

**D R A F T**

# PHASE II EAST BRANCH ENLARGEMENT FEASIBILITY REPORT

*Prepared for*  
California Department of Water Resources  
1416 Ninth Street, Room 550-1  
Sacramento, CA 95814

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URS Corporation  
1333 Broadway, Suite 800  
Oakland, CA 94612  
510-893-3600



MWH  
3321 Power Inn Road, Suite 300  
Sacramento, CA 95826

26814903.03800

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The Phase II East Branch Enlargement Feasibility Report presents the results of the URS/MWH cost analysis studies and provides a basis for further project design development. The feasibility studies focused on recommending the most economical combination of canal raise and hydraulic structure (including check structures and siphons) improvements to accommodate increasing flow in the East Branch of the California Aqueduct from the current (Phase I) 2,010 cfs to 2,876 cfs (Phase II) and to explore innovative designs to reduce cost. This report supplements DWR's East Branch Aqueduct Enlargement Study (DWR, 2004).

**Existing East Branch Aqueduct.** The East Branch Aqueduct is approximately 95 miles long, from the Alamo Powerplant/Cottonwood Chute Bypass in the north to the Mojave Siphon Powerplant in the south and includes approximately 91 miles of canal and 4 mile of siphons. The canal has a concrete-lined trapezoidal cross-section. The aqueduct system includes the Pearblossom Pumping Plant at the end of Pool 58. There are also 23 check structures, 15 siphons, several turnouts, and nearly 200 bridges, pipelines, overchutes, and culverts.

**Feasibility Design Criteria.** Prior to commencing feasibility design, general feasibility design criteria for Phase II of the East Branch Canal Enlargement were submitted in a September 2006 Design Criteria Memorandum by URS/MWH that addresses freeboard, hydraulic design, civil design, structural design, operations and deliveries, operations and maintenance, and seismic issues.

**Canal Flow Test and Hydraulic Modeling.** A flow test was conducted during May 5-10, 2006, to more accurately determine the values of hydraulic parameters for use in the HEC-RAS hydraulic model. Key to obtaining accurate measurements of the hydraulic parameters in the canal was the attainment of steady state flow in the canal.

The following four cases were evaluated using the calibrated HEC-RAS hydraulic model:

- DWR Full Enlargement Alternative 1 (Base Case)
- Base Case Plus Smooth Siphons
- Base Case Plus Added Check Bays
- Base Case Plus Added Check Bays and Smooth Siphons.

Water surface elevations for each of these cases were calculated and used to determine the raise of the canal banks necessary to increase the East Branch capacity to 2,876 cfs.

**Alternatives to Increase Canal Capacity.** The canal capacity will need to be increased, in part, by raising the canal banks. The amount of the raise required will vary depending upon the hydraulic operating scenarios ultimately selected and the height of existing canal lining and embankment. The following seven alternatives to raise the canal embankments were evaluated:

- Alternative 1: Earthfill embankment (similar to DWR's 2004 study)
- Alternative 2: Roller compacted concrete (RCC)
- Alternative 3: Spread footing wall
- Alternative 4: Concrete parapet
- Alternative 5: Earthfill embankment with retaining wall
- Alternative 6: Slipform wall

- Alternative 7: Precast panel wall

These alternatives were evaluated based on criteria that included implementability, operational flexibility, maintainability, reliability, and cost.

The earthfill embankment raise (Alternative 1) may be used where DWR already has right-of-way or in undeveloped areas where additional right-of-way can be obtained (i.e., developments do not encroach on the canal embankments). To improve seismic reliability, the earthfill embankment is also the only canal raise alternative that may be used where the canal traverses an active fault zone. Where developments encroach on the canal, Alternatives 2 through 7 were considered. For canal raise requirements greater than 8 inches, a precast panel system (Alternative 7) was found to have the lowest cost. Where the canal raise is less than 8 inches, a slip form wall or vertical curb was found to be more cost effective.

**Modifications to Canal Structures.** The enlargement of the East Branch Aqueduct will require modifications to the 11 siphons and 15 check structures upstream of Pearblossom Pumping Plant. Downstream of Pearblossom Pumping Plant, modifications to the Tejon siphon and the Antelope check structure and siphon are needed. This evaluation included adding two bays to existing two bay check structures and a single bay added to three bay check structures so that all check structures will have four bays. A single barrel will be added to the thirteen siphons.

Modifying the check structures while minimizing impacts to existing operations presents one of the greatest challenges to the East Branch Aqueduct Enlargement. In addition to adding a bay at each check location, the modification will include adding a siphon barrel at these check locations with siphons and modifying the existing radial gates and inlet and outlet transitions to accommodate the raised canal crest.

To increase canal capacity, overchutes can be handled in two ways. They can be raised to meet the desired freeboard or anchored in their existing position. However, overchutes cannot be raised without also raising the upstream flow transitions. In most cases, this would result in significant grading requirements at the upstream end of the overchute. Thus, anchoring overchutes in place was found to be more economical than raising them.

Bridges will need to be raised to a minimum clearance of 1-foot above the maximum water surface elevation to meet CALTRANS requirements.

**Cost Estimation Approach.** Costs were evaluated based on capital costs and life cycle costs. Construction and contingency costs make up a significant portion of capital costs. Costs for engineering, construction management, administration, legal, and other “soft costs” are also part of the capital cost and are generally estimated as a percentage of the construction cost. These “soft costs” are not included in this report and will need to be added based on DWR’s standard percentages. Life cycle costs are estimated by annualizing capital costs based on the estimated life of project elements and the 2007 Federal Water Resources discount rate (4.875%).

Pricing in this feasibility report was estimated in third quarter 2007 dollars. Unit costs were obtained from historical databases for similar projects. Vendor quotes were obtained for specialty items such as epoxy or polyurethane coatings for application in siphons. Right-of-way and environmental mitigation costs are outside the scope of this feasibility study and were not included in the construction cost estimates.

**Constructability Issues.** Due to the importance of maintaining the water supply in the East Branch Aqueduct, a shutdown of the facility for an extended period of time is not feasible. It is assumed that most work involved in expanding the capacity of the canal would be done at its current full flow capacity. The canal raise portion of the work is not expected to present a major impact to operations during construction. From an operations standpoint, the canal lining raise could likely be scheduled to occur anytime during the year. Similarly, although adding to the complexity, the overchutes can be anchored and bridges raised with a full canal.

The check/siphon modifications will present the majority of the scheduling challenges for the work. Modifications to checks and connecting new siphons will likely necessitate a reduction of flow. Multiple crews and shifts will likely be necessary to minimize the period of time that flow is reduced. Generally, this period would be from mid-October to mid-February.

**Evaluation of Scenarios to Increase Canal Capacity.** For scenario development, the general approach was to evaluate combinations of improvements to increase canal capacity to 2,876 cfs. Various combinations of improvements were evaluated using the calibrated HEC-RAS hydraulic model to check the effect that various improvements would have on canal water surface elevations.

Three scenarios were considered:

- Scenario 1: DWR 2004 Report Conditions, updated to reflect current costs, for comparison purposes; includes 16 check bays.
- Scenario 2: Canal Raise Alternative – includes pre-cast panel wall (in lieu of full embankment raise) in portions of the canal; includes 23 check bays.
- Scenario 3: Smooth Siphon Alternative - includes application of a smooth polyurethane or epoxy coating to the inside of all siphons to the structural improvements of Scenario 2 to reduce the height of canal raise.

The estimated construction costs (including 20 percent contingency but no “soft costs”), in third quarter 2007 dollars, and present values for the three scenarios are summarized below:

Cost	Scenario 1 – DWR 2004 Report – Updated	Scenario 2 – Canal Raise Alternative	Scenario 3 – Smooth Siphon Alternative
Construction Cost	\$372 million	\$363 million	\$347 million
Present Value	\$400 million	\$390 million	\$390 million

As shown in the above summary, the estimated construction costs of the three scenarios are similar (costs are within 7 percent of each other). Although Scenario 3 - Smooth Siphon Alternative could have a slightly lower initial cost than the other two scenarios, this scenario would require periodic reapplication of the polyurethane or epoxy coating. This product has only been in use since about 1993, so it does not have an extensive service record for this product. The estimated interval for reapplication could be between 10 and 30 years; 15 years was assumed for the life cycle analysis. This reapplication would need to be done during periods when the canal can operate at lower flow rates (mid-October to mid-February). Clearly, there are performance risks associated with this scenario that must be quantified if siphon coatings are to be further considered. At this time, there does not appear to be a clear benefit for this scenario.



The cost of Scenario 1 would be greater than shown above with the addition of right-of-way costs. Consideration of right-of-way costs may indicate that significant cost savings could be achieved by using precast panel walls, or similar walls (Scenario 2), instead of a full canal embankment raise (Scenario 1). Full canal embankment raises can only be used in areas where developments have not encroached on the East Branch canal embankments and where the canal crosses active fault zones. Further engineering and cost studies will need to be undertaken to confirm the most cost-effective canal raise system on a specific location basis.

To evaluate the cost tradeoff of using 16 check bays instead of 23 used for Scenario 2, but increasing the canal crest elevation, the cost of this variant of Scenario 2 was estimated. The total cost was found to be about \$362 million, which is nearly the same as for Scenario 2. Thus, no significant cost benefit was realized for this variant. Furthermore, for this variant, there would also be less operational flexibility than for Scenario 2 with the 23 check bays.

The overall construction duration for the three scenarios is estimated to be almost 7 years. Currently, modifications to Pearblossom Pumping Plant define the end of construction. It may be possible to shorten the completion date by starting work at Pearblossom ahead of the presently planned start date.

**Recommendations for Further Studies.** Additional studies and engineering are needed to further develop the concepts explored in this report for the Phase II East Branch Enlargement. Recommendations include:

- Conduct alternative analysis to optimize size of additional siphon barrels.
- Prepare alternative analyses on a site specific basis for construction of new check structures separated from existing check structures.
- Establish order of priorities for structures and related canal lining raises. Priority could be given to areas where capacity can be increased most cost effectively. For instance, the canal upstream of Pearblossom could be improved to provide increased capacity early in the East Branch Enlargement.
- Complete digital terrain model of the canal so that the cost of the earthfill embankment canal raise alternative can be estimated more accurately and compared to other alternatives.
- Complete right-of-way mapping for the canal and incorporate the cost of right-of-way into the overall scenario costs.
- Perform slope stability analyses of the canal embankments for the enlarged aqueduct capacity using properties of the embankment materials that are based on laboratory testing data.
- Confirm locations of earthfill borrow materials for use in canal embankments.
- Perform structural analyses of the culverts, radial gates and other features for the increased loading from the enlarged aqueduct.
- Perform hydraulic analyses to assess the performance of the transitions to the enlarged check structures.

## 1.1 BACKGROUND

The existing East Branch of the California Aqueduct (East Branch Aqueduct) is approximately 95 miles long, conveying water from the Alamo Powerplant to the Mojave Siphon Powerplant (Figure 1-1). The canal capacity at the Pearblossom Pumping Plant located near the midpoint of the reach is approximately 2,010 cfs. The East Branch Enlargement Project (Phase II) would increase capacity at this location to 2,876 cfs and to 3,149 cfs at Alamo Powerplant.<sup>1</sup> Current water demand projections indicate that design work for the Phase II Enlargement should begin by spring 2008, with the EIR process beginning in 2006.

The team of URS/MWH was retained by the California Department of Water Resources (DWR) to perform a feasibility study to recommend the best combination of canal raise and hydraulic structure improvements (including check structures and siphons) to accommodate the increased flow and to explore innovative designs to reduce the cost to implement such improvements.

In December 2004, a reconnaissance site visit was conducted to gather data and information on the proposed Phase II canal enlargement; to observe existing conditions of the canal and associated structures, including Pearblossom Pumping Plant and Alamo Power Plant; and to review issues associated with canal enlargement, specifically to gain an understanding of operations and of particular system bottlenecks. A summary of the findings of the field reconnaissance is included in URS/MWH (2004).

On February 14, 2005, a workshop was held between DWR, the State Water Contractors Association, Metropolitan Water District of Southern California (MWD), URS/MWH, and Bookman-Edmonston at DWR's office in Sacramento. The purpose of the workshop was to:

- Review the preliminary findings and conclusions from the December 2004 field trip.
- Confirm design criteria and operational and demand requirements.
- Develop a detailed work plan for a feasibility study to increase aqueduct capacity from 2,010 cfs (Phase I) to 2,876 cfs (Phase II).
- Explore innovative designs to reduce cost.

A summary of the discussions and actions resulting from the workshop is included in URS/MWH (2005).

Between May 5 and 10, 2006, a canal flow test was conducted to provide steady-state data that could be used to calibrate a hydraulic model of the canal by determining roughness coefficients for the canal, as well as head loss coefficients for the checks and siphons. The flow test plan is included in URS/MWH (2006a) and the flow test results are included in URS/MWH (2006b). The conclusions of the flow test are summarized below:

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<sup>1</sup> The needed capacity of the East Branch Aqueduct decreases from Alamo Powerplant to Pearblossom Pumping Plant due to turnouts along this reach. Between Pearblossom and the Mojave Powerplant, the needed capacity is essentially the same. Unless noted otherwise, canal capacity referred to within this text is the canal capacity at the Pearblossom Pumping Plant.

- The data indicate that the flow in the East Branch Aqueduct reached steady state conditions from Check 43 to Pearblossom Pumping Plant, with 2,010 cfs at Pearblossom Pumping Plant.
- The flow test data are adequate for use in calibrating the hydraulic model of the East Branch Aqueduct.

After completing the flow test, the HEC-RAS hydraulic model originally developed by U.C. Davis was calibrated to the results of the flow test. The hydraulic model calibration results were submitted to DWR on June 30, 2006 (URS/MWH, 2006c). Utilizing the calibrated model, URS/MWH evaluated alternatives to increase capacity of the East Branch Aqueduct to 2,876 cfs. This evaluation focused on the most economical combination of canal raise and hydraulic structure improvements to accommodate the increased flow.

URS/MWH issued the Design Criteria Memorandum (DCM) on September 29, 2006 (URS/MWH, 2006d). The purpose of the DCM is to present design criteria sufficient for feasibility design to increase the capacity of the East Branch Aqueduct to 2,876 cfs.

A meeting with DWR was held on April 23, 2007, to review the hydraulic model results and alternative canal raise concepts.

## 1.2 PURPOSE AND SCOPE

The purpose of this feasibility report is to document the results of the URS/MWH studies and provide a basis for further project design development. The feasibility studies focused on recommending the most economical combination of canal raise and hydraulic structure (including check structures and siphons) improvements to accommodate the increased flow from the current (Phase I) 2,010 cfs to 2,876 cfs (Phase II) and to explore innovative designs to reduce cost. This report supplements DWR's East Branch Aqueduct Enlargement Study (DWR, 2004).

The scope of work includes the following tasks:

- Confirm/update design criteria.
- Develop alternative components that were assembled into canal system scenarios.
- Perform hydraulic modeling, including calibrating the model to steady state conditions obtained from a canal flow test.
- Evaluate and shortlist alternatives to be included in scenario development.
- Develop feasibility designs.
- Estimate construction costs of alternative components and scenarios.
- Prepare this feasibility report.

This feasibility report was prepared in accordance with Agreement No. 4600003518 between DWR and URS, Task Order No. EBE-URS 3, dated August 10, 2005.

### 1.3 ORGANIZATION OF REPORT

After this introductory section, this report is organized into the following sections:

- Section 2 describes the existing project facilities.
- Section 3 presents the feasibility design criteria.
- Section 4 presents the results of the hydraulic modeling, including the results of a full-scale canal flow test.
- Section 5 discusses the development of alternative component concepts.
- Section 6 discusses the alternative components evaluation.
- Section 7 discusses the feasibility designs of the various scenarios.
- Section 8 presents the basis of the construction cost estimates.
- Section 9 describes construction considerations and schedule.
- Section 10 presents the conclusions and recommendations of this feasibility study.
- Section 11 presents the limitations of this study.
- Section 12 lists the references used to prepare this report.

The following supporting appendices are included at the end of the feasibility report:

- Appendix A presents aerial images of the existing East Branch Aqueduct.
- Appendix B includes spreadsheets of the dimensions of the existing canal facilities.
- Appendix C presents the Flow Test Plan (Appendix C1) and Flow Test Results (Appendix C2).
- Appendix D presents the hydraulic model results.
- Appendix E presents construction cost estimate backup data.

The East Branch Aqueduct is approximately 95 miles long, from the Alamo Powerplant/Cottonwood Chute Bypass in the north to the Mojave Siphon Powerplant in the south. Appendix A presents aerial images (from *Google Earth*<sup>TM</sup>) of the East Branch Aqueduct. The existing East Branch Aqueduct facilities are described in the East Branch Aqueduct Enlargement – Phase II (DWR, 2004), and a full description is not repeated here. Rather, a summary of existing canal facilities is provided in this section.

The canal has a concrete-lined trapezoidal cross-section. The depth of water within the aqueduct at the existing design flow is between 16 and 18 feet. From Pool 43 to Pool 58 (distance of about 51 miles), the concrete lining thickness is 4 inches, and from Pool 59 to Pool 66 (a distance of about 44 miles), the lining is 3 inches thick. DWR found the depth of sediment in the canal to be approximately 6 inches (DWR, 2004). No sediment was found in the siphons (URS, 2006).

The canal system includes the Pearblossom Pumping Plant at the downstream end of Pool 58. There are also check structures, siphons, and canal crossing structures as summarized below:

- **Check Structures:** The canal is divided into a number of pools that can be isolated by check structures. Inlets to the checks are equipped with radial gates, except at Pool 59 (uncontrolled). The check structures include:
  - 12 two-bay checks
  - 11 three-bay checks
- **Siphons:** Siphons are generally used to convey aqueduct flow in closed channel across natural drainage features. The siphons on the East Branch Aqueduct consist of 3 bays and 3 siphon barrels, except Big Rock Siphon (Pool 59) that has 3 bays and 2 siphon barrels, and the siphon at Pool 64 that has two bays and two box siphons. The siphons are summarized below:
  - 11 circular siphons
  - 4 box siphons
- **Canal Crossing Structures:** There are nearly 200 bridges, pipelines, overchutes, and culverts. The bridges range from small access bridges built by DWR to large Interstate highway bridges owned and maintained by the California Department of Transportation. The pipelines carry water, oil, gas, and cables. Overchutes are open-channel concrete flumes that pass runoff across the canal. Culverts are reinforced concrete structures that pass runoff beneath the canal.
- **Pearblossom Pumping Plant:** A total of nine pumps lift water 540 feet, which then flows by gravity to the Mojave Siphon Powerplant and into Silverwood Lake. The installed capacity at Pearblossom Pumping Plant is 2,525 cfs (includes one spare 375 cfs unit). There are three 375 cfs units, four 280 cfs units, and two 140 cfs units. The flow from these units is delivered to the downstream canal through three discharge lines.
- **Turnouts:** Water deliveries to the SWP contractors' turnouts from the canal are located uphill or downhill from the canal, and are either gravity or pumped turnouts.

Appendix B contains pertinent data on the canal facilities from the 2004 DWR study.

### 3.1 GENERAL

The purpose of this section is to present design criteria sufficient for feasibility design to increase the capacity of the East Branch Aqueduct to 2,876 cfs. The design criteria is from the Design Criteria Memorandum (DCM; URS, 2006d) that includes the results of the February 14, 2005, workshop.

### 3.2 CANAL CAPACITY

The East Branch Enlargement Project (Phase II) would increase capacity to 2,876 cfs at Pearblossom Pumping Plant. The canal would generally not be operated at steady state at this capacity, which could occur with the check gates out of the water. The canal will generally be operated with gates partially in the water using the controlled volume approach. The canal system pumps at Pearblossom will be operated at 2,876 cfs during non-peak power cost hours and at a lesser rate during peak power cost hours to take advantage of lower energy costs, resulting in an average discharge somewhat less than 2,876 cfs.

### 3.3 CONSTRUCTION PERIOD/WATER DELIVERIES

The construction period for work in the canal can be from mid-October through mid-February, a period of 4 months. Flow can be decreased during this period, but the canal cannot be taken out of service. The amount of flow reduction depends on water demand and other conditions. However, for the purpose of the feasibility study, a reduced uniform flow of 1,000 cfs will be assumed. Work in the canal may be done in alternate checks to maintain partial capacity.

### 3.4 FEASIBILITY DESIGN CRITERIA

Table 3-1 summarizes the general feasibility design criteria for Phase II of the East Branch Enlargement and includes the following categories:

- Freeboard
- Hydraulic Design
- Civil Design
- Structural Design
- Operations and Deliveries
- Operations and Maintenance
- Seismic Issues.

**Table 3-1 East Branch Enlargement (Phase II)  
General Feasibility Design Criteria**

Category	Criteria	Commentary
Freeboard	Canal: 2 feet lined plus 2 feet unlined	<ul style="list-style-type: none"> <li>• Some variation may be allowed.</li> <li>• Except at San Andreas Fault crossings, parapet walls may be used to replace unlined portion of freeboard:                             <ul style="list-style-type: none"> <li>- Include removable sections for maintenance access.</li> <li>- Water-tight seals are required at joints</li> </ul> </li> <li>• Canal sections designed for overtopping to spill basins may be used in lieu of meeting freeboard requirement [Potential sites are immediately upstream of Pearblossom (Pool 58) and Mojave Penstock (Pool 66)]</li> </ul>
	Elevated Canal Structures: Minimum 2-foot freeboard was criteria for original design, but a number of structures do not currently meet this criteria. Consider raising if less than 1-foot freeboard remains with the new design flow	<ul style="list-style-type: none"> <li>• If structure is raised, include seismic upgrade.</li> </ul>
	Bridges: <ul style="list-style-type: none"> <li>• Caltrans jurisdictional: 1-foot minimum; to be confirmed.</li> <li>• Non-Caltrans jurisdictional: to be determined on case-by-case basis.</li> </ul>	
	Overchutes: <ul style="list-style-type: none"> <li>• Negative freeboard may be acceptable; headloss to be checked.</li> <li>• Anchor if less than 1-foot freeboard is provided.</li> </ul>	<ul style="list-style-type: none"> <li>• Seal inverts to prevent leakage if less than 1-foot freeboard is provided.</li> </ul>
	Pipelines: <ul style="list-style-type: none"> <li>• Pipelines with less than 1-foot freeboard will be raised to provide minimum 2-foot freeboard.</li> </ul>	
	Check/siphon headwalls: <ul style="list-style-type: none"> <li>• Same freeboard requirements as for canal.</li> </ul>	

**Table 3-1 East Branch Enlargement (Phase II)  
General Feasibility Design Criteria**

Category	Criteria	Commentary
Hydraulic Design	Flow velocities in: Canal: Average at maximum discharge 3.25 fps (existing) and 3.5 fps (future enlargement).	
	Siphons: Current maximum is 6.6 fps at Antelope Siphon. With no modification, maximum velocity would be 9.5 fps at this siphon.	Decision to add additional barrel will be on basis of economics.
	Canal capacity (gates fully open): <ul style="list-style-type: none"> <li>• 3,149 cfs immediately downstream of Alamo Powerplant</li> <li>• 2,876 cfs downstream from Pearblossom Pumping Plant.</li> </ul>	Canal will normally be operated with gates partially in water, peaking at 2,876 cfs with lesser flows during portions of the day.
	Manning “n” values in canal = 0.017 for most of the concrete lined canal, with limited segments ranging up to 0.019.	<ul style="list-style-type: none"> <li>• From model as calibrated by steady state flow test (URS/MWH, 2006c).</li> <li>• Value includes the effect of sediment; depth of sediment in the canal is approximately 6 inches.</li> <li>• No sediment was found in siphons (URS, 2006).</li> </ul>
	Other losses (entrance, exit, across checks) – from existing data.	Base on steady-state flow test model calibration.
	Drawdown rate: 1-foot per day (normal operation) to 2 feet per day (repair).	Drawdown rates used by SFD are 1.5 feet/day upstream of Pearblossom and 2 feet/day downstream.
Civil Design	Canal slopes – interior and exterior slopes will be same as existing slopes. Stability factor of safety = 1.5.	Slope stability analyses to be performed to confirm adequate slope stability. If available from DWR, soil properties for the existing canal will be used. If not, typical soil properties for compacted sandy soils will be used.
	Lining thickness and reinforcement: <ul style="list-style-type: none"> <li>• 4 inches thick (Pool 43-58)</li> <li>• 3 inches thick, (Pool 59-66)</li> </ul>	<ul style="list-style-type: none"> <li>• Thickness to be same as current condition. Linings will be unreinforced.</li> <li>• The purpose of the lining is to prevent erosion, reduce seepage, and to reduce roughness coefficient.</li> </ul>



**Table 3-1 East Branch Enlargement (Phase II)  
General Feasibility Design Criteria**

Category	Criteria	Commentary
	Culverts	No reanalysis of original hydraulics and hydrology design. <ul style="list-style-type: none"> <li>• Extend culverts</li> <li>• Rebuild headwalls</li> <li>• Repair cracks/strengthen pipe for higher external loading.</li> </ul>
	Turnouts	Extend structure, raise gate operators
	Primary O&M road: Normal top width to be 20 feet, same as existing condition. Paved width to be 16 feet.	<ul style="list-style-type: none"> <li>• Asphalt concrete over aggregate base.</li> </ul>
	Secondary O&M road: Minimum width to be 12 feet.	<ul style="list-style-type: none"> <li>• Aggregate base over native materials.</li> </ul>
Structural Design	Concrete strength: 4,000 psi	Original strength was 3000 psi. New structures will use 4,000 psi concrete.
	Load combinations: <ol style="list-style-type: none"> <li>1. Dead + hydrostatic + uplift + backfill + live load</li> <li>2. Dead + hydrostatic + uplift + backfill + seismic</li> <li>3. Dead + backfill + construction live load</li> </ol>	Due to the limited increase in canal water surface elevation (3 ft. max.), limited structural calculations are anticipated.
	Bridges: Maintain same structural capacity as existing bridge.	
	Radial gates: Extend and strengthen where feasible, replace as necessary to accommodate higher levels within the canal.	
Operations and Deliveries	Controlled volume – gates in water	Provides flexibility to the system and makes operations more manageable.
	Controlled flow – gates out	Generally will not be used.
	Load rejection	SFD’s current practice to handle load rejection at the Mojave Siphon Power Plant is to close the gates at Check 65 and “feather” the gates upstream of this check to distribute the remaining incoming flow among the various pools and thereby maintain minimum freeboard throughout all the pools in the system.

**Table 3-1 East Branch Enlargement (Phase II)  
General Feasibility Design Criteria**

<b>Category</b>	<b>Criteria</b>	<b>Commentary</b>
Operations and Maintenance	Sediment removal: Will not be done for normal O&M.	Dredging was unsuccessful in past. May remove sediment at Pool 58 if it is determined that sediment adversely affects the backwater profile.
	Mechanical components (gates)	To accommodate increased water depth for enlarged canal.
Seismic Issues	San Andreas fault crossings – rupture potential	Seismic triggers will be used to close gates to isolate pools.
	Liquefaction potential	Identified as risk. Analysis to determine reaches with highest risk and measures to remediate are beyond project scope.

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## 4.1 FLOW TEST RESULTS

The purpose of the flow test was to calibrate the HEC-RAS flow model of the East Branch Aqueduct and to modify the existing DWR model. Modeling of the East Branch requires accurate parameters for use in the hydraulic model. The most important parameters for model calibration are canal and siphon roughness factors and contraction and expansion coefficients. Prior to the flow test, conducted from 22:00 May 5, 2006 to 08:00 May 10, 2006, there was concern that the parameters being used in the hydraulic model of the East Branch were not simulating actual conditions in the canal. The purpose of the flow test was to more accurately determine the values for these parameters. The flow test plan and test results are presented in Appendix C1 and Appendix C2, respectively.

An important factor for allowing the accurate measurement of the flow parameters in the canal was the attainment of steady state flow in the canal. At steady state, it can be assumed that water surface elevations at any point in the canal are a result of the hydraulic characteristics of the canal, therefore making it possible to confirm the hydraulic parameters for use in the canal model. This section describes the results of the East Branch flow test.

Prior to the beginning of the flow test, Southern Field Division (SFD) operators filled the canal reach from Pool 43 to Pool 58 (study reach) with 5,400 acre-feet of storage. Release of a constant flow from the Tehachapi second afterbay began at 22:00 on May 5, 2006. The constant flow was measured at Alamo Power Plant and Cottonwood Bypass as 2,140 cfs, to provide 2,010 cfs at Pearblossom Pumping Plant after subtracting turnout demands upstream from Pearblossom. Beginning at 22:00 on May 5, and continuing until 08:00 on May 7, the gates in the study reach and also downstream from Pearblossom Pumping Plant were operated to prevent excessive drawdown or encroachment into canal freeboard. At 08:00, on May 7, all gates in the study reach were fully open (i.e., 12 feet open).

