EAST ORANGE COUNTY WATER DISTRICT

PETERS CANYON AND VISTA PANORAMA RESERVOIR

CONDITION ASSESSMENT

FINAL
April 2019

3150 BRISTOL STREET, SUITE 500 • COSTA MESA, CALIFORNIA 92626 • P.714.593.5100 • F.714.593.5101
pw://Carollo/Documents/Client/CA/EOCWD/9711A00/Deliverables/TM01/Condition Assessment

EAST ORANGE COUNTY WATER DISTRICT

PETERS CANYON AND VISTA PANORAMA RESERVOIR

CONDITION ASSESSMENT

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 Scope

East Orange County Water District (EOCWD) has tasked Carollo Engineers (Carollo) to provide a seismic evaluation and condition assessment of two of its reservoirs: the 6 million gallons (MG) Peters Canyon Reservoir and the 150,000 gallon Vista Panorama Reservoir. The scope of work is based on the proposal letter issued by Carollo Engineers, dated November 2014.

As outlined in our scope of work the following tasks were performed:

1.1.1 Task 1 – Peters Canyon Reservoir

- Review existing reports and drawings
- Perform additional seismic calculations
- Perform an on-site external condition assessment and a slope stability analysis
- Develop and analyze retrofit/replacement alternatives
- Prepare a technical memorandum summarizing our findings

1.1.2 Task 2 – Vista Panorama Reservoir

- Review existing reports and drawings
- Perform additional seismic calculations
- Perform an on-site external assessment
- Develop and analyze retrofit alternatives
- Prepare a technical memorandum summarizing our findings

1.2 Background

1.2.1 Peters Canyon Reservoir

The Peters Canyon Reservoir was originally designed and constructed in 1963. The reservoir, situated on a hillside in east Orange County, is located just east of Jamboree Road. The site can be reached by driving east on Handy Creek Road, a private access road which intersects with Jamboree Road. An aerial view of the site is provided on Figure 1.

The reservoir is a rectangular hopper-bottom shaped partially buried reservoir. The reservoir bottom and sideliner walls are constructed of 4-inch (in) thick reinforced concrete slabs with thickened edges at the perimeters. The reservoir plan dimensions are approximately 280-ft-4-in in the east-west direction and 170-ft 4-in in the north-south direction. The reservoir above ground consists of 4 ft tall, 8 ft thick concrete masonry unit (CMU) perimeter retaining walls with eccentric footings. These CMU walls have cast-in-place concrete pilasters at the main glulam beam and purlin support locations along the perimeter wall. The roof structure is comprised of sawn lumber purlins and glulam beams. At the interior of reservoir, the beams are supported on precast concrete columns, which in-turn are supported on concrete spread footings. The purlins span in the east-west direction and the glulam beams span in the north-south direction. All the wood members were preservative-treated (Wolmanized). In the east-west direction, additional bracing is provided by precast concrete diagonal braced frames. The roofing membrane consists of a Zip-Rib style aluminum decking, which is attached to the wood-framing members using sliding connections that limit the ability of the roof the decking to perform as a functional structural diaphragm. At the center of the roof in the east-west direction, the roof framing is raised to accommodate venting. The roof framing slopes to the north and to the south for drainage.

The concrete liner walls and concrete columns are overlain by a Hypalon liner installed in 1995. The bottom elevation of the reservoir slopes from 767.25 feet (ft) on the east side to 765.25 ft on the west side. The high operating water level is at elevation 790 ft, which is a minimum depth of 22.75 ft. The top of the perimeter CMU wall is at elevation 792 ft. A wood pony wall is built-up along the east and west walls of the reservoir. The pony wall is framed with wood studs and preservative treated plywood sheathing. The structural roof slopes up to the middle of the reservoir. The average height of the roof above the bottom of the reservoir is about 28 ft.

In 1996, portions of roofing and structural wood framing were significantly damaged from high winds resulting in roughly one-third of the roofing and a significant amount of wood framing requiring replacement. A seismic assessment was conducted in 1999, which resulted in a seismic retrofit in 2000. The retrofit consisted of adding steel rod rock anchors to the base of each of the concrete piers along the north and south perimeter walls only. The seismic assessment report was not available for review, but it appears that these rock anchors were added to increase the sliding friction capacity of the pier footing to resist seismic loads imposed by the glulam beams in the direction parallel with those beams.

Additional structural and seismic evaluations were performed over the last few years in response to observations made during prior dive inspections.

Figure 1
Aerial View of Peters Canyon Reservoir

East Orange County Water District Condition Assessment

carollo
Engineers...Working Wonders With Water®

EAST ORANGE COUNTY WATER DISTRICT

1.2.2 Vista Panorama Reservoir

The Vista Panorama Reservoir is a 46.5-ft diameter circular, conventional (non-prestressed) reinforced concrete tank with a wood-framed roof. The tank was built in the 1930's. The tank is above ground and the wall height is approximately 12 ft. The wood roof beams are supported on a total of 7 interior steel columns. The storage capacity of the tank is approximately 150,000 gallons. An aerial view of the reservoir is provided in Figure 2.

1.3 Cost Estimates

To assist EOCWD with their planning efforts in this evaluation report, major scope items were identified and rough-order of magnitude costs were estimated for each. Cost estimates provided in this evaluation/study 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 (AACE International). These costs are anticipated to have an accuracy range of plus 50 percent to minus 30 percent and are for intended for planning purposes. 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 summaries from previous projects
- RS Means Heavy Construction Cost Data
- RS Means Building Construction Cost Data
- Manufacturer and contractor quotes

1.4 Data Collection and Review

Data and information necessary for use in the evaluation of Peters Canyon and Vista Panorama reservoirs, was obtained from the following documents provided in electronic format by from EOCWD:

- Construction drawings of 6 MG Reservoir and Filtration Plant, sheets 17 through 26, dated 1963.
- Construction drawings 6 MG Reservoir Liner, sheets 1 through 5, dated 1995.
- Construction drawings of 6 MG Reservoir Remedial Roof Construction, sheets S-1 and S-2, dated 1997.

20-EOWWD1-15F2-9711A00.AI

Figure 2
Aerial View of Vista Panorama Reservoir

East Orange County
Water District
Condition Assessment

carollo
Engineers...Working Wonders With Water®

EAST ORANGE COUNTY WATER DISTRICT

- Construction drawings of Reservoir Improvements, sheets 1 to 6, dated 2000.
- Tank Inspection Report for Peters Canyon Reservoir, by Dive/Corr, Inc., dated January 2010.
- East Orange County Water District – Field Report, by Potable Divers Inc., dated July 2011.
- East Orange County Water District - Engineering Evaluation of the Peters Canyon Six Million Gallon Reservoir report, by Richard Brady & Associates, dated October 2012.
- Peters Canyon 6 MG Reservoir Structural Peer Review, by AARK Engineering, dated September 2013.
- Beam Inspection – East Orange County Reservoir; Orange, California report prepared by Western Wood Structures, Inc., dated November 2013.
- East Orange County Water District – Results and Recommendations for Peters Canyon Reservoir Roof report prepared by Richard Brady & Associates, dated September 9, 2014.
- Geotechnical and Seismic Hazard Assessment Report, Prepared by American Geotechnical, Inc., dated April 2, 2014.
- Retrofit Design of Vista Panorama Reservoir Report, prepared by Simon Wong Engineering, dated February 15, 2012.

1.5 Site Visits

Carollo conducted site visits for Peters Canyon Reservoir and Vista Panorama reservoir on Wednesday December 3, 2014, and Thursday December 4, 2014, to gather as-built information and structural configurations. At the time, both the reservoirs were in service and observations of the interior structure were limited. Conditions observed during these site visits are described in the following sections. Photographs taken during these site visits are included in Appendix B of this report.

2.0 PETERS CANYON RESERVOIR

2.1 Review of Existing Drawings and Reports

Based on the review of the existing structural drawings and the reports prepared by Richard Brady & Associates (Brady), the following detailed conclusions about the condition of the various structural elements were made.

