Wholesale System 6.0 MG Reservoir Reconstruction Project

Preliminary Design Report

July 2026 / FINAL

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Abbreviations

AACEI	Association for the Advancement of Cost Engineering International
ACI	American Concrete Institute
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASTM	ASTM International
AWS	American Welding Society
AWWA	American Water Works Association
Carollo	Carollo Engineers
CBC	California Building Code
CEQA	California Environmental Quality Act
CML&C	cement mortar lined and coated
CP	control panel
DDW	Division of Drinking Water
District	East Orange County Water District
DSOD	Department of Safety of Dams
EOCWD	East Orange County Water District
EPS	extended period simulation
FRP	fiber-reinforced plastic
g	acceleration due to Earth's gravity
gpm	gallons per minute
GSWC	Golden State Water Company
HDPE	high-density polyethylene
HGL	hydraulic grade line
IRWD	Irvine Ranch Water District
IS	initial study
kpi	thousand pounds per square inch
MCE	maximum considered earthquake
MG	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
MWD	Metropolitan Water District of Southern California
MWDOC	Municipal Water District of Orange County
NFPA	National Fire Protection Association
NSF	National Sanitation Foundation
NTP	Notice to Proceed
OCFA	Orange County Fire Authority
OSHA	Occupational Safety and Health Administration

pcf	pounds per cubic foot
PDR	preliminary design report
PLC	programmable logic controller
psf	pounds per square foot
psi	pounds per square inch
Reservoir	Peters Canyon Reservoir
RSV	reservoir station
SCADA	supervisory control and data acquisition
SCE	Southern California Edison
SGMHA	site-specific ground motion hazard analysis
WS	welded steel

SECTION 1 INTRODUCTION AND PROJECT OVERVIEW

1.1 Introduction

Peters Canyon Reservoir (Reservoir) was originally constructed in 1963 and is the highest storage reservoir in the system managed by East Orange County Water District (District). The reservoir is located on a hillside east of Jamboree Road and Highway 261, within the existing Reservoir property. The reservoir access is via Handy Creek Road, which is a private road that intersects Jamboree Road. Handy Creek Road has a chain-link security gate at the entrance from Jamboree Road and crosses under the Highway 261 bridge. The project site is bound by Highway 261 to the west and north, Highway 241 to the east, and open space to the south.

The property parcel area measures approximately 6 acres and is accessible from Handy Creek Road via a relatively steep asphalt (asphaltic concrete) driveway with a site access gate. The reservoir site is perimeter-protected by an existing chain-link fence. The reservoir site includes the existing reservoir, a maintenance yard, and a trailer. The site is elevated and slopes towards the entrance from Handy Creek Road. The elevation difference is approximately 40 feet. Refer to Figure 1 for the reservoir vicinity map.

The existing 6.0-million-gallon (MG) reservoir is a concrete-lined hopper-bottom reservoir covered with a corrugated metal roofing system supported by wood framing and precast concrete columns. The reservoir is supplied by pump station OC-70, which is owned and operated by the Metropolitan Water District of Southern California (MWD) and is located along Jamboree Road within Peters Canyon Regional Park. The pump station (finished floor at an elevation of about 592 feet) pumps up to the 6.0-MG reservoir (base elevation of about 765 feet and overflow elevation of about 790 feet).

The inlet piping, outlet piping, and site drainage are located within the access driveways and perimeter pavement. On-site yard piping will be protected in place where it does not interfere with the new reservoir construction. Yard piping that connects to the existing reservoir will need to be removed and replaced. The existing yard piping will be rerouted to accommodate the footprints of the selected reservoirs. Some modifications and replacement of the existing asphalt-paved access road, and storm drains are also expected.

The reservoir, all yard piping, and appurtenances in contact with drinking water will be designed in accordance with the National Sanitation Foundation (NSF)/American National Standards Institute (ANSI) Standard 61-2016 Drinking Water System Components – Health Effects.

As a primary reservoir integral to the District's operations, numerous reports have been prepared in the past regarding its condition and repair. The reservoir has undergone several inspections along with various specific improvements. In 1997, portions of the roof were removed and reattached with improved hardware after significant damage caused by high winds. In 1999, a seismic evaluation resulted in a seismic retrofit in 2000, which consisted of adding rock anchors to the base of each perimeter concrete pier in order to provide increased resistance to seismic overturning perpendicular to north and south walls. Given the age and condition of the existing reservoir, the District would like to replace it and is looking for alternatives to do so.

Figure 1 Project Vicinity Map

In addition, the District's Local Hazard Mitigation Plans identified the current reservoir site and structure as susceptible to seismic and wildfire hazards. This report will identify ways the risks and hazards are proposed to be mitigated through materials analysis and selection, and site configuration.

This Preliminary Design Report (PDR) will evaluate the existing reservoir site and will prepare and identify the following design elements and considerations:

- Analyze and mitigate risks from both seismic activity and wildfires.
- Site layouts for a replacement reservoir, including allocations for support structures (electrical, communications/supervisory control and data acquisition [SCADA]) and access/maintenance vehicle parking area.
- Analyze any critical issues that would prevent a new reservoir from being constructed within the existing site.
- Analyze construction materials and associated methods (prestressed concrete, cast-in-place concrete, welded steel).
- Site improvements necessary for construction of the infrastructure (e.g., retaining walls, etc.).
- Phasing required to maintain water/fire service to existing customers during construction of the facilities.
- Water quality considerations, including the potential addition of a reservoir management system.

1.2 Existing Peters Canyon Reservoir Facility

The existing Reservoir is mostly buried and has plan dimensions that are approximately 280 feet 4 inches long and 170 feet 4 inches wide, and has a hopper bottom with a slope of 1-1/2 horizontal to 1.0 vertical ratio (1.5:1) on one side and a 2:1 slope on the other side. The reservoir also includes 12-inch square precast concrete columns spaced at approximately 21 feet on center in one direction and 41 feet on center in the other direction. The reservoir roof is comprised of an aluminum ZIP-RIB style aluminum decking that is supported by a wood-framed system that includes primary beams (5-1/4-inch by 21-inch glu-lam beams) spanning 41 feet over the precast concrete columns and sawn lumber purlins (2 by 12) spaced at 36 inches on center. The exterior perimeter wall of the reservoir is an approximately 4-foot high, 8-inch masonry block wall on all four sides.

The existing reservoir includes the following:

- Length: 280 feet 4 inches.
- Top of Foundation (floor of reservoir) Elevation: 765 feet.
- High Water Elevation: 790 feet.
- High Water Depth: 25 feet.

This reservoir is currently not listed under the Department of Safety of Dams (DSOD) jurisdiction.

The existing as-built plans of the reservoir do not show a floor drain at the low point area of the floor slab. The reservoir floor slopes down towards the outlet pipe on the north side. The existing reservoir has a grated overflow opening at the north end that spills directly into the access road and then into a grated storm drain catch basin. Refer to Figure 2 for the existing reservoir bottom plan and section, and Figure 3 for the reservoir roof plan.

The as-built plans of the existing reservoir are included in Appendix A of this PDR.

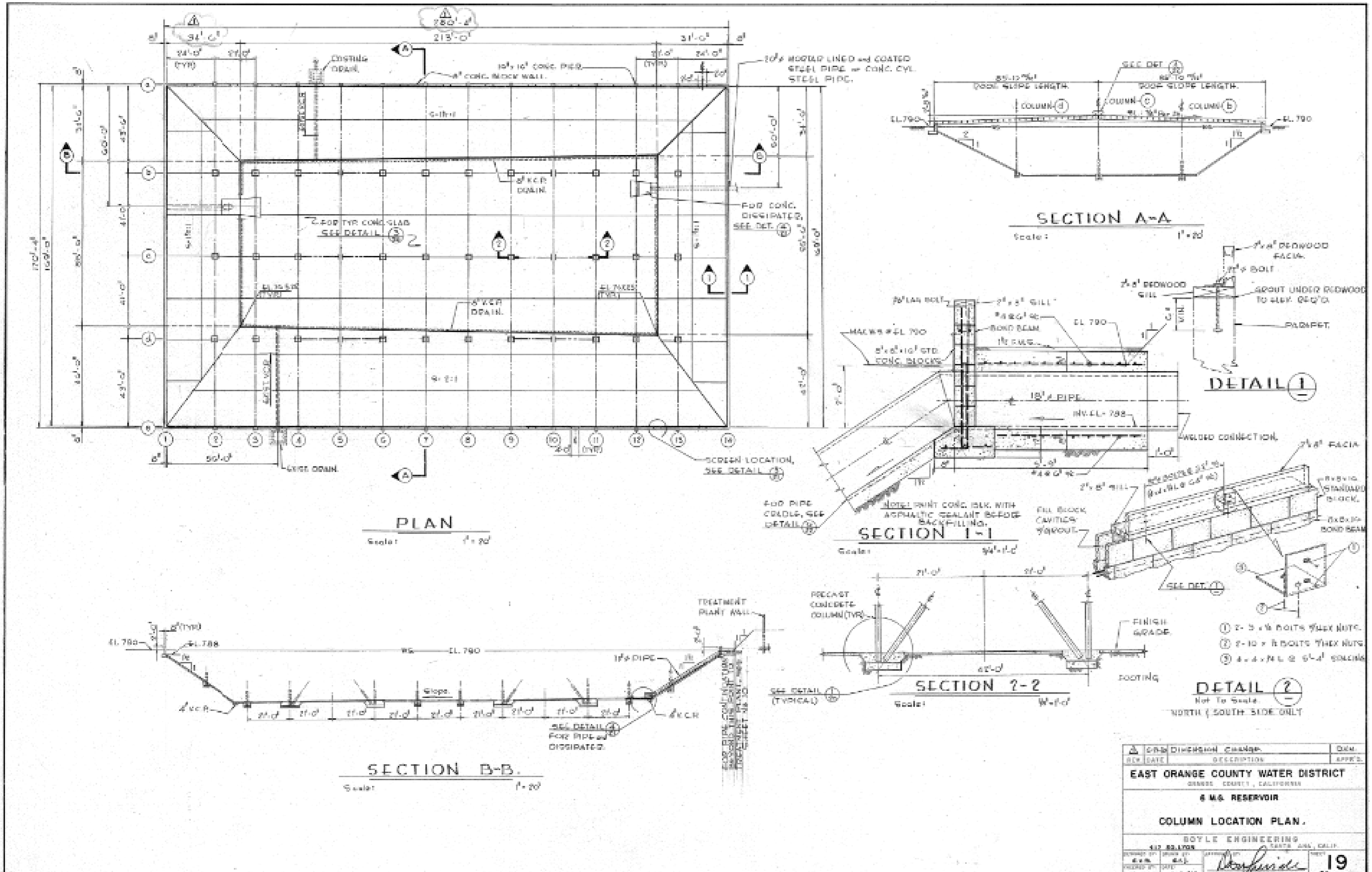


Figure 2 Existing Reservoir Bottom Plan

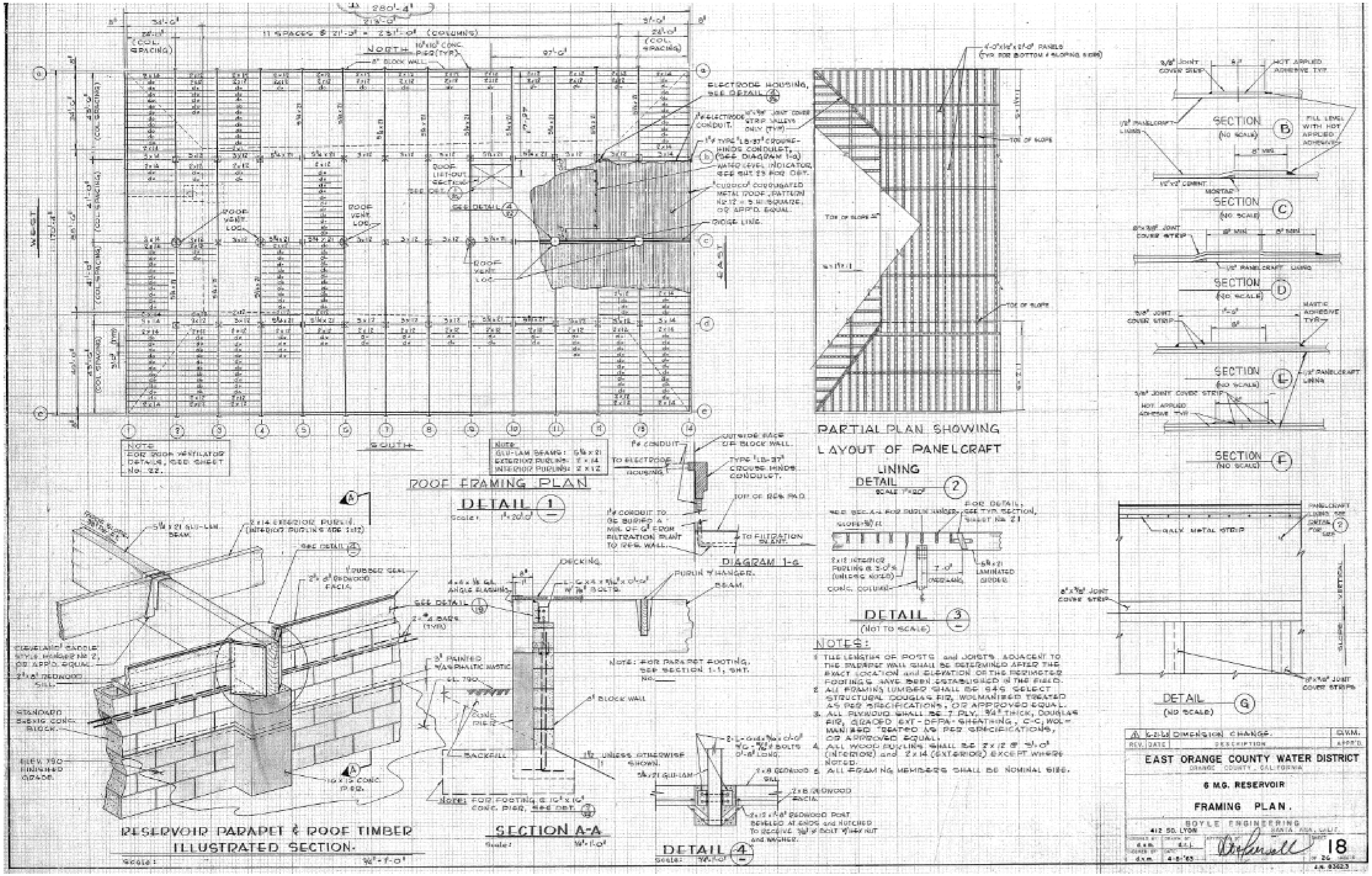


Figure 3 Existing Reservoir Roof Plan

1.3 Purpose of This Report

The Wholesale System Reservoir Seismic Reconstruction project will involve the demolition of the existing 6.0-MG Peters Canyon Reservoir and replacing it with one or two new reservoirs. This PDR describes the key features of the proposed reservoirs, including the evaluation of five potential alternatives and recommendations for the best value option. Analysis of construction costs and construction duration for each alternative is used to determine the best value option. This report also discusses other requirements, such as the probable construction costs, design criteria, schedule, and permitting, and includes several conceptual drawings for the four proposed alternatives to be used for cost comparisons between the options.

The District applied for funding through Federal Emergency Management Agency Disaster Number 4558. The scope of work included demolition of the existing reservoir and construction of a new replacement reservoir or reservoirs to reduce risk to critical infrastructure from both seismic activity and wildfires. To mitigate seismic risks, the reservoir structure will be designed to the latest codes and requirements and will include a site-specific seismic design hazard criterion. The existing roof is constructed of wood. To mitigate the potential for wildfire damage, the new structure will be constructed of concrete material to reduce potential impacts caused by wildfires in the area.

Five options were analyzed:

Option 1: One Single 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

- Phase 1 – 145-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Phase 2 – Not applicable.
- Total Volume – 3.0 MG.

Option 2: One Single 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

- Phase 1 – 3.0-MG cast-in-place rectangular concrete reservoir.
- Phase 2 – Not applicable.
- Total Volume – 3.0 MG.

Option 3: Two 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoirs

- Phase 1 – 115-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Phase 2 – 115-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Total Volume – 6.0 MG.

Option 4: One Single 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

- Phase 1 – 115-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Phase 2 – Not applicable.
- Total Volume – 3.0 MG.

Option 5: Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

- Phase 1 – 3.0-MG cast-in-place concrete reservoir (Cell No. 1).
- Phase 2 – 3.0-MG cast-in-place concrete reservoir (Cell No. 2).
- Total Volume – 6.0 MG.

Note: Options 3 and 5 are included to provide system redundancy.

Section 4 of this PDR will focus on comparing these five reservoir options in more detail and recommending the best-value option for the Peters Canyon Reservoir Replacement.

1.4 Available Studies and Reports

The following is a list of available record drawings, studies, and reports pertaining to the Peters Canyon Reservoir site provided by the District for review:

- Record Drawings (Appendix A).
- Peters Canyon and Vista Panorama Reservoir Condition Assessment by Carollo Engineers, dated April 2019.
- Engineering Evaluation of Peters Canyon 6.0 MG Reservoir by Richard Brady and Associates, dated July 2012.
- Geotechnical and Seismic Hazard Assessment Report for Existing 6.0 MG Reservoir by American Geotechnical, Inc., dated April 2, 2014.
- Summary of Findings for Geotechnical Investigation Services Existing 6.0 MG Reservoir, Handy Creek Road, Orange, California, prepared by American Geotechnical, Inc., dated March 11, 2014.

1.5 Project Team

Carollo Engineers (Carollo) has prepared this PDR. Mr. Graham Juby is the client service manager. Mr. Miko Aivazian, P.E., is the Project Manager and the point of contact for Carollo. The Project Engineer is Mr. Matthew Huckaby, P.E. The Hydraulics evaluation lead is Mrs. Adelina Pirijanyan, P.E. Carollo's structural lead is James Doering, P.E., S.E.

For East Orange County Water District, the Project Manager and point of contact is Mr. Bobby Young, P.E., Engineering Manager, with support from Jerry Mendzer, Operations Manager.

SECTION 2 HYDRAULIC ANALYSIS AND RESERVOIR SIZING

The District was originally formed to provide wholesale water service in central Orange County, including portions of the Cities of Tustin and Orange, a portion of Irvine Ranch Water District, and nearby unincorporated communities such as Lemon Heights, Cowan Heights, and Panorama Heights (North Tustin). The District took over Orange County Waterworks District No. 8 in 1985 and provides retail water service to approximately 1,200 service connections in the north Tustin area.

The District operates both a Wholesale Zone and a Retail Zone.

- The Retail Zone, located within the central west portion of the Wholesale Zone, includes wells, reservoirs, pump stations, pipelines, and customer connections.
- The Wholesale Zone system includes reservoirs, pump stations, transmission pipelines, flow control facilities, and metered connections serving five sub-agencies. The Peter's Canyon Reservoir serves the Wholesale Zone system.

To evaluate the storage requirements associated with replacing the existing Peters Canyon Reservoir, the District's existing Wholesale Zone hydraulic model was updated to reflect the demand projections and operational assumptions developed as part of the District's ongoing, separate Master Plan effort.

2.1 Water Supply

The District meets all its demands in the Wholesale Zone with treated imported water from MWD, delivered through Municipal Water District of Orange County (MWDOC).

The District sells imported water to five agencies: East Orange County Water District (EOCWD) Retail Zone, Irvine Ranch Water District (IRWD), Golden State Water Company (GSWC), City of Orange, and the City of Tustin. These agencies sell treated water at the retail level to residential, industrial, institutional, and commercial customers. Demand for imported water varies based on each agency's ability to meet groundwater needs from the Orange County Basin.

The Wholesale Zone is supplied by three imported water connections: OC-48, OC-43, and OC-70.

- OC-70 supplies the Peters Canyon Reservoir, which serves GSWC, the City of Orange, the City of Tustin, and IRWD, and provides water to the Paul A. Andres Tank and the Retail Zone. This service area is referred to as the upper wholesale zone.
- OC-48 and OC-43 primarily serve the City of Tustin through multiple turnouts and are collectively referred to as the lower wholesale zone.

During normal periods of water supply, member agencies rely primarily on local groundwater sources. During periods of groundwater operational constraints, member agencies have historically relied on the District to meet up to 100 percent of their total demand. However, in recent years, overall demands have declined substantially. Figure 4 presents the average monthly water supply from OC-70 between January 2021 and September 2025, illustrating a fairly steady reduction in peak-season demand from approximately 3,366 gallons per minute (gpm) in July 2021 to 1,978 gpm in July 2025, a decrease of roughly 41 percent.

This sustained reduction in higher-use months suggests a structural shift in demand rather than short-term variability. The trend has important implications for operational planning, system capacity utilization, and future infrastructure sizing, particularly during constrained groundwater conditions when imported supply reliability remains critical.

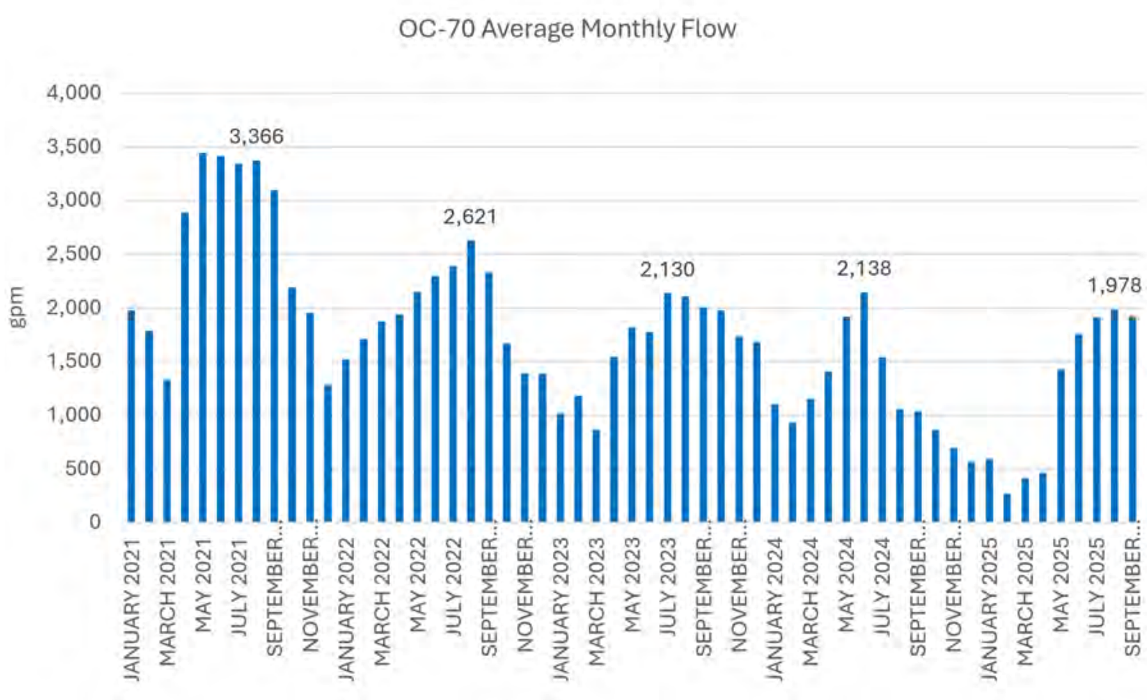


Figure 4 OC-70 Average Monthly Supply January 2021 to September 2025

2.2 Water Demand

The District is updating the Wholesale Zone Master Plan and, as part of that effort, has performed an historical analysis of the Wholesale Zone demands to determine and attempt to project realistic demands to appropriately size the proposed new reservoir.

The District provided the range of demands shown in Table 1, with demands totaled by Upper and Lower Zones. The Peters Canyon Reservoir serves the demands in the Upper Zone, which are projected to range from 1,007 gpm (low), 1,328 gpm (expected), and 2,531 gpm (high).

Table 1 Demand Estimates: Low, Expected, and High

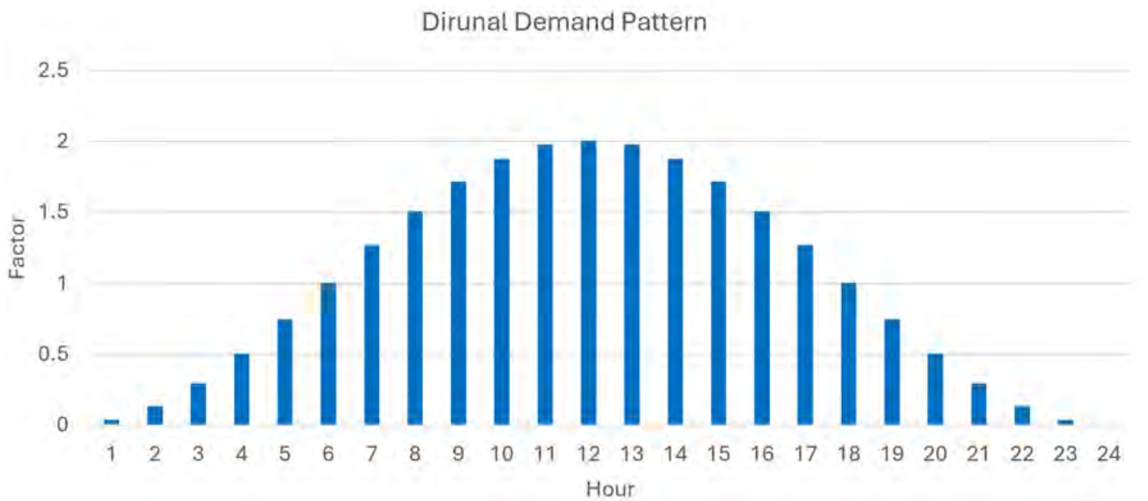
Agency Connection	Zone	Low Year Demand (gpm)	Expected Demand (gpm)	High Year Demand (gpm)
IRWD: Jamboree	Upper	0	0	190
City of Orange: Chandler Ranch	Upper	100	100	242
EOCWD: Retail Zone	Upper	0	75	497
GSWC: Peacock Hill	Upper	297	364	507
GSWC: Skyline	Upper	97	118	165
GSWC: FoxRun	Upper	297	364	507
City of Tustin: Newport Grey	Upper	218	307	424
City of Tustin: Newport Green	Upper	0	0	0
Upper Zone Total		1,007	1,328	2,531
GSWC: Cowan Heights	Lower	24	29	41
GSWC: Lemon Heights	Lower	24	29	41
City of Tustin: Prospect	Lower	89	126	173
City of Tustin: Foothills	Lower	89	126	173
City of Tustin: Hewes	Lower	256	361	498
City of Tustin: Ethelbee	Lower	175	247	340
City of Tustin: OC-43	Lower	262	853	1,237
Lower Zone Total		918	1,771	2,503
Total		1,925	3,099	5,034
Total Without Tustin: OC-43		1,663	2,246	3,797

2.3 Diurnal Patterns

Imported water is delivered to each agency through turnouts, with flow rates varying throughout the day according to a diurnal pattern. In hydraulic modeling, a diurnal pattern is a 24-hour demand curve that applies hourly multipliers to the average (base) demand to reflect typical daily fluctuations in water use.

An extended period simulation (EPS) models system performance over consecutive hours, typically 24 hours or more, capturing changes in demand, pump operations, tank levels, and controls over time. In an EPS, total demand consists of the base demand adjusted by the diurnal demand pattern.

The base demand represents the allocated customer usage at each model junction and is summarized in Table 1. The diurnal curve shown in Figure 5, provided by the District, was applied to all wholesale turnouts except for IRWD and the City of Orange, which were assumed to operate at a constant demand over the 24-hour period and therefore were not assigned a diurnal variation.



Source: District

Figure 5 Daily Diurnal Pattern

2.4 Reservoir Sizing

A storage reservoir/tank is sized to balance supply and demand within the service area while maintaining operational reliability and system stability. The required storage volume is typically determined by evaluating the following components: operational storage, emergency storage, fire storage (if applicable), and dead storage. Each term is defined below.

Operational Storage: Volume needed to balance differences between a relatively constant supply rate and variable customer demand over a 24-hour diurnal cycle.

