D R A F T R E P O R T

IN-DELTA STORAGE PROGRAM INTEGRATED FACILITIES SUPPLEMENTAL STRUCTURAL ENGINEERING DESIGN AND ANALYSIS

Prepared for

Department of Water Resources
901 P Street
Sacramento, CA 94236

May 6, 2005

URS Corporation
1333 Broadway, Suite 800
Oakland, CA 94612

26815054

TABLE OF CONTENTS

Section 1 Introduction 1

1.1 Background and Purpose of Study 1
1.2 Scope of Work 1
1.2.1 Task 1 – Structural Engineering Analysis and Design 1
1.2.2 Task 2 – Prepare Summary Report of Findings 3

Section 2 Design Criteria 1

2.1 Data for Site Facilities 1
2.2 Purpose 1
2.3 Design Codes 1
2.4 Design Loads 1
2.5 Design Methods 3
2.5.1 Reinforced Concrete Design 3
2.5.2 Deep Foundation Design 3
2.6 Design Considerations 3
2.6.1 General 3
2.6.2 Reinforced Concrete Design 3
2.7 Material Strengths 4
2.8 Material Coatings 4

Section 3 Geotechnical Design Analyses 1

3.1 Summary of Soil Conditions 1
3.2 Lateral Earth Pressures 1
3.3 Axial Pile Capacity 2
3.4 Lateral Pile Capacity 3
3.5 Sheet Pile Wall 4

Section 4 Structural Design Analysis 1

4.1 Box Culvert Structures 1
4.2 Retaining Walls 1
4.3 Pump Station 1
4.4 Vault Structures 2
4.5 Other Structures and Pile Requirements 2

Section 5 Conclusions and Recommendations 1

Section References 1

Tables

Table 2-1 Integrated Facility Elevations

Table 2-2 Integrated Facility Water Surface Elevations

Table 3-1 Lateral Earth Pressures and Seismic Loads for New Fill Materials

Table 3-2 Axial Pile Capacities and Pile Tip Elevations

Table 3-3 Summary of Lateral Loads and Deflections of Single Piles

Table 3-4 Summary of Sheet Pile Analysis Results

Table 4-1 Summary of Structural Design Analysis Results for Northwest Site of Webb Tract

Table 5-1 Summary Comparison of Pile Requirements

Figures

Figure 1 Site Vicinity

Figure 2 Webb Tract - Site Plan

Figure 3 In-Delta Storage Program - Stiff Soil Response Spectrum at Pile Depth of Fixity

Figure 4 Axial Pile Compression Capacity vs Depth

Figure 5 Axial Pile Tension Capacity vs Depth

Appendices

Appendix A Basis for Maximum Head Differential for Use in Sheet Pile Wall Design

Appendix B Summary of Structural Design Analysis Results (April 2003)

1.0 BACKGROUND AND PURPOSE OF STUDY

The Department of Water Resources (DWR) is conducting feasibility-level engineering and environmental studies under the Integrated Storage Investigations Program. As part of the project evaluations, DWR is evaluating the technical feasibility and conducting engineering investigations for the In-Delta Storage Program. The engineering investigations will aim at developing solutions to enhance project reliability through improved embankment design and consolidation of inlet and outlet structures.

As part of this feasibility study, the Department requested that URS Corporation (URS) perform supplemental structural engineering design and analysis of a newly proposed integrated facility at the northwest corner of Webb Tract. The supplemental structural engineering design and analysis consists of performing structural engineering design of inlet/outlet structures, pumping stations, sheet pile walls, and structural components of the fish screens; working with DWR staff, and preparing a brief report on the structural feasibility of the proposed facilities. This newly proposed location has been chosen to explore the benefits it may have over the integrated facility proposed on the northeast corner of the island. The design information will be used by DWR to estimate quantities and costs of the newly proposed integrated facility.

2.0 SCOPE OF WORK

The structural design criteria established in the URS Draft Report In-Delta Storage Program Integrated Facilities Structural Engineering Design and Analysis (April 2003) as amended in response to USBR review comments and the general facility arrangements as documented in the DWR Draft Report Integrated Facilities Engineering Design and Analyses (July 2003) was used. DWR provided a revised Table 2-1 (Integrated Facility Elevations) from the URS draft report that includes the elevations of the newly proposed integrated facility components.

The scope of work is described in Task Order IDS-0205-1747-013 issued by DWR dated February 16, 2005. The work required under this Task Order focused on the foundation design of all structural components of the newly proposed integrated facility, located at the northwest corner of Webb Tract.

3.0.0 Task 1 – Structural Engineering Analysis and Design

Prepare State feasibility level structural analysis and design in sufficient detail to allow a feasibility-level cost estimate for the newly proposed integrated facility to be completed. The integrated facility, as described in the In-Delta Storage Program's Draft Report *Integrated Facilities Engineering Design and Analyses (July 2003)* includes: a fish screen, three inlet/outlet structures, a pumping station and conduits, a bypass channel, a sheet pile wall and associated structural facilities. Structural design considers the existing and planned subsurface conditions at the proposed facility location on Webb Tract. Information related to existing subsurface conditions at the site are based on the findings from the URS Draft Report *Proposed Integrated Facility at Webb Tract, Supplemental Geotechnical Exploration (April 2005)* prepared under Task Order IDS-1004-1747-012. That report is based on the results of one boring, DH-1, drilled at the location of the proposed fish screen. A second boring, DH-2, located at the proposed pumping plant (850 feet south of DH-1), was also to be drilled. However, site conditions were

found to be too soft and wet for drill rig access during the winter of 2004-2005 and this boring was therefore not drilled.

Information related to the planned subsurface conditions at the site was based on the findings and recommendations of the DWR Independent Board of Consultants in their Report No. 2. DWR staff will complete all necessary feasibility level design drawings based on the results of the work performed under this task order.

Task 1.1 - Design Review: The design review task includes the following elements:

- Review the design criteria used in the previous structural design and analysis (URS Draft Report *In-Delta Storage Program Integrated Facilities Structural Engineering Design and Analysis, April 2003, performed under* Task Order IDS-0203-1841-015).
- Review the revised Table 2-1 (Integrated Facility Elevations) provided by DWR and determine which structural components of the newly proposed integrated facility do not fall within the size ranges detailed in Table 4-1 (Summary of Structural Design and Analysis Results). This review includes all structural components of the fish screen structure, the three inlet/outlet structures, the pumping plant superstructure and all previously identified structures associated with the pumping stations and conduits, and the sheet pile walls and bridge crossing designed as part of the bypass channel.
- Review the previous foundation design and reasons for selecting pile foundations.
- Review the DWR Independent Board of Consultants Report No. 2 recommendation to remove the peat soils within the footprint of the integrated facilities and replace with suitable foundation materials.
- Review Appendix C of the URS Draft Report *In-Delta Storage Program Earthwork Construction Cost Estimate (June 2003).*

Task 1.2 - Structural Design: As identified in Task 1.1, prepare State feasibility level structural analysis and design for the structural components of the newly proposed integrated facility that do not fall within the size ranges detailed in Table 4-1. Lateral earth pressures and seismic loads from Table 3-1 of the URS Draft Report *In-Delta Storage Program Integrated Facilities Structural Engineering Design and Analysis (April 2003)* will apply to this newly proposed integrated facility. The major difference in the design of the newly proposed integrated facility is the Midbay elevation. It has been lowered by approximately 5 feet as compared to the previous design to better accommodate pump head and hydraulic jump formation requirements. The sheet pile wall design was re-evaluated based on the removal of peat soils and replacement with mineral soils within the footprint of the integrated facility. The mechanical design and loading of the newly proposed integrated facility pumping station remained unchanged from the original design.