We generally concur with the conclusions presented by Brady in their 2012 and 2014 reports with the exceptions and additional issues set forth in this report. We have also reviewed supplemental reports that were prepared by AARK Engineering and Western Wood Structures and we have also noted exceptions and potential issues upon which we have provided expanded discussions and recommendations.

2.1.1 Hinge Connector and Glulam Beam Condition

Brady identified that the glulam beams at the glulam hinge connectors are damaged and in need of immediate repair to prevent a roof collapse. The investigation made by Western Wood Structures in November 2013 indicated general deterioration of the top lamination due to rot and some limited lamination damage at the hinge connector. The investigation also noted that there appeared to be some rotation of the hinge connector and indicated that the rotation was likely due to the softening of the top bearing lamination of the cantilevered beam end. This rotation in turn is driving the hinge connector into the bottom lamination of the cantilevered end causing it to split.

It is not clear why the glulam beams are apparently vulnerable to rot. The original construction drawings specify the “framing lumber” to be “Wolmanized treated,” which is a wood preservation process that adds chromate copper arsenate (CCA) as a pesticide to protect the lumber from rot and decay. It is possible that the glulam beams are not treated with this preservative. A simple visual observation can confirm whether these beams were treated or not. Lumber that has been treated will have small uniformly spaced incising marks (narrow thin penetrations) on all of the wood surfaces. Smaller lumber sections may appear to have a green tint to them as well.

If the glulam beams were not treated with any wood preservative, the glue used to structurally bond the individual beam laminates together might not be of a sufficiently durable grade for the environment within the reservoir. Degradation of the laminate glue can lead to structural failure of the beam. Delamination has not been identified in any surveys thus far.

Entry into the reservoir to observe framing conditions inside was not within our scope of work. If further information regarding the glulam beam conditions is desired, we recommend an interior survey be made of the roof framing. Since the laminations may continue to deteriorate, the damaged hinge connectors should be corrected as soon as practicable.

2.1.2 Roof Framing Hardware Corrosion

During our site visit, we viewed the interior of the reservoir immediately adjacent to the access door located at the west end of the reservoir. Corroded nails were observed at a few of the joist hangers, but the nails were in place. No other evidence of severe corrosion was observed. EOCWD staff provided a photograph of a collection of metallic debris that included stainless steel screws and heavily corroded nails that were retrieved from the bottom of the reservoir during the last cleaning, which occurred in 2013. It is not known if this debris is indicative of an on-going problem with connection hardware corrosion or if the debris was simply construction litter.

Given the age of the reservoir and the high relative humidity within the reservoir, it is likely that the roof framing fasteners that are embedded within the wood have corroded to some extent. Our visual observations are limited and we can only assume that corrosion is active. Corrosion of metals in wood will continue unabated inside of the reservoir.

Metals that are embedded in wood can experience accelerated corrosion rates compared to that of typical atmospheric corrosion, since the wood can retain moisture and also serve as an electrolyte, allowing the chemical reactions to occur. The following conditions are considered to accelerate corrosion of embedded metals in wood:

- A wood moisture content that exceeds 15 percent
- Relative humidity that exceeds 70 percent
- Use of wood preservatives that contain copper ions
- Use of dissimilar metals (hangers and fasteners of different materials)

The investigation made by Western Wood Structures indicated that the wood moisture content, the greatest single driving factor in corrosion of embedded metals in wood, was measured to be 6.5 percent at the time of the investigation. These conditions can change. The exposed ends of the glulam beams, if permitted to get wet, can wick large amounts of moisture into the beam along its full length. We recommend waterproofing the glulam beam-ends by providing a protective covering or mastic coating that will prevent water from entering in through the beam ends. The existing beam-ends are currently exposed and appear to be painted. Some appeared to be wet during our site visit.

It should also be pointed out that the aluminum Zip-rib roof system may include aluminum fasteners. Product literature for the wood preservative (CCA) used in the roof framing members, indicates that aluminum should not be used in direct contact with wood that has been treated with CCA. We understand that the aluminum can deteriorate when placed in contact with CCA.

We recommend that a corrosion survey be made of the interior framing connections within the interior of the reservoir. The conditions within the reservoir are conducive to corrosion and some evidence has been visually observed suggesting that fasteners are corroding and failing. Since the framed lumber members are supported on the glulam beams using face-nailed hangers, nail failure can result in collapse of the joist.

2.1.3 <u>Reservoir Roof Structure Seismic Load Path and Structural Diaphragm</u>

Wood-framed roofs are typically encountered in buildings, where plywood sheathing is provided and serves to transfer seismic loads acting on the roof structure to the lateral load resisting system by diaphragm action. This is the most common framing method to transfer seismic loads for a wood-framed roof. However, while a diaphragm can efficiently transfer these loads, not all wood-framed roofs require a diaphragm to transfer seismic loads. Often, wood roofs over reservoirs are designed such that seismic loads are transferred from the roof framing members through weak-axis bending (bending sideways). This can be feasible when the roof framing weight is relatively light and spans are not excessive. These loads are then transferred into beam lines that are braced by perimeter walls, drilled piers, large pilasters, or other lateral load resisting elements. This system can be present in both orthogonal directions.

Based on our work on existing reservoirs of similar design and construction, the seismic load paths provided for the roof structure usually consists of using the roof wood joists and beams as the main seismic load transferring members. For the Peters Canyon Reservoir, in the north-south direction, the original seismic load path to support the roof is provided by transferring the roof seismic loads generated in each bay, along each of the column lines, by imposing axial loads into the wood members. This will create tension and compression axial loads in the wood members based on the direction of seismic loads. The wood beams transfer these axial seismic loads to the perimeter concrete piers as an out-of-plane lateral load. The piers and the footing resist the loads and transfer them into the soil. For the wood beams to act as axial tension and compression members, the seismic load has to be transferred to these main column lines. This can be achieved by providing a structural diaphragm to transfer the seismic mass to these column lines within each bay. Alternatively, if no structural diaphragm is present, the seismic roof mass loads can be transferred by out-of-plane bending and/or through axial loading of individual purlins. If the wood members have sufficient capacity to resist the out-of-plane bending and axial loads, then a structural diaphragm for seismic loads may not be required to complete the seismic load path.

The seismic load path in the east-west direction is similar except that the seismic loads are transferred through the roof joists in axially to the roof beams, which then bend about their weak axis and transfer the loads to “collector” lines spaced at about 40 ft on center. These collector lines (total of 3 at the interior) are comprised of glulam beams that transfer loads axially into chevron-style braced frames (inverted V’s) that are comprised of precast concrete bracing members. Seismic loads are then transferred into the soil via isolated pad footings. The north and south walls are CMU walls, which resist east-west seismic loads in shear.

While the transfer of seismic loads using weak-axis bending and axial load transfer (as opposed to using a diaphragm) appears to be the intent of the original structural engineer, alternatively, the tension compression in wood members can be ignored and a structural diaphragm can be provided with sufficient capacity to transfer the seismic lateral loads to the perimeter 8-in thick CMU walls for the loads acting in each direction.

The seismic load values have increased over the decades based on advances in knowledge and information about new faults and intensity of earthquakes and the response of buildings to these earthquakes. The reservoir roof structure will most likely be found to be deficient if the seismic loads calculated per the 2013 CBC are considered. A more detailed analysis and capacity checks have to be performed if the existing structural roof system is to be left in place and a retrofit strengthening option is chosen as the ideal solution to this reservoir.