Emergency Storage: Volume required to maintain service during supply interruptions such as transmission shutdowns, turnout outages, or groundwater constraints. The District defined the emergency condition as follows: seven-day MWD shutdown of imported water supply.

Fire Storage: Volume required to meet fire flow demands while maintaining minimum system pressures. Typically governed by local fire code and agency standards. Per the District, the 6-MG Peters Canyon Reservoir is not required to provide fire storage. Based on discussions with the member agencies, each agency maintains its own dedicated local fire storage and does not rely on District-supplied water during fire flow events. Accordingly, fire storage was not included as a component of the reservoir storage requirement.

Dead Storage: Volume below outlet elevations or unusable storage due to hydraulic or water quality constraints.

Since the Peters Canyon Reservoir serves the Upper Zone of the Wholesale System, coordination with the member agencies identified that each agency provides water storage at the individual retail level to meet Operational and Fire Storage. To determine the required storage volume for the Peters Canyon Reservoir, the District performed an emergency storage evaluation as part of the Wholesale Master Plan Update, assuming a seven-day MWD supply shutdown scenario. Operational changes were incorporated to pump water from the District's 11.5-MG tank to the Peters Canyon Reservoir during the outage condition. The demand conditions applied in this scenario reflected reduced (low demand) conditions. The District has confirmed that this lower demand projection is representative of expected operating conditions during such an outage and is therefore appropriate for sizing the reservoir. The analysis evaluated system performance under full imported supply interruption conditions and quantified the storage volume necessary to maintain wholesale service over the defined outage duration. At the time of this report, the District was in the process of procuring a bypass pump that can be used to pump from the Andres Reservoir to the Peters Canyon Reservoir. Based on this evaluation, the required emergency storage volume was determined to be approximately 3 MG.

To confirm the daily operational adequacy of the 3-MG Peters Canyon Reservoir, the Wholesale Zone was evaluated under both expected and high-demand conditions.

Under normal system operations, OC-70 serves as the primary supply source to the Peters Canyon Reservoir. The District indicated that the OC-70 supply rate is manually adjusted based on observed water surface elevations in the Peters Canyon Reservoir and system pressures at the wholesale turnout locations. These operational adjustments are used to balance reservoir storage levels while maintaining adequate pressure throughout the Wholesale Zone.

Under normal operating conditions, the Newport control valve fills the 11.5-MG tank during the early hours of the day, which then supplies the Retail Zone. The 1-MG tank that is filled through the Newport intertie and can feed the City of Tustin turnouts does not feed the model. The Newport intertie is set as closed in the model; therefore, the 1-MG tank does not fill or empty and is disconnected from the system.

2.4.1 Assumptions

The following assumptions were used to model the new 3-MG tank:

- Tank Bottom Elevation: 765 feet.
- Tank Height: 24.8 feet.
- Tank Diameter: 144 feet.
- Tank Initial Level: 19 feet.
- Tank Minimum Level: 2 feet.
- Tank Shape: cylindrical.

The tank dimensions are key factors when performing the modeling because the tank height will affect the hydraulic grade line (HGL) of the upper Wholesale Zones and impact the pumping operation for the existing OC-70 pumps.

2.4.2 Expected Demand Scenario

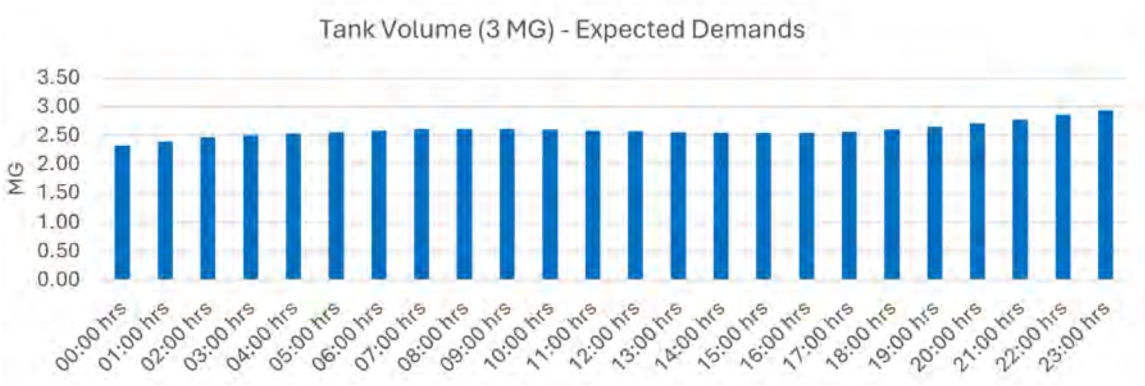
Table 2 outlines the modeling assumptions for a modeling scenario using the "expected" demands outlined in Table 1. These assumptions are key to determining an accurate tank size recommendation.

Table 2 Expected Demand Modeled Scenario Assumptions

Modeled Scenario	Modeled Time Setting	Base Demand(1)	OC-70 Supply	New Tank Size	New Tank Initial Water Level	11.5-MG Tank Initial Water Level
Expected Demand	24-Hour EPS	Expected Demand	Constant 1,500 gpm	3.0 MG	19 feet	20 feet

Notes:
(1) The expected demands are detailed by turnout in Table 1. Additionally, the diurnal curve applied to the base demands is shown in Figure 5.

This section outlines the modeling results for expected demand conditions with the tank sized at 3 MG, which is the tank size determined by the emergency storage analysis. As shown in Figures 6 through 8, the tank level fluctuates throughout the day as the tank empties and fills.



Source: Wholesale Zone Hydraulic Modeling Results

Figure 6 3.0-MG Tank Storage Volume Under Expected Demand Conditions (MG)

Source: Wholesale Zone Hydraulic Modeling Results

Figure 7 3.0-MG Tank Level Under Expected Demand Conditions (Feet)

Source: Wholesale Zone Hydraulic Modeling Results

Figure 8 3.0-MG Tank Head Under Expected Demand Conditions (Feet)

Under expected demand conditions, the pressures at each turnout are greater than the District's minimum required pressure of 40 pounds per square inch (psi). Turnout pressures are summarized in Table 3.

Table 3 3.0-MG Tank Pressure Results Under Expected Demand Conditions

Turnout	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)
GSWC - Foxrun	56	52	53
GSWC - Peacock	78	68	73
GSWC - Skyline	85	76	81
Retail Zone	146	146	146
City of Orange	53	50	51
Tustin – Newport Gray Vault	169	165	167
Tustin – Foothills	171	167	169
Tustin – Hewes	179	176	178
Tustin – Newport Green Vault	216	208	212
Tustin – Prospect	200	198	199
Tustin – Ethelbee	201	200	201
IRWD – Jamboree	53	51	52

2.4.3 High Demand Scenario

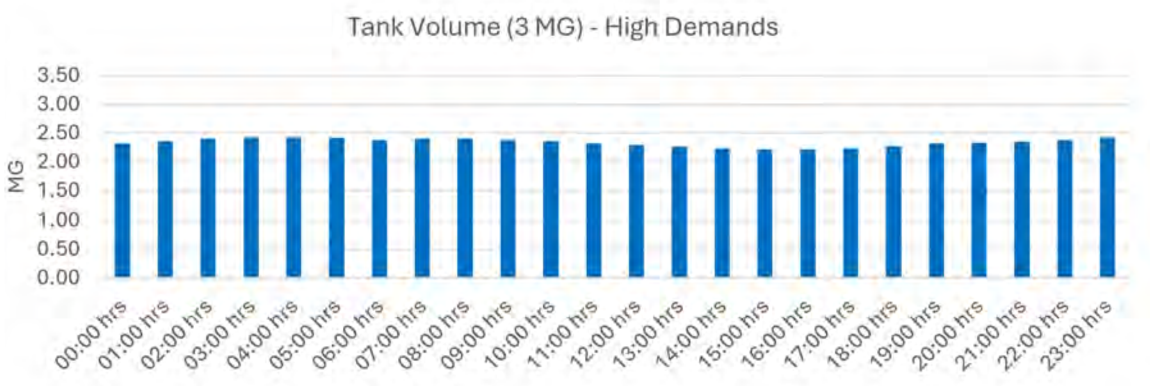
Table 4 outlines the modeling assumptions for a modeling scenario using the "high" demands outlined in Table 1. These assumptions are key in determining an accurately recommended tank size.

Table 4 High Demand Modeled Scenario Assumptions

Modeled Scenario	Modeled Time Setting	Base Demand(1)	OC-70 Supply	New Tank Size	New Tank Initial Level	11.5-MG Tank Initial Level
High Demand	24-Hour EPS	High Demand	Constant 2,200 gpm	3 MG	19 feet	20 feet

Notes:
(1) The high demands are detailed by turnout in Table 1. Additionally, the diurnal curve applied to the base demands is shown in Figure 5.

Figures 9 through 11 show the tank volume, level, and head over the 24-hour period. The results are similar to the expected demand modeling scenario because of the increase in supply from OC-70.



Source: Wholesale Zone Hydraulic Modeling Results

Figure 9 3.0-MG Tank Storage Volume Under High Demand Conditions (MG)

Source: Wholesale Zone Hydraulic Modeling Results

Figure 10 3.0-MG Tank Level Under High Demand Conditions (Feet)

Source: Wholesale Zone Hydraulic Modeling Results

Figure 11 3.0-MG Tank Head Under High Demand Conditions (Feet)

Under expected demand conditions, the pressures at each turnout are greater than the District's minimum required pressure of 40 psi. Turnout pressures are summarized in Table 5.

Table 5 3.0-MG Tank Pressure Results Under High Demand Conditions

Turnout	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)
GSWC - Foxrun	52	49	51
GSWC - Peacock	75	59	69
GSWC - Skyline	82	68	76
Retail Zone	146	143	145
City of Orange	50	47	49
Tustin – Newport Gray Vault	169	161	166
Tustin – Foothills	171	164	168
Tustin – Hewes	179	173	177
Tustin – Newport Green Vault	213	200	208
Tustin – Prospect	200	196	199
Tustin – Ethelbee	201	199	200
IRWD – Jamboree	51	50	50

2.5 Temporary Reservoir Sizing

When the existing 6.0-MG reservoir is taken out of service and demolished to make room for the replacement reservoir, there will be a need for a small temporary reservoir to provide some level of storage until the new reservoir is constructed.

2.5.1 Temporary Tank Calculation

Temporary water storage tanks are sized using a mass balance (equalization) approach that evaluates the difference between available supply and system demand over time, rather than as full daily supply storage. Storage is intended to buffer short-term inflow–outflow imbalances during peak demand periods, operational transitions, or construction activities. Sizing, therefore, focuses on short-duration demand peaks, with the maximum cumulative supply deficit defining the required storage volume. In addition to volume, the temporary tank must support minimum system pressures, acceptable pump cycling and operational stability, and adequate water turnover for water quality. Accordingly, an understanding of District operations during the construction period—when a temporary tank is in service at the Peters Canyon Reservoir—is critical.

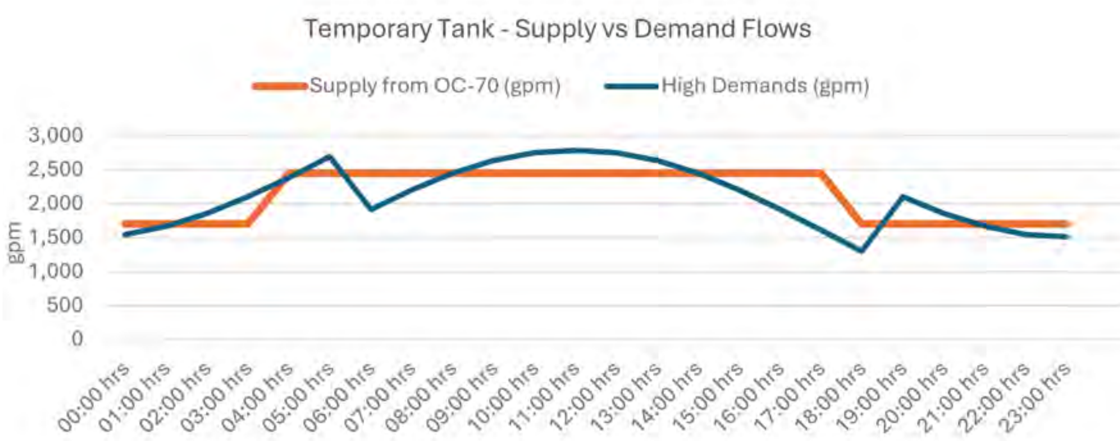
To size the temporary tank, a mass balance analysis was performed by evaluating the difference between supply from the OC-70 Pump Station and system demands over a 24-hour period. This analysis was used to determine the operational storage volume required to be maintained in the temporary tank. The calculations are summarized in Table 6 and the corresponding demand-versus-supply profile is shown in Figure 12.

Table 6 Temporary Tank Sizing Calculation

Time	Operational Storage					
	High Demands (gpm)	Supply from OC-70 (gpm)	Flow (Supply–Demand) (gpm)	Storage (mgd)	Storage Volume (MG)	Paul A. Andres Tank Flow Rate (Newport Valve) (gpm)
00:00 hours	1,550	1,700	150	0.2	0.01	1,083
01:00 hours	1,668	1,700	32	0.0	0.00	1,083
02:00 hours	1,857	1,700	-157	0.0	0.00	1,083
03:00 hours	2,104	1,700	-404	0.0	0.00	1,083
04:00 hours	2,387	2,450	63	0.1	0.00	1,083
05:00 hours	2,694	2,450	-244	0.0	0.00	1,083
06:00 hours	1,917	2,450	533	0.8	0.03	0
07:00 hours	2,200	2,450	250	0.4	0.02	0
08:00 hours	2,447	2,450	3	0.0	0.00	0
09:00 hours	2,636	2,450	-186	0.0	0.00	0
10:00 hours	2,754	2,450	-304	0.0	0.00	0
11:00 hours	2,789	2,450	-339	0.0	0.00	0
12:00 hours	2,754	2,450	-304	0.0	0.00	0
13:00 hours	2,636	2,450	-186	0.0	0.00	0
14:00 hours	2,447	2,450	3	0.0	0.00	0
15:00 hours	2,200	2,450	250	0.4	0.02	0
16:00 hours	1,917	2,450	533	0.8	0.03	0
17:00 hours	1,611	2,450	839	1.2	0.05	0
18:00 hours	1,304	1,700	396	0.6	0.02	0
19:00 hours	2,104	1,700	-404	0.0	0.00	1,083
20:00 hours	1,857	1,700	-157	0.0	0.00	1,083
21:00 hours	1,668	1,700	32	0.0	0.00	1,083

Time	Operational Storage					
	High Demands (gpm)	Supply from OC-70 (gpm)	Flow (Supply–Demand) (gpm)	Storage (mgd)	Storage Volume (MG)	Paul A. Andres Tank Flow Rate (Newport Valve) (gpm)
22:00 hours	1,550	1,700	150	0.2	0.01	1,083
23:00 hours	1,515	1,700	185	0.3	0.01	1,083
					0.205	

Notes:
mgd - million gallons per day



Source: Wholesale Zone Hydraulic Modeling Results

Figure 12 0.205-MG Temporary Tank Supply versus Demand Flows (MG)

2.5.1.1 Construction Operating Condition Assumptions

- OC-70 supply rate is shown in Table 6 and Figure 12. The supply rate is a key factor in sizing the temporary tank; as the supply rate approaches the demand rate, the required temporary storage volume decreases.
- The tank is sized under high-demand conditions.
- Same diurnal demand pattern reflects the diurnal provided by the District (Figure 5).
- Flow rate to fill the Paul A. Andres Tank is shown in Table 6. It was assumed that the Paul A. Andres Tank would be filled at the rate indicated; however, per District, the tank can alternatively be supplied using water from local wells. If the District modifies its operational approach for filling the Paul A. Andres Tank, the required volume of the temporary tank may be reduced.

Based on the assumed OC-70 supply rate, system demands, diurnal demand, and operational assumptions for supplying the Paul A. Andres Tank, the analysis indicates that a temporary storage tank with a capacity of approximately 205,000 gallons is required. Should the need arise, the District will have the ability to pump from the Andres Reservoir to the Peters Canyon Reservoir during the period that the temporary storage tank is in operation.

2.5.2 Temporary Tank Evaluation

The hydraulic model was used to evaluate system performance under the construction phase operating conditions described in the previous section. This section presents the modeling results, assuming a temporary storage tank with a capacity of 0.205 MG.

2.5.2.1 Assumptions

The following assumptions were used to model the temporary 0.205-MG tank:

- Tank Bottom Elevation: 800 feet.
- Tank Height: 14 feet.
- Tank Diameter: 50 feet.
- Tank Initial Level: 13 feet.
- Tank Minimum Level: 2 feet.
- Tank Shape: cylindrical.

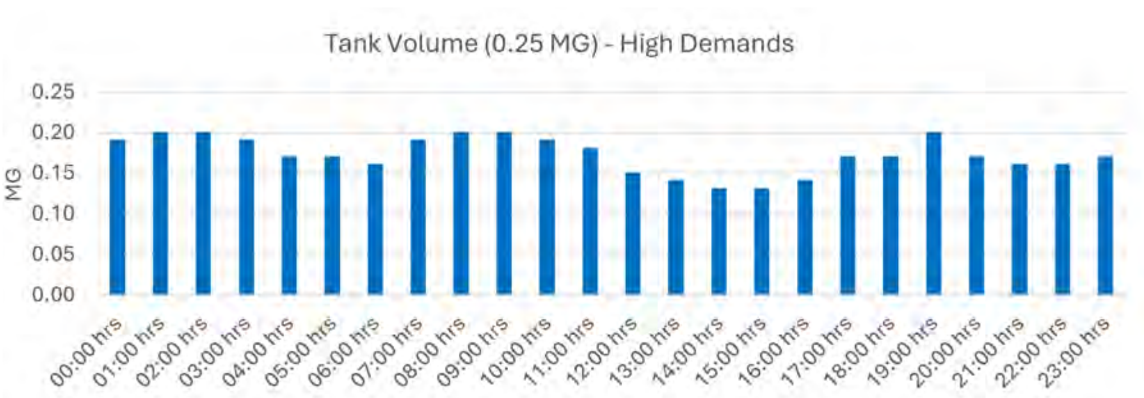
Table 7 outlines the modeling assumptions used for the temporary tank scenarios.

Table 7 Temporary Tank Modeled Scenario Assumptions

Modeled Scenario	Modeled Time Setting	Base Demand(1)	OC-70 Supply	New Tank Size	New Tank Initial Level	11.5-MG Tank Initial Level
Temporary Tank – High Demand	24-Hour EPS	High Demand	Average of ~ 2,100 gpm	0.205 MG	13 feet	20 feet

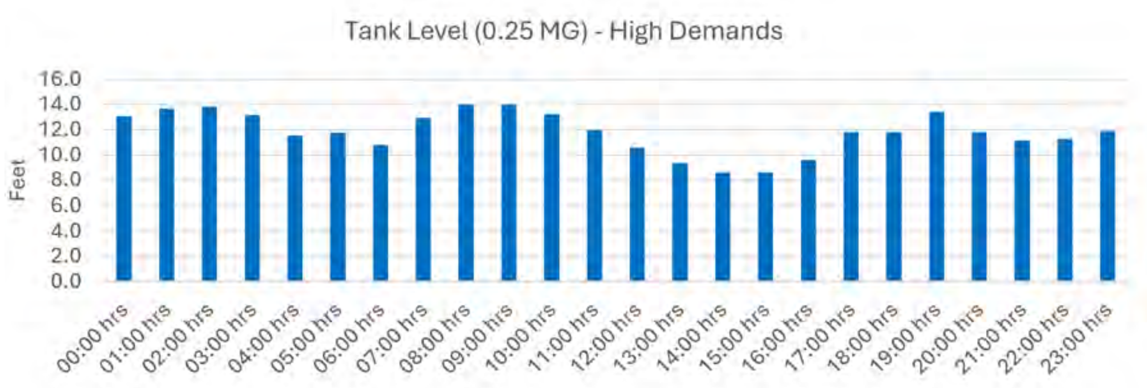
Notes:
(1) The high demands are detailed by turnout in Table 1. Additionally, the diurnal curve applied to the base demands is shown in Figure 5.

Figures 13 through 15 show the tank volume, level, and HGL over the 24-hour period.



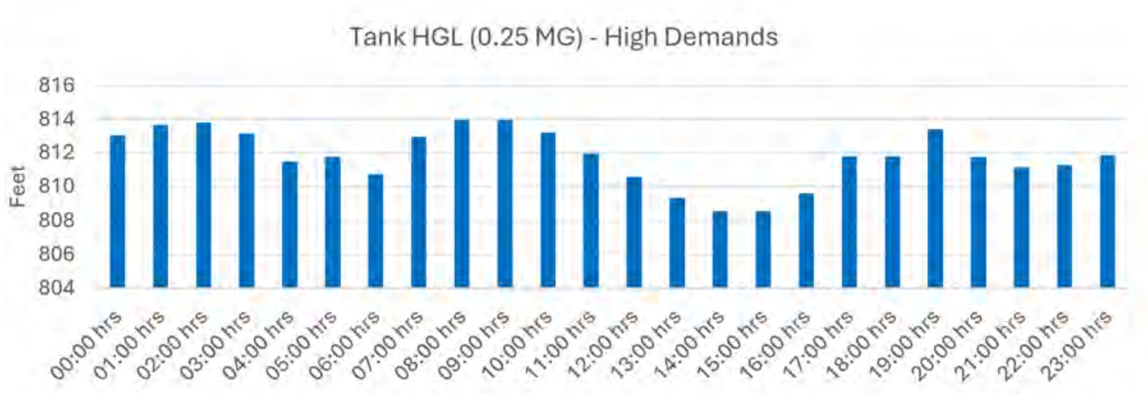
Source: Wholesale Zone Hydraulic Modeling Results

Figure 13 0.205-MG Temporary Tank Storage Volume Under High Demand Conditions (MG)



Source: Wholesale Zone Hydraulic Modeling Results

Figure 14 0.205-MG Temporary Tank Level Under High Demand Conditions (Feet)



Source: Wholesale Zone Hydraulic Modeling Results

Figure 15 0.205-MG Temporary Tank Head Under High Demand Conditions (Feet)

Under high demand conditions, the pressures at each turnout are greater than the District's minimum required pressure of 40 psi. Turnout pressures are summarized in Table 8.

Table 8 0.205-MG Temporary Tank Pressure Results Under High Demand Conditions

Turnout	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)
GSWC – Foxrun	65	60	63
GSWC – Peacock	87	71	81
GSWC – Skyline	94	80	88
Retail Zone	146	143	145
City of Orange	62	58	61
Tustin – Newport Gray Vault	169	161	166
Tustin – Foothills	171	164	168
Tustin – Hewes	179	173	177
Tustin – Newport Green Vault	225	212	220
Tustin – Prospect	200	196	199

Turnout	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)
Tustin – Ethelbee	201	199	200
IRWD – Jamboree	63	61	62

2.6 Discussion

The hydraulic modeling analysis indicates that under both the expected and high-demand scenarios, a 3.0-MG storage tank provides sufficient capacity to meet anticipated demands under normal operating conditions. For both scenarios, changes in reservoir volume, reservoir water level, and the hydraulic grade line originating from the reservoir remained within acceptable ranges. In addition, system pressures were consistently maintained above 40 psi.

Based on these results, it is concluded that a 3.0-MG reservoir located at the Peters Canyon Reservoir site, replacing the existing 6.0-MG reservoir, will provide adequate storage to support the District's anticipated operating scenarios. As noted previously, a 3.0-MG reservoir will also provide sufficient storage to meet system demands during an emergency condition, such as a seven-day MWD outage, provided that water is pumped from the Paul A. Andres Tank to the Peters Canyon Reservoir.

The system was also evaluated under construction-phase conditions, during which the existing 6.0-MG reservoir would be taken out of service and demolished to accommodate construction of the replacement reservoir. Under this condition, a temporary storage tank is required to provide limited operational storage during the construction period. Modeling results indicate that a temporary tank with a capacity of approximately 250,000 gallons would be required based on the assumed construction-phase operating conditions. It is recommended that the District review these assumptions to confirm and finalize the temporary tank size.

SECTION 3 FIELD INVESTIGATIONS AND UTILITY RESEARCH

3.1 Field Investigations

Carollo team visited the project site on September 25, 2025. The team walked around the reservoir to observe the exterior of the existing tank. A visual inspection of the tank interior was not performed by the team. The following are a few photos of the exterior of the existing reservoir (Figures 16 through 23).

Figure 16 Reservoir Roof and Inlet Pipe – South End

Figure 17 Reservoir Exterior – South End

Figure 18 Reservoir Access Road – Looking Northwest

Figure 19 Reservoir Access Road – Entrance Gate

Figure 20 Reservoir Overflow – North End

Figure 21 Reservoir Perimeter Monitoring Well

Figure 22 Reservoir Crack/Movement Monitoring

Figure 23 Curb Crack/Movement Monitoring

3.2 Existing Site Utilities

3.2.1 Existing Reservoir Inlet/Outlet Piping

The existing reservoir inlet pipe is a 24-inch cement mortar lined and coated (CML&C) steel pipe. The 24-inch inlet pipe reduces to 18-inch diameter prior to entering the reservoir on the south end. The reservoir outlet pipe is a 21-inch CML&C steel pipe and is located on the north end of the reservoir. There is also a 16-inch reservoir bypass pipeline parallel to the inlet pipe that was abandoned when the filtration plant was decommissioned. Refer to Figure 24 for the existing inlet/outlet piping plan and details. The existing inlet/outlet pipe will be modified and adjusted to fit the selected reservoir alternative.

3.2.2 Existing Site Storm Drainage System

Stormwater improvements were made by Carollo in 2015. Runoff travels from east to west into two grate inlets along the westerly curb. Runoff collected by the north inlet flows into the existing storm drain system. Runoff collected by the south inlet travels through a 15-inch high-density polyethylene (HDPE) pipe, a series of junction boxes, and a transition structure to connect to the existing storm drain system. Refer to Figure 25 for the Peters Canyon Reservoir Storm Water Drainage Improvements – Site Layout and Drainage Plan.