Task 1.3 - Foundation Design: The foundation materials at all the integrated facility site locations (Webb Tract and Bacon Island) 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 will be removed and replaced with suitable foundation materials (mineral soils). This will reduce problems associated with differential settlements between structural facilities and integrated facility embankments and improve the overall seismic performance of the facilities.

This fundamental change was made after the previous structural design was completed. This peat soil replacement, along with the existing subsurface conditions at the site (see Draft Report, *Proposed Integrated Facility at Webb Tract, Supplemental Geotechnical Exploration, URS, April 2005)*, was considered in the foundation design.

Based on the findings of the URS Draft Report *In-Delta Storage Program Embankment Design Analysis (June 2003)*, URS reviewed and confirmed material properties to be used for the replacement soil. Adequate foundation materials will be selected and designed for all integrated facility structural components at the newly proposed site such that settlement, cracking and tilting do not cause structural distress. The design will be based on the recommended material properties of replacement soils, to be agreed upon by URS and DWR, and existing subsurface conditions.

4.0.0 Task 2 – Prepare Summary Report of Findings

Prepare a report summarizing the findings of the supplemental structural design and analysis of the integrated facility structural components for the newly proposed site. The report updates the previous draft report (April 2003) and is a stand-alone report. The updated report summarizes new or modified design criteria and assumptions, procedures, and results related to the analysis of pertinent structural components and foundations for the integrated facility. A revised Table 2-1 and a revised Table 4-1 are also included. Based on the designs prepared under this Task Order, URS will offer a professional opinion on a comparison of the supplemental design and the results of the previously designed integrated facilities. This includes an opinion on the foundation design recommendations (considering the peat soil replacement) and how any suggested modifications may affect cost estimates of the previously designed integrated facilities. The effect on cost estimates is based on a qualitative assessment.

5.0 DATA FOR SITE FACILITIES

The site vicinity is shown on Figure 1 and the proposed facility location at the northwest corner of Webb Tract is shown on Figure 2. Refer to Table 2-1 for Integrated Facility Elevations. The general integrated facility arrangements prepared by DWR will be used in feasibility-level structural design.

6.0 PURPOSE

The structural design criteria are based on the preliminary design criteria prepared by DWR staff. The purpose of structural design criteria is to establish the structural design standards for the following structures:

- Inlet/Outlet Structures
- Pumping Stations
- Conduit Supports, Collars, Apron and Equipment Slabs, Cut-off Walls and Thrust Blocks
- Fish Screen Supports and Decks
- Bypass Channel Bridge Structure and Trash Rack
- Vaults
- Sheet Piling for Bypass Channels

7.0 DESIGN CODES

Reinforced concrete design shall be in accordance with the 2001 California Building Code (CBC) and American Concrete Institute ACI 318, Building Code Requirements for Reinforced Concrete (2002).

8.0 DESIGN LOADS

The loads and forces in this section shall apply to all structures, structural components and structural supports designed as part of this scope of work. The load types are summarized below:

- Dead Load (D)
  - For normal weight concrete, a unit weight of 150 pcf shall be assumed.
  - For structural steel, a unit weight of 490 pcf shall be assumed.
  - Equipment, trash racks, gates, fish screens and piping weights shall be based on information provided by DWR.
- Lateral Earth Pressures (H)
  - Lateral loads from soil, at-rest, active, passive, and seismic earth pressures shall be considered. Pressure diagrams shall be developed as a function of depth for idealized soil profiles based on site-specific soil properties at individual structure locations.

- Hydrostatic Loads (F)
  - For calculating lateral loads from water and buoyancy affects, a unit weight of 62.4 pcf shall be assumed and shall be considered for applicable tide levels and operating reservoir water levels as well as the 100-year flood stage level for storm events. The corresponding elevations are provided in Table 2-2.
  - Uplift pressures shall be considered for the following dewatering scenarios:
    - Fish Screen (at least one bay at a time)
    - Transition Pool (behind the fish screen rear stoplogs)
      - Sheet pile wall will need to support hydrostatic loads when transition pool is dewatered for maintenance purposes
    - Midbay (full dewatering)
    - Reservoir (periodically emptied)
    - All gate structures will have stoplogs provided to allow dewatering
- Hydrodynamic Loads (Q)
  - Hydrodynamic loads from water developed in a seismic event, although considered, were found not to be applicable.
- Wave/Wind Action Loads (P)
  - Sheet piling shall be designed to withstand wave/wind effects as defined in the Draft Report *Flooding Analysis (URS, 2003).*
- Live Loads (L)
  - Shall include the HS20 (Caltrans, 2000) vehicle load with impact.
  - Shall include 100 psf or 1000 pound concentrated load to account for foot traffic.
  - Shall include vibration effects resulting from operation of equipment.
- Seismic Design Criteria (E)
  - For this study, the very soft near-surface soils would be excavated and replaced with compacted engineered fill. Based on the preliminary foundation design analyses presented in this report, it is expected that the majority of the structures for the integrated facilities will be supported on driven pile foundations. These pile foundations will be founded in the stiffer and denser soils present beneath the near-surface clays and peat soils.
  - A smoothed horizontal acceleration response spectrum associated with a particular seismic hazard level was previously developed for the project *(Seismic Analysis, URS, April 2003).* The selected seismic event corresponds to ground motion having a 10% probability of exceedance in 50 years (i.e., return period of about 475 years). This target spectrum represents free-field motions for the outcropping stiff soil site condition. In order to develop the ground motions that are transmitted to the structure through the pile foundations, the target spectrum was deconvolved to the deeper stiff soil layer using the computer program SHAKE to obtain the ground motions at the pile depth of fixity. This

approach applies to the scenario of removing the soft peat soils and replacing them with engineered fill. The design response spectrum is shown in Figure 3.

9.0 DESIGN METHODS

10.0.0 Reinforced Concrete Design

All reinforced concrete design shall be in accordance with the ACI Strength Design Procedures and USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures (1992).

11.0.0 Deep Foundation Design

Deep foundation design shall be in accordance with USACE EM 1110-2-2906 (1991) and design of sheet pile foundations shall be in accordance with EM 1110-2-2504 (1994).