In a typical roof structure, structural diaphragms perform two main functions. The first being that, in high seismicity areas a structural diaphragm acts as the main horizontal lateral load transfer element to transfer the seismic load from the roof structure mass to vertical lateral load resisting elements. The second function of a structural diaphragm is provide lateral bracing to wood purlins, beams and girders to prevent compression edge lateral buckling when these members are resisting bending in their strong axis due to gravity, wind, and seismic loads. The National Design Specification for the design of wood members sets forth prescriptive recommendations for providing stability of bending members. Accordingly, 2 by 12 and 2 by 14 purlins will require a structural diaphragm on the compression edge and full depth blocking or diagonal cross bracing at every 8 ft on center to prevent lateral buckling under design loads. For the 3 by 14 and 5.25 by 21 glulam beams, no compression bracing is needed due to their smaller depth to width ratios which makes them more stable against lateral buckling. Note that under gravity dead and live loads the compression edge is on the top of the wood members. Under uplifting wind loads there could be a net tension uplift loads on the wood members causing compression on the bottom edge of the members. To prevent lateral buckling under wind loads additional full depth blocking may be needed at the 2 by 12 and 2 by 14 purlins as providing a structural diaphragm on the bottom edge of these members will not be practical. This lateral buckling might have been one of the reasons why roof purlins failed in 1996 under high wind loads. Use of full-depth blocking is standard practice and this reservoir appears to lack this provision.

Additionally, a structural diaphragm on the top edge will prevent weak axis buckling of 2 by 12 and 2 by 14, 3 by 12 and 3 by 14 purlins if axial loads are imposed on these members. The Zip-Rib aluminum roofing is not considered to be a structural diaphragm as it is not rigidly connected to the purlins and has sliding connections to let it expand and contract due to temperature changes.

There are two options for providing a structural diaphragm: a galvanized corrugated metal decking or plywood sheathing. The plywood sheathing if used has to be preservative treated to prevent decay due to the presence of moisture. Wood preservatives are pesticides that protect wood against attack by fungi, bacteria, or insects. The active ingredients found in wood preservatives may include pentachlorophenol (pent or PCP), creosote, copper, zinc, chromium, arsenic and other compounds. Use of wood preservatives is now restricted and regulated by the Environmental Protection Agency (EPA). The EPA has published recommendations regarding the use of various types of preservatives and has concluded that they should not be used where they may come into direct or indirect contact with public drinking water. Since water condensation often accumulates under the roof deck and the roof deck may leak when it rains, wood preservatives in the wood framing can potentially leach into the water, albeit in small amounts at a time. If the roof structure were to collapse into the reservoir due to an earthquake, the wood preservatives can leach into the water at potentially higher rates than in the supported position out of the water. Marine-grade plywood is a durable product, but manufacturer's recommend that this plywood be preservative-treated when its use will be subject to long-term exposure to moisture.

Due to the above-mentioned reasons, preservative treated plywood sheathing is not recommended to be used as a structural diaphragm for the Peters Canyon Reservoir. Instead, a galvanized metal decking should be used as a structural diaphragm.

Note that based on the existing drawings, the original wood members and plywood sheathing used around the perimeter were treated with a wood preservative called Wolman CCA (chromated copper arsenate). As mentioned above the EPA does not allow the use of preservative treated wood near public drinking water. Furthermore, the EPA has established guidelines for the safe handling and disposal of preservative treated products. Mitigation strategies that involve the demolition of the existing roof framing may need to specify disposal requirements and limitations regarding recycling of the wood framing members.

We contacted Western Wood Preservers Institute (WWPI) and we were informed that, given the age of the structure, any leaching of the wood preservative is likely to have diminished significantly since the time of construction. However, it is not known if leaching would occur and at what rate, if the wood framing were to collapse into the reservoir. It should also be noted that the repairs made in 1997 to the wind-damaged roof sections involved the introduction of new replacement wood framing that was treated with a wood preservative, which is assumed to be CCA.

It is advisable to conduct testing to verify the content of CCA in the existing wood-framed members. The American Wood Protection Association (AWPA) has established wood testing procedures that can be used to help identify the content of preservatives in wood. Testing for chemical content in water that has been exposed to CCA treated lumber may be an alternative means to verify if this is a valid concern or not.

In lieu of using wood preservatives, newer reservoir roof covers that have been constructed of wood framing have used sawn lumber and glue-laminated beams that have a natural resistance to decay. Redwood and Alaska Yellow Cedar are two wood species that have a natural resistance to decay. The Van Norman reservoir, owned and operated by the Los Angeles Department of Water and Power (LADWP), was constructed in August of 1992. LADWP engineers did not want to introduce additional chemicals to the water and chose to use Alaska Yellow Cedar, a naturally durable species of wood that is commercially available for use in structural wood framing applications.

2.2 Additional Seismic Calculations

2.2.1 Sloshing Wave Height and Freeboard

The sloshing action of the water within the reservoir during an earthquake can generate a maximum wave height at the perimeter of the structure. When insufficient freeboard is provided, the water can slosh and surcharge the bottom side of the roof framing at or near the perimeter of the structure. The surcharge force will be directly proportional to the amount of freeboard deficit. The associated loading to the underside of a roof structure can be substantial and cause significant damage or collapse, especially when the roof is framed with a lightweight material.

Since most reservoirs operate at their high water level for substantial amounts of time, the wave height was estimated assuming the reservoir is full. This wave height was determined using the equations and procedures set forth in American Society of Civil Engineers (ASCE) 7-10 and American Concrete Institute (ACI) 350.3. The higher of the two values determined from these two different codes was used conservatively for this evaluation. The sloshing wave height and the required freeboard were estimated to be 37 in. Currently the freeboard available to the underside of the aluminum roofing is 34 in. Given this small amount of surcharge, damage to the roof framing is not expected to be substantial at the sloshing levels estimated.

2.2.2 Soil Rod Anchor Retrofit at North-South Wall Piers

In 1999, steel rod rock anchors were installed at 24 pier footings along the north and south walls. The pier footings were also extended to accommodate the anchors. It appears that these rock anchors were installed to increase the frictional lateral load carrying capacity of the pier footing to resist lateral loads imposed by the glulam beams in a seismic event. The rock anchor provides a net vertically downward acting force on the pier footing which will result in a net increase in frictional capacity of the pier footing. We estimated that the lateral load imposed by the glulam beams on each of the pier footings is approximately 8.6 kips at service load level. The rock anchor with 30 kips of net axial design load, as shown on the structural drawings, is estimated to provide up to 13.8 kips (service level) of frictional capacity to resist the lateral load, assuming a soil friction coefficient of 0.3.

The reaction at the base of the pier footing can be applied in a direction toward the reservoir or away from the reservoir. For loads applied away from the reservoir, the full frictional resistance is expected to be developed. However, in the direction towards the reservoir, the bottom of the footing butts up against the reservoir sloped liner. Development of the full lateral load capacity of the footing is questionable and loading of the piers during an earthquake can potentially result in direct loading of the sloped liner, which can cause cracking or more significant damage to the liner.

The selection of rock anchor retrofit confirms the tension-compression axial load seismic load path described in Section 2.1.1 was used by the structural engineer to resist seismic loads in-lieu of providing a structural diaphragm.

2.2.3 Seismicity due to Recently Studied faults

Carollo Engineers contracted with Converse Consultants to evaluate the seismic design parameters for the Peters Canyon Reservoir site taking into account recent fault data for the nearby Peralta Hills Fault. The results of this study are provided in Appendix C to this report. The ground accelerations determined from this study are slightly higher (about 10 percent) than those values obtained through the current building code and the United States Geological Survey data.

Any future seismic evaluations, retrofit designs, and/or new designs at the site should use the values determined by this study.

2.3 Onsite External Assessment and Slope Stability Analysis

The exterior assessment of the reservoir was limited to a visual assessment of the reservoir from what could be seen from a walk around the perimeter of the reservoir. In general, the roofing appears to be in good condition with evidence of localized sealant applications. The following observations were made:

- The plywood panels located along the east and west sides of the reservoir are in relatively poor condition and appear to have evidence of moisture damage and potential rot. Replacement is recommended as conditions worsen. However, panels should not be replaced with preservative-treated members for reasons previously discussed. We recommend that panels be replaced with Marine grade plywood or an alternative material that is more durable. Marine grade plywood will likely last much longer than conventional plywood, but replacement should be anticipated at some time in the future as well due to degradation.