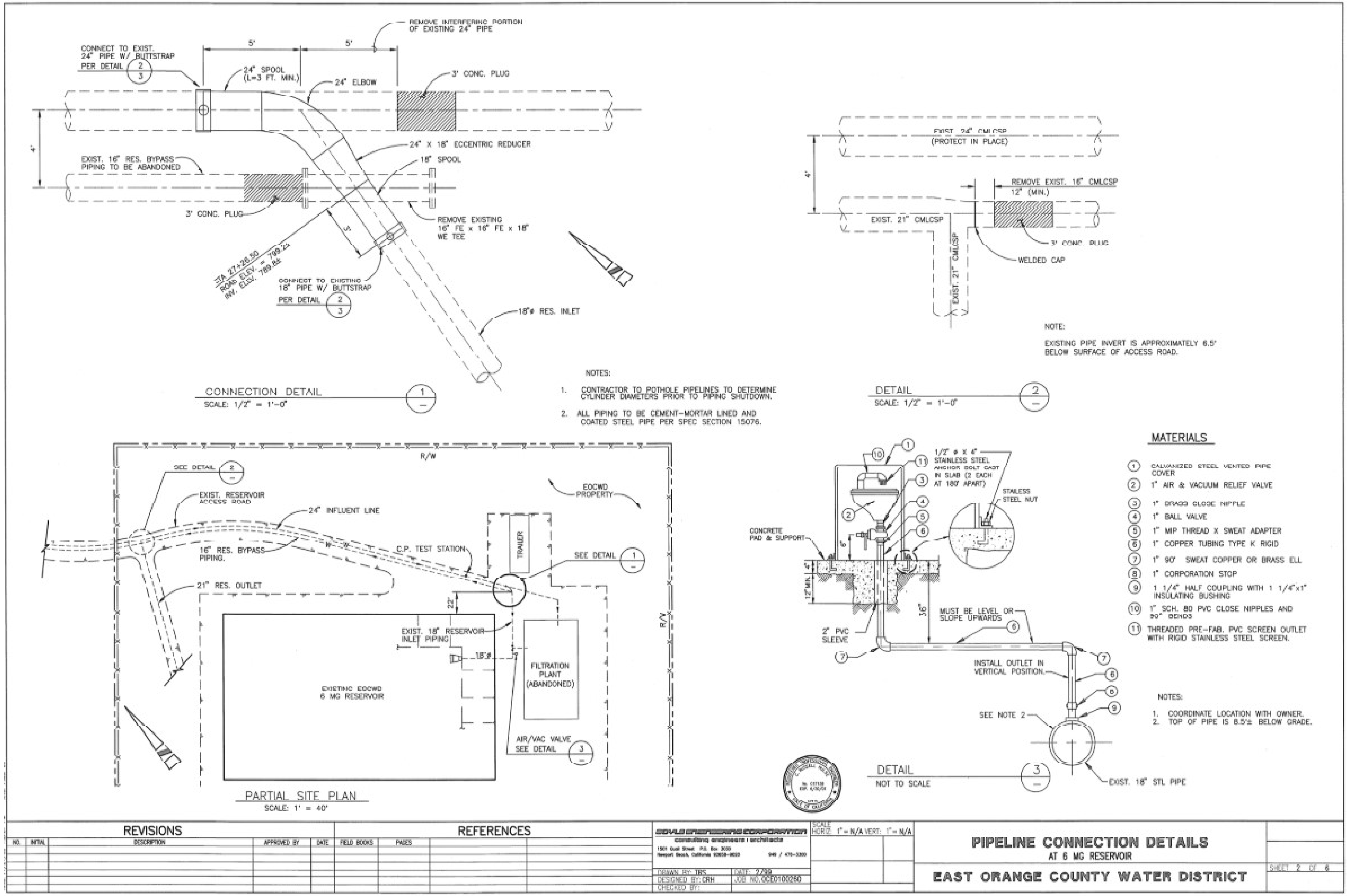


Figure 24 Existing Inlet/Outlet Piping

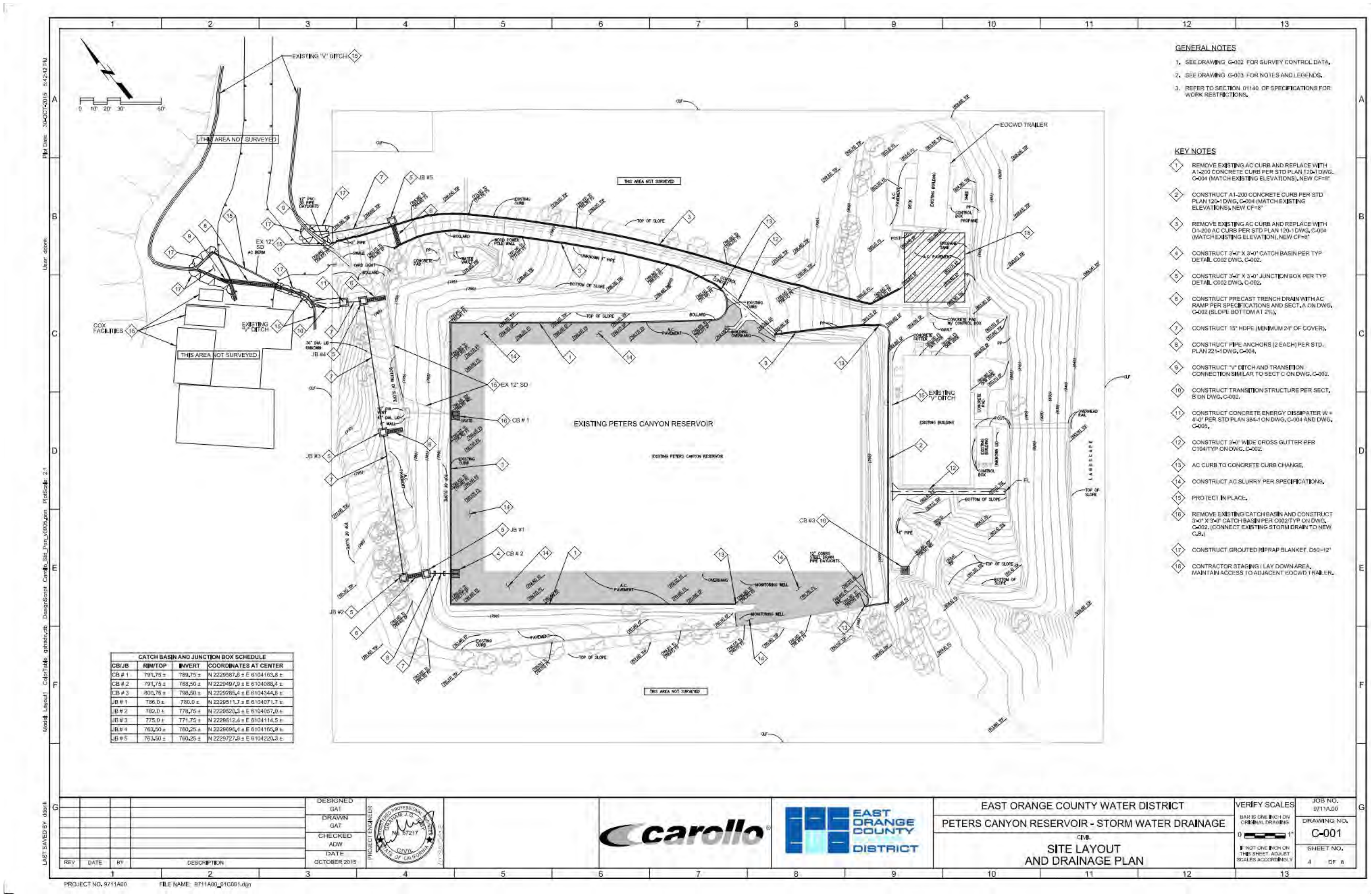


Figure 25 Existing Drainage

SECTION 4 RESERVOIR ALTERNATIVES

4.1 Analysis of Construction Materials and Selected Alternatives

A general analysis of construction materials and associated methods for a replacement reservoir (prestressed concrete, cast-in-place concrete, welded steel) follows.

Water storage tanks larger than 0.25 MG are typically constructed of welded steel (WS) per American Water Works Association (AWWA) D100, prestressed concrete per AWWA D110, or cast-in-place concrete per ACI 350-20. Welded steel tanks generally have lower capital costs than prestressed concrete tanks in the 1.0 to 2.0± MG capacity range. However, life cycle costs for WS tanks tend to be comparatively higher due to periodic interior and exterior recoating and repairs to address corrosion of the tank structure. WS tank recoating is typically recommended every 15 to 20 years, depending on service conditions, requiring the tank to be offline for several weeks or months, depending on the tank size.

- Prestressed and cast-in-place concrete tanks do not require interior or exterior coatings and typically have significantly lower life-cycle costs despite higher capital costs.
- Prestressed and cast-in-place concrete roof connections may be designed to withstand impact from a seismic sloshing wave, allowing for a greater operational storage volume (higher maximum HGL) than a welded steel tank of identical diameter and height.
- Prestressed concrete tanks can be designed with either a self-supporting dome roof or a column-supported flat (1.5 to 2 percent slope) roof. Compared to flat roofs, the domed roof system is less costly, does not require internal columns, has a reduced construction schedule, and has less stringent differential settlement constraints. The primary advantage of a flat roof is its lower visual profile, whereas a domed roof tank may not be acceptable due to site aesthetic concerns from nearby residential developments. Welded steel tanks are generally constructed with flat (1.5 to 2 percent slope) roofs supported by an interior central column and radial rafters.
- Prestressed and cast-in-place concrete tanks may also be partially or fully buried if required to meet structure height limitations, whereas welded steel tanks must be constructed at grade.
- Welded steel tanks conduct heat (thin steel shell), which results in thermal stratification (thermocline) within the stored water volume. Warmer water rises to the top, accelerating the loss of disinfectant residual, resulting in extended water age in unmixed tanks. Prestressed and cast-in-place concrete tanks are naturally insulated (thicker concrete shell) and do not experience thermocline to the same degree as welded steel tanks, resulting in a negligible impact on disinfectant residual loss.
- Prestressed and cast-in-place concrete tanks are inherently fire resistant as concrete is non-combustible. Welded steel tanks may lose structural integrity if exposed to high-intensity fires. Furthermore, interior or exterior coatings may be damaged if exposed to extreme heat, requiring a prolonged tank outage to recoat impacted surfaces.

During the first scoping meeting held with the District on September 25, 2025, several options for the reservoir were discussed, including the desired capacity of the reservoir. The District was of the opinion that the existing 6.0-MG reservoir may be too big given the operational scenarios previously discussed in this report. This led to the analysis detailed in Section 2, and the conclusion that a 3.0-MG reservoir would meet all of the requirements for the District. Based on this conclusion and the provided record drawings and maps, Carollo investigated five options for the replacement reservoir. These are:

- Option 1: One 145-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Option 2: One 3.0-MG rectangular cast-in-place concrete reservoir.
- Option 3: Two 3.0-MG circular prestressed concrete reservoirs.
- Option 4: One 115-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Option 5: Two 3.0-MG rectangular cast-in place concrete reservoir cells.

A welded steel tank option was not considered. In addition to the considerations mentioned above, this option would require extensive earthwork, site modifications, and retaining walls. Because of the extensive effort and high cost required to construct a welded steel tank and retaining walls, this option was removed from further evaluations.

Each reservoir option will be developed to maximize the storage capacity based on the hydraulic requirements, demands, elevation criteria, type/shape of the reservoir, and the existing site constraints. Hydraulic modeling and reservoir sizing is described in Section 2 of this PDR.

The following sections of this PDR will focus on comparing these five options and recommending the best alternative for the Peters Canyon Reservoir Replacement.

During construction of the selected alternative, a temporary tank will be required to maintain service to GSWC and the rest of the Upper Zone. The temporary tank will be a circular tank constructed of corrugated steel and have a 50-foot diameter and a 14-foot-high water column. Temporary inlet and outlet piping will be routed underground to connect with existing inlet and outlet infrastructure. Refer to Section 2 for an analysis of the temporary tank sizing. Refer to the site layouts for each option, which include a site layout of the temporary tank, located on the east side of the site.

4.1.1 Option 1 – One 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

This option includes the construction of one 3.0-MG circular prestressed concrete reservoir sited directly over the existing reservoir footprint. A significant reduction in the footprint area is expected. The existing inlet, outlet, and drain piping will require modifications to accommodate the new circular reservoir.

The reservoir will have the following features:

- Inside Diameter: 145 feet.
- Top of Foundation Elevation: 765 feet.
- High Water Elevation: 790 feet.
- High Water Depth: 25 feet.

A conceptual site layout of the prestressed concrete reservoir option is provided on Figure 26. A conceptual structural plan and section of the reservoir option is provided on Figure 27.

4.1.2 Option 2 – One 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

This option includes construction of one 3.0-MG rectangular cast-in-place concrete reservoir.

The cast-in-place reservoir for this option will be rectangular 130 feet long by 130 feet wide. The 3.0-MG reservoir for this option will have the following features:

- Length (inside): 130 feet.
- Width (inside): 130 feet.
- Top of Foundation Elevation: 765 feet.
- High Water Elevation: 790 feet.
- High Water Depth: 25 feet.

This option will not require raising the high-water elevation. In other words, the existing water elevation of 790 feet can be maintained and will have no impact on the existing pumping system.

Considering the existing reservoir site grading and side slopes, this size of a reservoir with a water depth of 25 feet will not place the reservoir under the DSOD jurisdiction. This option may be a viable option considering DSOD permitting will not be required.

The reservoir will be sited directly over the existing reservoir footprint. The footprint area is expected to be similar to the existing reservoir. The existing inlet, outlet, and drain piping will require some modifications to accommodate the two cells of the new reservoir.

A conceptual layout of the cast-in-place concrete reservoir option is provided in Figure 28. Conceptual structural bottom plan, top plan, and sections are provided in Figures 29, 30, and 31.

4.1.3 Option 3 – Two 115-Foot Diameter 3.0-MG Circular Prestressed Reservoirs

The construction of two 3.0-MG circular prestressed reservoirs would be phased. In Phase 1, the first tank will be constructed on the west side of the existing tank site and placed in service. The construction of the second 3.0-MG tank will be completed in Phase 2. This option will allow the District to have redundancy and operational flexibility. One reservoir may be taken out of service for maintenance when needed while the other reservoir remains operational. Both reservoirs will be sited directly over the existing reservoir footprint. The existing inlet, outlet, and drain piping will require some modifications to accommodate the new reservoirs.

The 3.0-MG reservoirs for this option will have the following features:

- Inside Diameter: 115 feet.
- Top of Foundation Elevation: 765 feet.
- High Water Elevation: 802 feet.
- High Water Depth: 37 feet.

This option will require raising the high-water elevation from 790 to elevation 802, an increase of 12 feet. Existing pumping system will need to be checked to make sure existing pumps can handle the additional head required. This will also create additional head in the distribution system when the reservoir is full and may require adjustment of system pressure controllers.

Considering the existing reservoir site grading and side slopes, this size of a reservoir even with 37-foot water depth will not place the reservoir under the DSOD jurisdiction. This option may be a viable option considering DSOD permitting will not be required.

A conceptual site layout of this option is provided in Figure 32. A conceptual structural bottom plan, top plan and section of Phase 1 is provided in Figure 33.

4.1.4 Option 4 – One 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

This option includes construction of one 3.0-MG circular prestressed concrete reservoir sited directly over the existing reservoir footprint. A significant reduction in the footprint area is expected. The existing inlet, outlet, and drain piping will require modifications to accommodate the new circular reservoir.

The reservoir will have the following features:

- Inside Diameter: 115 feet.
- Top of Foundation Elevation: 765 feet.
- High Water Elevation: 802 feet.
- High Water Depth: 37 feet.

A conceptual site layout of the prestressed concrete reservoir option is provided on Figure 34. A conceptual structural plan and section of the reservoir option is provided on Figure 35.

4.1.5 Option 5 – Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

This option includes construction of two cast-in-place rectangular concrete reservoirs in the following phases:

- Phase 1 – 3.0-MG cast-in-place concrete reservoir (Cell No. 1).
- Phase 2 – 3.0-MG cast-in-place concrete reservoir (Cell No. 2).

The cast-in-place reservoir for this option will be rectangular 260 feet long by 160 feet wide. The 6.0-MG reservoir for this option will have the following features:

- Length (inside): 260 feet (130 feet per cell).
- Width (inside): 130 feet.
- Top of Foundation Elevation: 765 feet.
- High Water Elevation: 790 feet.
- High Water Depth: 25 feet.

This option will not require raising the high water elevation. In other words, the existing water elevation of 790 feet can be maintained and will have no impact on the existing pumping system or distribution system pressure.

Considering the existing reservoir site grading and side slopes, this size of a reservoir with a water depth of 25 feet will not place the reservoir under the DSOD jurisdiction. This option may be a viable option considering DSOD permitting will not be required. This option will also provide the District redundancy and operational flexibility. One reservoir may be taken out of service for maintenance when needed while the other reservoir remains operational.

The reservoir will be sited directly over the existing reservoir footprint. The footprint area is expected to be similar to the existing reservoir. The existing inlet, outlet, and drain piping will require some modifications to accommodate the two cells of the new reservoir.

A conceptual site layout of the cast-in-place concrete reservoir option is provided in Figure 36. A conceptual structural bottom plan, top plan and sections is provided in Figures 37, 38, and 39.

4.1.6 Comparison of Options

Table 9 represents the pros and cons of each reservoir option. It should be noted that as described previously in this report, all options considered will greatly reduce the risks associated from seismic and wildfire vulnerabilities.

Table 9 Options Comparison Table

Pros and Cons of Options		
Option Number	Pros	Cons
Option 1 (Single Circular 145-foot Diameter 3.0-MG Prestressed Concrete Tank)	▪ Provides some area for use as maintenance yard. ▪ Less modification to inlet/outlet piping. ▪ Not in DSOD jurisdiction. ▪ Lower construction cost.	▪ Provides no operational redundancy/ flexibility. ▪ Provides no future capacity increase.
Option 2 (Single Rectangular 3.0-MG Cast-in-Place Concrete Tank)	▪ Provides some area for use as maintenance yard. ▪ Less modification and impact to the existing access roads. ▪ Not in DSOD jurisdiction. ▪ Lower construction cost.	▪ Provides no operational redundancy/ flexibility. ▪ Provides no future capacity increase. ▪ Major modification to the inlet/outlet piping.
Option 3 (Two 115-foot Diameter Circular 3.0-MG Prestressed Concrete Tanks)	▪ Provides twice the storage capacity of Options 1 and 2. ▪ Not in DSOD jurisdiction. ▪ Provides operational redundancy/ flexibility.	▪ Does not provide additional area for use as maintenance yard. ▪ Requires additional shoring during construction. ▪ Requires pumping system analysis and potential modifications due to higher top water elevation. ▪ 65% higher construction cost compared to Option 1 due to larger storage capacity. ▪ Major modification to the inlet/outlet piping.

Pros and Cons of Options		
Option Number	Pros	Cons
Option 4 (Single Circular 115-foot Diameter 3.0-MG Prestressed Concrete Tank)	▪ Provides some area for use as maintenance yard. ▪ Less modification to inlet/outlet piping. ▪ Not in DSOD jurisdiction. ▪ Lower construction cost.	▪ Provides no operational redundancy/ flexibility.
Option 5 (Two Rectangular 3.0-MG Cast-in-Place Concrete Tanks)	▪ Provides twice the storage capacity of Options 1 and 2. ▪ Not in DSOD jurisdiction. ▪ Provides operational redundancy/flexibility. ▪ Does not require pumping system modifications. HGL stays the same.	▪ Does not provide additional area for use as maintenance yard. ▪ Requires additional shoring during construction. ▪ 50% higher construction cost compared to Option 2 due to larger storage capacity. ▪ Major modification to the inlet/outlet piping.