12.0 DESIGN CONSIDERATIONS

13.0.0 General

All designs shall provide for adequate structural dimensions in accordance with the following:

- Stability with respect to sliding, overturning and uplift (USACE EM 1110-2-2502, 1989):
  - F.S. = 3.0 for D+H+F, overturning
  - F.S. = 1.5 for D+H+F, sliding and uplift
  - F.S. = 1.0 for D+E+F, overturning
  - F.S. = 1.1 for D+E+F, sliding
- Minimize differential settlement
- Control of scour
- Prevention of piping
- Pile foundations with the following factors of safety:
  - F.S. = 3.0 for D+L+H+F+P
  - F.S. = 1.7 for D+L+H+F+E
- Drainage provision where water may accumulate (including pumping plant floor slabs, stairwells, conduits, etc.)

14.0.0 Reinforced Concrete Design

Design shall provide for the appropriate concrete thicknesses and steel reinforcement patterns for structural members to resist bending moment, thrust and shear effects imposed by reasonable loads on the structure. The following factored load combinations shall apply:

- Load Combination 1: 1.3*{1.2*D+1.6*L+1.6*F+1.6*H}
- Load Combination 2: 1.4*D+1.4*L+1.4*F+1.5*E

15.0 MATERIAL STRENGTHS

Concrete

The minimum 28-day compressive strength of concrete for reinforced concrete structures shall be 4000 psi.

Reinforcing Steel

Reinforcing steel for concrete reinforcement shall conform to ASTM A615 or A706, fy=60 ksi. Plain wire for welded wire fabric shall comply with ASTM A82.

Sheet Pile Walls

Strength of sheet piling shall conform to ASTM A572, Grade 50.

Miscellaneous Steel Components

Steel components exposed to salt water and salt-water sprays shall conform to ASTM A36 (as specified in U.S. Army/TM 5-809-6 Air Force AFM 88-3, Chapter 6).

2.8 MATERIAL COATINGS

All steel components shall be either stainless steel, painted with an anti-corrosion coating system, or hot-dipped galvanized.

Table 2-1
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
			Northwest	Northeast			
Fish Screen	**Screen Dimensions**	Screen Length (vertical direction)	17	15	18	15	12
		Screen Width (horizontal direction)	10	10	10	10	10
	Elevations	Top of Screen	2.42	2.49	2.39	2.49	2.59
		Bottom of Screen (Sill) @ Screen Face	-14	-12	-15	-12	-9
		Top of Bottom Slab @ Downstream End	-14.3	-12.3	-15.3	-12.3	-9.3
		Deck (Top of Embankment)	11	11	11	10.2	10.4
	Overall	Total Facility Width	768	878	724	878	1120
		Number of Bays	35	40	33	40	51
		Clear Span Between Piers	20	20	20	20	20
Gate Structures	**Gate #1**	Sill Elevation (Top of Bottom Slab)	-14	-12	-15	-13	-8
		Deck Elevation (Top of Embankment)	11	11	11	10.2	10.4
	Gate #2	Sill Elevation (Top of Bottom Slab)	-18	-18	-18	-16	-16
		Deck Elevation (Top of Embankment)	11	11	11	10.2	10.4
	Gate #3	Sill Elevation (Top of Bottom Slab)	-15	-15	-16	-12	-8
		Deck Elevation (Top of Embankment)	11	11	11	10.2	10.4
Midbay		Floor Elevation	-29	-24	-24	-22	-22
Conduit	**Invert Elevations**	Reservoir Side	-14	-12	-12	-10	-10
		Bypass Channel Side	-14	-12	-12	-10	-10
Reservoir	**Finished Grade Elevations**	@ Gate #2 Outlet	-18	-18	-18	-16	-16
		@ Conduit Outlet	-18	-18	-18	-16	-16
Bypass Channel	**Finished Grade Elevations**	@ Conduit Outlet	-15	-15	-16	-12	-8
		@ Gate #3 Outlet	-15	-15	-16	-12	-8
		@ Connection to River Channel	-16	-16	-17	-13	-9
		Bottom Width	30	30	30	40	70
	Sheet Pile Wall	Top Elevation	11	11	11	10.2	10.4

* from DWR design criteria.

Table 2-2
Integrated Facility Water Surface Elevations*

Item	Description	Integrated Facility Location							
		Webb Tract San Joaquin River East and West		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
Plant Forebay & Afterbay Water Surface Elevation	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

* from DWR design criteria

This section describes the feasibility design geotechnical analyses performed for the In-Delta Storage Integrated Facilities. In these analyses, 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.

16.0 SUMMARY OF SOIL CONDITIONS

Boring DH-1 was drilled at Webb Tract in November 2004, and the details of this investigation are presented in the URS Draft Report *Proposed Integrated Facility at Webb Tract, Supplemental Geotechnical Report (April 2005)*. The subsurface conditions at the location of Boring DH-1 generally consist of a layer of very soft, low to high plasticity, highly compressible, black organic silt with organic debris and peat to a depth of about 18 feet. The near surface soil appears to be lighter in color due to higher degree of oxidation. The black organic silt and peat layer is underlain by alternating layers of alluvial gray and brown, silty sand (SM and SP-SM), clayey and sandy silt (ML), and silty clay (CL) to the bottom of the boring at 101.5 feet.

Silty Sand Soils: The relative density of the sandy soils ranged from loose to medium dense between about 18 feet and 29 feet below the ground surface. Below 29 feet, the sandy soils ranged from medium dense to dense, with SPT blow counts and Modified California sampler blow counts (corrected to SPT values by reducing the Modified California sampler blow counts by 20 percent) ranging from 12 to 37. Based on the laboratory testing data, the fines content (materials passing the No. 200 U.S. Standard Sieve) generally ranged from about 6% to 30% fines; one sample at 41 feet had a fines content of 2%.

Clayey and Silty Soils: The clayey and silty soils generally are low plasticity, with the plasticity index between 5 and 20. In-situ water contents range from about 22% to 38%. The consistency of the clayey and silty soils ranges from stiff to hard. The undrained shear strengths (S_u) from the unconsolidated undrained triaxial compression tests and the unconfined compression tests generally range from 2,000 psf to 5,400 psf above a depth of 90 feet. The normalized undrained shear strengths (S_u/effective overburden pressure, σ'_{vo}) of soils above 90 feet range from about 0.8 to 1.3. These values indicate that the soils are overconsolidated. The undrained shear strengths of two soil samples below this depth are about 1,200 psf to 1,300 psf with the normalized undrained shear strength of about 0.25. These undrained shear strengths indicate that these soils are normally consolidated.

Groundwater Conditions: The level of groundwater encountered at the time of drilling was about 4.5 feet below the ground surface. The groundwater levels are largely affected by the irrigation and drainage system within the island. The ground surface elevation within most of the island is lower than sea level, and the water level outside the perimeter of the island is higher than the ground surface of the island. Static groundwater levels were not recorded due to the immediate backfill of the boring with tremie grout.

17.0 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 URS Draft Report *Embankment Design Analysis (2003)*. The earth pressures in Table 3-1 are expressed as equivalent fluid weights, and are presented for

unsaturated (above groundwater level) and saturated (below groundwater level) conditions. The seismic loads presented in Table 3-1 are based on the design peak horizontal ground acceleration shown on Figure 3.

Table 3-1
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.