The gates in the study reach were maintained in the fully open condition through to the end of the flow test at 08:00 on May 10, 2006. Water level data were collected at 10 minute intervals from pressure transducers at the upstream and downstream end of each of the checks and siphons in the study reach. These data are plotted for the period of 0:00 on May 8, to 08:00 on May 10 in Appendix C2.

### 4.1.1 Flow Test Steady State Operation Rules and Criteria

The steady state operation rules and criteria for the flow test were:

1. The focus of the flow test was in pools upstream from Pearblossom Pumping Plant, but data was collected to Check 66.
2. The flow at the upper end of the canal was set to match a 2,010 cfs pumping rate at Pearblossom Pumping Plant, plus the turnout flows upstream from Pearblossom during the test. As far as was possible within the operating constraints of agencies taking water from the canal, constant turnout flows were maintained during the test period.
3. Operations did not schedule any flow changes during the period of testing. The test was run until steady flow was achieved throughout the canal upstream from the Pearblossom Pumping Plant (54.8 miles).

4. Gates were not operated in or downstream from the targeted pools during the measurement period. All gates were set to the full open position. All gate openings were recorded as part of the flow test.
5. Measurements were taken over a period of 24 hours after the canal achieved steady state conditions in all pools with particular attention in the targeted pools (50, 51, 58).
  - Steady state conditions were defined as continuous operation with the water surface maintained within a 0.2-foot band at both ends of a pool.
  - Water level measurements were taken in regular canal cross section upstream and downstream from transitions at siphons and checks.

#### 4.1.2 Determination of Steady State Flow

The operations of the aqueduct during the flow test followed the operation rules described above. Figure 4-1 shows the periods for which the sensor data over a 24-hour period varied within a band of 0.2 feet. As shown in Figure 4-1, a steady state was achieved for the 12-hour period May 9, 8:00 to May 9, 20:00. The average readings for each of the gages during that period were used to describe the steady state flow conditions in the Aqueduct upstream from Pearblossom Pumping Plant, and in particular, Pools 49, 50, 51, and 58.

#### 4.1.3 Conclusions of Flow Test

The data indicate that the flow in the East Branch of the California Aqueduct reached steady state conditions of 2,010 cfs from Check 43 to Pearblossom Pumping Plant during the May 8 to May 10, 2006 flow test. Data collected during the 12-hour period from 8:00 to 20:00 on May 9, 2006 are adequate for use in calibrating the hydraulic model of the East Branch Aqueduct.

### 4.2 HYDRAULIC MODELING

This section describes the hydraulic modeling methodology used to model the East Branch. The use of the data from the flow test to calibrate the model will also be described.

The HEC-RAS (USACE, 2006) model of East Branch was originally developed by U.C. Davis and additional information on bridges and overchutes was inserted by Bookman Edmonston. The HEC-RAS hydraulic model includes the East Branch canal from downstream of the Alamo Power Plant to Check 66, a distance of approximately 95 miles. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The HEC-RAS system contains a one-dimensional analysis component for steady flow water surface profile computations. This is the component used to model East Branch.

#### 4.2.1 Model Calibration

After completion of the flow test, sufficient data were available to calibrate the HEC-RAS steady state hydraulic model of the East Branch. Calibration began by examining data from Pools 49 to 51 and 58. Using the water surface elevations collected at the upstream and downstream ends of each of the pools, it was possible to determine the friction headloss in the pool. The results of this calibration effort indicated that the Manning's "n" roughness factor was around 0.017. Open

channel reference books (Chow, 1959) indicate that 0.017 should be used for lined canals with sediment on the bottom of the canal. Bathymetry surveys, to obtain information on sediment accumulation in the canal, were performed in early 2006 in Pools 50 and 51. The data generally indicates that the depth of sediment is on the order of 6 inches in the pools surveyed. This condition has also been noted when various pools have been drained for maintenance activities.

Calibration of the model started in Pool 58, upstream from Pearblossom Pumping Plant, and worked upstream by pool. The calibration process used in each pool was as follows:

1. Determine headloss for a pool as the difference between the flow test water surface elevation from the upstream and downstream ends of the pool.
2. Set Manning's  $n$  roughness coefficient for the canal to 0.017.
3. Run model and compare pool headloss in the model with flow test headloss.
4. If headloss is not within 0.2 feet of flow test data, modify roughness coefficient and rerun model. Continue process as needed until 0.2 foot criterion is achieved.
5. Conduct a similar process to adjust the headloss through the siphon or siphon and check at the upstream end of the pool.
6. The processes were repeated for each pool upstream from Pearblossom Pumping Plant.

Calibration results are given in Table 4-1. The table includes comparisons with observed water surface elevations from the flow test. With the exception of the upstream boundary (where the difference is 0.3 feet), the difference between modeled and observed water surface elevations is less than 0.2 feet, and mostly less than 0.1 feet.

#### 4.2.2 Boundary Conditions

Boundary conditions for the hydraulic model were specified by DWR. Since the East Branch is a canal operating at subcritical velocity, the downstream boundary conditions, at Check 66 and at Pearblossom Pumping Plant, are most important. The boundary condition water surface elevation at Check 66 was set at canal normal depth with a slope of 0.00008. The boundary condition water surface elevation at Pearblossom Pumping Plant was set at 2941.96 for the enlargement flow rate of 2,876 cfs at the pumping plant.

Table 4-1 Model Calibration Results

Canal Mile & Description	W.S. Elevation (ft)	Observed W.S. Elevation	Difference	Manning's n
<b>Tehachapi Afterbay</b>				
305.75 Pool 43 Start Canal	2962.80	2963.10	-0.30	0.017
308.09 Pool 43 End Canal	2962.10			0.017
308.12 Pool 43 Start Box #1	2962.01			0.014
308.17 Pool 43 End Box #1	2961.95			0.014
308.18 Pool 43 Start Canal	2962.01			0.017
309.68 Pool 43 End Canal	2961.56	2961.59	-0.03	0.017
309.69 Pool 43 Transition	2961.21			0.015
309.70 Check 43				
309.72 Pool 44 Transition	2961.11			0.015
309.73 Pool 44 Start Canal	2961.36	2961.44	-0.08	0.017
309.93 Pool 44 End Canal	2961.30			0.017
309.94 Pool 44 Start Box #2	2961.21			0.014
309.96 Pool 44 End Box #2	2961.18			0.014
309.97 Pool 44 Start Canal	2961.25			0.017
311.71 Pool 44 End Canal	2960.74			0.017
311.72 Pool 44 Start Box #3	2960.65			0.014
311.74 Pool 44 End Box #3	2960.62			0.014
311.75 Pool 44 Start Canal	2960.68			0.017
314.79 Pool 44 End Canal	2959.81	2959.81	0.00	0.017
314.80 Pool 44 Transition	2959.48			0.015
314.81 Check 44				
314.83 Pool 45 Transition	2959.38			0.015
314.84 Pool 45 Start Canal	2959.61	2959.54	0.07	0.017
319.72 Pool 45 End Canal	2958.19	2958.12	0.07	0.017
319.73 Pool 45 Transition	2957.86			0.015
319.74 Check 45				
319.75 Pool 46 Transition	2957.76			0.015
319.76 Pool 46 Start Canal	2957.96	2957.92	0.04	0.017
323.82 Pool 46 End Canal	2956.78	2956.75	0.03	0.017
323.83 Pool 46 Transition	2956.58			0.015
323.84 Check 46				
323.85 Pool 47 Box Trans	2956.53			0.015
323.86 Pool 47 Start Myrick Siphon	2956.28			0.014
324.03 Pool 47 End Myrick Siphon	2955.76			0.014
324.04 Pool 47 Box Trans	2955.85			0.015
324.05 Pool 47 Transition	2955.84			0.015
324.06 Pool 47 Start Canal	2955.91	2955.94	-0.03	0.017
326.75 Pool 47 End Canal	2955.14	2955.12	0.02	0.017
326.76 Pool 47 Transition	2954.96			0.015
326.77 Check 47				
326.78 Pool 48 Box Transition	2954.91			0.015
326.79 Pool 48 Start Willow Siphon	2954.65			0.014

Table 4-1 Model Calibration Results

Canal Mile & Description	W.S. Elevation (ft)	Observed W.S. Elevation	Difference	Manning's n
326.92 Pool 48 End Willow Siphon	2954.20			0.014
326.93 Pool 48 Box Transition	2954.28			0.015
326.94 Pool 48 Transition	2954.28			0.015
326.95 Pool 48 Start Canal	2954.34	2954.36	-0.02	0.017
330.80 Pool 48 End Canal	2953.30	2953.32	-0.02	0.017
330.81 Pool 48 Transition	2953.12			0.015
330.82 Check 48				
330.83 Pool 49 Box Transition	2953.07			0.015
330.84 Pool 49 Start Johnson Siphon	2952.80			0.014
330.94 Pool 49 End Johnson Siphon	2952.51			0.014
330.95 Pool 49 Box Transition	2952.60			0.015
330.96 Pool 49 Transition	2952.59			0.015
330.97 Pool 49 Start Canal	2952.65	2952.75	-0.10	0.018
335.91 Pool 49 End Canal	2951.15	2951.16	-0.01	0.018
335.92 Pool 49 Transition	2950.64			0.015
335.93 Check 49				
335.94 Pool 50 Transition	2950.53			0.015
335.95 Pool 50 Start Canal	2950.90	2950.95	-0.05	0.019
341.49 Pool 50 End Canal	2948.93	2948.90	0.03	0.019
341.50 Pool 50 Transition	2948.79			0.015
341.51 Check 50				
341.52 Pool 51 Box Transition	2948.74			0.015
341.53 Pool 51 Start Ritter Siphon	2948.55			0.014
341.68 Pool 51 End Ritter Siphon	2948.08			0.014
341.69 Pool 51 Box Transition	2948.19			0.015
341.70 Pool 51 Transition	2948.18			0.015
341.71 Pool 51 Start Canal	2948.27	2948.44	-0.17	0.019
342.05 Pool 51 End Canal	2948.15	2947.96	0.19	0.019
342.06 Pool 51 Transition	2947.98			0.016
342.07 Check 51				
342.08 Pool 52 Box Transition	2947.92			0.016
342.09 Pool 52 Start Leona Siphon	2947.73			0.014
342.41 Pool 52 End Leona Siphon	2946.88			0.014
342.42 Pool 52 Box Transition	2946.95			0.014
342.43 Pool 52 Transition	2946.95			0.014
342.44 Pool 52 Start Canal	2947.09	2947.00	0.09	0.016
343.72 Pool 52 End Canal	2946.82	2946.77	0.05	0.016
343.73 Pool 52 Transition	2946.23			0.014
343.74 Check 52				
343.75 Pool 53 Transition	2946.16			0.014
343.77 Pool 53 Start Canal	2946.58	2946.57	0.01	0.015
348.15 Pool 53 End Canal	2945.64	2945.67	-0.03	0.017
348.16 Pool 53 Transition	2945.41			0.015

Table 4-1 Model Calibration Results

Canal Mile & Description	W.S. Elevation (ft)	Observed W.S. Elevation	Difference	Manning's n
348.17 Check 53				
348.18 Pool 54 Box Transition	2945.37			0.015
348.19 Pool 54 Start Soledad	2945.19			0.014
348.45 Pool 54 End Soledad	2944.51			0.014
348.46 Pool 54 Box Transition	2944.58			0.015
348.47 Pool 54 Transition	2944.58			0.015
348.48 Pool 54 Start Canal	2944.71	2944.76	-0.05	0.017
350.23 Pool 54 End Canal	2944.38	2944.36	0.02	0.015
350.24 Pool 54 Transition	2943.88			0.015
350.25 Check 54				
350.26 Pool 55 Transition	2943.74			0.015
350.27 Pool 55 Start Canal	2944.06	2944.03	0.03	0.017
352.68 Pool 55 End Canal	2943.63	2943.63	0.00	0.017
352.69 Pool 55 Transition	2943.40			0.015
352.70 Check 55				
352.71 Pool 56 Box Transition	2943.35			0.015
352.72 Pool 56 St Cheseboro	2943.12			0.015
352.87 Pool 56 En Cheseboro	2942.67			0.015
352.88 Pool 56 Box Transition	2942.77			0.015
352.89 Pool 56 Transition	2942.79			0.015
352.90 Pool 56 Start Canal	2942.83	2942.87	-0.04	0.017
354.74 Pool 56 End Canal	2942.49	2942.47	0.02	0.013
354.75 Pool 56 Transition	2942.36			0.013
354.76 Check 56				
354.77 Pool 57 Box Transition	2942.28			0.013
354.78 Pool 57 St Littlerock	2942.06			0.013
354.92 Pool 57 En Littlerock	2941.72			0.013
354.93 Pool 57 Box Transition	2941.86			0.013
354.94 Pool 57 Transition	2941.86			0.013
354.95 Pool 57 Start Canal	2941.94	2941.97	-0.03	0.017
356.91 Pool 57 End Canal	2941.57	2941.61	-0.04	0.015
356.92 Pool 57 Transition	2941.20			0.015
356.93 Check 57				
356.94 Pool 58 Transition	2940.99			0.015
356.95 Pool 58 Start Canal	2941.22	2941.27	-0.05	0.017
360.54 Pool 58 End Canal	2940.63	2940.63	0.00	0.017
<b>Pearblossom Pumping Plant</b>				



### 4.2.3 Canal Parameters

The majority of the canal pools have a Manning's n value of 0.017. Pools 49 and 50 have significant sinuosity and have calibrated n value of 0.018 and 0.019, respectively. Pool 51 has a calibrated n value of 0.019, likely because it is a very short pool. The remaining pools have a Manning's n of 0.017.

### 4.2.4 Check Parameters

Check structures have a calibrated transition Manning's n value of 0.015. Contraction and expansion coefficients for the entrance and exit transitions are 0.1 and 0.2, respectively. In order to model the gates as they would be operated, each check gate in the system was set closed 2 feet from the fully-open position.

### 4.2.5 Siphon Parameters

Siphons have calibrated internal Manning's n values of 0.013. Calibrated contraction (entrance) and expansion (exit) coefficients are 0.03 and 0.05, respectively.

One method of lowering water surface elevation is to increase the flow through the siphons. This can be accomplished by applying a polyurethane or epoxy coating to the inside of the siphon. These coatings are smoother than stainless steel when applied to an even surface. Irregularities in the pipe greater than the thickness of the coating will result in frictional losses.

DWR implemented a test case on a 2-mile stretch of the South Bay Aqueduct in 1997. Polyurethane was sprayed on the interior of the 72-inch pipe. The majority of the pipe is concrete with shorter sections of steel. Scaling occurred at the mortar lining in the steel sections of the pipe. The steel sections were then troweled with epoxy. Tests performed before and immediately after completion showed that Manning's n dropped from 0.015 to 0.010.

Additional tests are pending to determine the long term effects of this alternative. One concern is that clays in the water will coat the interior of the pipe over time and decrease the flow. For the East Branch siphons, sand may scratch the coating resulting in increased friction.

The life cycle of these coatings is estimated to be 10 to 30 years. Little practical data was found to assess how long these coatings would last. Application of the coatings would require a shut down of a siphon barrel possibly for as long as one to two months. An inspection program will likely need to be implemented 5 to 10 years after the initial application.

Epoxy coated siphons (smooth siphons) were assumed to have Manning's n values of 0.011 based on DWR's experience with the South Bay Aqueduct, manufacturer's literature and on information from standard hydraulic reference books (Chow, 1959). The life of the coating was assumed to be 10 years.

### 4.2.6 Bridge, Overchute, and Pipeline Crossing Parameters

All structures crossing over a waterway are handled as bridges in HEC-RAS. The East Branch Aqueduct has roadway and railroad bridges, drainage overchutes, and pipeline crossings that are modeled in HEC-RAS. The major hydraulic distinction between a bridge and an overchute on

the East Branch is that the normal water surface elevation in the canal will be allowed to contact the overchutes, but all bridges will have at least 1-foot of freeboard between the normal water surface and the soffit of the bridge. In some cases the freeboard requirements for the bridge will require that the bridge be raised (see Section 5.5).

Overcrossing parameters that were entered into the hydraulic model include the following:

- Overcrossing modeling approach
- Piers, including number, location, shape, and width
- Elevation of the top of structure; i.e., diameter for pipelines, top of guardrail for roadway, and top of overchute
- Soffit elevation.

Establishing an overcrossing modeling approach consists of defining which methods the program will use for low flow computations and which for high flow (flow at or above the maximum low chord) computations. For low flow computations, the model was set to calculate energy loss using the energy (standard step), momentum, and Yarnell equations (Yarnell, 1934). The highest resulting energy loss is used in the model to set the low flow energy losses.

There are two options for calculation of energy losses from flows that come in contact with the underside of an overcrossing structure, energy (standard step) and pressure and weir. Each model simulation was run twice, once using the energy equation and once using the pressure and weir method. In general the pressure and weir method results in greater energy loss through the overcrossings for flows contacting the overcrossing soffit. For isolated overcrossings the effect of the greater energy loss is not significant, but some pools have many overcrossings close together and this can result in locally increased water surface elevations over the results from the runs using the energy equation. The water surface elevations shown in this section result from the use of the pressure and weir equations to calculate losses through the overcrossings when the water surface is in contact with the overcrossing soffit.

The energy loss from a pier is highly dependent on the shape of the pier. The energy loss (drag) coefficients used are from the HEC-RAS manual and are based on survey information describing the number, location, shape, and width of overcrossing piers.

The elevation of the top of the structure is likely not a factor in the analysis for the East Branch Aqueduct because it is highly unlikely that any of the overcrossings will be overtopped. However, the elevation of the soffit or underside of the overcrossing is important, particularly for some of the overchutes that will be in contact with the water during normal operations.

### 4.3 CASES EVALUATED

Four cases were evaluated using the model described above:

- DWR Full Enlargement Alternative 1 (Base Case)
- Base Case Plus Smooth Siphons
- Base Case Plus Added Check Bays
- Base Case Plus Added Check Bays and Smooth Siphons.

### 4.3.1 DWR Full Enlargement Alternative 1 (Base Case)

Full enlargement encompasses all structural modifications to the East Branch that are required to carry 2,876 cfs at Pearblossom. The structural modifications include:

- Canal lining raise
- Added siphon barrels
- Added check bays

For the canal lining raise, it was assumed that the new canal lining would have the same Manning's n value as the existing canal lining. Added siphons and checks were assumed to be the same size and overall construction as the existing siphons and checks.

### 4.3.2 Base Case Plus Smooth Siphons

This case was the same as the full enlargement case, but in addition all siphons and box culverts would have a smooth epoxy lining installed to reduce the Manning's n value from 0.013 to 0.011.

### 4.3.3 Base Case Plus Added Check Bays

For operation and maintenance purposes, it may be useful to have four check bays at each of the check structures. This case adds check bays where required such that each check structure has four check bays. While the additional check bays result in some lowering of the canal water surface (Figure 4-2), the principal reason for adding the check bays is to provide the same level of operational and maintenance flexibility at each of the checks.

### 4.3.4 Base Case Plus Added Check Bays and Smooth Siphons

This case is the same as the previous case but includes the installation of the epoxy lining in all siphons.

## 4.4 RESULTS

Figures 4-2 and 4-3 indicate the water surface elevation compared to the existing maximum water surface elevation, for each of the 4 cases described above for pools upstream and downstream of Pearblossom, respectively. The differences indicated on the figure were used to determine the required lining and/or embankment raise for each of the scenarios and the corresponding construction costs (see Section 8).

Tables 4-2 and 4-3 provide the results for the siphons and bridges, respectively. Tables containing results for all structures and pools, as well as complete model input files, are in Appendix D.

Table 4-2 shows that using a lining to reduce the friction in the siphons would provide significant reductions (around 4 feet) in the friction losses through the siphons. As shown in Figure 4-2 and in the tables in Appendix D, however, the resulting lowering of the water surface elevations at the upstream ends of the siphons are somewhat dissipated in the long pools between siphons as the water surface moves toward normal depth in the pool.

Table 4-4 (based on Table 4-3) summarizes the numbers of utilities, bridges, and overchutes that would need to be raised for each of the cases evaluated in Section 4.3.

**Table 4-4 Summary of Utilities, Bridges, and Overchutes to be Raised**

<b>Component to be Raised</b>	<b>DWR Full Enlargement Alternative 1 (Base Case)</b>	<b>Base Case Plus Smooth Siphons</b>	<b>Base Case Plus Added Check Bays</b>	<b>Base Case Plus Added Check Bays and Smooth Siphons</b>
Utilities	12	12	12	12
Bridges	33	25	31	21
Overchutes	71	68	71	67

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**Table 4-2 Siphon and Check Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)		Base Case Plus Smooth Siphons		Base Case Plus Added Check Bays		Base Case Plus Added Check Bays and Smooth Siphons	
	W.S. Elev (ft)	Head-loss	W.S. Elev (ft)	Head-loss	W.S. Elev (ft)	Head-loss	W.S. Elev (ft)	Head-loss
308.12 Pool 43 Box #1	2965.04	0.07	2964.72	0.04	2964.87	0.06	2964.56	0.04
309.70 Check 43	2964.46	0.07	2964.12	0.07	2964.40	0.05	2964.07	0.04
309.94 Pool 44 Box #2	2964.32	0.04	2963.98	0.02	2964.19	0.04	2963.86	0.02
311.72 Pool 44 Box #3	2963.77	0.04	2963.39	0.03	2963.62	0.04	2963.26	0.03
314.81 Check 44	2962.66	0.07	2962.20	0.07	2962.61	0.05	2962.17	0.04
319.74 Check 45	2961.17	0.07	2960.51	0.07	2961.10	0.04	2960.49	0.04
323.84 Check 46	2959.99	0.06	2959.09	0.06	2959.83	0.07	2958.99	0.05
323.86 Pool 47 Myrick Siphon	2959.49	0.91	2958.63	0.63	2959.33	0.90	2958.54	0.64
326.77 Check 47	2958.26	0.07	2957.50	0.06	2958.06	0.06	2957.38	0.05
326.79 Pool 48 Willow Siphon	2957.75	0.83	2957.03	0.59	2957.56	0.83	2956.92	0.59
330.82 Check 48	2956.31	0.07	2955.63	0.05	2956.06	0.06	2955.48	0.05
330.84 Pool 49 Johnson Siphon	2955.80	0.64	2955.16	0.48	2955.57	0.63	2955.02	0.48
335.93 Check 49	2953.93	0.20	2953.17	0.11	2953.76	0.08	2953.13	0.06
341.51 Check 50	2951.97	0.06	2950.86	0.05	2951.70	0.07	2950.61	0.06
341.53 Pool 51 Ritter Siphon	2951.57	0.70	2950.50	0.47	2951.30	0.70	2950.25	0.46
342.07 Check 51	2951.04	0.06	2950.14	0.06	2950.76	0.06	2949.88	0.06
342.09 Pool 52 Leona Siphon	2950.63	1.21	2949.76	0.79	2950.35	1.21	2949.50	0.79
343.74 Check 52	2949.28	0.05	2948.75	0.05	2949.16	0.04	2948.67	0.04
348.17 Check 53	2948.42	0.06	2947.74	0.06	2948.12	0.06	2947.48	0.05
348.19 Pool 54 Soledad Siphon	2948.02	1.01	2947.37	0.67	2947.73	1.01	2947.12	0.67
350.25 Check 54	2946.83	0.14	2946.45	0.12	2946.67	0.06	2946.34	0.06
352.70 Check 55	2946.07	0.06	2945.75	0.06	2945.93	0.05	2945.56	0.06
352.72 Pool 56 Cheseboro Siphon	2945.63	0.69	2945.32	0.50	2945.54	0.78	2945.14	0.49
354.76 Check 56	2944.76	0.09	2944.63	0.08	2944.62	0.08	2944.43	0.08
354.78 Pool 57 Littlerock Siphon	2944.33	0.54	2944.21	0.42	2944.24	0.61	2944.02	0.42
356.93 Check 57	2942.98	0.27	2942.98	0.27	2942.93	0.12	2942.93	0.12
<b>Pearblossom Pumping Plant</b>								
363.53 Pool 59 Tejon Siphon	3481.85	0.48	3480.95	0.37	3481.71	0.48	3480.86	0.36
366.09 Check 59	3480.75	0.19	3479.83	0.11	3480.59	0.17	3479.70	0.11
366.11 Pool 60 Big Rock Siphon	3480.41	2.49	3479.61	1.72	3480.27	2.49	3479.48	1.71
373.94 Check 60	3474.78	0.32	3474.75	0.31	3474.45	0.28	3474.43	0.27
379.00 Check 61	3472.14	0.19	3472.10	0.19	3472.26	0.08	3472.23	0.08
384.26 Check 62	3469.50	0.25	3469.43	0.25	3469.68	0.11	3469.62	0.11
389.50 Check 63	3466.54	0.19	3466.39	0.18	3466.92	0.09	3466.78	0.08
395.10 Check 64	3463.00	0.20	3462.65	0.21	3463.00	0.20	3462.65	0.21
395.12 Pool 65 Box #1	3463.47	0.04	3463.16	0.04	3463.47	0.04	3463.16	0.04
400.32 Check 65	3459.85	0.05	3458.85	0.06	3459.85	0.05	3458.85	0.06
400.34 Pool 66 Antelope Siphon	3459.29	3.44	3458.34	2.43	3459.29	3.44	3458.34	2.43
403.41 Check 66	3455.29	0.09	3455.29	0.09	3455.29	0.09	3455.29	0.09

# SECTIONFOUR

# Results of Hydraulic Modeling

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
307.23 300th St. W (Private) Bridge	2965.45	No	No	2965.16	No	No	2965.30	No	No	2964.99	No	No
307.24 14" Dia. Irrigation Line	2965.41	No	Yes	2965.11	No	Yes	2965.26	No	Yes	2964.95	No	Yes
307.85 14" Dia. Irrigation Line	2965.24	No	Yes	2964.93	No	Yes	2965.09	No	Yes	2964.77	No	Yes
308.95 Overchute 6' x 30'	2964.84	No	Yes	2964.52	No	Yes	2964.66	No	Yes	2964.35	No	Yes
309.52 Bridge - 280th St. W (Private)	2964.66	No	Yes	2964.33	No	Yes	2964.47	No	Yes	2964.15	No	Yes
309.53 Water pipe	2964.66	No	No	2964.32	No	No	2964.47	No	No	2964.14	No	No
310.12 8" Dia. Irrigation Line	2964.33	No	Yes	2963.99	No	Yes	2964.19	No	Yes	2963.87	No	Yes
310.52 Overchute 6' x 30'	2964.21	Yes	Yes	2963.86	No	Yes	2964.08	No	Yes	2963.74	No	Yes
311.66 260th St. W (Private) Bridge	2963.90	No	No	2963.51	No	No	2963.75	No	No	2963.38	No	No
311.84 12" Waterline (LADW&P)	2963.79	Yes	Yes	2963.41	Yes	Yes	2963.64	Yes	Yes	2963.28	Yes	Yes
312.12 255th St. W ( Elise Street ) Bridge	2963.56	No	Yes	2963.19	No	Yes	2963.41	No	Yes	2963.05	No	Yes
312.51 Overchute 6' x 15'	2963.44	No	Yes	2963.05	No	Yes	2963.28	No	Yes	2962.92	No	Yes
313.56 Highway 136 (State) Bridge	2963.13	No	Yes	2962.71	No	No	2962.96	No	Yes	2962.56	No	No
314.93 Lancaster Rd. (County) Bridge	2962.66	Yes	Yes	2962.20	Yes	Yes	2962.54	Yes	Yes	2962.10	Yes	Yes
315.18 Overchute 6'6" x 10'	2962.58	No	Yes	2962.12	No	Yes	2962.46	No	Yes	2962.02	No	Yes
316.08 Overchute 6' x 26'	2962.33	No	Yes	2961.84	No	Yes	2962.20	No	Yes	2961.73	No	Yes
316.7 Overchute 6' x 26'	2962.14	Yes	Yes	2961.63	No	Yes	2962.00	No	Yes	2961.51	No	Yes
317.87 Overchute 6' x 15'	2961.82	Yes	Yes	2961.26	No	Yes	2961.66	No	Yes	2961.13	No	Yes
318.66 205th St. W (State) Bridge	2961.60	No	Yes	2961.01	No	No	2961.43	No	Yes	2960.87	No	No
318.86 Overchute 6' x 15'	2961.55	Yes	Yes	2960.94	No	Yes	2961.38	No	Yes	2960.81	No	Yes
319.66 Overchute 7'6" x 39'	2961.32	Yes	Yes	2960.67	No	Yes	2961.14	No	Yes	2960.53	No	Yes
319.79 195th St. W (County) Bridge	2961.17	No	Yes	2960.51	No	No	2961.05	No	Yes	2960.44	No	No
320.02 Overchute 6' x 26'	2961.11	Yes	Yes	2960.44	No	Yes	2960.99	Yes	Yes	2960.37	No	Yes
320.53 Overchute 6'6" x 10'	2960.96	Yes	Yes	2960.26	No	Yes	2960.83	Yes	Yes	2960.18	No	Yes
321.08 Overchute 6' x 26'	2960.82	Yes	Yes	2960.09	No	Yes	2960.69	Yes	Yes	2960.01	No	Yes

