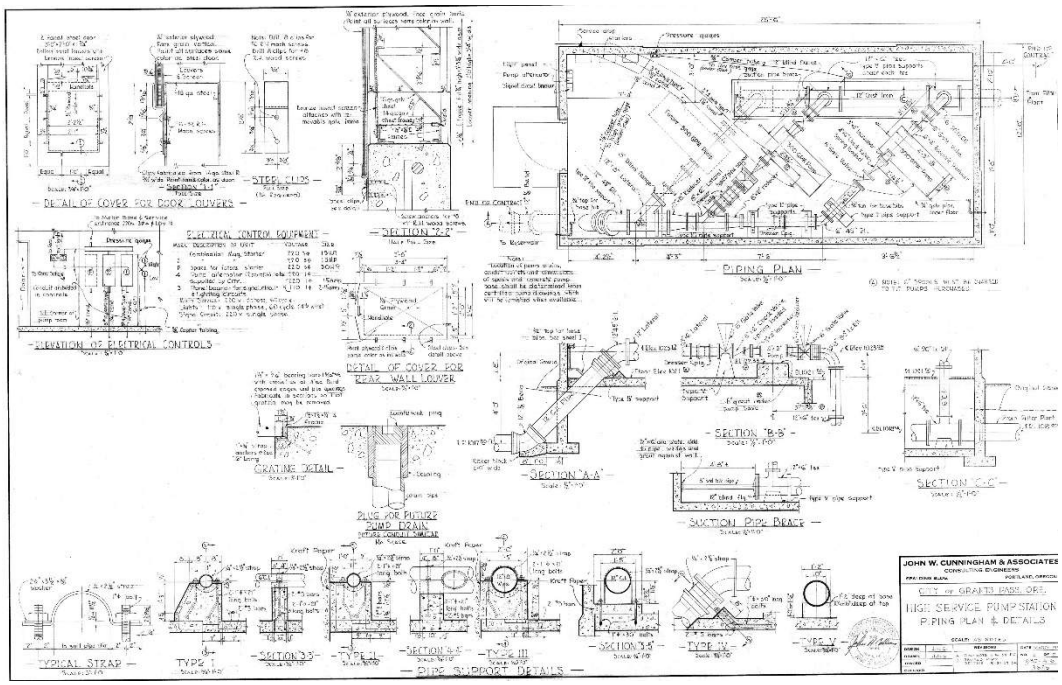


Figure 8-2: Piping Plan & Details

8.8 Observations Pictures

Figure 8-3: North Elevation

Figure 8-4: Typical Cracking at Exterior

Figure 8-5: Roof Ponding & Central Drain

Figure 8-6: Repaired Interior Cracking

Figure 8-7: Differential Settlement at Pump Base

Figure 8-8: Differential Settlement at Wall/Floor Slab

Figure 8-9: Concrete Damage

END OF SECTION

9 New Hope Pump Station

9.1 Description & Background

Based upon the documents provided by the City the New Hope pump station was constructed circa 2000; the 'as-built' drawings are dated July 2000. The pump station is rectangular, 42'x30' in plan.

The construction is reinforced CMU walls, partially grouted with vermiculite insulation, and a light framed wood premanufactured truss roof. The walls are 8" CMU blocks in running bond and are 12'-0" tall. The roof is a simple gable roof with attic access through an interior hatch and metal roofing over plywood sheathing. The foundation is a standard concrete strip footing located around the full perimeter.

On the interior of the pump station several concrete chases run down the length of the pump station along the interior. A crane rail runs along the approximate center of the building located above the pump locations. The crane is posted with a 6,000 lb load rating.

9.2 Visual Condition Assessment

We performed site visits to observe the as-built current condition of the pump station on February 13th, 2018. At this time both the interior and exterior of the structure was observed. In general, the building appears to be in very good condition as would be expected for its age.

The roofing is standing seam metal roof decking over plywood and appears to be in good shape. We accessed the 'attic' space within the premanufactured trusses and there were no obvious signs of leaks. It appears that at some point there was entry by birds, but it is our understanding from City staff on site that the entry point has been sealed. The 'as-built' drawings show gable end bracing and are noted at 18" o.c., however, field observations found gable end braces at only quarter points. In addition, the original drawings appear to show A35 clips at 48" o.c. and the as-built's call out clips at 24" o.c. We inspected a couple of locations and found the end wall clips appear to be at 48" o.c. Due to space constraints we were unable to observe the clips at the sidewalls and have assumed they match the original drawings.