18.0 AXIAL PILE CAPACITY

Due to the magnitude of the loads imposed by the structures, including uplift, the structures will need to be supported by precast prestressed concrete piles. This pile type is frequently used in marine applications, has good load-carrying capacity, can be installed efficiently, and is relatively economical. For feasibility design purposes, both 14-inch and 16-inch square precast prestressed piles were evaluated, which have allowable capacities of 45 tons and 70 tons, respectively.

The results of Boring DH-1 were used to compute axial pile capacity as a function of depth for the pumping plant structure with top of pile elevation at -32 (midbay floor elevation –29). Contributions from both skin friction and end bearing were included in the axial pile compression capacity calculations. Axial pile capacity is shown as a function of depth for both 14-inch and 16-inch piles on Figure 4. Hydrostatic uplift (tension) capacity was computed based on skin friction and is shown as a function of pile depth for both 14-inch and 16-inch piles on Figure 5.

Following the recommendations presented by the USACE in Design of Pile Foundations (1991) for “usual” loading conditions, a factor of safety of 3.0 for working loads was applied. A factor of safety of 1.7 is used for “extreme” loading conditions, i.e., seismic case. In accordance with USACE (1991), the use of pile load tests or a pile driving analyzer can reduce the required factors of safety. The use of a pile driving analyzer during pile installation would allow a factor of safety of 2.5 for working loads, resulting in higher pile tip elevations.

Pile tip elevations for 14-inch and 16-inch piles with top of pile elevation –32 are summarized in Table 3-2. For feasibility study purposes, these results can be applied to piles supporting retaining walls and other structures with higher top of pile elevations, as the engineered fill that will replace the peat soil would have similar skin friction capacity to the native soils below elevation –32. For these other structures, the calculated pile tip elevation should be raised by the same interval as the difference between elevation –32 and the top of pile for the structure under consideration.

Table 3-2
Axial Pile Capacities and Pile Tip Elevations
(for top of pile elevation –32)

Factor of Safety on Ultimate Load	Compression/ Tension	14-inch Pile		16-inch Pile	
		Working Load/Ult. Capacity (tons)	Pile Tip Elevation	Working Load/Ult. Capacity (tons)	Pile Tip Elevation
3.0	Compression	45 / 135	-72	70 / 210	-76
	Tension	45 / 135	-74	70 / 210	-79
2.5*	Compression	45 / 112.5	-66	70 / 175	-73
	Tension	45 / 112.5	-70	70 / 175	-76

* FS = 2.5 if a pile driving analyzer is used.

Table 3-2 shows that the pile tip elevations are controlled by uplift (tension) capacity. Also, as shown in the table, the use of a pile driving analyzer during construction would reduce pile lengths by about 5%.

19.0 LATERAL PILE CAPACITY

The lateral capacity of the 14-inch and 16-inch square precast piles was computed using the program LPILE (Ensoft, Inc., 2000). An idealized soil profile representing the conditions at the pumping plant was modeled, with top of pile elevation -32. LPILE options for fixed and free head conditions, nonlinear EI, and prestress forces were included in the analyses. The LPILE analyses were iterated to determine the loads at which 0.25-inch and 1-inch pile head deflection would be obtained for fixed and free head conditions, for both the 14-inch and 16-inch piles as shown in Table 3-3.

Table 3-3
Summary of Lateral Loads and Deflections of Single Piles

Pile Size (inches)	Lateral Load (tons) for 0.25-in. and 1.0-in. Deflection			
	Free-Head Case		Fixed-Head Case	
	0.25-inch	1.0-inch	0.25-inch	1.0-inch
14	6.4	11.3	12.5	20.8
16	7.7	13.2	14.0	25.8

These results can be applied to piles supporting retaining walls and other structures with different top of pile elevations, as the engineered fill that will replace the peat soil would have similar lateral resistance to the native soils below elevation –32. The preliminary foundation layout includes arrays of piles in groups at spacings of approximately 3 to 8 pile diameters on center. To account for group effects, group reduction factors were applied to the load capacities for single piles shown in Table 3-3. Group effects should be more rigorously analyzed during final design with pile group soil-structure interaction analyses. Further design refinements may also consider the use of batter piles, which can more efficiently resist lateral loads.

20.0 SHEET PILE WALL

The cantilever sheet pile wall that forms the bypass channel was analyzed. An idealized soil profile was modeled that consists of engineered fill to elevation –29 feet, underlain by stiff clays and dense sands. The top of the sheet pile wall was modeled at elevation +11, water in the bypass channel at elevation +4, and water in the transition pool at elevation –10 feet, resulting in maximum head differential of 14 feet. The basis for this head differential for sheet pile wall design is discussed in Appendix A.

The estimated lateral pressures due to the 14 feet of head differential between the channel and the transition pool would induce peak bending moments in the sheet piles in the range of approximately 125 kip-ft/ft. In accordance with standard sheet pile design practice, the sheet pile tip elevations were calculated using reduced passive earth pressures. The computation of section modulus was based on specifying Grade 50 steel, and applying a factor of safety of 1.5. For feasibility design, AZ-36 sheet pile sections (manufactured by Skyline Steel/Arbed) were selected. Table 3-4 presents the sheet pile wall recommended tip elevation, maximum bending moment, required section modulus, and recommended section. Further design analyses will be needed to verify that estimated deflections are tolerable. Corrosion protection/cathodic protection will be required for the sheet pile wall.

Table 3-4
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
Webb Tract, Northwest Site	-44	125	44.7	AZ 36

This section describes the feasibility-level design of the structural elements of the In-Delta Storage Integrated Facilities at the newly proposed northwest site on Webb Tract. These analyses applied the load combinations, factors of safety and design methodology defined in the Design Criteria (Section 2) for this project, and determined the structural requirements for the structural elements of the Integrated Facilities shown on drawings prepared by DWR. For comparison purposes, the structural design analysis results presented in the April 2003 report are included in Appendix B of this report. The design approaches used in these analyses are presented in the sections that follow.

21.0 BOX CULVERT STRUCTURES

Recognizing the similarities between the fish screen supports and decks, the bypass channel bridge structure and trash rack, and the inlet/outlet (I/O) structures, a reinforced concrete box culvert section was determined to be most appropriate.

In the April 2003 report, 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, as described in Section 2. For the fish screen structure, self-cleaning and operating loads from the cleaning unit equipment were also accounted for in the analysis.

The results of the analysis are shown in Table 4-1. Thickness requirements for the concrete members as well as main reinforcement requirements are provided. For the purposes of preparing a feasibility-level cost estimate, minimum reinforcing may be assumed for the other reinforcement.

22.0 RETAINING WALLS

Cantilevered reinforced concrete retaining walls were designed for use at the ends of the I/O structures, along the approaches to the bypass channel bridge structure, and at the outlet structures for the conduits. A range of wall heights was analyzed where the top of footing elevation was assumed to be two feet below the top of the adjacent apron. The walls were designed to resist lateral pressures from soil and water, including seismic effects, as described in Sections 2 and 3.