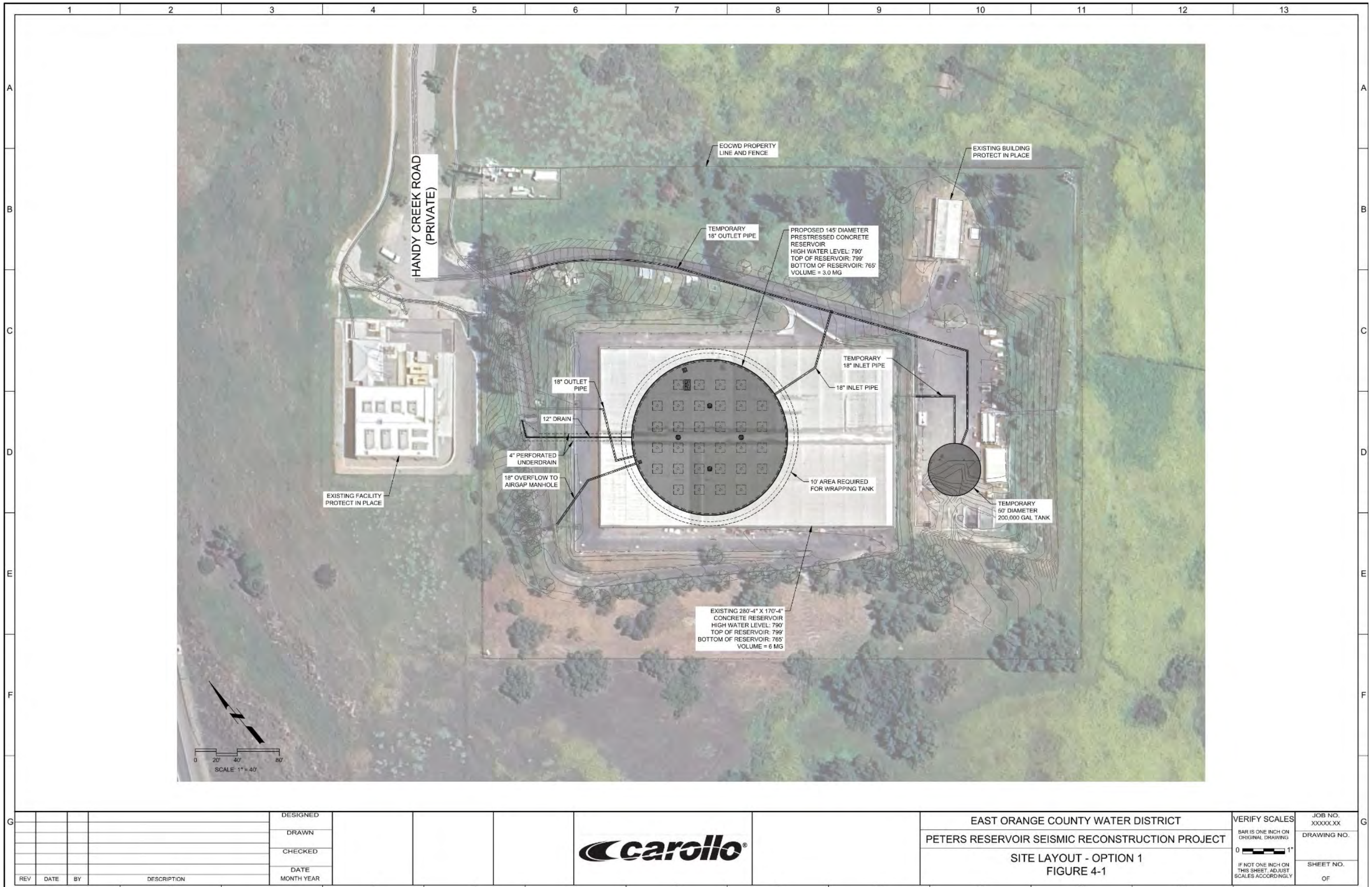


Figure 26 Option 1 Conceptual Site Layout

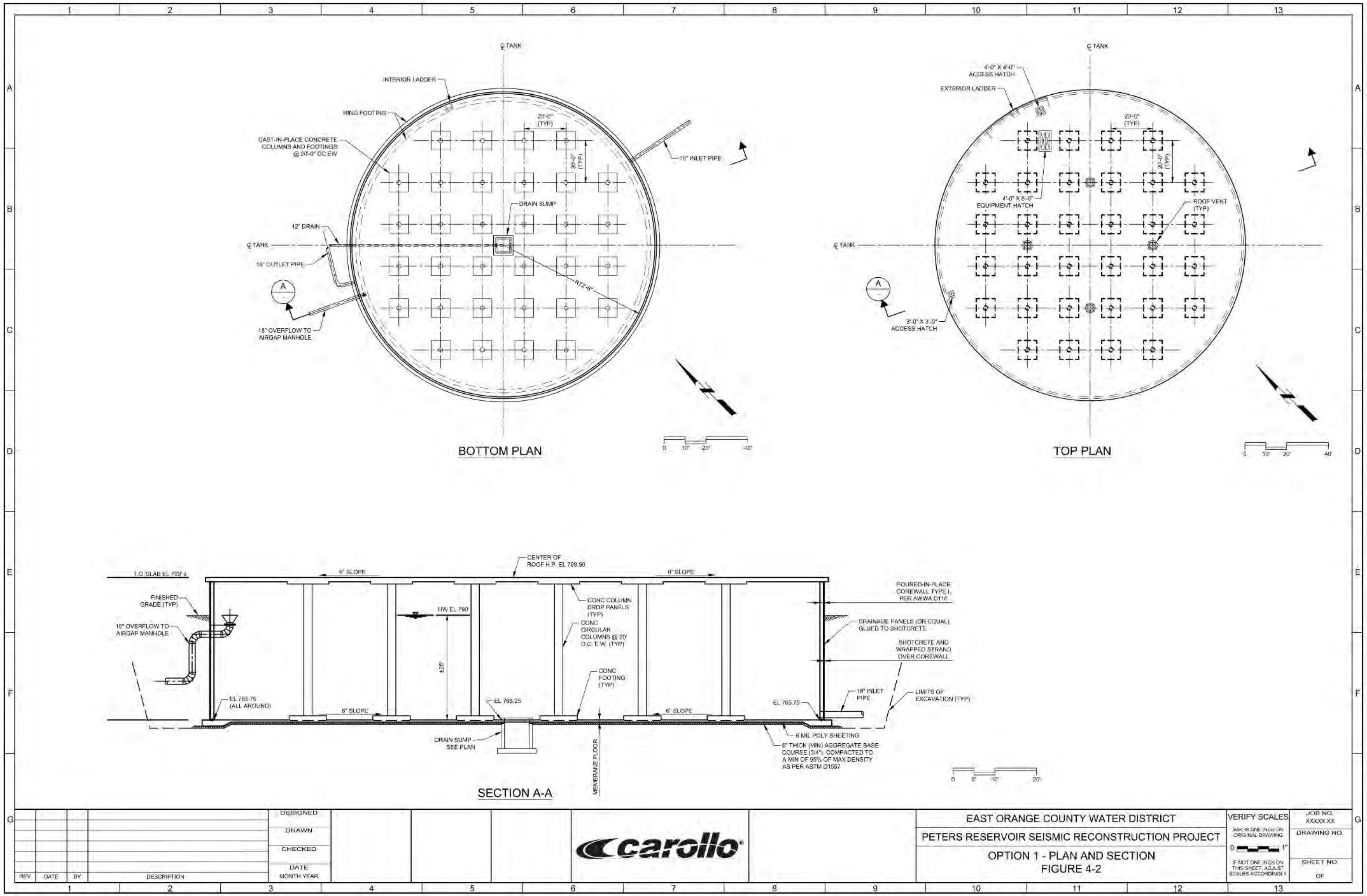


Figure 27 Option 1 Conceptual Structural Plan and Section

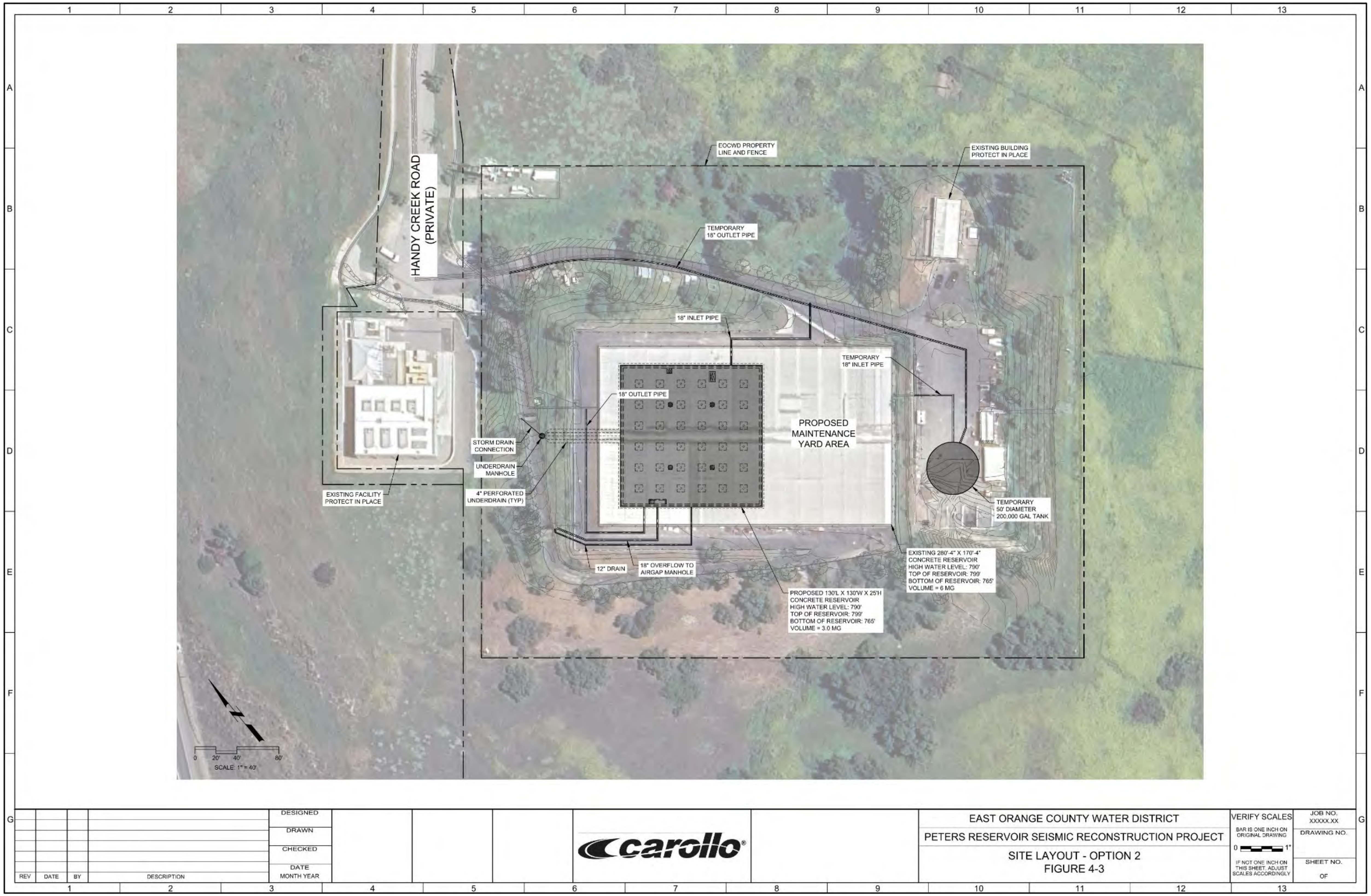


Figure 28 Option 2 Conceptual Site Layout

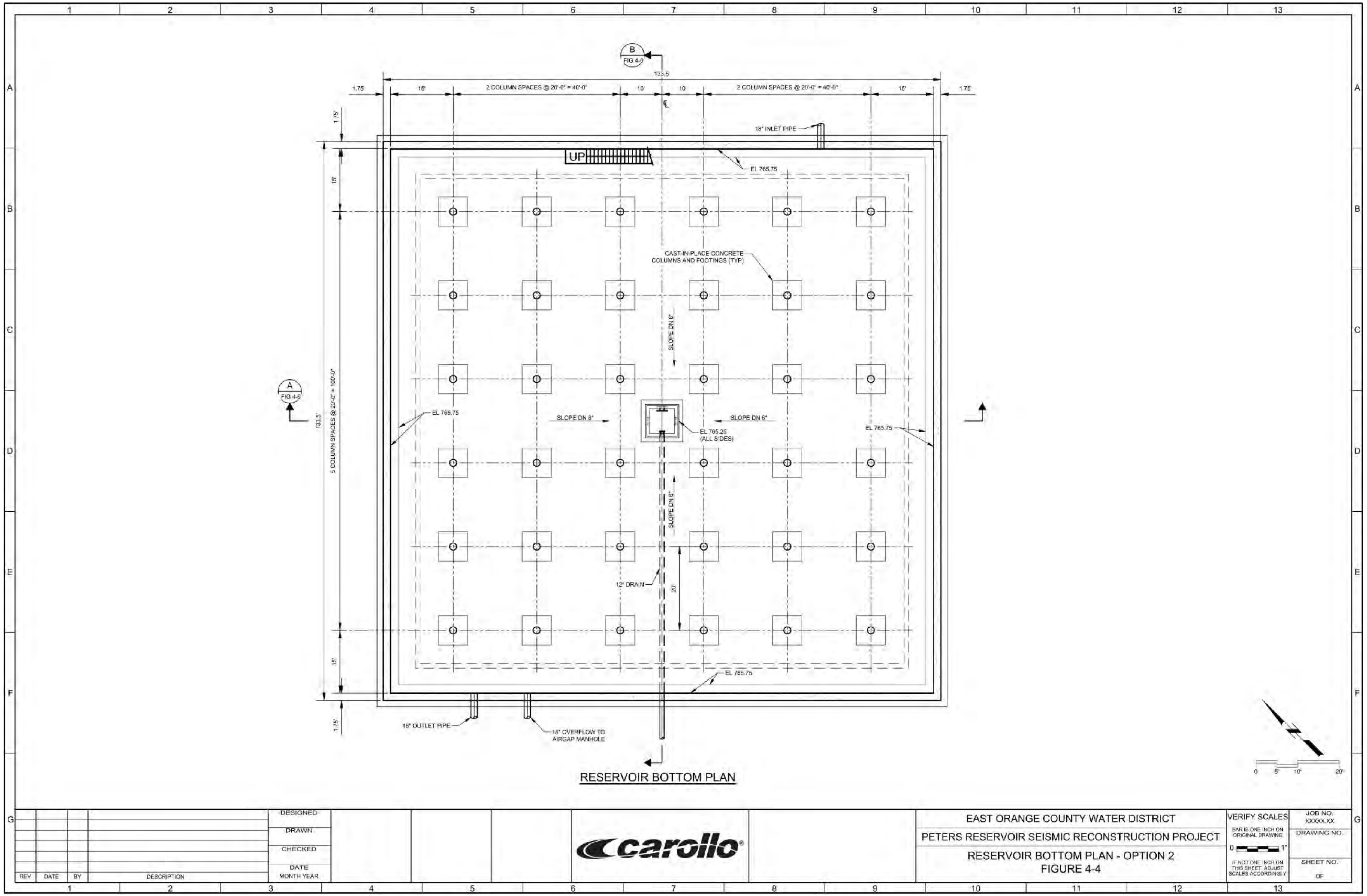


Figure 29 Option 2 Conceptual Structural Reservoir Bottom Plan

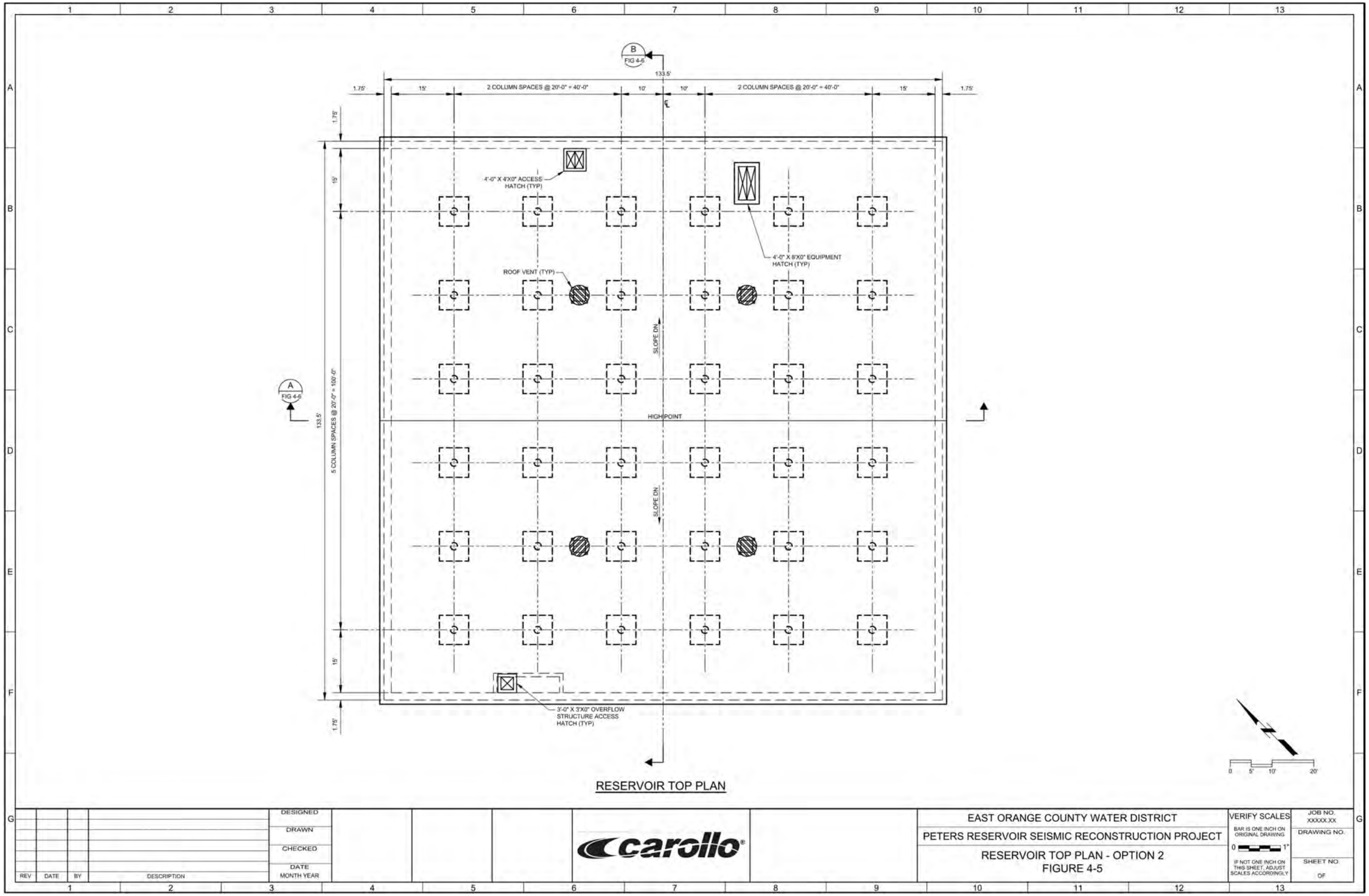


Figure 30 Option 2 Conceptual Structural Reservoir Top Plan

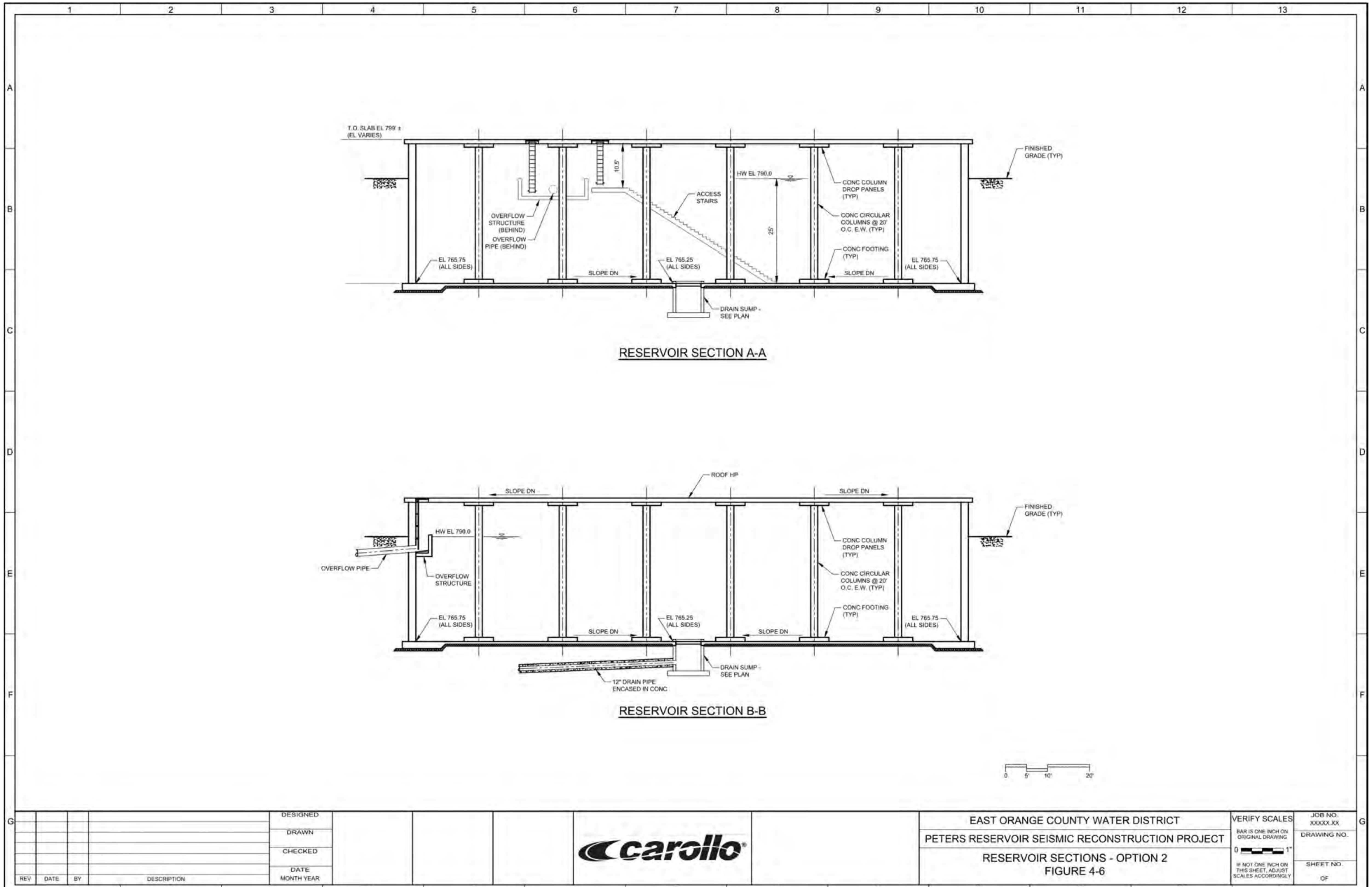


Figure 31 Option 2 Conceptual Structural Reservoir Sections

Figure 32 Option 3 Conceptual Site Layout

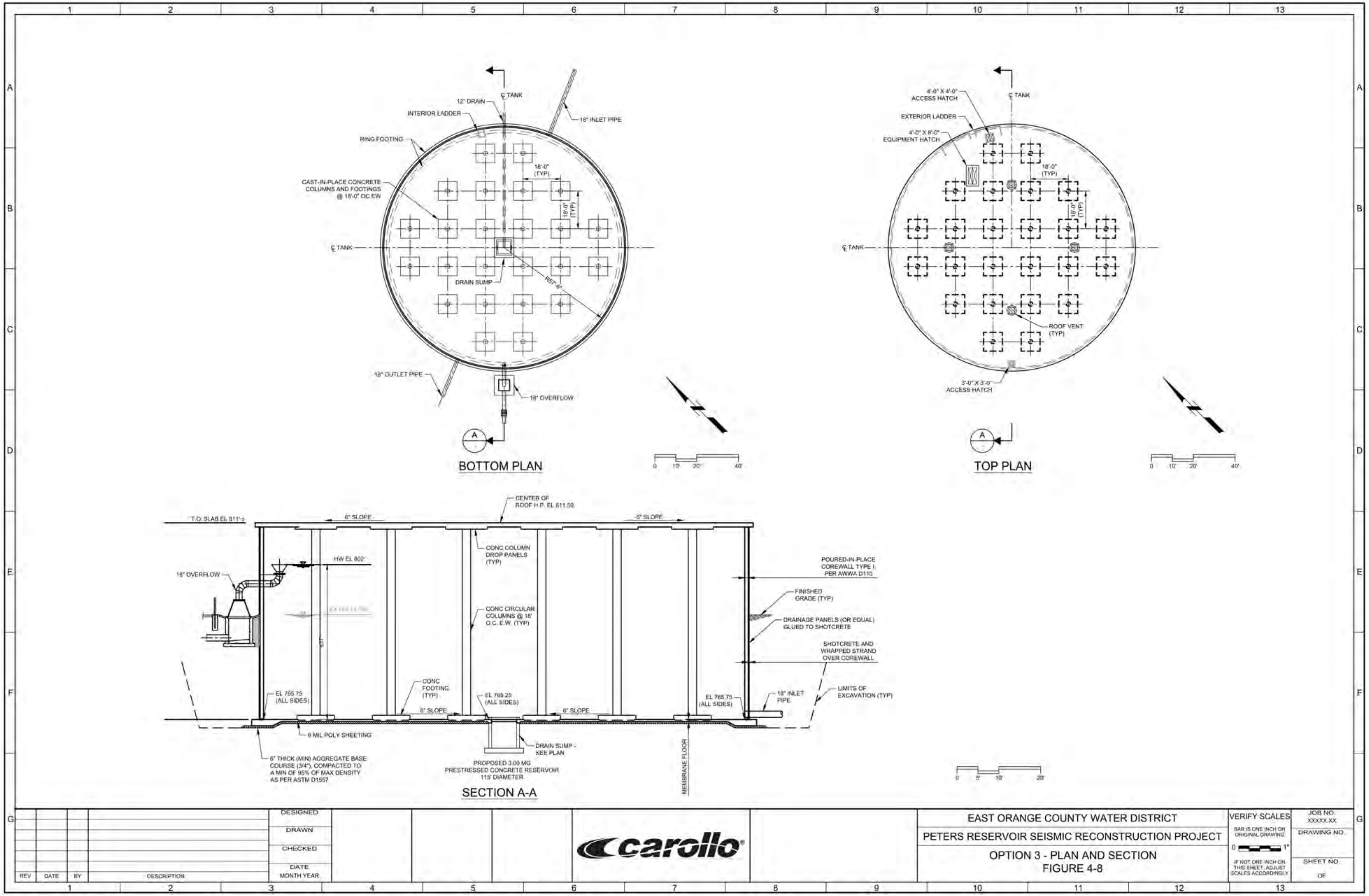


Figure 33 Option 3 Conceptual Structural Plans and Section

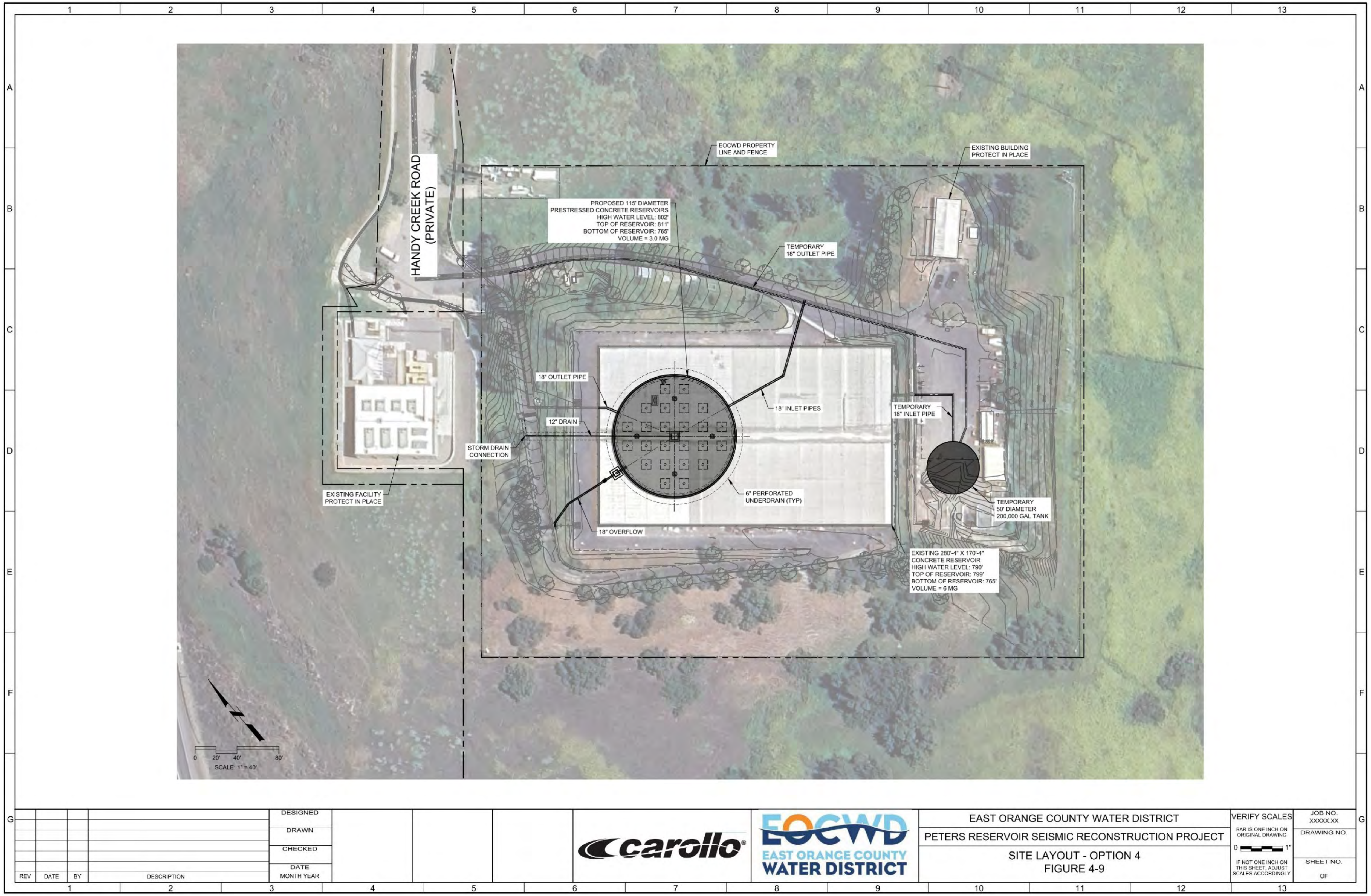


Figure 34 Option 4 Conceptual Site Layout

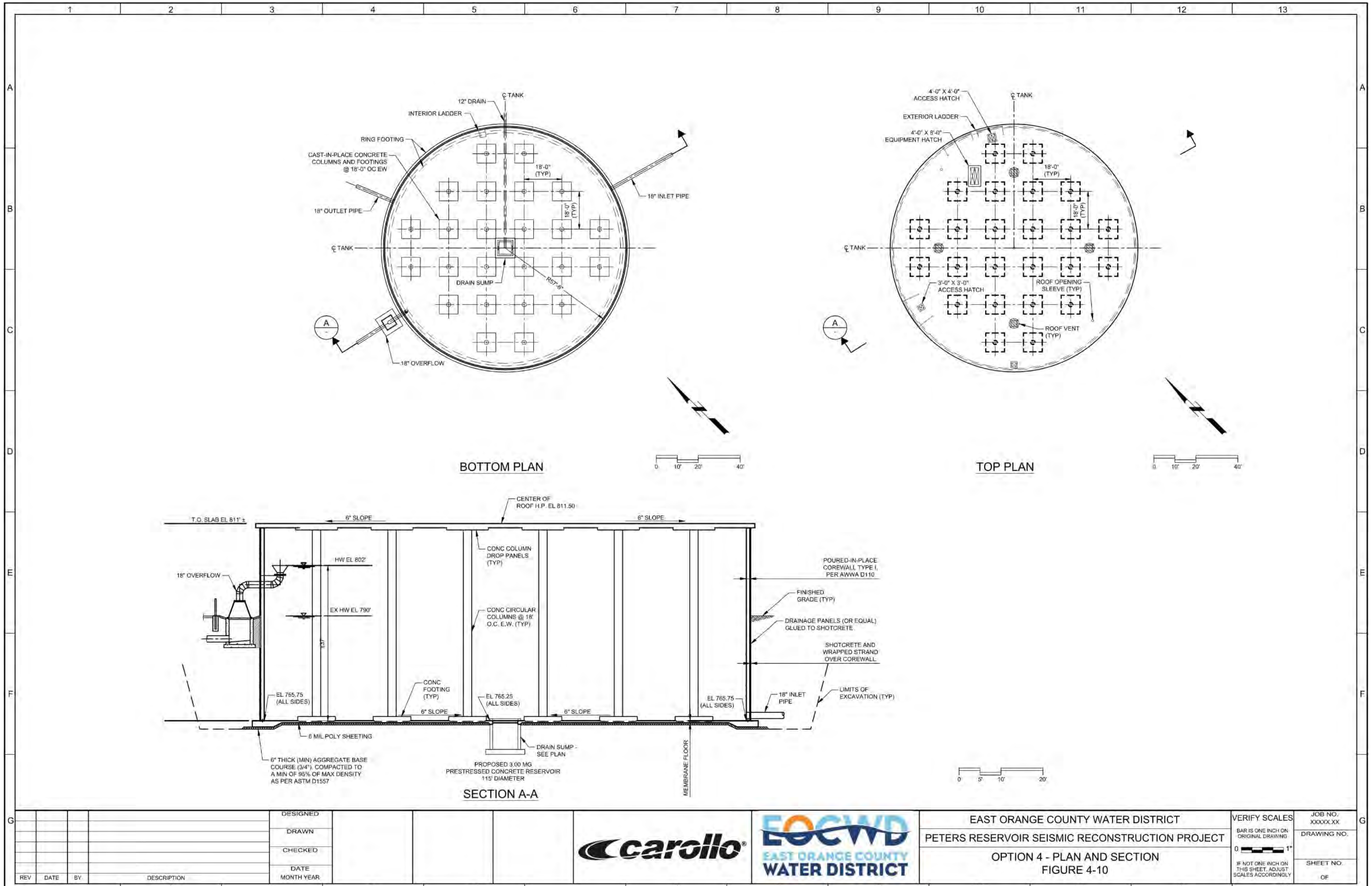


Figure 35 Option 4 Conceptual Structural Plans and Section

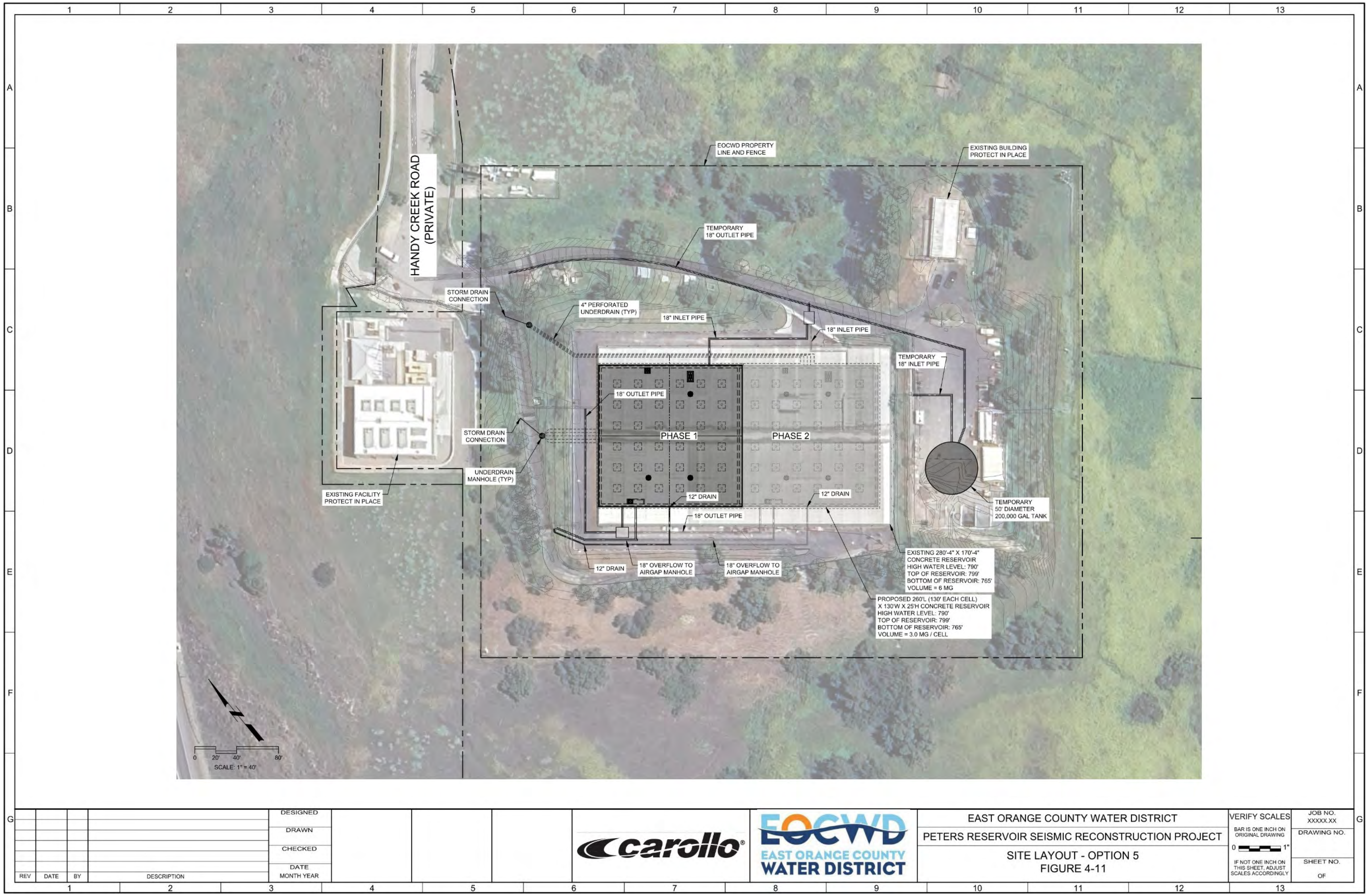


Figure 36 Option 5 Conceptual Site Layout

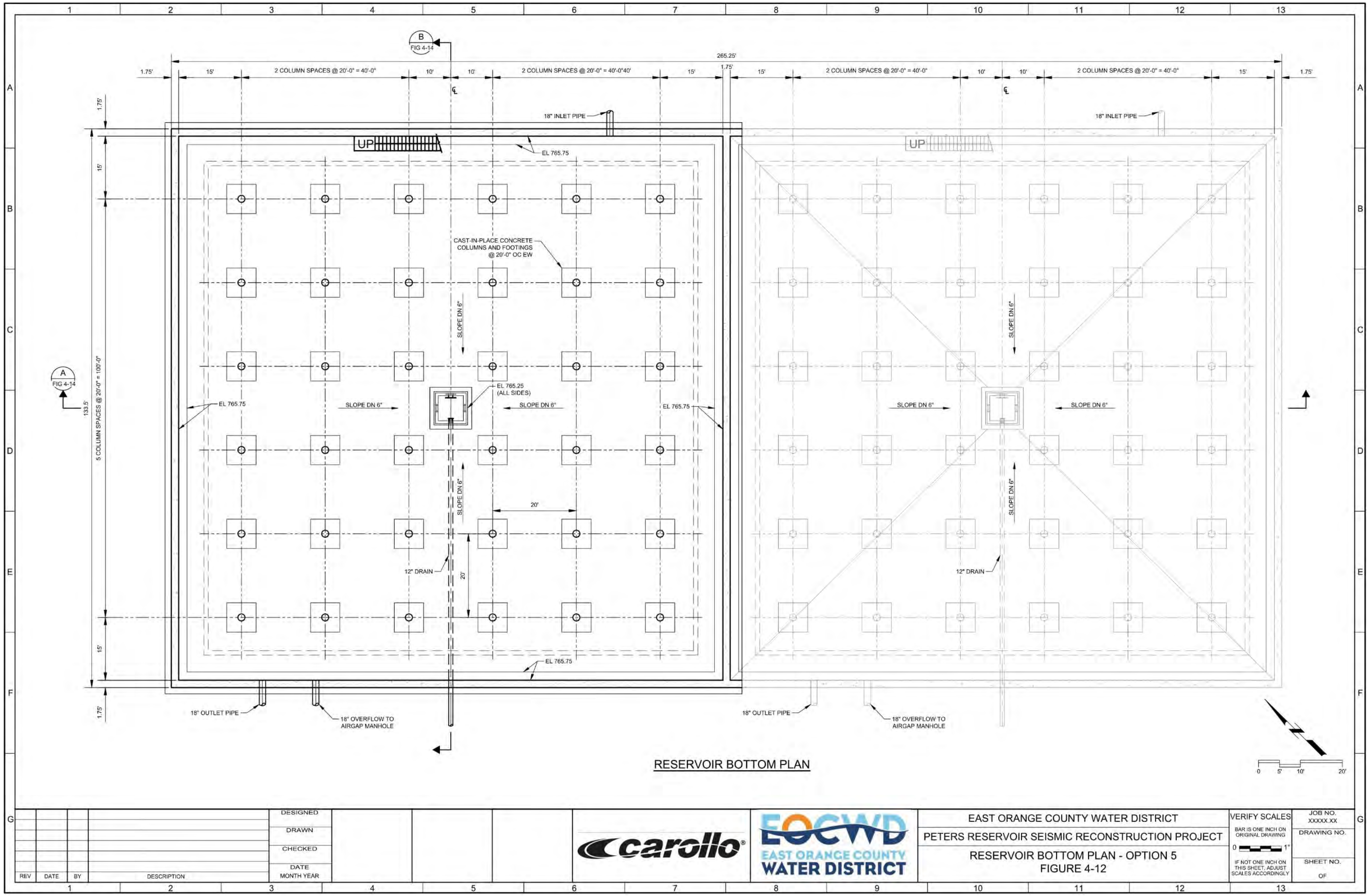


Figure 37 Option 5 Conceptual Structural Reservoir Bottom Plan

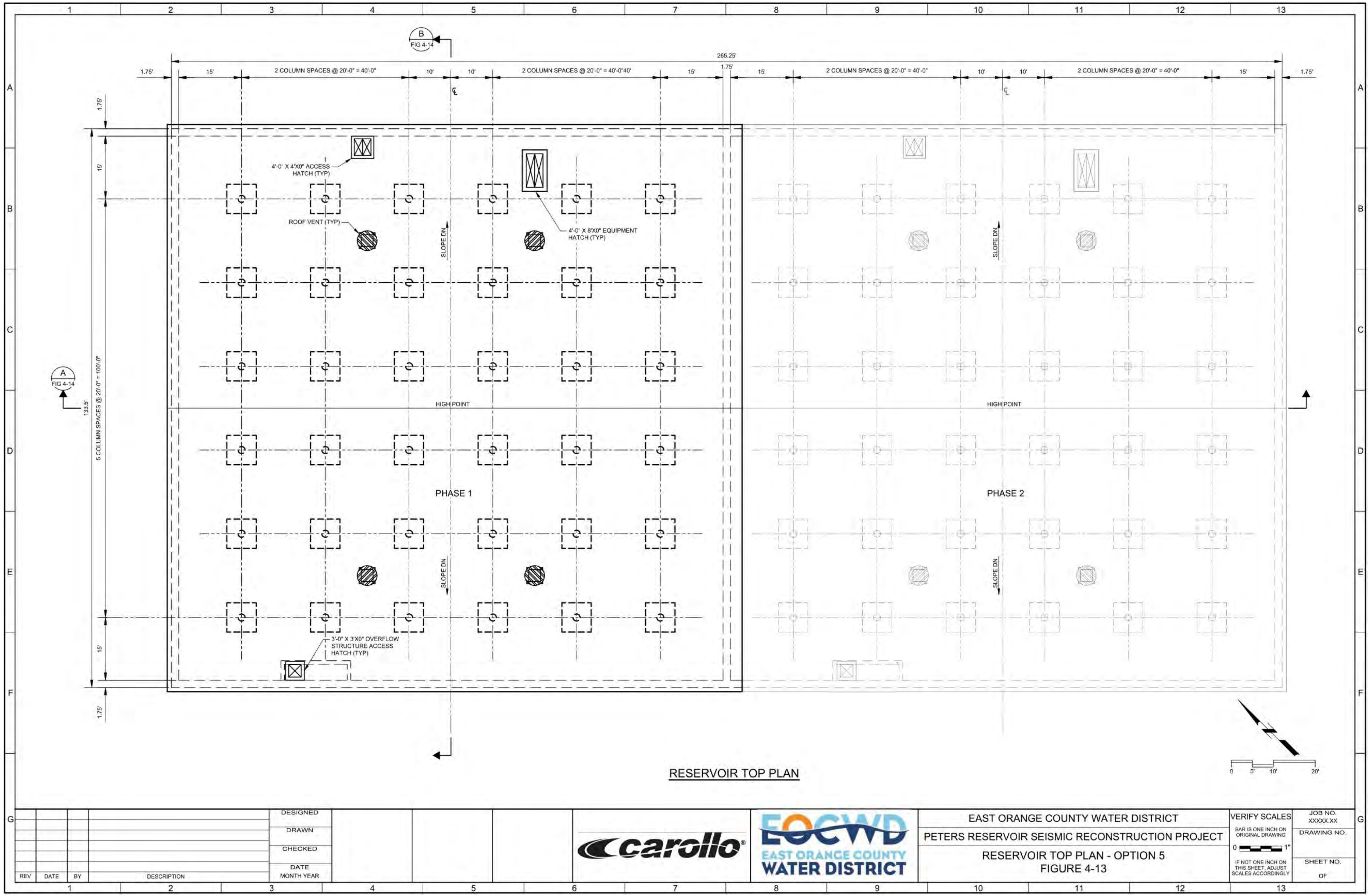


Figure 38 Option 5 Conceptual Structural Reservoir Top Plan

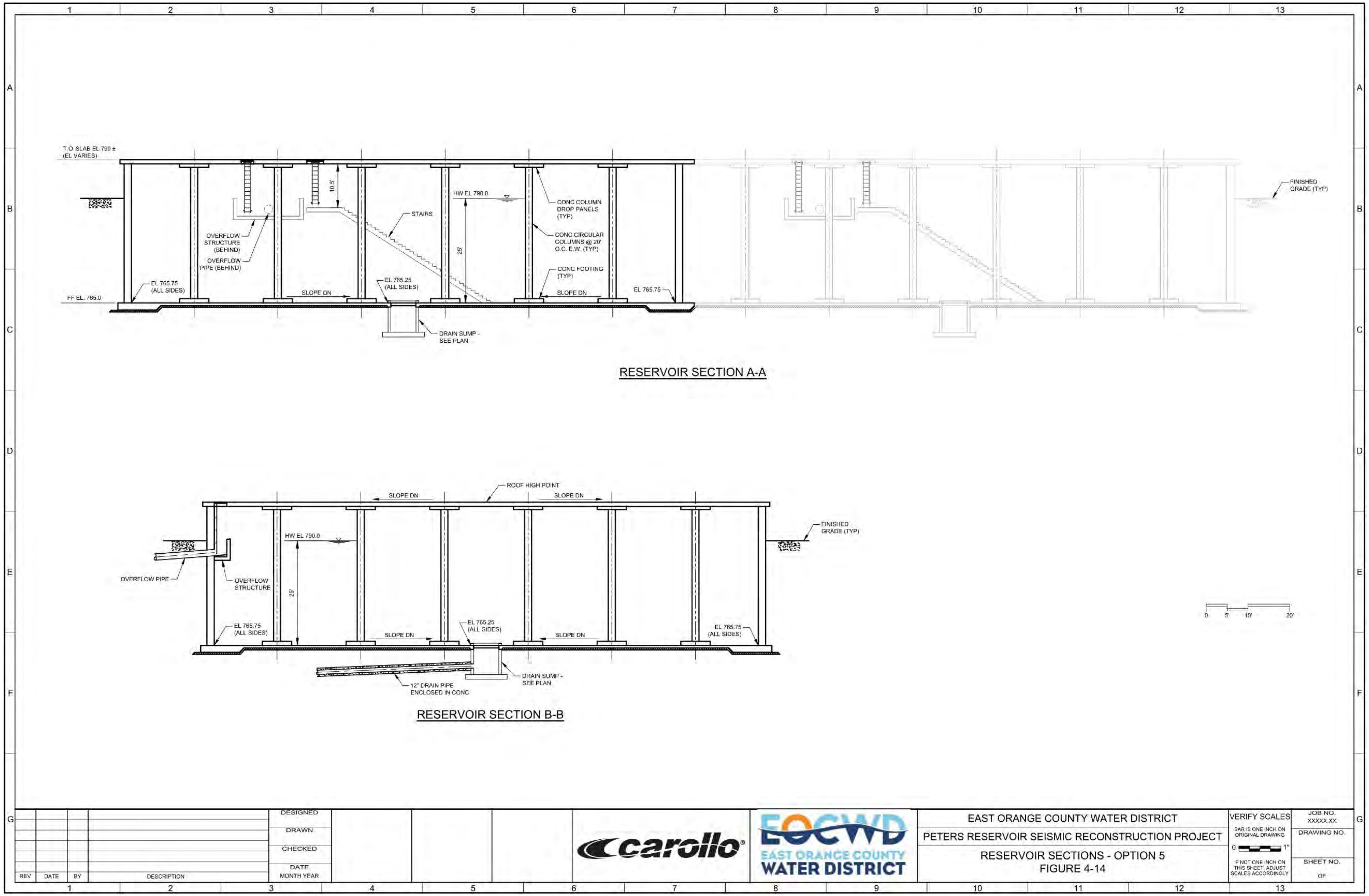


Figure 39 Option 5 Conceptual Structural Reservoir Sections

4.2 Inlet/Outlet Piping Configuration Considerations

4.2.1 Option 1 – One 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

Inlet Pipe: The existing reservoir inlet pipe is an 18-inch welded steel pipe that enters the reservoir on the southeast corner. The inlet pipe is placed above the reservoir floor along the hopper bottom and extends all the way into the reservoir floor. There is also a concrete flow dissipator at the end of the inlet pipe. The new reservoir in this option will have a flat bottom (no hopper) and will have the inlet pipe enter the reservoir through the floor or the wall of the reservoir.

For this option, the existing 18-inch inlet pipe will be extended to enter the new reservoir in a location near the southeast corner.

Outlet Pipe: The existing reservoir outlet pipe is a 21-inch welded steel pipe located on the northeast side of the reservoir and connects to the existing 21-inch welded steel pipe.

For this option, the new reservoir outlet pipe will be located on the opposite side of the inlet pipe to allow for sufficient mixing of the water within the reservoir. The outlet pipe will be an 18-inch welded steel pipe and will connect to exiting 21-inch reservoir outlet pipe within the reservoir site.

Reservoir inlet and outlet piping will be concrete encased if placed under the reservoir. See Figure 26 for conceptual layout of the inlet/outlet piping and Figure 40 for similar inlet and outlet piping details.

4.2.2 Option 2 – One 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

For this option, the reservoir inlet/outlet piping will be similar to Option 1.

Inlet Pipe: For this option, the existing 18-inch inlet pipe will be extended to enter the new reservoir in a location near the southeast corner.

Outlet Pipe: The new reservoir outlet pipe will be located on the opposite side of the inlet pipe to allow for sufficient mixing of the water within the reservoir. The outlet pipe will be a 18-inch welded steel pipe and will connect to exiting 21-inch reservoir outlet pipe within the reservoir site.

Reservoir inlet and outlet piping will be concrete encased if placed under the reservoir. See Figure 28 for conceptual layout of the inlet/outlet piping and Figure 40 for similar inlet and outlet piping details.

4.2.3 Option 3 – Two 3.0-MG Circular Prestressed Reservoirs

Inlet Pipe: The existing 18-inch inlet piping shall be modified in order to feed both reservoirs. A separate valving system and valve vault will be required to allow each reservoir to be filled via the same line when needed.

Outlet Pipe: Each 3.0-MG reservoir will have its own 18-inch outlet pipe. For both reservoir outlets, similar to the inlet piping, a separate valving system and valve vault will be required. The valve system will connect to the existing reservoir outlet piping system within the site.

See Figure 32 for conceptual layout of the inlet/outlet piping and Figure 40 and for similar inlet and outlet piping details.

4.2.4 Option 4 – One 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

For this option, the reservoir inlet/outlet piping will be similar to Option 1.

Reservoir inlet and outlet piping will be concrete encased if placed under the reservoir. See Figure 34 for conceptual layout of the inlet/outlet piping and Figure 40 for similar inlet and outlet piping details.

4.2.5 Option 5 – Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

For this option, the reservoir inlet/outlet piping will be similar to Option 3.

Inlet Pipe: The existing 18-inch inlet piping shall be modified in order to feed each reservoir cell. A separate valving system and valve vault will be required to allow each reservoir to be filled via the same line when needed.

Outlet Pipe: The new reservoir outlet pipe will be located on the opposite side of the inlet pipe to allow for sufficient mixing of the water within the reservoir. The outlet pipe will be an 18-inch welded steel pipe and will connect to the existing 21-inch reservoir outlet pipe within the reservoir site. Similar to the inlet piping, a separate valving system and valve vault will be required. The valve system will connect to the existing reservoir outlet piping system within the site.

See Figure 36 for conceptual layout of the inlet/outlet piping and Figure 40 for similar inlet and outlet piping details.

4.3 Reservoir Drain Piping

4.3.1 Option 1 – One 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

For this option, a central sump will be located in the center of the reservoir. The sump will be approximately 5 feet long by 5 feet wide and 4 feet deep. The reservoir floor will be sloped downwards toward the center sump. A 6-inch-high curb will be provided around the sump to prevent mud getting into the drainpipe. A mud gate will also be installed on each side of the sump within the curb. These mud gates are made of stainless steel plates and their purpose is to prevent sand and mud from flowing into the sump and drain valve during reservoir dewatering operations.

A valve will be installed within the sump and will remain closed during normal operation. The valve will only be closed when the reservoir is out of service and during cleaning operations. The reservoir drainpipe will be a 12-inch welded steel pipe that will run from the center sump and under the reservoir to a point of connection to the existing storm drain system. The drainpipe will be concrete encased under the reservoir floor.

See Figure 26 for a conceptual layout of the reservoir drain piping and Figure 41 for interior sump details.

4.3.2 Option 2 – One 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

Drain piping for this option will be similar to Option 1.

See Figure 28 for a conceptual layout of the reservoir drain piping and Figure 41 for interior sump details.

4.3.3 Option 3 – Two 3.0-MG Circular Prestressed Reservoirs

Drain piping for this option will be similar to Option 1, with one drain line for each reservoir. The drain lines will connection via tee connection on the north side of the tanks and flow towards the existing drainage system at the northwest corner of the site.

See Figure 32 for a conceptual layout of the reservoir drain piping and Figure 41 for interior sump details.

4.3.4 Option 4 – One 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

Drain piping for this option will be similar to Option 1.

See Figure 34 for a conceptual layout of the reservoir drain piping and Figure 41 for interior sump details.