The interior of the pump station appears to be in good condition with few notable structural items observed. It was observed at the flow meter that was post installed and through bolted to the wall that the CMU has spalled at the bolt locations behind the brackets, but this appears to be a cosmetic issue. The small pressure tank at the Southern corner does not appear to be anchored.

A self-serve water station was added to the pump station at the South elevation at some point. To facilitate this a penetration was cut through the South wall near the West corner. The penetration is approximately 21.5"x19" and is located 40" above the footing and 30" from the corner.

Along the exterior, the CMU appears to be in good condition. The only item of note was the joint sealant at the expansion joints in the CMU walls is cracking and may need maintenance.

9.3 Structural Analysis

The structural analysis consisted of seismic and gravity load analysis of the structural elements of the pump station under the most currently available code standards. Seismic loads were determined using the American Society for Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). In addition, the American Society for Civil Engineers, "Seismic Evaluation and Retrofit of Existing Buildings" code ASCE 41-13 was utilized as it provides a framework for performing structural evaluation of existing buildings that do not conform to current code requirements. ASCE41 provides an alternate procedure to analyze nonconforming buildings that is considered equivalent to current code.

9.3.1 Gravity Analysis

The manufactured truss roof was not analyzed but, given the roof's age and its condition during the visual assessment, the trusses should be sufficient to resist the design gravity loads provided in the construction drawings. Similarly, the details for the CMU lintels indicate that, if built to the specified design, they should also be more than sufficient to resist the design gravity loads.

The only elements that were analyzed in the gravity system were the strip footings. These footings were analyzed using the design loads provided in the construction drawings. If the footings were detailed correctly, then they should have more than enough capacity to resist the given design loads. Additionally, the actual bearing pressure from the design loads should be significantly below the 2,000 psf allowable pressure given in the construction documents.

9.3.2 Lateral Analysis

Lateral checks were performed in accordance to the Tier 1 provisions of ASCE 41. Based on the information available, the building type was assumed to be a reinforced masonry building with flexible diaphragms. The most stringent performance level for the structural components of a building under a Tier 1 investigation is Immediate Occupancy (IO), which is applicable for pump stations. The level of seismicity was determined to be high. These three criteria dictate which checklists are required for the Tier 1 Evaluation. The checklists are broken out into their own sections below, but the only items that were listed are the ones that are either noncompliant or unknown. For reference, each item on each checklist must be deemed either compliant, noncompliant, not applicable, or unknown.

<u>Immediate Occupancy Basic Configuration Checklist:</u> No items on this checklist are noncompliant or unknown, which should be expected given the age of the building.

<u>Immediate Occupancy Structural Checklist:</u> The only item on this list that is noncompliant is the out-of-plane wall anchorage at the gable end walls. There should be an adequate positive connection between the roof trusses and the walls that the trusses bear on, but the end walls that are parallel to the trusses do not appear to have adequate braces. The purpose of this connection is for the diaphragm to provide lateral support to these walls. Without these connections, separation and even roof collapse may occur in a seismic event.

9.4 Summary

The building is in in good condition overall and should require only minor upgrades to extend its service life and to make it compliant with current code requirements. From a structural standpoint, the only item that needs to be upgraded are the out-of-plane bracing for the gable end walls. Additionally, it may be prudent to verify that the in-plane roof-to-wall connections are sufficient, especially at the gable end walls where only half of the specified angle clips were installed.

9.5 Recommendations

If the service life of this pump station is to be extended, then the issues indicated in the Summary should be addressed. The seismic upgrades required are minor and would require less time and fewer resources than replacing the building.

9.6 Field Notes

Pump Station Condition Assessment

Project Name	Grants Pass Water System Evaluations	**PSE Job #**	17-368
Client	Murraysmith	**Owner**	City of Grants Pass
Observation Date	February 13, 2018	**Observation Time**	9.30 am (approx.)
PSE Evaluator(s)	Travis McFeron, PE, SE		
Weather	Overcast 45 degrees		

General Information

Structure	New Hope Pump Station
Address	Williams Highway
Date Constructed (if known)	2000, 'as-built' drawings
Date Retrofitted (if any)	n/a
Original Design Code	1994 UBC (Assumed), based on 'as-built' date, 1997 UBC wasn't adopted until 2000, so likely built under the 1994 UBC
Building Construction Type	CMU w/ Flexible wood roof diaphragm
Size	1,260 square feet (approx.)
Wall Height	12' to top of CMU
Adjacent Structures (?)	None
General Notes – Full drawing set wasn't available on site, Sheets S201 & S202 were missing from the drawings provided. We subsequently located them in the full Reservoir set. A penetration was cut into the South elevation wall for a self-serve water station. The penetration is 40" above grade and located 30" from the West corner. The size of the penetration is 21.5" wide by 19" tall.	