The results of the analysis are shown in Table 4-1. For the walls at the newly proposed Northwest site on Webb Tract, only the size of the footing and the number of 14-inch piles in the shortest walls were modified. If 16-inch piles are used, the spacing between rows of piles can be increased for all wall heights. Thickness requirements for concrete members as well as main reinforcement requirements are provided for various wall heights. For the purposes of preparing a feasibility-level cost estimate, minimum reinforcing may be assumed for the other reinforcement.

23.0 PUMP STATION

A feasibility-level design for the pump station was performed. In the previous study, a 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, as described in Sections 2 and 3. Significant equipment loads necessitated the use of reinforced concrete beam floor systems. The results of the analysis are shown in Table 4-1. Locations of the various elements are described in Table 4-1.

Member sizes and main reinforcement requirements are provided for various elements. For the purposes of preparing a feasibility-level cost estimate, minimum reinforcing may be assumed for the other reinforcement. At the newly proposed northwest site on Webb Tract, only the number of piles and thickness of the concrete walls and invert slab at the lowest level were modified.

The structural design analysis results presented in the April 2003 report, summarized in Appendix B, show a corrected number of piles for the pump station.

24.0 VAULT STRUCTURES

Feasibility-level designs were prepared for the vault structures that house mechanical equipment near the gates for the I/O structures 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. For the purposes of preparing a feasibility-level cost estimate, minimum reinforcing may be assumed for other reinforcement.

Pile requirements are also provided in Table 4-1. At the newly proposed northwest site on Webb Tract, no modifications to the structural requirements are needed. The use of 16-inch piles would not have a significant impact on the number of piles required.

25.0 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 4-1. Pile requirements are also shown in Table 4-1.

For all structures, a lateral displacement of ¼-inch was allowed at the pile head for static loads and 1-inch was allowed at the pile heads for dynamic loads. These allowable displacements represent an improvement in seismic performance compared to those presented in the April 2003 design by limiting maximum displacements for all structures to 1-inch. The pile heads were assumed to be fixed against rotation at the bottom of the structures.

Table 4-1
Summary of Structural Design Analysis Results for Northwest Site of Webb Tract

BOX CULVERT STRUCTURES

Element	Thickness (ft)	Main Reinf. Ratio	No. 14-inch Piles	No. 16-inch Piles
Roof slab	1.5	0.007	9 piles per 500 sq. ft.	8 piles 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 per Row	Spacing between Rows (14”/16” Piles)
		Main Reinf. Ratio	Width (ft)	Thickness (ft)		
6' to 15'	1.7	0.003	11	2.5	3	4’/5’
16' to 27'	2.0	0.016	30	3	6	4'/5’
28' to 37'	3.0	0.013	41	3	8	4'/5’

PUMPING PLANT

Location	Element	Dimensions (in)	Main Reinf. Ratio	No. 14-inch Piles	No. 16-inch Piles
Upper Level	Beam "A"	30 x 36	0.018	380 piles total	340 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	36	0.009		
	Columns	36 x 36	0.03		
	Invert Slab Thick.	36	0.005		

VAULT STRUCTURES

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

Table 4-1 (continued)

OTHER STRUCTURES					
Element	**Material**	**Volume**	**Location**	**Pile Supports**	**Reinf. Ratio**
Pipe Supports (not buried)	Concrete	3 cy/ea	Place support each side of valve and under valve and every 20 feet along pipe	Not required	0.0018
Collars (Buried Pipe Supports)	Concrete	3 cy/ea	Place one collar support every 15 feet	2 piles/each collar	0.005
Apron Slabs and Cut-off Walls	Concrete	1.25 ft. thick	As shown on DWR drawings.	Not required	0.003, each way, each face
Equipment Slabs	Concrete	2.0 ft. thick	As shown on DWR drawings.	4 piles/100 sq. ft.	0.005, each way, each face
Thrust Blocks	Concrete	20 cy/ea	Place at each bend	Not required	Not required

The objective of this study is to perform a sufficiently detailed feasibility-level structural analysis and design of the integrated facility at the newly proposed northwest site on Webb Tract for preparation of a feasibility-level cost estimate. This report presents the results of URS' feasibility structural engineering design of the In-Delta Storage Integrated Facility inlet/outlet structures, pumping stations, sheet pile walls, bypass channel bridge structure, and structural components of the fish screens. The structural design criteria and general facility arrangements prepared by DWR were used in this work.

The subsurface conditions at the proposed integrated facility at the northwest corner of Webb Tract consist of a layer of very soft, low to high plasticity, highly compressible, organic silt with organic debris and peat to a depth of about 18 feet. The organic silt and peat layer is underlain by alternating layers of alluvial silty sand, clayey and sandy silt, and silty clay to a depth of 100 feet (depth of boring). As part of the facility construction, the soft peat soil will be excavated and replaced with compacted engineered fill.

The structures will require pile foundations to resist hydrostatic uplift loads. Design recommendations for both 14-inch and 16-inch precast prestressed concrete piles have been developed. Curves showing axial compression capacity and uplift capacity as a function of pile depth are presented, and lateral load capacities at ¼-inch and 1-inch deflection are provided. For feasibility design purposes, either 14-inch or 16-inch square precast prestressed piles may be used. The pile selection should be based on cost considerations. Sixteen-inch piles would cost more than 14-inch piles, but fewer 16-inch piles would be required.

A comparison of 14-inch and 16-inch pile requirements, including the requirements for 14-inch piles in the April 2003 report, is presented in Table 5-1.

Table 5-1
Summary Comparison of Pile Requirements

Structure*	NW Corner Webb Tract		April 2003 Report
	14-Inch Pile Requirements	16-Inch Pile Requirements	14-Inch Pile Requirements
Box Culvert	9 piles 500 sq ft	8 piles 500 sq ft	9 piles 500 sq ft
Retaining Wall Heights: 6' – 15'	3 piles/row @ 4' row spacing	3 piles/row @ 5' row spacing	2 piles/row @ 5' row spacing
Retaining Wall Heights: 16' – 27'	6 piles/row @ 4' row spacing	6 piles/row @ 5' row spacing	6 piles/row @ 4' row spacing
Retaining Wall Heights: 28' – 37'	8 piles/row @4' row spacing	8 piles/row @5' row spacing	8 piles/row @4' row spacing
Pumping Plant	380 piles total	340 piles total	280 piles total

*For structure types not shown here, the quantities for piles did not change. For more detail, see Table 4-1.

It appears that the construction cost of the integrated facilities would not be significantly affected by removal of the peat. However, removing the peat would have the benefit of limiting maximum lateral seismic displacements of structures to 1-inch. Also, removing peat would substantially reduce settlement of the integrated facility embankments.

Cantilever sheet pile walls that form the bypass channel were evaluated. The sheet pile wall design accounts for the scenario of a 14-foot head differential between water in the bypass channel and the transition pool. An AZ-36 sheet pile section was selected.

Feasibility-level design structural analyses were performed and applied the load combinations, factors of safety, and design methodology defined in the design criteria for this project. Due to the similarities between the fish screen supports and decks, the bypass channel bridge structure and trash rack and the inlet/outlet structures, a reinforced concrete box culvert section was utilized. Cantilevered reinforced concrete retaining walls were designed for use at the ends of the inlet/outlet structures, along the approaches to the bypass channel bridge structure and trash rack, and at the outlet structures for the conduits. Feasibility designs were also performed for the pump station, vault structures, conduit supports, pipe collars, equipment slabs, apron slabs, cut-off walls, and thrust blocks. Flexible conduit connections may be needed in areas where movement can occur. The results of the structural analyses are summarized in Table 4-1.