4.3.5 Option 5 – Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

Drain piping for this option will be similar to Option 3, with one drain line for each reservoir cell. The drain lines will connection via tee connection on the south side of the tanks and flow towards the existing drainage system at the southwest corner of the site.

See Figure 36 for a conceptual layout of the reservoir drain piping and Figure 41 for interior sump details.

4.4 Reservoir Overflow Considerations

4.4.1 Option 1 – One 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

The reservoir overflow piping will be a vertical pipe placed inside the reservoir and supported by brackets that will be fastened to the reservoir wall, with top of pipe set at the high water level of 790 feet. The pipe will penetrate through the wall (or drop through the reservoir floor and be encased in concrete below the floor slab). The purpose of this overflow structure is to avoid filling the reservoir to the underside of the roof slab and eliminate pressurizing the reservoir roof slab. Overflow pipes should be properly sized and extend to ground level, complete with a weir box. If an underground drain is present, it should be sealed and an air break installed. An air gap between the overflow pipe and observation manhole will be provided and a screen and flapper valve, in accordance with AWWA, National Fire Protection Association (NFPA) 22, Occupational Safety and Health Administration (OSHA), and EPA, needs to be installed over the opening of the overflow pipe to prevent the ingress of contaminates into the water supply. An access hatch for observation will also be provided from the roof of the reservoir. See Figure 40 for typical overflow piping details. The airgap manhole will be placed at the bottom of the slope due to elevation requirements.

4.4.2 Option 2 – One 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

For Option 2, it is recommended that a concrete overflow structure (box type) be constructed within the reservoir. The advantage of a concrete overflow structure versus a pipe overflow is that concrete does not require future maintenance. The pipe-type overflow structures require periodic re-coating.

The reservoir overflow structure will be 3 feet wide by approximately 10 feet long, cantilevered from the reservoir wall. The purpose of this overflow structure is to avoid filling the reservoir to the underside of the roof slab. The three side walls of the overflow structure will be set at high water level of 790 feet. The bottom elevation of the overflow structure will be 786 feet. An 18-inch diameter overflow pipe will be provided from the overflow structure and will allow overflow into an observation manhole. An air gap between the overflow pipe and observation manhole will be provided and a screen and flapper valve, in accordance with AWWA, NFPA 22, OSHA, and EPA, needs to be installed at the opening of the overflow pipe to prevent the ingress of contaminates into the water supply. The bottom of the overflow structure will be grouted and sloped towards the overflow pipe to facilitate drainage. Figure 43 shows a preliminary plan and section of the overflow structure. An access hatch for observation will be provided from the roof of the reservoir. The airgap manhole will be placed at the bottom of the slope due to elevation requirements.

4.4.3 Option 3 – Two 3.0-MG Circular Prestressed Reservoirs

The Option 3 reservoir overflow will be similar to Option 1, with one overflow drain line for each reservoir. The overflow drain lines will connect via tee connection on the south side of the tanks and flow towards the existing drainage system at the southwest corner of the site. Due to the high water elevation of 802 feet for this option, there is sufficient clearance for the airgap manhole to be placed adjacent to each reservoir. See Figure 40 for typical overflow piping details.

4.4.4 Option 4 – One 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

The Option 4 reservoir overflow will be similar to Option 1. See Figure 40 for typical overflow piping details.

4.4.5 Option 5 – Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

The Option 5 reservoir overflow will be similar to Option 2, with one overflow drain line for each reservoir. The overflow drain lines will connect via tee connection on the south side of the tanks and flow towards the existing drainage system at the southwest corner of the site. See Figure 43 for typical overflow plan and section.

4.5 Reservoir Access

The main access into the reservoir interior of Options 1, 3, and 4 will be via stainless steel ladder located along the exterior wall of each reservoir. The access will be from a 4-foot-by-4-foot maintenance hatch located at the roof of each reservoir. The access doors will be located directly above the reservoir top slab. Each reservoir will also have one 4-foot-by-8-foot equipment access opening with hatch covers. The access hatch size is sufficient to allow lowering of a "bobcat" into the reservoir for maintenance and repairs when the reservoir is dewatered. The location of the equipment hatches will be coordinated with District operation's staff during the final design phase. Access to the reservoir roof will be via exterior ladder and a safety cage if needed. Three-foot-by-three-foot hatches will provide access to the overflow structure inside each reservoir. Figure 42 shows preliminary details of the reservoir access.

The main access into each reservoir cell of Options 2 and 5 (the rectangular reservoirs) will be via two sets of concrete stairs located along the north wall. The access will be from maintenance hatches at the roof of the reservoir. The access doors will be located directly above the reservoir top slab. The reservoir will also have two 4-foot-by-8-foot equipment access openings. The access hatch size is sufficient to allow lowering of a "bobcat" into the reservoir for maintenance and repairs when the reservoir is dewatered. The location of the equipment hatches will be coordinated with District operations staff during final design phase. Three-foot-by-three-foot hatches will provide access to the overflow structure inside each reservoir.

4.6 Underdrain Piping

The purpose of the underdrain piping is to remove groundwater and allow detection of any leakage as well as to determine approximately where the leakage may be coming from. A multi-zone underdrain piping system may be recommended under the selected reservoir. Underdrain piping will be of 4-inch perforated polyvinyl chloride piping.

In Options 1, 3, and 4 (circular prestressed tanks), all underdrain piping will be routed into an observation manhole located outside of the reservoir, allowing inspection of water due to leaks or ground water. A common observation manhole may be used for the reservoir overflow and underdrain piping. Figure 44 shows the preliminary reservoir underdrain layout for Option 1. Options 3 and 4 will have a similar underdrain layout.

In Options 2 and 5 (rectangular cast-in-place tanks), four zone underdrain piping systems will be recommended under the reservoir. The underdrain piping zones will be placed in the following regions under the reservoir:

- Zone 1: Around all exterior walls.
- Zone 2: Between zones 1 and 3.
- Zone 3: Between zones 2 and 4.
- Zone 4: Around the drain sump.

All underdrain piping from the four zones will be routed into an observation manhole located at the southwest end of the site, allowing inspection of water due to leaks or ground water. A common observation manhole will be used for the reservoir overflow and underdrain piping. Figure 45 shows the preliminary reservoir underdrain layout for Option 2. Option 5 will have a similar underdrain layout.

