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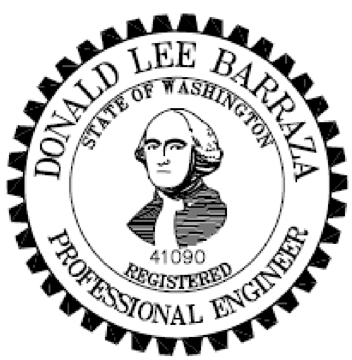
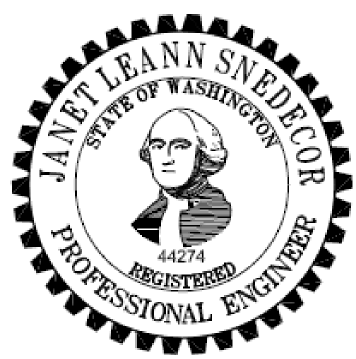
Minor Road Reservoirs Seismic and Structural Evaluation

August 2012





Prepared for

City of Kelso
203 South Pacific Avenue
Kelso, Washington 98626

K/J Project No. 1197022.00

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Executive Summary

This report documents the findings of a comprehensive review of the City of Kelso's (City's) Minor Road Reservoirs performed by Kennedy/Jenks Consultants. The City has expressed concerns pertaining to the structural integrity and longevity of the Minor Road Reservoirs and the site soils strength and stability. The reservoirs were originally built in 1924 and have a history of leaking; the source and magnitude of the leakage is unknown. Whenever possible, Kennedy/Jenks Consultants relied on documentation of previous field investigations by other consultants and repair and leakage test reports, rather than performing additional investigations, in order to assist with the timely completion of this report.

The concrete reservoirs have numerous cracks and spalls, many of them actively leaking. Attempts were made to repair leaks in the reservoirs in 2005, 2007, 2009, and 2011; however, leakage in the reservoirs continues and may be increasing in quantity. The condition of the reinforcing steel in the walls is unknown and due to the history of leakage, may be significantly corroded. A large amount of subsurface groundwater is present, possibly from leakage from the reservoirs. High groundwater may be contributing to migration of subgrade material beneath the reservoir foundations, increasing lateral earth pressure and buoyant forces on the reservoirs.

Based on previously completed work, the Troutdale Formation of northern portion of the site is relatively weak and could contribute to soil instability. Slope instability issues could result in damage to the retaining walls and adjacent property without implementation of ground stabilization measures. The backfill soils around the reservoirs are saturated as a result of leakage from the tank walls and floors, are susceptible to mobilization in an earthquake due to the sloping site. Soil migration could result in loss of support to the reservoir walls and floors. The resulting conditions do not satisfy required factors of safety associated with slope stability, and could damage roadways and structures located to the west of the reservoir site.

The walls of the reservoirs were found to be significantly below strength under normal operating conditions and are vulnerable to failure under multiple conditions during an earthquake event with a Richter Magnitude 6.0 or larger. The results of the probability of failure analysis given a specific magnitude event indicate the reservoirs are 100 percent likely to fail during a seismic event having a Richter magnitude of 6.0 or larger, and 10 percent and 12 percent likely to failure during a 5.0 or 5.5 event, respectively. The lack of reinforcing steel between the wall, wall footing, and the floor slab results in the walls relying entirely on passive pressure to resist base shear. Due to the small size of the footing, this passive pressure is insufficient to resist shear forces. It is likely that in an earthquake event, the outward movement of the walls and increased lateral earth pressure will result in additional separation in the wall to floor joint leading to increased leakage and further undermining of the subgrade material supporting the floors.

The reservoirs are also at risk of a catastrophic failure if the water level were to be lowered rapidly due to a pipeline break downstream or other outside event. The sudden loss of water inside the reservoirs could contribute to a buoyant force placed on the reservoir floors resulting in structural damage to the floors.

The City is currently unable to remove either of the reservoirs for inspection or repairs while maintaining the other reservoir in service due to piping and valving limitations. This significantly reduces the reliability that two reservoirs provide at the site. If one of the reservoirs were removed from service, it is probable that leakage from the reservoir remaining in service could increase buoyancy forces on the drained reservoir; thus, increasing the possibility of failure in the drained reservoir floor.

Based on the evaluation results summarized herein, the risk of reservoir failure is high, as a result of moderate or large earthquake or a sudden loss of water in the reservoirs. Reservoir failure could endanger the public and residents on the adjacent private property and impact the City's water supply and fire protection. Damage could be significant, in the inundation zone below the reservoirs, to North Minor Road, the Three Rivers Christian School/Cornerstone Junior/Senior High School, and Interstate Highway 5 located beneath the reservoirs. The potential liability is significant.

Results from the City's hydraulic model indicated it is possible to remove the Minor Road reservoirs from service without negatively impacting the distribution system. While the Minor Road Reservoirs are out of service, the Paxton Road reservoirs will become the Main Zone's source of supply. However, as the Paxton Road reservoirs are supplied by a single pipeline which crosses beneath Interstate 5. To improve system reliability and provide redundancy, it is recommended that City install a redundant freeway crossing to supply the Paxton Road reservoirs.

Kennedy/Jenks Consultants recommends that the reservoirs be replaced. An economic comparison of the three alternatives evaluated shows that it is more economically practical to replace the reservoir than it is to structurally and seismically strengthen the reservoir. If the City intends to maintain the existing reservoirs in service for an extended period of time, Kennedy/Jenks Consultants recommends additional inspection and temporary remedial actions to improve the condition of the reservoirs. Additional inspection and remediation would include draining the tanks while monitoring groundwater levels, and inspecting the concrete of the walls, floors, and reinforcing steel both above and below grade. Any defective concrete should be removed and replaced using epoxy pressure injection into cracked concrete.

Section 1: Introduction

1.1 Background

The City of Kelso (City) has expressed concerns pertaining to the structural integrity and longevity of the Minor Road Reservoirs and the site soils strength and stability. The Minor Road Reservoirs facility consists of two circular, partially buried, 1 million gallon (MG) reservoirs constructed in 1924. The reservoirs have a history of leaking; the source and magnitude of the leakage is unknown. The City commissioned a structural evaluation [Kramer Gehlen Associates Report dated 24 February 2010 (KGA 2010)] and a preliminary geotechnical issues report [(Shannon & Wilson, Inc. dated 30 June 2010 (Shannon & Wilson 2010)].

1.2 Purpose and Goals

The purpose, goals, and objectives of this seismic evaluation were to expand upon the findings of previous reports and to provide the following information to aid the City in its decision making process:

- Determine the failure risk thresholds based on current site and reservoir conditions.
- Identify what magnitude earthquake could contribute to failure of the reservoirs.
- Provide recommendations for either the strengthening or replacement of the reservoirs.
- If strengthening of the reservoirs is a feasible option, correlate the improvements, limitations, costs, and extended life of the reservoirs.

1.3 Scope of Services

Our scope of services was performed in accordance with contract executed by the City on 14 December 2011. An outline of the tasks included in the scope of services is summarized below:

Phase A – Evaluation and Recommendations
Task A.1 – Project Management and Project QC
Task A.2 – Review of Structural Evaluation and Geotechnical Investigation
Task A.3 – Field Observations, Condition Survey, Sampling, and Video Review
Task A.4 – Identify Pipeline and Seismic Rehabilitation Alternatives
Task A.5 – Distribution System Improvements
Task A.6 – Conceptual Level Cost Estimate
Task A.7 – Report Preparation

Phase B – Supplemental Tasks
Task B.1 – Reservoir Inspection
Task B.2 – Geotechnical Services

Section 2: Existing Construction and Documentation

This section provides a summary of the background information available on the Minor Road Reservoirs. Kennedy/Jenks Consultants reviewed available drawings, specifications, construction records, past reports by engineering and geotechnical consultants, service history reports on repairs to leaks in the reservoirs, and video reports provided by the City related to the Minor Road Reservoirs and their history. Work reviewed included the Kramer Gehlen Associates report dated 24 February 2010 (KGA 2010) (included as Appendix A) and Preliminary Geotechnical Issues Report by Shannon & Wilson, Inc. dated 30 June 2010 (Shannon & Wilson 2010) (included as Appendix B).

2.1 Reservoir Site Description

The Minor Road Reservoirs are located just north of the intersection of 7th Avenue North (Mt. Brynion Street) and North Minor Road on the northeastern side of Kelso, Washington. The reservoir site is shown on Figure 1 - Reservoir Location included in the Figures section of this report. Private residences are located on the northern, eastern, and southern sides of the reservoir site. The Three Rivers Christian School/ Cornerstone Junior/Senior High School is located just west of the reservoir site. The reservoir site is less than 300 feet east of Interstate 5 near the 7th Avenue North overpass.

The reservoir site slopes downward from east to west and the reservoirs are located between 160 and 200 feet above sea level and approximately 60 feet above Interstate 5. The reservoirs are partially buried with the tops of the concrete walls between 4 feet (eastern side) and 8 feet (western side) above finished grade. The reservoir site is unimproved with grass over most of the site; however, during a site visit on 1 December 2011, it was noted that the ground was extremely saturated and on the downhill (western) side of the reservoirs, contained ponded water, and was very soft in some areas. A retaining wall supports fill materials on the western side of the reservoir site adjacent to North Minor Road. The reservoir site is fenced.

2.2 Reservoir Description and Design Information

The Minor Road Reservoirs were analyzed based on information contained on two drawings identified as Waterworks Improvement Unit #6 General Plan and Reservoir Details dated July 1924. Copies of these drawings are included in Appendix C. Based on review of the drawings, the reservoirs have the following dimensions, configuration, and design elements:

- The reservoirs are circular, conventionally reinforced, concrete tanks with sloping hopper bottom configurations and triangulated aluminum strut and panel dome roofs.

- The reservoirs have 90’-0” inside diameters.

- The reinforcing in the walls and floors of the reservoirs is composed of square bars.

- The floors of the reservoirs are approximately 6-inch-thick concrete slabs-on-grade and may contain welded wire mesh reinforcing steel in portions of the floors. Based on review of the general plan, the sloping hopper bottom panels of the floor slab should typically contain one square ½-inch reinforcing bar, with 20-inch long lap splices, around the perimeter of the panel with approximately 3 inches of cover.

- The hopper bases of the reservoir consist of many small slab partitions, which do not appear to be doweled together and the perimeter slab partitions appear to be simply resting on top of the tapered footings.
- The reservoirs have 18'-6" tall exterior walls that are vertical on their inside face and taper in thickness from 9 inches thick at top of wall to 15 inches thick at base of wall. The height is approximately 19'-6" from the top of the walls to the bottom of the footings.
- The reservoirs walls have a small 5'-0" wide perimeter wall footing that are flat on the bottom and taper at both ends from approximately 8 inches thick to 20 inches thick where they meet the wall faces. The footings may have been constructed monolithically with the walls. Due to the small size of the footings, it is unlikely that the footings can provide significant resistance to rotation at the base of the walls; *therefore, the walls would most likely behave as and should be modeled as a hinged condition.*
- The primary horizontal hoop stress reinforcing in the walls is located on the exterior face of the walls of the reservoirs and varies over the height of the reservoir walls from a minimum of ½-inch square bars at 12 inches on center near the top of the wall to 1-inch square bars at 4 inches on center near the base of the wall. The secondary, temperature and shrinkage horizontal reinforcing steel in the walls on the interior face consists of either ½-inch square bars at 16 inches on center over the top 10'-8" of the wall and ½-inch square bars at approximately 12 inches on center over the remaining 7'-10" at the base of the wall.
- Vertical flexural, bending moment, reinforcing steel in the walls of the reservoirs consists of ¾-inch square bars at 12 inches on center on the exterior face of the walls and ½-inch square bars at 24 inches on center on the interior face of the walls. Based on the designation of the vertical bars on the exterior face of the walls, it is unlikely that all of the bars are of the same length and over the full height of the wall.
- The reservoir walls have a stepped 3-inch and 6-inch thickened coping which is primarily for architectural purposes at the tops of the walls.
- The roofs of the reservoirs are constructed of triangulated aluminum strut and panel dome covers.

2.3 Materials and Construction Information

As-built construction documents from the original construction of the reservoirs were not retained; thus, little information is available that could provide information on the construction methods, materials, and problems encountered.

The original technical specifications, page 25, provided the only information with regard to the materials used in construction of the reservoirs. Excerpts from the original contract specifications are included in Appendix C. Based on review of the information contained in the original specifications and KGA 2010 report, we know the following regarding materials used in the construction of the reservoirs:

- Reinforcing steel was specified as Billet Steel Concrete Reinforcement Bars of the ASTM Serial Designation A15-44, structural steel grade. ASTM A15 Structural Grade plain and deformed bars had tensile requirements that included a yield strength of 33,000 pounds per square inch (psi) and a minimum tensile strength of 55,000 psi.

- Based on the KGA 2010 report, concrete utilized in the construction of the reservoir was specified with a minimum 28-day compressive strength of 2,500 psi.

2.4 Reservoir Evaluation and Structural Calculations

The KGA 2010 report included the results of field observation and a set of structural calculations to evaluate the structural and economical feasibility of remediation solutions as well as replacement of the two existing reservoirs with new reservoirs. A summary of the notable findings contained in this structural evaluation are included below:

1. Visual observation revealed numerous cracks in the concrete walls above the ground, with several of them actively leaking. Some previously patched cracks were actively leaking.

2. 6.3 sack Concrete mix with an estimated 28-day strength of 2,500 psi to 3,000 psi was assumed in the calculations.

3. No reinforcing steel in base slab, except at one panel, was assumed.

4. The wall footing was not reinforced heavily enough to be considered “Anchored and Contained”, thus, the wall footing is considered “Unanchored Uncontained Flexible Base”. An “Unanchored Uncontained Flexible Base” condition is not currently permitted in national standards for seismic Design Categories D, E, and F. Kelso, Washington is in located in a Seismic Design Category D geographic area.

5. Both a free base-free top condition and a hinged base-free top condition were assumed for analysis. Free base, free top condition: The calculated overstress in the hoop reinforcement is 47 percent to 73 percent from top of wall to bottom of wall. Hinged base, free top condition: The calculated overstress in the hoop reinforcement ranges from 0 percent at the top and bottom of the tank to 307 percent in the upper halves of the tank.

6. The ring tension induced by combined seismic and static loads was calculated to result in overstressing of the hoop reinforcement at the top of the wall by 60 percent and at the mid-height of the wall by 86 percent. Hoop reinforcement stress is 160 percent and 186 percent of the calculated acceptable hoop stress.

7. Maximum wave height was calculated as approximately 2.0 feet. Based on the current available freeboard of 1.0 foot, there is potential for roof damage in a seismic event.

8. Lap splices for existing 1-inch square horizontal reinforcing bar is 4’-0”. A minimum splice length of 81-inch was calculated for similarly sized #9 steel reinforcing bars.

9. The condition of the existing reinforcing steel was unknown, which may be significantly corroded.

10. The report presented the following three alternatives and recommended that construction of new water reservoirs be seriously considered:

    a. Lining the Reservoirs Without Strengthening Measures: This alternative was developed in order to eliminate leaking and provide reservoirs that would be serviceable for an unknown number of years provided a design level seismic event did not occur.

    b. Lining the Reservoirs and Remedial Strengthening: This alternative was developed in order to eliminate leaking and to construct new walls and foundations inside the existing reservoirs that would resist earthquake forces. This alternative would require construction of 16-inch-thick walls with 24-inch-thick mat foundations. The volumetric capacity of the reservoirs would be reduced by 157,000 gallons. Pressure relief valves would need to be installed in mat foundations to prevent uplift.

    c. Construct New Water Reservoirs: This alternative was developed to provide one or more new replacement reservoirs sized for current demands and designed to meet current codes and standards.

2.5 Draft Preliminary Geotechnical Issues Report

The Shannon & Wilson 2010 Report (included as Appendix B) was intended to identify and discuss geotechnical issues at the Minor Road Reservoir site related to potential rehabilitation of existing reservoirs and/or a potential complete replacement of the existing reservoirs with a single, new 2 MG reservoir at 130-foot-diameter with vertical sidewalls up to at least 20 feet high with hopper bottom or a maximum of 30 feet high without hopper bottom. The report does not include geotechnical analyses or evaluations. The report does not provide detailed information to assist in the design of a new reservoir. No static and seismic lateral loading criteria, vertical soil bearing capacities, soil design parameters, and type or extent of necessary foundation and foundation material preparation is provided. The following summarizes key points in the report:

1. The report states that geotechnical explorations and characterization will likely be necessary on the eastern portion of the site due to the high variability of the subsurface conditions.

2. Basis of report relies on subsurface soil characterization contained in a geotechnical exploration program completed in December 2009.

3. Portions of the site are characterized as relatively weak, fine-grained Troutdale Formation. Confirmation is needed that the fine-grained soils of the Troutdale Formation are not liquefiable.

4. Slope stability is a concern primarily at the northwestern and northern sides adjacent to the North Reservoir, especially if a larger footprint reservoir is to replace the existing reservoirs.

5. High groundwater levels were noted in borings. There appears to be no subsurface drainage around the perimeter of the tanks to relieve hydrostatic pressure on walls and foundation.

6. Due to the large amount of subsurface groundwater movement on a continuous basis, subgrade material may have migrated over the years, resulting in voids under the foundation.

7. The Shannon & Wilson 2010 Report recommends conducting an assessment of foundation slabs with the tanks drained and indicates the potential need for ground stabilization. Due to concern of groundwater uplift, groundwater levels will have to be confirmed or a method of hydrostatic relief should be installed.

8. Under seismic conditions, differential settlement is expected in the existing Northern Reservoir between the northern and southern side of the reservoir due to the difference in foundation material.

9. Under seismic conditions, differential settlement should not be a concern for the Southern Reservoir but this needs confirmation with additional subsurface information.

10. If reservoirs are rehabilitated from the inside, significant remediation work such as grouting to fill voids, ground improvement, and/or an underpinning system is required for both reservoirs to eliminate potential of differential settlement and future structural problems. The report does not detail the rationale for these conclusions.

11. For new reservoir construction, over-excavation of the fine-grained Troutdale material and replacement with imported crushed rock is necessary.

12. In order to construct the new reservoir while retaining one existing reservoir in service, a stable foundation for the existing reservoir adjacent to the excavation will need to be maintained using lateral restraint shoring systems, ground improvement, and/or underpinning of the existing reservoir foundation.

13. The Shannon & Wilson 2010 Report recommends removing both reservoirs prior to building new reservoir.

2.6 Service History

The following documents related to leaks and repairs on the reservoirs were reviewed; copies of the leak detection and repair reports are included as Appendix D.

North Reservoir Leak Detection and Repair Project Report, Northwest Underwater Construction, 13 May 2005. The report shows that a total of 20 leaks were sealed using an epoxy based material.

North and South Reservoirs Leak Repair Summaries, Northwest Underwater Construction, 13 March 2007. The summary letter states that using potable underwater epoxy, 21 repairs were performed in the North Reservoir and 18 repairs were performed in the South Reservoir.

Leak Detection and Repair Report, 22 and 23 September 2009. Review of the report indicates that seven small leaks were detected in the North Reservoir, three in the reservoir wall and four in the reservoir floor. Twelve small leaks were detected in the South Reservoir, three in the reservoir wall and nine on the reservoir floor. All leaks were repaired using an epoxy based sealing product.

Leak Repair Notes, 7 September 2011. Review of notes show the two leak test reports dated 22 and 23 September 2009, respectively, lack good data and contain inaccurate statements. Subsequent to the 2009 repair attempt, the overall current leakage is about 45 gallons per minute (gpm). The dive team had little success in finding the leaks due to the many cold joints and minor pits and cracks.

Leak Repair Videos (dates unknown). We reviewed the videos of the divers locating and patching leaks in cracks, holes, pits, and cold joints. Some previously patched areas were observed to be leaking in the videos, which show that the surface applied leak repair method may only be suitable as a short-term fix.

Section 3: Piping and Distribution System Improvements

The as-built conditions of pipeline connections to the reservoirs, isolation valving, and distribution system were evaluated to develop recommendations for further investigation and improvements. The City's water system model was utilized to determine what, if any, distribution system improvements would be necessary to allow the Minor Road Reservoirs to be taken offline for rehabilitation or replacement.

3.1 Reservoir Piping

The reservoirs were each constructed with a 12-inch dedicated fill inlet, 16-inch dedicated outlet, 12-inch overflow, and a 12-inch drain pipe. The 16-inch outlet is situated near the bottom of the reservoir with the 12-inch inlet located approximately 6 feet above the 16-inch. The contract specifications indicate the drain pipe is concrete; the pipe materials of the inlet, outlet, and overflow are unknown, for discussion purposes, it is assumed they are cast iron. Copies of the original contract drawings and specifications are included in Appendix C.

Common headers for the inlet, outlet, and drain piping and isolation valves were installed in a valve pit approximately 32 feet deep by 6 feet 9 inches by 9 feet at the widest which narrows down to 3 feet by 9 feet. Isolation valves with rising stem operators were installed on each of the inlet, outlet, and the drain valves which were operated from the Gate House situated on top of the valve pit. Access to the valves for repair and maintenance is extremely difficult and considered a confined space. It is assumed that the valve pit walls were constructed as a monolithic concrete structure on a bottom slab. Based on the 1924 construction documents, the valve vault did not include provisions for a sump or drainage. It is possible that there is water entering the valve vault through the pipe penetrations and the wall to floor slab joint. Without a known path of drainage, there could be standing water in the bottom of the vault which could have contributed to corrosion of the piping and valves. It is anticipated that damp conditions in the valve pit will continue to be a long-term maintenance issue.

Some operational and piping configuration changes have occurred since the reservoirs were originally constructed. The City abandoned the 12-inch dedicated inlet in favor of utilizing the 16-inch as a common inlet/outlet. The 12-inch yard and distribution piping has been removed up to the valve pit; the piping in the valve pit and connected to the reservoirs have been abandon in place. According to City staff, the 16-inch valves in the valve pit are no longer operational; thus, a common isolation valve for both reservoirs was installed in a vault to the west of the valve pit and gate house. As a result, the City is no longer able to isolate one reservoir from the other for service and maintenance. To facilitate maintenance and allow for operational flexibility, the functionality of the 16-inch isolation valves should be restored.

As discussed previously, the reservoirs are leaking at a rate of approximately 45 gpm (September 2011), or approximately 23 MG per year. It is difficult to quantify the amount leaking from any one individual location. The condition of the pipe and pipe connections were not assessed during the previous leak detection efforts, in part due to accessibility issues. The least destructive and intrusive method to ascertain the condition of the pipes and connections would be to video survey the pipes from the reservoirs to the isolation valves. Given the age of the reservoir piping and the potential migration of the supporting base material, it is likely that the pipes and pipe connections are contributing to overall reservoir leakage. If the City decides to either repair or strengthen the reservoirs, it is recommended that the 16-inch inlet/outlet pipes and 12-inch drain pipes be lined using a cured-in-place pipe (CCIP) rehabilitation product such as Insituform, InsituMain®. Use of a CCIP product would extend the life of the pipes and minimize or eliminate pipe leakage. Installation of a CCIP liner would involve minimal disruption to the pipes and would eliminate the need to excavate beneath the reservoir floor to replace pipes. InsituMain® is capable of bridging over corrosion holes, pinholes, and joint gaps in the host pipe on a long-term basis. To be considered a cost-effective rehabilitation alternative, the host pipe must be deemed structurally sound.

3.2 Distribution System Improvements

The City's water system model, WaterCAD version 8i, was utilized to determine the potential system impacts of removing the Minor Road Reservoirs from service for either rehabilitation or replacement. The highest demand situation, maximum day demand plus fire flow, was utilized. It should be noted the analysis performed is for the static condition and does not reflect cyclical demand patterns.

Two scenarios were created to for this analysis; Scenario 1 – Minor Road Reservoirs in service, maximum day plus fire flow conditions, and Scenario 2 – Minor Road Reservoirs out of service, maximum day plus fire flow. Both scenarios assume that the Minor Road pump station will remain in service. The results from Scenario 1 were then compared to Scenario 2. Model results indicate there are no negative distribution system impacts associated with removing the Minor Road Reservoirs from service. Table 3-1 summarizes the flow conditions at selected model nodes for both scenarios analyzed. Figure 2 depicts the approximate model node location. As can be seen from the results, there is a decrease in available flow in Scenario 2 with the reservoirs out of service; however, this decrease is not considered significant.

Table 3-1: Summary of Fire Flow at Selected Nodes, Scenario 1 and Scenario 2

Node Description	Scenario 1 Fire Flow available (gpm)	Scenario 2 Fire Flow available (gpm)
Williams Street, near dead end (node J-9186)	2,710	2,685
Crescent Avenue & Lewis (node J-1266)	3,693	3,450
Grant & 6th Avenue, West Kelso (node J-1434)	3,552	3,331
Elm Street & 11th Avenue (node J-2120)	2,902	2,848
Sunrise & 13th Avenue (J-3020)	5,000	5,000
Allen Street, near High School (node J-1308)	5,000	5,000

The Minor Road Pump Station will remain in service while the reservoirs are offline. The pump station draws from the 16-inch supply pipe supplying the reservoirs and pumps to the upper zones of Williams-Finney and Behshel Heights. The pump station suction is located to the west (downstream) of the reservoir isolation valve; therefore, no piping modifications are anticipated in order to maintain service to the pump station while the reservoirs are out of service. Output from the Minor Road pump station is relatively unchanged with the Minor Road reservoirs out of service.

According to the model, the distribution system relies heavily on the water treatment plant and the Paxton Reservoirs. The model indicates approximately 7,300 gpm comes from the Paxton Reservoir under maximum day plus fire flow conditions. Currently, a single pipeline supplies the Paxton Road reservoirs. A portion of this pipeline is asbestos cement pipe that crosses beneath the freeway; making access for repair or replacement difficult. As the Paxton Road reservoirs provide a large portion of the City's storage, it is recommended the City consider the installation of a redundant supply/distribution line to the Paxton Reservoir.

Ideally, a second distribution pipeline would be installed along Kelso Drive to provide a more direct connection between the Minor Road and Paxton Road Reservoirs. However, there are other smaller projects that could be completed which would provide the redundant connection to the Paxton Road Reservoir. The City has a capital improvement project planned (City Number W-43) to install 4,800 linear feet (LF) of 8-inch-diameter pipe from Grade Street/Haussler Road pump station to the Carrolls Road pump station. It is recommended that the City elevate the priority of this project and upsize the pipe from 8 inches to 12 inches; this would provide a second Interstate crossing to supply the Paxton Road reservoirs. Several other potential improvement projects have been identified in the City's Draft 2012 Water System Plan that would improve service reliability to Paxton Road Reservoirs; these projects are summarized in Table 3-2.

Table 3-2: Recommended Distribution System Improvements

Project Location	Project Description	Benefit	Estimated Cost [(a)]
Grade St. waterline - Haussler pump station to Carrolls Rd. pump station	Install 4,800 LF of 12-inch-diameter pipe on Grade Street from Lower Haussler Pump Station to Carrolls Road Pump Station.	Improve service to Paxton Road Reservoir and future industrial service areas. Provide redundant connection crossing freeway to supply Paxton Road Reservoir.	$1,229,000
Paxton Res Transmission Main – Carrolls Rd. pump station to reservoir	Replace existing 16-inch asbestos concrete (AC) main with 16-inch ductile iron. Revise routing from Carrolls to pump station to Paxton reservoir to address easement encroachments issues. Project will require alignment analysis and possible easement acquisition.	Improve reliability of only transmission main to Paxton Reservoir.	$800,000
Grade St. Main Replacement	Replace existing 6-inch AC and 8-inch DI with 12-inch DI from 13th Avenue to Haussler Pump Station.	Improve service to Paxton Reservoirs, Haussler and Carrolls pump stations.	$461,000
S. Kelso Dr. from intersection of S. Kelso Dr. and 13th to Haussler Rd. pump station.	Install new 16-inch DI main connecting existing 10-inch at 13th/Manasco to 16-inch DI at Carrolls Road Pump Station.	Provide more direct redundant connection linking reservoirs in main service zone. Improve hydraulic connection between Minor Road Reservoirs and Paxton Reservoirs.	$1,638,000
Cedar St. Waterline Replacement – S. Pacific Ave. to Grade St.	Replace 2,400 LF of 8-inch and 10-inch pipe with 16-inch pipe on Cedar Street from South Pacific Avenue to Grade Street.	Improve service to the distribution system and provide supply capacity to future service areas.	$819,000

Note:

(a) Estimated Construction Cost Preliminary Planning Level. Estimates include sales tax (7.9 percent), contractor overhead and profit (OH&P) (15 percent), planning level estimate contingency (25 percent), and engineering/design and construction management (25 percent)

Section 4: Seismic and Structural Evaluation

4.1 Structural and Seismic Evaluation

Records indicate that the reservoirs were designed in 1924; however, there is little or no information on the codes and standards utilized in the design of the reservoirs. Seismic evaluation of the reservoirs was performed in accordance with the following codes and national standards:

1. *2009 International Building Code*. *International Code Council, Inc*. *February 2009*.
2. *ASCE Standard for Minimum Design Loads for Buildings and Other Structures*. *ASCE/SEI 7-10*. *American Society of Civil Engineers*. *2010.*
3. *Code Requirements for Environmental Engineering Concrete Structures and Commentary (ACI 350-06) An ACI Standard, American Concrete Institute*. *2006.*
4. *Seismic Design of Liquid-Containing Concrete Structures and Commentary (ACI 350.3-06) An ACI Standard*. *American Concrete Institute*. *2006.*
5. *PCA Design of Liquid-Containing Concrete Structures for Earthquake Forces, Portland Cement Association, Javeed A. Munshi*. *2002.*
6. *PCA Circular Concrete Tanks Without Prestressing Design Manual, Revised 1st Edition, Portland Cement Association, August W. Domel, Jr*. *1993.*

4.2 Geologic Conditions and Seismic Hazard

This section provides background information of the general subsurface conditions, geologic setting, and seismic hazards believed to exist at the reservoir site. Interpretations of the site conditions are based on review of several geologic and geotechnical reports for projects in the Kelso geographic area. More detailed information would require field exploration including drilling of exploratory borings at the reservoir site.

The near-surface geology in the project area has been mapped as Pleistocene-Pliocene age (5.3 million to 11.5 thousand ybp) (Quaternary to Pliocene) sedimentary bedrock of the Troutdale Formation (Phillips 1987). Bedrock of the Troutdale Formation generally consists of a moderately to weakly consolidated conglomerate, sandstone, and sandy siltstone. Geologic information for the project area was obtained from the *Geologic Map of the Mount St. Helens Quadrangle, Washington and Oregon* (Phillips 1987), published by the Washington State Department of Natural Resources. According to the above-referenced geologic map, near-surface deposits in the project area are mapped as alluvium. Deposits defined as alluvium typically consist of younger, unconsolidated, stratified units of silt, sand, and gravel. In some areas, alluvium may contain interbeds of peat and organic silt. The site is located near the confluence of the Columbia and Cowlitz Rivers and the alluvium was likely transported and deposited by both rivers. The alluvial unit is typically very soft/loose to stiff/medium dense, has low to moderate shear strength, and depending on its composition, can be moderately compressible.

The Pacific Northwest is seismically active and the Minor Road Reservoirs site has most likely been subjected to ground shaking from a moderate to major earthquake over the life of the facility. Earthquake size is determined using the moment magnitude scale, denoted as M_w. M_w is used in the seismology and earthquake engineering communities to quantify the size of medium to large earthquakes based on fault displacement and area of fault rupture. Moment magnitude is the successor of the Richter scale. It was developed to address shortcomings in the Richter scale associated with very large earthquakes. However, the moment magnitude scale correlates very closely with the Richter scale. The moment magnitude scale is now the most common measure for medium to large earthquakes (greater than 3.5).

The regional sources of seismicity affecting the Kelso area and hence, the potential for ground shaking, are controlled by three separate fault mechanisms:

- Large interface earthquakes [moment magnitude (M_w) 8 to 9]
- Relatively deeper, yet smaller, intraplate events (M_w 6.5 to 7.3) associated with the Cascadia Subduction Zone (CSZ)
- Relatively shallow crustal zone earthquakes (M_w 5.0 to 7.0).

The two most relevant sources of seismicity for Minor Road (considering the 2,475-year return period for design) are: (1) the CSZ, which is considered to be capable of generating M_w 8+ earthquakes; and (2) relatively shallow, crustal sources, which are considered capable of generating M_w 6.0 to 7.0 earthquakes. Descriptions of these potential earthquake sources are presented in Appendix E - Sources of Seismicity.

4.2.1 Seismic Evaluation

The procedures used for determining earthquake forces on the reservoirs were based on methods documented in ACI 350.3-06 and Chapter 11 of ASCE 7-10. These are the same methods and codes that would be utilized to design a new concrete reservoir in Washington at this time. Seismic hazard due to ground shaking is based on the location of the structure with respect to causative faults, the regional and site-specific geologic characteristics, and a selected earthquake hazard level. For this project, four different earthquake hazard levels were identified in order to evaluate the impacts on the reservoir structures associated with different magnitude earthquake events (magnitude 5.0, 5.5, 6.0 BSE-1, and 7.0 BSE-2). Basic Safety Earthquake (BSE)-1 (10 percent in 50 years) and BSE-2 (2 percent in 50 years) are measures of the hazard level associated with the probability of exceedance of a given event. BSE-1 is considered to be the lesser ground shaking event and BSE-2 is a more extreme event.

Mapped acceleration parameters were obtained for the Minor Road Reservoirs site from the U.S. Geological Survey (USGS) web site based on the site coordinates. The mapped acceleration parameters and other seismic design parameters utilized in the evaluation of the reservoirs are summarized in Table 4-1.

Table 4-1: Earthquake Hazard Level Summary

Earthquake Hazard Level Richter Magnitude	Probability of Exceedance	Mean Return Period	Peak Ground Acceleration at T = 0	Mapped Spectral Response Acceleration and Short Periods, Ss	Mapped Spectral Response Acceleration at 1 Second Period, S_1
5.0	50% in 50 years	77 years	0.096g	0.240	0.129
5.5	20% in 50 years	225 years	0.108g	0.270	0.146
6.0 (BSE-1) [(a)]	10% in 50 years	500 years	0.266g	0.665	0.3916
7.0 (BSE-2) [(a)]	2% in 50 years	2,500 years	0.399g	0.997	0.586

Note:

(a) Basic Safety Earthquake (BSE)-1 (10% in 50 years) and BSE-2 (2% in 50 years) is a measure of the hazard level.

The Richter magnitude shown in Table 4-1 is a measure of earthquake strength, which closely correlates to the moment magnitude scale (M_w). The magnitude of an earthquake depends on the length and breadth of the fault slip. Exact correlations do not exist between magnitude and acceleration; however, we made an approximate comparison between magnitude and acceleration using published data to facilitate ease of understanding by stakeholders.

The mean return period in Table 4-1 represents the average number of years between earthquake events of similar severity. New reservoirs designed in accordance with currently adopted codes and standards are intended to satisfy a performance objective of remaining serviceable with minimal repairs following a BSE-1 or Richter Magnitude 6.0 earthquake event and preventing collapse or loss of all water contents following a BSE-2 or Richter Magnitude 7.0 earthquake event. However, some municipalities and water agencies have established a higher performance objective for critical reservoirs that must remain operational following a BSE-2 or Richter Magnitude 7.0 earthquake event.

According to Phillips (1987), a mapped fault is located west of Interstate 5. It is unknown when movement last occurred along this fault. Movement of this fault could conceivably result in a surface rupture in the project area. Seismic evaluation of the Minor Road Reservoirs was performed in accordance with the 2009 International Building Code (IBC 2009). Site classification and soil properties are not known in sufficient detail to determine the site class; therefore, Site Class D, per Table 1613.5.2 of the IBC 2009 was used as a conservative assumption.

Site Class of D was conservatively selected based on our understanding of the probable site soil properties, similar work for the Paxton Road reservoir site, and due to insufficient information to make a less conservative determination of the site class. This value should be verified based on field investigations for either strengthening or replacement of the reservoirs. Site coefficients of Fa = 1.155 and Fv = 1.718 were utilized for adjusting the mapped spectral acceleration response parameters for regional and site specific geologic characteristics. The peak ground acceleration and spectral acceleration response parameters for the smaller earthquake events, Richter Magnitude, 5.0 and 5.5, were adjusted from the 10 percent in 50-year event.

The reservoirs were evaluated for loads due to the weight and pressure of the water at a maximum water surface of 17'-6" above the base of the walls inside the tanks. The reservoirs were not evaluated for the loads due to soil and the water in the soil surrounding the exterior of the tanks as these loads would primarily place the reservoirs in compression and are not considered a controlling load combination. The reservoirs were evaluated for loads from weight and pressure of the concrete and water resulting from horizontal and vertical acceleration due to an earthquake event. The combined effects of the lateral inertia force of the accelerating walls and roof, the lateral impulsive and convective forces associated with the weight of the stored liquid, and the increase in fluid pressure on the walls associated with the vertical acceleration of the fluid were evaluated. Even though the reservoirs have a "hopper" bottom configuration with an increased depth of water, approximately 28'-0" in the center of the tanks, the reservoir walls were evaluated based on a maximum depth of fluid of 17'-6" which is consistent with industry practice. The reservoirs were seismically evaluated for all four earthquake hazard level events shown in Table 4-1.

4.3 Summary of Structural and Seismic Evaluation Findings

The walls of the reservoirs were determined to be significantly under-strength when compared with current building codes and design aids for factored strength loads in the static load combinations and in the dynamic load combinations for earthquake with a Richter Magnitude of either 6.0 or 7.0 or higher. Graphic representations of the structural calculations are included in Appendix F – Structural Calculations. The Portland Cement Association (PCA) guidelines for Circular Concrete Tanks Without Prestressing were utilized in completing the structural calculations. Table 4-2 presents the results of the evaluation of different member actions and load combinations in the form of demand-to-capacity ratios. When a demand to capacity ratio exceeds the value of 1.0, then the member is overstressed.

Table 4-2: Summary of Wall Structural and Seismic Evaluation Findings

	Load Combination					
	Service Loads	Strength Loads [(a)]				
			Dynamic Earthquake Richter Magnitude			
Wall Member Action	**Static [(a)]**	**Static [(a)]**	5.0	5.5	6.0	7.0
Ring Tension – Hinged	0.95	**1.50**	0.72	0.73	0.86	0.97
Ring Tension – Free	0.65	**1.44**	0.93	0.95	**1.20**	**1.40**
Flexure – Vertical Bending	0.36	0.79	0.47	0.48	0.55	0.61
Concrete Shear Strength	0.29	0.49	0.37	0.38	0.43	0.47
Concrete Tensile Strength	**1.03**	-	**1.13**	**1.14**	**1.31**	**1.45**

Note:

(a) This table presents the results of the evaluation of different member actions and load combinations in the form of demand-to-capacity ratios. When a demand to capacity ratio exceeds the value of 1.0, then the member is overstressed.

Utilizing the wall structural and seismic evaluation findings, summarized in Table 4-2, the probability of failure for a specific Richter Magnitude earthquake event over a defined 50-year period was determined. The probability of reservoir failure is solely dependent on the ring tension wall member action. Other wall member actions, flexure (vertical bending), concrete shear strength, and concrete tensile strength would not result in reservoir failure but would manifest as excessive reservoir leakage. The probability of ring tension failure assuming a free base condition, of the reinforcing steel in the walls was determined based on the occurrence of a given earthquake event. The probability that both an earthquake and tank failure would occur simultaneously within 50 years was also calculated. The results of this probability of failure analysis are summarized in Table 4-3. As illustrated in Table 4-3, if an earthquake of 6.0 or larger were to occur, there is a 100 percent probability that the reservoirs would fail. There is only a 10 and 12 percent probability of failure for a 5.0 and 5.5 magnitude event, respectively.

Table 4-3: Probability of Tank Failure Based on Probability of Earthquake Event

Earthquake Hazard Level Richter Magnitude	Probability of Exceedance P(A)	Probability of Ring Tension Failure based on Magnitude P(B|A)	Probability of Earthquake and Tank Failure P(A∩B)
5.0	50% in 50 years	10%	5%
5.5	20% in 50 years	12%	2.5%
6.0 (BSE-1) [(a)]	10% in 50 years	100%	10%
7.0 (BSE-2) [(a)]	2% in 50 years	100%	2%

Note:

(a) Basic Safety Earthquake (BSE)-1 (10% in 50 years) and BSE-2 (2% in 50 years) is a measure of the hazard level.

4.3.1 Circular Conventional Reinforced Tapered Walls

The existing reservoir walls are classified as circular conventional reinforced tapered walls. As illustrated in Table 4-2, there are several conditions under which the wall members are overstressed under all loading conditions analyzed. The walls of the reservoirs were found to be significantly under-strength under normal operating conditions and are vulnerable to failure under multiple conditions during an earthquake event with a Richter Magnitude 6.0 or larger. The walls are not capable of resisting their current maximum water loads with an acceptable factor of safety, and are not capable of resisting forces associated with earthquakes. The fact that the reservoirs have been repaired on several occasions and continue to leak a large quantity of water further supports the conclusion that the walls are overstressed under static loading conditions.

Ring Tension: When the walls were evaluated in ring tension, assuming either a hinged base or free base, the walls are overstressed under static loading conditions based on factored loads in accordance with currently adopted building codes and standards. The walls were also overstressed, assuming free base conditions, under factored loads when Richter Magnitude 6.0 and 7.0 earthquake events induced increased hoop stresses in the walls associated with the sloshing water.

Based on the structural calculations, it is not surprising to observe that the walls are leaking significantly. Under static load combinations, the walls are approaching the capacity of the reinforcing steel in the walls under optimum conditions which assumes no corrosion of the reinforcing steel. Leakage was also observed near the top of the walls where demand-to-capacity ratios exceed 1.0 and stresses in the concrete have contributed to cracking and leakage in the walls. The demand-to-capacity ratio for hinged ring tension under static loading of 0.95 may be the most alarming of all of the numbers in this evaluation. The 0.95 ratio represents the original design condition and does not provide a very large factor of safety in the event any of the horizontal reinforcing steel has corroded resulting in a loss of cross-sectional area and resistance to hoop forces.

<u>Vertical Bending:</u> The walls were evaluated for vertical bending, assuming a hinged-base condition. Results indicate the walls are sufficiently reinforced to resist the vertical bending moments that would develop in the walls. This assumes the vertical reinforcing steel has not corroded. Further evaluation could be completed to determine the theoretical crack width that would accompany the vertical bending moments in the walls, but was not considered necessary for this evaluation.

<u>Concrete Shear:</u> Concrete shear strength was evaluated at the base of the walls. The results indicate the walls have sufficient cross-sectional area to resist the shear loads that would develop throughout the wall cross-section. The evaluation assumed the walls were hinged at their base with concrete compressive strengths as low as 2,500 psi.

<u>Concrete Tensile Strength:</u> Typically, in the design of reinforced concrete members, the tensile strength of concrete is not considered, as any significant cracking in a liquid containing structure is unacceptable. For this reason, the stress in the concrete from ring tension is kept at a minimum to prevent excessive cracking. As shown in Table 4-2, when the walls were evaluated in concrete tensile strength, assuming the walls were hinged at their base, the walls have insufficient tensile strength to satisfy requirements to meet criteria for limiting tensile strength in the concrete to 10 percent of f'c (concrete compressive strength). This is illustrated by the fact that the concrete is overstressed under all loading conditions evaluated. With the maximum concrete tensile stress values as high as 363 psi (or 7.26 times square root of f'c), this value would exceed recommendations by the American Concrete Institute (ACI) for limiting the value to 6 to 7 times for normal weight concrete.

4.3.2 Flat and Sloping Hopper Floors

A structural analysis was not performed on the flat and sloping hopper floors of the reservoirs; however, there are a few findings related to the original construction, layout, and configuration of construction joints in the floors of the reservoirs. The orientation and the size of the construction joint between the sloping floor panels and the sloping footing at the base of the perimeter walls could contribute significantly to leakage in the reservoirs if the sealant material placed in the joints is old, deteriorated, lost elasticity and/or compressibility, and the ability to recover its original thickness. This is also applicable to radial and circumferential construction joints in the flat and sloping hopper floors of the reservoirs.

Any attempt to investigate, repair, or strengthen the floors of the reservoirs should include lowering water levels outside of the reservoir below the bottom of the reservoir before lowering of the water level inside the reservoirs to prevent exterior hydrostatic pressure. Buoyant forces on the underside of the floor could cause damage to the floors or damage to the construction joints by forcing sealant material upward and out of the construction joints. The reservoirs could potentially be at risk of a catastrophic failure if the water level were to be lowered rapidly in the reservoirs due to a pipeline break downstream causing a sudden loss of water.

4.3.3 Triangulated Strut and Panel Aluminum Dome Roofs

The triangulated strut and panel aluminum dome roofs were not evaluated as part of the seismic and structural evaluation of the reservoirs. The aluminum dome roofs are not considered a controlling factor in the determination of whether the reservoirs are repaired or strengthened. An evaluation of the triangulated strut and panel aluminum dome roofs could result in the need to increase the anchorage forces between the roof structure and the walls to meet current building codes and design standards. If the reservoirs are rehabilitated instead of replaced, the panel aluminum dome roof anchorage would need to be evaluated as part of the rehabilitation design.

4.3.4 Earthquake Maximum Wave Oscillation and Freeboard

New reservoirs are designed with provisions to accommodate the maximum wave oscillation generated by earthquake acceleration. The maximum vertical displacement of the water surface in the reservoirs associated with the various earthquake magnitudes is summarized in Table 4-3.

Table 4-4: Earthquake Maximum Wave Oscillation

Dynamic Earthquake Richter Magnitude	5.0	5.5	6.0	7.0
Maximum Vertical Displacement (feet) [(a)]	0.5	0.6	1.5	2.2

Note:

(a) Vertical Displacement refers to wave height during a seismic event

Given the City's current maximum operating water depth of 17'-6", the Minor Road Reservoirs have 1'-0" of freeboard. A minimum of 2'-0" of freeboard is recommended to prevent damage of the dome roof panels. The City could either elect to adjust operating levels in the reservoir to provide 2'-0" of freeboard or assume that some of the panels will be damaged in a large magnitude earthquake.

Section 5: Rehabilitation or Replacement Alternatives

Previous reports have looked at several alternatives for repair, strengthening, or replacement of the reservoirs. A description of each of the alternatives for repair, strengthening, or replacement of the reservoirs is provided below along with the limitations, costs, and estimated life of the reservoirs.