- The glulam beam ends that are exposed on the north and south walls of the reservoir are painted and flashed around. The paint and caulking on the south side are in poor condition. We recommend capping the ends of the glulam beams with a heavy mastic coating and replacing the flashing with a durable material.

Carollo Engineers contracted with Converse Consultants to review a previous slope stability analysis prepared by American Geotechnical in April 2014 and incorporate new fault data for the Peralta Hills Fault. The results of that review are presented in Appendix C of this report.

2.4 Develop and Analyze Alternatives

2.4.1 Alternative 1 – New Aluminum Roof – System Vulnerability

In this alternative, a new aluminum roof structure will be installed with its own structural column structure for gravity support and lateral system for seismic load resistance. All existing components of the existing roof, including wood glulam beams, purlins, and concrete columns will be demolished and removed from the site. It was estimated by Brady that the total construction time for this option would be nine months.

Replacement of the existing reservoir roof assumes that the footprint of the existing reservoir would remain unchanged. The cost to re-size the reservoir was not considered in the estimate provided by Brady. Such work would involve additional demolition and earthwork that will increase the cost and extend the duration of construction. This alternative, as it is currently being evaluated, would not provide additional space on the site for future development. Alternatives that involve replacement of the reservoir with a circular tank inherently allow for site re-development.

2.4.2 Alternative 2 – Replacement with New Prestressed Concrete Reservoir

We reviewed the quotation from DN Tanks for a replacement reservoir dated September 28, 2014 and the Brady update report from September 2014 in regards to the budget and construction duration. As noted by Brady a new prestressed concrete reservoir will provide the highest reduction in seismic risk and a reliable source of water storage for a service life of at least 50 years.

The following elements need to be considered while evaluating the replacement PT tank alternative.

- Temporary isolation of Peters Canyon Reservoir
- Demolition of the existing reservoir
- New piping
- The prestressed tank diameter is 165 ft and hence the hydraulic grade line will need to be raised to 803 ft to maintain the same volume. The current hydraulic grade line is at 790 ft. This higher-grade line may require installation of a pressure-reducing valve.
- The smaller size prestressed concrete tank can be located towards the east or west end of the current reservoir and the additional space of approximately 140 ft can be used for other purposes.
- Tank appurtenances
- Backfill and site work

In order to mitigate the loss of reservoir capacity during construction, a much smaller temporary bolted steel tank or tanks can be provided on the site to meet the emergency/mandatory water supply requirements while the prestressed concrete tank is constructed. This assumes that the water storage capacity required to be maintained at the site is a fraction of the full storage volume.

Figures 3, 4, and 5 show a conceptual layout and sections of a new prestressed concrete tank describing the construction work anticipated. We used the layout shown in these Figures to estimate the rough construction cost. We located the tank towards the west end of the existing reservoir to provide additional space on the east end, which could be valuable space used for other necessary site developments, such as new or expanded water treatment facilities.

For planning purposes, the following simplifying assumptions have been made in the development of our cost and schedule estimate:

- The existing soils require no additional improvement or replacement.
- Deep foundations, such as piles, are not required.
- The cost of temporary steel bolted tank(s) was not included.
- The replacement reservoir will be partially buried.
- The bottom of a new reservoir will not be any deeper than the existing reservoir.
- Water quality improvements, provided by baffling and mixing, for example, are not considered.
- Dewatering is not considered.
- Cost of a pressure relief valve was not considered.
- Soft costs for permitting, engineering, etc. are not included.

Based on our estimates the total construction cost for a new prestressed concrete tank having a storage capacity of 5.8 MG along with the associated infrastructure will be $8.1 million. This cost involves a substantial amount of earthwork because the layout considered pushes the tank to the west to maximize space on the site to the east. If a smaller volume tank is feasible, the project cost would be reduced due to a smaller tank size and potentially less earthwork. Other potential tanks sizes that can reduce earthwork, increase available space on the site, and reduce the project cost are as follows:

- A 165-ft diameter prestressed concrete tank with a storage capacity of 4.0 MG with the high operating level matching existing (cost estimate of $6.4 million).