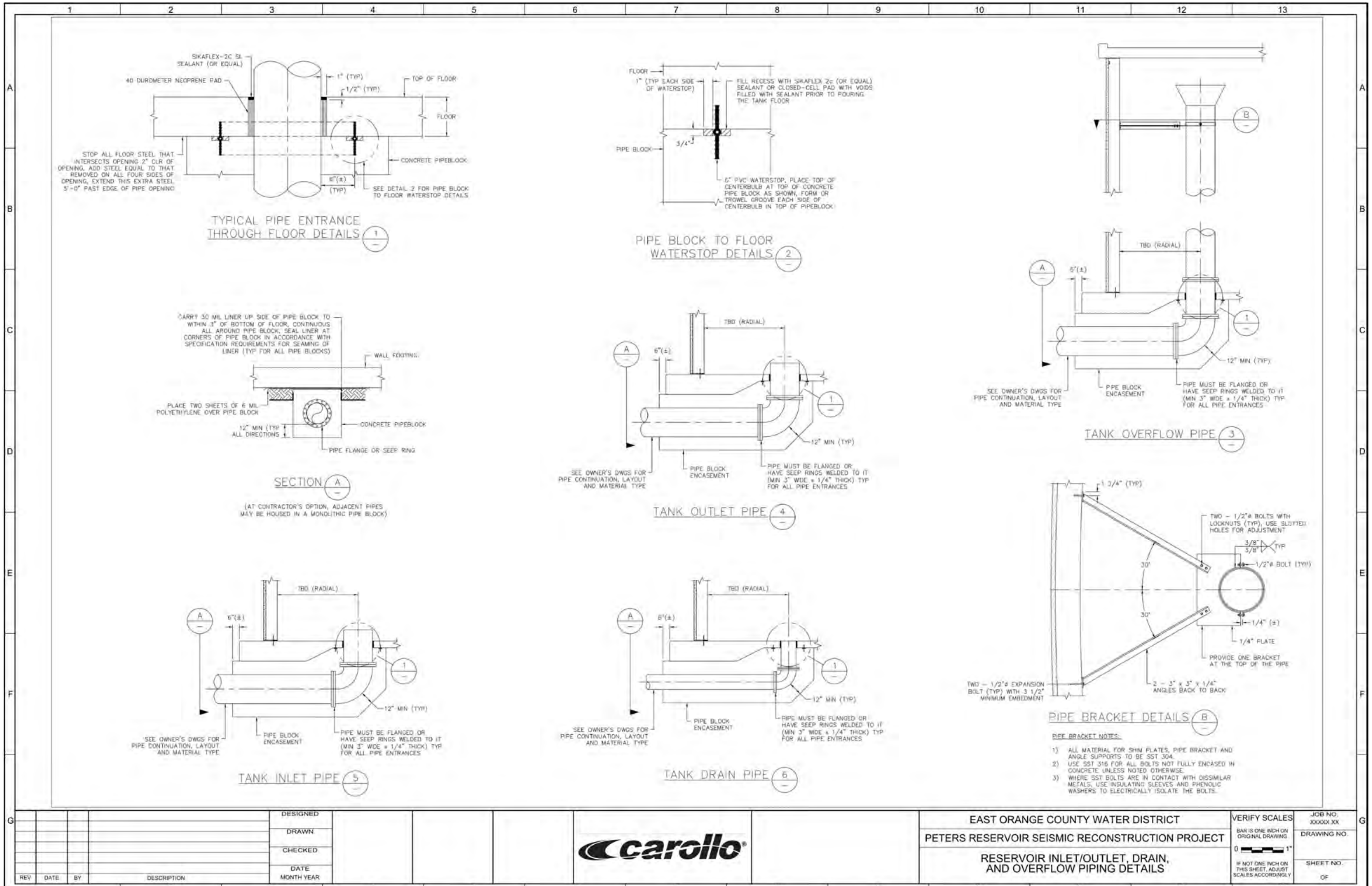


Figure 40 Reservoir Inlet/Outlet, Drain, and Overflow Piping Details

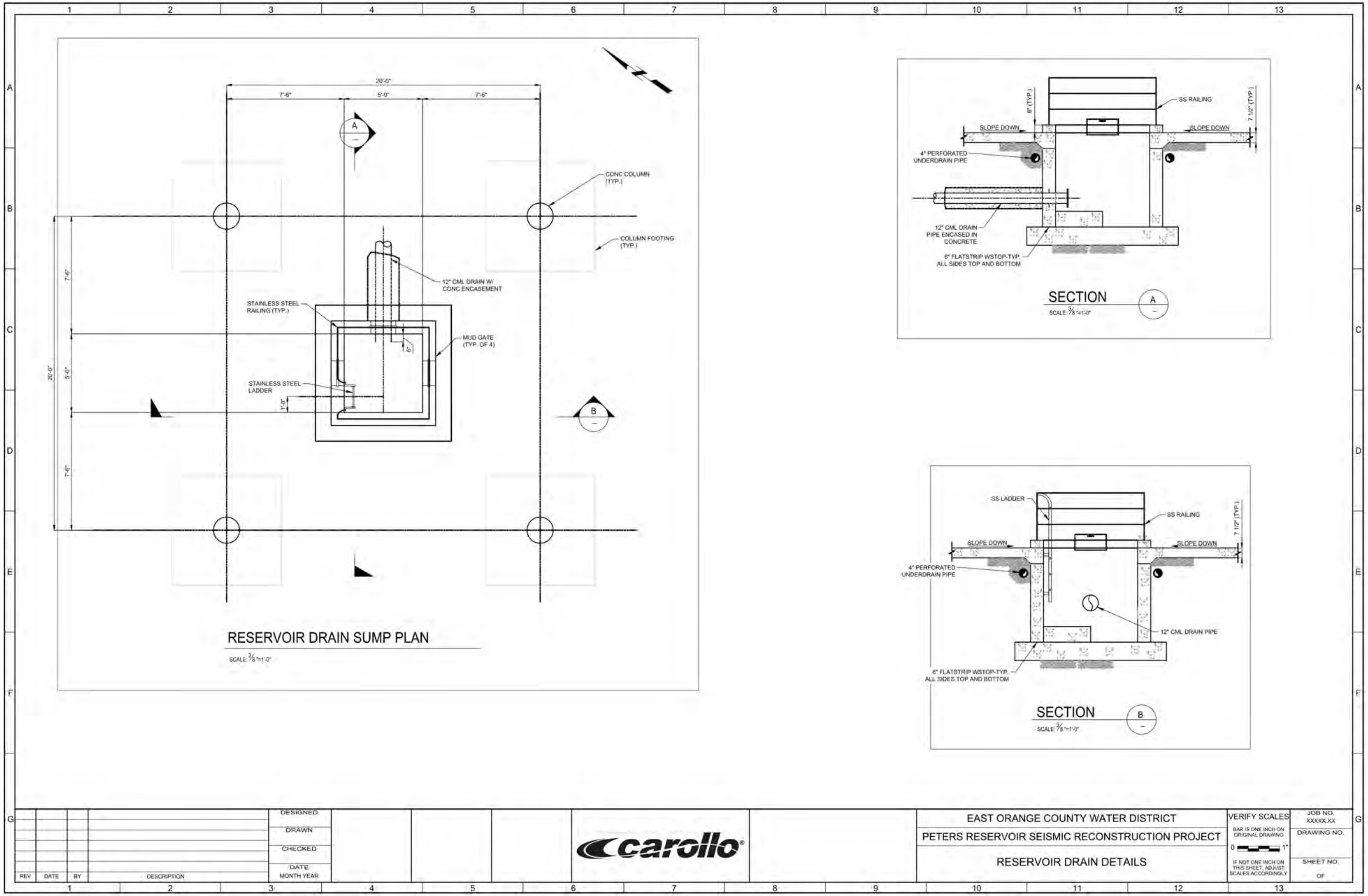


Figure 41 Reservoir Drain Sump Plan and Sections

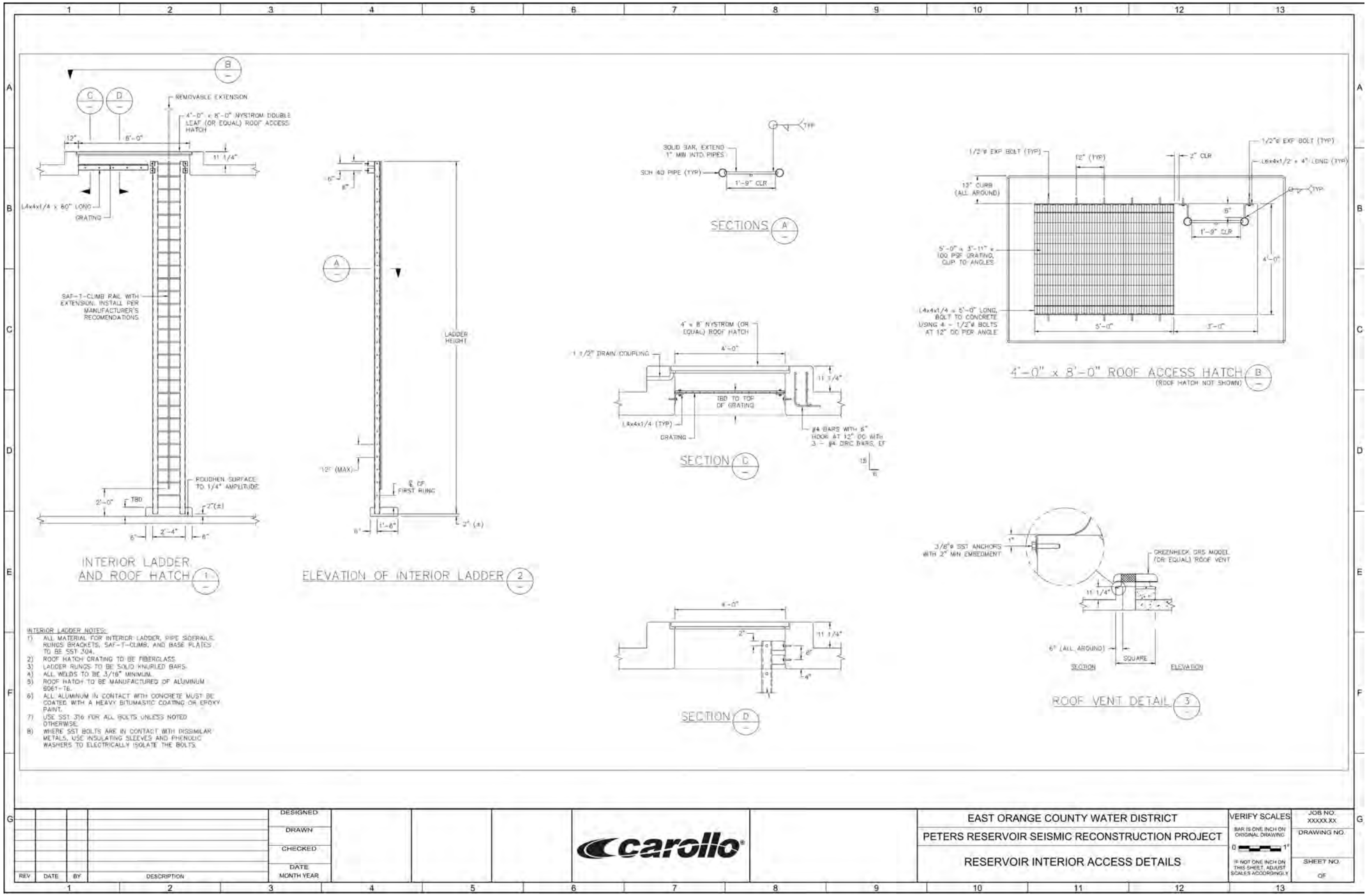


Figure 42 Options 1, 3, and 4 Reservoir Interior Access Details

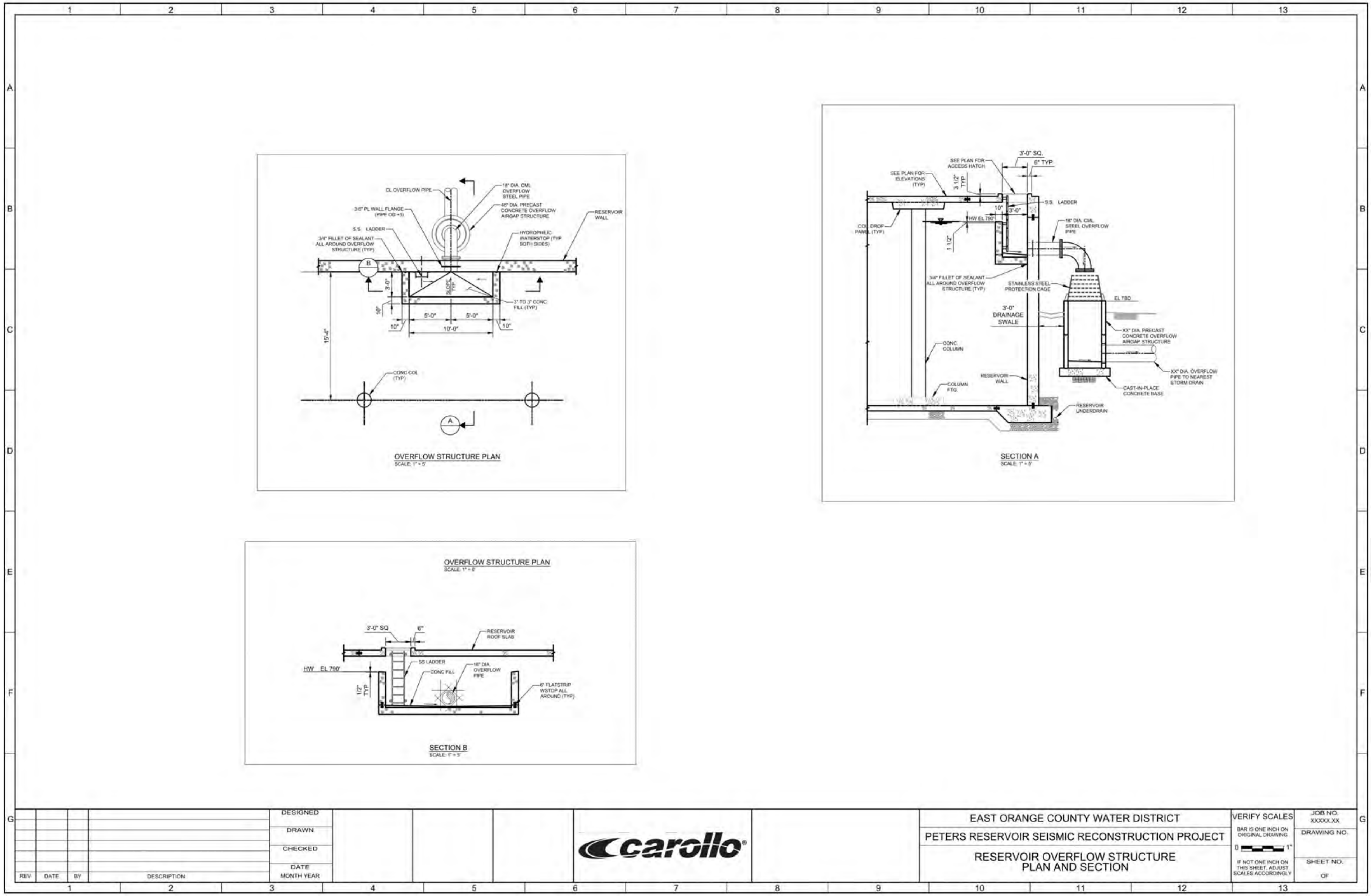


Figure 43 Options 2 and 5 Reservoir Overflow Details

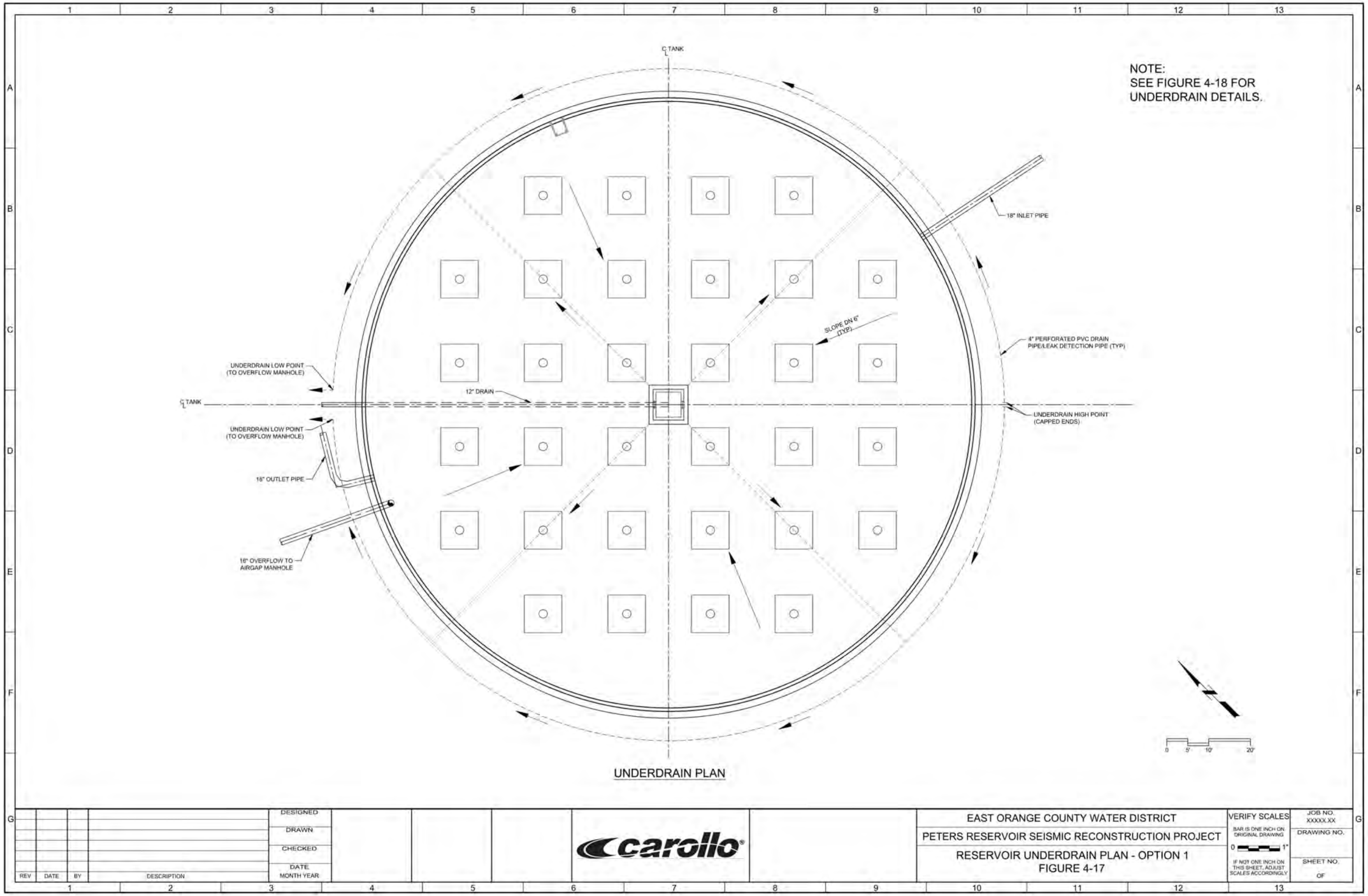


Figure 44 Option 1 Reservoir Underdrain System Layout

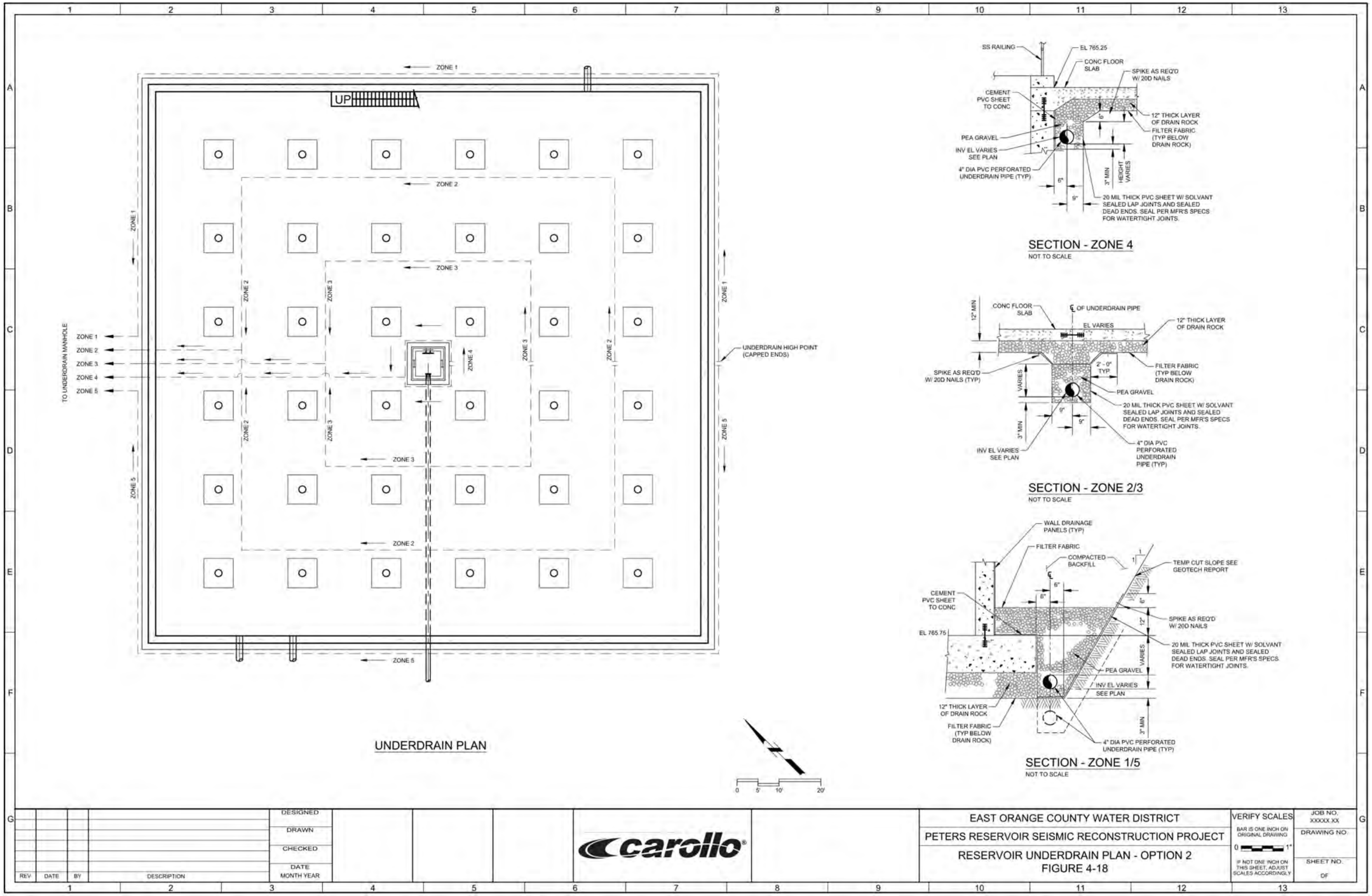


Figure 45 Option 2 Reservoir Underdrain System Layout

SECTION 5 RESERVOIR STRUCTURAL DESIGN CRITERIA

5.1 Introduction

The existing 6.0-MG potable water storage tank sits midway up a slope located at 8548 East Canyon View Avenue, Orange, California. An alternate site address in common use is 2323 Handy Creek Road, Orange, California. The aging water storage tank will be replaced in order to better serve customers and provide the desired emergency storage requirements. A previous seismic retrofit was performed on the tank site to stabilize the retaining wall at the easterly end of the tank. Boyle Engineering Corporation designed rock anchors in their 1999 Seismic Retrofit project. In May of 2015 Carollo Engineers conducted an evaluation of the roof and noted that repairs or replacement were required to address the corrosion of the roof connections. Ultimately, critical connections between roof members were replaced. The aging condition of the reservoir, along with identified seismic vulnerabilities, warrants replacement.

The project location is in a region of high seismic activity, and on a site with a number of seismic design concerns. The relevant primary and secondary seismic hazards are discussed in addition to the seismic response spectrum and the seismic behavior of fluid retaining structures. The design of the reservoir will require all of the discussed parameters, hazards, and pertinent design documents to be considered.

5.2 Design Codes and Standards

Reservoirs containing drinking water and water stored for emergency uses located in highly seismically active regions are subject to several design codes and standards. This project will be designed using the following documents. Some documents are material-specific and will not apply to all the design options evaluated.

- American Society of Civil Engineers (ASCE) 7-22: Minimum design Loads and Associated Criteria for Buildings and Other Structures (American Society of Civil Engineers, 2022).
- California Building Code (CBC) 2025 (California Building Standards Commission, 2025).
- American Concrete Institute (ACI) 318-19: Building Code Requirements for Structural Concrete (American Concrete Institute Committee 318).
- ACI 350-20: Code Requirements for Environmental Engineering Concrete Structures (American Concrete Institute Committee 350).
- ACI 350.3-20: Code Requirements for Seismic Analysis and Design of Liquid-Containing Concrete Structures (American Concrete Institute Committee 350).
- AWWA D110: Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks (American Water Works Association, 2018).
- AWWA D100: Welded Carbon Steel Tanks for Water Storage (American Water Works Association, 2021).

5.3 Dead Loads

Dead loads at a minimum will include the self-weight of permanent structures, structure finishes, appurtenances, mechanical and electrical equipment, piping, cable tray, conduit, ductwork, and equipment pads. The dead load will be estimated on the specified proportions of members and their common densities, some of which are identified in Table 10.

Table 10 Material Density for Estimating Dead Loads

Material	Density
Aluminum	170 pcf
Concrete, plain	145 pcf (downward)/140 pcf (to resist buoyancy)
Concrete, reinforced	150 pcf
Soil	110-120 pcf
Steel	490 pcf
Water	62.4 pcf

Notes:
pcf - pounds per cubic foot

5.4 Secondary Seismic Hazards

Secondary seismic hazards specific to the tank site were investigated.

Surface fault rupture is the visible deflection of the ground surface following an earthquake. Surface fault rupture was deemed a low risk due to the proximity of the site to the nearest fault zones.

Liquefaction is a near-surface phenomenon where the shear strength is dramatically reduced in granular cohesionless soils coupled with the presence of high ground water. A dynamic settlement analysis conducted by Converse Consultants in 2025 determined that the potential for liquefaction is negligible. Additionally, the dynamic settlement analysis was also conducted for the dry condition. This test concluded that dynamic dry seismic settlement was negligible. Lateral spreads, associated with liquefaction, can occur in loose cohesionless soils in the saturated condition that rest on competent soil or bedrock. Ground motion can cause the loose topsoil layer to rapidly deteriorate. This phenomenon was investigated as a part of the dynamic settlement analysis and it was determined that lateral spreading risk was very low.

Due to the cut sloping nature of the project site and the previous retaining wall stability retrofit project, the risk of seismically induced landslides is low to moderate.

5.5 Seismic Design Criteria

5.5.1 Seismic Risk Category

ASCE 7-22 tabulates the consequence of environmental loads being exceeded for a structure's occupants or end users through a risk category. A risk category of I would represent a low risk to human life, while risk category of IV represents a severe risk to human life. The reservoir serves the surrounding communities with drinking water and stores water necessary in case of an emergency situation, such as a long-term water supply outage. This function of the reservoir necessitates classification into risk category IV. A high-risk category dictates the necessity of a more stringent set of performance requirements for a structure's seismic response. Structures in risk category IV need to sustain minimal or only superficial damage following an earthquake due to their impact on the local population. This adjustment to the performance is achieved through the application of an importance factor to the required seismic loads.

5.5.2 Parameters

The mapped seismic parameters are available for the site through the ASCE Online Hazard tool (ascehazardtool.org). The input of the site coordinates returns the parameters based on the 2025 California Building Code and ASCE 7-22. The mapped seismic parameters are presented in Table 11.

Table 11 CBC 2025 and ASCE 7-22 Seismic Design Parameters

Parameter	Value
Site Coordinates	33.7765N, 117.7552 W
Risk Category	IV
Site Class	D (Stiff Soil)
Mapped Short Period (0.2 seconds) Spectral Response Acceleration, S_S	1.610 g
Mapped 1-Second Spectral Response Acceleration, S_1	0.500 g
MCE 0.2-Second Period Spectral Response Acceleration, S_{MS}	1.840 g
MCE 1-Second Period Spectral Response Acceleration, S_{M1}	1.260 g
Design Spectral Response Acceleration, S_{DS}	1.230 g
Design Spectral Response Acceleration for 1-Second Period, S_{D1}	0.840 g
Peak Ground Acceleration, PGA_M	0.650 g

Notes:
g - acceleration due to Earth's gravity; MCE - maximum considered earthquake

5.6 Site-Specific Design Criteria

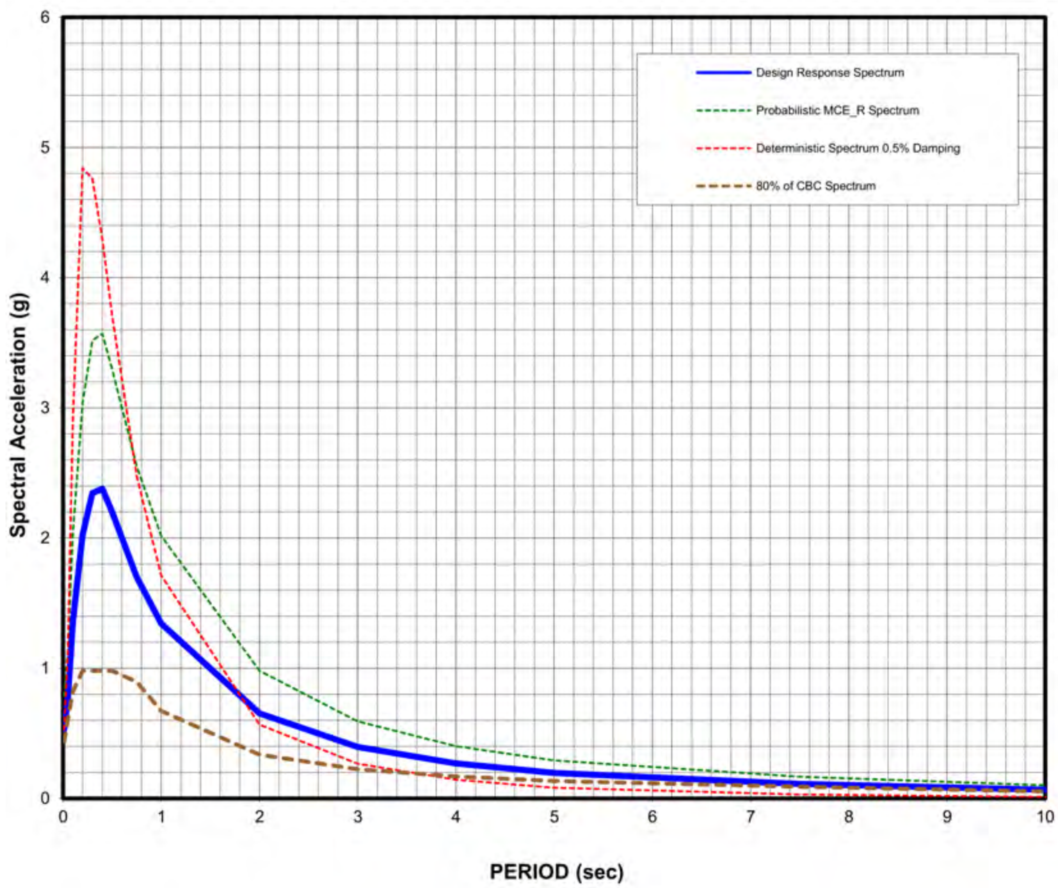
Site-specific design values are produced through a site-specific ground motion hazard analysis (SGMHA). The goal of SGMHA is to provide a seismic response spectrum tailored to the site's geological features and hazards. Although hazard maps are readily available, the response spectrum produced using the maps alone can underestimate or overestimate the response.

ASCE 7-16 required SGMHA be performed for all sites with soil site class D soil, due to the underestimation of long period ground motion. Alternatively, a 50 percent increase could be applied to the S1 parameter. Although ASCE 7-22 has altered the approach to appropriately attenuate the parameters, SGMHA can be useful to ensure the seismic parameters are relevant to the site and its use case.

