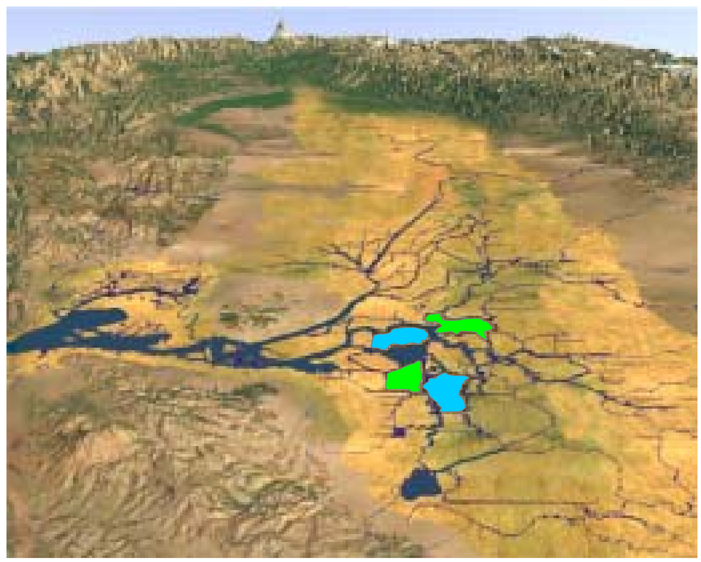
IN-DELTA STORAGE PROGRAM
STATE FEASIBILITY STUDY
DRAFT ENGINEERING INVESTIGATIONS SUMMARY

INTEGRATED STORAGE INVESTIGATIONS



CALIFORNIA BAY-DELTA AUTHORITY

Division of Planning and Local Assistance
Department of Water Resources
July 2003

ORGANIZATION

FOREWORD

We acknowledge the technical assistance provided by Reclamation in carrying out the role of federal lead agency for the CALFED Integrated Storage Investigations. Reclamation has not yet completed a full review of the State Feasibility Study reports. Reclamation will continue to provide technical assistance through the review of the State Feasibility Study reports and DWR will work with Reclamation to incorporate comments and recommendations in the final reports.

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1.1 General

In-Delta storage investigations were authorized under the CALFED Integrated Storage Investigations Program as defined in the CALFED Bay-Delta Program Record of Decision (ROD) and Implementation Memorandum of Understanding (MOU) signed by the CALFED Agencies on August 28, 2000. The ROD identified In-Delta storage as one of five surface storage projects to be studied. As a part of the In-Delta Storage investigations, the CALFED Agencies also decided to explore the lease or purchase of The Delta Wetlands (DW) Project, a private proposal by Delta Wetlands Properties Inc. to develop and market a water storage facility in the Sacramento-San Joaquin Delta (Delta). The proposed DW project included conversion of two islands, Webb Tract and Bacon Island, into “reservoir” islands and conversion of Bouldin Island and Holland Tract into “habitat” islands. The ROD included an option to initiate a new project if the DW Project proved cost prohibitive or technically infeasible.

The California Department of Water Resources and the CALFED Bay-Delta Program, with technical assistance from the U.S. Bureau of Reclamation, conducted a joint planning study to evaluate the DW project and other In-Delta storage options for contributing to CALFED water supply reliability and ecosystem restoration objectives. The main purpose of the investigations was to determine if the proposed DW project was technically and financially feasible. The joint planning study, completed in May 2002, concluded that the project concepts as proposed by DW were generally well planned. For ownership by DWR and USBR, however, the project as proposed by DW requires modifications and additional analyses before it is appropriate to “initiate negotiation with Delta Wetlands owners or other appropriate landowners for acquisition of necessary property” (CALFED ROD, page 44).

The Re-engineered In-Delta Storage project has the same reservoir and habitat islands as the proposed DW project. The design modifications include a re-engineered embankment design around the reservoir islands and four consolidated inlet and outlet structures (integrated facilities); two on each of the reservoir islands. The project islands and integrated facility locations are shown in Figure 1.1.

This report includes a description of how the feasibility-level engineering investigations are interlinked, as well as conclusions and recommendations resulting from the investigations. It also includes a detailed summary of each engineering investigation, including results.

1.2 Purpose and Need for In-Delta Storage

The purpose of In-Delta storage is to:

- help meet the ecosystem needs of the Delta,
- help achieve Environmental Water Account (EWA) and Central Valley Project Improvement Act (CVPIA) goals,
- provide water for use within the Delta, and
- increase reliability, operational flexibility and water availability for south of the Delta water use by the State Water Project (SWP) and the Central Valley Project (CVP).

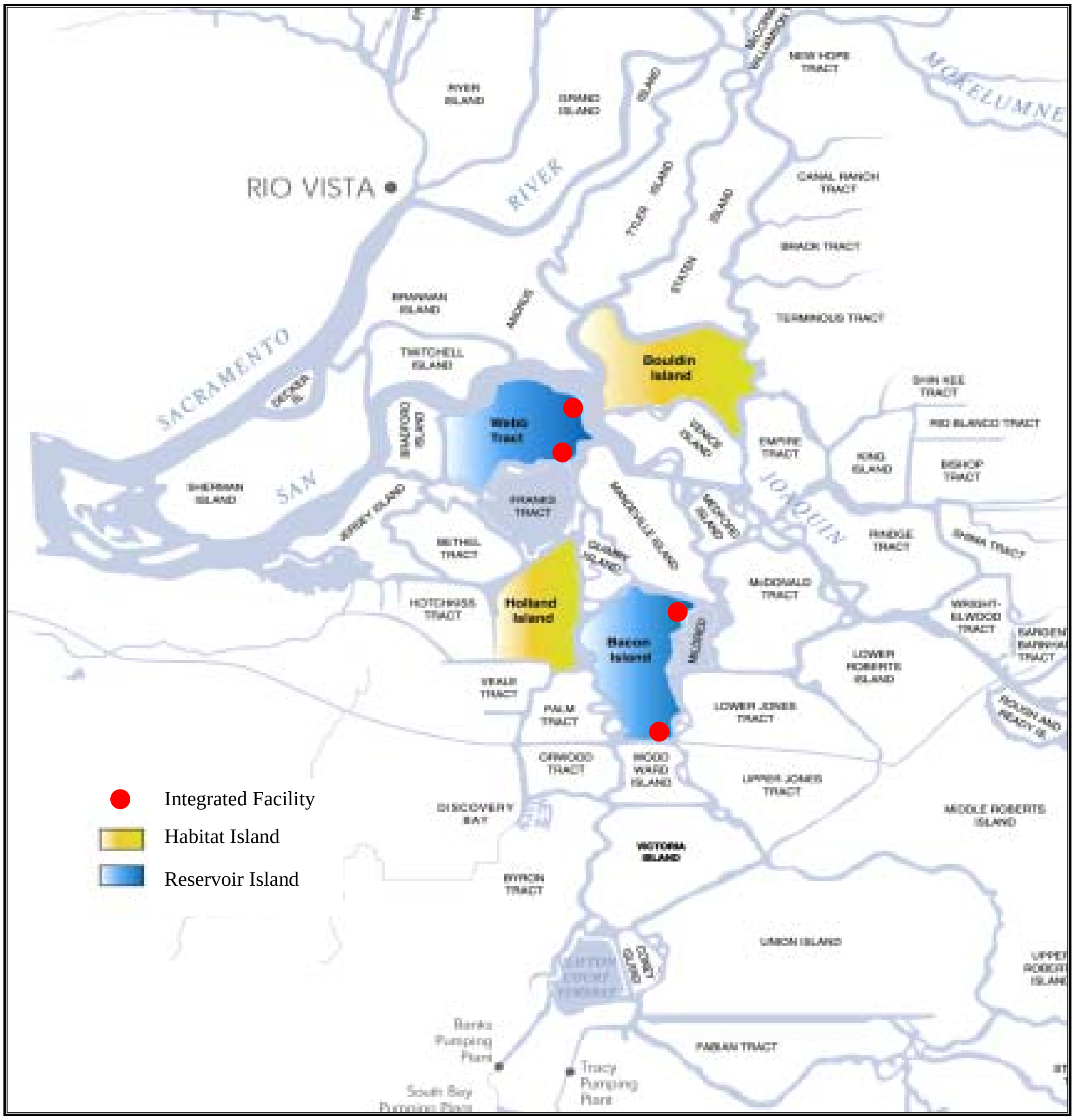


Figure 1.1 – In-Delta Storage Project Islands and Integrated Facility Locations

Improved operational flexibility would be achieved by providing an opportunity to change the timing of Delta exports and new points of diversion that could be selectively used to minimize impacts on fish. The In-Delta Storage Project would divert water from the Delta to Webb Tract and Bacon Island for storage during periods of high flow and low fish impacts. The stored water could be used to make up for export curtailments made during times most critical to listed fish species. New storage in the Delta could be useful to the California water system because it would:

- increase water supply reliability.
- improve system operational flexibility.
- allow reservoir space to be temporarily used for water transfers and banking.
- allow water to be stored and released to meet CVPIA and EWA goals and water quality constraints.
- allow surplus water to be stored during wet periods and when upstream reservoirs spill, permitting water to be stored in the Delta and released into the San Joaquin River and other in-stream channels for fisheries during dry periods.

1.3 Scope of Engineering Investigations

As mentioned earlier, the May 2002 joint planning study concluded that the project as proposed by DW requires modifications and additional analyses before it is appropriate to "initiate negotiation with Delta Wetlands owners or other appropriate landowners for acquisition of necessary property". As a result DWR developed a scope of work, which included conducting numerous engineering investigations to address these recommendations. The engineering investigations are covered in separate draft reports and are listed as follows:

- In-Delta Storage Program Draft Reports by URS Corporation. April 2003.
    - Flooding Analysis
    - Seismic Analysis
    - Embankment Design Analysis
    - Borrow Area Geotechnical Report
    - Integrated Facilities Structural Engineering Design and Analysis Report (located in the Integrated Facilities Engineering Design and Analyses report by DWR, Appendix B)
    - Embankments Construction Methods and Cost Estimating
    - Structures Construction Methods and Cost Estimating
    - Risk Analysis. May 2003.
- In-Delta Storage Program Draft Reports by California Department of Water Resources
    - Results of Geologic Exploration Program. January 2003.
    - Results of Laboratory Testing Program. January 2003.
    - Integrated Facilities Engineering Design and Analyses. April 2003.

The engineering investigations are all interlinked and all of them build upon information from others. The investigations were separated into four areas; field investigations, engineering design and analyses, construction methods and cost estimation, and risk analysis.

1.3.1 Field Investigations

The first phase of work was to conduct field investigations, which included hydrological investigations and geologic explorations. The information obtained from these investigations was used in the engineering design and analyses.

The hydrological investigations included literature review and a tidal analysis of river stages. In the tidal analysis, detailed statistical analyses of the available stage data were conducted to obtain historical distributions of the tidal stages near the integrated facility locations.

Geologic explorations were conducted to determine the soil properties of potential borrow sources on the reservoir islands and to evaluate the integrated facility foundation materials. The geologic data obtained from these explorations was used in the embankment design, borrow area investigations and integrated facilities structural design.

1.3.2 Engineering Design and Analyses

The second phase of work was to conduct engineering design and analyses, which included flooding and seismic analyses, embankment design, integrated facilities design, and borrow area delineation and quantity estimation. These investigations were used in the construction methods and cost estimation work and in the risk analysis.

Flooding Analysis: The purpose of the flooding analysis was to address the vulnerability and reliability of the existing conditions and In-Delta Storage re-engineered project under flood events. Freeboard requirements at Webb Tract and Bacon Island reservoirs were evaluated and embankment crest elevations of the reservoir islands were designed to protect the embankments from overtopping. Embankment breach analyses were also performed. The objective of this activity was to provide sufficient input to estimate the impacted areas and to quantify the consequences of failure from an uncontrolled release. Estimates for the probability of the re-engineered project embankments overtopping were completed as a part of the risk analysis.

Seismic Analysis: Under the seismic analysis, dynamic response analyses of the embankments were performed to calculate time histories of seismic-induced inertial force acting on the critical sliding masses. Seismic-induced permanent deformations of the embankments were estimated for the three ground motion levels selected for this study. The estimated deformations and their associated ground motion levels were used to evaluate the seismic risk of the proposed embankment alternatives and the probabilities of failure were estimated.

Embankment Design: Under the embankment design analysis, the vulnerability and reliability of the existing conditions and In-Delta Storage re-engineered project embankments were evaluated under operational demands by conducting extensive seepage and stability analyses.

Steady-state seepage conditions through transverse sections of the existing levees and re-engineered embankments at Webb Tract and Bacon Island were estimated and seepage control alternatives were analyzed.

The re-engineered project (“rock berm” and “bench”) embankment options have been evaluated by extensive stability analyses of the two sections selected to be representative of the lowest and highest elevations at which the base of the underlying peat layer is found in the two islands. Conditions evaluated in the stability analysis include end-of construction, long-term operation, sudden drawdown, and pseudo-static. Factors of safety were calculated and compared to the project's stability criteria, and the adequacy of the proposed project in regard to embankment stability was evaluated.

To meet the USBR Risk Analysis requirements, it was decided that the potential for erosion and piping had to be addressed. The probability of erosion and piping failures was determined and six alternatives were considered as solutions to reduce the chance for erosion and piping to occur. On the basis of factors that can contribute to erosion and piping, areas requiring control were identified and an evaluation was performed to select a preferred measure.

Borrow Area Delineation and Quantity Estimation: This investigation included identifying feasible borrow sites within Webb Tract and Bacon Island, assessing the suitability of the soils as borrow materials for earthwork, estimating the volume of borrow materials available from each identified location, and comparing the total quantity of suitable borrow material available at each island with the earthwork planned at the island.

Integrated Facilities Design: This investigation included developing a feasible design and layout for the integrated facilities. There are a total of four integrated facilities, two on Webb Tract and two on Bacon Island. The integrated facilities are consolidated control structures that will be used to control the diversion and release of water onto and off of the reservoir islands. A direct connection to Clifton Court Forebay from Bacon Island was also analyzed.

A number of hydraulic analyses were conducted to determine the overall layout of the integrated facilities. The objectives of the hydraulic analyses were to determine the size and optimize the configuration of the integrated facility components, and to develop flow rating curves for each integrated facility showing the percentage of time the design flow can be met by gravity flow only, pumped flow only, or a combination of gravity and pumped flow.

Mechanical designs were prepared for the pumping plants, conduits and gate structures and an electrical analysis was performed to size the transformers required to supply power to each integrated facility.

Structural analysis and design was prepared in sufficient detail to allow a feasibility-level cost estimate of the four proposed integrated facilities to be completed. In particular, structural analysis and design was completed for the structural components of the fish screen structure, the three gate structures, structures associated with the pumping stations and conduits, and for the sheet pile walls.

1.3.3 Construction Methods and Cost Estimation

The third phase of work was to analyze suitable construction methods, perform construction scheduling and estimate total project construction costs related to construction of both the "rock berm" and "bench" embankment options and construction of the four integrated facilities. Information developed under the construction methods and cost estimation work was used in the risk analysis.

Under the embankment construction methods and cost estimate investigation, quantity estimates for slope protection, piping protection and seepage control (pumping wells) were developed, but estimation of borrow material and fill quantities for all embankments were covered under the borrow area investigation.

Under the integrated facilities construction methods and cost estimate investigation, quantity estimates were developed for all integrated facility components, which include the fish screen facilities, gate structures, pumping stations, conduit pipes and associated outlet structures, bypass channel bridge structures, and sheet pile walls.

Applicable methods for constructing the various embankment, earthwork, and integrated facility components were reviewed and the most feasible methods were evaluated. Details on task sequencing and overall construction scheduling were also developed.

Market research was performed, including quotations from contractors and suppliers, to obtain relevant unit costs for acquiring different construction materials and transporting them to the project site, and the cost of labor and equipment required for placement of these materials, as applicable. Feasibility-level cost estimates were then prepared for constructing the earthwork components for the two embankment options and for constructing the four integrated facilities.

To include the impact of global warming and climate change, resulting sea level rise was considered for engineering costs estimates. Based on climate impact studies conducted by various agencies, climate change may cause a slow rise of 0.5 feet in the Delta water levels over the 50-year life of the project. This rise can be easily handled by normal embankment maintenance operations over the next 50 years and no additional costs were included in the cost estimates.

1.3.4 Risk Analysis

The fourth and final phase of work was to complete an overall risk analysis. The purpose of the risk analysis was to evaluate the risk and consequences of failure of the existing levees and In-Delta Storage re-engineered project embankments and integrated facilities under all loading events (operational, seismic, and flooding) and estimate the loss-of-life risk and economic losses through uncontrolled releases. The risk analysis was conducted in accordance with the general USBR risk analysis guidelines. The results of the analysis were used to evaluate the expected project performance relative to the “no action” alternative (i.e., existing levees).

To begin with, information regarding historical losses in past levee failures was reviewed and compiled. Then the zone of impact under each risk scenario was assessed and the environment and resources that would be impacted were identified. Finally, the economic losses associated with the consequences of an inward breach, an outward breach, and flooding a neighboring island were evaluated and the dollar values associated with these economic losses were estimated.

As mentioned above, the engineering investigations are all interlinked and all of them build upon information from others. Figure 1.2 is a flow diagram showing the interlinking relationships of the engineering investigations.

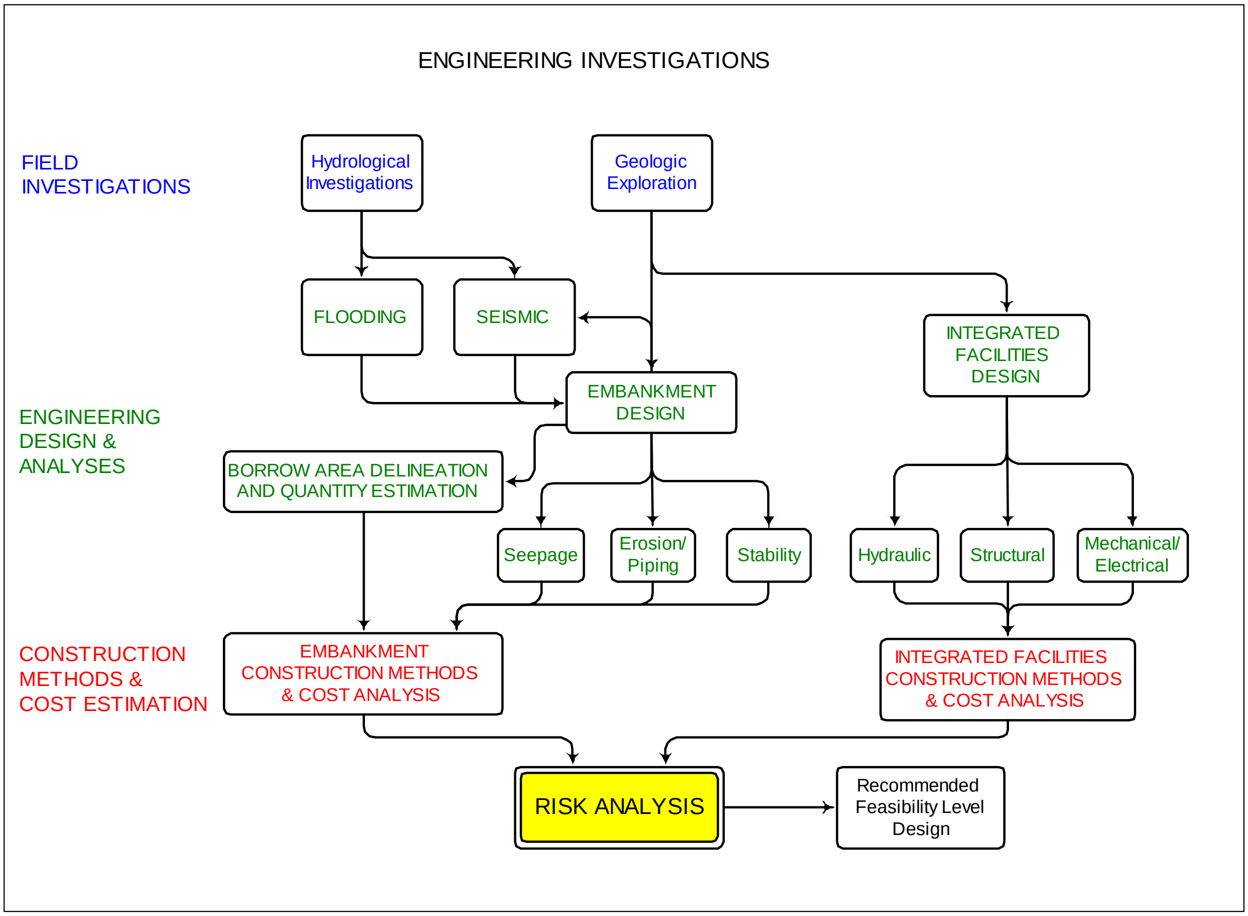


Figure 1.2 – Engineering Investigations Flow Chart

1.4 Independent Engineering Review

The Department of Water Resources retained a board of three consultants to review the feasibility level engineering designs for the proposed embankments and integrated facilities for the In-Delta Storage Program. The second meeting of the Independent Board of Consultants (IBC or Board) for the In-Delta Program was held on May 28 – 30, 2003. The Board was thoroughly briefed on all aspects of the feasibility level designs for In-Delta Storage Project features on May 28, 2003. May 29 was spent in the Delta visiting Webb Tract and the Brown Sand, Inc. pit for a demonstration of its below water level excavation procedures. The Board prepared their second report, *Independent Board of Consultants Report No. 2*, and presented their findings and recommendations to DWR on May 30. The IBC Report No. 2 is provided in Appendix A of this report for reference.

DWR has incorporated all of the Board's recommendations into the engineering investigations completed for the In-Delta Storage Program. The Board's recommendations are summarized below, along with the report name and location in which these can be found.

Embankment Design

- There may be both geometric and environmental difficulties in placement of the outboard continuous rock berm along some sections, and this will likely require use of the alternative "bench" configuration along some reaches. These locations should be identified and clearly delineated, and cost estimates should reflect mixed use of both types of sections.

Draft Earthwork Construction Cost Estimate. Section 2.2
Draft Engineering Investigations Summary. Section 1.5.2 and Section 8.2.1
Draft Executive Summary. Section 5.3.2.1

- Another issue that warrants additional consideration is the minimum freeboard required along the various reaches of the proposed embankments. In the cases wherein the embankment crest height is controlled by the combined considerations of water level plus wind driven wave set-up and ride-up, some additional minimum freeboard should be provided above the maximum run-up level.

  Draft Flooding Analysis. Section 3.3
  Draft Engineering Investigations Summary. Section 3.3.3

Embankment Erosion and Seepage Control

- Protection against piping at the reservoir side toe (and low on the embankment face) is a critical issue during the periods when the reservoirs are lower than the water levels in the adjacent sloughs…Addition of a second level of geotextile filter higher up, nearer to the final face of the final embankment, and at a later stage (after much of the initial settlement has occurred) would provide a significantly increased level of protection, and at relatively low cost.

  Draft Earthwork Construction Cost Estimate. Section 2.2 and Section 3.1

- A second type of erosion on the inboard (reservoir side) faces is potential erosion of the embankment faces due to both wave and wind forces…An alternative would be to provide some level of inboard side erosion protection, either over large areas or over selected areas of special importance or vulnerability. Prevailing winds, and storm winds, will be the key forces driving both wind and inboard wave erosion potential, and provision of coarse granular covers (gravels and/or rock) could be applied over selected reaches.

  Draft Earthwork Construction Cost Estimate. Section 2.2 and Section 3.1
  Draft Engineering Investigations Summary. Section 8.2.1 and Section 8.2.4
  Draft Executive Summary. Section 5.3.2.2

Integrated Facilities Design

- The embankments for the Integrated Facilities should be placed on mineral soils. An excavation plan needs to be developed for each Integrated Facility site to show what the rough excavation plan of the site will be before the layout is finalized for all the embankments surrounding the transition pool, midbay, bypass channel, and other compacted fills for structures. This plan of excavation should be a construction stage site plan that shows and describes the location of the sheet piling cofferdams, and dewatering facilities.

  Draft Earthwork Construction Cost Estimate. Section 5.3 and Appendix C
  Draft Engineering Investigations Summary. Section 8.3 (and subsections)
  Draft Executive Summary. Section 5.6

- It is important to emphasize the need for a hydraulic model design study during final design phase for the Integrated Facilities. This will be important to finalize design for the fish screen, the transition pool geometry, and the other hydraulic structures, as well as the specific setback location from the existing levee alignment.

  Draft Integrated Facilities Engineering Design and Analyses. Section 1.6
  Draft Engineering Investigations Summary. Section 1.5.4

Construction Methods and Cost Estimates

- A description of the most feasible construction methods suitable for the project should be developed. Additionally, an overall construction schedule needs to be presented, which summarizes the sequence of construction planned for the construction of embankments for Webb Tract and Bacon Island and for each of the Integrated Facilities. The costs required to dewater and maintain stability of the Integrated Facility sites during the rough excavation stage and the costs of the rough excavation and fill quantities should be included.

  Draft Earthwork Construction Cost Estimate. Section 5.3 and Appendix C
  Draft Engineering Investigations Summary. Sections 8.2.3, 8.3, 8.3.1 and 8.4.2
  Draft Executive Summary. Section 5.6

- Consider applying different contingencies to different project features to provide an overall contingency allowance that is representative of the feasibility level designs.

  Draft Engineering Investigations Summary. Section 8.5.2.1
  Draft Executive Summary. Section 5.6.1

Additional Recommendations

- "It should be clearly noted in such summary documents that the current designs do not provide for assured non-failure of the proposed storage facilities during strong seismic loading. Instead, the risk of failures (or breaches) of the proposed reservoirs are considered in the current planning and design as an acceptable level of risk…"

  Draft Executive Summary. Section 5.7

1.5 Conclusions and Recommendations

1.5.1 Flooding

Wave Runup Conclusions:

- The results indicate that the maximum wind wave runup plus setup is 1.8 feet for Webb Tract and 1.4 feet for Bacon Island; therefore, the freeboard required for the embankments around both Webb Tract and Bacon Island is 3 feet on the design flood event. The embankments would need to have crest elevations of +10.1 feet at Webb Tract and +10.3 feet at Bacon Island to have sufficient freeboard. This provides an additional freeboard above the maximum 100-year flood elevation ranging from 1.3 to 2.5 feet at Webb Tract and from 1.7 to 2.5 feet at Bacon Island.
- Based on the above design conditions, the wave runup plus setup values on the reservoir sides were estimated to be 2.0 feet and 2.2 feet for Webb Tract and Bacon Island, respectively. Therefore, with maximum reservoir water storage elevation at elevation +4.0 feet, both reservoir islands would have sufficient freeboard.

Embankment Breach Conclusions:

- Model results show that during an outward breach, the water surface directly across from the breach rises significantly. Peak velocities are observed on either side of the breach near the banks of the adjacent island levees. As would be expected, velocities are relatively small on either side of the breach adjacent to the reservoir island embankment due to the formation of eddies. During an inward breach of the reservoir, a similar flow pattern results, but the flow direction is reversed.
- During hypothetical outward and inward breach failures peak velocities are highest in narrow sloughs and velocities are lowest in wide sloughs. The results show that the levees adjacent to

narrow and medium slough sections would fail should the reservoir breach outward under the scenarios analyzed. Levee sections adjacent to wide slough sections would also fail during the outward breach except under the most favorable scenario analyzed. Under an inward breach failure, the adjacent island levees would not fail where the typical slough widths are medium or wide. However, the adjacent island levee would fail where the typical slough width is narrow.

- There would be no adjacent island levee failures due to overtopping caused by an inward or outward breach of a reservoir island embankment.

1.5.1 Seismic

Recommendations:

- Because liquefaction would lead to large deformations that would affect overall stability of the embankment, further investigation and evaluation of the existing levee materials are recommended. Depending on the extent of the potentially liquefiable sands within the existing levee, removal of the loose sands may need to be implemented.
- Due to the limitations of the QUAD4M computer program for large earthquake loads, a uniform assumption has been made for estimating the expected embankment deformation. Although this assumption is considered conservative, a more rigorous non-linear analysis would probably be useful and could provide more insight into the deformation patterns associated with large strains under the large earthquake shaking. This analysis could also provide more insight into the comparative performance of the embankment alternatives under the larger earthquakes.

1.5.2 Embankments

The Rock Berm Option, in combination with about 3,000 lineal feet of Bench Option, was chosen as the recommended design. The more expensive Bench Option would be used in areas where the slough is deep, the embankment slope on the slough is currently too steep to adequately place rock, or the placement of the rock may block a portion of the channel.

Seepage Conclusions:

- Seepage control by interceptor wells placed on the levees of the reservoir islands, as proposed, appears effective to control undesirable seepage effects. Other seepage control alternatives should be further investigated because of their potential engineering merits.

Seepage Recommendations:

- Increases in the permeability of the sand layer significantly increase calculated seepage volumes. Site specific pump tests located at potential seepage area on Webb Tract and Bacon Islands are recommended for design of the interceptor system.
- Pilot test borings should be drilled along those portions of Bacon Island and Webb Tract where interceptor wells are planned. Data gathered from the borings should be used for final design of the well system.
- During final design, Webb Tract and Bacon Islands should be surveyed for potential seepage problem areas. Potential seepage areas should be analyzed individually using parameters obtained from pump tests and additional borings.
- Test interceptor well sections should be installed and tested based on data collected from pump tests and pilot borings. Results of the test sections should be incorporated into the final design.

Stability Conclusions:

- Construction of the levee strengthening fills must be implemented in a manner to prevent stability failures due to the new fill loads. This will require carefully planned staged construction, and monitoring to observe the behaviors as the fill is placed. The staged construction will require a construction period estimated to extend over 4 to 6 years.
- Both the “rock berm” and “bench “option” can be constructed to meet the project’s required stability criteria. For some combinations of existing reservoir bottom elevation and base of peat elevation reservoir-side slope free draining toe berms are required to meet stability criteria.
- Based on the stability analysis presented in this section, the “rock berm” option appears to provide several advantages over the “bench” option as follows:
    - Calculated factors of safety for all analysis cases are greater than calculated for the “bench” option suggesting a lower probability of failure during normal operations.
    - Calculated yield accelerations are generally greater than for the “bench” option suggesting less earthquake induced deformation.
    - Fill volumes for new embankments are significantly less due to less consolidation deformation under new embankment and the absence of setback.

Stability Recommendations:

- Implement an extensive subsurface exploration program along the reservoir island levees, followed by stability evaluations and site-specific detailed design and construction to provide adequate embankment stability during design. These steps will be essential to achieve safety and effectiveness of the proposed embankment system.
- Conduct a survey of Webb Tract and Bacon Island to determine the extent and thickness of existing rockfill on the slough-side slopes. Where rockfill exists on the slough-side slopes, rock berm slopes required to meet stability criteria may be reduced.
- Implement a test fill section during design for the preferred embankment geometry at locations where the base of peat is located at elevations –20 feet and –40 feet. The test fill program would provide valuable information regarding consolidation rates and ultimate settlement for estimating the time required for staged construction.
- Include in the final design a filter fabric between the new embankment and existing levee to provide piping protection for materials that are up-gradient of the fabric. Determination of the locations along the reservoir embankments for filter fabric as a piping mitigation measure should be made during future engineering studies.

1.5.3 Borrow Areas

Conclusions:

- The estimated borrow material volumes available within 15 feet of the ground surface are sufficient to accommodate construction of the island embankments.

Recommendations:

- For further development of the In-Delta Storage embankments, supplemental drilling, laboratory testing, and CPT soundings should be performed in the potential borrow areas. Standpipe piezometers should be installed in selected borings to measure groundwater levels.

1.5.4 Integrated Facilities

- CVFFRT has evaluated the proposed fish screen facilities and agrees with the overall concept. DWR should organize a technical review committee for fish screen review during the final design phase.
- Sensitivity studies should be conducted to optimize the configuration, size, and elevation of the inlet and outlet structures, the pumping plant, and the conduit pipes.
- The design and layout of the integrated facilities is considered to include sufficient detail for a feasibility level assessment of cost. Physical hydraulic model design studies should be conducted during final design of the integrated facilities. This, along with a technical fish screens review committee, will be important to finalize the design of all integrated facility components and to determine the specific setback location from the existing levee alignment.
- Given the planned configuration of the pumping plant, a partial vacuum may form within the piping downstream of the pumps when the pumps are shut down. An analysis should be performed to determine the maximum vacuum that may occur and the pipe thickness should be sufficient to avoid collapse. This analysis will require dynamic modeling.
- An area assessment should be performed by PG&E to develop accurate distances to the nearest utility source that can handle the In-Delta Storage project's anticipated load and to determine the feasibility and cost associated with connecting power to the integrated facility sites.
- Additional analysis should be performed to refine the design of the conduit pipe outlet structures.
- Further structural engineering studies should be conducted to refine the design and extent of piles needed to support the integrated facility structures. The amount and extent of piles required may be reduced since the peat soils will be removed in the vicinity of the integrated facilities.

1.5.5 Construction Methods and Cost Estimation

Conclusions:

- The Rock Berm Option was found to cost about $69 million (excluding contingency) less than the Bench Option.
- The estimated construction cost, with contingencies, for the embankments ("Rock Berm" option using soil cement on 10:1 slopes), seepage control system, instrumentation, integrated facility embankments and structures, and miscellaneous items is $774.4 million.
- Based on the construction approach and the construction schedule that was developed for the In-Delta Storage project ("Rock Berm" option), it is estimated that 2 years would be required for engineering and final design and for the bid and award process, and 6 years would be required to construct the entire project (embankments, seepage control systems, and integrated facilities).

Recommendations:

- It is understood that earthwork construction to buttress Delta levees has not required dewatering of the borrow area excavations. Based on this experience, costs for well-point dewatering systems for excavation in the borrow areas were not included. However, pumping from the existing groundwater control system would continue throughout construction. Further design development should include field test excavations in the borrow areas at both Webb Tract and Bacon Island to confirm that dewatering systems are or are not needed for borrow excavations. This field test work should also include assessments of effort required to dry out the borrow materials sufficiently for use in embankment construction. The results of the field test work would be used to improve the reliability of the cost estimates. The costs associated with maintaining and operating the existing groundwater control system during construction should also be assessed and included the cost estimates.

- Overburden excavation has a significant effect on construction costs, especially for Bacon Island. Further field investigations of the borrow areas are recommended to better define the available material quantities and characteristics, and to confirm the required overburden excavation at the islands. These field tests would also be used to assess whether borrow excavations should be extended below the 15-foot limit used in the cost estimates for this study. Deeper excavations could be more efficient considering the amount of required overburden excavation.
- Further investigations should include a survey of the slough-sides of the levees to confirm the amount and extent of existing rockfill. This information would be used to evaluate where additional rockfill would be required.

1.5.6 Risk Analysis

- The expected dollar loss with flooding under existing conditions is large because multiple levee failures could occur during a period of 50 years under existing conditions. The total expected economic losses from flooding events, when added to other losses, results in the expected dollar risk of $131 million at Webb Tract under existing conditions. Similar calculations for Bacon Island result in the expected dollar risk of $177 million under existing conditions.
- The failure probability for the existing levee is higher than for the re-engineered alternatives by factors of 6 to 8. The expected dollar risk (without considering the loss of current resources on the project island) for the existing levee is higher than for the re-engineered alternatives by factors of 2 to 6.
- The probability of failure is about the same for the two embankment alternatives at both project islands. However, the expected dollar risk for the Rock Berm alternative is lower by about 30% than for the Bench alternative at both Webb Tract and Bacon Island. Additionally, the expected number of fatalities for the Rock Berm alternative is lower than for the Bench alternative by a factor of about 2.5 to 3, at both Webb Tract and Bacon Island.
- The failure probabilities, expected dollar risks, and expected number of fatalities for each alternative are about the same for both islands.
- The three loading events contribute to the overall failure probability and risk for each project alternative at the two project islands. For the two re-engineered alternatives, the operational loading contributes only 1% to 2% to the failure probability and expected dollar risk. The flooding and seismic loading contributes about 40% and 60%, respectively, to the failure probability and expected dollar risk for the re-engineered alternatives. Seismic loading contributes to the majority of the expected number of fatalities for the re-engineered alternatives. Flooding does not contribute to the fatality risk, because only an inward breach is possible under flooding and the fatality risk under an inward breach is negligible.

- For the existing levees at the candidate project islands, flooding contributes 62% to 74% to the failure probability. This is because of the relatively low crest elevation of the existing levees such that a 100-year flood is likely to cause overtopping. For the expected dollar risk for the existing levees, the operational loading has a major contribution, because of the potential water supply interruption from an inward breach of the existing levees.