Exterior Inspection Information

Site Slope	Flat
Liquefaction Potential (?)	Unknown (assumed no)

Slope Stability (?)	Unknown (assumed no), flat site
Foundation Condition	NA, not exposed, no signs of settlement
Roof Condition	Metal roofing – standing seam over 0.5" plywood Bottom of trusses have sheetrock Good condition No signs of any leaks
Wall Condition	CMU appears partially grouted with vermiculite insulation, could observe where the penetration for the self-serve water station was cut
Siding/Cladding	Split face CMU appears to be in good condition
Joints	Sealant at construction joint in CMU wall is cracked and may need repair in the near future, cosmetic/waterproofing not a structural issue.

Interior Inspection Information

Equipment	Unable to determine if electrical equipment is anchored
General Observation	Pressure tank at Southern corner doesn't appear to be anchored

9.7 Scans of Select Construction Documents

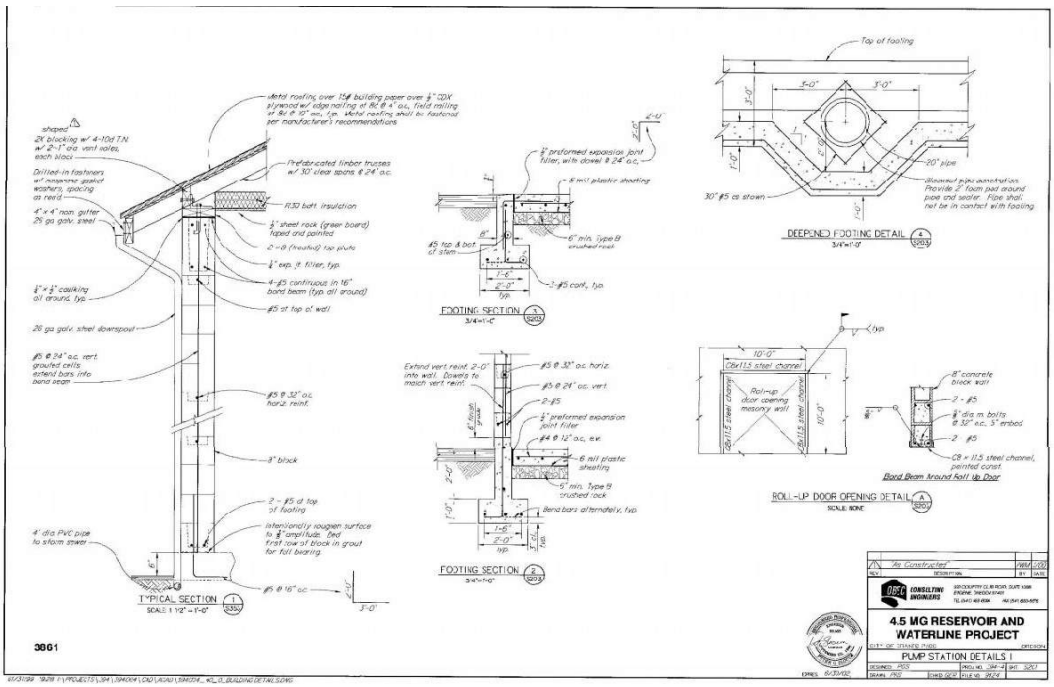


Figure 9-1: Pump Station Details 1

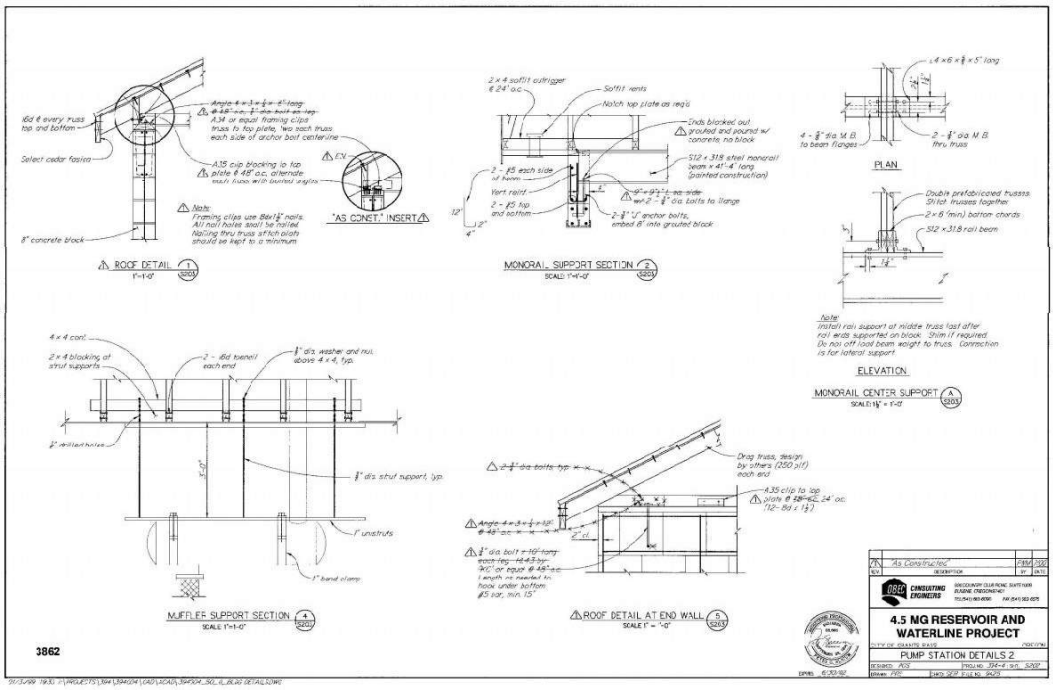


Figure 9-2: Pump Station Details 2

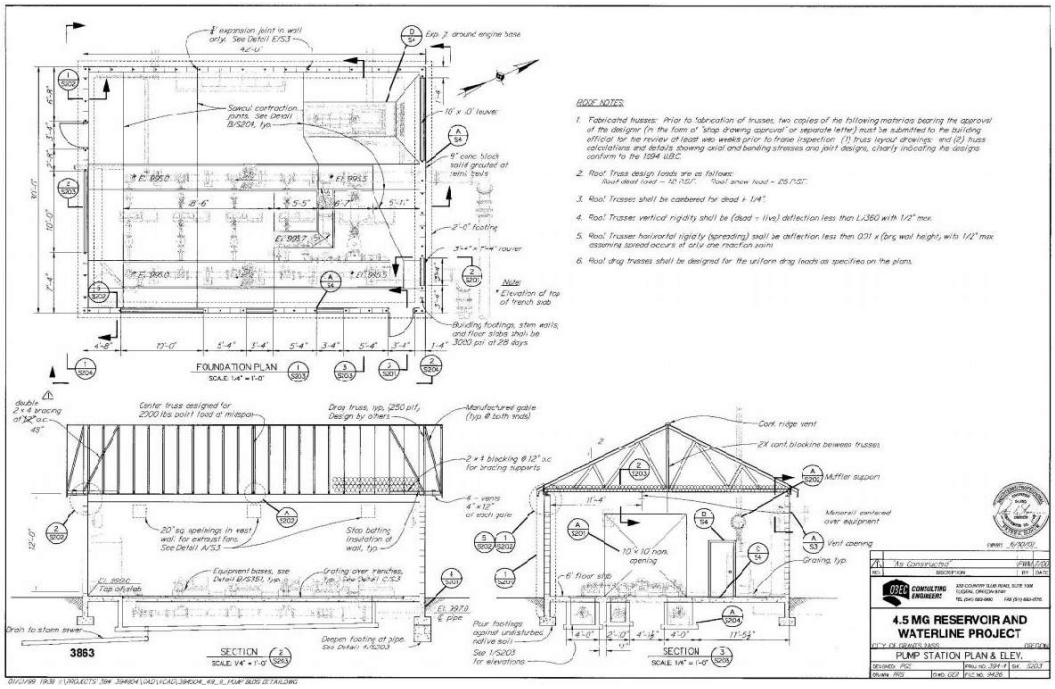


Figure 9-3: Pump Station Plan & Elevation

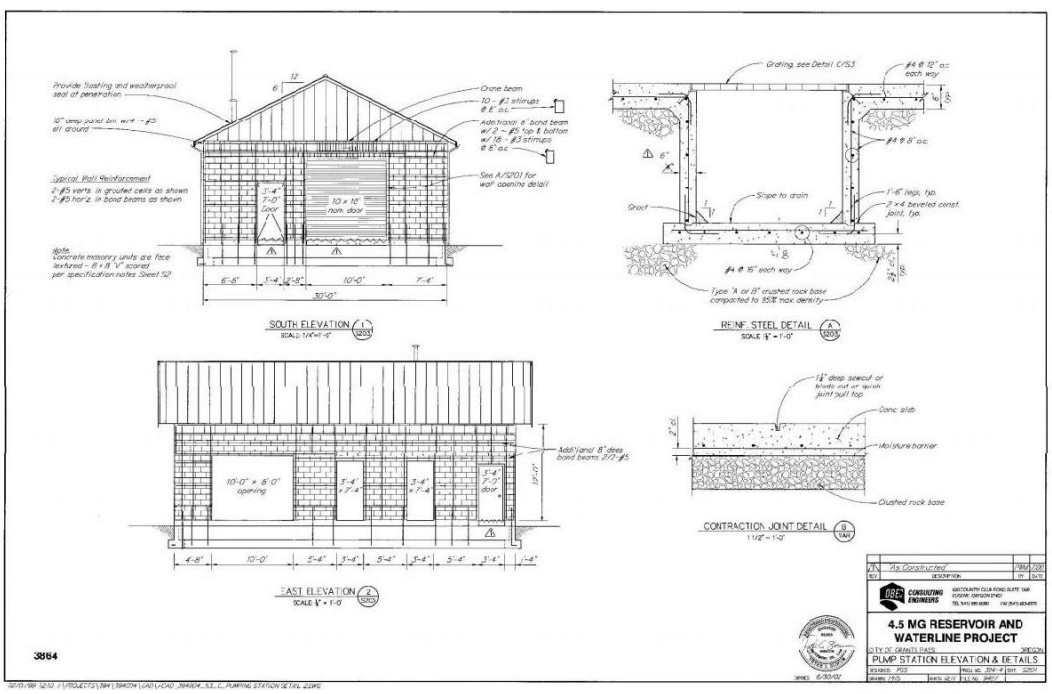


Figure 9-4: Pump Station Elevation & Details

