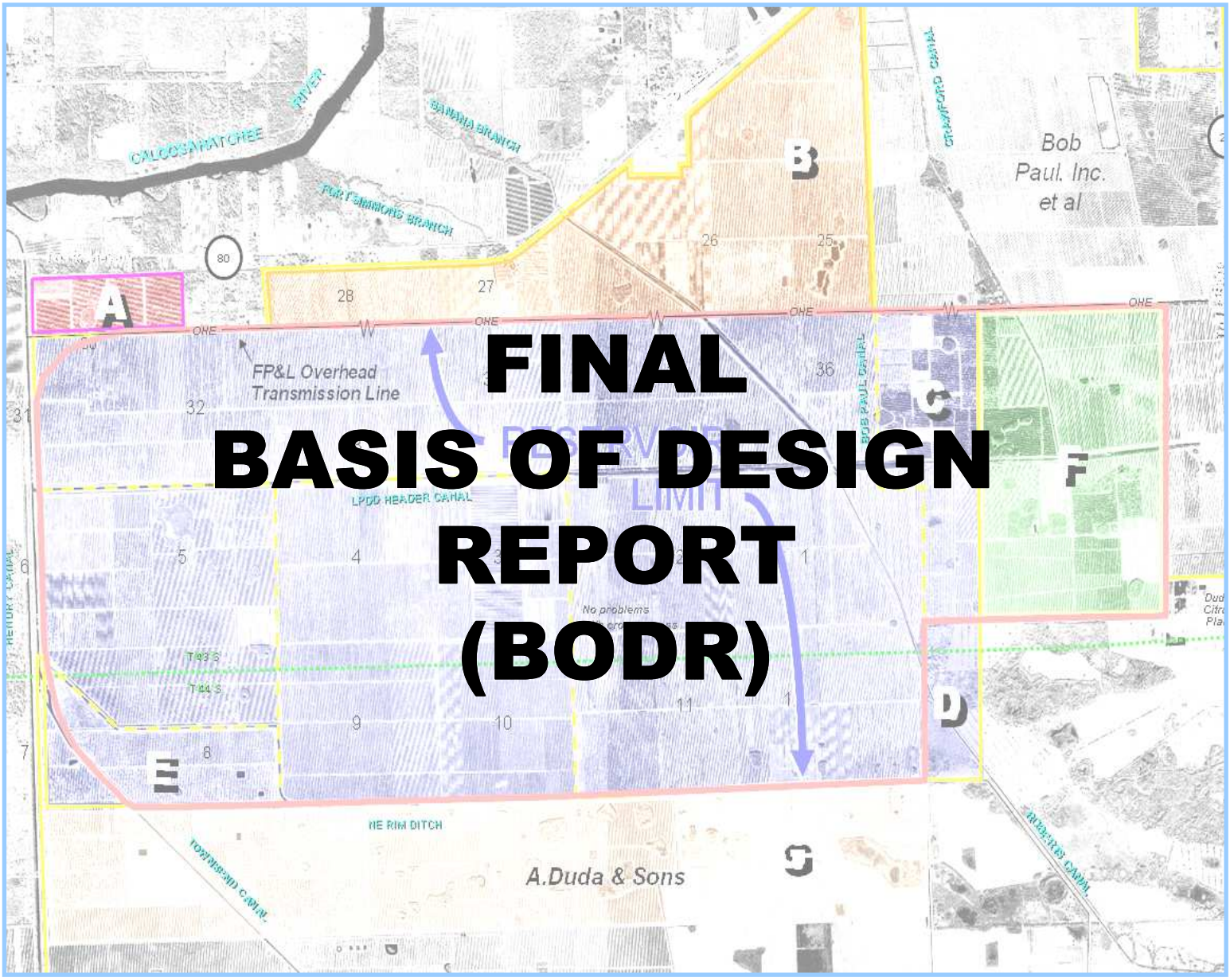
C-43 BASIN STORAGE RESERVOIR

South Florida Water Management District
Contract C-C10401P - W.O. 12



FINAL BASIS OF DESIGN REPORT (BODR)

Stanley Consultants INC.

APRIL 2006

TABLE OF CONTENTS

Executive Summary **1**

Section 1 – Introduction **8**

- **A. Authorization for BODR** 8
- **B. Purpose of BODR** 9
- **C. Scope of BODR** 9
- **D. Project Description** 9
  - 1) Purpose 9
  - 2) Project Scope 10
  - 3) Existing Facilities 12
  - 4) Proposed Facilities 12
- **E. Other Studies, Reports, and Projects** 17

Section 2 – Site Conditions **19**

- **A. Climate** 19
  - 1) Existing Conditions 19
  - 2) Potential Impacts 19
- **B. Land Use** 20
  - 1) Existing Conditions 20
  - 2) Potential Impacts 20
- **C. Geology and Soils** 20
  - 1) Regional 20
  - 2) C-43 West Storage Reservoir Site 22
- **D. Seismicity** 23
- **E. Surface Water Hydrology** 23
- **F. Ground Water Hydrology** 28
- **G. Environmental Conditions** 29
  - 1) Air Quality 29
  - 2) Noise 29
  - 3) Vegetation and Wetlands 30
  - 4) Fish and Wildlife 33
  - 5) Protected Species 34
  - 6) Water Management 37
  - 7) Water Quality 37
  - 8) Socioeconomics 39
  - 9) Recreational Resources 40
  - 10) Aesthetics 41
  - 11) Cultural/Archaeological Resources 41
  - 12) Hazardous, Toxic and Radioactive Waste 42
- **H. Existing Conditions** 46
  - 1) FP&L Transmission Lines 46
  - 2) FP&L Overhead Distribution Lines 47
  - 3) Local Irrigation Systems 47

4) Communications Utilities ... 47
I. **Surveying and Mapping** ... 48
1) General ... 48
2) Survey ... 48
3) Mapping ... 49

Section 3 – General Design Requirements ... 51
A. **Project Limits and Site Datum** ... 51
B. **Functional and Operational** ... 51
1) Lake Okeechobee Releases (Historical and Planned) ... 51
2) Irrigation Requirements (Historical and Planned) ... 52
3) Flood Flow Attenuation ... 53
4) Water Quality ... 54
5) Water Release Volume and Timing ... 55
C. **Service Life** ... 56
D. **Project Work Limits** ... 57
E. **Units and System of Measurement** ... 58
F. **Codes and Standards** ... 58
1) Site Work Design Criteria ... 58
2) Geotechnical Design Criteria ... 58
3) Architectural Design Criteria ... 59
4) Structural Design Criteria ... 60
5) Special Mechanical Equipment Design Criteria ... 63
6) HVAC, Plumbing and Fire Suppression ... 65
7) Plumbing Design Criteria ... 66
8) HVAC Design Criteria ... 67
9) Fire Protection and Detection Design Criteria ... 68
10) Electrical Design Criteria ... 69
11) Instrumentation and Controls Design Criteria ... 69
12) Telemetry System Design Criteria ... 71

Section 4 – Regulatory Considerations ... 72
A. **Federal Permits** ... 72
B. **State Permits** ... 72
C. **Local Permits** ... 75
D. **Other Considerations** ... 75
E. **Land Use/Zoning** ... 75
F. **PIR Coordination & NEPA EIS Coordination** ... 76

Section 5 – Hydrology ... 79
A. **Design Criteria** ... 79
B. **Hazard Classification** ... 79
C. **Design Storms and Floods** ... 80
D. **Reservoir** ... 81
1) Reservoir Sizing and Yield ... 82
2) Stage-Discharge ... 83
3) Inflow Design Flood(s) ... 83
4) Freeboard ... 83

Section 6 – Hydraulics 92
A. Design Criteria 92
1) Inflow pumps 93
2) Outflow Gates 94
3) Perimeter Canal 94
4) Routing of Flood Flows 94
5) Interior Cell Design of the Reservoir 95
6) Maintenance of Irrigation Flows to Agricultural Operations 95
B. Pump Requirements for the Reservoir Alternative B, Option 1 95
1) Tests Using 2000 Land Use 95
2) Tests Using 2050 Land Use 96
C. Pump Requirements for Alternative B, Option 2 102
D. Water Control Structures 108
1) Inlets/Outlets - Operations 108
2) Emergency Evacuation Requirements 111
3) Perimeter Canal 114
4) Impacts in Roberts Canal South of the Proposed Reservoir 120
5) Impact of the Proposed Project on S.R. 29 South of Labelle 122
6) Reservoir Seepage and Evaporation Losses 124
E. Canals/Culverts/Flow-Ways 125
1) Water Surface Profiles 125
2) Head Loss 127
3) Flows and Velocities 127
4) Energy Dissipation 127
F. Water Quality 129
G. Erosion Control 134

Section 7 – Subsurface Conditions 136
A. Insitu Materials 136
1) Geophysical Setting 136
2) Geotechnical Exploration 136
3) Stratigraphy 138
4) Laboratory Test Results 139
B. Groundwater 141
1) Groundwater 141
2) Field Permeability Tests 142
C. Excavations 143
D. Borrow 143
E. Foundations 145
F. Pipelines/Trenches 145
G. Design Parameters 145

Section 8 – Dam & Reservoir Design 148
A. Design Criteria 148
B. Slope Stability 150
C. Foundations/Bearing Capacity 152
D. Seepage Control 152

E. Settlement 154
F. Insitu Materials 155
G. Borrow 155
H. Dam Design 155
1) Perimeter Dam 155
2) Drainage Blanket 156
3) Interior Dam 157
I. Erosion Protection 158

Section 9 – Levees 160

Section 10 – Canals 161
A. Stability 161
B. Seepage Control 165
C. Erosion 165

Section 11 – Structural Design 166
A. Design Criteria 166
1) Loads and Forces 166
2) Stability 169
3) Vibration 169
4) Reinforced Concrete Design 170
5) Structural Steel Design 171
6) Aluminum Design 171
B. Water Control Facilities 172
1) Outlet Structures 172
2) Spillways/Weirs 175
3) Gates/Valves 178
4) Pump Stations 179
C. Culverts 179
1) Structures S-12 & S-13 179
2) Structure S-14 180
D. Control/Operations Buildings 180
E. Miscellaneous Structures 180
1) S.R. 80 Bridge over the Townsend Canal 180
2) Monitoring Stations 181

Section 12 – Site Civil Design 182
A. Site Access and Roadways 182
B. Site Layout 183
1) General Discussion 183
2) Alternate Reservoir Configurations 183
3) Dam Geometry 187
4) Canals 189
C. Stormwater Control/Site Drainage 191
1) During Construction 191
2) Permanent Construction 191
D. Utilities 192

1) Power lines ... 192
2) Local Irrigation System ... 193

Section 13 – Mechanical Design ... 194

A. Design Criteria ... 194
1) Pump Station 1 (PS-1) ... 194
2) Pump Station 2 (PS-2) (optional) ... 194
3) Major Equipment ... 195

B. Pump Station Discussion and Alternatives Comparison ... 201
1) Overview ... 201
2) Discharge Arrangement ... 202
3) Suction Channel ... 207
4) Pump Intake ... 209
5) Pumps ... 210
6) Net Positive Suction Head ... 212
7) Startup ... 214
8) Testing ... 215
9) Other Pump Issues ... 215
10) Engines ... 215
11) Discharge Apron ... 223
12) Dewatering System ... 223
13) Comparison of Options 1 and 2 ... 223

Section 14 – Electrical Design ... 226

A. Design Criteria ... 226
1) Project Electrical Features ... 226
2) General Electrical Design Requirements ... 227
3) Lighting Design ... 229
4) Pumping Station Engineering Guidelines ... 230

B. New Overhead Primary Lines ... 230
1) Utility Service by FP&L ... 231
2) Final Routing of New Lines will be Determined by FP&L ... 231
3) Design Coordination ... 231

C. Backup Power ... 232
1) PS-1 ... 232
2) PS-2 with Engine-Driven Pumps ... 232
3) PS-2 with Motor-Driven Pumps ... 232

D. Facility Electrical Systems ... 233
1) Power Distribution ... 233
2) Motors and Controllers ... 234
3) Wiring and Wiring Devices ... 234
4) Fire Detection System ... 235
5) Lightning Protection ... 236
6) Grounding ... 236

E. Spillway, Culvert and Monitoring Station Electrical Systems ... 236

F. Seepage/Ground Water Control (Pumps and Wells) ... 237

Section 15 – Instrumentation and Controls ... 238

A. Design Criteria 238
1) Instrumentation and Control Features 238
2) Ambient Conditions 240
3) Power Supplies 240
4) Control Signals 240
5) Level Monitoring in Deep Stilling Wells 241
6) Embankment Monitoring 241
7) Water Quality 242
8) References 242
B. Control Systems 242
1) Pump Stations (PS-1 and PS-2) 242
2) Gated Spillways and Gated Culverts 243
C. Monitoring Stations 244
1) Stilling Wells 244
2) Control Enclosure 244
3) Water Level Monitoring 244
4) Instruments 244
5) Data Transmission 245
D. Remote Telemetry – Microwave Radio 245
1) General 245
2) Existing System 245
3) Radio Path Study 246
4) Linking 246
5) Telemetry System Interface 246
E. Reservoir Telemetry – Spread Spectrum Radio 248
1) Data Radio Spread Spectrum Radios 248
2) Complementary Equipment 248
3) Antennas 248
4) SCADA Interface 248
F. Design Documentation 249
1) Control System Operating Description 249
2) Graphic Screen Requirements 249
3) Process and Instrumentation Diagrams 249
4) Control Schematics 249
5) I/O listing for each PLC/RTU 249
6) Construction Specifications 250

Section 16 – Architectural 251
A. Design Criteria 251
1) Introduction 251
2) Design Requirements 251
3) Life Safety 252
4) Material and Life Cycle 252
B. Exterior Architectural Features 252
1) Shell 252
C. Interior Architectural Features 253
1) Floor 253

2) Partitions 254
3) Interior Doors and Windows 254
4) Interior Finishes 254
5) Vertical Circulation 254
6) Interior Fixtures 255

Section 17 – HVAC, Plumbing and Fire Suppression 256
A. Design Criteria 256
B. HVAC 258
C. Potable Water 260
D. Sanitary System 260
E. Storm Water System 260
F. Fire Suppression System 260

Section 18 – Special Considerations 262
A. Environmental 262
B. Recreation 263
C. Access 265
D. Security 266
1) Fences and Gates 266
2) Site Monitoring, Pump Stations 266
3) Building Access, All Buildings 266

Section 19 – Public Impacts and Issues 268

Section 20 – Operation & Maintenance 269
A. Startup and Operation 269
B. O&M Manuals 270
C. Coordination Objectives 270

Section 21 – Value Engineering Reviews 272

Section 22 – Opinion of Probable Cost 273
A. Introduction 273
1) Perimeter Dam 273
2) Pump Stations 273
3) Control Structures 274
4) Mobilization and Contingencies 274
B. Cost as Function of Dam Height 275
1) General 275
2) Chart Criteria 276
C. Cost Model Development 278
1) Perimeter Dam 278
2) Control Structures 282
3) Pump Stations 282
D. Alternative A, Two-Cell Alternative Costs 283
1) Dam Embankment and Seepage Canal Costs 283
2) Control Structures 284
3) Pump Stations 285

E. Alternative B, Three-Cell Alternative Costs 286
1) Dam Embankment and Seepage Canal Costs 286
2) Control Structures 287
3) Pump Stations 288
F. Seepage 289
1) General 289
2) 2-Cell Option 289
3) 3-Cell Option 290
4) Slurry wall 290
G. Summary 290

Section 23 – Construction Coordination 291
A. Introduction 291
1) General 291
2) Perimeter Dam 291
3) Pump Stations 292
4) Control Structures 292
B. Cost Estimating and Budgeting 293
C. Constructability 293
D. Quality Assurance 293
E. Partnering 293
F. Construction Scheduling 294
G. Risk Management 294
H. Construction Permitting 294
I. Construction Sequencing 294
1) General 294
2) Clearing and Grubbing 295
3) Borrow Pits 295
4) Perimeter Dam 296
5) Pump Stations 296
6) Control Structures 296
J. Staging Areas 297
1) Introduction 297
2) Perimeter Dam 297
3) Pump Stations 297
4) Control Structures 297
K. Construction Access and Traffic Routing 298
L. Disposal 298

Section 24 – Emergency Action Plan 299
A. Dam Break Analysis for the Reservoir 299

Section 25 – Draft DDR Comments and Responses 307

Executive Summary

The Program Management Plan of the Central and Southern Florida Project Comprehensive Everglades Restoration Plan states:

> "The purpose of the C-43 Reservoir Project is to improve water deliveries to the estuary, provide for dry season flows, restore downstream salinity levels and ensure the availability of water for the natural system needs of the Caloosahatchee Estuarine System. Improvements in the distribution of water should result in improvements to the timing, quality and quantity of water deliveries to the inland ecosystems. Once the demands of the estuary are met, additional water resource benefits could be achieved by providing supplemental water resources for agriculture and urban users resulting in a reduction of demands placed on existing surface and ground water resources such as the C-43 Canal and Lake Okeechobee". [1]

The success of the C-43 West Storage Reservoir (Reservoir) project will, for the most part, be judged by how well the performance measures are met. The primary performance measure for purposes of this report is a more manageable flow range over the Franklin Lock (S-79) Structure on the Caloosahatchee River (C-43 Canal). A list of specific performance measures is provided in Section 6 of this report.

The stakeholders have been involved in the C-43 Basin Project Implementation Report (PIR). Public safety, during and after the construction of this project, must be of overriding importance. The Reservoir has been conceptually designed based on the latest design guidelines and standards referenced in the technical sections of this report. The conceptual Reservoir design has been developed to withstand hurricane force winds of over 150 mph. The Reservoir has also been designed to withstand combinations of both wind and rain. Any design criteria that is a result of the "Safe Impoundment Design Acceler8", Design Criteria Workshop, that is not currently included, will be incorporated into the final design. It can be stated that the goal of the Reservoir design is to prevent failure and endangerment of life.

The Reservoir presented in this Basis of Design Report (BODR) is one alternative site from a list of potential alternative sites within the C-43 Basin. It is contemplated that additional reservoir sites may be selected as a result of the C-43 Basin Storage Reservoir Report. It should be recognized that the Reservoir will be unable to meet all storage requirements for the entire C-43 Basin as currently proposed. Therefore, the projected flow management at the S-79 Structure will be further enhanced with the addition of other storage reservoirs within the C-43 Basin.

Section 1D 4) – Proposed Facilities of this report provides a comprehensive summary of the two alternatives under consideration for the Reservoir. The following is a brief overview of the alternatives:

[1] Central and Southern Florida Project Comprehensive Everglades Restoration Plan, Program Management Plan

Alternative A - Two-Cell Alternative

Alternative A is a two-cell Reservoir surrounded by a Perimeter Canal with two options for pump station configurations as indicated on **Figure 1D-03**, which is located at the end of the Executive Summary.

Option 1 uses a single pump station, PS-1, which is located in the northwest corner of the Reservoir. All water pumped to fill the Reservoir will be pumped from the Townsend Canal utilizing PS-1, which has a pumping capacity of 1,500 cfs. Locating the pump station in the northwest corner minimizes improvements necessary for the Townsend Canal.

Option 2 uses two pump stations, PS-1 and PS-2. PS-1 is again located in northwest corner of the Reservoir and has a capacity of 1,250 cfs. PS-2 is located in the southeast corner of the Reservoir. PS-2 has a pumping capacity of 500 cfs. PS-2 takes advantage of the higher water surface elevation in the Perimeter Canal at its location and thus the lower head requirement provides greater efficiency for this pump station as compared to PS-1.

Alternative B - Three-Cell Alternative

Alternative B is a three-cell Reservoir. The third cell is created by adding property to the east of the two-cell alternative and is currently property that is under the ownership of Bob Paul, Inc. The three-cell alternative is indicated by **Figure 1D-04** located at the end of the Executive Summary. Alternative B also has two pump station configuration options.

Option 1 uses a single pump station, PS-1, and Option 2 uses two pump stations, PS-1 and PS-2. The pump station capacities, locations and operating conditions are identical to Alternative A.

General Description of Alternatives

The two proposed alternatives are a result of many "footprint" configurations, economic and operational analysis, as well as consideration of social and environmental issues. In general, these two current alternatives include the following:

- The alternatives include a two-cell or a three-cell Reservoir with a "normal pool" storage capacity of 170,000 acre-feet. Additional capacity has been included in the conceptual design for contingency purposes. The interim Reservoir capacity of 170,000 acre-feet has been provided at this conceptual stage to allow for refinements in the design that will reduce capacity, while still providing at least 160,000 acre-feet of above ground storage. The final determination of the reservoir capacity will be determined during the 30% preliminary design. The two alternatives under consideration have been developed as a result of the analysis of numerous Reservoir sizes and cell configurations and represent the most economical options. Reservoir

capacity is further discussed in item 6) of the Recommendations, on page 6 of this Executive Summary.

- The water surface elevation is constant within the cells for each alternative. In Alternative A the water surface elevation is 39.2 ft. at "normal pool". In Alternative B the water surface elevation is 37.6 ft. at "normal pool". The maximum depths for both Alternatives occur in Cell 1. The average depth in Cell 1 for Alternative A is 19.1 ft. and for Alternative B it is 17.4 ft.

- A Perimeter Canal (seepage canal) is provided on the south, east, and north sides of the Reservoir. The existing Townsend Canal will act as a seepage canal on the west side of the Reservoir. In addition to seepage collection, the Perimeter Canal will also convey discharges from the Reservoir; convey surface runoff from the south and east of the Reservoir including improved drainage for S.R. 29; provide flows to the Crawford Canal, Banana Branch and Fort Simmons Branch; and maintain the surface water elevation in the expanded NE Rim Ditch that provides irrigation for the A. Duda and Sons (Duda) citrus operations, located to the south of the Reservoir.

- Improvements to the Townsend Canal are based on the necessary capacity to meet maximum pumping requirements to fill the Reservoir. Pre-storm and post-storm event flows have been run to analyze flows in the Townsend Canal and other local canals and tributaries. Storm event flows in the Townsend Canal will be decreased over existing conditions with the construction and operation of the Reservoir. The Reservoir outflow will be operated as needed to meet minimum flow requirements at the S-79 structure. The discharge gate operations in the modeled alternative call for water to be released from Cell 3 as a first priority, Cell 2 as a second priority, and from Cell 1 as a last priority. This approach would minimize the potential for circulation and increase the settling rate for water quality benefits.

- Each cell in both alternatives is designed to discharge independently through separate discharge structures (S-1 discharges Cell 1, S-8 discharges Cell 2). Structure S-5 is added for Alternative B only, to discharge Cell 3. Structure S-1 discharges into the Townsend Canal. Structures S-8 and S-5 discharge into the Perimeter Canal. These structures will be designed for incremental operation allowing required flows to be released to the C-43 Canal during periods of low flow i.e., the Reservoir outflow will be operated as needed to meet minimum flow requirements at the S-79 structure. Reservoir structure names used in the BODR are for analysis and design use only. The structure names will be formally changed and reflected in the final design and construction documents to avoid confusion with other existing structures that have the same name (e.g., S-8).

  These structures could also serve as design storm control structures for releases prior to and during a storm event. The storm releases must be balanced with the targeted maximum flow allowed over S-79, which is 4,500 cfs. An emergency spillway with a crest elevation based at the 25-year, 72-hour storm event is to be provided within each cell's discharge structure. The 25-year, 72-hour storm captured in the Reservoir

would be a "control release" after the storm event in order to restore the water surface to the normal pool elevation and not exceed 4,500 cfs at S-79.

- Structures S-12 and S-13 hydraulically connect the Reservoir Cells and are designed to be gated so that any cell can be "isolated" for operational or maintenance purposes.
- The average dam height for Cell 1 is 28.8 ft. for Alternative A, and 27.3 for Alternative B. Dam heights in other Cells are not as high.
- The project schedule is:
  - Final BODR completed in April 2006 and 30% design completed by June 2006
  - Final design and construction documents completed by April 2007
  - Start construction in July 2007
  - Construction completed by July 2010
  - Initiate operations July 2010

Table 1 is a summary comparison of the major elements of Alternative A and Alternative B with options.

Table 1 – Alternatives A and B Cost Comparisons

	No. of Cells (EA)	Pump Stations (EA)	Storage Capacity (AC-FT)	Average Depth in Cell 1 (FT)	Estimated Construction Cost [1.] (Millions)	Estimated Construction Cost per A-F Of Storage	Total Project Cost (Incl. Land To Be Purchased) [2.] (Millions)
Alternative A							
Option 1	2	1	170,000	19.1	$252.2	$1,484	$256.4
Option 2	2	2	170,000	19.1	$265.2	$1,560	$269.4
Alternative B							
Option 1	3	1	170,000	17.4	$276.9	$1,629	$294.7
Option 2	3	2	170,000	17.4	$289.1	$1,701	$306.9

[1.] The estimated construction cost is based on the recommended design elements for each alternative option.
[2.] Land not currently owned by the SFWMD is estimated at $6,500 per acre.

Recommendations

The following recommendations are made based on the analysis presented in this BODR.

1) The two-cell alternative, Alternative A, Option 1 (single pump station, PS-1) is the recommended Reservoir alternative. With the land cost (land not currently owned by the SFWMD) included in the alternative analysis and with refined topographic information, Alternative A is the most cost-effective alternative.

In addition to being the most cost-effective alternative, Alternative A has less environmental, social and economic impacts (smaller footprint and less land removed from citrus production). Alternative A, however, has a dam height that is slightly higher than Alternative B by approximately 1.7 ft.

Providing a single pump station saves approximately $13 million in total project costs. This savings is the primary reason for recommending Option 1. Any operational advantage of adding PS-2 does not justify an additional annualized cost of approximately $550,000 per year.

Although Option 2 costs considerably more than Option 1 it may have significant long-term advantages that may outweigh initial construction cost. Option 2 provides more pumping flexibility and pumping redundancy and gains the benefit of pumping with less head. Option 2 allows pumping into Cell 2 with PS-1 "down" and/or Cell 1 closed for maintenance. A detailed life cycle cost will be completed in the design phase to fully evaluate operation and maintenance costs.

PS-1 is natural gas powered and PS-2 is electric powered. This dual energy source provides for an operational safe guard. The applicable codes and standards for natural gas energy use will be reviewed and addressed during the 30% preliminary design.

It should also be noted that for Alternative A, parcel A1 (34 acres), parcel D1 (156 acres) and parcel I (456 acres) are not currently owned by the SFWMD. Please refer to **Figure 1D-01** at the end of the Executive Summary for the location of these parcels.

2) It is recommended that Pump Station (PS-1) be constructed as a 1,500 cfs pump station with two 500 cfs and two 250 cfs pumps. Please refer to **Figures 13B-01 and 13B-02** for the proposed plan and section at PS-1 located at the end of the Executive Summary.

   a) The discharge arrangement is to be In-the-Embankment with Check Valves. The cost comparison found three different arrangements to be very similar in life cycle cost. The In-the-Embankment with Check Valves arrangement is recommended based on shorter structure height, smaller footprint, and ease of construction.

   b) The pump engines are to be natural gas powered, based on the significantly lower fuel cost. It is recommended that SFWMD formalize an agreement with the gas utility so that design can continue utilizing natural gas engines.

   c) The pump station building shall be cast in place concrete walls with pre-cast panel construction. Further evaluation of construction method shall be performed during the 30% preliminary design phase.

d) The pump intakes are to be suction bell. The suction bell intake (K=0.04) is more efficient that the FSI (K=0.15). The siphonic recovery has lower headloss and fuel cost than the check valve arrangement, but the life cycle cost advantage remains with the proposed arrangement due to lower capital costs. The pump station valve type and arrangement design will be further evaluated in the 30% preliminary design.

3) Three structures are recommended to be gated spillway type structures (S-3, S-4, and S-15). Structures S-6, S-7, S-9, S-10, and S-11 will be weir structures. Structures S-1, S-2, S-5, S-8, S-12, and S-13 will be variations of gated culvert structures. It should be noted that S-12 and S-13 will include more than a single-gated structure at the location indicated. S-14 will be a free-flow culvert. Refer to **Figures 1D-03 and 1D-04** at the end of the Executive Summary for structure locations. All gated type structures include provision for maintenance dewatering.

4) The exterior and interior dams are to have a top width of 15 ft. The side slopes are 3 Horizontal: 1 Vertical (3H: 1V). The exterior dams are to be constructed from existing on-site material and have a low permeability core. A graded filter is to be provided on the exterior side of the dam to direct seepage. Soil cement protection is to be included on the interior side of all Reservoir slope faces (both sides on the interior separation dams) to prevent surface erosion. Refer to **Figures 8A-1 and 8A-2** at the end of the Executive Summary for the dam typical sections.

5) The freeboard for Cells 1 and 2 for both alternatives is approximately 10 ft. The freeboard for Cell 3 is approximately 7 ft. These freeboard heights are based on very detailed analyses of wind and wave height run-up. The freeboard for the interior dams has been set at 5 ft. The interior dam is protected with soil cement to prevent erosion, because it is recognized that at times water will "spill over" the interior dams. In addition to reducing the freeboard, the low permeability core and the graded filter have not been included in the interior dam design as these items are not required and construction costs can be reduced.

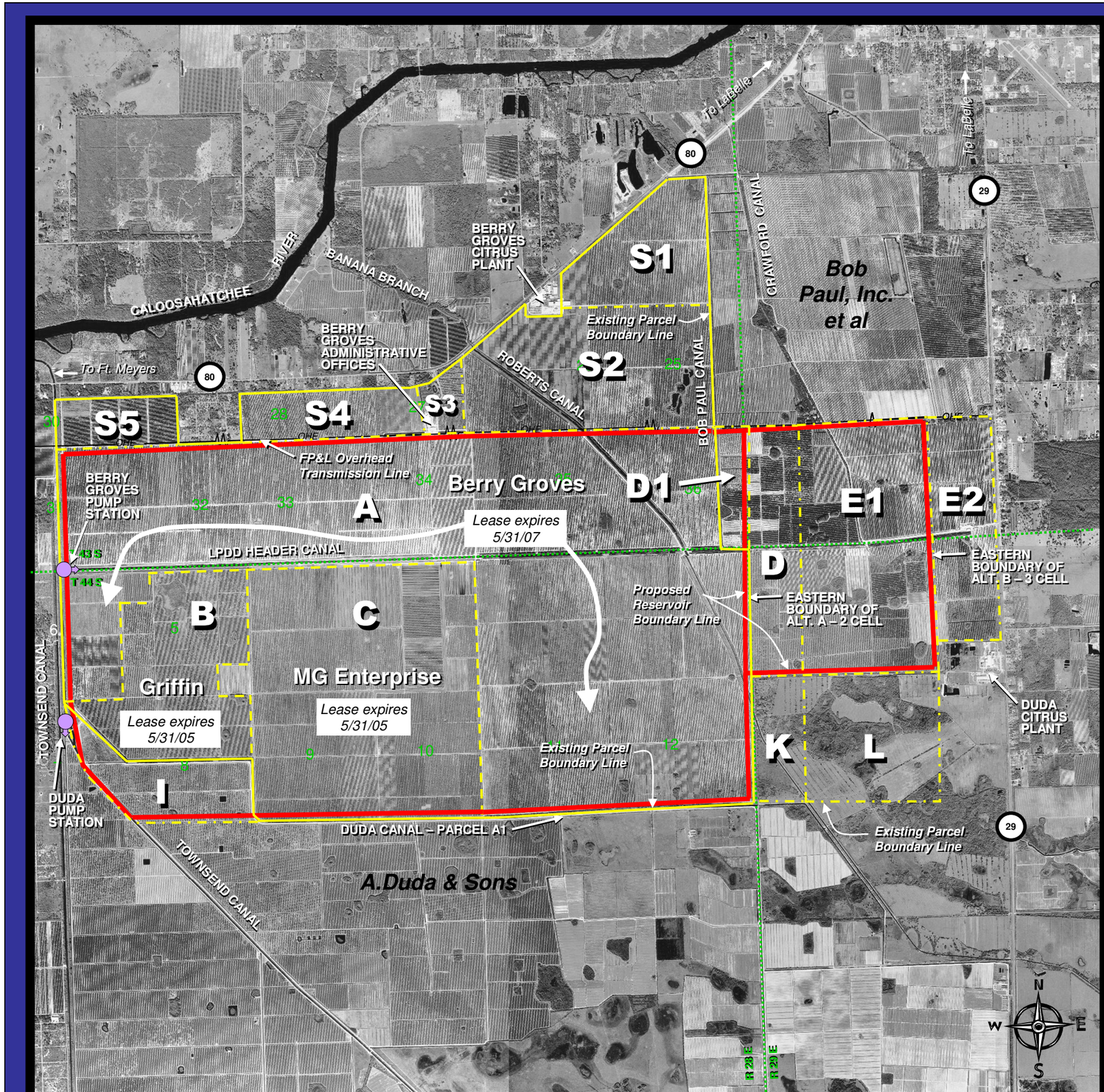
6) Both Alternative A and Alternative B are currently designed to contain 170,000 acre-feet of storage, which is in excess of the 160,000 acre-feet of storage specified in the project scope. It has been assumed that there may be some loss of storage volume during engineering refinements and therefore additional volume has been provided at this conceptual level of design.

Additional volume can be achieved by increasing the height of the dams. As indicated in **Section 22,** the cost per acre-foot of storage within the selected footprint can be further reduced by increasing the height of the dam. An approximate additional 10,000 acre-feet of storage can be provided for each additional foot of dam height. **Section 22** also provides an approximate total project cost for various storage volumes.

A decision needs to be made as to the Reservoir storage volume to be designed. Is the storage to be 160,000 acre-feet, or in excess of this amount? This is a District decision that should be made before the design moves forward. This decision should consider the analysis contained in this report, recommendations from the PIR (USACE is the lead agency), stakeholder interests and concerns, and the overall project budget. Alternative A can certainly be "engineered" to contain more than 160,000 acre-feet.

The reservoir has not been optimized based on storage volume requirements for the entire basin. The reservoir can be designed to store additional volume for less cost per acre-foot.

As presented in **Section 22,** the cost per acre-foot of storage decreases with the increase of storage volume.



C-43 West Storage Reservoir

AREA SUMMARY

Parcel A (SFWMD)	(GX100-001)	6,200 acres
Parcel B (SFWMD)	(QD100-001)	955 acres
Parcel C (SFWMD)	(QD100-005)	2,399 acres
Parcel D (Paul)	(GX100-009)	444 acres
Parcel E1 (Paul)	(GX100-015)	1,650 acres
Parcel E2 (Paul)		475 acres
Parcel I (Duda)	(QD100-006/009)	456 acres
Parcel K (Duda)	(QD100-008)	230 acres
Parcel L (Duda)	(QD100-007)	770 acres
Parcel A1 (Duda)		34 acres
Parcel D1 (Paul)		156 acres
Parcel S1 (SFWMD)	(GX100-008)	541 acres
Parcel S2 (SFWMD)	(GX100-013)	1,165 acres
Parcel S3 (SFWMD)	(GX100-012)	110 acres
Parcel S4 (SFWMD)	(GX100-011)	366 acres
Parcel S5 (SFWMD)	(GX100-010)	237 acres

Legend

- Proposed Reservoir Boundary
- Existing Parcel Boundary Line
- Existing Parcel Boundary Line
- A — Parcel Designation

Land Map & Existing Facilities

SOUTH FLORIDA WATER MANAGEMENT DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
PHONE: 561-686-8800
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406

Stanley Consultants co.

C-43 WEST STORAGE RESERVOIR
LAND MAP & EXISTING FACILITIES

CONTRACT NO.
C-C10401P

FIGURE
1D-01
Revised

C-43 WEST STORAGE RESERVOIR

ALTERNATIVE A – OPTION 1 (YELLOW)

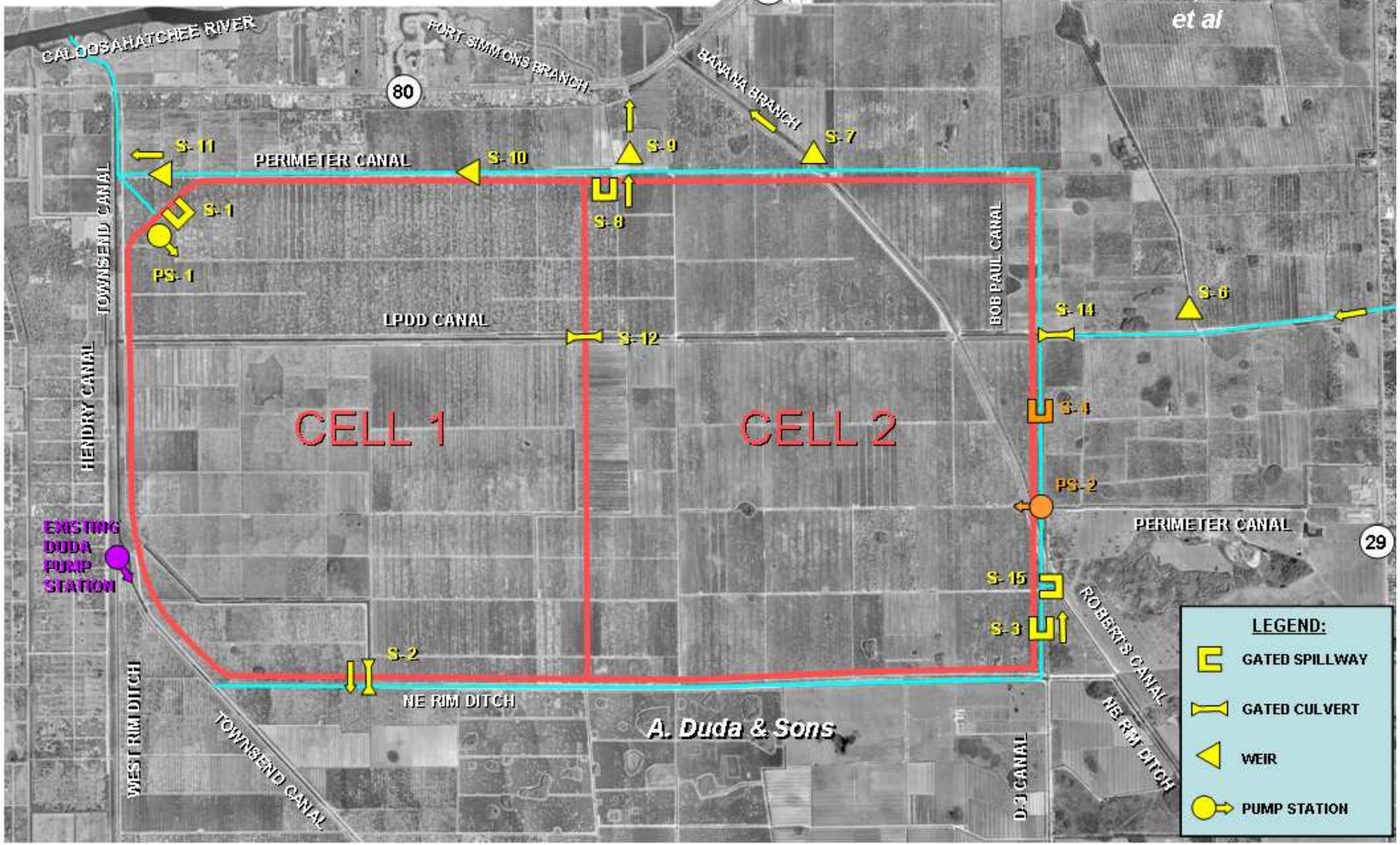
ALTERNATIVE A – OPTION 2 (YELLOW & ORANGE)

JANUARY, 2005

N W E S

Stanley Consultants INC.

Scale: NONE



SOUTH FLORIDA WATER MANAGEMENT DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
PHONE: 561-686-8800
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406

Stanley Consultants co.