Further design may consider the use of batter piles to resist lateral loads. In addition, further studies may indicate that pre-cast concrete construction for such elements as the box culvert structures may be more economical than cast-in-place construction.

American Concrete Institute (2002). ACI 318, Building Code Requirements for Reinforced Concrete.

California Building Code (2001).

Caltrans Bridge Design Specifications (2000).

Computers and Structures, Inc., Computer Program SAP 2000 (2003). Integrated Finite Element Analysis and Design of Structures, Version 8.1.6.

Department of Water Resources, In-Delta Storage Program's Draft Report on Engineering Investigations (2002). CalFed Bay-Delta Program. May.

Ensoft Inc. (2000). Computer Program LPILE Plus Version 4.0M.

URS Corporation (2003). In-Delta Storage Program, Flooding Analysis. April.

URS Corporation (2003). In-Delta Storage Program, Borrow Area Geotechnical Report. April.

URS Corporation (2003). In-Delta Storage Program, Seismic Analysis. April.

URS Corporation (2003). In-Delta Storage Program, Integrated Facilities Structural Engineering Design and Analysis. April.

URS Corporation (2003). In-Delta Storage Program, Embankment Design Analysis. June.

URS Corporation (2003). In-Delta Storage Program, Earthwork Construction Cost Estimate. June.

URS Corporation (2005). Proposed Integrated Facility at Webb Tract, Supplemental Geotechnical Report. April.

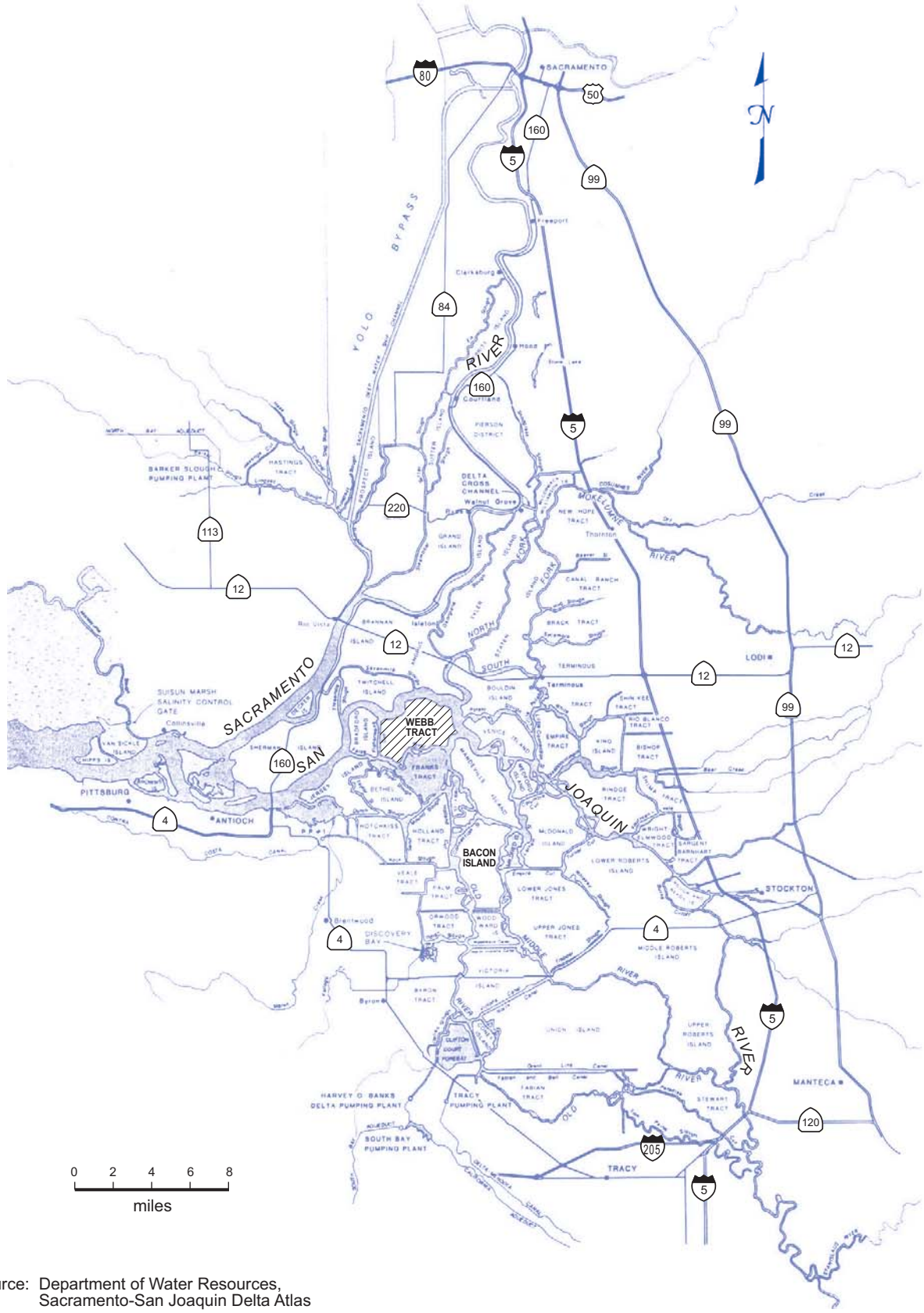
U.S. Army Corps of Engineers (1989). Retaining and Flood Walls, EM 1110-2-2502.

U.S. Army Corps of Engineers (1991). Design of Pile Foundations, EM 1110-2-2906.

U.S. Army Corps of Engineers (1992). Strength Design for Reinforced Concrete Hydraulic Structures, EM 1110-2-2104.

U.S. Army Corps of Engineers (1994). Design of Sheet Pile Walls, EM 1110-2-2504.

