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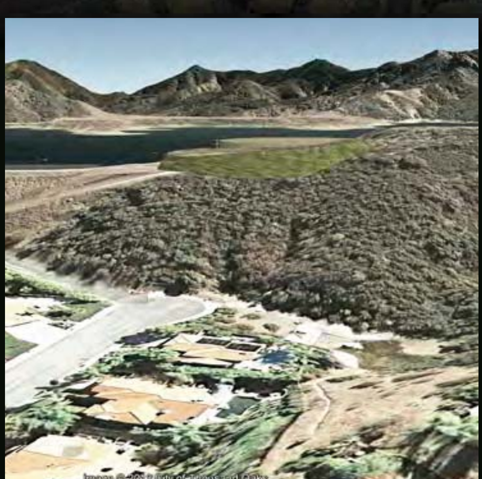
Las Virgenes Municipal Water District

Preliminary Design Report
5-Million-Gallon Water Storage Tank



Preliminary Design Report
5 Million Gallon Water Storage Tank

April 2013

Prepared for

Las Virgenes Municipal Water District
4232 Las Virgenes Road
Calabasas, CA 91302

AECOM Project No. 60283626

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Section 1: Introduction

1.1 Background

The Las Virgenes Municipal Water District (District) is a special district that provides potable water, wastewater treatment, biosolids composting and recycled water services to a 122 square mile service area in Los Angeles County. The service area includes the cities of Agoura Hills, Calabasas, Hidden Hills, and Westlake Village, as well as unincorporated areas of Los Angeles County.

The District purchases 100 percent of its potable water from Metropolitan Water District of Southern California. The District's potable water facilitates include 25 potable water tanks, 24 pump stations, 177 miles of transmission pipeline and 206 miles of distribution pipeline. The District also operates and maintains the 15 million gallon per day (MGD) Westlake Filtration Plant (WFP), which provides disinfection and filtration for the Las Virgenes Reservoir (Reservoir), an uncovered reservoir in Westlake Village.

The Reservoir serves as a non-potable water storage unit that is filled during the winter months. When water is needed in the summer months, raw water (RW) is drawn out of the Raw Water Reservoir (RWR) via vacuum pumps, filtered with diatomaceous earth (DE) filters, and disinfected using chloramination. The filtered water (FW) is pumped into the 1235-foot zone via the Westlake Pump Station (WPS) and released into the Las Virgenes distribution system.

The 2007 Potable Water Master Plan identified the Backbone Improvement Program to address a deficiency of 4 million gallons (MG) of storage in the western portion of the District. The Backbone Improvement Program contains several components that are to be implemented in phases; these phases include the following:

- Phase 1 - Pipeline improvements (upsizing) along the backbone pipeline (Agoura and Calabasas Pipeline Improvements projects)
- Phase 2 - 5 MG filtered water reservoir (Tank)
- Phase 3 - Expansion of the WFP from 15 MGD to 20.4 MGD
- Phase 4 - Upgrades to the WPS

The pipeline projects are already underway and completion of the Tank constitutes the next phase of the overall program. Once complete, this additional storage will correct existing and future storage deficiencies, as currently projected.

1.2 Report Overview

This Preliminary Design Report (PDR) addresses topics related to the Tank construction, and meets the requirements of the contract scope of work (provided as **Appendix A**). The major elements of this project are noted as follows:

- **Tank –** The 5 MG prestressed concrete tank will be located to the south of the existing WFP. The tank will have a floor elevation of 1065 feet, a height of 24 feet, and a diameter of 200 feet. The tank will include baffling to allow for plug flow and flexibility for future use to provide additional chlorine contact time.

- **Pipelines –** Two parallel 36-inch pipelines, inlet and outlet, will connect the Tank to the existing effluent filtered water pipeline at the WFP. The piping will be routed along the embankment of the Saddle Dam and held in place with steel piers or concrete block supports. A valve connection on the existing FW piping will allow for water to be gravity fed from the existing FW Reservoir (FWR) to the new Tank. The FW from the Tank will then be gravity fed back into the existing FW pipeline. Additional piping will be provided for winter fill and flexibility for increased fill rates in the future.

- **Berm –** To mitigate visual impact of the Tank to nearby residents, a berm will be constructed. The berm will be planted with native vegetation to match adjacent hillsides. The berm will not provide complete screening of the Tank, but will conceal a portion.

1.2.1 Technical Memorandums

For purposes of focusing the PDR on the key technical issues related to the design of the Tank project, several key topics are being addressed in parallel studies, via Technical Memorandums (TM). This allows for detailed review by outside agencies without impacting the design schedule.

Table 1-1 provides a summary of the technical memoranda for the project. The final technical memoranda are included as **Appendix B**.

TABLE 1-1
SUMMARY OF TECHNICAL MEMORANDUMS

Technical Memorandum	Focus
1	Hydraulics and Models
2	Blasting Plan Outline
3	Public Outreach Concepts
4	Blasting Mitigation Concepts
5	Landscape Concepts
6	Traffic Mitigation Concepts
7	Anticipated Permits

1.3 Report Structure

The project report has been separated into the following sections:

- **Section 2 – Tank Design:** This section provides a summary of the Tank location and dimensions, pipeline inlet and outlet, integration with existing WFP and key appurtenances. Operation conditions are presented for present and future conditions.

- **Section 3 – Conveyance:** This section details the hydraulics, alignment, and dam crossing for the new 36-inch inlet and outlet pipelines. Winter fill bypass alternatives and site pipeline routing and conflicts are also addressed.

- **Section 4 – Future Considerations:** Phases 3 and 4, WFP expansion and WPS improvements, respectively, are introduced along with consideration of proposed design elements on these subsequent phases.

- **Section 5 – Structural Design Criteria:** This section addresses tank construction issues, including design loads, applicable codes and standards, materials, and geotechnical concerns.

- **Section 6 – Electrical and Control Systems:** This section summarizes the electrical and control design additions related to the Tank and pipeline construction.

- **Section 7 – Preliminary Cost Estimates:** Construction costs are summarized in this section.

- **Section 8 – Construction Schedule:** The construction schedule is detailed in this section.

Section 2: Tank Design

This section provides a summary of the tank location and dimensions, pipeline inlet and outlet, integration with existing WFP and key appurtenances. Operation conditions are presented for present and future conditions.

2.1 Sizing/Design Criteria

The volume required for the Tank is determined by previous studies completed by the District, including most recently in the 2007 Potable Water Master Plan and summarized in several subsequent documents by the District[1]. The volume of the Tank was determined to be 5 MG. The sizing is based on the future storage deficit of 4.48 MG (2007 MP – 2030), and is considered the volume required for the future operation of the Tank. However, the Tank could operate at a lower volume of 3.45 MG, based on the 2010 UWMP – 2020 without conservation, until 2020. Based on this review of available documentation, the required Tank volume design criteria are summarized as follows:

- 5 MG total volume
- Minimum 4.5 MG operational volume (after 2020)
- Minimum 3.5 MG operational volume (initial to 2020)

Providing a range of required volume based on current and future demand provides flexibility in the initial design and allows for potential phasing of available volume. This approach allows for cost savings for this phase of project by avoiding certain modifications at the WFP, such as improvements to the FWR.

The Tank dimensions were calculated based on the required volume and the following limiting constraints:

- Minimum elevation of finished floor is controlled by the invert elevation of the existing FW pipeline at 1064.40 feet (Per C-7, WFP Record Dwgs). To provide proper draining, the tank finished floor minimum will be 1065.0 feet.
- The Tank has been presented to the public as not exceeding the roof level of the existing WFP. The building height of the existing WFP is noted as 1089 feet (Drawing A-6, WFP Record Dwgs), which was confirmed during the field survey. The Tank height will not exceed 1089 feet.
- The roof needs to be three feet above the high water level to allow for sloshing during seismic events.
- The roof slab is estimated to be approximately one foot thick.

Based on these constraints, the Tank dimensions will be 200 foot diameter with a side wall height of 24 feet. The FWR target level of 12 feet above finished floor of 1071 feet will provide the required volume, meet the design constraints noted above, and provide a cost effective

[1] David Lippman, District Director of Facilities & Operations, to John Mundy, District General Manager, 4 June 2012, *Backbone Improvement Program*

design. **Table 2-1** provides a summary of the proposed Tank volume at various levels and phases.

TABLE 2-1
SUMMARY OF 200' X 24' TANK

Elevation (feet)	Level (Feet)	Volume (MG)[(a)]	Description
1089	24		Roof Slab & Cross fall
1088	23		
1087	22	5.01	Freeboard (Sloshing)
1086	21	4.78	
1085	20	4.54	Phase 3, 4.78 MG (TANK ONLY)
1084	19	4.31	
1083	18	4.07	Phase 2, 4.62 MG Capacity (4.31MG TANK + 314,000 gallon FWR)
1082	17	3.84	
1081	16	3.61	
1080	15	3.37	
1079	14	3.14	
1078	13	2.90	
1077	12	2.67	
1076	11	2.44	
1075	10	2.20	
1074	9	1.97	
1073	8	1.73	
1072	7	1.50	
1071	6	1.27	
1070	5	1.03	
1069	4	0.80	
1068	3	0.56	
1067	2	0.33	
1066	1	0.10	
1065	0	0.00	

(a) Top of Westlake Filtration Plant Building Parapet wall is 1089.1 (source Penfield and Smith 2013 survey). The above volume totals reflect volume loss from columns and pedestals estimated at 0.036 MG. Volumes assumed a 1.5 % floor slope. A 1% floor slope increase the storage approximately 1%. Roof cross fall is ~ 12" minimum and roof thickness is approximately 12" (~24" total top reservoir to top wall).