C-43 WEST STORAGE RESERVOIR
ALTERNATIVE A – OPTION 1 & 2

CONTRACT NO.
C-C10401P

FIGURE
1D-03
Revised

C-43 WEST STORAGE RESERVOIR

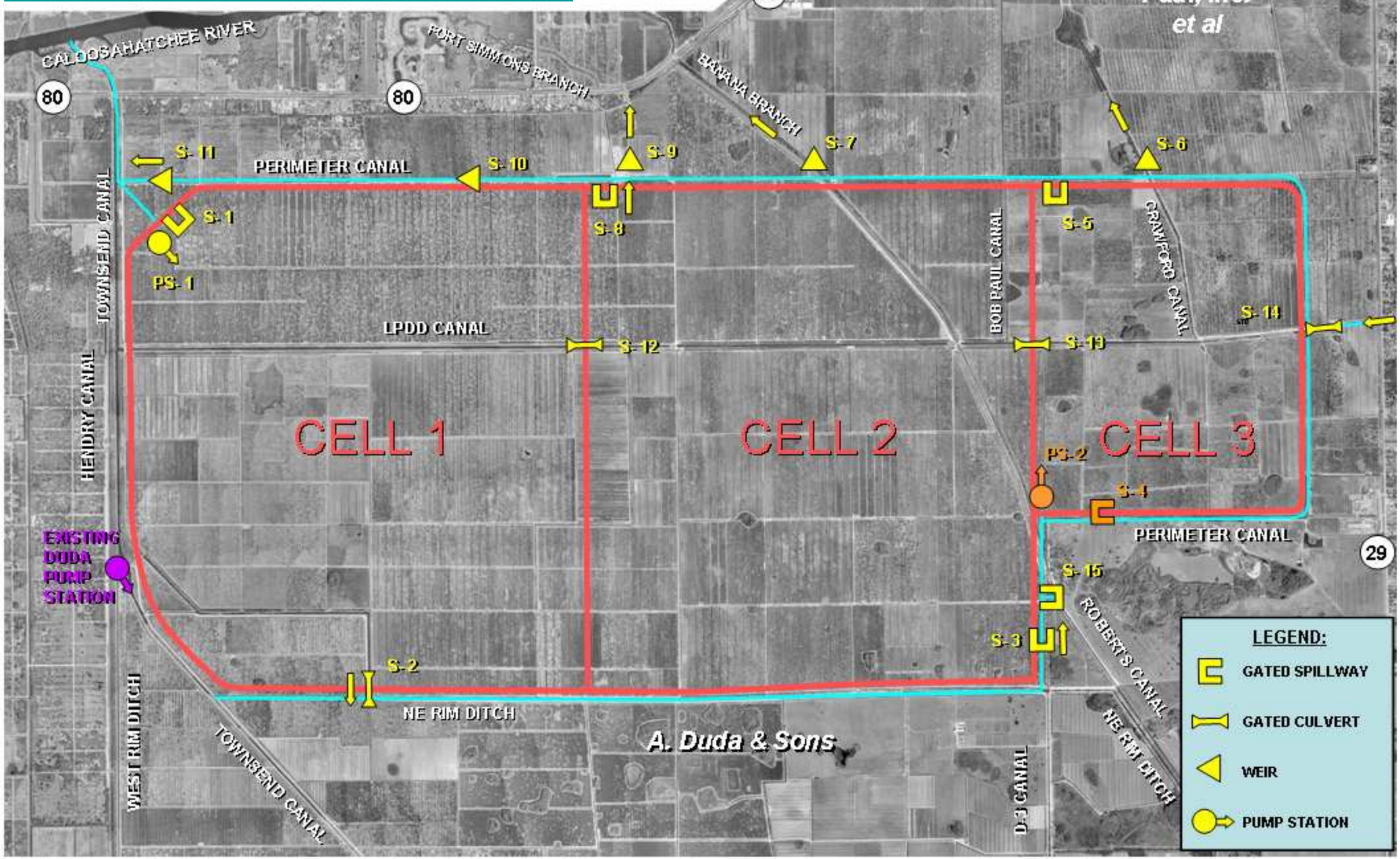
ALTERNATIVE B – OPTION 1 (YELLOW)

ALTERNATIVE B – OPTION 2 (YELLOW & ORANGE)

JANUARY, 2005

Scale: NONE

Stanley Consultants INC.



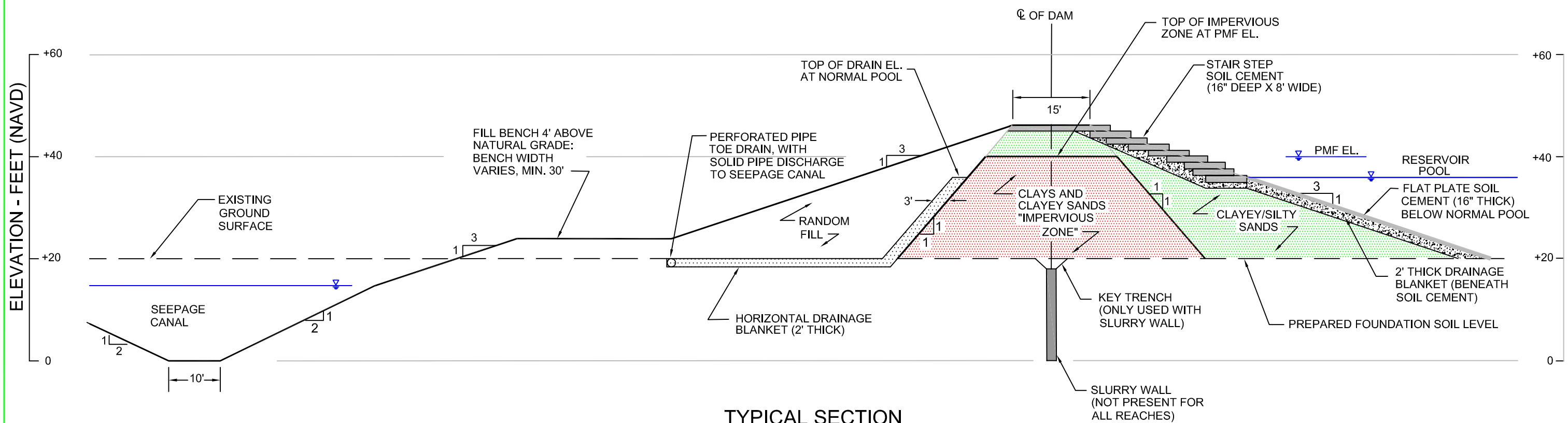
SOUTH FLORIDA WATER MANAGEMENT DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
PHONE: 561-686-8800
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406

Stanley Consultants co.

C-43 WEST STORAGE RESERVOIR
ALTERNATIVE B – OPTION 1 & 2

CONTRACT NO.
C-C10401P

FIGURE
1D-04
Revised



TYPICAL SECTION

0' 10' 20' 40'

1"= 20' H & V

3 CELL OPTION

ZONE	DISTANCE (MILES)	BENCH (FEET)	ELEVATIONS (FEET-NAVD)				
			GROUND	CANAL BOTTOM	CANAL WATER	TOP OF DAM	POOL
A1	1.7235	50	+20	0	+10	+47	+37.4
A2	3.5795	30	+20	0	+16	+48	+37.4
B	4.9242	30	+25	0	+16	+45	+37.4
C	5.5871	50	+25	+5	+23	+48	+37.4
D	2.4621	150	+20	-5	+2	+47	+37.4
E	1.0000	30	+25	0	+16	+48	+37.4

2 CELL OPTION

ZONE	DISTANCE (MILES)	BENCH (FEET)	ELEVATIONS (FEET-NAVD)				
			GROUND	CANAL BOTTOM	CANAL WATER	TOP OF DAM	POOL
A1	1.7235	50	+20	0	+10	+49	+39.4
A2	3.5795	30	+20	0	+16	+50	+39.4
B	0.0000	-	-	-	-	-	-
C	5.5871	50	+25	+5	+23	+50	+39.4
D	2.4621	150	+20	-5	+2	+49	+39.4
E	2.9167	50	+25	0	+16	+50	+39.4

C-43 WEST STORAGE RESERVOIR
TYPICAL SECTION - PERIMETER DAM

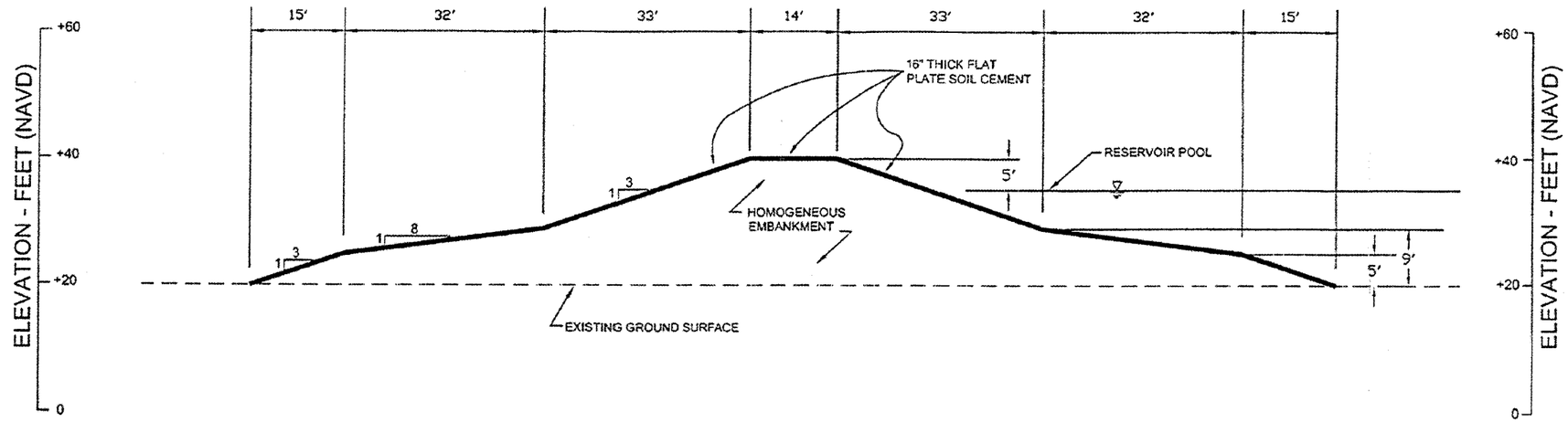
REVISION DESCRIPTION	DRAWN	REV.#	DATE
PER BUDR COMMENTS 2998	GD	1	4-20-06

ENGINEER: JAIME VELEZ
DRAWN: GARY D.
CHECKED: KEVIN AUBRY
DATE: 1-14-05
SCALE: 1"=20'

SOUTH FLORIDA WATER MANAGEMENT DISTRICT
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406-4680

GCME DEεI
6917 VISTA PARKWAY NORTH
SUITE# 10
WEST PALM BEACH, FL 33411
1225 OMAR ROAD
WEST PALM BEACH, FL 33405

FIGURE 8A-1



TYPICAL SECTION- INTERIOR DAM

0' 10' 20' 40'

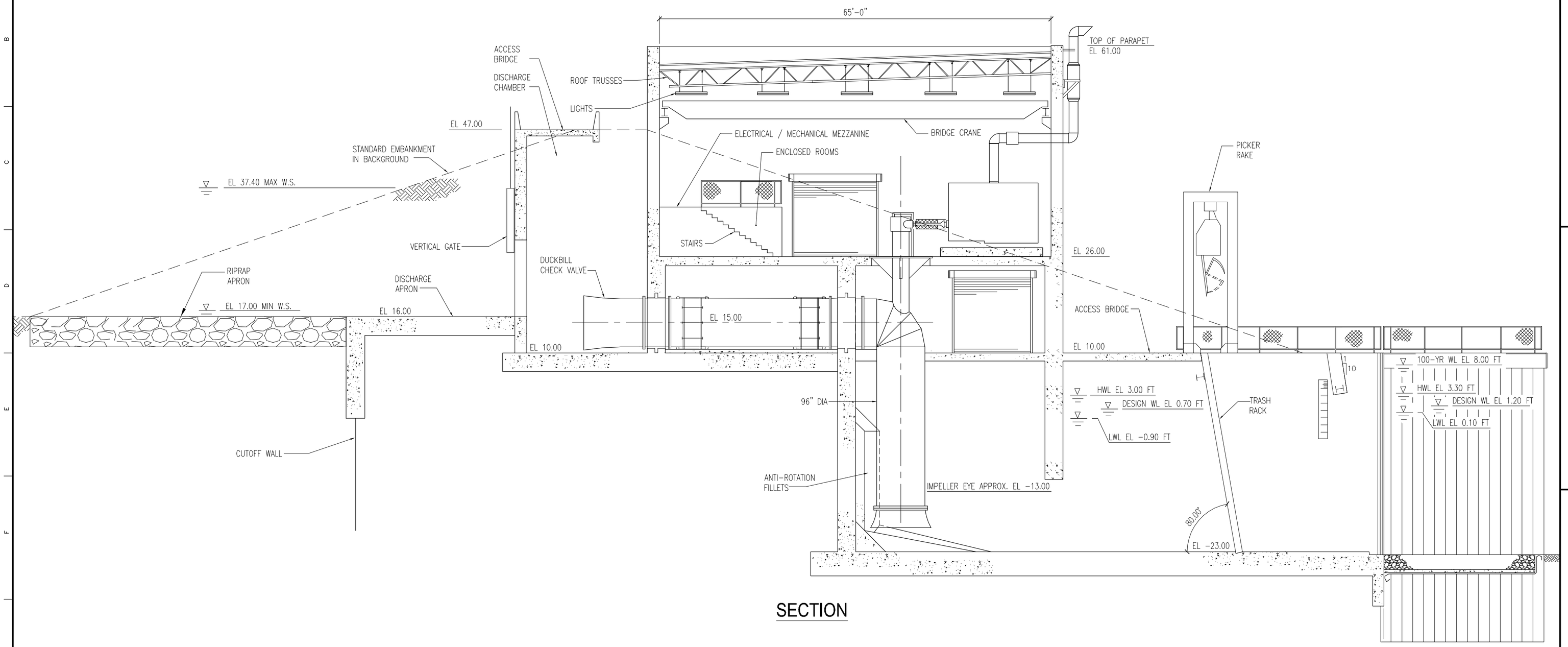
1"= 20' H & V

C-43 WEST STORAGE RESERVOIR
TYPICAL SECTION - INTERIOR DAM

SOUTH FLORIDA
WATER MANAGEMENT DISTRICT
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406-4680

GCME DE&T
6917 VISTA PARKWAY NORTH SUITE# 10 WEST PALM BEACH, FL 33411
1225 OAAK ROAD WEST PALM BEACH, FL 33405

FIGURE
8A-2



SECTION

FINAL BODR SUBMITTAL
APRIL 2006

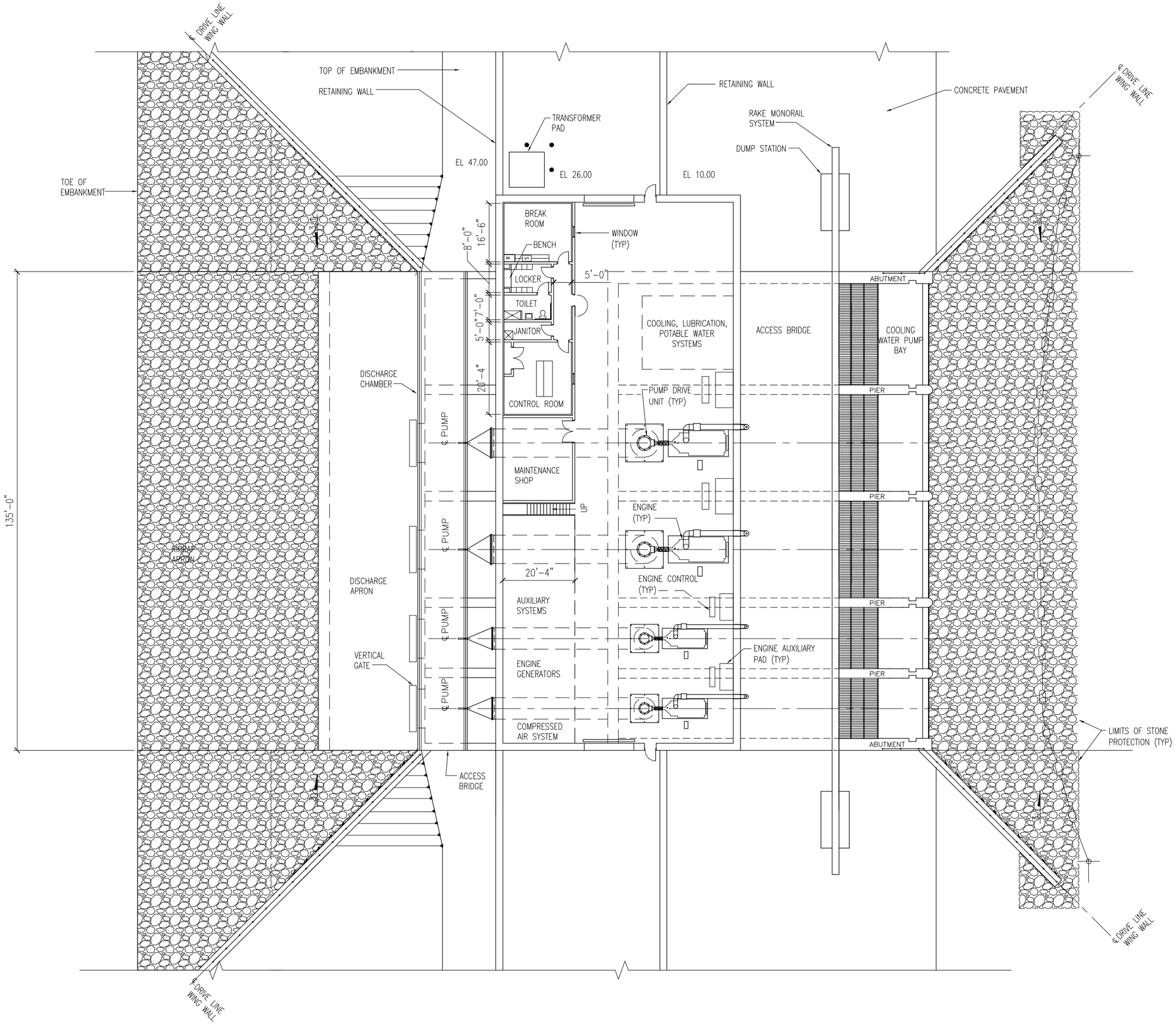
SOUTH FLORIDA WATER MANAGEMENT DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
PHONE: 561-686-8600
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406

Stanley Consultants INC.

C-43 WEST STORAGE RESERVOIR
PUMP STATION 1
CHECK VALVE ARRANGEMENT

CONTRACT NO.
C-C10401P

Figure
13B-01



PLAN

FINAL BODR SUBMITTAL
APRIL 2006

SOUTH FLORIDA WATER MANAGEMENT DISTRICT
ENGINEERING AND CONSTRUCTION DEPARTMENT
PHONE: 561-686-8600
3301 GUN CLUB ROAD
WEST PALM BEACH, FL 33406

Stanley Consultants INC.

C-43 WEST STORAGE RESERVOIR
PUMP STATION 1
CHECK VALVE ARRANGEMENT

CONTRACT NO.
C-C10401P

Figure
13B-02

Section 1 – Introduction

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 1 of the Appendices.

A. Authorization for BODR

In August 2002, the Governing Board of the South Florida Water Management District (SFWMD) approved the award of the Reservoir – Part 1 Project Implementation Report (PIR) A/E Support Contract to Stanley Consultants, Inc. (Stanley). The PIR will formulate, evaluate, and compare suites of alternatives such that a Selected Alternative Plan (SAP), formerly known as a tentatively selected plan, can be recommended.

Early efforts in the alternative evaluation process identified many sites to be further evaluated for use in the development of alternatives. These sites were considered in an initial screening process. The final alternative plans selected will cost-effectively meet the project goals and objectives.

The Berry Groves site, which included options with additional properties, was the highest ranking site in the site screening evaluation. These parcels include Berry Groves, Griffin Groves, and MG Enterprise Groves.

In October 2003 the SFWMD decided to proceed with the design and implementation of the Reservoir to provide early benefits to the Everglades Ecosystem. The expedited process, as formalized by the Acceler8 program in October 2004, will be performed on a dual track with the PIR process. The PIR will be led by the Corp of Engineers (the Corps-SFWMD partnership will be maintained). The SFWMD will proceed with the design and implementation of a reservoir on the parcels previously acquired by the SFWMD, the Florida Department of Environmental Protection (FDEP) and the US Department of the Interior. The SFWMD is currently performing parcel exchanges and acquisitions that will provide for a footprint for the Reservoir Project. Stanley Consultants was contracted to perform the 30% preliminary design services for the Reservoir Project.

This BODR has been developed under contract C-C104001P Work Orders WO02 through WO07 and WO12. These Work Orders include all engineering and support services to provide 30% preliminary design for the Reservoir and pump station. The Work Orders are as follows:

- Work Order 02 – Survey and Mapping Services; Authorized February 24, 2004
- Work Order 03 – Geotechnical Services; March 29, 2004
- Work Order 04 – Management; April 7, 2004

- Work Order 05 – HTRW, Environmental, Archaeological, and Economic Impact; April 14, 2004
- Work Order 06 – Telemetry, Contract Document Preparation, Constructability Review, Cost Estimates, and DDR Preparation; April 20, 2004
- Work Order 07 – Hydrology & Hydraulics, 30% Engineering Design; April 29, 2004
- Work Order 12 – Final Basis of Design Report; March 2006

The initial project scope in Work Order 06 included the preparation of a Design Development Report (DDR). This scope was been amended so that a BODR is provided in lieu of the DDR. This document represents the submittal of the Final BODR which incorporates technical comments/responses on the Pre-Final BODR. Appendix Volume III (Response to Comments) includes a list of Dr. Checks comments/responses on the Pre-Final BODR.

B. Purpose of BODR

The purpose of this BODR is to further analyze and evaluate the conceptual alternatives developed for the Berry Groves site as part of the site screening process of the C-43 Basin PIR. This analysis shall include various Reservoir configurations that ultimately will result in an efficient and cost effective Reservoir design that best meets the goals of the project.

C. Scope of BODR

The BODR analysis is to include the Berry Groves, Griffin Groves, and MG Enterprises Groves properties. All of these properties are currently owned by the SFWMD. The project will also consider Reservoir footprints that will include some property owned by A. Duda and Sons (Duda) and Bob Paul, Inc. in an effort to maximize storage volume while minimizing overall project costs. The Reservoir is to be designed to store approximately 160,000 acre-feet at normal pool elevation.

D. Project Description

1) Purpose

The C-43 Basin Storage Reservoir and Aquifer Storage and Recovery (ASR) Project purpose as provided in the 1999 Central and Southern Florida Project Comprehensive Review Study (Yellow Book) states:

> "The purpose of this feature is to capture C-43 Basin runoff and releases from Lake Okeechobee. These facilities will be designed for water supply benefits, some flood attenuation, to provide environmental water

supply deliveries to the Caloosahatchee Estuary, and water quality benefits to reduce salinity and nutrient impacts of runoff to the estuary. It is assumed that, depending upon the location of the facility and pollutant loading conditions in the watershed, the facility could be designed to achieve significant water quality improvements, consistent with appropriate pollution load reduction targets.

The Central and Southern Florida Project Comprehensive Everglades Restoration Plan (CERP) Program Management Plan identified the Yellow Book ASR component as a separate CERP project and identifies the C-43 Basin Storage Reservoir Project purpose as:

> "The purpose of the C-43 Basin Storage Reservoir Project is to improve water deliveries to the estuary, provide for dry season flows, restore downstream salinity levels and ensure the availability of water for the natural system needs of the Caloosahatchee Estuarine System. Improvements in the distribution of water should result in improvements to the timing, quality and quantity of water deliveries to the inland ecosystems. Once the demands of the estuary are met, additional water resource benefits could be achieved by providing supplemental water resources for agriculture and urban users resulting in a reduction of demands placed on existing surface and ground water resources such as the C-43 Canal and Lake Okeechobee." [2]

2) Project Scope

The C-43 West Storage Reservoir 30% Preliminary Design Scope of Work states the following:

- The C-43 West Storage Reservoir – Part 1 Project is one of the components of the CERP C-43 Basin Storage Reservoir Project and is scheduled for authorization in the Water Resources Development Act of 2006. The project as presented in the CERP is conceptual and additional detailed planning is required prior to design.

- The C-43 West Storage Reservoir – Part 1 Project is generally located in Hendry, Glades, Charlotte and Lee Counties with a total storage capacity of approximately 160,000 acre-feet. The concept presented in the CERP assumed 20,000 acres with water levels fluctuating up to eight feet above grade. However, the land area available will require 10 to 15 ft. of water storage depth.

- The purpose of the Project as defined in the CERP is to capture C-43 Basin runoff and releases from Lake Okeechobee. The facilities will be

[2] The Central and Southern Florida Project Comprehensive Everglades Restoration Plan Program Management Plan

designed for environmental water supply deliveries to the Caloosahatchee Estuary, water supply benefits, and some flood attenuation.

- The C-43 West Storage Reservoir site ranked very high in the initial site screening evaluation matrix. Of the two initial concept plans developed for this site, the selected plan was Option 2 as the basis, with some modifications:

  (a) 160,000 acre-foot reservoir with external levees and Perimeter Canals and internal levees to divide the reservoir into cells;

  (b) *3000 cfs pump station that pumps from Townsend Canal;

  (c) *500 cfs pump station that pumps from Roberts/Duda Canal;

  (d) *350 cfs pump station that pumps from Banana Branch;

  (e) *250 cfs pump station that pumps from Crawford Canal;

  (f) Three gated spillway structures;

  (g) Nine gated culvert structures;

  (h) Improvements to Townsend Canal;

  (i) Improvements to Banana Branch;

  (j) Improvements to Ft. Simmons Branch; and

  (k) Four-lane Bridge on S.R. 80 over Townsend Canal.

  * Additional site-specific hydraulic modeling was necessary to determine the proper sizing and necessity of the pump stations.

The above scope of work and assumptions were the starting point for this BODR development and evaluation. The 30% preliminary design began with a kickoff meeting held on May 7, 2004. Weekly design meetings and miscellaneous meetings have been held during the design process. Refer to Meeting Notes 1D-01 in Appendix **Section 1** for the meeting notes. These notes document the design decisions made during the design process.

3) Existing Facilities

The Reservoir site currently under consideration includes the Berry Groves property, the Griffin property, the MG Enterprise property, and a portion of the Duda property in the southwest corner of the proposed Reservoir. A portion of the Bob Paul, Inc. property is also included. Refer to **Figure 1D-01** in **Section 1** of the Appendices.

Essentially all of this property described above is in citrus production and has been in citrus production for many years. The Townsend Canal provides water for irrigation for all of the properties, except the Bob Paul property. The Berry Groves pump station, located at the intersection of the LPDD Canal and the Townsend Canal provides water for the Berry Groves, Griffin and MG Enterprises properties. The Duda pump station, located at the "bend" in the Townsend Canal west of the southwest corner of the Griffin property, lifts water from the Townsend Canal into the Townsend Canal south of the pump station and feeds a series of canals within the Duda property. The water elevation within the Duda property is maintained at approximately 23.5 ft. in elevation.

Irrigation water for the Bob Paul property is provided by a series of wells.

In addition to the internal irrigation canals that are included within each ownership within the project site, there are several canals and streams that serve more than internal irrigation purposes. The Townsend Canal, the NE Rim Ditch, the Roberts Canal, the Crawford Canal, the Banana Branch, and the Fort Simmons Branch serve as watershed conveyance systems and tributaries to the C-43 Canal. All provide environmental benefits.

Berry Groves has an existing citrus plant north of the project footprint along S.R. 80. The Duda citrus plant is east of the project footprint along S.R. 29. A large FP&L transmission line is located parallel and borders the entire northern boundary of the project footprint.

4) Proposed Facilities

As mentioned under **Section 1 D 2)**, the BODR proceeded from the Berry Groves Site Option 2, which was one of the site options developed as a result of the site screening process as part of the C-43 Basin Reservoirs PIR. Option 2 is indicated in **Section 1** of the Appendices as **Figure 1D-02**.

Much iteration of the overall property boundary, dam/levee locations and height, cell configurations, pump station(s) locations, and structure locations have occurred as the conceptual design has progressed. The design progression is documented throughout this report.

The interim Reservoir capacity of 170,000 acre-feet has been provided at this conceptual stage to allow for refinements in the design that will reduce capacity, while still providing at least 160,000 acre-feet of above ground storage.

(a) Alternative A, Two-Cell Alternative

At this, the Pre-Final stage of the BODR, two alternatives have resulted from the conceptual refinements. Alternative A is a two-cell alternative with two options and is indicated by **Figure 1D-03** in **Section 1** of the Appendices.

- Option 1 is the single pump station option (PS-1), which is located in the northwest corner of the Reservoir. All water pumped to fill the Reservoir will be pumped from the Townsend Canal utilizing PS-1. PS-1 has a pumping capacity of 1500 cfs is based on modeling requirements for an approximate 1050 square mile basin.

- Option 2 is the two pump station option with PS-1 and PS-2. PS-1 is again located in northwest corner of the Reservoir and has a capacity of 1250 cfs. PS-2 is located in the southeast corner of the Reservoir. PS-2 has a pumping capacity of 500 cfs based on flows generated by canals south of the project and the modeling requirements for an approximate 120 square mile basin.

The PS-2 pump station will normally pump during localized storm events to fill the Reservoir by utilizing excess run-off from the Townsend Canal (south of the Duda pump station), the NE Rim Ditch and the Roberts Canal. It is anticipated that during a normal year approximately one-half of the designed 170,000 acre-feet of storage can be pumped by PS-2.

The advantage of pumping utilizing PS-2 is that the water surface elevation in the Perimeter Canal at PS-2 will be maintained at or above 18 ft. The water surface elevation in the Townsend Canal at PS-1 is at approximately 3 ft. Thus, the estimated head advantage of pumping utilizing PS-2 is approximately 15 ft.

Figure 1D-03 in **Section 1** of the Appendices indicates the major design elements of the Reservoir for Alternative A. The water level in the two cells will be the same elevation for both Option 1 and 2, and when filled to the "normal pool" elevation, the water surface elevation will be at elevation 39.2 ft. The average water depth will be approximately 19.1 ft. in Cell 1 and 15.8 ft. in Cell 2, with the resulting storage of approximately 170,000 acre-feet.

A general description of the Reservoir design elements follows with more detailed descriptions and exhibits of individual elements in the body of this report. Evaluations will be completed in the design phase

using storm events equal to 5-day/10-year, 5-day/25-year and 5-day/ 100-year design storms.

Alternative A, the two-cell alternative, **Figure 1D-03** in **Section 1** of the Appendices is very conceptual, but represents the general location of dams, canals, pump stations and structures. The heavy red line indicates the Perimeter Canal and separator dam locations. The heavy blue line indicates the perimeter/conveyance canal system around the Reservoir and other canal improvements. The pump station and structures for Option 1 are indicated in yellow. The added pump station and structures for Option 2 are indicated in orange.

- As mentioned above, Alternative A is a two-cell alternative with two options. All "yellow" structures are essentially the same for both options. Only the pump stations and orange structures vary. PS-1 will pump from an approximate water surface elevation in the Townsend Canal of 3 ft. and will be utilized solely (Option 1) or jointly with PS-2 (Option 2) to fill the Reservoir to an approximate normal pool elevation of 39.2 ft.

- Structure S-1 will be adjacent to the pump station facility and will serve as a discharge structure from Cell 1 into the Townsend Canal to maintain adequate flows in the C-43 Canal during periods of low flows. Structure S-1 can also serve as a design storm control structure for releases prior to and during a storm event. The storm releases must be balanced with the maximum flow allowed over S-79. An emergency spillway with a crest elevation based on the 25-year, 72-hour storm event for Cell 1 will also be provided at Structure S-1.

- Structure S-2 serves as a discharge structure to provide water to the improved NE Rim Ditch. This structure will provide water to the NE Rim Ditch during periods when Duda would be operating their pump station on the Townsend Canal and water is being released from the Reservoir to the Townsend Canal. This structure eliminates the duplication of Duda pumping out of the Townsend Canal while the SFWMD is discharging into the Townsend Canal to maintain adequate flows at S-79.

- The improved NE Rim Ditch from the Townsend Canal to S-3 serves several purposes. It is designed as a seepage canal and will be an enlarged canal incorporating the existing NE Rim Ditch. The water surface elevation will be maintained at approximate elevation 23.5 ft. and within the operating range for irrigating the Duda groves. Structure S-3 will maintain the desired water level in the NE Rim Ditch.

- The seepage canal, labeled the "Perimeter Canal", from S-3 to S-11 also serves several purposes. It will serve to reduce flooding conditions south and east of the Reservoir, including flooding of the Duda property from the Townsend Canal, the D-3 Canal, the NE Rim Ditch south of the Reservoir; and from the Roberts Canal and property east of the Reservoir including flooding along and possibly east of S.R. 29. The Perimeter Canal will also maintain flows in the Crawford Canal, the Banana Branch, and the Fort Simmons Branch north of the Reservoir.

  With Option 1, water levels downstream of S-3 will typically be maintained near elevation 16. With Option 2, water levels between S-3 and S-4 will be maintained at elevation 18 and downstream of S-4 the water level will be maintained at elevation 16. Elevation 16 will provide for positive flows from the S.R. 29 drainage and provide adequate head for discharges into the Crawford, Banana Branch and Fort Simmons systems.

- S-4 is only required for Option 2 and provides a "pool" for PS-2 pumping.

- Structures S-6, S-7, and S-9 will be constructed to prevent flooding and to maintain historic flows in the Crawford Canal, Banana Branch, and Fort Simmons Branch that are necessary to maintain the existing environmental conditions.

- Structure S-10 will be a weir and maintain the normal water surface elevation in the Perimeter Canal at approximately 16 ft., except during releases or high rainfall events. The natural ground elevation, west of S-10, drops significantly to the Townsend Canal and the water surface elevation cannot be maintained at the 16 ft. elevation without flooding adjacent property.

- Structure S-11 will serve as a weir structure to eliminate impacts to the Perimeter Canal when PS-1 is pumping. The water surface elevation upstream of S-11 will be elevation 10 and as mentioned above will prevent flooding on adjacent property.

- Structure S-8 will be similar to S-1 and will serve Cell 2. S-8 can also serve as a design storm control structure for releases prior to and during a storm event, similar to S-1. The storm releases for S-8 and S-1 must be balanced with the maximum flow allowed over S-79 (4,500 cfs). An emergency spillway with a crest elevation based on the 25-year, 72-hour storm event for each cell will also be provided. In addition, structure S-8 can also serve as discharge structures to maintain the required flows through structures S-6, S-7 and S-9.

- Structure S-12 is anticipated to be a gated culvert structure and will maintain flows between the Reservoir cells. This structure will be gated so that flows can be blocked if needed during maintenance operations and the cells can be drained independently. There will be multiple structures at the S-12 location.

- Structure S-14 is a culvert structure to allow flows during storm events to enter the Perimeter Canal from the east, including improved drainage from S.R. 29. During low flows, water will flow east from the Perimeter Canal to provide adequate flow in the Crawford Canal.

- Structure S-15 is a gated structure that will maintain the water in the Roberts Canal upstream of the structure at an approximate elevation of 25.5. During storm events and high water, flows will enter the Perimeter Canal.

(b) Alternative B, Three-Cell Alternative

Alternative B is a three-cell alternative. The third cell is created by adding property to the east of the two-cell alternative and is currently property owned by Bob Paul, Inc. The three-cell alternative is indicated by **Figure 1D-04** in **Section 1** of the Appendices. Alternative B also has two options.

- Option 1 is the single pump station option (PS-1), which is located in the northwest corner of the Reservoir. All water pumped to fill the Reservoir will be pumped from the Townsend Canal utilizing PS-1. PS-1 has a pumping capacity of 1500 cfs.

- Option 2 is again a two pump station option with PS-1 and PS-2. PS-1 is again located in northwest corner of the Reservoir and has a capacity of 1250 cfs. PS-2 is located in the southeast corner of the Reservoir. PS-2 has a pumping capacity of 500 cfs.

As discussed under Alternative A, the PS-2 pump station will pump during storm events to fill the Reservoir by utilizing excess run-off from the Townsend Canal (south of the Duda pump station), the NE Rim Ditch and the Roberts Canal. Again, the advantage for using PS-2 is a 15 ft. head advantage from pumping out of the Perimeter Canal vs. the Townsend Canal.

The Pump Stations, structures, Perimeter Canal, etc. are very similar in both Alternative A and Alternative B. The discussion below will only address differences between Alternative A and Alternative B. Design elements that are contained in both alternatives and discussed under Alternative A, will not be repeated again.

A legend on **Figure 1D-04** in **Section 1** of the Appendices again indicates the symbols for each design element of the Reservoir. The water level in the three cells will be the same elevation for both Option 1 and Option 2, and when filled to the "normal pool" elevation, the water surface elevation will be at approximate elevation 37.7 ft. The average water depth will be approximately 17.4 ft. in Cell 1, 14.1 ft. in Cell 2 and 11.4 ft. in Cell 3, with the resulting storage of approximately 170,000 acre-feet. Again, this interim Reservoir capacity of 170,000 acre-feet has been provided at this conceptual stage to allow for refinements in the design that will reduce capacity, while still providing at least 160,000 acre-feet of above ground storage.

Starting at PS-1 on **Figure 1D-04** in **Section 1** of the Appendices, the design elements for Alternative B including S-2, S-3 and PS-2 remain in the same locations as Alternative A.

- For Alternative B, Option 2, S-4 is moved to the east of PS-2 along the Perimeter Canal that now is extended around the perimeter of Cell 3.

- S-5 is added as a discharge structure from Cell 3. Structure S-5 will be similar to S-1 and S-8 and will serve Cell 3. S-5 can also serve as a design storm control structure for releases prior to and during a storm event, similar to S-1 and S-8. The storm releases from S-5, S-8 and S-1 must be balanced with the maximum flow allowed over S-79. An emergency spillway with a crest elevation based on the 25-year, 72-hour storm event for each cell will also be provided.

- S-6 is moved to the north and outside of Cell 3 at the intersection of the Perimeter Canal and the Crawford Canal.

- Structure S-13 is anticipated to be a gated culvert structure and will maintain flows between the Reservoir Cell 2 and Cell 3. This structure will be gated so that flows can be blocked if needed during maintenance operations and the cells can be drained independently. There will be multiple structures at the S-13 location.

E. Other Studies, Reports, and Projects

- *C-43 Basin Storage Reservoir Project, Feasibility Scoping Meeting Conference Materials, December 2004*

- *Central and Southern Florida Project Comprehensive Everglades Restoration Plan, Program Management Plan, Data Management, February 26, 2002*

- *Central and Southern Florida Project Management Plan, Southwest Florida Feasibility Study, January 2001*
- *Caloosahatchee Water Management Plan, April 2000*
- *Lower West Coast Water Supply Plan, Planning Document Volume 1, April 2000*

Section 2 – Site Conditions

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 2 of the Appendices.

A. Climate

1) Existing Conditions

Annual rainfall in Hendry County averages 50 inches a year; the areas of Lake Okeechobee and Charlotte Harbor usually receive less rainfall than any other region within south Florida. More than half the rain falls in the wet season from June through September and is associated with thundershowers, squalls, and tropical cyclones. Afternoon thundershowers are frequent over land, where moisture-laden air from sea breezes and wetlands warms, and rises, and the moisture condenses to form clouds. The wet season often has a bimodal rainfall pattern with two maxima, one in early and one in late summer. Rainfall during the remainder of the year is usually the result of large frontal systems and is broadly distributed rather than localized. April and May typically have the lowest rainfall. Annual and seasonal rainfalls, however, vary from year to year. Droughts in south Florida can be fairly widespread while others can be localized, even over distances of only 30 miles. Tropical cyclones (hurricanes and tropical storms) produce the most severe weather conditions in south Florida; hurricane season lasts from June through November. The high tides and heavy rains associated with these storms can produce coastal and inland flooding, and strong winds can cause extensive damage.

The climate in south Florida in general, and Hendry County in particular, is subtropical and humid. Average temperatures are in the mid-70's °F annually, ranging from about 60 °F in midwinter to about 80 °F in summer. It is not unusual for humidity levels to approach 90% during summer months. The summer heat is tempered by sea breezes near the coast and by frequent afternoon and evening thundershowers. Freezes are not common in Hendry County and pattern and severity is erratic from year to year.

2) Potential Impacts

No impacts to regional climate are anticipated for the project.