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
321.54 180th St. W (Private) Bridge	2960.69	Yes	Yes	2959.93	No	Yes	2960.55	No	Yes	2959.85	No	Yes
321.74 Water pipe	2960.64	No	No	2959.86	No	No	2960.49	No	No	2959.77	No	No
322.28 Overchute 6' x 5'	2960.50	Yes	Yes	2959.68	No	Yes	2960.35	Yes	Yes	2959.60	No	Yes
322.65 Pipeline	2960.40	No	No	2959.56	No	No	2960.25	No	No	2959.48	No	No
322.73 170th St. W (County) Bridge	2960.38	Yes	Yes	2959.53	Yes	Yes	2960.22	Yes	Yes	2959.45	Yes	Yes
323.1 Overchute 6'6" x 10'	2960.28	Yes	Yes	2959.41	No	Yes	2960.12	Yes	Yes	2959.32	No	Yes
323.25 Ave. H (County) Bridge	2960.25	Yes	Yes	2959.37	Yes	Yes	2960.09	Yes	Yes	2959.28	Yes	Yes
323.53 Overchute 6' x 5'	2960.16	Yes	Yes	2959.28	No	Yes	2960.00	No	Yes	2959.18	No	Yes
324.46 Private Bridge	2958.94	No	Yes	2958.30	No	No	2958.78	No	Yes	2958.20	No	No
328.54 130th St. W (Private) Bridge	2956.97	No	No	2956.39	No	No	2956.76	No	No	2956.27	No	No
332.32 Private Bridge	2955.23	No	Yes	2954.67	No	Yes	2954.97	No	Yes	2954.51	No	No
333.76 90th St. W (Private) Bridge	2954.81	No	No	2954.18	No	No	2954.52	No	No	2954.00	No	No
334.03 Private Bridge	2954.72	No	No	2954.08	No	No	2954.42	No	No	2953.88	No	No
336.2 70th St. W (Private) Bridge	2953.82	No	Yes	2953.15	No	Yes	2953.65	No	Yes	2953.01	No	Yes
337.73 Goode Hill Rd. (County) Bridge	2953.32	No	No	2952.55	No	No	2953.11	No	No	2952.38	No	No
342.56 Platt Rd. (Private) Bridge	2949.84	No	Yes	2949.35	No	Yes	2949.55	No	Yes	2949.09	No	Yes
345.24 Ave. S (County) Bridge	2949.19	No	Yes	2948.63	No	No	2948.93	No	Yes	2948.40	No	No
345.38 6" Waterline (PWD)	2949.15	No	No	2948.59	No	No	2948.90	No	No	2948.36	No	No
345.71 Tierra Subida Ave. (County) Bridge	2949.08	No	Yes	2948.51	No	Yes	2948.82	No	Yes	2948.27	No	No
346.32 Overchute 6' x 10'	2948.95	Yes	Yes	2948.36	No	Yes	2948.68	Yes	Yes	2948.12	No	Yes
346.53 Barrel Springs O.C. (State) Bridge	2948.91	No	No	2948.30	No	No	2948.63	No	No	2948.07	No	No
346.77 Antelope Freeway (State) Bridge	2948.85	No	No	2948.23	No	No	2948.57	No	No	2948.00	No	No
347.12 8" Waterline (PWD)	2948.77	Yes	Yes	2948.14	Yes	Yes	2948.49	Yes	Yes	2947.90	No	Yes
347.57 Private Bridge	2948.68	Yes	Yes	2948.04	Yes	Yes	2948.40	Yes	Yes	2947.80	No	Yes
349.55 Pearblossom Hwy (County) Bridge	2947.24	Yes	Yes	2946.88	Yes	Yes	2946.93	Yes	Yes	2946.62	Yes	Yes

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
350.16 Barrel Springs Rd. (County) Bridge	2947.12	Yes	Yes	2946.75	No	Yes	2946.81	No	Yes	2946.48	No	Yes
350.74 Overchute 6' x 10'	2946.78	Yes	Yes	2946.43	Yes	Yes	2946.59	Yes	Yes	2946.26	Yes	Yes
351.16" Waterline (PWD)	2946.67	Yes	Yes	2946.37	Yes	Yes	2946.54	Yes	Yes	2946.20	Yes	Yes
351.22 40th St. E (County) Bridge	2946.56	Yes	Yes	2946.26	Yes	Yes	2946.43	Yes	Yes	2946.09	Yes	Yes
351.48 16" Waterline (PWD)	2946.51	Yes	Yes	2946.21	Yes	Yes	2946.38	Yes	Yes	2946.03	Yes	Yes
352.01 47th St. E (County) Bridge	2946.35	Yes	Yes	2946.04	Yes	Yes	2946.22	Yes	Yes	2945.86	No	Yes
352.96 4" Waterline (PWD)	2945.26	Yes	Yes	2945.14	Yes	Yes	2945.04	Yes	Yes	2944.95	Yes	Yes
353.07 Cheseboro Rd. (County) Bridge	2945.14	Yes	Yes	2945.02	Yes	Yes	2945.00	Yes	Yes	2944.83	Yes	Yes
355.57 77th St. E (County) Bridge	2944.04	Yes	Yes	2944.04	Yes	Yes	2943.84	Yes	Yes	2943.84	Yes	Yes
355.86 Overchute 6' x 10'	2943.97	Yes	Yes	2943.97	Yes	Yes	2943.76	Yes	Yes	2943.76	Yes	Yes
356.14 82nd St. E (County) Bridge	2943.71	Yes	Yes	2943.71	Yes	Yes	2943.50	Yes	Yes	2943.50	Yes	Yes
356.47 Overchute 6' x 20'	2943.63	Yes	Yes	2943.63	Yes	Yes	2943.41	Yes	Yes	2943.41	Yes	Yes
356.69 87th St. E (County) Bridge	2943.38	Yes	Yes	2943.38	Yes	Yes	2943.16	Yes	Yes	2943.16	Yes	Yes
356.81 Overchute 6' x 20'	2943.33	Yes	Yes	2943.33	Yes	Yes	2943.12	Yes	Yes	2943.12	Yes	Yes
357.61 Littlerock (State) Bridge	2942.69	Yes	Yes	2942.69	Yes	Yes	2942.69	Yes	Yes	2942.69	Yes	Yes
357.72 96th St. E (County) Bridge	2942.66	Yes	Yes	2942.66	Yes	Yes	2942.66	Yes	Yes	2942.66	Yes	Yes
358.75 106th St. E (County) Bridge	2942.40	Yes	Yes	2942.40	Yes	Yes	2942.40	Yes	Yes	2942.40	Yes	Yes
359.91 116th St. E (County) Bridge	2942.12	No	Yes	2942.12	No	Yes	2942.12	No	Yes	2942.12	No	Yes
359.93 8" Waterline (LACWW Dist. 24)	2942.09	No	Yes	2942.09	No	Yes	2942.09	No	Yes	2942.09	No	Yes
<b>Pearblossom Pumping Plant</b>												
362.6 121st St. E (County) Bridge	3482.71	No	No	3481.86	No	No	3482.58	No	No	3481.77	No	No
363.28 Private Bridge	3482.36	No	Yes	3481.46	No	No	3482.22	No	Yes	3481.35	No	No
364.56 Private Bridge	3481.53	No	Yes	3480.64	No	No	3481.38	No	Yes	3480.54	No	No
365.01 Private Bridge	3481.39	No	Yes	3480.47	No	No	3481.24	No	Yes	3480.36	No	No
365.73 14" Waterline (RCWC)	3481.18	Yes	Yes	3480.20	No	Yes	3481.02	Yes	Yes	3480.08	No	Yes



**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
365.89 Valyermo Rd. (County) Bridge	3481.02	No	No	3480.12	No	No	3480.86	No	No	3480.00	No	No
367.72 Overchute 6' x 16'	3478.14	No	Yes	3478.08	No	Yes	3478.00	No	Yes	3477.95	No	Yes
368.02 Overchute 6' x 16'	3478.03	No	Yes	3477.97	No	Yes	3477.89	No	Yes	3477.83	No	Yes
368.15 14" Waterline (LDRWC)	3477.97	No	Yes	3477.90	No	Yes	3477.82	No	Yes	3477.77	No	Yes
368.35 Overchute 6' x 10'	3477.89	No	Yes	3477.82	No	Yes	3477.74	No	Yes	3477.68	No	Yes
368.53 175th St. E (County) Bridge	3477.82	No	No	3477.75	No	No	3477.66	No	No	3477.60	No	No
368.98 Overchute 6' x 10'	3477.64	No	Yes	3477.57	No	Yes	3477.48	No	Yes	3477.41	No	Yes
369.63 Overchute 6' x 6'	3477.38	No	Yes	3477.31	No	Yes	3477.21	No	Yes	3477.14	No	Yes
370.15 Overchute 6' x 10'	3477.19	No	Yes	3477.11	No	Yes	3477.01	No	Yes	3476.94	No	Yes
370.73 Overchute 6' x 6'	3476.98	No	Yes	3476.89	No	Yes	3476.78	No	Yes	3476.70	No	Yes
371.2 Overchute 6' x 10'	3476.81	No	Yes	3476.72	No	Yes	3476.60	No	Yes	3476.52	No	Yes
371.49 Overchute 6' x 16'	3476.70	No	Yes	3476.60	No	Yes	3476.48	No	Yes	3476.39	No	Yes
371.73 Largo Vista Rd. (County) Bridge	3476.60	No	No	3476.50	No	No	3476.37	No	No	3476.28	No	No
371.89 Overchute 6' x 10'	3476.51	No	Yes	3476.42	No	Yes	3476.29	No	Yes	3476.19	No	Yes
372.51 Overchute 6' x 10'	3476.28	No	Yes	3476.18	No	Yes	3476.04	No	Yes	3475.94	No	Yes
372.71 213th St. E (County) Bridge	3476.20	No	No	3476.09	No	No	3475.95	No	No	3475.84	No	No
373.24 Overchute 6' x 6'	3476.00	No	Yes	3475.88	No	Yes	3475.73	No	Yes	3475.62	No	Yes
373.52 Overchute 6' x 6'	3475.90	No	Yes	3475.78	No	Yes	3475.62	No	Yes	3475.51	No	Yes
374.33 Overchute 6' x 6'	3474.92	No	Yes	3474.90	No	Yes	3474.64	No	Yes	3474.63	No	Yes
375.46 Overchute 6' x 16'	3474.48	No	Yes	3474.46	No	Yes	3474.16	No	Yes	3474.15	No	Yes
375.75 Overchute 6' x 6'	3474.35	No	Yes	3474.32	No	Yes	3474.02	No	Yes	3474.00	No	Yes
376.32 Overchute 6' x 6'	3474.12	No	Yes	3474.10	No	Yes	3473.77	No	Yes	3473.75	No	Yes
377.09 Overchute 6' x 6'	3473.82	No	Yes	3473.79	No	Yes	3473.43	No	Yes	3473.41	No	Yes
377.57 Overchute 6' x 6'	3473.63	No	Yes	3473.60	No	Yes	3473.22	No	Yes	3473.20	No	Yes
377.81 263rd St. E (County) Bridge	3473.53	No	Yes	3473.50	No	Yes	3473.12	No	No	3473.09	No	No

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
378.17 Overchute 7' x 24'	3473.39	No	Yes	3473.36	No	Yes	3472.96	No	Yes	3472.93	No	Yes
378.63 Overchute 7' x 24'	3473.18	No	Yes	3473.15	No	Yes	3472.72	No	Yes	3472.70	No	Yes
378.78 Oasis Rd. Bridge	3473.10	No	No	3473.06	No	No	3472.63	No	No	3472.60	No	No
378.85 Overchute 7' x 36'	3473.06	No	Yes	3473.03	No	Yes	3472.59	No	Yes	3472.56	No	Yes
379.48 Overchute 7' x 36'	3472.43	No	Yes	3472.39	No	Yes	3472.20	No	Yes	3472.17	No	Yes
379.64 Overchute 7' x 36'	3472.35	No	Yes	3472.31	No	Yes	3472.11	No	Yes	3472.08	No	Yes
380.01 Southern Pacific RR Bridge	3472.19	No	No	3472.15	No	No	3471.94	No	No	3471.91	No	No
380.22 Overchute 6' x 36'	3472.07	No	Yes	3472.03	No	Yes	3471.82	No	Yes	3471.79	No	Yes
380.93 Overchute 6' x 16'	3471.78	No	Yes	3471.74	No	Yes	3471.50	No	Yes	3471.46	No	Yes
381.84 Overchute 6' x 10'	3471.41	No	Yes	3471.36	No	Yes	3471.09	No	Yes	3471.05	No	Yes
381.94 Beekley Rd. Bridge	3471.34	No	No	3471.29	No	No	3471.02	No	No	3470.98	No	No
382.88 Overchute 6' x 16'	3470.97	No	Yes	3470.91	No	Yes	3470.61	No	Yes	3470.56	No	Yes
383.01 Sheep Creek Rd. Bridge	3470.89	No	Yes	3470.84	No	Yes	3470.53	No	No	3470.48	No	No
383.3 Overchute 6' x 24'	3470.78	No	Yes	3470.72	No	Yes	3470.40	No	Yes	3470.35	No	Yes
383.57 Overchute 6' x 20'	3470.65	No	Yes	3470.59	No	Yes	3470.26	No	Yes	3470.21	No	Yes
384.07 Johnson Rd. Bridge	3470.43	No	No	3470.36	No	No	3470.01	No	No	3469.95	No	No
384.5 Overchute 6' x 20'	3469.81	No	Yes	3469.74	No	Yes	3469.67	No	Yes	3469.61	No	Yes
384.76 Overchute 6' x 20'	3469.69	No	Yes	3469.62	No	Yes	3469.54	No	Yes	3469.48	No	Yes
385.05 Overchute 6' x 20'	3469.56	No	Yes	3469.49	No	Yes	3469.41	No	Yes	3469.35	No	Yes
385.23 Overchute 6' x 20'	3469.46	No	Yes	3469.39	No	Yes	3469.31	No	Yes	3469.24	No	Yes
385.83 Overchute 6' x 20'	3469.19	No	Yes	3469.11	No	Yes	3469.03	No	Yes	3468.96	No	Yes
386.12 Wilson Ranch Rd. Bridge	3469.06	No	No	3468.97	No	No	3468.89	No	No	3468.81	No	No
386.26 Overchute 6' x 20'	3469.00	No	Yes	3468.92	No	Yes	3468.83	No	Yes	3468.75	No	Yes
386.55 Overchute 6' x 20'	3468.86	No	Yes	3468.78	No	Yes	3468.68	No	Yes	3468.60	No	Yes
386.68 Overchute 6' x 20'	3468.79	No	Yes	3468.71	No	Yes	3468.61	No	Yes	3468.53	No	No

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
386.94 Overchute 6' x 20'	3468.67	No	No	3468.58	No	No	3468.48	No	No	3468.39	No	No
387.26 Overchute 6' x 20'	3468.52	No	No	3468.42	No	No	3468.32	No	No	3468.22	No	No
387.87 Overchute 6' x 20'	3468.24	No	No	3468.14	No	No	3468.03	No	No	3467.93	No	No
388.17 Caughlin Rd. Bridge	3468.10	No	No	3467.99	No	No	3467.87	No	No	3467.77	No	No
388.5 Overchute 6' x 10'	3467.95	No	No	3467.84	No	No	3467.72	No	No	3467.61	No	No
388.82 Overchute 5' x 24'	3467.78	No	Yes	3467.67	No	Yes	3467.54	No	Yes	3467.42	No	Yes
389.02 Duncan Rd. Bridge	3467.68	No	No	3467.56	No	No	3467.43	No	No	3467.31	No	No
389.3 Overchute 5' x 36'	3467.54	No	Yes	3467.41	No	Yes	3467.28	No	Yes	3467.14	No	Yes
389.6 Overchute 5' x 36'	3466.99	No	Yes	3466.86	No	Yes	3466.99	No	Yes	3466.86	No	Yes
390.42 36" High Pressure Gas Line	3466.57	No	Yes	3466.41	No	Yes	3466.57	No	Yes	3466.41	No	Yes
390.48 Overchute 5' x 24'	3466.53	No	Yes	3466.37	No	Yes	3466.53	No	Yes	3466.37	No	Yes
390.69 Overchute 5' x 36'	3466.42	No	Yes	3466.26	No	Yes	3466.42	No	Yes	3466.26	No	Yes
390.87 Overchute 5' x 36'	3466.32	No	Yes	3466.15	No	Yes	3466.32	No	Yes	3466.15	No	Yes
390.89 10" Waterline (RMCWD)	3466.27	No	No	3466.11	No	No	3466.27	No	No	3466.11	No	No
391.1 Goss Rd. Bridge	3466.17	No	No	3466.00	No	No	3466.17	No	No	3466.00	No	No
391.39 Overchute 6' x 24'	3466.02	No	Yes	3465.84	No	Yes	3466.02	No	Yes	3465.84	No	Yes
391.86 Overchute 6' x 24'	3465.78	No	Yes	3465.60	No	No	3465.78	No	Yes	3465.60	No	No
392.61 Overchute 6' x 10'	3465.40	No	Yes	3465.20	No	Yes	3465.40	No	Yes	3465.20	No	Yes
393.13 Overchute 6' x 10'	3465.13	No	No	3464.91	No	No	3465.13	No	No	3464.91	No	No
393.61 Hwy 395 (State) Bridge	3464.87	No	No	3464.63	No	No	3464.87	No	No	3464.63	No	No
393.96 Overchute 6' x 10'	3464.66	No	No	3464.40	No	No	3464.66	No	No	3464.40	No	No
394.84 Overchute 6' x 10'	3464.17	No	Yes	3463.88	No	Yes	3464.17	No	Yes	3463.88	No	Yes
395.34 Overchute 5' x 16'	3463.43	No	Yes	3463.11	No	Yes	3463.43	No	Yes	3463.11	No	Yes
395.7 Main St. Bridge	3463.22	No	No	3462.87	No	No	3463.22	No	No	3462.87	No	No
395.87 Overchute 6' x 16'	3463.10	No	No	3462.74	No	No	3463.10	No	No	3462.74	No	No

**Table 4-3  
Bridge and Overchute Modeling Results**

Canal Mile and Description	DWR Full Enlargement Alternative 1 (Base Case)			Base Case Plus Smooth Siphons			Base Case Plus Added Check Bays			Base Case Plus Added Check Bays and Smooth Siphons		
	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board	W.S. Elev (ft)	W.S. Contact Soffit	Inadequate Free-board
396.12 Overchute 6' x 16'	3462.94	No	Yes	3462.57	No	No	3462.94	No	Yes	3462.57	No	No
396.35 Overchute 6' x 16'	3462.79	No	Yes	3462.41	No	No	3462.79	No	Yes	3462.41	No	No
396.77 Overchute 6' x 16'	3462.53	No	No	3462.12	No	No	3462.53	No	No	3462.12	No	No
397.25 Overchute 6' x 16'	3462.23	No	No	3461.77	No	No	3462.23	No	No	3461.77	No	No
398.05 Maple Ave. Bridge	3461.71	No	No	3461.17	No	No	3461.71	No	No	3461.17	No	No
398.24 Mesquite Rd. Bridge	3461.55	No	No	3460.98	No	No	3461.55	No	No	3460.98	No	No
398.8 Cottonwood Ave. Bridge	3461.18	No	No	3460.54	No	No	3461.18	No	No	3460.54	No	No
399.57 Ranchero Rd. Bridge	3460.58	No	No	3459.80	No	No	3460.58	No	No	3459.80	No	No
399.8 Overchute 5' x 36'	3460.38	No	No	3459.53	No	No	3460.38	No	No	3459.53	No	No
400.03 Overchute 5' x 5'	3460.16	No	No	3459.24	No	No	3460.16	No	No	3459.24	No	No
400.12 Farmington Rd. Bridge	3460.04	No	No	3459.09	No	No	3460.04	No	No	3459.09	No	No
400.26 Overchute 5' x 16'	3459.92	No	No	3458.92	No	No	3459.92	No	No	3458.92	No	No
402.16 Las Flores Rd. Bridge	3455.92	No	No	3455.92	No	No	3455.92	No	No	3455.92	No	No

The following sections describe alternatives to raise the canal, modify/enlarge the checks and siphons, raise or anchor overchutes and bridges, and extend culverts or raise the headwalls.

## 5.1 CANAL RAISE CONCEPTS

The canal capacity will be increased by raising the canal banks. The amount of the raise required will vary depending upon the hydraulic operating scenarios ultimately selected and the height of existing canal lining and embankment. Generally, the amount of canal raise needed is greater in the pools upstream of Pearblossom Pumping Plant than in the pools downstream of Pearblossom Pumping Plant. Refer to Section 8 for profiles of existing and proposed water surface elevations for various scenarios. The increase in water surface elevations suggest that canal embankment raise requirements vary from approximately 2 to 3.7 feet upstream of Pearblossom Pumping Plant and from no raise to 2.8 feet downstream of Pearblossom Pumping Plant.

Depending upon the topography of the canal for the reach being raised, there could be different raise options for each side of the canal. For example, an embankment fill section would be treated differently than a cut section. The following paragraphs describe the different canal raise alternatives developed and reference conceptual figures for each.

### 5.1.1 Earthfill Embankment Alternative 1

This alternative (Figure 5-1) is the baseline alternative developed by DWR (2004) and is advantageous in areas where right-of-way is not restricted and borrow and/or spoil materials from previous canal construction are readily available near the canal. This alternative was used in the Phase I Enlargement for the East Branch Aqueduct, and uses a 4-inch thick concrete lining section.

An earthfill embankment is the only acceptable alternative in zones where active fault crossings are an issue. The locations of seismic zones are indicated in Table 5-1. Only the earthfill embankment is to be used within 0.2 miles of a Type I fault crossing. There are no restrictions on canal crossing for Type III fault status. (Type II fault crossings are not indicated in Table 5-1.)

The earthfill embankment alternative includes clearing and grubbing the exterior slopes, removing the existing road, raising the embankment with local materials, placing a new road and concrete lining and hydroseeding the exterior banks.

Where available, stockpiled materials from the original construction will be used for embankment fill. Otherwise, earthfill will need to be imported from off-site sources for embankment construction.

**Table 5-1 Fault Crossing Locations**  
 [Check 41 to Check 60 (Mojave Siphon); Ref: DWR, 4/27/2007]

Current Mile Post	1965 Map Mile Post	Fault Name	Fault Status	Location/Feature
203.30	305.0		III	
304.80	306.0		III	
336.20	337.0	Hitchbroom; SAFZ	I	
337.53 - 337.55		Hitchbroom; SAFZ	I	
341.65		San Andreas	I	Check 50
341.85		San Andreas	I	Ritter Siphon
342.07		San Andreas	I	Leona Siphon
342.20		San Andreas	I	Leona Siphon
342.50		San Andreas	I	Pool 52
342.66		San Andreas	I	Pool 52
346.01		Nadeau; SAFZ	I	
348.90		San Andreas	I	
349.50		San Andreas	I	
349.68		San Andreas	I	
350.27		San Andreas	I	Check 54
350.74		San Andreas	I	
351.77		San Andreas	I	
354.77	356.0	San Andreas	I	Little Rock Siphon
361.05	362.0		III	
361.30	362.2		III	
361.45	362.4		III	
361.55	362.5		III	
362.30	363.2		III	
363.95 - 364.20	365.0		III	
364.85	365.6		III	
364.95	365.6		III	

**Notes:**

1. "Data primarily compiled from California Aqueduct Fault Crossings," a Project Geology map and explanation dated 11/18/65.
2. SAFZ - San Andreas Fault Zone.
3. Fault Status: I - Active fault with defined surface expression, II - Potentially active or active fault with proximity to the Aqueduct considered a hazard, and III - Minor and/or ill-defined faults not considered significant.
4. Secondary references for this table:  
 E.J. Bortugno and T.E. Spittler, 1986, "Geologic Map of the San Bernardino Quadrangle," CDMG Regional Geologic Map Series, Map 3A, scale 1:250,000.  
 C.W. Jennings and R.G. Strand, 1969, "Los Angeles Sheet, Geologic Map of California," Olaf Jenkins Edition, CDMG, scale 1:250,000.  
 Project Geology Reports C-49 (April 1969), C-66 (April 1971), and C-70 (June 1974).

An average haul distance of 10 miles is assumed for canal embankment fill. Road costs are based on an 18-foot-wide road.

### 5.1.2 Roller Compacted Concrete Alternative 2

Roller compacted concrete (RCC) is a viable alternative that has been used extensively in dam and spillway modifications. The refill area behind the lip of the existing lining would be excavated to a width needed for the RCC paving machine, and RCC would be placed in approximate 12-inch lifts to the height needed for the top of the canal embankment (Figure 5-2). The width of the RCC block would be kept to a minimum to reduce the impact to the operation and maintenance road. Forming the sides of the RCC could be accomplished at an increased cost.

### 5.1.3 Spread Footing Wall Alternative 3

This alternative could be used in locations where the existing embankment will provide 2 feet of freeboard (Figure 5-3). The vertical wall placed immediately adjacent to the existing canal lining would be the height sufficient to provide an additional 2 feet of freeboard. Impacts to the existing operation and maintenance road are minimal.

### 5.1.4 Concrete Parapet Wall Alternative 4

This alternative (Figure 5-4) is similar to the earthfill embankment alternative and the spread footing wall alternative, except that the impacts to the existing operation and maintenance road are less. The wall height would be sufficient to provide the 2-foot unlined freeboard.

### 5.1.5 Earthfill Embankment With Retaining Wall Alternative 5

This alternative (Figure 5-5) would be used primarily on the outside of the operation and maintenance road and be beneficial to keep the downside slopes of the earthfill embankments from extending into rights-of-way.

### 5.1.6 Slipform Wall Alternative 6

This alternative (Figure 5-6) has a limited applicability for reaches downstream of Pearblossom Pumping Plant where the amount of the canal raise required is 8 inches or less and the existing unlined freeboard is less than 1.5 feet. It is essentially an enlarged concrete curb that is placed immediately adjacent to the existing concrete lining to provide for a small raise amount.

### 5.1.7 Precast Panel System Alternative 7

This alternative (Figure 5-7) would be applicable for canal raises 2-foot and less in height. Access to the canal for maintenance could be accommodated by temporarily removing a precast panel segment.

## 5.2 SERVICE ROADS

Primary and secondary maintenance roads are maintained on the top of the embankments. The primary service road is situated on the left or north side of the embankment and is 18 feet wide with a 6-inch asphalt concrete (AC) dike on the inboard side and a two foot shoulder on the outboard side. As-built drawings indicate the original section of the primary road has 2 inches of asphalt concrete over 4 inches of aggregate base. The secondary road on the right or south embankment is aggregate base (AB) only.

Raising the embankment will require replacing portions of the service road. The DWR Maintenance Department recommendation of a design pavement section for the primary road of 3 inches of AC over 5 inches of AB was used for the cost estimates. The paved width will be 16 feet.

## 5.3 CHECKS/SIPHONS

The enlargement of the East Branch Aqueduct will require modifications to the siphons and check structures upstream of Pearblossom Pumping Plant. Downstream of Pearblossom Pumping Plant, only modifications to the Tejon and Antelope siphons are needed for the Base Case (see Section 4.3.1). An additional bay will need to be added to sixteen check structures and an additional barrel will be added to thirteen siphons. The existing radial gates and the inlet/outlet transitions to checks and inlet/outlet transitions to siphons will also require modifications. Table 5-2 provides a summary of the check and siphon modifications for the Base Case (see Section 4.3.1).