Chapter 2: Field Investigations

2.1 Introduction

Field investigations that were conducted as part of the In-Delta Storage Program State Feasibility Study include hydrological investigations and geologic explorations. The hydrological investigations included literature review and a tidal analysis of river stages. Geologic explorations were conducted to determine the soil properties of potential borrow sources on the reservoir islands and to evaluate the integrated facility foundation materials. The information obtained from these investigations was used in the engineering designs and analyses conducted for this study.

2.2 Hydrological Investigations

The integrated facilities are located in tidally influenced areas where river stages vary in quantity and direction hourly and seasonally. To perform hydraulic analyses and design of the integrated facilities, information about river stage variation was required. Detailed statistical analyses of the available stage data were conducted to obtain historical distributions of the tidal stages near the integrated facility locations.

2.2.1 Tidal Analyses of River Stages

Stage variations in the channels adjacent to the integrated facilities were obtained from a DSM2 computer model simulation. For each day, high-high, low-high, low-low and high-low stages were extracted. Examples of these stage levels are presented in Figure 2.1. Statistical analyses were carried out to determine the mean, median, minimum, maximum and standard deviation of stages at each facility location. The results of these analyses are summarized in Tables 2.2(a) through 2.2(d).

Frequency analyses of the time series were carried out to determine the stages having different levels of probability of occurrences. Tidal stages and the probability occurrences are summarized in Table 2.2. The probability plots of the stages are shown in the Integrated Facilities Engineering Design and Analysis Report, Appendix A, Figures A.1 through A.4.

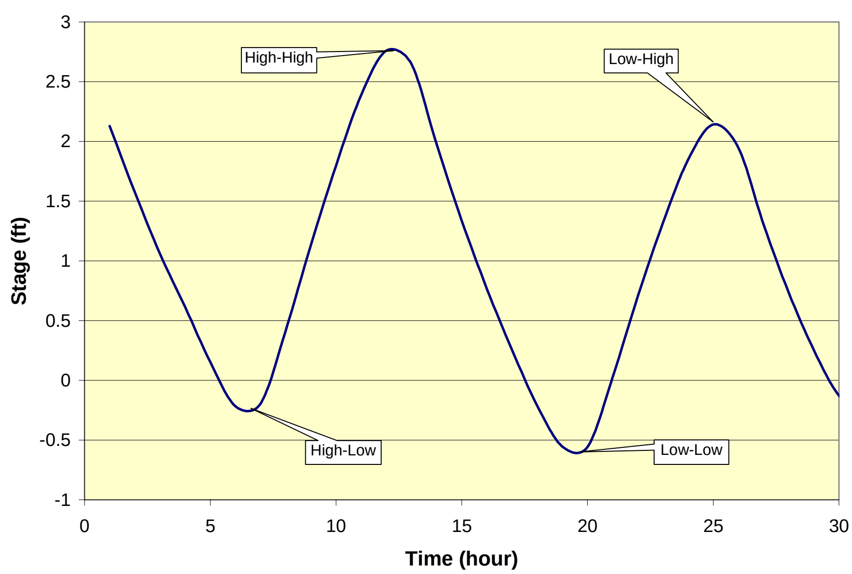


Figure 2.1 – Definition Figure for Tidal Stage

Table 2.1 – Summary of Statistical Analyses of Stage Time Series

(a) Webb Tract at San Joaquin River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.826	5.810	5.003	5.003
Minimum	1.331	0.639	-1.714	-1.398
Mean	3.109	2.237	-0.651	0.125
Median	3.080	2.129	-0.768	0.004
Std. Dev	0.615	0.527	0.552	0.641

(b) Webb Tract at False River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.36	5.57	4.21	4.87
Minimum	0.91	0.50	-1.54	-1.30
Mean	2.95	2.05	-0.44	0.36
Median	2.93	1.98	-0.54	0.27
Std. Dev	0.59	0.51	0.51	0.59

(c) Bacon Island at Middle River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.761	5.767	4.397	4.989
Minimum	1.325	0.687	-1.686	-1.368
Mean	3.16	2.27	-0.60	0.18
Median	3.139	2.181	-0.713	0.058
Std. Dev	0.61	0.52	0.53	0.62

(d) Bacon Island at Santa Fe Cut

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.826	5.81	5.003	5.003
Minimum	1.331	0.639	-1.714	-1.398
Mean	3.11	2.23	-0.65	0.12
Median	3.08	2.129	-0.768	0.004
Std. Dev	0.62	0.53	0.55	0.64

Table 2.2 – Exceedance Probability and Corresponding Stages at Intake Site

Facility Location	Tidal Stage (ft)			
	90% Low-Low	10% High-High	10% Low-High	50% Low-High
Webb Tract, San Joaquin River	-1	3.8	2.75	2.1
Webb Tract, False River	-1	3.8	2.75	2.1
Bacon Island, Middle River	-1.1	3.9	2.9	2.2
Bacon Island, Santa Fe Cut	-1.1	3.9	2.9	2.2

2.3 Geologic Exploration

The purpose of the geologic explorations was to determine the soil properties of potential borrow sources on the reservoir islands and to evaluate the integrated facility foundation materials. The geologic data obtained from these explorations was used in the embankment design, borrow area investigations and integrated facilities structural design.

Geologic explorations for potential borrow sources and integrated facility foundation evaluations were conducted on Webb Tract and Bacon Island by DWR, USBR, and URS. The geologic explorations were conducted in two phases. Phase I was conducted by USBR during August and September 2002 and consisted of Cone Penetrometer Test (CPT) borings ranging from 28 to 101 feet in depth. CPT soundings of 28 to 52 feet in depth were used for the characterization of borrow areas and materials on both islands, while 85 to 101 foot deep soundings were used determine foundation conditions beneath the proposed integrated facilities. Phase II of the investigation was conducted by DWR during September and October 2002 and consisted of drilling and sampling one 100 foot drill hole at each of the four integrated facility sites. These drill holes were also used to determine foundation conditions beneath the proposed integrated facilities. URS conducted additional explorations as a part of their borrow area investigations on Webb Tract and Bacon Island in December 2002. This included 20 drill holes (10 per island) ranging in depth from 15 to 19 feet below the existing ground surface.

After completion of the Phase I and Phase II field work, the CPT and bore-hole logs were compiled and used to develop geologic cross sections and isopach maps showing the thickness of soft and/or organic soils overlying potential borrow materials. Laboratory testing was then conducted on samples from the integrated facility locations by DWR’s Division of Engineering, Civil Engineering Branch. The URS exploratory boring samples were tested to evaluate their engineering properties for use in borrow material evaluations.

The CPT logs and the laboratory testing data are presented in the following In-Delta Storage Program draft reports completed by DWR in January 2003: 1) Results of Geologic Exploration Program and 2) Results of Laboratory Testing Program.

The locations of the CPT soundings are shown in Chapter 6, on Figure 6.1 (Webb Tract) and Figure 6.2 (Bacon Island).

Chapter 3: Flooding Analysis

3.1 Introduction

The purpose of the flooding analysis was to address the vulnerability and reliability of the existing conditions and In-Delta Storage re-engineered project under flood events. Freeboard requirements at Webb Tract and Bacon Island reservoirs were evaluated and embankment crest elevations of the reservoir islands were designed to protect the embankments from overtopping. Embankment breach analyses were also performed. The objective of this activity was to provide sufficient input to estimate the impacted areas and to quantify the consequences of failure from an uncontrolled release. Estimates for the probability of the re-engineered project embankments overtopping were completed as a part of the risk analysis.

3.2 Criteria and Parameters

3.2.1 Data Review

Historical data including flood and tide elevations in the Delta region were obtained from previous studies conducted by CALFED, DWR, U.S. Army corps of Engineers (USACE) and URS. Hydraulic data for the 50-, 100-, and 300-year flood events in the Delta were obtained from CALFED and USACE. These design flood stage data are presented in Table 3.1.

Table 3.1 – Flood Stage Data Estimated by CALFED and USACE

Reservoir Island	Average Design Flood Stage (feet – NGVD 1929)			
	50-year	100-year		300-year
	USACE	USACE	CALFED	USACE
Webb Tract	6.8	7.0	6.9	7.2
Bacon Island	6.9	7.2	7.2	7.5

3.2.2 Analysis Parameters

3.2.2.1 Embankment Geometry

The embankment crest of the proposed reservoir islands at Webb Tract and Bacon Island will be constructed to at least elevation +10.0 feet. For embankment sections adjacent to Franks Tract and Mildred Island, the following geometric shapes will be used for the slough side of the embankment:

- At Franks Tract and Mildred Island: A bank slope of 3:1 (H:V) with no berms (rock-berm option)
- At Franks Tract: A composite bank slope with a horizontal berm at elevation + 2.0 feet, the slope below the berm as 2.14:1 (H:V) and above the berm as 3:1 (H:V)
- At Franks Tract: A composite bank slope with a horizontal berm at elevation + 6.0 feet, the slope below the berm as 2.14:1 (H:V) and above the berm as 3:1 (H:V)

- At Mildred Island: A composite bank slope with a horizontal berm at elevation + 3.0 feet, the slope below and above the berm as 3:1 (H:V).

For slough side slopes of embankment sections that are not adjacent to Franks Tract and Mildred Island, two geometric options will be considered as follows:

- Rock-berm option with a bank slope of 3:1 (H:V) with no berms.
- Bench option with varying bench elevations and widths such that average slope ranges from approximately 3:1 (H:V) to 5:1 (H:V)

The embankment slope on the reservoir side is designed as 3:1 (H:V) above the maximum water surface elevation (WSEL) of +4.0 and 10:1 (H:V) below.

3.2.3 Analysis Criteria

3.2.3.1 Reservoir Stages and Slough Water Levels

Minimum and maximum storage water levels in Webb Tract and Bacon Island and minimum, average and average-high tide levels in the surrounding sloughs that were used in the analyses are presented in Table 3.2.

Table 3.2 – Reservoir Stages and Slough Water Levels

Reservoir Island	**Webb Tract**	**Bacon Island**
Minimum WSEL in Reservoir	-8.0 (1)	-8.0 (1)
Maximum WSEL in Reservoir	+4.0	+4.0
Minimum Tide Level in Slough	-1.0	-1.0
Average Tide Level in Slough	+1.5	+1.5
Average-High Tide Level in Slough	+3.5	+3.5

(1) This condition exists for about five months per year during the periods of emptying and filling of the reservoir (URS, 2001).

3.2.3.2 Breach Evaluation Criteria

Three geometric configurations were evaluated in the breach analyses to determine potential impacts on adjacent levees. Both reservoir islands are surrounded by sloughs of varying widths and depths. For the breach analysis, the sloughs surrounding Webb Tract and Bacon Island are categorized into three groups: narrow, medium, and wide. The slough bottom elevations range from about -18 to -50 feet and average about -25 feet. To provide conservative peak velocity estimates, the slough bottom elevations were set at -25 feet for the three typical channel widths. Slough widths and bottom elevations used in the analysis are presented in Table 3.3.

Table 3.3 – Typical Embankment Geometry for Breach Analysis

Reservoir Island	Crest Elevation	Slough Bottom Elevations			New Res. Bottom Elevation
		Narrow (400 ft)	Medium (1,000 ft)	Wide (3,000 ft)	
Webb Tract	+10.0	-25.0	-25.0	-25.0	-20.0
Bacon Island	+10.0	-25.0	-25.0	N/A	-18.0

To evaluate the impacts of the embankment breach scenarios on adjacent levees, a velocity of 5 feet per second (fps) was used as the threshold for failure of adjacent levees. To evaluate the impacts of overtopping of adjacent levees during a hypothetical reservoir breach, it is assumed that the crest elevations of adjacent levees are 8.0 feet.

3.2.3.3 Freeboard Criteria

The embankment crest elevations shall be the larger of the following two criteria (CALFED, 2002):

- The maximum reservoir water storage elevation (+4 feet MSL) plus the wind wave runup plus setup on the reservoir. If wind wave runup plus setup is less than 3 feet, then a freeboard of 3 feet should instead be added to the maximum water storage elevation, or
- The water surface elevation of the design flood event on the slough side plus the wind wave runup plus setup. If the wind wave runup plus setup is less than 3 feet, then a freeboard of 3 feet should instead be added to the water surface elevation of the design flood event.

3.3 Wave Runup and Reservoir Setup Analyses

3.3.1 Methodology

Freeboard requirements at Webb Tract and Bacon Island reservoirs were evaluated based on design flood stages and wind wave characteristics estimated for the Sacramento-San Joaquin Delta region. Using this information, embankment crest elevations of the reservoir islands were designed to protect the embankments from overtopping due to extreme flooding and wind loading conditions on the surrounding water bodies.

3.3.2 Wind Wave Analysis

Wave runup analyses for sloughs surrounding Webb Tract and Bacon Island were performed to estimate freeboard requirements for the reservoir embankments. Wave runup is defined as the vertical height above still-water level (SWL) to which water from an incident wave will run up the face of a structure. The wave runup analyses involved estimating wave characteristics such as wave height and wave period from wind velocities and reservoir fetch length.

3.3.2.1 Effective Fetch Length

The fetch lengths of the water bodies surrounding Webb Tract and Bacon Island reservoirs vary and were categorized into three typical lengths: short, medium, and long. Table 3.4 provides the approximate station locations of typical short, medium, and long fetch length categories and the stations adjacent to Franks Tract and Mildred Island. Figures 3.1 and 3.2 show the station locations (described in Table 3.1) for the Webb Tract and Bacon Island reservoirs, respectively.

Table 3.4 – Embankment Station Locations at Webb Tract and Bacon Island Reservoirs

Reservoir Island	Levee Station (feet)			
	Adjacent to Franks Tract and Mildred Island	Slough Section with "Short" Fetch Length	Slough Section with "Medium" Fetch Length	Slough Section with "Long" Fetch Length
Webb Tract	70+00 to 220+00(1)	590+00 to 680+00	0+00 to 70+00 220+00 to 290+00	290+00 to 350+00 350+00 to 590+00
Bacon Island	60+00 to 200+00(2)	200+00 to 250+00 620+00 to 700+00	0+00 to 60+00 250+00 to 350+00 350+00 to 570+00 570+00 to 620+00 700+00 to 750+00	N/A

(1) Section adjacent to Franks Tract
(2) Section adjacent to Mildred Island

The maximum effective fetch length for each category, in addition to the sections adjacent to Franks Tract and Mildred Island, were measured and used in the analysis (see Table 3.5).

Table 3.5 – Effective Fetch Length at Webb Tract and Bacon Island Reservoirs

Reservoir Island	Effective Fetch Length (miles)			
	Adjacent to Large Water bodies	Slough Section with "Short" Fetch Length	Slough Section with "Medium" Fetch Length	Slough Section with "Long" Fetch Length
Webb Tract	3.22 (1)	0.34	0.60	1.29
Bacon Island	2.04 (2)	0.39	0.69	N/A

(1) Effective fetch length adjacent to Franks Tract
(2) Effective fetch length adjacent to Mildred Island

3.3.2.2 Design Wind Velocity

Wind velocities for the "fastest mile of record" were obtained from generalized charts published by USACE (1976) and USBR (1981) and used to calculate average wind velocities associated with the minimum wind duration required to generate the reservoir wind wave spectrum. The estimated fastest mile of record wind velocities at the reservoir sites for winter, spring, summer and fall are 60, 56, 40, and 60 miles per hour, respectively.

3.3.2.3 Wind Wave Runup

The estimate of wave runup requires both wind wave and reservoir embankment characteristics. These characteristics are (1) minimum wind duration to generate the wind wave spectrum, (2) average wind velocity over water, (3) wind stress factor, (4) significant wave height, (5) wave period, and (6) slope and roughness characteristics of the embankment face.

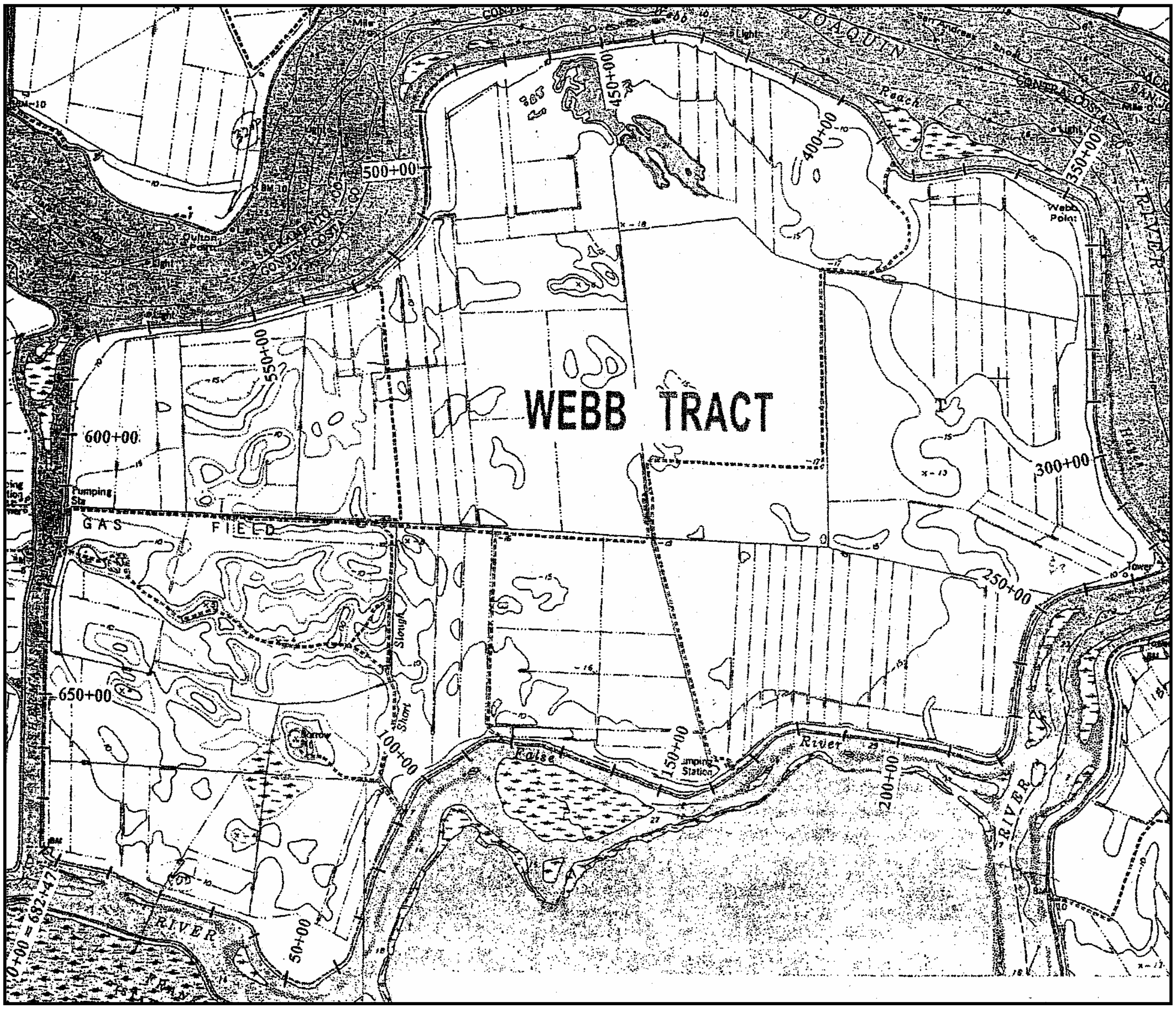


Figure 3.1 – Embankment Stations at Webb Tract Reservoir

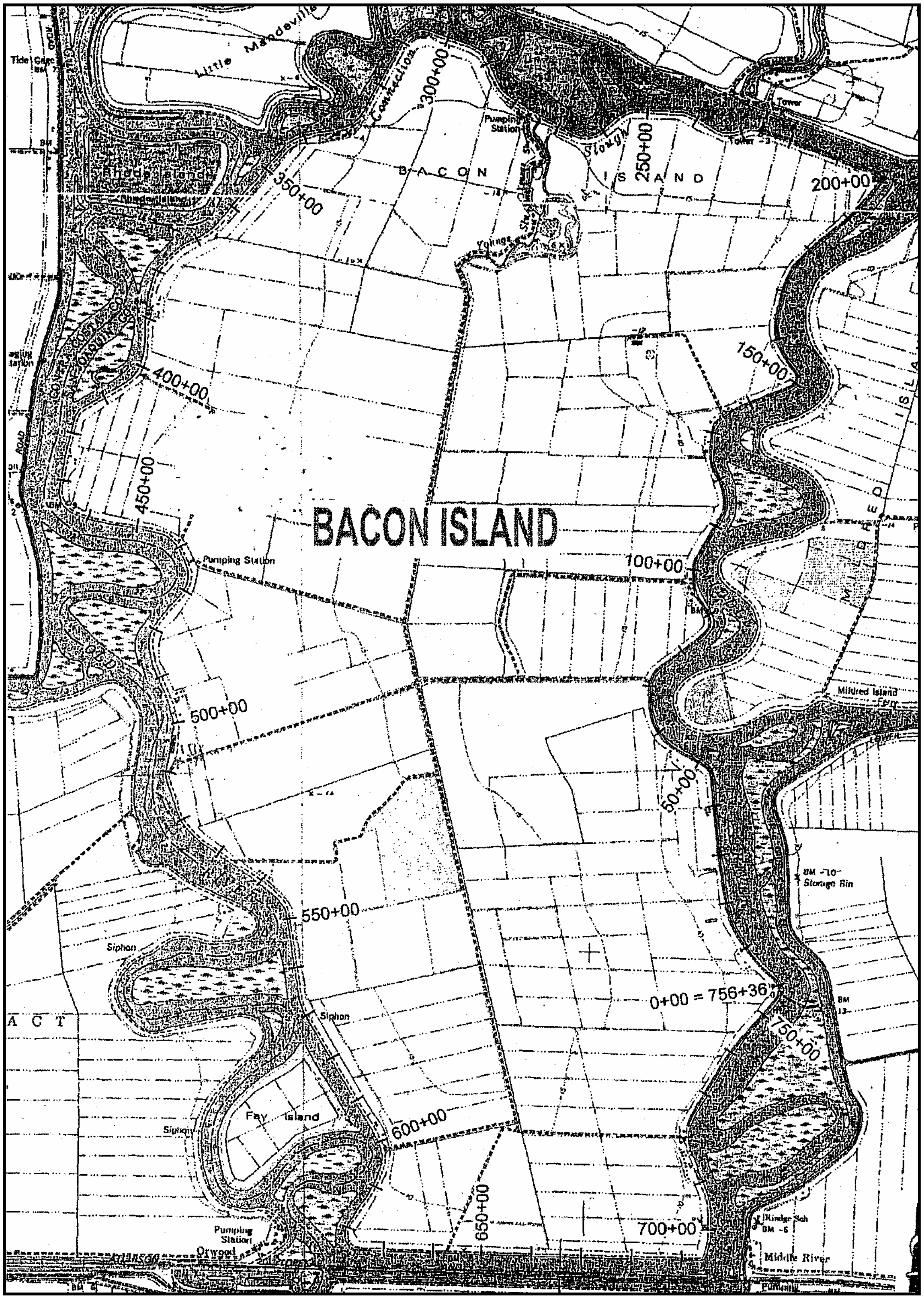


Figure 3.2 – Embankment Stations at Bacon Island Reservoir

Wind-wave runup values for this study are estimated using the embankment geometry described in Section 3.2.2.1 was used with three typical fetch lengths (short, medium, and long) for the slough side of the embankment with riprap armor in place. The remnant levees and marsh areas in Franks Tract were not considered in the wind wave run-up analysis. The full fetch across Franks Tract was used to calculate the wave runup on the slough side of the Webb Tract embankment.

3.3.2.4 Wind Setup

Wind setup is a general tilting of water surface due to shear stress caused by winds. Estimates of wind setup resulting from winds on the slough side of the Webb Tract and Bacon Island reservoirs were estimated using the procedure published by USBR (1981). The wind setup estimates require (1) average wind velocity over water, (2) effective slough side fetch length, and (3) average water depth at slough side.

3.3.3 Results

Wave action from wind, calculated by adding wave runup and wind setup, is used to evaluate the freeboard requirements at Webb Tract and Bacon Island reservoirs. Tables 3.6 and 3.7 present the 50-, 100-, and 300-year design flood stages (USACE, 1992), estimated wave runup plus setup values, and the resulting maximum flood elevations during 50-, 100-, and 300-year flood events at Webb Tract and Bacon Island reservoirs, respectively. As mentioned earlier, the freeboard requirement for the project is 3 feet on the 100-year flood stage or maximum wind wave runup plus setup, whichever is greater. The results indicate that the maximum wind wave runup plus setup is 1.8 feet for Webb Tract and 1.4 feet for Bacon Island; therefore, the freeboard required for the embankments around both Webb Tract and Bacon Island is 3 feet on the design flood event. The embankments would need to have crest elevations of +10.1 feet at Webb Tract and +10.3 feet at Bacon Island to have sufficient freeboard. This provides an additional freeboard above the maximum 100-year flood elevation ranging from 1.3 to 2.5 feet at Webb Tract and from 1.7 to 2.5 feet at Bacon Island. Tables 3.6 and 3.7 show that the crest elevations are also sufficient to prevent overtopping due to the 300-year flood event.

Table 3.6 – Estimated Wind Wave Runup and Reservoir Setup at Webb Tract Reservoir

Webb Tract Embankment Station	Wind Wave Runup + Setup (feet)	Design Flood Stage (USCAE, 1992) (feet)			Maximum Flood Elevation (feet) [Design Flood Stage + (Wind Wave Runup + Setup)]			Section used to Estimate Wave Runup *(See Tables 3.1 and 3.2)*
		50-year	100-year	300-year	50-year	100-year	300-year	
0+00 to 70+00	0.8 (1)	6.8	7.0	7.2	7.6	7.8	8.0	Medium
220+00 to 290+00	0.8 (1)	6.8	7.1	7.2	7.6	7.9	8.0	Medium
290+00 to 350+00	1.1 (1)	6.8	7.0	7.2	7.9	8.1	8.3	Long
350+00 to 590+00	1.1 (1)	6.8	7.0	7.2	7.9	8.1	8.3	Long
590+00 to 680+00	0.6 (1)	6.8	7.0	7.2	7.4	7.6	7.8	Short
70+00 to 220+00	1.8 (2) 1.8 (3) 0.6 (4)	6.8	7.0	7.2	8.6	8.8 8.8 7.6	9.0	Adjacent to Franks Tract

(1) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(2) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(3) A composite bank slope with a horizontal berm at + 2.0 feet and with quarrystone riprap.
(4) A composite bank slope with a horizontal berm at + 6.0 feet and with quarrystone riprap.

Table 3.7 – Estimated Wind Wave Runup and Reservoir Setup at Bacon Island Reservoir

Bacon Island Embankment Station	Wind Wave Runup + Setup (feet)	Design Flood Stage (USCAE, 1992) (feet)			Maximum Flood Elevation (feet) [Design Flood Stage + (Wind Wave Runup + Setup)]			Section used to Estimate Wave Runup *(See Tables 3.1 and 3.2)*
		50-year	100-year	300-year	50-year	100-year	300-year	
0+00 to 60+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
200+00 to 250+00	0.6 (1)	6.9	7.2	7.5	7.5	7.8	8.1	Short
250+00 to 350+00	0.8 (1)	6.9	7.1	7.5	7.7	7.9	8.3	Medium
350+00 to 570+00	0.8 (1)	6.9	7.2	7.5	7.7	8.0	8.3	Medium
570+00 to 620+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
620+00 to 700+00	0.6 (1)	6.9	7.3	7.5	7.5	7.9	8.1	Short
700+00 to 750+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
60+00 to 200+00	1.4 (2) 1.4 (3)	6.9	7.2	7.5	8.3	8.6 8.6	8.9	Adjacent to Mildred Island

(1) For average bank slope of 3:1 (H:V) with riprap (see Appendix B).
(2) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(3) A composite bank slope with a horizontal berm at +3.0 feet and with quarrystone riprap.

3.3.4 Reservoir Side Wave Runup and Setup Analyses

Wave runup and setup were also calculated for the reservoir sides of Webb Tract and Bacon Island to check the adequacy of the embankment freeboard due to wave action within the reservoirs. To estimate the wave runup and setup for the reservoir sides of Webb Tract and Bacon Island, the following design parameters were used:

- Calculated fetch lengths of 3.68 and 4.06 miles for Webb Tract and Bacon Island, respectively
- Fastest mile of record wind speed of 60 miles per hour
- Reservoir side embankment slope of 3H:1V above elevation +4.0 feet, with riprap armor assumed to be in place for both reservoir islands.

Based on the above design conditions, the wave runup plus setup values on the reservoir sides were estimated to be 2.0 feet and 2.2 feet for Webb Tract and Bacon Island, respectively. Therefore, with maximum reservoir water storage elevation at elevation +4.0 feet, both reservoir islands would have sufficient freeboard.

3.4 Embankment Breach Analysis

3.4.1 Methodology

- Embankment breach analyses was made to estimate peak discharges from inward and outward breaches of Webb Tract and Bacon Island reservoirs and the resulting peak velocities and water surface elevations that could occur in the adjacent sloughs.

As discussed in Section 3.2.3.2, the slough widths were categorized as wide, medium, and narrow. In the analyses, the following assumptions were made in simulating hydraulic flow conditions at the breach opening and the opposite levee facing the breach:

- Breach width of 400 feet was assumed based on previous dam breach studies for Webb Tract Reservoir (URS, 2000)
- Time to breach was assumed to be 1.0 hour
- The broad crested weir formula was used to calculate discharges through the breach opening under partially submerged conditions
- Under submerged conditions, the Bernoulli equation was used to calculate peak discharges across the breach opening accounting for head losses due to the sudden contraction and expansion of the flow through the breach
- The reservoir head during an outward breach considered the reduction in reservoir volume during the breach development time
- Breach was assumed to form in a straight reach of slough and develop perpendicular to the slough.

A two-dimensional hydrodynamic model (RMA-2) was used to determine the impacts of an outward reservoir breach of the embankment. The outcome from the analysis includes maximum flow velocities and maximum water surface elevations along the adjacent island levees.

For an inward reservoir breach, the higher peak discharges produce critical flows at the breach section. The RMA-2 model is not capable of simulating the critical flow regime, so normal flow conditions have been assumed to estimate flow velocities in the channel. The velocities near the adjacent islands are greatest on either side of the breach. As flow in the channel turns to pass through the breach, velocities at the adjacent island embankment are reduced, approaching zero.

Table 3.8 provides the hydraulic head differential across the reservoir embankments and peak discharges used in the breach analysis. Inward breach scenarios assumed that the reservoir is empty (at elevation –8.0 feet). This condition exists for about five months per year during the periods of emptying and filling of the reservoir (URS, 2001).

Table 3.8 – Hydraulic Head Differential and Peak Discharge

Breach Type	WSEL in Reservoir Island (feet)	WSEL in Slough (feet)	Head Differential (feet)	Peak Discharge (cfs)
Outward	+4.0	-1.0	5.0	95,000
Outward	+4.0	0.0	4.0	88,000
Outward	+4.0	+1.5	2.5	73,000
Inward	-8.0	+7.0	15.0	157,000
Inward	-8.0	+3.5	11.5	128,000

3.4.2 Results

Model results show that during an outward breach, the water surface directly across from the breach rises significantly. Peak velocities are observed on either side of the breach near the banks of the adjacent island levees. As would be expected, velocities are relatively small on either side of the breach adjacent to the reservoir island embankment due to the formation of eddies. During an inward breach of the reservoir, a similar flow pattern results, but the flow direction is reversed.

Peak velocities and water surface elevations estimated for narrow, medium, and wide slough sections are summarized in Tables 3.9 and 3.10, respectively. Peak velocities and water surface elevations presented are those observed near the adjacent island levee. Greater velocities are observed near the reservoir island breach.

Table 3.9 – Estimated Peak Velocities for Typical Slough Sections

Breach Type	Head Differential(1) (feet)	Maximum Velocity (ft/sec)		
		Wide (3,000 feet)	Medium (1,000 feet)	Narrow (450 feet)
Outward	5.0	6.2	9.2	12.3
	4.0	5.4	8.0	10.7
	2.5	4.1	6.1	8.1
Inward	15.0	1.0	2.9	6.0
	11.5	1.0	2.8	5.8

(1) See Table 3-8.

Table 3.10 – Estimated Water Surface Elevations for Typical Slough Sections

Breach Type	Head Differential[1] (feet)	Peak WSEL (feet)		
		Wide (3,000 feet)	Medium (1,000 feet)	Narrow (450 feet)
Outward	5.0	-0.1	0.5	1.7
	4.0	0.7	1.1	2.1
	2.5	1.9	2.1	2.7
Inward	15.0	7.0	7.0	7.0
	11.5	3.5	3.5	3.5

(1) See Table 3-8.

Figures 3.3 and 3.4 present the relationships between head differential and resulting peak velocity during a hypothetical outward and inward breach failure, respectively, for the three typical slough sections. These figures show that peak velocities are inversely proportional to the slough widths adjacent to the reservoir.

As discussed in Section 2.3.2, a velocity of 5 fps was selected as the threshold for failure of an adjacent levee. The results show that the levees adjacent to narrow and medium slough sections would fail should the reservoir breach outward under the scenarios analyzed. Levee sections adjacent to wide slough sections would also fail during the outward breach except under the most favorable scenario analyzed, where the head differential is 2.5 feet (see Table 3.9). Under an inward breach failure, the adjacent island levees would not fail where the typical slough widths are medium or wide. However, the adjacent island levee would fail where the typical slough width is narrow (see Table 3.9).

The average crest elevation of levees protecting adjacent islands is +8.0 feet (Section 2.3.2). Table 3.10 shows that there would be no adjacent island levee failures due to overtopping caused by an inward or outward breach of a reservoir island embankment.

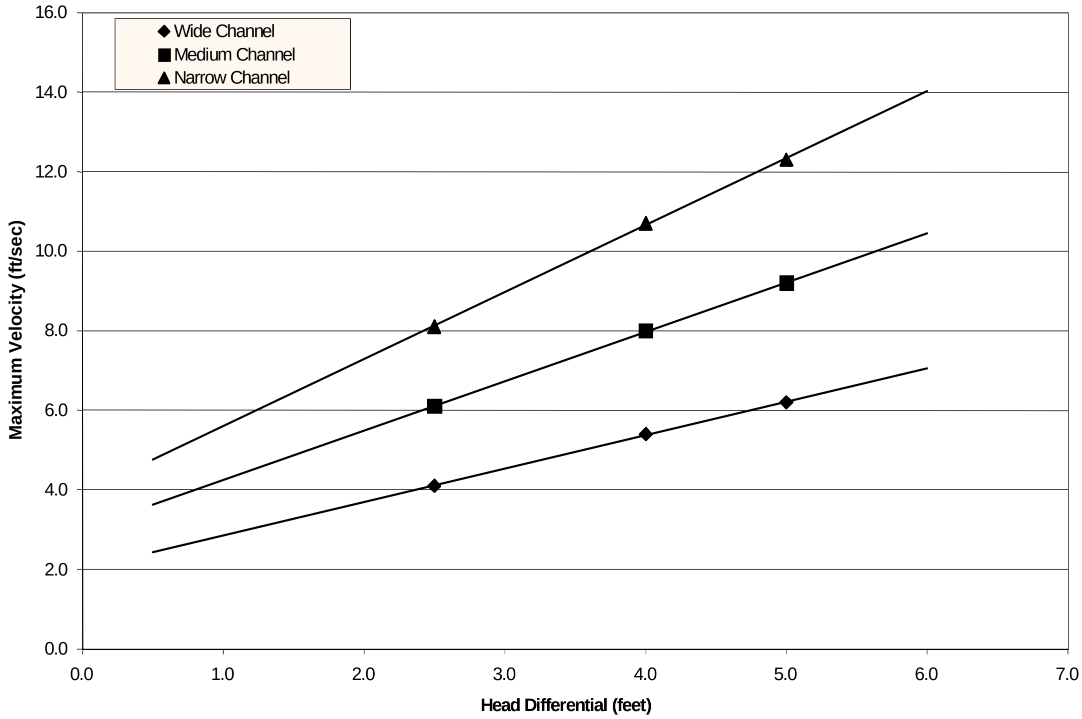


Figure 3.3 – Results of Outward Embankment Breach Analysis

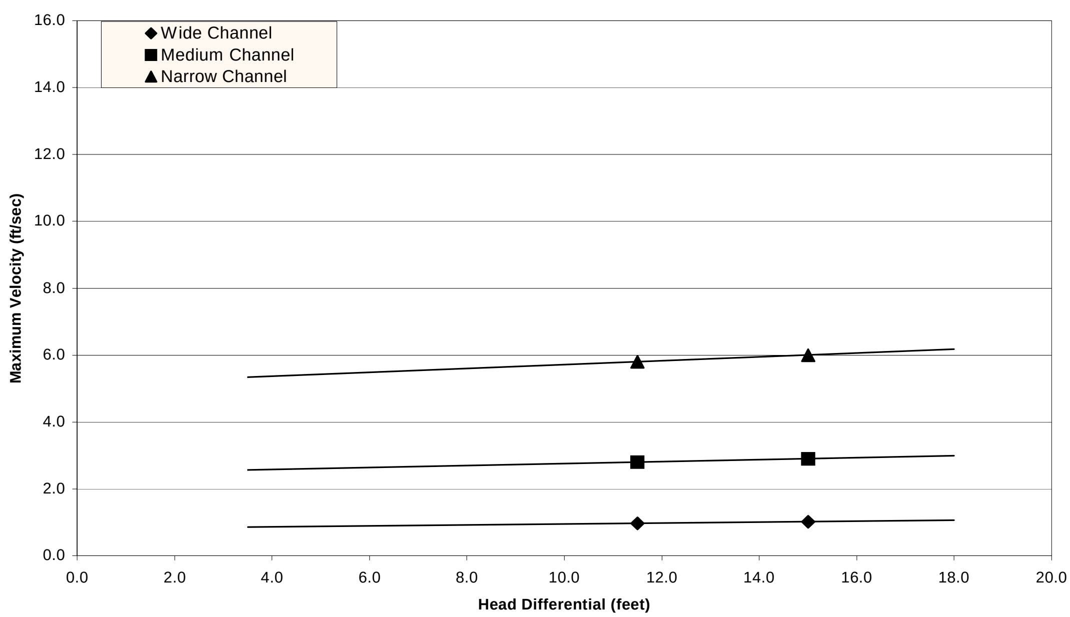


Figure 3.4 – Results of Inward Embankment Breach Analysis

Chapter 4: Seismic Analysis

4.1 Introduction

Under the seismic analysis, dynamic response analyses of the embankments were performed to calculate time histories of seismic-induced inertial force acting on the critical sliding masses. Seismic-induced permanent deformations of the embankments were estimated for the three ground motion levels selected for this study. The estimated deformations and their associated ground motion levels were used to evaluate the seismic risk of the proposed embankment alternatives and the probabilities of failure were estimated.