B. Land Use

1) Existing Conditions

Agricultural lands are prevalent in Hendry County; pastureland and crops (primarily citrus but also sugarcane and others) comprise the majority of the agricultural industry. Water systems including canals, streams, and ponds are common. Remaining natural areas include pine flatwoods (upland communities dominated by pine and saw palmetto) and a variety of wetland communities. Land use within the project boundary reflects the land use of the surrounding county.

Natural/biological features and land use within the approximate 12,000-acre project area were initially reviewed using the 2000 Florida Land Use, Cover and Forms Classification System (FLUCFCS) GIS data layer provided by the SFWMD. Citrus grove (FLUCFCS 221) covers approximately 96.4% of the study area, more than any other land cover type. Water (FLUCFCS 500 series) comprises approximately 1.6% of the site and includes extensive agricultural canals, classified as Streams and Waterways (FLUCFCS 510), and excavated ponds, classified as Reservoirs less than 10 acres (FLUCFCS 534). Wetland (FLUCFCS 600 series) comprises approximately 1.6% of the site and includes Mixed Wetland Hardwoods (617), Willow and Elderberry (618), Exotic Wetland Hardwoods (619), Cypress (621), Wetland Shrub (631), and Freshwater Marsh (641). The remaining 0.4% of the site consists of Spoil Areas (743) and one small pine flatwoods parcel (411). Refer to **Figure 2B-01** in **Section 2** of the Appendices for a land use map and to the Vegetation section of this report for additional information on the natural community types.

2) Potential Impacts

All current land use types within the project boundary will be converted to open water (likely to be considered as Reservoirs larger than 500 acres; FLUCFCS 531), fringing marsh wetlands (likely to be considered as freshwater marsh; FLUCFCS 641), and cell berms/maintenance roads (likely to be considered as Transportation, Communication and Utilities; FLUCFCS 800 series).

C. Geology and Soils

1) Regional

According to Dr. H. K. Brooks publication "Guide to the Physiographic Divisions of Florida" 1981, the C-43 Basin lies within the Southwestern Flatwoods District. It is comprised of the Caloosahatchee Valley along its east-

west trending axis, the De Soto Slope to the north and the Immokalee Rise to the south. These are described as follows.

- Caloosahatchee Valley – An ancient river valley filled with sands and shells from the Plio-Pleistocene age. The landform is terraced and generally made up of flatwoods and wet prairie.

- De Soto Slope – A sloping plain that varies in elevation between +90 and +30 ft. above mean sea level (MSL). This area is also characterized by wet prairie and flatwoods. The surface drainage is disrupted by swamps.

- Immokalee Rise – A broad area of land that is somewhat elevated above that surrounding it. Elevations are typically between +30 and +40 ft. above MSL. Landforms are comprised mostly of flatwoods with some wet prairie and cypress swamp.

Review of the Feasibility Scoping Meeting Conference Materials for the C-43 Basin Storage Reservoir Project indicates that the C-43 Drainage Basin of both Collier and Hendry Counties is characterized by terraced plains. The terraces basically control the topography of the C-43 Basin. Pleistocene age sands were deposited as three marine terraces, namely:

- The Talbot terrace, which ranges from 25 to 42 ft. above sea level and is considered the highest in elevation within the subject area.

- The Pamlico terrace, which ranges between 8 to 25 ft. above sea level.

- The Silver Bluff terrace, which is the lowest in elevation, is less than 10 ft. above sea level.

The deposits were formed during the Pleistocene age when sea levels rose and fell in response to glacial advance and retreat. When sea levels remain relatively constant for long periods, conditions become favorable for the development of shoreline features. The Pleistocene deposits in Southwest Florida are represented by the Anastasia Formation (a coquinoid limestone, sand and clay), alternating marine and freshwater limestone consisting of mollusks, interbedded clays, and sands of the Ft. Thompson Formation, Caloosahatchee Formation (shell hash limestone and clay), and the Hawthorn Group sediments.

Rock formations found within the C-43 Basin study area range in age from Recent to Miocene to Oligocene. These formations include (from land surface) the Ft. Thompson Formation, Tamiami Formation (which includes the Bonita Springs Marl and Pine Crest Sand members), the Hawthorn Group (which consists of the Peace River Formation within the upper unit and Arcadia Formation belonging to the lower Hawthorn Group sediments), and, to a lesser extent, the early Oligocene Series of the Suwannee Limestone.

The soils of the Caloosahatchee River Basin are dominated by somewhat poorly drained sandy soils. These soils are considered recent deposits of limestone origin, underlain by marl and or limestones. South of the river and in the coastal area the soil consists of poorly drained sands and organic muck and peat. They are very poorly drained soils underlain by marl and limestone.

2) C-43 West Storage Reservoir Site

The US Geological Survey 7.5 minute quadrangle maps for Alva and Sears, FL, dated 1958 and photo-revised in 1973, indicate that the Reservoir site is relatively flat and has ground surface elevations that vary between approximately +20 and +28 ft. with respect to the National Geodetic Vertical Datum of 1929 (NGVD), near the project's northwest and southeast corners, respectively. The site is transected and/or bounded by several canals, including the Townsend Canal (western project boundary), the Fort Simmons Branch (trending north-south through the center of the project), the Roberts Canal, which trends northwest-southeast in the eastern half of the Reservoir footprint, the LPDD Canal (east-west trending through the center of the site), and the NE Rim Ditch which borders the project to the south. The USGS quadrangle map for the project site is presented as **Figure 7A-1** in **Section 7** of the Appendices.

Research of the Soil Survey of Hendry County, Florida, published by the US Department of Agriculture – Soil Conservation Service indicates that the Reservoir site is underlain by soils that are typical of sloughs and flatwoods. The Soil Survey describes these soils in general terms as follows:

Oldsmar-Wabasso Association: nearly level, poorly drained soils that are sandy throughout and have organic staining in the subsoil.

Malabar-Pineda-Oldsmar Association: nearly level, poorly drained and very poorly drained, sandy soils that have a loamy (i.e. clayey) subsoil; some are underlain by limestone.

Boca-Riviera-Pineda Association: nearly level, poorly drained, sandy soils that have a loamy subsoil or a sandy and loamy subsoil underlain by limestone.

The Geologic Map of Florida, prepared by Dr. H. K. Brooks, indicates that the project site is underlain predominantly by the Pleistocene age Caloosahatchee Formation. This is described as a calcareous, shelly sand with diverse extinct tropical marine fauna, and may contain multiple caprock layers. Geology in the western fringes of the project boundary are mapped by Dr. Brooks as consisting of the lower Buckingham member and upper Ochopee member of the Tamiami Formation of Pliocene age. This is described as impure, clayey to sandy, marly limestone with phosphorite grains, soft to medium hard. Some fraction of the soil phosphorus will be soluble and will enter the water column when the

reservoir is flooded, however, most of this phosphorus will cycle through the system and will settle back into the sediment layer.

D. Seismicity

The Reservoir lies in an area that is relatively quiet from the standpoint of seismic activity. The US Geological Survey (USGS Earthquake Hazards Program) has reported two historical earthquakes in Florida. On February 6, 1780, a military outpost in northwest Florida experienced ground motion resulting in arms racks falling from walls in many barracks. This event occurred during a "fearful" storm that was accompanied by violent (electrical) weather and raging seas. The storm was believed by some to be a hurricane. On January 13, 1879, near St. Augustine, Florida, a trembling motion was preceded by a rumbling sound. Two shocks occurred, each lasting 30 seconds. This event was felt from a line joining Tallahassee, Florida to Savannah, Georgia on the north to a line joining Punta Rassa and Daytona Beach, Florida on the south. No known earthquakes have been recorded or reported in the project vicinity.

Reference: Seismicity of the United States, 1568-1989 (Revised), by Carl W. Stover and Jerry L. Coffman, U.S. Geological Survey Professional Paper 1527, United States Government Printing Office, Washington: 1993.

E. Surface Water Hydrology

The Reservoir area is approximately 12,000 acres and will be located just south of S.R. 80; approximately three (3) miles west of Labelle, FL. Hydrologic and hydraulic modeling is being performed using a both a sub-regional model and a site-specific model. The sub-regional model is used to evaluate watershed-wide impacts, and the site-specific model addresses local issues such as seepage and dam breach analysis. The sub-regional model domain covers 1,200 square miles and extends from the SFWMD structure S-77 at Lake Okeechobee to the structure S-79 at the Franklin Lock east of Fort Myers. The site-specific model domain of the Berry Catchment covers 120 square miles. The watershed area is shown in **Figure 2E-01**.

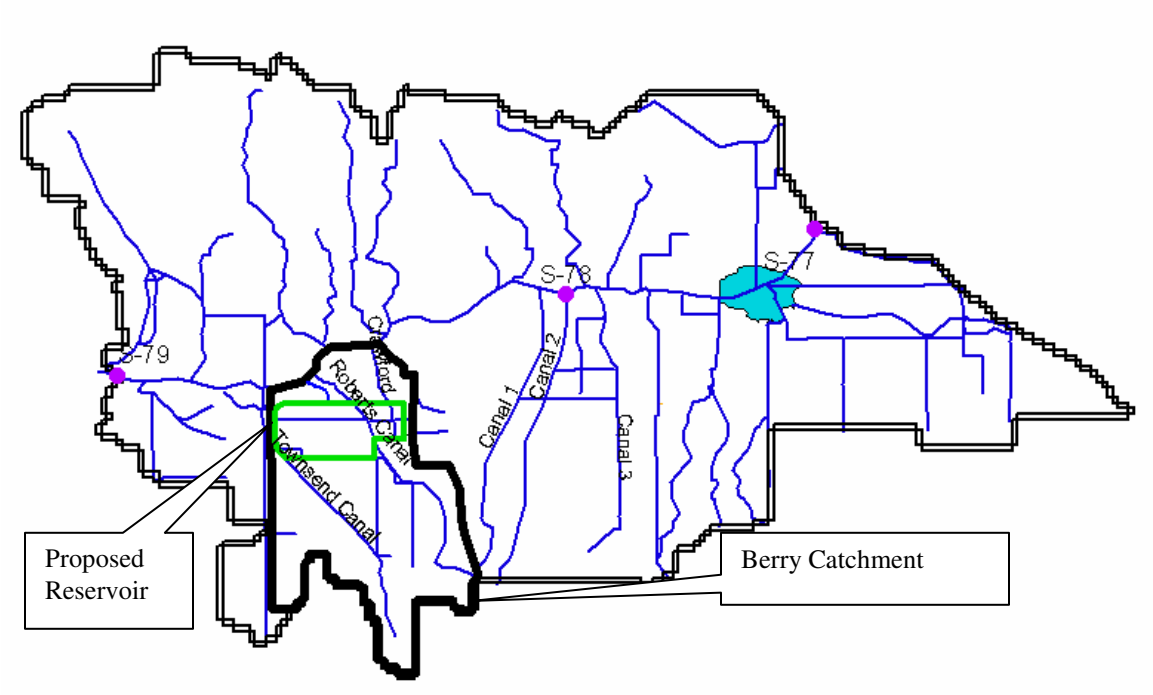


Figure 2E-01 – Area of C-43 Basin, Berry Groves Catchment, and the Proposed Reservoir

The C-43 Watershed MIKE 11/MIKE SHE regional model calibration was completed in July, 2004 (DHI, 2004). The calibrated model was modified for use in evaluating the proposed Reservoir. The calibration of the C-43 model is documented in (Draft) Updated Integrated Surface/Ground Water Model for C-43 Basin (DHI-2004). The model calibration for S-78 and S-79 are summarized in the cumulative flow plots presented in **Figures 2E-02 and 2E-03**. The flows in C-43 Canal are influenced by releases from Lake Okeechobee at S-77, irrigation within the watershed, and drainage of the agricultural lands.

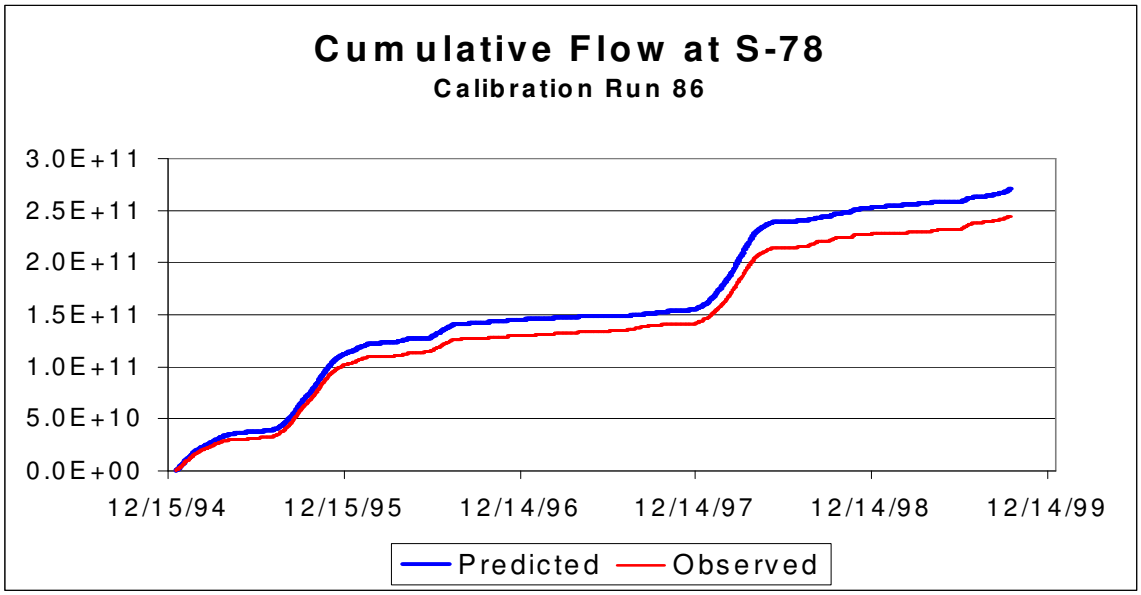


Figure 2E-02 Calibration of C-43 Model at S-78

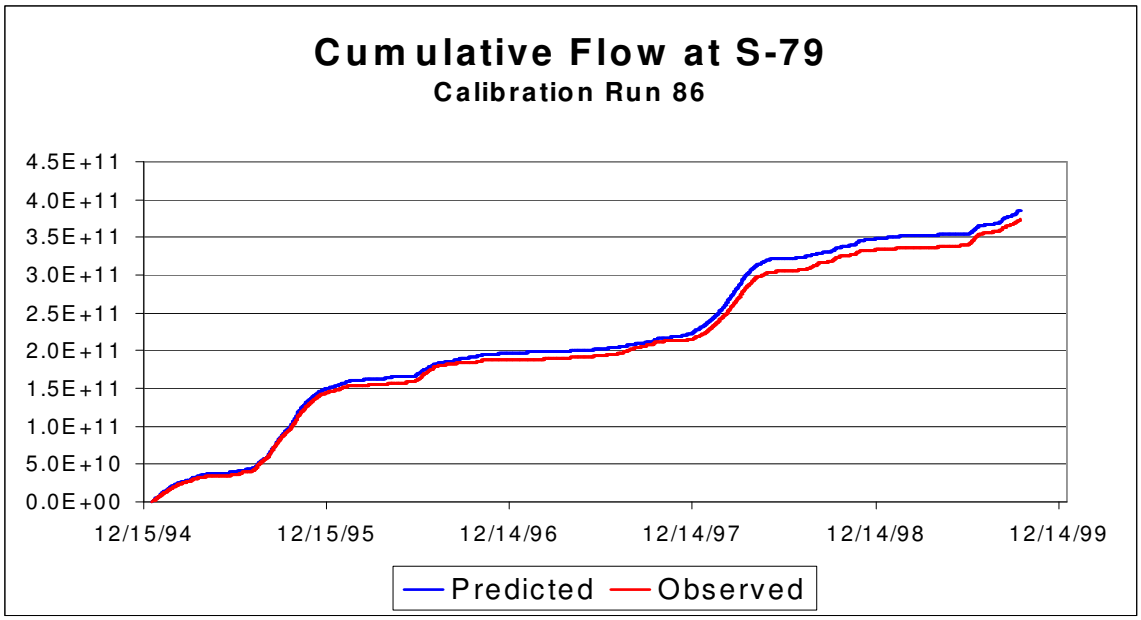


Figure 2E-03 Calibration of C-43 Model at S-79

C-43 Canal flows in the later part of the 20th century have ranged from very high to very low with periods of significant discharges from Lake Okeechobee and at different times from the 1,200 square mile watershed. **Figure 2E-04** illustrates observed flows from S-77, S-78, and S-79.

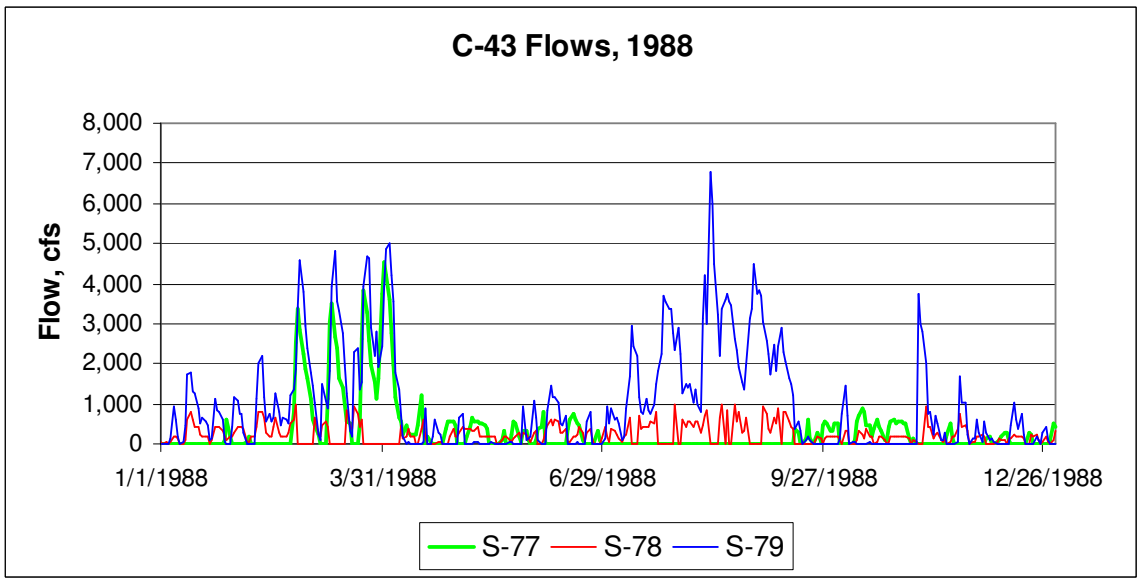


Figure 2E-04 – C-43 Flows during 1988

Figure 2E-04 illustrates that high flow periods in March 1988 were dominated by releases from S-77, while high flows in September were dominated by basin runoff. Interestingly, the high runoff in March 1988 was not reported at S-78, possibly due to high irrigation withdrawals in the sub-watersheds of Canals 1, 2, and 3 (see **Figure 2E-01**

for location) that returned flow to C-43 Canal downstream of S-78. Note that there are no measured flows for Canals 1, 2, and 3, and the irrigation pump capacity that move water upstream in Canal 3 are not definitively known. Operation routines for these pumps are also not known. During the development of the C-43 model, information was obtained from field drive-by surveys, off-the-record discussions with engineers familiar with the area, and microfiche files of SFWMD agricultural permits. Best available judgment was used during the model development, and the model results confirm the diversion of flows around S-78.

Figure 2E-05 presents flows for 1998, which was a year with spring season flows dominated by a major discharge from Lake Okeechobee and late season flows dominated by a very large discharge from the watershed and no outflow from the Lake. This figure illustrates that runoff from the C-43 Basin can generate very large peak flows in excess of 10,000 cfs.

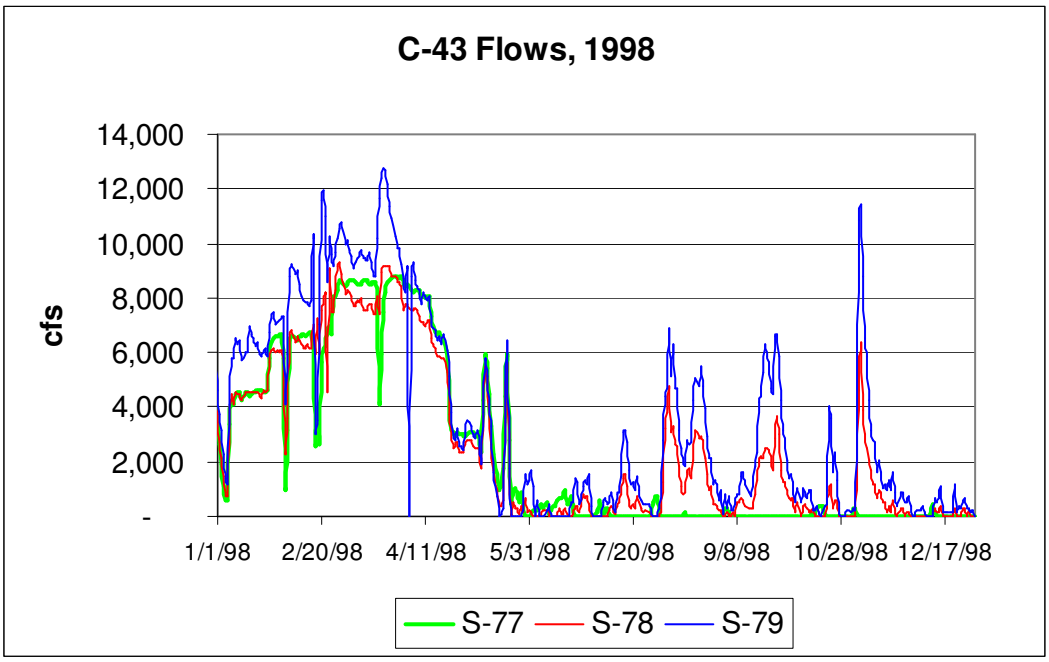


Figure 2E-05 – C-43 Flows for 1998

Figure 2E-06 confirms that the C-43 Basin can generate large flows without discharges from Lake Okeechobee, and the peak flow from the basin is over 14,000 cfs. Total runoff from 7/2/2001 to 11/4/2001 at S-79 was 728,000 acre-feet. Runoff for this same period above a daily release at S-79 of 600 cfs was 578,000 acre-feet.

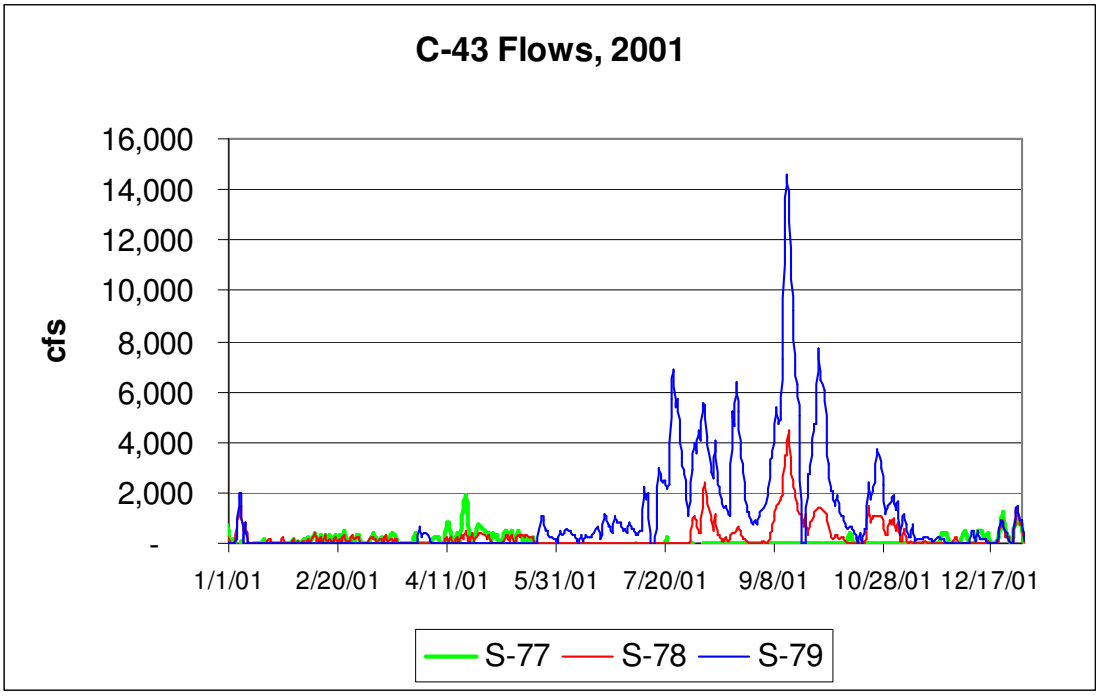


Figure 2E-06 – C-43 Flows for 2001

F. Ground Water Hydrology

Groundwater elevations vary from being very close to the surface near Lake Okeechobee and in wetlands such as Nicodemus Slough and Okaloacoochee Slough to being highly variable in areas at higher elevations. Groundwater elevations are presented for two locations near the proposed Reservoir location in **Figures 2F-01 and 2F-02** (taken from DHI, 2004).

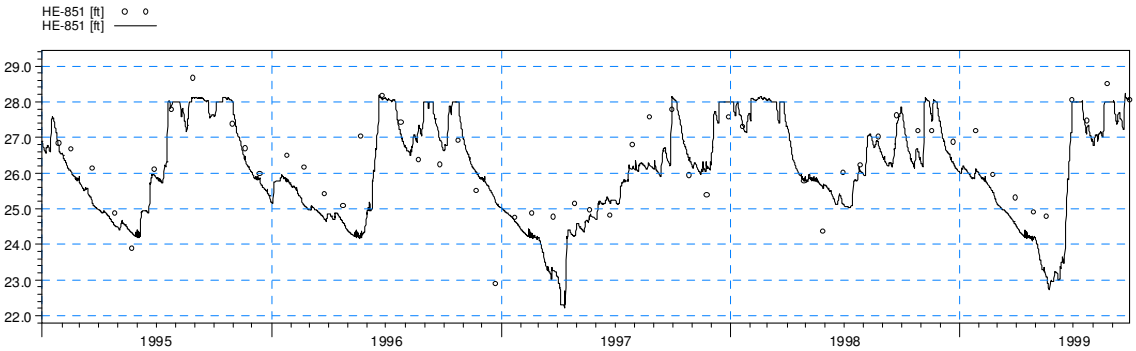


Figure 2F-01 – Simulated and Measured Surficial Aquifer Elevations at Well HE-851 Near Roberts Canal and Sears Road (Ground elevation = 28 ft NAVD)

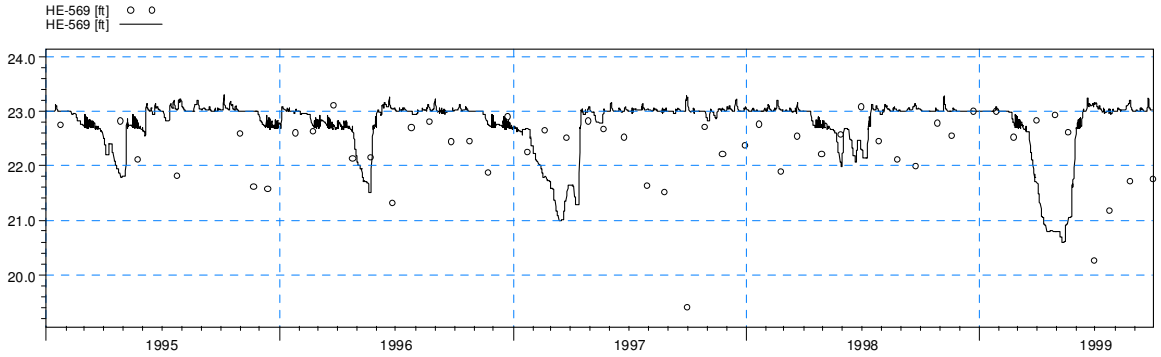


Figure 2F-02 – Measured and Simulated Surficial Aquifer Elevations Near the NE Rim Ditch Between Duda and Berry Groves Farms (Ground Elevation = 24 ft NAVD)

Well HE-851 is located near wetlands that have larger variations in groundwater elevations, while HE-569 is located between two citrus operations that experience less variation.

G. Environmental Conditions

1) Air Quality

A few common air pollutants are found all over Florida. The existing air quality within south Florida is considered good, and the region attains all National Ambient Air Quality Standards. The current sources of air pollution are area wide, resulting from autos in urban areas, land clearing and partly from various licensed emitters (SWFRPC, 2002). Larger industrial polluters are limited. There are citrus processing plants in the project area.

In the site's current condition, sources of air pollution include emissions from diesel-operated pumps, particulates from land manipulation activities, and airborne pesticides and herbicides, if applied by spray-application within the citrus groves at the project site. In the proposed condition, these sources will be eliminated. There may be short-term particulate dust emissions from land clearing and moving operations during construction, however, best management practices will be used to control any such emissions.

2) Noise

Within the major natural areas of south Florida, external sources of noise are limited and of low occurrence. Rural areas, such as the project area, typically have noise levels in the range of 34-70 decibels. In the site's current condition,

sources of noise include the pumps, when in operation, and heavy machinery used in normal farming practices. In the proposed condition, operation of pumps will continue, and there may be other noise generated from the operation of maintenance equipment from time to time. During construction, noise will be generated from the operation of construction equipment and pumps, but this will be short-term. All local and state noise regulations for construction will be adhered to during the construction phase.

3) Vegetation and Wetlands

(a) Existing Conditions

Natural vegetative communities in Hendry County consist primarily of pine flatwoods and are interspersed with forested and non-forested wetlands. Flatwoods are described as low-relief savannas that burn frequently. They are typically dominated by an open canopy of slash pine (*Pinus elliottii* var. *densa*) over a low open cover of scrub oak (*Quercus* spp.) on dry sites, saw palmetto (*Serenoa repens*) on moist sites, and a dense and very diverse herbaceous community on wet sites. Forested wetlands are dominated by cypress and/or mixed hardwoods. They occur in topographic depressions or on stream floodplains where there are long hydroperiods and moderate annual water table fluctuations. Herbaceous wetlands consist of relatively short and dispersed vegetation or woody shrubs.

Natural/biological features and land use within the project area were initially reviewed using the 2000 Florida Land Use, Cover and Forms Classification System (FLUCFCS) GIS data layer provided by the SFWMD. Citrus grove (FLUCFCS 221) covers approximately 96.4% of the study area, more than any other land cover type. Water (FLUCFCS 500 series) comprises approximately 1.6% of the site and includes extensive agricultural canals, classified as Streams and Waterways (FLUCFCS 510), and excavated ponds, classified as Reservoirs less than 10 acres (FLUCFCS 534). Spoil Areas (743) and one small pine flatwoods parcel (411) comprise 0.2%, respectively, of the site. Wetland (FLUCFCS 600 series) comprises approximately 1.6% of the site and includes Mixed Wetland Hardwoods (617), Willow and Elderberry (618), Exotic Wetland Hardwoods (619), Cypress (621), Wetland Shrub (631), and Freshwater Marsh (641). Refer to **Figure 2B-01** in **Section 2** of the Appendices for wetland locations.

A total of 70 wetland systems were identified in the study area. Many of the wetlands are highly disturbed by the surrounding land use practices of the active citrus groves. The majority of the wetlands are rim-ditched and connected to the internal ditch system of the groves. As a result, nuisance and exotic species (primarily Brazilian peppertree (*Schinus*

terebinthifolius) and primrose willow (*Ludwigia peruviana*) were abundant and present in all but one wetland system. However, several cypress wetlands were found to contain healthy interior systems with little or no nuisance species and a presence of several varieties of air plants (an indication of healthy systems). The following is a description of the wetland systems and the one natural upland system present on-site. Refer to the November 2004 Draft Wetlands Evaluation Report for more information.

Mixed Wetland Hardwood (617)

Mixed wetland hardwood forests contain a large variety of hardwood species tolerant of hydric conditions yet exhibit an ill-defined mixture of species. Wetlands in this classification contain laurel oak (*Quercus laurifolia*), live oak (*Quercus virginiana*), sabal palm (*Sabal palmetto*), and pond cypress (*Taxodium ascendens*) with an occasional swamp bay (*Persea palustris*), strangler fig (*Ficus aurea*), and/or slash pine (*Pinus elliottii*). The shrub layer typically reflects the canopy species, with the addition of marlberry (*Ardisia escallonioides*), American beautyberry (*Callicarpa americana*), and/or Brazilian pepper. Groundcover usually consists of swamp fern (*Blechnum serrulatum*), sword fern (*Nephrolepis exaltata*), and occasionally primrosewillow (*Ludwigia peruviana*), caesarweed (*Urena lobata*), and pickerelweed (*Pontederia cordata*). Level 4 FLUCFCS (Mixed Shrubs - 6172) was used when a dense shrub layer was encountered.

Willow and Elderberry (618)

These wetlands are populated by Carolina willow (*Salix caroliniana*), elderberry (*Sambucus nigra* var. *canadensis*), and occasional Brazilian pepper and/or primrose willow.

Exotic Wetland Hardwoods (619)

These wetland systems are typified by complete or almost complete cover of Brazilian pepper.

Cypress (621)

Cypress swamps in the project area are dominated by pond cypress and are frequently fringed with Brazilian pepper that occasionally populates the wetland centers. The subcanopy commonly consists of laurel oaks, strangler fig, and sabal palm. The shrub layer is generally composed of wax myrtle (*Myrica cerifera*), Carolina willow, elderberry, Brazilian pepper, and primrose willow. The majority of the groundcover, when present, is swamp fern, sword fern, maiden fern (*Thelypteris* spp.), and occasionally cinnamon fern (*Osmunda cinnamomea*), royal fern

(*Osmunda regalis* var. *spectabilis*), and muscadine (*Vitis rotundifolia*). Air plants (*Tillandsia* spp.) are occasionally present.

Wetland Shrub (631)

This community is associated with topographical depressions and poorly drained soil. Species observations include Carolina willow, Brazilian pepper, elderberry, primrose willow, and saltbush (*Baccharis halimifolia*), with occasional red maple (*Acer rubrum*) and melaleuca (*Melaleuca quinquenervia*).

Freshwater Marshes (641)

Freshwater marshes are composed primarily of herbaceous species rather than woody species. The majority of the marshes in the project study area were positioned within the deeper interior parts of a forested wetland. The vegetative composition generally included smartweed (*Polygonum hydropiperoides*), pickerelweed (*Pontederia cordata*), common fireflag (*Thalia geniculata*), swamp fern, maiden fern, sword fern, and netted chain fern (*Woodwardia areolata*). Level 4 FLUCFCS (6417) was used to denote freshwater marshes with dense layers of shrub, brush, and vines comprised of muscadine, primrose willow, and other shrubby species.

Pine Flatwoods (411)

One pine flatwoods community is located in the project area and is characterized by a young canopy of slash pine, with wax myrtle, melaleuca, and Brazilian pepper in the subcanopy. Groundcover species are diverse, but dominated by palmetto (*Serenoa repens*). Species noted primarily in the historic, logging skid trails included blue maidencane (*Amphicarpum muhlenbergianum*), peelback St. John's-wort (*Hypericum fasciculatum*), royal fern, foxtail (*Setaria geniculata*), southern beaksedge (*Rhynchospora microcarpa*), broomsedge bluestem (*Andropogon virginicus*), musky mint (*Hyptis alata*), coinwort (*Centella asiatica*), manyflower marshpennywort (*Hydrocotyle umbellata*), and various rushes (*Juncus* spp.).

(b) Potential Impacts

Wetland impacts resulting from this project will be approximately 190.7 acres or 1.7% of the project site. The vast majority of this acreage is forested wetlands, such as Exotic Wetland Hardwoods (619), Willow and Elderberry (618) and Cypress (621). Approximately 0.2 wetland acres consist of Open Water (500 series) and less than 0.5 wetland acres consist of Freshwater Marshes (641). Mitigation for wetland impacts will be provided pursuant to legislation developed specific to CERP

projects. Mitigation options, if necessary, will be explored further during the final design phase of the project. Very minor impacts to upland systems will occur within a pine flatwoods (411) parcel in the southwestern portion of the project area.

4) Fish and Wildlife

During site field surveys for wetlands and listed species, biologists noted all fish and wildlife, regardless of protection status. Observed wildlife included 7 species of invertebrates (2 snail species and 5 fish species), 3 species of amphibians, 8 species of reptiles, 42 species of birds, and 9 species of mammals. The majority of the species were common species native to Florida; however, 8 listed species were observed as were several non-native and/or nuisance species (such as feral hog and Cuban treefrog). The species observed (refer to the Listed Species Report) are typical for the land use of the study area and encompassing landscape.

Hendry County was historically dominated by a variety of wetland habitats and mesic and upland forests. This characteristic is still evident today, in portions of areas such as the Caloosahatchee Ecoscape, present to the east of the project area, as well as other protected and managed parcels including Okaloacoochee Slough State Forest and Management Area, Twelvemile Slough, and Devil's Garden. These properties are appropriate and large enough to support listed species including the Florida panther and black bear. However, much of Hendry County today has been converted for citrus production; the county has more citrus trees than any other county in Florida. As a result, wildlife in the project area, and in much of the county, consists of common species that are habitat generalists.

During field surveys, several fish species were observed in grove canals and ditches and were typical for the region. Species included those common to fresh and brackish water that are known to help control the mosquito population by consuming mosquito larvae and pupae, such as eastern mosquitofish (*Gambusia affinis*) and sailfin molly (*Poecilia latipinna*). Species also included popular sport fish such as largemouth bass (*Micropterus salmoides*) and bluegill (*Lepomis macrochirus*) common to the area.

The project area, as part of the Caloosahatchee Basin, is ecologically connected to the C-43 Canal and Estuary. The river has been significantly altered from channel dredging during the 1930's and 1950's. Natural bends and "oxbows" were replaced with a channel deep and wide enough to allow for boats to cross the peninsula of Florida. With the removal of the natural bends in the river, much of the aquatic grasses such as tape grass (*Vallisneria americana*) and littoral habitat that support fish and wildlife were similarly removed. Many of Florida's fishes rely on multiple aquatic habitats (including freshwater and estuarine ecosystems) for the habitat heterogeneity they provide for breeding areas, protection and food source, and the fishes of the Caloosahatchee region are

no exception. Fish and shellfish populations in the region have therefore declined over the past decades, due to habitat loss and fishing pressures.

An additional challenge to preserving the habitat of the Caloosahatchee ecosystem involves the amount and timing of water discharged from Lake Okeechobee through the C-43 Canal to the Estuary. Several aquatic plant species, and therefore the fish and other wildlife that depend on them for food and shelter, require appropriate salinity levels. Discharged water can negatively impact the estuarine system by significantly altering salinity levels, as well as contributing high levels of pesticides and turbidity to the ecosystem. One of the goals of the C-43 Storage Basin Project is to better regulate water discharge to the Caloosahatchee Estuary, in an effort to improve habitat conditions for plants, fish and wildlife.

(a) Potential Impacts

Due to the limited acreage of natural habitats, and relative commonality of the natural habitats and altered habitats (citrus groves) present adjacent to the project area, adverse impacts to fish and wildlife are not anticipated. Potential impacts include direct, indirect, and temporal effects. Species will be affected by temporary impacts, such as during construction, when water alterations, elevation changes, noise and equipment use may deter the species from foraging in the area. Invertebrates and wading bird species and their food sources will be affected temporally by the elimination of the matrix of internal agricultural ditches. However, following Reservoir construction, new habitat will be created for these species that will afford similar foraging opportunity. In addition, the Reservoir will aid in regulating fresh water flows to the C-43 Canal and downstream estuary, thereby providing an overall net benefit.