Table 2-1 illustrates that following completion of this project (Phase 2 of the overall Backbone Improvement Program) 4.62 MG of storage capacity will be available between the FWR and Tank. The Phase 3 volume designates the storage capacity following the WFP expansion, designated as Phase 3. The change in volume is due to the modification at the WFP. During that phase, the FWR will be bypassed and FW will be pumped directly to the Tank in lieu of first flowing through the FWR.

In summary, **Table 2-1** demonstrates that the required interim operational volume of 3.5 MG is provided by this project and the future 4.5 MG required operational volume will be provided by Phase 3.

2.2 Tank Piping

The inlet, outlet, and overflow piping will be routed through floor slab penetrations to minimize penetrations through the pre-stressed concrete walls. For corrosion protection all piping placed under the floor slab will be concrete encased. Flexible joints will be provided outside the wall footing to accommodate any movement caused by differential settlement or seismic activity. Minimal settlement is expected as rock is located below the Tank site.

2.2.1 Inlet/Outlet Piping

The inlet piping will be routed along the Tank floor to the side opposite the outlet piping and will include a 90 degree elbow, angled 45 degree from the Tank floor. The outlet pipe will consist of an opening in the floor slab with a railing surrounding the opening for operator safety during future maintenance within the Tank. The configuration of the inlet and outlet piping are illustrated on Drawing P.04 of **Appendix C**.

2.2.2 Overflow

The Tank overflow will be routed back to the Reservoir, where a concrete pad with rock features will be constructed, similar to the overflow provided for the existing RWR. With regards to noise from flowing water, AECOM does not anticipate any impacts to residents in the area. The closest residence is estimated to be 350 feet (horizontally) from the tank overflow. The elevation of the residence is at 990 feet. The elevation of the highest ground elevation between the overflow and residence is approximately 1063 feet. It is not anticipated that sound will travel with any significant measure from the tank overflow down towards the resident closest to the tank overflow.

The Tank overflow system must be designed to pass the maximum filling rate, which is 20.4 MGD (Phase 3 - WFP expansion). In addition, the overflow weir (inside of the Tank) may operate in the future as a means for filling the Reservoir which will not exceed this flow rate. **Table 2-2** shows the weir calculations for a 30-inch and 36-inch weir.

TABLE 2-2
SUMMARY OF OVERFLOW WEIR CALCULATIONS

Weir Overflow (MGD)	Circumference (feet)	Head Required (inches)	Weir Diameter (inches)
27.2	7.9	12.0	30
20.4		***10.0***	
17.6		9.0	
9.6		6.0	
3.4		3.0	
32.6	9.4	12.00	36
21.2		9.00	
20.4		***8.80***	
11.5		6.00	
4.1		3.00	

Based on the calculations provided in **Table 2-2**, a 30-inch overflow would require approximately 10-inches of hydraulic head to release the peak flow of 20.4 MGD and a 36-inch overflow would require less, approximately 9-inches. To reduce ponding and allow additional sloshing height, the Tank will include a 36-inch pipe.

2.2.3 Washdown Piping

A 1-inch Schedule 80 PVC pipe will be provided from the upstream side of the inlet isolation valve to the exterior and interior of the Tank. By providing the connection at this location, the Tank can be isolated and water can still be provided to this connection via the winter fill piping described in **Section 3.3**. The pipe will terminate at a hose bib in both locations. The connection will allow for wash down water to be supplied during tank maintenance and supply a temporary pump for irrigation of the landscaped berm. A power supply for the temporary pump will be provided at the PLC panel described in **Section 6**.

2.2.4 Sample Piping

The Tank will include four sampling points that will be drawn from four separate Tank locations, and at various elevations. The sample pipelines will be placed on top of the floor slab and then through the tank floor slab to avoid additional wall penetrations. The pipelines will consist of ¾ inch diameter stainless steel tubing. The four sample lines will terminate adjacent to the Tank. The sample pipelines will include labeling and isolation valves to provide for sampling by Operators.

The routing of the sample piping is illustrated on Drawing C.06 of **Appendix C**.

In addition, the layout will allow for future addition of a chlorine analyzer. To provide for this future use conduits will be installed for power and controls, and a drain will be included for sampling. Due to the chemical discharge of the future chlorine analyzer, a drain pipeline will be provided from the Tank to the existing sewer along the WFP access road.

2.2.5 Leak Detection Piping

An under drain system will be provided to detect any leakage through the floor slab or floor joints.

2.2.5.1 Under Drain System for Tank Leakage

The under tank leak detection system has two main components. The first component is under the tank and the second component is outside of the tank footprint (but buried).

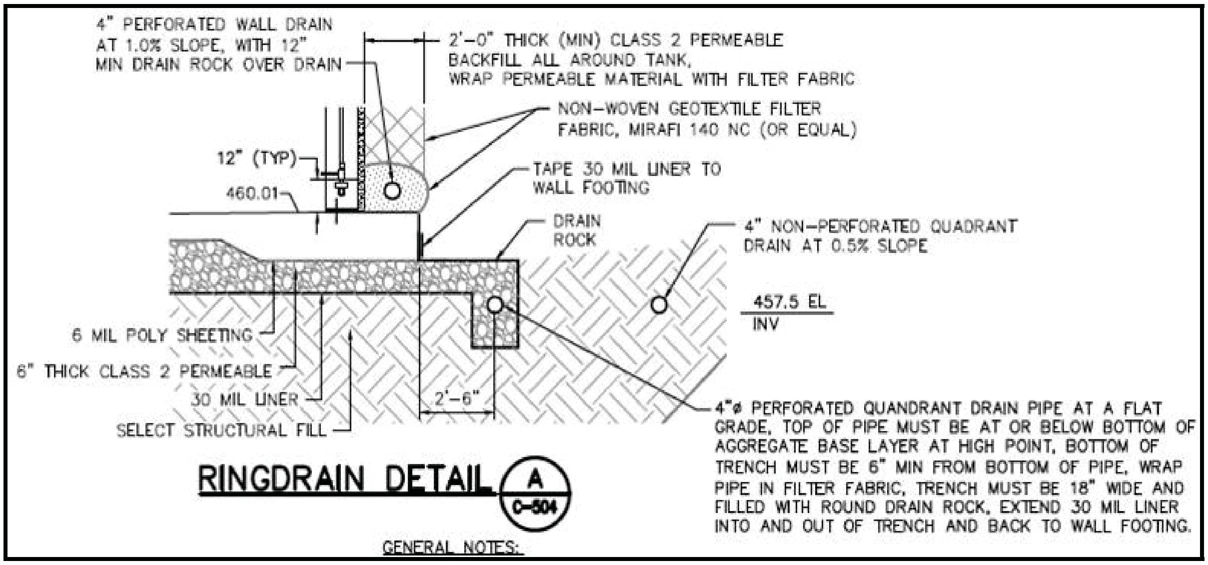
The under tank component create quadrants under the tank floor. Two inch diameter PVC pipe in the shape of a cross isolates each quadrant allowing the operator to identify where a leak in the floor is coming from. On top of the PVC pipe is a liner which outside delivers water to the Tank perimeter.

The outside portion of the leak detection system will consist of four separate perforated pipes bedded in gravel and filter fabric. These four pipes will collect water from the under tank system and route the water to the Reservoir via a small concrete headwall. The headwall will be marked with each quadrant name (i.e. NW – North Tank).

2.2.5.2 Under Drain System for Wall Leakage

If the Tank is backfilled, a leak detection system is recommended to monitor tank wall leakage. This leak detection system will consist of a vertical drainage panel placed around the Tank wall perimeter. Water collected from this panel will be routed to a perforated pipe bedded in gravel and filter cloth that rests on the Tank footing. This pipe will be placed adjacent to the under tank piping system outside of tank footprint. The backfill leak detection system will also be directed to the headwall as described in **Section 2.2.5.1**. A sample detail of the leak detection system is provided as **Figure 2-1**.

FIGURE 2-1
LEAK DETECTION SYSTEM DETAIL



2.2.6 Baffling

The Tank will operate with a continuous flow, which is unlike most of the District's distribution system reservoirs which have a diurnal flow pattern (fill during the night, and empty during the day). With this type of use, the internal design of the Tank should include provisions to avoid short circuiting of the inflow which can result in water quality impacts. The use of internal baffling can mitigate this water quality concern by controlling hydraulic retention time.

Table 2-3 provides a summary of the analysis conducted for two types of internal tank baffling systems.

TABLE 2-3
COMPARISON OF BAFFLING METHODS

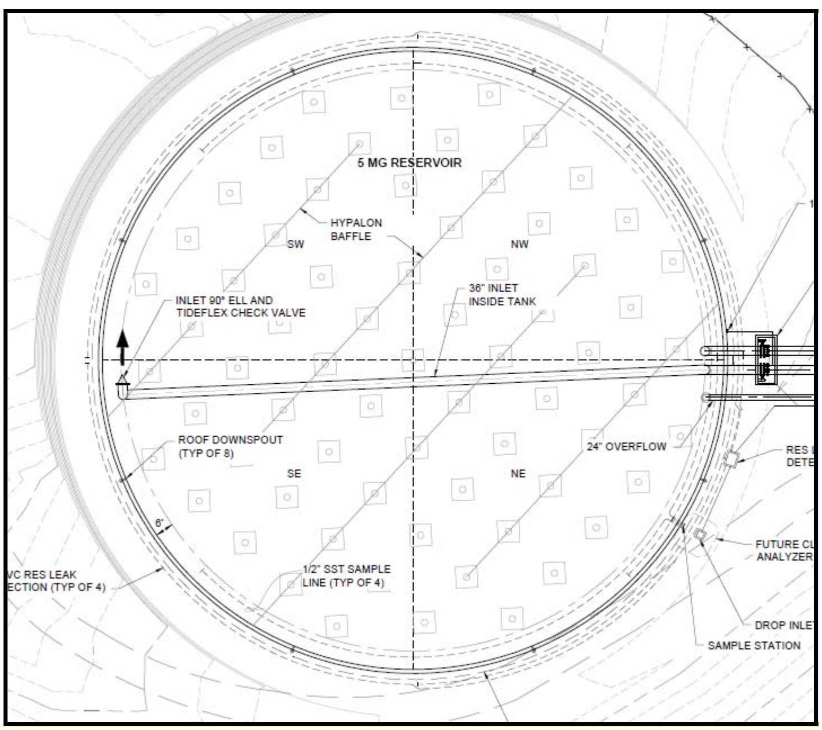
Type	Benefit	Challenge
Concrete baffling walls	• Potentially increase Baffling Factor from 0.5 to 1.0[a] • More durable and reduced long term maintenance costs	• Increased cost[c] • Increased construction schedule time, due to form work requirements[e] • More complex and labor intensive construction[f] • Lost storage of approximately 0.16 MG
Hypalon curtains	• Baffling Factor of 0.5(a) • Lower cost(b) • Proven approach (same used in RW Tank) • Flexibility for future modifications	• Potential O&M issue should fasteners fail • Useful life estimated to be greater than 20-30 years[d]