4.2 Dynamic Response Analysis

To begin with, information from relevant existing studies was reviewed. Analysis parameters were then developed which include embankment cross sections, material properties, and reservoir and slough water levels. Site-specific estimates of ground motions for future earthquake occurrences were developed and earthquake acceleration time histories were spectrally matched to the selected ground motion response spectra. Dynamic response analyses of the embankments were then performed to calculate time histories of seismic-induced inertial force acting on the critical sliding masses.

Review of the soil data indicates that there are some sections under the perimeter levees where the upper 5 feet of the underlying sand deposits may liquefy during earthquake events. In addition, part of the existing levee, on the island side, may contain loose sands, which have the potential to liquefy when they become saturated during the reservoir filling. One of the consequences of the liquefaction of the loose saturated sand is the reduction in shear resistance along the critical slip surface during earthquake shaking. In the context of this analysis, this translates into lower yield acceleration, ky, which in turn, induces larger deformations. Dynamic analyses for both cases involving non-liquefied and liquefied sandy layers were performed and embankment deformations for these cases were estimated.

4.2.1 Analysis Parameters

4.2.1.1 Embankment Cross Sections

Two embankment configurations were considered. The first configuration consists of building the embankment on the island side with a slough-side bench (bench option). This configuration results in a relatively off-set embankment from the existing levee, and provides for a flat slough side slope of 4H:1V or flatter. The second configuration consists of building the embankment on the existing levee and placing a rock toe berm on the slough-side slopes with an average slope of 3H:1V (rock berm option). These embankment configurations care presented in Chapter 5, Figures 5.1 and 5.2.

For each of these configurations, two cross-sections representing the variation in subsurface conditions (base of peat elevation at –20 feet, thinnest peat layer, and at –40 feet, thickest peat layer) were developed for analysis. These cross sections are considered to be representative at both Webb Tract and Bacon Island sites.

4.2.1.2 Material Properties

The selected dynamic soil properties used for the response analyses are summarized in Table 4.1. Plots of the selected G/Gmax and damping vs. shear strain relationships are presented in Figures 4.1 and 4.2.

4.2.1.3 Reservoir Stages and Slough Water Levels for Analyses

Two operating water elevation scenarios were selected to represent the normal fluctuation of water elevations in the reservoir and the slough, and are as follows (see Embankment Design Analysis Report):

- High Tide and Low Reservoir: a low reservoir water level and high slough water level at elevation +3.5 feet. This condition was assumed to prevail 2/3 of the time.
- Low Tide and High Reservoir: a high reservoir water level at elevation +4.0 feet and low slough water level at elevation –1 foot. This condition was assumed to prevail 1/3 of the time.

Table 4.1 – Dynamic Soil Parameters Selected for Analysis

Description		Moist Unit Weight (pcf)	K_{2max}	Shear Wave Velocity (ft/sec)	Modulus and Damping Curves
Embankment Materials					
New fills: sand		120	80	-	Sand[1]
Peat	- free-field	70	-	See note[4]	Peat[2]
	- under embankment				Peat[2]
Foundation Materials					
Sand	(non-liquefied)	120-125	80	-	Sand[1]
	(liquefied)	120-125	-	300-400	See Note [5]
Clay		127	-	1000	Clay[3]

Note: 1. Relationships of Kokusho (1980), function of confining pressure
2: Relationships of Wehling et al (2001)
3: Relationships of Vucetic and Dobry (1991) for PI = 50
4: Shear wave velocity was estimated using the following equations (Wehling et al. (2001):

$$v_s = \sqrt{\frac{G_{max}}{\rho}} \geq 75\ ft/\sec$$

$$\frac{G_{max}}{Pa} = 75.7\left[\frac{\sigma'_{1c}}{Pa}\right]^{0.87} OCR^{0.65}$$

Where Pa and σ'_{1c} are the atmospheric and effective vertical pressures, respectively
5. For liquefied sand, no reduction in G is allowed and the damping is fixed at 8%-10% of critical damping.

4.2.2 Earthquake Loads

A site-specific probabilistic seismic hazard analysis was performed for the current study to provide estimates of ground motions for future earthquake occurrences. A discussion of the approach, assumptions and results is presented in Attachment 1 to the Seismic Analysis report.

4.2.2.1 Earthquake Response Spectra

Three seismic events representing a small, a moderate, and a large earthquake in the region were considered. The three selected events correspond to ground motions having probabilities of exceedance in 50 years of about 69%, 10% and 2%, corresponding to ground motions with return periods of about 43 years, 475 years and 2,500 years, respectively. The 5%-damped response spectra represent free-field motions for the outcropping stiff soil site condition. The peak ground accelerations (PGA's) at the site are as follows:

- 43 year return period: 0.14g
- 475 year return period: 0.33g
- 2,500 year return period: 0.52g

4.2.2.2 Spectrally-Matched Time Histories

To perform the dynamic response analyses, earthquake acceleration time histories are needed as input. The same time histories as used in the previous URS, 2000 study were used. Table 2 lists these recorded motions along with their closest distances from the rupture planes and recorded peak accelerations.

Modifications to the natural time histories were necessary to develop acceleration time histories with overall characteristics that match the target response spectra. The two acceleration time histories were spectrally matched to the selected response spectra (i.e., response spectra for return periods of 43 years, 475 years and 2,500 years). See the URS Seismic Analysis Report, Section 2.3.2 for more details.

Table 4.2 – Summary of Earthquake Records Used in the Dynamic Response Analysis

Earthquake	M_w	Recording Station			Comp.	Recorded PGA (g)
		Distance (km)	Station	Site Condition		
1987 Whittier Narrows	6.0	18	Altadena – Eaton Canyon Station	Soil[a]	90°	0.15
1992 Landers	7.3	64	Fort Irwin Station	Soil[a]	0°	0.11

Note : a = Deep stiff soil site

4.2.3 Analysis Results

Dynamic response analyses were performed and the results are expressed in terms of average horizontal acceleration (K_{ave}) time histories of the potential (critical) slide masses within the embankments. The critical slide masses for each embankment alternative and for the two cross sections were identified in the static slope stability analyses (Embankment Design Analysis Report), and are presented in the Seismic Analysis report, Figures 15 through 18. The average horizontal acceleration was calculated by computing the dynamic response of the embankment and averaging various stresses within or close to the sliding surface. Examples of the calculated K_{ave} time history are presented in the Seismic Analysis report, Figures 19 through 22 for the 475-year return period ground motion.

4.3 Seismic Stability and Deformation Analysis

4.3.1 Methodology

Seismic-induced permanent deformations of the embankment slopes were estimated using the Newmark Double Integration Method (1965) and the Makdisi and Seed Simplified Procedure (1978). The Newmark Double Integration Method is based on the concept that deformations of an embankment will result from incremental sliding during the short periods when earthquake inertial forces in the critical slide mass exceed the available resisting forces. The simplified procedure of Makdisi and Seed (1978) was developed based on observations of dam performance during past earthquakes and analysis results.

4.3.2 Results

The results of the seismic deformation analyses for both the bench option and rock berm option are discussed below and are also summarized in the Seismic Analysis report, Table 10A.

4.3.2.1 Bench Alternative

The slope deformations calculated using the Newmark Double Integration Method for non-liquefied sandy soils are tabulated in the Seismic Analysis report, Tables 3 and 4 for Cross Section I (bottom of peat at elevation –20 feet) and Cross Section II (bottom of peat at elevation –40 feet), respectively. For the non-liquefied cases, the results of the analysis suggest that up to about 1.6 feet and 0.5 feet of slope deformations on the slough and reservoir sides, respectively, can be expected during an earthquake event having a 475-year return period. Under the 43-year return period ground motions, the seismic induced slope deformations are expected to be small. The Simplified Makdisi and Seed procedure was also used to estimate slope deformations for comparison purposes as shown in the Seismic Analysis report, Tables 3 and 4.

The results for the liquefied cases are tabulated in the Seismic Analysis report, Tables 5 and 6. As expected, under the 475-year return period event, much larger slope deformations were estimated. For Cross Section I, up to about 6 feet and 2.4 feet of deformations were calculated for the slough and reservoir slopes, respectively. Slough side slope deformations of about 17 feet and reservoir side slope deformations of about 6.5 feet were estimated for Cross Section II. Under the smaller ground motions of 43-year return period, maximum deformations of about 1.2 feet and 0.2 feet were calculated for the slough and reservoir slopes, respectively, for Cross Section I. The maximum slope deformations for Cross Section II were calculated to be about 3 feet, for the slough slopes, and 0.75 feet, for the reservoir slopes.

As noted in the Seismic Analysis report, Tables 3 through 6, convergence was not obtained for some of the cases with larger earthquakes (2500-year and some 475-year events). Further details are discussed in the Seismic Analysis report, Section 3.2.1. For the purpose of this study, a deformation of over 12 feet was assumed to have a 95 percent probability of embankment failure. This condition was considered to represent the expected embankment performance under severe earthquake events.

4.3.2.2 Rock Berm Alternative

For the rock berm option, the calculated slope deformations considering non-liquefied sandy soils are tabulated in the Seismic Analysis report, Tables 7 and 8 for Cross Section I (bottom of peat at elevation –20 feet) and Cross Section II (bottom of peat at elevation –40 feet), respectively. For the non-liquefied case, the results of the analysis suggest that up to about 0.6-foot of slope deformation can be expected during an earthquake event having a 475-year return period. Under the 43-year return period ground motions, the seismic induced slope deformations are expected to be small. The Simplified Makdisi and Seed procedure was also used to estimate slope deformations for comparison purposes as shown in the Seismic Analysis report, Table 7 and 8.

The results for the liquefied cases are tabulated in the Seismic Analysis report, Tables 9 and 10 for Cross Sections I and II, respectively. As expected, under the 475-year return period event, larger slope deformations were estimated, For Cross Section I, up to about 2.4 feet and 1 foot of deformations were calculated for the reservoir and slough slopes, respectively. Maximum deformations of about 6.3 feet and 1.3 feet were estimated for the reservoir and slough slopes of Cross Section II, respectively. Under the smaller ground motions of 43-year return period, maximum reservoir slope deformation of about 2 feet was calculated.

As noted in the Seismic Analysis report, Tables 7 through 10, convergence was not obtained for some of the cases with larger earthquakes (2500-year and some 475-year events). Similar to the bench option, a deformation of over 12 feet was assumed to have a 95 percent probability of embankment failure. This condition was considered to represent the expected embankment performance under severe earthquake events.

4.4 Estimated Probability of Failure

Estimated probability of failures for the various cross sections analyzed under the different earthquake scenarios were determined as part of the seismic analysis for use in the overall risk analysis. The modes of failure considered for this study included those caused by an earthquake event, such as seismic-induced slumping, slope failure, liquefaction-induced sliding and lateral spreading and other related secondary failures (i.e., piping through an open crack, etc.).

4.4.1 Embankment Fragility Curve

The embankment fragility curve developed by the Seismic Vulnerability Sub-Team (CALFED, 1998) was used for this study for both the liquefied and non-liquefied cases. This curve was then utilized to evaluate the probability of failure of an embankment cross section with given earthquake-induced deformations.

4.4.2 Failure Probability

Failure probabilities for the two project embankment alternatives (bench and rock berm options) and the two embankment cross sections (Cross Section I and II) were calculated by combining the various weights (probabilities) associated with reservoir and slough water levels, earthquake ground motion and liquefaction scenarios. Weights assigned to the reservoir and slough water level scenarios were estimated based on the percentage of time each scenario would occur annually. Weights for the earthquake ground motion scenarios were estimated by assuming a time-independent Poisson process for earthquake occurrence and a project life cycle of 50 years. In estimating the weights for the three ground motion scenarios, we assumed that the 43-year, 475-year and 2,500-year ground motions are represented by ground motions with return periods less than about 130 years, 130 years to about 1,000 years and greater than 1,000 years, respectively. The failure probabilities were calculated considering the contributions from the large/distant and moderate/near earthquakes and critical slide masses on the reservoir and slough sides. Weights for the liquefaction scenarios were selected based on judgment and evaluation of sampler blowcounts recorded in the sandy deposits.

Tables 11 through 14 in the Seismic Analysis report summarize the contributions of the various scenarios and provide estimates for the total probability of failure for each project alternative and each cross section for a 50-year life cycle. The bench alternative with peat at elevation –20 feet has about 19 percent chance of failure, while the cross section with peat at elevation –40 feet has about 28 percent chance of failure. For the rock berm alternative, the cross section with peat at elevation –20 feet has about 17 percent chance of failure, while the cross section with peat at elevation –40 feet has about 23.5 percent chance of failure.

4.5 Summary

This report presents the results of estimated seismic performance of the two embankment design alternatives, and addresses the probability of earthquake-induced embankment failure.

Table 10A in the Seismic Analysis report shows that the calculated seismic deformations are large for several conditions for the 475-year earthquake event. The results of the evaluation appear to suggest that the rock berm alternative would provide for a lower probability of failure than the bench alternative. The rock berm alternative is preferable to the bench alternative because it places the embankment over the existing levee and, therefore, makes use of the stronger peat under the levee as opposed to the weaker free-field peat. In addition, the rock berm alternative provides a more stable slough side slope.

Because liquefaction would lead to large deformations that would affect overall stability of the embankment, further investigation and evaluation of the existing levee materials are recommended. Depending on the extent of the potentially liquefiable sands within the existing levee, removal of the loose sands may need to be implemented.

Due to the limitations of the QUAD4M computer program for large earthquake loads, a uniform assumption has been made for estimating the expected embankment deformation. Although this assumption is considered conservative, a more rigorous non-linear analysis would probably be useful and could provide more insight into the deformation patterns associated with large strains under the large earthquake shaking. This analysis could also provide more insight into the comparative performance of the embankment alternatives under the larger earthquakes.

The calculation of the overall risk is presented in the URS Risk Analysis report. The risk analysis combines the probabilities of failure from various events (seismic, operational and flood) and their failure consequences.

Chapter 5: Embankment Design Analysis

5.1 Introduction

Under the embankment design analysis, the vulnerability and reliability of the existing conditions and In-Delta Storage re-engineered project embankments were evaluated under operational demands by conducting extensive seepage and stability analyses.

Steady-state seepage conditions through transverse sections of the existing levees and re-engineered embankments at Webb Tract and Bacon Island were estimated and seepage control alternatives were analyzed.

The re-engineered project (“rock berm” and “bench”) embankment options have been evaluated by extensive stability analyses of the two sections selected to be representative of the lowest and highest elevations at which the base of the underlying peat layer is found in the two islands. Conditions evaluated in the stability analysis include end-of construction, long-term operation, sudden drawdown, and pseudo-static. Factors of safety were calculated and compared to the project’s stability criteria, and the adequacy of the proposed project in regard to embankment stability was evaluated.

To meet the USBR Risk Analysis requirements, it was decided that the potential for erosion and piping had to be addressed. The probability of erosion and piping failures was determined and six alternatives were considered as solutions to reduce the chance for erosion and piping to occur. On the basis of factors that can contribute to erosion and piping, areas requiring control were identified and an evaluation was performed to select a preferred measure.

5.2 Analysis Parameters and Design Criteria

5.2.1 Data Review

Reports from several geotechnical and environmental studies that have been conducted at the two proposed reservoir sites and neighboring islands were reviewed for this analysis. In addition, we reviewed and incorporated data provided by DWR into the current study. These reports and data describe subsurface soil conditions encountered during various field and laboratory investigations. Previous field investigations included drilling and standard penetration testing (SPT), sampling, and cone penetration testing (CPT). Previous laboratory testing programs included engineering property determination of embankment material and foundation soil.

New field or laboratory work for the current study included a USBR exploration program consisting of 19 CPT soundings at Webb Tract and 18 CPT soundings at Bacon Island drilled in 2002, as described in Chapter 2. No other field or laboratory work was performed for this study.

5.2.2 Analysis Parameters

5.2.2.1 Subsurface Conditions

Longitudinal profiles of the subsurface conditions along the perimeter of both islands developed and described in URS (2001) were updated to include the CPT data obtained by USBR in 2002. These profiles were also compared with stick-log profiles provided by DWR. No significant changes from previous interpretations of the stratigraphy under the levees were observed.

The general stratigraphy of the levee and underlying soils of Bacon Island and Webb Tract are similar. The stratigraphy of the interior of the islands consists of a surficial soft, organic fibrous peat (PT) layer underlain by a silty sand (SM) aquifer, below which lies stiff lean clay (CL). These units are laterally continuous and vary in thickness from one part of the island to another. The silty sand layer is exposed in some portions of Webb Tract. Deeper sand aquifers are present below the stiff clay in some areas.

The levees are typically built of about 10 feet of sandy to clayey fill, placed on a mixture of clayey peat and peat fill that overlies the natural peat layer. The levee fill consists of inter-fingered layers of sand, peat, clay and clayey peat that are likely to have more sand on the land-side and peat and peaty clay on the slough side. Portions of the sandy reservoir side of the levee fill may be loose based on the methods of placement used during construction of the levees. The underlying peat is fibrous, soft, and highly compressible. Based on the available data it is not feasible to differentiate the clayey peat and peat fill from the natural peat, but the data suggests that the engineering properties of the materials are similar. Occasionally, up to 15 feet of fat organic clay (OH) are encountered between the peat and underlying silty sand layer. For this study, the clayey peat and peat fill, natural peat, and fat clay have been combined to make up one layer. The combined layer thickness ranges from 15 to 40 feet under the levees.

For the current study, the upper five feet of the sand layer is assumed to be potentially liquefiable under portions of the perimeters of both islands. This thickness was based on the borings and CPT soundings available for review.

The islands were divided into sections based on the elevation of the base of peat. Bacon Island has been divided into four sections with the base of peat elevation ranging from –20 feet to –40 feet and Webb Tract has been divided into four sections with the base of peat elevation ranging from –25 feet to –40 feet.

Previous evaluations have shown that peat thickness under the levees has the greatest influence on slope instability. For the current study, two cases representing the new embankment constructed over peat having the highest (smallest peat thickness) and lowest (largest peat thickness) base elevations were analyzed. The cases are considered to be representative of both islands due to the similarity of the stratigraphy of the islands. The cases are as follows:

- Peat at El. –20 feet with the bottom of levee fill at 0 feet
- Peat at El. –40 feet with the bottom of levee fill at 0 feet

5.2.2.2 Embankment Geometry

The existing levees will be raised and strengthened, generally on the island side, to form the embankments impounding the proposed reservoirs. The configuration for the new embankments around both islands has a crest elevation of +10 feet, with a final crest width of 35 feet. The inside slope of the reservoir is a composite slope. The slope above elevation +4 feet is 3H:1V and the lower slope is 10H:1V. Erosion protection covers the inside slope from elevation +3 to the crest. Two configurations were considered for the slough-side slope, the “rock berm” option and the “bench” option, and are described below.

“Rock Berm” Option

The “rock berm” option consists of constructing the new embankment on top of the existing levee as shown on Figure 5.1. The slough-side slope of the new embankment extends from the outboard crest of the existing levee toward the slough at a 3H:1V slope. Where the existing slough-side slope is steeper than 3H:1V, rock fill would be placed from the outboard crest of the existing levee outward to the bottom of the slough at a 3H:1V slope. Rockfill would also be placed from the outboard crest of the existing levee to the bottom of the slough at slopes flatter than 3H:1V where required to meet stability criterion. Free-draining reservoir side berms would be placed at the bottom of the reservoir-side slope toe where analyses of combinations of base of peat elevation and reservoir base elevation result in factors of safety that do not meet project criteria.

"Bench" Option

The "bench" option, shown on Figure 5.1, consists of a bench, created by removing a portion of the existing levee to an elevation varying between 0 and 6 feet and constructing the new embankment from the reservoir side of the bench at a slope of 3H:1V to the crest of the embankment. In addition to removing load from the slough side of the embankment in order to provide a stable slough-side slope, the bench provides opportunity for environmental mitigation. The bench shifts the crest of the new embankment towards the reservoir. Erosion protection covering the slough-side slope above the bench would consist of riprap and bedding. Free-draining reservoir side berms would be placed at the bottom of the reservoir-side slope toe where analyses of combinations of base of peat elevation and reservoir base elevation result in factors of safety that do not meet project criteria.

Existing Levee Geometry

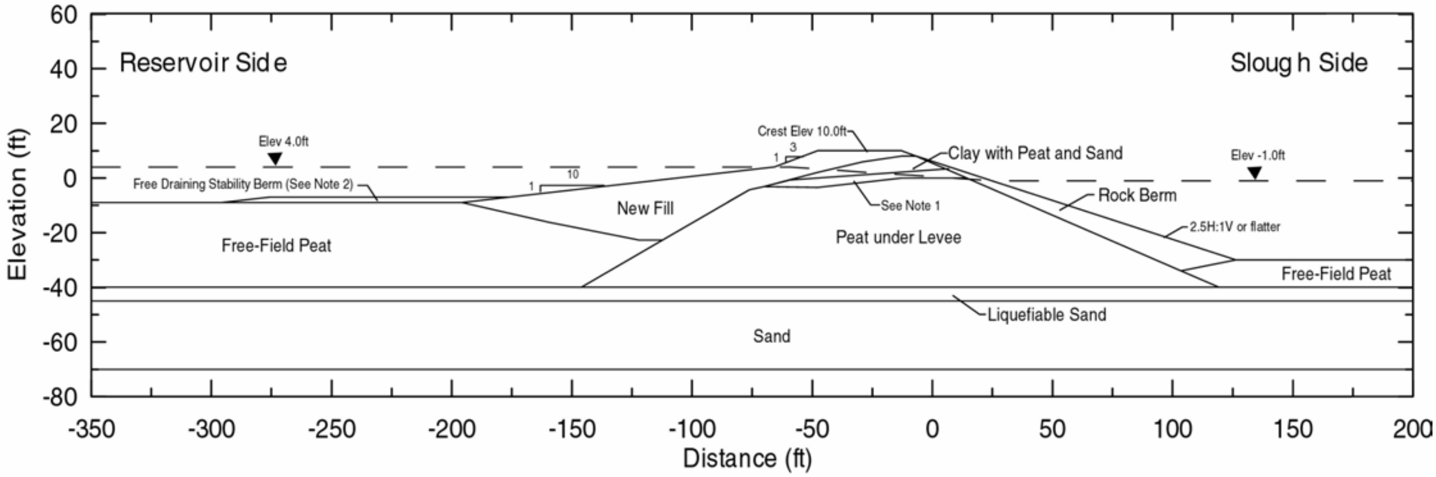
The geometry of the existing levees around Bacon Island and Webb Tract vary with respect to reservoir and slough side slopes, crest width, crest elevation, and slough bottom elevation. Rockfill exists on the slough-side slopes, but the extent and thickness of the rockfill is not known for certain. Therefore, the rockfill was not considered to be a continuous layer everywhere on the slough-side slopes and, as such, was not included in stability analyses.

Effect of Settlement

Construction of the new embankments over highly compressible organic soils in the foundation will result in significant settlement. Progressive placement of fill will be required to construct and maintain the final crest elevation resulting in substantial reduction of peat thickness under the embankments. The geometry of the new embankment fill and underlying peat for long term steady state stability conditions incorporate the deformation due to consolidation of the peat.

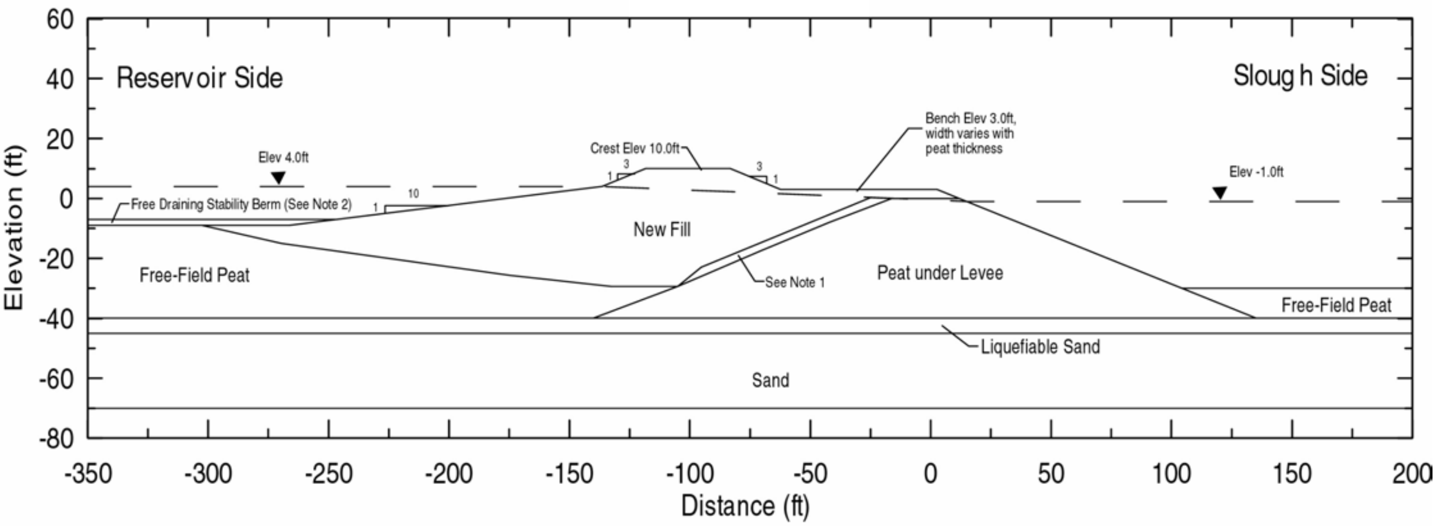
The finite element code program, Plaxis version 7.0, was used to estimate the deformed geometry at the end of consolidation. The deformed geometries of the new embankment for Case 2 (base of peat at –40 feet) for the "rock berm" and "bench" options are shown on Figure 5.1. Deformed geometries for Case 1 (base of peat at –20 feet) are shown on the stability analysis figures included in the Embankment Design Analysis report, Appendix A.

Rock Berm Option



Note 1: Portion of existing levee assumed to be potentially liquefiable.
Note 2: Free draining stability berm required to meet long term stability condition where base of peat is -30 feet or lower.

Bench Option



Note 1: Portion of existing levee assumed to be potentially liquefiable.
Note 2: Free draining stability berm required where liquefaction of upper sand occurs and base of peat is -40 feet or lower.

Figure 5.1 – Embankment Alternatives

5.2.2.3 Material Properties

Stress-Strain-Strength Properties

Material properties were based on the recent planning study entitled "In-Delta Storage Program, Draft Report on Engineering Investigations," dated May 2002, modifications for the strength of organic soils from data provided by DWR, and workshops held during the current study. A summary of the material properties used in the analyses is shown in Table 5.1. The typical location of the materials is shown on Figure 5.1.

For this study, it was assumed that peat under levee conditions applied to all peat that is located below a line projected along both the slough side and reservoir side levee slope through the peat to the underlying sand as shown on Figure 5.1. All peat outside of this limit was considered to be free field peat.

The strength of new embankment materials was reduced from shear strengths normally assigned for engineered sandy fills to account for shearing and cracking within the embankment fill during consolidation of the underlying peat and subsequent deformation of the new fill.

For the loose upper portion of the silty sand layer that exists in some portions of the islands, the post-liquefaction undrained residual shear strength was taken as 200 psf, based on the average estimated corrected penetration resistance (SPT). For portions of the island where this loose layer does not occur, this soil layer was assumed to have the same shear strength as the underlying sand.

An undrained shear strength of 200 psf, similar to that of the loose sand in the upper part of the silty sand layer, was assumed for evaluating the effect on stability for sandy portions of the existing levees on the reservoir side where the fill may be loose due to placement. No data from specific investigations for the density of the existing sandy levee fill were available for review.

Permeability

Generally, the coefficients of permeability for the various layers are the same as used in URS (2000). The coefficients of permeability used are summarized in Table 5.2. Further discussion of how the permeability coefficients were determined can be found in the Embankment Design Analysis report, Section 2.2.3.2.

5.2.2.4 Reservoir Stage and Slough Water Level

At each section and case analyzed, a combination of reservoir and slough water surface levels that produce critical conditions was used. A high slough water surface elevation, combined with a low reservoir elevation, is potentially the most critical to the island-side slope. A low slough water surface elevation, combined with a high reservoir elevation, is potentially the most critical to the slough-side slope.

Reservoir Stage

The reservoirs will operate at various levels during a typical calendar year. Patterns for reservoir levels were developed through operation studies. In a typical year, for a little less than two months (May and June), the reservoirs will be at their maximum operating water level (+4). During about five months (September through January of the following year), the reservoirs will be at their lowest operating level or will be empty. An intermediate constant reservoir stage at about –11 exists in the second week of February to the third week in March. In between these three periods of time, variations of the reservoir level will be approximately linear.

The maximum and minimum levels for the reservoir last for extended periods of time defining conditions that correspond to normal operation. The most critical of either the maximum or minimum reservoir levels were considered in the analysis cases described above.

Slough Water Level

Slough water levels vary with tide cycles and flooding events.

For the analysis of the long-term condition of the reservoir-side slope, it was assumed that the water level in the slough could reach peak flood level at least once during the design life of the reservoir. For the current study, a maximum peak flood elevation of +7.0 feet was used.

The sudden drawdown condition does not represent a “normal” condition. Therefore, it was combined with a flood condition less demanding than considered for the long-term condition. For the sudden drawdown analysis case, a slough water elevation of +6 feet was used.

For the stability evaluation of the slough-side slopes, the water surface level in the slough at an average low tide elevation (-1.0 feet) was used, which represents a reasonably conservative condition. Seismic conditions were analyzed for slough water levels corresponding to high (+3.5 feet), average (+1.5 feet), and low (-1.0 feet) tides.

The water elevations discussed above are tabulated along with the results of the stability analyses in Section 4.

5.2.3 Analysis Criteria

5.2.3.1 Seepage Analysis

A maximum acceptable gradient is 0.3, established by USACE (1997), at or near the toe of levee was used in this project to determine whether seepage mitigation measures are needed or not.

5.2.3.2 Stability Analysis

Because critical conditions may arise either on the slopes facing the slough side or the reservoir side, the factors of safety of both slopes were assessed. The following analysis conditions were evaluated.

End-of Construction

The end-of-construction scenario is the condition occurring immediately after placement of new fill on the reservoir island side of the levee. Fill is placed in thin layers and compacted. Immediately after fill placement, relatively impervious materials such as peat and clay in the levee and foundation will not have had sufficient time to dissipate construction-induced excess pore pressures. Hence, at the end of construction, undrained shear strengths are normally used to characterize the cohesive soils of the levee and foundation.

Long-Term Operation

The analysis of long-term levee stability involves the post-construction conditions when strength gain has occurred, and normal operation of the reservoir is in place. Two combinations of water levels (high reservoir and low slough water, and vise-versa) on the reservoir and slough sides were selected to produce the most critical load cases that could be encountered during such operation.

Sudden Drawdown

The sudden drawdown case affects the reservoir-side slope when the reservoir water level drops rapidly. Such condition may result from emergency drainage of the reservoir.

Because the drop in reservoir level can occur at a relatively rapid rate, the peat and other fine-grained soils would not have enough time to drain, so undrained strengths after long-term consolidation are used in the analysis.

Pseudo-Static Analysis

Pseudo static analysis is used to estimate the yield accelerations (K_y) for the most critical failure surfaces. The use of the calculated yield acceleration to estimate earthquake-induced deformation of the levee systems is discussed in the Seismic Analysis Report. Water levels on the island and slough sides were selected to produce critical cases. The strength of soil layers that are potentially liquefiable was taken as the undrained residual shear strength. Undrained shear strengths in potentially liquefiable soils were also used in computing post-seismic stability.

Evaluation Criteria

Criteria for the calculated factors of safety for each case of static stability are summarized in Table 5.3. These selected factors of safety are based on the significance of the project; the consequences of failure; uncertainties in estimated parameters; cases considered; and criteria from several agencies.

Table 5.1 – Material Properties

Material	Weight, γ, lb/ft^3		Undrained Shear Strength, lb/ft^2	Effective Strength		Total Strength	
	wet	Sat.		ϕ', degree	C', lb/ft2	ϕ_t, degree	C_t, lb/ft2
Rock Fill	140	140		40	0	40	0
New Fill [1]	110	120		30	0	30	0
Existing fill sand	110	110		30	0	30	0
Existing fill, sand with clay and peat	110	110		30	0	30	0
Peat under dam [2]	70	70	450	28	50	17	100
Free field peat [2]	70	70	200	20	50	13	100
Deep sand		125		36	0	36	0
Gray fat clay		85	250	25	0	30	100

[1] New fill shear strength properties are reduced to account for shearing within the embankment during consolidation of the underlying peat and subsequent deformation of the new fill.
[2] Peat shear strength values (provided by DWR on 9/30/02) are based on back calculations and data for similar islands.

Table 5.2 – Permeability of Soils Used in Seepage Analysis

Material	**Vertical Permeability Ky (cm/s)**	**Horizontal Permeability Kx (cm/s)**	**Ky/Kx**
Existing Sandy Fill (with clay and peat)	1×10^{-5}	1×10^{-4}	0.1
Existing Clayey Fill (Bay Mud)	1×10^{-7}	1×10^{-6}	0.1
Peat	1×10^{-6}	2×10^{-4}	0.005
Sand	1×10^{-4}	1×10^{-3}	0.1
Clay	1×10^{-6}	1×10^{-6}	1
Planned Fill (sand)	1×10^{-3}	1×10^{-3}	1

Table 5.3 – Minimum Factors of Safety for Static Stability

Case	Material Properties	Phreatic Surface	Minimum Factor of Safety
End of Construction	Unconsolidated undrained shear strength	Construction-induced excess pore pressures with high and low river elevations	1.3
Sudden Drawdown	Consolidated undrained shear strength	Rapid Drawdown from normal pool to dead storage with low river elevation (use phreatic surface from steady-state seepage with surface following the island slope.	1.2
Steady-State Seepage	Consolidated drained shear strength	Steady-state seepage under normal pool with low river elevation	1.5
Seismic - Post Liquefaction Stability	Consolidated Undrained -Based on SPT	Steady-state	1.1

5.3 Seepage Analysis

5.3.1 General

Seepage analyses for the reservoirs were previously performed as described in URS (2000). The sections analyzed in URS (2000) were reviewed and determined to be appropriate for the current study. The primary change in the current study is a reduction in the normal operating reservoir water elevation from +6.0 feet to +4.0 feet.