5) Protected Species

(a) Existing Conditions

Preliminary on-site surveys were conducted for all listed species with potential to occur within the project area. These species include: Eastern indigo snake (*Drymarchon corais couperi*), American alligator (*Alligator mississipiensis*), gopher tortoise (Gopherus polyphemus), gopher frog (Rana capito), bald eagle (*Haliaeetus leucocephalus*), Audubon's crested caracara (*Polyborus plancus audubonii*), Florida scrub jay (*Aphelocoma coerulescens*), snail kite (*Rostrhamus sociabilis*), Florida grasshopper sparrow (*Ammodramus savannarum floridanus*), red-cockaded woodpecker (RCW) (*Picoides borealis*), southeastern American kestrel (*Falco sparverius paulus*), Florida sandhill crane (*Grus canadensis pratensis*), wood stork (*Mycteria americana*), other listed wading birds

including limpkin (*Aramus guarana*), white ibis (*Eudocimus albus*), little blue heron (*Egretta caerulea*), snowy egret (*Egretta thula*), tricolored heron (*Egretta tricolor*), and the roseate spoonbill (*Platalea ajaja*), Florida panther (*Felis concolor coryi*), Florida manatee (*Trichechus manatus latirostris*), Florida black bear (*Ursus americanus floridanus*), Big Cypress fox squirrel (*Sciurus niger avicennia*), and Sherman's fox squirrel (*Sciurus niger shermanii*).

Review of the available data from the Florida Fish and Wildlife Conservation Commission (FFWCC), Florida Natural Areas Inventory (FNAI) and MERIT Subteam Florida Panther Zones identified the Florida panther (*Felis concolor coryi*) as a listed species that has been known to utilize habitat within the general region of the project area. Information from radio-collared individuals suggests significant habitat utilization just outside of the southeast project boundary. A parcel of land in this southeast corner was once included in the project boundary but has subsequently been removed given the panther habitat utilization. These individuals have also been tracked along the edges of the project boundary, likely as they were traveling/foraging to other areas. Approximately two-thirds of the project area falls within the Florida panther secondary habitat zone and a very small portion in the southeast corner of the project falls within the primary habitat zone. Refer to **Figure 2B-01** in **Section 2** of the Appendices for Florida panther MERIT zones on-site. No panthers were observed by SEA field ecologists.

Listed species that were observed by SEA ecologists include the Audubon's crested caracara (*Polyborus plancus audubonii*), wood stork (*Mycteria americana*), little blue heron (*Egretta caerulea*), tricolored heron (*Egretta tricolor*), snowy egret (*Egretta thula*), white ibis (*Eudocimas albus*) and American alligator (*Alligator mississipiensis*). Refer to **Figure 2B-01** in **Section 2** of the Appendices for listed species observed on-site. Approximately half of the study area falls within a wood stork Core Foraging Area (CFA). Apple snails (family Ampullariidae) were observed in many of the agricultural ditches; while apple snails are not a listed species, they are the primary food source for the snail kite (*Rostrhamus sociabilis*), an endangered species. Snail kites were not observed on-site but it is possible that snail kites utilize the area for foraging. The Breeding Bird Atlas identifies the locations of the nearest documented snail kite to be 5.7 miles to the southwest of the project area.

Listed species with known occurrences in the general vicinity of but not within the project boundary include the bald eagle (*Haliaeetus leucocephalus*), Florida scrub jay (*Aphelocoma coerulescens*), Florida black bear (*Ursus americanus floridanus*) and Florida manatee (*Trichechus manatus latirostris*). The nearest bald eagle nest to the

project site, as documented by the FFWCC, is approximately 2.25 miles to the east of the study area. A small scrub jay population has been identified by the FNAI approximately 1 mile to the northwest of the project boundary. Suitable habitat for the scrub jay was not found on-site; scrub jay survey field efforts were carried out in the one sub-optimal habitat located in the southwest corner of the project site but no individuals were observed. Black bear were identified by the FFWCC to occur approximately 2 miles to the north and northwest of the project boundary but no suitable habitat exists for the species on-site and no observations or signs of the species were noted during field surveys. Lastly, the Florida manatee has been noted by locals and representatives of the SFWMD to occasionally occur in the Townsend canal, located at the western edge of the property boundary. No individuals were noted at the time of field surveys although several individuals were observed at the weir of the canal outfall to the C-43 Canal. Refer to the November 2004 Draft Listed Species Report for additional information.

In addition to faunal surveys, the proposed project area was surveyed for protected flora. Small populations of wild pine (*Tillandsia fasciculata*), a state endangered species as listed by the Florida Department of Agriculture and Consumer Services' Chapter 5B-40, Florida Administrative Code, were observed within a few of the forested wetlands. The listing of this species is due to the introduction of the Mexican bromeliad weevil (*Metamasius callizona*) also known as the 'evil weevil' that eat and destroy large Tillandsia species. A single individual of giant orchid (*Pteroglossaspis ecristata*), a state threatened species may have been observed, however, the plant was not in flower, which is necessary for accurate species identification since wild coco (*Eulophia alta*), an unprotected terrestrial orchid, is remarkably similar vegetatively. Four commercially exploited species were observed within the proposed project area: Florida butterfly orchid (*Encyclia tampensis*), cinnamon fern, royal fern, and netted chain fern. The Florida butterfly orchid was limited to a single specimen, while the netted chain fern observation was limited to a population within one wetland area. The cinnamon fern and the royal fern were more common in the project area, as they are throughout the state.

(b) Potential Impacts

Due to the limited acreage of natural habitats, and lack of rare or endangered habitats, the number of individuals of threatened and endangered species is low and adverse affects to the listed plant species are not anticipated to result from the proposed project. Potential impacts to listed species include direct, indirect, and temporal effects. Listed species will be affected by temporary impacts, such as during construction, when water alterations, elevation changes, noise and

equipment use may deter the species from foraging in the area. Wading bird species and their food sources will be affected temporally by the elimination of the matrix of internal agricultural ditches. However, following Reservoir construction, new habitat will be created for these species that will afford similar foraging opportunity. Direct impacts to the Florida panther and possibly Florida black bear may occur if Reservoir construction impedes travel across the property.

Further consultation with USFWS on potential project impacts (direct, indirect, and cumulative) to threatened and endangered species and other Federal trust resources (e.g., migratory birds) will be necessary. With regard to surveys conducted for the preliminary Listed Species Report, the protect team did what they could during the months of the data collection timeframe. Additional efforts will be needed for permitting the project and producing a complete Biological Assessment.

6) Water Management

The C-43 Canal is the most important surface water resource in the Lower West Coastal region. The river is supplied by inflows from Lake Okeechobee and runoff from within its own watershed. The purpose of the Project as defined in the CERP is to capture C-43 Basin runoff and releases from Lake Okeechobee. The facilities will be designed for water supply benefits and environmental water supply deliveries to the Caloosahatchee Estuary for prevention of saltwater intrusion. The Reservoir may provide opportunities for some flood attenuation through operational changes to the Central & South Florida project and local drainage system. Additionally, it is assumed that, depending upon the location of the facility and the pollutant loading conditions in the watershed, the Reservoir and its facilities may provide limited water quality improvements by reducing salinity and nutrient impacts of runoff to the estuary.

A conceptual operating plan for managing the delivery of water to the estuary will be prepared when the final reservoir configuration is selected. The operating plan will be developed during the final design and will take into consideration results from the C-43 Pilot Project.

7) Water Quality

Water quality in the tributaries in the vicinity of the proposed Reservoir is influenced by the surrounding agricultural lands and water quality in C-43 Canal is influenced by Lake Okeechobee releases, agricultural runoff from lands east of the proposed Reservoir, and drainage from undeveloped lands. Sampling has been conducted in Townsend and Roberts Canals on six dates in 2002 and 2003 for nutrients and pesticides. C-43 Canal water quality data is available for 157 sampling dates from 1979 through 2003. Parameters include basic nutrients,

total suspended solids, and on selected dates metals and pesticides. Total Phosphorus (TP) data for C-43 Canal at S-78 is presented below in **Figure 2G-01**. TP data for Roberts Canal and Townsend Canal is presented in **Figure 2G-02**. TP concentrations are generally in the range of 0.1 mg/l for C-43 Canal at S-78 and range from 0.02 to 0.35 mg/l. TP concentrations appear to be more variable in Townsend Canal, however there are only six samples for Townsend Canal, therefore it is not statistically valid to compare average concentrations with S-78. Roberts Canal concentrations are generally lower than Townsend Canal concentrations. Given that the Townsend Canal watershed is almost entirely agricultural and Roberts Canal is a mix of agricultural lands and undeveloped lands and therefore similar to the overall C-43 Watershed, it is reasonable to assume that agricultural runoff will have higher nutrient concentrations than C-43 Canal at S-78.

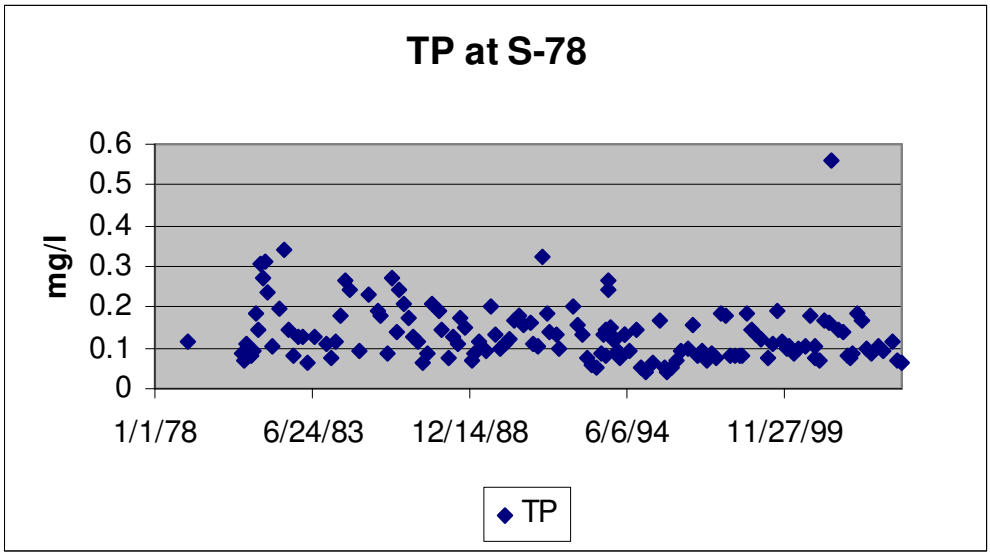


Figure 2G-01 – Measured TP Concentrations in C-43 Canal at S-78

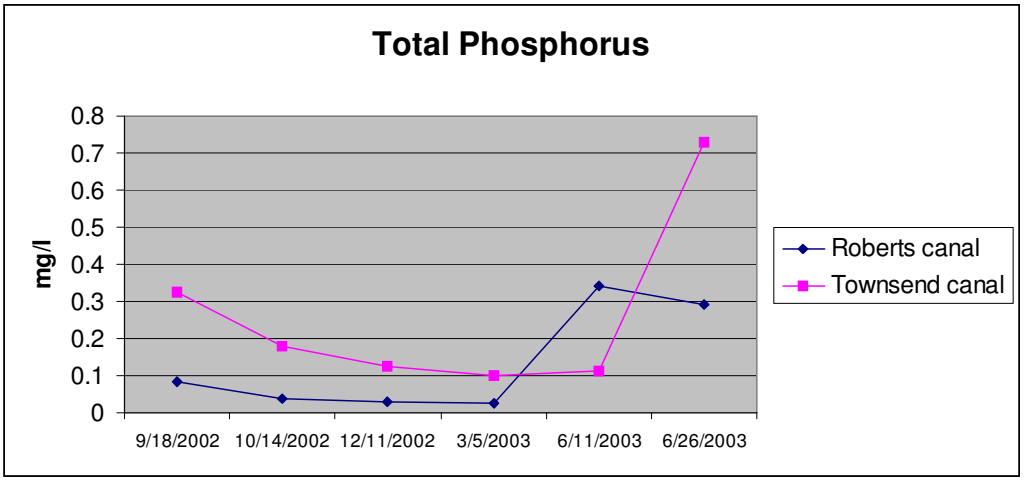


Figure 2G02 – TP concentrations in Roberts and Townsend Canals

8) Socioeconomics

(a) Existing Conditions

Hendry County, while not located directly on a Florida Coast, is beginning to realize impacts from the significant recent growth in coastal areas. As many areas become constrained to future growth, population expansion will inevitably push into Hendry County. The predominant industry in Hendry County is agriculture, and mostly citrus production. Hendry County also has significant portions of undeveloped land that is part of the natural system.

The current population of Hendry County is approximately 36,000 citizens based on 2000 Census Data. As previously mentioned, many of the local population are either directly or indirectly involved in agriculture. The per capita income of Hendry County is $13,663. The unemployment rate in Hendry County is over 11%. Property values in Hendry County vary significantly based on the location and land use. [3]

Hendry County is home to seasonal visitors. Most visitors camp at locations along the C-43 Canal at resorts or parks. These seasonal visitors contribute to the overall economy but do not significantly contribute to the tax base for the County.

(b) Potential Impacts

[3] C-43 Basin Storage Reservoir Project Feasibility Scoping Meeting Conference Materials, USACE, January 2005.

Development of the Reservoir Project will convert over 10,000 acres of citrus producing groves to a water storage facility. To fully measure the impact that this project will have on the local economy, a study has been initiated by the University of Florida, Institute of Food and Agricultural Sciences. The Study will utilize economic models to access impacts to the local economy based on the loss of citrus production. The Study will go further to estimate the impacts that building and operating a $300M + project will have on the region.

9) Recreational Resources

(a) Existing Conditions

Currently, no designated recreational areas or infrastructure are present within the project area; the project area consists solely of citrus grove operations. Nearby recreational areas include the LaBelle Nature Park, a small, forested park adjacent to the C-43 Canal that provides day-use facilities. The SFWMD owns the parcel and the Hendry-LaBelle Recreation Department manages the site. Future projects in the park may include restoration of a river "oxbow". Similar projects have been conducted in other areas of the river to restore some of the natural bends and habitats of the river. The Ortona Lock Recreation Area is located approximately 10 miles to the east of LaBelle and is situated directly on the C-43 Canal by the lock. The north recreational area provides day-use facilities and the south recreation area provides camping facilities. The USACE manages the sites.

From a regional recreational perspective, the C-43 Canal and Estuary provide significant resources for recreation. Sport fishing and boating are popular in the area. Included species are common snook (*Centropomus undecimalis*), largemouth bass (*Micropterus salmoides*), sunshine bass (*M. chrysops x M. saxatilis*), tarpon (*Megalops atlanticus*), black crappie (*Pomoxis nigromaculatus*) and butterfly peacock (*Cichla ocellaris*). The wetlands and streams throughout the LaBelle area that drain to the river contribute to these recreational resources by providing habitat and a food source for some fish species.

(b) Potential Impacts

Project construction is not anticipated to negatively affect recreational resources. It is probable that the project will, instead, provide positive benefits for recreation by increasing habitat for fish and other invertebrates. Management of the storage basin could allow for some direct recreational uses, if appropriate.

A conceptual recreation plan will be developed as part of the 30% preliminary design and will address activities which do not conflict or compromise the primary purpose of the reservoir.

10) Aesthetics

The aesthetic quality of any community is composed of its visual resources. The area of this proposed project is largely rural; agricultural land uses, primarily citrus groves and improved cattle pastures, are dominant features. The viewshed also includes some forested areas. In its current condition, it is somewhat tranquil and is generally pleasing to the senses.

The potential visual impacts from the project will affect the local community in two ways: the view of the Reservoir, and the view from the Reservoir. Because the area is sparsely populated, the most common views of the project will likely be from S.R. 80 and S.R. 29. In both cases, the project will be at a distance and in the background. The observer will be able to see vegetation adjacent to the seepage canal followed by the green sodded berm of the dam. Should a recreational component be included in the project, the observer standing on a dam will see the surrounding vista from a height of greater than 20 ft. above the existing land surface.

In summary, the proposed project is compatible with the surrounding area in terms of aesthetics because:

- It will not result in a dramatic change in visual resources;
- It will not negatively affect any existing area features that are currently considered critical to local visual resources;
- It may provide additional vistas of the surrounding area, if recreational components are incorporated; and
- Landscaping will be utilized to lesson the usual impacts of the project.

11) Cultural/Archaeological Resources

(a) Existing Conditions

A Phase I Cultural Resources Survey was completed in 2003 for the Reservoir site. At that time, the project boundary included all or portions of Sections 23 through 36 of Township 43 South, Range 28 East, and all or portions of Sections 1 through 12 of Township 44 South, Range 28 East. Four archaeological occurrences were identified but given the paucity of artifacts, these occurrences were not considered archaeological sites. Four historic structures were additionally noted but

none were considered to be eligible for the National Register of Historic Places. Refer to the June 2004 Draft Cultural Resources Survey of the C-43, Berry Groves Storage Reservoir Report for more information.

Since the archaeological study in 2003, the Reservoir Project boundary has been modified. Portions of the current project boundary that were not previously surveyed for archaeological resources included sections 31 and 32 of Township 43 South, Range 29 East, sections 5 and 6 of Township 44 South, Range 29 East, and the southern portions of sections 7 and 8 of Township 44 South, Range 28 East. These areas were assessed for probability of cultural resource presence and need for archaeological field survey. The southwest portion of the addition (portions of sections 7 and 8 of Township 44 South, Range 28 East) was determined to have at least a moderate probability for the presence of cultural resources and was therefore surveyed in November 2004. No cultural resource occurrences, sites, or structures were discovered during the additional field surveys. Refer to the December 2004 Addendum A to the Cultural Resources Survey of the C-43, Berry Groves Storage Reservoir Report for more information.

(b) Potential Impacts

No adverse impacts to archaeological resources will occur in the study site.

12) Hazardous, Toxic and Radioactive Waste

The various parcels that comprise the project site have been in citrus production for approximately 40 years. Over the operating life of these groves, fertilizers, herbicides and pesticides have been applied routinely as part of normal agricultural operations. During the 1960's to 1980's, pesticide use would have included the application of persistent organic compounds such as DDT and toxaphene. Present chemical application includes the use of copper compounds. Phase I and II Environmental Site Assessment (ESA) work has been completed for the site, and in addition, a Phase III Ecological Risk Assessment (ERA) has been completed for copper. The U.S. Fish & Wildlife Service (FWS) is currently completing a study to determine the potential for copper remaining in soils on the site to ultimately affect the snail kite, a federally protected species known to forage in the area.

Forty (40) potential contamination sites were identified within the overall project boundaries. However, only four or five sites are located within the Reservoir footprint. **Table 2G-01** lists the pertinent information for all areas of concern, as identified within previously completed reports. Sixteen (16) locations have the potential for hazardous material or petroleum contamination on land tracts

located within the project boundaries. Of the 40 potential contamination site, only 2 subset of sites are located within the Reservoir footprint.

The following designation has been assigned to each of the sites regarding if in the Reservoir footprint:

- YES Site located within proposed Reservoir footprint.
- NO Site not located within Reservoir footprint.
- – Designation not made as the site requires no action.
- P Possibility that site lies within the Reservoir footprint depending on selected alternative.

The 40 potential contamination sites identified within the project boundaries have been assigned a risk potential ranking. There are three categories used in determining this ranking, which are defined as follows:

- **NO.** Either no contamination was detected, or any contamination detected was below threshold limits that trigger remediation.
- **CONTINGENT**. Some remediation is required and completion is pending. The completion of the required corrective action would render the property suitable for the projected use.
- **YES**. Extensive remediation and or additional sampling are required. As of the date of this document, no concrete plan is in place for remediation.

Table 2G-01 – Potential Hazardous Material and Petroleum Contamination Sites

ID No.	Area of Concern	Environmental Concern	Corrective Action	Risk	Within Footprint
Tract No. 100-001; Jack Berry					
1	Pole Barns, A&B	None identified	NFA	NO	-
2	Building D, Paint Shed	None identified	NFA	NO	-
3	Vegetative Landfill	None identified	NFA	NO	-
4	Former Landfill	None identified	NFA	NO	-
5	Former Airstrip	None identified	NFA	NO	-
6	Shooting Range	None identified	NFA	NO	-
7	Former Spray Field	None identified	NFA	NO	-
8	"Bone Yard"	None identified	NFA	NO	-
9	Irrigation Pump Stations	None identified	NFA	NO	-
10	Building C, Machine Shop	None identified	impacted soil – excavation and off site disposal	YES	NO
11	Former Nursery	None identified	NFA	NO	-
12	Current Airstrip	None identified	NFA	NO	-
13	Former UST	Petroleum	Enter in Abandoned Tank program	CONTINGENT *No impact once clean up is completed	NO
14	Building G, Chemical Barn	None identified	NFA	NO	-
15	Building E, Maintenance Shop/Fuel Island	Petroleum	Impacted soil – excavation and off site disposal	YES	NO
16	Building F, Vehicle Repair Shop	Hazardous - Solvent based	Impacted soil – excavation and off site disposal	YES	NO
17	Equipment Washing Area	Hazardous - Solvent based	Impacted soil – excavation and off site disposal	YES	NO
18	Wastewater Treatment Ponds	Hazardous	Impacted soil – excavation and off site disposal	NO Clean up completed	-
Tract No. GX-100-001; Griffin Brothers Property					
19	Agricultural Lands	Metal and Pesticides	*Paraquat cease immediately *Aldicarb cease one year prior to flooding.	CONTINGENT *No impacts if paraquat & aldicarb cease	YES
20	Canal Sediments	Low levels Copper & pentachlorophenol	NFA	NO	-

ID No.	Area of Concern	Environmental Concern	Corrective Action	Risk	Within Footprint
21	Maintenance Area/Chemical Barn	soil impacted – agrochemical and petroleum.	133 tons of impacted soil – excavation and off site disposal	CONTINGENT *No impact once clean up is completed	YES
22	Burn Area	Soil Impacts - copper	121 tons of impacted soils – excavation and off site disposal	CONTINGENT *No impact once clean up is completed	YES
23	Fertilizer Mix/Load Area	Soil Impacts – Zinc	38 tons of impacted soils – excavation and off site disposal	CONTINGENT *No impact once clean up is completed	YES
24	Pump Stations	None identified	NFA	NO	-
25	Solid Waste	None identified	Remove piles of solid waste	NO	-
Tract No. GX-100-005; Winthrop Citrus Groves					
26	Citrus Grove Crop Area (cultivated area)	Metal and Pesticides	*Paraquat cease immediately *Aldicarb cease on year prior to flooding.	CONTINGENT *No impacts if paraquat & aldicarb cease	YES
27	Canal Sediments	Low level copper	NFA	NO	-
28	Agriculture Maintenance Area	Metal and Pesticides	*Paraquat cease immediately *Aldicarb cease on year prior to flooding.	CONTINGENT *No impacts if paraquat & aldicarb cease	YES
29	Exploratory oil & Gas Well	None identified	NFA	NO	-
30	Solid Waste	None identified	NFA	NO	-
31	Maintenance Area A	Petroleum and pesticides	653 tons of impacted soil – excavate and off-site disposal	CONTINGENT *No impact once clean up is completed	YES
32	Maintenance Area B	Petroleum	694 tons of impacted soil – excavate and off-site disposal	CONTINGENT *No impact once clean up is completed	YES
Tract No. GX-100-009; Bryan Paul Citrus, Inc.					
33	Pump Station	Visible presence of contaminations	Soils excavation adjacent to the main pump station and former pup station	CONTINGENT *No impact once clean up is completed	P
34	Former Nursery Barn	Arsenic, cadmium, chlorpyrifos and malathion	Soil to be excavated and disposed of off-site	CONTINGENT *No impact once clean up is completed	P

ID No.	Area of Concern	Environmental Concern	Corrective Action	Risk	Within Footprint
35	C-1 Mix Station	Metals and Petroleum	Impacts to Soil – excavation and disposal of surface soil near structure & sediment in catchment basin. Groundwater contamination – perform groundwater treatment	CONTINGENT *No impact once soil excavation and groundwater treatment is completed and 1 year of groundwater monitoring is required.	P
36	Auxiliary Tank Area	Metals	Impacted Soils – excavation and disposal of soil	CONTINGENT *No impact once clean up is complete	P
37	Graded Area Solid Waste	None identified	Solid waste present – scrap and removal of waste	NO	P
38	Burn Area	Metals	Removal of ash material is excavated and disposed of offsite	CONTINGENT *No impact once clean up is complete	P
39	Cultivated Area	Chlordane	13 acres high levels of Impacted soils. Options: Excavate, cover with clean fill, Soil inversion	YES	P
40	Canal Sediments	Metal	Excavate ash from bank of canal and sediments within the canal adjacent to burn area	CONTINGENT *No impact once clean up is complete	P
NFA = No Further Action					

H. Existing Conditions

The existing utilities found within the project boundary include Florida Power and Light (FP&L) overhead power transmission and distribution lines, local irrigation systems for the citrus groves and local pump stations used for drawing and distributing irrigation water from the Townsend Canal and the LPDD Canal.

1) FP&L Transmission Lines

Parallel FP&L overhead transmission lines are routed through the north portion of the site development area. As the project has developed, the location for the north dam is now adjacent to and south of the FP&L right-of-way. The project

as conceptualized should have no material impacts on the FP7L overhead transmission lines.

2) FP&L Overhead Distribution Lines

Existing FP&L medium voltage (MV) overhead primary lines within the project footprint which may be impacted by the project are described below and shown in **Figure 14A-01,** Area Electrical Plan in **Section 14** of the Appendices.

(a) Three-phase MV line along the Townsend Canal from S.R. 80 to the Duda Pump Stations at the southwest corner. This is normal supply to pump stations and must remain in service during all phases of construction.

(b) Three-phase MV line along the LPDD Canal and the Bob Paul Canal. This provides back-up to the pump stations along the Townsend Canal from the LaBelle Substation located at S.R. 29, and must be replaced to maintain reliable supply to the Duda Pump Station at the southwest corner of the Reservoir.

(c) Three-phase MV line east from the Townsend Canal for about one-half mile just north of the Duda Pump Station area.

3) Local Irrigation Systems

The local irrigation system consists of pump stations of varying sizes, canals and miles of irrigation pipe and tubing. There also exists a large pump station at the west end of the LPDD Canal that draws water from the Townsend Canal and discharges the water into the LPDD Canal. The LPDD Canal is located approximately one-mile south of the FP&L transmission line and runs the full east-west length of the project. Along the LPDD Canal there are small pump stations that are used to draw and distribute the irrigation water to the citrus groves.

4) Communications Utilities

No communication facilities are known to exist within the proposed construction area.

I. Surveying and Mapping

1) General

The survey and mapping of the Reservoir site was completed under a separate Work Order (W.O.) for this project. The survey and mapping effort includes full topographic (1-ft contours), hydrographic, and planimetric features to support the site development and the diversion and care of water during construction. The mapping was completed in early September 2004. The services provided included ground control surveys, flight planning, aerial photogrammetry, contour mapping, field edits to locate above grade utilities and location of soil boring.

2) Survey

The Florida State Plan Coordinate System NAD83 (1999) will be used as the coordinate base (horizontal control). All elevations will be based on NAVD 88. An offset value will be provided so that the elevations may be converted to NGVD 29.

In addition to the site mapping activities, a series of tasks associated with ground control survey were completed. These tasks include:

- Establish aerial targets for aerial photogrammetry;
- Locate and obtain elevation data on the proposed soil borings and piezometers;
- Establish horizontal and vertical control monuments that will be used during construction;
- Obtain field cross sections of existing canals as directed by the SFWMD and Stanley;
- Obtain hydrological field data (normal pool and seasonal high water) at locations as directed by the SFWMD and Stanley;
- Perform profile checks to verify mapping Digital Terrain Model (DTM) accuracy;
- Locate existing utilities as directed by the SFWMD and Stanley; and
- Complete ground surveys to supplement the mapping where the ground is obscured by heavy brush, tree cover or water.

The ground survey tasks have included obtaining cross section data in the existing canals for use in refining the hydraulic models, obtaining seasonal high

water elevations in existing tributaries north of the proposed Reservoir and compiling a base map from legal descriptions provided by the SFWMD that illustrated the lands currently owned by the SFWMD. **Figure 2I-01** in **Section 2** of the Appendices is the result of the legal description compilation.

3) Mapping

The mapping, completed in September 2004, provides flight planning, aerial photogrammetry and contour maps. The mapping consultant for this project was Spectrum (3Di Florida, LLC). The aerial photo limits cover approximately 44 square miles. The mapping limits provide coverage from just west of the Townsend Canal east to approximately 200 ft. east of S.R. 29, and from S.R. 80 south to approximately 1,500 ft. south of the NE Rim Ditch. Additionally a 500 ft. corridor centered about the Townsend Canal from S.R. 80 to the C-43 Canal is included in the mapping coverage. The mapping limits fall within the photo coverage and encompass approximately 28 square miles. For the photo and mapping limits, please refer to **Figure 2I-02** in **Section 2** of the Appendices. Based on the periodic updates delivered by the mapping consultant, it is likely that the compiled DTM file for the entire site will be in excess of 100 megabytes in size. The mapping effort includes the integration of the supplemental ground surveys completed in areas obscured by heavy brush, tree cover or water. Deliverables included:

- One electronic set of aerial photogrammetric contour maps with 1.0 ft. contour intervals for the project area at a scale of 1 inch = 200 ft., and related computer generated mapping data and spot elevations;
- One contact print with each negative;
- One aerial photo mosaic map at a scale of 1 inch = 1000 ft.;
- One set of aerial photos 9” x 9” contact prints;
- One set of photo indices showing the relative location of the individual exposures utilized in the mapping project;
- The appropriate number of sheet index maps, in electronic format, showing the relative location of each sheet in the mapping project. The map shall be a line drawing that will include, but not be limited to, the following: sections, townships, ranges, key topographic features such as shorelines and major roads as discernable from U.S. Geological Survey Quad maps;
- One set each of horizontal and vertical control description sheets in Micro Soft Word format;
- Flight control diagram(s); and

- An ASCI comma delimited file (X,Y,Z) in NAD 83/99 and NAVD 88 and an ASCI comma delimited file (X,Y,Z) in NAD 83/99 and NGVD 29 of all spot shots on this project on a CD disk.

Section 3 – General Design Requirements

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 3 of the Appendices.

A. Project Limits and Site Datum

In general the Reservoir Project limits are bounded by S.R. 80 to the north, S.R. 29 to the east, the NE Rim Ditch to the south and the Townsend Canal to the west. A more detailed discussion of the project limits can be found in **Section 3 D** and **Section 12 B**.

As stated earlier in this report the Datum for this project is NAD 83/89 and NAVD 88.

B. Functional and Operational

1) Lake Okeechobee Releases (Historical and Planned)

Measured inflows to the C-43 Watershed from S-77 at Lake Okeechobee were used to calibrate the model documented in DHI (2004). In the design of the proposed Reservoir these flows were modified to reflect the results of the SFWMD Model run CERP1RD (Interagency Modeling Center, 2004). This file represents the expected inflow to the C-43 Watershed from Lake Okeechobee with implementation of the CERP. The inflow from CERP1RD will be significantly less than historical inflows from S-77. This is illustrated in **Figure 3B-01**.

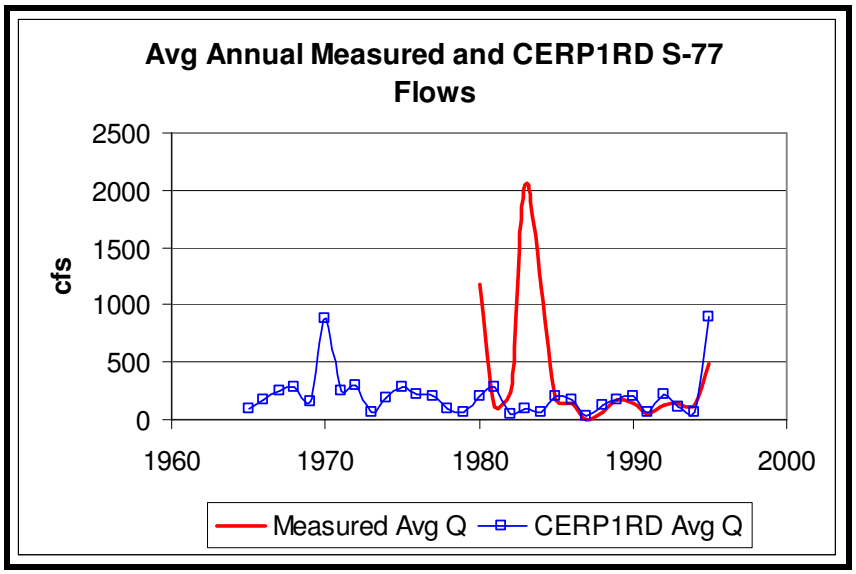


Figure 3B-01 – Average Annual Measured and CERP1RD S-77 Flows

2) Irrigation Requirements (Historical and Planned)

Irrigation of agricultural land is the major demand in the C-43 Basin (Note: this text is directly quoted from DHI, 2004). Irrigated land is 242,665 acres, or 32% of the watershed. Citrus is the main crop with significant amounts of sugar cane in the eastern portion of the watershed. Most of the irrigated farms have water use permits from SFMWD, and irrigation calculations specify the annual water use requirements of the farming operation. Sugar cane farms typically are permitted for 6-10 inches per year, and citrus farms are usually 10-12 inches per year. The sugar cane areas use less water because most of the sugar cane is grown in lowlands near Lake Okeechobee on soils with a significant amount of muck which tends to retain applied water. The citrus fields are usually on higher ground predominated by sandy soils and greater distances to the surficial aquifer. Sugar cane farms typically maintain the surficial aquifer within the one – two feet below land surface, while citrus field water tables can be as much as six feet below land surface. Irrigation demand has been evaluated separately by SFWMD using a spreadsheet program called AFSIRS. The SFWMD permit information and AFSIRS modeling were used to guide the calibration process. The model is calibrated to include withdrawals from the C-43. However, many of those withdrawals are lumped and dependent upon other model parameters. No comparisons have been made for specific irrigation pumps. Irrigation was calibrated based upon the volume of water (inches) applied to each particular land use.

Figure 3B-02 presents the annual irrigation by crop type for 1997, 1998, and 1999. Because the crop types are spread across the watershed and rainfall varies in intensity year-to-year differently across the basin, no trends in irrigation amounts can be made that correlate to rainfall. Irrigation was equal to 4-6 inches/year for the C-43 Basin. Irrigation will increase in the future condition because of additional agricultural and urban land development. Modeling of the 2050 Future Without Project Condition indicates that irrigation demand in the future will be equal to 9-30 inches/year for the C-43 Basin.

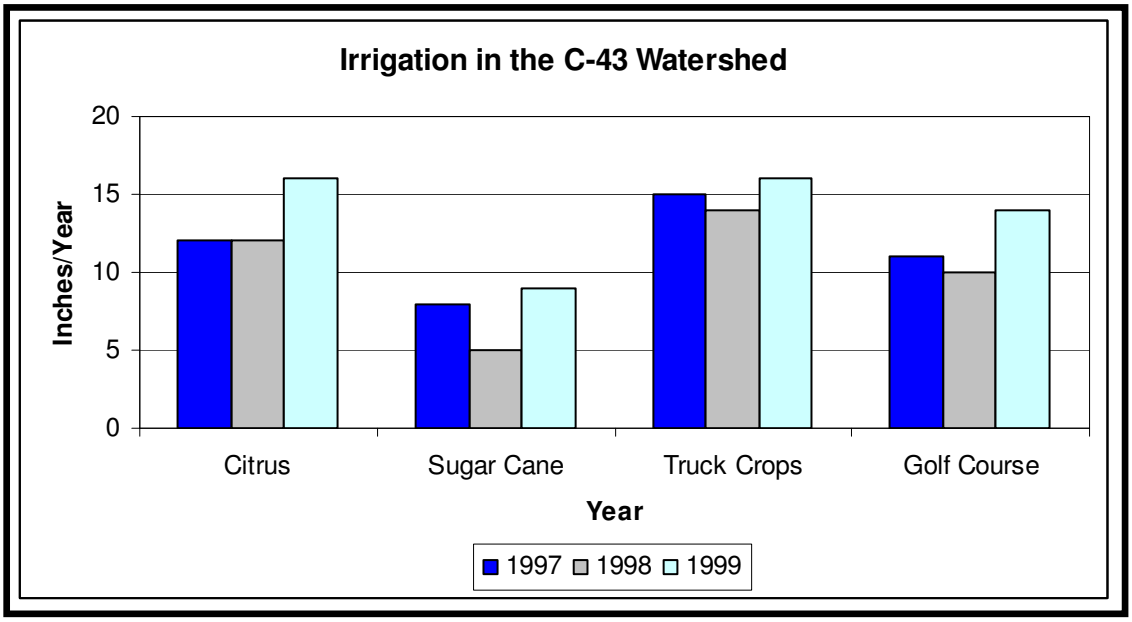


Figure 3B-02 – Annual Irrigation by Crop for the C-43 Watershed for 1997, 1998, and 1999 (from DHI, 2004)

3) Flood Flow Attenuation

DHI (2004) provided a map (see Figure 2E-01 from DHI, 2004) of detention basins used by agricultural operators in the C-43 Watershed. These detention basins provide as much as 1.3 inches of storage in portions of the C-43 Basin agricultural lands, however detention basins are not used in all agricultural areas. Furthermore, if rainfall exceeds 1.3 inches in a relatively short period of time (say 1-2 weeks), farmers will begin drainage pumping immediately. This results in relatively high flows during wet seasons that have higher than normal precipitation. Figure 2E-05 presented in **Section 2** of this report illustrates this effect. The proposed Reservoir is intended to reduce peak flows to the Caloosahatchee River estuary.

Tributaries of C-43 Canal often generate significant flows. **Figure 3B-03** illustrates the range of flows for the tributaries in the Berry Groves catchment area (note that these data are predicted flows from the calibrated MIKE SHE/MIKE 11 model of the C-43 Watershed). Daily flow can exceed 2,500 cfs, and the annual runoff ranges from 27,000 to 140,000 Ac Ft/yr. The tributaries in the immediate vicinity of the Reservoir can provide a significant percentage of the inflow to the Reservoir during wet years, but will provide relatively small amounts of flow during dry years. Utilizing runoff from Canals 1 and 2 (east of Roberts Canals) would increase runoff in the range of 40,000 to 178,000 Ac Ft/yr. It may not be possible to divert flows from Canals 1 and 2 to the southeast corner of the Reservoir inflow pumping station PS-2.

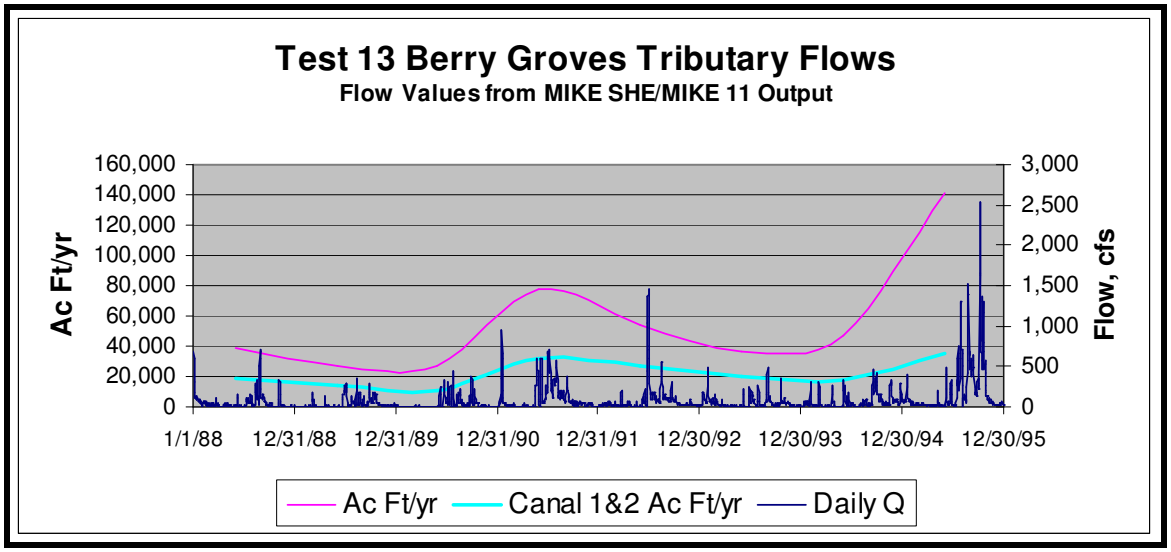


Figure 3B-03 – Berry Groves Catchment Tributary Flows

4) Water Quality

The reservoir will be designed according to the Category B projects identified in CGM 21. Specifically, the project does not provide water quality improvement features, but water quality will be addressed during design. In addition, evaluation criteria must be developed for assessing and comparing the water quality improvement of alternatives and adversely impacting water quality shall be treated as a constraint that must be avoided. Water quality parameters included in the water quality modeling effort are TP and TN.