**Table 5-2 Check and Siphon Modifications – Base Case**

Check or Siphon No.	Inlet to Siphon	No. of Check Bays		No of Siphon Barrels		Siphon Length	Siphon Size
		Exist.	Prop.	Exist.	Prop.	Ft.	Ft.
RCB#1	-	-	-	3	4	275	12.5 x.16
Check 43	No	2	3	-	-	-	-
RCB#2	-	-	-	3	4	125	12.5 x.16
RCB#3	-	-	-	3	4	155	12.5 x.16
Check 44	No	2	3	-	-	-	-
Check 45	No	2	3	-	-	-	-
Check 46	Myrich	3	4	3	4	1,061	13
Check 47	Willow	3	4	3	4	925	13
Check 48	Johnson	3	4	3	4	626	13
Check 49	No	2	3	-	-	-	-
Check 50	Ritter	3	4	3	4	1,024	13
Check 51	Leona	3	4	3	4	1,802	13
Check 52	No	2	3	-	-	-	-



Table 5-2 Check and Siphon Modifications – Base Case

Check or Siphon No.	Inlet to Siphon	No. of Check Bays		No of Siphon Barrels		Siphon Length	Siphon Size
		Exist.	Prop.	Exist.	Prop.	Ft.	Ft.
Check 53	Soledad	3	4	3	4	1,496	13
Check 54	No	2	3	-	-	-	-
Check 55	Cheseboro	3	4	3	4	896	13
Check 56	Littlerock	3	4	3	4	866	13
Check 57	No	2	3	-	-	-	-
<b>Pearblossom Pumping Plant</b>							
Pool 59	Tejon	-	-	3	4	669	2x12, 1x13 & new 1x13
Check 65	Antelope	3	4	3	4	3,763	2x11, 1x12 & new 1x12

**Note:**

Checks 49, 52, 54, and 57 have exterior counterfort walls.

### 5.3.1 Check Structure Modifications

Modifying the check structures presents one of the greatest challenges to the East Branch Aqueduct Enlargement. An additional check bay having the same size as the existing bay is needed at the check structures listed in Table 5-2. The existing check structures 43 through 45 are 16 feet wide by 16 feet high, and the rest of the check structures are 13 feet wide by 14.75 feet high. The check structure bays range from 27 feet to 50 feet in length. The existing inlet and outlet transitions are warped transitions 30 feet and 45 feet to 65 feet in length, respectively, with an overall top width (including canal prism) of up to 113 feet.

The present concept is for the new check bays to be placed as close to the existing structure as possible to minimize the extent of transition modifications and right-of-way required. The existing inlet and outlet transitions would need to be modified to accommodate the increased check structure width as well as an increase in height to accommodate the increased flow depth. This would need to be done utilizing a cofferdam that spans the length of the transitions impacted and attaches securely to the check structures. This will also involve demolition of the transitions.

Several of the existing check structure bays were designed with exterior counterfort walls that will make it more complicated to place the new check bay immediately adjacent to the existing wall. All of the transitions are warped transitions with counterforts. It is recommended that a detailed review of the loading criteria for the existing check structures be conducted and/or a structural analysis be performed to assess the limitations and construction constraints for the additional check work.

Figures 5-8 and 5-9 depict concepts for 3-bay check structures without and with counterfort walls respectively. Figures 5-10 and 5-11 depict concepts for 4-bay check structure siphon-inlet and siphon-outlet, respectively.

The need to maintain flow deliveries while constructing the additional check bays is an important consideration that may drive the design and placement of the new check bays with regard to the existing structure. Alternatives to bypass flow will need to be developed and analyzed with respect to reliability, constructability, and cost. Alternative concepts to maintain flow deliveries include:

- In the event that the check structure exterior walls were not designed for water loading without soil backfill, additional provisions might be needed. It may be possible to provide an “internal” prefabricated bypass that sits inside a check bay to accommodate operational water loads while the soil behind the existing exterior wall is removed to construct the new check bay. This may not be needed for the counterfort type walls. An alternate, more practical approach may be to design temporary bracing members to support the existing structure under the temporary loading condition.
- Each existing check bay has stop-log slots and, therefore, may be readily removed from service. A single check bay may be stop-logged, and removed from service to construct the adjacent new bay, and a pipe bypass constructed on the opposite side of the structure. The size of the pipe bypass would need to be determined based on the required flow deliveries along with the number of checks under construction simultaneously.

An alternative approach to adding a check bay is to construct a new check structure entirely separate from the existing check structure (see Figure 5-12). Short channel connections would be needed to tie into the existing canal upstream and downstream of the existing transitions. The O&M road would also need to be widened. This arrangement would simplify the cofferdam needed, and the modifications to the existing inlet and outlet transitions described previously would not be required. However, as indicated on Figure 5-12, excavation into hillsides and routing of the secondary O&M road around the excavation would involve a significant area. Measures will be needed to reduce the impact to the existing control building situated adjacent to each check structure.

This alternative approach would need to be evaluated on a case-by-case basis, taking into consideration the existing right-of-way limitations as one of the primary issues. This arrangement would, however, have less constructability issues than construction immediately adjacent to the existing check structure and should be evaluated during the next phase of the analysis.

### 5.3.2 Radial Gates

The height of the existing radial gates will have to be increased to match the increased flow depth to prevent overtopping the gates. The gates would be removed from the existing structure, and the structural frame strengthened and face plates extended as needed. Alternatively, the gates could be replaced. During the Phase I Enlargement, the heights of the radial gates were increased by adding raised sills. A raise of up to 1-foot 9 inches was accomplished by adding a structural steel frame/bottom seal embedded in concrete. The side seals were also extended by embedding seal plates in a raised superstructure wall.

Present plans for this enlargement call for the raised sill to be removed, and the overall gate modified to account for the total increased flow depth for both Phases I and II. Depending upon the amount of additional depth needed at each structure, it may be possible to remove the existing sill and replace it with a raised sill of the appropriate height needed for the Phase II Enlargement. This may result in cost savings for the gate modification and its associated hoist and electrical controls (if the height of the gate can remain the same). However, this may result in raising the water surface elevation. The cost of raising the water surface elevation would need to be evaluated against the cost savings by reducing the gate modifications needed. The gate side seals and superstructure/operating platform would still need to be raised the appropriate amount for the increased flow depth. The reinforced concrete structure is reported to be capable of withstanding larger forces than existing criteria (DWR, 2004). Further analyses of the existing concrete structure and the existing radial gates will be required.

A model run was made to assess the potential increase in water surface elevation that might result from raising the sills under the gates at the check structures. For this model run, the maximum sill was set to 3 feet and this raise was applied to the gates with existing sills. Existing gates without sills were given a sill height of 1.5 feet, and new gates had no added sills. Model results showed an average water surface elevation increase of 0.2 feet immediately upstream of the 3-bay check structures, and 0.09 feet at the four-bay checks.

### 5.3.3 Siphons

An additional barrel will need to be added at each location shown in Table 5-2. Present planning is for the new barrel to be identical in size to the existing barrels. There are three reinforced concrete box (RCB) siphons with 3 barrels, each 12.5 feet wide by 16 feet high, upstream of Pearblossom Pumping Plant that will have one new barrel added. Except for the Tejon and Antelope siphons, the rest of the siphons are triple 13-foot diameter pipes that will have one new pipe added. Operating heads range from approximately 20 feet for the RCBs to over 100 feet for the pipes. The 13-foot diameter siphon pipes will most likely be reinforced concrete cylinder pipe (RCCP) per AWWA C300. A pipe manufacturer reports that 13-foot diameter RCCP will need to be transported via truck in pipe lengths under 16 feet. During future engineering studies, it may be useful to evaluate using 12-foot diameter RCCP as it can be transported in approximately 24-foot lengths. A rough estimation of potential cost savings is approximately 25% of the cost of the larger diameter pipe.

The existing inlet and outlet transitions would need to be modified to accommodate the additional siphon barrel as well as an increase in height to accommodate the increased flow depth. Figures 5-10 and 5-11 depict concepts for 4-bay check structure siphon-inlet and siphon outlet respectively. Figure 5-13 depicts the 4-barrel RCB siphon concept. Shoring systems would be required to allow the existing siphon barrel adjacent to the new barrel to remain in operation during construction. Tie-ins for the inlet and outlet transitions would need to be appropriately scheduled to occur during low flow periods.

## 5.4 OVERCHUTES

Overchutes are open channel, gravity flow, concrete flumes that pass runoff over the canal. To increase canal capacity, overchutes can be handled in two ways. They can be raised to the desired freeboard or anchored in their existing position. However, overchutes cannot be raised

without also raising the upstream flow conditions and causing environmental impacts. In most cases, this will result in significant grading requirements at the upstream end of the overchute.

Raising or anchoring overchutes with clearances of less than 1-foot was considered (DWR, 2004). Not raising the overchutes will mean that some overchutes will be partially submerged in the canal. Securing the overchutes from buoyant and lateral forces is achievable with epoxy dowels. The anchorages will need to consider uplift and lateral loads due to transient waves from plant shutdowns. Additional head losses will be encountered at each submerged soffit that have been accounted for in the hydraulic calculations.

Keeping the overchutes at the present elevations has the advantages of avoiding the additional costs of altering the uphill flow patterns (raising the overchutes would raise flow levels on the uphill side of the canal), raising the overchutes, and constructing approach and tailrace extensions. Anchoring is the preferred option from an economic standpoint. Allowing the soffits of the overchutes to protrude into the canal water surface (negative freeboard) is conceptually acceptable, but will depend on the results of hydraulic modeling to assess headloss across these structures. In addition to anchoring, the joints in the overchute invert would need to be sealed to prevent canal water from entering the overchutes during high flow periods in the canal.

This study assumes that all overchutes will remain in place and not be raised. All overchutes within two feet of the maximum water surface elevation can be secured to their abutments and piers with epoxy dowels for relatively small cost (\$20,000 each). Figures 5-14 and 5-15 illustrate a typical overchute anchoring concept. Additionally, where the embankment is raised, the headwalls of the inlet and outlet will need to be raised.

## 5.5 BRIDGES

State and county bridges will need to be raised to a minimum clearance of ten inches above the maximum water surface elevation to meet CALTRANS requirements. Private bridges will also likely need to be raised to a similar standard. If bridges are raised, they will need to meet current seismic standards.

The cost of raising or replacing the bridge will depend on the bridge type. Bridges with spans that are built integrated into the abutments and piers will likely need to be replaced. Bridges with spans that are not integral to the vertical support structure may be raised by placing jacks on the abutments and piers, jacking the structures, and placing reinforced concrete that is doweled into the existing abutments and piers.

Record drawing information on most of the bridges was not available for this study. A concrete girder type bridge was assumed for all bridges for which record drawing information was not available. A search of available record drawings from state, county, and local agencies will be needed for design.

## 5.6 CULVERTS

The headwalls on culverts will need to be extended in locations where the canal embankment is to be raised. The cost associated with this work is approximately \$121,000 for a two-foot headwall adjustment. In some cases an adjustment may be avoided by implementing a raise alternative that avoids raising the embankment.

Increasing the water surface elevation will apply additional loads on existing culverts. Some culverts show cracks and may need repair (URS/MWH, 2005); a structural liner may be needed to strengthen and repair some culverts. Depending on structural lining thickness, the linings would decrease the flow capacity of the culverts causing higher water surface elevations at the headwalls. The headwalls may need to be raised to account for the increased water surface elevation. The costs of repair and strengthening are not included in this report.

Piping of the canal embankment soils through open cracks in the culverts is also an issue and must be prevented. One method of repair includes installing a laminate repair in the crack locations. DWR is considering a laminate system for the Mojave Bypass (URS/MWH, 2005).

### 5.7 SPILL BASINS

It may be possible to relax the 2-foot unlined freeboard at Pool 58 (Pearblossom) or Pool 66 by using spill basins to store water from transient waves resulting from load rejection. Spill basins would likely be under Division of Safety of Dams (DSOD) jurisdiction. Implementation of these spill basins as an alternative may be pursued in the next stage of the enlargement project. Spill basin costs have not been developed and are not included in this report.

**6.1 GENERAL**

Screening criteria were developed in order to evaluate the canal raise alternatives described in Section 5.1. This section presents the screening criteria employed and the resulting evaluation for the canal raise concepts. The feasibility designs described in Section 8 utilize alternatives with the lowest estimated costs that are suitable for implementation for the canal reach under review.

**6.2 EVALUATION CRITERIA**

Alternatives evaluation criteria, contents, and commentary are summarized in Table 6-1. Evaluation criteria include the following:

- Implementability
- Operational Flexibility, Maintenance, and Reliability
- Cost
- General Environmental Impacts – This criterion is addressed separately from this Feasibility Report.

**Table 6-1 Criteria and Contents for Alternatives Evaluation**

No.	Criteria	Contents	Commentary
1	Implementability	Alignment Constructability/risk Geotechnical considerations Construction schedules Right-of-way (ROW)	Location requirements exist. In-canal construction period is from mid-October through mid-February. Water deliveries must not be shut down. Right-of-way limitations exist.
2	Operational Flexibility, Maintainability & Reliability	Operational flexibility Operational reliability/performance Seismic reliability/performance Maintainability	Confidence in good performance; vulnerabilities in performance will be considered.
3	Cost	Capital costs Life cycle costs	Current pricing levels are used.

**6.3 EVALUATION**

Table 6-2 presents a summary matrix of the alternatives evaluated for the canal raise. Each of the criteria listed in Table 6-1 was defined under several sub-criteria to provide a mechanism to evaluate the alternative canal raise concepts.

The alternatives implementability are initially described in terms of locations where each alternative might be best suited and also where each alternative could not be used (i.e., restrictions exist at fault crossing locations). A sub-criterion for implementability is constructability/risk that considers geotechnical issues and construction limitations that might occur due to unanticipated field conditions, or potential equipment limitations that might impact construction quality. Each alternative was also evaluated on its ability to tie into check structures readily and whether additional structural improvements would be needed to make this transition.

Availability and cost of right-of-way is a significant issue and each alternative was evaluated for this sub-criterion.

The operational flexibility, maintainability and reliability features of each canal raise alternative was evaluated for impact to the operation and maintenance road and for potential increased joint lining and surface maintenance. The cost evaluation includes a capital cost estimate per lineal foot of canal raise, and a qualitative assessment on life cycle considerations that would affect the overall cost of the facilities.

The earthfill embankment raise (Alternative 1) may be used where developments do not encroach on the canal embankments. This is also the only canal raise alternative that may be used where the canal traverses an active fault zone. Where developments encroach on the canal, Alternatives 2 through 7 are considered, as indicated in Table 6-2. For canal raise requirements greater than 8 inches, the precast panel system (Alternative 7) has the lowest cost.

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**Table 6-2  
Alternatives Evaluation of Canal Raise Concepts**

Criteria	Alternative 1 Earthfill Embankment	Alternative 2 Roller Compacted Concrete	Alternative 3 Spread Footing Wall	Alternative 4 Concrete Parapet	Alternative 5 Earthfill Embankment w/Retaining Wall	Alternative 6 Slipform Wall	Alternative 7 Precast Panel System
<b>Implementability</b>							
	<ul style="list-style-type: none"> <li>➤ Use in cut areas</li> <li>➤ Use where embankment fill material is readily available</li> <li>➤ ROW constraints may preclude use of this alternative in locations</li> <li>➤ Slope stability needs to be confirmed</li> <li>➤ Investigation on added loads to existing canal culverts needed</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use where Earthfill Embankment Alternative is not possible due to ROW constraints</li> <li>➤ Use where embankment fill material is not readily available</li> <li>➤ Do not use in fault crossing locations</li> <li>➤ Slope stability needs to be confirmed for existing embankment</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use where Earthfill Embankment Alternative is not possible due to ROW constraints</li> <li>➤ Use where embankment material is not readily available</li> <li>➤ Do not use in fault crossing locations</li> <li>➤ Slope stability needs to be confirmed for existing embankment</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use where Earthfill Embankment Alternative is not possible due to ROW constraints</li> <li>➤ Use where embankment material is not readily available</li> <li>➤ Do not use in fault crossing locations</li> <li>➤ Slope stability needs to be confirmed for existing embankment</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use where Earthfill Embankment Alternative is not possible due to ROW constraints</li> <li>➤ Use where embankment material is not readily available</li> <li>➤ Do not use in fault crossing locations</li> <li>➤ Slope stability needs to be confirmed for existing embankment</li> <li>➤ Investigation on added loads to existing canal culverts needed</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use only where height of embankment raise is 8 inches or less</li> </ul>	<ul style="list-style-type: none"> <li>➤ Use where Earthfill Embankment Alternative is not possible due to ROW constraints</li> <li>➤ Use where embankment material is not readily available</li> <li>➤ Do not use in fault crossing locations</li> <li>➤ Slope stability needs to be confirmed for existing embankment</li> </ul>
<b>Constructability/Risk</b>							
	<ul style="list-style-type: none"> <li>➤ Adequate quality and quantity of borrow sources need to be located</li> <li>➤ Erosion and sediment control for earthfill embankment construction will be needed</li> <li>➤ Need to protect adjacent properties from earthfill construction operations</li> <li>➤ Sliver fills on small embankment raises may be difficult to place and compact</li> <li>➤ Canal lining could be shotcrete or machine lined</li> </ul>	<ul style="list-style-type: none"> <li>➤ 4-foot to 6-foot wide lifts selected; (limitation is vibratory roller requirements)</li> <li>➤ Cracking will need to be controlled; heat rise in RCC, especially during the summer months, will be required to reduce the potential for thermal cracking</li> <li>➤ Sealing of cracks will be required</li> </ul>	<ul style="list-style-type: none"> <li>➤ Unformed back wall adjacent to existing canal lining lip may present problems with voids underneath canal lip if soil cannot stand up sufficiently</li> </ul>	<ul style="list-style-type: none"> <li>➤ Requires most concrete placement forming</li> </ul>	<ul style="list-style-type: none"> <li>➤ Canal lining could be shotcrete or machine lined</li> </ul>	<ul style="list-style-type: none"> <li>➤ None identified</li> </ul>	<ul style="list-style-type: none"> <li>➤ Potential issue with achieving compaction and seal at panel – foundation interface may be resolved by designed detailing</li> <li>➤ Potential fit-up problems between posts and panels</li> <li>➤ Canal lining portion could be shotcrete or machine lined</li> </ul>
<b>Check Structure Tie ins</b>							
	<ul style="list-style-type: none"> <li>➤ Requires modification (lining extension or parapet wall)</li> </ul>	<ul style="list-style-type: none"> <li>➤ Requires modification (lining extension or parapet wall)</li> </ul>	<ul style="list-style-type: none"> <li>➤ Facilitates tie-in</li> </ul>	<ul style="list-style-type: none"> <li>➤ Facilitates tie-in</li> </ul>	<ul style="list-style-type: none"> <li>➤ Requires modification (lining extension or parapet wall)</li> </ul>	<ul style="list-style-type: none"> <li>➤ Requires modification (lining extension or parapet wall)</li> </ul>	<ul style="list-style-type: none"> <li>➤ Facilitates tie-in</li> </ul>

Table 6-2  
Alternatives Evaluation of Canal Raise Concepts

Criteria	Alternative 1 Earthfill Embankment	Alternative 2 Roller Compacted Concrete	Alternative 3 Spread Footing Wall	Alternative 4 Concrete Parapet	Alternative 5 Earthfill Embankment w/Retaining Wall	Alternative 6 Slipform Wall	Alternative 7 Precast Panel System
<b>Right-of-Way (ROW)</b>							
	<ul style="list-style-type: none"> <li>➤ Has greatest need for additional ROW requirements</li> <li>➤ May have potential for environmental issues for additional ROW needed</li> <li>➤ ROW purchase may have schedule impacts</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements (assuming reduced O&amp;M road width is acceptable)</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements</li> </ul>	<ul style="list-style-type: none"> <li>➤ No additional ROW requirements</li> </ul>
<b>Operation &amp; Maintenance (O&amp;M)</b>							
	<ul style="list-style-type: none"> <li>➤ Maintains present lining and embankment configuration; no reduction in capability to access canal lining</li> <li>➤ Canal lining joints will require maintenance to provide for a water seal</li> </ul>	<ul style="list-style-type: none"> <li>➤ Changes present lining configuration with reduced capability to access canal lining</li> <li>➤ Reduces O &amp; M road width. This impact may be minimized by forming vertical wall at road side – for added cost</li> <li>➤ RCC cracking may result in water sealing issues that will require maintenance</li> </ul>	<ul style="list-style-type: none"> <li>➤ Changes present lining configuration with reduced capability to access canal lining</li> <li>➤ Canal lining and vertical wall joints will require maintenance to provide for a water seal</li> </ul>	<ul style="list-style-type: none"> <li>➤ Changes present lining configuration with reduced capability to access canal lining</li> <li>➤ Canal lining and transverse wall joints will require maintenance to provide for a water seal</li> </ul>	<ul style="list-style-type: none"> <li>➤ Maintains present lining and embankment configuration; no reduction in capability to access canal lining</li> <li>➤ Canal lining will require maintenance to provide for a water seal</li> </ul>	<ul style="list-style-type: none"> <li>➤ Short curb presents slightly reduced capability to access canal lining</li> <li>➤ Canal lining and vertical wall joints will require maintenance to provide for a water seal</li> </ul>	<ul style="list-style-type: none"> <li>➤ Changes present lining configuration with reduced capability to immediately access canal lining (panels need to be removed from posts to access canal lining)</li> <li>➤ Canal lining and joints will require maintenance to provide for a water seal; sealing issues at base of panels will require maintenance</li> </ul>
<b>O &amp; M Road Impact</b>							
	<ul style="list-style-type: none"> <li>➤ No impact to road width; it meets standard width criteria</li> <li>➤ Communication facilities in road will likely be impacted</li> <li>➤ O &amp; M road culverts will need to be replaced (inlets and outlets will be raised)</li> </ul>	<ul style="list-style-type: none"> <li>➤ O&amp;M road width will decrease approx. 1.5 ft to 2 ft. per foot of canal raise</li> <li>➤ Communication facilities in road will likely be impacted</li> <li>➤ O &amp; M road culverts will need to be modified (minor)</li> </ul>	<ul style="list-style-type: none"> <li>➤ May slightly increase width</li> <li>➤ Communication facilities in road will likely be impacted</li> <li>➤ O &amp; M road culvert outlets will need to be modified (minor)</li> </ul>	<ul style="list-style-type: none"> <li>➤ May slightly increase width</li> <li>➤ Communication facilities in road will likely be impacted</li> <li>➤ O &amp; M road culvert outlets will need to be modified (minor)</li> </ul>	<ul style="list-style-type: none"> <li>➤ No impact to road width; it meets standard width criteria</li> <li>➤ Communication facilities in road will likely be impacted</li> <li>➤ O &amp; M road culverts will need to be replaced (inlets and outlets will be raised)</li> </ul>	<ul style="list-style-type: none"> <li>➤ No impact to road width; it meets standard width criteria</li> <li>➤ Communication facilities in road will probably not be impacted</li> <li>➤ O &amp; M road culverts will need to be modified (minor)</li> </ul>	<ul style="list-style-type: none"> <li>➤ Slight impact to road width; it most likely meets standard width criteria</li> <li>➤ Communication facilities in road will probably not be impacted</li> <li>➤ O &amp; M road culverts will need to be modified (minor)</li> </ul>
<b>Cost Estimate (see Note 1)</b>							
	<ul style="list-style-type: none"> <li>➤ \$337 per linear foot (20 foot embankment height)</li> <li>➤ \$287 per linear foot (10 foot embankment height)</li> </ul> Additional costs for: <ul style="list-style-type: none"> <li>➤ ROW</li> <li>➤ Canal culvert inlets and outlets extensions</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$202 per linear foot</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$266 per linear foot</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$226 per linear foot</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$895 per linear foot</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$110 per linear foot (Use only where height of embankment raise is 8 inches or less)</li> </ul>	<ul style="list-style-type: none"> <li>➤ \$219 per linear foot</li> </ul>

Notes:

1. Cost estimate is Class 4 estimate as defined in AACE, 2005 (-20% to +35%). 30% contingency is included. Estimate is for a raise to one side of the canal (w/primary O&M road). Estimate is based on 2-foot embankment (or unlined freeboard) raise and 1.5 foot canal lining raise, unless noted otherwise). Embankment fill haul distance is 15 miles.

## 7.1 GENERAL

The structure of the cost estimates for each scenario are similar to the estimate found in Table 13-1 “Contractor Payment, Full Enlargement 2876 cfs, Alternative 1” of the DWR 2004 report. Costs for the Alamo Powerplant and Cottonwood Chute Bypass are not within the scope of the study and are not included. Costs for the Pearblossom Pumping Plant are included and are the same for each scenario. Costs for Pools 43 to 58 (pools upstream of the Pearblossom Pumping Plant) and Pools 59 to 66 (pools downstream of the Pearblossom Pumping Plant) are combined.

The general approach was to evaluate scenarios of combinations of improvements to increase canal capacity to 2,876 cfs. Various combinations of improvements were evaluated using the calibrated HEC-RAS hydraulic model (see Section 4.0) to check the effect that various improvements would have on canal water surface elevations.

Three scenarios were considered:

- Scenario 1: DWR 2004 Report Conditions, updated to reflect current costs, for comparison purposes.
- Scenario 2: Canal Raise Alternative.
- Scenario 3: Smooth Siphon Alternative.

This study assumes that for all scenarios, the south bank of the East Branch Aqueduct, being generally on the uphill side, will be raised with an earth embankment.

## 7.2 SCENARIO 1: DWR 2004 REPORT CONDITIONS

Embankment quantities were based on DWR (2004) for the left and right banks of each pool. The height of the embankment along the canal varies and detailed information was not available for use in this report. Further refinement will be required to improve quantity estimates and to determine the most cost effective arrangement of alternatives<sup>2</sup>.

The recommendations given in the 2004 DWR report were used as the basis of comparison with the scenarios provided in this report. The 2004 report included the following improvements:

- Raise the lining to provide the minimum two feet of lined freeboard using the earthfill embankment alternative described herein.
- Raise the embankments and service roads to provide for a minimum of four feet total lined and unlined freeboard.
- Raise the bridges, overchutes, and pipelines to provide adequate clearance between the overcrossings and the water surface elevation.
- Raise the walls around the platforms and radial gates of the existing check structures.
- Provide an additional bay to all two bay check structures between Check 43 and Check 58 and provide bypass for flows during construction.

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<sup>2</sup> DWR is currently working on a digital terrain model of the canal that would be used for further evaluations.

- Provide an additional bay and siphon barrel to all siphons, except for Big Rock Siphon.
- Remove raised sills from all check structures that are being modified and modify radial gates, hoists and electrical work.
- Install two 375 cfs pump units with appurtenances at Pearblossom Pumping Plant, and construct one discharge pipeline and air chamber.

The updated Base Case water surface elevation (Section 4.3.1) as calculated by the hydraulic model (see Section 4) resulted in additional earthwork embankment and bridges to be replaced compared to that assumed in the 2004 DWR report.

### 7.3 SCENARIO 2: CANAL RAISE ALTERNATIVE

Scenario 2 includes the improvements of Scenario 1 but includes additional bays at check structures (total of 4 bays per check structure for a total of 23 check bays) and substitutes alternative canal raise components into the alignment in lieu of raising the embankment. The substitution is done in a manner that takes advantage of each component cost advantages in order to minimize the overall cost of the project.

Areas where the embankment is in a fault crossing zone or which have more than two feet of raise are to be raised with an earth embankment.

The water surface elevation used is Base Case Plus Added Check Bays (Section 4.3.3). Scenario 2 includes the following canal raise alternative components:

- |                                   |   |
|-----------------------------------|---|
| • Mile Post (MP) 305.75 to 323.84 | Earth Embankment                                |
| • MP 323.84 to 324.06             | Myrick Siphon                                   |
| • MP 324.06 to 326.77             | Earth Embankment                                |
| • MP 326.77 to 326.95             | Willow Siphon                                   |
| • MP 326.95 to 330.82             | Earth Embankment                                |
| • MP 330.82 to 330.97             | Johnson Siphon                                  |
| • MP 330.97 to 348.18             | Earth Embankment                                |
| • MP 348.18 to 348.48             | Soledad Siphon                                  |
| • MP 348.48 to 352.71             | Earth Embankment                                |
| • MP 352.71 to 352.90             | Cheseboro Siphon                                |
| • MP 352.90 to 354.76             | Earthen Embankment                              |
| • MP 354.76 to 354.95             | Littlerock Siphon                               |
| • MP 354.95 to 360.54             | Earth Embankment                                |
| • MP 360.54 to 361.89             | Pearblossom Plant                               |
| • MP 361.89 to 363.52             | Earth Embankment                                |
| • MP 363.52 to 363.66             | Tejon Siphon                                    |
| • MP 363.66 to 367.00             | Earth Embankment                                |
| • MP 367.00 to 393.96             | Precast Panel Wall                              |
| • MP 393.96 to 398.05             | Slip Form Wall                                  |
| • MP 398.05 to 400.32             | No Improvement Necessary (Sufficient Freeboard) |
| • MP 400.32 to 401.04             | Antelope Siphon                                 |
| • MP 401.04 to 403.41             | Precast Panel Wall                              |

The precast panel wall was selected as the lowest cost solution to areas where the existing embankment is above the proposed top of lining. The slip form wall was selected where the embankment raise required is less than 8 inches.

Figure 7-1 shows the raise requirements and applied alternatives along the alignment. This figure also shows the seismic zones along the alignment.