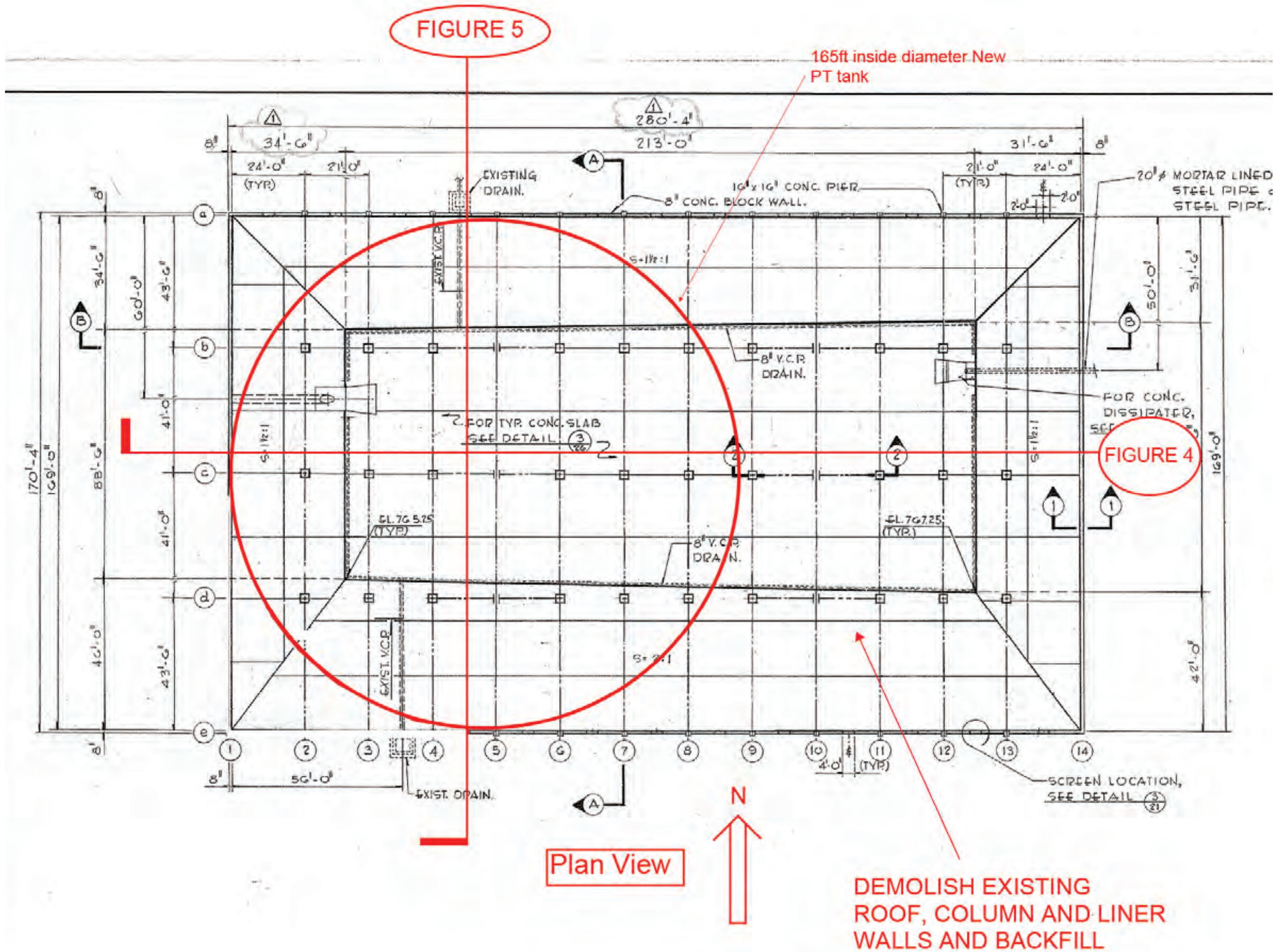

Figure 3
Peters Canyon Reservoir - Layout of New PT tank

East Orange County Water District
Condition Assessment

carollo
Engineers...Working Wonders With Water®

EAST ORANGE COUNTY WATER DISTRICT

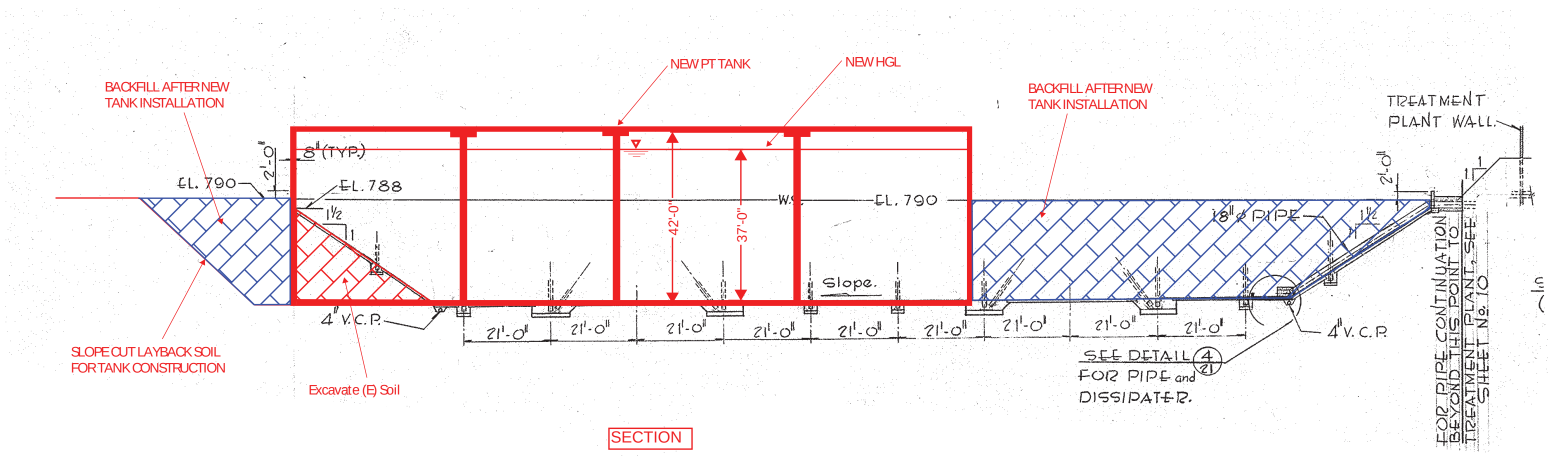

Figure 4
Peters Canyon Reservoir - East West Section of New PT tank

East Orange County Water District
Condition Assessment

carollo
Engineers...Working Wonders With Water®

EAST ORANGE COUNTY WATER DISTRICT

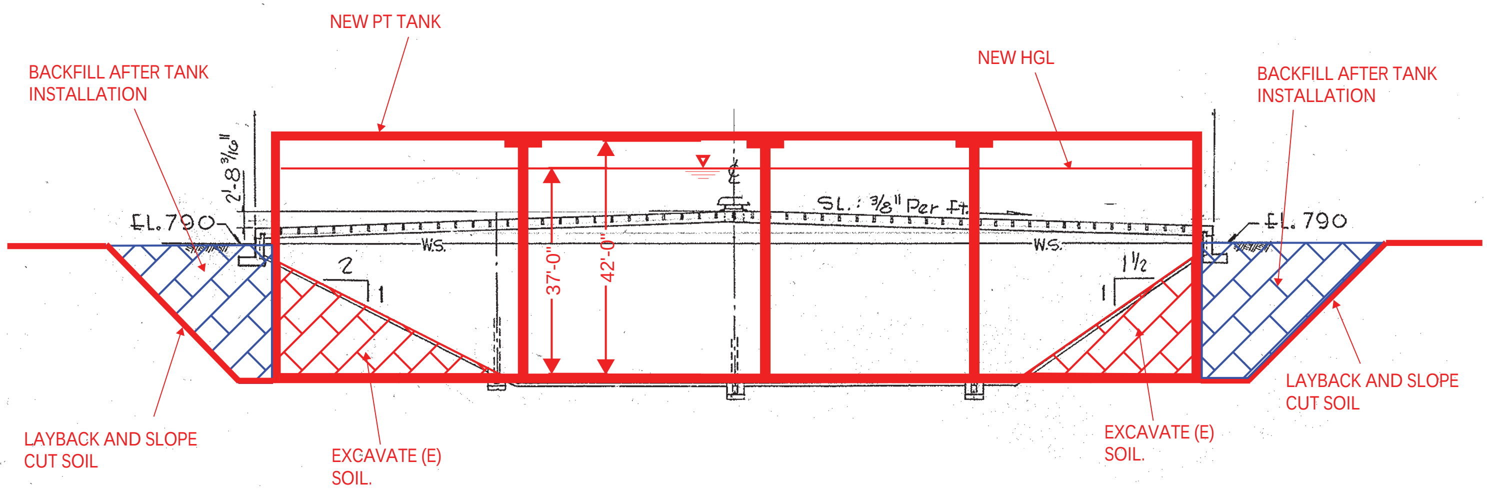

Figure 5
Peters Canyon Reservoir -North South Section of new PT tank

East Orange County Water District
Condition Assessment

carollo
Engineers...Working Wonders With Water®

EAST ORANGE COUNTY WATER DISTRICT

- A 143-ft diameter prestressed concrete tank with a storage capacity of 3.0 MG with the high operating level matching existing (cost estimate of $5.6).

We estimate that the construction of a new prestressed concrete tank will take about 12 months excluding the construction of a new road around the tank. Therefore, depending on the tank size and layout of the tank at the site, the cost to replace the existing reservoir with a prestressed concrete tank can vary from $5.6 million to $8.1 million.

2.4.3 Alternative 3 – Flexible-membrane Floating Cover - Hypalon

Flexible-membrane linings and floating covers can be incorporated into many types of water-storage facilities, both new construction and rehabilitated structures. The water industry began using floating covers using ethylene-propylene-diene monomer (EPDM) synthetic rubber and less commonly polyvinyl chloride (PVC), during the mid-1960s. Since then, improvements in design, materials, and operating procedures have resulted in hundreds of successful floating-cover installations in the United States and throughout the world. In the United States, more recently, most have been constructed using reinforced elastomeric membranes of chlorosulfonated polyethylene (CSPE-R), commonly know as Hypalon™. The following sections provide a detailed review of Hypalon as a floating cover, feasibility of using it at Peters Canyon, and a cost estimate for planning purposes.

2.4.3.1 Background Information

Hypalon is the registered trademark for a series of chlorosulfonated polyethylene (CSPE) synthetic rubbers manufactured by DuPont Dow Elastomers. Vulcanizates of Hypalon are highly resistant to the deteriorating effects of ozone, oxygen, weather, heat, oil, and chemicals. It can be compounded to give excellent mechanical properties-for example, high tensile strength and abrasion resistance. Several types and grades of Hypalon are available for a variety of end use requirements. One such end use is in the potable water industry for lining and floating covers over reservoirs. Hypalon used as a floating cover is manufactured into plies that are combined over reinforced polyester scrim layer or layers. It is available in up to 5 ply layers. Hypalon as a floating cover is available in 36, 45, and 60 mil thicknesses and up to 90 mils by special order.

Hypalon as a floating cover is fabricated by heat welding smaller panels into larger panels in a fabrication facility. The panels are finally field seamed together into one large cover using either heat-welding or solvent adhesives.