With and Without Project N loading information will be provided for the average wet and average dry season. There will be large error bars associated with this analysis. TN removal in reservoirs and wetlands is highly variable due to de-nitrification in bottom sediments, nitrogen fixation when N:P ratios are low and sufficient bio-available P is present in the water column. The reservoir retention time is considered during the water quality calculations.

If concentrations of key water quality constituents of the water released from the basin will be higher than the concentrations of those constituents in the C-43 Canal, then a stormwater treatment area could be required for the reservoir. Because water quality data does not exist for a reservoir of this size and depth variation in South Florida, the dynamics of pollutant retention is not known. Therefore, the modeling of water quality will utilize a number of potential removal mechanisms and a range of possible outflow concentrations will be reported. The analysis will include nutrient inputs from the tributaries south of the reservoir and nutrient inputs from the C-43 Canal. The reservoir

water quality assessment will consider the routing of water within the proposed reservoir cells and the changes in depth of water (which affects the hydraulic retention time).

5) Water Release Volume and Timing

The pre-condition analysis presented above in **Section 3 B 1) and 3)** indicates that available runoff from the C-43 Watershed ranges from approximately 160,000 acre-feet to 580,000 acre-feet. It should therefore be expected that there will be years that the Reservoir storage will not be filled and there will be years when the Reservoir could be filled many times over. During an extended drought, the Reservoir does not have sufficient capacity to meet minimum flow requirements at S-79. During a wetter than average year, the Reservoir will reach full capacity before the wet season is over and therefore, it can no longer be used to reduce peak flows during late season events. This Reservoir is one component of an overall plan for restoring hydrology of the C-43 Basin. Additional components of the C-43 Project Implementation Report (in process by others) will provide additional benefits that cannot be provided by this project.

One of the major objectives of the proposed Reservoir is to reduce peak flows to the Tidal C-43 Canal and to increase baseflows. This change in the frequencies of high and low flows is expected to improve aquatic habitat in the Caloosahatchee River estuary. **Figure 3B-04** presents the existing, 2050 Future Without Project, and target flow distributions at S-79. The target flow ranges were recommended in a study by SFWMD (SFWMD, 2003) and are based on flow target EST05. The existing flows are modeled flows for the 1988 – 1995 period. The existing flows are the same as "pre-condition" flows.

In the modeling of the proposed Reservoir, a number of different release rates were tested (see **Section 6** for additional discussion) ranging from 300 – 600 cfs. The predicted flows at S-79 with the Reservoir releases were averaged monthly and compared to the target flows presented in **Figure 3B-04**. The release structure dimensions and operating rules were gradually modified with the objective of matching the target percentages as close as possible. See **Section 6** for additional information.

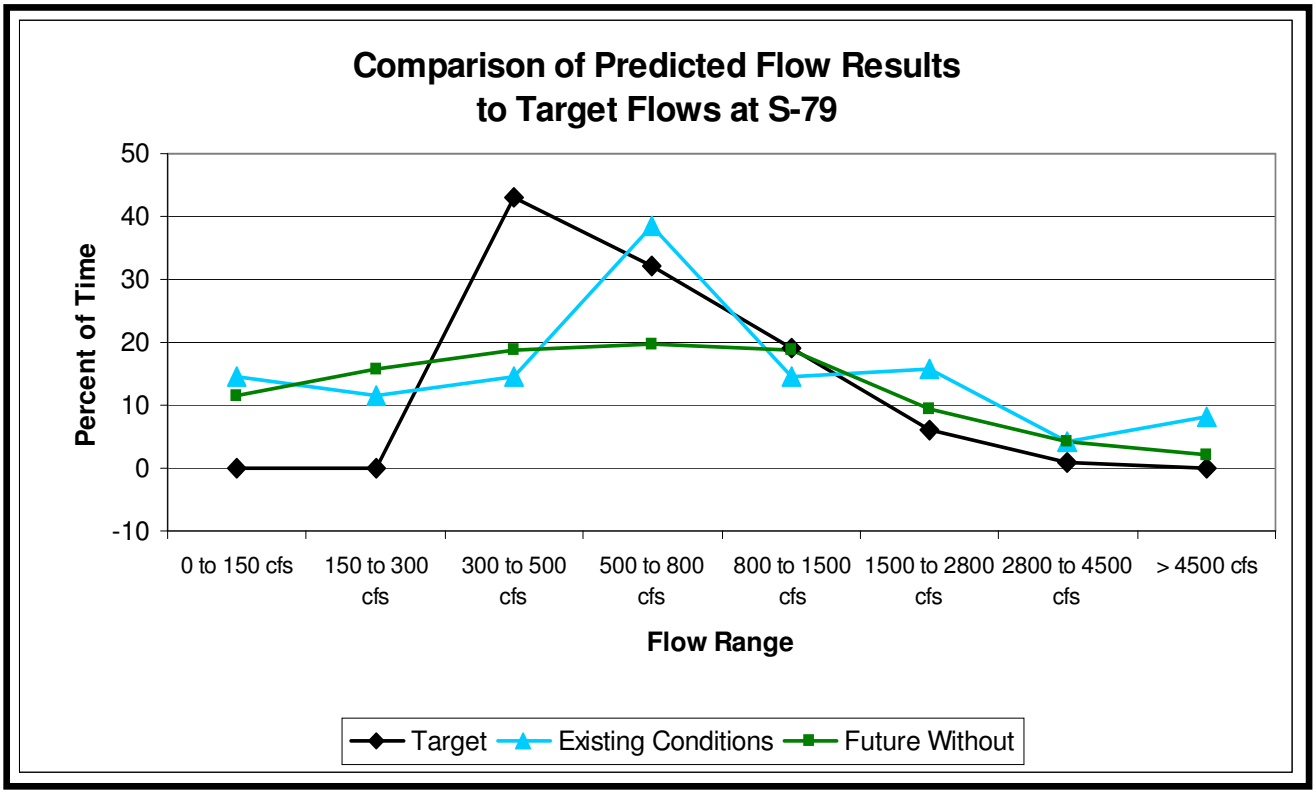


Figure 3B-04 – Existing, 2050 Future without Project, and Target Flow Distributions at S-79

C. Service Life

According to EM-1110-2-3104, EM-1110-2-3105, and Major Pumping Station Engineering Guidelines, the design life of the pump stations will be 50 years. By following the guidelines outlined in these reference documents, this design life can be achieved. The architectural and structural design of the pump stations will incorporate elements that require minimum maintenance and repair.

The mechanical equipment will require rehabilitation or replacement at least once during the 50 year service life. The engines and pumps will operate intermittently, in the range of 500 to 2000 hours per year. At this usage, following the manufacturer's maintenance schedule, the engines would go through one major overhaul in the 50 year service life. With proper preventative maintenance, the engine blocks should last beyond the 50 year service life of the pump station. Pump impellers and shafts will be stainless steel to provide long service life.

D. Project Work Limits

As discussed in the Survey and Mapping section of this report one of the survey tasks was to compile an exhibit that represents the lands owned by the SFWMD. **Figure 2I-01** in **Section 2** of the Appendices illustrates those findings.

Regardless of which alternative is selected for the Reservoir Project the work limits, with one exception, will fall within the land that is or will be owned and/or controlled by the SFWMD. The exception will be in the area along the NE Rim Ditch centerline which is land that is owned by A. Duda & Sons. The proposed construction in that area includes widening and deepening the existing NE Rim Ditch and incorporating it into the proposed perimeter/seepage canal system around the Reservoir. An easement agreement between the SFWMD and A. Duda & Sons will need to be in place prior to the NE Rim Ditch expansion.

In general the project work limits for Alternative A are described as follows:

Along the north side of Reservoir the limits will be the south line of the existing FP&L easement within the east half of Section 31 and Sections 32, 33, 34, 35 and 36 of Township 43S and Range 28E. The east limits will be along the east lines of Section 36 of Township 43S and Range 28E and Sections 1 and 12 of Township 44S and Range 28E. The southern limits will be the south line of Sections 8, 9, 10, 11 and 12 of Township 44S and Range 28E. Generally the western work limits will be defined by the west line of the eastern half of Section 32 of Township 43S and Range 28E and the west line of the eastern half of Sections 6 and 7 of Township 44S and Range 28E. The work limits in the southwest corner of the Reservoir will be contained within the land owned by the SFWMD and is not intended to impact the existing pump station owned by A. Duda & Sons.

The work limits for Alternative B are identical to Alternative A with the following additions:

The north limits will be the south line of the existing FP&L easement within Section 31 and the west ¾ of Section 32 of Township 43S and Range 29E. The east limit will be along the east line of the west ¾ of Section 32 of Township 43S and Range 29E and the west ¾ of Section 5 of Township 44S and Range 29E. The east work limits will continue to be along the east line of Section 12 of Township 44S and Range 28E. The southern limits will be expanded to include the south line of Sections 5 and 6 of Township 44S and Range 29E.

There will be work done beyond the described limits but that work will be limited to improvements to or in some of the existing channels and canals affected by the Reservoir construction. Channel improvements are expected to occur to the Townsend Canal and at the existing S.R. 80 roadway bridge, and the addition of a fixed weir in each of the Ft. Simmons Branch, the Banana Branch and the Crawford Canal and channel improvements to the LPDD Canal between the Reservoir and S.R. 29.

E. Units and System of Measurement

The units and system of measurement will be in English.

F. Codes and Standards

1) Site Work Design Criteria

Codes and standards: design and specification of all work shall be in accordance with latest laws and regulations of the federal SFWMD, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *AASHTO - American Association of State Highway and Transportation Officials*
- *ADAAG – ADA Accessibility Guidelines for Buildings and Facilities*
- *AI - Asphalt Institute.*
- *ANSI - American National Standards Institute, Inc.*
- *ASTM - American Society for Testing and Materials*
- *FDOT - Florida Department of Transportation.*
- *FHWA – Federal Highway Administration*
- *MUTCD - Manual on Uniform Traffic Control Devices*
- *SFWMD - South Florida Water Management District*
- *UFAS - Uniform Federal Accessibility Standards*
- *USACE - United States Department of the Army Corps of Engineers*

2) Geotechnical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *American Society for Testing and Materials (ASTM).*

- *Florida Building Code – 2003.*
- *Florida Department of Transportation (FDOT).*
- *South Florida Water Management District (SFWMD).*
- *United States Department of the Army Corp of Engineers (COE).*
  - *EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams*
  - *EM 1110-2-1902, Slope Stability*
  - *EM 1110-2-1913, Appendix G, Use of Soil Cement for Levee Protection*
- *Federal Highway Administration (FHWA), Publication No. NGHI-00-0045 "Soil and Foundation Workshop".*

Recommended and recognized standards from other organizations shall be used where required and approved to serve as guidelines for the design, fabrication, and construction when not in conflict with the standards referenced herein.

3) Architectural Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *Florida Building Code – Latest Edition*
- *Florida Fire Prevention Code – Latest Edition*
- *NFPA 13 - Automatic Sprinkler System*
- *NFPA 72 - National Fire Alarm Code*
- *NFPA 80 - Standard for Fire Doors and Fire Windows*
- *NFPA 101 - Life Safety Codes*
- *OSHA 29 CFR*
- *Florida Accessibility Code – Latest Edition*
- *ADAAG – Americans with Disabilities Act Architectural Guidelines (Title III)*

- *AAMA 1503 (1998) – Voluntary Test Method for Thermal Transmittance and Condensation Resistance of Windows, Doors and Glazed Wall Sections*
- *ASTM C 755 (2002) - Water Vapor Retarders for Thermal Insulation*
- *ASTM E 108 (2000) - Fire Tests of Roof Coverings*
- *ASTM E 413 (1987; R 1999) - Rating Sound Insulation*
- *ASTM E 96 (2000e1) - Water Vapor Transmission of Materials*
- *ASTM E 966 (2002) - Airborne Sound Insulation of Building Facades and Facade Elements*
- *UL - Underwriters' Laboratories, Inc.*

4) Structural Design Criteria

(a) General References

- *CERP Standard Design Manual, June 6, 2003*
- *ER 1110-2-1150, Engineering and Design for Civil Works Projects*
- *ER 1110-2-1200, Plans and Specifications for Civil Works Projects*
- *SFWMD Standard Details and Guideline Drawings*

(b) Stability Design

- *EC 1110-2-6058, Stability Analysis of Concrete Structures*
- *EM 1110-2-2100, Stability Analysis of Concrete Structures (draft)*
- *EM 1110-2-2504*, *Design of Sheet Pile Walls*
- *EM 1110-2-2503*, *Design of Sheet Pile Cellular Structure*
- *EM 1110-2-3104, Stability Design Criteria for Pump Stations*
- *ETL 1110-2-256, Sliding Stability for Concrete Structures*
- *ETL 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures*
- *ETL 1110-2-352, Stability of Gravity Walls, Vertical Shear*

(c) Wind Design

- *SFWMD Major Pumping Station Design Guidelines, Section 2.7.2*
- *Florida Building Code, 2001 with 2003 Revisions*
- *ASCE 7-98, Minimum Design Loads for Buildings and Other Structures*

(d) Seismic Design

- *TI 809-04, Seismic Design for Buildings*
- *FEMA 302, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*
- *International Building Code (IBC) 2000*
- *EM 1110-2-6050, Response Spectra and Seismic Analysis for Concrete Hydraulic Structures*
- *FEMA 350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*
- *AISC, Seismic Provisions for Structural Steel Buildings*

(e) Concrete Structures Design

- *EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures*
- *EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures*
- *EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures*
- *EM 1110-2-2005, Standard Practice for Shotcrete*
- *EM 1110-1-2009, Architectural Concrete*
- *ACI 315, Details and Detailing of Concrete Reinforcement*
- *ACI 318-02, Building Code Requirements for Structural Concrete*
- *ACI Manual of Concrete Practice*
- *Concrete Reinforcing Steel Institute Handbook*
- *PCI Design Handbook, Pre-cast and Pre-stressed Concrete*

(f) Steel Structures Design

- *EM 1110-1-2101, Working Stresses for Structural Design*
- *EM 1110-2-2105, Design of Hydraulic Steel Structures*
- *AISC, Manual of Steel Construction, ASD Ninth Edition*
- *AISC, Manual of Steel Construction, Load & Resistance Factor Design*
- *Steel Deck Institute, Design Manual for Composite Decks, Form Decks and Roof Decks*
- *Steel Joist Institute, Standard Specifications and Load Tables For Steel Joists and Joist Girders*
- *American Welding Society, Structural Welding Code-Steel*

- *American Welding Society, Structural Welding Code-Stainless Steel*

(g) Aluminum Structures Design

- *The Aluminum Association, Aluminum Design Manual*
- *American Welding Society, Structural Welding Code-Aluminum*

(h) Masonry Structures Design

- TM 5-809-3, *Masonry Structural Design for Buildings*
- *ACI 530*, *Building Code Requirements for Masonry Structures*
- *ACI 530.1*, *Specification for Masonry Structures*

(i) Service Bridge Design

- *American Association of State Highway Traffic Officials (AASHTO), Standard Specification of Highway Bridges*
- *Florida Department of Transportation (FDOT*), *Standard Specifications for Road and Bridge Construction*

(j) Culvert Design

- *EM 1110-2-2902, Conduits, Culverts and Pipes*
- *FDOT, Standard Specifications for Road and Bridge Construction*

(k) Concrete Channel Design

- *EM 1110-2-2007*, *Structural Design of Concrete Lined Flood Control Channels*

(l) Pumping Station Design

- *EM 1110-2-3102*, *General Principles of Pumping Station Design and Layout*
- *EM 1110-2-3104*, *Structural and Architectural Design of Pumping Stations*
- *TI 809-02*, *Structural Design Criteria for Buildings*
- *TM 5-809-12*, *Concrete Floor Slabs on Grade Subjected to Heavy Loads*

(m) Spillway Design

- *EM 1110-2-2400*, *Structural Design of Spillways and Outlet Works*
- *EM 1110-2-2701*, *Vertical Lift Gates*

(n) Retaining Wall Design

- *EM 1110-2-2502*, *Retaining and Flood Walls*
- *EM 1110-2-2504*, *Design of Sheet Pile Walls*
- *TR-01-1*, *State of the Practice in the Design of Tall, Stiff, and Flexible Tieback Retaining Walls*

(o) Closure Structure Design

- *EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Protection Projects*

(p) Highway Bridges

- *AASHTO LRFD Bridge Design Specifications*, *2nd Edition, 1998, with Interims through 2004*
- *FDOT Structures Manual*, *January 2003*
- *FDOT Standard Specification for Road and Bridge Construction*, 2004

5) Special Mechanical Equipment Design Criteria

Design References

- *HI 9.8, Pump Intake Design, 1998, Hydraulic Institute.*
- *EM 1110-2-3102, General Principles of Pumping Station Design and Layout, 1995, US Army Corps of Engineers.*
- *EM 1110-2-3104, Structural and Architectural Design of Pumping Stations, 1989, US Army Corps of Engineers.*
- *EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, 1999, US Army Corps of Engineers.*
- *Major Pumping Station Engineering Guidelines, 2004, South Florida Water Management District.*
- *CERP Standard Design Manual, 2003, US Army Corps of Engineers Jacksonville District and South Florida Water Management District.*

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *Hydraulic Institute Standard 9.8-1998 – Pump Intake Design.*
- *Hydraulic Institute Standard 2.1-2.6-2000 – Standards for Vertical Pumps.*
- *EM 1110-2-3102, General Principles of Pumping Station Design and Layout, 1995, US Army Corps of Engineers.*
- *EM 1110-2-3104, Structural and Architectural Design of Pumping Stations, 1989, US Army Corps of Engineers.*
- *EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, 1999, US Army Corps of Engineers.*
- *Major Pumping Station Engineering Guidelines, 2004, South Florida Water Management District.*
- *CERP Standard Design Manual, 2003, US Army Corps of Engineers Jacksonville District and South Florida Water Management District.*
- *AWWA Manual M11 Steel Pipe: A Guide for Design & Installation, Fourth Edition.*
- *Diesel Engine Manufacturers Association (DEMA)*
- *American Society of Testing and Materials (ASTM)*
- *Heat Exchange Institute (HEI)*
- *American Gear Manufacturers Association (AGMA)*
- *American Bearing Manufacturers Association (ABMA)*
- *American Petroleum Institute (API)*
- *Underwriters Laboratories Inc. (UL)*
- *National Fire Protection Association (NFPA)*
- *American Association of State Highway and Transportation Officials (AASHTO)*
- *American Society of Mechanical Engineers (ASME)*

6) HVAC, Plumbing and Fire Suppression

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *ICC International Building code - 2003.*
- *ICC International Mechanical Code - 2003.*
- *ICC International Plumbing Code - 2003.*
- *ICC International Fire Code - 2003.*
- *AABC - Associated Air Balance Council.*
- *ADC - Air Diffusion Council.*
- *AGA - American Gas Association.*
- *AMCA - Air Movement and Control Association.*
- *ANSI - American National Standards Institute.*
- *ARI - Air Conditioning and Refrigeration Institute.*
- *ASHRAE - American Society of Heating, Refrigeration, and Air Conditioning engineers.*
- *ASME - American Society of Mechanical Engineers.*
- *ASME - American Society of Plumbing Engineers.*
- *ASTM - American Society for Testing Materials.*
- *AWS - American Welding Society.*
- *AWWA - American Water Works Association.*
- *EJMA - Expansion Joint Manufacturers Association.*
- *ETL - Electrical Testing Laboratory.*
- *HI - Hydraulics Institute.*
- *MSS - Manufacturers Standardization Society of the Valve and Fittings Industry.*

- *NBS - National Bureau of Standards.*
- *NEBB - National Environmental Balancing Bureau.*
- *NEMA - National Electrical Manufacturer's Association.*
- *NFPA - National Fire Protection Association.*
- *OSHA - Occupational Safety and Health Administration.*
- *PFI - Pipe Fabrication Institute.*
- *SMACNA - Sheet Metal and Air Conditioning Contractors' National Association.*
- *UL - Underwriters' Laboratories, Inc.*

7) Plumbing Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *ICC International Building code - 2003.*
- *ICC International Mechanical code - 2003.*
- *ICC International Plumbing Code - 2003.*
- *AGA - American Gas Association.*
- *ANSI - American National Standards Institute.*
- *ASHRAE - American Society of Heating, Refrigeration, and Air Conditioning Engineers.*
- *ASME - American Society of Mechanical Engineers.*
- *ASPE – American Society of Plumbing Engineers.*
- *ASTM - American Society for Testing Materials.*
- *AWS - American Welding Society.*
- *AWWA - American Water Works Association.*
- *ETL - Electrical Testing Laboratory.*

- *HI - Hydraulics Institute.*
- *MSS - Manufacturers Standardization Society of the Valve and Fittings Industry.*
- *NBS - National Bureau of Standards.*
- *NEMA - National Electrical Manufacturer's Association.*
- *NFPA - National Fire Protection Association.*
- *OSHA - Occupational Safety and Health Administration.*
- *PFI - Pipe Fabrication Institute.*
- *UL - Underwriters' Laboratories, Inc.*

8) HVAC Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *ICC International Building code - 2003.*
- *ICC International Mechanical Code - 2003.*
- *AABC - Associated Air Balance Council.*
- *ADC - Air Diffusion Council.*
- *AGA - American Gas Association.*
- *AMCA - Air Movement and Control Association.*
- *ANSI - American National Standards Institute.*
- *ARI - Air Conditioning and Refrigeration Institute.*
- *ASHRAE - American Society of Heating, Refrigeration, and Air Conditioning Engineers.*
- *ASME - American Society of Mechanical Engineers.*
- *ASTM - American Society for Testing Materials.*

- *AWS - American Welding Society.*
- *EJMA - Expansion Joint Manufacturers Association.*
- *ETL - Electrical Testing Laboratory.*
- *HI - Hydraulics Institute.*
- *MSS - Manufacturers Standardization Society of the Valve and Fittings Industry.*
- *NBS - National Bureau of Standards.*
- *NEBB - National Environmental Balancing Bureau.*
- *NEMA - National Electrical Manufacturer's Association.*
- *NFPA - National Fire Protection Association.*
- *OSHA - Occupational Safety and Health Administration.*
- *PFI - Pipe Fabrication Institute.*
- *SMACNA - Sheet Metal and Air Conditioning Contractors' National Association.*
- *UL - Underwriters' Laboratories, Inc.*

9) Fire Protection and Detection Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *ICC International Building Code - 2003.*
- *ICC International Fire Code – 2003*
- *NFPA - National Fire Protection Association.*
- *OSHA - Occupational Safety and Health Administration.*
- *UL - Underwriters' Laboratories, Inc.*

10) Electrical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *ANSI C2, National Electrical Safety Code.*
- *ANSI C84.1, Electric Power Systems and Equipment – Voltage Ratings.*
- *ANSI A117.1, Buildings and Facilities – Providing Accessibility and Usability for Physically Handicapped People.*
- *ANSI/IEEE Std. 242, Recommended Practice for Protection and Coordination of Industrial and Commercial Power Systems. (The Buff book.)*
- *IES Lighting Handbook, Reference Volume and Application Volume.*
- *NEMA - National Electrical Manufacturer's Association.*
- *NFPA 70, National Electrical Code.*
- *NFPA 72, National Fire Alarm Code.*
- *NFPA 78, Lightning Protection Code.*
- *NFPA 101, Code for Safety to Life from Fire in Buildings and Structures.*
- *Uniform Federal Accessibility Standards.*
- *UL 268, Smoke Detectors for Fire Protective Signaling Systems.*
- *U.S. Army Corps of Engineers Technical Standards, TI-800-01.*
- *IEEE C62.41 Surge Voltage in Low Voltage AC Power Circuits.*

11) Instrumentation and Controls Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *NEMA 250 (1997) Enclosures for Electrical Equipment (1000 Volts Maximum)*

- *NEMA ICS 1 (2000) Industrial Control and Systems: General Requirements.*
- *NEMA ICS 2 (2000) Industrial Control and Systems: Controllers, Contactors, and Overload Relays Rated 600 volts.*
- *NEMA ICS 4 (2000) Industrial Control and Systems: Terminal Blocks.*
- *NEMA ICS 6 (1993; R 2001) Industrial Control and Systems: Enclosures.*
- *NFPA 70 (2002) National Electrical Code.*
- *UL 1059 (2001) Terminal Blocks.*
- *UL 508 (1999; Rev thru Dec 1002) Control Equipment.*
- *ANSI C37.90 (1989) Relays and Relay Systems Associated with Electric Power Apparatus.*
- *ANSI C37.90.1 (1989) Surge Withstand Capability (SWC) Test for Protective Relays and Relay Systems.*
- *EIA ANSI/EIA/TIA-232-F (2002) Interface Between Data Terminal Equipment and Data Circuit-Terminating Equipment Employing Serial Binary Data Interchange*
- *IEEE C62.41 (1991) Recommended Practice for Surge Voltages in Low-Voltage AC Power Circuits*
- *IEEE Std 100 (2000) IEEE Standard Dictionary of Electrical and Electronics Terms.*
- *IEEE Std 802.4 (1990; R 1995) information Processing Systems, Local Area Networks: Part 4: Token Passing Bus Access Method and Physical Layer Specifications.*
- *IEC 61131-3 (2003) Programmable Controllers – Part 3: Programming Languages.*

12) Telemetry System Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the State of Florida and the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- *EIA ANSI/TIA/EIA-222-F (1996) Structural Standards for Steel Antenna Towers and Antenna Supporting Structures.*
- *EIA ANSI/EIA/TIA-232-F (2002) Interface Between Data Terminal Equipment and Data Circuit Terminating Equipment Employing Serial Binary Data Interchange.*
- *EIA ANSI/EIA-310-D (1992) Racks, Panels, and Associated Equipment.*
- *47 CFR 15 Radio Frequency Devices.*
- *CERP Standard Design Manual, 2003, US Army Corps of Engineers Jacksonville District and South Florida Water Management District.*

Section 4 – Regulatory Considerations

This Section of the BODR summarizes the regulatory and permitting requirements as they apply to the Reservoir. A significant amount of work has been initiated regarding the permitting effort for this project. The emphasis of subsequent work activities associated with regulatory and permitting requirements will be to maintain appropriate communications with the U.S. Army Corps of Engineers (USACE) and other Federal agencies, state, and local

The following is a list of applicable permits and associated guidance that has been assembled for this project.

A. Federal Permits

- ***Section 404 – Dredge & Fill Permit***

  Contact: Ms. Tori White

  U.S. Army Corps of Engineers

  South Permits Section Office, SESAJ-RD-SS

  4400 PGA Boulevard, Ste. 500

  Palm Beach Gardens, FL 33410-2933

- ***Coast Guard Permit***

  Contact: Mr. Joe B. Embers

  Chief Planning and Marine Information

  Aide to Navigation and Waterways Management Branch

  Seventh Coast Guard District

  909 SE First Avenue, Ste 406

  Miami, FL 33131

B. State Permits

- ***Comprehensive Everglades Restoration Plan Act Regulation Act (CERPRA), Chapter 373.1501 approval order and Chapter 373.1502 construction and operation permit***

  Contact: Ms. Temperince M. Morgan

  Environmental Manager

  Florida Department of Environmental Protection

  Water quality Standards and Special Project Program

  2600 Blair Stone Rd, MS 3560

  Tallahassee, FL 32399

- ***Well Construction Permit***
- ***Consumptive Use Permit***

  Contact: Ms. Temperince M. Morgan
  Environmental Manager
  Florida Department of Environmental Protection
  Water quality Standards and Special Project Program
  2600 Blair Stone Rd, MS 3560
  Tallahassee, FL 32399

  Mr. Thomas Colios
  Senior Supervising Hydrogeologist
  South Florida Water Management District
  3301 Gun Club Road
  West Palm Beach, FL 33406

- ***Florida Inland Navigation District (FIND)***

  Contact: Mr. David Roach
  Executive Director
  Florida Inland Navigation District
  1314 Marcinski Road
  Jupiter, FL 33477

- ***Florida Department of Transportation (FDOT) Right-of-Way Permit***

  Contact: Michael Rippe
  Director, Southwest Area Office
  Florida Department of Transportation, District 1
  2295 Victoria Avenue, Ste. 292
  Ft. Myers, FL 33901

- ***Clean Air (Title V) Permit***

  Contact: Scott Sheplak, Program Administrator
  Florida Department of Environmental Protection
  Division of Air Response Management
  2600 Blair Stone Rd, MS 3560

Tallahassee, FL 32399

- ***National Pollution Discharge Elimination System (NPDES) Generic Permit for Stormwater Discharge from Large and Small Construction Activities***

  Contact: FDEP NPDES Stormwater Program (850-245-7522),
  http://www.dep.state.fl.us/water/stormwater/npdes/construction3.htm#permit

  Ms. Temperince M. Morgan

  Environmental Manager

  Florida Department of Environmental Protection

  Water Quality Standards and Special Project Program

  2600 Blair Stone Rd, MS 3560

  Tallahassee, FL 32399

- ***National Pollution Discharge Elimination System (NPDES) General Permit for the Discharge of Produced Ground Water from any Non-Contaminated Site Activity***

  Contact: Ms. Temperince M. Morgan

  Environmental Manager

  Florida Department of Environmental Protection

  Water Quality Standards and Special Project Program

  2600 Blair Stone Rd, MS 3560

  Tallahassee, FL 32399

- ***Florida Department of Environmental Protection (FDEP) Water Facilities Regulation***

  Contact: Ms. Temperince M. Morgan

  Environmental Manager

  Florida Department of Environmental Protection

  Water Quality Standards and Special Project Program

  2600 Blair Stone Rd, MS 3560

  Tallahassee, FL 32399

- ***Florida Fish and Wildlife Conservation Commission (FWCC) Migratory Bird Nest Removal Permit***

  Contact: Protected Species Permit Coordinator

  Florida Fish and Wildlife Conservation Commission

  Division of Wildlife

620 South Meridian Street

Tallahassee, FL 32399

C. Local Permits

- Hendry County Building Permit

D. Other Considerations

- *Manatee Protection*
- *Florida Endangered Species, Threatened Species, and Species of Special Concern*
- *Habitat Management Guidelines for the Bald Eagle in the Southeast Region*
- *Available Options to Address the Presence of Gopher Tortoises on Lands Stated for Development*
- *Osprey Nest Removal Policies*
- *Florida Breeding Birds Atlas Guidelines for Safe Dates for Selected Breeding Codes*

A compendium of proposed permits and regulatory compliance guidance documents has been prepared as reference for this and other Acceler8 projects. The Acceler8 Permitting Support Group has assembled a comprehensive permitting manual for all Acceler8 projects. The manual provides a summary of all permits that could potentially be required for this and other Acceler8 projects. The document also includes permit application forms, contact information, and other guidance information that project teams can utilize in preparation for permits for Acceler8 projects. Subsequent phases of project development will include preparation of permitting documents and submission of all associated back-up required for permitting purposes.

Fish and wildlife enhancement recommendations that are compatible with Reservoir operations may be considered in the final design. A 2004 Draft Initial CERP Update Planning Aid Letter (U.S. Fish and Wildlife Service) which focuses on compatibility with reservoir and STAs facilities management and operation for their principal purposes of water storage and nutrient removal identifies examples of enhancement such as maintaining deep water zones, perpendicular flow features, underwater structures, diverse plan communities (e.g., submerged aquatic vegetation) and vegetative buffers may be incorporated into the reservoir design without compromising reservoir function.

E. Land Use/Zoning

The current land use for the Reservoir site is designated as “Agriculture”. The planned use for this site is proposed as “Water Resource – Reservoir”. Hendry County has developed a Comprehensive Plan amendment known as “West LaBelle Land Use Study” to address growth in the area, and has submitted the Study to the Florida Department of Community Affairs. The study includes approximately 18,000 acres of land located west

of LaBelle, and includes the project area. In addition, Hendry County is proposing to amend it's Comprehensive Plan to create new land use designations.

Development of the Reservoir Project will require a land use change from Hendry County for the intended land use. All proposed land use changes must maintain consistency with the ongoing Comprehensive Plan Amendment activities in Hendry County. Ongoing correspondence with Hendry County will be required during design development. The contact person in Hendry County is:

Ms. Mary Ham

Director, Planning & Zoning Department

25 E Hicpochee Avenue

P.O. Box 2340

LaBelle, FL 33975

F. PIR Coordination & NEPA EIS Coordination

Design development for the Reservoir Project requires coordination with the ongoing PIR as being prepared cooperatively by the USACE and the SFWMD (PIR Team). **Figure 4F-01** in **Section 4** of the Appendices outlines a flow diagram that has been developed to indicate the key interdependencies between the Acceler8 design development strategy and the PIR plan formulation process. Other coordination processes are also interwoven into the flow diagram, including USFWS Coordination, U.S. Department of Interior (DOI) coordination, and FDEP coordination. The following is a summary of key coordination points between the Acceler8 team as it relates to the PIR and other associated processes:

- Coordination of project schedules and key milestones
- Sharing of project data and elimination of redundancy of effort
- Coordination between Draft PIR/EIS development and 404 regulatory process to ensure that the publicly-noticed review periods for the 404 Permit Application and Draft PIR/EIS can be concurrent
- Coordination between the Acceler8 Team, PIR Team, and U.S.F.W.S. to ensure that the Biological Assessment and Section 7 consultation are completed and reviewed in a timely manner
- Coordination in scheduling SFWMD Governing Board (GB) briefings for the Selected Alternative Plan, Draft PIR/EIS, and SFWMD GB approval of the Letter of Support with PIR activities to ensure that GB briefings do not create schedule delays.
- Coordination between the Acceler8 Team, PIR Team, and the FDEP regarding 1501 and 1502 permits.

- Coordination between Acceler8 Team and PIR Team NEPA experts to ensure that post-NEPA design changes and enhancements do not trigger a need to re-open NEPA.

- Between the Acceler8 Team, PIR Team and DOI to make sure that land use change is set to take effect immediately upon 404 permit receipt.

- Coordination with FP&L on utility relocations.

The following is a list of key activities that have been completed to date for the Reservoir site:

- Completion of Phase I and Phase II Environmental Assessments, including identification of 17 sites requiring remedial actions

- Ongoing Ecological Risk Assessment, including soil inversion for Copper impacted soils

- Threatened and Endangered Species Evaluation

- Wetland Evaluation Report and Wetlands Rapid Assessment Procedure (WRAP) assessment completed

- Phase I and Phase II Cultural Resources Assessment

- 1501 Pre-Application Meeting

- Planning Aid Letters provided by USFWS in April 2002 & August 2003

- State Historical Preservation Office (SHPO) coordination in process

Subsequent coordination with the PIR team and the agencies will continue through subsequent design phases. The following is a list of key personnel associated with the PIR process and NEPA/EIS coordination:

- SFWMD PIR Project Manager: Janet Starnes

- USACE PIR Project Manager: Debbie Wegmann

- USACE PIR Professional Engineer Lead: Graham Story

- USACE PIR Planning Technical Lead: David Apple

- USACE Environmental Lead: Susan Conner

- USACE Economics Lead: Kevin Wittman

- USACE Regulatory Lead: Tori White

- USACE Geotechnical Lead: Margie Zhang
- USACE Real Estate Lead: Tony Ayuso
- USACE H&H Lead: Brian Cornwell
- USACE Design Lead: Stan Pilinski
- USFWS Lead: Joyce Mazourek
- Florida Fish & Wildlife Conservation Commission: James Beever

Section 5 – Hydrology

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 5 of the Appendices.

A. Design Criteria

The design criteria for required freeboard height of dams have as an objective that the dam will not create a threat of loss of life or inordinate property damage. Failure of the dam at the Reservoir would create such a threat. Therefore, the dam height will be designed with appropriate freeboard and structural capacity, such that the dam will safely pass the Inflow Design Flood (IDF) computed from Probable Maximum Precipitation (PMP) occurring over the watershed above the dam site (USACE regulation ER 1110-8-2 (FR), March 1991). A freeboard analysis was performed using the PMP for 6, 12, 24, 48 and 72 hours as a design storm. A 120 mph maximum design wind speed (adjusted wind speed of 146-153 mph for different fetch lengths) near the project site per Florida Building Code was used in the analysis with the reference elevation as the normal operating level. **Section 5 D 4)** discusses this analysis in more detail. **Section 8** discusses the geotechnical/structural capacity issues.

B. Hazard Classification

With the passage of the National Dam Safety Program Act of 1996, Public Law 104-303, the ad hoc Interagency Committee On Dam Safety (ICODS) and its subcommittee were reorganized to reflect the objectives and requirements of this law. In 1998, the newly convened Guidelines Development Subcommittee completed work on the update of five major guidelines, including one for "Hazard Potential Classification System for Dams" (FEMA 94, October 1998).

The hazard potential for a dam is defined as the possible adverse incremental consequences that result from the release of water or stored contents due to failure of the dam or operation of the dam or appurtenances. The hazard potential of a dam is based on consideration of the effect of a failure during both normal and flood flow conditions. There are three classification levels of the hazard potential for a dam: low hazard potential, significant hazard potential and high hazard potential (FEMA 94, October 1998).

Dams assigned a low hazard potential classification are those dams where failure or operation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

Dams assigned a significant hazard potential classification are those dams where failure or operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns.

Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

Dams assigned a high hazard potential classification are those dams where failure or misoperation will probably cause loss of human life.

If the dam of a Reservoir were to fail during a flood flow condition, and the resultant incremental flood flow would create a potential for loss of life or serious damage to property, the dam would have high hazard potential classification.

Since Highways S.R. 80 and S.R. 29 are adjacent to the Reservoir as well as some residential development, a high hazard potential classification was identified for the analysis. The PMP event was designated as the design storm for the Reservoir with a high hazard potential classification.

The breach inundation study was completed using projected land use data for 2050. The data suggests that the inundated area will consist primarily of a high density urban land use. The data does not suggest how many permanently occupied buildings would be in any particular area.

C. Design Storms and Floods

The IDF for the Reservoir was developed in accordance with ER 1110-8-2 (FR), March 1991. In Section 7b, Safety Dam Standard 1 requires structural designs that will safely pass an IDF computed from PMP occurring over the watershed above the dam. Since the Reservoir isn't a classic dam with a contributing watershed above it, the contributing watershed area used to compute the IDF was simply the area of the impoundment itself. The upper limit of the IDF is the probable maximum flood. The all-season PMP estimates for specified durations were developed using maximum observed areal precipitation from the U.S. Weather Service's Hydrometeorological Report(HMR) No. 51 (Schreiner and Riedel, 1980). Figures in HMR No. 51 show that the smaller the drainage area, the larger the PMP values. According to the HMR No. 51, the PMP values for the Reservoir should be based on the southernmost isolines shown on the figures. The rise from the normal pool elevation resulting from the IDF due to the PMP for 6, 12, 24, 48 and 72 hours was estimated to be about 32", 38.7", 47.1", 51.8" and 55.7 inches, respectively. These data were added to the normal pool level for the condition of maximum flood level in estimating the freeboard. The overflow spillway crest will be set 11 inches above the normal pool elevation based on the SFWMD permitting requirement to store a 25-year, 72-hour rainfall event that occurs at the time the Reservoir is at the normal pool elevation.

D. Reservoir

Modeling work to determine the Reservoir size and operation required changes to the calibrated C-43 Model. Changes made to the model are summarized below:

- The three cells of the proposed Reservoir were added to the MIKE 11 network as cross sections, inflow pumps, and outflow gates. Submerged gates convey water between the cells of the Reservoir.

- The irrigation command areas for Berry Groves and portions of the Bob Paul Ranch were removed.

- Portions of Roberts Canal and Crawford Canal that will be covered by the Reservoir were removed from the model and replaced with a Perimeter Canal around the proposed Reservoir. Gated spillways were added to the Perimeter Canal to control inflow from Roberts Canal and the Duda Northeast Rim Ditch, and weirs were added to control outflows to Crawford Canal, Roberts Canal (also known as Banana Branch), Fort Simmons Branch, and Townsend Canal.