#### 7.4 SCENARIO 3: SMOOTH SIPHON ALTERNATIVE

Scenario 3 includes the improvements from Scenario 2 but adds application of a smooth polyurethane or epoxy coating to the inside of the siphons. This coating results in lower water surface elevation, less embankment costs and in some areas, the ability to construct alternatives other than the earth embankment. For instance, the canal between MP 317 to MP 339 requires a raise of over 2 feet and, without the smooth siphons, an earth embankment would be required as in Scenario 2. However, with the smooth siphons, a lower water surface elevation would result, and this canal section would be under the 2-foot raise limit for the non-earth embankment alternatives.

The precast panel wall (see Section 5.1.7) was selected as the lowest cost solution in areas where the existing embankment is above the proposed top of lining. The slip form wall was selected where the embankment raise required is less than 8 inches.

The water surface elevation used is Base Case Plus Added Check Bays and Smooth Siphons (Section 4.3.4). Scenario 3 includes the following canal raise alternative components:

- MP 305.75 to 323.84      Precast Panel Wall
- MP 323.84 to 324.06      Myrick Siphon
- MP 324.06 to 326.77      Earth Embankment
- MP 326.77 to 326.95      Willow Siphon
- MP 326.95 to 330.82      Earth Embankment
- MP 330.82 to 330.97      Johnson Siphon
- MP 330.97 to 336.40      Earth Embankment
- MP 336.40 to 337.33      Precast Panel Wall
- MP 337.33 to 337.75      Earth Embankment
- MP 337.75 to 341.45      Precast Panel Wall
- MP 341.45 to 348.18      Earth Embankment
- MP 348.18 to 348.48      Soledad Siphon
- MP 348.48 to 352.71      Earth Embankment
- MP 352.71 to 352.90      Cheseboro Siphon
- MP 352.90 to 354.76      Earthen Embankment
- MP 354.76 to 354.95      Littlerock Siphon
- MP 354.95 to 360.54      Earth Embankment
- MP 360.54 to 361.89      Pearblossom Plant
- MP 361.89 to 363.52      Precast Panel Wall
- MP 363.52 to 363.66      Tejon Siphon
- MP 363.66 to 393.13      Precast Panel Wall
- MP 393.13 to 396.56      Slip Form Wall

- MP 396.56 to 400.32 No Improvement Necessary (Sufficient Freeboard)
- MP 400.32 to 401.04 Antelope Siphon
- MP 401.04 to 403.41 Precast Panel Wall

Figure 7-2 shows the freeboard and applied alternatives along the alignment. This figure also shows the seismic zones along the alignment.

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## 8.1 GENERAL

Costs for a facility can be evaluated based on capital costs and life cycle costs. Construction and contingency costs make up a significant portion of the capital costs. Costs for engineering, construction management, administration, legal, and other “soft costs” are also part of the capital cost and are generally estimated as a percentage of the construction cost. These “soft costs” are not included in this report and will need to be added based on DWR’s standard percentages. Life cycle costs are estimated by annualizing capital costs based on the estimated life of project elements and the current Federal Water Resources discount rate and adding annual operation and maintenance (O&M) costs. The estimation of capital costs and life cycle costs are discussed in the following sections.

## 8.2 CAPITAL COST

### 8.2.1 Construction Cost

The construction cost estimate is a Class 4 estimate as described by the Association for the Advancement of Cost Engineering (AACE, 2005) as follows:

*“Class 4 estimates are generally prepared based on limited information and subsequently have fairly wide accuracy ranges. They are typically used for project screening, determination of feasibility, conceptual evaluation, and preliminary budget approval. Typically, engineering is from 1% to 15% complete,....”*

Typical accuracy ranges for Class 4 estimates are -15% to -30% on the low side, and +20% to +50% on the high side (AACE, 2005).

The major elements are defined based on the conceptual designs presented in Section 5. Cost back-up of these elements is included in Appendix E.

Various measures to increase capacity will result in the need to acquire additional property and/or rights-of-way (ROW). ROW costs will be provided by DWR. Where ROW acquisition is an obvious high cost (such as encroachment into built-out areas), measures to minimize the need to acquire additional ROW are considered (refer to Section 5.1).

The EIR will not be available for the Phase II Feasibility Study. Therefore, this feasibility study does not address environmental impacts. Environmental mitigation costs will need to be addressed separately, after completion of the EIR.

### 8.2.2 Unit Costs

Unit costs are summarized in Appendix E. Unit costs were obtained from historical databases for similar projects. Vendor quotes on specialty items, such as epoxy or polyurethane coatings for application in siphons, were obtained.

Pricing in this feasibility report was estimated in third quarter 2007 dollars. Escalation indices are based on U.S. Bureau of Reclamation (USBR) Construction Cost Trends (CCT) for canals

([http://www.usbr.gov/pmts/estimate/-cost\\_trend.html](http://www.usbr.gov/pmts/estimate/-cost_trend.html); see Appendix E). Costs used from the DWR Report (2004) are shown as cost period January 2001.

Unit cost assumptions are listed below:

- Import fill assumed a 15-mile one-way haul. For canal pools with stockpiles of fill from the original construction, this distance was reduced to 2 miles.
- Asphalt concrete (AC) thickness is 3 inches. AC unit weight = 2.0 tons/cy.
- Aggregate base (AB) thickness is 5 inches. AB unit weight = 1.8 tons/cy.
- Unlisted items are based on 5% of direct cost totals.
- General conditions calculated at 10% of direct costs.
- Exterior embankment slopes are based on 2H:1V.
- Rebar is non-epoxy coated.
- Hydromulch seeding assumes truck access and no post watering.
- Edge forming excluded for RCC.
- Pricing assumes one contractor mobilization per contract.
- Pricing assumes competitive market conditions.

### 8.2.3 Contingency

A conceptual design has unlisted items, quantities, requirements and constraints that have not been fully identified, or else are not fully investigated or designed. In later stages of design, the scope of the project also tends to expand as more detail is developed and as regulatory agencies undertake more detailed reviews.

A design contingency is, therefore, incorporated into and becomes an integral part of the estimated construction cost to accommodate those features of the work that cannot be adequately assessed due to the partially developed design. The amount of contingency reflects both the degree of risk associated with uncertainties, particularly with respect to geotechnical conditions, and the completeness of the design detail for the major elements of work. The design contingency is based on, and added to, the subtotal of construction costs because it represents an unknown portion of the total estimated construction cost. The contingency will decrease as the project moves forward into final design as more information becomes available, project requirements become better defined, and more of the design detail is captured in the subtotal of construction costs.

During preparation of this estimate, the potential risks associated with several major constructability issues were considered that include surface and groundwater management, borrow area development, haul distances, and operation of the canal during construction. A 20 percent design contingency was used for this conceptual-level estimate. This percentage is in accordance with DWR's contingency allowance for their 2004 East Branch Aqueduct Enlargement Study (DWR, 2004).



The 20 percent contingency does not include construction growth contingency that should be planned to pay for the cost of owner-directed changes after the project is under contract, changed conditions that occur or are encountered during construction, and other unforeseen conditions or changes. Construction growth costs are often associated with unexpected, or variations in expected, subsurface conditions.

### 8.3 LIFE CYCLE COSTS

A present worth analysis was performed to estimate life cycle costs. An amortized cost over the estimated lifespan of the project is provided as an indication of annual budget requirements. A 50-year project life was assumed for present worth analysis. The discount rate for Federal Water Projects for 2007 of 4.875 percent was used in the calculation of present value (see <http://www.economics.nrcs.usda.gov/cost/priceindexes /rates.html>).

The analysis was undertaken using discounted cash flow analysis. The process of discounting enables the direct comparison of amounts of money that accrue in different time periods. Discounting gives greater weight to initial costs and benefits, and less weight to those in the future.

Life cycle costs were estimated for each component, an annual cost was estimated for that component, and then all the component costs were summed to obtain a total annualized cost. This total annualized cost was converted to a present value using a 50-year project life and a discount rate of 4.875 percent. In estimating life cycle costs, O&M costs are not included in the scenarios.

### 8.4 SUMMARY OF SCENARIO COSTS

The estimated construction costs (including contingency but no “soft costs”), in third quarter 2007 dollars, and present values for the three scenarios are summarized in Table 8-1.

Although Scenario 3 - Smooth Siphon Alternative could have a slightly lower initial cost than the other two scenarios, this scenario would require periodic reapplication of the polyurethane or epoxy coating. This product has only been in use since about 1993, so it does not have an extensive service record for this product. The estimated interval for reapplication could be between 10 and 30 years; 15 years was assumed for the life cycle analysis. This reapplication would need to be done during periods when the canal can operate at lower flow rates (mid-October to mid-February).

The cost of Scenario 1 would be greater than shown above with the addition of right-of-way costs. Consideration of right-of-way costs may indicate that significant cost savings could be achieved by using precast panel walls, or similar walls (Scenario 2), instead of a full canal embankment raise (Scenario 1). Full canal embankment raises are not expected to be used in areas where developments have encroached on the East Branch canal embankments. Embankment raises are the only option acceptable where the canal crosses active fault zones. Further engineering and cost studies will need to be undertaken to confirm the most cost-effective canal raise system on a specific location basis.

To evaluate the cost tradeoff of using 16 check bays instead of 23 used for Scenario 2, but increasing the canal crest elevation, the cost of this variant of Scenario 2 was estimated. The

total cost was found to be about \$362 million, which is nearly the same as for Scenario 2 (see Table 8-1).

The costs for Scenario 1 are greater than the costs indicated for Alternative 1 in the DWR 2004 report for the following main reasons:

- Unit costs in the DWR 2004 report were escalated to 2007 using USBR Construction Cost Trends.
- Updated embankment quantities from DWR are approximately 20 percent greater than indicated in the DWR 2004 report.
- Water surface elevations from the calibrated HEC-RAS model are higher than the DWR water surface elevations, leading to greater canal raise requirements and cost.
- The primary road cross section was increased per the recommendation of DWR Maintenance to 3-inches AC over 5-inches AB from the original 2-inches AC over 4-inches AB.
- Some bridges with insufficient freeboard may have integral reinforcement between the decking, piers/abutments, precluding a raise and requiring a bridge replacement. Bridges to be raised will need to meet current seismic requirements. In the absence of bridge design data, it was assumed that 50 percent of the bridges would be replaced and 50 percent would be raised. These factors increase the bridge cost from the original assumptions.
- Mobilization and demobilization costs were increased from less than 1 percent in the DWR 2004 report to 5 percent.

**Table 8-1  
Summary of Scenario Costs**

Revised 4/29/08

Item	Unit	2007 Unit Cost (a)	Estimated Lifecycle	Scenario 1 DWR 2004 Report Conditions (Base Case Water Surface Elevations)			Scenario 2 Canal Raise Alternative			Scenario 3 Smooth Siphon Alternative		
				Quantity (d)	2007 Construction Cost (a*d)	Annualized Cost with Contingency	Quantity (e)	Cost (a*e)	Annualized Cost with Contingency	Quantity (f)	Cost (a*f)	Annualized Cost with Contingency
<b>B &amp; D Canal</b>												
1 Mobilization and Demobilization <sup>4</sup>	EA	5%	50	1	12,774,000	823,498	1	12,426,104	801,070	1	11,801,550	760,807
2 Raise Embankment <sup>3</sup>	CY	23	100	4,198,686	96,569,767	5,698,144	3,540,274	81,426,291	4,804,597	2,304,919	53,013,128	3,128,064
3 Compacted Embankment	CY	33	100	292,008	9,636,269	568,593	246,217	8,125,168	479,429	160,301	5,289,945	312,136
4 Raise Concrete Lining	CY	400	50	37,397	14,958,640	964,335	33,485	13,393,804	863,455	26,597	10,638,945	685,858
6.5 Remove and Replace Primary Road	FT	60	15	485,496	28,918,929	3,315,150	309,038	18,408,101	2,110,231	167,746	9,991,891	1,145,430
7 Add One Bay Check Structures <sup>1</sup>	EA	908,072	50	16	14,529,147	936,647	23	20,885,649	1,346,430	23	20,885,649	1,346,430
8 Add Single Barrel Siphon <sup>1</sup>	EA	3,178,492	50	8	25,427,935	1,639,256	8	25,427,935	1,639,256	8	25,427,935	1,639,256
8.1 Add Single Barrel Siphon (Tejon)	EA	2,022,677	50	1	2,022,677	130,395	1	2,022,677	130,395	1	2,022,677	130,395
8.2 Add Single Barrel Siphon (Antelope)	EA	13,002,921	50	1	13,002,921	838,256	1	13,002,921	838,256	1	13,002,921	838,256
9 Add Three R.C. Box Siphon <sup>1</sup>	LF	3,756	50	555	2,084,802	134,400	555	2,084,802	134,400	555	2,084,802	134,400
10 New Radial Gates and Radial Gate Hoists <sup>1</sup>	EA	211,883	25	16	3,390,134	285,040	23	4,873,318	409,746	23	4,873,318	409,746
11 Modify Existing Radial Gate and Check <sup>1</sup>	EA	15,135	50	41	620,516	40,003	41	620,516	40,003	41	620,516	40,003
12 Remove Raised Concrete Sill at Check <sup>1</sup>	EA	12,108	50	54	653,812	42,149	54	653,812	42,149	54	653,812	42,149
13 Modify Existing Radial Gate Hoist and Electrical <sup>1</sup>	EA	75,673	25	41	3,102,578	260,863	41	3,102,578	260,863	41	3,102,578	260,863
14 Bridges <sup>2</sup>	EA	655,876	75	33	21,643,908	1,302,854	31	20,332,156	1,223,894	20	13,117,520	789,609
15 Overchutes <sup>1</sup>	EA	20,000	50	71	1,420,000	91,543	71	1,420,000	91,543	67	1,340,000	86,385
16 Raise Pipelines <sup>1</sup>	EA	126,450	50	12	1,517,405	97,822	12	1,517,405	97,822	12	1,517,405	97,822
17 Raise 121" Steel Pipeline <sup>1</sup>	LS	224,801	50	1	224,801	14,492	1	224,801	14,492	1	224,801	14,492
18 Extend Culvert Inlets and Outlets <sup>1</sup>	EA	121,076	30	106	12,834,080	987,620	67	8,169,426	628,662	37	4,434,353	341,237
19 Hydromulching <sup>1</sup>	AC	9,178	20	100	917,803	87,442	64	584,220	55,660	35	317,114	30,212
20 Traffic Control and Detour <sup>1</sup>	LS	2,003,869	50	1	2,003,869	129,183	1	2,003,869	129,183	1	2,003,869	129,183
21 Slip Form Wall	LF	84	50	0	0	0	21,595	1,813,997	116,942	18,110	1,521,274	98,072
23 Precast Panel System	LF	119	30	0	0	0	154,862	18,428,626	1,418,137	291,773	34,720,963	2,671,881
24 Smooth Coating for Siphons	SF	14	15	0	0	0	0	0	0	1,801,827	25,225,584	2,891,760
<b>C Pearblossom Pumping Plant</b>												
1 Furnish and install pump units <sup>1</sup>	EA	6,276,793	25	2	12,553,585	1,055,498	2	12,553,585	1,055,498	2	12,553,585	1,055,498
2 Furnish and install motors <sup>1</sup>	EA	5,803,598	25	2	11,607,195	975,926	2	11,607,195	975,926	2	11,607,195	975,926
3 Furnish and install valves <sup>1</sup>	EA	2,045,589	50	2	4,091,179	263,745	2	4,091,179	263,745	2	4,091,179	263,745
4 Install 11'-0" discharge line <sup>1</sup>	JOB	13,161,846	50	1	13,161,846	848,501	1	13,161,846	848,501	1	13,161,846	848,501
<b>Discount Rate: 4.875%</b>				Subtotal:	\$309,667,797	\$21,531,356	Subtotal:	\$302,361,980	\$20,820,285	Subtotal:	\$289,246,353	\$21,168,116
<b>Contingency: 20%</b>				20% Con-	\$61,933,559		20% Con-	\$60,472,396		20% Con-	\$57,849,271	
<b>Project Lifecycle (Years): 50</b>				<b>Total:</b>	<b>\$371,601,356</b>		<b>Total:</b>	<b>\$362,834,375</b>		<b>Total:</b>	<b>\$347,095,623</b>	
				<b>Present Value:</b>	<b>\$400,000,000</b>		<b>Present Value:</b>	<b>\$390,000,000</b>		<b>Present Value:</b>	<b>\$390,000,000</b>	

**Notes:**

- 1 Unit Cost is escalated from the DWR East Branch Enlargement Report Costs for 2001.
- 2 Bridge cost is the average between the cost of replacing and raising the bridge.
- 3 Updated embankment quantity from DWR
- 4 Mobilization and Demobilization cost excludes C Pearblossom Pumping Plant.
- 5 Design, Environmental and Right of Way costs are not included

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## 9.1 CONSTRUCTABILITY CONSIDERATIONS

Constructability considerations include the following:

- Seasonal operation of the canal will need to be considered. Construction may be scheduled when flow requirements downstream are less critical. Shutdown of turnouts along the canal may be limited to very short periods, or alternative water supplies may need to be provided. Coordination with DWR Operations staff during design and construction will be extremely important.
- Acceptable earthfill sources for canal embankment construction will need to be located.
- During construction on the embankments most work will likely take place while the canal remains at near full or full capacity. Constructability considerations for the different canal raise alternatives are presented in Table 6-2.
- Construction at the existing check structures presents a significant challenge for the work. If it is decided to modify the existing transitions, the cofferdams needed to construct the transitions while maintaining flow capacity will most likely be expensive and complicated. The cofferdams would need to span the length of the transitions being demolished and reconstructed and must be adequately sealed at its connection points.
- Construction access to the canal embankments is a consideration, particularly in areas where developments encroach on the canal.
- If it is decided to coat the interior of the siphons, the remaining siphons may remain operational while each siphon is coated in turn. The economic impacts of the reduced flows during construction are not included in this analysis.

## 9.2 OPERATIONS DURING CONSTRUCTION

Due to the importance of maintaining the water supply in the East Branch Aqueduct, a full shutdown of the facility for an extended period of time is not feasible. It is assumed that most work involved in expanding the capacity of the canal would be done at its current full flow capacity. The canal raise portion of the work will not present much impact to operations during construction. From an operations standpoint, the canal lining raise could likely be scheduled to occur anytime during the year. Similarly, although adding to the complexity, the overchutes can be anchored and bridges raised with a full canal.

The check/siphon modifications will present the majority of the scheduling challenges for the work. Modifications to checks and connecting new siphons will necessitate a reduction of flow. For instance, at a two-bay check, one bay will likely need to be closed while the sill is being raised. Multiple crews and shifts will likely be necessary to minimize the period of time that flow is reduced. Generally, this period would be from mid-October to mid-February.

## 9.3 CONSTRUCTION SCHEDULES

The need to have early additional incremental capacity was discussed at the February 2005 workshop. It was agreed at this workshop that the East Branch Enlargement should be built in one stage to a capacity of 2,876 cfs, unless there are some modifications that could be done early,

for example, in vulnerable areas (URS/MWH, 2005). There do not appear to be advantages to building the enlargement incrementally.

Construction of embankments and linings along the embankments should be able to proceed at the same time as the construction of the checks and siphons provided allowance is made for the transportation of materials between pools. Application of coatings to the interior of siphons can be done at the same time the check structures are taken out of service for improvements. Modifications to bridges and overchutes may also be done independently of other construction.

The scenario schedules presented on Figures 9-1, 9-2, and 9-3 are at a conceptual level. Assumptions used to prepare the scenario schedules include:

- DWR report (2004) durations and start date relationships were used as a starting point to prepare the scenario schedules.
- For the scenarios, the DWR durations were prorated based on quantity changes, except for anchoring overchutes, which would be faster than raising the structures.
- Polyurethane or epoxy coatings can be applied to siphon barrels at the rate of 100 linear feet per day per crew (based on vendor quote).
- Precast panel walls could be built at the rate of about 320 linear feet per day per crew (based on vendor quote).
- Most improvements would be done concurrently. This may entail simultaneous contracts for added checks, check gate modifications, embankments, siphons, etc.
- Multiple contracts will be issued for different segments of the work.
- Work requiring reduced flows will occur during periods of low demands (mid-October through mid-February).
- Manpower, materials, and equipment are available in sufficient quantities to meet the needs of the schedule.
- All regulatory, design and right-of-way requirements are met in a timely fashion.

The overall construction duration for the three scenarios is about 2500 days (6.8 years). Currently, modifications to Pearblossom Pumping Station define the end of construction. It may be possible to shorten the completion date by starting work at Pearblossom ahead of the planned schedule.

## 10.1 CONCLUSIONS

This report presents the results of the URS/MWH cost analysis studies and provides a basis for further project design development. The report focuses on recommending the most economical combination of canal raise and hydraulic structure (including check structures and siphons) improvements to accommodate increasing flow in the East Branch of the California Aqueduct from the current (Phase I) 2,010 cfs to 2,876 cfs (Phase II) and to explore innovative designs to reduce cost. This report supplements DWR's East Branch Aqueduct Enlargement Study (DWR, 2004).

The canal capacity will need to be increased, in part, by raising the canal banks. The amount of the raise required will vary depending upon the hydraulic operating scenarios ultimately selected and the height of existing canal lining and embankment. The following seven alternatives were evaluated to raise the canal embankments:

- Alternative 1: Earthfill embankment (similar to DWR's 2004 study)
- Alternative 2: Roller compacted concrete (RCC)
- Alternative 3: Spread footing wall
- Alternative 4: Concrete parapet
- Alternative 5: Earthfill embankment with retaining wall
- Alternative 6: Slipform wall
- Alternative 7: Precast panel wall

These alternatives were evaluated based on criteria that included implementability, operational flexibility, maintainability and reliability, and cost.

The earthfill embankment raise (Alternative 1) may be used where DWR already has right-of-way or in undeveloped areas where additional right-of-way can be obtained (i.e., developments do not encroach on the canal embankments). To improve seismic reliability, the earthfill embankment is also the only canal raise alternative that may be used where the canal traverses an active fault zone. Where developments encroach on the canal, Alternatives 2 through 7 were considered. For canal raise requirements greater than 8 inches, a precast panel system (Alternative 7) was found to have the lowest cost. Where the canal raise is less than 8 inches, a slip form wall or vertical curb was found to be more cost effective.

The enlargement of the East Branch Aqueduct will require modifications to the 11 siphons and 15 check structures upstream of Pearblossom Pumping Plant. Downstream of Pearblossom Pumping Plant, modifications to the Tejon siphon and the Antelope check structure and siphon are needed. This evaluation included adding two bays to existing two bay check structures and a single bay added to three bay check structures so that all check structures will have four bays. A single barrel will be added to the thirteen siphons.

Modifying the check structures while minimizing impacts to existing operations presents one of the greatest challenges to the East Branch Aqueduct Enlargement. In addition to adding a bay at each check location, the modification will include adding a siphon barrel at these check locations

with siphons and modifying the existing radial gates and inlet and outlet transitions to accommodate the raised canal crest.

To increase canal capacity, overchutes can be handled in two ways. They can be raised to meet the desired freeboard or anchored in their existing position. However, overchutes cannot be raised without also raising the upstream flow transitions. In most cases, this would result in significant grading requirements at the upstream end of the overchute. Accordingly, anchoring overchutes in place was found to be more economical than raising them. As such, the anchorage concept was adopted for this feasibility study. Bridges will need to be raised to a minimum clearance of 1-foot above the maximum water surface elevation to meet CALTRANS requirements.

The canal raise portion of the work will not present much impact to operations during construction. From an operations standpoint, the canal lining raise could most likely be scheduled to occur anytime during the year. The check/siphon modifications will present the majority of the scheduling challenges for the work. Due to the importance of maintaining the water supply in the East Branch Aqueduct, a full shutdown of the facility for an extended period of time is not feasible. It is assumed that most work involved in expanding the capacity of the canal would be done at its current full flow capacity. This requirement may add considerable difficulty to the expansion of checks and raising of bridges. However, modifications to checks and siphons will likely necessitate a reduction of flow. This reduction in flow will depend on the number of similar structures that can remain in operation during construction. Multiple crews and shifts may be necessary to minimize the period of time flow is reduced (mid-October to mid-February).

For scenario development, the general approach was to evaluate combinations of improvements to increase canal capacity to 2,876 cfs. Various combinations of improvements were evaluated using the calibrated HEC-RAS hydraulic model to check the effect that various improvements would have on canal water surface elevations. Three scenarios were considered:

- Scenario 1: DWR 2004 Report Conditions, updated to reflect current costs, for comparison purposes; includes 16 check bays.
- Scenario 2: Canal Raise Alternative – includes precast panel wall (in lieu of full embankment raise) in portions of the canal; includes 23 check bays.
- Scenario 3: Smooth Siphon Alternative - includes application of a smooth polyurethane or epoxy coating to the inside of all the siphons to the structural improvements of Scenario 2 to reduce the height of canal raise.

The estimated construction costs (including 20 percent contingency but no “soft costs”), in third quarter 2007 dollars, and present values for the three scenarios are summarized below:

Cost	Scenario 1 – DWR 2004 Report – Updated	Scenario 2 – Canal Raise Alternative	Scenario 3 – Smooth Siphon Alternative
Construction Cost	\$372 million	\$363 million	\$347 million
Present Value	\$400 million	\$390 million	\$390 million



As shown in the above summary, the estimated construction costs of the three scenarios are similar (costs are within 7 percent of each other). Although Scenario 3 - Smooth Siphon Alternative could have a slightly lower initial cost than the other two scenarios, this scenario would require periodic reapplication of the polyurethane or epoxy coating. This product has only been in use since about 1993, so it does not have an extensive service record for this product. The estimated interval for reapplication could be between 10 and 30 years; 15 years was assumed for the life cycle analysis. This reapplication would need to be done during periods when the canal can operate at lower flow rates (mid-October to mid-February). Clearly, there are performance risks associated with this scenario that must be quantified if siphon coatings are to be further considered. At this time, there does not appear to be a clear benefit for this scenario.

The cost of Scenario 1 would be greater than shown above with the addition of right-of-way costs. It appears that significant cost savings could be achieved by using precast panel walls, or similar walls (Scenario 2), instead of a full canal embankment raise (Scenario 1). Full canal embankment raises can only be used in areas where developments have not encroached on the East Branch canal embankments and where the canal crosses active fault zones. Further engineering and cost studies will need to be undertaken to confirm the most cost-effective canal raise system on a specific location basis.

To evaluate the cost tradeoff of using 16 check bays instead of 23 used for Scenario 2, but increasing the canal crest elevation, the cost of this variant of Scenario 2 was estimated. The total cost was found to be about \$362 million, which is nearly the same as for Scenario 2. Thus, no significant cost benefit was realized for this variant. Furthermore, for this variant, there would also be less operational flexibility than for Scenario 2 with the 23 check bays.

The overall construction duration for the three scenarios is estimated to be about 2500 days (6.8 years). Currently, modifications to Pearblossom Pumping Plant define the end of construction. It may be possible that work at Pearblossom could be initiated earlier than originally planned to shorten the overall project schedule.

## 10.2 RECOMMENDATIONS FOR FURTHER STUDIES

Further studies and engineering are needed to develop the concepts explored in this report for the Phase II East Branch Enlargement. Recommendations include:

- Conduct alternative analysis to optimize size of additional siphon barrels.
- Prepare alternative analyses on a site specific basis for construction of new check structures separated from existing check structures.
- Establish order of priorities for structures and related canal lining raises. Priority could be given to areas where capacity can be increased most cost effectively. For instance, the canal upstream of Pearblossom could be improved to provide increased capacity early in the East Branch Enlargement.
- Complete digital terrain model of the canal so that the cost of the earthfill embankment canal raise alternative can be estimated more accurately and compared to other alternatives.
- Complete right-of-way mapping for the canal and incorporate the cost of right-of-way into the overall scenario costs.

- Perform slope stability analyses of the canal embankments for the enlarged aqueduct capacity using properties of the embankment materials that are based on laboratory testing data.
- Confirm locations of earthfill borrow materials for use in canal embankments.
- Perform structural analyses of the culverts, radial gates and other features for the increased loading from the enlarged aqueduct.
- Perform hydraulic analyses to assess the performance of the transitions to the enlarged check structures.

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URS/MWH represents that the services were conducted in a manner consistent with the standard of care ordinarily applied as the state of practice in the profession within the limits prescribed by our client. No other warranties, either expressed or implied, are included or intended in this report.

The following lists the main limitations and data gaps of this feasibility study:

- Topographic mapping was not available to use for estimating earthwork quantities.
- Elevations of existing canal lining and embankments were provided by DWR.
- Limits of right of way were not available for this feasibility study.
- Right-of-way costs and environmental mitigation costs are not included.
- Canal raise alternatives included feasibility level structural analyses. Detailed structural analyses were not conducted.
- Costs for engineering, construction management, administration, legal, and other “soft costs” are not included in this report.