5.1 Further Investigations, Repair, or Strengthening

Any attempt to investigate, repair, or strengthen the Minor Road Reservoirs should begin with an assessment of the condition of the concrete and reinforcing steel in the walls of the reservoirs. To facilitate investigation and limit damage to the reservoirs, the following steps should be taken:

- Place Piezometers around the two reservoirs to monitor and measure water levels around the reservoirs. The piezometers should be installed to a depth below the level of the reservoir(s) floor.
- Remove backfill placed around the perimeter of the walls of the reservoirs by excavating to the bottom of the walls, soil beneath the floor would not be disturbed. The valve pit is in close proximity to the reservoirs. Due to limited space, additional shoring and alternative excavation methods may be required to prevent damage to either structure as a result of excavation activities.
- Drain reservoir. Water levels outside and inside the structures should be closely monitored throughout this operation and water levels inside the reservoirs maintained at a higher than outside water level.

After successfully draining reservoirs, the condition of the reinforcing steel inside and outside the structures could be investigated through exploratory concrete removal or coring. If the condition of the reinforcing steel is determined to be favorable, then attention could focus on the repair of cracks, spalls, and other leaking areas in the concrete walls potentially attributable to poor quality concrete. If the condition of the reinforcing steel is determined to be unfavorable due to significant corrosion, then this would strengthen the recommendation for complete replacement of the reservoir structures. After repair completion, replace removed backfill with compacted structural fill.

5.2 Alternative No. 1 – Repair of the Reservoirs

Description: Alternative No. 1 – Repair of the Reservoirs would be limited to maintaining the reservoir structures in their present structural condition and preventing further deterioration.

<u>Reservoir:</u> Repairs would be limited to replacing or correcting those portions of the structure necessary to provide water containment or provide structural support for a waterproofing system. Leakage from the reservoirs would be minimized through the addition of an interior waterproofing system consisting of one of the following:

- An unbonded geomembrane
- Hypalon liner attached to the tops of the interior walls of the reservoirs
- A bonded fluid applied urethane or asphaltic emulsion sprayed on the floors and walls of the reservoir.

Piping: This alternative would also include the rehabilitation of the individual reservoir inlet/outlet and drain piping using a CIPP product and replacement of the individual isolation valves. The reservoirs will need to be completed drained in order to install complete this rehabilitation work. Pipe rehabilitation of the 16-inch inlet/outlet and 12-inch drain would occur from the point of penetration at each reservoir to respective common headers for each piping system. New individual isolation valves would be installed in the valve pit. The condition of the common piping headers will need to be assessed at the time the individual pipes are rehabilitated. Depending on the assessed condition, it may be necessary to replace a portion of the common headers to effectively minimize leakage from the reservoir system. The previously abandoned 12-inch piping would need to be removed from the valve pit to improve access to the 16-inch inlet/outlet piping and the 12-inch drain piping. Additionally, condition of the previously abandoned 12-inch pipe between the reservoirs and the valve pit would be checked to confirm it was properly abandoned and is not currently a source of leakage.

Limitations:

The following concerns ***would still need to be addressed*** with this alternative:

- Protect the water supply from a loss of water storage in a seismic event. If significant ground movement were to occur, it is possible that an unbonded liner or fluid applied liner could be damaged, torn, or loosen structural support that could result in damage to the liner and loss of contents stored in the reservoirs.
- Foundation support: Correct the loss of support to the floors of the reservoirs and potential settlement once the water level in the surrounding subgrade is lowered.
- Prevent groundwater from entering the reservoirs and getting between the unbounded or loose liner and the structural floors and walls of the reservoirs.
- Correct the insufficient freeboard in the reservoirs. City would need to operate reservoirs at a lower water level to prevent damage to the reservoir roofs.
- Operation and maintenance challenges: The installation of an unbonded or loose liner can be a significant maintenance and operational challenge for the reservoirs if water gets into the void space between the liner and the concrete structure. If a bonded fluid applied waterproofing membrane is installed on the walls and floors of the reservoirs there are limitations as to the size of cracks the liner is capable of spanning.
- Water age and tank mixing: Additional improvements would be necessary to address the effects of a common inlet/outlet and improve reservoir turnover.

Extended Life: Properly installed and warranted, the addition of a waterproofing system liner could extend the life of the reservoirs for an additional 10 to 20 years.

Estimated Cost: The estimated construction cost of this alternative is $1.8 million.

5.3 Alternative No. 2 – Strengthening of the Reservoirs

Description: Alternative No. 2 – Strengthening of the Reservoirs would include structural and seismic strengthening of the walls, wall footings, wall to floor connections, and anchorage of the dome roof structures to satisfy currently adopted building code and national standards. In addition to the strengthening improvements, either structural or non-structural systems, such as waterproofing membrane liners or coating systems as previously presented for Alternative No. 1, would be incorporated to minimize leakage from the reservoirs. This alternative would protect the water supply from a loss of water in a seismic event.

Seismic strengthening of the walls would be to the maximum level capable of resisting the Richter Magnitude 7.0 earthquake event. Strengthening of the reservoirs walls could be with achieved by one of the following methods:

1. A new 3-inch-thick minimum prestressed concrete reinforced wall constructed around the perimeter of the reservoirs with pneumatic (shotcrete) protecting the high strength galvanized prestressing strand; refer to Figure 3 located in the Figures section of this report.

2. A new 5-inch-thick minimum reinforced concrete wall constructed around the perimeter of the reservoirs placed with either conventional concrete placement methods or pneumatic (shotcrete) methods and additional conventional reinforcing as shown on Figure 4 located in the Figures section of this report.

Due to the proximity of the reservoirs to the valve pit, it may be necessary to partially demolish the valve pit and gate house.

<u>Piping:</u> This alternative would also include the rehabilitation of the individual reservoir inlet/outlet and drain piping using a CIPP product and replacement of the individual isolation valves. The reservoirs will need to be completed drained in order to install complete this rehabilitation work. Pipe rehabilitation of the 16-inch inlet/outlet and 12-inch drain would occur from the point of penetration at each reservoir to respective common headers for each piping system. New individual isolation valves would be installed in the valve pit. The condition of the common piping headers will need to be assessed at the time the individual pipes are rehabilitated. Depending on the assessed condition, it may be necessary to replace a portion of the common headers to effectively minimize leakage from the reservoir system. The previously abandoned 12-inch piping would need to be removed from the valve pit to improve access to the 16-inch inlet/outlet piping and the 12-inch drain piping. Additionally, condition of the previously abandoned 12-inch pipe between the reservoirs and the valve pit would be checked to confirm it was properly abandoned and is not currently a source of leakage.

Limitations:

The following concerns <u>*would still need to be addressed*</u> with this alternative:

- Foundation support: Correct the loss of support to the floors of the reservoirs and potential settlement once the water level in the surrounding subgrade is lowered.

- Prevent groundwater from entering the reservoirs and getting between the liner and the structural floors and walls of the reservoirs.
- Correct the insufficient freeboard in the reservoirs. City would need to operate reservoirs at a lower water level to prevent damage to the reservoir roofs.
- Operation and maintenance challenges: The installation of an unbonded or loose liner can be a significant maintenance and operational challenge for the reservoirs if water gets into the void space between the liner and the concrete structure. If a bonded fluid applied waterproofing membrane is installed on the walls and floors of the reservoirs, there are limitations as to the size of cracks the liner is capable of spanning.
- Water age and tank mixing: Additional improvements would be necessary to address the effects of a common inlet/outlet and improve reservoir turnover.

This alternative could reduce the amount of waterproofing system liner or coating placed on the interior walls of the reservoirs but would still require the systems installed on the floors of the reservoirs. This alternative could include the addition of pressure relief valves in the floors of the reservoirs to prevent structural damage to the floor from a buoyancy event.

Extended Life: With structural improvements the life of the reservoirs could be extended for an additional 20 to 40 years.

Estimated Cost: The estimated construction cost of this alternative is $2.4 million.

5.4 Alternative No. 3 – Replacement of the Reservoirs

Description: Alternative No. 3 – Replacement of the Reservoirs would include construction of a new approximately 2.0 MG strand-wound circular prestressed concrete reservoir in accordance with the building code and national standards. The City could choose to construct a single 1.0 MG replacement reservoir and then attempt to rehabilitate one of the existing reservoirs, but the costs of constructing the one reservoir and rehabilitating the second would be prohibitive. This alternative would allow the City to abandon and demolish the existing aging reservoirs. This alternative would also allow the City to make adjustments in the hydraulic profile of the new reservoir by adjusting the maximum and minimum water levels and optimize the capacity of the reservoir to accommodate future demands. The estimated cost for this alternative includes the costs of demolition of both of the existing reservoirs on the site.

Limitations: There are no limitations associated with this alternative.

Extended Life: A new conventional or prestressed concrete reservoir designed to satisfy the building code and national standards for water storage reservoirs would have a design life of 40 to 80 years.

Estimated Cost: The estimated construction cost of this alternative is $4.1 million including demolition of the two existing reservoirs.

Table 5-1: Evaluation of Rehabilitation or Replacement Alternatives

Alternative	Description	Advantages	Limitations	Extended Life	Estimated Cost	Annual Cost
No. 1	Repair Reservoirs	• Leakage eliminated with liner or coating. • Leak eliminated by replacement/rehabilitation of connecting supply and drain piping. • Least construction cost.	• Repairs limited to water containment. • Damage to liner or coatings thru settlement or earthquake. • No protection from earthquakes. • Requires additional investigation, dewatering, and excavation around reservoirs. • Requires exploration removal to identify material condition including corrosion. • Does not protect floors from settlement or buoyancy. • Limited ability to find and correct lost foundation support material. • Maintenance required for loose liners if water is allowed in space between structure and liner. • Water level needs to be lowered to protect against sloshing damage. • Limitations in liners spanning joints or cracks. • Liability to City associated with inundation of adjacent private properties, schools, and Interstate 5.	10 to 20 yr	$1.8 M	$154,000 to $311,000
No. 2	Strengthen Reservoirs	• Leakage in walls eliminated through construction of new exterior wall. • Leakage in floors eliminated with liner or coating. • Leak eliminated by replacement/rehabilitation of connecting supply and drain piping. • Walls reinforced to resist largest Richter Magnitude 7.0 earthquake forces.	• High construction cost. • Damage to liner or coatings thru settlement. • Requires additional investigation, dewatering, and excavation around reservoirs. • Requires exploration to identify material condition including corrosion. • Does not protect floors from settlement or buoyancy unless pressure relief valves are included. • Limited ability to find and correct lost foundation support material. • Maintenance required for loose liners if water is allowed in space between structure and liner. • Water level needs to be lowered to protect against sloshing damage.	20 to 40 yr	$2.4 M	$113,000 to $234,000
No. 3	Replace Reservoirs	• New structure designed to recently adopted codes and standards. • New prestressed concrete reservoir with improved seismic performance design detailing. • Abandon and/or demolish existing reservoirs. • Hydraulics/mixing and capacity can be optimized. • Low maintenance.	• Highest construction cost.	40 to 80 yr	$4.1 M	$75,000 to $179,000

5.5 Engineer's Opinion of Probable Construction Cost

Conceptual level engineer's opinion of probable construction costs for the three alternatives are included in Table 5-1 and the detailed backup information for these cost estimates is included in Appendix G. The probable construction costs are based on repairs or strengthening for two reservoirs or replacement with one new reservoir including demolition of the two reservoirs. The probable construction costs are based on unit costs for sitework, earthwork, concrete, and miscellaneous materials, labor and equipment obtained from R.S. Means 2012 Cost Data, quoted information received from material suppliers, fabricators, and contractors, and information obtained from similar type projects.

The conceptual level opinion of probable cost include a 10 percent markup on costs for mobilization, insurance and other front end Special Provision requirements; sales tax (7.9 percent), contractor overhead and profit (15 percent), estimate contingency (20 to 40 percent)and an escalation of 2 percent to the midpoint of construction. A 40 percent estimate contingency was used for Alternatives 1 and 2 due to a higher level of unknowns associated with the repair and strengthening options. The estimate contingency of 20 percent was used on the replacement option due to the conceptual nature of the replacement design, Conceptual level opinions of probable cost are considered to have an accuracy range of +50 percent to -30 percent.

5.6 Economic Analysis

In order to evaluate the time value of money and the equivalence of the three alternatives, an economic analysis was performed to determine which alternative had the best time value of money. An economic analysis was performed for: 1) repairing the existing reservoirs, 2) strengthening the existing reservoir, or 3) replacing the existing reservoirs. In the economic analysis, several Discount Rates, ranging from 1 to 3 percent, were utilized to account for the difference between the rate of return on invested money and the inflation rate. Assumptions used in the economic analysis are summarized below:

Alternative No. 1 – Repair Reservoirs: Annualized cost for the repairs would be paid over the 10- to 20-year remaining life of the reservoirs; a new reservoir would be constructed in 10 to 20 years to replace the existing reservoirs. Payment for the repair and eventual replacement would start now and end in 90 to 100 years.

Alternative No. 2 – Strengthen Reservoirs: Annualized cost for the strengthening would be paid over the 20 to 40-year remaining life of the reservoirs; a new reservoir would be constructed in 20 to 40 years to replace the existing reservoirs. Payment for the strengthening and eventual replacement would start now and end in 100 to 120 years.

Alternative No. 3 – Replace Reservoirs: A new reservoir would be constructed now and it would be paid for over the 40 to 80-year life of the structure.

The results of the economic evaluation indicate it is more cost efficient to replace the reservoirs rather than repair or strengthen the reservoirs. Based on the economic analysis, Alternative No. 1 – Repair of the Reservoirs is not considered a favorable option.

Section 6: Findings and Recommendations

6.1 Summary of Findings

As shown in past leakage test reports and in other documentation attached to this report, the reservoirs have several problems that need to be addressed to avoid a potential failure. The structural and seismic deficiencies in the reservoirs along with other potential risks are summarized below:

1. The circular conventional reinforced tapered walls are under-reinforced for circumferential hoop stresses. The amount of under-reinforcement in the walls is greater in the top one-third of the walls than in the bottom of the walls.

2. The thickness of the walls is insufficient to maintain the tensile stress in the concrete at acceptable levels under either static or dynamic loading conditions.

3. The weight of the wall is capable of resisting the bending moment and the overturning moment on the tank associated with an earthquake; however, as previously noted in other studies, the small size of the footings means that the subgrade materials may be overstressed in bearing resulting in excessive rotation of the walls and separation of the joint with the floors.

4. The lack of reinforcing steel between the wall, wall footing, and the floor slab results in the walls relying entirely on passive pressure to resist base shear, which due to the small size of the footing is insufficient. It is likely that in an earthquake event, the outward movement of the walls will increase lateral earth pressure, resulting in additional separation in the joint with the floors leading to leakage of water from the reservoirs and undermining of the subgrade material supporting the floors. In a seismic event, the lack of reinforcing steel is considered the greatest contributor to the risk of failure.

5. Construction joint age between the flat and sloping hopper floor panels and between the sloping hopper floor panels and perimeter walls is most likely contributing significantly to leakage from the reservoirs. Construction joint and expansion joint filler materials should be inspected and replaced.

6. Insufficient freeboard at current maximum operating water depth could lead to roof damage in a seismic event.

7. The City is currently unable to remove either one of the reservoirs for inspection and repairs while maintaining service in the other reservoir due to valving issues. This drastically reduces the reliability that two reservoirs provide at the site.

8. Reservoir leakage could contribute to high groundwater and a buoyancy failure of the drained reservoir floor. The reservoirs could be at risk of a catastrophic failure if the water level were to be lowered rapidly or if a pipeline were to break downstream causing a sudden loss of water. Under normal operating conditions, buoyancy failure is considered the greatest risk.

9. The reservoirs are located in close enough proximity to a seismic fault region that strong ground motion could contribute to failure of the reservoirs.

10. The reservoirs are not capable of resisting their current maximum water loads with an acceptable factor of safety, and are not capable of resisting forces associated with earthquakes. The reservoirs are vulnerable to significant damage and loss of contents. Failure to act promptly could be very expensive and unsafe, especially if a moderate or large earthquake were to occur.

11. The walls of the reservoirs were found to be vulnerable to failure under multiple conditions during an earthquake event with a magnitude 6.0 or larger.

12. The reservoirs are approaching 90 years in age; a 50-year usable life for this type of facility would be considered acceptable.

13. The backfill soils around the tanks are saturated as a result of leakage from the walls, floors, and possibly connecting the pipe, making soils susceptible to mobilization in an earthquake due to the sloping site. This movement could result in loss of support to the tank walls and floors and does not satisfy required factors of safety associated with slope stability. Soil movement could damage roadways and structures located to the west of the reservoir site. At some point, a catastrophic failure of one or both of the reservoirs may occur, endangering the public and residents on the adjacent private property below the inundation zone and affecting the water supply and fire protection. Damage could be significant to North Minor Road, the Three Rivers Christian School/Cornerstone Junior/Senior High School, and Interstate 5 located beneath the reservoirs. The potential liability is significant.

14. It appears that damage to the walls of the tanks has significantly increased over time. Concrete holes and spall areas are significantly larger in 2007 reports documenting repairs when compared with repairs completed in 2005.

15. Minor Road Reservoirs can be taken offline without negatively impacting the City's Main distribution zone. The Paxton Reservoirs would become the City's primary storage while the Minor Road reservoirs are offline.

16. Paxton Road Reservoirs are supplied by a single 16-inch asbestos concrete pipeline which crosses beneath Intestate 5.

6.2 Recommendations

Kennedy/Jenks Consultants recommends that the reservoirs be replaced. An economic comparison of the three alternatives shows that it is more economically practical to replace the reservoir than to either repair or structurally and seismically strengthen the reservoir. Kennedy/Jenks Consultants further recommends that the two reservoirs be replaced with a single reservoir with approximately 2.0 MG capacity, designed in accordance with the currently adopted codes and standards for water containment structures for potable water.

If the City intends to maintain the existing reservoirs in service for an extended period of time, Kennedy/Jenks Consultants recommends additional inspection and temporary remedial actions to improve the condition of the reservoirs. Additional inspection and remediation would include draining the tanks, while monitoring groundwater levels, inspecting the concrete of the walls, floors and reinforcing steel both above and below grade. Any defective concrete should be removed and replaced using pressure injection into cracked concrete. The procedures outlined in the further investigations section should be followed to minimize potential damage to the existing structures. Kennedy/Jenks Consultants should be notified if significant spalling, cracking, corrosion, or leakage is found during such an investigation.

While the model results indicate the City can remove the Minor Road Reservoirs from service without negatively impacting the City Main service zone, it is recommended that the City complete flow testing prior to removing the Minor Road Reservoirs from service for replacement. Flow testing should be completed both with the reservoirs on and offline. To improve reliability and provide redundancy, it is further recommended that the City install a redundant Interstate 5 pipeline crossing to supply Paxton Road as discussed in Section 3 of this report.

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February 24, 2010

Mr. Tom Gower
Gibbs and Olson, Inc.
P. O. Box 400
Longview, WA 98632

RE: City of Kelso – Reservoir Evaluation
KGA Project No. 10-051-00

Dear Tom:

Our firm has been retained to perform structural evaluations of two, identical, 90-foot diameter concrete water reservoirs in Kelso, Washington that are currently leaking water. The desired results from these evaluations would be to make recommendations whether the reservoirs could be:

Lined to stop the leaking,

Lined and strengthened to remedy any strength deficiencies found during the evaluation, or

Whether there are factors that would make replacing the existing reservoirs with new water holding structures the most feasible.

<u>Bases of Evaluations:</u>

We have based our analyses, evaluations and recommendations on the items and publications listed below:

- My visual observations during a site visit on January 19, 2010
- The existing construction documents and project specifications dated June and July 1924 and approved by Geo. H. Norris, City Engineer and Albert Morris, Mayor
- The 2006 International Building Code
- ACI-318-08 "*Building Code Requirements for Structural Concrete and Commentary an ACI Standard*"
- ACI-350-06 "*Code Requirements for Environmental Engineering Concrete Structures and Commentary (ACI-350R-06) An ACI Standard*"
- ACI-350.3-06 "*Seismic Design of Liquid-Containing Concrete Structures and Commentary An ACI Standard*"
- PCA Publication "*Design of Liquid-Containing Concrete Structures for Earthquake Forces*" – 2002
- PCA Publication "*Circular Concrete Tanks Without Prestressing*"
- Excel program developed by Kramer Gehlen & Associates. This program analyzes circular concrete tanks based on criteria in the publication listed immediately above for static load cases.
- Hand calculations for seismic forces on the reservoirs. These calculations are approximations based on criteria in the ACI and PCA publications identified above.

Site Observations:

On January 19, 2010 I visually observed the two concrete reservoirs in the company of Mr. Mike Kardas, P.E., Civil Engineer for the City of Kelso, and yourself.

The tanks are circular in shape and constructed of reinforced concrete. They both sit on a sloping site and are partially buried. The tops of the concrete wall heights above the finish grade vary from 4 feet on the uphill side of the tanks to approximately 8 feet on the downhill sides of the structures. There are light-weight domed roofs over the concrete reservoirs that appear to be constructed of fiberglass and aluminum.

I observed many cracks in the concrete walls above the ground, with several of them actively leaking. There are places where the cracks had been patched in an attempt to stop the leakage. The patching worked with varying degrees of success, with some of the patched locations still actively leaking.

I also observed several areas of spalled concrete, some of which had been patched. I did not notice leaking at the concrete spalls.

While walking around to the downhill side of the reservoirs, it was noted the ground became very soft due to soil saturation. There was water at the ground surface ponding in depressions in the soil.

You mentioned during our site observation that a gravel-filled trench drain, with a drain pipe, had been installed across the downhill side of the reservoirs and the pipe had been day-lighted into an existing road side ditch several hundred feet to the north. You and I walked to where the pipe emptied into the ditch. The flow rate was steady but no one had determined what the cubic feet per minute rates were.

You also mentioned during our visit that the water coming out of the drain pipe had been tested and found to contain fluoride. The water in the reservoirs also has fluoride. This is evidence that the water is coming from leaks in the reservoirs. The flow rates I observed emerging from the cracks in the exposed concrete surfaces were considerably less than the amount of water flowing from the drain pipe, which leads me to believe the cracking below grade is greater than above grade.

Review of Existing Drawings and Specifications:

Review of the existing drawings indicates the reservoirs to have inside diameters of 90-feet. The exterior walls of the tanks are vertical and taper from 9” thick at the tops to 15” thick at the bottoms, and are approximately 18’-6” from the wall tops to the tops of the bottom slabs of the tanks. The bottoms of the tanks are approximately 6” thick concrete slabs. The drawings do not call out reinforcing being typical in all areas; however, they do note ”Mesh reinforcing in this slab panel”. This leads me to believe the slabs are not reinforced in all areas. The drawing shows footings poured monolithically with the walls. The footings are 5’-0” wide, flat on the bottoms and taper at both ends from 8” to approximately 20” where they meet the walls.

The drawings show concrete wall sections that indicate the wall reinforcing, with both horizontal hoop reinforcing and vertical reinforcing. The reinforcing all is noted as square bar. A copy of this wall section is included in the Appendix (see sheet SUP-1).

Review of the specifications indicates the reinforcing was to have been ASTM A-15 – Structural Grade reinforcing and ". . .shall provide a mechanical bond with concrete at frequent intervals". ASTM A-15 – Structural Grade reinforcing has a yield strength FY=33,000 pound per square inch (psi). The mechanical bond in the specifications refers to deformed bar. A sheet showing ASTM A-15 design criteria is included in the Appendix (see sheets SUP-2 and SUP-3).

The concrete specifications indicate three classes of concrete, Class B, Class C and Class E. My interpretation of the specifications is that the reservoirs would be constructed of Class B concrete. This is the one with the highest cement content per cubic yard and likely would attain the highest design strength. It is specified to have 1.57 barrels of cement per cubic yard. A barrel of cement is 376 pounds (this would be referred to in today's terms as a 6.3 sack mix). By experience with old structures, this would, in our opinion, likely produce a 28-day design strength of 2500 psi and possibly up to 3000 psi.

Analyses:

As there is no reinforcing in the base slabs except at one panel, no connections of the walls to the concrete base slabs are indicated on the drawings and the footings do not appear to be reinforced heavily enough to assume that the bases of the walls are "Anchored and Contained". This "Unanchored Uncontained Flexible Base" condition would not be permitted in regions of high seismicity, (seismic design categories D, E and F). Kelso is in Category D. A diagram showing this condition is included in the Appendix (see sheets PCA-1 and PCA-2).

For the lack of restraint of the wall base, the assumption was made to check the hoop tension forces that vary linearly from zero at the tops to 62.4 x H x D/2 at the bottoms. Hoop tension is a force developed in circular structures by the outward pressure of the water pushing against the sides of the tank walls while being resisted by the horizontal reinforcing in the walls trying to hold the walls together.

If the condition existed that there were large mat footings to restrain the wall bases from deflecting laterally, the hoop tension stresses in the walls would vary linearly from zero at the tops to a maximum value at a location 0.2 x H to 0.3 x H above the tank bottoms, then return to zero again at the wall bottoms.

An analysis was performed with the Excel spreadsheet program developed in-house by KGA, with the bases restrained from moving laterally and the walls hinged at the bases. The hinged base conditions were assumed due to the minimal sizes of the footings and minimal footing reinforcing being sufficient to resist rotations in the footings. This analysis was done to see what the stresses would be in the horizontal reinforcing with this assumption: The actual wall forces are somewhere in between the two analysis methods.

Hand calculations were performed to determine the seismic forces in the walls and the wave heights in the tanks during a design seismic event, utilizing well-established methodology in ACI-350, and charts and tables published in PCA publication "*Design of Liquid-Containing Concrete Structures for Earthquake Forces*".

Additionally, the lap splice lengths were computed by hand following criteria in ACI-318, and were compared to office-generated tables.

Results of Analyses:

Static Pressure Analyses:

The results from the hand analyses studying the static fluid pressures that vary from zero fluid pressure at the tops to 62.4 x H at the bottoms indicated large hoop tension over-stresses at all levels in the reservoirs except at the top two bands of reinforcing. These over-stresses ranged from 47% to 73%, assuming the sanitary coefficients required today are applied to these conditions.

I am unable to find the design standards from 1924. Perhaps they did not have the "sanitary coefficients" that are in use today to give a higher safety factor for tensile stress design and use only the working stress design safety factors on the rebar yield. If that were the case, only one of the bands would be over-stressed. These pages can be found in the calculations portion of the Appendix (see sheets HT-1 through HT-3)

The results of the Excel spread sheet analyses, where the bases were modeled as being restrained from lateral movements, also indicated over-stresses in the circular horizontal steel. These ranged from 0% at the tops and bottoms of the tanks to 307% at the upper halves of the tank wall hoop stresses in the circular horizontal steel. The Excel analysis results can be found in the calculations section of the Appendix (see sheets EXCEL-1 and EXCEL-2).

These force levels were typically smaller near the bottoms and higher near the tops of the tank than with the linear variations of force analysis; however, they still indicate the reservoir walls are under-reinforced.

Hoop stress failures are tensile failures of the horizontal circular steel. Tensile failures are a brittle (sudden) failure mode. For this reason the design codes have mandated significant safety factors on this element of design.

Review of the reinforcement shown on the drawings and doing hand calculations with linear hoop stresses increased from the tops to the bottoms, indicates the original design engineer must have made this same assumption as the steel area in the wall is significantly larger at the bottoms of the walls than the tops`.

Seismic Analyses:

The seismic forces analyses to the walls were performed by hand as mentioned above. These results were combined with the forces due to static pressure with the load combinations as prescribed by the design codes.

The results of the seismic analyses are an approximation of the maximum hoop tensions as mentioned in the PCA publication " Design of Liquid-Containing Concrete Structures for Earthquake Forces". However, the results of the analyses provide good indications of the tensile hoop stresses. They do continue to say that ". . .for shallow tanks (D>>H) out-of-plane bending effects are small and can be neglected".

The only reasonable way to get results that are not an approximation is to model the tank with a finite element computer program.

The seismic calculations show that the lateral pressures of the water against the walls increase at all levels along the full height. This increases the hoop tension forces in the reinforcing as well (see calculation sheets SEIS-1 through SEIS-9).

Wave Height Analyses Due to a Design Seismic Event:

Calculations of the maximum wave heights due to a design earthquake were done by hand calculations based on the PCA publication on seismic design of concrete structures. The calculated maximum wave heights are approximately 2.0 feet. With the operating freeboard of 1'-0" there is potential for damage to the roof system.

As the water in the reservoir is potable and does not contain hazardous materials, overflow of the tank is not a critical issue and the forces that the roofs could resist are likely much smaller than the concrete walls can resist. While the roofs may be severely damaged, the concrete structure damage would be minimal due to wave action (see the wave height calculations on sheet SH-1 in the Appendix).

Lap Splice Calculations:

Calculations were performed to compare the current lap splice length requirements with the lap splices shown on the contract documents.

The 1" square horizontal reinforcing bar with a 4'-0" lap splice shown on the contract documents was compared to a #9 steel reinforcing bar (also 1.0 square inches in cross-sectional area). The hand calculations indicate an 81" lap splice length would be required using current design standards. Lap splices have steadily increased over the years, and our office is not surprised by this large difference in lap length (see calculation sheet LAP-1).

It should be mentioned that the deformation requirements to provide mechanical anchorages of the bars were not as strict then as they are now, making the short lap splice all the more critical.

Recommendations and Conclusions:

Lining the Reservoirs Without Strengthening Measures:

The reservoirs could be lined to prevent leaking and be serviceable for an unknown number of years, provided a design level seismic event does not occur. If a design level earthquake were to occur, one could expect separation of the concrete tank bottoms and the walls, as the base slabs are neither reinforced nor connected to the walls. This could cause major leaking from the voids that would occur.

It is highly likely that tensile hoop stresses would increase to failure levels, causing splitting of the concrete walls and additional leaking. At that point the reservoirs would be unusable.

This does not address the rusting that may have occurred in the reinforcing steel over the past 86 years, nor the additional corrosive action that will continue to take place due to water infiltration from the outsides of the tank walls through the existing cracks in the walls.

If the Owner wishes to take this course of action, it would, in our opinion, be prudent to investigate the levels of rusting that have taken place to date by chipping out at several crack locations that are currently leaking. The tanks should be drawn down to investigate rust levels at lower levels on the tanks.

This still will not reveal the level of corrosion that has occurred in the reinforcing in the layer of steel near the outside face of the wall. That could only be discovered by excavating around the outside of the tank and doing exploratory work.

Lining the Reservoirs and Remedial Strengthening:

The reservoirs may be able to be strengthened to be able to resist current code requirements due to forces imposed by soil, liquids and earthquake. It may be possible to construct new walls and foundations inside the existing reservoirs.

The walls would be approximately 16" thick and the mat footings would be approximately 24 inches thick. This would result in reductions of tank capacities of approximately 157,000 gallons.

This option would be dependent on what a geotechnical study would give as foundation options and bearing capacities. This design option would need to rely on pressure relief valves to prevent buoyancy and uplift pressure from damaging the tank bottoms in the event the tanks were empty during a high ground water event.

Construct New Water Reservoirs:

Building new reservoirs or a single larger one to replace the two would give the Owner a new tank sized for the current demands and projected future demands. The new tank would be designed to current code levels for all loadings, soil, fluid pressures, roof snow load and forces created due to seismic loading.

In our opinion, the Owners should seriously consider this option. One of the two current reservoirs could be left operable during the construction of the new reservoir.

The following pages are photographs taken during our site visit.

I trust the above information is satisfactory for your needs. Please telephone our office if you would like clarification or additional input relative to our opinions.

Sincerely,

David Goff

David Goff, P.E., S. E.
Kramer Gehlen & Associates, Inc.

Enc.
py

APPENDIX

PHOTOGRAPHS

Figure 1: Two Reservoirs

Figure 2: Two Reservoirs At Uphill Side Looking South. Notice The Standing Water Against The Near Reservoir Wall.

Figure 3: Water At Ground Surface At Downhill Side Of South Reservoir.

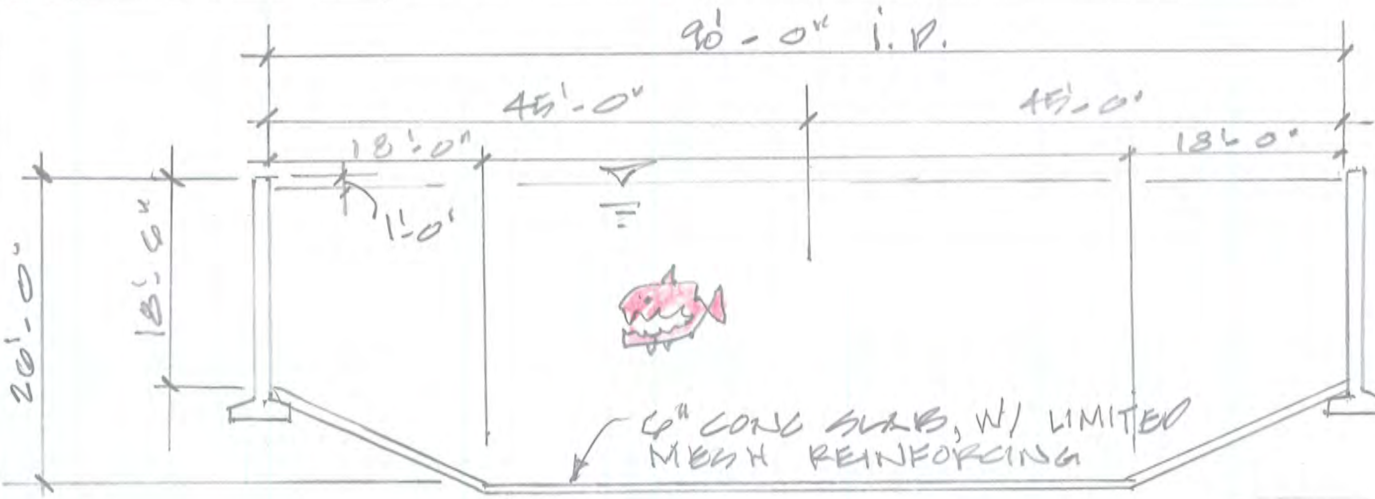
Figure 4: Active Leak, One Of Many (Typical Of Both Reservoirs).

Figure 5

Figure 6: Locations Where Attempts To Patch Leaks Have Not Been Successful.

Figure 7: The Water Flowing From The Drainage System.

CALCULATIONS



DESIGN CRITERIA:

REBAR ASTM A-15 $F_y = 40$ KSI

CONCRETE

$\gamma_{H_2O} = 62.4$ PCF

EFP SOIL = 90 PCF/FT

EFP SEISMIC = 25 PCF INVERTED @ $\bar{h} = 0.6h$ ↑

$\mu_f = 0.35$

PASSIVE PRESSURE = 200 PCF/FT

ZIP CODE 98624

PROJECT KELSO WATER RESERVOIR | TANK SECTION & DESIGN CRITERIA

CLIENT GIBBS & OLSON

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DATE 2-22-2010
DESIGN DG
PROJECT NO. 10
SHEET RE-1

* FTG IS NOT REINFORCED HEAVILY ENOUGH TO FIX BASE AGAINST LATERAL MOVEMENT & ROTATION

$P = (17.0')(62.4) =$

$= 1061$ PSF

HOOP TENSION$_{BOTTOM}$ $= 1061\left(\frac{90'}{2}\right) = 47.74^{k}$

$F_{TU} = 1.65(1.7)(47.74) = 133.9$

$A_{S\,WS} = 1.65(47.74) \div (0.5)(33) = 4.8$ /FT

$A_{S\,ULT} = 133.9 \div 0.9(33) = 4.5$ in²/FT

A_{STL} PROVIDED $= 3\,in^2 + 0.25\,in^2 = 3.25\,in^2$/FT

*

AVERAGE OVER 3'-8" @ BOTTOM

$P_1 = 835$

$P_2 = 1061$

$F_T = \left[\frac{1061 + 835}{2}\right](45')(3.66') = 156.1^{k}$

$A_{S\,WS} = 15.6\,in^2$ $> 20\%$

A_S PROVIDED $= 13.0\,in^2$

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PROJECT: KECO

CLIENT: GIBBS & OLSON

✓ HOOP TENSION
BASE FREE TO MOVE

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DATE: 2-23 2010

DESIGN: DG

PROJECT NO. 10061

SHEET: HT-1

1 / 3

NEXT SECTION 3 - 1" ⌀ @ 7"

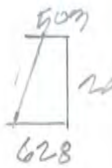
$$F_{HOOP} = \left(\frac{629+738}{2}\right)\left(\frac{21}{12}\right)(45') = 53.8^k$$

$A_{s\,WS} = 1.65(53.8) \div 16.5\,ksi = 5.4\ IN^2$ (54% OVER)

$A_{s\,ULT} = 1.65(1.7)(\quad) \div 0.9(33) = 5.1\ in^2$

✱ $A_{PROVIDED} = 3(1) + 2(0.25) = 3.5\ in^2$ N.G

NEXT SECTION 3 - 1" ⌀ @ 8" o.c.



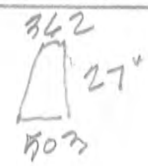
$$F_{HOOP} = \left(\frac{628+503}{2}\right)2(45) = 50.9^k$$

$A_{s\,WS} = (\quad)(\quad) \div 16.5 = 5.1\ in^2$ (56% OVER)

$A_{s\,ULT} = (\quad)(\quad)(\quad) \div 0.9(33) = 4.8\ in^2$

✱ $A_{s\,PROVIDED} = 3.25\ in^2$ < N.G.

NEXT SECTION 3 - 7/8" ⌀ @ 9" o/c:



$F_{HOOP} = 43.8^k$

$A_{s\,WS} = 4.4\ in^2$ 72% OVERSTRESSED

$A_{s\,ULT} = 4.1\ in^2$

$A_{s\,PROVIDED} = 0.77\,in^2 \times 3 + 0.25 = \underline{2.56\ in^2}$

N.G.

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	CLIENT G⌀O	
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NEXT SECTION 3-3/4" Ф @ 10" o.c.

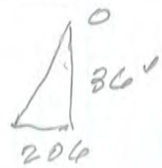
$F_{HOOP} = 32.0^k$

$A_{S_{WS}} = 3.2 \text{ in}^2$

$A_{S_{ULT}} = 3.02 \text{ in}^2$

$A_{S_{PROVIDED}} = 3(0.56 \text{ in}^2) + 2(0.25) = 2.18 \text{ in}^2$ OK

TOP SECTION 3-1/2" Ф @ 12" o.c. $+ 3(0.25 \text{ in}^2) = 1.5 \text{ in}^2$



$F_{HOOP} = 13.91^k$

$A_{S_{WS}} = 1.4 \text{ in}^2$

$A_{S_{ULT}} = 1.31 \text{ in}^2$

$A_{S_{PROVIDED}} = 1.5 \text{ in}^2$ OK

KGA KRAMER GEHLEN ASSOCIATES	PROJECT KELSO	✓ HOOP TENSION
	CLIENT G & O	
	CONSULTING ENGINEERS – STRUCTURAL/CIVIL 400 Columbia St. Suite 240 Vancouver, WA 98660-3413 360 / 693-1621 503 / 289-2661 Fax: 360 / 696-1572 www.kga.cc	DATE 2-23-10 · DESIGN KG · PROJECT NO. 10081- · SHEET HT-3 · 3 / 3

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Project:	*KELSO RESERVOIR EVALUATION*	*By:*	*D GOFF*
Location:	*TANK WALL WITH INSIDE FACE STEEL MODELED*	*Project No.:*	*10-051*
Client:	*GIBBS AND OLSON*	*Last Update:*	*24-Oct-03*
	CONCRETE CIRCULAR TANKS		

Dimensions			Properties		
H	17.5	ft	Es	29.0E+06	psi
D	90.0	ft	f'c	3,000	psi
t	12.0	in	fy	33,000	psi
Cover	2.00	in	C	0.0003	
H^2/Dt	3.4		K_1	400	psi
Sanitary Load Factors			β_1	0.85	
Tension	1.65		Ec	3.1E+06	psi
Flexure	1.30		n	9	
Shear	1.30				
Applied Loads					
ω	62.4	pcf			
p	-	pcf			
Strength Reduction Factor - ϕ					
Tension	0.90				
Flexure	0.90				
Shear	0.85				

Base Fixity
- ● Base Hinged
- ○ Base Fixed

Base Shear		
Service Base Shear (kips/ft)	Ultimate Base Shear (kips/ft)	ϕVn (kips/ft)
4.81	10.62	2.78

Ring Tension Triangular Load

Wall Point	Bar Size	As (in^2)	No. Bars	Spacing (in.)	ρ	Service Ring Tension Top Free-Base Hinged (kips/ft)	Ultimate Ring Tension Top Free-Base Hinged (kips/ft)	ϕNn (kips/ft)	
0.0H	No. 5	0.25	1	14.88	0.00174	2.51	7.04	7.43	
0.1H	No. 5	0.25	1	14.88	0.00174	7.96	22.34	7.43	
0.2H	No. 5	0.25	1	14.88	0.00174	13.25	37.18	7.43	
0.3H	No. 7	0.56	1	12.80	0.00391	18.27	51.24	16.71	307%
0.4H	No. 8	1.05	1	9.00	0.00731	22.46	63.00	31.28	
0.5H	No. 9	1.50	1	8.00	0.01042	25.64	71.91	44.55	61%
0.6H	No. 9	1.71	1	7.00	0.01190	26.69	74.87	50.91	
0.7H	No. 9	2.00	1	6.00	0.01389	25.00	70.13	59.40	18% O.S.
0.8H	No. 9	2.40	1	5.00	0.01667	19.85	55.69	71.28	
0.9H	No. 9	3.00	1	4.00	0.02083	11.23	31.50	89.10	

Bending Moment Triangular Load

Wall Point	Bar Size	Spacing (in.)	As (in^2/ft)	As min (in^2/ft)	As max (in^2/ft)	ρ	d (in)	Service Moment Top Free-Hinged Base (kips-ft/ft)	Ultimate Moment Top Free-Hinged Base (kips-ft/ft)	ϕMn (kips ft/ft)
0.1H	No. 5	30.00	0.12	0.01	4.15	0.00086	9.69	0.09	0.21	2.92
0.2H	No. 5	30.00	0.12	0.04	4.15	0.00086	9.69	0.45	1.00	2.92
0.3H	No. 5	30.00	0.12	0.08	4.15	0.00086	9.69	1.01	2.24	2.92
0.4H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	1.70	3.76	2.92
0.5H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	2.61	5.76	2.92
0.6H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	3.65	8.08	2.92
0.7H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	4.50	9.95	2.92
0.8H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	4.65	10.27	2.92
0.9H	No. 5	30.00	0.12	0.70	4.15	0.00086	9.69	3.46	7.64	2.92
1.0H	No. 5	30.00	0.12	-	4.15	0.00086	9.69	-	-	2.92

EXCEL - 1

KGA KRAMER GEHLEN ASSOCIATES	Project: KELSO RESERVOIR EVALUATION	By: D GOFF
CONSULTING ENGINEERS – STRUCTURAL / CIVIL	Location: TANK WALL WITH OUTSIDE VERT STEEL MODELED	Project No.: 10-051
400 Columbia Street, Suite 240 Vancouver, WA 98660-3117	Client: GIBBS AND OLSON	Last Update: 24-Oct-03
(360) 693-1821 (503) 289-2661 Fax: (360) 696-1572	CONCRETE CIRCULAR TANKS	

Dimensions		
H	17.5	ft
D	90.0	ft
t	12.0	in
Cover	2.00	in
H^2/Dt	3.4	
Sanitary Load Factors		
Tension	1.65	
Flexure	1.30	
Shear	1.30	
Applied Loads		
ω	62.4	pcf
p	-	pcf
Strength Reduction Factor - ϕ		
Tension	0.90	
Flexure	0.90	
Shear	0.85	

Properties		
Es	29.0E+06	psi
f'c	3,000	psi
fy	33,000	psi
C	0.0003	
K_1	400	psi
β_1	0.85	
Ec	3.1E+06	psi
n	9	

Base Fixity
- ● Base Hinged
- ○ Base Fixed

Base Shear		
Service Base Shear (kips/ft)	Ultimate Base Shear (kips/ft)	ϕVn (kips/ft)
4.81	10.62	5.39

Ring Tension Triangular Load

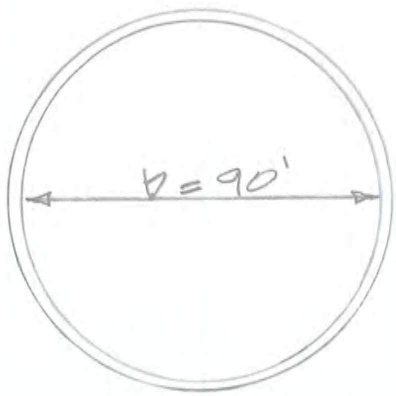
Wall Point	Bar Size	As (in^2)	No. Bars	Spacing (in.)	ρ	Service Ring Tension Top Free-Base Hinged (kips/ft)	Ultimate Ring Tension Top Free-Base Hinged (kips/ft)	ϕNn (kips/ft)
0.0H	No. 5	0.25	1	14.88	0.00174	2.51	7.04	7.43
0.1H	No. 5	0.25	1	14.88	0.00174	7.96	22.34	7.43
0.2H	No. 5	0.25	1	14.88	0.00174	13.25	37.18	7.43
0.3H	No. 7	0.56	1	12.80	0.00391	18.27	51.24	16.71
0.4H	No. 8	1.05	1	9.00	0.00731	22.46	63.00	31.28
0.5H	No. 9	1.50	1	8.00	0.01042	25.64	71.91	44.55
0.6H	No. 9	1.71	1	7.00	0.01190	26.69	74.87	50.91
0.7H	No. 9	2.00	1	6.00	0.01389	25.00	70.13	59.40
0.8H	No. 9	2.40	1	5.00	0.01667	19.85	55.69	71.28
0.9H	No. 9	3.00	1	4.00	0.02083	11.23	31.50	89.10