	• Less construction schedule impact [e]	

(a) EPA Guidance Manual Disinfection Profiling and Benchmarking (Table 3-2, Baffling Classifications and Factors)
(b) Hypalon Baffle costs are approximately $375-$400/lineal foot installed. Approximately $215,000 at $385/LF.
(c) Concrete baffle walls for the Tank are estimated at $800,000 - $850,000. Costs for straight baffle walls are estimated to be - $54 per square foot of surface area and $268 per lineal foot of wall footing. Curved baffle walls - $59 per square foot of surface area $268 per lineal foot for wall footing. Total Cost for the Tank = $860,000.
(d) Current estimates of Hypalon service life are estimated at a 25 year life span. There are several projects out there (reservoirs) where existing hypalon baffles are showing minor wear at 15 years. There is no documentation that AECOM is aware of where the existing length of services is greater than 15 years (source DN Tanks).
(e) Concrete baffling constructions adds three (3) weeks time to the completion time of the tank construction. Hypalon add approximately two (2) weeks to the overall schedule. In addition, concrete baffle walls complicate roof shoring efforts as the baffle walls are constructed prior to the roof shoring system installation.
(f) Based on a baffle wall height of 23 feet, preliminary baffle wall and footing sizes are estimated to be 18-inch thick, with reinforcing steel, and a footing that is 18-inch high and 5-foot wide.

Based on the analysis provided in **Table 2-3,** the Tank design will include hypalon curtains anchored between the roof columns, supported by a stainless steel anchoring system into the floor. The hypalon will be NSF Standard 61 certified. **Figure 2-2** illustrates the baffle design layout, using hypalon curtains, and the preliminary column layout for the Tank.

FIGURE 2-2
PRELIMINARY BAFFLE LAYOUT



The Tank will include four (4) baffle walls, each at approximately 20 feet tall. The total linear distance of baffle walls is approximately 560 feet, equating to an area of 11,200 square feet. The width of the created channel is 20 feet and the approximate centerline path of the configuration provides for a total travel length of 780 feet. Based on the design illustrated in **Figure 2-2**, the baffles will provide a relatively even height to width cross section.

2.3 Location

The general location of the Tank was determined as part of the Project Alternatives Study for the 1235-ft Backbone Improvements (October 2009) and subsequent evaluations by the District. With the elevation and diameter of the Tank fixed, the placement of the Tank was dictated by identifying the location which limits blasting, maintains the existing access road, and provides ample space for construction and a berm for screening. The location selected is illustrated in Drawing C-02 in **Appendix C**.

Although the location limits the quantity of blasting required, approximately 14,700 cubic yards of material must still be excavated. To minimize cost and traffic impacts, the excavated material will be used to overlay the existing embankment on the east of the site and also the berm. This buildup on the embankment will create a level dry storage area for use by the Contractor during construction, which is needed due to site constraints. Any balance of material will be used to further expand this proposed "rock fill" area or for final grading. A summary of the calculated cut/fill based on the Tank location shown in Drawing C-02 and described above is summarized in **Table 2-4**.

TABLE 2-4
PRELIMINARY CUT/FILL ESTIMATES

Activity	Volume (CY)[a]
Tank Excavation	14,700
Berm	5,500
Rock Fill Area	8,200
Final Grading	300
Net	**700**[b]

(a) Shrinkage of earthwork not considered because of existing rock. No subsidence has been included, as site is rock and should not subside. Swelling of earthwork may be possible as rock is broken into fragments and the additional void space may cause swelling.
(b) Net cut will be added to "Rock Fill Area", resulting in balanced earth work.

2.3.1 Tank Backfill

Backfilling the Tank or not backfilling has costs, maintenance and aesthetic implications which are evaluated in this section. A summary of the analysis conducted is provided in **Table 2-5**.

TABLE 2-5
COMPARISON OF BACKFILL OPTIONS

Type	Benefit	Challenge
Backfill	• Easier access to top of reservoir • Minimize need to maintain coatings at covered portion • Potentially provide a more transitioned landscape (aesthetic)	• Addition materials testing • Cost of backfill placement • Tank sliding considerations (requires keyway) • Backfill settlement • Vertical leak detection system required • Requires temporary storage of rock fill and additional labor to move twice
No Backfill	• More cost effective • Shorter construction schedule	• Safety fencing long term to protect slope • Long term maintenance costs of Tank coating • Paving around the tank • Slope stability concerns • Potential graffiti target

As noted in **Table 2-5**, the option to not provide backfill has a cost benefit. The items contributing to this additional cost are as follows:

- **Backfill** - Preliminary earthwork calculations indicate approximately 3,000 cubic yards of backfill are required to bring the site grade back to the existing grade around the south and west Tank perimeter. This cost is estimated at approximately \$5 - \$8 per cubic yard or approximately **\$15,000 - \$24,000** total. Time to complete this backfill is estimated to be 15 working days (three weeks) added to the overall Construction Schedule. This has a direct cost increase for additional field management and testing time.
- **Vertical Leak Detection System** - A vertical leak detection system is recommended for backfilled tanks, the cost is estimated at approximately **\$25,000**.
- **Keyway** - Backfilling the tank to the original grade will increase the sliding potential of the Tank. Differentials in backfill height of eight feet or more create forces that can cause the Tank to slide when the Tank is near empty, simultaneous with a seismic event. If the Tank is backfilled to the original line and grade, a keyway (deepening of concrete footing) or extended (horizontal footing) will be required to mitigate this sliding. These extra costs are estimated to be approximately **\$30,000** (source DN Tanks). Not backfilling the tank reduces the sliding potential where adequate friction is developed between the Tank and rock.

There are additional costs associated with not backfilling, such as roof access (ladder), access paving and additional drainage. However, based on the analysis presented in **Table 2-5**, the design will not include backfilling of the Tank.

2.4 Design Standard

The AWWA D110 Standard for Prestressed Concrete Water Tanks outlines minimum design parameters for four alternative types of prestressed tanks depending on the type of construction and circumferential prestressing methods employed. These four types of tanks are:

- Type I: Cast-in-place concrete with vertical prestressed reinforcement.

- Type II: Shotcrete with a steel diaphragm.
- Type III: Precast concrete with a steel diaphragm.
- Type IV: Cast-in-place concrete with a steel diaphragm.

Two types (Type I and Type III) of the above listed AWWA D110 Tanks are being constructed in the high seismic zone areas of the Western U.S. and are the subject of this opinion of comparison which is described in the following sections. (Tank Types II and IV are not considered due to the lack of local construction experience.)

2.4.1 Type I Tank: Cast-in-Place Concrete with Vertical Prestressing

The following general observations are noted for Type I Tanks:

- **General Wall Construction**: Reinforced cast-in-place concrete, usually 10 to 12 inches thick, with shotcrete exterior coating. Mild strength steel reinforcement is contained within the cast-in-place walls. All wall joints, spaced between 24 and 32 feet on center, contain cast-in-place rubber water stop. Horizontal mild steel reinforcing is continuous through the vertical wall joints. The interior surface of the perimeter wall typically has a steel form finish which minimizes surface contamination and aids with cleaning.
- **Vertical Pre-stressing:** The vertical high strength pre-stressing steel rods are tensioned to install a residual compressive force in the vertical direction. This is to avoid vertical tension and development of horizontal cracks in the concrete core perimeter wall.
- **Horizontal Pre-stressing:** Pre-stressing cables consist of seven-wire galvanized strands. The cables are wrapped horizontally around the cast-in-place walls with a computerized controlled reel/drum type wrapping machine which automatically tensions the cable to specified pre-stressing forces. An instantaneous visual readout and hard copy of the critical cable tensioning stress is available.
- **Shotcrete Cover**: Shotcrete, wet mix, is applied by automatic control equipment mounted on the same equipment as the automated wrapping machine. The shotcrete application is directionally controlled for enhanced coating of pre-stressing cable. Minimum cover over pre-stress strands is 1.5 inches.
- **Roof Structure**: Various roof structure types may be accommodated. Usually, a cast-in-place flat roof (with joints doweled and waterstopped) supported by columns is provided. Domed roof systems may also be provided.
- **Floor Slab**: The AWWA D110 discusses floor designs as either a “structural floor” or “membrane floor”. This standard also references ACI 318 and ACI 350R relative to reinforcing steel placement and minimum clearances. Type I tanks call for a structural floor slab of 6 to 8 inches with a minimum of No. 4 bars each direction. This floor slab has a minimum concrete cover thickness over the reinforcement of 3 inches to the ground and 2 inches to hydraulic surfaces. This standard conforms to ACI 350R for Environmental Engineering Concrete Structures for minimum reinforcement cover.
- **Perimeter Wall to Flooring Detail**: The Type I tank utilizes a continuous cast-in-place water stop between the perimeter wall-to-footing flexible joint for positive water tightness. The perimeter wall is seismically connected to the footing with embedded seismic cables cast in place to both the footing and wall. The seismic cables provide

a positive connection and transfers the tank lateral seismic forces from the wall to the footing.