9.8 Observations Pictures

Figure 9-5: New Hope – South Elevation

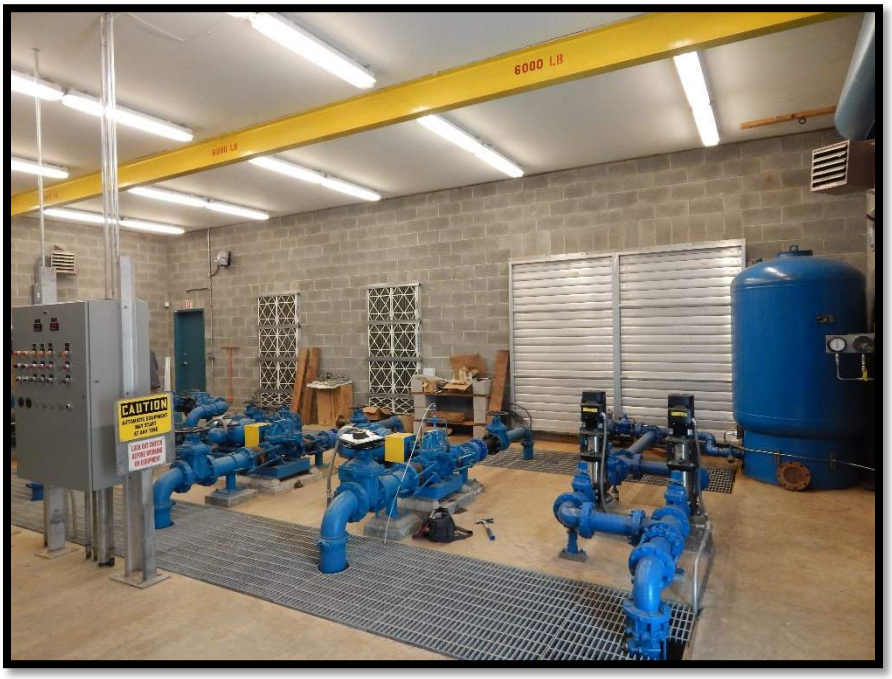


Figure 9-5: Interior View

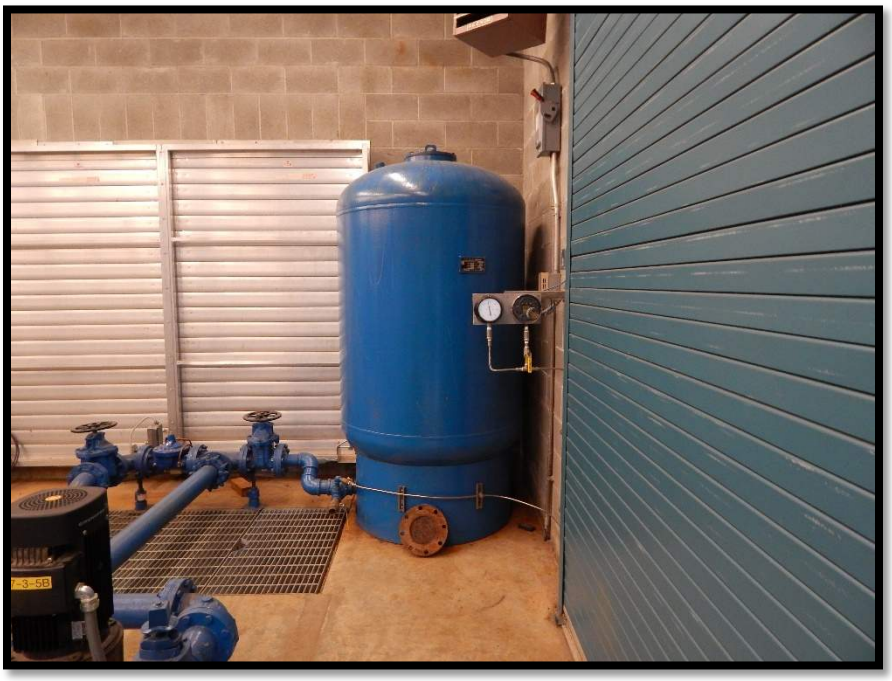


Figure 9-5: Unanchored pressure Tank

Figure 9-5: Cracked Sealant at Vertical Construction Joint

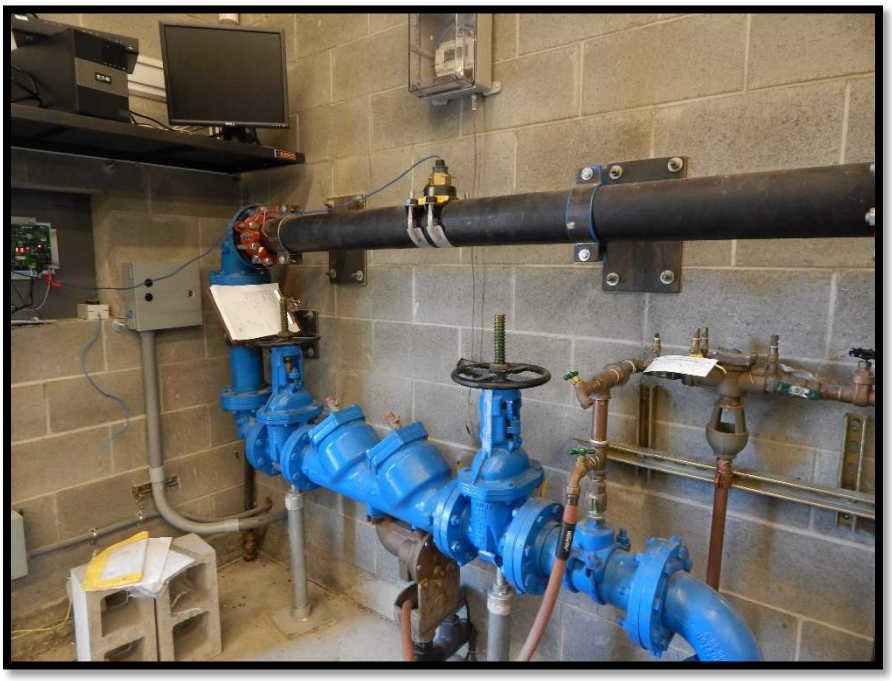


Figure 9-5: New penetration added for water filling station (Top left of Photo)