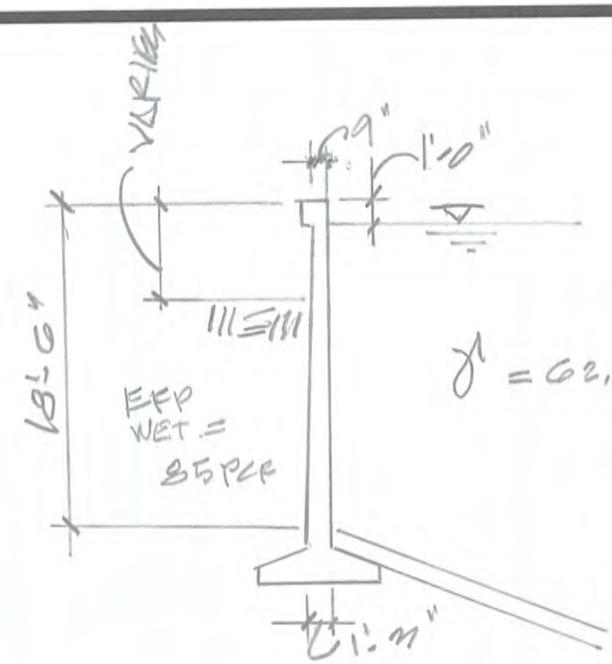
Bending Moment Triangular Load

Wall Point	Bar Size	Spacing (in.)	As (in^2/ft)	As min (in^2/ft)	As max (in^2/ft)	ρ	d (in)	Service Moment Top Free-Hinged Base (kips-ft/ft)	Ultimate Moment Top Free-Hinged Base (kips-ft/ft)	ϕMn (kips ft/ft)
0.1H	No. 7	30.00	0.24	0.01	4.10	0.00167	9.56	0.09	0.21	5.49
0.2H	No. 7	30.00	0.24	0.04	4.10	0.00167	9.56	0.45	1.00	5.49
0.3H	No. 7	30.00	0.24	0.08	4.10	0.00167	9.56	1.01	2.24	5.49
0.4H	No. 7	30.00	0.24	0.13	4.10	0.00167	9.56	1.70	3.76	5.49
0.5H	No. 7	30.00	0.24	0.20	4.10	0.00167	9.56	2.61	5.76	5.49
0.6H	No. 7	30.00	0.24	0.70	4.10	0.00167	9.56	3.65	8.08	5.49
0.7H	No. 7	30.00	0.24	0.70	4.10	0.00167	9.56	4.50	9.95	5.49
0.8H	No. 7	30.00	0.24	0.70	4.10	0.00167	9.56	4.65	10.27	5.49
0.9H	No. 7	30.00	0.24	0.70	4.10	0.00167	9.56	3.46	7.64	5.49
1.0H	No. 7	30.00	0.24	-	4.10	0.00167	9.56	-	-	5.49

EXCEL 2

B-479-6249
98626





ZIP CODE 98626

$S_S = 0.896\,g$

$S_1 = 0.348\,g$

RIGID NON-BUILDING

$R = 2.0$, $I = 1.25$, $S_{DS} = 0.682$, $S_{D1} = 0.395$

$C_S = 0.3\,S_{DS}\,I = 0.256\,W$

WEIGHTS:

$\frac{D}{H_L} = \frac{90'}{17.5'} = 5.1$ (USE 5.0 TO SIMPLIFY)

FROM FIG. 4-4(b)

$\frac{W_i}{W_L} = 0.24$ & $\frac{W_c}{W_L} = 0.72$

PROJECT KELSO	SEISMIC	
CLIENT G & O		
	DATE 2-22-2010	DESIGN DG
	PROJECT NO. 10051.00	SHEET SEIS-1

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Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Zip Code = 98626
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Data are based on a 0.05000000074505806 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	0.863	(Ss)
1.0	0.340	(S1)

Period (sec)	Maximum Sa (g)	
0.2	0.896	(Ss)
1.0	0.348	(S1)

Period (sec)	Minimum Sa (g)	
0.2	0.858	(Ss)
1.0	0.313	(S1)

JEIS-1B

IBC2006 (1613), ASCE 7-05 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Response Spectral Acc. (0.2 sec) S_s =	89.60%g	= 0.896g	Figure 22-1 through 22-14
Response Spectral Acc.(1.0 sec) S_1 =	34.80%g	= 0.348g	Figure 22-1 through 22-14
Soil Site Class	D	Table 20-3-1, Default = D	
Site Coefficient F_a =	1.142		Table 11.4-1
Site Coefficient F_v =	1.704		Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.023	(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 0.593	(11.4-2)
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	= 0.682	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.395	(11.4-4)
Building Occupancy Categories	IV, Essential		Table 1-1
Design Category Consideration:	Flexible Diaphragm		with dist. between seismic resisting system >40ft
Seismic Design Category for 0.1sec	D		Table 11.6-1
Seismic Design Category for 1.0sec	D		Table 11.6-2
S1 < .75g	NA		Section 11.6
Since Ta < .8Ts (see below), SDC =	D	Control (exception of Section 11.6 does not apply)	

Comply with Seismic Design Category D

IRC, Seismic Design Category = D1 — T-R301.2.2.1.1

12.8 Equivalent lateral force procedure

A. BEARING WALL SYSTEMS — T-12.2-1

Seismic Force Resisting Systems 2. Ordinary reinforced concrete shear walls

C_t =	0.02	x = 0.75		T-12.8-2
Building ht. H_n =	18	ft	Limited Building Height (ft) = NP	
C_u =	1.400	for S_{D1} of 0.395g	Table 12.8-1	
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.175	12.8-7	T_L = .266 Sec
Calculated T shall not exceed ≤	Cu.Ta	= 0.245		Use T = 0.245 sec.
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.464	Control (exception of Section 11.6 does not apply)	
Is structure Regular & ≤ 5 stories ?	Yes		12.8.1.3	
Response Spectral Acc.(0.2 sec) S_s =	0.896g		*Max Ss ≤ 1.5g*	
F_a =	1.14			
@ 5% Damped Design S_{DS} =	$⅔(F_a.S_s)$	= 0.682g	(11.4-3)	
Response Modification Coef. R =	4		Table-12.2-1	
Over Strength Factor Ω_o =	2		foot note g	
Importance factor I =	1.5		Table 11.5-1	
Seismic Base Shear V =	$C_s W$			

$$C_s = \frac{S_{DS}}{R/I} = 0.256 \qquad (12.8\text{-}2)$$

$$\text{or need not to exceed, } C_s = \frac{S_{D1}}{(R/I).T} = 0.606 \qquad \text{For } T \le T_L \qquad (12.8\text{-}3)$$

$$\text{or } C_s = \frac{S_{D1}T_L}{T^2(R/I)} \quad \text{N/A} \qquad \text{For } T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than =	0.01			(12.8-5)
Min C_s =	$0.5S_1I/R$	N/A	For $S_1 \ge 0.6g$	(12.8-6)
Use C_s =	0.256			
Design base shear V =	0.256 W	Control		

T-12.14-1

SKG-1C

$$W_L = \frac{\pi (90)^2 \times 17.5' \times 62.4}{4(1000)} = 6947^k$$

$$W_i = 0.24(6947^k) = 1667^k$$

$$W_c = 0.72(6947^k) = 5002^k$$

WALL WEIGHT (ASSUME 1'-0" THICK FOR SIMPLICITY):

$$W_W = \frac{3.14(91)(18.5)(150)}{(1000)} = 793^k$$

ROOF WT

ACCOUNT FOR "DOME"

$$W_R \cong (8\ PSF)\frac{(\pi)(90)^2}{4(1000)} \times 1.2 = 61^k$$

PERIOD:

$$T_i = \frac{2\pi}{\omega_1} = \frac{2\pi}{107.8} = \underline{0.0583\ SEC}$$

$$\omega_1 = C_L \frac{12}{H_L}\sqrt{\frac{E_c}{\rho_c}} = \omega_1 = (0.192)\frac{12}{17.5}\sqrt{\frac{3122 \times 10^6}{4.66}} = 107.8\ \frac{RAD}{SEC}$$

$$C_L = 10\, C_W \sqrt{\frac{t_w}{12r}} = 10(0.129)\sqrt{\frac{12}{12(45)}} = 0.192$$

$C_W = 0.129$ (FIG 4-10).

$\rho_c = 4.66\ \#\text{-}SEC^2$ (MASS DENSITY OF CONCRETE)

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		PROJECT NO. 10091	SHEET SEIS-2

RESPONSE SPECTRUM METHOD (SECTION 4.2.3)

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.395}{0.682} = 0.58 \text{ SEC} \leftarrow$$

$$T_0 = 0.2 T_S = 0.116 \text{ SEC}$$

SINCE $T_i < T_0 \Rightarrow (0.0583 < 0.116)$

$$S_{a_i} = S_{DS}\left[\frac{0.6 T_i}{T_0} + 0.4\right] = 0.682\left[\frac{0.6(0.0583)}{0.116} + 0.4\right] = 0.48$$

$$C_{si} = \frac{S_{ai}}{R} = \frac{0.48}{2} = \underline{0.24} \leftarrow$$

$$V_i = C_{si}(W_W + W_R + W_i) = 0.24[2521^k] = 605^k \leftarrow$$

$$V_c = 179^k$$

$$V_T = \sqrt{179^2 + 605^2} = 631^k$$

* SINCE 80% OF $1089^k = 871^k > 631^k$ USE $\underline{\underline{871^k}} = V_T \leftarrow$

✓ SLIDING:

$\mu_f \cong 0.35$

$$F_f = (0.35)(793^k + 61^k) = 290^k$$

PASSIVE PRESSURE = 200 PCF/FT $d_{min} \cong 10'$

$$F_p = 200(10)(\tfrac{10}{2})(90') = 900^k$$

$F_R = 900^k + 290^k \cong 1190^k > 871^k$ TANK DOES NOT SLIDE

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	CLIENT G+O		
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		PROJECT NO. 10051	SHEET SEIS-3

✓ OVERTURNING:

$M_i = C_{si}(W_w h_w + W_R h_r + W_i h_i) =$

$M_i = 0.24\left[793\left(\frac{18.5}{2}\right) + 61(18.5) + 1667(6.56')\right] = 4656'^k$

$h_i = (0.375)(17.5') = 6.56'$ ← FIG 4.5(b)

$\frac{H_i}{H_L} = 0.375$ (FIG 4-5b)

$M_c = C_{sc} W_c H_c = (0.0358)(5002)(0.53)(17.5) = 1661'^k$

$H_c = 0.53(17.5) = 9.28'$

$M_T = \sqrt{4656^2 + 1661^2} = \underline{4943'^k}$ ←

DON'T ADD H_2O WT AS FLOOR INADEQUATE

RESISTING MOMENT ≥ $45'(793^k + 61^k) = 38{,}340'^k > 4943$

* TANK DOES NOT OVERTURN

✓ DESIGN WALLS FOR IN-PLANE LOADING:

$V_c = \frac{0.8 V_u}{\pi r} = \frac{0.8(939^k)}{\pi(45)} = 5.31$ K/FT ACI 350.3

$V_u = 1.0E = 1.0(871^k + $ $0.03(10)\left(\frac{10}{2}\right)(90') =$

$V_u = 1.0(939^k) = 939^k$

NOMINAL SHEAR STRENGTH $= A_{cv}(\alpha_c \sqrt{f'_c} + \rho_n f_y) =$

$\frac{h_w}{l_w} = \frac{18.5}{\pi(90)} = 0.065 < 1.5 \quad \alpha_c = 3.0$

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$$T_c = \frac{2\pi}{\lambda}\sqrt{D} = 0.725\sqrt{90} = \underline{6.9\ \text{SEC}}$$

$$\frac{2\pi}{\lambda} = 0.725 \quad \text{(FROM FIG 4.9(b))}$$

BASE SHEAR

$$V_i = C_{si}(W_w + W_R + W_i) = (0.426)(\overbrace{793^k + 61^k + 1667^k}^{2521^k}) = \underline{1074^k}$$

$$C_{si} = \frac{S_{DS} I}{R} = \frac{0.682(1.25)}{2} = 0.426 \leq \frac{S_{D1} I}{R T_i} = \frac{(0.395)(1.25)}{2(0.0583)} = 4.23$$

$*$ USE $C_{si} = 0.426$

$$V_c = C_{sc}(W_c) = (0.0358)(5002) = \underline{179^k}$$

$$C_{sc} = \frac{S_{DS} I}{R} = \frac{0.682(1.25}{2} = 0.426 \leq \frac{S_{D1} I}{R T_c} = \frac{(0.395)(1.25)}{2(6.9)} = 0.036$$

$*$ USE $C_{sc} = 0.0358$

TOTAL BASE SHEAR PER ACI 350.3:

$$V_T = \sqrt{V_i^2 + V_c^2} = \underline{1089^k}$$

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DESIGN DG

PROJECT NO. 10051

SHEET SEIS.5

SEISMIC CATEGORY "D"

$\phi V_n = 0.6(12)(15")\left[3\sqrt{3000} + \rho_n F_Y\right] = 77.1\ k/ft > 5.31^k/ft$

$t_{BASE} = 15"$ $\quad +(0.0166)(33000)$

$\rho_n = \dfrac{1 in^2 (+E)}{4(15)} = 0.0166$

$F_Y = 33\ ksi$

* SHEAR STRENGTH IS OK *

DESIGN WALLS FOR OUT OF PLANE LOADING

$M_U = 0.75\left[1.4D + 1.7L + 1.7F + \dfrac{1.87E}{1.4}\right] =$

$M_U = 1.3F + 1.0E$

$M_F =$

WALL INERTIA

$P_W = C_{si} W_W = 0.24(793^k) = 190.3^k$

ROOF INERTIA

$P_R = 0.24(61^k) = 14.6^k$

IMPULSIVE

$P_i = C_{si} W_i = 0.24(1667^k) = 400.1^k$

CONVECTIVE

$P_c = C_{sc} W_c = (0.0358)(5002^k) = 179.1^k$

KGA	PROJECT KELSO			
	CLIENT G & O			
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		PROJECT NO. 10051	SHEET SEIS-6	

$P_{WY'}$:

$$P_{WY'} = \frac{P_W}{2H_W} = \frac{190.3^k}{2(18.5')} = 5.14^k/FT$$

$$P_{iY} = \frac{P_i\left[4H_L - 6h_i\right] - \left[(6H_L - 12h_i) \times y/H_L\right]}{2H_L^2} =$$

AT BOTTOM OF WALL

$$P_{iY_0} = \frac{P_i\left[4(17.5) - 6(6.56)\right] - \left[(6(17.5 - 12(6.56)) \times \frac{0}{17.5}\right]}{2(17.5)^2} =$$

$P_{iY_0} = 0.050 P_i$

AT TOP OF WALL

$$P_{iY_{17.5}} = \frac{P_i\left[\overset{30.64}{4(17.5) - 6(6.56)}\right] - \left[(\overset{26.28}{6 \times 17.5 - 12 \times 6.56})\left(\frac{17.5}{17.5}\right)\right]}{2(17.5)^2} =$$

$P_{iY_{17.5}} = 0.0071\ P_i$

CONVECTIVE PRESSURES

$$P_{cY} = \frac{16\ P_{cY}}{9\pi r} \cos\theta = 16($$

$$P_{cY_0} = P_c \frac{\left[4(17.5) - 6(9.28)\right]}{2(17.5)^2} \times \cancel{\cos\theta} = 0.0234 P_c$$

$$P_{c_{17.5}} = \frac{P_c\left[4(17.5) - 6(9.28)\right] - \left[6(17.5) \overset{-6.36}{-} 12(9.28) \times \frac{17.5}{17.5}\right]}{2(17.5)^2} \times \cancel{\cos\theta}$$

$P_{c_{17.5}} = 0.0338\ P_c \cos\theta \Rightarrow MAX @ \theta = 0° = 0.0338 P_c$

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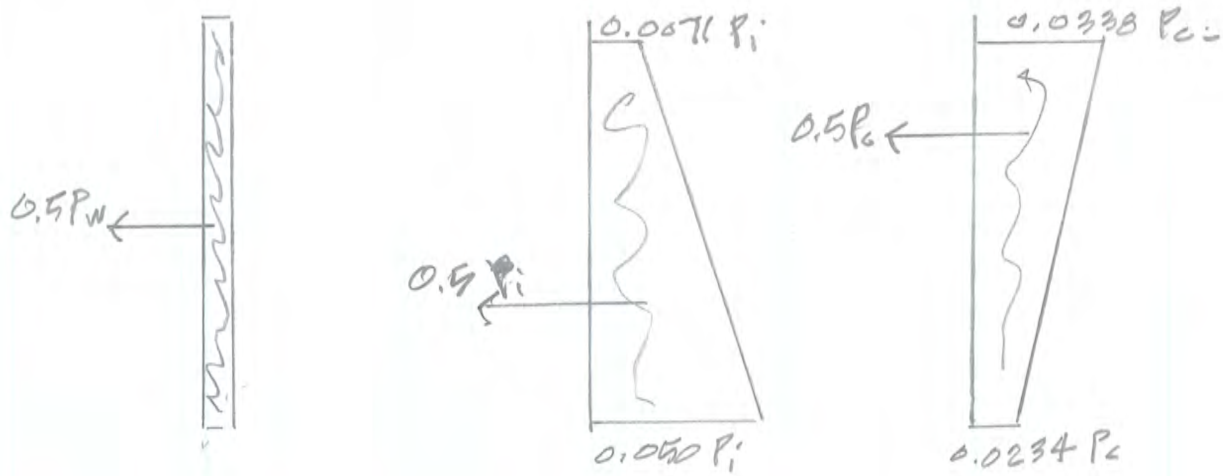
SHEET 4 ~~REMB~~-7



PRESSURE DUE TO INERTIA

$$P_w = \frac{5.14^k (1000)}{\pi (45)} = 36 \text{ PSF}$$

PRESSURE DUE T IMPULSIVE

@ BOTTOM $P_i = \dfrac{2 P_{i_y}}{\pi r} \cos\theta = \dfrac{2(0.050)\, P_i\, (1)}{\pi\ 45}(1000) = 283 \text{ PSF}$ (P_i = 400.1)

@ TOP $P_i = \dfrac{0.0071}{0.050}(283) = 40 \text{ PSF}$

PRESSURE DUE TO CONVECTIVE

@ BOTTOM $P_c = \dfrac{2(0.0234)(179.1)(1000)}{\pi \times 45} = 59 \text{ PSF}$

@ TOP $P_c = \dfrac{0.0338}{0.0234}(59) = 85 \text{ PSF}$

PROJECT KELSO			
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FINAL PRESSURES

$P_{TOP} = 62.4(0) + 36 + 40 + 85 = 161$ PSF

$P'_{BOTTOM} = [62.4(17.5)] + (36 + 283 + 59) = 1092 + (378)$

$1.7F + 1.0E =$

@ TOP $P_T = 161$ PSF

@ BOTTOM $P_B = 1.7(1092) + 378 = 1798$ PSF

STATIC PRESSURE

$P_B = 1.7(1092) = 1856$ PSF < 1798 PSF $\underline{OK}$

THESE VALUES DO NOT HAVE SANITARY COEFFICIENTS IN, BUT COMPARING APPLES TO APPLES

ULTIMATE RING TENSION @ TOP

$F_{UT} = (161)\frac{(90')}{2} = 7.25^k \times 1.65 = 11.95^k$

$A_{ST} = \frac{11.95}{0.9(33)} = 0.40\ in^2/FT > (0.5'')^2 = 0.25\ in^2$

* 60% OVERSTRESSED ←

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	CLIENT: G & O		
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		PROJECT NO.: 10051	SHEET: SEIS. 9

✓ RING TENSION @ MID-HT:

$$P_U = 1.7(1.65)(62.4)\left(\frac{17.5'}{2}\right) + 1.0\left(\frac{161 + 378}{2}\right)(1.65) = 1976 \text{ PSF}$$

$$F_{U_{HOOP}} = 1976\left(\frac{90'}{2}\right) = 88.93^{k}$$

$$A_{S_{REQ'D}} = \frac{88.93}{(0.9)(33)} = 2.99 \text{ in}^2/\text{FT}$$

$$A_{S_{PROVIDED}} \cong \frac{(1.0 \text{ in}^2)(12)}{7.5''} = \underline{\underline{1.6 \text{ in}^2}} < \underline{2.99 \text{ in}^2}/\text{FT}$$ 86% OVER STRESSED

KGA KRAMER GEHLEN ASSOCIATES	PROJECT KELSO			
	CLIENT G & O			
	CONSULTING ENGINEERS – STRUCTURAL/CIVIL 400 Columbia St. 360 / 693-1621 Suite 240 503 / 289-2661 Vancouver, WA Fax: 360 / 696-1572 98660-3413 www.kga.cc	DATE 2-24-10	DESIGN DG	
		PROJECT NO. 10061	SHEET SKG.10	

$d_{max} = \frac{D}{2} C_c I$

$D = 90'$, $I = 1.25$ $\qquad \frac{D}{H_L} = \frac{90'}{17'} = 5.3$

$C_c = \frac{2.4 S_{DS}}{T_c^2} = 0.0332 \qquad \le 1.5 S_{DS} = 1.5(0.682) = 1.02$

$T_c = \left(\frac{2\pi}{\lambda}\right)\sqrt{D} = (0.74)\sqrt{90} = \underline{\underline{7.02}} > \frac{1.6}{0.579} = \underline{\underline{2.79}} \therefore$

FIG 9.3.4(b)
ACI 350.3-06

$W_c = \frac{\lambda}{\sqrt{D}} =$

$\lambda = \sqrt{3.68 g \tanh\left[3.68\left(\frac{H_L}{D}\right)\right.} =$

$H_L = 17'$ $\qquad S_{DS} = 0.682$ $\qquad S_{D1} = 0.395$

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.395}{0.682} = \underline{\underline{0.579}} \leftarrow$

ZIP CODE = 98626

* USE $C_c = \underline{0.0032} \leftarrow$

$d_{max} = \frac{90}{2}(0.0332)(1.25) = \underline{\underline{1.9'}} > 1.0'$

* FREEBOARD IS INSUFFICIENT FOR SLOSHING HEIGHT

KGA KRAMER GEHLEN ASSOCIATES	PROJECT: KELSO RESERVOIRS	✓ FREE BOARD FOR SEISMIC SLOSHING	
	CLIENT: GIBBS & OLSON		
	CONSULTING ENGINEERS – STRUCTURAL/CIVIL 400 Columbia St. Suite 240 Vancouver, WA 98660-3413 360 / **693-1621** 503 / **289-2661** *Fax: 360 / 696-1572* www.kga.cc	DATE: 1-27-2010	DESIGN: DG
		PROJECT NO.: 10051	SHEET: SH-1

LAP LENGTH

$F_y = 33$ ksi (33,000)

$f'_c \cong 3000$ psi

$$l_d = d_b \left\{ \frac{3}{33} \frac{f_y}{\sqrt{f'_c}} \frac{\Psi_t \Psi_e \Psi_s \lambda}{\left(\frac{c_b + K_{TR}}{d_b}\right)} \right\} = \left[\frac{3}{33} \frac{33000}{\sqrt{3000}} \frac{(1.0)(1.3)(1.0)(1.0)}{\left(\frac{2+0}{1.13}\right)} \right] d_b$$

$l_d = 40.2\, d_b = (40.2)(1.13) = 45.4'' \times 1.5 = \underline{81''}$

$d_b =$

$$A = \frac{\pi d^2}{4} \Rightarrow \sqrt{\frac{1.0(4)}{\pi}} = d_b = 1.13 \text{ in}$$

$$c_b = \frac{4''}{2} = 2''$$

$K_{TR} = 0$

$\Psi_T = 1.3$

$\Psi_e = 1.0$

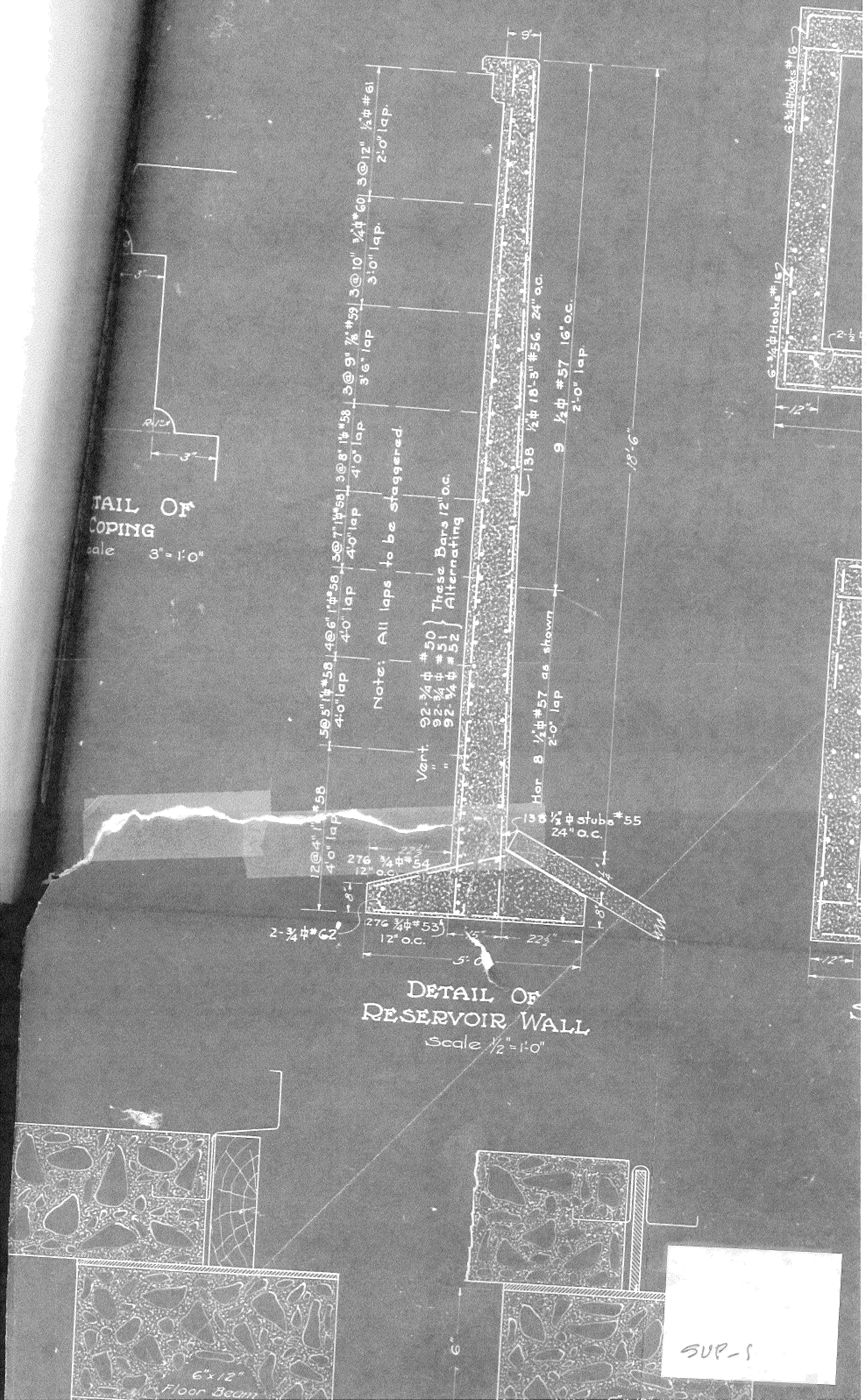
$\Psi_s = 1.0$

$\lambda = 1.0$

* KGA LAP SPLICE TABLES INDICATE $L_d = 81''$ OK

KGA KRAMER GEHLEN ASSOCIATES	PROJECT KELSO	LAP SPLICE CHECK	
	CLIENT GIBBS & OLSON		
	CONSULTING ENGINEERS – STRUCTURAL/CIVIL 400 Columbia St. 360 / 693-1621 Suite 240 503 / 289-2661 Vancouver, WA Fax: 360 / 696-1572 98660-3413 www.kga.cc	DATE 2-24 2015	DESIGN DG
		PROJECT NO. 10051	SHEET LAP-1

FIGURES, CHARTS AND SPECIFICATIONS



6-3/4 Φ Hooks #16
6-3/4 Φ Hooks #15
TAIL OF
COPING
cale 3" = 1'-0"
R 1/2"
3"
9"
18'-6"
138 1/2 Φ 18'-3" #56. 24" o.c.
9 1/2 Φ #57 16" o.c.
2'-0" lap
Hor 8 1/2" Φ #57 as shown
2'-0" lap
138 1/2 Φ stubs #55
24" O.C.
Vert. 92-3/4 Φ #50
" 92-3/4 Φ #51
" 92-3/4 Φ #52
These Bars 12" o.c.
Alternating
Note: All laps to be staggered.
12@4" 1" Φ #58
4'-0" lap
5@5" 1" Φ #58
4'-0" lap
4@6" 1" Φ #58
4'-0" lap
3@7" 1" Φ #58
4'-0" lap
3@8" 1" Φ #58
4'-0" lap
3@9" 7/8" Φ #59
3'-6" lap
3@10" 3/4" Φ #60
3'-0" lap
3@12" 1/2" Φ #61
2'-0" lap.
276 3/4 Φ #54
12" o.c.
276 3/4 Φ #53
12" o.c.
2-3/4 Φ #62
22 1/2"
15"
5'-0"
8"
DETAIL OF
RESERVOIR WALL
Scale 1/2" = 1'-0"
6"x12"
Floor Beam
12"
6"
SUP-1

MATERIAL:

Unless otherwise specified, reinforcing steel shall meet the requirements of the standard Specification for Billet Steel Concrete Reinforcement Bars of the American Society for Testing Materials Serial Designation A15-14, structural steel grade.

STRUCTURAL STEEL:

All structural steel shall conform to the Standard Specifications for Structural Steel for Bridges, adopted by the American Society for Testing Materials, 1916, or any later revision thereof.

All structural steel shall be made by the open hearth process.

IRON CASTINGS:

All iron castings shall meet the requirements of the Standard Specifications of the American Society for Testing Materials. Serial Designation A-48-18.

STEEL CASTINGS:

All steel castings shall meet the requirements of the American Society for Testing Materials, Serial Designation on A27-16 (Class B.)

When purchased from warehouse in small lots reinforcement may, at the discretion of the Engineer, be accepted subject to the bending test only.

BENDING, STORING AND PLACING:

All steel shall be cut and bent by careful, competent men. It shall be bent cold to templates, which shall not vary appreciably from the shape and dimensions shown on the plans. All sharp bends will be avoided, and in no case shall a bend be of less radius than 3 diameters of the bar.

Steel shall be stored in a protected place and in a manner which will protect it from being bent or badly rusted. Before being placed in the forms, all steel shall be thoroughly cleaned from all rust, flakes, dirt, oil, or other material which will tend to lessen the bond strength with concrete.

All points of intersections of bars shall be firmly wired, making the whole reinforcing system rigid. Stirrups shall be firmly wired to the main reinforcement. The whole reinforcing system shall be held away from the forms by cement cubes or other approved form of spacer. Concrete shall be placed as soon as possible after the steel is placed, but no concrete shall be poured until the amount and position of steel is inspected by the Engineer. Any concrete placed contrary to this provision may be rejected.

SPLICING REINFORCING BARS:

All reinforcing bars shall be as long as possible, and splicing shall be done only when indicated on the approved plans. All deformed bars shall be lapped at least 40 diameters and thoroughly and firmly wired at the splices. Plain bars (when allowed) shall be lapped 50 diameters. Laps in adjacent bars shall be staggered.

ORDERING REINFORCING STEEL:

The Contractor shall order reinforcing steel at his own risk, with reference to length, bending an number of bars; and should check his orders from the plans.

ANCHOR BOLTS:

Anchor bolts of such size, number and dimension, as shown on the detailed plan, and as called for in this specification, shall be furnished by the Contractor. They shall be accurately set, either in the original concrete or in drilled holes after the concrete is set. If drilled, the holes shall be at least 1 inch larger in diameter than the anchor bolt, to afford ample room for "grouting in." If set in the original concrete, a gas pipe should be used at least 2 inches larger in diameter than the anchor bolt, using a heavy steel washer as an anchorage for the bolt at the bottom. This pipe must in all cases be filled with grout after the plates are set in their true position.

SUP-2

Search

Return to index: [Subject] [Thread] [Date] [Author]

RE: Rebar fy - Intermediate Grade Steel

[Subject Prev][Subject Next][Thread Prev][Thread Next]

- **To**: seaint(--nospam--at)seaint.org
- **Subject**: RE: Rebar fy - Intermediate Grade Steel
- **From**: "Collier, Randy" <rhcollier(--nospam--at)winkinc.com>
- Date: Thu, 9 Feb 2006 16:34:26 -0600

Juan,

Try this:

RE-BAR	Structural	Intermediate	Hard
Tensile	55-75 Kips	70-90 Kips	80 Kips min
Yield (min)	33 Kips	40 Kips	50 Kips

Randall Collier, P.E.

From: Juan Carlos Morales [mailto:jcmorales1564(--nospam--at)yahoo.com]
Sent: Thursday, February 09, 2006 4:23 PM
To: seaint(--nospam--at)seaint.org
Subject: Rebar fy - Intermediate Grade Steel

I am analyzing a reinforced concrete structure designed and built in 1960-1962. The only reference I have found in the drawings regarding the grade of steel reinforcement is a note indicating "intermediate grade steel". I need fy, the yield strength, for the analysis.

I have done some research in the internet and the current applicable standard for typical steel reinforcement, ASTM A615, was established in 1968. It replaced ASTM A15 which was withdrawn in 1969. As far as I have researched, ASTM A15 includes grades "structural, intermediate and hard". This seems like a good-enough lead to purchase it. But before I make the investment, is anyone familiar with ASTM A15? Are the "structural, intermediate, and hard" grades accompanied by an fy.

Regards,

Juan C. Morales, P.E.

- Prev by Subject: Rebar fy - Intermediate Grade Steel
- Next by Subject: RE: Rebar fy - Intermediate Grade Steel

SUP-3

CHAPTER 2
General

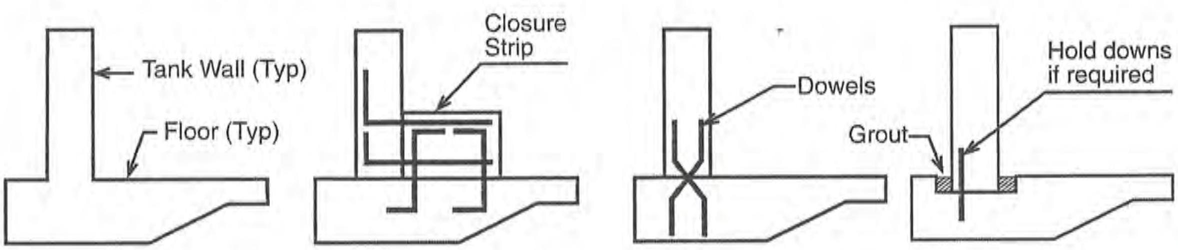
2.1 TYPES OF LIQUID-CONTAINING STRUCTURES

i. **Rectangular**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))

ii. **Circular without Prestressing**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))

iii. **Circular with Prestressing**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))
   3. **Flexible Base**
      a. Anchored (Fig. 2-2(a))
      b. Unanchored, Contained (Fig. 2-2(b))
      c. Unanchored, Uncontained (Fig. 2-2(c))

Liquid-containing structures essentially fall into two categories of behavior based on their wall-to-footing connection: the non-sliding or the rigid base (Fig. 2-1) and the flexible base (Fig. 2-2). The non-sliding base typically uses a fixed or hinged wall-to-footing connection. The flexible base typically uses a base pad between the wall and the footing and allows varying degrees and types of movement depending upon whether the wall is anchored, unanchored contained or unanchored uncontained in the footing (Fig. 2-2). This type of connection is only used for circular prestressed tanks. The type of base connection is likely to influence the seismic response of a liquid-containing structure and its effect should be properly included in modeling, design and detailing.

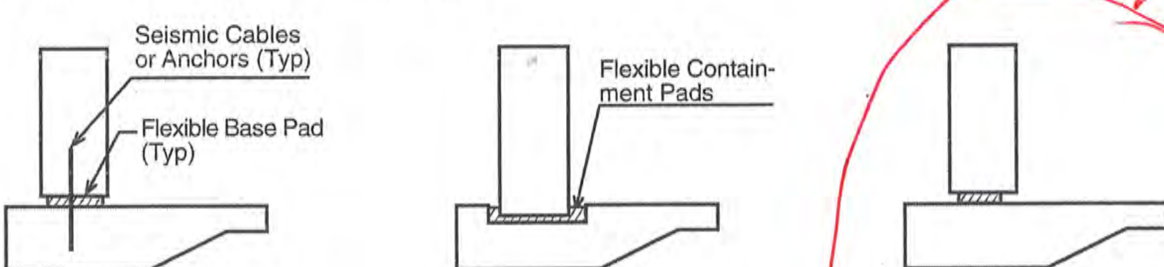


(a) FIXED

(b) HINGED OR PINNED

Figure 2-1. Nonflexible Base Connections

ESSENTIALLY WHAT WE HAVE



(a) ANCHORED

(b) UNANCHORED, CONTAINED FLEXIBLE BASE

(c) UNANCHORED, UNCONTAINED FLEXIBLE BASE

Figure 2-2. Flexible Base Connections

PCA-1

Non-sliding Base. Tanks that have a fixed or hinged connection between the walls and the foundation are essentially non-sliding type. Steel reinforcement or keying action ensures the non-sliding behavior, as shown in Fig. 2-1.

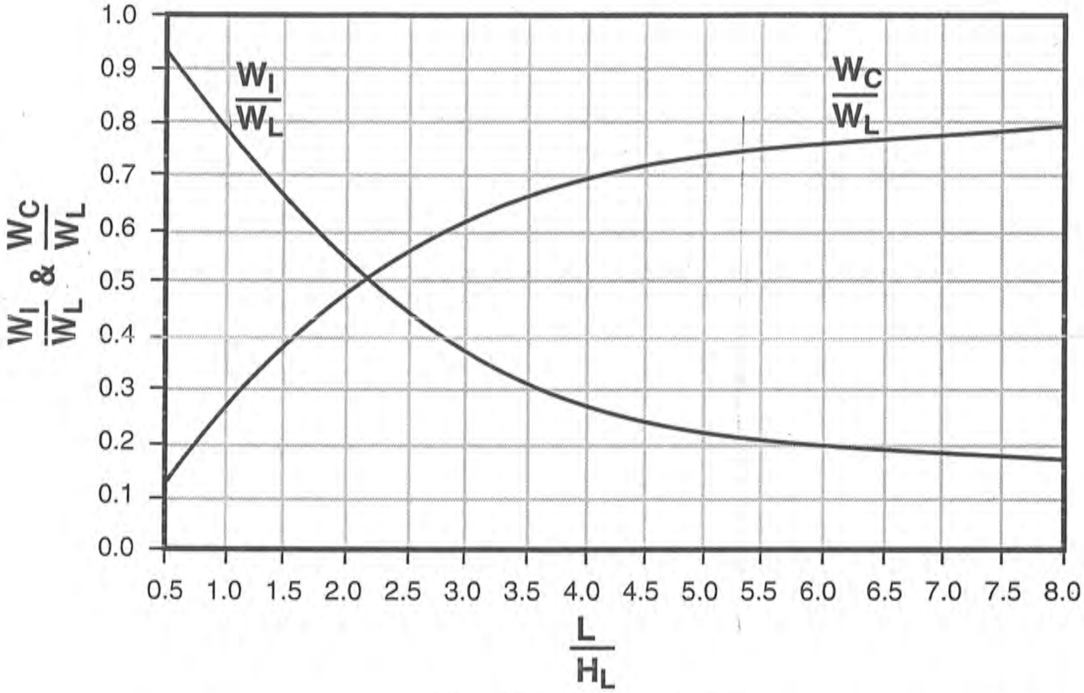
Anchored Flexible Base. Tanks with flexible base that use some kind of anchorage between the wall and the footing allow radial movement but restrict the tangential movement at the base of the structure. Typically, anchorage is achieved with strand cables embedded in the wall and the footing (Fig. 2-2(a)). Compressible sleeves are used over anchor cables at the base joint to allow radial wall movement.

Unanchored Contained Flexible Base. These tanks use an unanchored wall contained by a concrete curb as shown in Fig. 2-2(b). This type of connection allows limited radial and tangential movement.

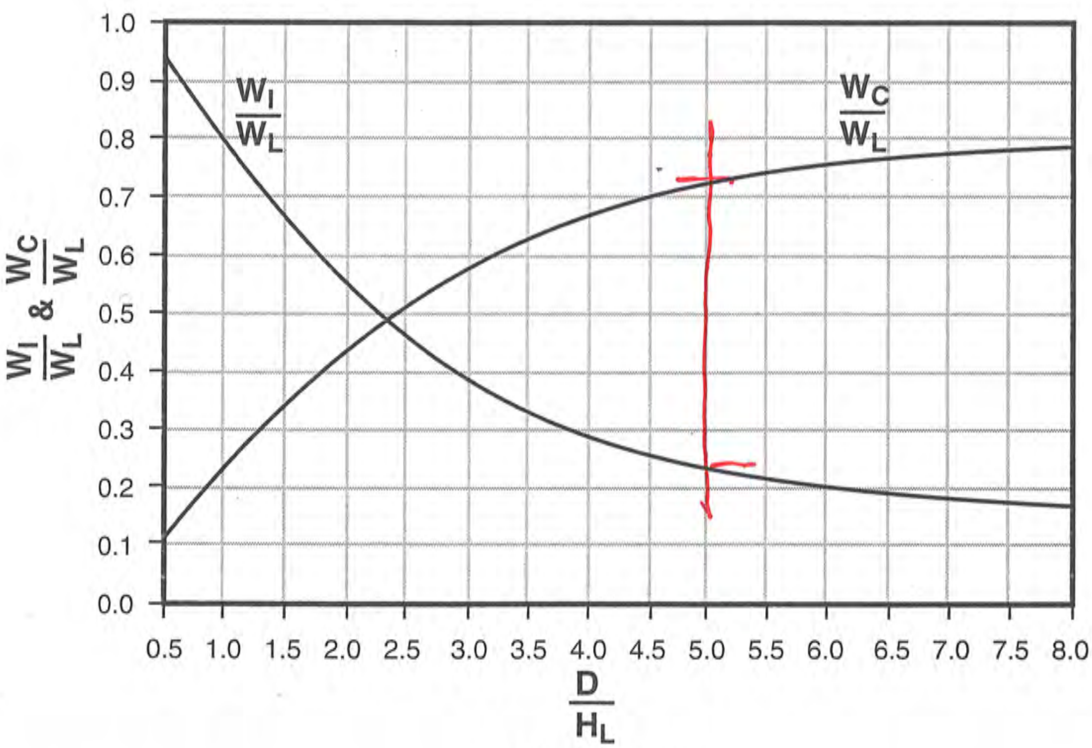
Unanchored Uncontained Flexible Base. These allow an unlimited radial and tangential movement of the joint since no anchorage or containment of the walls is involved (Fig. 2-2(c)). This type of tank is not permitted in regions of high seismicity (UBC zones 3 and 4) for obvious reasons of potentially uncontrolled movement during a seismic event.

NOW SEISMIC DESIGN CATEGORY "D" IS ESSENTIALLY UBC ZONE 3

PCA - 2



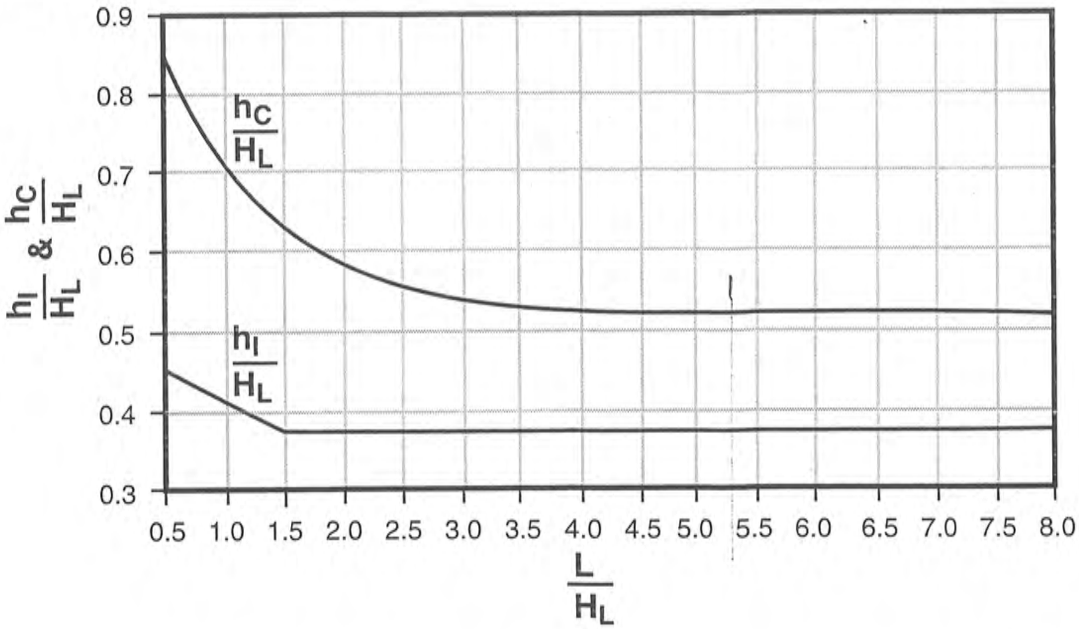
(a) Rectangular Tanks



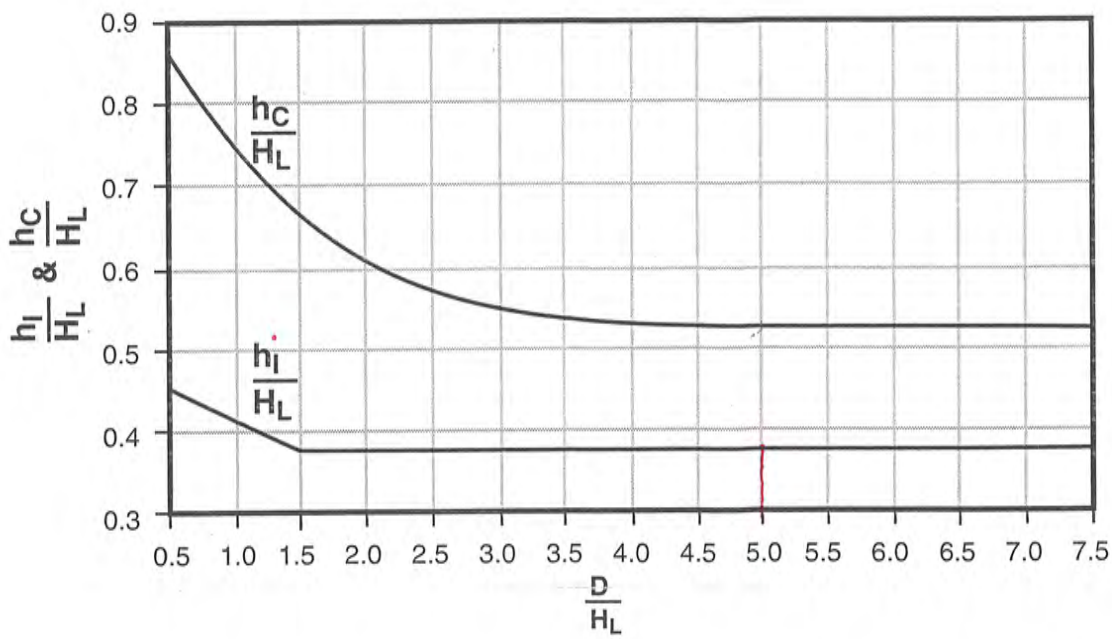
(b) Circular Tanks

Fig. 4-4 Impulsive and Convective Weights for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)

PCA-3



(a) Rectangular Tanks



(b) Circular Tanks

Fig. 4-5 Effective Height of Impulsive and Convective Weights for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)

PCL-4

Table 4-1 Importance Factor, I (Table 4(c), Ref. 3-6)

Tank Use	Factor I
Tanks containing hazardous materials (1)	1.5
Tanks that are intended to remain usable, for emergency purposes after an earthquake; or tanks that are part of lifeline systems	1.25
All other tanks	1.0

(1) For tanks containing hazardous materials, engineering judgment may require a factor I > 1.5 to account for the possibility of an earthquake greater than the design earthquake.