5.3.2 Methodology

The computer program SEEP/W (Geo-Slope International Ltd., 1994) was used to estimate steady-state seepage conditions through transverse sections of the existing levees at Webb Tract and Bacon Island. SEEP/W uses a two-dimensional finite element method to model seepage conditions and assumes that flow through both saturated and unsaturated media follows Darcy’s Law. Using the SEEP/W mesh generation program, finite element meshes were generated to model the multiple seepage conditions considered for the levees on Webb Tract and Bacon Island.

The SEEP/W analysis program was used to evaluate the steady-state phreatic surface location, the head distribution throughout the model, and flow quantities at particular locations. The SEEP/W contouring program was used to generate head distribution diagrams. Phreatic surfaces, total head contours and flux quantities are presented in the Embankment Design Analysis Report.

5.3.3 Analysis Sections

Three sections were considered for the seepage analysis, two at Webb Tract and one at Bacon Island. The sections at Webb Tract were selected at Stations 630+00 and 260+00 to represent the narrow (400 feet) and wide (1,200 feet) slough, respectively. The section at Bacon Island was selected at Station 665+00 to represent an average slough width (700 feet), which is more common around the islands.

In addition to the above three analysis sections, the two sections at Webb Tract were evaluated assuming the sand is exposed in the island interior.

5.3.4 Analysis Conditions

For each section, three seepage conditions were evaluated: (1) existing conditions, (2) full reservoir with no pumping at the interceptor wells, and (3) full reservoir with required pumping at the interceptor wells. Existing conditions were first analyzed to evaluate the pre-reservoir seepage conditions. Full reservoir conditions without underseepage remediation were analyzed as an intermediate condition to estimate the impacts of the reservoirs on neighboring islands. Full reservoir conditions with pumping at the interceptor well system were analyzed to evaluate the efficiency of the proposed interceptor well system and to estimate the minimum pump rate (in gallons per minute per foot of levee) required to reestablish pre-reservoir seepage conditions at the far levee.

5.3.5 Boundary Conditions

The primary boundary conditions affecting the seepage models include the constant head boundaries imposed by presence of the slough, the full reservoir, and the groundwater conditions within the adjacent island. The slough was modeled as having a constant elevation head of –1.0 feet. For the full reservoir condition, a constant normal operating reservoir water level of +4 feet was used. Sensitivity with respect to slough water level was analyzed for Webb Tract station 630+00 using a high tide (+3.5 feet) and full reservoir conditions. The cross sections considered for seepage analysis together with water elevations used in both reservoir and slough sides are summarized in Table 5.5.

The far-field boundary condition at the neighboring island under existing conditions was estimated using a groundwater level at about 2 feet below the average ground elevation of the island.

For the full reservoir condition with pumping at the interceptor wells, a constant flow boundary was placed through the sand aquifer at the location of the well line. This boundary condition was used to represent the average flow rate along the well line during pumping, and was varied until the pre-reservoir conditions were re-established.

5.3.6 Results

The analysis results are summarized for each case in Table 5.6. The table presents the following:

- The average total head (in feet) in the sand aquifer at the near levee (Webb Tract or Bacon Island) centerline
- The average total head (in feet) in the sand aquifer at the far levee (adjacent island) centerline
- The flow rate through the sand aquifer at the far levee centerline
- Exit gradient at the land-side toe of the far levee
- The corresponding pump rates for individual interceptor wells spaced at 160 and 200 feet
- Discussion of the findings for each cross-section are presented below.

Webb Tract Station 630+00. This cross-section was considered to be a critical seepage condition for Webb Tract, as the adjacent island levee is only about 400 feet away. The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet. Under existing conditions a significant head loss within the channel peat occurs, indicating the importance of the channel peat's influence on the seepage rates under the levees.

Under full reservoir conditions with no seepage remediation, there is about a five-foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.2 under existing conditions to 0.6 under full reservoir conditions, which indicates a potential for sand boils and piping of levee material to occur on the neighboring island. Using interceptor wells, the minimum pump rate needed to re-establish pre-reservoir conditions at the adjacent island is about 6 gpm for wells spaced at 160 feet and 7.5 gpm for 200 feet.

Sensitivity under high tide (+3.5 feet) and full reservoir conditions was also checked for this cross-section for comparison with the low tide and full reservoir conditions discussed above. The total head within the sand aquifer at each levee under existing seepage conditions is at about elevation –12 feet. Under full reservoir conditions with no seepage remediation, there is about a five-foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.2 under existing conditions to 0.6 under the full reservoir case.

For conditions where the sand aquifer is exposed within Webb Tract near the new embankment with no seepage remediation, there is a six-foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates that gradients of 0.7 exist at the ground surface under full reservoir conditions. Under gradients of this magnitude, there would likely be sand boils and piping of levee material on the neighboring island. Under full reservoir conditions with pumping at the interceptor wells, the minimum pump rate needed to re-establish the pre-reservoir conditions at the adjacent island is about 8.7 gpm for wells spaced at 160 feet and 10.8 gpm for wells spaced at 200 feet.

Webb Tract Station 260+00. This cross-section was considered to be one with the widest slough (1200 feet). The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet. Under existing conditions a significant head loss within the channel peat occurs, indicating the importance of the channel peat's influence on the seepage rates under the levees.

Under full reservoir conditions with no seepage remediation, there is about a two-foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.1 to 0.2 from the exiting condition to the full reservoir case, respectively. Under full reservoir, these gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage flows could increase by about 1.6 times. Under full reservoir conditions with pumping at the interceptor wells, the minimum pump rate needed to re-establish pre-reservoir conditions at the adjacent island is estimated to be about 5.7 gpm for wells spaced at 160 feet and 7.2 gpm for wells spaced at 200 feet.

For conditions where the sand aquifer is exposed within Webb Tract near the new embankment with no seepage remediation, there is a three-foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates that under full reservoir, the exit gradient is about 0.2. These gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage flows would increase by about 1.6 times. Under full reservoir conditions with pumping at the interceptor wells, the minimum pumping rate needed to re-establish the pre-reservoir conditions at the adjacent island is about 8.8 gpm for wells spaced at 160 feet and 10.9 gpm for wells spaced at 200 feet.

A sensitivity analysis allowing the water level to vary from –1 feet to +3.5 feet in elevation showed an insignificant difference between the two cases.

Bacon Island Station 665+00. This cross-section was considered to be an average representative slough width of 700 feet. The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet.

Under full reservoir conditions with no seepage remediation, there is about a two and one-half foot increase in the total head beneath the far levee. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.20 to 0.30 from the exiting condition to the full reservoir case, respectively. Under full reservoir, these gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage would increase by about two times. Although not calculated, the minimum pumping rate needed to re-establish pre-reservoir conditions at the adjacent island is estimated to be about 6 gpm for wells spaced at 160 feet or about 7.5 gpm for wells spaced at 200 feet.

5.3.7 Seepage Control Alternatives

Potential seepage control measures for the In-Delta-Storage islands include interceptor wells, slurry cut-off walls, reservoir floor clay blanket, and collector trenches/French drains in the neighboring islands among others. These techniques vary in cost, constructibility, feasibility, and operation and maintenance. A brief discussion of these alternatives and their advantages and limitations is presented below.

5.3.7.1 Interceptor Wells

This solution relies on a series of active extraction wells located on the crest of the reservoir island embankments. The wells are actively operated to draw the aquifer down such that seepage flows in the neighboring islands are maintained to the same levels as pre-project conditions. This solution would require well spacing varying from 160 feet to 200 feet or greater. Assuming 200-feet as an average representative well spacing, the pumping rate to re-establish the existing condition (pre-project) would be about 8 gpm per well. The excess seepage flow into the neighboring islands, absent any pumping, would be on average 2 to 4 gpm per 100-foot section of levee.

The major limitations associated with active pumping to control seepage are the required operation and maintenance to keep the interceptor wells in good operating conditions. Based on general experience, one can expect that 50 percent of the wells would be replaced every 50 years. Because extraction wells may cause migration of fines, the proper well design and construction would be needed to minimize desitling the aquifer. In conjunction with these potential problems, periodic monitoring of well performance and surveying for subsidence are required.

5.3.7.2 Slurry Wall

The slurry cut-off wall is one of the most common solutions used for under-seepage control. It is a passive solution that requires no maintenance. However, considering the soft nature of the peat layers within the existing levee and foundation, the construction of slurry cut-of walls could become challenging because of the potential squeezing soft strata within the slurry trench. Experience with slurry walls along flood control levees in the Sacramento region has often resulted in leaks of slurry during construction. Because of the potential challenges associated with the construction of this technique, test sections would need to be conducted to validate the feasibility and constructability of slurry walls in the Delta.

Compared to the interceptor wells solution, the slurry cut-off method could be as much as 2 to 3 times more expensive.

5.3.7.3 Reservoir Floor Clay Blanket

The reservoir floor clay blanket is considered for comparison purpose. A 1000-foot long and three-foot thick clay blanket extending from the toe of the embankment toward the center of the reservoir would be needed to provide under-seepage control. Although this solution also offers a passive seepage control measure that would not require operation and maintenance, it could however, be exposed to the potential risks of drying and cracking if not maintained continuously under water.

This method would require a large volume of imported clay and the cost could be as high as six times that of the interceptor wells.

5.3.7.4 Collector Trench

Collector trenches constructed along the landside toe of the adjacent levees would be an effective method of collecting excess seepage and protecting against piping due to high exit gradients. The collector trenches would penetrate the overlying peat to the underlying sand aquifer. This alternative is also a passive seepage control system and would not require operation and maintenance. Given the low seepage flows that would occur (2 to 4 gpm), the excess seepage would be accommodated by discharging flows from the collector trench into the local drainage ditches within the neighboring islands. This technique is highly effective and readily constructed.

The major limitation of such a solution is the requirement for an encroachment permit within the neighboring islands and for long-term agreements with the neighboring island owners (including possibly some cost sharing of pumping effort to drain the islands). In other words, the seepage control measure would not be on State owned land, and hence access could become an issue.

The collector trench solution is one of the most attractive of the four on the basis of engineering merits alone. It comes at the lowest cost among the four alternatives, and is approximately one third the cost of the interceptor wells.

5.3.8 Summary of Findings

The seepage analyses conducted for three cross sections taken along the Webb Tract and Bacon Island levees shows that the proposed reservoir islands may increase the water table beneath the levee at adjacent islands 2 to 3.5 feet, and that flooding may occur in the neighboring islands in the absence of a seepage control system. Seepage flows at the neighboring island will increase by 1.5 to 2.5 times for an operating reservoir level of +4 feet. Exit gradients will also increase with greater increases where slough widths between the reservoir and the adjacent islands are narrower. At the narrowest section analyzed (Webb Tract Section 630+00) exit gradients increase to levels that could cause sand boils and piping.

A properly functioning seepage control system can be used to minimize the effects of the proposed reservoirs on adjacent islands, including the potential for rises in the ground water table or flooding. Interceptor wells (spaced at 160-feet with pump rates of about 6 to 8 gpm) are recommended for seepage control based on cost for alternatives that can be constructed within the reservoir areas. The interceptor well concept generally appears to be able to mitigate seepage problems induced by the proposed reservoirs. Proper design, construction, and maintenance will be key to the success of the interceptor well system.

Table 5.4 – Soil Properties Used in Seepage Analysis

Cross Section	Soil Layer	Approximate Soil Layer Thickness (feet)	Horizontal Hydraulic Conductivity K_x (cm/s)	Vertical Hydraulic Conductivity K_y (cm/s)
Webb Tract Sta. 260+00	Fill Material [1]	12	1×10^{-4}	1×10^{-5}
	Peat	32	2×10^{-4}	1×10^{-6}
	Sand	40	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Webb Tract Sta. 630+00	Fill Material [2]	10	1×10^{-4}	1×10^{-5}
	Fill Material [3]	5	1×10^{-6}	1×10^{-6}
	Peat	20	2×10^{-4}	1×10^{-6}
	Sand	45	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Bacon Island Sta. 665+00	Fill Material [4]	20	2×10^{-4}	1×10^{-6}
	Peat	18	2×10^{-4}	1×10^{-6}
	Sand	22	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}

[1] Clay with Peat and Sand
[2] Sand
[3] Clay
[4] Peat

Table 5.5 – Cross Sectional Models Used in Seepage Analysis

Cross Section	Water Elevation Slough (feet)	Water Elevation Reservoir (feet)
Webb Tract Sta. 260+00	-1 -1	Empty +4
Webb Tract Sta. 630+00	-1 -1 +3.5 [1] +3.5 [1]	Empty +4 Empty +4
Bacon Island Sta. 665+00	-1 -1	Empty +4

[1] Average high tide used. Reservoir full stage does not correspond to highest water stages typically between December through February.

Table 5.6 – Seepage Analysis Results

Location	Condition	Head in Sand at Near Levee CL (feet)	Head in Sand at Far Levee CL (feet)	Flow rate at Far Levee CL (gpm/ft)	Exit Gradient at Far Toe of Far Levee	Pumping Rate Required For Wells (gpm)	
						160’ spacing	200’ spacing
Webb Tract - Station 630+00	Existing	-12	-12	0.0045	0.21	NA	NA
	full reservoir	-2	-7	0.0115	0.57	NA	NA
	full reservoir w/pumping	-11	-12		0.23	6	7.5
	full reservoir high tide +3.5	-1	-6.5		0.64	NA	NA
	full reservoir exposed sand	0	-6.5		0.64	NA	NA
	full reservoir exposed sand w/pumping	-11	-12		0.24	8.7	10.8
Webb Tract - Station 260+00	existing	-11.5	-11.5	0.0056	0.13	NA	NA
	full reservoir	0.5	-9.5	0.0090	0.24	NA	NA
	full reservoir w/pumping	-10	-11.5		0.13	5.7	7.2
	full reservoir exposed sand	1.5	-9		0.25	NA	NA
	full reservoir exposed sand w/pumping	-11	-11.6		0.13	8.8	10.9
Bacon Island - Station 665+00	existing	-10.5	-10.5	0.0032	0.23	NA	NA
	full reservoir	0	-8	0.0067	0.34	NA	NA

5.4 Stability Analysis

5.4.1 Methodology

The stability of the embankments was analyzed using the limit equilibrium method based on Spencer’s procedure as coded in the computer program UTEXAS3 (Wright (1992)). In Spencer's procedure, side forces acting on all slice interfaces are assumed to have the same inclination. The trial-and-error solution coded in the program involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium conditions are satisfied. UTEXAS3 was used to compute factors of safety using either circular or general shaped, noncircular shear surfaces.

5.4.2 Results

Failure surfaces for all sections analyzed are included in the Embankment Design Analysis Report, Appendix A.

5.4.2.1 End-of-Construction

An analysis reflecting end-of-construction conditions was conducted for the "rock berm" option using the most critical case, Case 2 (base of peat at –40 feet). Slough water and reservoir groundwater levels were also selected to assess a critical condition.

The analysis indicates that the height of embankment that can be constructed in a single stage is dependent on the location of the boundary between the peat under levee and free field peat. A sensitivity analysis was performed by setting the material strength for peat under the levee to be equivalent to free-field peat and observing the location of the resulting critical failure surfaces for different fill heights.

Based on the sensitivity analysis, embankment construction should be staged using a 10H:1V reservoir-side slope, with the first stage being no greater than 8 to 10 feet in height. Successive construction stages are assumed to be allowed after eighty percent of consolidation resulting from the previous stage of construction has occurred. Three and eighteen months will be required for Case 1 and Case 2, respectively.

The above results indicate the need for careful planning and constructing the embankments in stages over several seasons (4 to 6 years). These results confirm that building up the embankments too rapidly could result in slope failure.

5.4.2.2 Long-Term Normal Operation

"Rock Berm" Option

Analyses were performed on the slough side for the best, average, and worst slough-side slopes for Case 1 (base of peat at –20 feet) and Case 2 (base of peat at –40 feet). Results indicate that for all of the cases considered stability criteria can only be met by adding a rock berm on the slough-side toe of the existing levee.

On the reservoir side stability criteria were met for Case 1, but not for Case 2. The addition of a thin horizontal reservoir-side toe berm of free draining material to Case 2 was required to meet stability criteria.

The results are presented in Tables 5.7 and 5.8 along with water surface elevations assumed on either side of the embankment.

"Bench" Option

Analyses were performed using average slough-side slopes to assess what combinations of bench width and elevation proposed for the project would meet stability criteria. Critical failure surfaces passing through both the bench and the crest were considered and the analyses were performed assuming full reservoir during low tide conditions for Case 1 and Case 2. The results of the analyses are presented in Table 5.9.

Generally, higher bench elevations decrease the calculated factor of safety for surfaces assumed to pass through the crest of the embankments and increase the calculated factor of safety for surfaces passing through the bench. Increased bench widths (i.e., shifting of the embankment crest towards the reservoir) increase the calculated factor of safety for surfaces passing through the crest. The analyses indicate that bench elevations in excess of 3 feet do not meet stability criteria. Where benches having elevation of 6 feet are desired and where slough-side slopes are steeper than the average cases analyzed, rock berms should be placed on the toe of the existing levees in order to meet stability criteria.

Long-term stability calculations towards the reservoir assumed a slough side bench elevation of 3 feet and bench widths required to meet stability criteria were 31 feet for Case 1 and 65 feet for Case 2. These bench elevations and widths were used for the remaining analyses of the "bench" option.

The results are presented in Tables 5.10 and 5.11 along with water surface elevations assumed on either side of the embankment.

5.4.2.3 Sudden Drawdown

"Rock Berm" Option

The results of sudden drawdown are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of sudden drawdown. Computed factors of safety range from 1.6 to 1.5 for Case 1 and Case 2, respectively. The results are summarized in Tables 5.7 and 5.8.

"Bench" Option

The results of sudden drawdown are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of sudden drawdown. Computed factors of safety range from 1.4 to 1.3 for Case 1 and Case 2, respectively. The results are summarized in Tables 5.10 and 5.11.

5.4.2.4 Psuedo-Static Analyses

The pseudo-static analyses were performed to estimate the yield accelerations (K_y) to be used in the seismic risk analysis (see Seismic Analysis Report). Yield accelerations were determined assuming non-liquefaction and liquefaction in the upper sand layer.

"Rock Berm" Option

Yield accelerations for portions of the islands where the upper portion of the sand layer does not liquefy range from 0.14 to 0.27 for Case 1 and from 0.09 to 0.12 for Case 2. Yield accelerations where the upper sand does not liquefy are only slightly sensitive to water levels in the reservoirs and slough. Where liquefaction occurs in the sand, yield accelerations are more sensitive to water levels in the reservoir and slough. The yield accelerations range from 0.03 to 0.12 for Case 1 and from 0.04 to 0.07 for Case 2. The results are summarized in Tables 5.7 and 5.8.

"Bench" Option

Yield accelerations for the "bench" option are sensitive to water levels in the reservoir and slough for all cases analyzed. Yield accelerations for portions of the islands where the upper portion of the sand layer does not liquefy range from 0.1 to 0.14 for Case 1 and from 0.06 to 0.09 for Case 2. Where liquefaction occurs in the sand, yield accelerations range from 0.03 to 0.07 for Case 1 and from 0.01 to 0.08 for Case 2. The results are summarized in Tables 5.10 and 5.11.

5.4.2.5 Post-Liquefaction Stability Analysis

Post-liquefaction stability analyses considered both circular and non-circular failure surfaces passing through the "liquefied layer. The development of earthquake-induced excess pore pressures in the existing levee materials was not considered, which is potentially unconservative. However, the entire loose sand layer was assumed liquefied, which is conservative.

"Rock Berm" Option

Computed factors of safety range from 1.9 to 3.0 for Case 1 and Case 2, respectively for those portions of the island where liquefaction of the upper portion of the sand layer does not occur. Where liquefaction of the upper portion of the sand layer occurs the computed factors of safety range from 1.3 to 2.4. The results are summarized in Tables 5.7 and 5.8.

"Bench" Option

Computed factors of safety range from 1.6 to 2.8 for Case 1 and from 1.5 to 2.1 for Case 2, for those portions of the island where liquefaction of the upper portion of the sand layer does not occur. Where liquefaction of the upper portion of the sand layer occurs the computed factors of safety range from 1.2 to 1.9 for Case 1 and from 1.1 to 1.7 for Case 2. For Case 2, a 2-foot layer of horizontal free draining fill was placed on the reservoir-side slope toe to increase the factor of safety from 1.0 to 1.1 for reservoir empty and high tide conditions. The results are summarized in Tables 5.10 and 5.11.

5.4.3 Summary of Findings

Stability criteria can be met for embankments having either the "rock berm" or "bench" configurations as slough-side slopes. For both configurations, where the base of peat is deep, minor modification to the sections are required in order to meet all stability criteria. Specifically, the "rock berm" option requires a free draining horizontal stability berm at the reservoir-side slope toe to meet criteria for long-term conditions for base of peat at elevations of –30 feet or lower. The "bench" option requires a free draining horizontal stability berm at the reservoir-side slope toe to meet post seismic stability criteria for those portions of the perimeter of the islands where liquefaction of the upper sand layer occurs and the base of peat elevation is -40 feet. End-of-construction stability analysis indicates that the embankments will require staged construction with the first stage being limited to a height of between 8 to 10 feet. Successive stages could be placed after eighty percent consolidation has occurred. This would occur after three months and 18 months, for peat with base elevations of –20 feet and –40 feet, respectively.

Based on the stability analysis presented in this section, the "rock berm" option appears to provide several advantages over the "bench" option as follows:

- Factors of safety for long term conditions toward the slough are higher, 2.0 to 1.8 compared with 1.6 to 1.5, suggesting less probability of an outward breach.
- Factors of safety for long term conditions toward the reservoir are higher, 1.9 to 1.7 compared with 1.6 to 1.5, suggesting less probability of an inward breach.
- Factors of safety for sudden drawdown conditions within the reservoir are higher, 1.6 to 1.5 compared with 1.4 to 1.3.
- Yield accelerations (and factors of safety for post seismic conditions) are equal or greater for nearly all conditions analyzed suggesting less deformation during earthquake events.

Table 5.7 – Stability Analysis Results "Rock Berm" Option [1] (Base of Peat at El. –20 feet)

Existing Slough Side Slope	Rock Berm Slope	Condition	Water Elevation		Side Slope Considered	F.S.	Ky
			Slough	Reservoir			
1.4H : 1V (worst)	none	long term	-1.0	4.0	Slough	0.9	--
	3H : 1V	long term	-1.0	4.0	Slough	2.4	--
2.6H : 1V (average)	none	long term	-1.0	4.0	Slough	1.1	--
	3H : 1V	long term	-1.0	4.0	Slough	2.0	--
		long term	7.0	empty	Reservoir	1.9	--
		sudden drwdn	6.0	4.0/empty	Reservoir	1.6	--
2.6H : 1V w/o liquifiable sand layer	3H : 1V	seismic	-1.0	4.0	Reservoir	2.8 [2]	0.14
		seismic	-1.0	4.0	Slough	2.7 [2]	0.25
		seismic	3.5	Empty	Reservoir	2.1 [2]	0.14
		seismic	3.5	Empty	Slough	3.0 [2]	0.27
2.6H : 1V w/ liquefiable sand layer	3H : 1V	seismic	-1.0	4.0	Reservoir	1.8 [2]	0.07
		seismic	-1.0	4.0	Slough	1.6 [2]	0.08
		seismic	3.5	Empty	Reservoir	1.3 [2]	0.03
		seismic	3.5	Empty	Slough	2.0 [2]	0.12
5H : 1V (best) [3]	none	long term	-1.0	4.0	Slough	1.4	--
	2' layer rock fill	long term	-1.0	4.0	Slough	1.8	--

[1] Slough bottom = -25 feet.
[2] post-seismic factor of safety
[3] Slough bottom = -20 feet

Table 5.8 – Stability Analysis Results, “Rock Berm” Option [1] (Base of Peat at El. –40 feet)

Existing Slough Side Slope	Rock Berm Slope	Condition	Water Elevation		Side Slope Considered	F.S.	Ky
			Slough	Reservoir			
2.35H : 1V (worst)	none	long term	-1.0	4.0	Slough	1.1	--
	3H : 1V	long term	-1.0	4.0	Slough	1.8	--
2.6H : 1V (average)	none	long term	-1.0	4.0	Slough	1.2	--
	3.5H : 1V	long term	-1.0	4.0	Slough	1.8	--
		long term	7.0	empty	Reservoir	1.2	--
		long term	7.0	empty	Reservoir [2]	1.7	--
		sudden drwdn	6.0	4.0/empty	Reservoir [2]	1.5	--
2.6H : 1V w/o liquifiable sand layer	3.5H : 1V	seismic	-1.0	4.0	Reservoir [2]	2.6 [3]	0.09
		seismic	-1.0	4.0	Slough	1.9 [3]	0.11
		seismic	3.5	Empty	Reservoir [2]	2.0 [3]	0.09
		seismic	3.5	Empty	Slough	2.3 [3]	0.12
2.6H : 1V w/ liquefiable sand layer	3.5H : 1V	seismic	-1.0	4.0	Reservoir [2]	2.4 [3]	0.06
		seismic	-1.0	4.0	Slough	1.4 [3]	0.04
		seismic	3.5	Empty	Reservoir [2]	1.4 [3]	0.04
		seismic	3.5	Empty	Slough	1.8 [3]	0.07
3.5H : 1V (best)	none	long term	-1.0	4.0	Slough	1.3	--
	4H : 1V	long term	-1.0	4.0	Slough	1.6	--

[1] Slough bottom = -30 feet.
[2] with u/s 2 foot thick horizontal rock berm
[3] post-seismic factor of safety

Table 5.9 – Stability Analysis Results, “Bench” Option - Sensitivity to Bench Elevation and Width

Peat Condition	Bench Elevation	Bench Width (feet)	Factor of Safety	
			Crest	Bench
Peat at –20 feet	6.0	34.0	1.33	2.00
	0.0	27.0	2.60	1.36
	3.0	31.0	1.64	1.63
Peat at –40 feet	6.0	36.0	1.36	1.39
	2.0	36.0	1.99	1.33
	2.0	65.0	1.94	1.46
	3.0	60.0	1.68	1.48
	3.0	65.0	1.68	1.52

[1] Long-term Condition (towards slough)
[2] Average slough-side slope used.
[3] Reservoir at 4.0 feet
[4] Water surface in slough at –1.0 feet

Table 5.10 – Stability Analysis Results, “Bench” Option [1] (Base of Peat at El. –20 feet)

Condition	Water Elevation		Side Slope Considered	Levee Crest		Bench [2]	
	Slough	Reservoir		Ky	F.S	Ky	F.S.
long-term	7.0	empty	Reservoir	--	1.6	--	--
	-1.0	4.0	Slough	--	1.6	--	1.6
sudden drawdown	6.0	4.0/empty	Reservoir	--	1.4	--	--
seismic w/o liquifiable sand layer	-1.0	4.0	Reservoir	0.14	2.8	--	--
	-1.0	4.0	Slough	0.095	1.6	0.094	1.7
	1.5	-2.5	Reservoir	0.13	2.5	--	--
	1.5	-2.5	Slough	0.120	1.8	0.12	2.2
	3.5	empty	Reservoir	0.14	2.1	--	--
	3.5	empty	Slough	0.11	1.8	0.12	2.6
seismic w/liquifiable sand layer	-1.0	4.0	Reservoir	0.07	1.9	--	--
	-1.0	4.0	Slough	0.027	1.2	0.094	1.7
	1.5	-2.5	Reservoir	0.05	1.5	--	--
	1.5	-2.5	Slough	0.053	1.4	0.12	2.2
	3.5	empty	Reservoir	0.027	1.2	--	--
	3.5	empty	Slough	0.063	1.5	0.12	2.6

[1] Average Slough-Side Slope used.
[2] Bench Elevation = 3.0 feet

Table 5.11 – Stability Analysis Results, “Bench” Option [1] (Base of Peat at El. –40 feet)

Condition	Water Elevation		Side Slope Considered	Levee Crest		Bench [2]	
	Slough	Reservoir		Ky	F.S	Ky	F.S.
long-term	7.0	empty	Reservoir	--	1.5	--	--
	-1.0	4.0	Slough	--	1.5	--	1.7
sudden drawdown	6.0	4.0/empty	Reservoir	--	1.3	--	--
seismic w/o liquifiable sand layer	-1.0	4.0	Reservoir	0.094	2.1	--	--
	-1.0	4.0	Slough	0.06	1.5	0.08	2.0
	1.5	-2.5	Reservoir	0.086	1.9	--	--
	1.5	-2.5	Slough	0.085	1.7	0.125	2.5
	3.5	empty	Reservoir	0.07	1.5	--	--
	3.5	empty	Slough	0.078	1.7	0.110	2.7
seismic w/liquifiable sand layer	-1.0	4.0	Reservoir	0.058	1.6	--	--
	-1.0	4.0	Slough	0.009	1.1	0.082	2.0
	1.5	-2.5	Reservoir	0.075	1.7	--	--
	1.5	-2.5	Slough	0.029	1.2	0.125	2.5
	3.5	empty	Reservoir	0.015	1.1	--	--
	3.5	empty	Slough	0.03	1.3	0.110	2.7

[1] Average Slough-Side Slope used.
[2] Bench Elevation = 3.0 feet

5.5 Estimated Probability of Failure

5.5.1 General

The probability of embankment failure during normal operations is the aggregate of the probability of failure of identifiable failure modes. These failure modes include: 1) internal erosion and piping due to high exit gradient caused by excessive seepage, 2) erosion through cracks in the existing levees and engineered embankment caused by differential settlement or unstable slopes, and 3) overtopping caused by slumping or loss of freeboard due to slope failure or excessive settlement.

Houston and Duncan (1978) predicted the aggregate annual probability of failure of the existing levees (0.02 for Bacon Island and 0.05 for Webb Tract) based on 27 years of historical observation of levee failures in the Delta. The predicted probabilities were calculated for 40 years of continued use of the islands for farming where the island elevations would continue to subside at a rate of 3 inches per year.

The engineered embankments will be much improved compared to the existing levees because the long-term factor of safety for stability meets the adopted design criteria of 1.5 or higher and seepage exit gradients will be 0.3 or lower. Based on the improvement of the engineered embankments over the existing levees, it is judged that the annual probability of failure would be approximately 100 times smaller than for the existing Bacon Island levees. Because the new embankments for both islands would be designed to meet the same criteria, the annual probability of failure was assumed to be the same. For this study, the annual probability of failure for the new embankments was estimated to be 2×10^{-4}.

The contribution to the annual risk of failure from the different failure modes is described in the following paragraphs. The Embankment Design Analysis Report, Appendix B, provides further discussion on calculation of probabilities of failure.

5.5.2 Probability of Failure Due to Internal Erosion

Failure from internal erosion can occur due to high exit gradients caused by excessive seepage or through cracks in the existing levee or new embankment that may form during consolidation of the underlying soft soils. For the purposes of this evaluation, it has been assumed that protection against internal erosion due to cracking consisting of filter fabric between the existing levee and new embankment would be installed at selected locations around the islands where there is a greater likelihood of cracking to occur during construction. The filter fabric would provide piping protection for materials that are up-gradient of the fabric. Alternatives for mitigation measures against internal erosion failures due to cracking and piping are discussed in the Embankment Design Analysis Report, Appendix C.

The probability of failure from internal erosion due to high exit gradient caused by excessive seepage or cracking during normal operations was calculated using the method described in USBR (1997). The method requires the identification of steps leading to failure, assigning a probability of the occurrence of those steps, and multiplying the probability of occurrence of each of those steps to obtain the total probability of failure. The steps identified and the probability of each of the steps occurring are outlined in Appendix B. The calculated annual probability of failure due to internal erosion during normal operations (not including flood events) is 1.27×10^{-4} (considering weighted contribution for inward and outward flows).

5.5.3 Probability of Seepage Failure During Flood Events

During high flood stage, a greater head difference between the water surface in the reservoir and the adjacent slough can exist compared to normal operations. Exit gradients at the toe of the new embankments during high flood stage (up to 300 year event) were calculated to be 25 percent higher (0.25 compared with 0.20) than during normal operations. These gradients are still less than those that could cause sand boils or piping. To estimate the probability of failure due to internal erosion during high flood stage, the contribution of different flood stages was proportioned to the percent change in the corresponding exit gradient as shown in Appendix B. On an annualized basis, the probability of failure due to internal erosion during flooding events is estimated to be 0.27×10^{-4}. The combined annual probability of failure due to seepage-induced piping under all tide and flood stages below elevation +10 feet is estimated to be $(1.27+0.27)x10^{-4}$ or $1.54x10^{-4}$.

5.5.4 Probability of Overtopping Caused by Slope Failure or Excessive Settlement

The probability of failure due to overtopping caused by slumping or loss of freeboard due to slope failure or excessive settlement can be calculated as the difference between the annualized aggregated probability of failure and the probabilities of failure due to internal erosion during normal operations included high flood events. This calculated probability of failure is estimated to be $(2 – 1.54)x10^{-4}$ = $0.46x10^{-4}$. This calculated probability of embankment failure should be relatively lower due to the following:

- The embankments are designed for a long-term factor of safety of 1.5 and higher.
- Foundation soil strengths and embankment strengths are based on back-calculated strengths from a failure on Webb Tract and likely represent some of the lower strengths for the islands.
- There will be opportunity to assess settlement and stability during the five-year construction period. Areas of the islands that exhibit settlement or stability problems could be addressed during construction.

5.6 Summary and Conclusions

5.6.1 Seepage

The findings from the seepage analysis were based on two representative sections for Webb Tract and one section for Bacon Island. The cross sections at Webb Tract island were selected for the “narrowest” and “widest” slough width across reservoir island and neighboring island. The section across Bacon Island represents a case that lies in-between the “narrowest” and “widest” cases of Webb Tract. These cross sections represent somewhat a bounding of the seepage conditions. The following major findings emerged from the seepage evaluations.

- Seepage mitigation measures should be considered to control undesirable seepage flooding effects on adjacent islands that may occur as a result of the reservoirs.
- Seepage control by interceptor wells placed on the levees of the reservoir islands, as proposed, appears effective to control undesirable seepage effects. Well spacings of a minimum of 160 feet would be required where the adjacent slough is the narrowest. Wider well spacings could be used at other locations. The required pumping rates of about 6 to 8 gpm appear to be reasonable and manageable.
- Success of an interceptor well system will be a function of proper design, construction, maintenance, and monitoring.
- Other seepage control alternatives should be further investigated because of their potential engineering merits.

Based on the results of the current study, the following recommendations are made:

- Sensitivity analysis reported in URS (2000) demonstrated that increases in the permeability of the sand layer significantly increase calculated seepage volumes. Site specific pump tests located at potential seepage area on Webb Tract and Bacon Islands are recommended for design of the interceptor system.

- Pilot test borings should be drilled along those portions of Bacon Island and Webb Tract where interceptor wells are planned. Data gathered from the borings should be used for final design of the well system.
- During final design, Webb Tract and Bacon Islands should be surveyed for potential seepage problem areas. Potential seepage areas should be analyzed individually using parameters obtained from pump tests and additional borings.
- Test interceptor well sections should be installed and tested based on data collected from pump tests and pilot borings. Results of the test sections should be incorporated into the final design.

5.6.2 Embankment Configuration

Two configurations for the project's embankments have been evaluated by extensive stability analyses of two sections selected to be representative of the lowest and highest elevations at which the base of the underlying peat layer is found in the two islands. Stability analyses were performed for the more severe situations expected at the reservoir islands. The calculated factors of safety have been compared to the project's stability criteria, and judgments were made of the adequacy of the proposed project in regard to embankment stability. The resulting conclusions and recommendations are:

- Construction of the levee strengthening fills must be implemented in a manner to prevent stability failures due to the new fill loads. This will require carefully planned staged construction, and monitoring to observe the behaviors as the fill is placed. The staged construction will require a construction period estimated to extend over 4 to 6 years.
- Both the "rock berm" and "bench "option" can be constructed to meet the project's required stability criteria. For some combinations of existing reservoir bottom elevation and base of peat elevation reservoir-side slope free draining toe berms are required to meet stability criteria.
- Based on the stability analysis presented in this section, the "rock berm" option appears to provide several advantages over the "bench" option as follows:
    - Calculated factors of safety for all analysis cases are greater than calculated for the "bench" option suggesting a lower probability of failure during normal operations.
    - Calculated yield accelerations are generally greater than for the "bench" option suggesting less earthquake induced deformation. Deformations are addressed in the Seismic Analyses Report.
    - Fill volumes for new embankments are significantly less due to less consolidation deformation under new embankment and the absence of setback.
- A probability of failure of the embankments during normal operations based on engineering judgement was presented.