- Detailed surveyed cross sections were obtained for S.R. 29 Canal, Crawford Canal, Roberts Canal, Fort Simmons Branch, and Townsend Canal.

- The S.R. 80 bridge over Townsend Canal was added to the model, and culverts for Fort Simmons Branch were added to the model.

- A site-specific model was developed with a finer grid to simulate seepage in greater detail. The site-specific model area is identified as the Berry Groves Catchment in **Figure 2E-01** in **Section 2** of the Report. The site specific model was developed for only the Alternative B options.

- Alternative B, Option 1, includes a pump station from Townsend Canal just south of S.R. 80. Because the invert elevation of Townsend Canal is below sea level, water from C-43 Canal can flow upstream into the Townsend Canal and can be pumped into the Reservoir from this pump station. The Townsend Canal bridge and canal cross sections were modified to handle the higher inflow pumping requirements of the Reservoir for a number of scenarios. However, the existing bridge and canal are adequate for conveying flows to the proposed Reservoir (see discussion below). **Figure 1D-04** in **Section 1** of the Appendices shows the conceptual layout of Alternative B, Option 1.

- Alternative B, Option 2 is basically the same as Option 1 except that there is an additional pump station in the Perimeter Canal at the southeast end of the Reservoir near the junction with Roberts Canal. The south pump station discharges into Cell 3 and the structure S-4 is utilized to provide a raised source of water for the pump. **Figure 1D-04** in **Section 1** of the Appendices also shows the conceptual layout of Alternative B, Option 2.

- Inflows to the C-43 Watershed from S-77 at Lake Okeechobee were modified to reflect the results of the South Florida Water Management Model (SFWMD) run CERP1RD (Interagency Modeling Center, 2004).

- The land use file used in the model is a 2050 land use file provided by SFWMD in December, 2004 (e-mail from Tim Liebermann, 12/3/2004).

- Rainfall to be used in the model is a grid rainfall file with daily rainfall values provided by the SFWMD that covers the period of 1965 – 2000. Tests of various structure operations for the proposed Reservoir will use short time periods ranging from 6 months to 7 years (see **Figure 5D-01**), and the recommended scenario will be tested using a 30-year period of record corresponding to the SFWMM run CERP1RD.

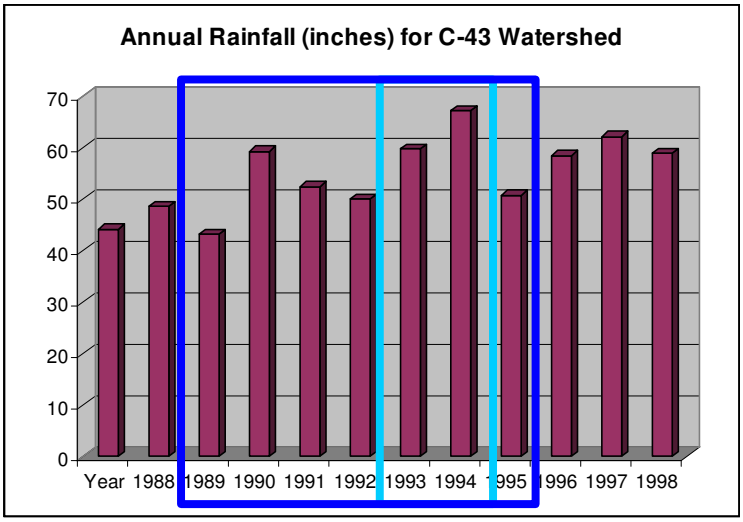


Figure 5D-01 – Rainfall used for Determining Initial Design of Berry Groves Reservoir Hydraulic Control Structures

1) Reservoir Sizing and Yield

The Reservoir size was determined primarily from prior work conducted by the PIR Project Delivery Team (PDT). The desired Phase 1 storage Reservoir volume was set initially at 160,000 to 170,000 acre-feet, and the benefit of this Reservoir was determined by this project, as discussed below. As mentioned above in **Section 3 B 5)**, this Reservoir is the first phase of the overall C-43 PIR, and any ecosystem or water supply targets not achieved by this Reservoir will be met by additional reservoirs currently under evaluation by the PIR Team. Subsequent discussions in this section will provide quantitative information regarding the performance of the Reservoir in meeting the peak flow attenuation and base flow augmentation goals.

2) Stage-Discharge

The Reservoir, Alternative B, is planned for a normal maximum operating stage of 37.7 ft. The bottom elevation of Cell 1 ranges from 26 at the southeast corner of the cell to 16 ft. at the northwest corner. Gates will be used to release water from the Reservoir. The discharge released from the Reservoir will be determined based on the flow at S-79, and the outflow gates will open as necessary to match the necessary augmentation flow. The gate opening, conforming to standard orifice flow principles, will open the necessary amount to provide the desired flow.

3) Inflow Design Flood(s)

Figures 2E-04 to 2E-06 in **Section 2E** of this Report illustrates that peak flows during runoff periods in the C-43 Canal at S-79 range from 1,000 to 15,000 cfs. The peak observed discharge at S-79 is 19,000 cfs. The maximum capacity considered for total pump station capacity at the proposed Reservoir was 3,500 cfs. The target percentage of monthly average flows above 4,500 cfs at S-79 is 0%. The total pump station capacity will be determined through an analysis of modeling results (how often is the total pump station capacity used) and cost factors (how much additional cost for each pump station unit).

4) Freeboard

There have been numerous laboratory tests and field investigations conducted over the years to estimate freeboard (wind set up and wave run up from the wave-induced height). These estimates were primarily determined by empirical methods with different forms of models or figures and tables.

Wind setup is the tilting of the Reservoir water surface caused by the movement of the surface water under the action of the wind and is a significant factor in the design height of a dam. Wind set up can be estimated using three analytical methods: Zieder Zee formula (USACE manual EM 1110-2-1420, July 1989), Bretschneider model (Ippen, 1966), and Sibul's formulation (Brater, 1996).

The Zuider Zee formula is a frequently used equation to determine the wind setup of a Reservoir and is presented in USACE manual EM 1110-2-1420, October 1997. The wind setup estimated from this formula is a function of the wind velocity, fetch length and the average depth of the Reservoir along the fetch. Wind setup is larger for higher wind speed, longer fetch distance and in shallow Reservoirs with rough bottoms. Since the average water depths of the Reservoir range from 14.3 to 17.6 ft. for various Reservoir options, the Zieder Zee formula was considered to be appropriate and was adopted for the wind set up calculation. The average water depth varies based on the alternative and associated dam slopes being evaluated.

The empirical equations suggested by Ahrens and McCartney (1975) were used to estimate the run up on structures protected by various types of primary armor faces. In this method, the wave run up is estimated as a non-linear function of the dam slope, incident wave height and wave length. The ACES software package was developed based on these equations, and provides an estimate of wave run up on rough and smooth impermeable surfaces (Leenknecht, et al, 1992). The empirical coefficients associated with soil cement of a sloped surface are not available. The Shore Protection Manual (SPM) 1984 published by USACE Waterways Experiment Station (WES) with the USACE engineering manuals EM 1110-2-1420 and EM 1110-2-1414 provides a wave run up analysis that is performed for the interior slopes of the Reservoir to assure that overtopping of the dam does not occur. The Water Preserve Area (WPA) feasibility study (2001) and Everglades Agricultural Area Storage A-1 Reservoir Levee Optimization Study (2004) of Central and Southern Florida Project and C-44 Reservoir and STAs Project (2004) used the SPM and engineering manuals to estimate wave run up. The guidelines and procedures presented in these manuals have received general acceptance and were selected for use in estimating freeboard requirements for the Reservoir.

In accordance with EM 1110-2-1420, the freeboard is defined as the vertical distance between the crest of a dam and some specified pool level, usually the normal operating level or the maximum flood level. Therefore, the freeboard of the Reservoir was determined under two scenarios as shown in **Figure 5D-02** in **Section 5** of the Appendices.

- using a probable maximum wind condition with the reference elevation as the normal operating level
- using a reduced wind condition, slower than the probable maximum wind condition, with the maximum Reservoir level

The detailed calculations of all the Reservoir options with the backup materials can be found in **5D-01** in **Section 5** of the Appendices.

The calculation option I-B.2 is now referred to as the two-cell alternative, Alternative A. The calculation option II-B.2 is now referred to as the three-cell alternative, Alternative B.

The Reservoir dam must have sufficient freeboard above the maximum pool still-water elevation so that waves cannot wash over the top of the dam. The objective in selection of design freeboard is to assure that failure of the dam will not result from wind setup, wave action, uncertainties in analytical procedures, and uncertainties in project function in combination with the most critical pool elevation (ER 1110-8-2(FR)). Based on the guidelines in the USACE manuals, the key parameters including water fetch length, wind velocity, average water depth, slope angle, and slope roughness were used in establishing the freeboard allowance and in determining the final dam height. Only the key parameters of

two potential Reservoir options, I-B.2 two cells and II-B.2 three cells of constant water surface and varied top of bank elevation will be presented in the following discussion.

A very detailed freeboard analysis is provided in **Calculation 5D-01** in **Section 5** of the Appendices which includes four two-cell options as well as four three-cell options. Alternative A and Alternative B were recommended for detailed comparisons based on this analysis.

(a) Water Fetch Length

The characteristics of wind-generated waves are influenced by the distance that wind moves over the water surface in the "fetch" direction. The water fetch length is typically constrained and easily determined from the configuration of the Reservoir. The fetch lengths for the Reservoir are geometrically constrained by land availability. EM 1110-2-1414 of July 89 recommends a fetch calculated based on the average fetch within a 24° radius from the origin. Therefore, the fetch length was estimated as the radial average of path lengths over an arc of 24 degrees, drawn at 3-degree intervals, centered on the longest path to the opposite dam. The length of each path was given equal weight in the calculated average. The fetch length for Reservoir option I-B.2 of two cells was estimated to be about 3.55 miles. The fetch length for Cell 1, 2 and 3 of Reservoir option II-B.2 of three cells was calculated to be 3.55, 3.53 and 2.11 miles, respectively.

(b) Wind Velocity

A severe tropical cyclone is called a hurricane when the maximum sustained wind speeds reach 74 mph. Hurricane winds may reach wind speeds of more than 150 mph in Central and South Florida. The Saffir-Simpson Hurricane Scale defines hurricane strength by categories. A Category 1 storm is the weakest hurricane with wind speeds of 74 to 95 mph; a Category 5 hurricane is the strongest with wind speeds over 155 mph. Frequently, the right side of a hurricane is the most dangerous in terms of wind setup and wave run-up. The wind speeds of a few major hurricanes that formed in August and September 2004 and hit the Central and South Florida near the project site area are compared with the design wind speeds employed in the freeboard estimate.

In accordance with the National Weather Service Forecast Office, NOAA storm summary report, Hurricane Charley was a Category 4 (winds 131-155 mph) and made landfall along the Lee County coastal line and moved northeast through South West and Central Florida with wind speeds around 140 mph during the late afternoon and evening hours of August 13, 2004. The maximum wind gusts inland ranged from 45 to 112 mph, including 112 mph in Punta Gorda of Charlotte County

and 95 mph in Fort Meyers of Lee County about 45 and 25 miles from the project site, respectively. The wind speed reduced to about 104 mph in Arcadia of Desoto County, about 30 miles from Punta Gorda around 5:30 PM.

Frances was a Category 4 hurricane with winds of 145 mph before its center reached the Florida east coast near Sewalls Point, Martin County after midnight on September 5, 2004 as a Category 2 storm (winds 96-110 mph). It continued moving west-northwestward to the northeastern Gulf of Mexico and moved across the central Florida after almost 24 hours. Frances was downgraded to a tropical storm and made a final landfall near St. Marks on the afternoon of September 6, 2004. The maximum wind gust of 48 mph was measured in Fort Meyers of Lee County.

On September 25, the large eye of Hurricane Jeanne made landfall on the east coast of Florida near Stuart as a category 3 (winds 111-130 mph). The peak winds (gusts) near the Reservoir from Jeanne across West Central and Southwest Florida ranged from 50 to 59 mph.

The 3-second gust wind speed was estimated to be about 120 mph at the project site based on the Florida Building Code published by Florida Department of Community Affairs and the Florida Building Commission as shown in **Figure 5D-03** in **Section 5** of the Appendices. The freeboard analysis for Scenario 1 was performed using a 120 mph maximum design wind speed near the project site, adjusted based on the ratios in the "Wind Setup" section of Calculation 5D-01, as the wind velocity over water is higher than that over land surfaces (EM 1110-2-1420). The adjusted wind speed used for calculating wave height generation for over water has a range of 108% to 130% of the maximum probable 120 mph for the computed effective fetch. For example, the adjusted wind velocity of 153 mph for a fetch length of 3.55 miles for Cell 1 in both options I-B.2 and II-B.2 was used in the wind setup and wave run-up analysis. The adjusted maximum wind velocity (146-153 mph) used in the freeboard analysis for Scenario 1 under both options is similar to that of the major hurricanes given in **Table 5D-01**.

Hurricane/Option	Maximum Wind Velocity(mph)	Maximum Rainfall(inches)
Charley	145	8
Frances	105	15.8
Jeanne	120	13
Easy	125	38.7
I-B-2	116	32
I-B-2	108	38.7
I-B-2	97	47.1
I-B-2	92	51.8
I-B-2	87	55.7
II-B-2	116	32
II-B-2	108	38.7
II-B-2	96	47.1
II-B-2	91	51.8
II-B-2	86	55.7

Table 5D-01 – Hurricane Winds and Rainfall

Note: The above maximum wind velocity for two options was estimated by area-weighted average.

In the analysis of Scenario 2, the all season PMP values for various durations ranging from 6 to 72 hours were used under reduced wind conditions. Since the reduced wind condition was not well defined, the wind velocity was determined by an iterative simulation given the same freeboard that was estimated from Scenario 1. The velocity used in Scenario 2 to arrive at the same freeboard ranges from 87-116 mph for option I-B.2 and 86-116 mph for option II-B.2, respectively. These results are shown in the above table for five maximum Reservoir levels under PMP of 32”– 55.7”. Hurricane Easy is the worst case of an actual hurricane. However, the maximum wind speed and rainfall were measured at the coast of Cedar Key and Yankeetown, respectively. The wind data was collected from October 21, 1992 to October 2, 2004 at a weather monitoring location S78W about 14 miles east of project site. The SFWMD’s DBHYDRO database indicated that the maximum wind speed of 44 mph was recorded on 9/26/04. Based on the evaluation of the historical wind data near the Reservoir, the velocity used in scenario 2 is acceptable for the application in both two-cell and three-cell, constant water depth alternatives.

The design wind speed will be adjusted for the coefficient of drag when the final design criteria of freeboard determination are available.

(c) Water Depth

The height of the dam is derived from the Reservoir water depth plus the minimum required freeboard. The average water depth was estimated

based on the elevations of the bottom of the Reservoir and the volume requirements. The bottom elevation of the Reservoir ranges from about 16 to 26 ft. of 1988 NAVD. The average water depth ranges from 14.3 ft. in Reservoir option II-B.2 (Alternative B). The average water depth of Cell 1 and 2 in Reservoir option I-B.2 (Alternative A) was estimated to be about 19.1 ft. and 15.8 ft., respectively. These values were used in the wind setup and wave run-up analyses. Water depth in the Reservoir affects wave generation. For a given set of water fetch length and wind velocity, wave heights will be smaller and wave periods shorter if generation takes place in shallow water rather than in deep water. The waves can be defined by their relationship of water depth (d) to deep-water wave length (L_0). The wave characteristics are independent of depth if the ratio d/ L_0 is greater than 0.5. $L_0=gT^2/6.28$ where T is wave period. However, some energy is dissipated due to bottom friction if d/ L_0 is less than 0.5. In other words, for the same wind conditions and water fetch length, a shallow-water wave would have a lower height than a deep-water wave. USACE recommends a shallow-water assumption if the relative depth (d/ L_0) is less than 0.5 (EM1110-2-1414). This criterion was used in the freeboard analysis.

The project team understands that the SFWMD is developing the design criteria and guidelines on the combination of precipitation and wind speeds for the freeboard determination. There are 5 cases under consideration: (1) PMP combined with the 100-year wind (~105 mph), (2) a 100-year precipitation combined with a Category 5 Hurricane wind (~155 mph), (3) a 200 mph probable maximum wind combined with the normal maximum pool level, (4) a historical storm specific event, and (5) PMP combined with 125 mph sustained winds (proposed by USACE). The project team will revise the freeboard calculation when the design criteria of freeboard determination are available.

(d) Dam Slope

Besides water fetch length, wind velocity and water depth in the Reservoir, the wave run-up level depends on the slope angle of the dam. Most dams have slopes between 3H: 1V, 4H: 1V and 5H: 1V. The wave run-up level will decrease if the dam slope decreases and other parameters remain the same. For the wave run up analysis, all interior slopes of dam were assumed to be 3H: 1V. The estimates of wave run up were corrected by the dam slope. Data of run up correction for scale effects from the Shore Protection Manual were adopted in the estimate of wave run up. The estimated freeboard for options I-B.2 (Alternative A) and II-B.2 (Alternative B) under slope 3H: 1V is about 1.5 ft. and 1.3 ft. greater than that under slope 4H: 1V, respectively. However, the average earthwork quantities estimated for 4H: 1V was about 1.8 millions cubic yards more than that for 3H: 1V. Therefore, the dam

slope of 3H: 1V was the selected one in estimating the freeboards of all the Reservoir options as the 3H:1V slope is the most economical.

(e) Slope Roughness

A rough riprap or any other slope protection layer on a dam tends to reduce the height of run-up. If the thickness of riprap or slope protection material is relatively small in comparison with wave magnitude and the underlying surface is relatively impermeable, the height of run-up may closely approach heights attained on smooth dams of same slope. However, if the riprap or any other slope protection layer is sufficiently rough and thick, further run-up will be almost eliminated (EM1110-2-1420). Therefore, the slope roughness factor for the interior slopes of dam was mainly determined based upon slope protection materials and characteristics. Multiple layers of compacted soil-cement are selected for the dam slope protection to dissipate the energy in the waves and reduce the height of the wave run-up. It consists of a series of 16 inches thick by 8 ft. wide horizontal layers of soil cement with a 3H: 1V slope to the exposed stair-step face and with random fill adjacent to the 1H: 1V slope of the central clays/clayey sands impervious core in the dam. A 16 inch thick flat plate of soil cement was proposed to be installed below the normal Reservoir pool level. More detailed information is available in **Figure 8A-1** in **Section 8** of the Appendices. The rough slope run-up correction factor (r) was assumed to be 0.7 for various slope characteristics from Table 7-2 in Shore Protection Manual and Table 2-2 in EM 1110-2-1614. This factor was used to adjust the estimate of the wave run-up height.

(f) Summary of Results

For Scenario 1, the estimated freeboard heights for the Reservoir are 9.7 ft. and 10 ft. for Cell 1 and 2 of Reservoir option I-B.2 (Alternative A), respectively shown in **Table 5D-02** in **Section 5** of the Appendices . The freeboard heights of 9.9 ft., 10.3 ft. and 7.3 ft. were estimated for Cell 1, 2, and 3 in Reservoir option II-B.2 (Alternative B), respectively shown in **Table 5D-02** in **Section 5** of the Appendices. The procedure for calculating the wind setup and wave run-up analysis for both candidate Reservoir options is presented and summarized in **Table 5D-02** shown in **Section 5** of the Appendices. Wave height and wave period calculations are also presented in the table. The wave run-up estimates include not only the maximum fetch distance in any direction, but also include the correction for the upstream dam slope. The estimate was based on the fetch distance in the direction of the maximum wind speed.

The results indicate that the Reservoir option I-B.2 (Alternative A) is more economic than Reservoir option II-B.2 (Alternative B) for the constant water surface elevation condition based on the estimated

earthwork quantities. A much more detailed "dam section" cost analysis is provided in **Section 22**. The results of freeboard analysis for option I-B.2 (Alternative A) show that the maximum top of dam elevations are 48.9 ft. and 49.3 ft., for Cell 1 and 2, respectively. Since the interior dams will be armored on both sides and overtopping is not an issue, the elevations of interior dam will be based on normal pool elevation plus 5 ft. The final estimated freeboard will include dam settlement that should be applied to account for consolidation of foundation and dam materials. Detailed settlement analysis along the dam alignment is given in **Section 8**.

REFERENCES

- *Ahrens, J. P. and McCartney B. L., Wave Period Effect on the Stability of Riprap, Proceedings of Civil Engineering in the Oceans/III, ASCE, pp. 1019-1034, 1975*
- *Brater, E. F., H. W. King, J. E. Lindell, and C. Y. Wei, "Handbook of Hydraulics", McGraw-Hill Book Company, Inc., 1996.*
- *ER 1110-2-1155 Engineering and Design – Dam Safety Assurance Program, September 1997*
- *ER 1110-8-2(FR) Engineering and Design – Inflow Design Floods for Dams and Reservoirs, March 1991*
- *EM 1110-2-1420 Engineering and Design – Hydrologic Engineering Requirements for Reservoirs, October, 1997*
- *EM 1110-2-1414 Engineering and Design – Water Levels and Wave Heights for Coastal Engineering Design, July 1989*
- *EM 1110-2-1603 Hydraulic Design of Spillways, January 1990*
- *EM 1110-2-1614 Engineering and Design –Design of Coastal Revetments, Seawalls, and Bulkheads, June 1995*
- *EM 1110-2-2300 Earth & Rock-Fill Dams General Design & Construction Considerations, July 1994*
- *EM 1110-2-2302 Construction with Large Stone, October 1990*
- *FEMA, Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams, October 1998.*
- *Ippen, A. T., "Estuary and Coastline Hydrodynamics", McGraw-Hill Book Company Inc., 1966.*

- *Leenknecht, D. A., A. Szuwalski and A. Sherlock, Automated Coastal Engineering System, Technical Reference, Coastal Engineering Research Center, Waterways Experiment Station (WES), Version 1.07, 1992*

- *Schreiner, L. C. and J. T. Riedel, Probable Maximum Precipitation Estimates for the United States East of the 105th Meridian, National Oceanic and Atmospheric Administration, August 1980*

- *Shore Protection Manual – Volume 1 & 2, Published by Waterways Experiment Station (WES), 1984*

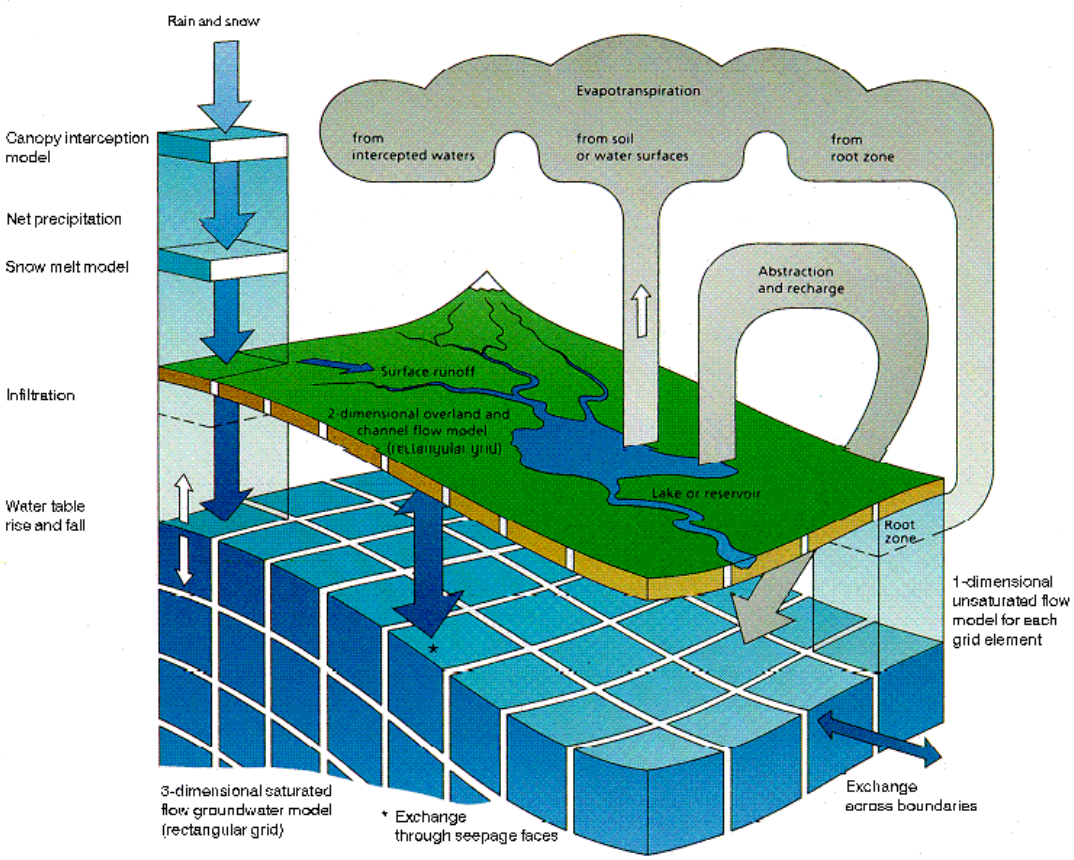
Section 6 – Hydraulics

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 6 in the Appendices.

A. Design Criteria

The purpose of this project is to conduct hydrologic and hydrologic modeling to determine the optimal reservoir configuration to meet ecosystem restoration goals without sacrificing existing withdrawals from the C-43 Canal. The MIKE SHE/MIKE 11 hydrologic model was used in evaluation of reservoir design alternatives.

MIKE SHE is a dynamic modeling tool for the analysis, planning and management of a wide range of water resources and environmental problems related to surface water and groundwater. MIKE SHE gives unique possibilities for modeling of integrated surface water / ground water or drainage system / ground water aspects. The modeling system is a physically based distributed meaning that all important processes influencing the land phase of the hydrological cycle are accounted for using the mathematical differential equations governing the particular process.



The models modular structure allows for certain processes to be studied in detail using selected modules. The basic water movement module is well suited for detailed water balance studies. In addition to the basic module, MIKE SHE offers a wide range of add on modules allowing for detailed studies of: irrigation systems, variable land use, environmental impact studies and the combined transport of natural and anthropogenic contaminants. Powerful pre-processing and results presentation tools are included in the MIKE SHE software package.

MIKE 11 is a modeling system for modeling rivers, channels and floodplains in a fully dynamic, one-dimensional flow description. It models the hydraulic behavior in terms of discharge and water levels as well as dambreak analyses, sediment transport, water quality and other issues. MIKE 11 is an integral part of MIKE SHE and the exchange of water between channels and flooded areas is modeled as inundation above channel banks and as the interaction between surface water and groundwater. MIKE 11 can handle a wide variety of structures and allows the user to define logical rules to control the operation of gate and pump structures. The model utilizes many kinds of boundary conditions including discharge and stage. The model is also FEMA approved.

The combined MIKE SHE/MIKE 11 modeling systems is well suited for the modeling of the interaction between surface water and groundwater. The system is able to utilize all available information in a combined context and all data for calibration can be used. The modeling software is developed with Graphical User Interfaces and is very user-friendly. The modeling system has strong presentation capabilities including various possibilities for visualization and even animation of results.

The reservoir will consist of levees, inflow pumps, outflow gates, and a Perimeter Canal to collect reservoir seepage and safely convey high flows from tributaries that flow through the footprint of the proposed reservoir.

1) Inflow pumps

Two inflow pumping scenarios were considered during the design. Option 1 has one pump station (PS-1) at the northwest corner of the reservoir, and Option 2 has two pump stations (PS-1 and PS-2), with PS-2 located near the southeast corner of the reservoir. Pump capacity at PS-1 was not known at the beginning of design and was varied from 1,000 to 3,500 cfs. Pump sizes considered range from a small number of large pumps to a mix of larger and smaller pumps. Pump capacity at PS-2 ranged from 500 to 600 cfs, and pump sizes considered range from one big pump to a number of smaller pumps. During initial modeling, Townsend Canal was found to be a limiting factor for conveying the initial assumed inflow rates to the reservoir inflow pump station PS-1 from the C-43 Canal. The need for expansion of the Townsend Canal was evaluated during the design process.

2) Outflow Gates

The outflow gate dimensions were established to safely convey the desired outflow rates, which will typically be less than 1,000 cfs. Overflow and underflow gates were evaluated, and combinations of gates were evaluated. Initially, only one outflow gate was considered, and then more gates were included to allow for longer reservoir hydraulic retention times (to enhance pollutant removal) and to allow for water supply releases for agricultural users. Gate operations were modified during design to minimize the frequency of low flows at S-79.

3) Perimeter Canal

Re-routing of existing Roberts Canal is necessary to construct the reservoir, and a Perimeter Canal is proposed to collect seepage and to route Roberts Canal and Crawford Canal flows around the proposed reservoir. The Perimeter Canal invert elevation was established at the top of the clay aquiclude so that the Perimeter Canal could collect seepage from the reservoir. The width of the canal was established so that there would be sufficient conveyance capacity for flood flows from Roberts and Crawford Canals as well as some runoff from the S.R. 29 Canal. The Perimeter Canal also provides flow releases to the downstream portions of Crawford Canal, Roberts Canal, and Fort Simmons Branch. The control structures considered for these tributaries will have to release small amounts of water for baseflow and also release large amounts of water during floods.

4) Routing of Flood Flows

A typical in-line reservoir completely blocks the flows of the inflow river, and a flood routing study is needed to determine if failure of the reservoir during a low-frequency high rainfall event will endanger lives of downstream residents. The proposed Reservoir will only be filled if there is sufficient capacity to receive the inflows. There are no gravity inflows to the reservoir besides rainfall. Accordingly, the reservoir does not have to be designed to handle runoff from a large low-frequency rainfall event. The levee top-of-bank elevation has been set sufficiently high so that the entire rainfall volume from a 25-year event can be contained. It was assumed that each cell of the reservoir should have a weir to convey runoff from a Probable Maximum Precipitation event in the event that the outflow gates cannot be opened.

The 30% preliminary design phase will include an analysis of the Probable Maximum Flood (PMF) flow from south and east of the reservoir including routing to evaluate the impact of a PMF on the east and south embankments of the reservoir. If the PMF is not contained by the Perimeter Canal, the embankment toes can be eroded.

5) Interior Cell Design of the Reservoir

The two main alternatives considered for the reservoir configuration were a two-cell reservoir (Alternative A) and a three-cell reservoir (Alternative B). The two-cell reservoir will have the same capacity as the three-cell reservoir, and modeling was conducted only on the three-cell reservoir alternative.

6) Maintenance of Irrigation Flows to Agricultural Operations

A major citrus operation remains south of the proposed reservoir site. Townsend Canal provides flood conveyance for agricultural runoff and irrigation conveyance during dry weather periods. Townsend Canal flows to the north to the C-43 Canal during runoff periods and to the south during irrigation. Runoff leaves the farm at elevation 23.5 ft., and irrigation water is pumped from lower Townsend Canal at elevation 3 ft. up to elevation 23.5 ft. The Reservoir should be operated in a manner that allows for continued functioning of the citrus operations. Factors to consider are:

- preserving inflow conveyance in Townsend Canal or provision of an alternative mechanism for water delivery,
- locating inflow pump stations near the source of runoff to save on pumping costs, and
- providing adequate seepage control so that the Reservoir does not cause excessively high rates of seepage from the Reservoir into the farm.

B. Pump Requirements for the Reservoir Alternative B, Option 1

Alternative B Option 1 was developed to evaluate the effectiveness of the Reservoir with one pump station at the northwest corner of the Reservoir. A total of 33 runs were performed to determine the Reservoir hydraulic control structures. The first 16 runs were performed prior to the availability of 2050 land use and updated SFWMM S-77 outflows. The final 17 runs were performed after receipt of 2050 land use and CERP1RD S-77 flows.

1) Tests Using 2000 Land Use

Approximately 11 runs were conducted using a two-year period of simulation to determine basic structure dimensions and pump station capacities. Inflow pump capacity ranged from 2000 to 3500 cfs, outflow gate dimensions were tested, and outflow gate operating criteria were tested. Once the basic issues were resolved, the following runs were performed using 2000 land use information and D13R S-77 flows to refine the structure dimensions and operations:

- Option 1 Test 12 – one 2000 cfs inflow pump, three outflow gates
- Option 1 Test 14 – four 500 cfs pumps, six outflow gates
- Option 1 Test 15 – four 500 cfs pumps, S-79 gate operations modified
- Option 1 Test 16 – two 250 and three 500 cfs pumps, S-79 not modified

Test 15 is a modification of Test 14 that includes modified gate operations for S-79. S-79 will open 3 inches if flow at S-79 is less than 300 cfs and the S-79 headwater elevation is higher than the tailwater elevation.

The performance of iteration Alternative B Option 1 Tests 14-16 are presented in **Table 6B-01**. It can be seen that Test 15 does a fair job of meeting both the peak flow reduction and base flow augmentation targets. Improvement is needed to better match the target percent of flows in the 0–500 cfs range and this iteration maintains flows between 150 to 300 cfs four percent of the time.

Target		Percent Exceeded			
Flow Range	Target Exceedance %	Test 14	Test 15	Test 16	Calibration
0 to 150 cfs	0%	2.08	2.08	3.13	8.33
150 to 300 cfs	0%	6.25	4.17	8.33	8.33
300 to 500 cfs	43%	31.25	36.46	29.17	16.67
500 to 800 cfs	32%	32.29	29.17	31.25	20.83
800 to 1500 cfs	19%	20.83	21.88	18.75	8.33
1500 to 2800 cfs	6%	4.17	4.17	6.25	16.67
2800 to 4500 cfs	1%	2.08	2.08	2.08	8.33
> 4500 cfs	0%	1.04	1.04	1.04	12.50

Table 6B-01 – Comparison of Alternative B Option 1 to Target Flows at S-79

Table 6B-01 was presented in the August, 2004 draft DDR report and confirmed that the Reservoir will have a beneficial effect on hydrology of the C-43 Basin. Subsequent work performed after the August, 2004 report addressed outstanding issues to refine the design of the proposed Reservoir.

2) Tests Using 2050 Land Use

The final 17 test runs utilized 2050 land use and S-77 outflows from SFWMM CERP1RD. A number of runs were performed to confirm that the Reservoir structural features would still be appropriate with revised land use data and S-77 inflows. The results from test runs 26 through 33 are discussed below to document the configuration of Alternative B Option 1. **Table 6B-02** presents a summary of the factors modified in test runs 26 through 32. Inflow pump

capacity at PS-1 (at the northwest corner of the Reservoir) ranges from 2000 down to 1000 cfs. Outflow gates were opened when S-79 flows exceeded 800, 650, and 600 cfs.

Run Name	Flow at S-79	Pump Rate at PS-1	Pump Rate at PS-2
C-43_BerryFarm_Alt1_Test32_2050 and Test 33-2050	> 650 cfs	250 cfs	0 cfs
	> 800 cfs	500 cfs	0 cfs
	>900 cfs	1000 cfs	0 cfs
	>1000 cfs	1500 cfs	0 cfs
C-43_BerryFarm_Alt1_Test31_2050	> 600 cfs	250 cfs	0 cfs
	> 750 cfs	500 cfs	0 cfs
	> 900 cfs	1000 cfs	0 cfs
	>1000 cfs	1500 cfs	0 cfs
C-43_BerryFarm_Alt1_Test30_2050	> 600 cfs	250 cfs	0 cfs
	>800 cfs	500 cfs	0 cfs
	>1000 cfs	1000 cfs	0 cfs
	>1100 cfs	1500 cfs	0 cfs
C-43_BerryFarm_Alt1_Test29_2050	>800 cfs	250 cfs	0 cfs
	>900 cfs	500 cfs	0 cfs
	>1000 cfs	1000 cfs	0 cfs
C-43_BerryFarm_Alt1_Test27_2050	>800 cfs	250 cfs	0 cfs
	>900 cfs	500 cfs	0 cfs
	>1000 cfs	1000 cfs	0 cfs
	>1100 cfs	1500 cfs	0 cfs
C-43_BerryFarm_Alt1_Test26_2050	>800 cfs	250 cfs	0 cfs
	>900 cfs	500 cfs	0 cfs
	>1000 cfs	1000 cfs	0 cfs
	>1100 cfs	1500 cfs	0 cfs
	>1200 cfs	2000 cfs	0 cfs

Table 6B-02 – Alternative B Option 1 Trigger Flows at S-79 that Control Outflow Gate Operations and Pump Station Capacities

Figure 6B-01 illustrates that pump station capacities of 1000 and 2000 cfs were found to be inferior to a capacity of 1500 cfs. The 1000 cfs pump station resulted in higher frequencies of high flows and did not perform as well in meeting the 300-500 cfs flow range. The 2000 cfs pump station did not perform as well as the 1500 cfs pump station in the 500 to 1500 cfs flow range. **Figure 6B-02** illustrates that there is no significant difference in Reservoir elevations for 2000 or 1500 cfs, and that the Reservoir elevation for 1000 cfs is significantly lower. Based upon these results, it was concluded that the optimal capacity for PS-1 was 1500 cfs.

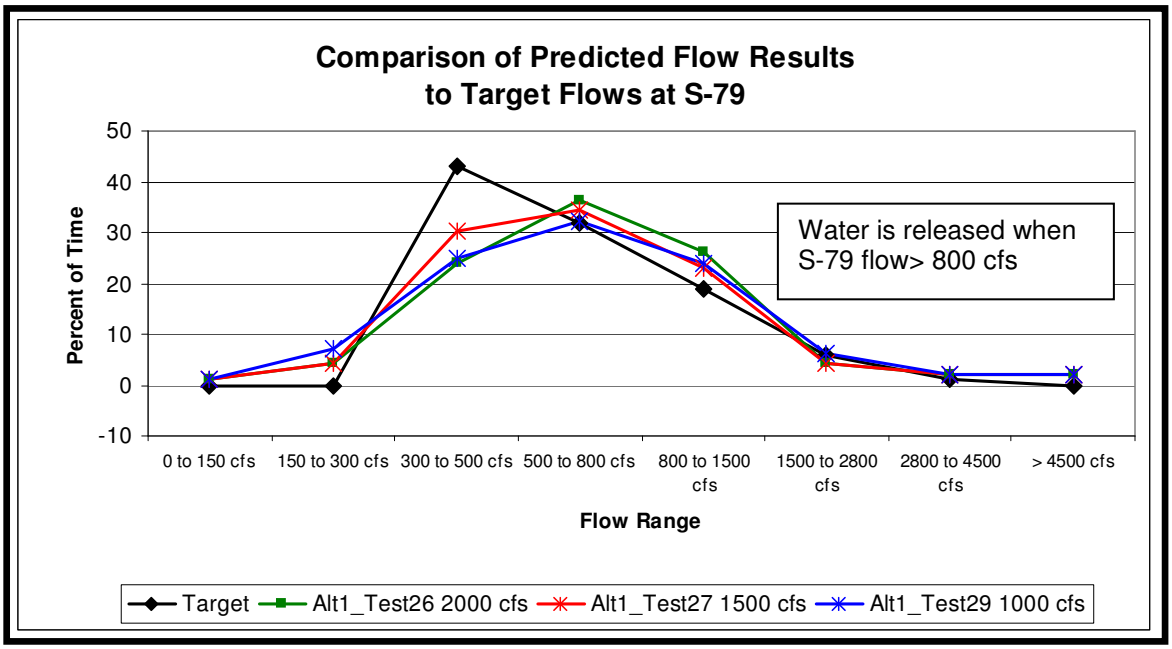


Figure 6B-01 – Performance of Reservoir Alternatives Varying Inflow Pump Station Capacity

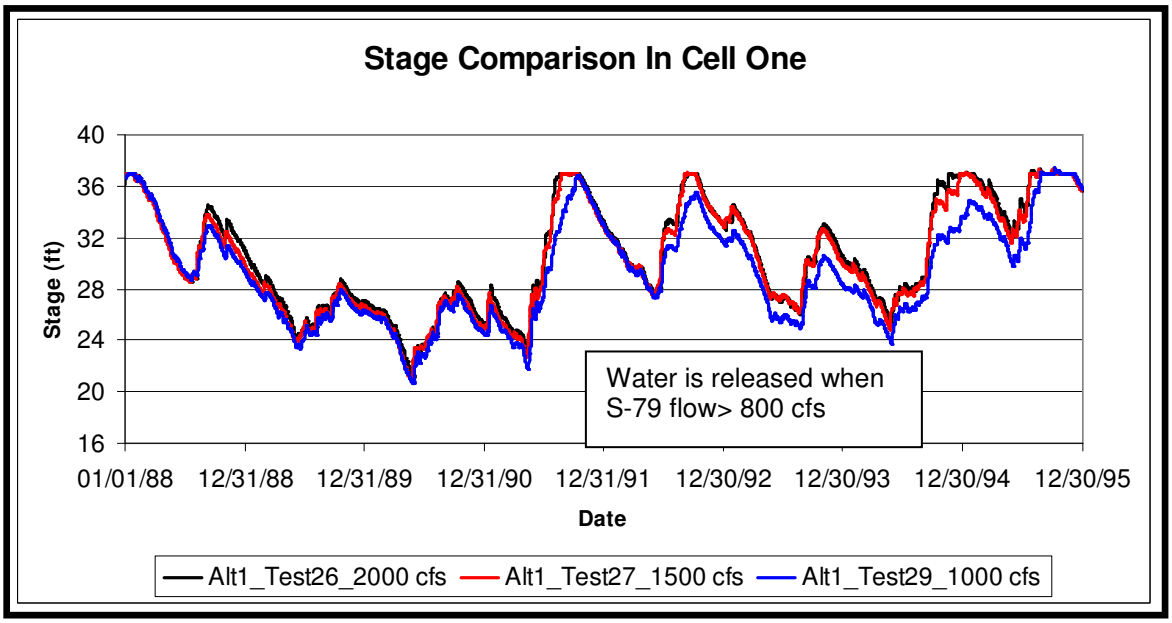


Figure 6B-02 – Reservoir Elevation for Pump Station Capacities of 1000 – 2000 cfs

Water is released from the Reservoir through a number of outflow gates to augment low flows at S-79. As presented above in **Table 6B-02**, the trigger flow at S-79 used to open gages was varied to determine the optimum operating criteria. The flow at S-79 is higher than the target flow distribution in the 500 to 800 cfs range when the trigger flow is 650 cfs (see **Figure 6B-03**), however the differences between these two test runs is relatively slight.