The two most common floating cover designs are the “defined sump” and “Mechanically Tensioned”. The defined sump covers use floats and weights to create rainwater collection sumps in the cover and to accommodate changes in water level. Tensioned covers use cables attached to tensioners to control slack as the cover moves up and down. Tensioned covers are mechanically tensioned around the periphery, which eliminates sumps and the main panel cover is maintained flat.

Hypalon floating covers have been used by numerous water districts within United States and elsewhere successfully for about 35 years. Below are a list of general advantages and disadvantages associated with Hypalon floating covers:

Advantages

- The capital cost of fabrication and installation is low compared to any other traditional structural roof system.
- Hypalon is flexible and hence easy to install and can be installed in a short duration of time.
- Hypalon is durable and possess desirable properties for a potable tank cover, such as resistance to chemicals, extreme temperatures, and UV degradation.
- Manufacturers provide standard warranty and extended warranty can be negotiated at the time of order for the material. Warranty for material from the manufacturer is typically 30 years. The warranty from the bonding installer is about 1 to 2 years.
- Hypalon covers have proven repair and maintenance procedures.

Disadvantages

- The service life of a floating cover is around 20 to 25 years, which is shorter than a service life of a new structural roof framing system.
- Floating covers will have additional life cycle costs associated with operation and maintenance. Some of these include regular inspections, dewatering of rain water, disinfection of interior covers, cleaning of the cover regularly to remove foreign objects, inspection of pumps and auxiliary equipment, immediate repair of pin-holes, etc.
- Although some floating-cover designs incorporate inflation or suspension as a means of providing access for interior cleaning, operators should remember that even a moderate wind could cause severe damage or destruction of the cover under certain conditions.
- Modifications to the existing reservoir have to be performed to accommodate a floating cover roof. Some of the modifications may involve new concrete curbs, chafing strips, baffle-walls and perimeter anchor posts.
- Maintenance crew and personnel have to undergo training to safely and properly operate a floating cover roof.
- Hypalon covers need to be inflated for inspection and cleaning inside of the reservoir.

- Inspection of the floating cover may be required on a frequent basis to immediately repair any damage to the floats and membrane for proper operation of the floating cover.

Cover owners may choose to use their own crew to perform yearly maintenance or they can hire the installer. A detailed non-technical account about the design, operation, and maintenance is provided in AWWA Manual M25, Flexible-Membrane Covers and Linings for Potable-Water Reservoirs. It is highly recommended that EOCWD personnel review this document to get an understanding of installation and operation of a floating cover system before a decision is made to pursue this alternative.

2.4.3.2 Peters Canyon Reservoir Floating Cover Feasibility

Based on our review of the existing Peters Canyon Reservoir and the site around, it is our conclusion that a Hypalon floating cover is a feasible alternative provided it is properly designed and installed. The drawings and specifications for a cover system should be prepared by a licensed engineer having specialized knowledge and experience in the design and operation of flexible-membrane floating-cover systems. The following list provides a summary of the things to consider for designing, installing and operating a floating cover specific to this reservoir. The typical advantages and disadvantages associated with floating covers described in the previous section should be considered in conjunction with the following:

- Demolition of the existing structural roof for the installation of a new floating roof cover will likely damage the existing Hypalon liner on the floor and slopes of the reservoir. A new Hypalon liner on the reservoir floor and slopes may need to be installed to replace the damaged liner.
- Anticipate improvements and repair of the existing concrete liner slab and piping.
- The concrete columns have concrete fill around them at the column foundation. The removal of the concrete column will require placing of new concrete fill and joint at these locations.
- Along the perimeter 4-ft tall CMU wall, existing wall vent openings and voids created by removal of Glulam beam seats and purlins will need to be infilled with concrete.
- Existing perimeter CMU wall will have to be checked for the loads imposed by the Hypalon cover to verify if any additional strengthening of the wall is required.
- 10-ft of access around of the perimeter of the reservoir will be required for installation of the floating cover. This may require removal of existing soil and installation of shoring on the east side of the reservoir adjacent to the existing water treatment plant.

- Environmental conditions such as local vegetation, nearby trees and likely tree debris, animals, smog and other conditions should be reviewed.
- If the surrounding area is prone to animal intrusion that could cause damage to the cover, a security fence should also be designed and provided.
- The slopes of the reservoir are 1.5:1 (horizontal to vertical) on three sides and 2:1 on the fourth side. The ideal slope for maintenance crews without needing any special harnesses would be 2:1 and flatter (moderate slope). So on the three surfaces where the slope is 1.5:1, safety gear may need to be incorporated for maintenance crews to access the reservoir cover.
- The required minimum freeboard for a sloshing wave due to the design-level earthquake for this reservoir is approximately 2 ft in the east-west direction and 3.1 ft in the north-south direction. Floating covers are not typically designed for surcharge loads from sloshing waves. If the floating cover is installed such that the top of the cover is flush with the top of the perimeter wall at elevation 792 ft, the water level will have to be dropped down from current operating level of 790 ft to 789 ft. This will result in a loss of about 356,000 gallons of storage capacity.

We estimate that the total construction cost of a new floating cover will be on the order of $2.4 million. In addition a yearly operation and maintenance cost $12,000 per year shall be considered. In addition, the service life of a floating cover is around 25 years. Since other structural replacement alternatives have a greater expected service life, for comparison purposes, an additional replacement cost should be considered. We performed a life cycle cost analysis over a 40 year interval that includes operation and maintenance costs and capital costs. An interest rate and inflation rate of 3 percent and 2 percent were assumed, respectively. Based on our calculations, an additional $2.7 million should be added to the current capital cost to account for the replacement of the floating cover at year 25. Therefore, the total cost of the floating cover reservoir will be on the order of $5.1 million. We estimate that the total construction time needed for installing a floating cover will be approximately five months.

The floating cover system that was considered in this evaluation assumes that the cover would be placed over the footprint of the existing reservoir. The cost to resize the reservoir was not estimated. Such work would involve additional demolition and earthwork that will increase the cost and extend the duration of construction. This alternative, as it is currently being evaluated, would not provide additional space on the site for future development. Alternatives that involve replacement of the reservoir with a circular tank inherently allow for site redevelopment.

The cost of providing temporary tankage during construction was not evaluated for the floating cover or the prestressed concrete tank alternatives.

3.0 VISTA PANORAMA RESERVOIR

3.1 Review of Existing Drawings and Reports

Based on the review of the report prepared by Simon Wong Engineering (SWE) and our site visit, the following are our detailed conclusions about the condition of the various structural elements. We generally concur with the conclusions presented by SWE in their report with the exceptions and additional issues set forth in this report. The SWE proposed recommendations to mitigate leakage in the tank is a feasible alternative.

3.1.1 Existing Wall Thickness and Reinforcing

There are no structural drawings available to review the construction details of this reservoir. The SWE report notes that the thickness of the tank walls are 12 to 13 in with one layer of horizontal reinforcing based on a concrete core sample provided by EOCWD. The core sample indicates that the concrete tank wall has one horizontal layer of 5/8 in square rebar at 4 in on center located near the outside face of the wall with 1 in of concrete cover. There was no vertical rebar present in the core sample.

We measured the thickness of the concrete wall by measuring the distance of the wall exterior and interior faces relative to the roof hatch opening. Based on our measurements the wall thickness is approximately 8 in. We conclude that the concrete core may not be representative of the Vista Panorama tank walls. Furthermore, the surfaces of the concrete core sample did not appear to match that of the tank, which is painted pink. Also, evidence of patching of a concrete core was not evident based on our visual observations. Accurately knowing the size and spacing of the wall reinforcing steel is paramount to determining the structural adequacy of the tank construction with respect to seismic loading. If the core sample used as the basis of the reinforcing steel is not associated with the wall, to confirm the as-built conditions, we recommend that additional cores be taken through both horizontal and vertical steel by first mapping the rebar locations inside the wall using a rebar location detector.