Based on the results of the current study, the following recommendations are made:

- Implement an extensive subsurface exploration program along the reservoir island levees, followed by stability evaluations and site-specific detailed design and construction to provide adequate embankment stability during design. These steps will be essential to achieve safety and effectiveness of the proposed embankment system.
- Conduct of the subsurface exploration program should include sample collection and laboratory testing designed to evaluate the potential for liquefaction of the reservoir side of the existing levees, the variation of the strength of peat under levee and free field peat and the transition between them, and the change in strength in the peat as it consolidates under the new embankment.

- Conduct a survey of Webb Tract and Bacon Island to determine the extent and thickness of existing rockfill on the slough-side slopes. Where rockfill exists on the slough-side slopes, rock berm slopes required to meet stability criteria may be reduced.
- Implement a test fill section during design for the preferred embankment geometry at locations where the base of peat is located at elevations –20 feet and –40 feet. The test fill program would provide valuable information regarding consolidation rates and ultimate settlement for estimating the time required for staged construction. The test fills should be monitored using piezometers, settlement survey monuments, and visual observation during and after construction.
- Include in the final design a filter fabric between the new embankment and existing levee to provide piping protection for materials that are up-gradient of the fabric. Determination of the locations along the reservoir embankments for filter fabric as a piping mitigation measure should be made during future engineering studies.

Chapter 6: Borrow Area Delineation and Quantity Estimation

6.1 Introduction

This investigation included identifying feasible borrow sites within Webb Tract and Bacon Island, assessing the suitability of the soils as borrow materials for earthwork, and estimating the volume of borrow materials available from each identified location. The total quantity of suitable borrow material available at each island is compared with the earthwork planned at each island in the construction and cost estimates investigation.

For the purpose of this study a "feasible borrow site" is defined as a site where the top surface of geotechnically-acceptable borrow soil deposits occurs within a depth of 15 feet below existing ground surface and where dewatering requirements related to borrow operation are expected to be low.

6.2 Site Conditions

Site locations, accessibility and surface conditions are described in detail in the URS Flooding Analysis report.

6.3 Review of Existing Data

As part of this study, the following documents were reviewed:

- Borrow Sites, Staged Filling and Slough-side Slope Stability, Delta Wetlands Reservoirs, Contra Costa County and San Joaquin County, California, dated July 25, 2002, prepared by Hultgren-Tillis Engineers.
- Bureau of Reclamation Cone Penetrometer Test data for the In-Delta Storage Program conducted during August to September 2002.
- Preliminary Geotechnical Investigation, Delta Wetlands Project, Sacramento-San Joaquin River Delta, Volume 1 of 2, dated February 15, 1989, prepared by Harding Lawson Associates (HLA).

These documents include data and information related to material handling and to potential borrow areas and volumes of borrow material available on Webb Tract and Bacon Island.

The locations of the USBR CPT soundings and the HLA borings are shown on Figure 6.1 (Webb Tract) and Figure 6.2 (Bacon Island).

The subsurface soil data presented in the previous studies indicates that a layer of peat and fat clay of variable thickness was encountered in the upper part of the soil stratigraphy within the islands. The thickness of this layer ranges from a few feet to about 40 feet. This peat and clay layer is underlain by a layer of gray silty fine sand and sandy silt, which is suitable for borrow materials. In some areas on the west side of Webb Tract, sand is exposed at the ground surface.

6.4 Field Exploration

A field exploration program was conducted for this study and included a field reconnaissance and geotechnical exploratory borings and sampling.

A geotechnical and environmental field reconnaissance on Webb Tract and Bacon Island was conducted during December 5 and 6, 2002 to identify the borehole locations and to examine a 50-foot radius circle around each drilling site for potential burrows or surface cracks. The drilling locations were adjusted to maintain a minimum of 50-foot radius clear of burrows or surface cracks and were located on disturbed areas, either on or adjacent to farm roads or within active agricultural fields.

Ten exploratory borings were drilled on each island during December 11 and 12, 2002. These borings totaled 165 linear feet and ranged in depth from 15 to 19 feet below the existing ground surface. The borings are designated W-1 to W-10 for Webb Tract and B-1 to B-10 for Bacon Island and are shown on Figure 6.1 (Webb Tract) and Figure 6.2 (Bacon Island).

A URS engineer logged the soil cuttings and samples in the field and visually classified the soils as the drilling proceeded. Samples of the subsurface materials were obtained at selected depths in the borings using a Standard Penetration Test (SPT) split-spoon sampler. Soil samples were also collected of the potential borrow materials that were visually classified as sand, silty sand, clayey sand, sandy clay, or sandy silt.

The recovered samples were then taken to the URS geotechnical laboratory in Pleasant Hill for further visual examination and testing.

6.5 Laboratory Testing

Logs of Borings were prepared based on the field logs, the visual examination in the laboratory, and the laboratory testing results. Further laboratory testing was conducted on selected soil samples obtained from the exploratory borings to evaluate their engineering properties for use in the borrow material evaluations. The following laboratory tests were performed on the selected soil samples:

- Grain size analyses
- Water content determination
- Atterberg limits determination

The logs of borings and the geotechnical laboratory test results are presented in Appendix A and Appendix B of the URS Flooding Analysis report, respectively.

6.6 Subsurface Conditions

6.6.1 Subsurface Soil Conditions

On both islands, there is a highly organic soil and peat layer that ranges from a few feet to more than 15 feet thick. On Webb Tract, this layer is underlain by gray, silty sand (SM and SP-SM) that extends to the depth explored. This material varied in consistency from loose to medium dense and contained interbedded thin layers of gray sandy silt. On Bacon Island, the organic soil and peat layer is underlain by a gray, silty sand (SM) layer that extends to the depth explored. This material varied in consistency from loose to medium dense and contained interbedded thin layers of gray silty clay. The silt and clay contents and the water contents in the materials encountered in Bacon Island are higher than for the materials encountered in Webb Tract.

6.6.2 Groundwater Conditions

The level of groundwater encountered at the time of drilling in the borings in Webb Tract varied from about 2 feet to 9 feet below the ground surface with most levels around 2 feet to 5 feet below the ground surface. The level of the groundwater encountered at the time of drilling in the borings in Bacon Island varied from about 3 feet to 13 feet below the ground surface. Based on the water levels measured at the time of drilling, the groundwater levels in Bacon Island are deeper than in Webb Tract. The groundwater levels are largely affected by the irrigation and drainage system within the islands. Static groundwater levels were not recorded due to the immediate backfill of the borings with soil cuttings. Accordingly, the static water levels are expected to be shallower than those measured at the time of drilling.

6.7 Estimated Available Borrow Volumes

The potential borrow areas in Webb Tract and Bacon Island were delineated based on maintaining a distance of at least 1500 feet between the borrow areas and the crests of the existing island levees, and encompassing areas that have no more than 15 feet to the top of potential sandy borrow materials. The borrow area delineations are shown on Figures 6.1 and 6.2 for Webb Tract and Bacon Island, respectively. These figures also show the depths to the top of sandy borrow materials adjacent to the borings and CPT's.

Table 6.1 summarizes the acreage of the potential borrow areas, estimated volume to remove peat and other unacceptable overburden soils, estimated borrow material volumes available within 15 feet of the ground surface, and ratios of overburden volume to borrow volume.

Table 6.1 – Summary of Available Borrow Volume Estimates

Estimated Area/Volume	Webb Tract	Bacon Island
Delineated Area (acres)	2330	2620
Volume of Overburden Excavation (CY)	36.9 million	49.6 million
Volume of Potential Borrow Materials within 15 feet of the Ground Surface (CY)	19.5 million	13.8 million
Ratio of Overburden Volume to Borrow Volume	1.9:1	3.6:1

6.8 Borrow Development Considerations

The borrow area limits shown on Figures 6.1 and 6.2 are the maximum potential areas within each island. Specific areas within each island to be utilized would depend on the contractor's operation plans and excavation conditions encountered during construction to make use of the most readily available materials. A trade-off would need to be made between haul distance and excavation of overburden materials.

It is expected that the contractor would develop sections within each borrow area to minimize haul distances and to make use of the materials with the least amount of overburden stripping. The ratios of overburden volume to borrow volume shown in Table 6.1 indicate that there would be a significant amount of stripping required to obtain the borrow materials. Stripped materials would be stockpiled for subsequent placement in the depleted sections of the borrow areas.

It is anticipated that the sandy borrow materials would be mined by excavators, mostly below groundwater level, and stockpiled to drain, since groundwater may be as shallow as 2 feet or 3 feet below the ground surface. Moisture conditioning of the soils may require disking and aerating. After the soils are moisture conditioned for compaction, they would be hauled to the embankment locations along the perimeters of the islands.

6.9 Recommendations

For further development of the In-Delta Storage embankments, supplemental drilling, laboratory testing, and CPT soundings should be performed in the potential borrow areas. Standpipe piezometers should be installed in selected borings to measure groundwater levels.

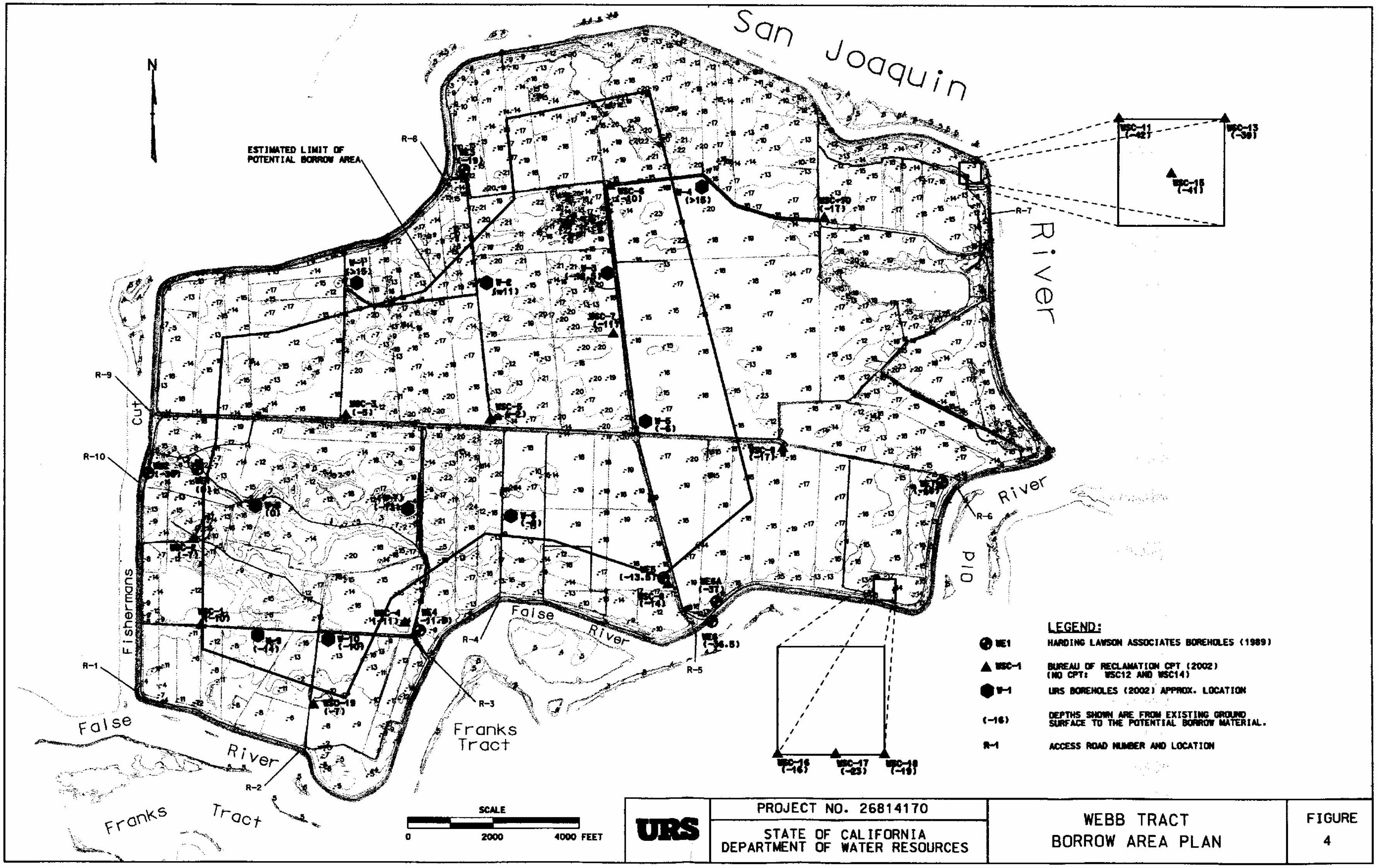


Figure 6.1 – Webb Tract Borrow Areas

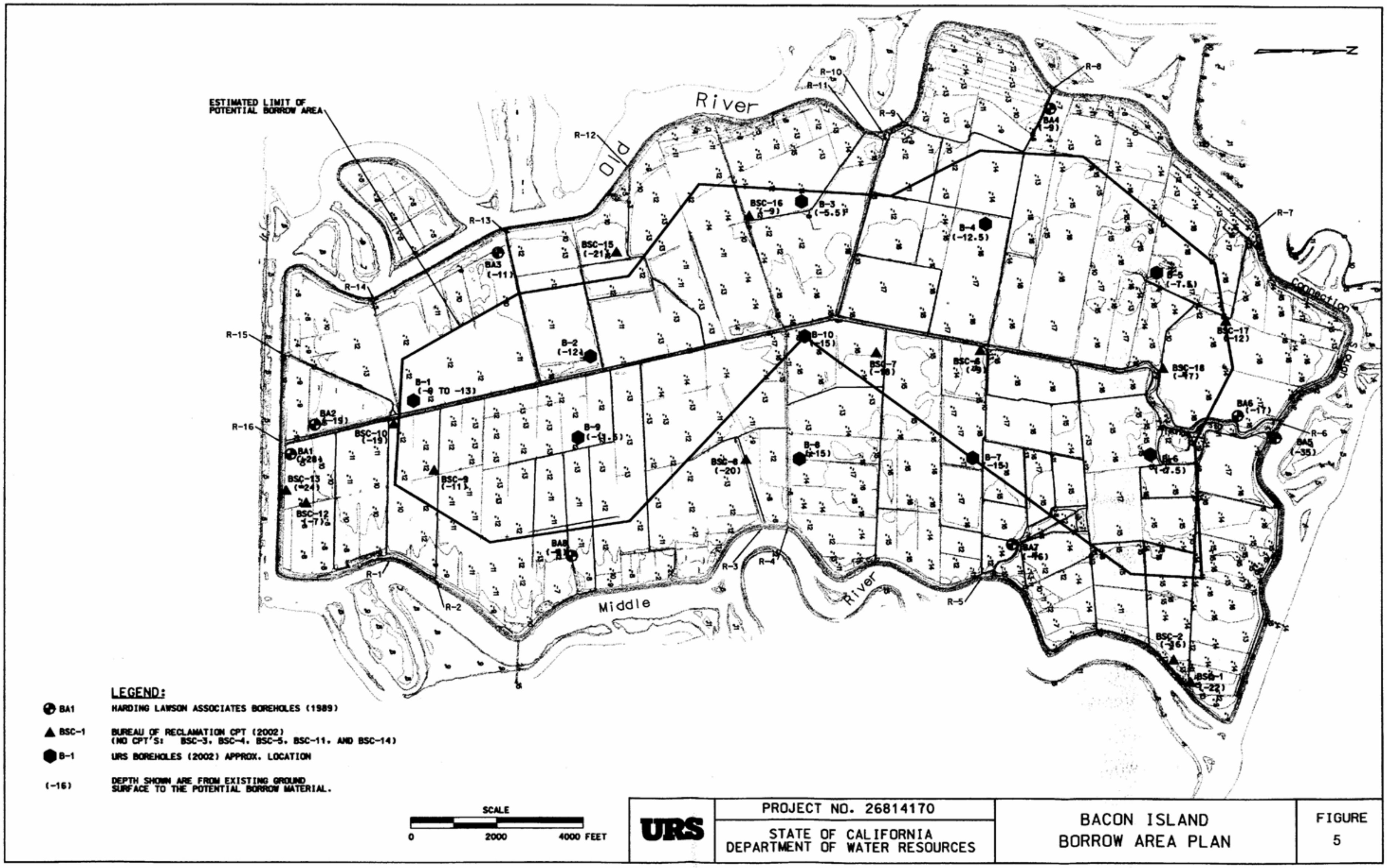


Figure 6.2 – Bacon Island Borrow Areas

Chapter 7: Integrated Facilities Engineering Design and Analyses

7.1 Introduction

The integrated facilities are consolidated control structures that will be used to control the diversion and release of water onto and off of the reservoir islands. There are a total of four integrated facilities, two on Webb Tract and two on Bacon Island. Figure 7.1 shows a 3-dimensional illustration of a typical integrated facility.

The key features of each integrated facility are as follows:

- The fish screen is isolated from the other controls with a transition pool
- Storage diversions and releases can occur when the river and reservoir are at different levels, allowing for year-round operations
- Diversions and releases are optimized with gravity flow and pumping combinations
- Required flow under gravity is possible with small head differences
- Low midbay level and pumping units allow for complete drainage of reservoir when necessary

A number of hydraulic analyses were conducted to determine the overall layout of the integrated facilities. Mechanical designs were prepared for the pumping plants, conduits and gate structures and an electrical analysis was performed to size the transformers required to supply power to each integrated facility. Structural analysis and design was prepared in sufficient detail to allow a feasibility-level cost estimate of the four proposed integrated facilities to be completed. The main components of the integrated facilities are described below. All design drawings for the integrated facility components are provided in the *Integrated Facilities Engineering Design and Analyses* report, *Appendix C.*

Fish Screen Facility: The fish screen facility is located at the entrance to the integrated facility and is oriented adjacent and parallel to the river channel. The objective of the fish screen facility is to pass the design diversion rate over a range of water levels in both the river channel and the reservoir while protecting juvenile fish from entrainment, impingement and migration delay.

Transition Pool: The transition pool is located immediately downstream of the fish screen facility. The purpose of the transition pool is to separate the fish screen from the other operational controls, create a smooth transition of flow from the very wide section of the fish screen facility to the narrow section at Gate #1, and act as a settling basin to prevent excess suspended silt from entering the reservoir.

Gate Structures: Each integrated facility consists of three gate structures. Each gate structure operates strictly by gravity flow and serves a unique purpose in the integrated facility operations. Gate #1 is used strictly during diversion operations to regulate flows into the midbay. Gate #2 is used to regulate the flow of water from the midbay to the reservoir during diversion operations. Gate #2 can also be used to regulate the flow of water out of the reservoir and into the midbay during release operations. Gate #3 is used strictly during release operations to regulate flows from the midbay into the bypass channel.

Midbay: The midbay is located at the center of the integrated facility gate structures and pumping plant. The midbay serves as a flow regulation pool during diversion and release operations. It also serves as a forebay for the pumping plant when it is operating.

Pumping Plant and Conduit: The pumping plant is located adjacent to the midbay on the side opposite to Gate #1 and the conduit pipes stretch from the reservoir side of the integrated facility to the bypass channel. The pumping plant serves two main purposes: (1) to supplement diversion and release gravity flows when sufficient head is not available at the gate structures to meet the desired flow rates by gravity and (2) to meet the desired flow rate when the net head is zero or negative. The pumping plant consists of five pumping units, three pumps with a capacity of 400 cubic feet per second (cfs) each and two pumps with a capacity of 150 cfs each, totaling a maximum pumping capacity of 1500 cfs. The pumped flows will be routed through the conduit pipes, which are used to discharge water into the reservoir and bypass channel during diversion and release pumping operations, respectively. The conduit pipes can also be used for gravity flow releases to supplement the gravity flow releases through Gate #3.

Bypass Channel: The bypass channel is used to convey reservoir releases into the river. Reservoir releases enter the bypass channel at its upstream end through the conduit pipes and/or through Gate #3. The bypass channel is isolated from the fish screen facility and transition pool by a structural sheet pile wall. A vehicle access bridge spans the bypass channel and is connected on one end to the integrated facility embankment and on the other end to the fish screen structure.

Embankments: Engineered embankments will surround the integrated facility on the reservoir side and will surround the midbay on all sides. All embankments will have 3H:1V side slopes on both the interior and exterior slopes. Riprap slope protection will be placed on all embankments along the entire slope.

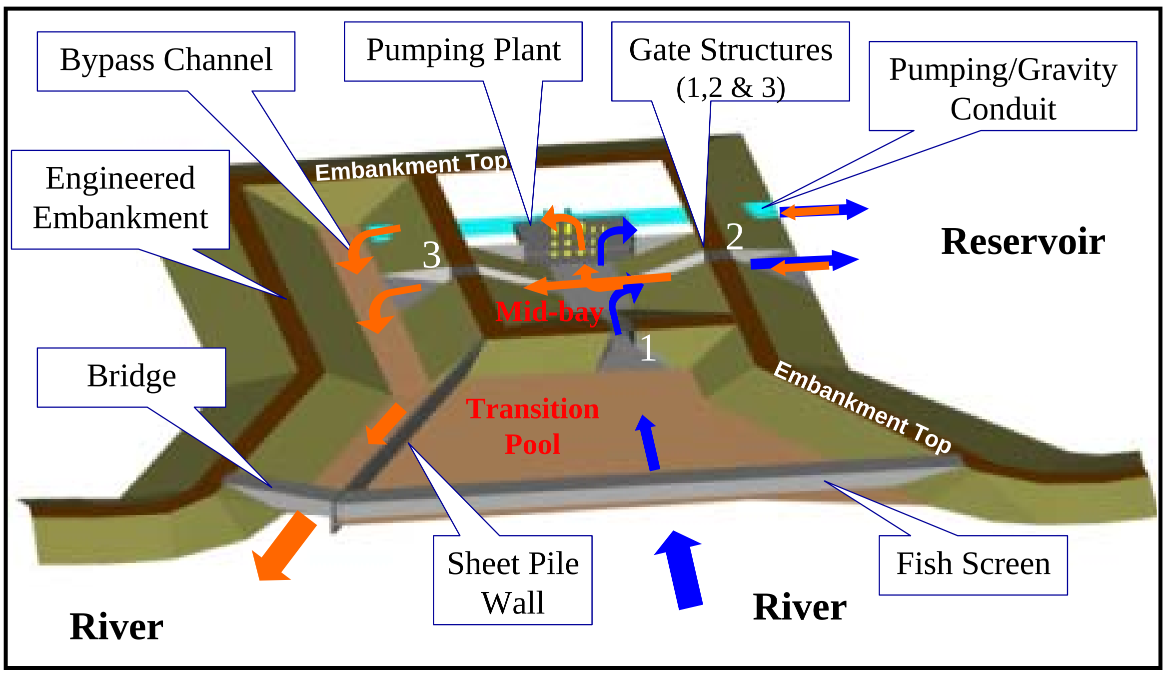


Figure 7.1 – 3-Dimensional Illustration of a Typical Integrated Facility

7.1.1 Direct Connection to Clifton Court Forebay

A direct connection to Clifton Court Forebay (CCF) from the Bacon Island Santa Fe Cut Integrated Facility was considered to supply make-up Environmental Water Account (EWA) water for EWA imposed SWP/CVP export curtailments. This direct connection would be in addition to the proposed configuration and operation of the Bacon Island Santa Fe Cut Integrated Facility. Pumping units along with a conveyance system and an outlet channel would be added to the proposed facility to convey 900 cfs directly to CCF. This direct connection cannot be justified due to costs outweighing the benefits, but it may be considered as a part of the proposed fish screens at the new CCF intake, reducing the required screen size of the new CCF intake. With that said, the cost of this direct connection will not be added to the overall In-Delta Storage Project cost. Instead, the cost of this direct connection could be counted as an avoided cost of the proposed fish screens at the new CCF intake project, if deemed justifiable. Details of the design and cost of this connection are provided in the *Integrated Facilities Engineering Design and Analyses* report, *Appendix D*.

7.2 Fish Screen Facility

The objective of the intake structure is to divert the required flow over the desired range of water levels in the channels and in the reservoir with hydraulic efficiency. The intake structure should also divert the design discharge under constraints imposed by operational and environmental considerations. Specifically, the intake structure should not hamper the movement of the fish species present in the river channel. The fish screens are the part of the intake structure intended to effectively protect juvenile fish from entrainment, impingement, and migration delay.

7.2.1 Design Criteria

NMFS and DFG have established a number of criteria for the design and operation of fish screens installed at diversion points. The criteria are related to biological considerations, and hydraulics and hydrologic requirements for fish screening structures. The criteria listed below, was used to design the fish screen structures at the In-Delta Storage Project intake sites.

- The screen should allow diversions up to 1500 cubic feet per second (cfs) at low stage and 2250 cfs at high stage.
- The screen face shall be placed parallel to the river flow and adjacent bank lines. The intake facility should be designed to minimize or eliminate areas of reverse flow or slack water. These areas are predator habitat.
- The structure must allow migrants to move freely in the channel adjacent to the screen area. The transition between the fish screen structure wing walls and the channel embankment should be smooth.
- For self-cleaning screens and for all flow conditions, the approach velocity shall not exceed 0.2 ft/sec. The approach velocity is the water velocity 3 inches in front and perpendicular to the screen face.
- The approach velocity in front of the screen should be distributed uniformly across the face of the screen.
- The flow velocity component parallel to the screen face, known as sweeping velocity, must be twice the approach velocity (0.2 ft/sec).
- NMFS recommends an upper limit of 60 seconds as the desirable fish passage time at approach velocities of 0.4 ft/sec. Fish passage time is defined as the length of time a fish is in front of the screen. For approach velocity of less than 0.4ft/sec, longer contact time may be applied with NMFS approval.
- For vertical profile bar type fish screens, the screen openings should not exceed 0.0689 inches in width (1.75 mm).
- The screen material shall provide a minimum of 27 percent open area.

- For all hydrologic conditions, the screen material should be strong enough to withstand the water pressure caused by differential head over the screen faces. The fish screen material used should be corrosion resistant and antifouling.
- The head difference to trigger fish screen cleaning shall be a maximum of 0.1 feet. To avoid flow impedance and violations of approach velocity criteria, a cleaning frequency of 5 minutes is desired.
- Structural features shall be provided to protect the fish screens from large debris.

Intake Site Water Levels

- The fish screen facility will be designed for a 100-year return period maximum river stage. To account for climate change effects, the system performance will be checked with a 300-year return period flood. The design flood levels and their respective return periods were taken from the Flooding Analysis Report and are summarized in Table 7.1.

Table 7.1 – Design Flood Levels

Facility	Flood Stage (ft)	
	100-year	300-year
Webb Tract, SJR	7.0	7.2
Webb Tract, False River	7.0	7.2
Bacon Island, Middle River	7.2	7.5
Bacon Island, Santa Fe Cut	7.3	7.5

The diversion capacity of each fish screen facility will be 1500 cfs for low stage operations and 2250 cfs for high stage operations. The facility will be operated year-round and it will be able to deliver the design discharge at low stage as well as high stage conditions. The low stage condition is defined as the tidal stage when the river stage is at 90 percent level of the Low-Low tidal stage. The high stage condition is defined as the tidal condition when the river stage is at or above the 50 percent level of the Low-High tidal stage. The probability levels and the corresponding stages are summarized in Chapter 2, Table 2-3. These two tidal stages will be used to determine screen height, total width of the fish screen facility, and top level of the steel face wall.

7.2.2 Design and Layout

7.2.2.1 General Layout

The fish screens will be placed in a location where the river alignment is fairly straight such that the fish screen face is nearly parallel to the adjacent riverbank. This will minimize the contact time of the migrating fish species with the fish screen facility. The velocity distribution in front of the screen should be uniform to ensure proper functioning of the screens. Each facility is located in areas influenced by tides, and depending upon the tidal cycle and hydrology the channel velocity will fluctuate and may even change its direction. Therefore, the criteria dealing with the sweeping velocity and passage time requirements may be applicable to the screens for flows in two tidal directions.

The components of each fish screen facility will include a log boom, fish screen, cleaning device, adjustable baffles, debris collection and removal system, reinforced concrete box culvert structural section, and an access road. The layout plan and cross section of the proposed fish screen facility are shown in the Integrated Facilities Engineering Design and Analysis Report, Appendix C.

7.2.2.2 Fish Screen Sill Elevation

The fish screen sill elevation is based on the site and channel topography and nature of site soils. Table 7.2 summarizes the selected sill level for each site. Lowering the sill level would provide additional screen height and less overall screen width, but it may exacerbate the accumulation of sediments in the fish screen facility.

Table 7.2 – Intake Site Topographic and Sill Elevations

Facility	Average Topographic Elevation	Sill Elevation	Top of Deck/Levee Elevation (ft)
Webb Tract, SJR	-10	-12	11.0
Webb Tract, False River	-13	-15	11.0
Bacon Island, Middle River	-10	-12	10.2
Bacon Island, Santa Fe Cut	-7	-9	10.4

7.2.2.3 Fish Screen Facility Width

The width of the intake facility should be sufficient to pass 1500 cfs during low flow conditions and 2250 cfs during high flow conditions in the river channel.

For the design flow of 1500 cfs, the minimum gross wetted screen area required is 7500 ft^2. The intake facility is designed to deliver this flow at minimum slough stage, which is the 90 percent probability of exceedance level for the low-low tidal stage. The difference between the 90 percent low-low tidal stage and the fish screen sill level gives the effective fish screen height. Using this height, the limiting approach velocity, and a design flow of 1500 cfs, the required fish screen width was determined.

For the design flow of 2250 cfs, the minimum gross wetted screen area required is 11,250 square feet. The intake facility is designed to deliver this flow when the slough stage is higher than the 50 percent probability of exceedance level of the low-high tidal stage. Following the same procedure described above, but for a screen height relative to the 50 percent exceedance level of the low-high tidal stage, the required fish screen width was determined.

Once the required screen width for each site was determined under both flow conditions, as described above, the results were compared and the larger of the two screen widths was chosen for the design.

For all sites, the required width of screen is too large for a single screen, so the facility will be divided into bays. Each bay will have a clear span of 20 feet and will be separated by a 2-foot wide concrete pier. In addition to separating the bays, the piers will support the fish screens, stop logs, adjustable baffles, mechanical equipment and deck slab. The total number of bays required, along with the total width of each facility and the screen top elevations are given in Table 7.3.

Table 7.3 – Number of Bays, Total Width and Screen Top Elevations

Facility	Number of Bays	Total Width of Intake Facility (ft)	Top-of-Screen Elevation (ft)
Webb Tract, SJR	40	878	2.49
Webb Tract, False River	33	724	2.39
Bacon Island, Middle River	40	878	2.49
Bacon Island, SF Cut	51	1120	2.59

7.2.2.4 Screen Type and Layout

The fish protection system for all intake sites will be passive screen. The screen will be vertical profile bar made of type-304 stainless steel and the clear opening between vertical bars will not exceed 0.0689 inches (1.75 mm). The vertical bars will be configured such that the flatter side of the bar faces upstream, which will reduce the chances of the panel becoming clogged.

Each bay will have two screen panels installed side-by-side across the bay separated by a guide-rail channel section placed at the center, so each screen will be 10 feet wide.

Screen guides will be provided for removing and reinstalling the panels for inspections and maintenance. The vertical profile bars will be supported at intermediate points by the channel sections to minimize deflection. The top and bottom of the screen will be strengthened by a screen panel structure frame. Lifting eye cutouts will be provided in the screen frame to facilitate removal of the screen panel. The screens will be kept in place by gravity and they will move along the screen guides.

7.2.2.5 Screen Top

The top of the screen will extend to a minimum elevation of the 50 percent probability of exceedance for the low-high tide level. This is to ensure that the top of the screen will extend to a level that the lifting eyes become visible for a portion of time during most days. This screen height will allow regular maintenance and repair activities to be performed during the low tide periods. The actual top of screen elevation was rounded to an increment suitable to accommodate manufacturing requirements. The top-of-screen elevations are given in Table 7.3.

7.2.2.6 Steel Face Wall

From the top of the fish screen to the top of the intake structure deck (engineered embankment elevation), a steel face wall will be constructed of 1¼-inch thick steel plates. The steel face wall will prevent excess flows from passing through the facility above the fish screen when the river is at higher stages and will also protect the deck slab from wave run-ups. The steel face wall can also be used as a stop log (by lowering it to the sill), preventing water from entering the fish screen facility when maintenance and inspections are required. Stop log slots will also be provided at the downstream end of the concrete piers. The interface between the face wall and fish screen will be sealed such that the resulting openings are smaller than the allowable fish screen opening.

7.2.2.7 Log Boom

A floating log boom will be provided to prevent floating debris from clogging the fish screen. The floating log boom will be supported by a series of dolphin piles driven in the channel bed and will be equipped with a suspended debris fence. The log boom fence will be inspected and cleaned regularly by divers.

7.2.2.8 Cleaning Device and Frequency

At each site, three to four single stroke vertical scraping type cleaning units on single rail systems will be installed. The number of units is based on the overall length of the fish screen. The cleaning device will be manufactured by Kuenz America (Type TRCM E 35), Atlas Polar Cleaning Systems (Type ST8100 Hydro-brush) or from some other established manufacturer. A manufacturer will be selected based upon the performance reliability of the installed devices by that manufacturer.

To avoid flow impedance and violations of approach velocity criteria, a cleaning frequency of five minutes is desired. Contingent upon DFG/NMFS/USFWS approval, the time lags between the cleaning periods could be higher than the recommended time of 5 minutes.

7.2.2.9 Debris Removal System

Once the debris has been pulled out from the screen it must be properly collected and transported to the disposal site. Thus, each cleaning brush will be accompanied with a collecting dumpster. These dumpsters, when filled, will be transported to the disposal site.

The fish screen, cleaning device and debris removal systems are an integrated system. It is preferred, then, that the entire system be fabricated by one manufacturer. If, however, different manufacturers supply various components of the screening facility, close coordination shall be ensured. The selection of the manufacturer will be decided based upon the reliability of performance of the installed devices by the manufacturer.

7.2.2.10 Intake Structure and Deck

A reinforced concrete box culvert section was determined to be most appropriate for the fish screen structure. Structural design details are presented in Section 7.6 and in the Integrated Facilities Engineering Design and Analysis Report, Appendix B. The top slab of the box culvert structure will act as a bridge deck along the top of the intake facility and will be wide enough to accommodate the cleaning rails, walkway, debris-removal system and fish screen handling cranes. Lighting and safety railings will be provided along the deck to allow operation of the facility 24 hours a day. The deck and the levee top will have the same elevation, as given in Table 7.2.

7.2.2.11 Sediment Handling and Removal

To reduce sedimentation, the bottom of the intake channel will be sloped towards the river. The sediments that are carried through and deposited inside the fish screen may be flushed out from behind the screens by using high pressure water jets through pipes installed within the base slab and concrete sill or by another suitable method (to be determined during final design). The flushed-out (or re-suspended) sediments will then be carried downstream of the fish screen structure and deposited in the Transition Pool. The Transition Pool will be dredged as needed to remove accumulated sediments.

7.2.2.12 Stop Log Guides and Adjustable Baffles

Stop log slots will be provided at the downstream end of the concrete piers. With these stop logs in place and the steel face wall lowered, maintenance and inspections can be performed for each bay individually.

A second set of slots will also be provided at the downstream end of the concrete piers for the installation of adjustable baffles. These adjustable baffles will help provide uniform flow through the entire width of the fish screen facility.

7.2.2.13 Scour and Erosion Protection

The channel bed upstream and downstream of the fish screen structure will be provided with a riprap blanket for protection against scouring and erosion. On the downstream side (behind the screen) where the flow velocity is expected to be low, the riprap blanket will be extended laterally to a distance of

10 feet beyond the edge of the fish screen piers. On the upstream side, the riprap blanket will be extend 10 to 20 feet from the beginning of the fish screen slab toward the river channel.

7.2.3 Summary

The proposed fish screens will be vertical profile bar type and will be continuously cleaned to prevent excessive debris buildup. The design meets applicable design criteria set forth by the California Department of Fish and Game (DFG) and the National Marine and Fisheries Service (NMFS). The total width of the fish screen facilities varies from 724 feet to 1,120 feet, depending on facility location. A summary of the fish screen specifications for all sites is presented in Table 7.4.