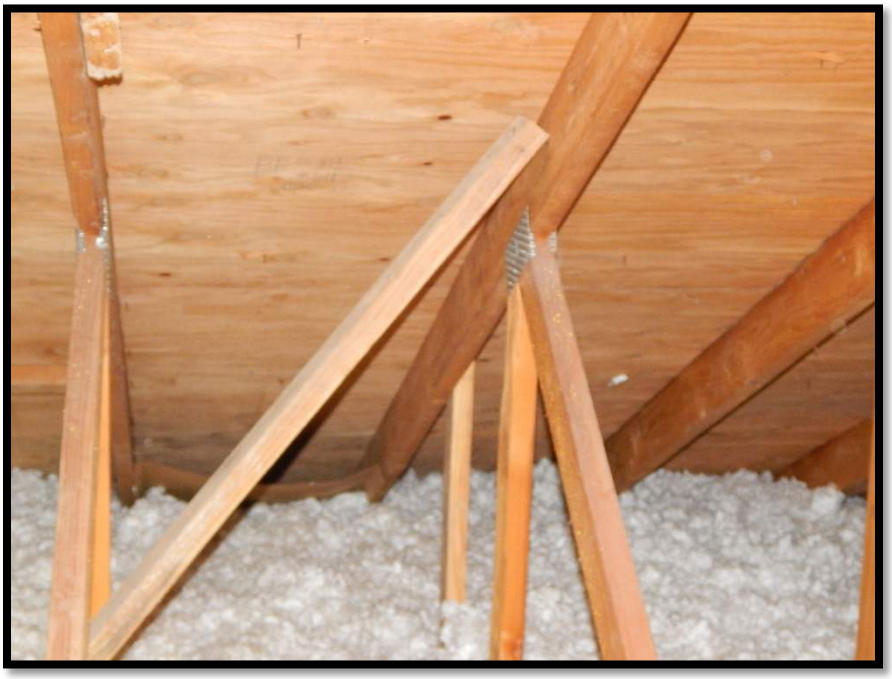


Figure 9-6: Typical Gable end Brace

END OF SECTION

10 Limited Pump Station Condition Assessments

10.1 Description & Background

In addition, to the full structural evaluations for the Reservoirs and pump stations contained in the earlier portions of this report, a limited visual condition assessment of (10) additional pump stations was performed. The limited condition assessments consisted of a site visit to each site to observe the existing as-built condition at which time photographs were taken and any structural items were noted. Review of the original construction documents was not part of the visual condition assessment, nor was any analytical analysis of the existing structures. The visual assessments were performed on January 31st, 2018. The following report sections list each Pump Station reviewed with the date of constructed as reported by the City, in the order visited, along with any notable structural items.

10.2 Williams Crossing Pump Station – Circa 2005

Williams crossing is a clamshell style pump station, which has had a conventional light frame wood enclosure stick framed around it on a concrete slab. Within the enclosure is a small pressure tank as well which is bolted down to the slab. The clamshell pump station appears to be in good condition and no structural concerns were noted during the visual inspection. The wood enclosure structure appears to be in good condition. It was noted that the surrounding grade is in contact with the wood siding which may cause damage to the wood. In addition, one area of the concrete slab/footing is being undermined. It is recommended that the grade be adjusted as needed in these areas.

Figure 10-1: Grading Issues

10.3 Meadow Wood Pump Station – Circa 2002

Meadow Wood pump station is a light frame wood structure built to resemble a residential structure to fit within the neighborhood surroundings. The structure appears to be in good condition and appears to

have been built relatively recently and is expected to be consistent with current code requirements for structural design. No structural issues were noted during the visible inspection. The only item noted, was that the exterior grade is close to the bottom of the wood trim and should be kept clear to prevent damaging the wood.

10.4 Panoramic Loop Pump Station – Circa 2006

Panoramic Loop pump station is a pre-packed manufactured pump station with an integral enclosure and was built relatively recently. The pump station sits on a structural slab and is bolted down. Based upon the appearance and the age of construction it appears to be in good condition and appears to have been and is expected to be consistent with current code requirements for structural design. No structural issues were noted during the visible inspection.

10.5 Harbeck Pump Station – Circa 1999

Harbeck pump station is a clam shell style pump station supported on a structural slab. A generator is also located on the same site, but it sits on a separate structural slab. Both the pump station and the generator are bolted to the structural slabs. Both appear to be in good structural condition and no structural items were noted. Based upon the appearance and the age of construction it is likely that the pump station is comparable to current code structural requirements. The small diameter anchor bolts for the generator may not meet current code requirements, but would require further analysis. If the generator back-up is required additional analysis of the anchorage is recommended. Note, this would require obtaining information on the generator size and weights as well as the existing anchor bolts if available.

10.6 Hilltop Pump Station – Circa 2009

Hilltop pump station is a clam shell style pump station supported on a structural slab as well as a separate pre-manufactured pump station enclosure. A generator, propane tank, and a pressure tank are also installed on the same structural slab. All of the items appear to be anchored to the structural slab. All of the items appear to be in good condition. Some minor areas of corrosion are present on some of the piping and base metal. These do not appear to be structural concerns at this time, but may require maintenance to prevent future problems. No other structural concerns were observed during our inspection.