3.1.2 Proposed new concrete or masonry water containment wall

A 3 ft tall concrete or masonry wall was proposed by SWE for delaying the water flow onto the street in case of reservoir rupture due to a major earthquake. The following are the additional factors, which need to be considered for proposal:

- The available space around the existing concrete tank is limited and it may not be feasible to build a concrete wall around the tank. Existing controls that are located very near to the wall of the tank would need to be relocated.
- In an event that the existing tank was to rupture, the proposed wall may not be effective to restrain water resulting in an uncontrolled release that can damage

existing improvements at the site (footing scour and erosion) and an unknown impact to the adjacent private residences.

Based on this, we conclude that this proposed new wall may not be effective in mitigating potential release and damage.

3.2 Additional Seismic Calculations

We performed seismic analyses and capacity checks of the existing structure to identify potential structural deficiencies. We made certain assumptions for the structural parameters to perform this evaluation, as information about existing construction was limited. As discussed in Section 3.1 we determined that the thickness of the concrete wall is approximately 8 in based on our field measurements. The information regarding reinforcing in the wall is not known to be reliable at this time, therefore two scenarios for the rebar layout in the wall were considered to obtain the range of wall capacities possible. The two scenarios we evaluated are

- Case 1 – Assumed 5/8 in square rebar at 4 in on center in both horizontal and vertical direction. Assumed that the vertical rebar is in the middle of the wall.
- Case 2 – Assumed 5/8 in square rebar at 12 in on center in both horizontal and vertical direction. Assumed that the vertical rebar is in the middle of the wall.

The analysis was performed using ASCE 7-10, ACI 350-06 and ACI 350.3-06. Table 1 provides a summary of the material properties assumed based on the age of construction using the data provided in ASCE 41-13.

Table 1 Material Properties – Concrete and Reinforcing Steel
Peters Canyon and Vista Panorama Reservoir Condition Assessment
East Orange County Water District

Material	Property	Code/Source Reference
Concrete	$f'_c = 2{,}000\ psi$ $f'_{ce} = 3{,}000\ psi$	ASCE 41-13 Tables 10-1 and 10-2
Reinforcing Steel	$f_{ymin} = 33\ ksi$ $f_{ye} = 41\ ksi$	ASCE 41-13 Tables 10-1 and 10-3

Based on the assumed rebar size and spacing the following three cases are possible. These three cases describe the structural elements demand capacity ratios and corresponding operating storage capacity for the tank. Use of ultrasonic pulse echo or taking field concrete cores of the tank are recommended to verify the rebar size and spacing for comparison against the assumptions and conclusions described in this report. Once the as-built information is verified, the structural deficiencies from a seismic load resistance perspective can be finalized and appropriate mitigation techniques can be explored. The three cases identified below provide a range of behavior of the existing concrete tank in a seismic event:

3.2.1 Case 1

In this case, we assumed that the rebar is 5/8-in square at a spacing of 4 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at the high operating level of 11.8 ft above the floor of the tank. The storage volume at this height will be 150,000 gallons. The structural walls are capable of resisting the seismic loads imposed on them for this condition. The seismic sloshing wave height is approximately 3.4 ft. The available freeboard for this case is 0.2 ft and is not sufficient to accommodate the 3.4 ft sloshing wave height. The wood-framed roof will be damaged and will likely collapse due to the surcharge loads imposed by a sloshing wave. The spillage caused by this sloshing wave may also cause secondary damage by eroding the soil underneath existing structures near the tank and compromise their load carrying capacity. A mitigation strategy would be to lower the operating height of the water to 8.6 ft above the base of the tank. The volume at this reduced operating water level would be 109,000 gallons. Alternatively, replacement of the roof framing system with one that is capable of resisting the surcharge load or increasing the wall height to provide the necessary freeboard would allow utilization of the full storage capacity.

3.2.2 Case 2

In this case, we assumed that the rebar is 5/8-in square and spaced at 12 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at the high operating level of 11.8 ft above the floor of the tank. The storage volume at this height will be 150,000 gallons. The horizontal hoop rebar for this case is overstressed by 62 percent from height 2.5 ft to 9.5 ft (6.9 ft of wall height) measured from the top of the tank. The seismic sloshing wave height is approximately 3.4 ft. The available freeboard for this case is 0.2 ft and is not sufficient to accommodate the 3.4 ft sloshing wave height. The wood-framed roof will be damaged and will likely collapse due to the surcharge loads imposed by a sloshing wave. The spillage caused by this sloshing wave may also cause secondary damage by eroding the soil underneath existing structures near the tank and compromise their load carrying capacity. A mitigation strategy would be to lower the operating height of the water to 8.6 ft above the base of the tank. The volume at this operating water level would be 109,000 gallons. Alternatively, replacement of the roof framing system with one that is capable of resisting the surcharge load or increasing the wall height to provide the necessary freeboard would allow utilization of the full storage capacity.

3.2.3 Case 3

In this case, we assumed that the rebar is 5/8-in square and spaced at 12 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at a high operating level of 8.6 ft above the floor of the tank. The storage volume at this height would be 109,000 gallons. For this case, the existing tank rebar assumed is sufficient to resist the seismic loads imposed and the seismic sloshing wave height is less than the freeboard provided.

3.3 Onsite External Assessment

The exterior assessment of the reservoir was limited to a visual assessment of the reservoir from what could be seen from a walk around the perimeter of the reservoir and through the roof hatch. In general, the roof wood structure appears to be in good condition. The tank wall appears to be in good condition. The water level was being operated at around 3 ft so no visible leakage was observed. We concur with the SWE documented issues with water leakage and rusting of rebar. The following additional observations were made:

- The wood members do not seem to have been preservative treated for long-term durability. Decay and rot is expected in the future as the wood is exposed to moisture inside the tank and may have to be replaced at some time in the future. We recommend making regular inspections of the condition of the roof framing and the hardware.
- Based on the review of the photos taken by SWE, the steel posts at the bottom of the tank do not seem to be positively anchored to the floor slab. We recommend that a positive connection be provided by using anchor bolts from the base plate to the concrete floor slab.

3.4 Develop and Analyze Alternatives

3.4.1 Alternative 1 – Maintain Current Operating Condition - System Vulnerability

Due to concerns that EOCWD has about the condition of the reservoir and its potential vulnerability to leakage, it is currently operated with only 5,000 gallons of water in it. In the current partial operation condition, the water height would be at 3 ft above the base. One alternative would be to continue operating in this mode, but this is not the best use of the storage volume.

3.4.2 Alternative 2 – Seismic Rehabilitation

A complete full compliance seismic rehabilitation should be based on the structural deficiencies of the existing structure. Performing a complete seismic analysis of the existing structure would involve performing a detailed analysis and checking the existing foundation system, tank walls, and wood-framed roof structure. In Section 3.2 the tank walls, which are the main structural elements, have been analyzed in detail to identify potential structural deficiencies. The available structural element information of the tank structure is very limited and is required to provide the most suitable seismic retrofit alternatives. Based on the observed deficiencies a feasible seismic retrofit alternative may involve strengthening portions of the tank walls. This may be accomplished by one of the following or similar approaches:

- Provide supplemental circumferential reinforcing steel in the areas where the hoop rebar is deficient along the exterior of the tank wall. This approach would also require epoxied dowels into the existing wall at regular intervals. The reinforcing steel would need to be encapsulated in shotcrete or a formed concrete wall.

- Provision of an exterior steel shell and/or steel beam sections is another approach that appears to have been proposed by Premier Tank. This approach appears to be capable of providing ample supplemental reinforcing steel in the form of steel plate and beam sections. However, bonding of this retrofit should require small dowels, which can be located between the shell and the wall. The steel should be left exposed and properly coated for protection from corrosion. Attempting to shotcrete cover or finish over with a cementitious material is not advised as the difference in the coefficient of thermal expansion between the steel and large sections of steel may cause brittle finishes to crack and spall.