Table 7.4 – Summary of Fish Screen Specifications

Item	Required Specification
Required Gross Wetted Area	7,500 sq.ft. during low stage 11,250 sq.ft during high stage
Screen Open Area	50 percent of Gross Wetted Area
Screen Type	Vertical Profile Bar
Screen Opening	0.0689 inches (1.75 mm)
Length of Individual Screens	15, 18, 15, and 12 ft *
Screen Size	Screen Length x 10 ft
Permissible Bending	1/8 inch
Screen Material	Type 304 Stainless Steel
Side Support	Steel Channel Guide
Vertical Support and Removal	Gravity and through lifting eyes
Cleaning Type	Single Stroke Vertical Scrapper
Number of Cleaning Units	3, 3, 3, 4 *
Debris Removal	Continuous Type

* Screen lengths and number of cleaning units are for Webb Tract @ SJR, Webb Tract @ False River, Bacon Island @ Middle River and Bacon Island @ SF Cut, respectively.

The Central Valley Fish Facilities Review Team (CVFFRT) has evaluated the proposed fish screen facilities and agrees with the overall concept. CVFFRT recommends that DWR organize a technical review committee for fish screen review during the final design phase.

7.3 Gate Structures and Midbay

The objective of the gate structures and midbay area is to control the flow, under varied slough and reservoir stages, into and out of the reservoir with hydraulic efficiency. This section describes the design criteria, general layout, and hydraulic design of the gate structures and the midbay area of the integrated facility.

7.3.1 Design Criteria

Hydraulic design criteria for the gate structures and midbay area are listed below.

1. Velocity in unlined sections should not exceed 3 ft/sec. This will prevent scouring.
2. Energy dissipation structures or stilling basins should be provided to prevent damage to the sections of the integrated facility downstream of point of control.

7.3.2 General Layout

Each integrated facility consists of three gate structures and one midbay area. Gate #1 is used for inflow (diversions) only, Gate #2 is used for both inflow and outflow (releases), and Gate #3 is used for outflow only. All three gates will be vertical lift slide gates that can be regulated mechanically or manually. The total number of gate structures required in the facility was selected to maximize the use of gravity flow. In particular, Gate #3 was added to achieve maximum gravity flow releases under the year-round reservoir operations as modeled in the CALSIM-II daily model.

The midbay area serves as a transition pool for all three gates. Flow into and out of each gate structure is directed using smooth and straight transitions to minimize hydraulic losses and cavitation potential.

Energy dissipaters will be used to dissipate excess energy at the downstream end of each gate structure. Gate #1 has only one energy dissipater on the downstream side of the gate sill. Gate #3 has only one energy dissipater which is located between the gate sill and the bypass channel. Gate #2 has energy dissipaters on both sides of the gates.

7.3.2.1 Gate Location and Sill Levels

The gate structures are centered along the sides of the midbay, providing for uniform flow as water approaches the gate structures and reducing the head loss.

The sill levels for Gate #1 and Gate #3 were determined based the existing ground elevations and slough bed levels at each integrated facility location. The sill level of Gate #2 was selected to achieve the desired level of reservoir emptying. Table 7.5 summarizes the selected sill elevations for each gate structure.

Table 7.5 – Gate Structure Sill Elevations

Item	Integrated Facility Elevation (ft)			
	Webb Tract, San Joaquin River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe River
Gate #1	-12	-15	-13	-8
Gate #2	-18	-18	-16	-16
Gate #3	-15	-16	-12	-8

7.3.2.1 Midbay Floor Level

The midbay floor is designed to meet submergence requirements of the pumps and is deep enough to both empty the reservoir to the desired level and to allow flow through the gate structures to form a hydraulic jump within the midbay. Calculations for the hydraulic tail water depth requirements are given in the Integrated Facilities Engineering Design and Analysis Report, Appendix A.

The bottom and sides of the midbay will be covered with riprap. Table 7.6 summarizes the midbay floor elevations and minimum required midbay water levels during diversions.

Table 7.6 – Midbay Floor Elevations and Minimum Required Water Levels During Diversion

	Integrated Facility			
	Webb Tract, SJR River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe Cut
Midbay Floor Elevation (ft)	-24	-24	-22	-22
Minimum Recommended Water Level in Midbay (ft)	-14	-17	-15	-10

7.3.3 Hydraulic Design

7.3.3.1 Gate Selection

The number of gate panels and the maximum gate openings were fixed to maximize gravity flow through the gates and are shown in Table 7.7. Gate design procedures are summarized in the Integrated Facilities Engineering Design and Analysis Report, Appendix A. All of the gates will be vertical-type mechanically driven painted steel roller gates with hydraulic cylinder actuators, however, in case of power failure, they will be equipped for manual operation.

Table 7.7 – Number of Gate Panels, Gate Width, and Gate Height

Gate No.	Number of Gate Panels	Gate Panel Width (ft)	Maximum Gate Opening (ft)
Gate #1	3	12	10
Gate #2	3	12	10
Gate #3	2	12	8

7.3.3.1 Energy Dissipation

Gate #1 Energy Dissipaters

A very high head differential is possible at Gate #1 when slough levels are relatively high compared to reservoir levels. In this case, a large amount of energy must be dissipated in the midbay just downstream of Gate #1. This energy can be dissipated by a submerged jump downstream of Gate #1 provided a minimum depth of water is available in the midbay.

Water surface profiles were generated first by assuming an empty midbay level and maximum flow on the slough side and then by computing the minimum tail water depth required to dissipate the energy through a submerged hydraulic jump. The Froude Number for all integrated facilities was calculated to be in the range of 4.5 to 4.6, which suggests a steady hydraulic jump. An S2 profile was developed and combined with the minimum tail water depth to generate the final water surface profiles as shown in the Integrated Facilities Engineering Design and Analysis Report, Appendix A. The recommended minimum tail water levels are summarized in Table 7.7.

Gate #2 Energy Dissipaters

As previously mentioned, Gate #2 is designed as a two-way hydraulic structure connecting the midbay to the reservoir, so it has energy dissipaters on both sides of the gates.

The floor on the reservoir side is used to dissipate energy during the diversion of water into the reservoir. The reservoir may be empty during some diversion periods, indicating that there is no minimum tail water depth available on the downstream side of the gate to dissipate excess energy through a hydraulic jump. Because of this, the floor on the reservoir side of Gate #2 is designed to dissipate energy even without adequate tail water depth. Starting from Gate #2, the floor will expand at a 45-degree transition angle. The horizontal concrete slab extending from the gate sill to the reservoir floor is about 52 feet long and will have an end sill followed by 10 to 20 feet of riprap protection. As the flow passes through the transition, its velocity will be reduced to the permissible limit of 3 ft/sec.

During the release of water from the reservoir to the midbay, the energy dissipation downstream of Gate #2 will be achieved with a minimum tail water depth as described for Gate #1.

Gate #3 Energy Dissipaters

The outlet extending from the sill of Gate #3 into the bypass channel will consist of a flared concrete transition. This outlet transition, along with sufficient tail water depth provided by normal slough levels, will dissipate the energy and when the water reaches the bypass channel the velocity will be within the permissible limit of 3 ft/sec. The design of this outlet is similar to that of Gate #2.

7.3.3.1 Flow Rating Curves

Flow rating curves were developed for both diversion and release operations at all integrated facility locations for Gate #1 (inflow only), Gate #2 (inflow only), and Gate #3 (outflow only). Each rating curve shows the percentage of time the design flow can be met by gravity flow only, pumped flow only, or a combination of gravity and pumped flow. Each rating curve also shows the corresponding total head required between the reservoir and the slough. Information from the DSM2 and CALSIM-II computer models and gate geometry was combined to develop the curves. DSM2 Hydro provided hourly slough stage data at each integrated facility location, whereas CALSIM-II provided reservoir stage, and inflow and outflow data on a daily basis.

7.3.4 Miscellaneous Design Features

- A concrete face wall will be provided from the top of the gate opening to the deck slab level.
- Hydraulic cylinder actuators along with hydraulic power units were chosen to operate the mechanically driven painted steel roller gates.
- To stop the flow of debris and floating particles, trash racks will be provided at the beginning of Gate #2.
- In addition to normal operating (service) gates, stop log slots will be provided at each gate to allow maintenance and inspections of the gate, gate slides, or gate sill areas.
- The inlet/outlet gates will be protected against suspended silt load. The transition pool upstream of Gate #1, having very low velocities, will act as a settling basin to trap sediments before they enter the reservoir during diversion operations.
- A dredged and graded trapezoidal outlet channel will extend from Gate #2 to the lower elevations of the reservoir to allow for maximum drainage of the reservoir.

7.4 Pumping Plant and Conduit Pipes

The pumping plant serves to:

- supplement diversion and release gravity flows when sufficient head is not available to meet the desired flow rate, and
- meet the entire flow rate when no head (or negative head) is available.

The pumping plant consists of five vertical-type pumping units (three 400 cfs and two 150 cfs units), totaling a maximum pumping capacity of 1500 cfs. The smaller pumps, having lower submergence requirements than the larger pumps, can be used to pump water out of the reservoir at lower elevations, allowing flexibility in operations when needed.

The conduit pipes will be used to discharge water into the reservoir and bypass channel during diversion and release operations, respectively. For both operational conditions, the flow direction is controlled by two butterfly valves installed in each conduit pipe. For diversions to be made through the pumping plant, the valves closest to the bypass channel will be closed and the valves closest to the reservoir will be open. The opposite is true for releases. The conduit pipes can also be used for gravity flow releases to supplement the gravity flow releases through Gate #3. This can be achieved by opening both butterfly valves in each conduit pipe.

The pumping plant layout, design and layout of the conduit pipes, and selection of mechanical and electrical equipment is discussed in more detail throughout this section.

7.4.1 Design Criteria

7.4.1.1 Pumping Plant Design Criteria

The following design criteria will be applied in the hydraulic design of the pumping plant:

1) The pumping plant shall supply water under the following cases:
   a) Diversions
      i) Pumping only: for diversions into the reservoir when the reservoir level is the same as or higher than the river level.
      ii) Combination (pumping and gravity): for diversions into the reservoir when the reservoir level is lower than the river level, but gravity flow is not enough to achieve the desired level of diversions.
   b) Releases
      i) Pumping only: for releases from the reservoir when the reservoir level is the same as or lower than the river level.
      ii) Combination (pumping and gravity): for releases from the reservoir when the reservoir level is higher than the river level, but gravity flow is not enough to achieve the desired level of releases.
2) Pumping unit submergence requirements shall be met at all times during pumping operations. This will help to prevent cavitation of the pumps.
3) The inlet from the midbay to each pumping unit shall provide a smooth transition to minimize head loss. A formed suction intake shall be used to ensure a smooth transition.
4) The intake basin shall be configured to avoid vortex formation in the midbay and to minimize flow separation.
5) The pumping station shall be designed to allow for maximum drainage of the reservoir.

6) The midbay shall be completely drained for maintenance operations. This will require the design and installation of a smaller sump pump and discharge conduit.
7) The forebay and afterbay water surface elevations for the proposed pumping plant are given in Table 7.8.

Table 7.8 – Pumping Plant Forebay and Afterbay Water Surface Elevations

Forebay and After Bay Water Surface Elevation	Integrated Facility Location							
	Webb Tract San Joaquin River		Webb Tract False River		Bacon Island Middle River		Bacon Island Santa Fe Cut	
	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River
Maximum	6.8	4	6.4	4	6.8	4	6.8	4
Normal	-1		-1		-1.1		-1.1	
Minimum	-1.7	-18	-1.5	-18	-1.7	-16	-1.7	-16

7.4.1.2 Conduit Pipe Design Criteria

The following design criteria will be applied in the design of the conduit:

1) The conduit pipes shall be designed to flow under pressure and shall be sufficient to pass a total flow rate of 1500 cfs.
2) The conduit pipes shall have a minimum slope of 1-foot per 1000-feet to allow for drainage. Where conduit pipe sections have no slope, a drainage system shall be installed.
3) A cathodic protection system shall be considered in combination with protective coatings to ensure adequate protection and longevity of conduit, pumping units, gates, valves and other appurtenances.
4) A trash rack shall be provided at the reservoir side of the intake/discharge conduit.
5) All concrete conduit pipes should be manufactured in accordance with ASTM C76M specifications.

7.4.2 Pumping Plant and Conduit Layout

The pumping plant consists of three 1500 hp pumps with a capacity of 400 cubic feet per second (cfs) each and two 800 hp pumps with a capacity of 150 cfs each, totaling five pumping units with a maximum pumping capacity of 1500 cfs. This combination of pump units was selected based on the proposed year-round operation of the facilities.

Stop log slots will be provided in front of each pumping plant intake. This will allow individual pumping units to be shut down and serviced while the rest of the units continue operating. A gantry crane will also be provided to facilitate required maintenance and inspections of the pumps, valves, motors and gears.

7.4.2.1 Plant Superstructure

The pumping plant superstructure will consist of three levels, an upper level, middle level and lower level. The upper level will support the right angle gears, pump motors, and gantry crane, and will have covered openings that provide access to the pump units and the discharge valves. The upper level will not be housed (enclosed) by a supported structure, in other words, it will be exposed to the elements. The switch yard will be located on the top of the embankment adjacent to the upper level. The middle level will support the pump units, pump discharge pipe, butterfly valves (and associated floor mounted hydraulic actuators) and a hydraulic power unit (HPU). The middle level will also house the motor control room and office. The lower level will house and provide access to the formed suction intakes. The pumping plant will also have a service elevator at both ends of the superstructure, providing access to all three levels.

7.4.2.2 Piping Layout

The conduit pipes consist of two eight-foot diameter pipes and one six-foot diameter pipe. Pump Unit No. 1 will split its discharge between the two 8-foot conduit pipes, with half of its flow going to each pipe. Pump Unit No. 2 will discharge into one of the 8-foot conduit pipes and Pump Unit No. 3 will discharge into the other. Both 150 cfs pumps (Pump Units No. 4 and No. 5) will discharge into the 6-foot conduit pipe.

There are two butterfly valves installed in each conduit pipe and the direction of flow through the each pipe is controlled by the joint operation of the two butterfly valves. The butterfly valves in each conduit pipe are aligned with one another and are housed in a valve vault. Each valve vault contains a hydraulic power unit to operate the floor-mounted hydraulic actuators, which open and close the butterfly valves.

7.4.2.3 Conduit Pipe Design

The conduit pipes were designed to carry a combined design discharge of 1500 cfs under a maximum permissible velocity of 12 ft/sec. A variety of pipe sizes, configurations, and materials were considered to optimize the pipe sizes for various hydrologic and operating conditions. The chosen configuration consists of one 6-foot and two 8-foot diameter pipes. Considering their size, strength, economy and other factors, Double-Gasket Spigot type precast reinforced concrete pipes are being recommended for all integrated facility locations.

Given the variation of available head between the reservoir and the river, gravity flow capacity through the conduit pipes was determined by the energy balance approach. The capacity calculations include pipe friction losses and minor head losses. Hydraulic design procedures are given in the Integrated Facilities Engineering Design and Analysis Report, Appendix A.

7.4.2.4 Trash Racks and Stop Logs

To stop the flow of debris and floating particles, trash racks will be provided at both ends of the conduit pipes. The trash racks will be made of anti-fouling steel and the clear spacing between the bars should not be more than 2 inches. This spacing requirement will also prevent adult/predator-sized fish from exiting the reservoir. For easy placement and removal, the trash racks will be placed in slots. The racks will remain in place by gravity.

Stop log slots will be also be provided at each end of the conduit pipes, so inspection and maintenance of the conduits can be carried out.

7.4.2.5 Energy Dissipaters

The exit velocities at both ends of the conduit pipes are high (up to 12 ft/sec) and due to the nature of the peat soils underwater erosion may occur downstream of the outlet structures. Baffled apron drop structures will be used to dissipate excess energy at both ends of the conduit pipes under both submerged and un-submerged outlet conditions. Baffled apron drops were selected for two reasons: (1) they do not require a downstream water surface for satisfactory performance and (2) they can function under a wide variation of downstream water surface elevations. All three conduit pipes will discharge into a common energy dissipation structure. The design of the baffled apron is based on USBR specifications and is summarized in the Integrated Facilities Engineering Design and Analysis Report, Appendix A.

7.4.3 Mechanical Engineering Design

7.4.3.1 General

In order to reduce plant construction costs, no oil room or maintenance bay will be provided. It is assumed that any oil purifying or major maintenance will be done at another plant. This is similar to how the South Bay Pumping Plant operates.

7.4.3.2 Pump Selection

A hydraulic analysis was performed to calculate the total dynamic head (or maximum pumping head) that the pumps must be able to operate against. The total dynamic head includes static head, pipe friction head losses, and minor head losses from valves and fittings. A summary of total dynamic head for each pumping plant is given in Table 7.9.

Table 7.9 – Total Dynamic Head for Each Integrated Facility Pumping Plant

		Total Dynamic Head			
Case	Flow	Webb Tract, SJR River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe Cut
Diversions	150 cfs	16.2	16	16.2	16.2
	400 cfs	9.9	9.7	9.9	9.9
Releases	150 cfs	35.3	34.9	35.3	35.3
	400 cfs	23	22.5	23	23

All pumping units will be vertical-type mixed flow pumps driven by a floor-mounted fixed speed motor connected to a right angle gear (to minimize the vertical height of the plant). A formed suction intake (FSI) will be mounted to each pump below the impeller to eliminate vortex formation in front of the pump.

When the reservoir level is low and submergence requirements for the 400 cfs pumps are not met, pumping will be limited to the 150 cfs pumps. At minimum head, both 150 cfs pumps may have to be operated in order to maintain sufficient friction head in the common 6-foot conduit pipe. Otherwise, the head would be too low and the pumps would experience vibration or cavitation problems. Under low head conditions, it may be necessary to throttle with the butterfly valves to prevent vibration or cavitation problems.

7.4.3.3 Valve Selection

AWWA Class 75B butterfly valves were chosen to control the flow through both the pump discharge pipes and the conduit pipes. Hydraulic actuators were chosen to operate the butterfly valves. A hydraulic power unit will be provided in the pumping plant to control all of the valves in the plant. Similarly, a hydraulic power unit will be provided in each vault. The hydraulic power units in the pumping plant and in the valve vaults will serve as backup to each other by running hydraulic and control lines between the vaults and the plant.

7.4.3.4 Gantry Crane

A gantry crane is required to lift the pumps, motors, and right-angle gears for maintenance. Since removal of the valves and actuators is anticipated to be required much less frequently, it was assumed that these would be moved using a mobile crane.

7.4.3.5 Heating, Ventilation, and Air Conditioning

It is assumed that heating and air conditioning will be limited to the office, control room, and motor control room. Only ventilation will be provided to the rest of the plant. A summary of the HVAC equipment to be used in the pumping plant is provided in Table 7.10.

Table 7.10 – Pumping Plant HVAC Equipment Summary

Location	Equipment	Quantity
Motor Control Room	10 Ton Cooling Only Air Conditioning Unit	2
	3.3 Kw Electric Unit Heater	1
Office	Split System Heat Pump, 1.2 Tons Cooling and 9 MBh heating	1
Control Room	Split System Heat Pump, 2.0 Tons Cooling and 1.8 MBh heating	1
Mid & Lower Levels	Ventilation Fans (5400 CFM each)	2

7.4.3.6 Miscellaneous

An 850 gpm 15-hp sump pump will be provided to empty the midbay area for maintenance purposes, such as dredging and cleaning the midbay, performing maintenance at the gate structures, and performing maintenance on the formed suction intakes.

Combination air valves will be provided just downstream of the pump discharge valve. Each 150 cfs pump discharge pipe (4-foot-6 inch pipe) will contain a 12-inch air valve and each 400 cfs pump discharge pipe (7-foot pipe) will contain an 18-inch air valve.

7.4.4 Electrical Engineering Design

Feasibility level electrical engineering design for the electrical components of the integrated facility was completed and the major equipment recommendations are discussed in this section.

7.4.4.1 Transformer Sizing

A transformer sizing simulation was completed using the EDSA Micro Corporation Advanced Power Flow Program. The simulation considered only motor loads. The transformer loading consisted of the three 1500 hp motors and the two 800 hp motors at the 4160 voltage level and the results of the transformer sizing simulation indicate the need for a 7.5 MVA transformer. The 7.5 MVA transformer is at 83% of its capacity, which leaves adequate power for other low voltage loads.

7.4.4.2 Utility Source

A PG&E area assessment was not performed in this study. The nearest utility source that can handle the In-Delta Storage project's anticipated load of 7.5MVA is estimated to be, at most, six miles from the project islands.

7.4.4.3 Equipment Layout

The major electrical equipment includes a control room and a switchyard containing a transformer, circuit breaker and a disconnect switch. The control room is located on the middle level of the pumping plant and has a minimum ceiling height requirement of twelve feet to accommodate the switchgear, ductwork, overhead raceway, and all other associated electrical equipment that will be installed. The switchyard is located on the embankment in front of the pumping plant valve vaults. A summary of the major electrical equipment is provided in Table 7.11.

Table 7.11 – Major Electrical Equipment

5kV Metal Clad Switchgear
Vacuum or SF6 Circuit Breakers
7.5/8.85 MVA, 230kV-4.16kV, OA/FA rated service transformer
Programmable Logic Controllers
Microprocessor based multifunction relay protection for the motors, switchyard equipment, and feeders
Modbus Plus communication protocol
Low Voltage Motor Control Center
Low Voltage Distribution Center

7.4.4.4 Recommendations

Fixed speed motors were chosen to drive the pumps. This type of motor uses across-the-line starting, which causes a large in-rush current, typically five to six times the full load amperage of the motor. This will cause stress on the motors and the feeders, which could be eliminated by the use of variable frequency drives. Therefore, it is recommended that variable frequency drives be considered. Variable frequency drives provide many advantages including energy savings, reduced equipment wear and stress, and increased efficiency. Modern clean power variable frequency drives reduce harmonics, are reliable and provide excellent field performance. Variable frequency drives inherently provide motor and feeder protection. Also, since the pumping head may vary greatly, more precise motor speed control may be required to operate the pumps in their optimum range. Lastly, options such as remote operation and monitoring over a network using a protocol such as Modbus plus are easily configurable with modern variable frequency drives.

An area assessment should be performed by PG&E to develop accurate distances to the nearest utility source that can handle the In-Delta Storage project's anticipated load of 7.5MVA.

7.5 Bypass Channel

The bypass channel is used to convey reservoir releases into the river and is shown in Figure 1.4. Reservoir releases enter the bypass channel at its upstream end through the conduit pipes and/or through Gate #3. The bypass channel is isolated from the fish screen facility and transition pool by a structural sheet pile wall.

7.5.1 Design Criteria

The following criteria were used in the design of the bypass channels.

- The bypass channel should be designed to accommodate a maximum flow rate of 2250 cfs. Because the project site is located in the areas of tidal influences, the bypass channel should be able to pass the maximum flow during the lowest tide levels.
- To prevent bank erosion, channel degradation, and scouring along the sheet pile wall, flow velocities within the channel should not exceed 3 feet per second (ft/sec). If the channel velocities exceed 3 ft/sec, adequate bed and slope protection should be provided.
- Adequate freeboard should be provided within the channel to provide maximum protection during times when the maximum release flow of 2250 cfs coincides with the highest tide levels (300-year flood level).
- Trash and other floating debris should not be allowed to enter or reside in the bypass channel.

7.5.2 Channel Design

The channel design included selecting channel bed elevations based on island topography and selecting the most efficient channel geometry given the overall layout of the integrated facility. The design procedures are discussed in more detail in this section and the results are presented in Table 7.12.

7.5.2.1 Bed Level

The bed level of the bypass channel on the upstream side equals the invert elevation of the outlet structures (Gate #3 and the pipe conduit). The invert elevations of the outlet structures were set according to the topography at each site. This was done to minimize the height of the structures. The adopted bypass channel bed levels for each integrated facility are given in Table 7.12.

7.5.2.2 Channel Geometry

At the upstream end, the channel section is trapezoidal with a side slope of 3H:1V. On the downstream end, one side of the bypass channel will continue as a sloped section while the other side will consist of the vertical sheet pile wall.

Sheet Pile Wall

The bypass channel is isolated from the fish screen and transition pool area by a vertical sheet pile wall. The top of the sheet pile wall will extend to the top of the embankment. The sheet pile wall will be designed so water will not flow freely between the bypass channel and the transition pool; however, the wall will not be completely sealed.

Channel Bottom Width

The channel was designed to accommodate a maximum flow of 2250 cfs while keeping the flow velocity within the permissible limit. The size of the riprap and Manning's roughness coefficient are interdependent. Manning's roughness coefficients of 0.02 and 0.025 were used for the channel bed and channel sides, respectively. The required bottom width of the channel was determined for a design discharge of 2250 cfs with both minimum and maximum slough levels. Since the bottom elevation of the channel was determined based upon the existing topography, the controlling situation occurred when the slough levels are lowest and the bypass channel is discharging the maximum flow. The U.S Army Corps of Engineers HEC-RAS program was used to determine the required channel geometry.

7.5.2.3 Slope Protection

Both sides, as well as the bottom of the bypass channel will be lined with rock riprap to prevent bank erosion.

7.5.2.4 Access Bridge and Trash Rack

The vehicle access bridge will allow access to the fish screen from both ends as well as allow traffic to move from one side of the facility to the other. The bridge has a deck width of 15 feet and spans across the bypass channel. The bridge is designed as a simple box culvert and has vertical abutments and intermediate piers to hold the trash racks in position. The trash racks will have clear openings of not more than 2 inches to prevent the attraction and egress of adult-sized fish into and out of the bypass channel. The trash racks will prevent the flow of debris and adult-sized fish from entering into the intake facility.

Table 7.12 – Summary of Bypass Channel Design

<table>
<tr><th colspan="3" rowspan="2">Bypass Channel Component</th><th colspan="4">Integrated Facility Location</th></tr>
<tr><th>Webb Tract, San Joaquin River</th><th>Webb Tract, False River</th><th>Bacon Island, Middle River</th><th>Bacon Island, Santa Fe Cut</th></tr>
<tr><td colspan="3">Upstream Bed Level (ft)</td><td>-15</td><td>-16</td><td>-12</td><td>-8</td></tr>
<tr><td colspan="3">Downstream Bed Level (ft)</td><td>-16</td><td>-17</td><td>-13</td><td>-9</td></tr>
<tr><td colspan="3">Sheet Pile Wall Top Elevation (ft)</td><td>11</td><td>11</td><td>10.2</td><td>10.4</td></tr>
<tr><td colspan="3">Bottom Width (ft)</td><td>30</td><td>30</td><td>40</td><td>70</td></tr>
<tr><td rowspan="4">Side Slopes</td><td rowspan="2">Upstream End</td><td>Left Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
<tr><td>Right Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
<tr><td rowspan="2">Down-stream End</td><td>Left Bank</td><td>Vertical</td><td>Vertical</td><td>Vertical</td><td>Vertical</td></tr>
<tr><td>Right Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
</table>

7.6 Structural Design

Design criteria, design basis and assumptions, and design procedures were established and State feasibility level structural analysis and design was prepared in sufficient detail to allow a feasibility-level cost estimate of the four proposed integrated facilities to be completed. In particular, structural analysis and design was completed for the structural components of the fish screen structure, the three gate structures, structures associated with the pumping stations and conduits, and for the sheet pile walls. Using the geological laboratory information provided by DWR, precast prestressed concrete piles were designed for each structure such that settlement, cracking and tilting do not cause structural distress. Feasibility level design drawings related to the structural components and foundations of the integrated facilities were also developed.

7.6.1 Design Criteria

Details on design codes, design loads, design methods and considerations for reinforced concrete design and deep foundation design, and material strengths and coatings are provided in the Integrated Facilities Engineering Design and Analysis Report, Appendix B. Table 7.13 presents a summary of all integrated facility design elevations that were used in the structural analyses.

7.6.2 Geotechnical Design Analyses

In the geotechnical analyses performed for the Integrated Facilities, lateral earth pressures were calculated for design of the structures, structure foundation alternatives were evaluated, axial and lateral capacities for pile foundations were developed, and design analyses were performed for the sheet pile wall.

7.6.2.1 Summary of Soil Conditions

The subsurface conditions at the four integrated facility sites are similar, and consist of soft clays and peat soils overlying denser and stiffer interbedded sands and clays. Cone penetration tests (CPTs) recently performed by the Bureau of Reclamation were used to characterize the stratigraphy and strength profile with depth at the I/O structure sites. The soil conditions at the four integrated facility sites consist of soft soils overlying stiffer and denser clays and sands and the depth of soft soils are summarized below:

- Webb Tract, San Joaquin River: approximately 40 feet
- Webb Tract, False River: approximately 20 feet to 25 feet
- Bacon Island, Middle River: approximately 20 feet
- Bacon Island, Santa Fe Cut: approximately 25 feet to 30 feet

Table 7.13 – Integrated Facility Elevations

Structural Component	Item	Description	Location: Webb Tract San Joaquin River	Webb Tract False River	Bacon Island Middle River	Bacon Island Santa Fe Cut
Fish Screen	Screen Dimensions	Screen Length (vertical direction)	15	18	15	12
		Screen Width (horizontal direction)	7.5	7.5	7.5	7.5
	Elevations	Top of Screen	2.49	2.39	2.49	2.59
		Bottom of Screen (Sill) @ Screen Face	-12	-15	-12	-9
		Top of Bottom Slab @ Downstream End	-12.3	-15.3	-12.3	-9.3
		Deck (Top of Embankment)	11	11	10.2	10.4
	Overall	Total Facility Width	933	768	933	1108
		Number of Bays	54	44	54	64
		Clear Span Between Piers	15	15	15	15
Gate Structures	Gate #1	Sill Elevation (Top of Bottom Slab)	-12	-15	-13	-8
		Deck Elevation (Top of Embankment)	11	11	10.2	10.4
	Gate #2	Sill Elevation (Top of Bottom Slab)	-18	-18	-16	-16
		Deck Elevation (Top of Embankment)	11	11	10.2	10.4
	Gate #3	Sill Elevation (Top of Bottom Slab)	-15	-16	-12	-8
		Deck Elevation (Top of Embankment)	11	11	10.2	10.4
Midbay		Floor Elevation	-24	-24	-22	-22
Conduit	Invert Elevations	Reservoir Side	-12	-12	-10	-10
		Bypass Channel Side	-12	-12	-10	-10
Reservoir	Finished Grade Elevations	@ Gate #2 Outlet	-18	-18	-16	-16
		@ Conduit Outlet	-18	-18	-16	-16
Bypass Channel	Finished Grade Elevations	@ Conduit Outlet	-15	-16	-12	-8
		@ Gate #3 Outlet	-15	-16	-12	-8
		@ Connection to River Channel	-16	-17	-13	-9
		Bottom Width	30	30	40	70
	Sheet Pile Wall	Top Elevation	11	11	10.2	10.4

7.6.2.1 Lateral Earth Pressures

It is anticipated that the integrated facility structures will be founded in new fill material placed for the embankment construction. Computation of earth pressures in new fill are based on the soil properties presented in the Embankment Design Analysis Report (URS, 2002). The earth pressures in Table 7.14 are expressed as equivalent fluid weights and the seismic loads are based on the design peak horizontal ground acceleration.

Table 7.14 – Lateral Earth Pressures and Seismic Loads for New Fill Materials

Case	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	Active Case (pcf)	Passive Case (pcf)	At-Rest Case (pcf)	Seismic Loads [2] (lbs/ft)
Unsaturated	110	30	0	37	330	55	11 H^2
Saturated [1]	120	30	0	82	173	91	12 H^2

Notes:

1) Active and at-rest equivalent fluid pressures for saturated case include hydrostatic pressure of 62.4 pcf
2) For seismic loads, H is the height of the wall in ft, expressed in lbs/ft of wall, and acts at a height of 0.6 H above the base of the wall.

7.6.2.2 Axial Pile Capacity

Due to the magnitude of the loads imposed by the structures, and the very soft near-surface soils, the structures will need to be pile-supported. Precast prestressed concrete piles are recommended as they are frequently used in marine applications, have good load-carrying capacity, can be installed efficiently, and are relatively economical. For preliminary design purposes, a 14-inch square precast prestressed pile was selected, which has an allowable capacity of 45 tons.

The cone penetration test results at each of the four integrated facility sites were interpreted using the LCPC method of Bustamante and Gianeselli (1982) to obtain pile capacity versus depth diagrams. Contributions from both skin friction and end-bearing were included in the capacity calculations. Given a factor of safety of 3.0 and a working load of 45 tons, the capacity versus depth diagrams were evaluated to determine the depth at which an ultimate capacity of 135 tons could be achieved. The ultimate capacity was attained at the following pile tip elevations:

- Webb Tract, San Joaquin River Integrated Facility (northern facility): -65 feet
- Webb Tract, False River Integrated Facility (southern facility): -50 feet
- Bacon Island, Middle River Integrated Facility (northern facility): -70 feet
- Bacon Island, Santa Fe Cut Integrated Facility (southern facility): -65 feet

Uplift capacity is calculated as 70 percent of downward capacity to account for deduction of end-bearing capacity. The analyses take into account downdrag forces acting on the piles to account for consolidation of the new fill.

7.6.2.3 Lateral Pile Capacity

The lateral capacity of the 14-inch square precast piles was computed using the program LPILE (Ensoft, Inc., 2000). A soil profile representing the average of the four integrated facility sites and the average pile tip elevation of –65 was modeled. Three pile head elevations at –11, –16, and –21 feet were considered to represent the range of pile head elevations that will be used for the integrated facility structures. Pile head load-deflection curves were developed for these three cases.

7.6.2.4 Sheet Pile Wall

The cantilever sheet pile wall that forms the bypass channel was also analyzed. Average soil conditions consisting of soft clay/peat to elevation –30 feet, underlain by stiff clays and dense sands were modeled. The top of the sheet pile wall was modeled at elevation +11, water in the bypass channel at elevation +7, and the scenario of the pool dewatered to the sill elevation at the respective structures (ranging from – 8 to –15 feet). The lateral pressures due to the 15 to 22 feet of head differential induce bending moments that were estimated to be in the range of approximately 291 to 520 kip-feet/foot. In the absence of tieback anchors, these high bending moments will be resisted by a combination H-pile/sheet-pile wall.

In accordance with standard sheet pile design practice, the sheet pile tip elevations calculated for equilibrium have been increased by 30 percent. The computation of section modulus is based on specifying Grade 55 steel, and applying a factor of safety of 1.5. Table 7.15 presents the sheet pile wall maximum bending moments, tip elevations, required section modulus, and recommended HZ section.

Table 7.15 – Summary of Sheet Pile Analysis Results

Structure	Recommended Sheet Pile Wall Tip Elevation (ft)	Maximum Bending Moment (k-ft/ft)	Section Modulus (in^3/ft)	Recommended Section (HZ Wall System)
Webb Tract San Joaquin River	-59	429	140.4	HZ 975A – 14/AZ13
Webb Tract False River	-62	520	170.2	HZ 975D – 14/AZ13
Bacon Island Middle River	-60	461	150.9	HZ 975B – 14/AZ13
Bacon Island Santa Fe Cut	-54	291	95.2	HZ 775B – 12/AZ18

7.6.3 Structural Design Analyses

This section describes the feasibility-level design of the structural elements of the In-Delta Storage Integrated Facilities. These analyses applied the load combinations, factors of safety and design methodology established for this project to determine the structural requirements for the structural elements of the Integrated Facilities. All results of the structural analysis are shown in Table 7.16. Detailed drawings of all Integrated Facility components are provided in the Integrated Facilities Engineering Design and Analysis Report, Appendix C.

7.6.3.1 Box Culvert Structures

Recognizing the similarities between the fish screen supports and decks, the bypass channel bridge structure and trash rack, and the gate structures, a reinforced concrete box culvert section was determined to be most appropriate.

A 2-D finite element model SAP 2000 (Computers and Structures, Inc., 2003) of the structure was used for the analysis. The structures were designed to carry HS-20 live loads, dead loads from trash racks, screens and gates as well as lateral pressures from soil and water, including seismic loads where appropriate. For the fish screen structure, self-weight and operating loads from the cleaning unit equipment were also accounted for in the analysis. The results of the analysis are shown in Table 7.16.

7.6.3.2 Retaining Walls

Cantilevered reinforced concrete retaining walls were designed for use at the ends of the gate structures, along the approaches to the bypass channel bridge structure, and at the conduit outlet structures. A range of wall heights was analyzed and designed to resist lateral pressures from soil and water, including seismic effects. The results of the analysis are shown in Table 7.16.

7.6.3.3 Pump Station

A feasibility-level design for the pump station was performed. Where required, the SAP 2000 finite element model of the structure was used for the analysis. Exterior walls were designed to resist lateral pressures from soil and water, including seismic effects. Significant equipment loads necessitated the use of reinforced concrete beam floor systems. The results of the analysis are shown in Table 7.16.