Table 4-2 Response Modification Factor, R_w (Table 4(d), Ref. 3-6)

AVERAGE = 3.70

Type of Structure	R_{wi}		R_{wc}
	On or Above Grade	Buried(1)	
(a) Anchored, flexible-base tanks	4.5	4.5(2)	1.0
(b) Fixed or hinged-base tanks	2.75	4.0	1.0
(c) Unanchored, contained or uncontained tanks(3)	2.0	2.75	1.0
(d) Elevated Tanks	3.0	—	1.0

(1) Buried tank is defined as a tank whose maximum water surface is at or below ground level.
(2) R_{wi} = 4.5 is the maximum R_{wi} value permitted to be used for any liquid-containing concrete structure.
(3) Unanchored, uncontained tanks shall not be built in Zone 2B or higher.

where $\frac{2\pi}{\lambda}$ can be obtained from Fig. 4-9(a) for a given L/H_L of the tank. (L = length of tank in direction of analysis (ft)).

4.7.2 Circular Tanks

(a) **Non-sliding Base.** The following equations can be used to determine the impulsive period of fixed or hinged base circular tanks with or without prestressing:

$$T_I = \frac{2\pi}{\omega_I}$$

where

$$\omega_I = C_L \frac{12}{H_L}\sqrt{\frac{E_C}{\rho_C}}$$

$$C_L = 10C_W\sqrt{\frac{t_w}{12r}}$$

and ρ_c = mass density of concrete (4.66 lb-sec²/ft⁴), t_w = thickness of wall (in.), r = radius of tank (ft), E_C = modulus of elasticity of concrete (lb/in.²), C_W is given in Fig. 4-10 in terms of D/H_L.

(b) **Flexible Base.** The following equations can be used to determine the impulsive period T_I of flexible base circular prestressed tanks:

$$T_I = \sqrt{\frac{8\pi W}{gDk_a}} \quad \leq \text{ 1 second for anchored tanks and 2 seconds for unanchored tanks}$$

Note that ACI 350.3 specifies a limit of 1.25 seconds on both anchored and unanchored maximum periods.

$$W = W_W + W_R + W_I$$

$$k_a = 144\left[\frac{A_S E_S \cos^2\beta}{L_S S_b} + \frac{2G_p w_p L_p}{t_p S_p}\right] \quad \text{For anchored flexible tanks}$$

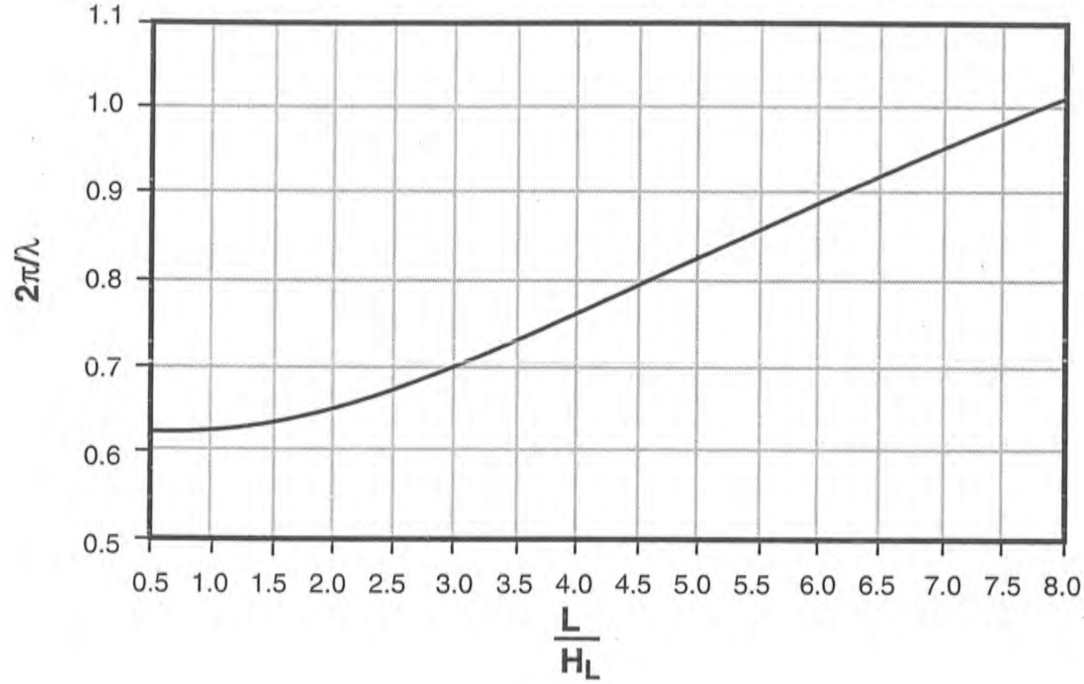
$$k_a = 144\left[\frac{2G_p w_p L_p}{t_p S_p}\right] \quad \text{For unanchored flexible tanks}$$

where A_s = cross-sectional area of cable/strand (in.²), E_s = modulus of elasticity of cable/strand (ksi), β = angle of cable/strand with horizontal, L_s = effective length of cable/strand taken as sleeve length plus 35 times the diameter (in.), S_b= spacing between cable sets (in), S_p = spacing of elastomeric pads (in), G_p = shear modulus of elastomeric pads (ksi), t_p = thickness of elastomeric bearing pad (in.), L_p = length of individual elastomeric pad (in.) and w_p = width of elastomeric pad in radial direction (in.), and k_a = spring constant (k/ft²).

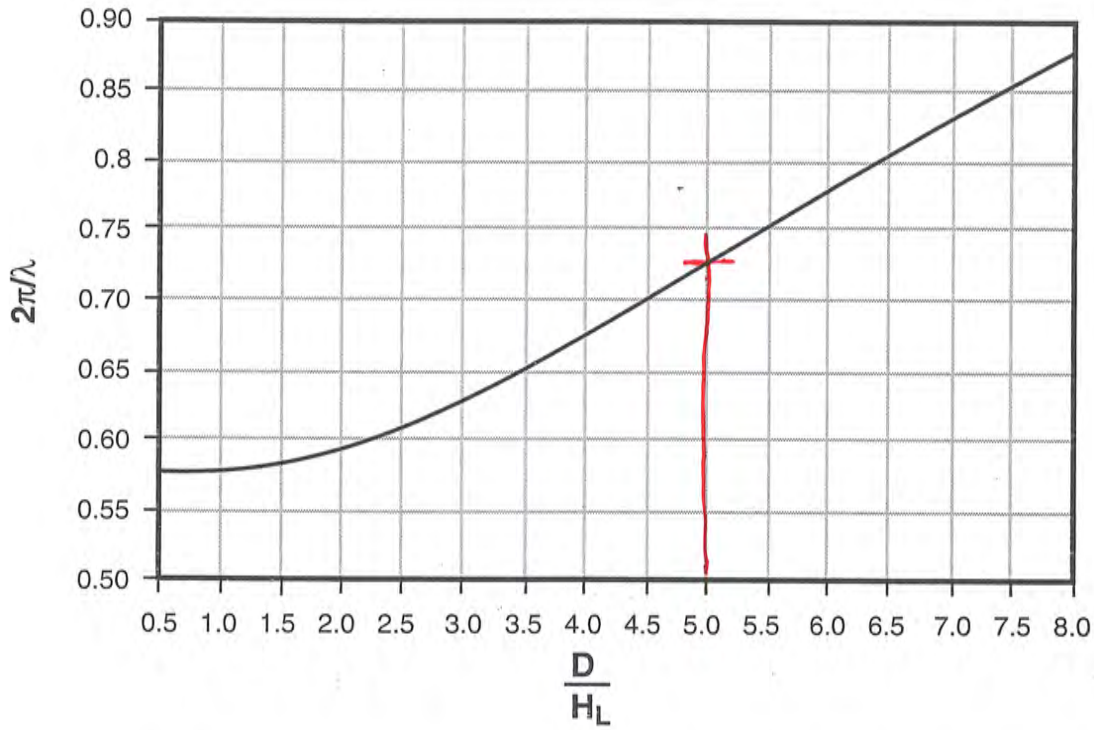
The convective period T_C for both non-sliding and flexible base tanks can be determined using the following equation:

$$T_C = \frac{2\pi}{\lambda}\sqrt{D}$$

REA. 5



(a) Rectangular Tanks



(b) Circular Tanks

Fig. 4-9 Charts for Obtaining Factor $\frac{2\pi}{\lambda}$ for Computation of Convective Period (T_c) for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)

PCS-6

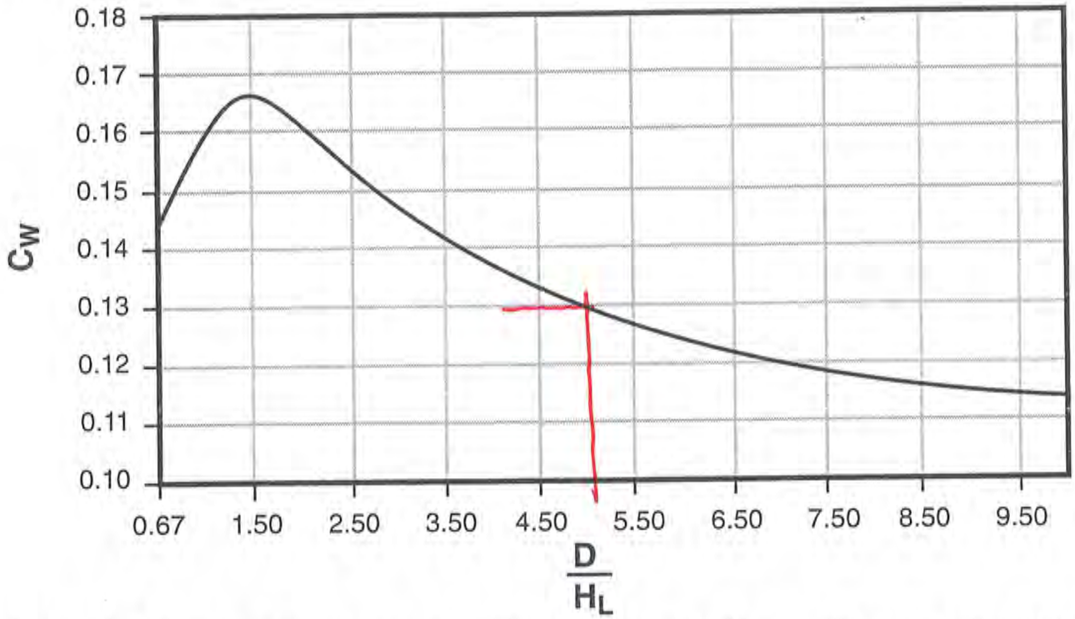


Fig. 4-10 Chart for Obtaining Factor C_w for Computation of Impulsive Period (T_i) of a Non-Sliding Circular Tank (Adapted from Ref. 3-6)

The value of $\frac{2\pi}{\lambda}$ may be obtained from Fig. 4-9(b) for D/H_L of a tank for both rigid and flexible base tanks (D = diameter of tank (ft), H_L = height of liquid (ft)).

4.8 VERTICAL ACCELERATION

The effect of vertical accelerations should be included in the design of tank components. In the absence of more detailed analysis, the magnitude of vertical acceleration is generally taken as two-thirds of the horizontal acceleration. The effects of vertical acceleration as recommended in ACI 350.3 are computed as follows:

The hydrodynamic pressure per foot height of the tank is

$$p_{hy} = \ddot{u}_V q_{hy}$$

where $q_{hy} = \gamma_L(H_L\text{-}y)$ lbs/ft, unit hydrostatic pressure at level y above tank base

$\ddot{u}_v$ = magnitude of vertical acceleration associated with the vertical period (T_V) of the structure and γ_L = specific weight of contained fluid.

The period associated with the vertical motion (T_V) of the circular tank is computed as follows:

$$T_V = 2\pi\sqrt{\frac{\gamma_L D H_L^2}{24 g t_w E_c}}$$

4.9 FREEBOARD

The anticipated unrestrained sloshing height should be computed to determine any sloshing pressure on the tank roof, wall and the joint between roof and the wall. Note that tanks with inadequate freeboard will experience uplift pressures on the roof due to liquid sloshing. Tanks in seismic zones 3 and 4 and tanks designed for importance factor greater than 1.0 should either have adequate freeboard d_{max} (Fig. 4-2a) or should be designed for the forces due to restrained sloshing and vertical acceleration effects. The sloshing height may be computed by using the following equations which are based on concepts similar to those given in ACI 350.3.

IBC 2000 Method

$$d_{max} = \frac{S_{D1}I}{1.4T_C}\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = \frac{S_{D1}I}{1.4T_C}\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

UBC '97 Method

$$d_{max} = \frac{C_V I}{1.4T_C}\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = \frac{C_V I}{1.4T_C}\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

UBC '94 Method

$$d_{max} = ZC_C I\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = ZC_C I\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

PC8-7

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

ALASKA
CALIFORNIA
COLORADO
FLORIDA
MISSOURI
OREGON
WASHINGTON

June 30, 2010

Gibb & Olson, Inc.
1405 17th Avenue, Suite 300
Longview, Washington 98632

Attn: Mr. Thomas Gower, PE

**RE: "DRAFT" PRELIMINARY GEOTECHNICAL ISSUES REPORT
MINOR ROAD WATER RESERVOIR SITE
KELSO, WASHINGTON**

Dear Mr. Gower:

INTRODUCTION

This preliminary report presents our identification and discussion of the key geotechnical issues at the Minor Road reservoir site related to potential rehabilitation of existing reservoirs and a potential complete replacement of the existing reservoirs with a single new reservoir. This report is based on the background information discussed below. Also, this report identifies recommended additional work to characterize the site in more detail and/or provide other information judged to be necessary to complete a geotechnical predesign of both options mentioned above. This discussion of issues should be considered preliminary in nature, since this study contained no geotechnical analysis or evaluations, and additional geotechnical explorations and characterization will likely be necessary on the eastern portion of the site due to the high variability of the subsurface conditions.

BACKGROUND

The site location is shown on the Vicinity Map, Figure 1. The subsurface soil characterization, to provide a basis of this assessment, is contained in a geotechnical exploration program completed

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www.shannonwilson.com

in December 2009, and summarized in a Shannon & Wilson Geotechnical Data Report (GDR), dated April 2010. The exploration locations are shown in Figure 2, the Plan of Explorations. Using this subsurface information, we prepared a Generalized Subsurface Profile through the site, shown in Figure 3 with the profile location shown on Figure 2. The detailed descriptions of the various soil layers encountered are contained in the GDR. For reference, the logs of the borings and the Soil Classification and Log Key are contained in Appendix A. Also, photographs of the site are contained in Appendix B.

The existing reservoirs consist of two identical concrete, partially buried circular structures, each 90-foot diameter, with a storage capacity of about 1-million gallons. The existing reservoirs have a wall height of 18.5 feet and a hopper bottom with net vertical height of 7.5 feet. Based on existing records and information compiled in a report by Kramer Galen Associates (KGA), titled City of Kelso, Reservoir Evaluation, dated February 24, 2010, the existing foundation depths and configurations are shown on Figure 3. The existing reservoir appears to be about 86 years old with construction having begun in 1924.

From discussions with Gibbs & Olson and the City, we understand a new replacement reservoir would have a storage capacity of 2-million gallons, with the same overflow as the existing reservoirs. The probable new reservoir configuration would be a partially buried, circular, concrete reservoir about 130 feet in diameter with vertical sidewalls up to at least 20 feet high, for a minimum storage capacity of approximately 2-million gallons; however, the concept of a 30-foot-deep side wall may also be studied. In the evaluations below, issues during construction of a new reservoir are considered for the options of keeping one tank operational and for taking both reservoirs out of service.

ASSUMPTIONS RELATED TO THE ISSUES

We assume, the current building codes require water reservoirs be designed for earthquake loading; therefore, we have attempted to separate issues into static and seismic. For both rehabilitation of existing reservoirs and a new reservoir, we assumed a long-term design life of at least 50 years would be required. For both reservoirs, we have assumed the overflow elevation would remain the same as the existing reservoirs. We have focused our list of issues on the reservoir structures, and have not considered issues related to existing or new pipelines.

For a new reservoir, we have assumed that there is the possibility that one or both of the existing reservoirs can be taken off line; we understand this depends on whether or not the Paxton Road reservoir has been constructed. Also for a new reservoir, we understand a wall height up to a maximum 30 feet with no hopper bottom may be considered.

EXISTING RESERVOIRS ISSUES AND REHABILITATION

The preliminary list of geotechnical issues for rehabilitation of the existing reservoirs is discussed below.

Overall Site Stability

Static Condition. It is likely the existing slope stability will not meet required design factors of safety, primarily on the northwest and north slopes adjacent to the northern reservoir. The fine-grained upper soils described as Fill and portions of the fine-grained Troutdale Formation appear to be relatively weak and would require buttressing and/or replacement with stronger material, such as imported crushed rock to provide additional slope stability and to protect the reservoir walls and possibly the foundations. With the probable hoop tension overstressed condition of the existing walls, mentioned in the KGA report, any loss of backfill lateral pressure support could cause failure of the reservoir walls and/or the connection between the walls and the foundation.

Seismic Condition. During seismic loading, the slope stability will be less stable than the static condition and the slope would likely experience sufficient movement and result in loss of support to the reservoir walls, due to strength reduction caused by cyclic seismic loading.

Walls

Static Conditions. There appears to be no subsurface drainage along the buried walls to relieve hydrostatic pressure on the walls from high groundwater and/or perched water conditions. It is most probable that the static lateral earth pressure is significantly higher than what was assumed in the original design calculations (pre 1924). The adverse impact of higher loads would relate to the current and future structural integrity of the walls. Also, it is possible that the lateral earth pressure of the soil only would have been used in the original design.

Seismic Condition. During seismic loading, the significant increased lateral earth pressure above a static condition from backfill would not have been accounted for in the original design. The adverse impact of this higher loading would relate to the current and future structural integrity of the walls.

Foundations

Static Condition. Due to the large amount of groundwater moving in the subsurface over the history of the reservoirs, a condition could exist where foundation subgrade soil fine particles have migrated causing voids to exist under the existing foundations. In order to be assured of a stable foundation, an assessment of the slab and potential need for ground stabilization should be done. Assessments of the existing slabs would require removal of the water inside the reservoirs; therefore, there is a concern about hydrostatic uplift pressures under the existing slab. Either the groundwater levels would have to be confirmed to be below the slab level or a method of hydrostatic relief should be installed.

Seismic Condition. During seismic cyclic loading, the foundation soils will likely undergo some loss of strength (softening) in the southern portion of the North reservoir and the entire foundation of the South reservoir. Since the northern portion of the North reservoir is founded partially on dense to very dense gravels; therefore this portion of the reservoir should not lose strength and essentially no movement should occur. However, for the portion of the North reservoir founded on fine-grained soils, under seismic loading, there is concern that movement would create differential settlement across the southern portion of the slab. Since the South reservoir appears to be on similar soils, it is unlikely that significant differential settlement would occur; however, additional subsurface information is needed on the eastern side of the reservoir to confirm this. Also, confirmation is needed that the fine-grained soils of the Troutdale Formation are not liquefiable.

Remedial Construction Issues

Groundwater/Hydrostatic Uplift Control. For remediation of the existing reservoirs, access to a dry reservoir would require removal of the water inside the reservoirs, and there is a concern about hydrostatic uplift pressures under the existing slab could cause damage or failure of the slab. Either the groundwater levels would have to be confirmed to be below the slab level, or a method of hydrostatic relief should be installed.

Other Reservoir Issues. Finding potential voids under the slabs will be difficult. If the reservoirs were rehabilitated from inside, there still would be significant remediation work needed under the existing slabs to fill potential voids and in the fine-grained soils, to eliminate the potential of differential settlement and future structural problems. This would likely require grouting to fill voids and ground improvement and/or an underpinning system. This remediation would be needed for both reservoirs. The cost for this foundation remediation would be significantly high. As compared to a new reservoir foundation, the remediation of one existing reservoir would likely be higher.

NEW RESERVOIR ISSUES

The preliminary discussion of geotechnical issues for constructing a new reservoir is provided below.

Overall Site Stability

Static Condition. The backfill for a new reservoir will need to consider slope stability issues primarily on the northwest and north sides adjacent to the North reservoir, due to the fine-grained upper soils described as Fill and portions of the fine-grained Troutdale Formation which appear to be relatively weak. Due to the site constraints and the larger footprint of a new reservoir, any backfill and slope buttressing would likely require imported crushed rock, to provide sufficient slope stability and minimize the buttressing footprint.

Seismic Condition. Seismic loading and the need to maintain lateral resistance would be accounted for in the design of a new reservoir.

Walls

Static Conditions. As mentioned above, the backfill for a new reservoir would likely be imported crushed rock which would incorporate subsurface drainage along the buried reservoir walls.

Seismic Condition. During seismic loading, the increased lateral earth pressure would be considered in the design.

Foundations

Static Condition. In order to be assured of a stable foundation with essentially no differential settlement, the entire foundation of the new reservoir will need to be placed on non-compressible material. This will require over-excavation of the fine-grained Troutdale (sandy silt, silt, and clayey silt) to be replaced with imported crushed rock. Over-excavation appears to be the most feasible option, because the depth to the gravel portion of the Troutdale Formation or the sandstone of the Cowlitz Formation is not significant.

Seismic Condition. With over-excavation of the fine-grained soils and replacement with crushed rock, the foundation soils should perform adequately during seismic cyclic loading.

New Reservoir Construction Issues

One Existing Reservoir Out of Service Option. The excavation for the new reservoir will require an excavation depth that will undermine and create an unstable condition for the existing reservoir foundation. Therefore, it will be necessary to maintain a stable foundation for the existing reservoir adjacent to the new excavation using mitigation techniques such as special lateral restraint shoring systems, ground improvement, and/or underpinning the existing reservoir foundations adjacent to the excavation. Groundwater control will be required for this excavation. Consideration of the construction and groundwater control impacts on the existing reservoir will be necessary. Also, the current available land parcel may not be of sufficient size for the new reservoir footprint and construction activities while maintaining operation of one of the existing reservoirs; therefore, a temporary construction easement outside the property lines or a permanent easement, or property acquisition may be required.

Both Existing Reservoirs Out of Service Option. When considering permitting, impacts/acquisition of adjacent property, design, constructability and costs, and risk management, the preferred method would be to remove both existing reservoirs to construct a new reservoir. The excavation for the new reservoir will require shoring and groundwater control. Consideration should be given to the deep shoring methods that can be constructed without impact to adjacent property and the need to be compatible, minimizing impacts on the existing reservoir. Taking both reservoirs out of service can only occur if the Paxton Road Reservoir is constructed.

ADDITIONAL WORK RECOMMENDED

Below is recommended additional geotechnical work beyond this Issues Report in order to complete a predesign phase of either rehabilitation of the existing reservoirs or construction of a new reservoir. Not mentioned is additional work related to preliminary and final design.

Geotechnical Exploration, Testing, and Analysis

Existing Reservoirs. We recommend additional borings (up to 3 borings), laboratory testing, and analysis be done for complete site characterization, especially at the north end of the site and on the eastside of the existing reservoirs to determine the properties of the subsurface soil and the depth of the soil contacts between the fine-grained and the gravelly Troutdale Formation, and the sandstone of the Cowlitz Formation. Also, additional laboratory testing and analysis is needed to confirm that the fine-grained Troutdale Formation is not liquefiable during a code-based design earthquake event.

New Reservoir. We recommend similar additional explorations, testing, and analysis on the northern and eastern portion of the site. However, the testing and analysis for a new reservoir will be more cost effective, because the problematic fine-grained soils will be removed and replaced with non-compressible, imported crushed rock.

Evaluation of Existing Reservoirs' Condition

Existing Reservoirs. If the existing reservoirs are planned to be rehabilitated, we anticipate significant additional geotechnical work would be required to support the structural engineer in evaluating the various structural elements of the existing reservoirs, and more detailed evaluation of the issues and approaches for rehabilitation of the existing reservoirs.

New Reservoir. If a new reservoir replaces the existing reservoirs, we anticipate minor additional geotechnical work would be required to support the civil and structural engineers in evaluating the impacts of demolition of one of the existing reservoirs if one reservoir is kept operational during the construction of a new reservoir. If both existing reservoirs are taken off line during construction of the new reservoirs, essentially no additional evaluation of the existing reservoirs' condition will likely be necessary.

Geotechnical Predesign Phase

We assume the decision would be made for rehabilitation of existing reservoirs or a new reservoir before a predesign phase of work is started. Some of the geotechnical work we recommend for this stage is a site specific seismic hazard evaluation according to the IBC Code requirements and providing predesign level recommendations for seismic design parameters and seismic resistance methods and values. Other work would include evaluation and recommendations for foundation systems, underdrains for the foundation and drains for the buried walls, backfill materials and earth pressures on buried walls, site grading and slope stabilization, surface drainage, and conceptual methods for excavations, shoring, and dewatering.

CONCLUSIONS

Considering the geotechnical conditions at the site and significant design issues, construction challenges, and probable high cost to rehabilitate the existing reservoirs, in our opinion, the preferred geotechnical option would be construction of a new reservoir. While constructing a new reservoir, the preferred approach would be to take both existing reservoirs off line. However, it appears feasible to keep one of the reservoirs operational, but this alternative would add significant cost to the project for the reasons discussed above.

LIMITATIONS

This geotechnical preliminary report provides our opinion and interpretations made from existing geotechnical data. No testing, analyses, design evaluations, or recommendations were made in preparation of this report. This report should be considered a planning-level document only.

This report was prepared for the exclusive use of the Gibbs & Olson and the City of Kelso. This report should not be made available to prospective contractors, since it is based on opinions and interpretations made from the Geotechnical Data Report described above. This report is not as a warranty of subsurface conditions.

Within the limitations of the scope, schedule, and budget, the data available for the preparation of this report, the opinions and interpretations presented in this report are in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no warranty, either expressed or implied.

Unanticipated ground conditions are commonly encountered and cannot be fully disclosed by merely taking samples from exploration points. Unexpected ground conditions, particularly in this geologic setting, should be anticipated.

The scope of our geotechnical services did not include any environmental assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site, or for evaluation of disposal of contaminated soils or groundwater, should any be encountered, except as noted in this report.

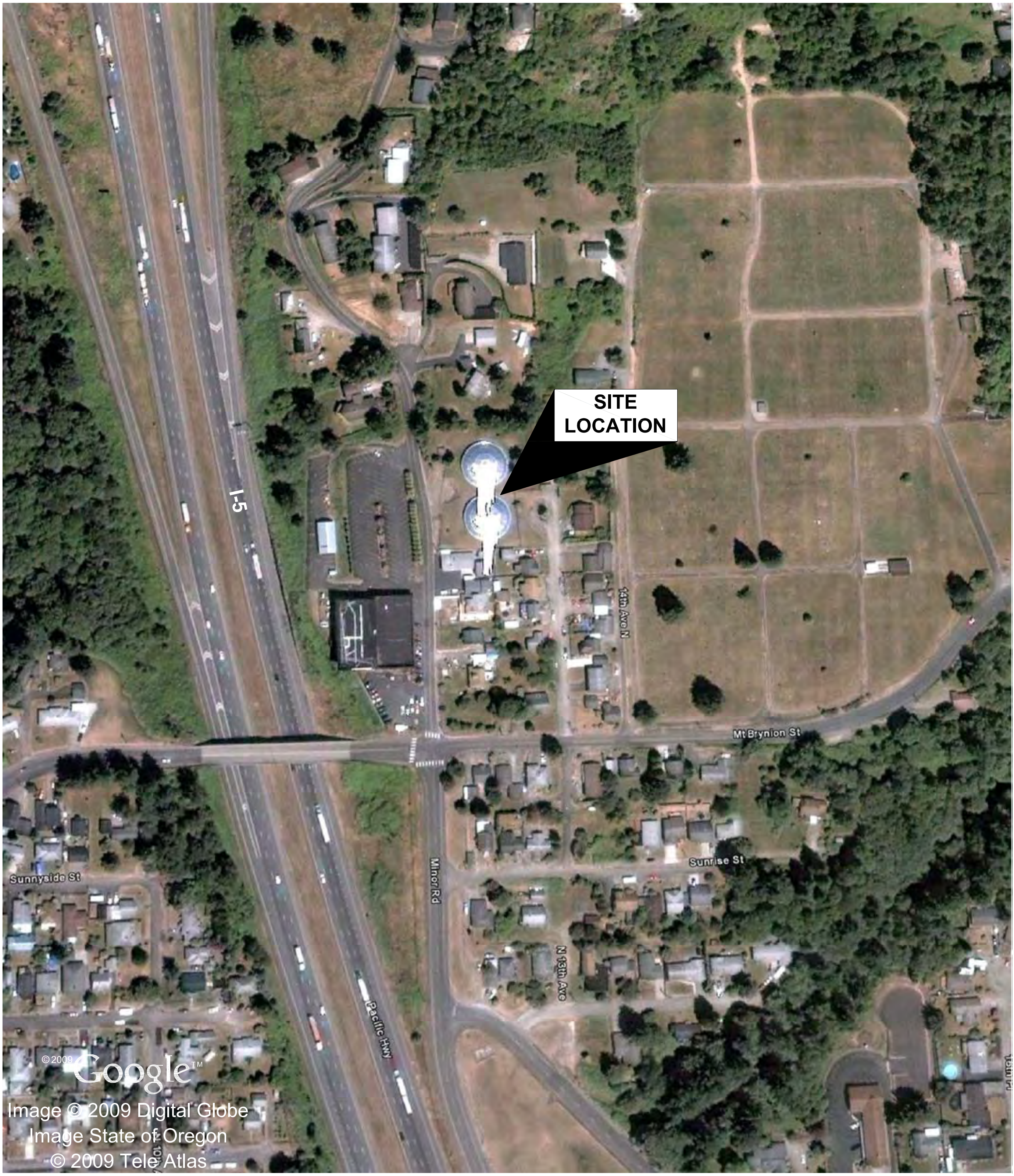
Shannon & Wilson, Inc. has included "Important Information About Your Geotechnical Report," (Appendix C), to assist you and others in understanding the use and limitations of our report.

SHANNON & WILSON, INC.

Jerry L. Jacksha, PE
Senior Associate

JLJ/YWL/rlf

Enc: Figure 1 Vicinity Map
Figure 2 Plan of Explorations
Figure 3 Generalized Subsurface Profile
Appendix A – Soil Classification, Log Key, and Boring Logs (Taken from GDR)
Appendix B – Miscellaneous Reservoir Photographs
Appendix C – Important Information About Your Geotechnical Report



0 250 500

Scale in Feet
(Scale is Approximate)

N

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.

Minor Road Reservoir
Kelso, Washington

VICINITY MAP

June 2010 24-1-03576-002

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FIG. 1

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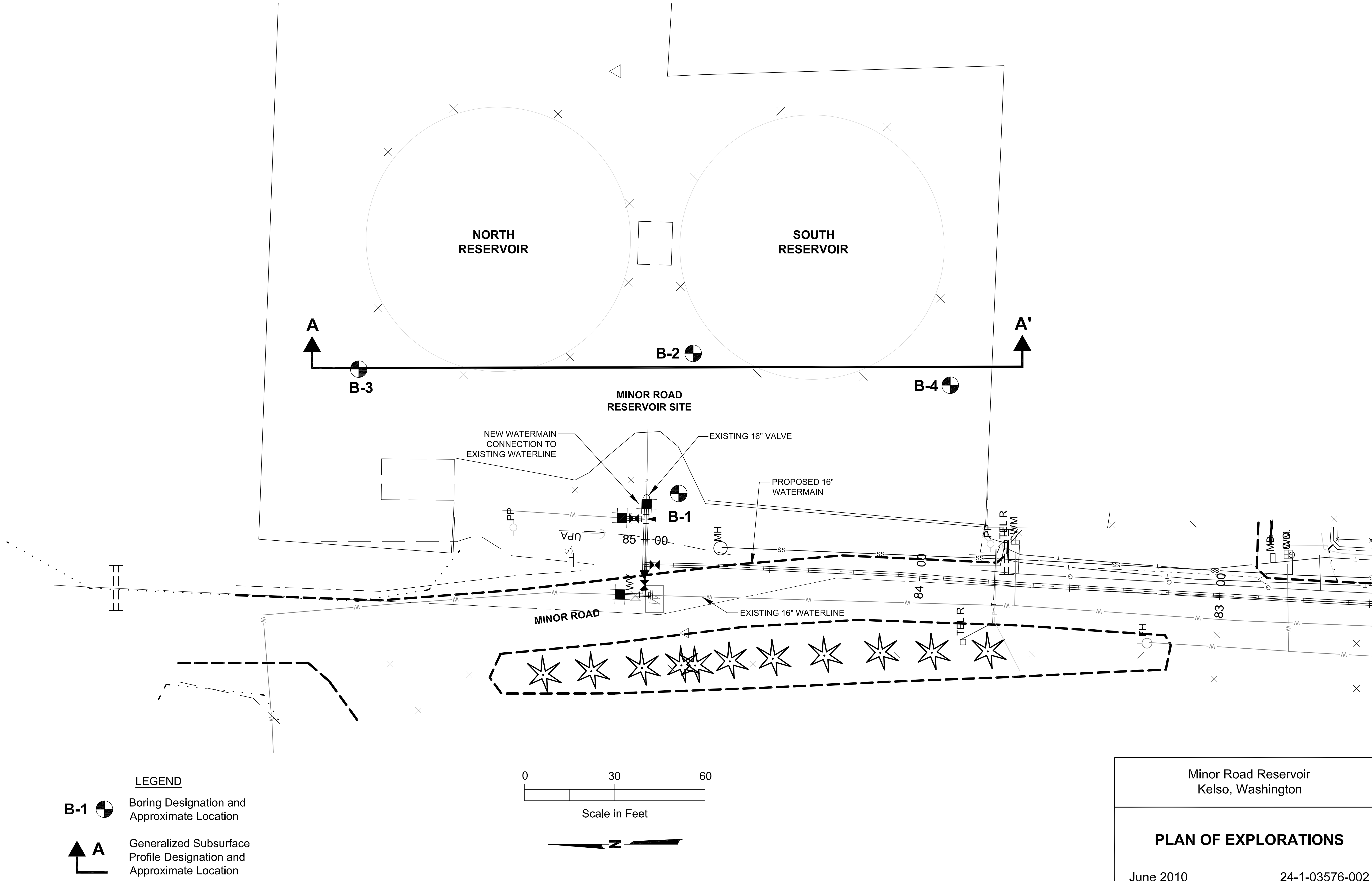

Minor Road Reservoir
Kelso, Washington

PLAN OF EXPLORATIONS

June 2010 24-1-03576-002

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FIG. 2

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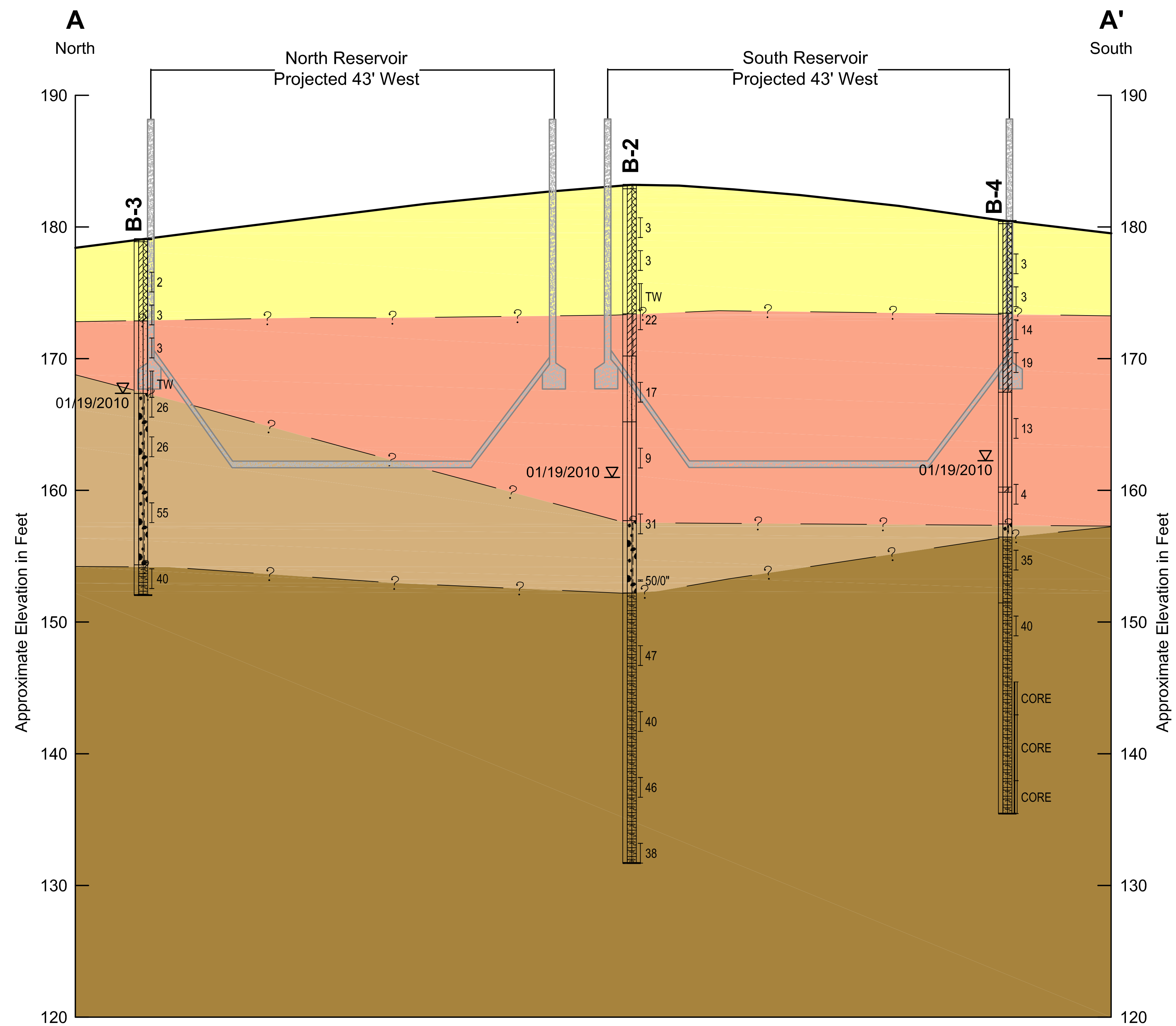
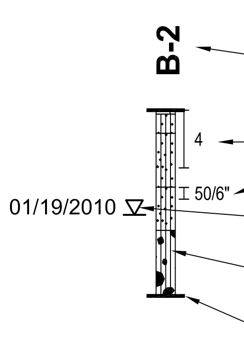


PROFILE LEGEND



Designation of Boring

Sample and Penetration Resistance in Blows/Foot or Blows/Inches Driven

Water Level, Date Measured

USCS Symbol (see chart, right)

Bottom of Boring

Fill - Fine-Grained Soils

Troutdale Formation - Sandy Silt, Silt, Silty Clay

Troutdale Formation - Silty, Sandy, Gravel

Cowlitz Formation - Sandstone

0 30 60

Horizontal Scale in Feet

0 10 20

Vertical Scale in Feet

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. Boring B-1 not shown because it is at a much lower elevation.
3. Boring B-2 projected 5' west and B-4 projected 6' east.
4. Reservoir wall and foundation elevations below ground surface are approximate.

Minor Road Reservoir
Kelso, Washington

GENERALIZED SUBSURFACE PROFILE

June 2010 24-1-03576-002

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FIG. 3

File: I:\WIP\Projects\24-1 Portland\3500\3576 Minor Rd Reservoir\Drafting\002\Site Plan_Profile.dwg Date: 06-29-2010 Author: rlf

APPENDIX A

SOIL Classification, Log Key, and Boring Logs
(Taken from GDR)

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOIL *(more than 50% retained on No. 200 sieve)*	Gravel *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravel *(less than 5% fines)*	GW	Well-graded gravel, gravel, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravel, gravel-sand mixtures, little or no fines
		Gravel with Fines *(more than 10% fines)*	GM	Silty gravel, gravel-sand-silt mixtures
			GC	Clayey gravel, gravel-sand-clay mixtures
	Sand *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sand *(less than 5% fines)*	SW	Well-graded sand, gravelly sand, little or no fines
			SP	Poorly graded sand, gravelly sand, little or no fines
		Sand with Fines *(more than 10% fines)*	SM	Silty sand, sand-silt mixtures
			SC	Clayey sand, sand-clay mixtures
FINE-GRAINED SOIL *(50% or more passes the No. 200 sieve)*	Silt and Clay *(liquid limit less than 50)*	Inorganic	ML	Inorganic silt of low to medium plasticity, rock flour, sandy silt, gravelly silt, or clayey silt with slight plasticity
			CL	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay, silty clay
		Organic	OL	Organic silt and organic silty clay of low plasticity
	Silt and Clay *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silt, micaceous or diatomaceous fine sand or silty soils, elastic silt
			CH	Inorganic clay or medium to high plasticity
		Organic	OH	Organic clay of medium to high plasticity, organic silt
HIGHLY-ORGANIC SOIL	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols (symbols separated by a hyphen, i.e., SP-SM, SAND with silt) are used for coarse-grained soils with 10 percent fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols (symbols separated by a slash, i.e., CL/ML and GW/SW) indicate that the soil may fall into one of two possible basic groups.
3. In log solid line designate estimated unit boundaries and dashed line are changes within the unit.

Minor Road Reservoir
Kelso, Washington

SOIL CLASSIFICATION AND LOG KEY

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FIG. 3
Sheet 2 of 2

POST 07 BORING CLASS2 MINOR RD RESERVOIR GINT.GPJ SWNEW.GDT 1/21/10

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

Major constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).

Modifying (secondary) constituents precede the major constituents (i.e., silty SAND) and compose 15 to 45 percent, by weight, for fine-grained soils and 30 to 45 percent, by weight, for coarse-grained soils.

Minor constituents follow major and modifying constituents (i.e., silty SAND with gravel) and compose 10 percent, by weight, for fine-grained soils and 10 to 25 percent, by weight for coarse-grained soils.

Trace constituents follow all other constituents and are labeled "trace" (i.e., silty SAND with trace gravel). Trace constituents comprise 5 percent, by weight of coarse-grained soils and 5 to 10 percent, by weight of fine-grained soils.

Percentages are based on estimating amounts to the nearest 5 percent.