The SGMHA performed for the tank site determines the average shear wave velocity of the site in the upper 30 meters. VS30 was calculated to be 301 meters per second or 987.5 feet per second. This value correlated the standard proctor test blow counts with the shear wave velocity. Ultimately, this results in a seismic response spectrum that meets the requirements set forth in ASCE 7-22. This response spectrum is two thirds of the MCE spectrum. The MCE is an earthquake that has a 2 percent probability of exceedance in 50 years and a 2,475-year return interval. These results for the 0.5 percent and 5.0 percent damping response spectrum are presented in Table 12. The 0.5 percent response spectrum is displayed in Figure 46 and the 5 percent response spectrum in Figure 47.

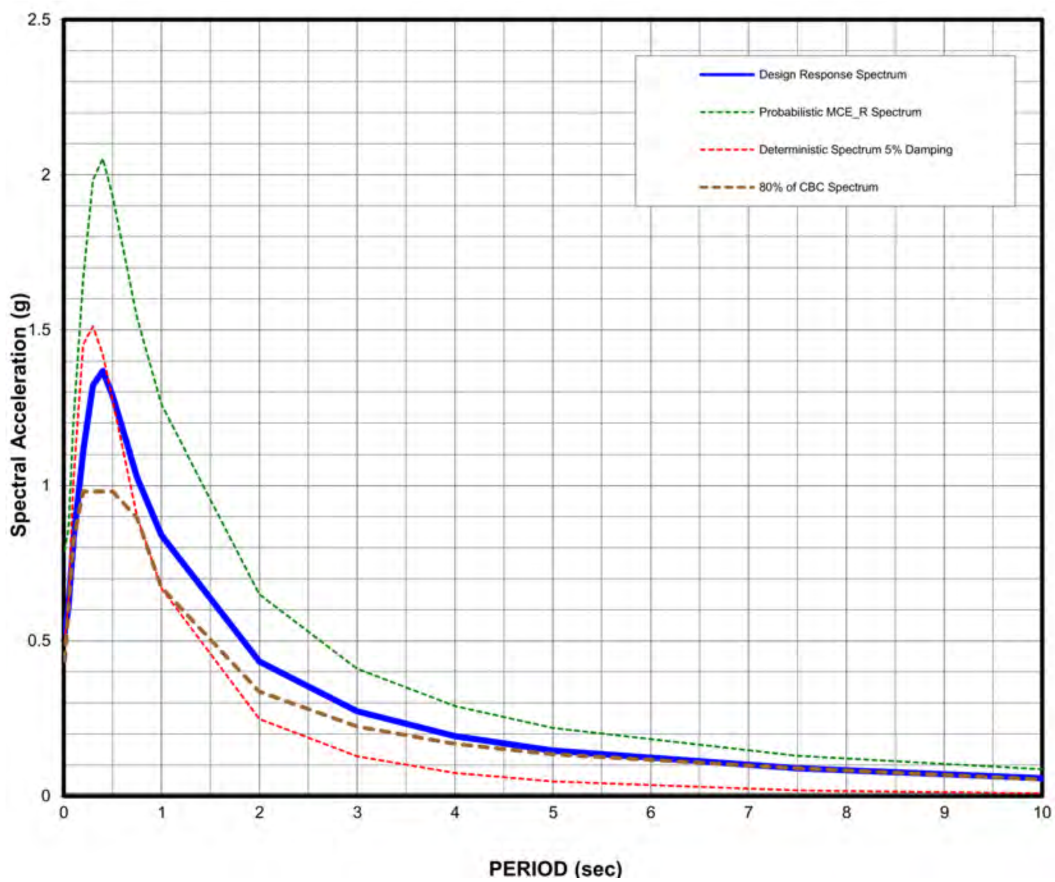
Table 12 CBC 2025 and ASCE 7-22 Site-Specific Seismic Design Parameters

Parameter	Design Parameters	80% of CBC Design
MCE_R (0.5% damped) spectral response acceleration for short period adjusted for site class, S_{MS}	3.212 g	1.472 g
MCE_R (0.5% damped) spectral response acceleration for 1-second period adjusted for site class, S_{M1}	2.016 g	1.008 g
Design spectral response acceleration (0.5% damped) acceleration at short periods, S_{DS}	2.141 g	0.981 g
Design spectral response acceleration (0.5% damped) at 1-second period, S_{D1}	1.344 g	0.672 g
MCE_R (5% damped) spectral response acceleration for short period adjusted for site class, S_{MS}	1.847 g	1.472 g
MCE_R (5% damped) spectral response acceleration for 1-second period adjusted for site class, S_{M1}	1.299 g	1.008 g
Design spectral response acceleration (5% damped) acceleration at short periods, S_{DS}	1.231 g	0.981 g
Design spectral response acceleration (5% damped) at 1-second period, S_{D1}	0.866 g	0.672 g



Source: Updated Geotechnical Letter Report, Converse Consultants

Figure 46 Site-Specific Response Spectrum, 0.5 Percent Damping



Source: Updated Geotechnical Letter Report, Converse Consultants

Figure 47 Site-Specific Response Spectrum, 5 Percent Damping

5.7 Reservoir Seismic Response

Unlike the seismic response of a building, the primary source of seismic loads in large water storage tanks is not generated from the inertia of the structure, but rather from hydrodynamic forces imparted onto the structure by the contained fluid. Hydrodynamic loading is composed of two dynamic modes: the short period impulsive response and the long period convective response.

The forces due to the impulsive hydrodynamic mode act as both a pressure force on the leading edge of the tank and as an opposing force on the trailing edge of the tank. Seismic ground motion accelerates the tank walls laterally. The retained fluid acts as a solid mass rigidly attached to the walls, due to the rapid change in direction associated with the high frequency vibrations that cause this hydrodynamic mode. These oscillations generate net lateral forces acting in the same cardinal direction on the interior face of the walls. Due to the short period nature of the impulsive hydrodynamic mode, the imposed forces change direction several times every second. The rapidly alternating directional changes of the impulsive forces are a result of the short period oscillations of ground motion.

The convective forces are imparted onto the walls due to the oscillations of the fluid. The fluid oscillations are generated by the impulses of the walls accelerating the fluid within the tank. The oscillating nature of the convective hydrodynamic mode can be visualized as a sloshing wave. The period of the sloshing wave is relative to the depth to diameter ratio or depth to length ratio of the tank. Generally, the convective period is several seconds long. The convective component of hydrodynamic force is based on the 0.5 percent damping spectrum, which differs from both the impulsive forces and inertial forces generated from the seismic mass of the tank structure. The impulsive and tank inertial forces are damped assuming a 5 percent damping, which is a typical damping ratio for most structures that are not fluid bearing or externally damped.

Vertical ground motions cause a piston like pounding phenomenon of the contained fluid. The vertical motion of the fluid causes a pulsating effect where the fluid hydrostatic pressure is increased due to the oscillating vertical acceleration. This motion occurs at the short period frequency component of the ground motion spectrum.

While generally, seismic loads due to structure inertia are far less than hydrodynamic loading, they cannot be ignored. The total seismic mass of the structure, including the walls, roof, and any appurtenances are the source of the inertial seismic loads. The seismic load case for design is taken as the square root sum of the squares of the impulsive, convective, vertical, and inertial seismic forces.

5.8 Material Selection

All materials in direct contact with potable water will need to comply with NSF 61. Any submerged metal appurtenances are to be constructed from stainless steel. Concrete, steel, and stainless steel properties are summarized in Tables 13 through 15.

Table 13 Concrete Properties

Concrete	
Structural Concrete	4,000 psi at 28 days
Concrete Fill and Encasement	2,500 psi at 28 days
Electrical Conduit Encasement	3,000 psi at 28 days
Pavement, Curbs, Gutters, and Sidewalks	2,500 psi at 28 days
Cement:	Type II Portland Cement per ASTM C150
Weight	Normal weight concrete = 150 pcf
Reinforcing Steel	ASTM A615, Grade 60 (ASTM A706, Grade 60, for welded bars)
Prestressing Wire	Galvanized 7 wire strand, Grade 250 ASTM A416 and A475
Concrete Cover on Rebar	Minimum required per ACI 318 and ACI 350

Notes:
ASTM - ASTM International

New steel design strengths for members and associated strengths shall be as listed in Table 14.

Table 14 Steel Properties

Steel	
Rolled Sections	▪ ASTM A992: Shapes W, WT » ASTM 36: Shapes S, ST, M, MT, HP, C, MC, L
Plates	▪ ASTM A36
Hollow Structural Sections	▪ Square and Rectangular: ASTM A500, Grade C, Fy = 50 ksi ▪ Round: ASTM F593
Pipe Sections	▪ ASTM A53, Grade B, Fy = 35 ksi
Bolts	▪ ASTM F3125, Grade A325, high-strength bolts with load indicator washers
Anchor Bolts	▪ ASTM F1554, Grade 36 Galvanized (in interior dry areas) » ASTM F593, Stainless Steel Type 316 (in areas considered wet or corrosive)
Welding	▪ E70XX low hydrogen electrodes, operators AWS qualified

Notes:
AWS – American Welding Society; ksi – thousand pounds per square inch

Table 15 Stainless Steel Properties

Stainless Steel	
General	ANSI Type 316/316L
Shapes and bars	ASTM A276
Bolts	Match alloy of the structural members connected
Anchor bolts	Type 316/316L: ASTM F593, Grade B8M, Class 1, Heavy Hex
Welding	Type 316L: E316L-15 electrodes

5.9 Roof Selection

Circular tank options can be constructed as either a column-supported flat roof or a domed roof. Either option can provide the required roof access and appurtenances. Rectangular tanks can be constructed with a flat roof or pitched roof. Flat rectangular roofs can be constructed out of either concrete or a beam-supported steel roof deck. All roofs would be designed for a minimum roof live load of 20 pounds per square foot (psf), as required by ASCE 7-22.

The load combinations will follow those listed in CBC 2025, with modifications as shown in ASCE 7-22, Chapter 2. Table 16 lists some of the critical load combinations that will be considered.

Table 16 Foundation Design Parameters

Parameter	
Shallow Footing Allowable Bearing Capacity	2,500 psf
Coefficient of Friction	0.35
Matt Foundation Bearing Capacity	3,500 psf
Passive Earth Pressure	220 psf
Active Earth Pressure	40 psf
At-Rest Earth Pressure	60 psf
Seismic Earth Pressure	22H psf (where H = the retained height in feet)

5.10 Foundation Design Parameters

The foundation for the tank will be designed according to the parameters in the report provided by Converse Consultants on November 26, 2025. The foundation design parameters are summarized in Table 16. The soil was tested for corrosion and sulfide content. The sulfide content is classified as S0 (ACI 318-19 Table 19.3.1.1), which indicates the sulfide exposure is negligible. The chloride exposure classifies the site soil as C1, or moderately corrosive. The foundation design parameters are summarized in Table 16.

5.11 Conclusion

The project area is located in a region of high seismicity. As a result, the design requirements have changed throughout the years. The project will consider all of the factors previously discussed to design the reservoir. In order to provide a successful and resilient reservoir; the final design project must appropriately incorporate and consider all of the discussed seismic design parameters. The wall thicknesses for the various tank construction options have been summarized in Table 17.

Table 17 Concrete Wall Thickness

Concrete	Wall Thickness
Option 1 – 145-foot Diameter Circular Prestressed Concrete Tank	12 inches
Option 2 – Rectangular Cast-in-Place Concrete Tank	26-inch Perimeter Walls
Option 3 – Two 115-foot Circular Prestressed Concrete Tanks	12 inches
Option 4 – 115-foot Circular Prestressed Concrete Tank	12 inches
Option 5 – Two Rectangular Cast-in-Place Concrete Tanks	26-inch Perimeter Walls

SECTION 6 ADDITIONAL DESIGN CONSIDERATIONS

6.1 Electrical, Instrumentation, Communication, and SCADA

The existing electrical service is provided by Southern California Edison (SCE). Daily average electrical use over the last year was approximately 9 kilowatt hours (kWh). The electrical source is located near the entrance to the reservoir site at the northwest corner. The existing 60-amp service disconnect breaker is currently maxed out. Existing facilities that require electrical service include pumps, chlorine analyzer, cathodic protection rectifier, gate operator, lighting, and a recently added PAX mixer system. See Figure 48 for photo of the existing electrical source.

Figure 48 Existing SCE Electrical Source

The electrical system for the new reservoir is not expected to require a significantly larger load than the existing; however, it is assumed that the SCE service will require upgrading and replacing of most of the existing equipment during the reservoir replacement project. Further analysis of electrical load requirements will be performed during final design.

The existing instrumentation and controls system utilizes Schneider Control Expert as its programmable logic controller (PLC) and Ignition for its SCADA. The PLC controls and monitors the following process areas:

- Reservoir Station (RSV): monitors tank level/flow, chlorine residual, and discharge pressure.
- Control Panel (CP): monitors panel intrusion, power conditions (automatic transfer switch, uninterruptible power supply, phase monitor and power fail) and their respective alarms. See Figure 49 for existing SCADA CP.

Figure 49 Existing SCADA Panel

It is anticipated that the proposed instrumentation and controls system for the new reservoir will utilize similar PLC and SCADA systems. See Appendix A for as-builts and Appendix D for previous reports regarding the existing systems.

6.2 Water Quality Considerations

6.2.1 Disinfection Residual

As mentioned earlier, the District currently operates a 6.0-MG reservoir. The water pumped into the reservoir is drinking water provided by MWD that contains a chloramine residual. To maintain a residual in the 6.0-MG reservoir and in the discharge pipe from the reservoir, the District has found that there is a need to add a small amount of sodium hypochlorite solution to the inlet flow. This is done using a temporary dosing system shown in Figure 50.

Figure 50 Temporary Hypochlorite Dosing System at Peters Canyon Reservoir

Moving forward, the size of the replacement reservoir will be smaller, as discussed in Section 4. This will certainly help to reduce the overall retention time of the water in the reservoir and increase the daily turnover rate, which will assist in maintaining the disinfection residual. However, it is recommended that a permanent sodium hypochlorite dosing station be included as part of the project to give the District the flexibility to add chlorine to the reservoirs as needed. Preliminary design criteria for chlorine dosing system are shown in Table 18. It is anticipated that the system would be set up with either chemical totes, drums, or permanent fiber-reinforced plastic (FRP) storage tanks, placed under a shade canopy. In either case each storage tank would be mounted on a load cell that will transmit the weight of the tank to SCADA for monitoring chemical usage and to determine when more chemical is needed. An example is shown in Figure 51.

Table 18 Preliminary Design Criteria for Sodium Hypochlorite Dosing System

Description	Criteria	Comment
Chemical Details		
- Name	sodium hypochlorite	NSF approved
- Concentration, %	12.5	
Chemical Storage Tanks		
- Number	2	one duty, one standby
- Material	plastic	permanent FRP tanks are also an option
- Volume (each), gallons[1]	330	tote
- Load Cells	Yes	each tote/tank positioned on a load cell
Dosing Pumps		
- Number	2	one duty, one standby
- Type	hose pump	blue-white or similar
- Capacity, gallons per hour[2]	0.5 to 3.0	
- Drive	variable speed	
Residual Chlorine Analyzer		
- Number	1	located in outlet pipe
- Probe Design	insertion type	Endress & Hauser or similar

Notes:
(1) A 330-gallon tote is expected to last for 5 days at a maximum dosing rate of 2.5 gallons per hour to add 3 milligrams per liter (mg/L) of hypochlorite to the reservoir.
(2) Dosing pump sized to provide a dose of between 1 mg/L and 3 mg/L of hypochlorite to the reservoir.

Figure 51 Example of a Chemical Tote Positioned on a Load Cell

Totes have the advantage that offloading of chemicals via pipes is avoided, but they would need to be lifted into position using a forklift or truck hoist and it would need to be confirmed that the chemical delivery vehicle can deliver totes to the reservoir site. If tote delivery is not possible then 55-gallon drums could be used. The criteria in Table 18 are based on the use of 330-gallon totes and a 3.0-MG reservoir.

An in-line total chlorine analyzer will be provided in the outlet pipe from the reservoir to continuously measure the chlorine residual in the water leaving the reservoir. This information will be transmitted to the District's SCADA system for remote monitoring. The dosage rate of hypochlorite solution will be adjustable either locally or via SCADA.

6.2.2 Other Water Quality Considerations

Because the water being provided to the District comes from MWD and already meets all drinking water quality standards, and because the retention time within the new Peters Canyon Reservoir will effectively reduce by about 40 percent, it is not anticipated that any further water quality parameters will need to be monitored or controlled at the new reservoir site.

Nitrification can become a potential issue if the water age in the new reservoirs gets much longer than about seven days. Because of the reduced size of the new reservoirs, nitrification is not anticipated to become an issue. However, if water age is a concern, then the mixing pattern in the new reservoir could be checked using computational fluid dynamics (CFD) modeling to determine the water age profile under different operating conditions. This would confirm whether or not a permanent reservoir mixing system will be needed, or whether the location of the inlet and outlet pipes can be relocated to improve mixing.

SECTION 7 PERMITTING AND ENVIRONMENTAL (CEQA) REQUIREMENTS

7.1 Permitting

The following agencies will require permits and/or approvals for the Peters Reservoir Replacement Project:

- State Water Resources Control Board – Amendment to Domestic Water Supply Permit; Construction General Permit for storm water runoff.
- Orange County Public Works/Orange County Fire Authority (OCFA) – building permit.
- South Coast Air Quality Management District - Compliance with Rule 403 (dust control) during construction activities.
- Division of Drinking Water (DDW).
- Division of Safety of Dams (confirmed to be not applicable for the four options analyzed in this PDR).

7.2 CEQA Requirements

At the time of preparation of this PDR, the initial California Environmental Quality Act (CEQA) documents for this project were not started. CEQA requirements will be discussed in detail within the separately provided Initial Environmental Study and Proposed Mitigated Negative Declaration as part of another contract (if required) and could begin in the early stages of final design phase for the project. The Initial Study (IS) will serve to identify the site-specific impacts, evaluate their potential significance, and determine the appropriate document needed to comply with the CEQA regulations.

SECTION 8 CONSTRUCTION COST ESTIMATES

To assist the District with their planning efforts, Carollo has developed a planning level construction cost estimate for each reservoir option listed in Section 4.

Cost estimates provided in this report are considered to be a Class 5 estimate as defined in "Recommended Practice 18R-97 Cost Estimate Classification System for the Process Industries," published by the Association for the Advancement of Cost Engineering International (AACEI). These costs are anticipated to have an accuracy range of +50 percent to -30 percent and are intended for planning purposes only. Cost estimates do not include soft costs, such as engineering consulting fees and permitting. Costs were estimated using the following resources:

- Our proprietary cost database.
- Cost summary from previous projects.
- RS Means Heavy Construction and Building Construction Cost Data.
- Manufacturer quotes.
- Bid summaries from recent projects.

The following sections will discuss the cost estimates for each of the Options 1, 2, 3, 4, and 5.

8.1 Construction Costs for Option 1 – One 145-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

The various capital costs associated with Option 1 include the following items:

- Division 1 costs, including contractor mobilization/demobilization.
- Demolition of the existing 6.0-MG reservoir.
- Construct new 3.0-MG circular prestressed concrete reservoir.
- Construct yard piping modifications for reservoir inlet, outlet overflow, and drain piping.
- Construct new paving and modifications for the driveways and access.
- 30 percent contingency.

Table 19 is a summary of the conceptual estimated construction cost for Option 1, with total construction cost of approximately $28.0 million. This estimate does not include any design engineering and/or construction inspection services.

Table 19 Option 1 Construction Cost Estimate Summary

EOCWD Peters Reservoir Replacement - Option No. 1 Planning Level Cost Estimate						
Item	Description	Qty	UOM	Unit Cost	Total Cost	Comments
GENERAL						
1	Mobilization	1	LS	$1,298,130	$1,298,130	10% of Construction Cost
2	Traffic Control	1	LS	$100,000	$100,000	Allowance
3	Stormwater Mitigation	1	LS	$150,000	$150,000	Allowance
4	Misc Enviornmental Mitigation	1	LS	$100,000	$100,000	Allowance
5	Permitting	1	LS	$100,000	$100,000	Allowance
6	200,000 Gallon Temporary Tank (50 ft Dia) and Associated Piping	1	LS	$900,000	$900,000	Allowance
SITE WORK						
7	Clearing and Grubbing	1	LS	$100,000	$100,000	
8	Demolition and Removal of Existing Reservoir and Appurtenances	1	LS	$1,500,000	$1,500,000	
9	Shoring	8,400	SF	$85	$714,000	
10	Backfill and Compaction	36,500	CY	$30	$1,095,000	
11	AC Paving	4,900	SY	$45	$220,500	Includes repaving of (E) access road
12	Misc Site Improvements	1	LS	$400,000	$400,000	
TANK						
13	Prestressed Concrete Tank, 145-ft Dia, 25-ft Water Depth, 8-ft	1	EA	$6,500,000	$6,500,000	
14	Tank Accessories (Vents, Hatches, Ladders, Guardrail)	1	LS	$150,000	$150,000	
15	Chlorination Station	1	LS	$200,000	$200,000	If Needed
Yard Piping and Appurtenances						
16	Inlet Piping (18")	110	LF	$420	$46,200	
17	Outlet Piping (18")	70	LF	$420	$29,400	
18	Drain Piping (12")	190	LF	$200	$38,000	
19	Overflow Piping (18")	105	LF	$400	$42,000	
20	Overflow Airgap Manhole	1	EA	$15,000	$15,000	
21	PVC Perforated Underdrain Piping (4")	1015	LF	$80	$81,200	
Misc. Costs						
21	Instrumentation	1	LS	$200,000	$200,000	Allowance
22	Electrical	1	LS	$200,000	$200,000	Allowance
23	Testing and Commissioning	1	LS	$100,000	$100,000	
	Subtotal Direct Construction Costs				$14,279,430	
	General Conditions			15%	$2,141,915	15% of Subtotal of Direct Construction Cost
	Sales Tax (10% of Material Costs = 5% of Direct Cost)			5%	$713,972	5% of Subtotal of Direct Construction Cost
	Construction Contingency			30%	$5,140,595	30% of Subtotal of Direct Construction Cost
	Contractor OH&P			12%	$2,673,109	12% of Subtotal of Direct Construction Cost
	Mid 2028 Project Escalation for Schedule B (6% per year)			12.4%	$3,083,699	41.9% of Subtotal
	Total Construction Costs				$28,032,719	

8.3 Construction Costs for Option 2 – One 3.0-MG Rectangular Cast-in-Place Concrete Reservoir

The various capital costs associated with Option 2 include the following items:

- Division 1 costs, including contractor mobilization/demobilization.
- Demolition of the existing 6.0-MG reservoir.
- Construct one new 3.0-MG, Phase 1 cast-in-place reservoir.
- Construct yard piping modifications for reservoir inlet, outlet, overflow, and drain piping.
- Construct new paving for the driveways and access.
- 30 percent contingency.

Table 20 is a summary of the conceptual estimated construction cost for Option 2, with a total construction cost of approximately $29.9 million. This estimate does not include any design engineering and construction inspection services.

Table 20 Option 2 Construction Cost Estimate Summary

EOCWD Peters Reservoir Replacement -Option No. 2 Planning Level Cost Estimate						
Item	**Description**	**Qty**	**UOM**	**Unit Cost**	**Total Cost**	**Comments**
GENERAL						
1	Mobilization	1	LS	*$1,386,780*	**$1,386,780**	*10% of Construction Cost*
2	Traffic Control	1	LS	*$100,000*	**$100,000**	*Allowance*
3	Stormwater Mitigation	1	LS	*$150,000*	**$150,000**	*Allowance*
4	Misc Enviornmental Mitigation	1	LS	*$100,000*	**$100,000**	*Allowance*
5	Permitting	1	LS	*$100,000*	**$100,000**	*Allowance*
6	200,000 Gallon Temporary Tank (50 ft Dia) and Associated Piping	1	LS	*$900,000*	**$900,000**	*Allowance*
SITE WORK						
7	Clearing and Grubbing	1	LS	*$100,000*	**$100,000**	
8	Demolition and Removal of Existing Reservoir and Appurtenances	1	LS	*$1,500,000*	**$1,500,000**	
9	Shoring	13,000	SF	*$85*	**$1,105,000**	
10	Backfill and Compaction	38,000	CY	*$30*	**$1,140,000**	
11	AC Paving	4,800	SY	*$45*	**$216,000**	*Includes repaving of (E) access road*
12	Misc Site Improvements	1	LS	*$400,000*	**$400,000**	
TANK						
13	Cast in Place Concrete Tank, 130-ft SQ, 25-ft Water Depth, 8-ft Freeboard	3400	CY	*$2,000*	**$6,800,000**	
14	Tank Accessories (Vents, Hatches, Ladders, Guardrail)	1	LS	*$150,000*	**$150,000**	
15	Chlorination Station	1	LS	*$200,000*	**$200,000**	*If Needed*
Yard Piping and Appurtenances						
16	Inlet Piping (18")	140	LF	*$400*	**$56,000**	
17	Outlet Piping (18")	200	LF	*$400*	**$80,000**	
18	Drain Piping (12")	270	LF	*$180*	**$48,600**	
19	Overflow Piping (18")	140	LF	*$400*	**$56,000**	
20	Overflow Airgap Manhole	1	EA	*$15,000*	**$15,000**	
21	PVC Perforated Underdrain Piping (4")	1640	LF	*$80*	**$131,200**	
22	Underdrain Manhole	1	EA	*$20,000*	**$20,000**	
Misc. Costs						
21	Instrumentation	1	LS	*$200,000*	**$200,000**	*Allowance*
22	Electrical	1	LS	*$200,000*	**$200,000**	*Allowance*
23	Testing and Commissioning	1	LS	*$100,000*	**$100,000**	
	Subtotal Direct Construction Costs				**$15,254,580**	
	General Conditions			15%	**$2,288,187**	**15% of Subtotal of Direct Construction Cost**
	Sales Tax (10% of Material Costs = 5% of Direct Cost)			5%	**$762,729**	**5% of Subtotal of Direct Construction Cost**
	Construction Contingency			30%	**$5,491,649**	**30% of Subtotal of Direct Construction Cost**
	Contractor OH&P			12%	**$2,855,657**	**12% of Subtotal of Direct Construction Cost**
	Mid 2028 Project Escalation for Schedule B (6% per year)			12.4%	**$3,294,286**	**41.9% of Subtotal**
	Total Construction Costs				**$29,947,089**	

8.4 Construction Costs for Option 3 – Two 3.0-MG Circular Prestressed Reservoirs

The various capital costs associated with Option 3 include the following items:

- Division 1 costs, including contractor mobilization/demobilization.
- Demolition of the existing 6.0-MG reservoir.
- Construct two new 3.0-MG circular prestressed concrete reservoirs.
- Construct yard piping modifications for reservoir inlet, outlet, overflow, and drain piping.
- Construct new paving for the driveways and access.
- 30 percent contingency.

Table 21 is a summary of the conceptual estimated construction cost for Option 3, with a total construction cost of approximately $45.6 million. This estimate does not include any design engineering and construction inspection services.