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**Figures**



Ref: DWR, 1999

SCALE: NTS

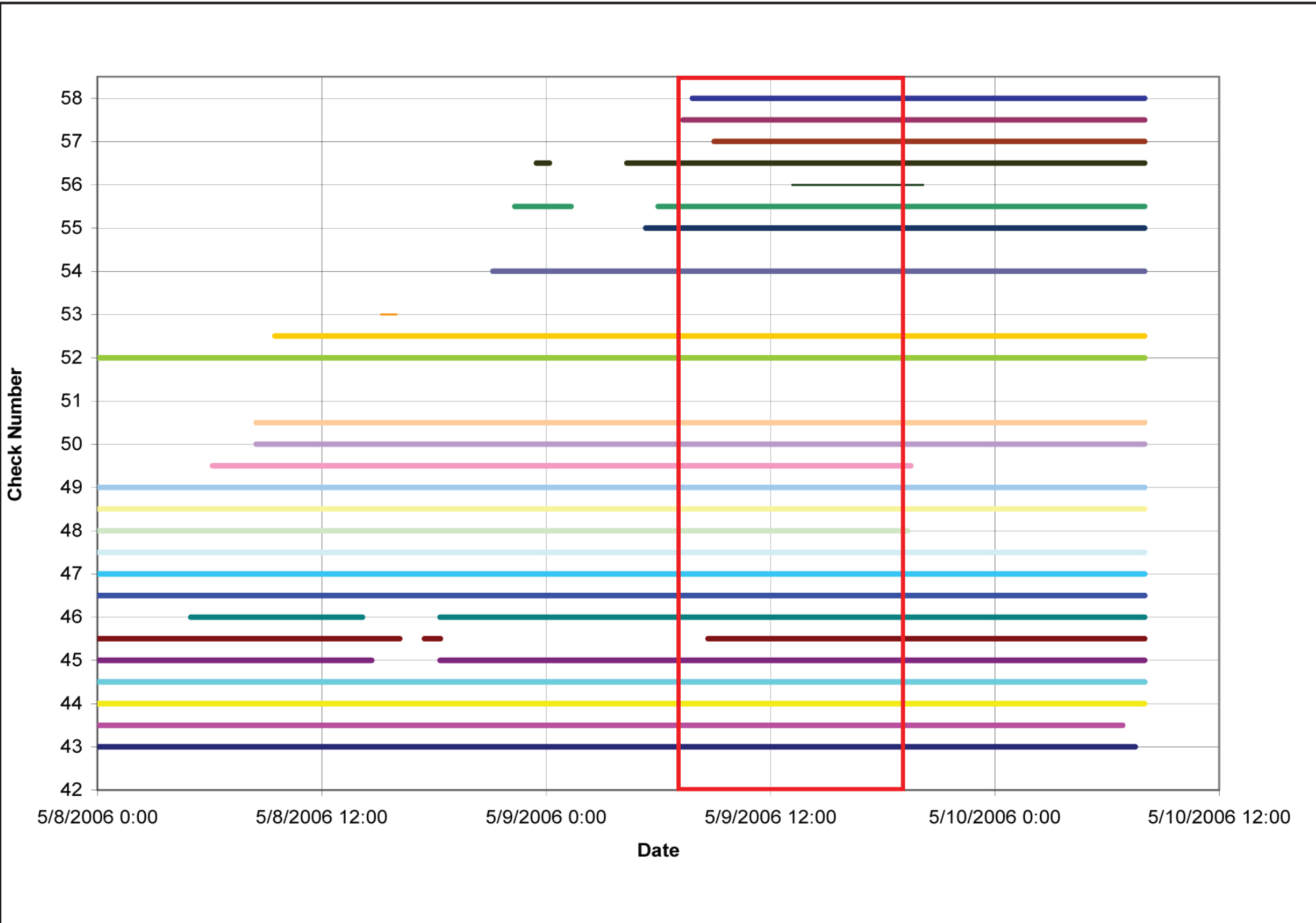


PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

LOCATION MAP

Figure  
1-1



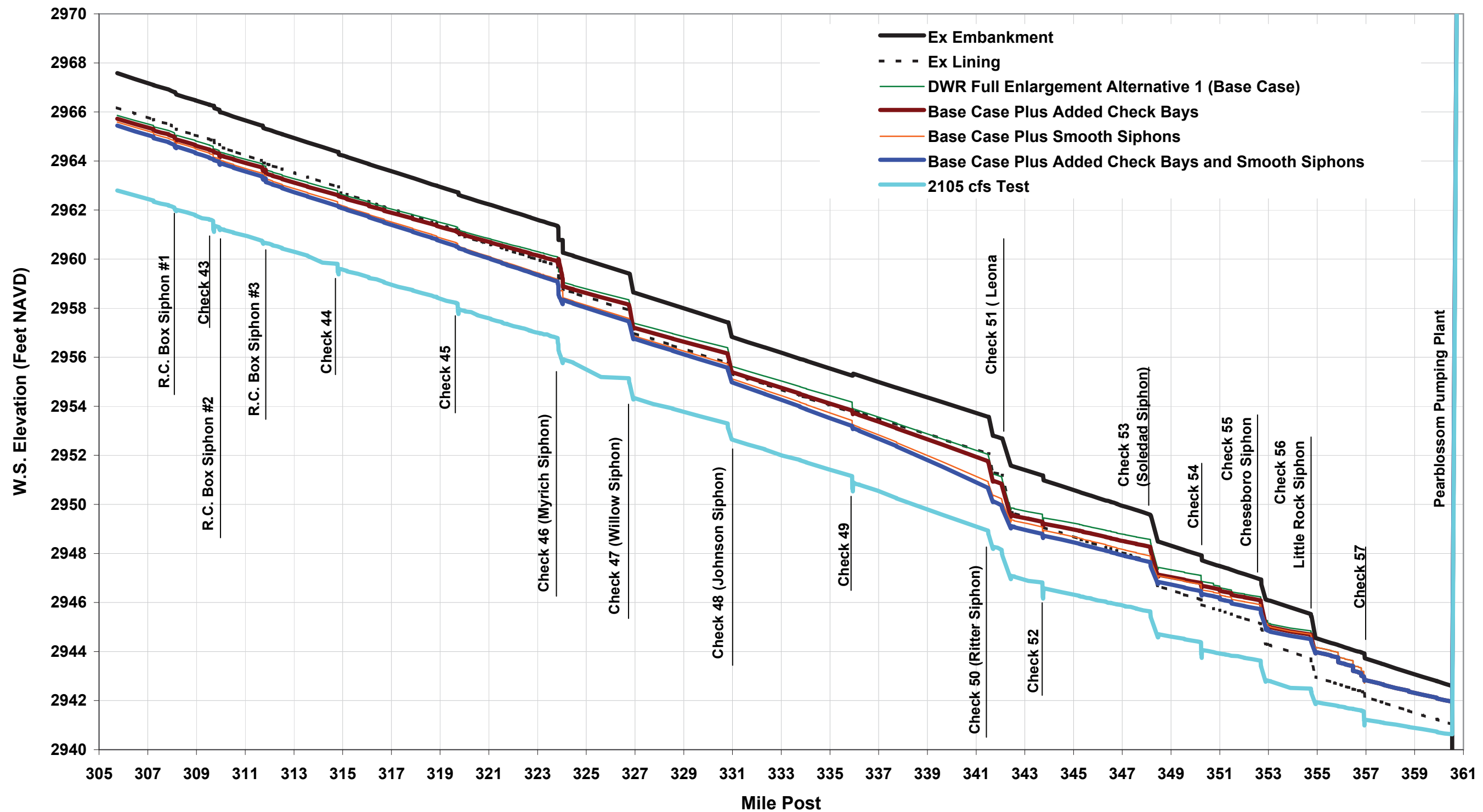
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PROJECT NO. 26814903  
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 EAST BRANCH ENLARGEMENT PROJECT

FLOW TEST,  
 0.2 FOOT STEADY  
 STATE BAND

Figure  
 4-1



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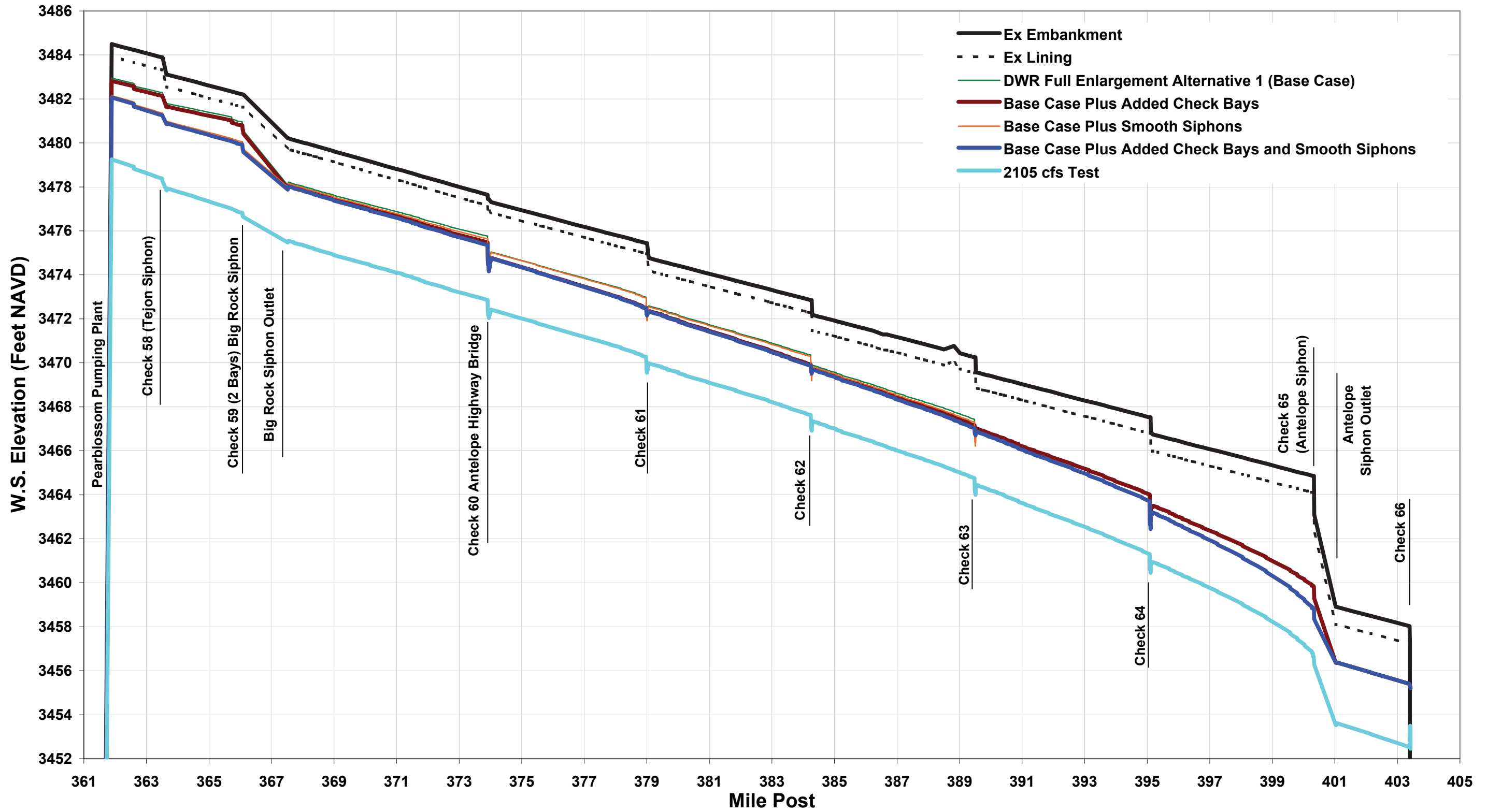


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 EAST BRANCH ENLARGEMENT PROJECT

WATER SURFACE  
 ELEVATIONS -  
 POOLS 43-58

Figure  
 4-2





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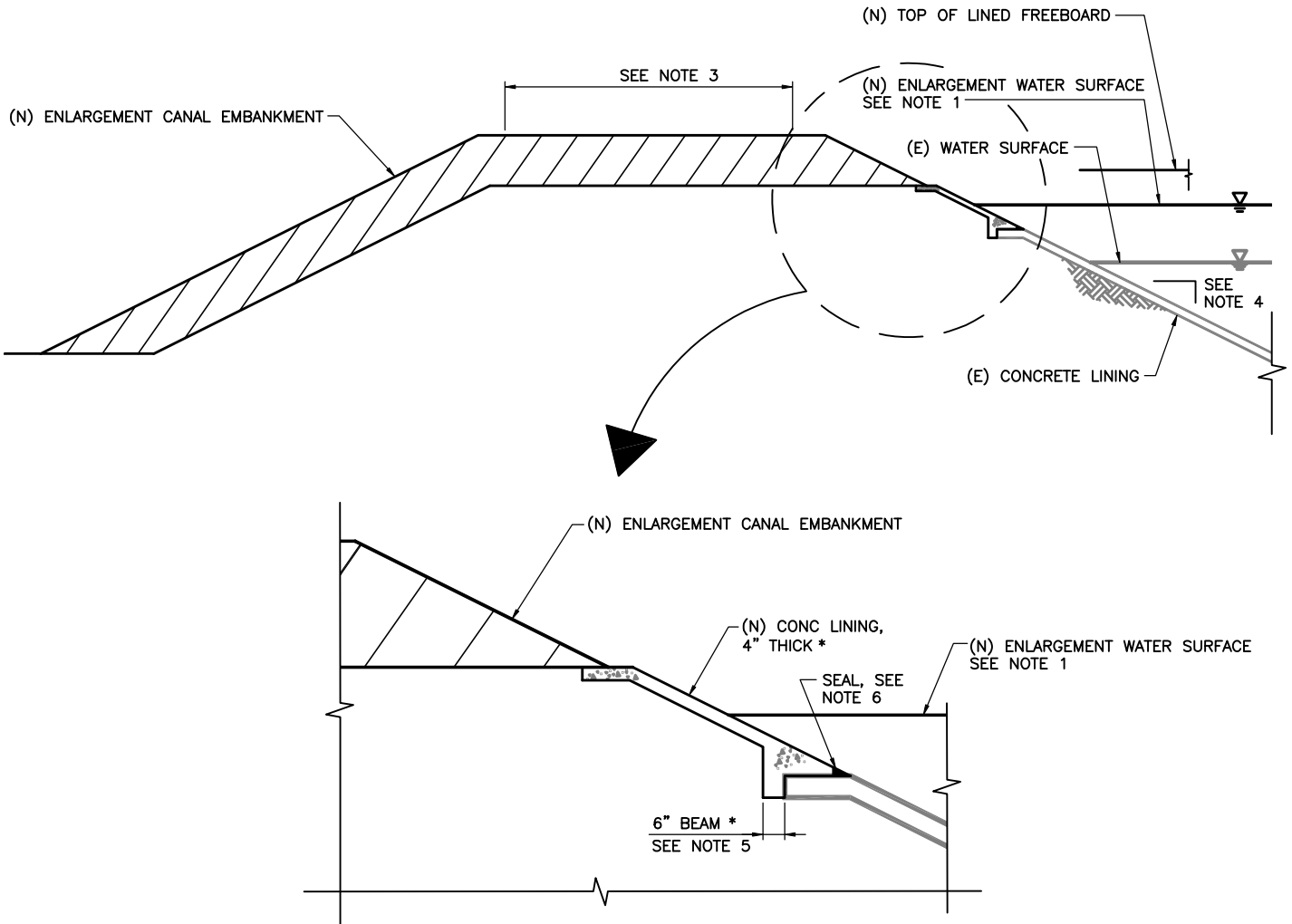
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PROJECT NO. 26814903  
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 EAST BRANCH ENLARGEMENT PROJECT

WATER SURFACE  
 ELEVATIONS -  
 POOLS 59-66

Figure  
 4-3



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
  - a.) POOLS 56, 57, AND 58 ABOVE EXISTING CONCRETE LINING.
  - b.) REMAINING POOLS BELOW EXISTING CONCRETE LINING.
2. FREEBOARD: 2 FT. CONCRETE LINED, 4 FT. TOTAL ABOVE THE DESIGN FLOW WATER SURFACE.
3. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
4. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE.
5. DISCONTINUE BEAM AT VERTICAL JOINTS TO CONTROL CRACKING.
6. ELASTOMERIC SEAL @ LINING SURFACE, OR HYDROPHILLIC SEAL @ MIDPOINT EXISTING CONCRETE LINING LIP.
7. \* DENOTES THAT THESE ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008

Ideateworks Apr. 25, 2008 - 11:47am S:\URS Corp\East Branch Aqueduct\Figures\URS MWH Figure 5-1.dwg

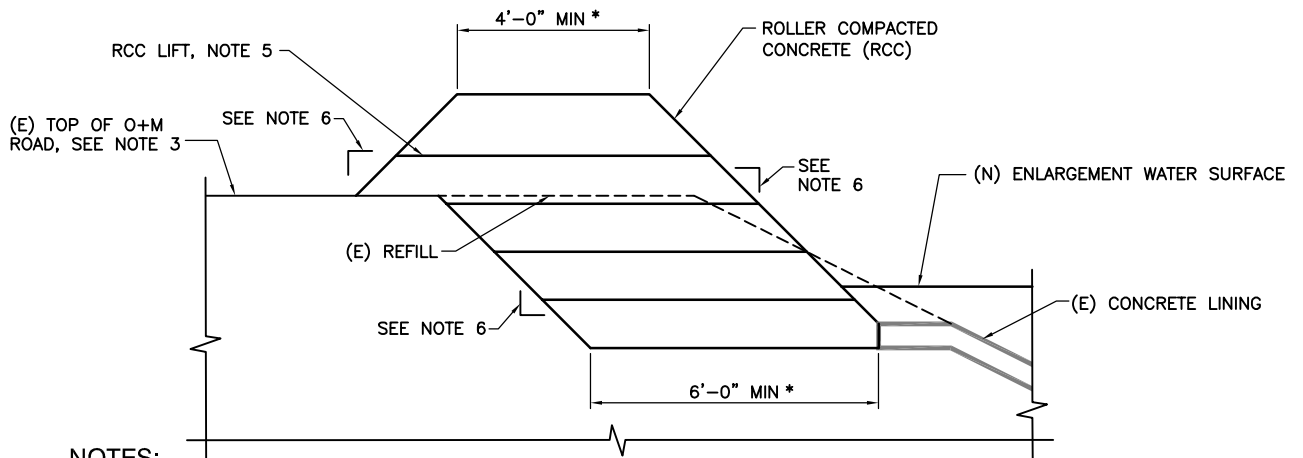
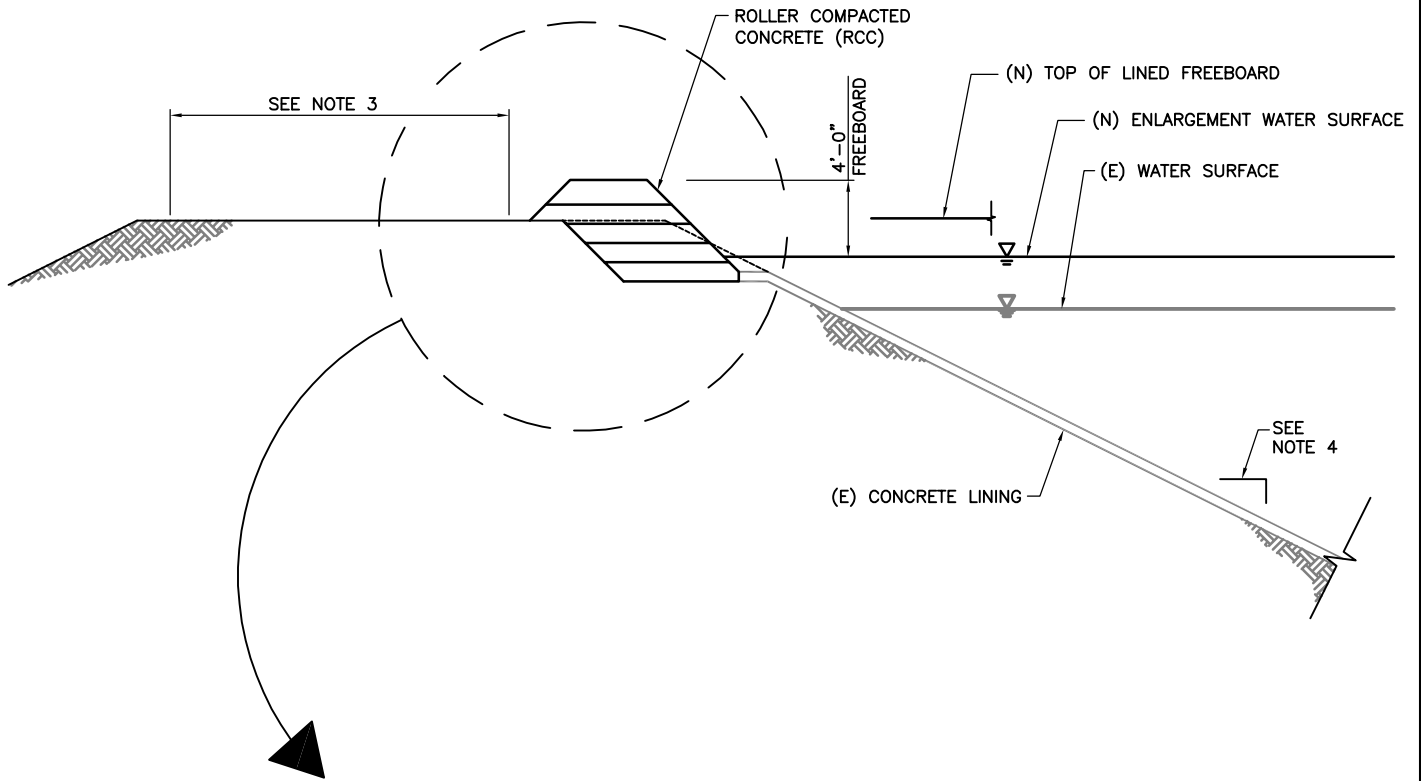


PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
EARTHFILL EMBANKMENT ALTERNATIVE

FIGURE  
5-1



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. FREEBOARD: 2 FT. CONCRETE LINED, 4 FT. TOTAL ABOVE THE DESIGN FLOW WATER SURFACE.
3. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY. FOR EACH FT OF CANAL RAISE, A DECREASE OF APPROX 1.5 TO 2 FT OF ACCESS ROAD RESULTS.
4. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE.
5. RCC DEPICTED WITH 12 INCH LIFTS.
6. SLOPE 1H:1V SHOWN; 3/4H:1V MAY BE POSSIBLE.
7. \* DENOTES THAT THESE ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008

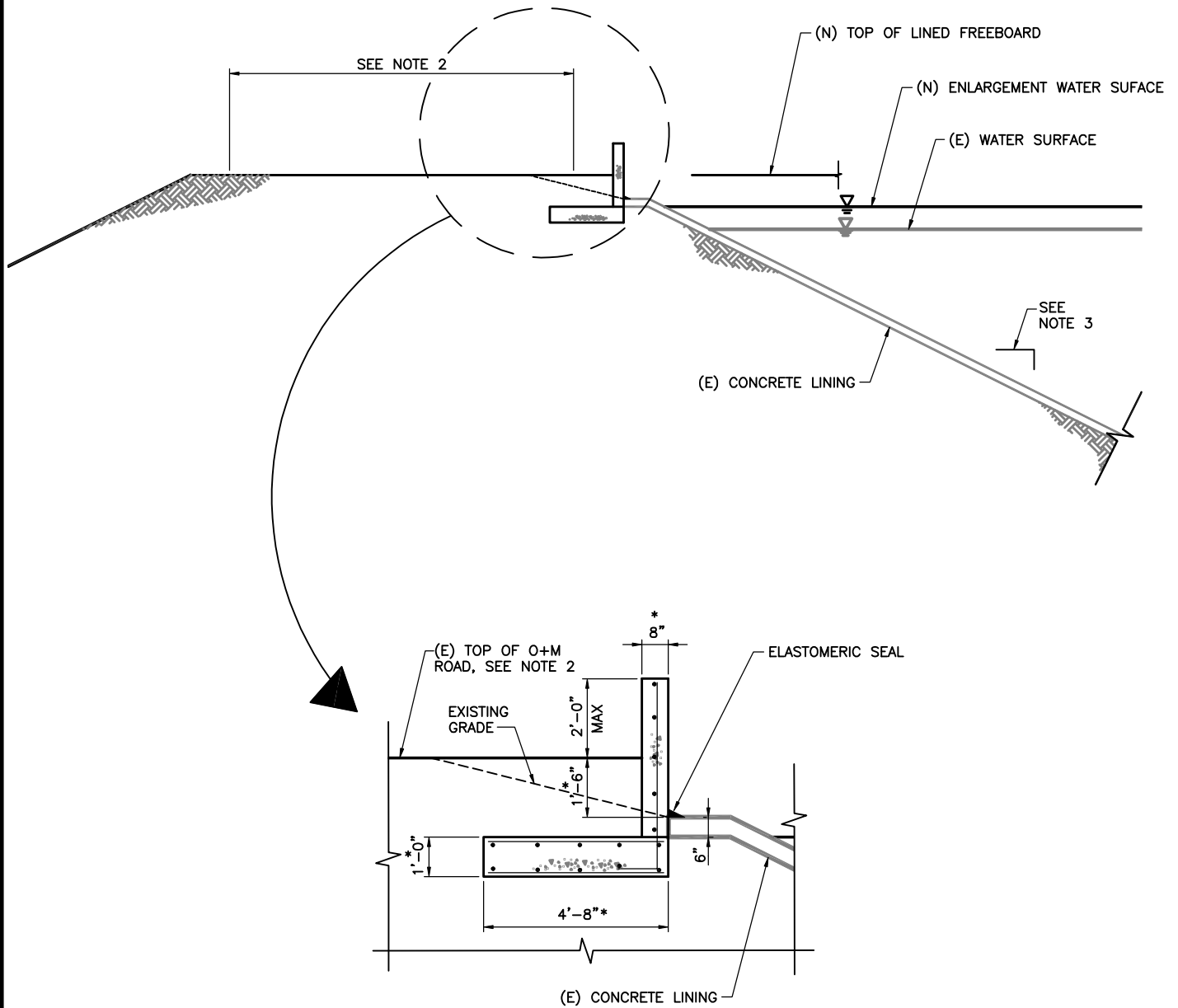
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PROJECT NO. 26814903  
 CALIFORNIA DEPARTMENT OF WATER RESOURCES  
 EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
 ROLLER COMPACTED CONCRETE ALTERNATIVE

FIGURE  
 5-2

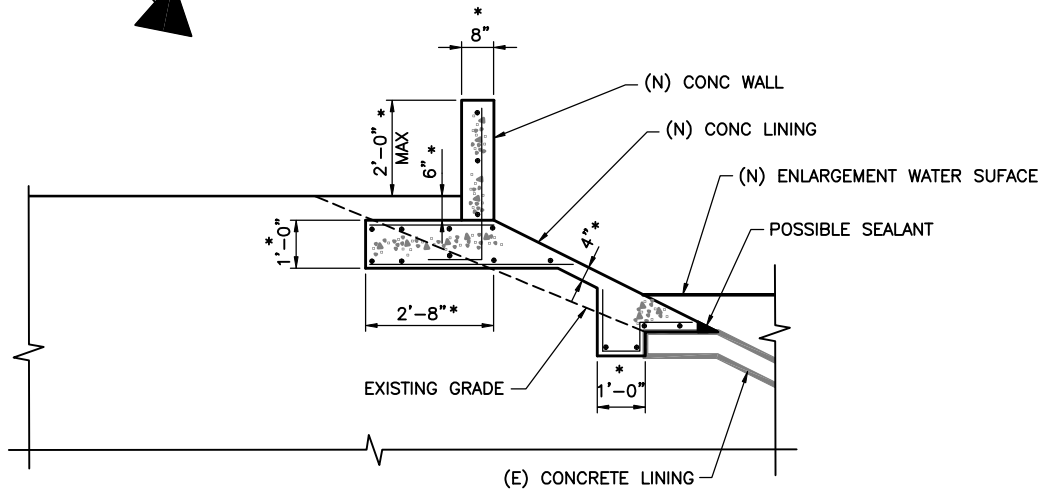
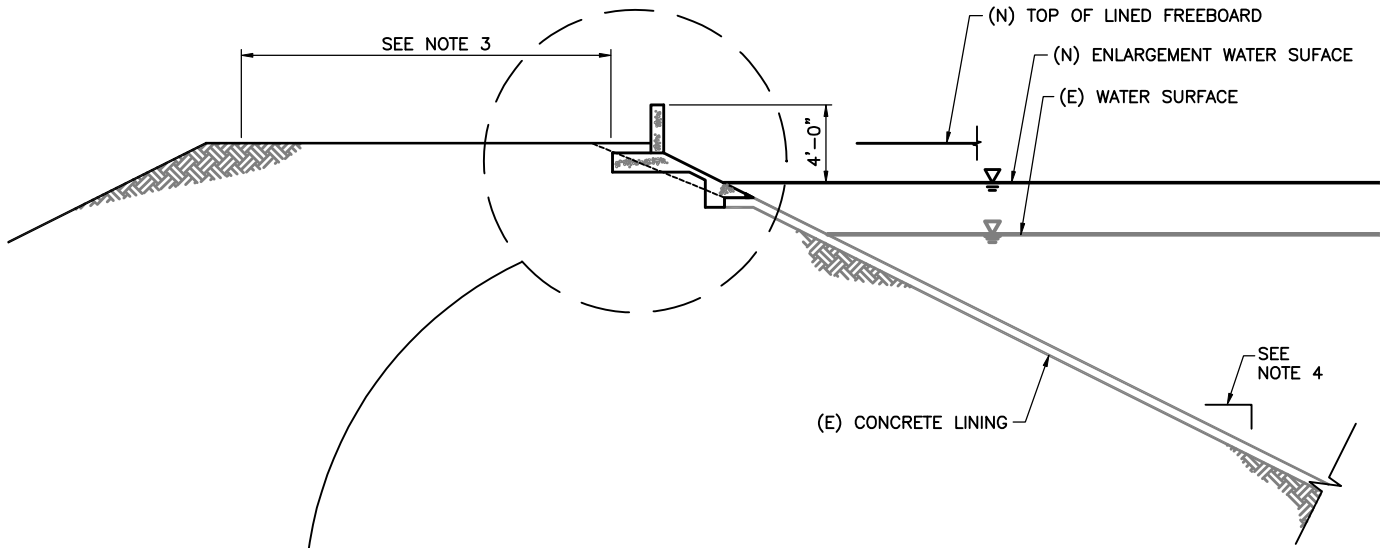