2.4.2 Type III – Precast Concrete Wall Panels (Site Cast) with Vertical Steel Diaphragms

The following general observations are noted for Type III Tanks:

- **General Wall Construction:** The precast wall construction consists of a corrugated steel panel with 4 inches of concrete. A shotcrete exterior coating is applied after wall panel erection. The precast wall panels (8 to 12 feet wide) are erected and joined with a vertical cast-in-place closure pours approximately 10 to 15 inches in width. The joint is covered with a 10 gauge steel panel skip welded to the precast steel panel and sealed with a polysulfide sealant.
- **Vertical Pre-stressing:** Not required. Steel diaphragm stops water.
- **Horizontal Pre-stressing:** Nongalvanized high strength pre-stressing wire (6 to 9 gauge) is wrapped around the tank using a “pull through” type die/brake system. This system operates on the principal that when sufficient tension in the wire is developed, the wire will neck down and pass through the die. Tension in the wire is checked with a harping device which checks the tension in the wire by either a sonic resonance or deflection pull force. The tension in the wire is checked after wrapping is completed.
- **Shotcrete Cover:** Shotcrete is applied by hand-held application similar to standard shotcrete methods. Minimum cover over pre-stress wire is 1.0 inch.
- **Roof Structures:** Various roof structure types may be accommodated. Usually a 3 inch thick pre-cast segmental dome roof without waterstop and without columns is supplied.
- **Floor Slab:** The normal floor slab is 4 inches thick (membrane floor) and has No. 4 bars each way centered in the slab with 1.5-inch clear to ground or hydraulic surfaces.
- **Perimeter Wall-to-Footing Detail:** The Type III flexible joint at the wall-to-footing connection utilizes a cast-in-place water stop in the footing. A concrete curb is cast at the interior to the wall base only by the adhesion with a second waterstop to the wall panel. No cast-in-place water stop is provided between the ringwall footing and wall. Seismic cables (between the wall and footing) are cast into the footing and are attached to the precast wall by shotcreting.

2.4.3 Summary and Recommendations

Table 2-6 provides a summary and comparison of Type I and Type III tank design.

TABLE 2-6
TYPE I VS TYPE III TANK DESIGN

Description	Type I	Type III
Wall Construction	• CIP reinforced concrete (usually 10-12” thick) • CIP water stop • Continuous horizontal steel	• Precast corrugated steel panel with 4” concrete

Vertical Pre-stressing	• Vertical pre-stressing steel rods	
Horizontal Pre-stressing	• Galvanized cable • Automatic cable tensioner	• Non-galvanized wire • Pull through die for tension
Shotcrete Cover	• Wet mix prior to cable • Machine applied; 1.5" cover	• Standard shotcrete hand held application; 1" cover
Floor	• Structural floor typically 5" or greater (3" clr to soil and 2" clr to wtr)	• Structural floor typically 4" or greater
Floor to Wall	• Flexible joint with continuous waterstop • Seismic cables cast in footing/wall, to roof	• Flexible joint; waterstop is related to concrete adhesion • Seismic cables cast in footing and attached to wall during shotcreting

Although the two tank types discussed above and summarized in **Table 2-6** come under the same AWWA Standard, there are considerable differences between the two types of pre-stressed concrete water tanks, including:

- Type I utilizes galvanized cables versus uncoated wire used for Type III.
- There is no vertical compressive force in a Type III tank.
- Wall joints in a Type I tanks have positive water stop, cast in the concrete.

Most importantly, there is considerably less experience with a Type III tank in Seismic Zone 4. The behavior of the Type III tank is not as well known as Type I in Zone 4. While the cost of a Type III tank may be less than a Type I tank, behavior during a major earthquake and overall reliability for a Type III tank as compared with a Type I tank has not been established. This is significant for the District's project since the site is located over an existing Dam. Providing a safe and reliable tank design in this area is of paramount importance. Based on this difference and those noted above, the Tank design will be based on AWWA D110 Type I.

2.5 Aesthetics

The Initial Study/Mitigated Negative Declaration for the Backbone Improvement Project (October 2009) included several requirements related to aesthetics for the Tank that are summarized as follows:

- Engineered berm will provide visual screening for a portion of the Tank
- Berm shall be planted with native vegetation (requires Vegetation Restoration Plan).
- Earth-tone colors and materials will be used to encourage the Tank to blend into the surrounding backdrop
- No large expanses of glass or other reflective materials will be used
- Low-level nighttime security lighting (use motion-sensor) will be provided
- Lighting shall be shielded so as to avoid spill-over onto adjacent land uses

The design elements noted above will be incorporated into the detailed design. The Vegetation Restoration Plan is included in TM No. 5, Landscape Concepts (**Appendix B**). In addition, the aforementioned study included two renderings intended to illustrate the visual impacts from the Tank project, included as **Figure 2-3**. As part of the 50 percent design effort, these renderings will be updated as needed to reflect the updated design details.

FIGURE 2-3
INITIAL RENDERINGS (TO BE UPDATED AT 60%)

Source: Figure 15 and 16 from the LVMWD Backbone System Improvement Project – Initial Study/Mitigated Negative Declaration (2009)

2.5.1 Tank Roof

Based on discussions with District staff (March 20, 2013), the following features will be included in the Tank roof design:

- Ladders – two (2) ladders will be provided, one by the inlet and one by the outlet. Final placement will take into account aesthetics.
- Finish – Tank roof will be white concrete. An additional coating (hot mop and pea gravel) can be added at a future date if deemed necessary.
- Access – two (2) access hatches, minimum 5 foot x 5 foot, will be provided. One by each ladder. No intrusion switches will be provided but a locking feature will be included.
- Drainage - scuppers will connect to galvanized down drains that will convey run off away from the Tank to the proposed onsite curb and gutters. These curb and gutters will direct run off to the storm drain system that will be directed to the Reservoir.

2.6 Integration

The following section outlines the current operation of the FWR and the proposed integration of the Tank with the WFP.

2.6.1 Existing FWR Operation

The FWR is a concrete structure (rectangular, covered) located at the north end of the Filter Building. **Table 2-7** provides a summary of the FWR characteristics.

TABLE 2-7
FWR CHARACTERISTICS

Description	Units	Value
Wall Height	feet	18
Dimensions	feet	51'-10" x 67'-6"
Overflow	feet	15.25
Volume at Overflow	gallons	386,000
Pipe Inlet/Outlet	inches	30
Weir Height	feet	10
Finished Floor Elevation	feet	1071.0

Inflow enters the FWR via a 30-inch pipe penetrating the floor near the southeast corner (FW effluent line from DE Vacuum Filter System). From the 30-inch pipe, the flow passes over a weir wall 10 feet high into the main portion of the basin (purpose of the weir wall is to provide a relatively constant discharge head for the DE Vacuum Filter Pumps).

The water level is monitored via a diaphragm type pressure transmitter which is connected to a three inch pipe through the FWR south wall; % water level is indicated and signal is indicated at the transmitter and recorded at the WFP SCADA system; high-high, low, and low-low level alarms are indicated and are adjustable. Water level is controlled by automatic control of the FW Pumps at the WPS.

In the automatic mode, sequencing of pump motor and engines on/off and changing of engines speed is performed automatically by the WFP SCADA system to maintain water level in the FWR within a preset control band. The following provides a bulleted list of the various alarm/control settings.

- 3.0 feet (Emergency Pump Shutdown)
- 5.0 feet (Alarm)
- 10.5 feet (Operating Level +/- 6-inches)
- 12.0 feet (WPS Pumps stepped on)
- 14.75 feet (Filters Hold)

2.6.2 In-Series Operation with FWR

The original concept outlined in the Project Alternatives Study for the 1235-ft Backbone Improvements (2009) included abandonment of the FWR and potentially replacing or modifying (VFDs) for the Vacuum Pumps to account for the additional elevation of the new Tank. Based on detailed review of this concept, the following issues were identified:

- **Additional cost** – although these efforts will be required for Phase 3, **Section 2.1** demonstrates that the Tank will meet the storage requirements without these efforts during the initial construction (Phase 2).
- **Efficiency** – By combining these additional efforts with the WFP expansion (Phase 3), all the work planned for the WFP can occur simultaneously, allowing for internal WFP systems (i.e. instrumentation, electrical, etc) to be "touched once". This minimizes duplicated efforts and reduces overall program cost.
- **Disinfection** – the current system provides for adequate contact time required for disinfection. Modifying the FWR would impact current disinfection efforts and require UV upgrades to occur as part of this project (Phase 2).
- **Operation** – Modifying the FWR would result in a greater impact to overall operational procedures and require more operator time to become versed in the new operation.

To avoid these impacts, Phase 2 design will utilize the FWR and new Tank in series until Phase 3 (Filter Plant Upgrades) is completed. **Table 2-8** provides a summary of the FWR volume, combined with the Tank volume, assuming an in-series configuration.

TABLE 2-8
FWR OPERATING PARAMETERS

SCADA System	Existing	Elevation (feet)	Tank Volume (gallons)			Difference (gallons)
			FW Tank	5 MG Tank	Total	
Low-low (alarm)	3	1074	79,000	2,115,000	2,194,000	79,000
Low	5	1076	131,000	2,585,000	2,716,000	52,000
Target level	**10.5**	**1081.5**	**275,000**	**3,877,000**	**4,152,000**	**144,000**
High	12	1083	314,000	4,230,000	4,544,000	39,000
High-high (alarm)	14.75	1085.75	386,000	4,876,000	5,262,000	72,000

(a) Based on FW tank dimension of 51'-10" x 67'-6"
(b) Finished Floor elevation of FW Tank is 1071.0 feet.
(c) Finished Floor elevation of 5 MG Tank is 1065.0 feet.
(d) Based on Tank dimension of 200 feet in diameter.
(e) Headloss or column loss is not accounted for in Table 2-8.