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	Fill
Vibrating Wire	

Minor Road Reservoir
Kelso, Washington

SOIL CLASSIFICATION AND LOG KEY

February 2010 24-1-03576-001

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FIG. 3
Sheet 1 of 2

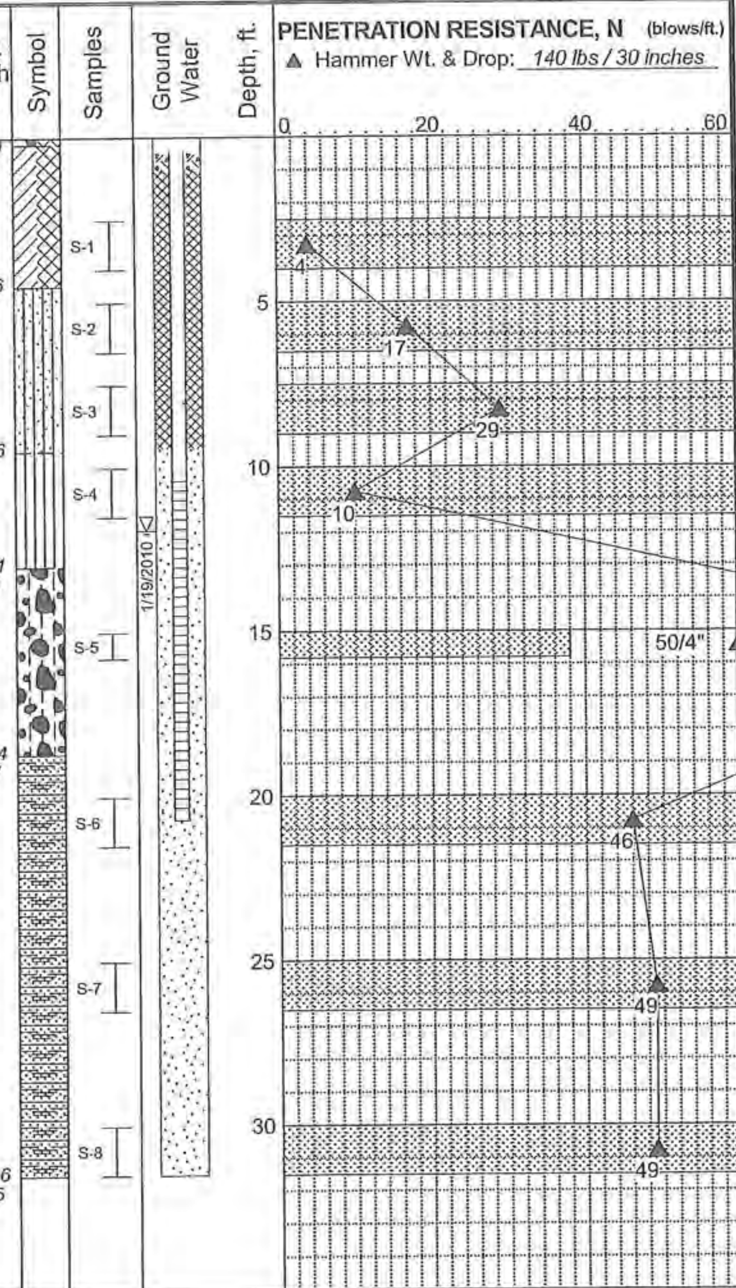
Total Depth:	31.5 ft.	Northing:	~ 308,369 ft.	Drilling Method:	HSA and Mud Rotary
Top Elevation:	172.13	Easting:	~ 1,032,342 ft.	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-55 Track rig
Horiz. Datum:	NAD83 St.Pl. ft.	Offset:	~	Other Comments:	WA Dept of Ecol. Well Tag #APL759

Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. / Depth (ft.)
GRAVEL base coarse; parking lot base. (GP)	171.9 / 0.2
FILL	
Soft tan-brown and gray silty CLAY trace of gravel and sand; moist, medium plasticity, fine to coarse sand, angular to subrounded gravel. (CL)	167.6 / 4.5
Medium dense to very stiff tan-brown to tan sandy SILT trace of clay; moist, non-plastic to low plasticity, fine sand, micaceous, slight iron oxidation and red-brown staining, occasional medium to high plasticity zones. (ML)	162.6 / 9.5
TROUTDALE FORMATION	
Loose red-brown to gray-tan sandy SILT; moist to wet, non-plastic, fine sand, micaceous, slight iron oxidation and staining to 10.3 feet, occasional clayey zones. (ML)	159.1 / 13.0
Very dense brown to red-brown silty sandy GRAVEL; moist, fine to coarse sand, rounded to angular(fragments) gravel, dark brown staining on gravel. (GM)	153.4 / 18.7
SANDSTONE: Very low to low strength (R1-R2) red-orange and tan, fine to medium grained sandstone, trace silt and clay, stratified, widely to very widely spaced joints (none observed), slight iron oxidation and staining, feldspathic and micaceous sand grains, fine shell fragments.	
COWLITZ FORMATION	
Bottom of Boring Completed January 08, 2010.	140.6 / 31.5



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

● % Water Content
Plastic Limit ——●—— Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-1

February 2010 24-1-03576-001

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FIG. 4

Total Depth: 51.5 ft.	Northing: ~ 308,360 ft.	Drilling Method: Mud Rotary	Hole Diam.: 6 in.
Top Elevation: 183.19	Easting: ~ 1,032,398 ft.	Drilling Company: Western States	Rod Type: NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment: CME-55 Track rig	Hammer Type: Automatic
Horiz. Datum: NAD83 St.Pl. ft.	Offset: ~	Other Comments: WA Dept of Ecol. Well Tag #APL760	

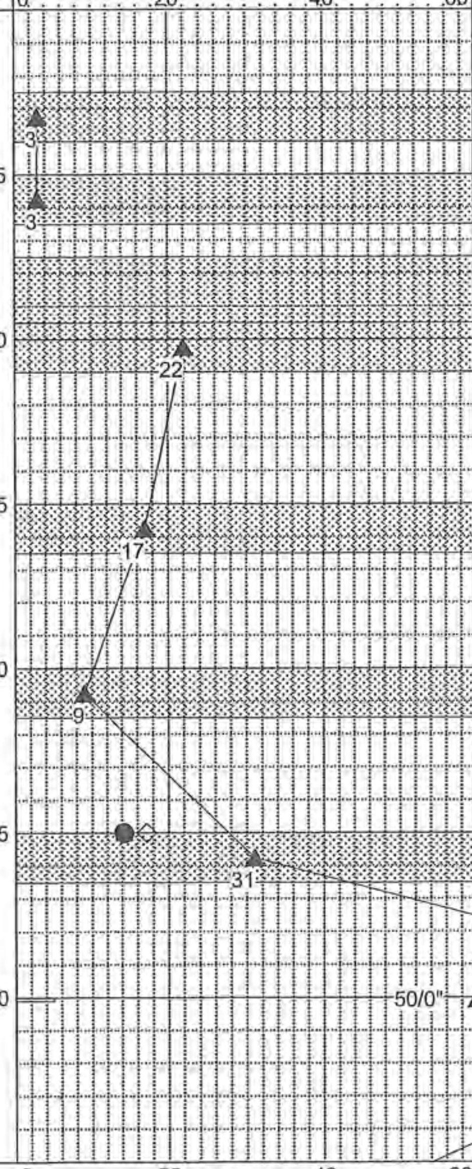
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples	Ground Water
Sod, approximately 4-inches thick.	182.9 / 0.3		
Soft brown silty CLAY trace of sand and gravel; moist, medium plasticity, fine to coarse sand, subangular to rounded gravel, occasional pockets of brown-gray silt, disturbed texture. (CL) **FILL**		S-1, S-2, S-3	
Very stiff brown and gray silty CLAY to clayey SILT trace of sand; moist, low to medium plasticity, fine sand, occasional gray clay zones, relict fine gravel or iron oxide nodule deposits. (CL/ML) **TROUTDALE FORMATION**	173.4 / 9.8	S-4	
Medium dense brown sandy SILT trace of gravel and clay; moist, non-plastic to low plasticity, fine to medium sand, fine dark brown stained rounded gravel, micaceous, slight iron oxidation and red-brown staining. (ML)	170.2 / 13.0	S-5	
Stiff brown SILT trace of clay and sand; moist to wet, low plasticity, fine sand, micaceous, slight iron oxidation and staining, occasional clayey zones. (ML)	165.2 / 18.0	S-6	1/19/2010
Dense to very dense brown silty sandy GRAVEL; moist, non-plastic fines, fine to coarse sand, rounded gravel, dark brown stained gravel clasts. (GM)	157.7 / 25.5	S-7, S-8	
SANDSTONE: Very low to low strength (R1-R2) tan with occasional red-orange zones, fine to medium grained sandstone, trace silt.	152.2 / 31.0		

CONTINUED NEXT SHEET

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Recovery (%)
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit

∇ 22.2

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-2

February 2010 | 24-1-03576-001

SHANNON & WILSON, INC.
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FIG. 5
Sheet 1 of 2

MASTER_LOG_E_MINOR RD RESERVOIR GINT.GPJ SHAN_WIL.GDT 2/10/10 Log: CKS Rev: Typ: CKS

Total Depth: 51.5 ft.	Northing: ~ 308,360 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation: 183.19	Easting: ~ 1,032,398 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	CME-55 Track rig	Hammer Type:	Automatic
Horiz. Datum: NAD83 St.Pl. ft.	Offset: ~	Other Comments:	WA Dept of Ecol. Well Tag #APL760		

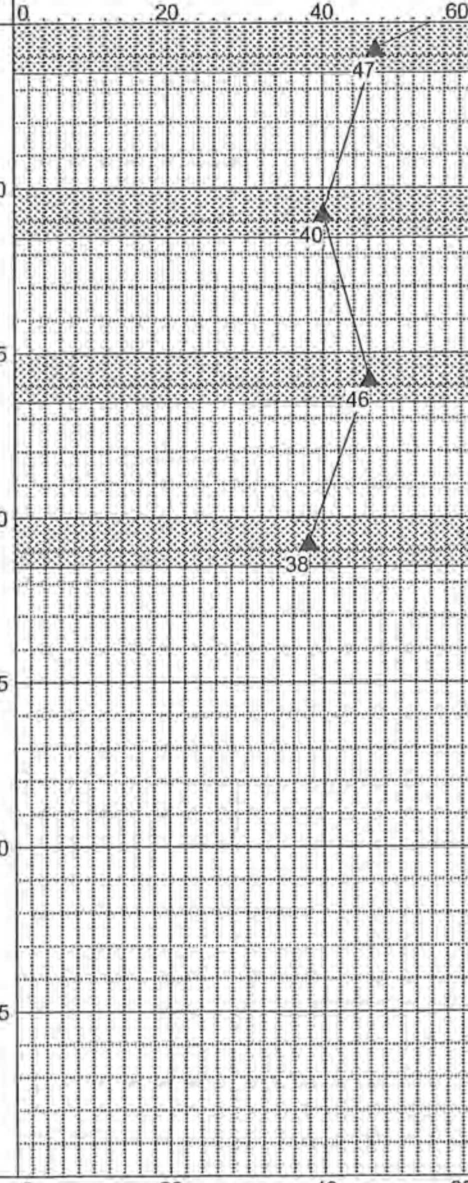
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
continued, SANDSTONE: Very low to low strength (R1-R2) tan with occasional red-orange zones, fine to medium grained sandstone, trace silt, stratified, widely to very widely spaced joints, micaceous sand grains. **COWLITZ FORMATION**			S-9 S-10 S-11 S-12		40 45 50
Bottom of Boring Completed January 11, 2010.	131.7 51.5				55 60 65

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-2

February 2010 24-1-03576-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 5
Sheet 2 of 2

MASTER_LOG_E MINOR RD RESERVOIR GINT.GPJ SHAN_WIL.GDT 2/10/10 Log: CKS Rev: Typ: CKS

Total Depth:	27 ft.	Northing:	~ 308,466 ft.	Drilling Method:	Mud Rotary
Top Elevation:	179.20	Easting:	~ 1,032,388 ft.	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-55 Track rig
Horiz. Datum:	NAD83 St.Pl. ft.	Offset:	~	Other Comments:	WA Dept of Ecol. Well Tag #APL761

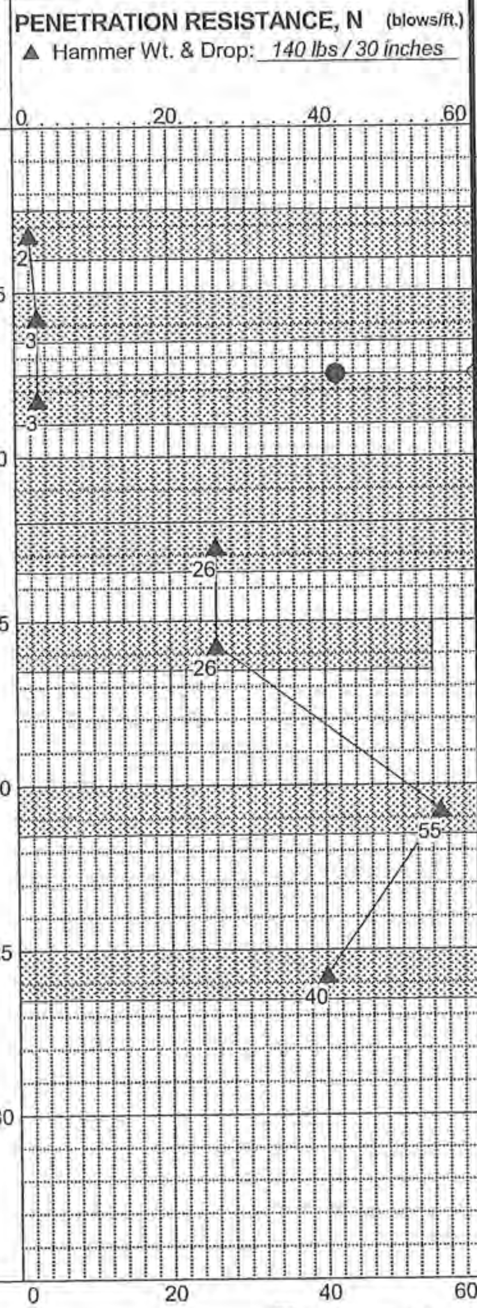
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)	Samples
Sod, approximately 3 to 4-inches thick.	179.0 / 0.2	
Very soft to soft brown silty CLAY trace of sand; moist, low to medium plasticity, fine to coarse sand, occasional pockets of brown silt and silty SAND, occasional organic debris, disturbed texture. (CL) **FILL**	173.0 / 6.2	S-1, S-2
Very loose brown sandy SILT trace of gravel; moist to wet, non-plastic to low plasticity, fine to medium sand, rounded gravel with dark brown staining, occasional organic flecks. (ML) **TROUTDALE FORMATION**	167.5 / 11.7	S-3, S-4
Medium dense grading to very dense red-brown to gray-brown silty sandy GRAVEL; moist, non-plastic to low plasticity fines, fine to coarse sand, rounded gravel with red-brown and dark brown staining, micaceous, slight iron oxidation, occasional to scattered severely weathered gravel clasts. (GM)	154.5 / 24.7	S-5, S-6, S-7
SANDSTONE: Very low to low strength (R1-R2) red-orange to tan, fine to medium grained sandstone, trace silt, stratified, widely to very widely spaced joints, slight iron oxidation and red-brown staining, micaceous sand grains. **COWLITZ FORMATION**	152.2 / 27.0	S-8
Bottom of Boring Completed January 12, 2010.		

Ground Water: 1/19/2010 ▽



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ⊢●⊣ Liquid Limit

▽ 11.7

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-3

February 2010 24-1-03576-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6

MASTER LOG E MINOR RD RESERVOIR GINT.GPJ SHAN WIL.GDT 2/10/10 Log: CKS Rev: Typ: CKS

Total Depth:	45 ft.	Northing:	~ 308,268 ft.	Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	180.47	Easting:	~ 1,032,395 ft.	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-55 Track rig
Horiz. Datum:	NAD83 St.Pl. ft.	Offset:	~	Other Comments:	WA Dept of Ecol. Well Tag #APL762

Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

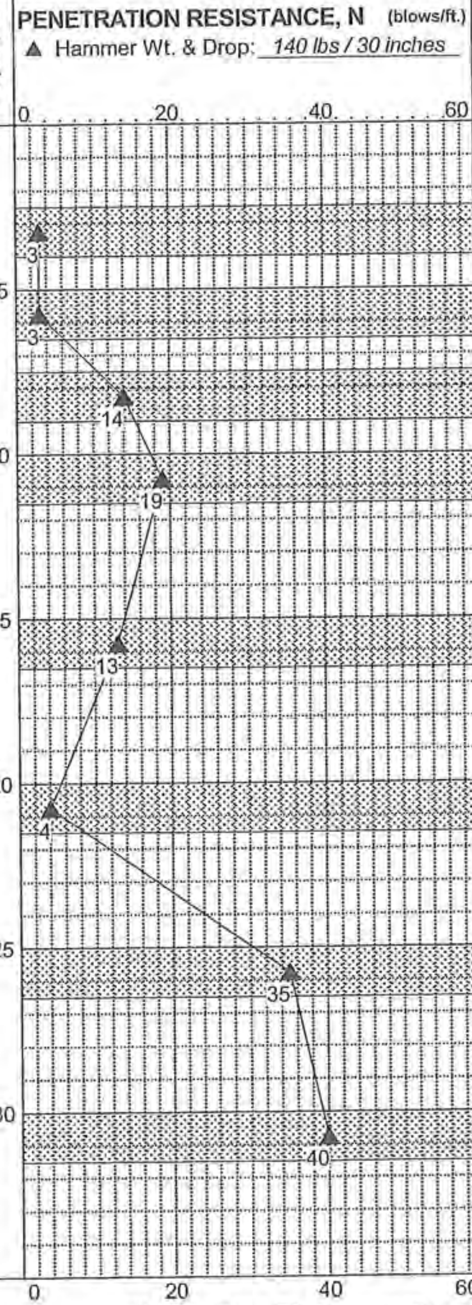
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)	Samples
Sod, approximately 3-inches thick.	180.3 / 0.2	
Soft brown and gray silty CLAY trace of sand; moist to wet, medium plasticity, fine to coarse sand, occasional organic debris and roots, slight iron oxidation and red-orange staining. (CL) **FILL**	173.5 / 7.0	S-1, S-2
Stiff gray and orange-brown silty CLAY trace of sand; moist, medium to high plasticity, fine to coarse sand, micaceous, occasional angular severely weathered gravel fragments at 11 to 11.5 feet, slight to moderate iron oxidation and staining. (CL/CH) **TROUTDALE FORMATION**	167.5 / 13.0	S-3, S-4
Medium dense grading to soft red-brown and tan-brown clayey SILT to sitly CLAY with sand trace of gravel; moist, wet below 20.6 ft, low plasticity with occasional medium plasticity zones, fine to medium sand, subrounded gravel, slight to moderate iron oxidation and staining, quick dilatency in the clayey SILT zones, CLAY layer from 20.2 to 20.6 feet.	159.9 / 20.6	S-5, S-6
(ML/CL)	157.5 / 23.0	
GRAVEL layer indicated by cuttings and drilling action. (GM)	156.5 / 24.0	
Dense red-brown and light gray clayey silty SAND; moist, low plasticity fines, fine to medium sand, slight iron oxidation and red-brown staining. (SM/SP) **COWLITZ FORMATION**	151.5 / 29.0	S-7
SANDSTONE: Very low to low strength (R1-R2) gray and brown to gray, fine to medium grained sandstone, faintly stratified in upper 2 feet to stratified, widely to very widely spaced joints (none observed), slight iron		S-8

Ground Water: 1/19/2010

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

▽ 18.2.

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-4

February 2010 24-1-03576-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7
Sheet 1 of 2

MASTER LOG E MINOR RD RESERVOIR GINT.GPJ SHAN WIL.GDT 2/10/10 Log: CKS Rev: Typ: CKS

Total Depth: 45 ft.	Northing: ~ 308,268 ft.	Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation: 180.47	Easting: ~ 1,032,395 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	CME-55 Track rig	Hammer Type:	Automatic
Horiz. Datum: NAD83 St.Pl. ft.	Offset: ~	Other Comments:	WA Dept of Ecol. Well Tag #APL762		

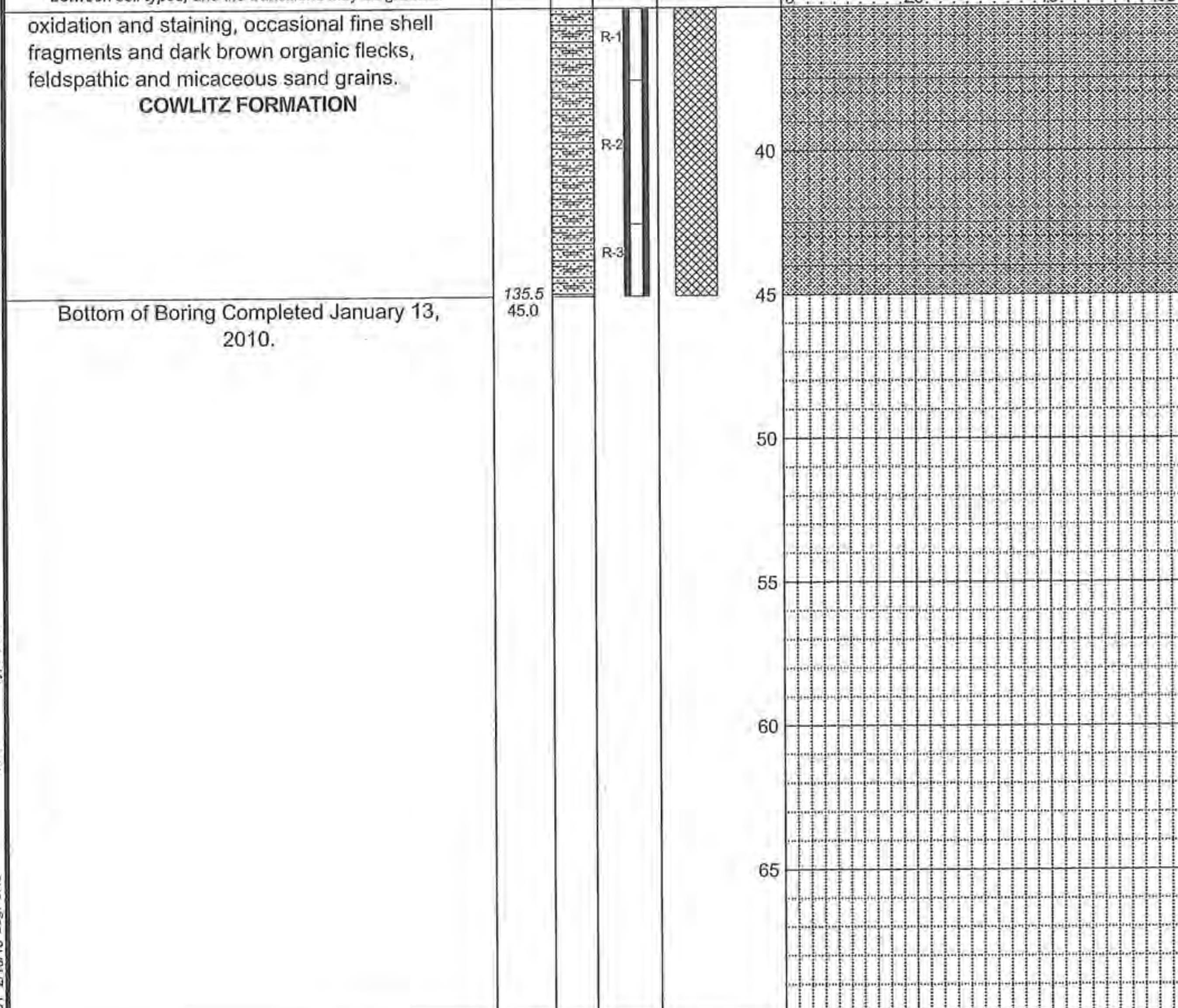
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



RQD (%) Recovery (%)

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit

LEGEND

* Sample Not Recovered

Standard Penetration Test

Rock Core

Piezometer Screen and Sand Filter

Bentonite-Cement Grout

Bentonite Chips/Pellets

Bentonite Grout

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.
5. UCS: Unconfined Compressive Strength, PSI: Pounds per Square Inch.

Minor Road Reservoir
Kelso, Washington

LOG OF BORING B-4

February 2010 24-1-03576-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7
Sheet 2 of 2

MASTER_LOG_E MINOR RD RESERVOIR GINT.GPJ SHAN_WIL.GDT 2/10/10 Log: CKS Rev: Typ: CKS

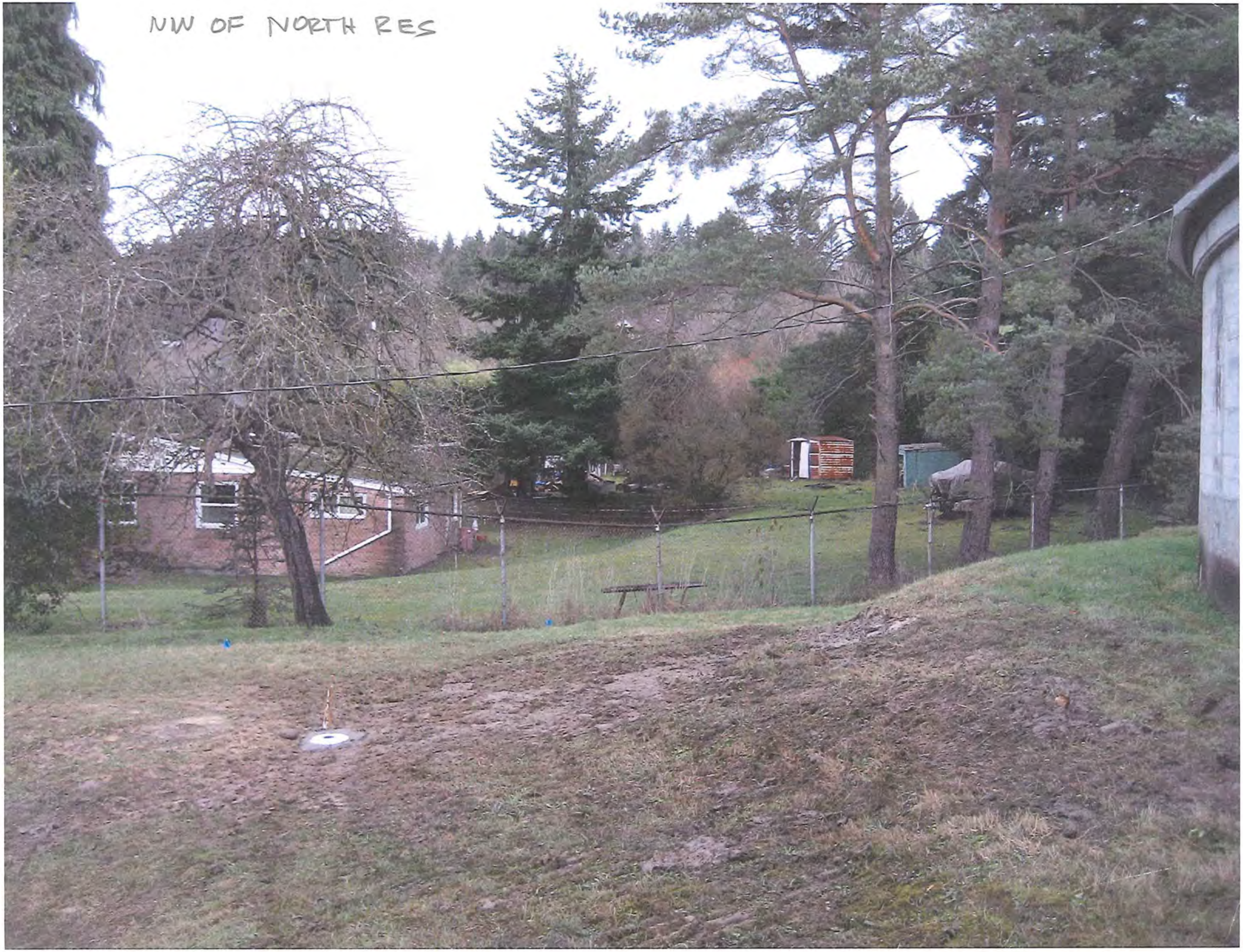
APPENDIX B

MISCELLANEOUS RESERVOIR PHOTOGRAPHS


NORTH RES

NORTH RES

NORTH RES


NW OF NORTH RES

SOUTH RES


SOUTH RES

SOUTH RES

APPENDIX C

IMPORTANT INFORMATION
ABOUT YOUR GEOTECHNICAL REPORT

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based on interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

UNIT NO. 6.---NEW RESERVOIR:

The furnishing of all material and labor for the construction and complete installation of a twin reinforced concrete reservoir of an approximate combined capacity of 2,000,000 gallons of water within the area described as Lots 1, 2, 3, 4, 5, 18, 19, 20, 21, 22, of Block 1, in the Sunrise Addition to the City of Kelso, Washington, including all connections for said reservoir with supply line and discharge line, equipping and connecting said reservoir with overflow and drain lines. The construction of a valve house, including the installation of all valves, fittings and control apparatus incident to the completion of this unit according to plans and specifications.

UNIT NO. 7.---DISCHARGE LINE---PUMP HOUSE TO RESERVOIR:

The furnishing of all labor and material necessary to the construction and complete installation of an 8 inch and 12 inch discharge line, from the pump house to both the old and new reservoir, making connections at the pump house and laying discharge lines southwesterly along the river bank to a point opposite Fir street produced, thence in an easterly direction along Fir street and across Hillsdale Addition to the north line of Sunrise Addition, connecting the 12-inch line to the new reservoir at this point, continuing easterly with the 8-inch line to a connection with old reservoir, including furnishing and installing all valves, pipe, structures and equipment incident thereto, all according to plans and specifications.

UNIT NO. 8.---NEW RESERVOIR DISTRIBUTING LINE:

The furnishing of all labor and material for the construction and complete installation of a system of distributing mains from the 2,000,000 gallon reservoir, southerly and westerly along the county road to Donation street; thence west along Donation street to a connection with the present water system at Front street; from the intersection of Donation and Fourth street southerly along Fourth street to Oak street; thence easterly along Oak street to Fifth street; thence southeasterly along Grade street to a connection with present mains at Vine street, including cutting and re-connecting to existing mains, separating high head and low head mains, furnishing and connecting fire hydrants, valves and other fittings and appurtenances, all according to plans and specifications.

UNIT NO. 9.---ADDITIONAL SUPPLY LINE, WEST KELSO:

The furnishing of all labor and material for the complete installation of an 8-inch supply line from a connection at the foot of Donation street in East Kelso across the Cowlitz River and southerly along the county road and Front street to a connection with the present distributing system in West Kelso at the intersection of Front and Fir street, including all labor and material and equipment incident to crossing the Cowlitz river, furnishing and connecting all fire hydrants, valves and other fittings and appurtenances, all according to plans and specifications.

UNIT No. 10.---OLD RESERVOIR DISTRIBUTING MAINS:

The furnishing of all labor and material for the complete installation of a system of distributing mains from the present 200,000 gallon reservoir, southerly and westerly along the county road to a connection with the present system at Seventh and Donation streets; the construction of 4-inch and 6-inch mains on Columbia and Cowlitz streets, furnishing and connecting fire hydrants, valves, and other fittings and appurtenances, all according to plans and specifications.

Concrete Structures

DESCRIPTION:

Portland Cement concrete structures shall be built where called for on plans or where directed in writing by the Engineer.

CLASSES OF CONCRETE:

There will be the following general classification of concrete: Class B, Class C, and Class E. Each class will in general be used as hereinafter specified and elsewhere where shown on the plans.

Class B. Concrete. Class B concrete shall be mixed in the proportions of one part cement, two parts sand, and four parts broken stone or gravel. Class B concrete shall be used in all reinforced concrete other than that covered by Classes C and E. The

coarse aggregate may be 2½-inch,
plans.

Class C Concrete. Class C conc
cement, three parts sand, and five
shall be used in all footing blocks n

Class E Concrete. Class E Conc
(10%) of cement added. Class E c
is placed under water.

The number of barrels of cem
es of concrete is as follows:

Class B Concrete: 1.57 barrels
Class C Concrete: 1.22 barrels
Class E Concrete: 1.34 barrels

CONCRETE MATERIALS:

Portland Cement. The cement u
quality, dry and free from lumps and
useage has proved to possess the pro
tended. It shall be delivered on the
properly labeled, and must be well
livered on the work in advance, in s
tunity of making tests before the cem
be stored separately. Cement stored
months, must be held for retest. If
storage, as shown by the tests of this
added to the mix at the Contractor's
may be rejected.

The cement shall meet the requi
land cement, adopted by the Americ
tion C9-21, with all subsequent ame
ciety.

Fine Aggregate. Sand shall co
which will not disintegrate when exp
loam, sticks, organic matter, and oth
the Engineer. Sand shall be tested f
test. It shall be well graded from c
retained on a one hunderd (100) me
nor more than thirty-five (35) per ce
per cent shall pass a screen having sq
tween wires.

Sand shall be of such quality th
cement and three (3) parts by wei
quettes, shall at the age of twenty-ei
with similar briquettes of Standard
proportion and of the same consiste
be used in Class B concrete and at le
in Class C, and E concrete, and if n
cement shall be added to increase the
additional cement shall be furnished

Coarse Aggregate. Coarse Aggre
a French Coefficient of wear of not l
thin, elongated or laminated pieces, st
clay, or other materials adhering to t
brushed off with the hand or remove
move the sand and shall be well gra
to secure a minimum of voids. Agg
may be rejected. Gravel shall be wa

For Class B concrete the size of
will pass a revolving screen having
one and one-half (1½) inches, or thr

; for the construction and complete instal-
r of an approximate combined capacity of
escribed as Lots 1, 2, 3, 4, 5, 18, 19, 20, 21,
he City of Kelso, Washington, including all
ne and discharge line, equipping and con-
ain lines. The construction of a valve
es, fittings and control apparatus incident to
ans and specifications.

HOUSE TO RESERVOIR:

necessary to the construction and complete
ge line, from the pump house to both the
at the pump house and laying discharge
a point opposite Fir street produced, thence
across Hillsdale Addition to the north line
line to the new reservoir at this point, con-
nection with old reservoir, including fur-
ures and equipment incident thereto, all ac-

BUTING LINE:

al for the construction and complete in-
from the 2,000,000 gallon reservoir, south-
Donation street; thence west along Dona-
water system at Front street; from the
southerly along Fourth street to Oak
fth street; thence southeasterly along Grade
Vine street, including cutting and re-con-
ead and low head mains, furnishing and con-
ngs and appurtenances, all according to

E, WEST KELSO:

d for the complete installation of an 8-inch
Donation street in East Kelso across the
y road and Front street to a connection with
so at the intersection of Front and Fir
l equipment incident to crossing the Cow-
hydrants, valves and other fittings and ap-
ecifications.

BUTING MAINS:

for the complete installation of a system of
00 gallon reservoir, southerly and westerly
the present system at Seventh and Dona-
l 6-inch mains on Columbia and Cowlitz
rants, valves, and other fittings and appur-
ations.

Structures

hall be built where called for on plans or

sification of concrete: Class B, Class C, and
l as hereinafter specified and elsewhere

ll be mixed in the proportions of one part
ken stone or gravel. Class B concrete shall
n that covered by Classes C and E. The

coarse aggregate may be 2½-inch, 1½-inch or ¾-inch maximum size, as shown on the plans.

Class C Concrete. Class C concrete shall be mixed in the proportions of one part cement, three parts sand, and five parts broken stone or gravel. Class C concrete shall be used in all footing blocks not reinforced, pier shafts and heavy walls.

Class E Concrete. Class E Concrete shall be a Class C concrete with ten per cent (10%) of cement added. Class E concrete shall be used for all work where concrete is placed under water.

The number of barrels of cement required per cubic yard for the various classes of concrete is as follows:

Class B Concrete: 1.57 barrels of cement per cu. yd.
Class C Concrete: 1.22 barrels of cement per cu. yd.
Class E Concrete: 1.34 barrels of cement per cu. yd.

CONCRETE MATERIALS:

Portland Cement. The cement used shall be a true Portland cement of the best quality, dry and free from lumps and all foreign material, it shall be a cement which useage has proved to possess the proper qualifications and uniformity for the work intended. It shall be delivered on the work in the original packages, in good condition, properly labeled, and must be well protected from rain and dampness. It shall be delivered on the work in advance, in such quantities as to afford the Engineer an opportunity of making tests before the cement shall be used. Each shipment or car lot shall be stored separately. Cement stored by the Contractor for a period longer than two months, must be held for retest. If the cement has lost strength during the period of storage, as shown by the tests of this department, sufficient additional cement must be added to the mix at the Contractor's expense, to overcome such loss, or the cement may be rejected.

The cement shall meet the requirements of the standard specifications for Portland cement, adopted by the American Society for Testing Materials, Serial Designation C9-21, with all subsequent amendments and additions thereto adopted by said society.

Fine Aggregate. Sand shall consist of clean, hard, durable, uncoated particles, which will not disintegrate when exposed to the weather, and shall be free from clay, loam, sticks, organic matter, and other impurities. It shall be washed if required by the Engineer. Sand shall be tested for organic matter by the standard colorimetric test. It shall be well graded from course to fine. Ninety-five (95) per cent shall be retained on a one hunderd (100) mesh screen. Not less than seventeen (17) per cent nor more than thirty-five (35) per cent shall pass a No. 28 sieve. One hundred (100) per cent shall pass a screen having square openings measuring one-fourth (¼) inch between wires.

Sand shall be of such quality that mortar composed of one (1) part of Portland cement and three (3) parts by weight of sand in question when made into briquettes, shall at the age of twenty-eight (28) days, have a strength ratio, compared with similar briquettes of Standard Ottawa sand mixed at the same time, in the same proportion and of the same consistency, of one hundred (100) per cent for sand to be used in Class B concrete and at least eighty-five (85) per cent for sand to be used in Class C, and E concrete, and if not testing below seventy-five (75) per cent enough cement shall be added to increase the strength ratio to eighty-five (85) per cent. Such additional cement shall be furnished at the Contractor's expense.

Coarse Aggregate. Coarse Aggregate shall consist of broken stone or gravel having a French Coefficient of wear of not less than 8. Coarse Aggregate shall be free from thin, elongated or laminated pieces, sticks, and other foreign matter, and containing no clay, or other materials adhering to the pieces in such quantity that it cannot be lightly brushed off with the hand or removed by dipping in water. It shall be screened to remove the sand and shall be well graded between limits as hereinafter specified so as to secure a minimum of voids. Aggregates having an excess of any one sized particles may be rejected. Gravel shall be washed if required by the Engineer.

For Class B concrete the size of coarse aggregate shall be such that all particles will pass a revolving screen having circular openings, two and one-half (2½) inches, one and one-half (1½) inches, or three-fourths (¾) inch in diameter as specified on the plans; and that ninety-five per cent (95%) will be retained on a screen having circular holes one-fourth (¼) inch in diameter.

In classes C, and E, concrete the size of coarse aggregate shall be such that all particles will pass a revolving screen having circular openings three (3) inches in diameter and ninety-five per cent (95%) will be retained on a screen having circular holes one-fourth (¼) inch in diameter.

For Class C concrete a certain amount of large boulders, may where shown on the plans, be included in the concrete, provided they are hard, clean rock of strength equal to the coarse aggregate. Each boulder shall be entirely surrounded by the concrete and be separated at least six (6) inches from any adjacent boulder from the surface of the concrete. In no case shall any boulder in its largest dimension be greater than one-half the thickness of the section of the concrete.

PROPORTIONING COARSE AND FINE AGGREGATES:

In Class B concrete the relative amounts of approved fine and coarse aggregates may be changed in order to develop the greatest possible strength. That is to say in a 1-2-4 mixture for example the relative portions 2 and 4 may be changed, keeping however the original 1-6 mixture, and the specified amount of cement per cubic yard of concrete. The changes in the proportioning of the concrete shall be made only at the direction and under the supervision of the Engineer.

Water. The water used in mixing concrete shall be fresh, clear, free from oil, acid, alkalies, vegetable or other foreign matter.

Measuring. All materials shall be accurately measured by volume, one barrel of cement as packed by the manufacturer shall be considered as 3.8 cubic feet by measure. Fine and coarse aggregate shall be measured loose. The Contractor shall provide and keep on the job a one-cubic foot box, and measure the wheelbarrows or other containers when requested by the Engineer, or the Engineer may require the use of measuring boxes.

HYDRATED LIME:

All hydrated lime shall be manufactured from freshly burned quicklime, thoroughly hydrated and not containing over 25% of water of hydration. All hydrated lime upon test shall be 97½% pure. Equal parts of hydrated lime and cement shall be thoroughly mixed and made into a pat, which shall be kept under moist cloth for 24 hours. The pat shall then be boiled for 5 hours. Any sign of swelling or disintegration may be cause for rejection of the hydrated lime. A fineness test should show 95% of hydrated lime passing a 100-mesh screen.

Hydrated lime, when required, shall be incorporated in the mixture in the amount specified. This amount, given as a percentage of the cement, shall be treated as additional material and not as replacing any cement. Generlly the hydrated lime shall not exceed five (5) per cent by volume of the cement. In special cases not to exceed ten (10) per cent may be used.

Where hydrated lime is specified on the plans, compensation for same shall be included in the contract price for the concrete.

Where hydrated lime is not specified on the plans, but is required by subsequent order of the Engineer, compensation will be allowed on the basis of extra work.

Consistency. Sufficient water shall be used, in mixing plain concrete to produce a mixture which will flatten and quake when deposited in place but not enough to cause it flow, and in mixing concrete in which reinforcement is to be embedded, to produce a mixture which will flow sluggishly when worked and which at the same time can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar. In no case shall the quantity of water used be sufficient to cause the collection of a surplus in the forms.

Mixing. Unless hand-mixing is specifically permitted by the Engineer, the mixing shall be done in a batch mixer of a type approved by the Engineer, and which will insure the uniform distribution of the materials throughout the mass so that the mixture is uniform in color and smooth in appearance. The mixing shall continue for a minimum time of one (1) minute, after all the ingredients are assembled in the drum, during which time the drum shall revolve at the speed for which it was designed, but shall not make less than twelve (12) nor more than (18) revolutions per minute. The volume of the mixed materials used per batch shall not exceed the manufacturer's rated capacity of the drum. The mixer shall be of sufficient capacity to make the largest continuous run necessary as shown by the pouring diagrams. In large structures requiring long continuous runs, a duplicate mixing plant may be required by the Engineer as an emergency against breakdowns. All machinery, as mixers, elevators, pumps, etc., accessory to the mixing plant must be in first class mechanical condition. The Contractor shall immediately repair or replace any machinery or plant equipment which is insufficient for its purpose.

The entire contents of the drum shall be discharged before any materials are placed therein for the succeeding batch.

Hand Mixing. Hand mixing shall be performed as follows: The Contractor shall provide water-tight platforms of an approved size and form. The sand and cement, each in its proper proportion, shall be thoroughly mixed dry until the color of the mixture is uniform throughout.

The coarse aggregate in the proper proportion, shall then be spread evenly over the dry mixed sand and cement. The entire mixture shall then be turned with shovels, adding water from time to time, until a uniform color is obtained throughout and the mixture is plastic and quakes under the blows of a tamper.

Placing. Concrete shall not be placed in foundations until the Engineer has approved the depth and character of the foundation; nor shall concrete be placed in forms until the engineer has approved them and checked the placing of the steel.

Concrete shall be placed as rapidly as possible after the mixing process is complete. In no case shall a period of more than 30 minutes elapse between the completion of the mixing and the placing in the forms. All mortar or concrete that has taken an initial set before being placed shall be removed, and not used in the work. No retempering of mortar or concrete shall be allowed. Concrete shall be so conveyed and placed that there will be no distinct separation of the different ingredients. Special care must be taken in placing all concrete to keep as perfect a mixture as possible throughout the whole process of placing. The concrete shall be carefully placed around and under the reinforcing steel in such a way as not to displace the steel from its true position. All concrete shall be placed in continuous horizontal layers, and all rock or sand pockets must be avoided. When such pockets do occur, they must be removed at the Contractor's expense. Plastering will not be allowed. All concrete shall be worked with a "puddling" or "spudding" bar or tool immediately after it is placed in order to insure the most dense mix possible and to obtain a smooth surface. All concrete shall be poured in the sequence given on the pouring diagram, each numbered section requiring continuous pouring until its completion.

Dropping concrete any considerable distance, or in large quantities and running it down long inclined slopes on the forms will not be allowed. Pouring through spouts, either vertical or inclined, will in general not be allowed. When absolutely necessary, the method and manner of doing so will be subject to the approval of the Engineer and must not be done without his permission.

POURING CONCRETE IN COLD WEATHER:

No concrete shall be poured when the atmospheric temperature is below 35 degrees Fahr., unless permission to do so is granted by the Engineer. When permission is granted by the Engineer, all concrete poured in freezing weather shall be done as follows:

The Contractor shall at his own expense, equip himself with sufficient canvas and framework, or other type of housing, to if necessary completely enclose the structure being poured, in such a way that the air surrounding such structure can be kept at a temperature of not less than 45 degrees Fahr. for a period of 5 days after the pouring is complete.

Sufficient heating apparatus, such as stoves, salamanders or steam equipment and fuel, to furnish all required heat, shall be furnished by the Contractor at his expense.

All water used for mixing concrete shall be heated to a temperature of a least 70 degrees, but not over 150 degrees Fahr.

Aggregates shall be heated either by steam or by dry heat to a temperature of at least 70 but not more than 150 degrees Fahr.

The temperature of the mixed concrete should not be less than 60 degrees Fahr. at the time of placing in forms.

In case of extreme weather, the Engineer may at his discretion, raise all the lower limiting temperatures for water, aggregate, and mixed concrete.

The Contractor assumes all risk connected with the pouring of concrete during freezing weather. The permission to pour concrete during such time, given by the Engineer, will in no way guarantee the results to the Contractor. Should concrete poured under such conditions prove unsatisfactory in any way, the Engineer shall still have the power to reject same just as if it had been poured without his permission.

CONCRETE IN WATER:

In no case shall concrete be placed in running water. Where placed under water it shall be so placed within the confines of a water tight compartment or caisson.

Concrete placed in still water inside an open form or caisson shall be placed by means of a "tremie," or closed bottom dump bucket.

Concrete placed under water shall be mixed with more water than is ordinarily permissable in order to produce better flowability and shall be a 1-3-5 mix with the addition of 10% more cement.

When the concrete is placed by means of a tremie the following added precautions shall be taken:

Keep the mouth of the tremie buried in the concrete at the bottom end.

Keep the tremie full to the top. In plac ing concrete through a tremie, two distinct handling devices must be used; one to raise, lower and place the tremie, the other to deliver the concrete to the tremie. When a batch is dumped in the tremie at the top, the tremie should be slightly raised, but not out of the concrete at the bottom, until the batch discharges to the bottom of the hopper, or the top of the tremie tube. The flow is then stopped by lowering the tremie.

Keep the concrete surface in the form as nearly level as possible.
Pour continuously until the required thickness of seal is placed.
When placed by means of a closed bottom dump bucket:
See that the bucket is full and completly closed before lowering into the water.
Lower slowly through the water until the bucket rests on the bottom.
Raise the bucket very slowly during the discharge travel, the object being to keep the water as still as possible at the point of discharge, and to agitate the mixture as little as possible.

In either method, if, for any unavoidable reason it is necessary to discontinue the pouring before the required thickness is placed, the Contractor will be required to pump out the form and remove all "scum," "sediment," or "laitance" before proceeding with the work.

CONSTRUCTION JOINTS:

At all construction joints allowed on the pouring diagram and at others which are unavoidably necessary, the following precautions shall be taken in joining old and new concrete:

Whenever a construction joint is made, it shall be either horizontal or vertical, or if the main reinforcement is inclined, normal to the direction of the main reinforcement.

If the section is subject to shear, sufficient material as a key, shear steel, or in some cases boulders, shall be provided to transmit such shear past the construction joint at the Contractor's expense.

Before placing adjoining concrete, the old face shall be perfectly cleaned and roughened and approved by the Engineer. Just immediately prior to placing the new concrete, the old face shall be coated with a thin layer of neat cement, and is subject to approval by the Engineer.

Where a joint must stand a hydrostatic pressure, a corrugated copper or zinc diaphragm sheet may be required.

Construction joints shall be avoided whenever possible, but when they are unavoidably necessary, they shall be so located as to have the least possible effect on the strength of the structure. Construction joints at more frequent intervals than shown on the pouring diagram will be allowed only upon permission of the Engineer, and when so allowed, shall be at the point of minimum shear and normal to the direction of the main reinforcement.

In general expansion joints in slabs, rails and walls shall be constructed with an approved type of felt and asphaltum or tar construction of the thickness shown on the plans.