**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
3. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE.
4. \* DENOTES THAT THESE ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. FREEBOARD: 2 FT. CONCRETE LINED, 4 FT. TOTAL ABOVE THE DESIGN FLOW WATER SURFACE.
3. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
4. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE.
5. \* DENOTES THAT THESE ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008

IdeateWorks Apr. 25, 2008 - 11:47am S:\URS Corp\East Branch Aqueduct\Figures\URS MWH Figure 5-4.dwg

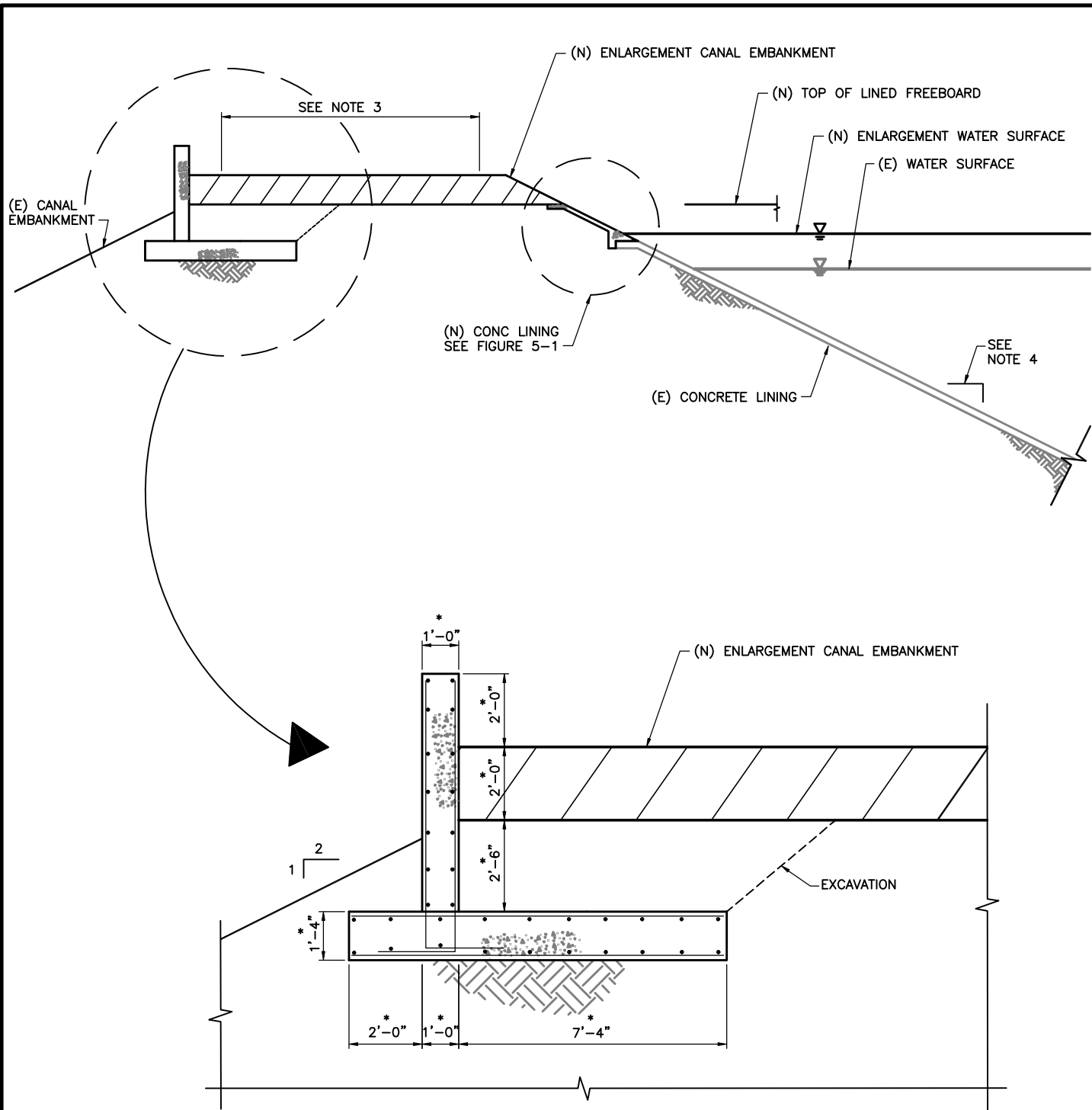


PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
CONCRETE PARAPET ALTERNATIVE

FIGURE  
5-4



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. FREEBOARD: 2 FT. CONCRETE LINED, 4 FT. TOTAL ABOVE THE DESIGN FLOW WATER SURFACE.
3. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
4. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE.
5. \* DENOTES THAT THESE ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008

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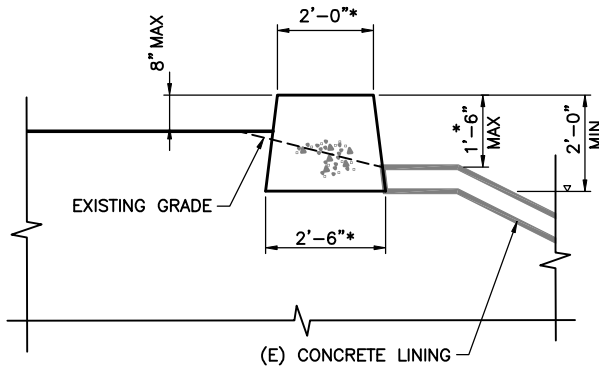
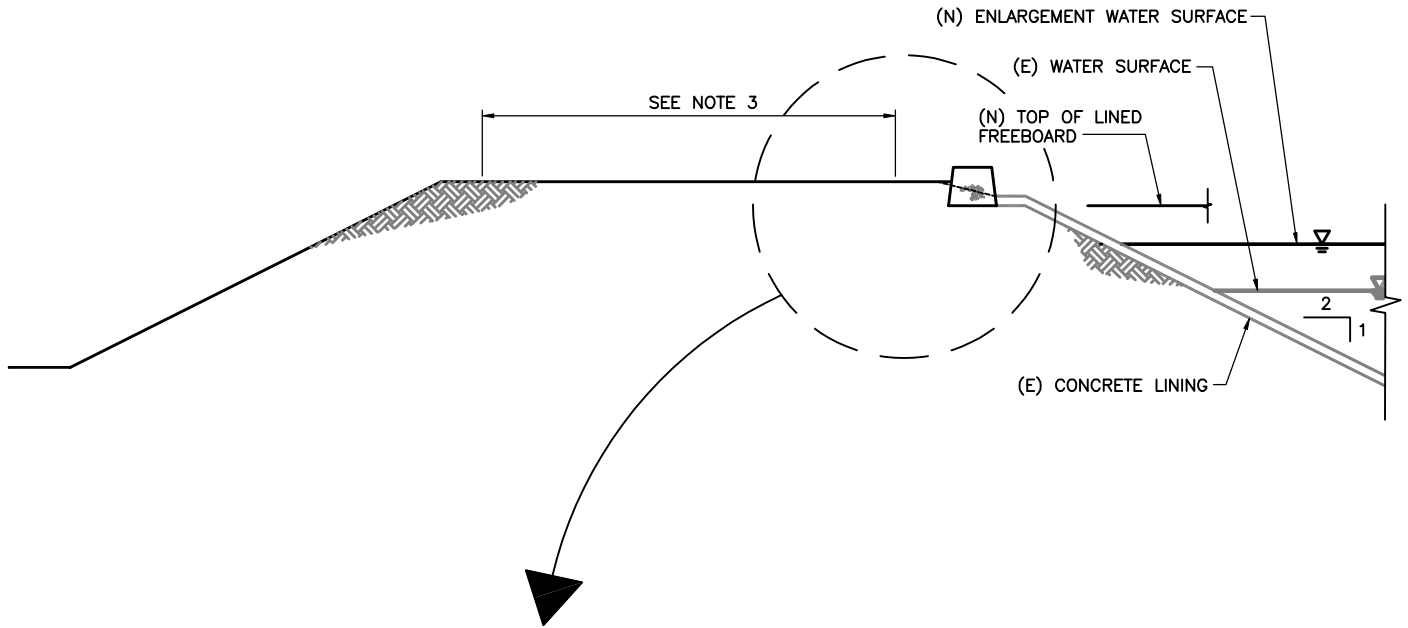


PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
EARTHFILL EMBANKMENT WITH RETAINING  
WALL ALTERNATIVE

FIGURE  
5-5



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. ALTERNATIVE IS ONLY FOR WHERE THE EXISTING CONCRETE LINING FREEBOARD IS ADEQUATE.
3. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
4. \* DENOTES THAT THESE DIMENSIONS ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.

SCALE: NONE

APRIL 25, 2008

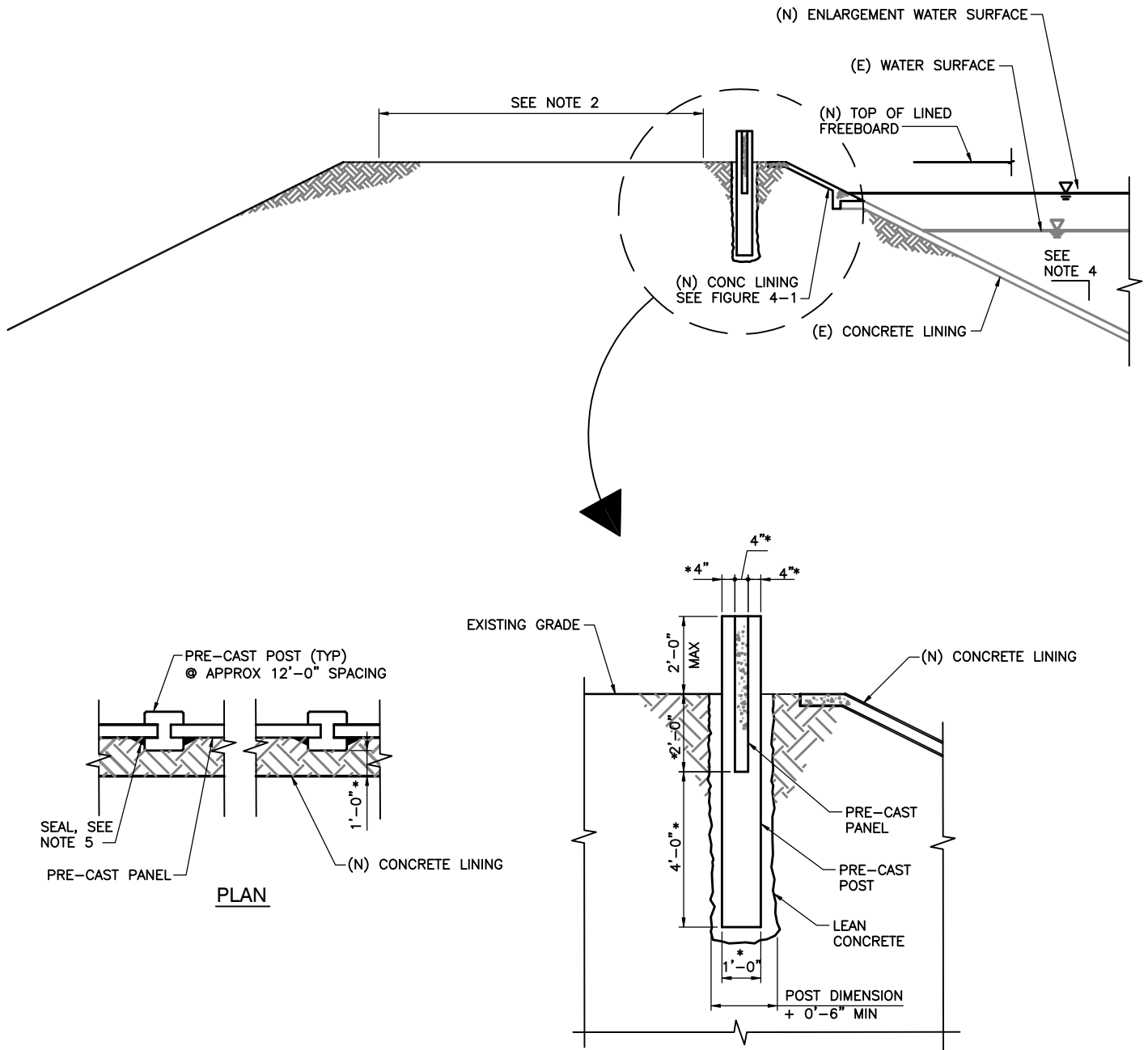
Ideas@urs | Apr. 25, 2008 - 11:48am | S:\URS Corp\East Branch Aqueduct\Figures\URS MWH Figure 5-6.dwg



PROJECT NO. 26814903  
 CALIFORNIA DEPARTMENT OF WATER RESOURCES  
 EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
 SLIPFORM WALL

FIGURE  
 5-6



**NOTES:**

1. WATER SURFACES SHOWN ARE FOR NORMAL OPERATING CONDITIONS AT DESIGN FLOW.
2. NORMAL ACCESS ROAD WIDTH: 16 FT. PRIMARY, 12 FT. SECONDARY.
3. \* DENOTES THAT THESE DIMENSIONS ARE PRELIMINARY DIMENSIONS USED FOR THE PURPOSES OF PRELIMINARY ESTIMATING.
4. SLOPE: 3H:1V FOR POOLS 52 TO 55, 2H:1V ELSEWHERE
5. ELASTOMERIC SEAL @ PANEL TO POST INTERFACE, TYPICAL.

SCALE: NONE

APRIL 25, 2008

I:\e\projects\2008\04\25\URS Corp\East Branch Aqueduct\Figures\URS MWH Figure 5-7.dwg



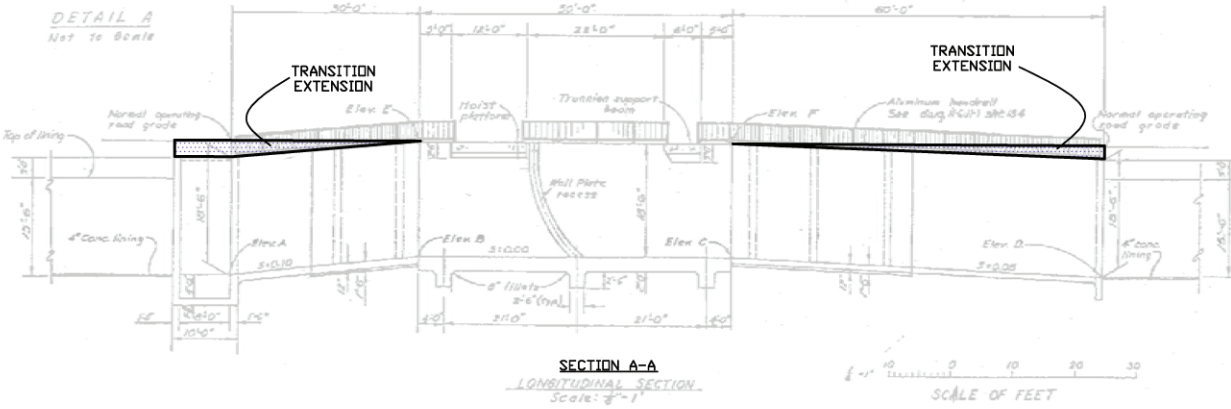
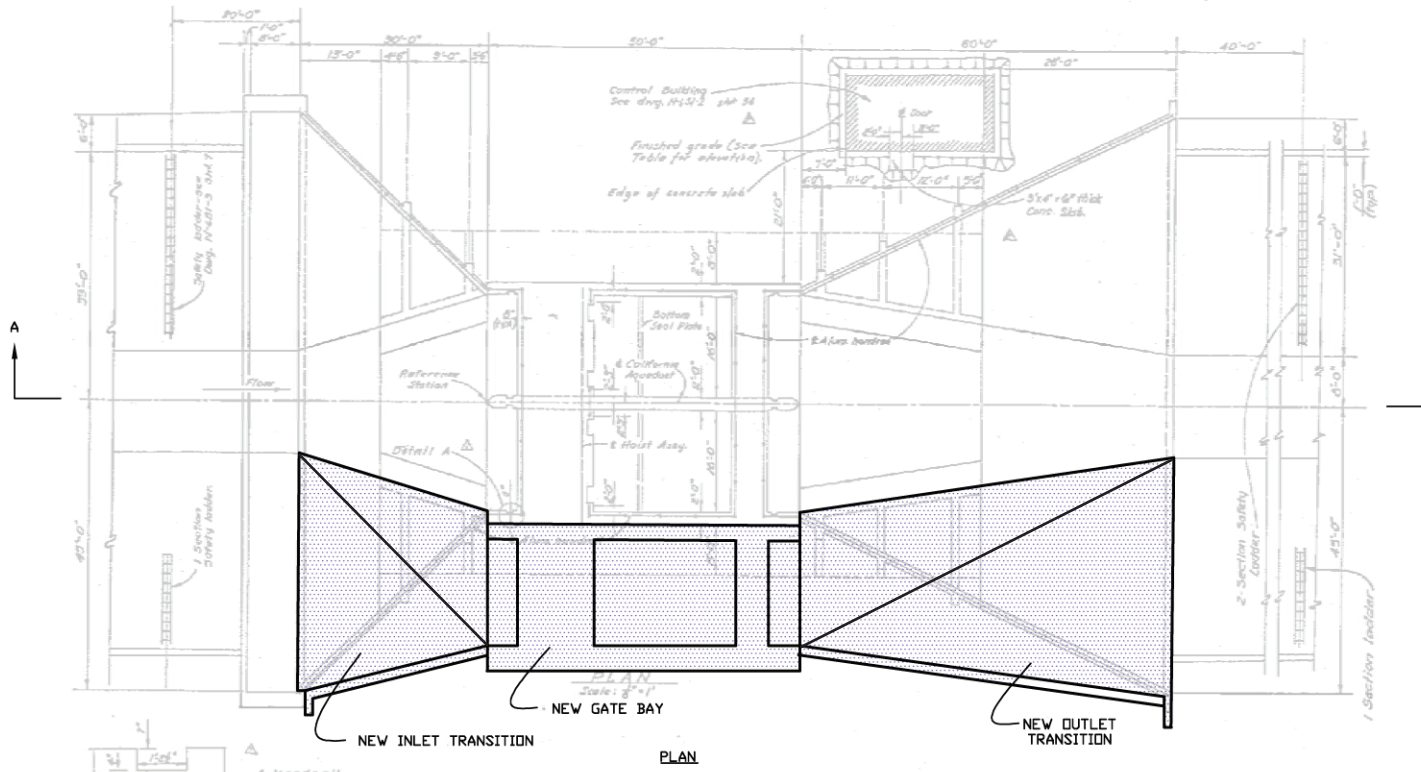
PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

CANAL RAISE CONCEPTS  
PRECAST PANEL SYSTEM

FIGURE  
5-7





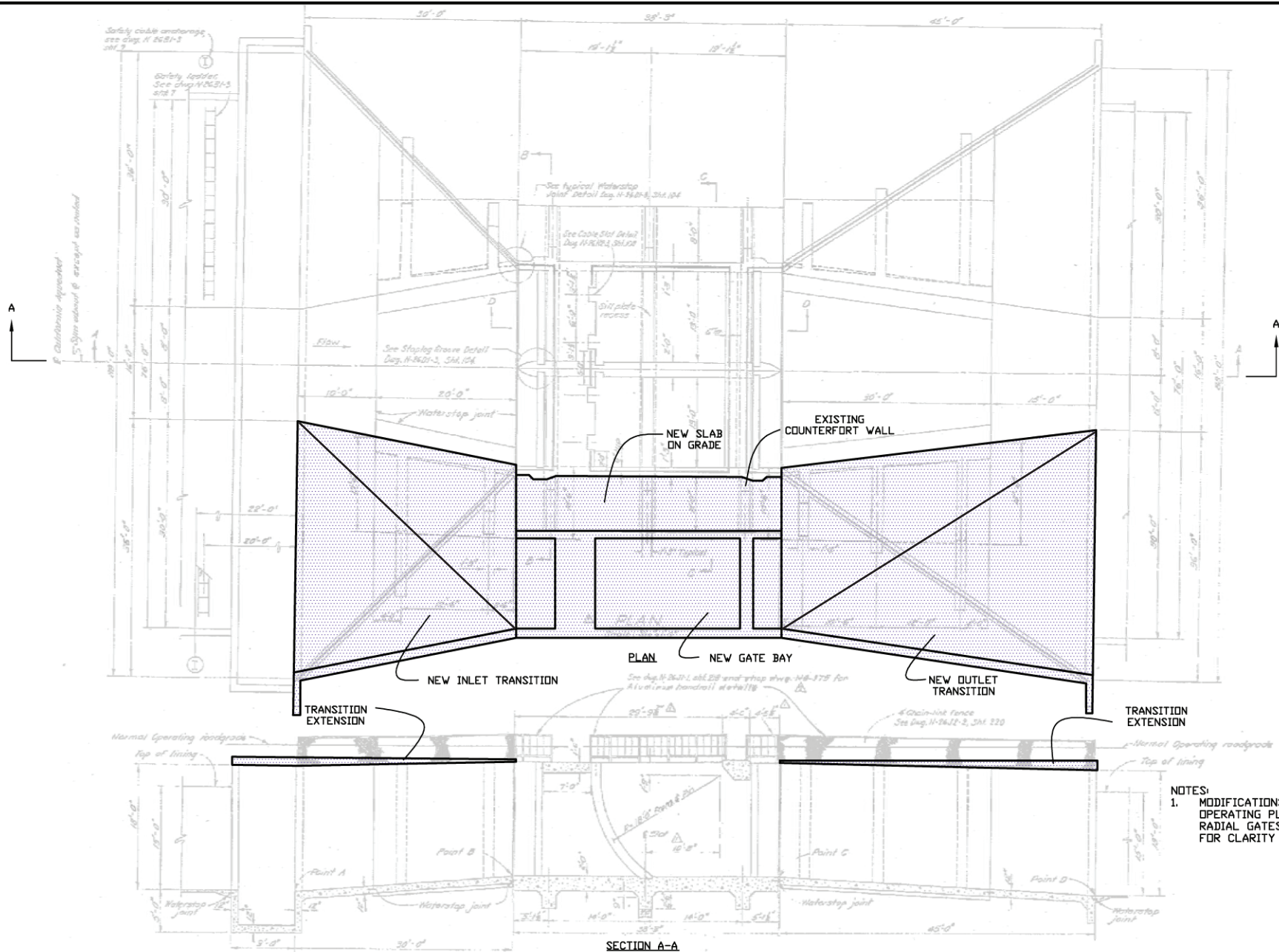


NOTES:  
 1. MODIFICATIONS TO EXISTING OPERATING PLATFORM AND RADIAL GATES NOT DEPICTED FOR CLARITY

Excerpt From DWG No. N-6D1-1, SHT No. 38, SPEC No. 88-21

SEPTEMBER 2007

SCALE:  NTS	 	PROJECT NO. 26814903	3-BAY CHECK STRUCTURE	FIGURE 5-8
		CALIFORNIA DEPARTMENT OF WATER RESOURCES EAST BRANCH ENLARGEMENT PROJECT		



NOTES:  
 1. MODIFICATIONS TO EXISTING OPERATING PLATFORM AND RADIAL GATES NOT DEPICTED FOR CLARITY

Excerpt From DWG No. N-26D1-1, SHT No. 102, SPEC No. 88-21

SECTION A-A  
 SECTION A-A  
 Scale 3/4" = 1'-0"