Although **Table 2-8** demonstrates that the FWR and Tank in-series operation meets the initial 3.5 MG *operational* volume requirement, the District could modify existing FWR set points to further increase stored volume. **Table 2-9** provides a summary of a potential modified control settings and the impact on stored water during normal operation.

TABLE 2-9
PROPOSED FWR OPERATING PARAMETERS

SCADA System	Proposed	El. (ft)	Tank Volume (gallons)			Difference (gallons)
			FWR	Tank	Total	
Low-low (alarm)	11.5	1082.5	301,000	4,112,000	4,413,000	3,003,000
Low	12	1083	314,000	4,230,000	4,544,000	131,000
Target level	**13**	**1084**	**340,000**	**4,465,000**	**4,805,000**	**261,000**
High	14	1085	366,000	4,700,000	5,066,000	261,000

High-high (alarm)	15	1086	393,000	4,935,000	5,328,000	261,000

(a) Based on FWR dimension of 51'-10" x 67'-6"
(b) Finished Floor elevation of FWR is 1071.0 feet.
(c) Finished Floor elevation of Tank is 1065.0 feet.
(d) Based on Tank dimension of 200 feet diameter.
(e) The FWR overflows through 5 openings spaced between the filtered water and raw water reservoirs. The 6" x 4' openings provide a high water elevation of 1086.25.

Both tables demonstrate that an in-series operation will provide the necessary volume required to meet the storage deficit identified by the District.

2.6.3 Seasonal Tank Operation

To meet the storage deficit the new Tank is required to have adequate volume all year. However, since the WFP only operates a portion of the year (summer) an alternate supply will need to be available to fill the Tank during the off period (winter). To address this seasonal change, the following operation modes are noted:

- **Summer** – WFP is be operational, and Tank will need to maintain the required storage. No additional CT is anticipated in Phase 2. FW will flow from the FWR into the Tank and then down to the WPS for distribution to the Backbone System.
- **Winter (Reservoir Fill)** – WFP will be offline and the District will be filling the Reservoir as is presently operated, through the connection at the WPS. To provide water to the Tank and maintain water quality, a new connection to the onsite domestic water pipeline, which is described in **Section 3.3**, will supply water to the Tank. The water will be supplied at a constant flow, approximately 2 MGD, which will result in water flowing through the Tank overflow into the Reservoir. This additional flow must be accounted for in the District's regular Reservoir fill operations.
- **Winter (No Reservoir Fill)** – When the WFP is offline and the District is not filling the Reservoir, the Tank will be filled similar to the Winter Fill scenario except the Tank filling will be stopped short of the overflow and partially drained by pumping the water into the distribution system via the WPS. The frequency of the filling/draining will be dictated by Tank water quality and Equestrian/Morrison tank levels.

2.7 Site Civil

The following section provides an overview of miscellaneous site improvements related to the Tank construction.

2.7.1 Site Paving

AECOM recommends that the access to and around the Tank be paved. There are several benefits to paving the access road and Tank perimeter, which are noted as follows:

- Reduce construction dust
- Reduced erosion potential during and after construction
- Clear delineation of site boundaries
- Pave around the tank to provide good grade and drainage control.

AECOM recommends that the Contractor pave the access across the dam in the initial mobilizing of the Contractor's work. Drawing C-01 of **Appendix C** illustrates the extents of the paving area. Once the site improvements are complete and the Tank is in service, the

Contractor will cap (overlay) the road across the dam and complete the remainder of site paving.

2.7.2 Site Storm Drain

AECOM investigated directing site run off to the existing storm drain system in the lower access road near security gate and also to the Reservoir. A summary of the analysis is provided:

- **Directing Run off to Existing Storm Drain System (to offsite facilities)** - AECOM investigated directing run off to the existing storm drain system. The Construction to complete this would encroach over the existing saddle dam south leg. It is not anticipated it would encroach into the structural support of the dam. In the developed condition, if run off is directed to the offsite storm drain system, the developed flow may slightly increase from the amount of runoff directed there today causing potential impacts. This is because the time of concentration to develop run off would increase and the developed flow is therefore slightly larger. The Initial Study/Mitigated Negative Declaration (October 2009) discusses that there would not be any downstream storm water impacts, which may conflict with this approach. Additionally, this storm drain proposal may create concern with the DSOD as excavations for storm drain installation would encroach in and around the existing saddle dam.

- **Directing Run off to the Westlake Reservoir** - AECOM researched the MS4 permit requirements for potable water reservoirs. According to the Regional Water Quality Control Board (RWQCB), the MS4 permit does not limit the District's ability to allow run off from the project area to run into the Reservoir. In addition, a portion of the proposed project site already drains to the Reservoir. Based on the 2010 Sanitary Survey, the Reservoir has a tributary watershed of approximately 575 acres; the project site represents only a small fraction of the surface area draining to the Reservoir.

Based on the requirements of the Initial Study/Mitigate Negative Declaration, requirements of the MS4 Permit and the cost savings provided by continuing drainage of the area to the Reservoir, AECOM recommends that onsite grading be designed to drain to the Reservoir. In addition, AECOM recommends that some level of long term treatment be provided via a catch basin insert or other treatment mechanism. This will provide some level of pretreatment prior to water entering the potable water reservoir. This approach will be reviewed with Department of Public Health prior to the 60 percent design submittal.

During Construction, a Storm Water Pollution Prevention Plan (SWPPP) to contain and treat the contaminants during construction (mostly PH and turbidity) will be required as part of the Contract Documents. This will be prepared and implemented by the Contractor. The District may be required to upload the storm water reporting to the RWQCB if the Contractor's SWPPP shows that impacts to downstream drainage facilities (i.e. Torchwood Place) will occur.

Section 3: Conveyance

As noted in Section 2, the Tank will operate in series with the FWR requiring that a new pipeline be constructed connecting the FWR and Tank, and subsequently, the Tank to the WPS. In addition, a means of filling the Tank during the winter period will be required. These pipelines and the corresponding hydraulics are addressed in **Section 3**.

3.1 Hydraulics

Although the previous section indicated values for available storage within the FWR and Tank, these estimates were based on static conditions. In operation, the system hydraulics will result in hydraulic head loss and a corresponding reduction in actual stored volume in the Tank. The following analysis is used to determine the minimum pipe sizing required for the Tank inlet/outlet piping.

Detailed graphs illustrating hydraulic grade lines for the following scenarios are included as TM No.1, Hydraulics and Models (**Appendix B**):

1. Plant Flow 20.4 MGD, 30-inch pipe, three (3) FWR elevations
2. Plant Flow 18 MGD, 30-inch pipe, three (3) FWR elevations
3. Plant Flow 15 MGD, 30-inch pipe, three (3) FWR elevations
4. Plant Flow 20.4 MGD, 36-inch pipe, three (3) FWR elevations
5. Plant Flow 18 MGD, 36-inch pipe, three (3) FWR elevations
6. Plant Flow 15 MGD, 36-inch pipe, three (3) FWR elevations

Table 3-1 provides a summary of the impacts each scenario has on the Tank storage volume.

TABLE 3-1
TANK INLET/OUTLET PIPELINE SIZING

New Pipe Diameter (inches)	Velocity (ft/sec)	Plant Flow (MGD)	Tank Volume (MG)			Feet above FW (E) Top of Pipe		
			FWR @ 15.25 ft	FWR @ 14 ft	FWR @ 8 ft	FWR @ 15.25 ft	FWR @ 14 ft	FWR @ 8 ft
30	6.4	20.4	2.1	1.8	0.4	-1.6	-2.9	-8.9
	5.7	18	2.7	2.4	1.0	2.9	1.7	-4.3
	4.7	15	3.4	3.1	1.7	7.9	6.6	0.6
36	4.5	20.4	3.3	3.0	1.6	8.2	7.0	1.0
	3.9	18	3.7	3.4	2.0	10.6	9.4	3.4
	3.3	15	4.1	3.8	2.4	13.3	12.0	6.0

Table 3-2 provides an overview of the costs associated with various pipe diameter alternatives for the main pipeline (2,260 feet).

TABLE 3-2
PIPE COST COMPARISON

Diameter (inches)	Material Price ($/LF)	Labor Price ($/LF)	Pipeline Cost ($)[a]	Valve Cost ($)	Total Cost ($)
30	171	214	869,535	40,000	909,535
33	186	233	945,810	52,000	997,810
36	203	254	1,032,255	64,000	1,096,255

(a) Costs based on discussions with Northwestern Pipe Company (February 2013).

The following analysis is based on the hydraulic analysis and costs noted in the previous tables:

- The in-series approach will not be capable of meeting either the short-term or long-term storage requirements after plant expansion (Phase 3). Thus, the plant expansion requires the bypassing of the FWR and pumping directly from the vacuum pumps to the Tank.
- A 30-inch pipe solution could be viable if the operating range for the FWR is adjusted. However, this approach provides limited flexibility in operation.
- The additional cost of upsizing from a 30-inch ID to 36-inch ID concrete lined steel pipe is estimated at approximately $200,000 for 2,000 feet of pipeline.

The 36-inch pipeline alternative provides additional short-term storage, flexibility in timing of implementing Phase 3, and reduced energy costs. The Tank inlet and outlet pipes will be designed using a 36-inch diameter pipeline.