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FORMS:

All forms shall be set true to the lines designated, and the interior shape and dimensions shall be such that the finished concrete shall coincide exactly with the plans for the structure. In general all form work for concrete structures shall be as follows:

FORM FOOTINGS:

All form footings must be properly designed to carry the maximum load that can come upon them. They shall be as near unyielding as possible under full load. Where pilings are necessary, they shall conform in all respects, as to bearing power, with standard specifications for piling. In cases of footings on rock or coarse sand and gravel, grouting may be required to insure uniform bearing.

POSTS:

All posts or columns supporting form work shall be designed according to the straight line formula for long columns, and in no case shall unit stresses or requirements herein specified be exceeded. Particular attention is called to end and transverse bearing on wood. All systems of supports shall be provided with wedges or other form of release which will permit of the uniform release and take up of forms.

STRINGERS AND BEAMS:

All stringers and beams used to support form work shall be particularly rigid; they shall show very small deflection under full load. To guard against undue deflection maximum bending fibre stress (tension) per square inch is limited to 1,200 pounds for fir and 12,000 pounds for steel.

BRACING:

All bracing shall be as rigid as possible and where there is any likelihood of movement, braces shall be provided with wedges to take up such displacement.

FACE LUMBER:

All sheeting shall be matched or tongue and grooved lumber of good quality free from loose knots, shakes or warped surfaces; it shall be dressed on one side and two edges and present a true plane surface to the concrete. All forms shall be as near watertight as possible. In narrow or thin walls where it is difficult to clean the bottom of forms, a board shall be left out to facilitate cleaning. All exposed corners shall be chamfered ¾ of an inch by placing a triangular strip in the form corners. For the purpose of form design concrete shall be assumed to exert on vertical surfaces an equivalent fluid pressure of 80 pounds per square foot for heights which may be poured in one hour continuous run. Horizontal surfaces shall be designed to withstand an equivalent liquid pressure of 150 pounds per square foot for each foot of height that may be reached in one hour's run. Oiling of forms may be required in certain cases when shown on the plans. The oil shall be applied several days before concrete is poured and shall be of such quantity that it is fully absorbed by the wood and will not discolor the surface of the concrete. In no case shall concrete be poured, in any form, before the form has been checked up by the Engineer.

WETTING OF FORMS:

Forms shall be thoroughly wetted in advance of pouring the concrete. No standing water will be permitted in the forms.

REMOVAL OF FORMS:

Various parts of the form work shall not be removed until the following time, after pouring has elapsed:

(a) Columns, wall faces (Not yet supporting loads) 3 to 4 days.

(b) Mass piers and abutments, 2 days.

(c) Sides of beams and girders, 5 to 7 days.

(d) Removal of arch centering or shoring under any beam or girder subject to bending stress, 28 days.

In special cases (d) may be modified to a greater or lesser time, but in no case less than 21 days.

Forms shall always be removed from columns before removing supports from beneath beams and girders, in order to determine the condition of the concrete column.

Forms shall not be removed from under concrete which has been placed at a temperature under 50 degrees Fahr., without first determining if the concrete has properly set, without regard to the time element.

In no case shall forms, centers, or falsework, be removed at any time without the approval and direction of the Engineer in charge.

CURING CONCRETE:

All floors of concrete and wall surfaces exposed to premature drying, shall as soon as possible after pouring, be covered with burlap or sand, and shall be kept damp continuously for a period according to the judgment of the Engineer, up to 10 days. The covering material shall not be removed from the surface it is covering for 14 days.

FINISHING:

The top surface of structures shall be formed of mortar of the same proportions and of the same quality of cement and sand as that which forms the matrix of the concrete, and shall be finished by cutting off the excess with a straight edge and rubbing the surface until smooth.

After removal of forms, all concrete shall show a smooth dense face. Any concrete which is porous shall be removed at the expense of the Contractor. No plastering of any surface will be allowed.

The lagging shall be removed as soon as practicable after the concrete has been poured and all lips and edgings where form boards have met, shall be removed at once with a sharp tool. Wires shall be clipped and set back at least one-fourth (¼) inch with a punch beneath the surface of the concrete. All holes shall be filled with a 1-2 mortar and floated to an even uniform surface.

All exposed surfaces shall be thoroughly washed with water and then treated with a neat cement wash or grout, composed of neat cement and water mixed to a creamy consistency. After the neat wash has set for twenty-four (24) hours, it shall be rubbed off with a coarse carborundum stone, applying water ahead of the stone. The entire surface shall then be washed and kept damp for several days.

PIERS, WALLS AND ABUTMENTS:

All general stipulations herein set forth as to method, materials, forms, concrete, finishing, foundations, etc., shall fully apply on the construction of piers, walls and abutments.

Piers, walls and abutments shall be poured according to the pouring diagram.

Ordinary piers, not subject to great horizontal shear will be poured in horizontal layers, as directed under "Pouring Concrete."

No backfilling for walls and abutments will be made until the concrete is at least 21 days old. This time may be increased to 28 days by the Engineer.

All pouring of concrete under water shall be done according to methods hereinbefore set forth. No pumping will be allowed from a cofferdam in which concrete is being poured. Cofferdams and caissons shall be kept to such heights as not to overflow and to exclude water.

Form work for piers shall be stiff and true to form, according to standard specifications under forms.

PAYMENT:

Concrete will be paid for at the contract price for the various classes of concrete, payment being made for the actual volume in place contained within the lines of the plans furnished by the Engineer. Contract price will include furnishing all materials for concrete, and all materials and labor incidental to construction of forms and falsework, and all labor, equipment, and, and other work incidental to placing the concrete and finishing same.

Excavation for foundation will be paid for at the contract unit price for "Excavation for Structures" as hereinafter specified.

Reinforcement

DESCRIPTION:

Concrete reinforcement shall consist of round or square bars, and unless otherwise specified, shall provide a mechanical bond with concrete at frequent intervals, and shall have a net sectional area equivalent to the section of a plain square bar of the size indicated on the drawing. Note: Square twisted bars are not considered to have a mechanical bond.

MATERIAL:

Unless otherwise specified, reinf
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STRUCTURAL STEEL:

All structural steel shall conforn
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IRON CASTINGS:

All iron castings shall meet the r
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STEEL CASTINGS:

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BENDING, STORING AND PLACIN

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SPLICING REINFORCING BARS:

All reinforcing bars shall be as
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ORDERING REINFORCING STEEL

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ANCHOR BOLTS:

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MEASUREMENT AND PAYMENT

All reinforcing steel shall be
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MATERIAL:

Unless otherwise specified, reinforcing steel shall meet the requirements of the standard Specification for Billet Steel Concrete Reinforcement Bars of the American Society for Testing Materials Serial Designation A15-14, structural steel grade.

STRUCTURAL STEEL:

All structural steel shall conform to the Standard Specifications for Structural Steel for Bridges, adopted by the American Society for Testing Materials, 1916, or any later revision thereof.

All structural steel shall be made by the open hearth process.

IRON CASTINGS:

All iron castings shall meet the requirements of the Standard Specifications of the American Society for Testing Materials. Serial Designation A-48-18.

STEEL CASTINGS:

All steel castings shall meet the requirements of the American Society for Testing Materials, Serial Designation on A27-16 (Class B.)

When purchased from warehouse in small lots reinforcement may, at the discretion of the Engineer, be accepted subject to the bending test only.

BENDING, STORING AND PLACING:

All steel shall be cut and bent by careful, competent men. It shall be bent cold to templates, which shall not vary appreciably from the shape and dimensions shown on the plans. All sharp bends will be avoided, and in no case shall a bend be of less radius than 3 diameters of the bar.

Steel shall be stored in a protected place and in a manner which will protect it from being bent or badly rusted. Before being placed in the forms, all steel shall be thoroughly cleaned from all rust, flakes, dirt, oil, or other material which will tend to lessen the bond strength with concrete.

All points of intersections of bars shall be firmly wired, making the whole reinforcing system rigid. Stirrups shall be firmly wired to the main reinforcement. The whole reinforcing system shall be held away from the forms by cement cubes or other approved form of spacer. Concrete shall be placed as soon as possible after the steel is placed, but no concrete shall be poured until the amount and position of steel is inspected by the Engineer. Any concrete placed contrary to this provision may be rejected.

SPLICING REINFORCING BARS:

All reinforcing bars shall be as long as possible, and splicing shall be done only when indicated on the approved plans. All deformed bars shall be lapped at least 40 diameters and thoroughly and firmly wired at the splices. Plain bars (when allowed) shall be lapped 50 diameters. Laps in adjacent bars shall be staggered.

ORDERING REINFORCING STEEL:

The Contractor shall order reinforcing steel at his own risk, with reference to length, bending an number of bars; and should check his orders from the plans.

ANCHOR BOLTS:

Anchor bolts of such size, number and dimension, as shown on the detailed plan, and as called for in this specification, shall be furnished by the Contractor. They shall be accurately set, either in the original concrete or in drilled holes after the concrete is set. If drilled, the holes shall be at least 1 inch larger in diameter than the anchor bolt, to afford ample room for "grouting in." If set in the original concrete, a gas pipe should be used at least 2 inches larger in diameter than the anchor bolt, using a heavy steel washer as an anchorage for the bolt at the bottom. This pipe must in all cases be filled with grout after the plates are set in their true position.

All anchor bolts shall be either "swaged," or have substantial steel washers at the bottom.

MEASUREMENT AND PAYMENT OF REINFORCING STEEL:

All reinforcing steel shall be measured and paid for by the pound at the contract price per pound for reinforcement.

Reinforcing steel shall include all plain or deformed bars embedded in concrete; all expansion plates and all anchor bolts. No allowance will be made for spreaders,

Reservoir and Appurtenances

All work to be done in connection with the construction of the reservoir not specifically covered in this section of these specifications shall be done according to the specifications covering the same class of work as found elsewhere in these specifications.

Earthwork shall include all excavations and embankments, pits and trenches. Grade and slope stakes will be set by the City Engineer for the guidance of the Contractor as may be necessary during the progress of the work, and all excavations and embankments, pits and trenches shall be carefully and accurately made in accordance therewith. Excavations shall be kept carefully drained, trenches and pits shall be securely timbered when necessary, and the slipping and erosion of slopes beyond the finished surfaces shall be prevented.

CLASSIFICATION:

Grading shall be classified under the heads of solid rock excavation, loose rock excavation and common excavation.

Solid rock excavation will include all rock in masses that cannot be removed without drilling and blasting and all boulders containing more than one-half cubic yard.

Loose rock excavation will include rock and boulders containing one cubic foot or more and less than one-half cubic yard, cemented gravel and hardpan. Cemented gravel will be defined as gravel caused to cohere by infiltrated calceneous or silicious matter or by the effect of such infiltration combined with pressure, and which cannot be loosened by picking or plowing with a six-horse team. Hardpan is defined as a mixture of clay sand pebbles and small rock, so firmly compacted that it cannot be loosened by picking or by plowing with a six-horse team.

Common excavation will include all surface stripping and all excavated material not included under the head of solid and loose rock excavation.

SURFACE STRIPPING:

Surface soil and all earth which contains vegetable or other perishable matter and all soft or objectionable material, shall be removed from any portion of the reservoir site and deposited in spoil banks as ordered by the City Engineer.

EXCAVATION, EMBANKMENT AND WASTE:

Excavation shall include all materials taken from within the limits of the work contracted for. The surplus beyond what is necessary to form the contiguous embankment may be disposed of in widening the embankment uniformly around each reservoir. All material remaining after the requirements set forth herein have been met shall be disposed of by the Contractor.

Excavation must not be carried down to, or below grade. All excavation must be left slightly above grade and finished by hand. Embankments shall be made from the excavated materials and shall be placed in horizontal layers of one foot in thickness, beginning at the outer slope of the bank and gradually building up in successive layers to the required height. Embankments shall be water settled and compacted according to the Engineer's instructions, the process meeting the requirements produced by the nature and condition of the excavated material.

PAYMENT:

Measurement will be made to lines twelve inches distant laterally from the neat lines of structure.

Grading will be paid for by the cubic yard of excavation at the respective price bid for the various classes of excavation provided for, and the price paid for such excavation shall be full compensation for excavating material, forming contiguous embankments or disposing of it as directed by the Engineer, and finishing the embankment slope and ditches as may be directed.

VALVES:

All gate valves shall conform to the specifications for gate valves as given in the section on Watermains and Appurtenances and in addition, where shown on the plans or where required by the Engineer, they shall be equipped with hand wheels, extension stems, pedestals or such combination of equipment as is necessary for the proper and convenient working of the valves.

HANDWHEELS:

Where shown on the plans the valves shall be provided with a handwheel, of ample dimensions, on top of the stem. An arrow will be cast in the metal in plain view indicating the direction the wheel is to be turned to open the valve.

EXTENSION STEMS:

Where indicated on the plans each valve shall be provided with an extension stem of wrought iron or steel, with a socket to fit the nut on the valve or pinion stem, and squared head or coupling at the upper end to connect securely with the hand wheel. The lower end of the extension shall be provided with a hole through it and also through the nut on the valve for a cotter pin. Before being installed the extension stem shall be painted with two coats of "P. & B." paint or its equal. The length of the extension stem shall be determined after the valves and pedestals have been placed in position.

PEDESTALS:

The Contractor shall provide and securely place in position cast iron pedestals for all gate valves where shown on the plans. All pedestals must be of a design and make approved by the Engineer. They shall be securely fastened in position by means of bolts or lag screws. After being set they shall be painted with not less than two coats of paint complying with the specifications for paint for iron. The color of the paint to be as the Engineer may direct.

PAYMENT:

Payment for all extension stems, handwheels, and pedestals shall be included in the rate bid for valves in place and shall include the furnishing of all material and labor required to completely install and paint the valves and appurtenances.

DRAIN:

Where indicated on the plan a 12-in. concrete sewer pipe drain shall be laid. The pipe shall conform in all respects to the standard specifications for concrete sewer pipe drain. The pipe shall be laid to the line and grade given and solidly bedded in the ground and shall be set with cement mortar. The mortar to be mixed of one part of cement to two parts of sand. The drain line shall be provided with fittings as may be required. At the outfall a concrete header containing not less than one cubic yard of concrete shall be constructed as directed by the Engineer.

Payment for the concrete sewer pipe drain shall be made at the rate bid per lineal foot in place and shall include furnishing all material, and all the labor necessary for all trenching, backfilling and all labor required in laying the pipe, and also the concrete header at the outfall.

SUB-DRAINS:

Sub-drains shall be laid where shown on the plans. All such pipe to conform to the standard specifications therefor and tobe of a make approved by the Engineer. The drains shall be laid true to the lines and grade given and the trenches shall be carefully formed with a minimum depth of 9-in. for the four-inch laterals. The size of the gravel used may range from a coarse sand to gravel not over one-half inch in diameter. After the tile is laid the trench shall be filled with the gravel as specified, and firmly compacted. The two six-inch trunk drain lines shall be connected with the reservoir drain through the concrete walls in the drain pit pit of the valve house, as shown on plans. Under the slope floor of the reservoir the joints of the six-inch trunk drain line shall be set with mortar as specified above.

Payment for sub-drains will be made at the rate bid per lineal foot which price will include furnishing all material and the laying of the pipe and all backfill including the gravel and all labor connected therewith.

EXPANSION JOINTS:

Expansion joints shall be provided in the concrete floor slabs and at other points as shown on the plans.

For horizontal expansion joints or where prepared filler is shown, Carey elastite, or its equal ¼-inch in thickness shall be used.

The copper diaphragm shall be used where shown and bent as indicated on the plans. The copper sheet forming the diaphragm shall be of 24 B&S gauge, hot rolled soft sheet copper 12-inches wide, and shall be embedded in the concrete as shown on the

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of 0 degrees Fahr. After the walls
shall also be filled with asphalt of t
phalt, or similar material shall ext

Payment for the copper diap
for copper sheets, which shall incl
forming the diaphragms as well as

Payment for the elastite or sin
for at the rate bid per linear foot
clude the furnishing of all materi
the satisfaction of the Engineer.

VALVE HOUSE:

A valve house of wood frame
The dimensions and construction
house in all its details. The house
paint complying with the specifica
by the Engineer, and the roof to
the Engineer. All downspouts an
tion to be of the quality shown or
as directed or as indicated on the
Payment for the valve house a
for the valve house complete.

FENCE:

After all other work is compl
weighing not less than ....lbs per
included in the reservoir site, said
design and equipped with suitable
shall be painted with two coats of
shall be of sound fir timber, 6-in.
to the satisfaction of the Engineer.
and not less than thirty inches in
thoroughly tamped. At the corner
shall be substantially braced. All
shall be tightly stretched.

Payment for the fence will be
in place which shall include all ma
the fence.

plans. All joints and intersections shall be water tight, securely made with a soldered joint or otherwise, subject to the approval of the Engineer.

After the floor is poured the joint above the copper diaphragms shall be filled with hot rock asphalt or similar material approved by the Engineer, of such quality that it will not run at a temperature of 140 degrees Fahr. or become brittle at a temperature of 0 degrees Fahr. After the walls are poured the joint between the wall and floor slab shall also be filled with asphalt of the same material. The filling of all joints with asphalt, or similar material shall extend to the full depth of the expansion joints.

Payment for the copper diaphragm in place shall be at the rate bid per pound for copper sheets, which shall include all soldering and labor required in placing and forming the diaphragms as well as the furnishing of all material.

Payment for the elastite or similar material and the hot asphalt filling will be paid for at the rate bid per linear foot of expansion joint complete in place, which shall include the furnishing of all material and all labor necessary to complete the joint to the satisfaction of the Engineer.

VALVE HOUSE:

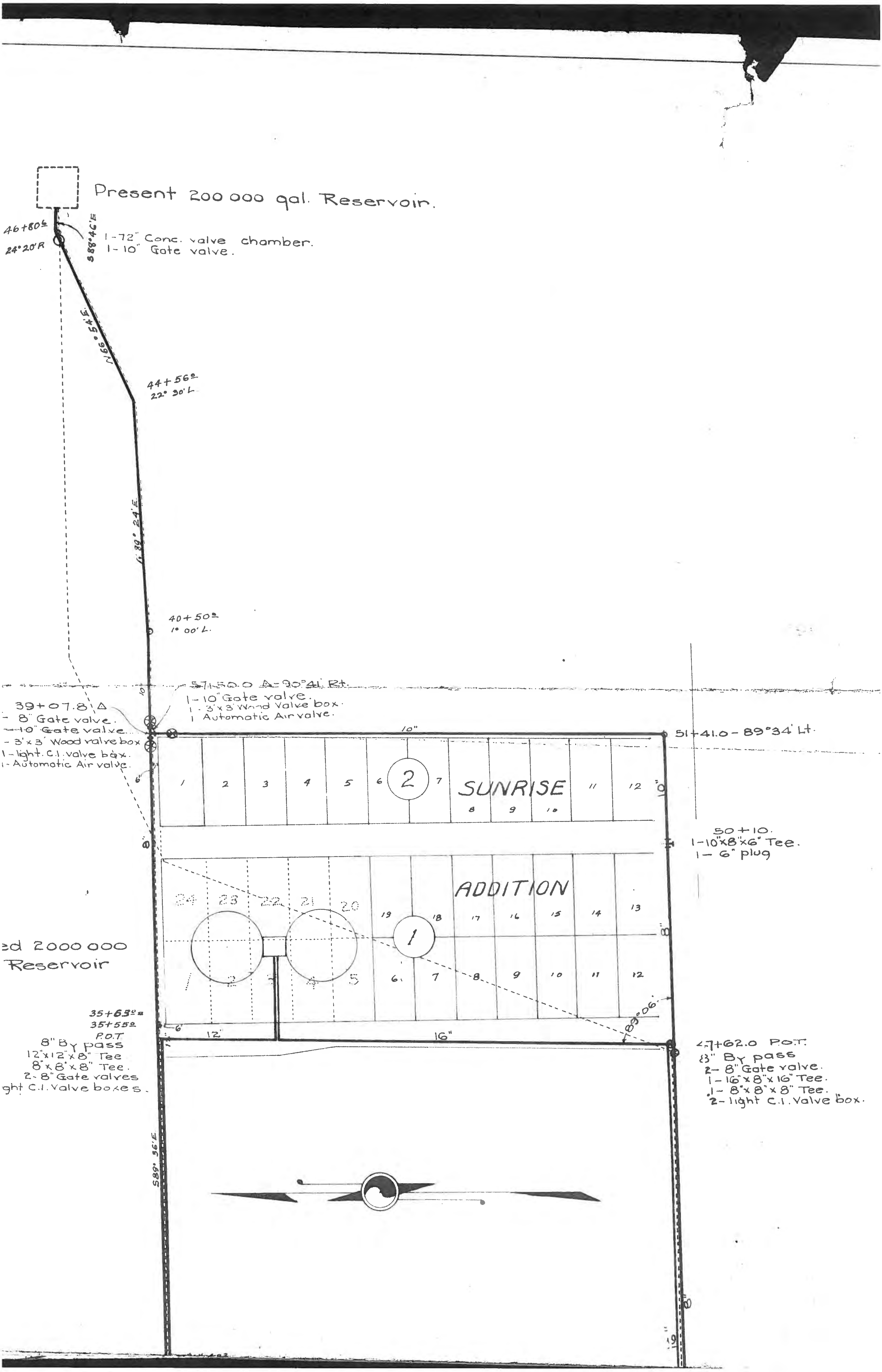
A valve house of wood frame construction shall be built as shown on the plans. The dimensions and construction details as shown will govern the construction of the house in all its details. The house shall be painted inside and out with two coats of paint complying with the specifications for paint for wood, the color to be determined by the Engineer, and the roof to be stained by the Contractor to the satisfaction of the Engineer. All downspouts and drains, millwork and hardware of every description to be of the quality shown or to the satisfaction of the Engineer and to be placed as directed or as indicated on the plans.

Payment for the valve house and all appurtenances will be made at the rate bid for the valve house complete.

FENCE:

After all other work is completed a six foot woven wire fence of galvanized wire, weighing not less than ....lbs per 100 square feet shall be placed around the ten lots included in the reservoir site, said fence to be provided with two gates of standard design and equipped with suitable brass locks. All exposed metal parts of the fence shall be painted with two coats of paint as specified in paint for iron. The fence posts shall be of sound fir timber, 6-in. by 6-in. in size and creosoted or otherwise treated to the satisfaction of the Engineer. Posts shall be set not more than eight feet apart and not less than thirty inches in the ground. All backfill around the posts shall be thoroughly tamped. At the corners and the gates and otherwise as necessary the posts shall be substantially braced. All wire shall be securely fastened to the posts and shall be tightly stretched.

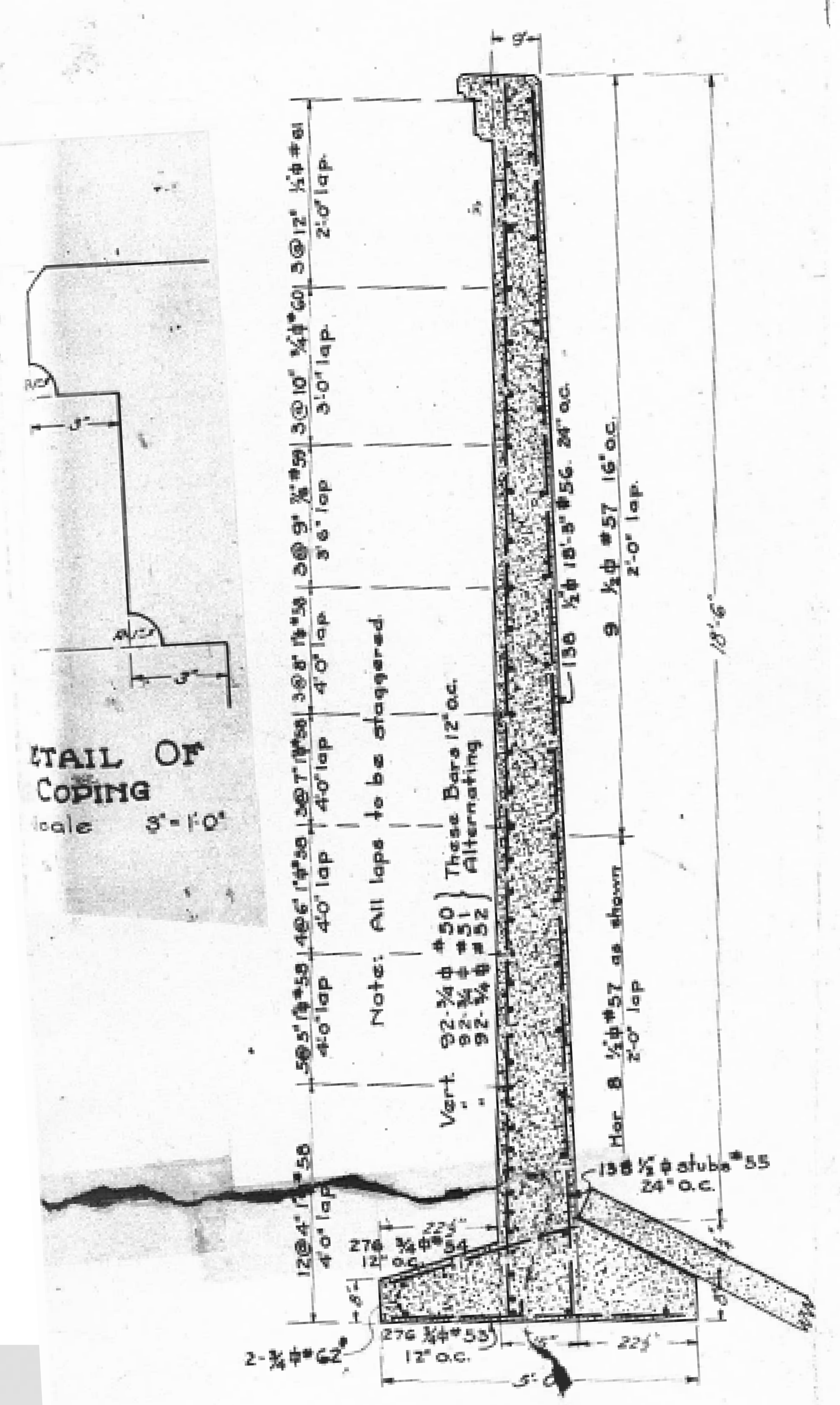
Payment for the fence will be made at the rate bid per lineal foot for the fence in place which shall include all material, and labor necessary for the completion of the fence.



Present 200 000 gal. Reservoir.
46+80 6
24°20'R
S88°46'E
1-72" Conc. valve chamber.
1-10" Gate valve.
N66°54'E
44+56 2
22° 30'L
N89°24'E
40+50 2
1° 00'L.
57+50.0 Δ=90°41' Rt.
1-10" Gate valve.
1- 3'x3' Wood Valve box.
1 Automatic Air valve.
39+07.8 Δ
8" Gate valve.
10" Gate valve.
3'x3' Wood valve box
1-light. C.I. valve box.
1-Automatic Air valve.
10"
51+41.0 - 89°34' Lt.
SUNRISE
ADDITION
50+10.
1-10"x8"x6" Tee.
1- 6" plug
ed 2000 000
Reservoir
35+63 5
35+55 2
P.O.T
8" By pass
12"x12"x8" Tee
8"x8"x8" Tee.
2-8" Gate valves
ght C.I. Valve boxes.
12"
16"
89°06'
47+62.0 P.O.T.
8" By pass
2- 8" Gate valve.
1- 16"x8"x16" Tee.
1- 8"x8"x8" Tee.
2- light C.I. Valve box.
S89°36'E


a
b
c
e
f
4" Sub drain
4'-0" Rad
23'-6" Rad
18'-0"
From Pumps
To Mains
Rad 42'-6"
This half same as other half
All Floor Slabs Reinforced like this
½" Ø Bars 20" lap
SECTIONAL ELE ION
CITY OF KELSO, WN.
CITY ENGINEER'S OFFICE
WATERWORKS IMPROVEMENT UNIT #6
GENERAL PLAN
Scale 3/16" = 1'-0"
July 1924
APPROVED:
APPROVED:
City Engineer
Mayor.

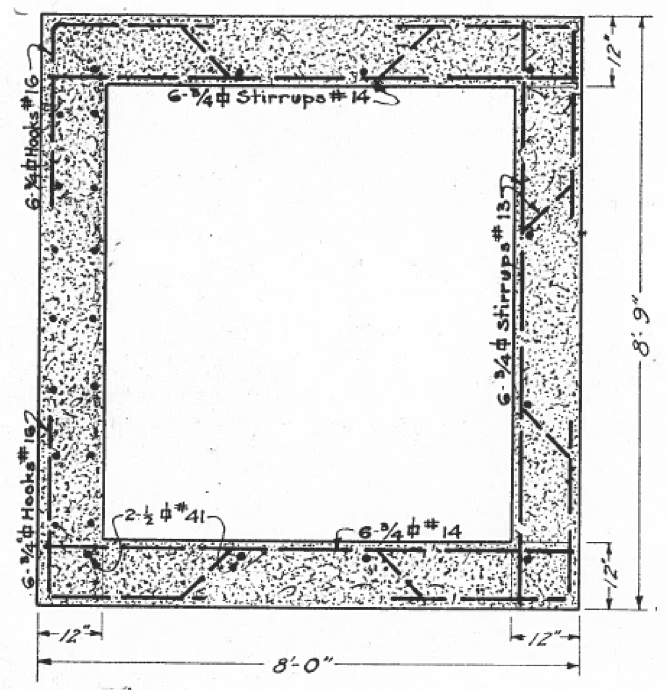
a



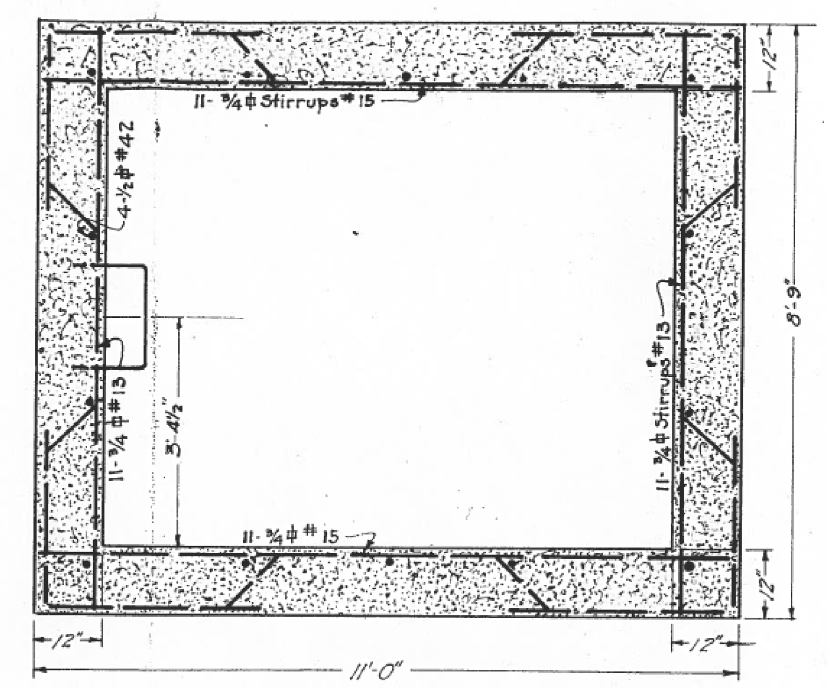
DETAIL OF COPING
Scale 3" = 1'-0"

DETAIL OF RESERVOIR WALL
Scale ½" = 1'-0"

b

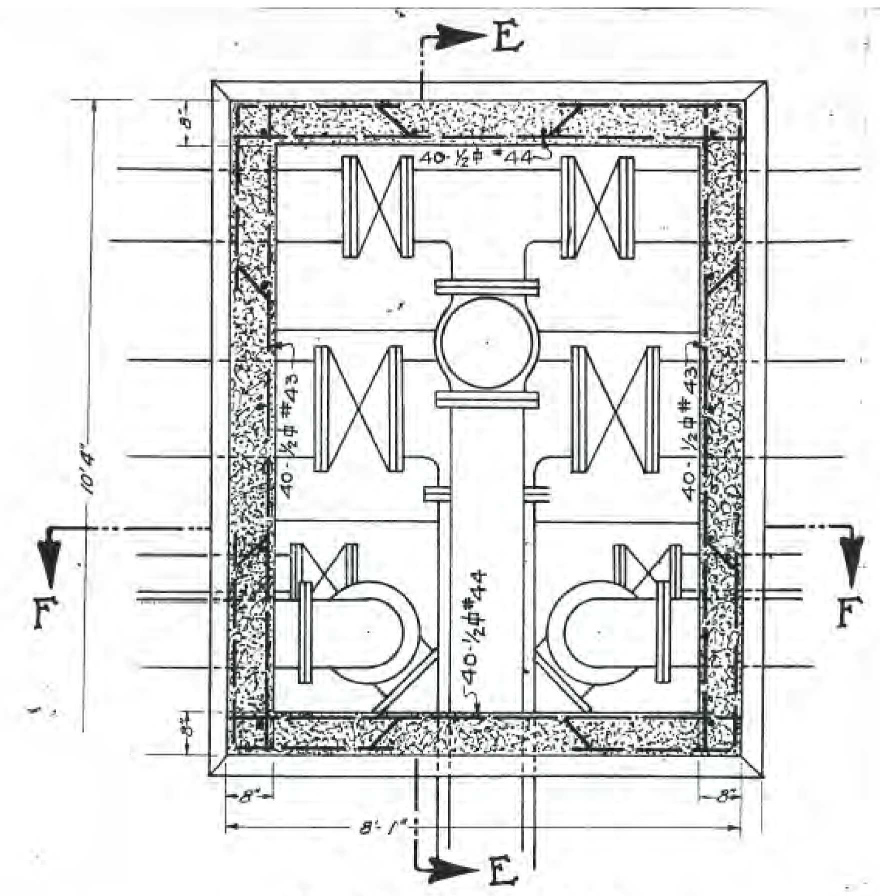


SEC. C-C

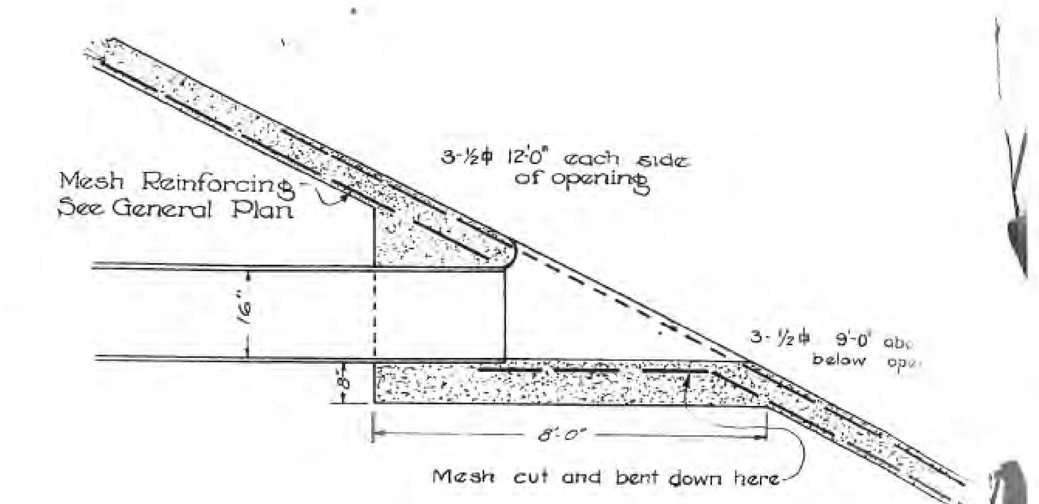


SEC. B-B

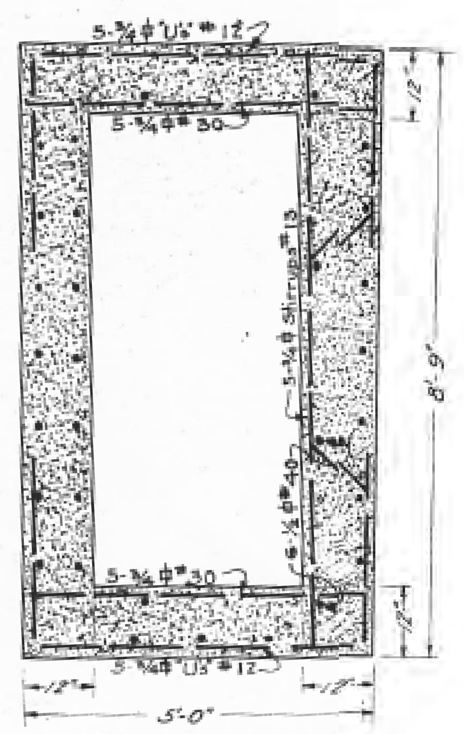
c



PLAN SEC. A-A

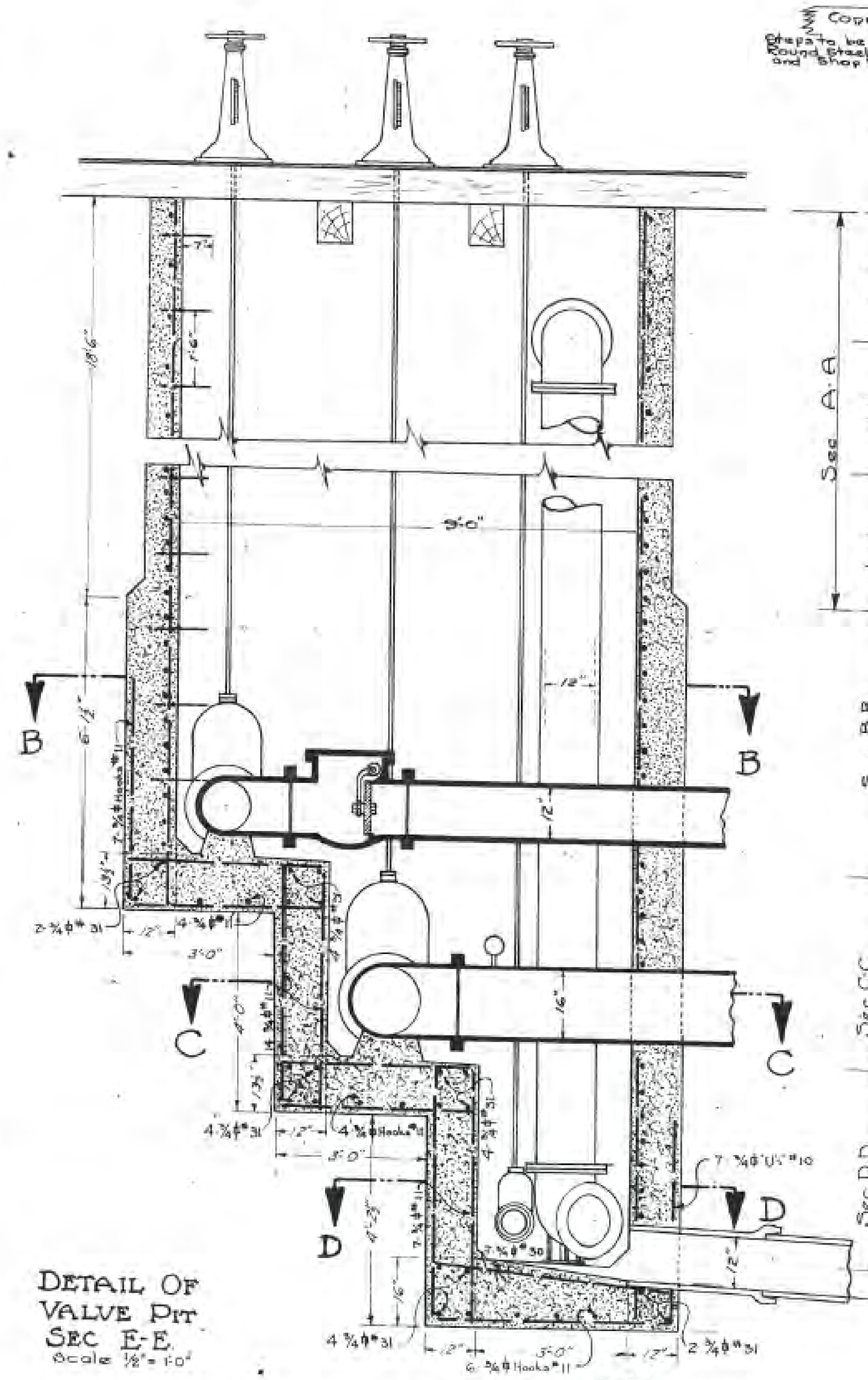


OUTLET DETAIL
Scale ½" = 1'-0"

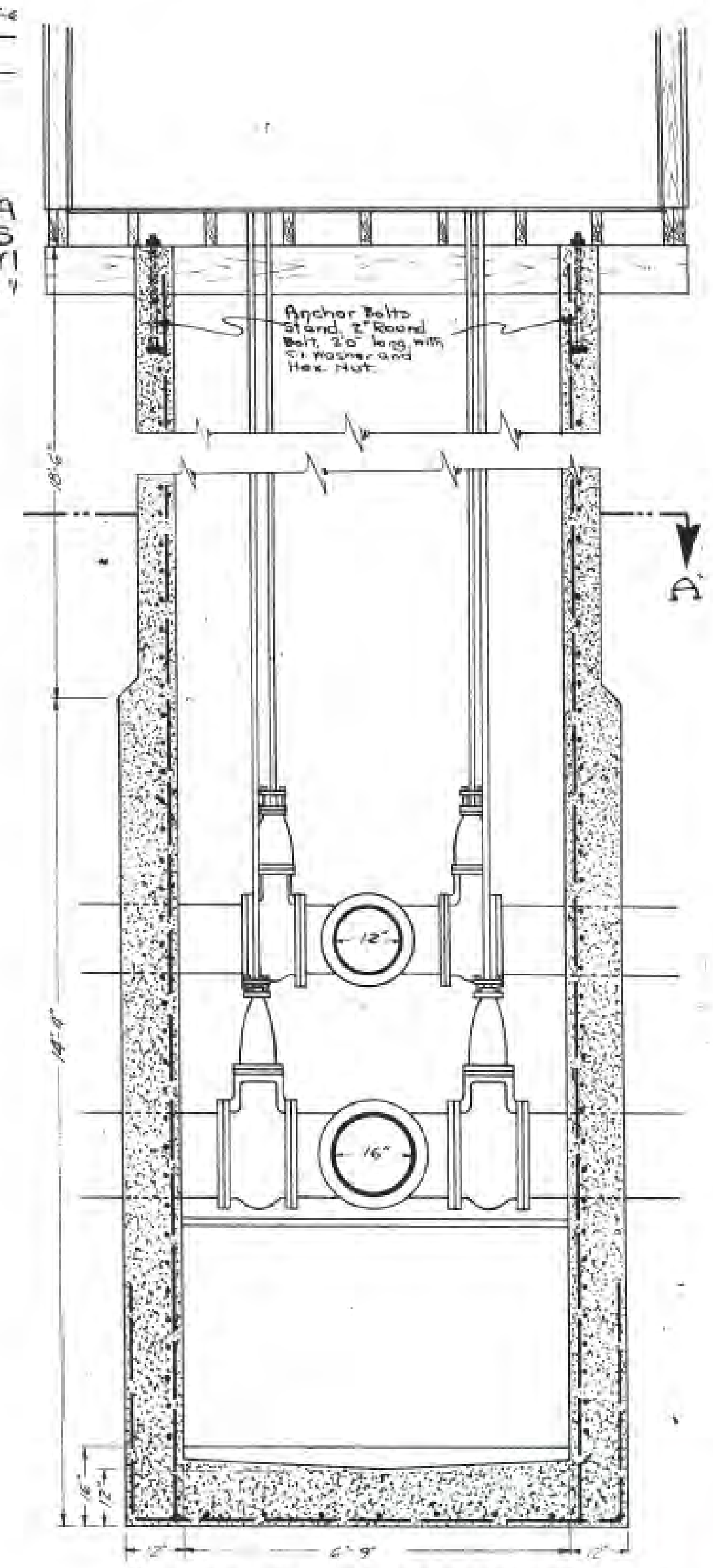


SEC. D-D

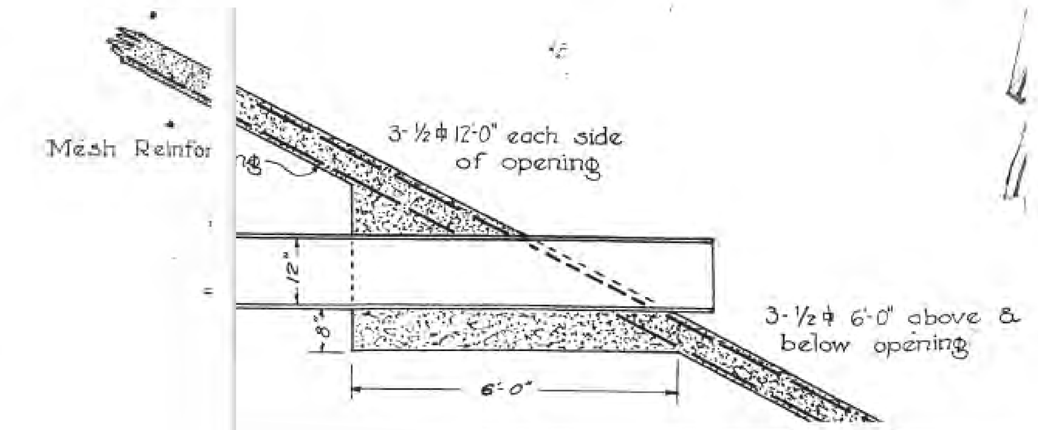
DETAIL OF STEPS INTO VALVE PIT
Scale ½" = 1'



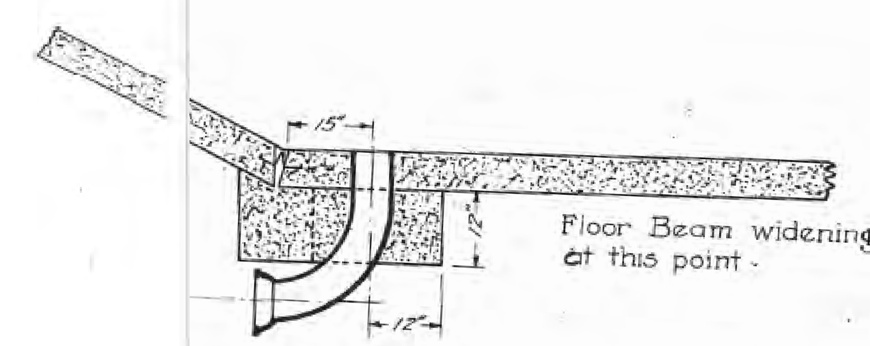
DETAIL OF VALVE PIT SEC E-E
Scale ½" = 1'-0"