7.6.3.4 Vault Structures

Feasibility-level designs were prepared for the vault structures that house mechanical equipment near the gates and for the vault structures that house the butterfly valves in the conduit pipes. Approximate member sizes and main reinforcement requirements are provided for various elements. The results of the analysis are shown in Table 7.16.

7.6.3.5 Other Structures and Pile Requirements

Feasibility-level designs for conduit supports, pipe collars, equipment slabs, apron slabs, cut-off walls and thrust blocks were performed and structural requirements for these elements are provided in Table 7.16.

Pile requirements are shown in Table 7.16. Except for the retaining walls, a lateral displacement of 1-inch was assumed at the pile heads. A 1½-inch lateral displacement for the retaining walls was assumed. The pile heads were assumed to be fixed against rotation at the bottom of the structures.

7.6.4 Recommendations

Further studies may indicate the desirability to use larger piles than the 14-inch piles evaluated for this study. Larger piles would decrease the number of piles required and they would have a higher lateral capacity, thus providing for economy. Further design may also consider the use of batter piles to resist lateral loads. The design presented in this study includes cast-in-place concrete elements. Further studies may indicate that pre-cast concrete construction for such elements as the box culvert and the bridge to the fish screen structure may be more economical.

Table 7.16 – Summary of Structural Design Analysis Results

BOX CULVERT STRUCTURES			
Element	Thickness (ft)	Main Reinf. Ratio	No. Piles
Roof slab	1.5	0.007	9piles per 500 sq. ft.
Exterior wall	2.0	0.011	
Interior wall	2.0	0.007	
Foundation slab	3.0	0.003	

RETAINING WALL STRUCTURES						
Wall Height	Thickness (ft)	Base of Wall (1H:15V batter)	Footing		No. Piles	Spacing
		Main Reinf. Ratio	Width (ft)	Thickness (ft)	per Row	btwn Rows
6' to 15'	1.7	0.003	10	2.5	2	5'-0"
16' to 27'	2.0	0.016	30	3	6	4'-0"
28' to 37'	3.0	0.013	41	3	8	4'-0"

PUMPING PLANT				
Location	Element	Dimensions (in)	Main Reinf. Ratio	No. Piles
Upper Level	Beam "A"	30 x 36	0.018	100 piles total
	Beam "B"	18 x 24	0.018	
	Floor Slab	7	0.009	
	Wall Thickness	12	0.009	
Middle Level	Beam "C"	18 x 24	0.018	
	Floor Slab	7	0.009	
	Wall Thickness	18	0.011	
	Columns	36 x 36	0.03	
	Invert Slab Thick.	18	0.005	
Lower Level	Wall Thickness	24	0.009	
	Columns	36 x 36	0.03	
	Invert Slab Thick.	24	0.005	

VAULT STRUCTURES					
Wall Height (ft)	Thickness at Base of Wall (ft)	Base of Wall (1H:15V batter)	Invert Slab		No. Piles
		Main Reinf. Ratio	Thickness (ft)	Reinf. Ratio	per 100 sf
9	1.7	0.003	2.5	0.008	4
28	3	0.011	3	0.011	4

Table 7.16 – Summary of Structural Design Analysis Results (Continued)

OTHER STRUCTURES					
Element	**Material**	**Volume**	**Location**	**Pile Supports**	**Reinf. Ratio**
Pipe Supports (not buried)	Concrete	3cy/ea	Place support each side of valve and under valve and every 20 feet along pipe	Not required	.0018
Collars (Buried Pipe Supports)	Concrete	3cy /ea	Place one collar support every 15 feet	2 piles/ each collar	.005
Apron Slabs and Cut-off Walls	Concrete	1.25 ft. thick	As shown on DWR drawings.	Not required	.003, each way, each face
Equipment Slabs	Concrete	2.0 ft. thick	As shown on DWR drawings.	4 piles/ 100 sq. ft.	.005, each way, each face
Thrust Blocks	Concrete	20cy/ea	Place at each bend	Not required	Not required

Chapter 8: Construction Methods and Cost Estimation

8.1 Introduction

The purpose of this work was to analyze suitable construction methods, perform construction scheduling and estimate total project construction costs related to construction of both the "rock berm" and "bench" embankment options and construction of the four integrated facilities. Information developed under the construction methods and cost estimation work was used in the risk analysis.

8.2 Island Embankment Construction Methods and Cost Estimation

Under the island embankment construction methods and cost estimation, quantity estimates for embankment fill, slope protection, piping protection and seepage control (pumping wells) were developed. Suitable construction methods and sequencing was then developed, and cost estimates were prepared.

8.2.1 Basis of Quantity Estimates

The proposed reservoirs on the islands will be developed by constructing embankments against the existing levees to crest elevation +10. The two options (as shown in Chapter 5, Figure 5.1) that were considered for construction cost estimation are the Rock Berm Option and Bench Option. The Rock Berm Option consists of placing rockfill on the slough-side of the levee to provide for stability (URS, 2003a). For the Bench Option, a bench would be excavated at elevation +3.0 to provide for stability. The Rock Berm Option includes 3,000 lineal feet of embankments on each island that are configured as the Bench Option to reduce the size of slough-side rockfill sections. The locations of the 3,000 lineal feet of Bench Option are shown in the *Earthwork Construction Cost Estimate* report, on *Figures 2 and 3*. Riprap and riprap bedding would be placed on the upper portion of the slough-side slopes to protect the embankment slopes from wave erosion.

The "rock berm" and "bench" embankment options both include embankment fill on the reservoir side that would be obtained from excavations in borrow areas within the islands. The reservoir-side slopes would be 3H:1V from the crest to elevation +4.0, and the slope would be 10H:1V below elevation +4.0. Riprap (2.5 feet thick) underlain by riprap bedding (1.0 foot thick) would be placed from the crest to elevation +3.0 to protect the steeper part of the slope from wave erosion. Riprap (2.0 feet thick below elevation +3.0) would also be placed on the north and west facing 10:1 slopes, which are the general prevailing wind and storm wind directions. For the "rock berm" option slough side riprap (2.0 feet thick) underlain by riprap bedding (1.0 foot thick) would be placed from the new embankment crest to the existing levee crest and a rock berm would be place on the remaining slope based on stability analyses. For the "bench" option slough side riprap (2.0 feet thick; 2.5 feet thick adjacent to Franks Tract & Mildred Island) underlain by riprap bedding (1.0 foot thick) would be placed from the new embankment crest to elevation +3.0 feet.

A heavy-duty woven filter fabric would be located between the existing levee and new embankment fill to mitigate piping potential as indicated in the Embankment Design Analysis report (URS, 2003a). In addition, woven filter fabric would be placed on the 10:1 slopes. Where not covered by riprap, the filter fabric would be covered by a 2-foot thick layer of compacted sandy fill, requiring continual periodic maintenance to repair erosion.

Both options have a seepage control system consisting of 50-foot deep interceptor wells spaced at about 200 feet along the crest of the embankments.

The estimated earthwork quantities for Webb Tract and Bacon Island "rock berm" and "bench" options are presented in Tables 8.1 through 8.4.

Table 8.1 – Quantity Estimate for Webb Tract (Rock Berm Option)

Item	Units	Estimated Quantity
Excavation	CY	0
Embankment Fill	CY	4,600,000
Reservoir Riprap Bedding	CY	74,000
Reservoir Riprap above el. +3	CY	185,000
Reservoir Riprap on 10:1 slope	CY	300,000
Slough Riprap Bedding	CY	7,500
Slough Riprap	CY	15,000
Slough Rockfill	CY	405,000[1]

[1]Includes a 20% increase due to loss from under-water placement.

Table 8.2 – Quantity Estimate for Webb Tract (Bench Option)

Item	Units	Estimated Quantity
Excavation	CY	500,000
Embankment Fill	CY	10,000,000
Reservoir Riprap Bedding	CY	74,000
Reservoir Riprap above el. +3	CY	185,000
Reservoir Riprap on 10:1 slope	CY	300,000
Slough Riprap Bedding	CY	55,000
Slough Riprap	CY	110,000
Slough Rockfill	CY	0

Table 8.3 – Quantity Estimate for Bacon Island (Rock Berm Option)

Item	Units	Estimated Quantity
Excavation	CY	0
Embankment Fill	CY	5,100,000
Reservoir Riprap Bedding	CY	80,000
Reservoir Riprap above el. +3	CY	200,000
Reservoir Riprap on 10:1 slope	CY	284,000
Slough Riprap Bedding	CY	8,500
Slough Riprap	CY	17,000
Slough Rockfill	CY	240,000[1]

[1]Includes a 20% increase due to loss from under-water placement.

Table 8.4 – Quantity Estimate for Bacon Island (Bench Option)

Item	Units	Estimated Quantity
Excavation	CY	480,000
Embankment Fill	CY	10,100,000
Reservoir Riprap Bedding	CY	80,000
Reservoir Riprap above el. +3	CY	200,000
Reservoir Riprap on 10:1 slope	CY	284,000
Slough Riprap Bedding	CY	65,000
Slough Riprap	CY	130,000
Slough Rockfill	CY	0

8.2.2 Basis of Construction Cost Estimates

The estimates for the island embankments were prepared in accordance with a Class 4 engineer's construction cost estimate as defined by the Association for the Advancement of Cost Engineering International (AACE, 1997). The typical expected accuracy range for this class estimate is –15% to –30% on the low side and +20% to +50% on the high side. Construction pricing for the project is in March 2003 U.S. dollars. An experienced construction cost estimator with construction and hard-dollar contract bid experience prepared this cost estimate.

A survey of imported materials, based on preliminary material gradations that were developed, was conducted to obtain material costs.

The assumptions used in the construction cost estimates are as follows:

- Costs for project management, administration and quality control staffing are based on usual wages and salaries for the area.
- Prevailing wage rates were used to estimate labor costs.
- General and administrative (G&A) cost is 5% of the direct cost; profit is 10% of the direct cost plus G&A cost; and bond is 1% of the direct cost plus G&A cost plus profit.
- Costs have not been included for maintaining and operating the existing dewatered condition of the interior of the islands during construction.
- A barge dock unloading facility will be constructed for unloading riprap bedding, riprap, and rockfill materials.
- A 20% yield factor was used to estimate the required borrow excavation volume to provide the required in-place embankment fill volumes indicated in Section 4.
- Overburden excavation volumes were estimated based on the borrow area exploration work at Webb Tract and Bacon Island (URS, 2003c). For Bacon Island, the overburden volume was estimated at 3.6 times the required borrow excavation volume. For Webb Tract, the borrow excavation was assumed to be in the western part of the island where the overburden is the thinnest, and the overburden volume was estimated at 1.3 times the required borrow excavation volume. Costs developed for removal of overburden assume that excavated overburden is wasted in adjacent borrow pits where excavation has been completed.
- Pricing for riprap bedding, riprap, and rockfill is from local commercial material suppliers with allowance for delivery to the islands by barge.
- Rockfill for the Rock Berm option will be placed underwater; due to underwater placement, a loss factor of 20% was assumed.
- It is understood that earthwork construction to buttress Delta levees has not required dewatering of the borrow area excavations (Hultgren-Tillis, 2002 and 2003). Based on this experience, costs for groundwater dewatering systems (e.g., well-points) for excavation in the borrow areas were not included. However, pumping from the existing groundwater control system would continue throughout construction. Construction costs presented in this report allow for drainage ditch and sump excavation and sump pumping.
- A 5-year embankment construction period was assumed due to the weak peat soil foundation (Embankment Design Analysis, URS, 2003a).

- A contingency allowance has not been included. DWR will include a contingency allowance in the project cost estimates.

8.2.3 Construction Approach and Schedule

The construction approach that follows is the engineer's general assessment of how the construction could proceed for the "rock berm" option. However, each contractor would have its own approach to optimize construction and minimize costs. The main construction activities are discussed below.

- Mobilization: Mobilization includes securing required permits, transporting equipment to the site, and setting up temporary facilities (offices, storage areas, water supply, power, etc.).
- Clearing, Grubbing and Site Preparation: This activity will include clearing and grubbing the site, stripping the peat, and excavating drainage ditches and sumps in borrow area paddock areas. The peat will be stockpiled near the paddock excavations and replaced in the paddocks as the borrow materials become exhausted. The peat could be excavated by large excavators (equipped with wide, low contact pressure tracks) or drag-lines. The stability of the borrow excavation slopes with adjacent heavy equipment would need to be evaluated.
- Borrow Area Excavation and Embankment Fill Construction: Constructing haul roads, the barge dock unloading facility, and other temporary construction are included in this activity. Haul roads would require ongoing grading, maintenance, and dust control. Excavation of borrow materials would be accomplished by large excavators and hauled by trucks along haul roads to stockpiles. Moisture conditioning to dry out the materials would be done in the stockpiles by disking and aerating the materials prior to hauling the materials to the embankments by scrapers or trucks. Bulldozers would spread the materials and rollers would compact the materials in lifts. The maximum fill differential elevation would need to be limited to reduce the potential for foundation failure during construction. Therefore, the fill would need to be placed in horizontal lifts around the entire island perimeter prior to beginning another lift. For embankment construction to be completed in 5 years, approximately 1.9 million cubic yards of earthfill per year would need to be placed in the embankments, on average, in both reservoir islands combined (about 5,400 cubic yards per day per island). For estimating purposes, earthfill operations would generally take place 5 days per week, 8 hours per day.
- Rockfill on Slough side: Placement of rockfill would be accomplished by placing rock with cranes from barges.
- Riprap and Bedding: Riprap and bedding on the reservoir and slough sides would be placed by excavators, lagging behind the embankment fill placement. Bulldozers would also be used to spread the riprap and bedding materials.
- Placement of Filter Fabric: Woven filter fabric would be placed on the reservoir side of the existing levees during the first two years of embankment placement to serve for erosion and piping control during construction (see Section 2.2). Filter fabric would also be placed on the 10:1 slopes of the new fill.
- Road Base: The road base would be placed on the embankment crests after they have been topped out.
- Instrumentation: Vibrating wire piezometers and survey points would be installed at selected locations as the embankments are placed to monitor embankment performance during construction. Inclinometers and final survey points would be installed at the completion of embankment construction. Due to the length of the reservoir island embankments (total of 27 miles) and the need

for a comprehensive monitoring program, an automated data acquisition system (ADAS) is included.

- Seepage Control System: Well drilling would begin after the embankments have been completed. This work would occur during the sixth year of construction to allow for some settlement prior to well installation; this would reduce the potential for damage to the wells.

The construction schedule prepared for the “rock berm” option is shown in the *Earthwork Construction Cost Estimate* report, *Appendix C*. The schedule reflects a total construction duration of 6 years, working about 8 months per year (between April and November). The contractor would need to keep a work force on site to monitor, maintain and repair the earthworks during the winter months. The schedule shows the basic sequence of construction activities and that work on both islands would proceed concurrently. The schedule also indicates the engineering and bidding periods.

8.2.4 Construction Cost Estimates

The construction cost estimate for the “rock berm” option is summarized in Table 8.7. The “rock berm” option was chosen as the preferred alternative for the island embankments based on both stability analysis results and cost. More details on the cost estimates for the “rock berm” and “bench” options are given in the *Earthwork Construction Cost Estimate* report, URS June 2003.

The island embankment cost estimates shown in Table 8.7 reflect a change from the cost estimates reported in the *Earthwork Construction Cost Estimate* report, URS June 2003. The URS cost estimates include riprap protection on the north and west facing 10:1 reservoir side slopes, which are the general prevailing wind and storm wind directions. The costs in Table 8.7 are based on using 12-inches of soil cement with bentonite mix for the 10:1 reservoir side slope protection in place of the riprap included by URS. Successful improvements using soil cement to protect the embankments at Clifton Court Forebay (CCF) have been made recently (2002). Clifton Court Forebay has much steeper side slopes than the 10:1 reservoir side slopes proposed for this project and CCF is under Division of Safety of Dams jurisdiction. Based on these considerations, soil cement is an appropriate and cost effective measure to protect the 10:1 reservoir side embankment slopes.

Unit costs used for the CCF work completed in 2002 were obtained from DWR’s Division of Engineering and used to estimate the costs for placing soil cement on the 10:1 reservoir side slopes. A unit cost of $8.65 per square yard was used for 12-inch thick soil cement. Quantities of soil cement required total 514,286 square yards for Webb Tract and 486,857 square yards for Bacon Island. This translates into a reduction in island embankment (rock-berm option) construction costs (based on using riprap) of roughly $9,772,000 for Webb Tract and $9,250,000 for Bacon Island, which reduces the total embankment construction cost from about $227 million to $208 million.

8.3 Integrated Facility Earthworks Construction Methods and Cost Estimation

The foundation materials at the integrated facility site locations are similar to those of the islands. The upper 5 to 25 feet of materials consist of peat soils and soft clays, which overly stiffer and denser interbedded sands and clays. Prior to constructing the integrated facility structures and embankments, the soft soils need to be removed. This section describes the excavation plan and associated costs for removing the soft soils and replacing them with suitable materials.

The work required to replace the soft soils includes the following:

- Furnish and install sheet piling around the pumping facilities and embankment area prior to excavation. Dewater the area around the pumping facilities, gate structures and embankments prior to excavation.
- Excavate the facility and embankment areas.
- Utilizing materials from the borrow areas, place and compact the earthwork for the facilities and embankments.
- Furnish and install the rip-rap and bedding material for the embankments surrounding the facilities.

This work will be performed simultaneously with that of the integrated facility structures construction.

8.3.1 Method of Construction

Sheet Piling Installation

The first phase of this work would include installing the sheet piling required for dewatering during excavation and backfill work. The length of the sheet piling for the Webb Tract, San Joaquin River Pumping Plant is approximately 4,500 lineal feet with a height of 37 feet. This would encompass the entire area within the boundaries of the service road that goes around the pumping structure and parallel to the location of the fish screen on the delta side. The lengths of the sheet piling for the other facility on Webb Tract and the Bacon Island facilities are approximately 4,000 to 4,200 lineal feet each with heights ranging from approximately 33 to 35 feet.

Dewatering Methods

Dewatering during excavation and construction of the Integrated Facilities will consist of initially placing sheet piling approximately 40 feet beyond the outer limits of the excavation footprints. Dewatering wells will be placed on 50-foot centers between the sheet piling and the toe of the excavation pit. After excavation of the peat, drainage ditches and sump pumps will be installed adjacent to the toe of the excavation pits, which will help keep the foundation drier during initial embankment and foundation placement.

It is assumed that water pumped out of the dewatering wells will not be pumped directly into the delta channels. It is likely that detention basins would need to be constructed so that sediments can settle before water is allowed to enter the delta.

Excavation Plan

The basic excavation plan for the Integrated Facilities is to remove all peat soil and soft clays that overlie the denser and stiffer inter-bedded sands and clays. The extent of the excavation at each site is generally just outside of the foundation footprint for the embankments and concrete structures. It is not necessary to excavate the soft soils within the transition pool area. The estimated peat depth for removal varies from 5 feet around the fish screen area to 25 feet around the gate structures, pumping plant and embankments. The average existing ground elevation and the estimated elevation of the top of the denser and stiffer inter-bedded sands and clays (bottom of excavation pit) at each facility are shown in Table 8.5.

Table 8.5 – Integrated Facility Site Elevations

Approximate Elevations	Webb Tract		Bacon Island	
	San Joaquin R.	False River	Middle River	Santa Fe Cut
Average Existing Ground Elevation	-16	-15	-14	-9
Bottom of Excavation Pit Elevation	-32	-28	-29	-21

Large excavators, weighing in the 150,000 to 200,000 pound range, and trucks will remove the peat and place it in a stock pile area for eventual placement into the borrow area excavation pits. Roadway ramps of 10% grade will be required along the sides of the excavation to provide access for construction equipment.

Earthwork Construction

Earthwork construction includes transporting borrow materials from stockpiles, placing, and compacting them at the integrated facility. The interior embankments are assumed to have a 35-foot wide crest with 3:1 rip-rapped side slopes. Borrow material for the embankments will come from the same borrow area utilized for embankment construction around the island perimeters. The compacted embankments will be placed up to the elevations of the facility structure foundations prior to construction of the structures. Embankment construction will then continue simultaneously with the structure construction.

Schedule

The schedule for construction of the integrated facility earthworks is included in the overall project construction schedule and is shown in the *Earthwork Construction Cost Estimate* report, *Appendix C*.

8.3.2 Construction Cost Estimates

The construction cost estimate for excavation and embankment construction at the integrated facility sites is summarized in Table 8.7. More details on the cost estimates for excavation and embankment construction at the integrated facility sites are given in Appendix C of the *Earthwork Construction Cost Estimate* report, URS June 2003. The cost estimates include the following:

- Project mobilization and demobilization costs.
- Project indirect costs (project staff, jobsite facilities, utilities, equipment, bonds, and insurances)
- Labor, materials, and equipment to furnish, install, and remove dewatering wells.
- Labor, materials, and equipment to furnish, install, and remove sheet piling.
- Labor, and equipment to excavate integrated facilities site.
- Labor, and equipment to relocate, place, and compact the embankment.
- Labor, materials, and equipment to furnish and place the rip-rap and rip-rap bedding materials.

8.4 Integrated Facility Structures Construction Methods and Cost Estimation

Under the integrated facility structures construction methods and cost estimation, quantity estimates were developed for all integrated facility components, which include the fish screen facilities, gate structures, pumping stations, conduit pipes and associated outlet structures, bypass channel bridge structures, and sheet pile walls. Suitable construction methods and sequencing was then developed, and cost estimates were prepared.

8.4.1 Basis of Quantity Estimates

Quantities derived for the integrated facility cost estimates were based on the design drawings provided in the *Integrated Facilities Engineering Design and Analyses* report, *Appendix C*. The design drawings include: general site layouts; structural details and concrete outlines for all concrete structures; retaining wall structures; bridges; foundation piling and plans; pumping plant arrangements and mechanical equipment; piping arrangements and supports; gate control structures and mechanical equipment; energy dissipation structures; valve vaults and valves; permanent sheet piling; rip rap and slope protection; stoplogs; fish screens and associated equipment, controls, and cleaning facilities; and general arrangements.

Although details of the sites are similar, each site differs by structure heights, invert elevations, fish screen widths, and/or existing site conditions. Structural design details from the Structural Design report were also used. Cost estimates for some mechanical and electrical equipment were obtained by DWR and input directly into the cost estimates.

The estimated quantities and itemized list of materials for each site are included in the detailed Cost Estimate spreadsheets as shown in the *Integrated Facility Structures Construction Cost Estimate* report, *Appendix A*.

8.4.2 Basis of Construction Cost Estimates

The estimates for the integrated facility structures were prepared in accordance with a Class 4 engineer's construction cost estimate as defined by AACE International.

Construction pricing for the project is presented as May 2003 U.S. dollars. Pricing is accomplished with unit pricing from published and internally developed and maintained historical databases and from crew makeup for the construction of the types of facilities for this project. All unit pricing is factored for location, contractor markups, and other project-specific criteria. Material pricing was obtained from vendor or supplier quotations, current similar types of cost estimates, and cost estimator experience. Average crew make-ups are assumed to be utilizing trade labor and construction equipment in the construction of the facilities and related activities, taking into account the logic, methods, and procedures for developing costs as are typical for the construction industry.

The assumptions used in the construction cost estimates are as follows:

- Costs for construction project management, administration and quality control staffing are based on usual wages and salaries for the area.
- Current prevailing wage rates were used to estimate trade labor costs.
- General and administrative (G&A) cost is 5% of the direct cost; profit is 10% of the direct cost plus G&A cost; and bond is 1% of the direct cost plus G&A cost plus profit.
- Construction windows are assumed to be year round with some allowance for weather related problems. There are no significant fisheries or pile driving restrictions since these activities will occur in the dry.

- The existing levee is to remain in place as the main cofferdam, and is to be removed as part of the overall earthwork portion of the project after construction of the structures and features of the project.
- The majority of the setback /ring levees around the pumping plants, along the bypass channels, and around the gate structures are assumed to be constructed along with the facilities. The portions of these that tie-in or connect to the integrated facilities will be constructed as the construction of these is completed or proceeds out of the base ground elevations. Foundations are assumed to be removed and replaced with proper materials and they are adequately consolidated and that significant earth movements and any possible settlements are accounted for. Cost for levee embankments and foundation work is not included in this estimate.
- A dewatering system is to be used throughout most of the construction period (30 months of dewatering) and is accounted for elsewhere in the overall project estimate. Water is assumed to be returned into the adjacent channels as long as it meets water quality requirements.
- All construction is assumed to be completed in the dry and from land based equipment, with the exception of the existing levee removal (included in the earthwork estimate).
- The barge unloading facility constructed as part of the earthworks and general project mobilization setup for the project was assumed to be used for the receiving of raw materials, structural items, and mechanical equipment used for the construction of the facilities.
- Concrete assumed to be produced on the island by means of a portable batch plant that will be set-up at a staging location, using raw materials that have been trucked and barged to the project location. It has been assumed that a single portable batch plant would serve as supplier for each of the two facilities being constructed on that island. There would be a requirement of two portable batch plants if construction is concurrent on the two islands, Webb Tract and Bacon Island.
- A 3-year construction period was assumed for the construction of each of the overall project facilities, two on the Webb Tract and two on Bacon Island. This assumes that the setback/ring levee is constructed prior to integrated facility construction.
- At completion of the integrated facilities, the existing levee will be removed and formed to the grade as shown on the plans. The material will be placed on the landside of the levee within one-half mile of the intake. Material removed below the waterline will be dredged and placed in same general area. Water resulting from the decanting process will be returned to the adjacent Delta channels when it meets water quality standards. Minor grade shaping will be completed after the material is completely dried.
- Any Contingency, Engineering, Legal, and Administration allowance's will be accounted for by DWR when all portions of the cost estimates are combined for the overall project.

Schedule

The schedule for construction of the integrated facility structures is included in the overall project construction schedule and is shown in the *Earthwork Construction Cost Estimate* report, *Appendix C*.

8.4.3 Construction Cost Estimates

The construction cost estimates for the four integrated facilities structural components are summarized in Table 8.7. More details on the cost estimates for the integrated facilities structural components are given in the *Integrated Facility Structures Construction Cost Estimate* report, CH2MHILL June 2003.

8.5 Summary of Cost Estimates

8.5.1 Annual Operation and Maintenance Costs

Table 8.6 shows annual operation and maintenance costs for the project. The basis of the annual operation and maintenance costs used for the In-Delta Storage Program are summarized below.

Table 8.6 – Annual Operation and Maintenance Cost Summary

Item	Amount
1. Embankment Maintenance	$ 837,000
2. Integrated Facilities and Fish Screen Maintenance	$ 400,000
3. Pump Operations	$ 983,000
4. Seepage Control System	$ 610,000
5. Habitat Islands, Fishery Monitoring and O&M	$ 1,700,000
6. Invasive Weed Control on Reservoir Islands	$ 722,000
7. Recreation	$ 265,000
8. Cultural Resources Mitigation	$ 10,000
9. Property Taxes	$ 346,000
Total Operation and Maintenance Costs	**$ 5,873,000**

Levee Maintenance:
Levee maintenance cost of $29,000 per mile was provided by Department of Water Resources, Division of Planning and Local Assistance, Central District Office. 1998 Cost Index of 1.059 was used. Length of levees used for Bacon Island and Webb Tract were 14.35 miles and 12.92 miles, respectively.

Integrated Facilities and Fish Screen Maintenance:
This cost was estimated by In-Delta Storage Program by assuming two persons per year for 4 facilities plus equipment replacement cost.

Pump Operations:
CALSIM-II study was used to estimate pump operation in kw-hr. Unit cost of $0.14/kw-hr was used to estimate power costs. 1999 Cost Index of 1.044 was used.

Seepage Control System:
This cost was based on 5% of the cost of installing a ‘Seepage Control System’. This includes the cost of interceptor wells, monitoring wells, electrical and control systems and filter fabric for piping mitigation.

Habitat Island monitoring and Operation and Maintenance:
It is based on the maintenance costs at Vic Fazio Wildlife Area and Stone Lakes National Wildlife Refuge. This includes the cost of invasive weed control on the habitat islands.

Fisheries Mitigation and monitoring:
This cost is based on mitigation and monitoring required for USFWS & NMFS biological opinions and DFG ITP.

Invasive weed control on reservoir islands:
This cost is based on DWR’s aquatic weed control at Clifton Court Forebay.

Recreation:
This cost was provided by CH2M Hill assuming public ownership of the Project.

Cultural Resources Mitigation:
This cost was provided by DWR's Division of Environmental Services. It is in compliance with Historic Properties Management Plan for the life of the project.

Property Taxes:
Delta Wetlands Properties provided information on property taxes. Property tax for Holland Tract is $61,177, Bouldin Island is $98,774, Webb Tract is $92,678 and for Bacon Island is $93,441.

8.5.2 Total Project Construction Costs

Table 8.7 shows total project construction costs for the "rock berm" option, including contingencies and costs for engineering design, construction management, administration and legal. Detailed estimates of Items 1 through 5 are provided in the *Earthwork Construction Cost Estimate* report, URS June 2003 and detailed estimates of Item 6 are provided in the *Integrated Facility Structures Construction Cost Estimate* report, CH2MHILL June 2003. Details on some of the miscellaneous costs (Item 7) are provided in the *Draft Environmental Evaluations* report, DWR June 2003.

8.5.2.1 Cost Contingencies

Project contingency costs were assumed to vary for embankment earthwork and integrated facilities construction. Generally contingencies are equal to 20% of the base construction estimates. For embankments earthwork and Integrated Facilities earthwork , a value of 25% was used due to uncertainties of material estimates. For Integrated Facility structures, mechanical and electrical components, a 20% contingency cost was included.

The engineering, construction administration and legal costs vary depending upon the study level.

	Range	Average
Engineering Design	6-10%	8%
Construction Administration	6-10%	8%
Legal	2-5%	3%
Total	10-25%	19%

Detailed cost estimates presented in engineering cost reports, include a value of 5% for contractors engineering, construction and administrative management. For the final adjustment to Engineering, Construction Management and Legal costs, an additional 20% of the subtotal of the base construction estimates plus contingencies was added. This cost component would account for project planning as well as engineering design (final design) and construction management. Lastly, legal and administrative costs associated with land acquisition, construction contracts and infrastructure relocation are also considered in this component. Both of these assumptions are typical for projects of this magnitude.

Table 8.7 – Total Project Construction Cost Summary

Item	Amount	
1. Island Embankments [1]		
Webb Tract (Rock Berm Option)	$ 87,428,000	
Bacon Island (Rock Berm Option)	$ 90,067,000	
2. Seepage Control System	$ 12,200,040	
3. Instrumentation	$ 3,000,000	
4. Mobilization for Embankment Construction [2]	$ 14,986,000	**$ 207,682,000**
5. Integrated Facility Embankments [3]		
Webb Tract @ San Joaquin River	$ 19,585,500	
Webb Tract @ False River	$ 17,357,300	
Bacon Island @ Middle River	$ 18,974,950	
Bacon Island @ Santa Fe Cut	$ 15,250,150	**$ 71,168,000**
6. Integrated Facility Structures [3]		
Webb Tract @ San Joaquin River	$ 36,830,697	
Webb Tract @ False River	$ 35,002,266	
Bacon Island @ Middle River	$ 36,694,504	
Bacon Island @ Santa Fe Cut	$ 38,415,855	**$ 146,944,000**
7. Miscellaneous		
Land Acquisition	$ 60,000,000	
Mitigation	$ 34,450,000	
Demolition & Hazardous Materials Clean Up	$ 8,000,000	
PG&E Pipeline & Electrical Relocation	$ 15,000,000	
Permits	$ 300,000	**$ 117,750,000**
SUBTOTAL		**$ 543,544,000**
Contingency for Island Embankment Earthwork (25%)		**$ 44,374,000**
Contingency for Facilities Earthwork (25%)		**$ 17,792,000**
Contingency for Facility Structures and Others (20%) [4]		**$ 31,014,000**
Contingency for Miscellaneous (15%) [5]		**$ 8,618,000**
Subtotal with Contingencies		**$ 645,342,000**
Costs for Eng Design, Const Mgmt, Admin & Legal [6]		**$ 129,069,000**
TOTAL COST		**$ 774,411,000**
Annual Operation & Maintenance Costs [7]		$ 5,873,000

1 Costs are based on using 12-inches of Soil Cement for 10:1 reservoir side slope protection
2 Includes mobilization for island embankments, seepage control and instrumentation
3 Costs include mobilization at each facility
4 "Others" include Seepage Control System and Instrumentation and does not include mobilization costs
5 Excludes Land Acquisition and Permits Costs
6 This cost is 20% of Subtotal with Contingencies
7 A description and breakdown of the Annual O&M Costs are provided in the Engineering Summary Report

8.5.3 Effect of Climate Change on Cost Estimates

To include the impact of global warming and climate change, resulting sea level rise was considered for engineering costs estimates. Based on climate impact studies conducted by various agencies, climate change may cause a slow rise of 0.5 feet in the Delta water levels over the 50-year life of the project. This rise can be easily handled by normal embankment maintenance operations over the next 50 years and no additional costs were included in the cost estimates.

Chapter 9: Risk Analysis

9.1 Introduction

The purpose of the risk analysis was to evaluate the risk and consequences of failure of the existing levees and In-Delta Storage Re-engineered project embankments and integrated facilities under all loading events (operational, seismic, and flooding) and estimate the loss-of-life risk and economic losses through uncontrolled releases. The risk analysis was conducted in accordance with the general USBR risk analysis guidelines. The results of the analysis were used to evaluate the expected project performance relative to the "no action" alternative (i.e., existing levees).

9.2 Assumptions and Limitations

The following is a list of assumptions and limitations used to evaluate probabilities and consequences of failure and to calculate the project risk:

- Probability of simultaneous occurrence of two major events (flooding and earthquake for this analysis) is negligible. This is a common assumption in risk analysis.
- The reservoir would operate at or near full level (elevation +4') during the months of April through June and at or near empty level (elevation –15') during the months of July through March.
- Probability of more than two simultaneous breaches of the embankment within each island due to flooding or earthquake is negligible.
- Probability of more than one simultaneous breach of the embankment within each island due to operational loading is negligible.
- Probability of failure of the levee on each neighboring island given that the embankment fails during flooding or earthquake is 100%. That is, if an earthquake or a flood causes the embankment to fail, it would also cause the levees on neighboring islands to fail. This is a reasonable assumption because the embankment would be an engineered project designed to withstand the expected seismic and flooding loading. In contrast, most of the existing levees are not engineered structures and hence would be much more vulnerable to seismic and flooding events. Thus, if an earthquake or flood were strong enough to cause the engineered embankment to fail, it would cause the levees to fail as well.
- The simultaneous failure of the project embankment as well as the existing levees would cause system-wide hydraulic changes in the Delta. As stated above, if an earthquake or flood causes the failure of the embankment, it is also assumed to cause the failure of the levees on neighboring islands. In such a scenario, the overall impact of the system-wide hydraulic changes to water quality could be substantial. However, the incremental impact of the embankment failure to water quality by itself (for example, increased salinity) would not be significant. This is a reasonable assumption because the volume of water that would be drawn from the slough into the reservoir, or released from the reservoir into the slough, would be only a small portion of the total volume of water that would be drawn into all the other islands. Therefore, the impact to Delta water quality is analyzed only under an operational failure, but not under a failure due to flood or earthquake loading.
- Given a failure of the embankment due to operational loading and an outward breach that floods the slough, there is a finite probability that a levee on a neighboring island would fail due to flood wave impact. This probability of levee failure depends on the slough width (with higher probabilities for narrower sloughs) and also on the probability of successful flood fighting measures on the neighboring islands.

- During a flooding event, relatively little boating activity is assumed to be present in the slough.
- Only direct costs and benefits are included in the economic analysis. Indirect and induced local economic effects (the "ripple" effects) are not considered.
- Only readily available and published information is used to estimate economic losses from a failure of the embankment or a levee on a neighboring island (no field surveys were conducted). Where necessary, professional judgment is used to supplement available information to estimate economic losses.

9.3 Risk Analysis Methodology

Following the general USBR guidelines for risk analysis, risk may be defined as the product of the probability of a loading event, times the probability of system failure when subjected to the loading event, times the consequences of system failure.