3.2 Inlet/Outlet Piping

Construction of the inlet and outlet piping requires consideration of several factors including the following items:

- Alignment
- Vertical Profile
- Pipe Bedding and trench section
- Cover
- Crossings

3.2.1 Alignment

Construction of the Tank inlet/outlet in the lower access road (south of the WFP) is not preferred for several reasons. Existing infrastructure in this road consists of an 8-inch sanitary sewer, underground electrical conduits and an existing 12-inch storm drain. North of the 8-inch sewer there is minimal room to construct both pipelines. Additionally, sewer and water separation criteria could not be met without significant costs in protecting the proposed water mains (i.e. casing).

Following the upper access road requires only that the utilities in the lower access road be crossed not paralleled. The crossings that are anticipated are as follows:

- 18-inch Storm Drain (go over)
- 10-inch Sanitary Sewer (go over)

- Underground Electrical (should be potholed, most likely go under)

Based on this analysis, the inlet and outlet pipe alignment is located along the upper access road.

3.2.2 Vertical Profile

The vertical profile of the pipeline is constrained by three main factors. The reservoir finished floor elevation (1065), elevation of dam (the lowest point on profile) and the connections at the WFP. Air vacuum valves will be placed at the high points of the pipeline (at WFP). These appurtenances will allow air to enter and leave the pipelines. Blow off valves will be placed at the low point to flush the pipeline (at the dam).

3.2.3 Pipe Bedding and Trench Section

Horizontal placement of the pipelines can be installed either in a parallel trench or on top of each other. The parallel configuration will provide ease of access for maintenance and should provide for a shorter construction duration. Additionally, vertical placement further complicates utility crossings and the dam crossing. AECOM recommends that the pipelines be installed parallel and horizontally spaced to each other.

Both the inlet and outlet will be placed with approximately 8-inch to 12-inch of horizontal separation. The Contractor may excavate a larger trench at pipe connections to allow for welding inside the pipe. This narrow trench spacing reduces excavation costs. The pipes will be backfilled in cement slurry that should allow future ease of excavation. The slurry will eliminate the need for providing cathodic protection of the pipeline because the pipe will be protected by mortar (lining) and the cement slurry backfill.

3.2.4 Cover

Cover over each of the pipelines will vary but will not be less than three feet to reduce live loading over the pipeline. Cover at the reservoir will be a minimum of three feet below the structural section of the reservoir slab (approximately four feet of cover below Tank finish floor). Cover over the pipeline in drive areas will be a minimum of 3 feet. The overall trench depth is expected to be seven feet +/- in order to provide three feet of cover and bedding support for the pipelines. Using cement slurry to backfill the pipeline will reduce impacts to the existing access road and will reduce the overall construction duration.

3.2.5 Dam Crossing

The saddle dam is regulated by the Division of Safety of Dams (DSOD). Two pipelines (one inlet and one outlet) will cross the existing saddle dam, which is approximately 450 feet in length. The pipelines will be placed on pipe supports spaced at 35 feet to 40 feet on center. Considerations of the pipe and pipe support design include the following items:

- Type of pipe support, cost, and DSOD approval
  - Place on cast in place concrete supports (13 locations)
  - Place on Piles (13 locations, 26 piles)
- Casing Installation - Protection of the saddle dam in the event of pipe failure
- Pipe Material
- Seismic restraint of pipe
- Draining of pipeline

- The high water line of the Reservoir
- Thermal expansion
- Cathodic protection

AECOM has completed preliminary discussions with the DSOD regarding the proposal to construct the above ground pipe supports across the saddle dam. Above ground dam pipe crossings have been proposed and approved by the DSOD before in the State of California. The two proposed pipe support methods are further detailed in the following sections.

3.2.5.1 Cast In-Place Pipe Supports

Cast in place pipe supports would be placed on the surface of the existing east face of saddle dam with minimal disruption to the existing dam surface. The pipe supports would consist of a reinforced concrete support, approximately 11 feet wide, 18-inches thick and would require a pad footing (width to be determined). A shear key to counter horizontal loads is required.

Construction of the pipe supports is not anticipated to present a challenge to the Contractor because most of the work is done at the pipe support location without disrupting traffic across the dam. This would assist the Contractor in scheduling activities. Risks to the District during construction of the concrete pipe support would be if the Contractor spills concrete into the Reservoir.

Additional information that would be required to complete the design of the concrete pipe support would be passive earth pressures from the project Geotechnical Engineer for the shear key. The shear key provides resistance to lateral movement of the pipe and pipe support during a seismic event. Questions remain as to how deep the shear key needs to be and how much friction would develop underneath the pad footing. In order to provide this information, additional geotechnical sampling may be required.

3.2.5.2 Pile Supports

Pile supports would consist of a circular steel pile or an H Beam (both steel) as described in Fugro's Preliminary Geotechnical Engineering (**Appendix D**). The total depth of each pile is estimated to be 30 feet. Construction of the piles across the saddle dam is anticipated to take a similar amount of time as those for the concrete pipe supports.

The preliminary design of the piles was based on using materials specified in the Contract Specifications for material gradations. AECOM has requested the Dam Construction Report from the District, which should detail the saddle dam construction. Additionally, due to the proximity of the pile adjacent to the high water line, the pile design was completed using a saturated condition, this reduces the overall friction resistance, which increases the length of pile. Piles are currently proposed to be pushed in and would push through several feet of the core dam material.

AECOM recommends the use of pile supports due to difficulties in meeting the required friction (sliding) for the concrete pipe supports. However, feedback from the DSOD will be the primary decision maker as to how or if the pipe is constructed across the dam.

3.2.5.3 Casing and Seismic Restraint

The inlet and outlet will be placed in a casing, for the portion crossing the saddle dam, to provide a primary containment vessel in the event either pipeline were to leak or rupture across the dam. The purpose of this casing is to monitor and control leakage from the pipelines in the event a seismic event ruptures the pipeline. The annular space between the pipe and carrier pipe will not be filled in. Because the existing filtration plant piping is steel, it is recommended to continue using steel pipe across the saddle dam. The casing would be constructed of steel as well.

3.2.5.4 Thermal Expansion

Thermal expansion of the above ground casing pipeline will be considered in the detailed design. Preliminary calculations indicate as much as three inches of expansion/contraction may be possible given temperature extremes of freezing (32 degrees Fahrenheit) to 110 degrees Fahrenheit in summer months. To mitigate impacts from this expansion/contraction, a restrained flexible coupling will be provided to allow movement of the pipeline. The carrier pipelines will be cooled due to the water within the pipelines. Thermal expansion will also need to be considered when the Contractor is making the connections at each end of the saddle dam and when the pipeline is taken out of service.

3.2.5.5 Cathodic Protection

Cathodic Protection of the above ground pipe, casing and piles (if piles are used) will be developed in the detailed design. There are two types of cathodic protection that can be used; these include Galvanized Bags (passive) and Impressed Current (active). AECOM recommends the use of galvanized bags as cathodic protection due to the reduced cost and effectiveness of this method of providing cathodic protection. In addition, the steel pipeline crossing the dam should be cement mortar lined and coated, and be installed inside a steel casing. The exposed exterior of the steel casing should be painted in lieu of mortar since damage from the sun may cause cracking of the mortar coating.

Based on discussions with District staff (March 20, 2013), the pipeline will not require cathodic protection but the pilings (if used) will be provided with cathodic protection.

3.3 Winter Fill Piping

As noted in **Section 2.6.3** a means of providing potable water to the Tank is required during winter operations, when the WFP is offline. Two alternatives were identified and include:

- **Alternative 1** – Connect to the existing 8-inch domestic water supply pipeline located at the northeast corner of the WFP site.

- **Alternative 2** – Provide a new connection at the WPS that would allow water to be supplied to the Tank via the existing 30-inch FW pipeline from the WPS to the Tank.

A summary of the hydraulics related to the two alternatives is summarized in **Table 3-3**.

TABLE 3-3
BYPASS ALTERNATIVES

Description	Units	Alternative 1 Existing 8-inch DW	Alternative 2 New WPS Connection
Length	feet	1685	1735
Max. Velocity[(a)]	feet/sec	9.9	5
Distribution HGL[(b)]	feet	1240	1240
Flow	gpm	1550	4900
Headloss	feet	30	2
Final HGL	feet	1210	1238
Volume per Day	MG	2	7
Tank Turnover	Days	2.2	0.7

(a) Table 3-7, Potable Water Master Plan Update 2007 (AECOM), notes 5 feet per second (new) and 10 feet per second (existing) for pipeline flow velocity, maximum.

(b) A pressure of 67 psi was noted at the existing domestic water connection during a site visit to the WFP on 2 February 2013. This corresponds to a HGL of 1227 feet. The difference of 13 feet from the HGL noted in Table 3-1 results in a final HGL of 1196 which is 111 feet more than the 1085 feet overflow elevation. Provides for additional losses from the meter, control valve and backflow preventer

Alternative 1 provides a 2.2 day turnover period and is considered adequate. AECOM recommends Alternative 1 for this phase. Alternative 2 should be re-evaluated during Phase 4 (WPS upgrades).

The new 8-inch connection will be routed from the existing 8-inch at the northeastern corner of the WFP. The connection will be made within the landscaped area and will include an automated valve, magnetic flow meter and backflow preventer, all located above ground in the landscaped area. The downstream piping will be routed along the access road and connect to the new FW inlet piping downstream of the new isolation valve. The preliminary layout of the piping is illustrated on Drawing P-05 of **Appendix C**.

Section 4: Future Considerations

As noted in **Section 1.1** the Backbone Improvements Program includes multiple phases, summarized as follows:

- Phase 1 - Pipeline improvements along the backbone pipeline (Agoura and Calabasas Pipeline Improvements projects)
- Phase 2 - 5 MG filtered water reservoir (Tank)
- Phase 3 - Expansion of the WFP from 15 MGD to 20.4 MGD
- Phase 4 - Upgrades to the WPS

This section addresses considerations of these future phases and the potential benefits or impacts that modifications during this phase may result in subsequent phases.