ELEVATION OF VALVE PIT SEC F-F
Scale ½" = 1'-0"



Scale ½" = 1'-0"



DRAIN OUTLET DETAIL
Scale ½" = 1'-0"

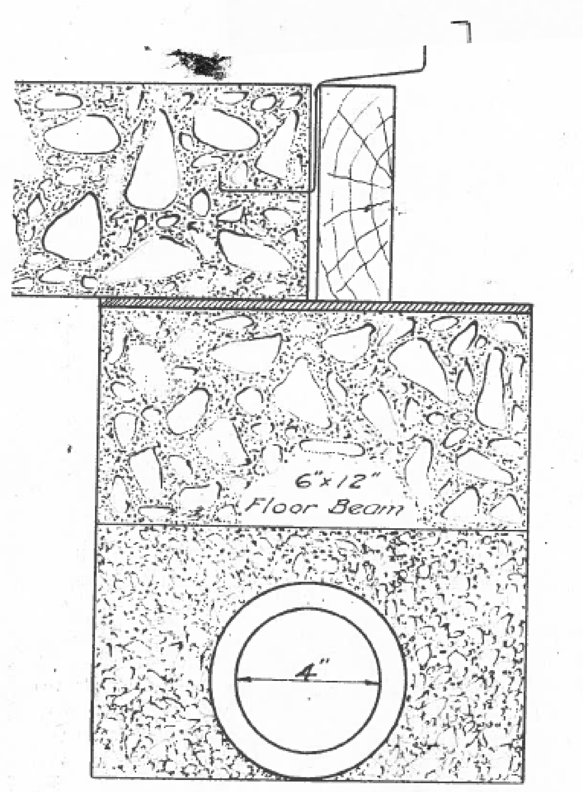


Fig. 1. Showing Expansion Joint While Pouring First Slab

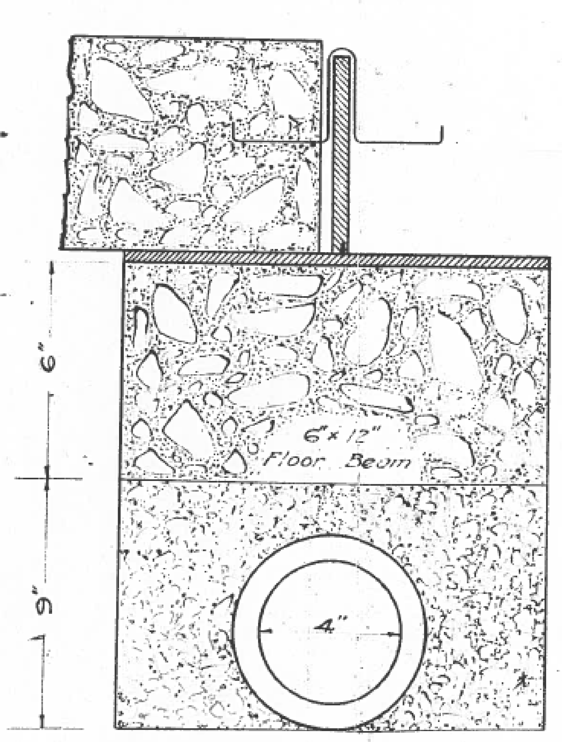


Fig. 2. Showing Expansion Joint Ready to Pour Second Slab

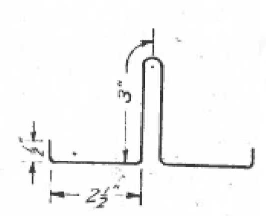


Fig. 3 Showing Bending of Copper Strip

EXPANSION JOINT DETAILS
Scale 3" = 1'-0"

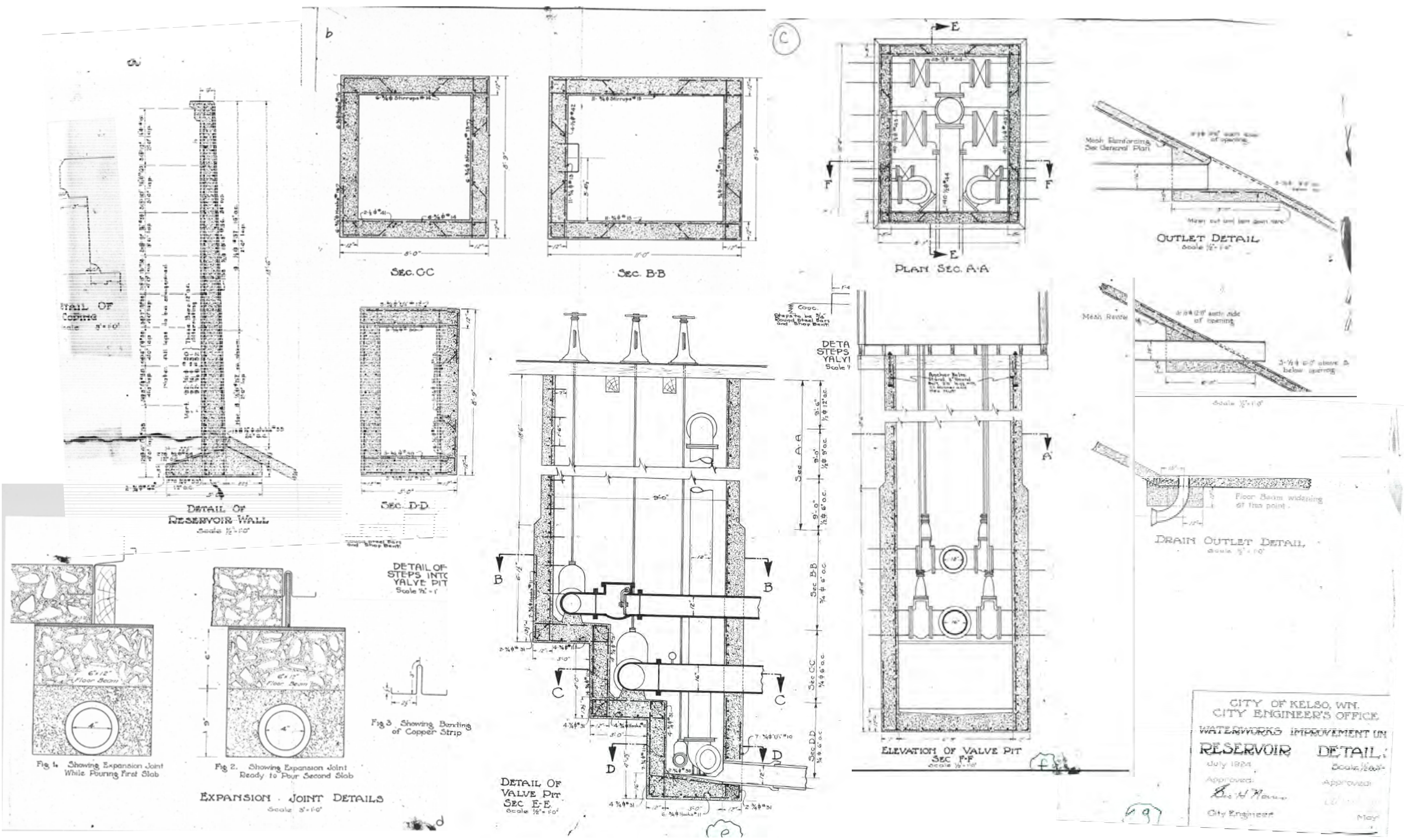
d

e

f

g

CITY OF KELSO, WN.
CITY ENGINEER'S OFFICE
WATERWORKS IMPROVEMENT UN
RESERVOIR DETAIL
July 1924
Scale ½" & 3"
Approved:
Geo. H. Harris
City Engineer
Approved:
May


Detail Of Coping
Detail Of Reservoir Wall
Scale ½" = 1'0"
Sec. C-C
Sec. B-B
Sec. D-D
Plan Sec. A-A
Outlet Detail
Mesh Reinforcing See General Plan
Drain Outlet Detail
Floor Beam widening at this point.
Detail Of Steps Into Valve Pit
Steps to be ¾" Round Steel Bars and Shop Bent
Detail Of Valve Pit Sec E-E
Elevation Of Valve Pit Sec F-F
Fig. 1. Showing Expansion Joint While Pouring First Slab
Fig. 2. Showing Expansion Joint Ready to Pour Second Slab
Fig. 3 Showing Bending of Copper Strip
Expansion Joint Details
Scale 3" = 1'0"
6"x12" Floor Beam
City Of Kelso, Wn.
City Engineer's Office
Waterworks Improvement
Reservoir Detail
July 1924
Approved
City Engineer

City of Kelso, Minor Rd. Reservoirs leak repair.
Notes: 9-7-11 PLR

Kelso scheduled for four days of dive cleaning time for leak repairs provided by **"state bid dive firm"**

Two man dive crew spent 4 days working on the north and south reservoirs.
9-21-09, 9-22-09, 9-23-09 & 9-25-09.

We contracted for 4-days of dive time.

Very little progress was made on stopping the leaking. Operator measurements put the total volume of leakage stopped at about 5 GPM. Plant operators can measure the flow leaking into the under drain system of the reservoirs. The surface leakage is measured visually.

Overall the leakage has not changed much from the last repair attempt in 2009 remaining steady at about 25 GPM leaking out of the under drain.

Recent main and valve replacements at Minor Road required the reservoirs to be shut down and this combined with the main being cut we know that we had no valve leakage. This allowed an overall "leak test" of the reservoirs and after calibration checks to the two level transducers, this put the overall current leakage at about 45 GPM.

The two leak test reports dated 9-22-09 & 9-23-09 are not good data. And contain many inaccurate statements. The statement about 7 leaks repaired on the south and 12 repaired on the north are correct and that is about it. About 15 feet of the seams / joints where leaks were found had epoxy repairs made. (There are several 100 yards of seams) Also the statement that epoxy was applied to any area that showed concrete failure is false.

A new ladder is in place in the North Reservoir. The new ladder was a result of the roof replacement and done as a change order. We have a new ladder to install in the South Reservoir in storage at the Operations Center.

Sediment was very minor in the both reservoirs at the time of the dive and cleaning was not necessary and this 1/16 inch of sediment was helpful in finding potential leaks on the floor and seems between the concrete slabs. The reservoir does not need cleaned and this can be verified by looking into the hatch with a light.

Summery: The dive firm was obtained on state bid at a good price. The magnitude of the project is that it is difficult to find the leaks as even a 1/8 inch hole can be leaking at a few gallons per minute. There are many places on the floors and walls that have small pits that have to all be checked and many cold joints that also have irregularities that have to be checked.

We had the dive team focus on the walls where we have seepage leaving the reservoirs and they had little success in finding the leaks.

The reservoir was kept on – line during the leak testing and the main valve was not closed and dye injected into the main 16 inch fill feed lines to see if flow was present. If we plan to schedule another dive project to look for leaks again we should plan on scheduling a full shutdown of the reservoirs. We should be able to get a full shutdown with the new valves that were installed in 2011. Knowing if this line leaks would be helpful in determining future projects to repair the reservoirs.

My guess is that we are getting the leakage from many small seeps and these are difficult for a diver to locate.

City of Kelso

REPAIR & LEAK TEST REPORT

SOUTH RESERVOIR

REPAIR & LEAK TEST REPORT
South Reservoir-Minor Road
City of Kelso
9-22-09

The S Reservoir is a semi-buried concrete water tank with a 1 million gallon liquid capacity.

Leak testing was performed on 9-22-09 using AWWA Procedures and Standards.

LEAK TEST RESULTS: 7 small leaks were detected, 3 on shell wall, 4 on floor

Repairs were made using Aquatapoxy A-6 part A & B

REPAIRS: All leaks were repaired with Aquatapoxy, substantial difference in GPM loss after application. Epoxy was applied to any area that showed concrete failure as well as all seems and floor joints.

We inspected the following internal items as a courtesy; please consider our recommendations.

1. Internal Ladder is in very poor condition; heavy corrosion. 100%
   Replace

2. Internal Plumbing 1 is an inlet/outlet in poor condition; blistering. 80%
   Replace or Repair

3. Internal Plumbing 2 is of unknown origin in very poor condition; heavy blistering. 100%
   Replace or Repair

4. Internal Overflow is in very poor condition; heavy corrosion. 100%
   Replace or Repair

5. Drain is in very poor condition; heavy blistering. 100%
   Replace

6. 1" of layered sand and silt present.
   Clean as soon as possible

SUMMARY

Advanced Diving recommends cleaning your tank as soon as possible then leak test again. The sediment does hinder the divers' ability to perform the leak test at 100%. The shell walls need to have a full and detailed inspection with additional leak testing performed on them.

City of Kelso

REPAIR & LEAK TEST REPORT

NORTH RESERVOIR

REPAIR & LEAK TEST REPORT
North Reservoir-Minor Road
City of Kelso
9-23-09

The N Reservoir is a semi-buried concrete water tank with a 1 million gallon liquid capacity.

Leak testing was performed on 9-23-09 using AWWA Procedures and Standards.

LEAK TEST RESULTS: 12 small leaks were detected, 3 on shell wall, 9 on floor

Repairs were made using Aquatapoxy A-6 part A & B

REPAIRS: All leaks were repaired with Aquatapoxy, GPM loss after application not known. Epoxy was applied to any area that showed concrete failure as well as all seems and floor joints.

We inspected the following internal items as a courtesy; please consider our recommendations.

1. Internal Plumbing 1 is an inlet/outlet in fair condition; blistering. 65%
   Monitor for further blistering

2. Drain is in fair condition; blistering. 70%
   Monitor for further blistering

3. 1/2" of layered sand and silt present.
   Clean as soon as possible

SUMMARY
Advanced Diving recommends cleaning your tank as soon as possible then leak test again. The sediment does hinder the divers' ability to perform the leak test at 100%.

NORTHWEST UNDERWATER CONSTRUCTION, LLC.

800 NE. TENNEY RD., SUITE 110-111, VANCOUVER WA. 98685
TOLL FREE 866-270-1114 **FAX** 360-993-5581 **EMAIL** adam@nwuwconst.com

March 13, 2007

Adam Krausman
800 NE. Tenney Road # 110-111
Vancouver, WA. 98685
360-381-0368 Mobile
360-993-5581 Fax
adam@nwuwconst.com
Visit us on the web at www.nwuwconst.com

City of Kelso
Paul Reebs
Public Works Department
203 S. Pacific Ave., Suite 205
Kelso, WA 98626

Minor Road Reservoirs

Northwest Underwater conducted underwater surveys to detect leaks in North and South reservoir. All measurements on chart are of repair area. Northwest Underwater used potable water underwater epoxy for all repairs. Project manager was Adam Krausman.

EXPLANATION OF CHART:
(W = WALL, B=BASE PANELS)
EXAMPLE
F-3-1
F IS THE I.D. FOR "FLOOR"
3 IS THE NUMBER OF THE PANEL
1 IS THE NUMBER LEAKS IN THE AFORE MENTIONED PANEL
Also note: Floor numbers and Wall numbers location coincide per the plans.
(EX: F-1 is below W-1)

NORTHWEST UNDERWATER
CONSTRUCTION, LLC.

800 NE. TENNEY RD., SUITE 110-111, VANCOUVER WA. 98685
TOLL FREE 866-270-1114 **FAX** 360-993-5581 **EMAIL** adam@nwuwconst.com

NORTH TANK

	I.D	*DESCRIPTION*		*STATUS*
1	F-16-1	12"	Slight flow, Potable water epoxy paint	REPAIRED
2	F-4-1	14"	Slight flow, Potable water epoxy paint	REPAIRED
3	F-5-1	12"	Slight flow, Potable water epoxy paint	REPAIRED
4	F-7-1	10"	Slight flow, Potable water epoxy paint	REPAIRED
5	F-7-2	13"	Slight flow, Potable water epoxy paint	REPAIRED
6	F-7-3	10" x 5"	Slight flow, Potable water epoxy paint	REPAIRED
7	F-8-1	15"	Heavier flow, Oakum, Painted epoxy	REPAIRED
8	F-8-2	20"	Slight flow, Potable water epoxy paint	REPAIRED
9	F-8-3	18"	Slight flow, Potable water epoxy paint	REPAIRED
10	F-8-4	14"	Slight flow, Potable water epoxy paint	REPAIRED
11	F-8-5	43"	Slight flow, Potable water epoxy paint	REPAIRED
12	F-9-1	17"	Slight flow, Potable water epoxy paint	REPAIRED
13	F-10-1	20"	Heavier flow, Oakum, Painted epoxy	REPAIRED
14	F-10-2	11.5"	Slight flow, Potable water epoxy paint	REPAIRED
15	F-10-3	10"	Slight flow, Potable water epoxy paint	REPAIRED
16	F-10-4	9"	Slight flow, Potable water epoxy paint	REPAIRED
17	F-12-1	13"	Slight flow, Potable water epoxy paint	REPAIRED
18	F-13-1	12"	Slight flow, Potable water epoxy paint	REPAIRED
19	F-13-2	11"	Slight flow, Potable water epoxy paint	REPAIRED
20	W-8-1	5" x 7"	Medium flow, Oakum, Painted epoxy	REPAIRED
21	W-14-1	3" x 3"	Slight flow, Potable water epoxy paint	REPAIRED

NORTHWEST UNDERWATER
CONSTRUCTION, LLC.

800 NE. TENNEY RD., SUITE 110-111, VANCOUVER WA. 98685
TOLL FREE 866-270-1114 **FAX** 360-993-5581 **EMAIL** adam@nwuwconst.com

SOUTH TANK

1	F-1-1	11"	Slight flow, Potable water epoxy paint	REPAIRED
2	F-1-2	21"	Slight flow, Potable water epoxy paint	REPAIRED
3	F-1-3	9"x 4"	Heavier flow, Oakum, Painted epoxy	REPAIRED
4	F-5-1	16"	Slight flow, Potable water epoxy paint	REPAIRED
5	F-5-2	10"	Medium Leak, Oakum, Painted epoxy	REPAIRED
6	F-6-1	9"	Slight flow, Oakum, Painted epoxy	REPAIRED
7	F-7-1	18"	Slight flow, Potable water epoxy paint	REPAIRED
8	F-9-1	7"	Slight flow, Potable water epoxy paint	REPAIRED
9	F-9-2	7"	Slight flow, Potable water epoxy paint	REPAIRED
10	F-9-3	10" / 6"	Slight flow, Potable water epoxy paint	REPAIRED
11	F-11-1	20"	Medium leak, Oakum, Painted epoxy	REPAIRED
12	F-13-1	10"	Medium leak, Oakum, Painted epoxy	REPAIRED
13	W-1-1	3" x 2"	Heavy flow, Oakum, Painted epoxy	REPAIRED
14	W-2-1	8" x 5"	Slight flow, Potable water epoxy paint	REPAIRED
15	W-4-1	6.5" x 2"	Slight flow, Potable water epoxy paint	REPAIRED
16	W-5-1	2" x 1" / 7" x 6" –	Slight flow, Oakum, Painted epoxy	REPAIRED
17	W-12-1	8" x 3"	Medium leak, Oakum, Painted epoxy	REPAIRED
18	W-13-1	5" x 5"	Slight flow, Oakum, Painted epoxy	REPAIRED

If you have any questions, please contact us. It was a pleasure working with you as always. If we can help you in the future, please do not hesitate to contact us.
Toll Free: 866-270-1114
Cell: 360-381-0368

Adam Krausman

SECTION VI: HEALTH HAZARD DATA

Primary Routes of Entry:

EYES: Severe eye irritant. May cause burns. Seek medical attention.
SKIN: Severe skin irritant. May cause injury to skin following prolonged or repeated contact. Repeated exposure may cause sensitization of the individual.
INHALATION: Excessive inhalation is likely to cause irritation of mucous membranes.
INGESTION: May cause headache, nausea, vomiting, bleeding of the gastrointestinal tract and vomiting of blood. Oral toxicity not available on compound. Seek immediate medical attention.
SYSTEMIC and OTHER EFFECTS: Product can be alkaline, corrosive and irritating to skin, ears, eyes and mucous membranes. May cause injury upon prolonged contact and repeated contact.

Emergency and First Aid Procedures:

EYES: Flush with large quantities of water for at least 15 minutes. Consult a physician.
SKIN: Wash immediately with soap and water. If irritation or sensitization occurs, remove individual from further contact with material.
INHALATION: Remove to fresh air if affects occur. Consult a physician.
INGESTION: If this product is swallowed, give large quantities of water. Do not induce vomiting. Seek medical advice.

SECTION VII: PRECAUTIONS FOR SAFE HANDLING AND USE

Steps to Be Taken in Case Material is Released or Spilled: Keep sources of ignition and hot metal surfaces isolated from the spill. Material may flow slowly. Scrape into containers for disposal.
Waste Disposal Methods: Dispose of according to all local, state and federal regulations.
Precautions to Be Taken in Handling and Storing: Keep containers closed when not in use. Avoid prolonged or repeated contact with skin. DO NOT handle or store near flame, heat or strong oxidants. Do not store in direct sunlight. Avoid prolonged storage above 38 deg C (100 deg F).

SECTION VIII: CONTROL MEASURES

RESPIRATORY: To avoid overexposure, wear a properly fitted NIOSH/MSHA organic vapor respirator.
VENTILATION: General mechanical ventilation is sufficient for most conditions. Local exhaust ventilation may be necessary for some operations.
EYES: Use chemical safety glasses, splash-proof eye goggles or goggles with full faceshield.
CLOTHING/GLOVES: Use nitrile or other chemically resistant gloves. Wear clean, long-sleeved, body covering clothing and rubber boots.

SECTION IX: TRANSPORT DATA

Proper Shipping Name: Amines, liquid, corrosive, n.o.s. (modified cycloaliphatic amine)
Hazard Class: Corrosive Material - 8
Identification Number: UN 2735
Packing Group: III

SECTION X: DISCLAIMER

RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES WITH RESPECT TO ANY INFORMATION PRESENTED HEREIN, ALL OF WHICH IS PROVIDED "AS IS". TO THE MAXIMUM EXTENT PERMITTED BY LAW, RAVEN EXPRESSLY EXCLUDES ALL WARRANTIES, OBLIGATIONS, REPRESENTATIONS, LIABILITIES, TERMS AND CONDITIONS (WHETHER THEY ARE EXPRESS OR IMPLIED, OR ARISE IN CONTRACT, STATUTE, OR OTHERWISE, AND IRRESPECTIVE OF THE NEGLIGENCE OF RAVEN, ITS EMPLOYEES OR AGENTS) IN CONNECTION WITH THE INFORMATION PRESENTED HEREIN. RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES AS TO MERCHANTABILITY, FITNESS FOR PURPOSE, NONINFRINGEMENT OR CONFORMITY WITH DESCRIPTION OR SAMPLE.

MATERIAL SAFETY DATA SHEET

Trade Name: **AquataPoxy A-7 - Part B**

SECTION I

Manufacturer: Raven Lining Systems
13105 East 61st Street, Suite A
Broken Arrow, OK 74012

Emergency Telephone #: 800-424-9300
Chemtrec

Revision Date: 3/15/06
Reason for Revision: New Address

Information Telephone #: 918-615-0020
800-324-2810

SECTION II: INGREDIENT INFORMATION

INGREDIENT	CAS NUMBER	PEL	TLV
Microcrystalline Silica (quartz)	14808-60-7	0.1 mg/m³ (respirable)	0.1 mg/m³ (respirable fraction)
Aliphatic Amine	N/A		

SARA Title III, Section 313 ingredients: NONE

The specific chemical identity is being withheld as a trade secret.

SECTION III: PHYSICAL DATA

Boiling Point: > 200 deg F
Vapor Pressure: N/A
Vapor Density: N/A
Solubility in Water: negligible
Appearance and Odor: Mastic consistency - bland odor

Specific Gravity: 1.20 - 1.25
Melting Point: N/A
Evaporation Rate: Not established
% Volatile by Volume: <1%

SECTION IV: FIRE & EXPLOSION HAZARD DATA

Flash Point: >200 deg F, Pensky-Martin Closed Cup **Fire Hazard Classification (OSHA/NFPA):** Class III B.
Extinguishing Media: Foam, CO_2, Dry Chemical, Water Spray
Special Fire Fighting Procedures: The use of self-contained breathing apparatus is recommended for firefighters. Water may be helpful in keeping adjacent containers cool.
Unusual Fire and Explosion Hazards: May generate toxic or irritating combustion products. Sudden reaction and fire may result if product is mixed with an oxidizing agent.

SECTION V: REACTIVITY DATA

Stability: Stable
Incompatibility: Strong acids and bases, selected epoxy resins and strong oxidizing agents.
Hazardous Decomposition or Byproducts: Thermal decomposition in the presence of air may yield carbon monoxide, carbon dioxide and nitrogen oxides.
Hazardous Polymerization: Will not occur.

SECTION VI: HEALTH HAZARD DATA

Primary Routes of Entry:

EYES: May cause slight transient (temporary) eye irritation. Corneal injury is unlikely.
SKIN: May cause allergic skin reaction in susceptible individuals. Prolonged exposure not likely to cause significant skin irritation. Repeated exposure may cause skin irritation. A single prolonged exposure is not likely to result in the material being absorbed through skin in harmful amounts.
INHALATION: Vapors are unlikely due to physical properties.
INGESTION: Single dose oral toxicity is low. The oral LD50 for rats is >4000 mg/kg. No hazards anticipated from ingestion incidental to industrial exposure.
SYSTEMIC and OTHER EFFECTS: Except for skin sensitization, repeated exposures to low molecular weight epoxy resins of this type are not anticipated to cause any significant adverse effects.

Carcinogenicity: A poorly characterized sample of low molecular weight epoxy resin of this type (diglycidyl ether of bisphenol A) has been reported to produce skin cancer in a highly sensitive strain of mice. However, high levels of impurities (including a known animal skin carcinogen) compromise the validity of the findings. Epoxy resin that is representative of current manufacturing processes is not believed to be a cancer hazard to humans. Results of mutagenicity tests in animals have been negative. Has been shown to be negative in some invitro ("test tube") mutagenicity tests and positive in others.

Emergency and First Aid Procedures:

EYES: Flush with large quantities of water for at least 15 minutes. Consult a physician.
SKIN: Wash off in flowing water or shower.
INHALATION: Remove to fresh air if affects occur. Consult a physician.
INGESTION: No adverse effects anticipated by this route of exposure incidental to proper industrial handling.

SECTION VII: PRECAUTIONS FOR SAFE HANDLING AND USE

Steps to Be Taken in Case Material is Released or Spilled: Keep sources of ignition and hot metal surfaces isolated from the spill. Material may flow slowly. Scrape into containers for disposal.
Waste Disposal Methods: Dispose of according to all local, state and federal regulations.
Precautions to Be Taken in Handling and Storing: Keep containers closed when not in use. Avoid prolonged or repeated contact with skin. DO NOT handle or store near flame, heat or strong oxidants. Do not store in direct sunlight. Avoid prolonged storage above 38 deg C (100 deg F).

SECTION VIII: CONTROL MEASURES

RESPIRATORY: Respiratory protection should not be needed. If respiratory irritation is experienced, use an approved air-purifying respirator.
VENTILATION: General mechanical ventilation is sufficient for most conditions. Local exhaust ventilation may be necessary for some operations.
EYES: Use safety glasses, splash-proof eye goggles or goggles with full faceshield.
CLOTHING/GLOVES: Use impermeable gloves to prevent skin irritation in sensitive individuals. Wear clean, long-sleeved, body covering clothing.

SECTION IX: TRANSPORT DATA

Proper Shipping Name: not regulated
Hazard Class: N/A
Identification Number: not regulated
Packing Group: N/A

SECTION X: DISCLAIMER

RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES WITH RESPECT TO ANY INFORMATION PRESENTED HEREIN, ALL OF WHICH IS PROVIDED "AS IS". TO THE MAXIMUM EXTENT PERMITTED BY LAW, RAVEN EXPRESSLY EXCLUDES ALL WARRANTIES, OBLIGATIONS, REPRESENTATIONS, LIABILITIES, TERMS AND CONDITIONS (WHETHER THEY ARE EXPRESS OR IMPLIED, OR ARISE IN CONTRACT, STATUTE, OR OTHERWISE, AND IRRESPECTIVE OF THE NEGLIGENCE OF RAVEN, ITS EMPLOYEES OR AGENTS) IN CONNECTION WITH THE INFORMATION PRESENTED HEREIN. RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES AS TO MERCHANTABILITY, FITNESS FOR PURPOSE, NONINFRINGEMENT OR CONFORMITY WITH DESCRIPTION OR SAMPLE.

MATERIAL SAFETY DATA SHEET

Trade Name: **AquataPoxy A-7 - Part A**

SECTION I

Manufacturer: Raven Lining Systems
13105 East 61st Street, Suite A
Broken Arrow, OK 74012

Emergency Telephone #: 800-424-9300
Chemtrec

Revision Date: 3/15/06
Reason for Revision: New Address

Information Telephone #: 918-615-0020
800-324-2810

SECTION II: INGREDIENT INFORMATION

INGREDIENT	CAS NUMBER	PEL	TLV
Epoxy Resin	25085-99-8		
Silica*	7631-86-9	20g/m³	10 mg/m³
*Hazard as dust only			

SARA Title III, Section 313 ingredients: None

The specific chemical identity is being withheld as a trade secret.

SECTION III: PHYSICAL DATA

Boiling Point: > 200 deg F
Vapor Pressure: N/A
Vapor Density: N/A
Solubility in Water: negligible
Appearance and Odor: Mastic consistency - faint epoxy odor

Specific Gravity: 1.24
Melting Point: N/A
Evaporation Rate: Not established
% Volatile by Volume: <1%

SECTION IV: FIRE & EXPLOSION HAZARD DATA

Flash Point: >200 deg F, TCC Method
DOT Flammability Classification: non-flammable
Extinguishing Media: Foam, CO_2, Dry Chemical
Special Fire Fighting Procedures: The use of self-contained breathing apparatus is recommended for firefighers. Water may be helpful in keeping adjacent containers cool. Avoid spreading burning liquid with water used for cooling purposes.
Unusual Fire and Explosion Hazards: Keep work areas free of hot metal surfaces and other sources of ignition.

SECTION V: REACTIVITY DATA

Stability: Stable
Incompatibility: Strong acids and bases, selected amines, oxidizing agents.
Hazardous Decomposition or Byproducts: Thermal decomposition in the presence of air may yield carbon monoxide and carbon dioxide.
Hazardous Polymerization: Will not occur.

COHESANT MATERIALS

13105 East 61st Street
Suite A
Broken Arrow, OK 74012
Telephone: 877-615-264Fax: 918-615-0140

Packing slip

Packing slip .....: 10043

Date ....: 12/20/2006

Ship to: NORTHWEST UNDER WATER CON:
800 NORTHEAST TENNY RD
SUITE 110-111
Vancouver, WA 98685

Sold to: NORTHWEST UNDER WATER CON:
800 NORTHEAST TENNY RD
SUITE 110-111
Vancouver, WA 98685

Your ref. .:
Ship via ..: UPS 3 Day
Terms ....: Freight Prepaid & Added

Order account ......: 10095
Requisition ....: CITY OF KELSO
Order date ....: 12/20/2006
Sales order ....: 100077
IRS payer record ....: MWierzchow

Item number	Description	Ordered	Delivered	Remaining quantity
A7A-1G	A7, PART A 1 GAL	2.00	2.00	
Quantity : 2.00 Serial number : 1004601				
A7B-1G	A7, PART B 1 GAL	2.00	2.00	
Quantity : 2.00 Serial number : 1002601				
A6B-1G	A6, PART B 1 GAL	1.00	1.00	
Quantity : 1.00 Serial number : 1120601				
A6A-1G	A6, PART A 1 GAL	1.00	1.00	
Quantity : 1.00 Serial number : 1106602				

ORDER HAS BEEN PAID WITH A CREDIT CARD.

Receipt: ____________________

City of Kelso
North Reservoir Leak Detection and Repair

Project Report

City of Kelso
PO Box 819
Kelso, WA 98626

Attn:
Mr. Paul Reebs
Water Treatment Plant Supervisor

May 4, 2005

Prepared By:

Northwest Underwater Construction, LLC

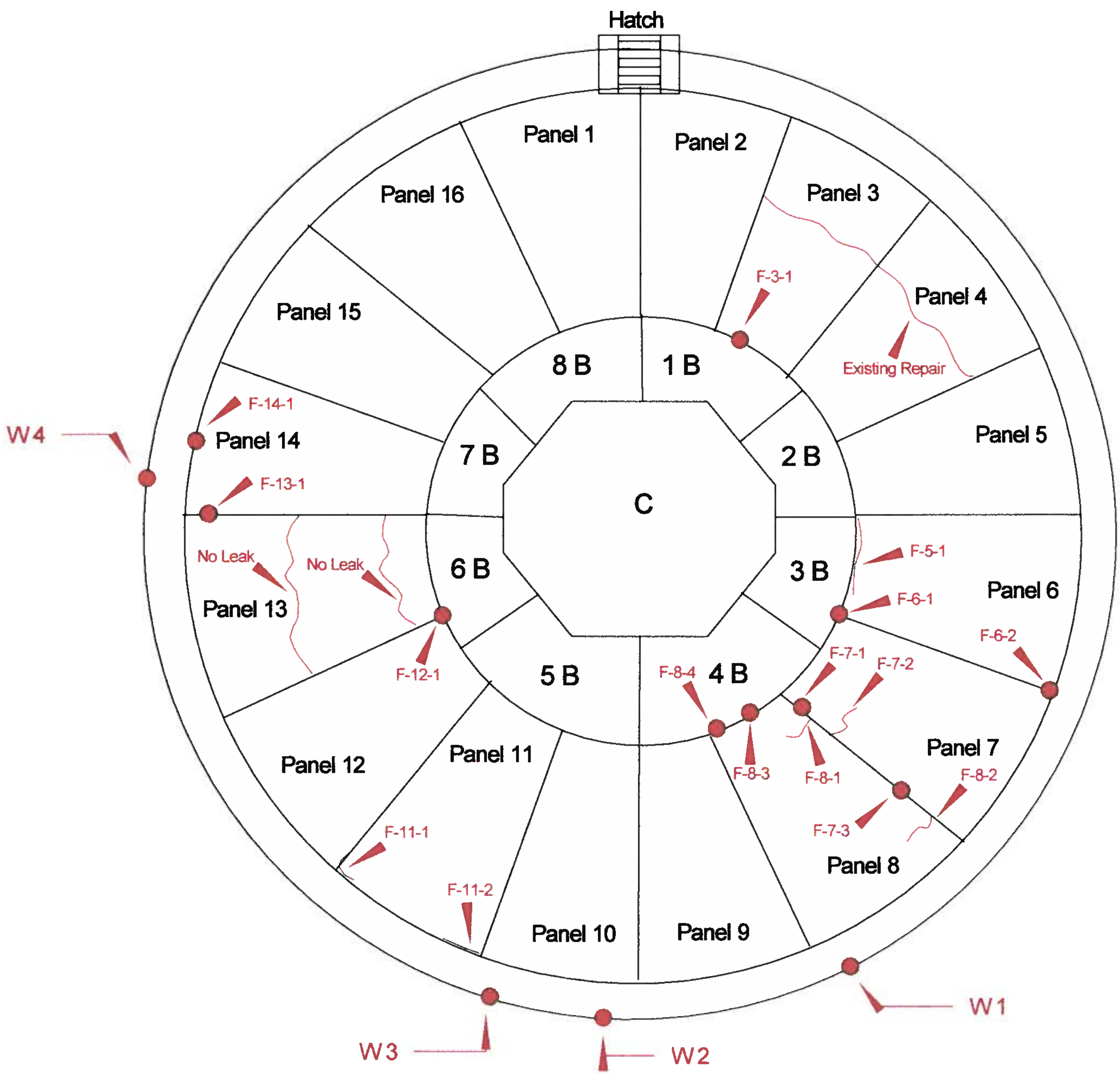
Oregon CCB# 149331
Washington State Contractors # NORTHUC994PJ

Jesse Hutton
(360) 518-3641

Northwest Underwater Construction LLC

800 Ne Tenney Rd. Ste. 110 - 111 Vancouver, WA 98685
Voice (360) 518-3641 Fax (360) 993-5581 Email info@nwuwconst.com

Date: May 04, 2005
Customer: City of Kelso
Location: City of Kelso - North Potable Water Reservoir
Illustration: Map of repairs made by NUC divers. Please refer to the attached report for detailed information about each point.



Not to Scale - For General Reference Only

NORTHWEST UNDERWATER CONSTRUCTION, LLC.

800 NE. TENNEY RD., SUITE 110-111, VANCOUVER WA. 98685
TOLL FREE 866-270-1114 **FAX** 360-993-5581 **EMAIL** info@nwusconst.com

May 13, 2005
City of Kelso – North Potable Water Reservoir Leak Repair

Project Report/Data

NUC mobilized personnel, equipment and materials to perform underwater leak detection and repair operations. All diving operations were conducted according to AWWA standards for potable water diving. All sealant materials have been approved for potable water use (please see attached MSDS information).

North Clear Well Potable Water Tank

EXPLANATION OF CHART:
EXAMPLE:
F-3-1
F *IS THE I.D. FOR "FLOOR" (W = WALL)*
3 *IS THE NUMBER OF THE PANEL*
1 *IS THE NUMBER LEAKS IN THE AFORE MENTIONED PANEL*

Item	*I.D*	*DESCRIPTION*	*STATUS*
1	F-3-1	5" L ON BOTTOM EXPANSION JOINT OF PANEL	REPAIRED
2	F-5-1	98" L X 1/8" AT WIDEST POINT ON BOTTOM EXPANSION JOINT BETWEEN PANELS 5 & 6	REPAIRED
3	F-6-1	2" X 2" BOTTOM CORNER OF PANELS 6 & 7	REPAIRED
4	F-6-2	35" L AT TOP EXPANSION JOINT OF THE CORNER OF PANEL 6 & 7	REPAIRED
5	F-7-1	7" L ON THE CENTER JOINT BETWEEN PANELS 7 & 8	REPAIRED
6	F-7-2	27" X 16" LEAK EXTENDS INTO FRACTURE EXTENDING INTO PANEL 7 AND ALONG EXPANSION JOINT	REPAIRED
7	F-8-1	11" L EXTENDS ALONG FRACTURE INTO PANEL 8	REPAIRED
8	F-7-3	13" L ON EXPANSION JOINT BETWEEN PANELS 7 & 8	REPAIRED
9	F-8-2	10" L ON FRACTURE EXTENDING INTO PANEL 8	REPAIRED
10	F-8-3	10" L ON BOTTOM EXPANSION JOINT OF PANEL 8	REPAIRED
11	F-8-4	12" L ON BOTTOM EXPANSION JOINT OF PANEL 8	REPAIRED

NORTHWEST UNDERWATER CONSTRUCTION, LLC.

800 NE. TENNEY RD., SUITE 110-111, VANCOUVER WA. 98685
TOLL FREE 866-270-1114 **FAX** 360-993-5581 **EMAIL** info@nwusconst.com

May 13, 2005
City of Kelso – North Potable Water Reservoir Leak Repair

12	F-11-1	86" L ALONG TOP EXPANSION JOINT BETWEEN PANELS 11 & 12 (STARTED OUT 19" X 6" LEAK AND INCREASED AS REPAIRS WERE MADE)	REPAIRED
13	F-11-2	17" L TOP EXPANSION JOINT OF PANEL 11	REPAIRED
14	F-12-1	16" L BOTTOM EXPANSION JOINT AT CORNER OF PANELS 12 & 13	REPAIRED
15	F-13-1	18" L EXPANSION JOINT BETWEEN PANELS 13 & 14	REPAIRED
16	F-14-1	20" L TOP EXPANSION JOINT OF PANEL 14	REPAIRED
17	W-1	1" X ½" DIA. HOLE 1' OFF BOTTOM WALL – FLOOR JOINT	REPAIRED
18	W-2	½" DIA. HOLE 6' OFF BOTTOM WALL- FLOOR JOINT	REPAIRED
19	W-3	1" DIA. HOLE 6' OFF BOTTOM WALL – FLOOR JOINT 3' TO THE RIGHT OF W-2	REPAIRED
20	W-4	1 ½" X 1 ½" DIA. HOLE 7' OFF BOTTOM WALL – FLOOR JOINT (UPON REPAIRING LEAK THE CONCRETE APPEARED TO BE DETERIORATING IN THIS AREA. UPON CLEANING FOR REPAIR THE HOLE BEGAN TO GET LARGER. AFTER REPAIR WAS MADE WITH EPOXY, THE CONCRETE APPEARED TO BE MORE STABILIZED.)	REPAIRED

If there are any questions regarding this inspection report please call Adam Krausman @ (360) 381-0368. Thank you very much for your business.

MATERIAL SAFETY DATA SHEET

Trade Name: **AquataPoxy A-7 - Part A**

SECTION I

Manufacturer: Raven Lining Systems
1024 North Lansing
Tulsa, OK 74106

Emergency Telephone #: 800-424-9300
Chemtrec

Revision Date: 8/01/01

Information Telephone #: 918-584-2810
800-324-2810

SECTION II: INGREDIENT INFORMATION

INGREDIENT	CAS NUMBER	PEL	TLV
Epoxy Resin	25085-99-8		
Silica*	7631-86-9	20g/m^3	10 mg/m^3

*Hazard as dust only

SARA Title III, Section 313 ingredients: None

The specific chemical identity is being withheld as a trade secret.

SECTION III: PHYSICAL DATA

Boiling Point: > 200 deg F
Vapor Pressure: N/A
Vapor Density: N/A
Solubility in Water: negligible
Specific Gravity: 1.24
Melting Point: N/A
Evaporation Rate: Not established
% Volatile by Volume: <1%
Appearance and Odor: Mastic consistency - faint epoxy odor

SECTION IV: FIRE & EXPLOSION HAZARD DATA

Flash Point: >200 deg F, TCC Method
Extinguishing Media: Foam, CO_2, Dry Chemical
DOT Flammability Classification: non-flammable
Special Fire Fighting Procedures: The use of self-contained breathing apparatus is recommended for firefighers. Water may be helpful in keeping adjacent containers cool. Avoid spreading burning liquid with water used for cooling purposes.
Unusual Fire and Explosion Hazards: Keep work areas free of hot metal surfaces and other sources of ignition.

SECTION V: REACTIVITY DATA

Stability: Stable
Incompatibility: Strong acids and bases, selected amines, oxidizing agents.
Hazardous Decomposition or Byproducts: Thermal decomposition in the presence of air may yield carbon monoxide and carbon dioxide.
Hazardous Polymerization: Will not occur.

SECTION VI: HEALTH HAZARD DATA

Primary Routes of Entry:

EYES: May cause slight transient (temporary) eye irritation. Corneal injury is unlikely.
SKIN: May cause allergic skin reaction in susceptible individuals. Prolonged exposure not likely to cause significant skin irritation. Repeated exposure may cause skin irritation. A single prolonged exposure is not likely to result in the material being absorbed through skin in harmful amounts.
INHALATION: Vapors are unlikely due to physical properties.
INGESTION: Single dose oral toxicity is low. The oral LD50 for rats is >4000 mg/kg. No hazards anticipated from ingestion incidental to industrial exposure.
SYSTEMIC and OTHER EFFECTS: Except for skin sensitization, repeated exposures to low molecular weight epoxy resins of this type are not anticipated to cause any significant adverse effects.

Carcinogenicity: A poorly characterized sample of low molecular weight epoxy resin of this type (diglycidyl ether of bisphenol A) has been reported to produce skin cancer in a highly sensitive strain of mice. However, high levels of impurities (including a known animal skin carcinogen) compromise the validity of the findings. Epoxy resin that is representative of current manufacturing processes is not believed to be a cancer hazard to humans. Results of mutagenicity tests in animals have been negative. Has been shown to be negative in some invitro ("test tube") mutagenicity tests and positive in others.

Emergency and First Aid Procedures:

EYES: Flush with large quantities of water for at least 15 minutes. Consult a physician.
SKIN: Wash off in flowing water or shower.
INHALATION: Remove to fresh air if affects occur. Consult a physician.
INGESTION: No adverse effects anticipated by this route of exposure incidental to proper industrial handling.

SECTION VII: PRECAUTIONS FOR SAFE HANDLING AND USE

Steps to Be Taken in Case Material is Released or Spilled: Keep sources of ignition and hot metal surfaces isolated from the spill. Material may flow slowly. Scrape into containers for disposal.
Waste Disposal Methods: Dispose of according to all local, state and federal regulations.
Precautions to Be Taken in Handling and Storing: Keep containers closed when not in use. Avoid prolonged or repeated contact with skin. DO NOT handle or store near flame, heat or strong oxidants. Do not store in direct sunlight. Avoid prolonged storage above 38 deg C (100 deg F).

SECTION VIII: CONTROL MEASURES

RESPIRATORY: Respiratory protection should not be needed. If respiratory irritation is experienced, use an approved air-purifying respirator.
VENTILATION: General mechanical ventilation is sufficient for most conditions. Local exhaust ventilation may be necessary for some operations.
EYES: Use safety glasses, splash-proof eye goggles or goggles with full faceshield.
CLOTHING/GLOVES: Use impermeable gloves to prevent skin irritation in sensitive individuals. Wear clean, long-sleeved, body covering clothing.

SECTION IX: TRANSPORT DATA

Proper Shipping Name: not regulated
Hazard Class: N/A
Identification Number: not regulated
Packing Group: N/A

SECTION X: DISCLAIMER

RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES WITH RESPECT TO ANY INFORMATION PRESENTED HEREIN, ALL OF WHICH IS PROVIDED "AS IS". TO THE MAXIMUM EXTENT PERMITTED BY LAW, RAVEN EXPRESSLY EXCLUDES ALL WARRANTIES, OBLIGATIONS, REPRESENTATIONS, LIABILITIES, TERMS AND CONDITIONS (WHETHER THEY ARE EXPRESS OR IMPLIED, OR ARISE IN CONTRACT, STATUTE, OR OTHERWISE, AND IRRESPECTIVE OF THE NEGLIGENCE OF RAVEN, ITS EMPLOYEES OR AGENTS) IN CONNECTION WITH THE INFORMATION PRESENTED HEREIN. RAVEN MAKES NO REPRESENTATIONS OR WARRANTIES AS TO MERCHANTABILITY, FITNESS FOR PURPOSE, NONINFRINGEMENT OR CONFORMITY WITH DESCRIPTION OR SAMPLE.