- Provision of post-tensioned 7-wire strand encapsulated in a shotcrete finish may be feasible provided that the additional post-tensioning does not induce excessive bending moments in the vertical reinforcing steel of the tank. This system imposes an active compressive load on the tank shell that can be designed to limit the development of tensile forces, thereby reducing the demand on the existing reinforcing steel. The feasibility of this retrofit approach would need to be verified by checking the required pre-compression force required and the additional bending moments and shear forces that would be induced by the post-tensioning.

The SWE report is limited to providing recommendations to prevent leakage issues with the tank and does not provide any recommendations for seismic retrofit of the existing tank. The Premier tank quotation does not seem to be based on an engineered solution to rectify existing structural deficiencies. In our opinion, the Premier tank proposed steel plates, angle and wide flange hoop at the top seems to be an excessive strengthening which may or may not rectify the structural deficiencies in the existing structure. We recommend that as-built structural information be identified through non-destructive testing and a limited number of core samples. Using the as-built information, a more detailed structural analysis using the Section 3.2 analysis as the basis to provide an appropriate full compliance seismic retrofit option.

3.4.3 Alternative 3 – Construct a new replacement reservoir

The most appropriate replacement reservoir for the Vista Panorama would be a bolted or welded steel tank. Steel tanks typically have a lower capital cost for at-grade construction compared to other alternatives and can be erected relatively fast. However, these tanks require a protective coating on the interior and exterior to protect it from corrosion. The tank will require a recoating every 20 to 30 years. Estimates for recoating a tank can be highly variable depending on the type of coating, condition of the tank steel, and air quality regulations. Cathodic protection can also provide additional protection.

The seismic performance of properly designed steel tanks can be excellent, provided those inherent vulnerabilities are carefully addressed. Such vulnerabilities include the tendency for the shell to buckle, excessive pipe restraint, tank uplift, and sloshing of water surcharge to the tank roof. These vulnerabilities were manifested in the 1994 Northridge Earthquake and subsequent large earthquakes throughout the world since that time. Numerous welded steel tanks failed with collapse, severe damage, foundation scouring, and loss of the tank contents. Steel tanks designed in accordance with current AWWA D100 standards are anticipated to have a significantly improved seismic performance compared to its predecessors. Fittings at pipe inlets and outlets should include flexible connections that allow for differential movement between the tank and the surrounding grade.

Leakage from steel tanks are expected to be minimal provided the tank is maintained in excellent condition.

When considering steel tanks, to understand the true cost of ownership, a life cycle cost analysis is recommended. A life cycle cost over a period of 40 years has been estimated. In development of our cost estimates we made the following assumptions:

- The existing soils require no additional improvement or replacement.
- Deep foundations, such as piles, are not required.
- Vista Panorama Reservoir may be taken offline, demolished as required, and replaced without the need to maintain temporary water storage to offset the lost volume during construction.
- The bottom of a new reservoir will not be any deeper than the existing reservoir.
- Dewatering is not considered.
- Soft costs for permitting, engineering, etc. are not included.

The total estimated replacement cost, including life cycle costs, for a welded steel tank is on the order of $500,000. The total estimated replacement cost including life cycle cost for a bolted steel tank is on the order of $400,000. We estimate that the total construction duration will be three months.

4.0 CONCLUSION

Our scope of services included a variety of tasks that included assessment of existing structural conditions, review of evaluations prepared by other consultants, evaluation of potential seismic vulnerabilities not addressed to date, re-evaluation of slope stability and seismic design parameters incorporating data from the Peralta Hills Fault, and evaluations of different replacement and retrofit alternatives for both the Peters Canyon Reservoir and the Vista Panorama Reservoir. The goal in carrying out these tasks is to assist EOCWD by providing useful information and recommendations that will help advance efforts to secure a safe and reliable water supply system.

4.1 Peters Canyon Reservoir

In general, our findings concur with previous findings for the Peters Canyon Reservoir. However, we have identified some additional issues that EOCWD should be aware of for the Peters Canyon Reservoir, which include the following:

- The existing wood-framed roof is constructed with CCA-treated wood. CCA is pesticide that is used as a wood preservative. Its use has come under the regulation of the EPA, which does not support its use where direct or indirect contact with potable water is possible. Decisions to retrofit the existing wood-framed roof structure should be carefully considered against the current regulations and any potential or otherwise perceived impact on water quality.
- Corrosion of the existing roof fasteners is expected to continue unabated. Deterioration of these connections is anticipated to worsen over time.
- The seismic load paths evaluated by others followed the more traditional concept, assuming the need for a diaphragm. Our evaluation of the existing load paths looked at load transfer through weak-axis bending and axial load transfer. We generally concur that provision of a diaphragm will resolve multiple vulnerabilities, both seismic and wind related.
- The nearby Peralta Hills Fault marginally increases the code-based seismic ground accelerations at the site. However, the slope stability evaluation results from the previous study by American Geotechnical remain valid.

To provide for a safe and reliable water supply, we believe that the Peters Canyon Reservoir should be seismically retrofitted or replaced to address a number of vulnerabilities and material issues identified. While a seismic retrofit is a potential path forward to help rectify the vulnerabilities, we believe the material concerns may be an over-riding consideration for EOCWD and its stakeholders. Accordingly, we also evaluated the feasibility of a reservoir replacement with a prestressed concrete tank or installation of a floating cover over the existing reservoir.

For planning purposes, we estimate that the cost to rehabilitate or replace the existing reservoir will be in the range of $5 to $8 million, depending on the alternative selected. Understanding that parallel work is underway to establish the land requirements at the site for a potential new water treatment facility, and modeling work is also underway to establish the long-term water storage needs of the District, a clear choice on the best alternative cannot yet be made. Once the land use needs at the site have been established and the storage volume needs confirmed, a further analysis to select the best rehabilitation/replacement approach can be made.

4.2 Vista Panorama Reservoir

Our findings for the Vista Panorama reservoir suggest that the reservoir requires additional non-destructive testing to identify as-built conditions. In the absence of detailed as-built information, we evaluated the tank based on visual observations and some assumptions regarding construction and materials. Based on these results, the tank should continue to be operated at a reduced height, although we conclude that the water level could be increased from the current depth of 3 ft to a maximum of 8.6 ft, which would increase the storage volume to around 100,000 gallons.

In addition, the reservoir does require some form of seismic retrofit or replacement. For planning purposes, we estimate that the cost of this work will range from $100,000 for a retrofit and up to $400,000 for replacement.

5.0 REFERENCES

American Concrete Institute (2006), ACI 350-06, “Code Requirements for Environmental Engineering Concrete Structures and Commentary.”

American Concrete Institute (2006), ACI 350.3-06, “Seismic Design of Liquid-Containing Concrete Structures and Commentary.”

American Forest & Paper Association/American Wood Council (2005), ANSI/AF&PA National Design Specification for Wood Construction

American Plywood Association (1995), “The Great Cover-Up: Engineered Wood Products Cover Reservoir.”

American Society of Civil Engineers (2010), ASCE 7-10, “Minimum Design Loads for Buildings and Other Structures.”

American Society of Civil Engineers (2013), ASCE 41-13, “Seismic Evaluation and Retrofit of Existing Buildings.”

American Water Works Association (2011), AWWA D100, “Welded Carbon Steel Tanks for Water Storage.”

American Water Works Association (2000), AWWA Manual M25, “Flexible-Membrane Covers and Linings for Potable-Water Reservoirs.”

Environmental Protection Agency (2011), “Pesticides: Regulating Pesticides: Chromated Copper Arsenate (CCA),” http://www.epa.gov/oppad001/reregistration/cca/

Reed Construction Data LLC (2014), “RSMeans Heavy Construction Cost Data, 28th Annual Edition.”

Reed Construction Data LLC (2008), "RSMeans Building Construction Cost Data, 66th Annual Edition."

-This Page Left Blank Intentionally-