Figures



Source: Department of Water Resources, Sacramento-San Joaquin Delta Atlas

URS	Project No. 26814887 STATE OF CALIFORNIA DEPARTMENT OF WATER RESOURCES	SITE VICINITY	FIGURE 1

26814170.01200/012103/gos

URS	26814887 STATE OF CALIFORNIA DEPARTMENT OF WATER RESOURCES	Webb Tract - Site Plan	FIGURE 2

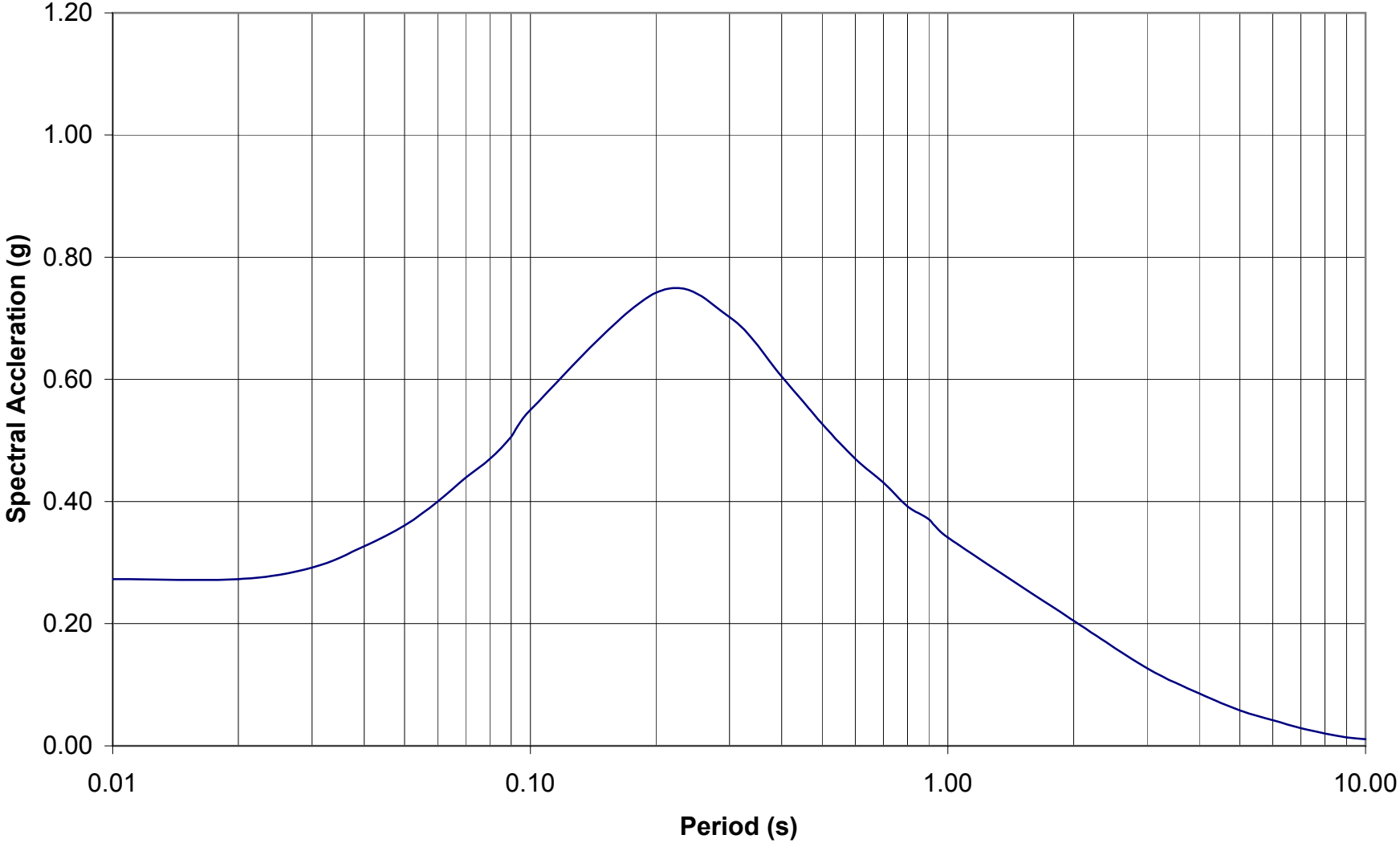
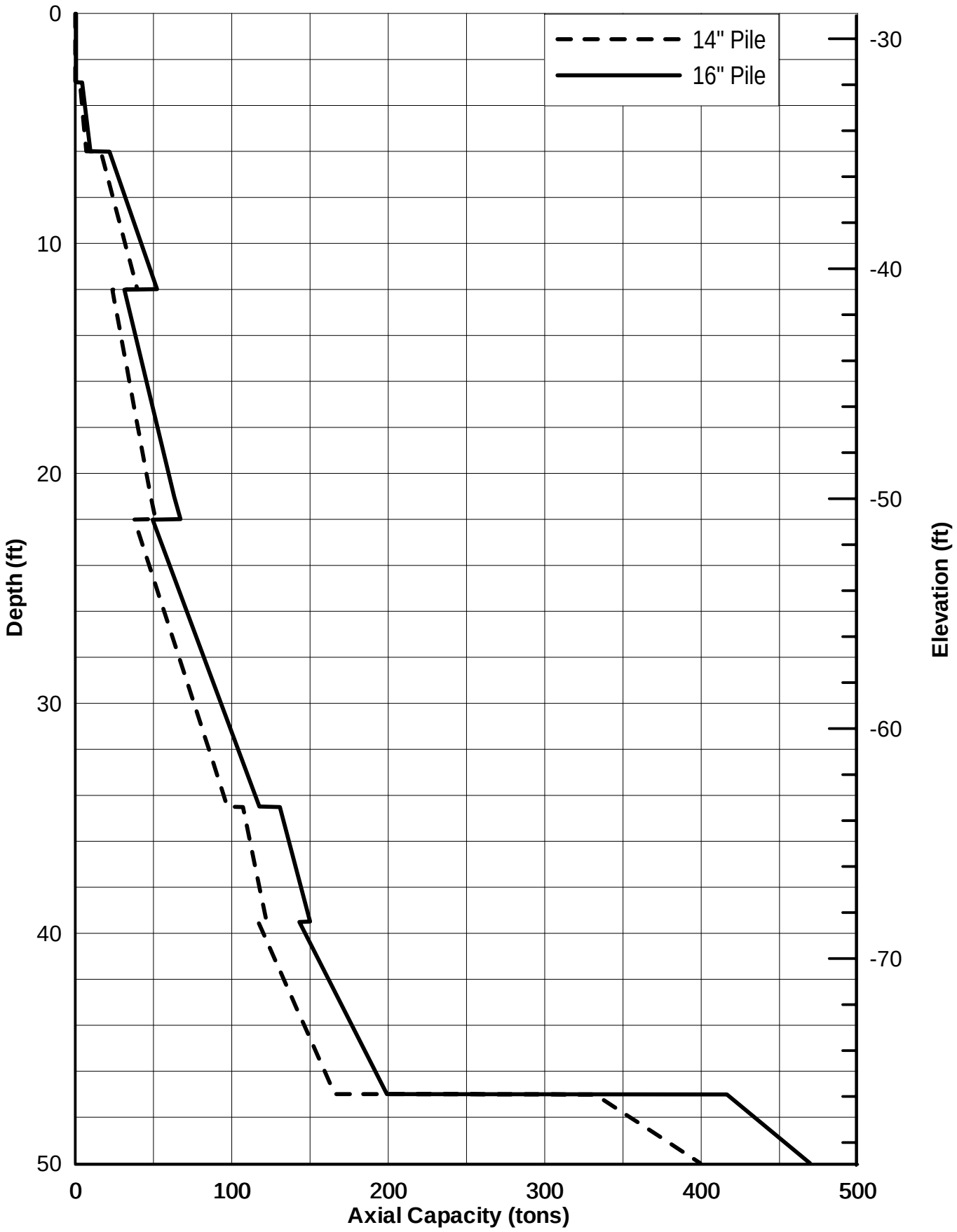
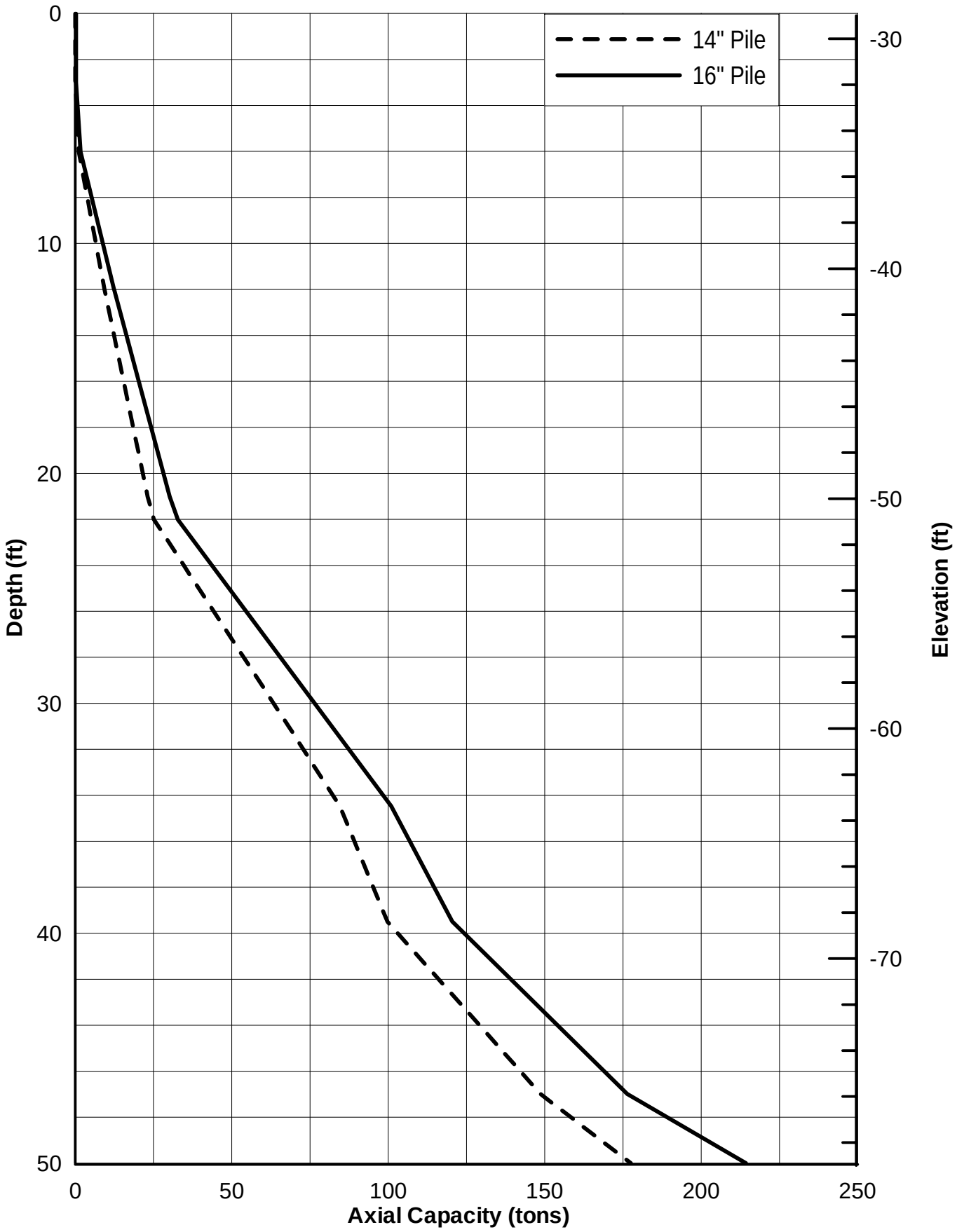