4.1 Phase 3 – WFP Expansion

As noted in the Project Alternatives Study for the 1234-ft Backbone Improvements (October 2009), the District intends to expand the existing WFP from the current 15 MGD capacity to 18 MGD, with a maximum production rate of 20 MGD. As noted in the aforementioned document this will have impacts on the current disinfection process, requiring additional CT (concentration x time) for disinfection.

Existing disinfection includes injecting sodium hypochlorite, with a target of 0.3-0.5 mg/L, at the RWR entry and then dosing again at the entry point to the FWR, at a target of 2.3 mg/L. The detention time in the FWR provides the required contact time (CT) to achieve the required 1 log of giardia inactivation required for DE treatment plants. After the CT requirements are satisfied, ammonia is added to convert the free chlorine to chloramines, and minimize the formation of additional disinfection by-products (DBPs). For chloramination, aqueous ammonia is injected as the water leaves the FWR.

The District uses a Wallace & Tiernan Penwalt residual analyzer (Micro/2000 Analyzer) for monitoring chlorine residual within the WFP. The following sampling points are noted:

1. Entry to the RWR (to be completed 2013)
2. Exit to the RWR
3. Entry to the FWR
4. Exit to the FWR (to be completed 2013)

Sample points 1 and 4 will utilize a multi-unit Wallace & Tiernan residual analyzer. Sample points 2 and 3 each have independent units.

For future disinfection, sample points will be required for future UV but can be accessed at areas of exposed pipe and do not require consideration for this phase, with exception to power/controls. Relative to continued chlorination in the future, a new sample point and ammonia injection point will be required downstream of the Tank. To accommodate this future consideration the following will be provided as part of this project:

- **UV Power/Controls** – conduit will be provided from the Electrical Room to the DE Storage area.

- **Ammonia Vault** – a vault will be provided downstream of the Tank outlet and will include a 1-inch sample pipeline and a 3-inch PVC pipeline for future routing of a ¾ inch ammonia tubing, consistent with the existing pipe sizing as shown on Sheet M-403 of the 2005 Disinfection Upgrade project. An outlet for ammonia injection and sample point along the existing 30-inch and within the provided vault will not be part of this phase.

- **Future Connection** – the 30-inch pipeline will be extended from the existing ammonia vault to the north side of the DE Storage area. The intent is to provide a future exterior connection for future piping and UV equipment to be located along the FWR side of the DE storage area (west wall).

4.2 Phase 4 – Westlake Pump Station Expansion

To use the higher capacity of the expanded WFP (Phase 3), modifications are also needed to the WPS (Phase 4). During this future phase the bypass modifications addressed in Section 3.3 can be provided which require additional modifications at the Tank site to allow for the bypassed flow to be routed from the Tank FW outlet (which is the connection at the WPS) to the Tank FW inlet.

To minimize costs of these future modifications the following are to be provided as part of this phase:

- Conduit sized for future power/controls of motorized valves required at pipe island.

- Two (2) tees and blind flanges at the 36-inch Tank FW inlet

- One (1) tee and blind flange at the 36-inch Tank FW outlet

- Space in site plan to accommodate future construction

Section 5: Structural Design Criteria

This section describes the structural and seismic design criteria for the new Tank.

5.1 Applicable Codes and Standards

This section outlines the primary documents that will be used for the structural design phase of the project. Where conflicts occur between two or more of the documents presented, the more stringent requirements will apply.

The strength, serviceability, and quality for materials and design procedures will meet the expectations of the following codes and standards:

- AWWA D-110, Type I;
- California Building Code (CBC), 2010 Edition;
- Applicable portions of the International Building Code (IBC), 2009 Edition;
- American Concrete Institute (ACI), ACI 318 - Building Code Requirements for Structural Concrete;
- American National Standards Institute/American Society of Civil Engineers (ANSI/ASCE) ANSI/ASCE 7 - Minimum Design Loads for Buildings and Other Structures;
- Code of Federal Regulations (CFR), 24 CFR Part 1910, Occupational Safety and Health Standards (OSHA);
- ACI 350 - Code Requirements for Environmental Engineering Concrete Structures;
- American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges - Vehicle and Traffic Loads;
- ACI 350.3 Seismic Design of Liquid Containing Concrete Structures and Commentary;
- Criteria for structural-seismic design obtained from the geotechnical evaluation completed for the project by the District (CBC Chapter 18); and
- ACI 301 Specifications for Structural.

5.2 Materials

Reinforced concrete will be used for the following purposes:

- 5 MG Tank
- Concrete Pipe Supports
- Concrete equipment pads.

All concrete materials will conform to the requirements of the ACI 318, ACI 350, and Chapter 19 of the CBC. Specific requirements for the project are as follows:

- Class C Concrete: Compressive strength (f_c) = 3,250 pounds per square inch (psi) for concrete fill, duct bank encasement, and where noted;
- Class A Concrete: f_c = 4,000 psi for all structural concrete, unless otherwise noted;
- Nominal maximum aggregate size: 1-inch (maximum, for design);
- Reinforcing Steel: American Society for Testing and Materials (ASTM) A706 or ASTM A615, Grade 60; and
- ASTM C 33: Appendix XI, Requirements for Testing for Potentially Alkali Reactive Aggregates.

Structural aluminum materials will be used at the tank structure. Stainless steel (SST) anchors will be used for structural connections to concrete. Type 316 SST will be used in general service areas subject to wash down, splashing, or precipitation.

5.3 Design Loads

Design loads for each structure will be in accordance with the applicable codes previously presented. The criteria below are a partial summary of the requirements.

Dead loads will consist of the weight of the structure and all equipment of a permanent or semi-permanent nature, including process equipment, piping, electrical wiring, and lighting.

Uniform and concentrated live loads will conform to ASCE 7 Table 4-1. Values from the tables will be applied in accordance with ASCE Chapter 2. Loadings for typical uses are listed below:

- Catwalks and Stairways: 100 pounds per square feet (psf) uniform load and 300-pound concentrated load. The concentrated load will be applied to the midspan of the grating or stair tread for calculation purposes;
- Equipment or Storage Room Floors: 250 psf uniform load unless stored materials justify a higher design loading;
- Roofs: 20 psf uniform load for non-concrete roofs and 50 psf for concrete roofs;
- Process Areas: 250 psf uniform load and data from specific equipment manufacturer for concentrated loads; and
- Unrestricted Vehicular Access: AASHTO HS-20 load.

Loads on any required vehicle barriers will conform to ASCE Section 4.4.

The structural/seismic design of facilities on this project will conform to the requirements outlined in the geotechnical report provided by Fugro (**Appendix D**).

Seismic design loads will be in accordance with the requirements of Chapter 12 of ASCE-7. The importance factor for elements of structures, nonstructural components, and equipment supported by structures (Seismic Importance Factor for Structures [I]), will be in accordance with ASCE Table 11.5-1. Site-specific coefficients S_s and S_1 will be based on the site-specific information from the geotechnical report.

Seismic design loads in water retaining structures will follow ACI 350.3-01.

Where pumps and pipes penetrate walls or slabs with embedded flange or collar, pipe thrust will be evaluated. The method of transfer of thrust load into the structure will be considered including local shear stresses, as well as overall stresses in the element. The greater of the maximum surge pressure and internal piping test pressure will be the basis for calculated thrusts during final design.

All structures and components thereof will be designed to sustain all dead loads and other loads discussed previously within the stress limitations of CBC Chapters 16, 19, 20, 21, and 22 of the CBC. Applicable loads will be combined using strength design or allowable stress design. Live loads will be arranged to cause maximum shear and bending moments along the member. Where applicable, both thermal expansion/contraction loads and assembly/erection loadings will be incorporated into the design.

Thermal loads due to change in temperature shall be considered in the design; such forces will include, but not be limited to, those caused by piping expansion or contraction.

5.4 Seismic Structural Design Approach

Structures will be designed to resist sliding and overturning due to loads induced by seismic forces. Resistance to sliding may include frictional resistance, as well as passive pressure on the opposite wall of the structure. A safety factor of 1.25 will be used for lateral load conditions that include seismic loads.

Structural elements, nonstructural components, and equipment supported by structures will be designed to resist a minimum lateral seismic force F_p as presented in Chapter 13 and 15 of ASCE-7.

The attachment and anchorage of the following equipment need not be detailed:

- Equipment components weighing less than 400 pounds and not located more than 4 feet above the roof or floor, and which have an importance factor of 1.0;
- Temporary equipment; and
- Equipment weighing less than 20 pounds and less than 5 pounds per foot.

Anchorage and other supports for equipment will be designed to resist seismic forces occurring at each of the three principal directions separately as well as simultaneously. The horizontal seismic loads in both principal directions and vertical seismic load will be combined. When including vertical loads results in a less conservative design, the vertical effects will be neglected. However, the combined effect of horizontal and vertical seismic loads will not be less than that defined in Section 13.3 of ASCE-7.

When the strength design method is used to analyze or design a concrete structure subjected to seismic loads, the sanitary durability coefficients presented in Section 9.2.8 of ACI 350 for environmental structures will be included.

5.5 Deferred Submittals

Specific equipment to be furnished by manufacturers will require submittal reviews. The submittal data supplied by the manufacturers will require seismic design analysis per ASCE Chapters 13 and 15.

5.6 Special Inspections

The project specifications will contain requirements for structural testing and special inspections per Chapter 17 of the CBC. The following elements of construction work will require special inspections:

- Earthwork;
- Structural concrete;
- Structural steel fabrication;
- Concrete reinforcement;
- Anchor bolt installation; and
- Foundation piles (if used).