SEPTEMBER 2007

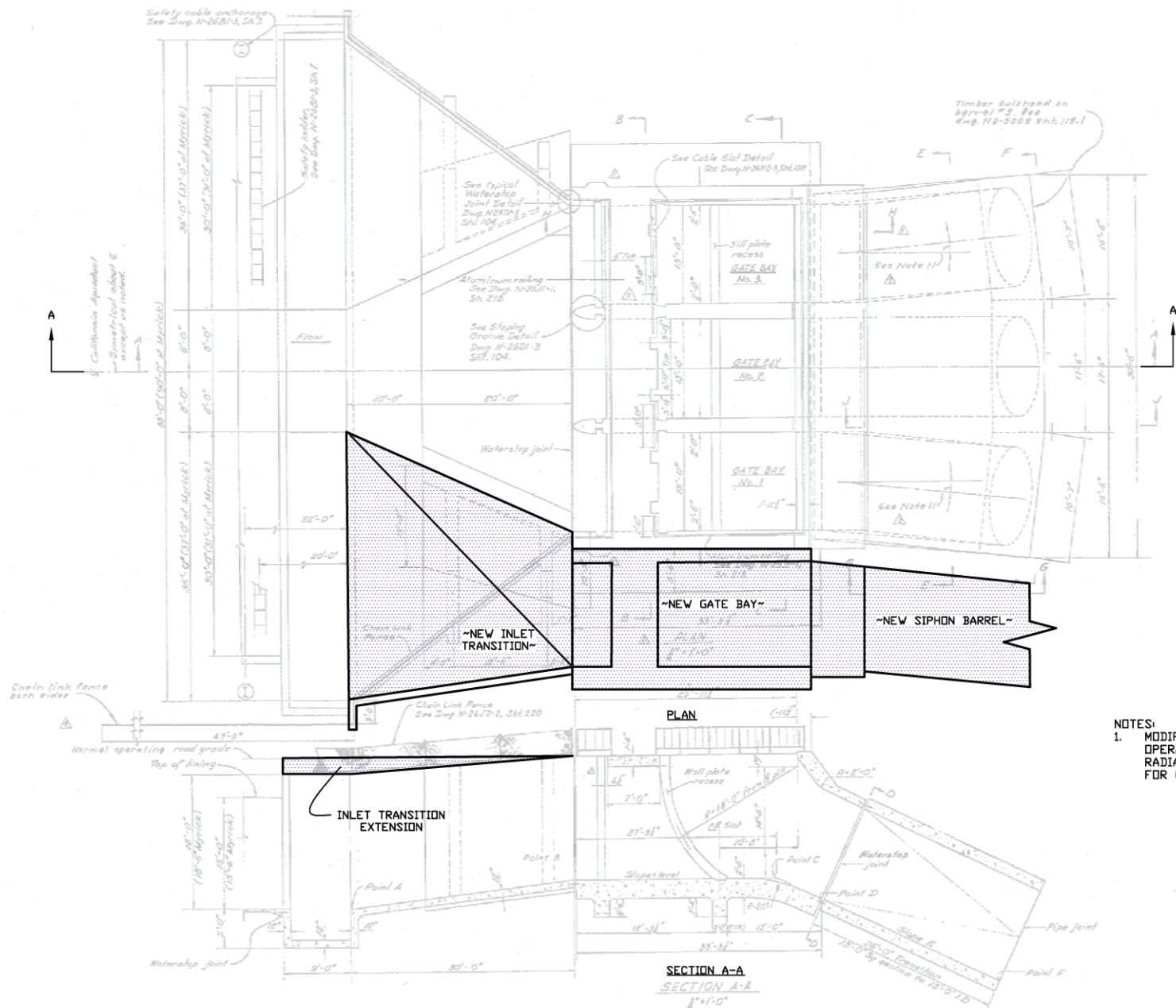
SCALE:  
 NTS



PROJECT NO. 26814903  
 CALIFORNIA DEPARTMENT OF WATER RESOURCES  
 EAST BRANCH ENLARGEMENT PROJECT

3-BAY CHECK STRUCTURE  
 WITH EXISTING COUNTERFORT WALL

FIGURE  
 5-9



NOTES:  
 1. MODIFICATIONS TO EXISTING OPERATING PLATFORM AND RADIAL GATES NOT DEPICTED FOR CLARITY

Excerpt From DWG No. N-26N2-1, SHT No. 106, SPEC No. 88-21

SEPTEMBER 2007

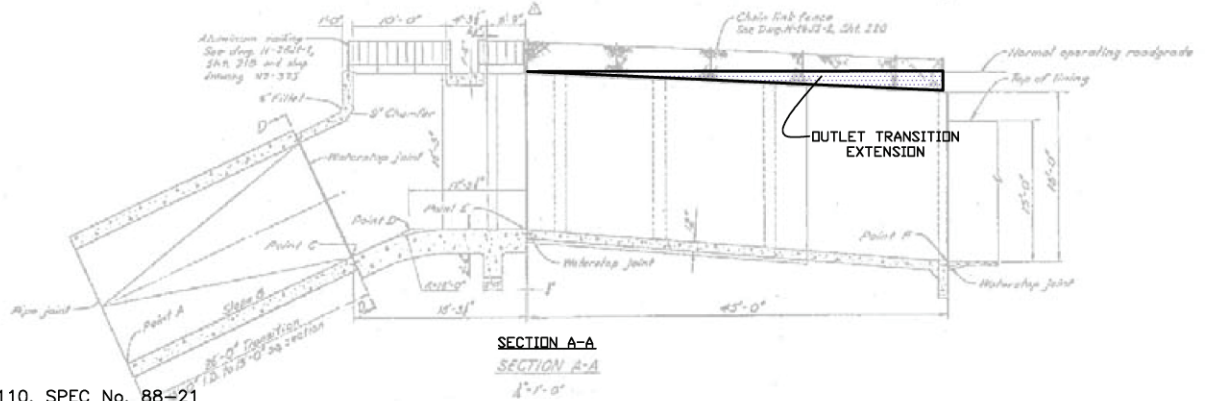
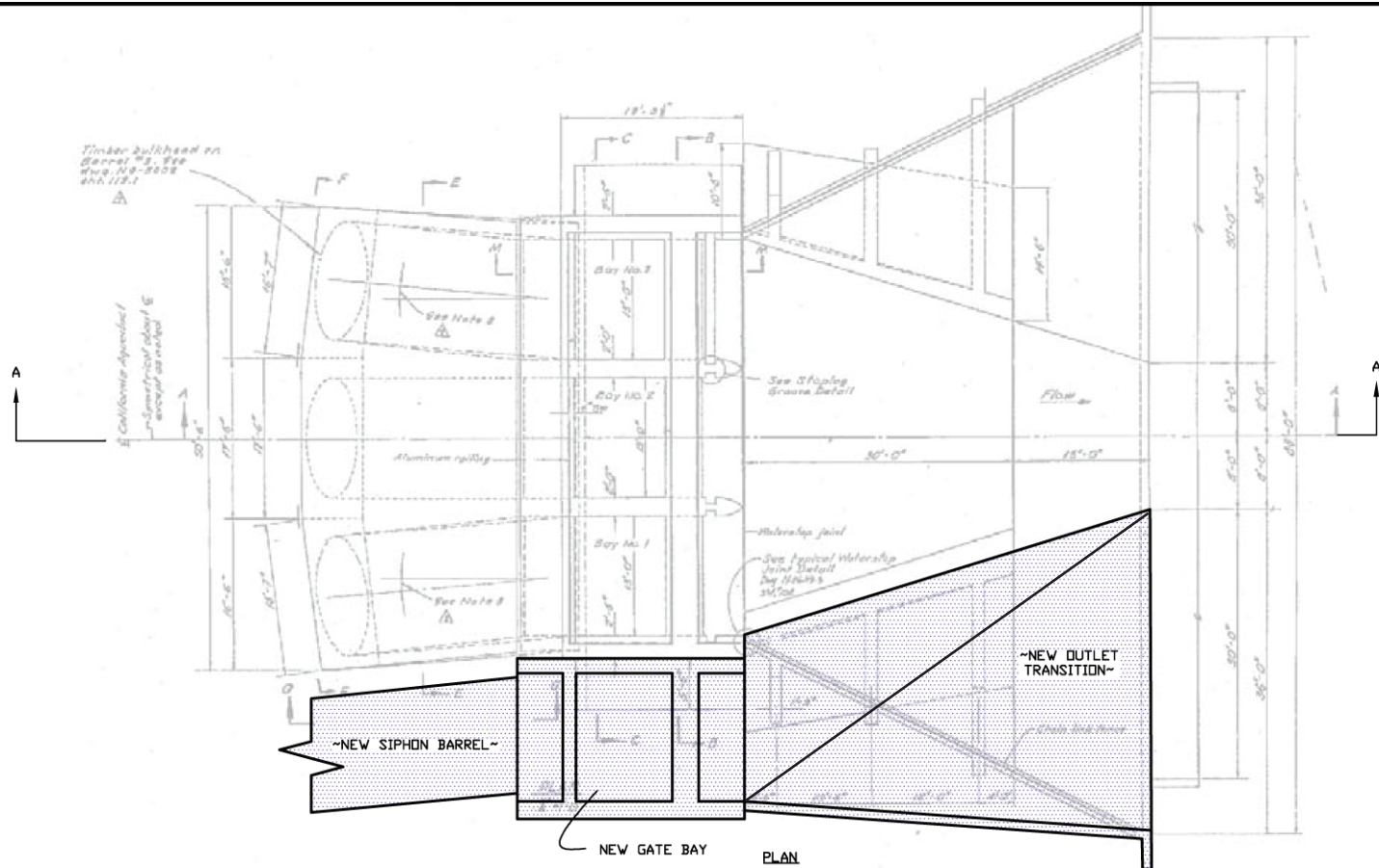
SCALE:  
 NTS



PROJECT NO. 26814903  
 CALIFORNIA DEPARTMENT OF WATER RESOURCES  
 EAST BRANCH ENLARGEMENT PROJECT

4-BAY CHECK STRUCTURE  
 SIPHON-INLET

FIGURE  
 5-10



NOTES:  
 1. MODIFICATIONS TO EXISTING OPERATING PLATFORM AND RADIAL GATES NOT DEPICTED FOR CLARITY

Excerpt From DWG No. N-26N5-1, SHT No. 110, SPEC No. 88-21

SEPTEMBER 2007

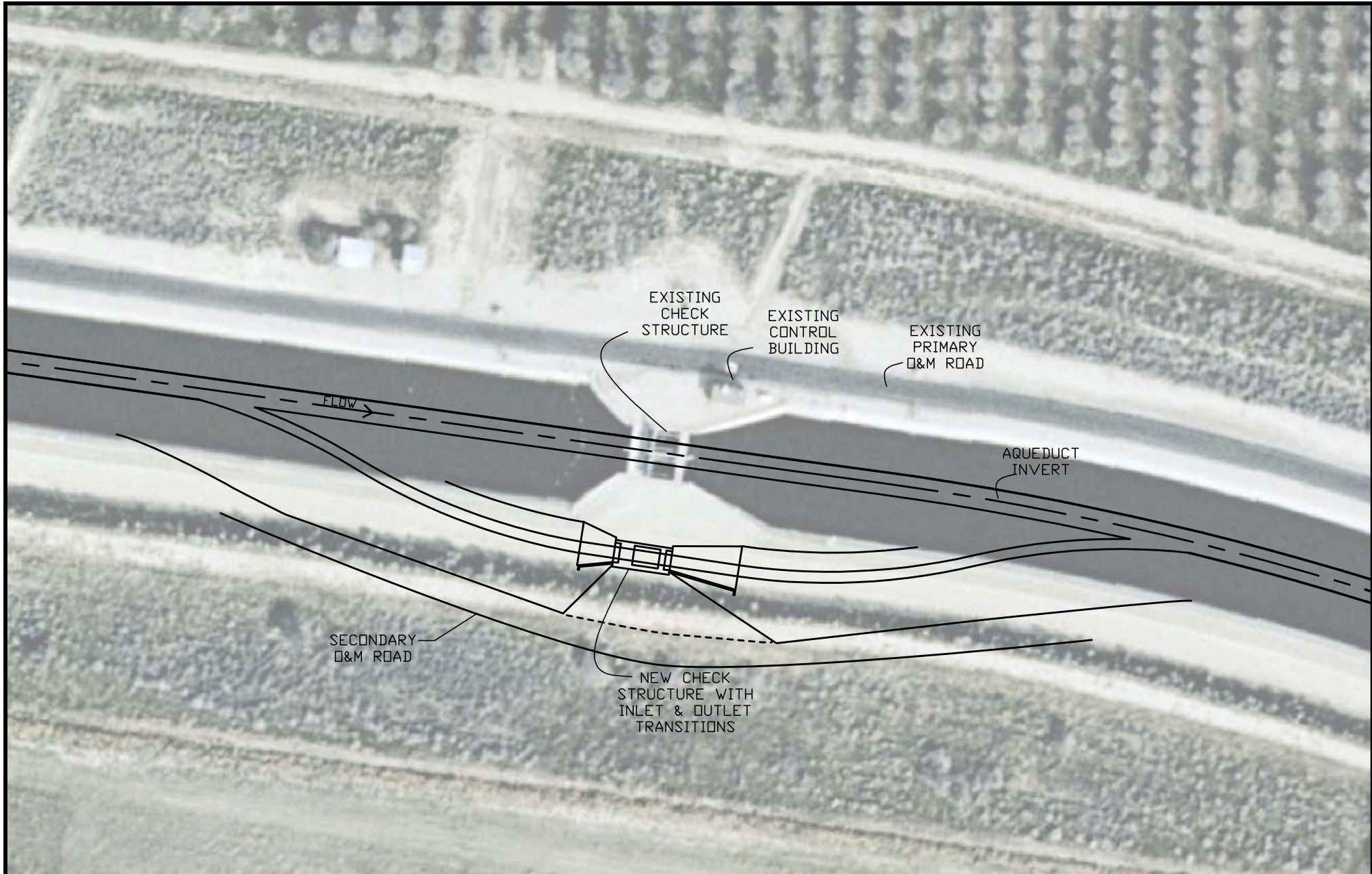
SCALE:  
 NTS



PROJECT NO. 26814903  
 CALIFORNIA DEPARTMENT OF WATER RESOURCES  
 EAST BRANCH ENLARGEMENT PROJECT

4-BAY CHECK STRUCTURE  
 SIPHON-OUTLET

FIGURE  
 5-11



AERIAL PHOTO FROM TIF IMAGE CA\_EastBranch\_Check\_43\_SP583

SEPTEMBER 2007

SCALE:  
NTS

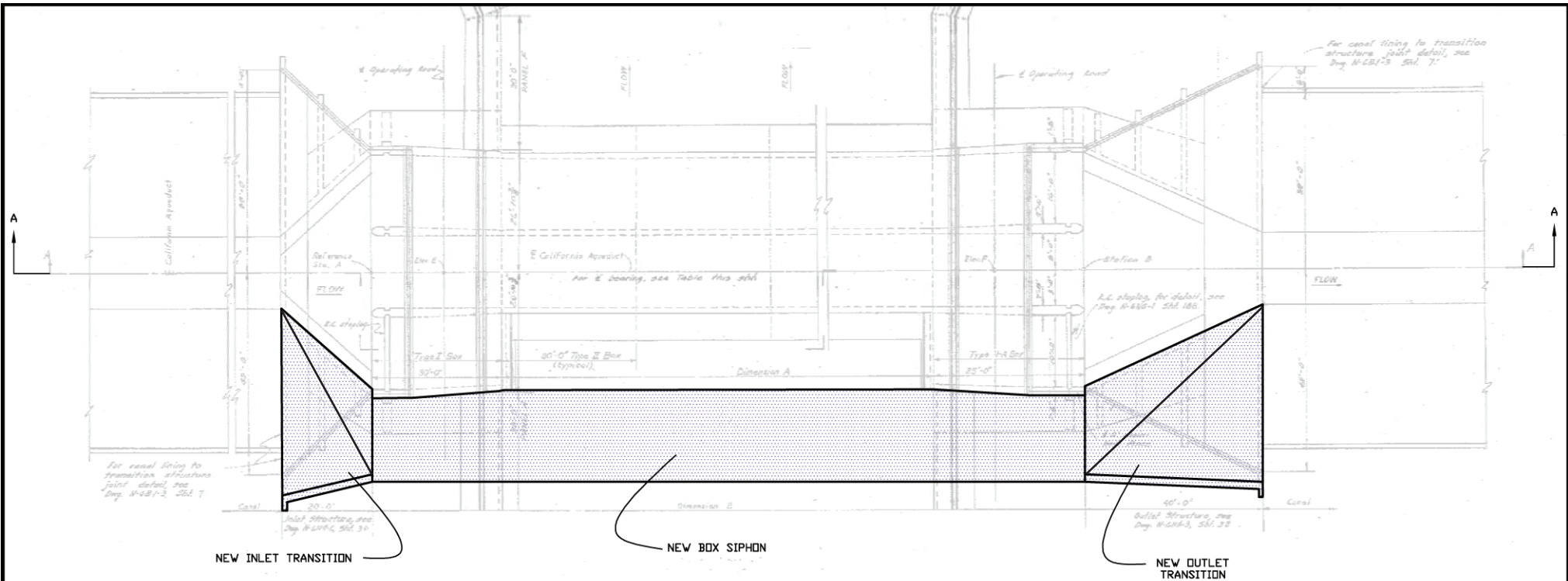


PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

ALTERNATIVE  
3-BAY CHECK STRUCTURE

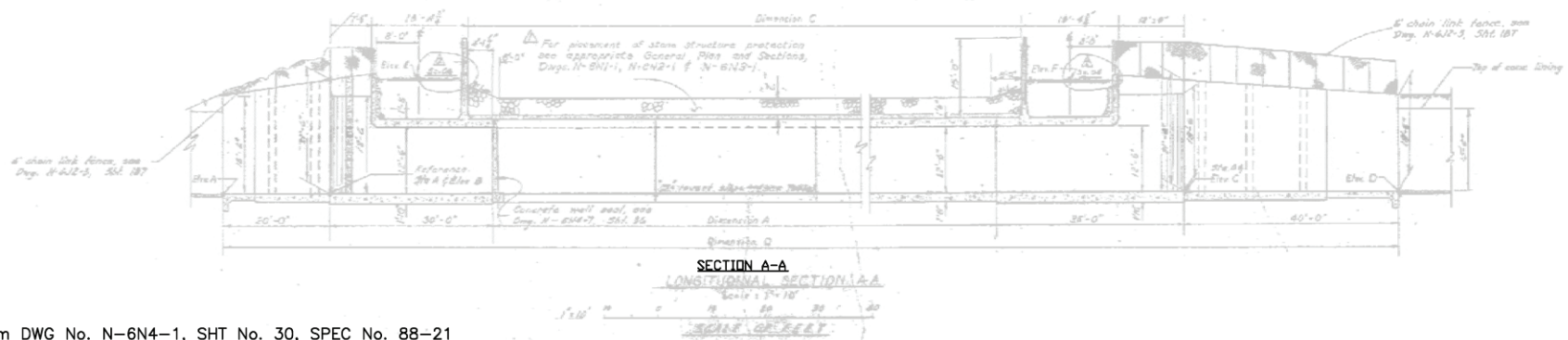
FIGURE  
5-12



PLAN

PLAN  
Scale: 1"=10'

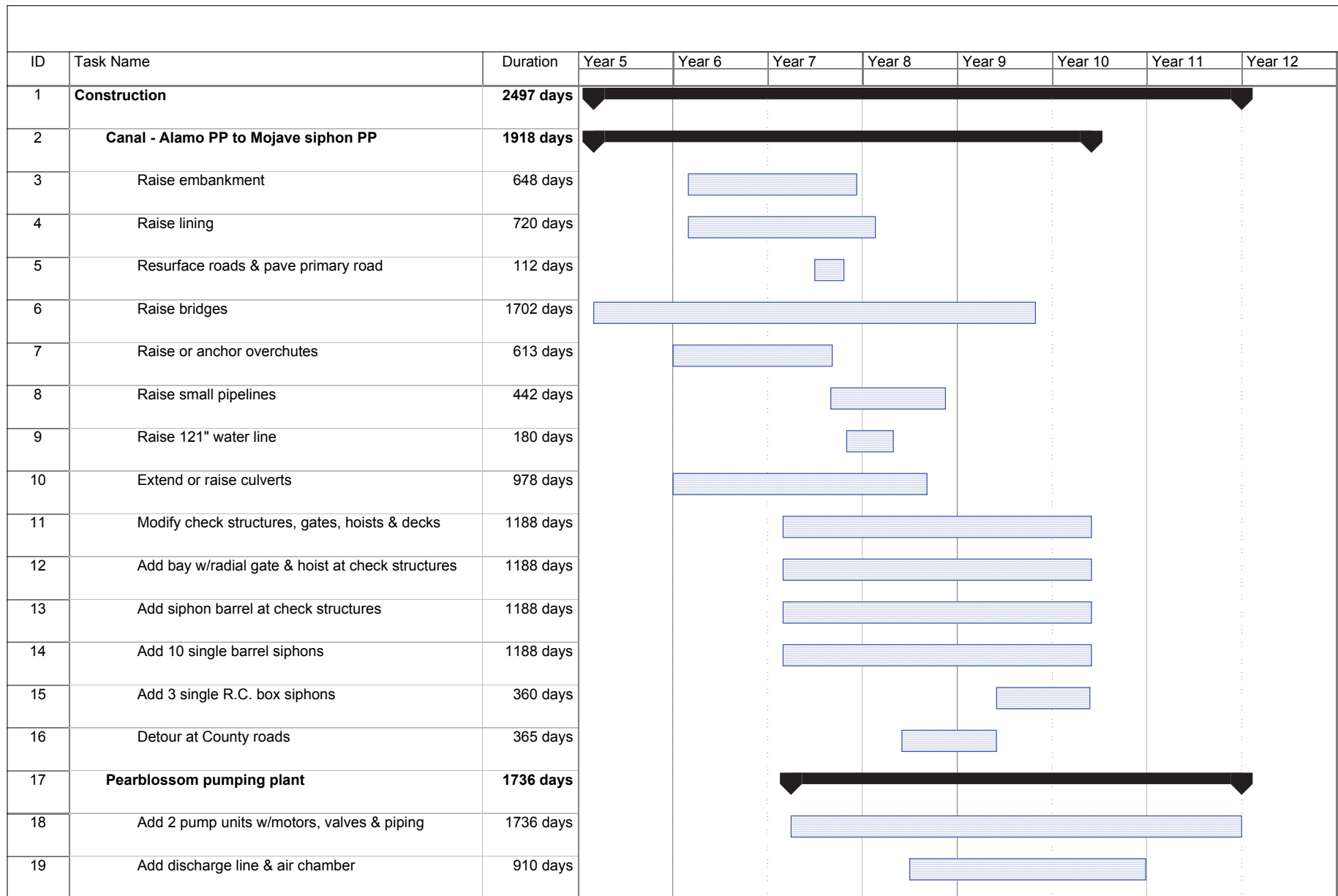
NOTES:  
1. MODIFICATIONS TO EXISTING OPERATING PLATFORM AND RADIAL GATES NOT DEPICTED FOR CLARITY



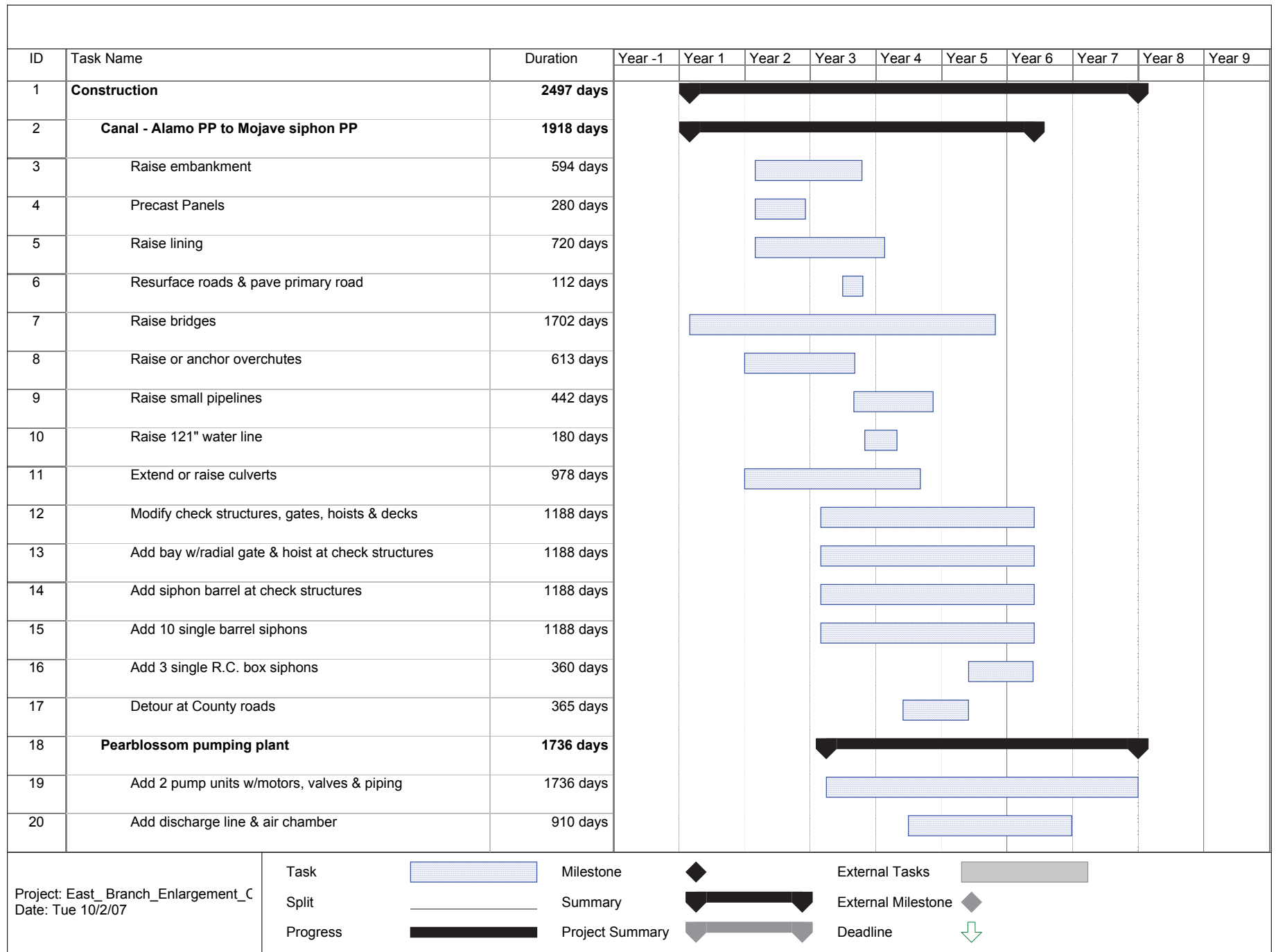
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SEPTEMBER 2007

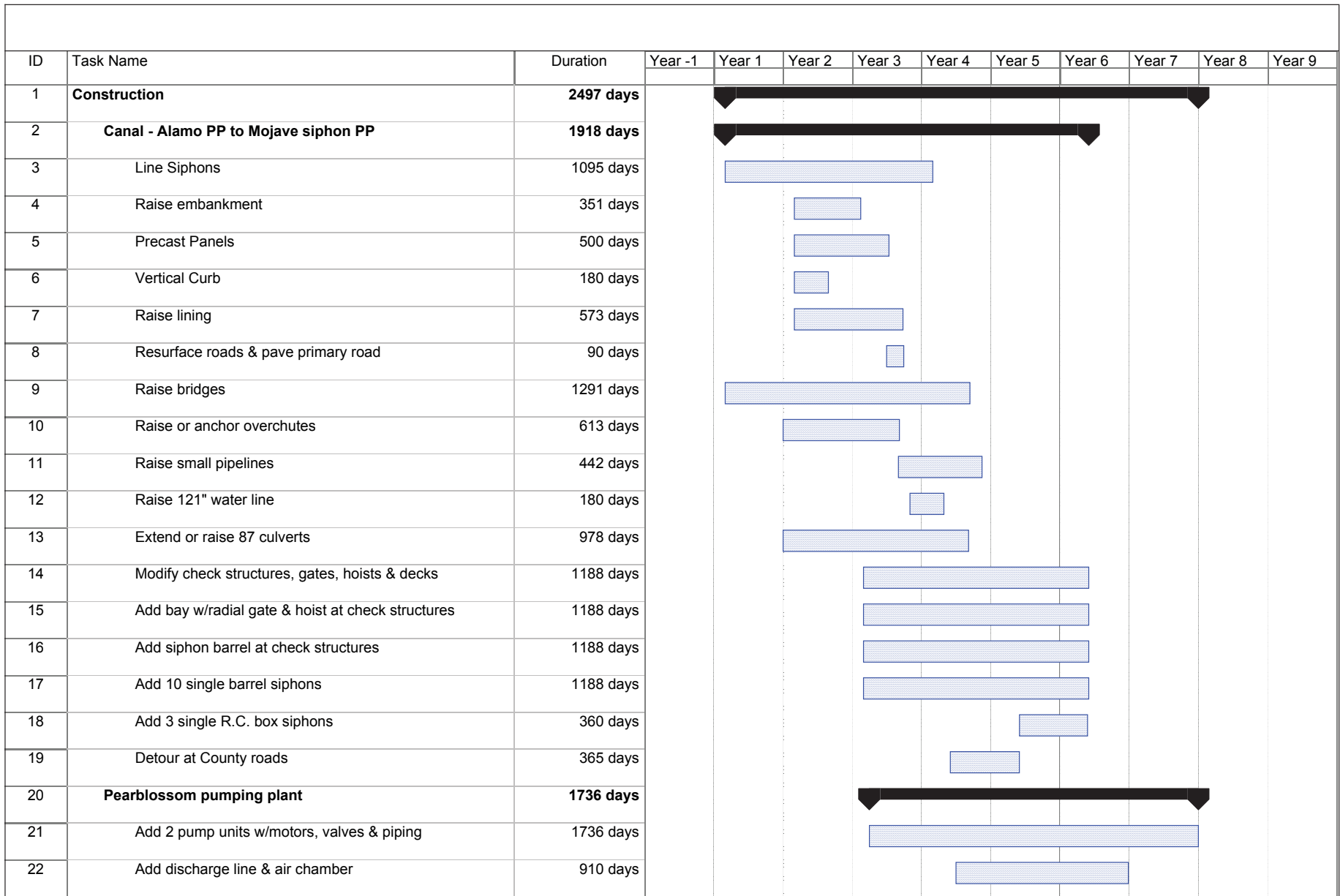
SCALE:  NTS		PROJECT NO. 26814903  CALIFORNIA DEPARTMENT OF WATER RESOURCES EAST BRANCH ENLARGEMENT PROJECT	4-BARREL BOX SIPHON	FIGURE 5-13
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Project: East_Branch_Enlargement_C Date: Tue 10/2/07	Task		Milestone		External Tasks	
	Split		Summary		External Milestone	
	Progress		Project Summary		Deadline	







Project: East_Branch_Enlargement_C Date: Tue 10/2/07	Task		Milestone		External Tasks	
	Split		Summary		External Milestone	
	Progress		Project Summary		Deadline	

SCALE: NTS



PROJECT NO. 26814903

CALIFORNIA DEPARTMENT OF WATER RESOURCES  
EAST BRANCH ENLARGEMENT PROJECT

SCENARIO 3  
CONSTRUCTION  
SCHEDULE

Figure  
9-3

DRAFT

**Appendix A**  
**Existing East Branch Aqueduct**

DRAFT

**Appendix B**  
**Data For Existing Canal Facilities**

DRAFT

**Appendix C**  
**Flow Test Plan and Flow Test Results**

DRAFT

**Appendix C1  
Flow Test Plan**

DRAFT

**Appendix C2**  
**Flow Test Results**

DRAFT

**Appendix D**

Hydraulic Model Results

DRAFT

**Appendix E**  
**Cost Estimate Backup**