Table 21 Option 3 Construction Cost Estimate Summary

EOCWD Peters Reservoir Replacement - Option No. 3 Planning Level Cost Estimate						
Item	Description	Qty	UOM	Unit Cost	Total Cost	Comments
GENERAL						
1	Mobilization	1	LS	*$2,113,150*	**$2,113,150**	*10% of Construction Cost*
2	Traffic Control	1	LS	*$100,000*	**$100,000**	*Allowance*
3	Stormwater Mitigation	1	LS	*$150,000*	**$150,000**	*Allowance*
4	Misc Enviornmental Mitigation	1	LS	*$100,000*	**$100,000**	*Allowance*
5	Permitting	1	LS	*$100,000*	**$100,000**	*Allowance*
6	200,000 Gallon Temporary Tank (50 ft Dia) and Associated Piping	1	LS	*$900,000*	**$900,000**	*Allowance*
SITE WORK						
7	Clearing and Grubbing	1	LS	*$100,000*	**$100,000**	
8	Demolition and Removal of Existing Reservoir and Appurtenances	1	LS	*$1,500,000*	**$1,500,000**	
9	Shoring	22,000	SF	*$85*	**$1,870,000**	
10	Backfill and Compaction	31,500	CY	*$30*	**$945,000**	
11	AC Paving	4,500	SY	*$45*	**$202,500**	*Includes repaving of (E) access road*
12	Misc Site Improvements	1	LS	*$400,000*	**$400,000**	
TANK						
13	Prestressed Concrete Tank, 115-ft Dia, 37-ft Water Depth, 8-ft	2	EA	*$6,250,000*	**$12,500,000**	
14	Tank Accessories (Vents, Hatches, Ladders, Guardrail)	1	LS	*$300,000*	**$300,000**	
15	Chlorination Station	1	LS	*$400,000*	**$400,000**	*If Needed*
Yard Piping and Appurtenances						
16	Inlet Piping (18")	240	LF	*$400*	**$96,000**	
17	Outlet Piping (18")	450	LF	*$400*	**$180,000**	
18	Drain Piping (12")	500	LF	*$180*	**$90,000**	
19	Overflow Piping (18")	355	LF	*$400*	**$142,000**	
20	Overflow Airgap Manhole	2	EA	*$15,000*	**$30,000**	
21	Valve Vaults and Appurtenances	2	EA	*$40,000*	**$80,000**	
22	PVC Perforated Underdrain Piping (4")	1950	LF	*$80*	**$156,000**	
23	Underdrain Manhole	2	EA	*$20,000*	**$40,000**	
Misc. Costs						
22	Instrumentation	1	LS	*$300,000*	**$300,000**	*Allowance*
23	Electrical	1	LS	*$300,000*	**$300,000**	*Allowance*
24	Testing and Commissioning	1	LS	*$150,000*	**$150,000**	
	Subtotal Direct Construction Costs				**$23,244,650**	
	General Conditions			15%	**$3,486,698**	**15% of Subtotal of Direct Construction Cost**
	Sales Tax (10% of Material Costs = 5% of Direct Cost)			5%	**$1,162,233**	**5% of Subtotal of Direct Construction Cost**
	Construction Contingency			30%	**$8,368,074**	**30% of Subtotal of Direct Construction Cost**
	Contractor OH&P			12%	**$4,351,398**	**12% of Subtotal of Direct Construction Cost**
	Mid 2028 Project Escalation for Schedule B (6% per year)			12.4%	**$5,019,773**	**41.9% of Subtotal**
	Total Construction Costs				**$45,632,826**	

8.5 Construction Costs for Option 4 – One 115-Foot Diameter 3.0-MG Circular Prestressed Concrete Reservoir

The various capital costs associated with Option 4 include the following items:

- Division 1 costs, including contractor mobilization/demobilization.
- Demolition of the existing 6.0-MG reservoir.
- Construct new 3.0-MG circular prestressed concrete reservoir.
- Construct yard piping modifications for reservoir inlet, outlet, overflow, and drain piping.
- Construct new paving for the driveways and access.
- 30 percent contingency.

Table 22 is a summary of the conceptual estimated construction cost for Option 4, with a total construction cost of approximately $27.7 million. This estimate does not include any design engineering and construction inspection services.

Table 22 Option 4 Construction Cost Estimate Summary

EOCWD Peters Reservoir Replacement - Option No. 4 Planning Level Cost Estimate						
Item	**Description**	**Qty**	**UOM**	**Unit Cost**	**Total Cost**	**Comments**
GENERAL						
1	Mobilization	1	LS	*$1,283,970*	**$1,283,970**	*10% of Construction Cost*
2	Traffic Control	1	LS	*$100,000*	**$100,000**	*Allowance*
3	Stormwater Mitigation	1	LS	*$150,000*	**$150,000**	*Allowance*
4	Misc Enviornmental Mitigation	1	LS	*$100,000*	**$100,000**	*Allowance*
5	Permitting	1	LS	*$100,000*	**$100,000**	*Allowance*
6	200,000 Gallon Temporary Tank (50 ft Dia) and Associated Piping	1	LS	*$900,000*	**$900,000**	*Allowance*
SITE WORK						
7	Clearing and Grubbing	1	LS	*$100,000*	**$100,000**	
8	Demolition and Removal of Existing Reservoir and Appurtenances	1	LS	*$1,500,000*	**$1,500,000**	
9	Shoring	6,900	SF	*$85*	**$586,500**	
10	Backfill and Compaction	44,400	CY	*$30*	**$1,332,000**	
11	AC Paving	5,700	SY	*$45*	**$256,500**	*Includes repaving of (E) access road*
12	Misc Site Improvements	1	LS	*$400,000*	**$400,000**	
TANK						
13	Prestressed Concrete Tank, 115-ft Dia, 37-ft Water Depth, 8-ft	1	EA	*$6,250,000*	**$6,250,000**	
14	Tank Accessories (Vents, Hatches, Ladders, Guardrail)	1	LS	*$150,000*	**$150,000**	
15	Chlorination Station	1	LS	*$200,000*	**$200,000**	*If Needed*
Yard Piping and Appurtenances						
16	Inlet Piping (18")	140	LF	*$420*	**$58,800**	
17	Outlet Piping (18")	25	LF	*$420*	**$10,500**	
18	Drain Piping (12")	140	LF	*$200*	**$28,000**	
19	Overflow Piping (18")	90	LF	*$400*	**$36,000**	
20	Overflow Airgap Manhole	1	EA	*$15,000*	**$15,000**	
21	PVC Perforated Underdrain Piping (4")	830	LF	*$80*	**$66,400**	
Misc. Costs						
22	Instrumentation	1	LS	*$200,000*	**$200,000**	*Allowance*
23	Electrical	1	LS	*$200,000*	**$200,000**	*Allowance*
24	Testing and Commissioning	1	LS	*$100,000*	**$100,000**	
	Subtotal Direct Construction Costs				**$14,123,670**	
	General Conditions			15%	**$2,118,551**	**15% of Subtotal of Direct Construction Cost**
	Sales Tax (10% of Material Costs = 5% of Direct Cost)			5%	**$706,184**	**5% of Subtotal of Direct Construction Cost**
	Construction Contingency			30%	**$5,084,521**	**30% of Subtotal of Direct Construction Cost**
	Contractor OH&P			12%	**$2,643,951**	**12% of Subtotal of Direct Construction Cost**
	Mid 2028 Project Escalation for Schedule B (6% per year)			12.4%	**$3,050,062**	**41.9% of Subtotal**
	Total Construction Costs				**$27,726,938**	

8.6 Construction Costs for Option 5 – Two 3.0-MG Rectangular Cast-in-Place Concrete Reservoirs

The various capital costs associated with Option 5 include the following items:

- Division 1 costs, including contractor mobilization/demobilization.
- Demolition of the existing 6.0-MG reservoir.
- Construct one new 3.0-MG Phase 1 cast-in-place reservoir (Cell No. 1).
- Construct one new 3.0-MG Phase 2 cast-in-place reservoir (Cell No. 2).
- Construct yard piping modifications for reservoir inlet, outlet, overflow, and drain piping.
- Construct new paving for the driveways and access.
- 30 percent contingency.

Table 23 is a summary of the conceptual estimated construction cost for Option 5, with a total construction cost of approximately $46.0 million. This estimate does not include any design engineering and construction inspection services.

Table 23 Option 5 Construction Cost Estimate Summary

EOCWD Peters Reservoir Replacement - Option No. 5 Planning Level Cost Estimate						
Item	Description	Qty	UOM	Unit Cost	Total Cost	Comments
GENERAL						
1	Mobilization	1	LS	*$2,128,940*	**$2,128,940**	*10% of Construction Cost*
2	Traffic Control	1	LS	*$100,000*	**$100,000**	*Allowance*
3	Stormwater Mitigation	1	LS	*$150,000*	**$150,000**	*Allowance*
4	Misc Enviornmental Mitigation	1	LS	*$100,000*	**$100,000**	*Allowance*
5	Permitting	1	LS	*$100,000*	**$100,000**	*Allowance*
6	200,000 Gallon Temporary Tank (50 ft Dia) and Associated Piping	1	LS	*$900,000*	**$900,000**	*Allowance*
SITE WORK						
7	Clearing and Grubbing	1	LS	*$100,000*	**$100,000**	
8	Demolition and Removal of Existing Reservoir and Appurtenances	1	LS	*$1,500,000*	**$1,500,000**	
9	Shoring	25,500	SF	*$85*	**$2,167,500**	
10	Backfill and Compaction	14,100	CY	*$30*	**$423,000**	
11	AC Paving	2,800	SY	*$45*	**$126,000**	*Includes repaving of (E) access road*
12	Misc Site Improvements	1	LS	*$400,000*	**$400,000**	
TANK						
13	Two CIP Concrete Tanks, 130-ft SQ, 25-ft Water Depth, 8-ft Freeboard	6450	CY	*$2,000*	**$12,900,000**	
14	Tank Accessories (Vents, Hatches, Ladders, Guardrail)	1	LS	*$300,000*	**$300,000**	
15	Chlorination Station	1	LS	*$400,000*	**$400,000**	*If Needed*
Yard Piping and Appurtenances						
16	Inlet Piping (18")	230	LF	*$400*	**$92,000**	
17	Outlet Piping (18")	320	LF	*$400*	**$128,000**	
18	Drain Piping (12")	455	LF	*$180*	**$81,900**	
19	Overflow Piping (18")	280	LF	*$400*	**$112,000**	
20	Overflow Airgap Manhole	1	EA	*$15,000*	**$15,000**	
21	Valve Vaults and Appurtenances	2	EA	*$40,000*	**$80,000**	
22	PVC Perforated Underdrain Piping (4")	4050	LF	*$80*	**$324,000**	
23	Underdrain Manhole	2	EA	*$20,000*	**$40,000**	
Misc. Costs						
22	Instrumentation	1	LS	*$300,000*	**$300,000**	*Allowance*
23	Electrical	1	LS	*$300,000*	**$300,000**	*Allowance*
24	Testing and Commissioning	1	LS	*$150,000*	**$150,000**	
	Subtotal Direct Construction Costs				**$23,418,340**	
	General Conditions			15%	**$3,512,751**	**15% of Subtotal of Direct Construction Cost**
	Sales Tax (10% of Material Costs = 5% of Direct Cost)			5%	**$1,170,917**	**5% of Subtotal of Direct Construction Cost**
	Construction Contingency			30%	**$8,430,602**	**30% of Subtotal of Direct Construction Cost**
	Contractor OH&P			12%	**$4,383,913**	**12% of Subtotal of Direct Construction Cost**
	Mid 2028 Project Escalation for Schedule B (6% per year)			12.4%	**$5,057,282**	**41.9% of Subtotal**
	Total Construction Costs				**$45,973,806**	

SECTION 9 CONSTRUCTION SEQUENCING AND PROJECT DESIGN SCHEDULE

9.1 Construction Sequencing

The major construction sequences will be broken into two phases. The first phase will include demolition and replacement of the existing 6.0-MG reservoir.

Phase 1 will consist of:

1. Mobilization – During mobilization, the Contractor will move construction equipment, tools, and personnel to the reservoir site to begin the work and implement construction noise and vibration mitigation measures.
2. A temporary tank will be installed to ensure continuous service during construction.
3. Isolation of existing reservoir and ancillary components from the distribution system.
4. Initial Demolition and Excavation – During this phase of the work, the Contractor will remove the existing reservoir. All other on-site facilities will be protected in place, unless otherwise noted on plans.
5. Shoring and over-excavation prior to installation of reservoir foundation.
6. Install reservoir underdrain system and foundation.
7. Grading - Trench new underground piping and conduits for connections to new reservoir. During this phase of the work, the Contractor will conduct excavation, place pipe bedding material, install new pipe, place trench backfill, and rough grading.
8. Construct reservoir walls and roof.
9. Reservoir hydraulic testing and disinfection.
10. Startup and Commissioning – During startup and commissioning, each piece of equipment will be tested for conformance with the intent of the facility, the manufacturer's requirements, and operability. After testing is complete, each piece of equipment and system will be operated to determine and commissioned to operate according to the requirements of the manufacturer, contract documents, and specifications.

Phase 2 of the project (where applicable) will consist of:

1. Mobilization – During mobilization, the Contractor will move construction equipment, tools, and personnel to the reservoir site to begin the work and implement construction noise and vibration mitigation measures.
2. Excavation, shoring, and over-excavation (if applicable) prior to installation of reservoir foundation.
3. Install reservoir underdrain system and foundation.
4. Grading - Trench new underground piping and conduits for connections to new reservoir. During this phase of the work, the Contractor will conduct excavation, place pipe bedding material, install new pipe, place trench backfill, and rough grading.

5. Construct reservoir walls and roof.
6. Reservoir hydraulic testing and disinfection.
7. Startup and Commissioning – During startup and commissioning, each piece of equipment will be tested for conformance with the intent of the facility, the manufacturer's requirements, and operability. After testing is complete, each piece of equipment and system will be operated to determine and commissioned to operate according to the requirements of the manufacturer, contract documents, and specifications.

9.2 Design and Construction Schedule

We have prepared a preliminary Design and Construction Phase schedule presented in Figure 52. The schedule includes Final Design, Bidding, and Construction.

The Final Design Phase will continue for six months and be completed in March 2027. It will include deliverables at 60 percent, 90 percent, and 100 percent, followed by a Bid Set Submittal.

Following the submittal of deliverables to the California Office of Emergency Services (Cal OES) in June 2027, the Bidding Phase will begin and take approximately three months to complete.

The Construction Phase will follow after the Bidding Phase with a Notice to Proceed (NTP). The construction duration is estimated at approximately 18 months.

ID	Task Name	Duration	Start	Finish
1	**NOTICE TO PROCEED**	0 days	Mon 8/24/26	Mon 8/24/26
2	**Final Design**	**155 days**	**Mon 8/24/26**	**Fri 3/26/27**
3	**2.7 Prepare 60% Design Submittal**	**55 days**	**Mon 8/24/26**	**Fri 11/6/26**
4	Prepare 60% Design PS&Es	35 days	Mon 8/24/26	Fri 10/9/26
5	EOCWD Review of 60% Submittal	20 days	Mon 10/12/26	Fri 11/6/26
6	**2.1.1 Prepare 90% Design Submittal**	**45 days**	**Mon 11/9/26**	**Fri 1/8/27**
7	Prepare 90% Design PS&Es	25 days	Mon 11/9/26	Fri 12/11/26
8	EOCWD Review of 90% Submittal	20 days	Mon 12/14/26	Fri 1/8/27
9	**2.1.2 Prepare Final Design Submittal**	**55 days**	**Mon 1/11/27**	**Fri 3/26/27**
10	Prepare Final Design PS&Es	20 days	Mon 1/11/27	Fri 2/5/27
11	EOCWD Review of Final Design Submittal	20 days	Mon 2/8/27	Fri 3/5/27
12	Prepare Bid Set	15 days	Mon 3/8/27	Fri 3/26/27
13				
14	**Environmental / Historic Review**	**152 days**	**Tue 9/1/26**	**Wed 3/31/27**
15				
16	**Deliverables to CalOES**	**11 days**	**Fri 5/28/27**	**Fri 6/11/27**
17				
18	**Bidding Phase**	**65 days**	**Mon 6/28/27**	**Fri 9/24/27**
19	Bid Period	40 days	Mon 6/28/27	Fri 8/20/27
20	Receive Bids	0 days	Fri 8/20/27	Fri 8/20/27
21	Bid Evaluation	25 days	Mon 8/23/27	Fri 9/24/27
22	Notice of Award	0 days	Fri 9/24/27	Fri 9/24/27
23				
24	**Construction Start (Contractor NTP)**	**390 days**	**Fri 10/22/27**	**Fri 4/20/29**
25	Contractor NTP	0 days	Fri 10/22/27	Fri 10/22/27
26	Construction Permits/Approvals	25 days	Mon 10/25/27	Fri 11/26/27
27	Mobilization	0 days	Fri 12/17/27	Fri 12/17/27
28	Procurement/Submittals	240 days	Mon 10/25/27	Fri 9/22/28
29	Clearing/Grubbing/Demolition	50 days	Mon 12/20/27	Fri 2/25/28
30	Excavation/Shoring	50 days	Mon 2/28/28	Fri 5/5/28
31	Inlet/Outlet Piping Installation	80 days	Mon 5/8/28	Fri 8/25/28
32	Reservoir Installation	150 days	Mon 5/8/28	Fri 12/1/28
33	Civil/Site	60 days	Mon 12/4/28	Fri 2/23/29
34	Electrical/Instrumentation Equipment Installation	40 days	Mon 12/4/28	Fri 1/26/29
35	Testing and Startup	30 days	Mon 1/29/29	Fri 3/9/29
36	Punch List Preparation	30 days	Mon 3/12/29	Fri 4/20/29
37	O&M Manuals	30 days	Mon 1/29/29	Fri 3/9/29
38	Commissioning	30 days	Mon 3/12/29	Fri 4/20/29
39	Substantial Completion	0 days	Fri 4/20/29	Fri 4/20/29

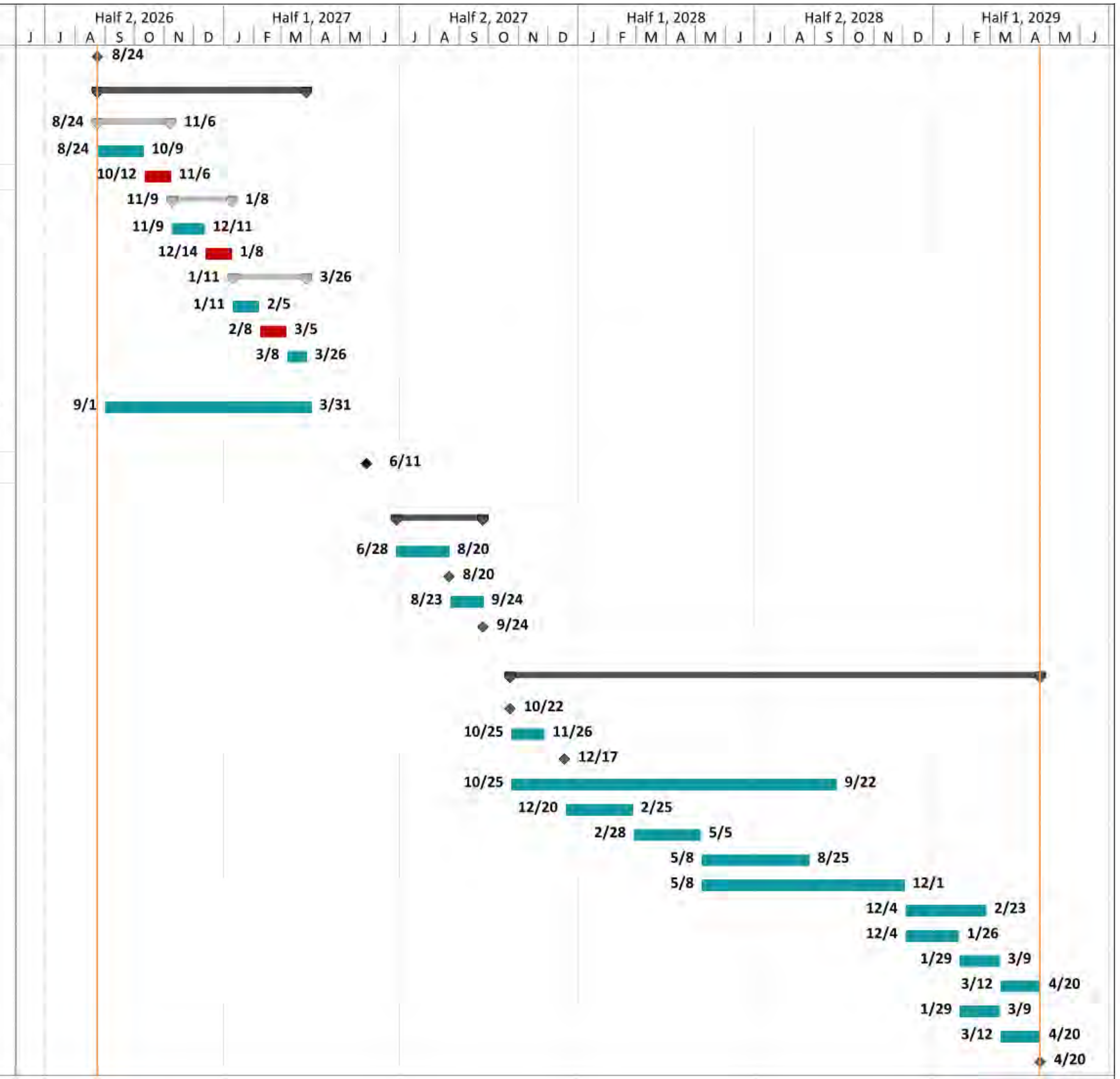


Figure 52 Final Design and Construction Phase Schedule

SECTION 10 RECOMMENDATIONS

10.1 Comparison of Costs

Section 4 of this Preliminary Design Report evaluated five reservoir alternatives for the Peters Canyon Reservoir Replacement. These included:

- Option 1: One 145-foot diameter 3.0-MG circular prestressed concrete reservoir.
- Option 2: One 3.0-MG cast-in-place rectangular concrete reservoir.
- Option 3: Two 115-foot diameter 3.0-MG each circular prestressed concrete reservoirs (6.0 MG total after Phase 2 completion).
- Option 4: One 115-foot diameter 3.0-MG circular prestressed concrete reservoir
- Option 5: Two 3.0-MG (each cell) cast-in-place rectangular concrete reservoirs (6.0 MG total after Phase 2 completion).

Comparing the cost estimates shown in Section 8, Tables 19, 20, 21, 22, and 23, Option 1 provides 3.0-MG capacity at maximum buildout with total estimated construction costs of $28.0 million. Option 2 provides 3.0-MG capacity at a cost of approximately $29.9 million. Option 3 provides 6.0-MG capacity at a cost of approximately $45.6 million. Option 4 provides 3.0-MG capacity at a cost of approximately $27.7 million. Option 5 provides 6.0-MG capacity at a cost of approximately $46.0 million. Actual costs will not be fully understood until after the Bidding Phase is complete.

10.2 Evaluation of Options

Options 1, 2, and 4 are single-cell tanks that do not provide redundancy. However, Options 1, 2, and 4 provide additional space for a maintenance yard. On a per-gallon basis, the costs of Options 1 and 2 are $9.33 and $9.97 per gallon, respectively. The cost for Option 4 is $9.23 per gallon. Options 1 and 4 will require an access road around the prestressed reservoir for maintenance.

Options 3 and 5 are two-cell tanks that provide redundancy. On a per-gallon basis, the costs of Options 3 and 5 are $7.60 and $7.67 per gallon, respectively. Option 3 will require an access road around the prestressed reservoir for maintenance.

Options 1, 2, and 5 maintain the same high-water level as the existing reservoir and therefore would not require any hydraulic analysis or modifications to the existing pump station.

The increased high-water level of Options 3 and 4 could require hydraulic analysis and potential modifications to the existing pump station. Although it should be noted that the existing pump station will be able to pump to the temporary tank, so the modifications, if any, would be minimal.

10.3 Recommendation of the Best Option

As previously stated, all the options presented will mitigate risks associated with seismic and wildfire hazards. Based on our technical evaluations of the five options, we recommend that the District consider Option 4 as the single reservoir option and Option 5 as the dual reservoir option. We recommend

Option 4, as it has the lowest cost among the options and provides space for a maintenance yard. We recommend Option 5, as Options 5 and 3 have similar costs, but Option 5 would not require any hydraulic analysis or potential modifications to the existing pump station.

The District has indicated they do not require redundancy and are interested in the additional area that will become available for a maintenance yard. This preference would eliminate Option 5 (two 3.0-MG rectangular reservoirs) that would occupy the entire site. Therefore, we recommend that the District move forward with the design of Option 4 (one 115-foot diameter, 3.0-MG prestress concrete tank).

The benefits of Option 4 - Single Circular 115-foot Diameter 3.0-MG Prestressed Concrete Tank include:

1. Less construction cost compared to other options.
2. Reduced operational costs.
3. Better site utilization to maximize storage.
4. Accessibility for operations and maintenance, including space for a maintenance yard.
5. A more streamlined permitting process.
6. No DSOD jurisdictional requirements.

Figure 53 presents a conceptual layout of the recommended Option 4.

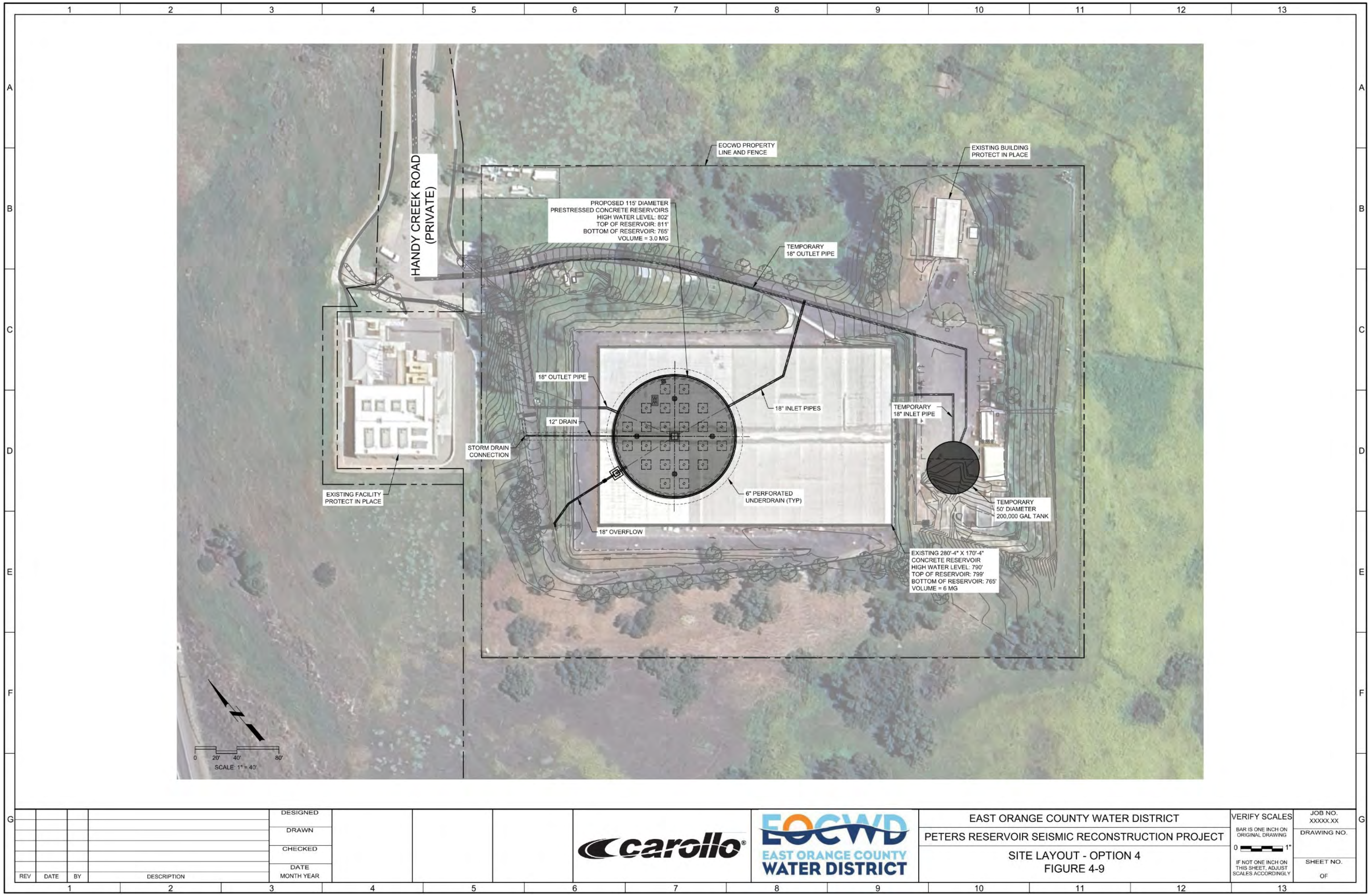


Figure 53 Recommended Option 4 Site Layout