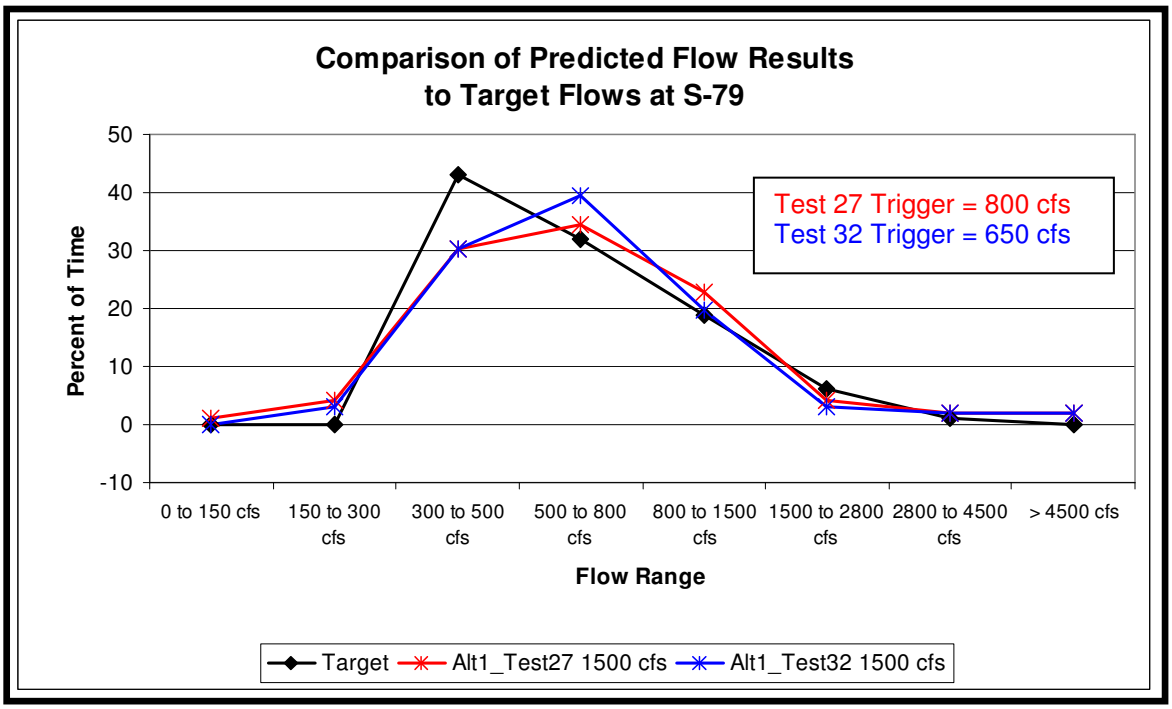


Figure 6B-03 – S-79 Flow Distribution for Different Reservoir Outflow Trigger Flows

Tests 26 through 32 assumed that the Townsend Canal and the S.R. 80 Bridge over the Townsend Canal would be larger than the existing dimensions because the inflow pump station capacity was set at 3,000 cfs. Also, the initial runs did not have recent surveyed cross sections for the Townsend Canal. Because there is a significant cost to expanding the bridge and canal dimensions, a test was conducted to determine if the larger canal and bridge were actually needed, as shown in **Figure 6B-05**.

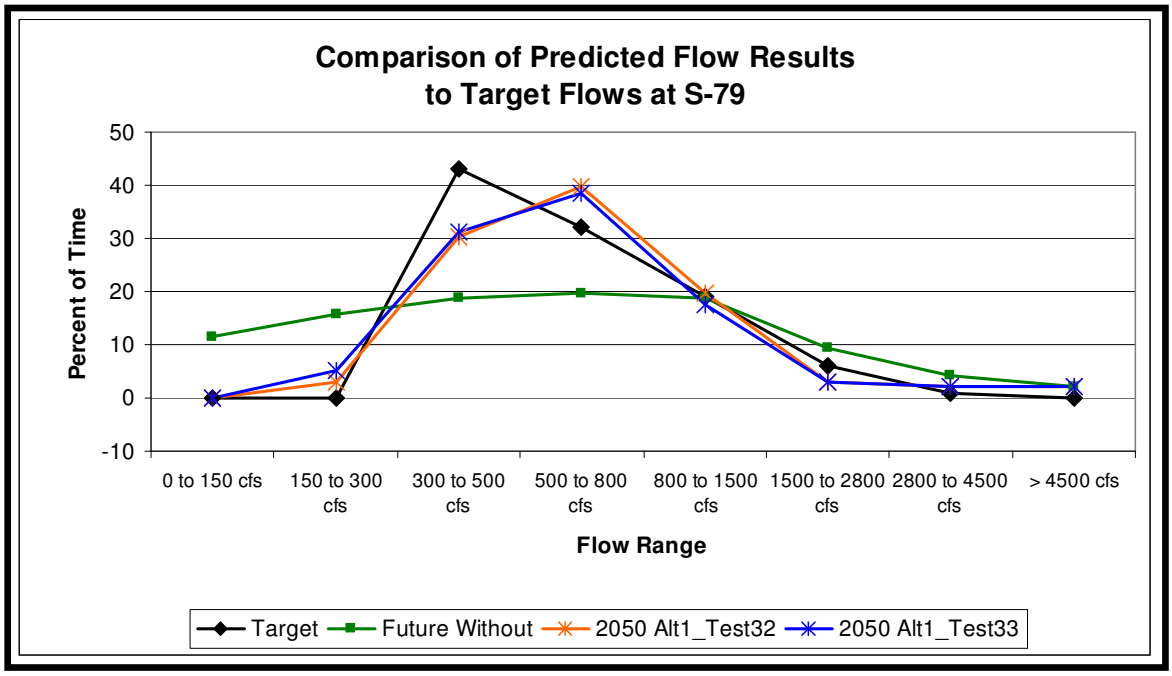


Figure 6B-04 – S-79 Flow Distributions for the Existing (Test 33) and Expanded (Test 32) Bridge and Canal

Figure 6B-04 illustrates that the S-79 flows are not significantly different between Test 33 (existing conditions for bridge and canal dimensions) and Test 32 (larger bridge and canal dimensions). Therefore, the project does not include major expansion of the Townsend Canal or the S.R. 80 Bridge, but is still being evaluated. **Figure 6B-05** illustrates that the water surface profile has a negative slope for Test 33. The water surface profile is flat for an enlarged Roberts Canal that was evaluated in Test 32 (figure not shown). Dense growths of Brazilian Pepper are commonly observed in the Townsend Canal south of S.R. 80. The Brazilian Pepper should be removed frequently from the existing canal banks so that head loss along the Townsend Canal is kept to a minimum. The specifics of Alternative B, Option 1, Test 33 are presented in **Table 6B-03.**

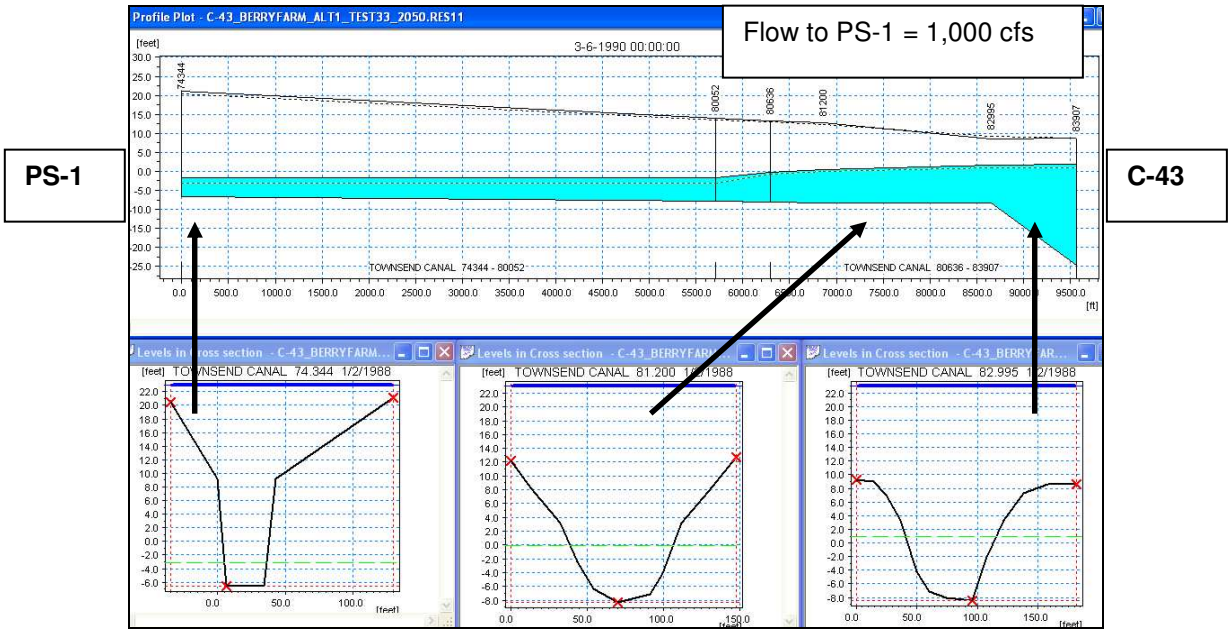


Figure 6B-05 – Water Surface Profile for Townsend Canal Using Existing Canal Dimensions for Alternative B Option 1 Test Run 33

Feature	Capacity	Operation Rule
Inflow Pump Station #1	2 x 500 + 2 x 250 cfs	Operate if water level in reservoir < 37.0 ft and WL in Townsend canal > -3.25 ft and flow at S-79 > 600 cfs
Cell 1 Outflow Gate S-1 (2 gates)	Inv. = 20.0 ft, W = 12 ft Inv = 16.0 ft, W = 22 ft	Opens if flow at S-79 < 300 cfs and water level in reservoir < 27 ft Lower gate only opens to drain reservoir.
Cell 2 Outflow Gate S-8 (2 gates)	Inv. 25.0 ft, W = 18 ft Inv. 20.0 ft, W = 12 ft	Opens if flow at S-79 < 300 and head in Cell 2 is < 32. Peak flow at 500 cfs, close if S-79 >400 cfs
Cell 3 Outflow Gate S-5	Inv. 30 ft, W = 12 ft Inv. 25 ft, W = 12 ft	Open if flow at S-79 < 400 cfs, and head in Cell 3 is >27 ft. Peak flow at 500 cfs, close if S-79 > 400 cfs.
Cell 1 Outflow Gate S-2	Inv. 23 ft, W = 7 ft	Opens if WL in NE Rim Ditch < 23.29 ft and head in Cell 1 > 29.5 ft
Perimeter Canal	BW = 10 ft at 3.5 ft TW = 130 ft at 23.5 ft	Maintain at 16 ft upstream of S-10 Maintain at 10 ft. upstream of S-11
S-10 Gate in Perimeter Canal	Five 5-ft gates Sill 13. 3 ft H	Overflow gates. Starts to open if WL >16.5 ft in 6-inch increments. Close if WL < 16.5 ft
S-11 Trapezoidal Weir	60 ft W at 10 ft, 150 W at 14 ft	No operation
Perimeter Canal Pump	20 cfs	Pump if WL > 15.75 ft
S-6, -7, and -9 Weir & Orifice	Inv. 16.5 ft W = 1 ft, W = 30 f t at 17.25 ft Orifice 1-ft dia. at elev. 14 ft.	S-6 – Crawford Canal S-7 – Roberts Canal S-9 – Fort Simmons Branch
S-15 (3 underflow gates)	Sill 22 ft, W = 12 ft, Max opening is 6 ft	Open if Roberts Canal WL > 25.5 ft
S-3 (3 underflow gates)	Sill = 16 ft., W = 12 ft, max opening is 10 ft. (gate is 10 ft high)	Open if NE Rim Ditch WL > 23.5 ft., Duda PS is not on, and Cell 1 WL>29.5.
Culverts	Dimensions	
S-12 between Cells 1 and 2	Inv. 22.9 ft, two 10x10 ft	Modeled as culverts. Will be gates that are normally open.
S-13 between Cells 2 and 3	Inv. 22.97 ft, two 10 x 10 ft	

Table 6B-03 – Features of Alternative B Option 1 Test 33

C. Pump Requirements for Alternative B, Option 2

Alternative B Option 2 is similar to Alternative B Option 1, however there is an additional inflow pump station at the southeast corner of the Reservoir. The idea for this alternative is to pump as much tributary runoff into the Reservoir as possible, thereby reducing pumping costs (because the head differential from the pump invert and the Reservoir is low) and maximizing pollutant retention in the Reservoir (catch the water before it gets diluted by C-43 Canal). This pump station (PS-2) moves flows directly from the Roberts Canal and has the ability to capture excess flows released to the east from the Duda Farm NE Rim Ditch. A gate at the eastern end of the NE Rim Ditch opens during runoff periods to convey water from the Duda farm toward PS-2.

As with Alternative B Option 1, a number of tests were conducted for Alternative B Option 2. The first 18 tests were conducted with 2000 land use and D13R S-77 flows. These simulations determined basic pump operations, outlet dimensions, etc. The initial tests evaluated the structural options presented in **Table 6C-01**.

Test Runs	Outflow Gate Opening Flow Trigger	PS-1 Flow	PS-2 Flow	Outlet Gates
1-8	350-500 cfs	2500 – 3500 cfs	600 cfs	3-10'x10' or 13'x13'
14, 15	400 cfs (Test 15 had modified S-79 operation)	4 x 500 cfs	1 x 600 cfs	3-15x20, 2-15x22, 1-15x24
17	Same as Test 15	2x250, 3x500 cfs	1 x 600 cfs	Same as Test 15
18	Same as Test 15	2x250, 2x500 cfs	3x200 cfs	Same as Test 15

Table 6C-01 – Initial Test Runs for Option B Alternative 2

Test 17 performed best in meeting the peak flow reduction targets, and, in subsequent refinements after receipt of 2050 land use and updated S-77 outflows, evaluated methods to improve performance in meeting minimum flow requirements.

Tests conducted after receipt of 2050 land use and updated S-77 outflows are presented below in **Tables 6C-02 and 6C-03**.

Test Runs	Outflow Gate Opening S-79 Flow Trigger	PS-1 Flow	PS-2 Flow	Outlet Gates
18-21	PS-1: 800-1000 cfs PS-2: 600-700 cfs	1500-2000 cfs	3x200 cfs	Tested irrigation releases to NE Rim Ditch

Table 6C-02 – Details for Alternative B Option 2 Test Runs 18 to 21

Run Name	Flow at S-79	Pump Rate at PS-1	Pump Rate at PS-2
C-43_BerryFarm_Alt2_Test26_2050 Test 27 = Test 26 + Structure S-15	> 600 cfs	0 cfs	250 cfs
	> 750 cfs	0 cfs	500 cfs
	> 800 cfs	250 cfs	500 cfs
	> 900 cfs	750 cfs	500 cfs
	>1000 cfs	1250 cfs	500 cfs
C-43_BerryFarm_Alt2_Test25_2050	>600 cfs	0 cfs	250 cfs
	>750 cfs	0 cfs	500 cfs
	>800 cfs	250 cfs	500 cfs
	>900 cfs	500 cfs	500 cfs
	>1000 cfs	1000 cfs	500 cfs
C-43_BerryFarm_Alt2_Test24_2050	>600 cfs	0 cfs	250 cfs
	>750 cfs	0 cfs	500 cfs
	>800 cfs	250 cfs	500 cfs
	>900 cfs	500 cfs	500 cfs
	>1000 cfs	1000 cfs	500 cfs
	>1100 cfs	1500 cfs	500 cfs
C-43_BerryFarm_Alt2_Test23_2050	>800	0 cfs	300 cfs
	>900	0 cfs	600 cfs
	>1000 cfs	250 cfs	600 cfs
	>1100 cfs	500 cfs	600 cfs
	>1200 cfs	1000 cfs	600 cfs
	>1300 cfs	1500 cfs	600 cfs
C-43_BerryFarm_Alt2_Test22_2050	>700 cfs	0 cfs	300 cfs
	>800 cfs	0 cfs	600 cfs
	>900 cfs	250 cfs	600 cfs
	>1000 cfs	500 cfs	600 cfs
	>1100 cfs	1000 cfs	600 cfs
	>1200 cfs	1500 cfs	600 cfs
C-43_BerryFarm_Alt2_Test22_2050	>800 cfs	250 cfs	0 cfs
	>900 cfs	500 cfs	0 cfs
	>1000 cfs	1000 cfs	600 cfs

Table 6C-03 – Details for Alternative B Option 2 Test Runs 22 to 26

The test runs conducted after the receipt of the 2050 land use and updated S-77 outflows evaluated pump station capacity for PS-1 and trigger flows for turning on PS-1 and PS-2. The operating controls for PS-2 were modified from test runs 1-18. In the initial test runs, PS-2 turned on when flows into the Perimeter Canal exceeded threshold levels. This resulted in filling of the Reservoir during periods when flow in the C-43 Canal at S-79 was below desired flows. During these times, runoff from the Roberts Canal and the NE Rim Ditch should bypass the Reservoir and flow into the C-43 Canal. Test Run 20 and subsequent runs utilized operating controls for PS-2 that were based on the C-43 Canal flow. Test 24 and 27 are compared in **Figure 6C-01**. Test 24 has PS-1 at 1500 cfs, and Test 27 has PS-1 at 1250 cfs. Test 27 also has Structure S-15 on Roberts Canal near intersection with the Perimeter Canal. This structure was added to minimize draining of wetlands in Roberts Canal south of the proposed Reservoir. PS-2 inflows are

less with structure S-15 due to less than optimal gate operations associated with maintaining water levels upstream of the structure, and possibly due to the smaller capacity of PS-1.

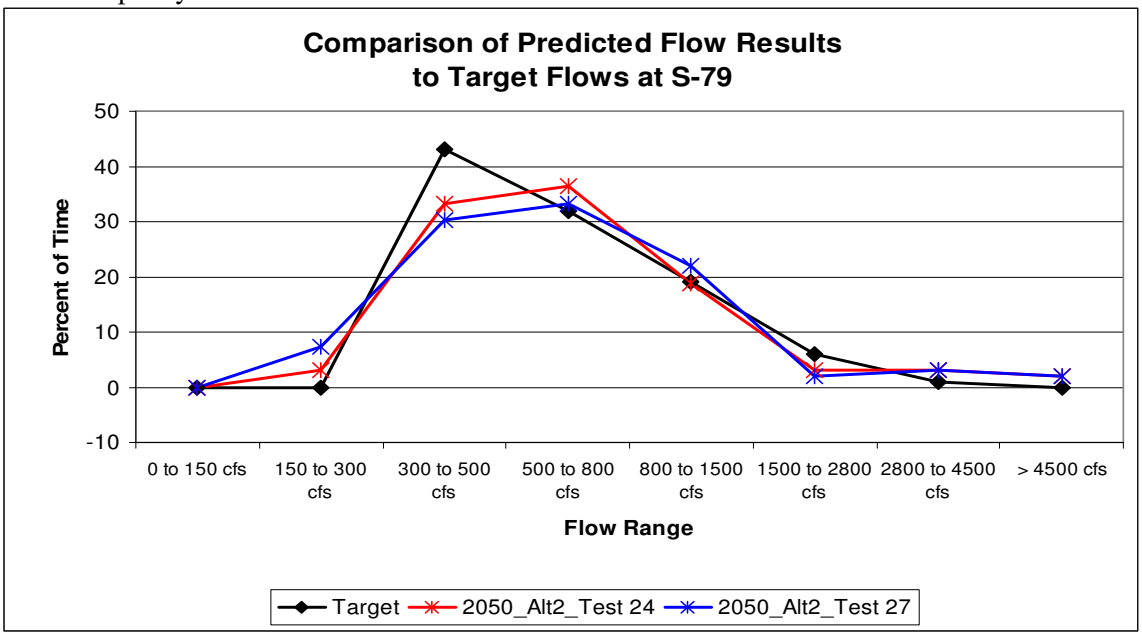


Figure 6C-01 – Comparison of Alternative B Option 2 Test 24 to Option 2 Test 27

Figure 6C-02 presents the results of Alternative B Option 1 Test Run 33 to Alternative B Option 2 Test Run 27. Reservoir elevations for these test runs are presented in **Figure 6C-03**. It can be seen that there are small differences between Option 1 (one pump station) and Option 2 (two pump stations). Because the differences are relatively small, selection of the best option will be based on other factors, such as cost, environmental benefits, and ease of operation.

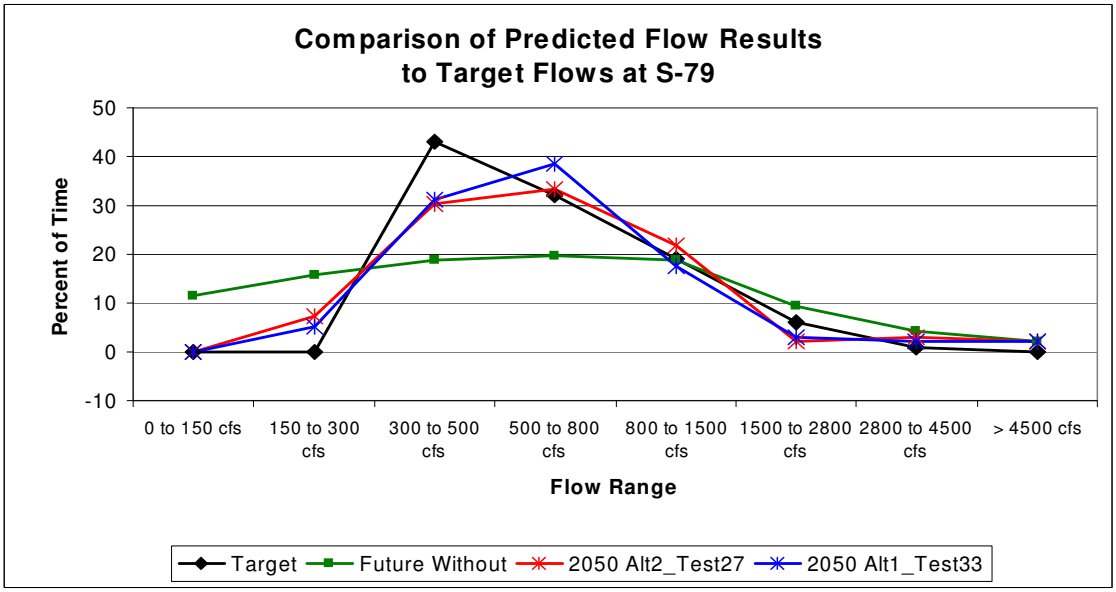


Figure 6C-02 – Comparison of Alternative B Option 1 Test 33 to Option 2 Test 27

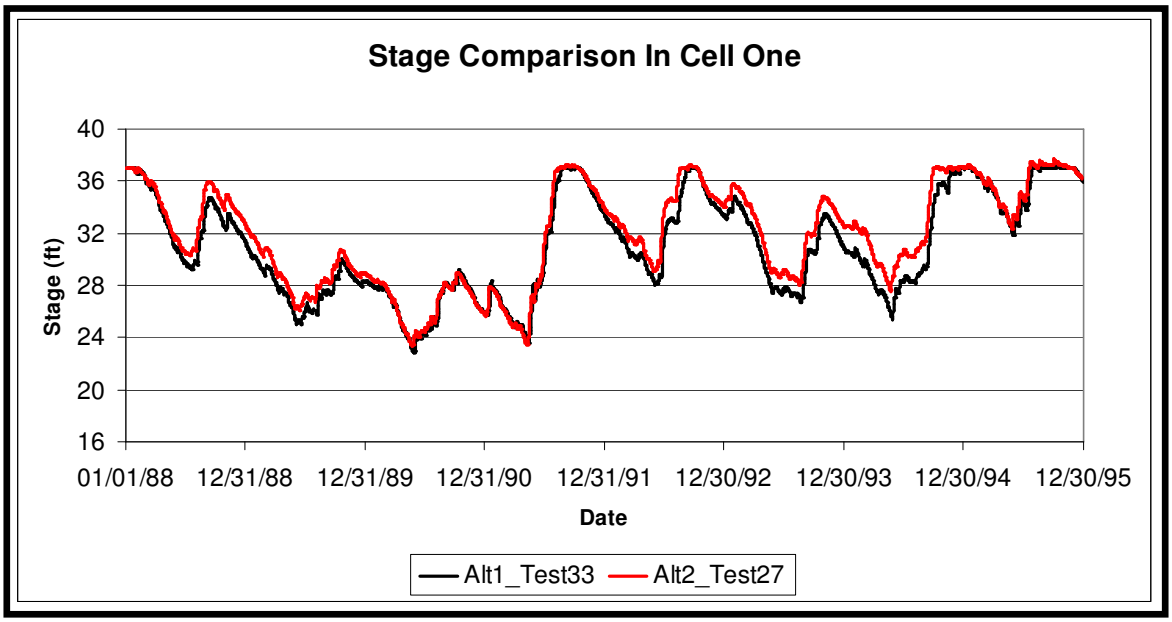


Figure 6C-03 – Reservoir Elevations for Alternative B Option 1 Test 33 vs Option 2 Test 27

The details of Alternative B Option 2 are provided in **Table 6C-04**.

Figure 6C-04 illustrates that peak flows in 1995 were higher than target peak flow performance measures. 1995 was an extremely wet year. The peak flow reduction target may be met with a simulation of a longer period of record, or it is possible that the peak flow reduction target cannot be met with the Reservoir due to volume limitations. The wet season runoff volume above 1,200 cfs was 2.1 million acre-feet (this number was calculated by taking S-79 measured flows for June 1 through December 31 and subtracting 1,200 cfs from any flows that exceeded 1,200 cfs). It is instructive to note that the 1999 analysis by DHI for one Reservoir did not meet the base flow or peak flow reduction targets (DHI, 1999). Flow at S-79 exceeded 12,000 cfs three times in 1995; therefore one Reservoir with a 2,000-3,000 cfs pump station should not be expected to control the peak flows experienced in 1995. In the 1999 analysis, the only scenario to meet both the baseflow and peak flow reduction targets was the 1999 Alternative 4, which consisted of nine Reservoirs distributed across the C-43 Basin. This Alternative used a 2020 land use file that had a greater amount of irrigated land than the land use file used in the current analysis. The greater amount of irrigated land resulted in greater demands during the dry season, which explains the greater storage capacity requirement calculated in 1999 (DHI, 1999).

Feature	Capacity	Operation Rule
Inflow Pump Station #1	2 x 500 + 2 x 250 cfs	Operate if water level in reservoir < 37.0 ft and WL in Townsend canal > -3.25 ft and flow at S-79 > 800 cfs
Cell 1 Outflow Gate S-1 (2 gates)	Inv. = 20.0 ft, W = 12 ft Inv = 16.0 ft, W = 22 ft	Opens if flow at S-79 < 300 cfs and water level in reservoir < 27 ft Lower gate only opens to drain reservoir.
Cell 2 Outflow Gate S-8 (2 gates)	Inv. 25.0 ft, W = 18 ft Inv. 20.0 ft, W = 12 ft	Opens if flow at S-79 < 300 and head in Cell 2 is < 32. Peak flow at 500 cfs, close if S-79 >400 cfs
Cell 3 Outflow Gate S-5	Inv. 30 ft, W = 12 ft Inv. 25 ft, W = 12 ft	Open if flow at S-79 < 400 cfs, and head in Cell 3 is >27 ft. Peak flow at 500 cfs, close if S-79 > 400 cfs.
Cell 1 Outflow Gate S-2	Inv. 23 ft, W = 5 ft	Opens if WL in NE Rim Ditch < 23.29 ft and head in Cell 1 > 29.5 ft
Perimeter Canal	BW = 10 ft at 3.5 ft TW = 130 ft at 23.5 ft	Maintain at 16 ft upstream of S-10 Maintain at 10 ft. upstream of S-11
S-10 Overflow Gate in Perimeter Canal	Five 5-ft gates Sill 13. 3 ft H	Overflow gates. Starts to open if WL >16.5 ft. in 6-inch increments. Close if WL < 16.5 ft
S-11 Trapezoidal Weir	60 ft W at 10 ft, 150 W at 14 ft	No operation
Perimeter Canal Pump	20 cfs	Pump if WL > 15.75 ft
S-6, -7, and -9 Weir & Orifice	Inv. 16.5 ft W = 1 ft., W = 30 f t. at 17.25 ft. Orifice 1-ft. dia. at elev. 14 ft.	S-6 – Crawford Canal S-7 – Roberts Canal S-9 – Fort Simmons Branch
S-4 on Perimeter Canal (2 underflow gates)	Sill 10 ft, W = 10 ft, H = 10 ft.	Open if U/S WL (between S-4 and S-3) is >18 ft
Inflow Pump Station PS-2	2 x 250 cfs	Pump if H>16 and S-79 Q > 600 cfs
S-15 on Roberts Canal (2 overflow gates)	Sill 22 ft, W = 12 ft, Max opening is 6 ft	Open if Roberts Canal WL > 25.5 ft
S-3 (1 underflow gate)	Sill = 16 ft, W = 12. ft, max opening is 10 ft. (gate is 10 ft high)	Open if NE Rim Ditch WL > 23.5 ft, Duda PS is not on, and Cell 1 WL>29.5.
Culverts	Dimensions	
S-12 between Cells 1 and 2	Inv. 22.9 ft, two 10x10 ft	Modeled as culverts. Will be gates that are normally open.
S-13 between Cells 2 and 3	Inv. 22.97 ft, two 10 x 10 ft	

Table 6C-04 – Features of Alternative B Option 2 Test 27

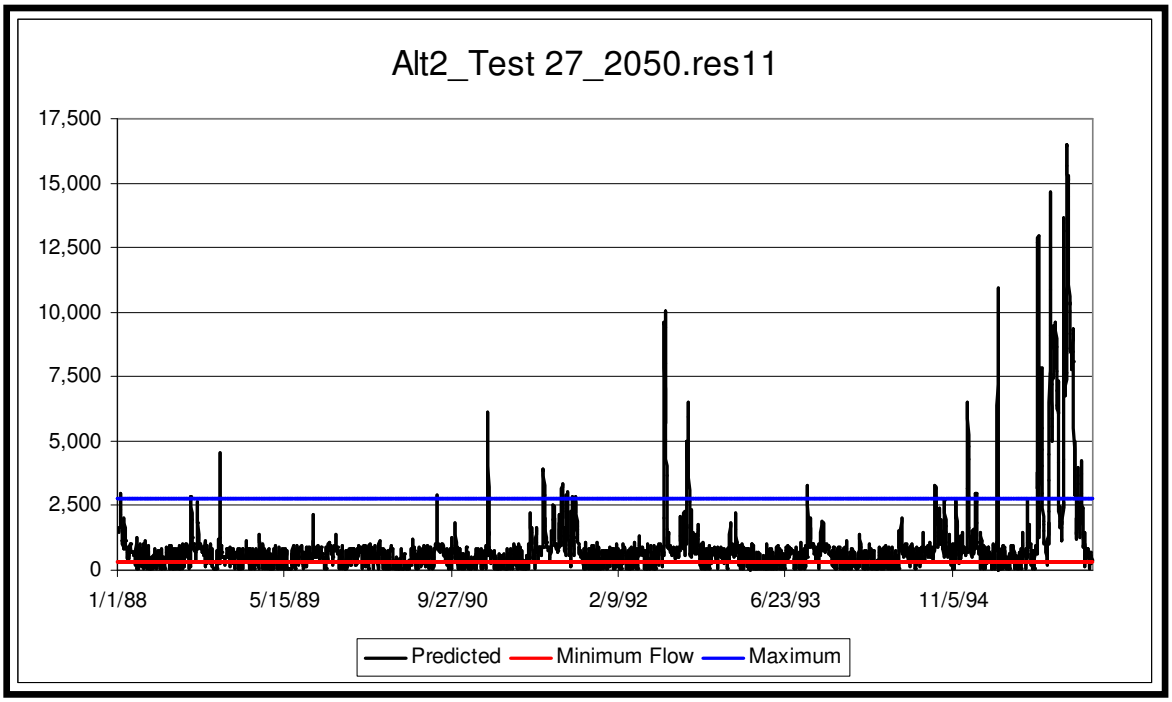


Figure 6C-04 – Simulated Flows at S-79 for Alternative B Option 2

D. Water Control Structures

1) Inlets/Outlets - Operations

The flow requirements for the hydraulic conveyance features of the Berry Groves reservoir are summarized in this section. The results of the model run Alternative B Option 1 Test 33 for an eight (8) year period from 1988 through 1995 that includes both dry and wet conditions will be used to define these flows. **Figure 6D-01** illustrates the PS-1 (northwest corner of Reservoir near junction with Townsend Canal) inflows (negative) and outflows (positive). The peak inflow at PS-1 is 1,500 cfs. The inflow time-series represents pump rates predicted at the structure when plotted every 24 hours. In the model, the pump operations are dependent upon flows at the S-79 structure. Reservoir discharge ranges from 0 to 600 cfs. The inflow pump station PS-1 is comprised of two 250 cfs pump stations, and two 500 cfs pump stations. One 250 cfs pump station turns on when the flow at S-79 exceeds 650 cfs, and the second 250 cfs pump station turns on when the flow at S-79 exceeds 800 cfs. The 500 cfs pump stations turn on when S-79 flow exceeds 900 and 1,000 cfs. The outflow gates open during periods when low flows are observed at S-79. When modifying the operations of the existing gated structure, either the speed at which a gate opens or the trigger for opening the gate is modified. Dimensions for these gates and

pumps for Options 1 and 2 are summarized above in **Tables 6B-03 and 6C-04,** respectively.

Pump hours for the pumps that deliver water to the Reservoir are presented in **Figure 6D-02** for PS-1 and **Figure 6D-03** for PS-2.

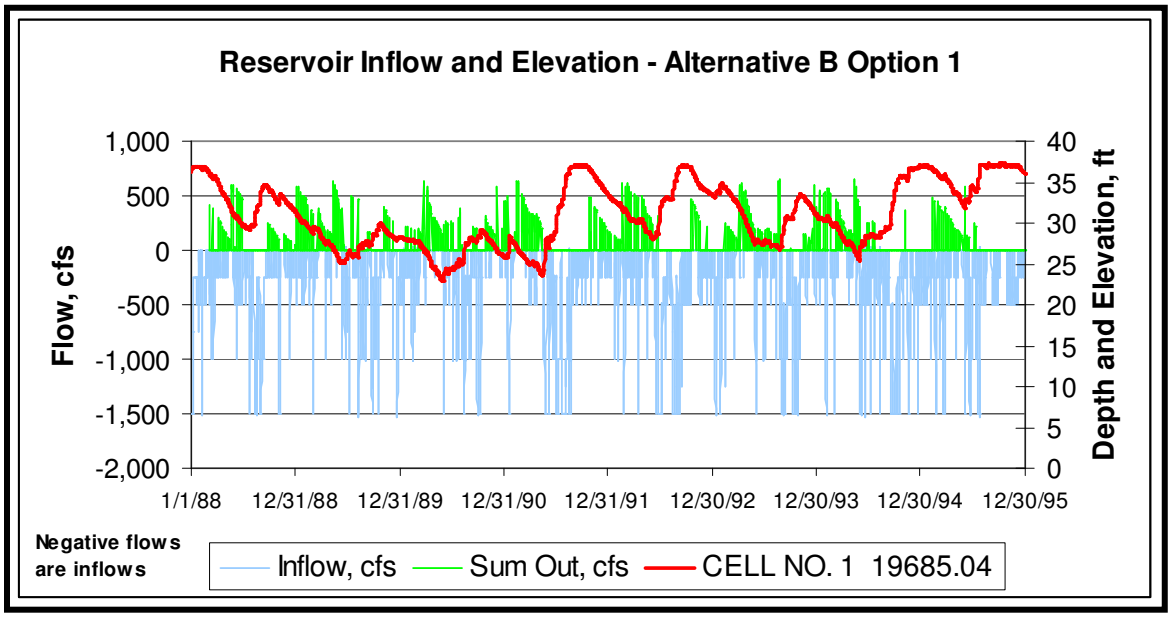


Figure 6D-01 – Flows at major inflow and outflow points for the Berry Groves Reservoir

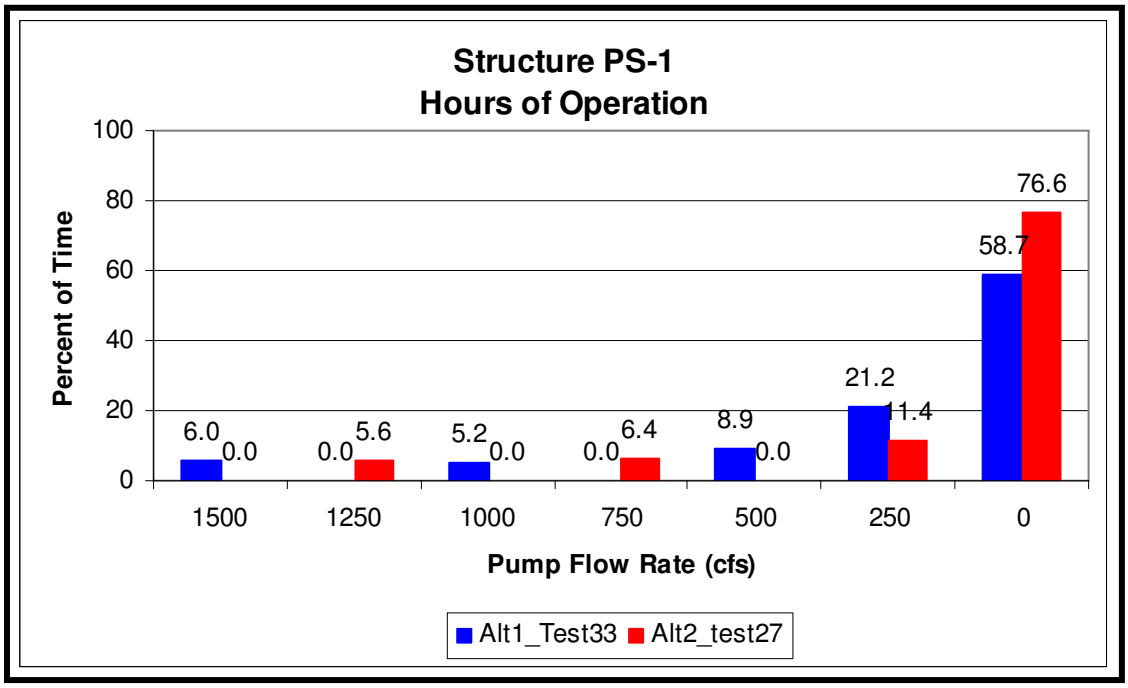


Figure 6D-02 – Hours of Operation for PS-1

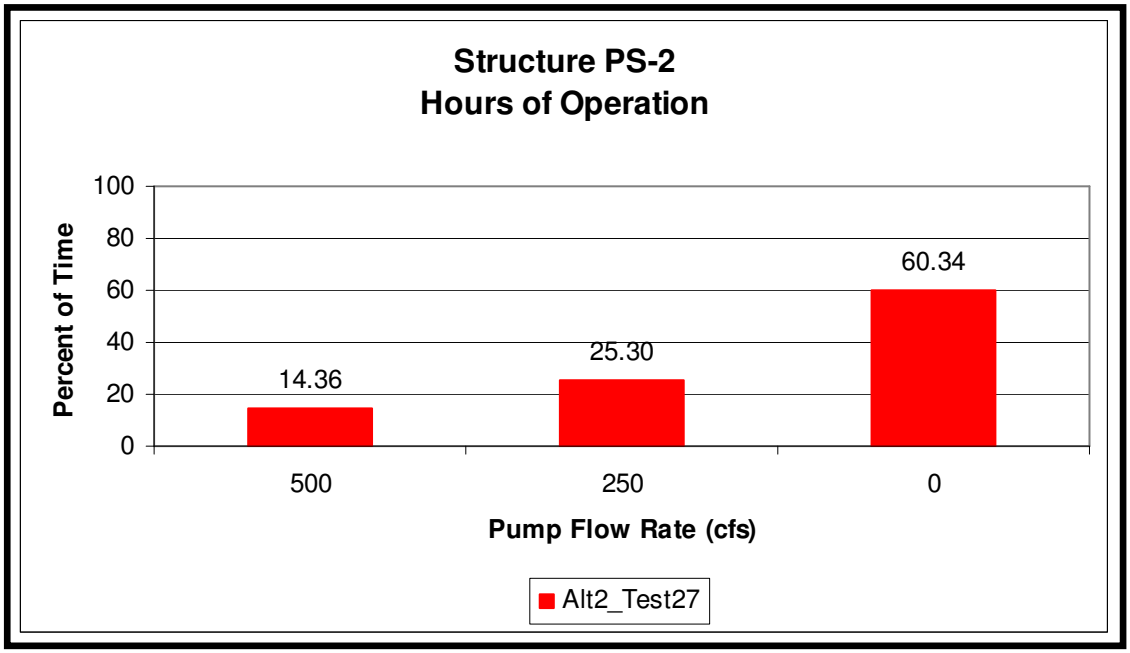


Figure 6D-03 – Hours of Operation for PS-1

The inflow pump station PS-1 is located on the Townsend Canal, and the source of water for this pump station is the C-43 Canal and runoff from the Perimeter Canal. Townsend Canal dimensions were evaluated during the design process, and inflow pumping was tested with both existing dimensions and an enlarged Townsend Canal, as shown previously in **Section 6B** in **Figure 6B-05**.