An "event tree" model was used to represent the chronological sequence of events from the occurrence of a loading event to the embankment failure to consequences of failure. This model was applied to each of the two reservoir islands – Webb Tract and Bacon Island. The main steps in implementing the model for each reservoir island were as follows:

- Identify alternative projects for the project embankment.
- Identify loading events.
- Characterize alternative load levels of each loading event.
- Characterize alternative operational scenarios.
- Evaluate the probability of a breach for each combination of loading event, load level, and operational scenario.
- Evaluate probabilities of alternative breach scenarios given the occurrence of a breach.
- Evaluate the expected consequences of each breach scenario.
- Integrate the information from the previous steps to calculate the risk of failure.

A brief description of each step is provided in the Risk Analysis report.

9.4 Evaluation of Consequences of Failure

9.4.1 Consequences of Inward Breach of Project Embankment

The economic losses associated with the following consequences of the inward breach of the project embankment were evaluated. The dollar values associated with these economic losses are summarized in Table 9.1.

Table 9.1 – Consequences of Inward Breach

	Rock Berm Alternative	Bench Alternative	No Action
Cost of Breach Repair ($000)	2,900	4,000	1,000
Unit Cost of Repairing Interceptor Well ($000/well)	40	40	40
Expected Number of Interceptor Wells Impacted by a Breach	5	5	0
Expected Cost of Repairs to Interceptor Wells ($000)	200	200	0
Unit Cost of Repairing Integrated Facility ($000/facility)	500	500	500
Probability of Damage to Integrated Facility	2.9%	2.9%	0.0%
Expected Cost of Repairs to Integrated Facilities ($000)	14.3	14.3	0.0
Total Repair Cost ($000)	**3,114**	**4,214**	**1,000**
Cost of Fish Entrainment Recovery ($000)	**10**		
Volume of Water Loss (acre-foot)	75,000		
Unit Cost of Acquiring and Pumping to Make Up for the Water Supply during Service Interruption ($/acre-foot)	70		
Total Cost of Making Up the Water Supply during Service Interruption, ($000)[1]	**5,250**		
Notes:			
(1) This cost impact is assumed only for an operational failure during July through November.			

9.4.1.1 Embankment Repair

The nature and extent of the potential damage to the embankment was assessed under each breach scenario and the cost to repair the embankment and restore its functionality was estimated. These costs were estimated for both project alternatives – Rock Berm and Bench.

9.4.1.2 Damage to Equipment

The damage to the interceptor wells and integrated facilities was assessed under each breach scenario and the cost to repair the damage and restore functionality was estimated. Failure of the interceptor wells without a breach (i.e., due to malfunctioning) was not analyzed. The probability of simultaneous failure of multiple wells due to malfunctioning was judged to be negligible.

9.4.1.3 Impact to Fish

Fish may be trapped inside the reservoir once the breach is repaired. The cost of seining the fish and transporting them back into the slough was estimated.

9.4.1.4 Impact to Water Quality and Water Supply

The flow of the Delta water into the reservoir could increase the salinity of the Delta water at the pumping stations for Contra Costa Water District (CCWD) and also possibly for the State Water Project (SWP) and Central Valley Project (CVP), causing pumping to be interrupted. The duration of pumping interruption was assumed to be four days based on discussion with CCWD. The corresponding loss of water supply would have to be made up from emergency sources. The cost of acquiring and pumping the make-up water was estimated under this scenario.

We estimated that CCWD would have to use about 25,000 acre-feet of water from the emergency storage in the Los Vaqueros Reservoir during the period of high salinity and the SWP and CVP would lose about 50,000 acre-feet during an inward breach. Thus, the total volume of water that would have to be made up following an inward breach during the period of low fresh water flows would be 75,000 acre-feet (25,000 acre-feet for CCWD and 50,000 acre-feet for the SWP & CVP combined). The cost of acquiring and pumping the make-up water was estimated to be $70/acre-feet.

9.4.1.5 Flooding of Project Island from a Breach of Existing Levee

This scenario addresses the probability and consequences of failure of the candidate project islands under the ”no action” (i.e., existing levee) condition. In this scenario, Webb Tract and Bacon Island are assumed to be operated as farming islands.

A breach of the existing levee on a project island (i.e., Webb Tract or Bacon Island) would flood the island and impact the current resources and infrastructure. The economic losses from these impacts were estimated. To provide a proper comparison between the estimated risks of the re-engineered project and existing levees, the consequences of flooding the project island were excluded for all alternatives.

The risk of loss of life due to flooding was considered to be insignificant because of limited exposure and sufficient warning time.

9.4.2 Consequences of Outward Breach of Project Embankment

9.4.2.1 Embankment Repair

The nature and extent of the potential damage to the embankment was assessed under each breach scenario and the cost to repair the embankment and restore its functionality was estimated. Separate repair costs were estimated for the two project alternatives – Rock Berm and Bench. Based on quantity and cost estimates the unit cost of breach repair per foot was calculated to be $2,400/lineal foot. The width of a breach was assumed to be 1,000 feet. The cost of breach repair at Webb Tract was, therefore, calculated to about $2.4 million. An additional cost of $0.5 million was assumed for foundation repair. The total breach repair cost for the Rock Berm alternative was then estimated at $2.9 million. A similar calculation was made for the Bench alternative at Webb Tract and the resulting breach repair cost was estimated at $4 million. For each alternative, the same breach repair cost was assumed for both reservoir islands.

9.4.2.2 Damage to Equipment

The probabilities of damaging the interceptor wells and integrated facilities were assessed under each breach scenario and the cost to repair the damage and restore functionality was estimated.

The cost of replacing each well was assumed to be $40,000 in this analysis. The probability of significant damage to the integrated facility when subjected to an embankment breach would be 50%. The cost of repairing such a facility for both Rock Berm and Bench alternatives was estimated to be 1% of the construction cost, which is about $500,000. These repair costs were also used for Bacon Island for the Rock Berm and Bench alternatives.

9.4.2.3 Impact to Fish

An outward breach may damage the fish habitat in the slough. A response to damaged fish habitat may involve repairing the habitat or enhancing an off-site area associated with a natural functioning Delta system. An equivalent restoration effort to repair a damaged spawning pool in an eastside Delta tributary was estimated to be $500,000. The probability that a habitat restoration action would be required was assessed to be relatively small (10%). Therefore, the expected cost of addressing fish impact was calculated to be (0.1 x $500,000=) $50,000.

9.4.2.4 Loss of Water from the Reservoir

Approximately 35,000 acre-feet of water would be lost from an outward breach of the project embankment. This water would have to be subsequently pumped back into the reservoir. The cost of acquiring and pumping the make-up water was assumed to be $70 per acre-foot.

9.4.2.5 Impact to Water Supply

An outward breach may impact the quality of water in the Delta by increasing the total organic carbon (TOC) in the water. Because of a concern about potential health impacts of drinking contaminated water, Contra Costa Water District may interrupt the pumping operations from the Delta, disinfect the contaminated water and blend it with water from Los Vaqueros Reservoir. The impact at SWP and CVP pumping intakes was assumed to be minimal because their intakes are some 15 miles away from the potential area of impact. Making assumptions similar to those for an inward breach, the total volume of water that would have to be made up by CCWD following an outward breach was estimated to be 25,000 acre-feet and the cost of acquiring and pumping the make-up water was assumed to be $70/acre-foot.

9.4.2.6 Impact to Marinas and Recreational Water Activities

The flood into the slough could cause damage to the facilities and infrastructure at the marinas in the impacted area. The probability that an outward breach would damage each marina was estimated based on the width of the slough separating the reservoir island embankment and the marina. The estimated probabilities were 50%, 10%, and 0%, respectively for narrow (less than 1,000 feet wide), medium (1,000 feet to 2,000 feet wide), and wide (greater than 2,000 feet wide) sloughs. If a marina were to be damaged, the repair cost and loss of revenues was estimated to be $200,000.

9.4.2.7 Loss of Life

Based on the results of breach analysis and engineering judgment, the zone of impact was assumed to extend half a mile centered on the breach location. The expected number of fatalities given an outward breach was calculated based on the expected number of people within the vulnerability zone and the assumed fatality rate. The results are summarized in the Risk Analysis report, Table 9.2.

For purposes of cost-benefit analysis, government agencies have recommended the use of “value of a statistical life (VSL)”. The VSL is the amount of money one would be “willing to pay” (i.e., willing to invest in a safety improvement action) in order to reduce the expected number of fatalities by one. This concept is appropriate to use in justifying a project that is expected to provide safety benefits (i.e., to reduce the expected number of fatalities). By no means should the VSL be misconstrued as the worth of a human life. Based on guidelines provided by the U. S. Department of Transportation and U. S. Environmental Protection Agency, a VSL of $3 million was used in this analysis.

Table 9.2 – Expected Loss of Life From Outward Breach

Possible Months for Outward Breach	Time of Week	Time of Day	Proportion of a Year in This Scenario	Average Number of People in Vulnerability Zone	Fatality Rate	Expected # of Fatalities	Value of a Statistical Life, VSL ($000)	Expected Value of Loss of Life ($000)
April through June	Friday-Sunday	Day Time	0.21	3	10%	0.063	3,000	**189**
		Night Time	0.21	1	10%	0.021	3,000	**63**
	Monday-Thursday	Day Time	0.29	1.2	10%	0.0348	3,000	**104.4**
		Night Time	0.29	0.4	10%	0.0116	3,000	**34.8**
		Total				**0.1304**		**391**

9.4.3 Consequences of Flooding of Neighboring Islands

In the event of an outward breach on the reservoir-island embankment caused by operational loading, the levee on the island adjoining the breach may also fail, resulting in flooding of the island. Such a failure could occur due to the impact of waves generated from the reservoir island breach. The probability of failure of the levee depends on the width of the slough separating the two islands and on the success of any flood fighting measures that may be undertaken. The greater the width of the slough separating the two islands, the less severe would be the threat to the integrity of the neighboring island levee and lower would be the probability of a levee breach on the neighboring island.

The economic losses from various consequences of flooding a neighboring island were estimated. The approach to estimating the various losses are described in the sections below and the dollar values are summarized in the Risk Analysis report, Table 6. For the sake of completeness, Table 6 also includes the various losses from flooding the project islands, although these losses were not included in the estimated dollar risk.

The risk of loss of life from the flooding of a neighboring island was considered to be insignificant. This is because there should be sufficient warning time to any individuals inside the neighboring island following a breach of the reservoir island and the individuals should be able to evacuate.

9.4.3.1 Damage to Levee, Buildings, and Infrastructure

The costs of repairing or replacing damaged levees, buildings, and infrastructure facilities were estimated.

The data on existing levees on the candidate project islands was used to roughly estimate the breach repair cost for the levees on the neighboring islands. This resulted in a breach repair cost of about $1 million for 1000-foot-long breach. The number of buildings on adjacent islands was estimated and it was assumed that the unit flood damage repair cost would be about half of the replacement cost, or $100 per square foot. The total flood damage repair costs were calculated assuming that the average building size is approximately 2,000 square feet.

The unit cost of road or rail replacement was assumed to be $1 million per mile. The replacement cost of a bridge was estimated to be $500,000 and the bridge repair cost was assumed to be about 5% of the replacement cost, or $25,000.

9.4.3.2 Impact to Agricultural Resources

Economic losses were estimated from the destruction of existing crops and the loss of future farming during the period in which the land could not be used for farming.

Crop acreages were calculated using GIS data developed by the California Department of Conservation's Farmland Mapping and Monitoring Program (California Department of Conservation, 2002). The crop area estimates do not include land identified as grazing land, urban and built-up land, other land, or water.

Total estimated losses are based upon the assumption that two crop seasons would be affected (current and subsequent). Losses would vary depending upon the season of inundation. There is no specific data on crop types in the study area, but it is reasonable to assume that at least 70% of the crops are summer field crops that would be affected by inundation if the breach occurred between March 1 and November 1. The remaining 30% of cropland may consist of orchards, alfalfa, or other perennial crops that would be affected by inundation during the winter months. The estimated value of the loss would be approximately $640 per acre.

9.4.3.3 Impact to Natural Habitats

Natural habitat area was estimated using the California Natural Diversity Data Base (CNDDB) GIS data (CDFG, 2002). An average cost of habitat restoration in the Delta was assumed to be similar to the cost of the approved habitat restoration plan in the CALFED Ecosystem Restoration Program, which is $50,000/acre (CALFED, 2002).

9.5 Comparison of Failure Risks of Existing Levee and Re-Engineered Project

Table 9.3 shows a comparison of the failure probabilities and risks under the "no-action" alternative (i.e., existing levee) and the two re-engineered alternatives at Webb Tract and Bacon Island.

In comparing the expected dollar risk under the existing levee to the In-Delta Storage (IDS) Project alternatives, the economic losses from the flooding of the project island were not included. This is appropriate because for the IDS Project, the loss of current resources would not be related to the risk of failure of the project embankment and hence this consequence is logically a part of the project cost. Since the loss of current resources on the project island is not considered for the IDS Project alternative, a consistent risk comparison requires that the loss not be considered for the "no-action" alternative (existing levee) as well. However, for a stand-alone (i.e., non-comparative) evaluation of the risk of the existing levee, this loss may be included. Table 9.3 shows the expected dollar risk of the existing levee failure under both scenarios; that is, including and excluding the economic losses caused by the impact to current resources on the project island.

The expected dollar loss with flooding under existing conditions is large because multiple levee failures could occur during a period of 50 years under existing conditions. It is assumed that after a levee failure that causes flooding of a project island, the levee would be repaired and the island would be redeveloped to its current land uses. To illustrate the estimation of the economic losses from flooding of a project island under existing conditions, consider Webb Tract. Table 6 shows that the economic losses from flooding of Webb Tract would be about $21 million. Under existing conditions, the annual probability of an inward breach causing flooding of Webb Tract is about 10% (5% from flooding and 5% from operating loading). Thus, over a project life of 50 years, about 5 inward breaches that cause flooding of Webb Tract would be expected. The total expected economic losses from five flooding events at Webb Tract under existing conditions would be about $100 million. This loss from flooding when added to other losses results in the expected dollar risk of $131 million under existing conditions, as shown in Table 9.3. Similar calculations for Bacon Island result in the expected dollar risk of $177 million under existing conditions as shown in Table 9.3.

Referring to Table 9.3, the failure probability for the existing levee is higher than for the re-engineered alternatives by factors of 6 to 8. The expected dollar risk (without considering the loss of current resources on the project island) for the existing levee is higher than for the re-engineered alternatives by factors of 2 to 6.

A comparison of the two re-engineered alternatives shows that the probability of failure is about the same for the two alternatives at both project islands (see Table 9.3). The expected dollar risk for the Rock Berm alternative is lower by about 30% than for the Bench alternative at both Webb Tract and Bacon Island. The expected number of fatalities for the Rock Berm alternative is lower than for the Bench alternative by a factor of about 2.5 to 3, at both Webb Tract and Bacon Island.

A comparison of the risks for the two candidate project islands shows that the failure probabilities, the expected dollar risks, and expected number of fatalities for each alternative are about the same for both islands (see Table 9.3).

Table 9.4 shows the contributions of the three loading events to the overall failure probability and risk for each project alternative at the two candidate project islands. For the two re-engineered alternatives, the operational loading contributes only 1% to 2% to the failure probability and expected dollar risk. This is because the failure probability for the re-engineered alternatives under operational loading is very small. The flooding and seismic loading contributes about 40% and 60%, respectively, to the failure probability and expected dollar risk for the re-engineered alternatives. The probability of failure under flooding is mostly due to overtopping, while the contribution of piping/internal erosion to the probability of failure is minor. With regard to the expected number of fatalities for the re-engineered alternatives, almost all of the contribution is from seismic loading. Flooding does not contribute to the fatality risk, because only an inward breach is possible under flooding and the fatality risk under an inward breach is negligible.

For the existing levees at the candidate project islands, flooding contributes 62% to 74% to the failure probability. This is because of the relative low crest elevation of the existing levees such that a 100-year flood is likely to cause overtopping. For the expected dollar risk for the existing levees, the operational loading has a major contribution, because of the potential water supply interruption from an inward breach of the existing levees.

The estimated risk for each reservoir island may be used in a cost-benefit analysis of the IDS Project. The benefits of the IDS Project include environmental enhancement, water revenues from users, improved water quality, and recreation. An evaluation of these benefits can be found in a DWR report

(DWR, 2002). These benefits may be compared to the project cost and the expected consequences of failure analyzed in this report.

Table 9.3 – Comparison of Risks under Re-Engineered Project Alternatives and Existing Levees

Reservoir Island	Annual Failure Probability			Expected Dollar Risk during 50 Years ($000)			Expected Number of Fatalities during 50 Years		
	Rock Berm	Bench	Existing Levee	Rock Berm	Bench	Existing Levee	Rock Berm	Bench	Existing Levee
Webb Tract	0.0213	0.0225	0.1740	2,085	2,972	13,152 w/o flooding losses 131,175 w/ flooding losses	0.0025	0.0064	Insignificant
Bacon Island	0.0217	0.0231	0.1440	2,112	3,059	7,231 w/o flooding losses 176,650 w/ flooding losses	0.0025	0.0073	Insignificant

Table 9.4 – Risk Contributions of Loading Events

Reservoir Island	% Contribution to Annual Failure Probability			% Contribution to Expected Dollar Risk during 50 Years ($000)				% Contribution to Expected Number of Fatalities during 50 Years		
	Rock Berm	Bench	Existing Levee	Rock Berm	Bench	Existing Levee w/o Flooding Losses	Existing Levee w/ Flooding Losses	Rock Berm	Bench	Existing Levee
Webb Tract -Flooding -Seismic -Operational -Total	 42 57 1 100	 39 60 1 100	 62 9 29 100	 39 59 2 100	 37 61 2 100	 21 4 75 100	 45 7 48 100	 0 99 1 100	 0 100 0 100	N/A
Bacon Island -Flooding -Seismic -Operational -Total	 41 58 1 100	 38 61 1 100	 74 12 14 100	 39 59 2 100	 36 62 2 100	 38 7 55 100	 64 10 26 100	 0 98 2 100	 0 99 1 100	N/A

Appendix A: Independent Board of Consultants Report No. 2

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INDEPENDENT BOARD OF CONSULTANTS
REPORT NO. 2
IN-DELTA STORAGE PROGRAM

May 30. 2003

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Subject: **In-Delta Storage Program**
Board of Consultants Meeting No. 2

Gentlemen:

As scheduled by Program Manager, Mr. Tirath Pal Sandhu, by e-mail on March 21, 2003 the second meeting of the Independent Board of Consultants for the In-Delta Program was held on May 28 – 30, 2003. The meeting was held according to the Agenda (Attachment A). The purpose of the meeting was to brief the Board and obtain comments on the feasibility level engineering designs for the proposed embankments and integrated facilities for the In-Delta Storage Program. As indicated on the Agenda, the Board received a thorough briefing on all aspects of the feasibility level designs for In-Delta Storage Project features on May 28, 2003 in DWR offices in Sacramento. May 29 was spent in the Delta visiting Webb Tract and the Brown Sand, Inc. pit for a demonstration of its below water level excavation procedures. Numerous stops were made to discuss existing and proposed embankment issues, view proposed borrow areas for embankment materials, and to review and discuss construction excavation, dewatering, and staging of construction at two sites considered for integrated facilities. Accompanying the group in the Delta were Dave Forkel, Delta Wetlands Properties, Inc. and Ed Hultgren, Hultgren-Tillis Engineers, who provided valuable insight into operation of island properties, potential borrow sites, and levee construction and maintenance issues. This report was prepared and presented to the Department May 30, 2003. A list of attendees is included as Attachment B.

Prior to the meeting, Board members were provided copies of the following documents for review:

- Independent Board of Consultants for In-Delta Storage Program-Second Meeting-May 28-30, 2003 – Information Package
- In-Delta Storage Program Embankment Analysis Draft Report, April 2003, URS Corporation
- In-Delta Storage Program Flooding Analysis Draft Report, April 2003, URS Corporation
- In-Delta Storage Program Seismic Analysis Draft Report, April 2003, URS Corporation
- In-Delta Storage Program Borrow Area Geotechnical Report Draft Report, April 2003 URS Corporation
- In-Delta Storage Program Integrated Facilities Engineering Design and Analyses Draft Report, April 2003, DWR and URS Corporation
- In-Delta Storage Program Earthwork Construction Cost Estimate Draft Report, April 2003, URS Corporation
- In-Delta Storage Program Feasibility Study Results of Geologic Study Program, January 2003, DWR
- In-Delta Storage Program Feasibility Study Results of Laboratory Testing Program, January 2003, DWR
- In-Delta Storage Program Integrated Facility Structures Construction Cost Estimate Draft Report, May 2003, CH2MHILL
- In-Delta Storage Program Risk Analysis Draft Report, May 2003, URS Corporation

During the formal presentations and field inspection numerous questions were asked and answered and informal discussions were held regarding the project feasibility designs and cost estimates. The major issues are discussed in the Board's responses to the following questions presented to the Board:

Question 1

Based on the presentations and the reports provided, are the engineering design analyses for the embankment design adequate "for feasibility level design"?

Response

To answer this question for the engineering and designs currently developed for the In-Delta Storage Program, as proposed by the Department, there needs to be a definition of "Feasibility Level Design" that would specifically apply to this project. This is particularly significant because of the context of the project with respect to the overall importance of the Bay Delta in connection with both the environment and the water supply of California, and the complicated review process that will be required to obtain the final approvals required to finance, design, construct and operate the project.

The Board proposes that the following definition be considered:

"Feasibility level design is taking the preliminary investigations, engineering studies (including operational understanding and descriptions), analyses, design drawings, cost estimates (including construction cost estimates and other capital costs), and O&M cost estimates for all facilities to a level of technical detail wherein no major changes or surprises will occur as the project moves into final design, construction, and operation."

Based on this definition, and based on the reports that were currently provided to the Board, the Board believes that, subject to comments in following paragraphs, the embankment design meets the feasibility level design requirements. The embankment cross-sections, as currently proposed, represent technically feasible designs. Similarly, static and seismic stability analyses based on these sections represent a suitable basis for risk assessment at this stage. These are, however, not likely to represent final design sections, as further refinements and optimization will be warranted.

There may be both geometric (over-steepened slopes due to scour or dredging) and environmental difficulties in placement of the outboard continuous rock berm along some sections, and this will likely require use of the alternative "bench" configuration along some reaches. These locations should be identified and clearly delineated, and cost estimates should reflect mixed use of both types of sections. Some reduction in costs, and some further improvement in both static and seismic stability, may be achievable if tensile reinforcement (e.g.: Tensar geogrid, or similar) is included in the "bench" type of cross section. This reinforcement is relatively inexpensive, and might result in net cost reduction if overall section fill volumes can be reduced. These types of refinements can be considered during later stages of design.

Another issue that warrants additional consideration is the minimum freeboard required along the various reaches of the proposed embankments. In the cases wherein the embankment crest height is controlled by the combined considerations of water level (tide plus runoff) plus wind driven wave set-up and ride-up, some additional minimum freeboard should be provided above the maximum run-up level.

Question 2

Does the Board consider the geotechnical investigations to delineate the borrow areas for embankment fill materials and the integrated facility locations adequate for assessment of availability of borrow materials and integrated facilities feasibility level foundation design?

Response

The geotechnical investigations performed to delineate the prospective borrow areas for the principal embankment fill materials (fine sands and silty sands) are adequate and suitable for feasibility stage studies, and demonstrate local availability of sufficient materials for the project.

Similarly, the geotechnical site investigations for the integrated facilities are suitable for feasibility level studies, and these serve to adequately demonstrate the technical feasibility of locating these proposed facilities at the four locations currently under consideration. Additional site investigations will be needed at these four sites, in the next stage of design studies, to provide a more detailed basis for refinement of designs and development of more detailed construction plans and specifications.

Question 3.

Based on the information presented in engineering design reports, is there a need for erosion and seepage control in the re-engineered embankments? Is the proposed solution acceptable to the Board?

Response

Yes, there is a need for both erosion and seepage control on the re-engineered embankments. The proposed solutions, as presented to date, represent a significant improvement, but further refinements are warranted.

Protection against piping at the reservoir side toe (and low on the embankment face) is a critical issue during the periods when the reservoirs are lower than the water levels in the adjacent sloughs. The current proposal consists of use of geotextile filter fabric to be located at the inboard side interface between the existing levee sections and the proposed new embankment fills. This is a good idea, and will provide some level of protection against reservoir side piping instability. As noted in the draft design documents, however, this filter fabric will be vulnerable to tearing as a result of anticipated differential settlements of the enlarged embankments.

Addition of a second level of geotextile filter higher up, nearer to the final face of the final embankment, and at a later stage (after much of the initial settlement has occurred) would provide a significantly increased level of protection, and at relatively low cost. This location (and timing) would reduce differential movements to which the geotextile would be subjected, and would improve accessibility for localized repairs if necessary. The geotextile selected should be optimized for its ability to safely withstand differential movements without tearing or rupture.

Inspection and maintenance will still be needed, both routinely over time and also at times of unusually high differential water levels (e.g.: during high flood stages in the outboard sloughs when inboard reservoir levels are low.) With suitable operations and maintenance provisions, protection against inboard piping erosion should be adequate.

A second type of erosion on the inboard (reservoir side) faces is potential erosion of the embankment faces due to both wave and wind forces. Wave erosion, with a filled or partially filled reservoir, may be amenable to handling largely as a maintenance issue, with erosion damage being repaired during times of low reservoir levels. Similarly, wind erosion (primarily during times of low reservoir levels) might also be addressed as a maintenance issue. It should be noted that wind erosion potential for the proposed project embankments is different than for most existing Delta levees because periodic inundation of the inboard sides will suppress the growth of vegetation that provides protection against wind erosion of typical existing inboard levee faces.

An alternative would be to provide some level of inboard side erosion protection, either over large areas or over selected areas of special importance or vulnerability. Prevailing winds, and storm winds, will be the key forces driving both wind and inboard wave erosion potential, and provision of coarse granular covers (gravels and/or rock) could be applied over selected reaches.

The currently proposed embankment cross sections make no special provision for protection against piping erosion on the outboard side faces under differential gradients due to conditions of full reservoir (Elev. +4 ft.) vs. outboard slough levels. This appears appropriate, as seepage lengths are long, and differential water levels are limited. If proposed reservoir storage levels increase at a later stage in design, this should be revisited.

Provisions for protection against wave erosion on the outboard faces appear generally suitable at this stage for the currently proposed embankment sections.

Question 4

Based on the draft reports on Integrated Facilities hydraulic and structural design and presentations, are the integrated facilities design studies adequate for the feasibility level of evaluations?

Response

Based on the definition proposed by the Board, the designs currently provided by the Department for the Integrated Facilities do not completely meet the feasibility level design requirements. The investigations that have been conducted to date, however, do provide the data and information needed to supplement the current designs with additional engineering studies to meet the requirements.

The main element missing is an excavation plan at each site identifying the magnitude of excavation required before the construction of all other facilities would take place. As recommended by the Board in its first meeting in December 2001, the embankments for the Integrated Facilities should be placed on mineral soils, not on the peat. Excavation and removal of compressible organic materials is necessary in order to (a) reduce problems associated with differential settlements between structural facilities and IF embankments, and (b) improve seismic performance of the current pile-supported structural facilities, which would otherwise be subjected to relatively large lateral displacements of the pile heads during seismic loadings.

An excavation plan needs to be developed for each site to show what the rough excavation plan of the site will be before the layout is finalized for all the embankments surrounding the transition, pool, mid bay, bypass channel, and other compacted fills for structures. This plan of excavation should be a construction stage site plan that shows and describes the location of the sheet piling cofferdams, and dewatering facilities. The temporary embankment construction required to stabilize the existing levee embankment should also be shown and described. This layout is needed to be able to estimate the rough excavation quantities and other work required to complete the cost estimate.

Additionally, it is important to emphasize the need for a hydraulic model design study during final design phase for the integrated facilities. This will be important to finalize design for the fish screen, the transition pool geometry, and the other hydraulic structures, as well as the specific setback location from the existing levee alignment.

Question 5

Does the Board consider the information on the construction methods and costs sufficient to estimate feasibility level costs for the project?

Response

Based on the reports provided for review and the presentations made to the Board, the information provided on the construction methods is not sufficient and the cost estimates are not totally complete. Some additional work needs to be performed to adequately describe the most feasible construction methods suitable for the project (as specified in the scope of work to the Department's consultants).

Additionally, an overall construction schedule needs to be presented, which summarizes the sequence of construction planned for the construction of embankments for Webb Tract and Bacon Tract and each of the Integrated Facilities. Major construction operational activities should also be shown, such as mobilization, installation of transportation facilities required, construction of temporary camp site facilities for equipment maintenance, borrow pit overburden excavation and dewatering facilities. The time frames required for engineering, final design and tendering should also be shown.

In conjunction with the development of a construction schedule, a write-up needs to be provided to describe at least one (or more if time permits) feasible construction methods, including equipment identification, suitable for the construction of the embankments, as well as for the construction of the Integrated Facilities. The current reports just provide "Cost Estimate Assumptions", which is not an evaluation of construction methods and equipment. This is especially important because of the location and accessibility of the project, and the uniqueness of the construction requirements.

In addition, with regard to the Integrated Facilities, there are some missing items in the cost estimates. These include the costs required to perform the dewatering and maintaining stability of the site during the rough excavation stage and the estimated cost of the rough excavation and fill quantities.

Lastly, the Board recommends that the Department consider applying different contingencies to different project features to provide an overall contingency allowance that is representative of the feasibility level designs. For example, contingency allowances of 20 percent are considered adequate for mechanical and electrical equipment components if actual budget quotes have been obtained for potential suppliers. Higher contingencies however should be applied to the embankment construction and construction of the integrated facilities, because of the unknowns that could result from the future activities during the EIR/EIS phase and final design phase (e.g.: model studies for the integrated facilities, etc.)

Question 6

Based on the presentations and reports completed by URS, is the evaluation of the risk and consequences of failure adequately evaluated?

Response

Based on the presentations and reports completed by URS, the evaluation of risk and consequences appears generally suitable and adequate for this feasibility level stage of design studies. These are complex issues, and they often require significant judgmental input. Opinions will always vary among engineers as to the precise values appropriate at any particular element of the risk analyses. The Board has some specific recommendations for some of these elements, and additional modifications of risk may be warranted as design sections and other details continue to be refined. Overall, however, the risk analyses appear well-structured and well-considered, and the results appear to provide a reasonable basis for assessment of the elements of risk addressed.

It is not clear, from the current report, whether or not ongoing subsidence has been included within the risk assessment. If the risk assessment for the “do nothing” case is based only on current geometry and elevations, then it systematically underestimates risk over the next 50 years and should be adjusted. It should be noted that the same is not true for the re-engineered storage reservoir scheme however, as island subsidence (due to oxidation and loss of peaty/organic soils, etc.) will be significantly reduced by the proposed storage operations.

The “expected dollar risk during 50 years” in Table 15 of the draft URS risk analysis report for the Existing Levee Case(s) “w/flooding losses” appear to be misleading, as they appear to incorporate the risk associated with potential failures of Mandeville and other adjacent islands. Mandeville and these other adjacent islands are indeed at risk, both with regard to seepage, overtopping and seismic failures, but this risk is not significantly affected by the construction (or non-construction) of the proposed In-Delta Storage facilities. These values should be deleted from this summary table (leaving in place the risk costs "w/o flooding losses"), and these risks should be addressed elsewhere. The flooding losses for inundation of Webb Tract and Bacon Island themselves, however, are significantly affected by the construction or non-construction of the proposed storage facilities, and these risk costs should be included in this table.

The risk analyses as performed are generally appropriate for consideration of the current project elements, but they do not address the broader issues of overall risk of the Delta with regard to both environmental consequences and/or water use consequences associated with potential Delta-wide seismic fragilities. This would be difficult to assess within the current scope of URS' work, but it should be noted in summary documentation currently being prepared for subsequent review and decision making that potentially significant benefits of the proposed project include providing two strategically well located, defensible and/or rapidly reparable islands which provide significant potential flexibility with regard to both reduction of overall Delta seismic fragility, as well as potential response to a significant and damaging seismic scenario.

Question 7

Based on the presented materials, does the Board have any other comments on the work completed to date and planned for the future or specific comments on the Program Management and Coordination?

Response

It is understood that the Department has a scheduled date to complete the Draft Feasibility Report, ready for review by the Science Panel by June 30, 2003. In the Board's opinion this is a very tight schedule considering work required to completely finalize the feasibility level designs and cost estimates suitable for presentation in the report.

Finally, it should be clearly noted in such summary documents that the current designs do not provide for assured non-failure of the proposed storage facilities during strong seismic loading. Instead, the risk of failures (or breaches) of the proposed reservoirs are considered in the current planning and design as an acceptable level of risk. Such breaches would be significantly less costly to repair than typical failures of "existing" Delta levees, as embankment widths are greater and differential water elevations between the reservoirs and adjacent sloughs are greatly reduced during periods of reservoir storage. Also important is the reduction of the consequences of potential failures during low flow periods in the sloughs (Summer and Fall). During these periods, the reservoirs would be full or at least partially full, so that potential failures would not result in drawing water into the failed islands, resulting in increased salinity levels. Instead, fresh water would be released, with beneficial impact on salinity levels into what would be a damaged overall Delta system, and minimization of scour damage would facilitate rapid repair of potential failures on the two project islands.

These are potentially very significant project benefits, but their value is difficult to assess, and depends to some extent on the actions that may be taken to reduce seismic vulnerability of appurtenant islands, levees, and other Delta facilities.

Concluding Remarks:

The Board appreciates the arrangements and courtesies extended by DWR for this meeting. The Board appreciates the high quality of the reports supplied by DWR and its Consultants prior to the meeting. The presentations summarizing the feasibility studies were excellent, and discussions and response to questions were sufficiently adequate, allowing the Board adequate information with which to formulate its responses to the questions posed.

Respectfully Yours,

Raymond B. Seed Alan L. O'Neill John Williams

Appendix A
AGENDA

Wednesday, May 28, 2003

Large Conference Room, 2nd Floor
Bonderson Building, 901 – P Street, Sacramento

08:00-08:15	Introductions Welcome to the Board Purpose of Meeting	(Harder/Cowin)
08:15-08:30	Review of Agenda Charge to the Board	(Roberts)
08:30-09:00	Program Management and Coordination	(Sandhu)
09:00-09:30	In-Delta Storage Engineering Studies and Investigations	(Arrich)
09:30-09:45	Break	
09:45-10:30	Embankments Design	(Salah-Mars/Roadifer)
10:30-11:00	Risk Analysis	(Salah-Mars/Kulkarni)
11:00-11:30	Erosion and Seepage control	(Forrest)
11:30-12:00	Borrow Area Delineation and Borrow Quantity Estimation	(Forrest)
12:00-13:00	Lunch (on own)	
13:00-13:45	Integrated Facilities Engineering Design	(Arrich)
13:45-14:00	Integrated Facilities Mechanical and Electrical Design	(Meininger)
14:00-14:15	Integrated Facilities Structural Design	(Johnson)
14:15- 14:45	Construction Methods and Costs	(Forrest/Hays/Lawson)
14:45-15:00	Questions for the Board	(Roberts)
15:00-15:15	Field Trip Briefing	(Sandhu)

Wednesday, May 28, 2003 (cont.)

15:15-16:00	General Discussion and Questions from the Board	(all)
16:00-17:00	Consulting Board Executive Session (Optional)	(Board)
18:30-20:00	Dinner	

Thursday, May 29, 2003

07:30	Load vehicles at Resource Building
07:30-09:30	Travel to Project Site
10:00-10:15	Board ferry to Webb Tract
10:15-12:00	Tour Webb Tract
12:00-12:45	Lunch (sack lunches provided)
13:00-13:15	Board ferry
13:15-14:30	Tour Bacon Island
14:30-16:00	Brown Sands Excavations Demo
16:00-17:30	Return to Resources Building
18:30-20:00	Dinner
20:00-	Board Executive Session

Friday, May 30, 2003

08:30-14:00	Consulting Board Executive Session	(Board only)
14:00-15:00	Presentation of Board's Recommendations	(Board)
15:00	Adjournment	

Appendix B

ATTENDANCE
May 28, 2003

Consulting Board Members

Alan O'Neill
John Williams
Ray Seed

URS

Mike Forrest
Said Salah-Mars
John Roadifer
Tracy Johnson
Ram Kulkarni

CH2M Hill

Darryl Hayes
Bob Lawson

USBR

David Lewis
Becky Morfitt

Department of Water Resources

Division of Engineering

Les Harder
Cosme Diaz
Brent Lamkin
Jasmine Doan
John Meininger

Division of Planning and Local Assistance-HQ

Mark Cowin
Steve Roberts

Division of Planning and Local Assistance- In-Delta Section

Tirath Pal Sandhu
Jeremy Arrich
Amy Bindra
Ganesh Pandey
Dainny Nguyen