Section 6: Electrical and Control Systems

A telemetry system will be provided for new and future monitoring and controls of the Tank and appurtenances. The telemetry system will consist of an Instrument Control Panel (ICP) and fiber optic network. The ICP will house a PLC system for monitoring and control, circuit breakers for site power, and fiber optic equipment for SCADA communications.

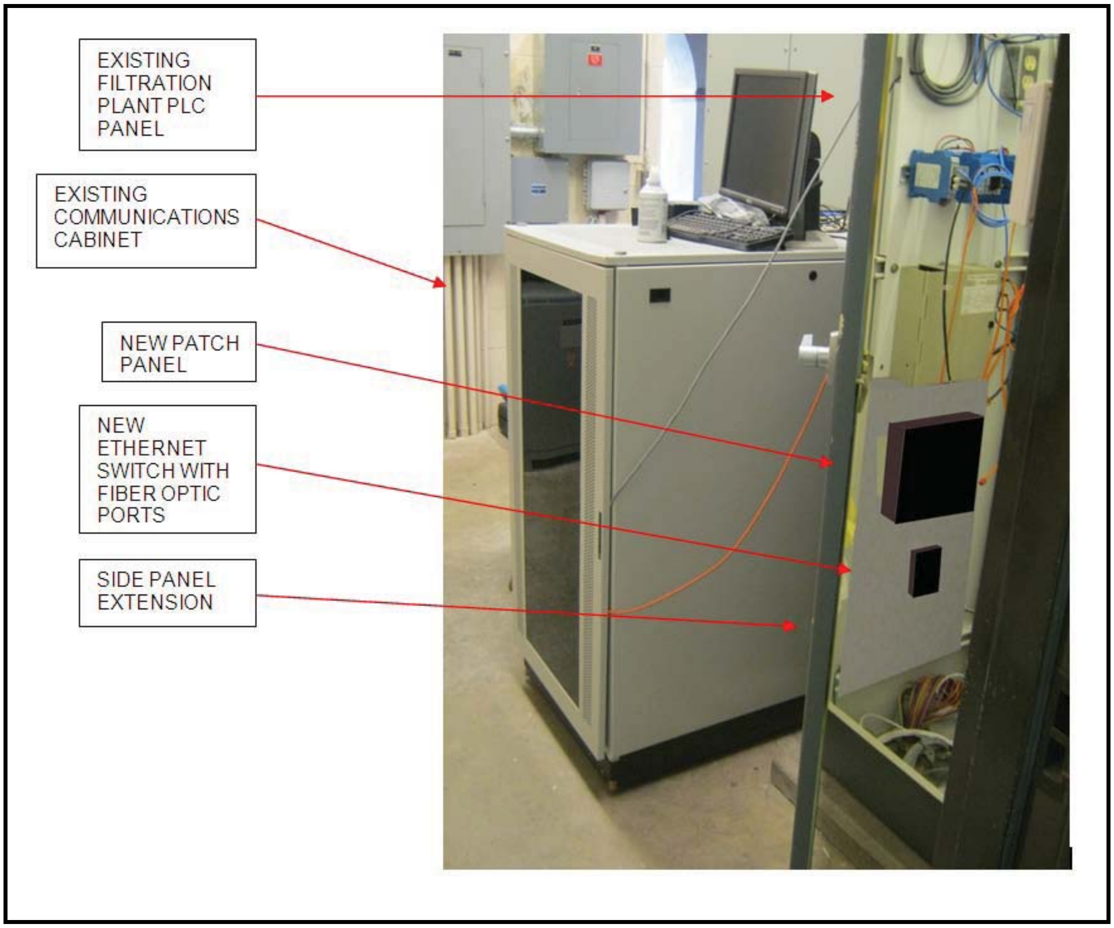
Ideally, the PLC would be identical to the existing Filtration Plant PLCs (Schneider Electric/Modicon Compact series). However, these have been discontinued therefore AECOM recommends providing a Schneider Electric M340 PLC at the Tank site. The M340 is a replacement model for the Compact PLC by the same manufacturer. The PLC will be provided with spare I/O and empty slots to accommodate future I/O requirements.

The ICP will be a freestanding stainless steel enclosure located adjacent to the reservoir. Since the site is small and power requirements are minimal, the ICP will contain additional circuit breakers for site lighting and a utility receptacle, for powering a temporary irrigation pump or washdown pump, in lieu of providing a separate electrical panel and receptacle. The receptacle will be accessible externally from the panel wall with a weather proof cover so that access inside the panel would not be required.

By providing a PLC with SCADA communications at the Tank site, it will allow greater versatility for control and monitoring of future site upgrades (motorized valves, chemical analyzers, security cameras, etc.) compared to running copper conductors with spares between the Tank site and the WFP. An Ethernet switch with fiber optic ports and fiber optic patch panel will be located inside the ICP. A 12-strand fiber cable will run from the ICP to the existing Filtration Plant PLC Panel inside the Filtration Plant where an identical Ethernet switch and patch panel will be provided. An Ethernet cable will routed from the Filtration Plant PLC Panel to the existing SCADA Ethernet switch, located in the adjacent Communications Cabinet, which will connect the new PLC to the SCADA network. Via the fiber system, the new PLC will interface with SCADA using Modbus/TCP protocol.

The proposed modifications to the Filtration Plant PLC are provided in **Figure 6-1**.

FIGURE 6-1
FILTRATION PLANT PLC PANEL MODIFICATIONS



AECOM proposes the new PLC be identified as PLC-50. Final identification of the new PLC will be determined during the detailed design. The new PLC will monitor the following parameters:

- Tank Level (pressure transmitter, including manual gauge)
- Check Valve Fully Closed Position
- Check Valve Vault Flood
- Check Valve Vault Intrusion

The operator will be able to view the above parameters via SCADA system. Control of any devices at the Tank site will not be required.

The Tank winter fill valve (located at the WFP) will be interfaced with the existing Filtration Plant PLC. During a site visit (5 February 2013), it was confirmed that the PLC has enough spare I/O to accommodate interfacing requirements. The PLC and SCADA will be modified to allow operator to command the valve open and close and also view the valve status.

Modifications to the existing PLC and SCADA system will be coordinated with the District.

Instrumentation devices will be open to qualified make and models meeting specification requirements. The specifications will be based off the following devices:

- Reservoir Level Transmitter – Rosemount Model 2088
- Flood Switch – Gems LS-270
- Ethernet Switch – N-Tron Model 305FX
- Fiber Optic System – Corning or Commscope

Drawings E-01 through E-06 and N-01 through N-03 in **Appendix C** provide additional detail to described approach to power and control systems.

Section 7: Preliminary Cost Estimates

An estimate of probable construction costs is summarized in **Table 7-1** and the complete version is attached in **Appendix E**.

TABLE 7-1
ESTIMATE OF PROBABLE CONSTRUCTION COST

Description	Qty.	Unit	Unit Price ($)	Total Cost ($)
General Conditions	1	LS	-	1,565,000
General Requirements	1	LS	-	297,000
Blasting	15,500	CY	40	620,000
Access Road	23,850	SF	9.25	221,000
Other Sitework	1	LS	-	383,000
Piling Over Dam	26	EA	6,500	169,000
Tank	0.69	$/Gal	5,000,000	3,450,000
Baffles	560	LF	375	210,000
36" Welded Steel Pipe	1,260	LF	255	321,000
36" Welded Steel Pipe & 42" Casing	1,000	LF	545	545,000
Slurry Backfill	1,294	CY	150	194,000
Pipeline Accessories	1	LS	-	456,000
Finishes	1	LS	-	38,000
Electrical	1	LS	-	138,000
Instrumentation	1	LS	-	46,000
			Subtotal	**8,653,000**
			Escalation (3% per year)	260,000
			Total	**8,900,000**

Note: Opinion of Probable Cost is only an estimate of possible construction costs for budgeting purposes. This estimate is limited to the conditions existing at issuance and is not a guaranty of actual price or cost. Uncertain market conditions such as, but not limited to; local labor or contractor availability, wages, other work, material market fluctuations, price escalations, force majeure events and developing bidding conditions, etc. may affect the accuracy of this estimate. AECOM is not responsible for any variance from this Opinion of Probable Cost or actual prices and conditions obtained.

The Project Alternative Study for the 1235-ft Backbone Improvements (October 2009) provided a conceptual level opinion of probable cost of $6.6M which escalates to $7.7M using an escalation of 3 percent for five years (construction in 2014). The remaining difference of $1.2M (16%) can be attributed to the following changes which were required to both meet public concerns and challenges presented in the preliminary design effort:

- Original evaluation assumed a single sunken 36-inch pipe, while this PDR determined an inlet and outlet (two pipes) are required for District plug flow operation and that an alignment over the saddle dam provides less contamination risk and simpler O&M.

- Due to the height elevation constraints of the Tank (cannot exceed WFP height for aesthetics) and floor elevation constraints (FW pipe connection at WFP), the Tank size became shorter and wider which resulted in increased cost.
- To mitigate public impacts due to construction dust, which has been a problem during past projects at the WFP, the current design provides for asphalt over the access road and dam crossing, which was not considered in the original conceptual evaluation.

The design does provide for additional benefits not originally considered. Most notably, the current design eliminates the need for UV disinfection during this phase and potentially future phases. The cost estimated in the 2009 study, updated for current value, is approximately $1M.

Section 8: Construction Schedule

The proposed construction schedule is included in **Appendix F**. The overall project duration is estimated to be thirteen (13) months. Expected durations for key events are as follows:

- Dam paving – 10 days
- Blasting for tank/pipeline excavation and berm rough grading – 25 days
- Paving adjacent to tank – 6 days
- Tank construction – 155 days
- Pipeline installation – 80 days
- Baffling and interior tank piping – 15 days
- Treatment plant connections – 15 days
- Tank fill and leak testing – 15 days