Figure 3 - In-Delta Storage Program - Stiff Soil Response Spectrum at Pile Depth of Fixity



URS	Project # 26815054 Webb Tract North DWR	Axial Pile Compression Capacity vs. Depth	FIGURE 4



URS	Project # 26815054 Webb Tract North DWR	Axial Pile Tension Capacity vs. Depth	FIGURE 5

Appendix A

Basis for Maximum Head Differential for Use in Sheet Pile Wall Design

APPENDIX A

BASIS FOR MAXIMUM HEAD DIFFERENTIAL FOR USE IN SHEET PILE WALL DESIGN[1]

The Maximum water surface elevation of 6.8 feet in Table 2-2 is the historical maximum measured High-high tide level for Webb Tract along the San Joaquin River (extracted from a DSM2 model simulation). Similarly, the historical minimum measured Low-low tide level is -1.7 feet. Statistically, the High-high tide level only goes above 3.8 feet in 10% of the years. The URS Flooding Analysis report shows a USACE 300-year design flood stage of 7.2 feet. Given this information, the following operating water surface elevations should be considered for the design of the cantilevered Sheet Pile Wall:

Operating Water Levels:

Max Operating W.S. Elev. (Transition Pool): 7.2 feet

Max Operating W.S. Elev. (Bypass Channel): 7.2 feet

Min Operating W.S. Elev. (Transition Pool): -1.7 feet

Min Operating W.S. Elev. (Bypass Channel): -1.7 feet

Note: Both the bypass channel and the transition pool are connected to the adjacent river channel at all times, so the two areas will always be at the same water surface elevation (unless dewatering the transition pool).

Maintenance Water Levels

It is assumed that there will never be a need to dewater the bypass channel because maintenance dredging could be accomplished from the adjacent integrated facility embankment. Maintenance dredging of the transition pool may be accomplished by drag-line dredging or other known methods. If necessary, the transition pool would only be partially dewatered for maintenance dredging, which would require stop logs to be placed in all fish screen bays. It is assumed that the transition pool would be partially dewatered (down to approx. -10 feet), leaving approximately 4 feet of water depth in the transition pool. This would never be done in the middle of winter or during a flood event. Instead, partial dewatering would occur when Delta channel water levels are at their lowest, which is during the summer months when water levels are at a maximum of +5 feet during High-high tide (although exceeding +4 feet in only 4% of the years and averaging +3 feet). For purposes of designing the cantilevered sheet pile wall, the maximum head differential occurs when the maximum water surface elevation in the bypass channel is assumed to be +4 feet and the minimum water surface elevation in the transition pool is assumed to be -10 feet, for a maximum head differential of 14 feet.

---

[1] Provided by DWR.

Appendix B

Summary of Structural Design Analysis Results

(April 2003)

Table 4-1
Summary of Structural Design Analysis Results (April 2003)

BOX CULVERT STRUCTURES

Element	Thickness (ft)	Main Reinf. Ratio	No. Piles
Roof slab	1.5	0.007	9 piles 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~~ 280 piles total (corrected)
	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) Main Reinf. Ratio	Invert Slab		No. Piles per 100 sf
			Thickness (ft)	Reinf. Ratio	
9	1.7	0.003	2.5	0.008	4
28	3	0.011	3	0.011	4

Table 4-1 (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