MATERIAL SAFETY DATA SHEET

Trade Name: **AquataPoxy A-7 - Part B**

SECTION I

Manufacturer: Raven Lining Systems
1024 North Lansing
Tulsa, OK 74106

Emergency Telephone #: 800-424-9300
Chemtrec

Revision Date: 8/01/01

Information Telephone #: 918-584-2810
800-324-2810

SECTION II: INGREDIENT INFORMATION

INGREDIENT	CAS NUMBER	PEL	TLV
Microcrystalline Silica (quartz)	14808-60-7	0.1 mg/m^3 (respirable)	0.1 mg/m^3 (respirable fraction)
Aliphatic Amine	N/A		

SARA Title III, Section 313 ingredients: NONE

The specific chemical identity is being withheld as a trade secret.

SECTION III: PHYSICAL DATA

Boiling Point: > 200 deg F
Vapor Pressure: N/A
Vapor Density: N/A
Solubility in Water: negligible
Appearance and Odor: Mastic consistency - bland odor

Specific Gravity: 1.20 - 1.25
Melting Point: N/A
Evaporation Rate: Not established
% Volatile by Volume: <1%

SECTION IV: FIRE & EXPLOSION HAZARD DATA

Flash Point: >200 deg F, Pensky-Martin Closed Cup
Fire Hazard Classification (OSHA/NFPA): Class III B.
Extinguishing Media: Foam, CO_2, Dry Chemical, Water Spray
Special Fire Fighting Procedures: The use of self-contained breathing apparatus is recommended for firefighters. Water may be helpful in keeping adjacent containers cool.
Unusual Fire and Explosion Hazards: May generate toxic or irritating combustion products. Sudden reaction and fire may result if product is mixed with an oxidizing agent.

SECTION V: REACTIVITY DATA

Stability: Stable
Incompatibility: Strong acids and bases, selected epoxy resins and strong oxidizing agents.
Hazardous Decomposition or Byproducts: Thermal decomposition in the presence of air may yield carbon monoxide, carbon dioxide and nitrogen oxides.
Hazardous Polymerization: Will not occur.

Appendix E – Sources of Seismicity for Kelso, Washington Region

The Pacific Northwest is seismically active area. The regional sources of seismicity affecting the Kelso area, and hence the potential for ground shaking, are controlled by three separate fault mechanisms: Cascadia Subduction zone, Intraplate Zone and Shallow Crust Earthquakes. Descriptions of these potential earthquake sources are presented below.

E.1 - Cascadia Subduction Zone

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between the two plates is dipping to the east, and therefore, becomes deeper toward Portland, Oregon. At the easternmost portion of the interface zone, which is thought to be capable of generating strong ground motions, the interface between these two plates is located at a depth of approximately 20 to 25 kilometers (km).

Quantifying the seismicity and hazard posed by the CSZ is subject to several uncertainties, including the size of the maximum credible earthquake as described by the Mw of the event; the rate of seismicity associated with CSZ earthquakes of various magnitudes; and the nature of the ground motions associated with CSZ earthquakes (M_w is used in the seismology and earthquake engineering communities to quantify the size of larger earthquakes and is based on fault displacement and area of fault rupture). Geologic evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon and Washington coast and in offshore landslide deposits (turbidities). This paleoseismic data has been used to infer the size of prehistoric earthquakes as well as their rate of recurrence. Sequences of interlayered peat and sand have been interpreted to be the result of large ($M_w > 8$) subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago.

E.2 - Intraplate Zone

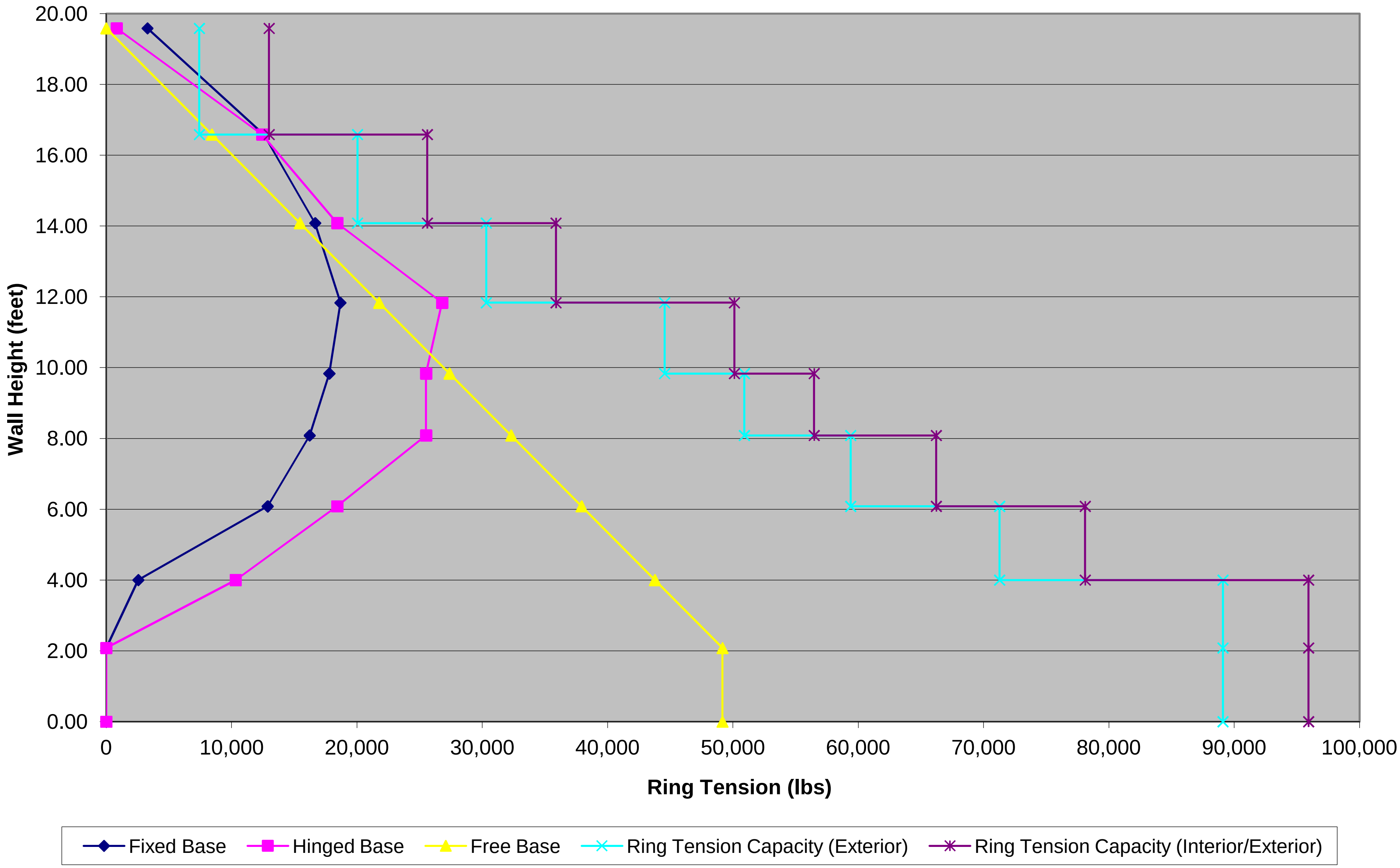
The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below western Washington. Very low levels of seismicity have been observed within the intraplate zone in southwest Washington. However, much higher levels of seismicity within this zone have been recorded in northern Washington and California. Several reasons for this seismic quiescence were suggested by Geomatrix (1995), and include changes in the direction of subduction between Washington and British Columbia, as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia M_w 7.1, 1965 Puget Sound M_w 6.5, and 2001 Nisqually M_w 6.8 earthquakes. Based on the data presented within the Geomatrix report (1995), an earthquake of M_w 7.25 has been chosen to represent the seismic potential of the intraplate zone. The long return period postulated for intraplate earthquakes results in a very small contribution to the overall hazard in the vicinity of the project site.

E.3 - Shallow Crustal Earthquakes

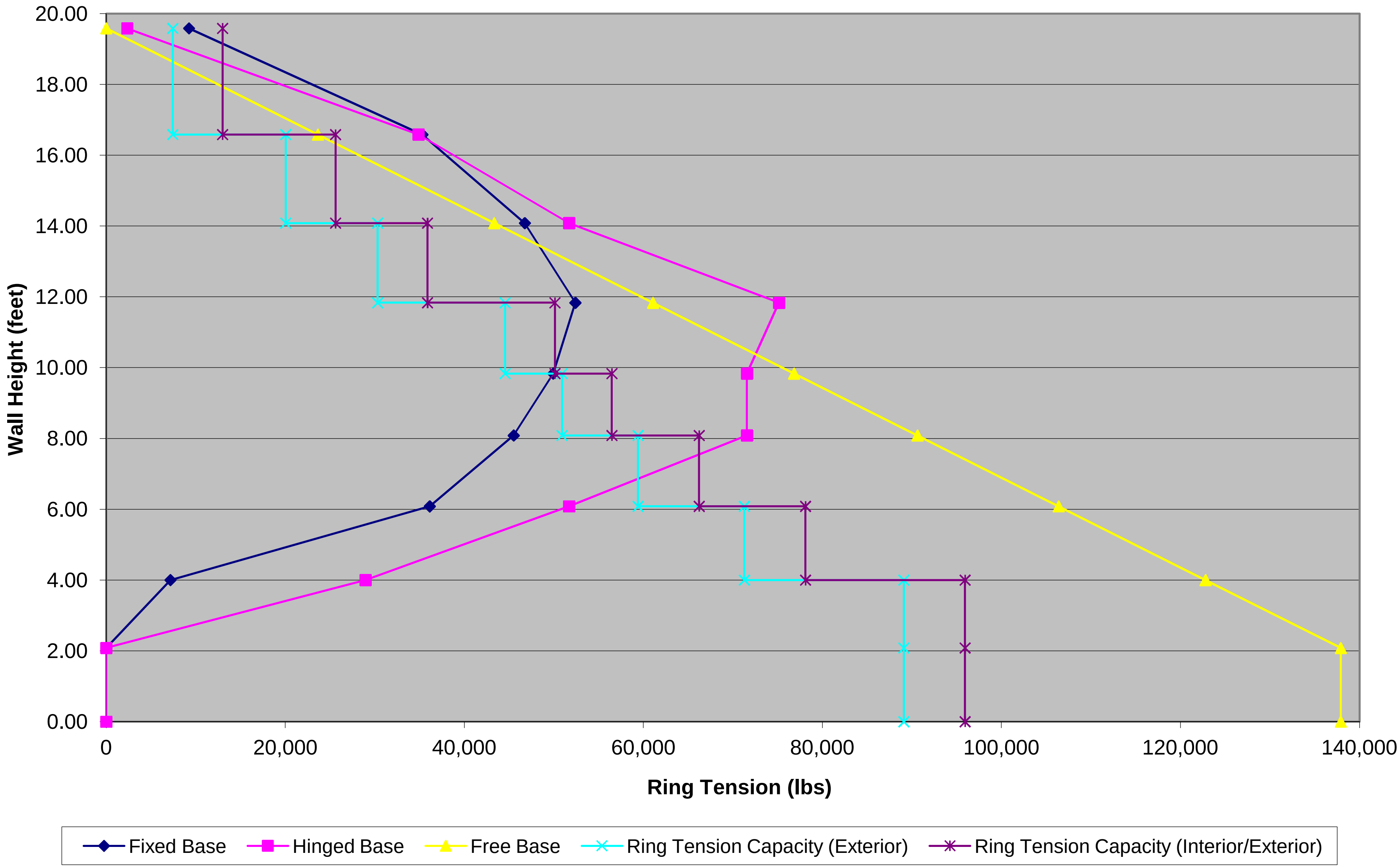
The third source of seismicity that can result in significant ground shaking within the greater Cowlitz County area is near-surface crustal earthquakes occurring within the

North American Plate. The 1993 Scotts Mills (M_w 5.6) and Klamath Falls (M_w 6.0) earthquakes are examples of rather shallow crustal earthquakes. The characterization of the local crustal earthquake sources includes known faults thought to be active in region and consideration of possible seismicity that may occur in the region along unmapped sources. The crustal earthquakes that occur along currently unmapped faults in the region have been referred to in seismic hazard investigations as “randomly occurring” earthquakes, “aerial sources”, or “gridded seismicity”.

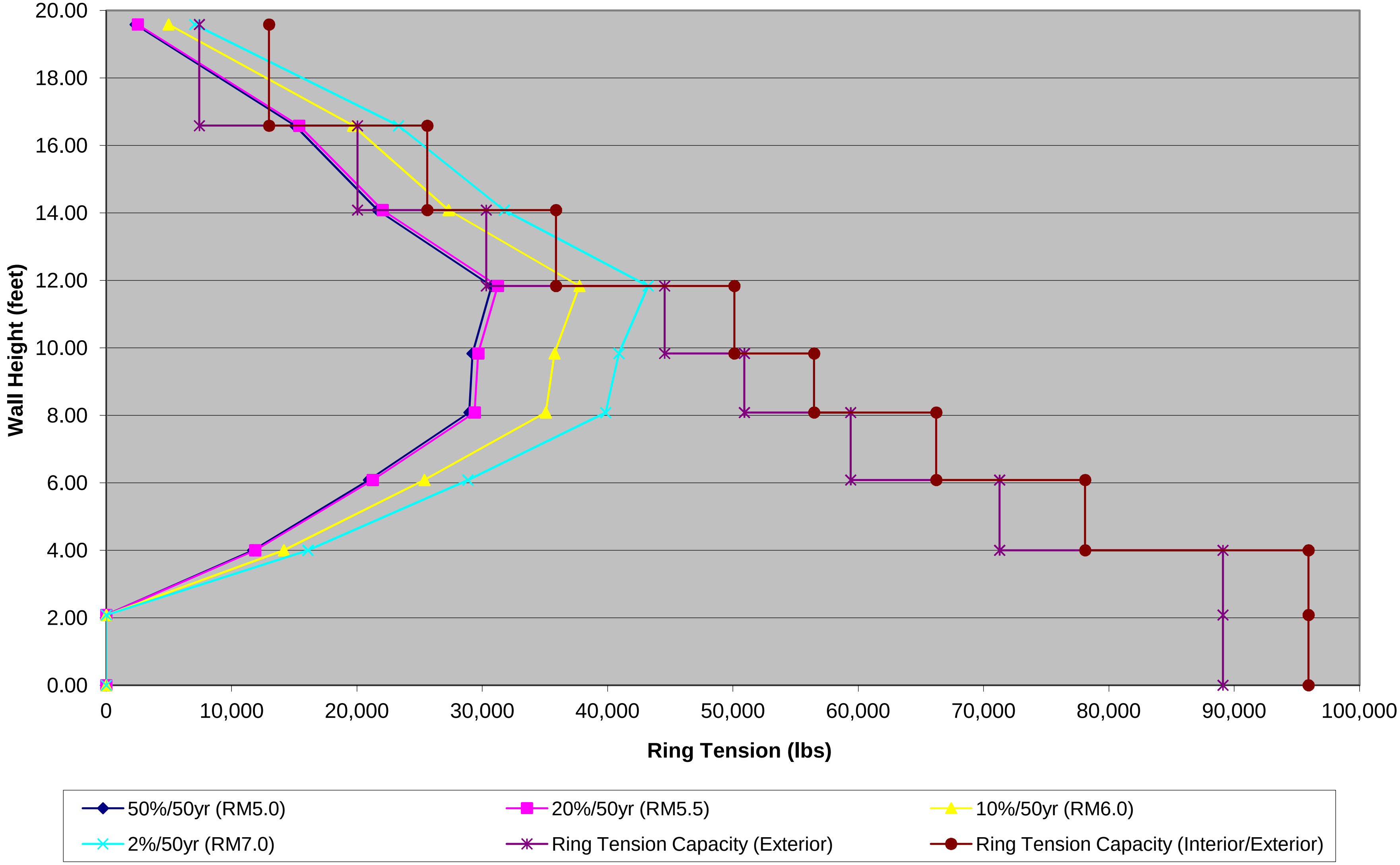
Service Hydrostatic Ring Tension Envelope



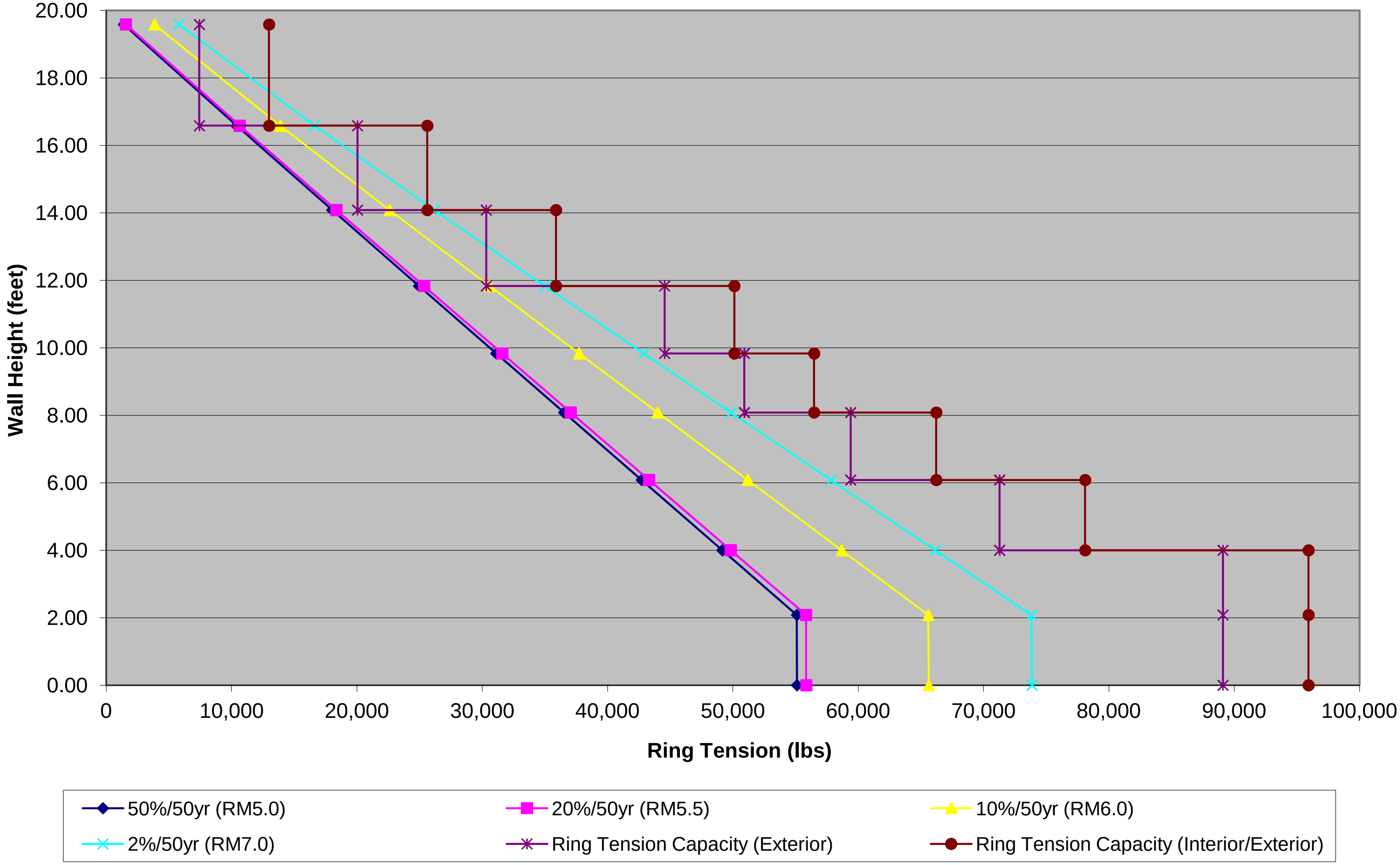
Strength (Factored 1.7 x 1.65) Hydrostatic Ring Tension Envelope

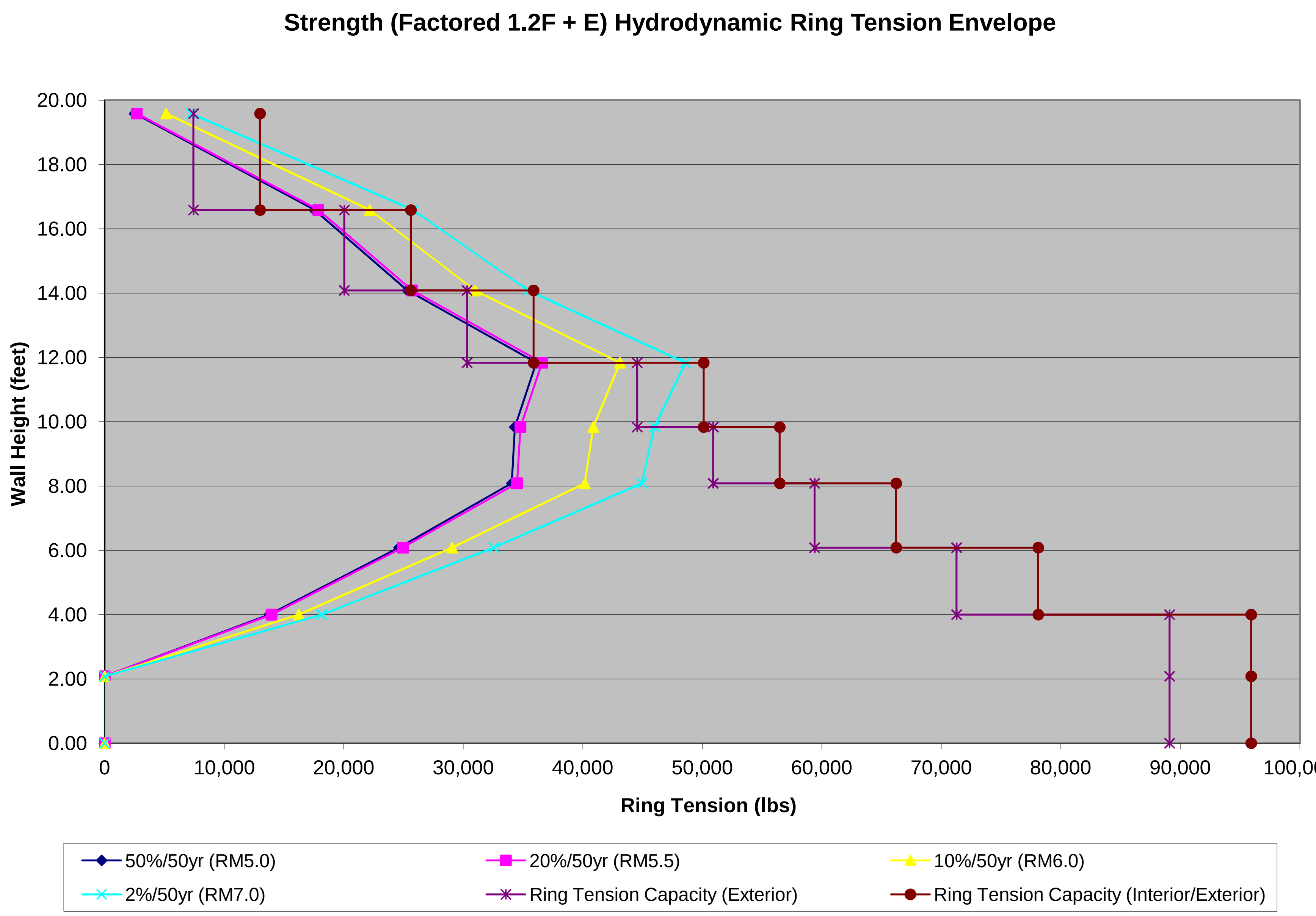


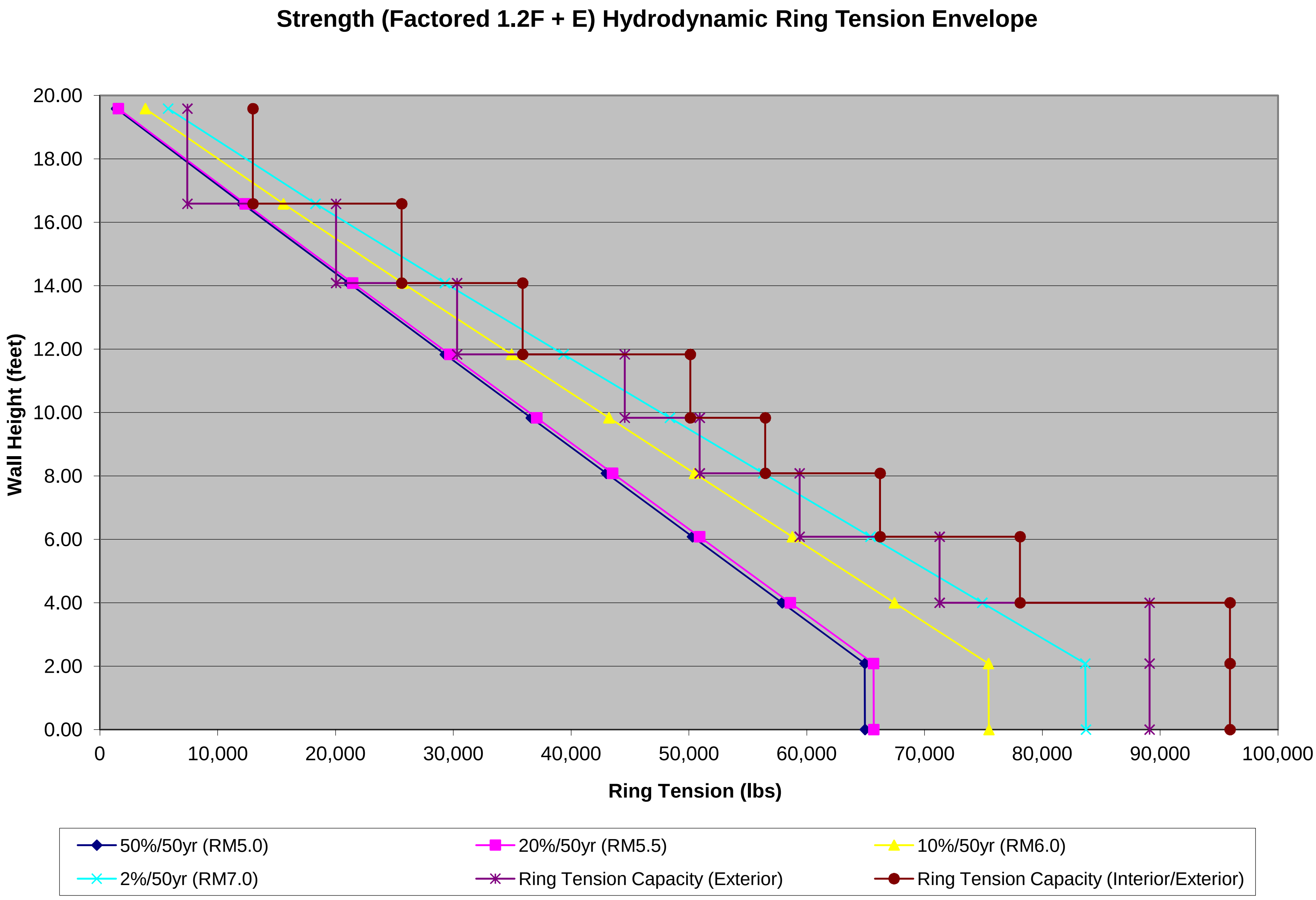
Service Hydrodynamic Ring Tension Envelope

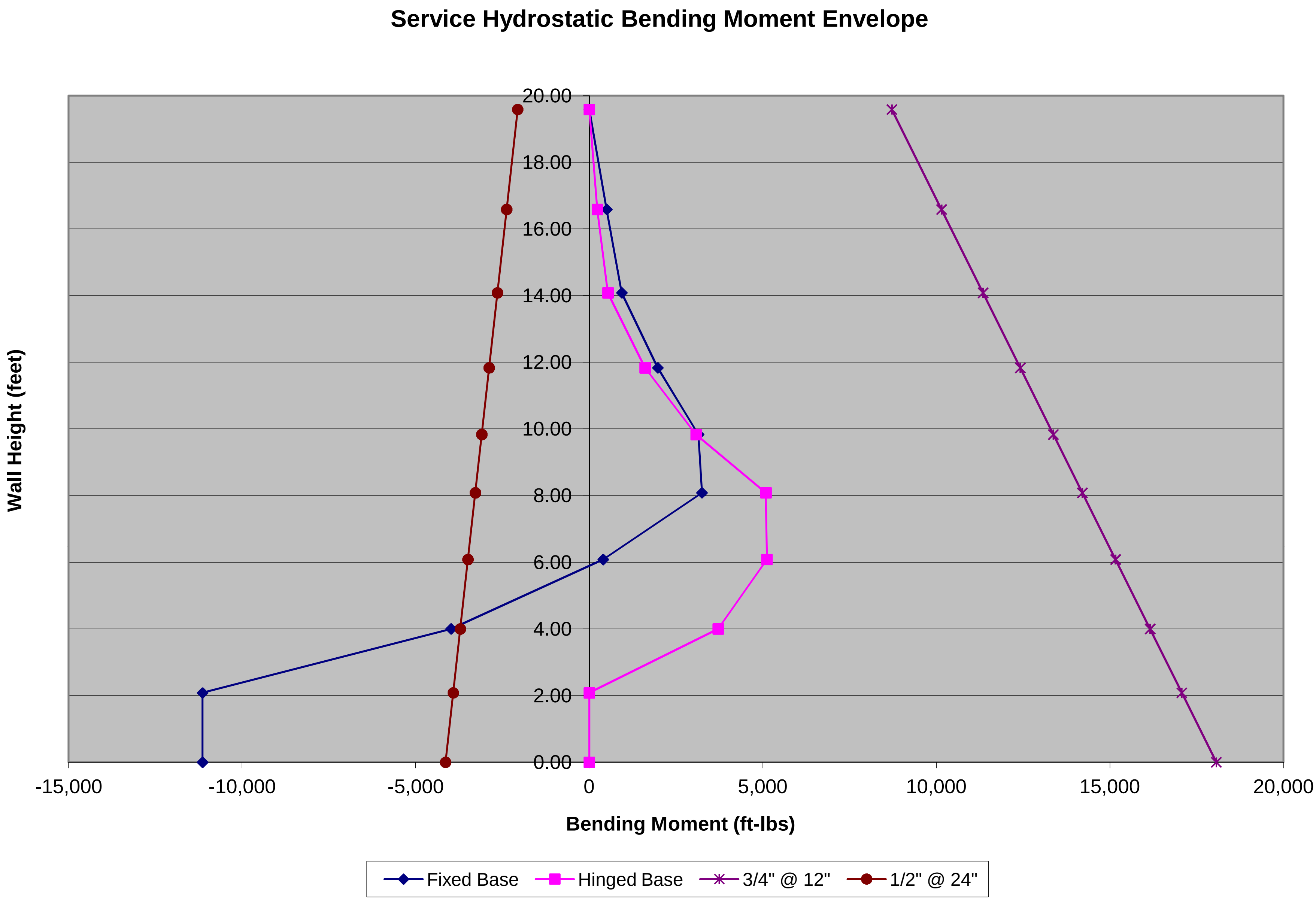


Service Hydrodynamic Ring Tension Envelope

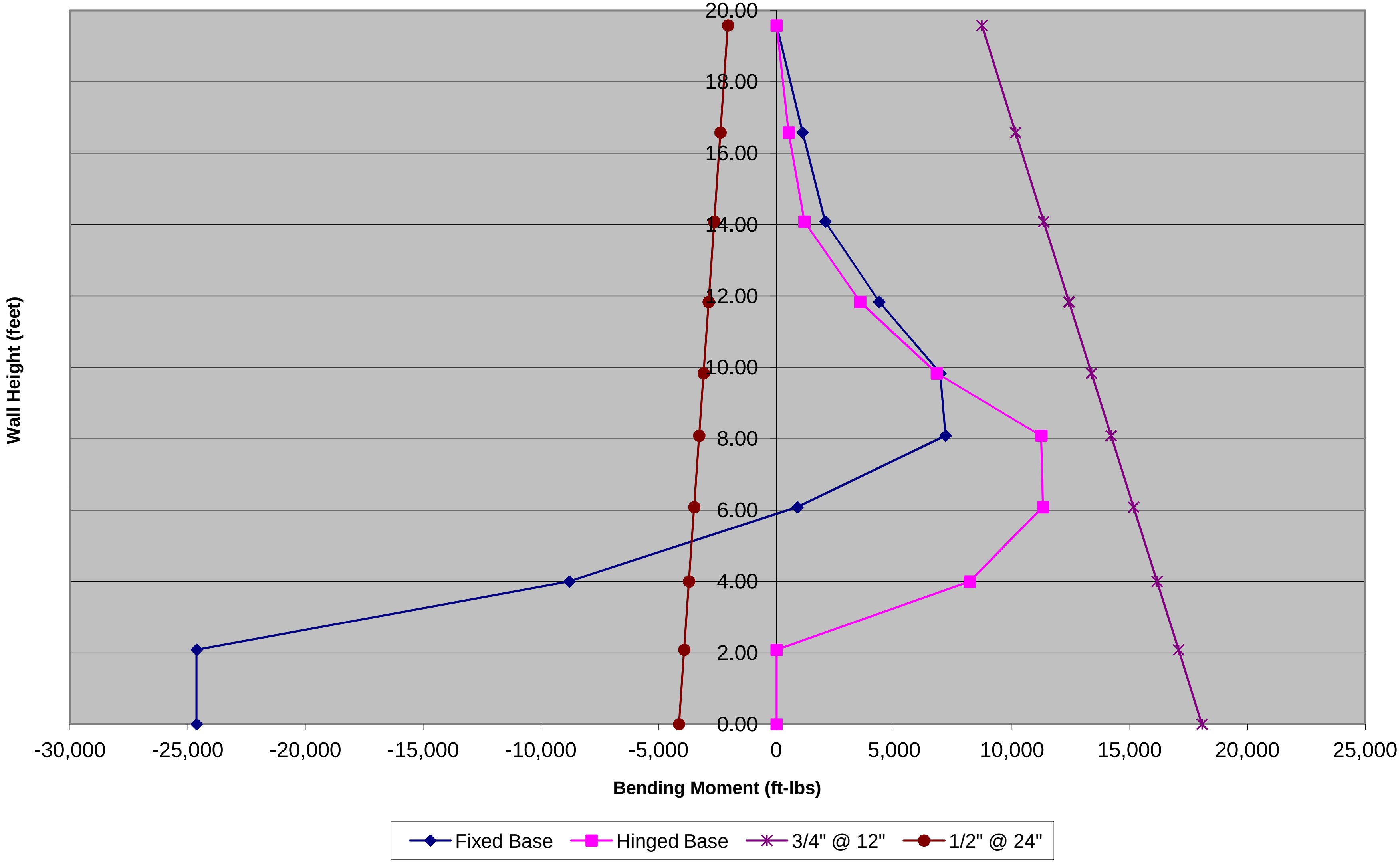



Strength (Factored 1.2F + E) Hydrodynamic Ring Tension Envelope
Wall Height (feet)
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
90,000
100,000
Ring Tension (lbs)
50%/50yr (RM5.0)
20%/50yr (RM5.5)
10%/50yr (RM6.0)
2%/50yr (RM7.0)
Ring Tension Capacity (Exterior)
Ring Tension Capacity (Interior/Exterior)

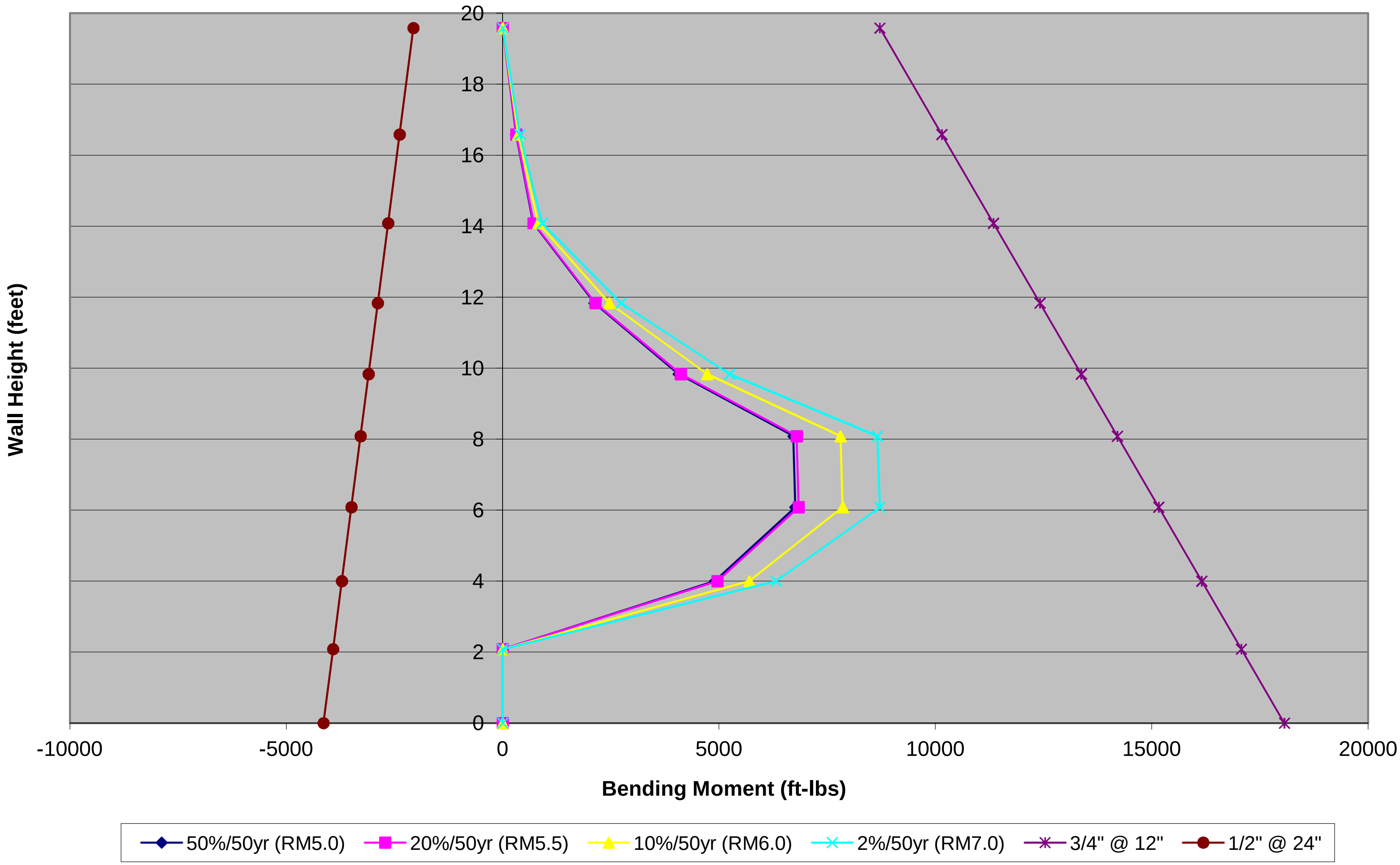

Strength (Factored 1.2F + E) Hydrodynamic Ring Tension Envelope
Wall Height (feet)
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
90,000
100,000
Ring Tension (lbs)
50%/50yr (RM5.0)
20%/50yr (RM5.5)
10%/50yr (RM6.0)
2%/50yr (RM7.0)
Ring Tension Capacity (Exterior)
Ring Tension Capacity (Interior/Exterior)


Service Hydrostatic Bending Moment Envelope
Wall Height (feet)
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
-15,000
-10,000
-5,000
0
5,000
10,000
15,000
20,000
Bending Moment (ft-lbs)
Fixed Base
Hinged Base
3/4" @ 12"
1/2" @ 24"

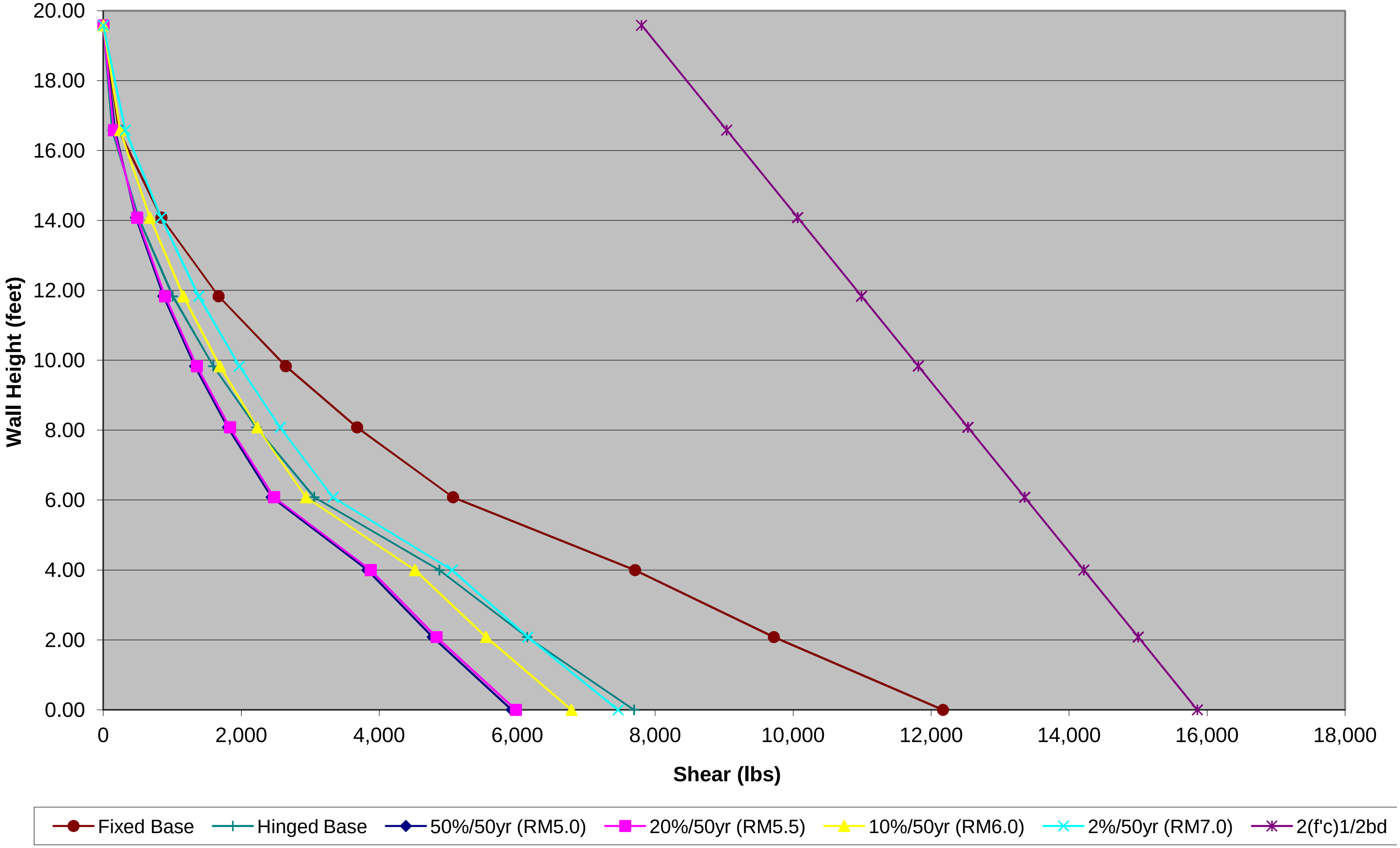
Strength (Factored 1.7 x 1.3) Hydrostatic Bending Moment Envelope



Strength (Factored 1.2F + E) Hydrodynamic Bending Moment Envelope



Shear Strength vs Capacity Envelope



Capital Recovery and Present Worth Formula for Annual Cost Calculations

P1 =	$1,809,000
P2 =	$2,406,000
P3 =	$4,133,000

Alternative No. 1 - Repair Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
Repair Cost						
(A/P, DR, 10)	0.105582	0.111327	0.1172305	0.1232909	0.129505	0.135868
New Reservoir Cost						
(P/F, DR, 10)	0.905287	0.820348	0.7440939	0.6755642	0.613913	0.5583948
(A/P, DR, 90)	0.016903	0.024046	0.0322556	0.0412078	0.050627	0.0603184
Annual Cost	$254,242	$282,918	$311,267	$338,090	$362,730	$384,991

Alternative No. 2 - Strengthen Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
Strengthen Cost						
(A/P, DR, 20)	0.055415	0.061157	0.0672157	0.0735818	0.080243	0.0871846
New Reservoir Cost						
(P/F, DR, 20)	0.819544	0.672971	0.5536758	0.4563869	0.376889	0.3118047
(A/P, DR, 100)	0.015866	0.023203	0.0316467	0.040808	0.050383	0.0601774
Annual Cost	$187,069	$211,679	$234,139	$254,012	$271,545	$287,316

Alternative No. 3 - Replace Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
New Reservoir Cost						
(A/P, DR, 40)	0.030456	0.036556	0.0432624	0.0505235	0.058278	0.0664615
Annual Cost	$125,873	$151,085	$178,803	$208,814	$240,864	$274,686

Capital Recovery and Present Worth Formula for Annual Cost Calculations

P1 =	$1,809,000
P2 =	$2,406,000
P3 =	$4,133,000

Alternative No. 1 - Repair Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
Repair Cost						
(A/P, DR, 20)	0.055415	0.0611567	0.067216	0.073582	0.0802426	0.0871846
New Reservoir Cost						
(P/F, DR, 20)	0.819544	0.6729713	0.553676	0.456387	0.3768895	0.3118047
(A/P, DR, 100)	0.015866	0.0232027	0.031647	0.040808	0.0503831	0.0601774
Annual Cost	$153,986	$175,168	$194,012	$210,083	$223,640	$235,267

Alternative No. 2 - Strengthen Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
Strengthen Cost						
(A/P, DR, 40)	0.030456	0.0365557	0.043262	0.050523	0.0582782	0.0664615
New Reservoir Cost						
(P/F, DR, 40)	0.671653	0.4528904	0.306557	0.208289	0.1420457	0.0972222
(A/P, DR, 120)	0.014347	0.0220481	0.03089	0.040365	0.0501437	0.0600552
Annual Cost	$113,103	$129,223	$143,227	$156,308	$169,655	$184,038

Alternative No. 3 - Replace Reservoirs

DR or i	1%	2%	3%	4%	5%	6%
New Reservoir Cost						
(A/P, DR, 80)	0.018219	0.0251607	0.033112	0.041814	0.0510296	0.0605725
Annual Cost	$75,299	$103,989	$136,851	$172,818	$210,905	$250,346