10.7 Hefley Pump Station – Circa 1996

Hefley pump station is a partially buried pump station built circa 1996 per the City. The pump station is CMU walls with a concrete floor and concrete slab roof. The pump station is accessed through a Bilco style hatch on the side of the pump station. The exterior of the pump station appears to be in good condition with no structural issues noted. Similarly, the interior of the pump station appears to be in good condition. There is a substantial amount of efflorescence built up on several of the walls. This doesn't appear to be a structural concern at this time. This may have occurred shortly after construction or may be indicative of water transmission through the walls. We recommend cleaning the walls of efflorescence to see if it returns which would be indicative of transmission through the walls that may need to be addressed. If

moisture is transmitting through the walls it is recommended that this be addressed to prevent potential damage to the reinforcing steel within the walls.

Figure 10-2: Efflorescence through CMU walls

10.8 Champion Pump Station – Circa 1982

Champion pump station is partially buried concrete pump station with a flat concrete lid. Several statues are located on top of the roof slab. These do not appear to be substantial or a structural concern. The pump station is fairly old, and it is our understanding that it has been prone to flooding in the past. The structure appears to be in fair to good condition with cracking and small chipped concrete areas typical of the age of construction. There were no visual items noted as obvious signs of concern. However, given the structure's age it was designed and constructed before modern seismic detailing requirements came into effect. Therefore, it will not meet current code requirements for seismic detailing by inspection. If the pump station is critical and/or part of long-term planning, we recommend further evaluation to determine if there are structural concerns in regard to seismic resiliency. Given the age of construction, we recommend any evaluation be performed using the methods within ASCE 41, "Seismic Evaluation and Retrofit for Existing Buildings".

10.9 North Valley Pump Station – Circa 1983

North Valley pump station is a small buried steel caisson pump station. Based upon a visible observation of the interior condition it appears to be in good condition. Based upon the configuration and style of construction it is expected to meet or be comparable to current structural requirements for current code. However, since it is a buried steel structure corrosion is a potential concern and should be monitored as required. We were unable to observe the buried earth side of the structure. It is our understanding that active cathodic protection is used to maintain corrosion resistance, this should be maintained as required.

10.10 Starlite Pump Station – Circa 1982

Starlite pump station is a concrete vault built into the hillside. The pump station is mostly buried with exposed roof hatches protruding through the hillside and is accessed by a hatch on the side of the vault exposed on the lower side of the hillside. The concrete has areas of small cracking, but these appear to be consistent with the expected performance and age of the structure. Generally, the concrete appears to be in good condition and there are no visual signs of immediate structural concerns or significant deterioration that warrants immediate action.

However, given the structure's age it was designed and constructed before modern seismic detailing requirements came into effect. Therefore, it is expected that it will not meet current code requirements for seismic detailing by inspection. If the pump station is critical and/or part of long term planning we recommend further evaluation to determine if there are structural concerns in regard to seismic resiliency. Given the age of construction, we recommend any evaluation be performed using the methods within ASCE 41, "Seismic Evaluation and Retrofit for Existing Buildings".

It was noted that the floor was wet in a fairly large area so there appears to be either ongoing leaking or issues with water infiltration that may warrant further investigation and/or maintenance. The exterior grade has sloughed around the roof hatches and bark chips and vegetation are impinging on the hatches and should be cleared back to keep the hatches free of deleterious materials. It appears that the entry hatch and some of the roof hatches may be leaking and areas of corrosion and staining are evident. These appear to be non-structural concerns, but should be maintained as needed.

10.11 Laurel Ridge Pump Station – Circa 2014

Laurel Ridge pump station has two clam shell style pump stations. It is our understanding that the newest one was built circa 2014, the age of the older one wasn't known on site. Both clam shell pump stations are located on a structural slab. In addition, a generator is located on the same structural slab. The structural slab has several cracks, but these do not appear to be a structural concern and are cosmetic only.

The newer clam shell and the generator are anchored to the structural slab. The older clamshell is built onto a steel skid and anchor bolts were not observable, however, may be located internally. The clam shells appear to be in fair to good condition with no structural issues noted. If the older clam shell is not anchored, it would need anchorage added to meet current seismic code requirements.

The anchorage for the generator appears undersized and may not meet current code requirements, but would require further analysis. If the generator back-up is required additional analysis of the anchorage is recommended. Note, this would require obtaining information on the generator size and weights as well as the existing anchor bolts if available.

END OF SECTION