The reservoir outflow gates for Alternative B Option 1 (one pump station in cell 1) are programmed to release water first from cell 3, then cell 2, and then cell 1. This operation is intended to maximize pollutant retention in the reservoir. **Figure 6D-04** presents a graph of the volume discharged through the three cells.

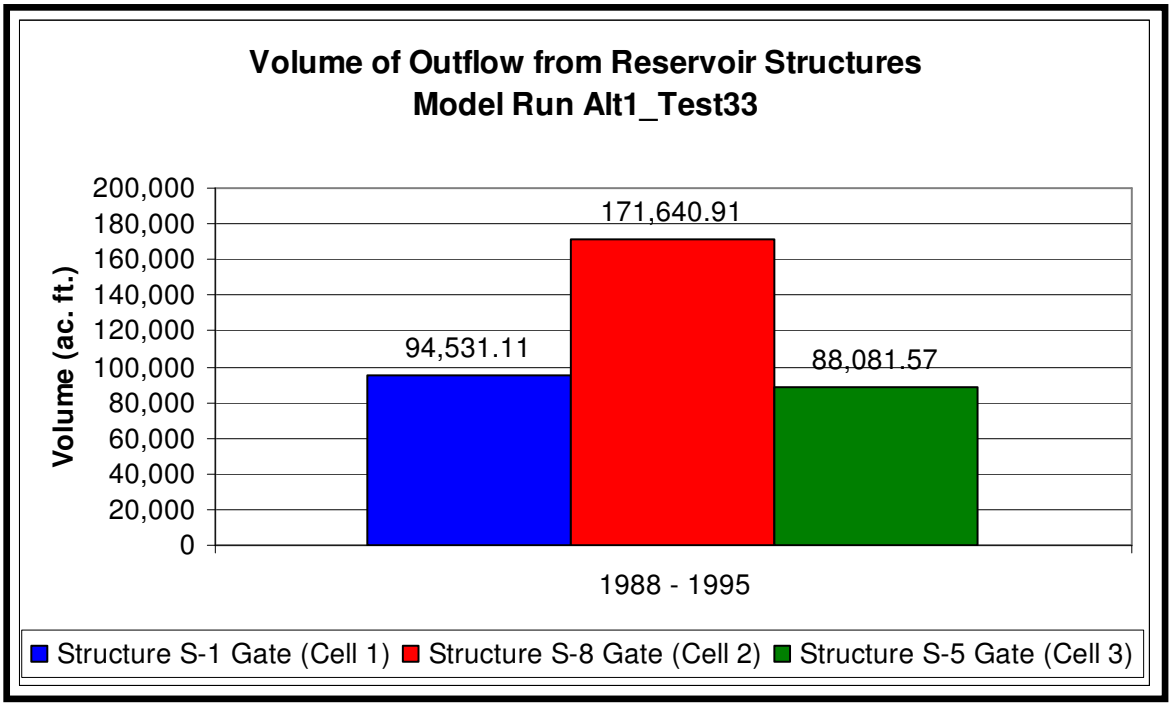


Figure 6D-04 – Volumes Discharged from the Three Cells of the Proposed Reservoir

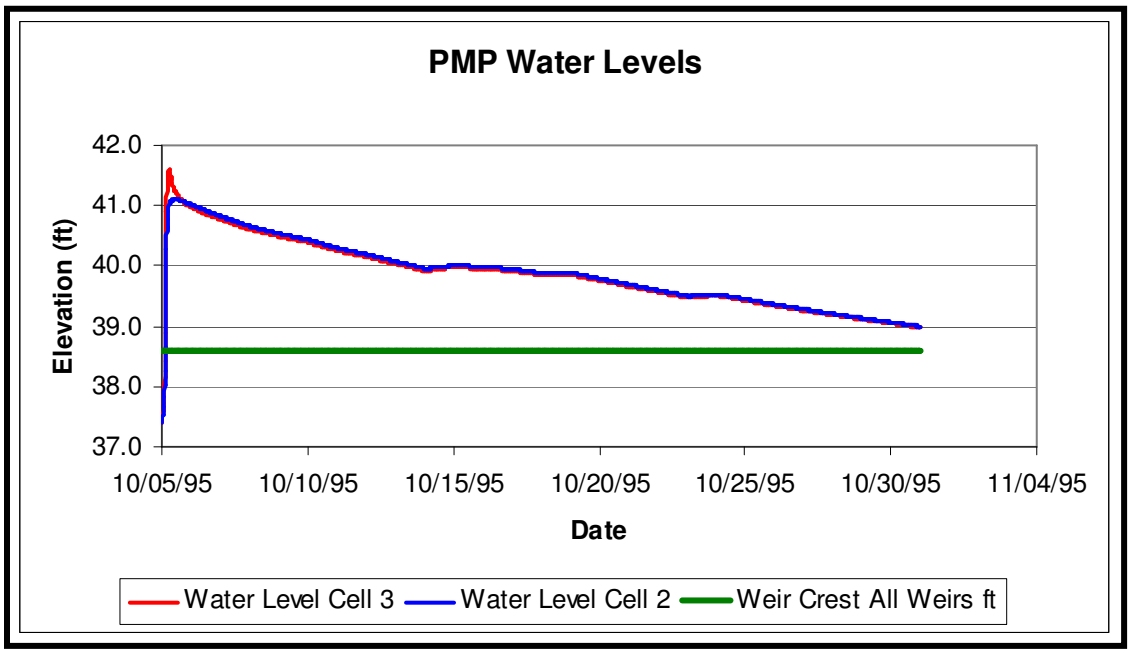
2) Emergency Evacuation Requirements

The Reservoir has emergency overflow weirs for each cell, and the 10-ft. freeboard can store the entire volume of a Probable Maximum Precipitation event discussed in **Section 5 D 4**). The emergency overflow weirs are all set at elevation 38.6 ft., which is slightly higher than the Reservoir full level plus 11 inches. Eleven inches is the approximate depth of 25-year, 3-day storm. The Cell 1 weir is 35 ft. wide, and the Cell 2 and 3 weirs are 25 ft. wide. The top of bank elevation is approximately 49 ft. for Alternative A and approximately 47 ft. for Alternative B. Therefore, these weirs can flow at a maximum depth of 10 ft. The Perimeter Canal can easily handle the PMP outflows, therefore there are no emergency evacuation requirements.

The reservoir will have an outflow during a Probable Maximum Precipitation Event. The peak stages and depths for the three outflow points are shown in **Figure 6D-05**. This figure illustrates that the combined flow from these three gates is less than 1000 cfs, which is less than the flow that is observed in the tributaries in the vicinity of the proposed reservoir. Because existing flows in the tributaries are significantly greater than the reservoir PMP outflows, the outflow gates and the proposed Perimeter Canal can safely convey the PMP. If the PMP falls on more than the area of the reservoir and falls on the entire Berry Groves Catchment (120 square miles), significant out-of-bank flooding will be experienced in the area with and without the proposed reservoir.

The presence of the reservoir will result in mounded groundwater levels in areas adjacent to the reservoir. Potential positive benefits to this include reduced irrigation pumping costs while potential negative impacts include increased drainage pumping costs. This issue has not yet been quantified but will be during a subsequent phase of the study. It is the intent of the project to control groundwater levels outside the project by controlling the water levels in the Perimeter (seepage) Canal. The groundwater levels within the Duda property will be maintained by controlling the water surface elevation in their canal system.

During the optimization process, approximately 32 runs were completed to evaluate the 3-cell, 2-pump alternative. During modeling activities, the final run for this set-up was referred to as Alternative 1_Test32. Approximately 27 runs were completed to evaluate the 3-cell, 1-pump alternative. During modeling activities, the final run for this setup was referred to as Alternative2_Test26. The simulation, Alternative2_Test26 is most representative of the selected design. Final runs for Alternative A, the two cell alternative, will be made prior to re-starting the 30% preliminary design, now anticipated for June 2006.



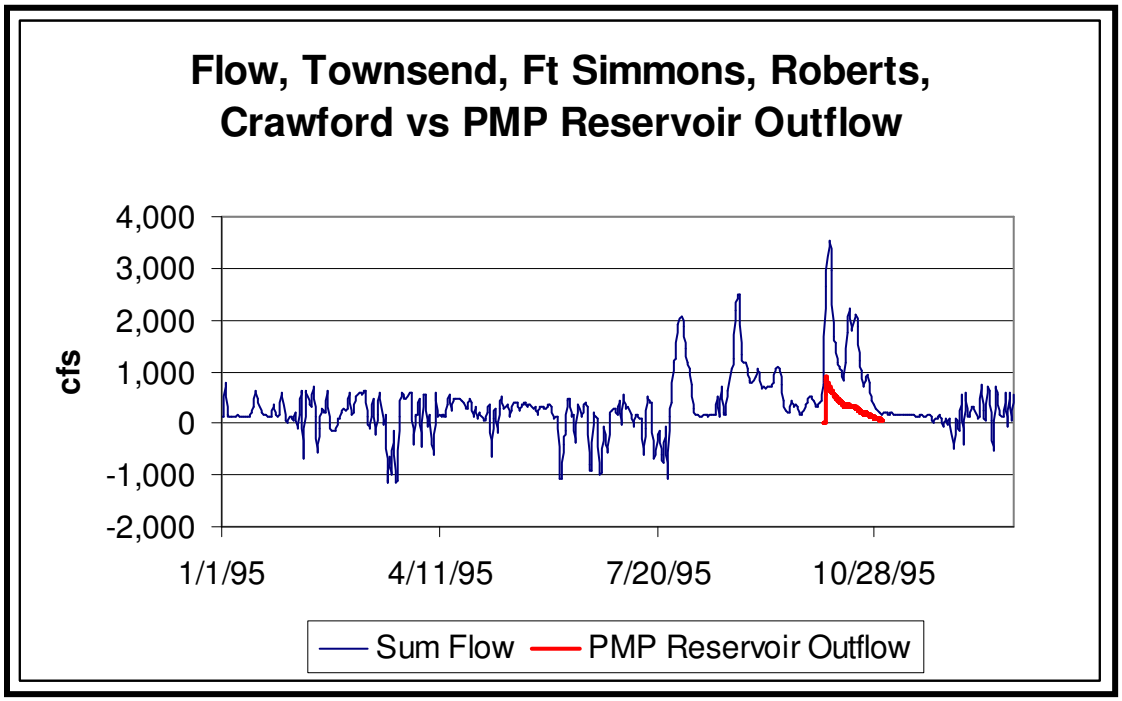


Figure 6D-05 – PMP Reservoir Water Levels and Outflows from Cells 1, 2, and 3 and Simulated 1995 Existing Conditions Total Flow from Townsend Canal, Fort Simmons Branch, Roberts Canal, and Crawford Canal

3) Perimeter Canal

The Perimeter Canal has been designed to intercept seepage from the proposed Reservoir. The invert elevation was selected to be just above the top of the clay aquiclude that exists underneath the site. The system is designed so that the Perimeter Canal will intercept any excess seepage from the Reservoir.

The Perimeter Canal is controlled by a weir (S-10) at 1.8 miles east of the junction with Townsend Canal that maintains a normal water surface elevation of 16 ft. in the Perimeter Canal upstream of that weir. Downstream of this weir, the water surface of the perimeter control canal is maintained at 10 ft. This modification was added because the natural ground elevation is in the range of 13 ft. in the northwest corner of the proposed Reservoir. The design team established the 10 ft. control elevation in the last 1.8 miles of the Perimeter Canal so that the Perimeter Canal elevation would not be higher than adjacent surficial ground water elevations north of the proposed Reservoir. The modeling includes a pump that returns 20 cfs to the Reservoir from the Perimeter Canal to counteract this seepage. This value was selected based on an inspection of the Perimeter Canal flows during baseflow conditions for a simulation that did not include the seepage return pump.

The proposed Reservoir will utilize the NE Rim Ditch on the Duda Farm as a seepage control ditch, and an underflow gate will be constructed at the eastern end of the NE Rim Ditch. A number of changes were made in the area of the NE Rim Ditch to account for the addition of the Reservoir. These changes are summarized below:

- This canal will be expanded so that the bottom elevation is close to the top of the clay aquiclude, plus the width is larger than the existing ditch.

- Operating controls for the main Duda pump station used for irrigation were modified so that the pump does not operate until the water level in the Townsend Canal south of the pump station drops below 23.0 ft. and the water level in the Townsend Canal north of this pump station is >0.5 ft. and S-2 is closed.

- An outflow gate S-2 was added to Cell 1 of the Reservoir to provide irrigation water to the NE Rim Ditch (see **Table 6C-04** for details).

The water levels in the NE Rim Ditch are presented in **Figure 6D-06**, and irrigation pumping for the main pump station in the Townsend Canal (this is a private pump station operated by Duda) is presented in **Figure 6D-07**. **Figure 6D-06** illustrates that NE Rim Ditch elevations are more constant than existing conditions with no periods of low water levels. **Figure 6D-07** illustrates that pump requirements of the Duda main pumping station will be considerably less than for existing conditions. This is because the Reservoir maintains higher

groundwater levels than existing conditions in the area south of the Reservoir, and there is additional capacity for irrigation from the expanded NE Rim Ditch.

The project team does not believe there will be an impact o the irrigation system. It is proposed that the water levels in the NE Rim Ditch be maintained at the elevation preferred by Duda. The water levels in the NE Rim Ditch can be controlled by pumping from the Townsend or releasing to the Townsend if the NE Rim Ditch is separate from the Perimeter Canal. If the NE Rim Ditch is included in the Perimeter Canal, water levels can be maintained by the Townsend canal pump station and/or S-2 to raise the level and S-3 and the existing Townsend structures to lower the level. In summary, the water surface elevation in the NE Rim Ditch can be easily controlled.

Structure S-2, intended for irrigation supply to Duda, was never opened during the simulation because Structure S-2 could not be opened unless the Reservoir elevation was higher than elevation 29.5 ft. The periods of low water levels coincided with periods when the Reservoir elevation was less than 29.5 ft. An analysis was conducted of groundwater elevations within the Duda Farm to assure that the Reservoir did not cause lower groundwater levels. **Figures 6D-08 and 6D-09** are plots of ground water elevations at two locations within the Duda Farm. **Figure 6D-10** presents the location of these two plots.

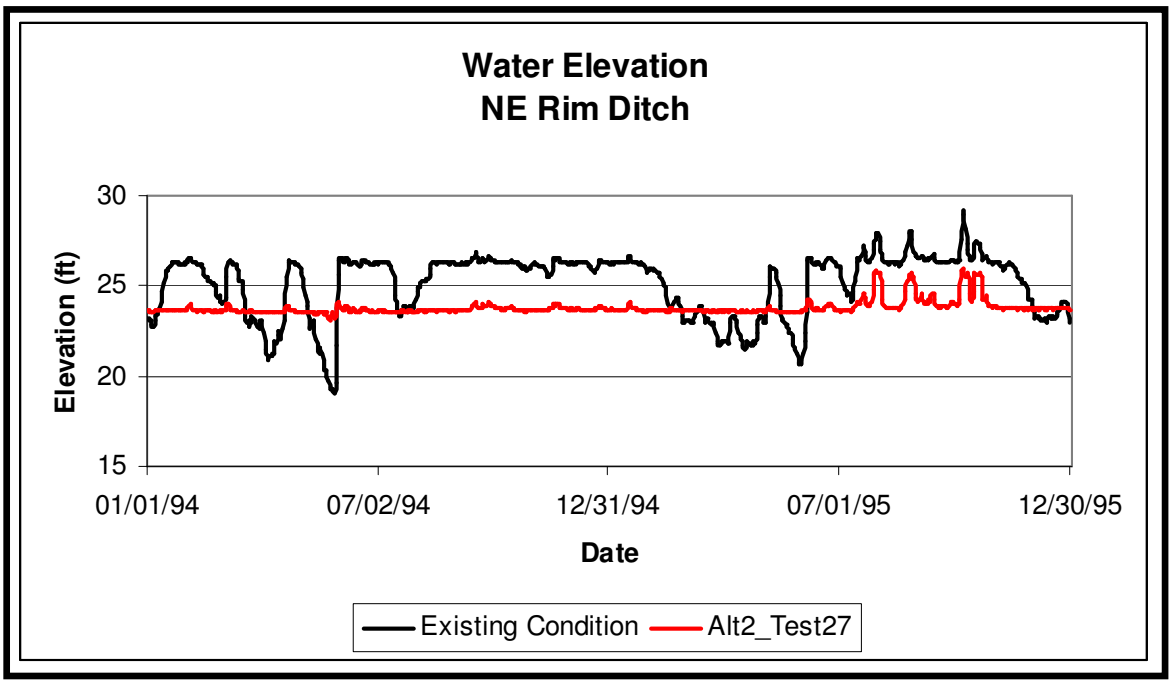


Figure 6D-06 – Water levels in the NE Rim south of the proposed Reservoir with and without the proposed Reservoir

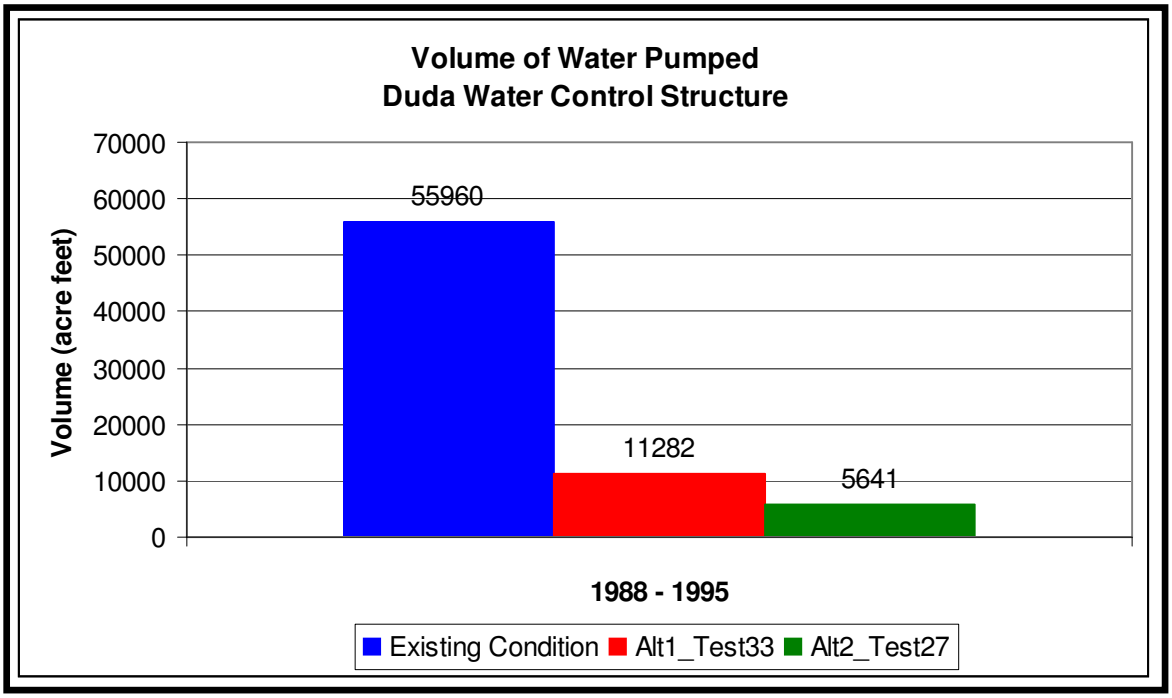


Figure 6D-07 – Pumped Volume at the Duda Irrigation Pumping Station on the Townsend Canal south of the proposed Reservoir with and without the proposed Reservoir

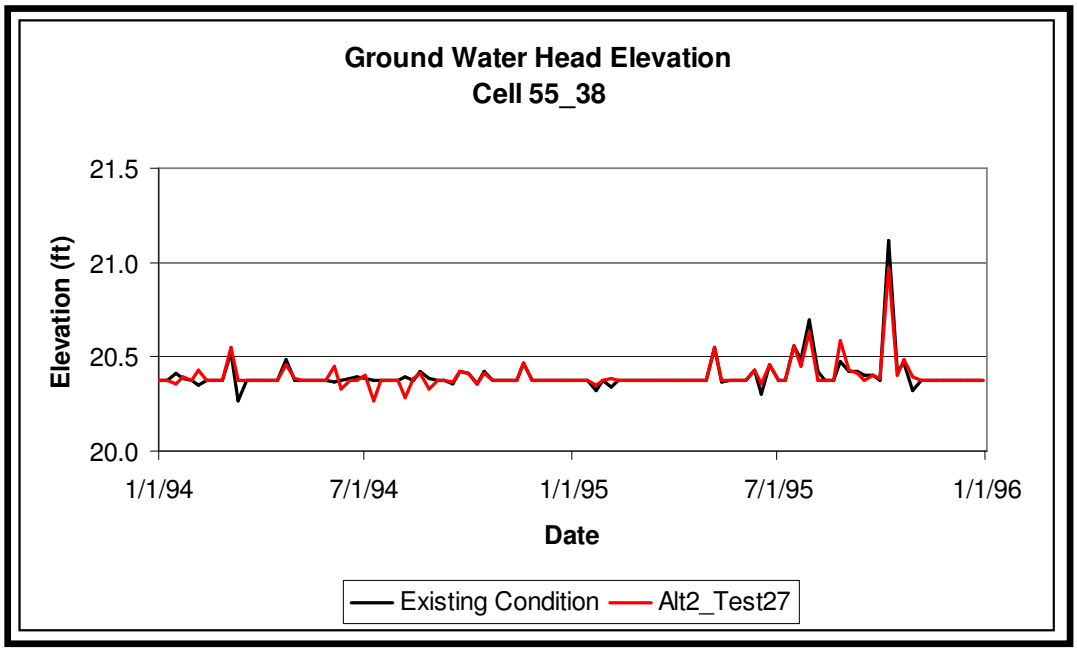


Figure 6D-08 – Plot of Groundwater Elevations in Cell 55_38 in the Duda Farm

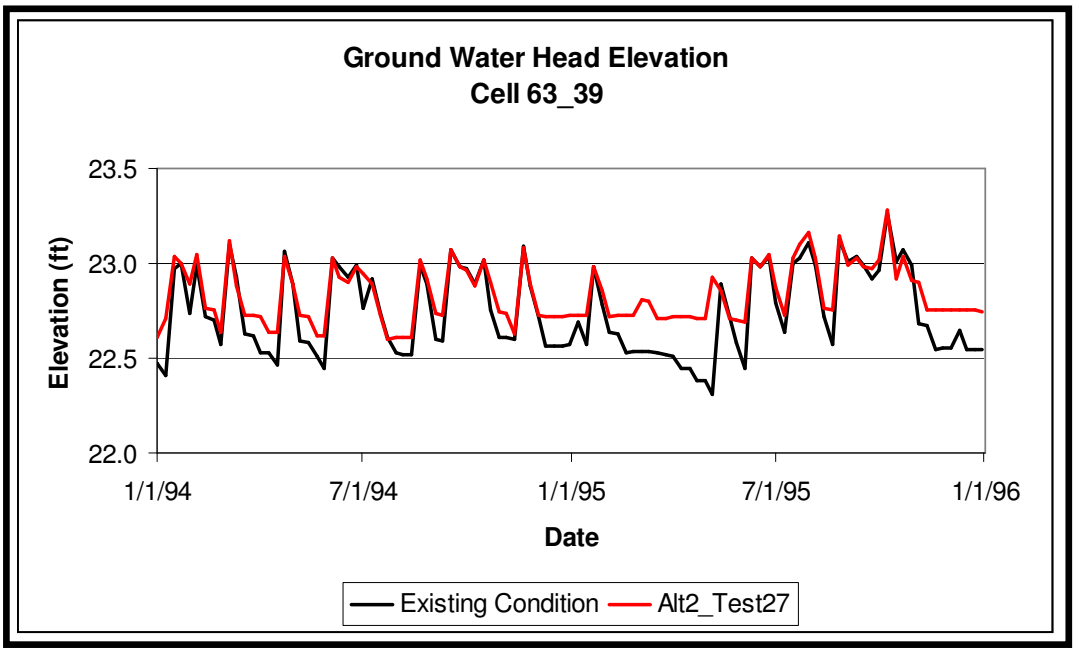


Figure 6D-09 – Plot of Groundwater Elevations in Cell 55_38 in the Duda Farm

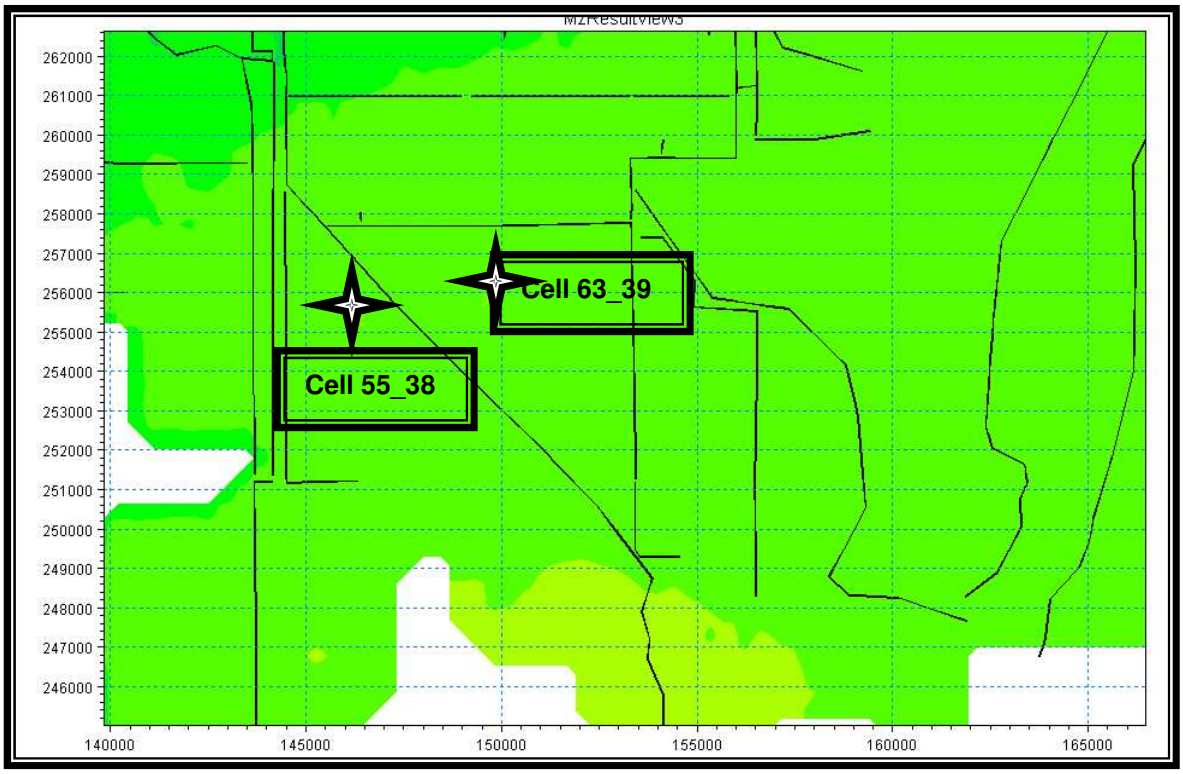


Figure 6D-10 – Location of Groundwater Plots in Duda Farm

Using the NE Rim Ditch as the seepage collector canal means that the SFWMD and the Corps (if this is incorporated into the federal project) would share operational control of this canal with Duda (private land owner to the south). The Corps may not be willing to participate in a project where control of a canal is shared with a private party. Or conversely, the private party (Duda) may not agree to the constraints placed upon the canal by the Federal partner. For these reasons, a separate Perimeter/Seepage canal paralleling the NE Rim Ditch is still under consideration. A separate canal would eliminate conflicting uses and overlapping maintenance responsibilities for the NE Rim Ditch. If a separate canal is utilized, the SFWMD will maintain the Perimeter Canal and Duda would maintain the existing NE Rim Ditch.

Water from the Perimeter Canal can discharge to either Crawford Canal, Roberts Canal, Fort Simmons Branch, or Townsend Canal. Because there will be constant water surface maintained in the Perimeter Canal, control structures were added to regulate discharges to these canals. Because there are no long-term flow measurements in Crawford Canal, Roberts Canal, and Fort Simmons Branch, it is not possible to determine if the control structures that regulate flows to these canals are appropriately sized. If the flow data remain unavailable during subsequent design phases, the S-9 structure will be modified so that the flows are consistent with the predicted, existing (Without-Project) condition flow regime. **Figures 6D-11 through 6D-13** present flows for existing conditions and the With-Project condition, however the existing condition flows were not calibrated to measured flow data. Monitoring of existing conditions would be necessary to determine if the proposed flows to these canals are appropriate.

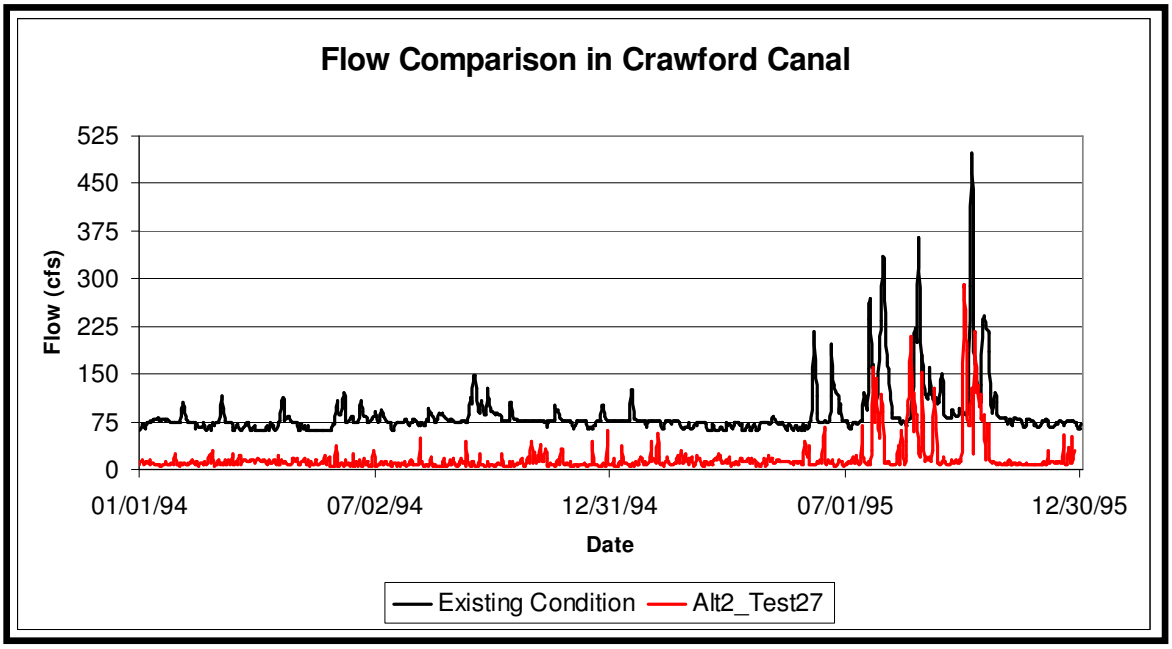


Figure 6D-11 – Flow Comparison in Crawford Canal

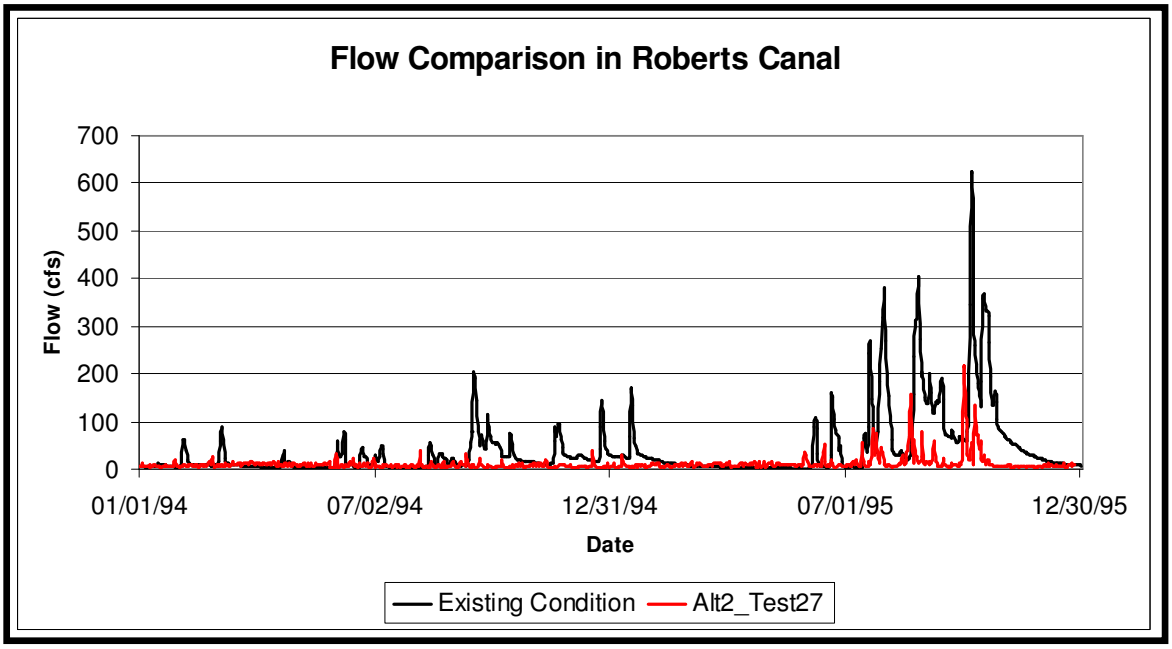


Figure 6D-12 – Flow Comparison in Roberts Canal

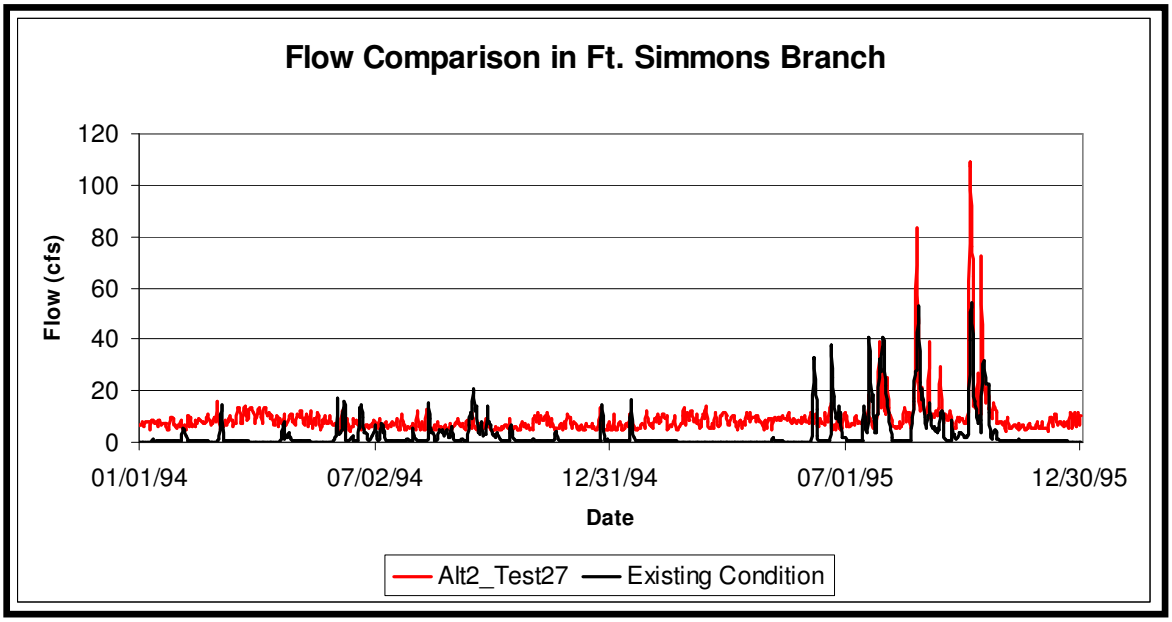


Figure 6D-13 – Flow Comparison in Fort Simmons Branch

4) Impacts in Roberts Canal South of the Proposed Reservoir

The Roberts Canal will be substantially affected by the Reservoir. The flow in Roberts Canal will be diverted into the Perimeter Canal and re-enters the existing alignment north of the proposed Reservoir. Control structure S-15 (see **Table 6C-04**) has been included in the proposed project to minimize dry-out of wetlands and flooding of farm lands. The effect of this structure is presented in **Figure 6D-14**, and the map of groundwater level differences for this area is presented in **Figure 6D-15**. The water level difference map presented in **Figure 6D-15** shows the impact of the changes in Roberts Canal on adjacent wetlands. It can be seen that there is no difference between existing conditions and proposed conditions during the summer wet season. A profile of water levels in Roberts Canal with and without the project is presented in **Figure 6D-16**. Structure S-15 appears to safely convey flood flows to minimize flooding problems and provides higher water levels during base flow periods.

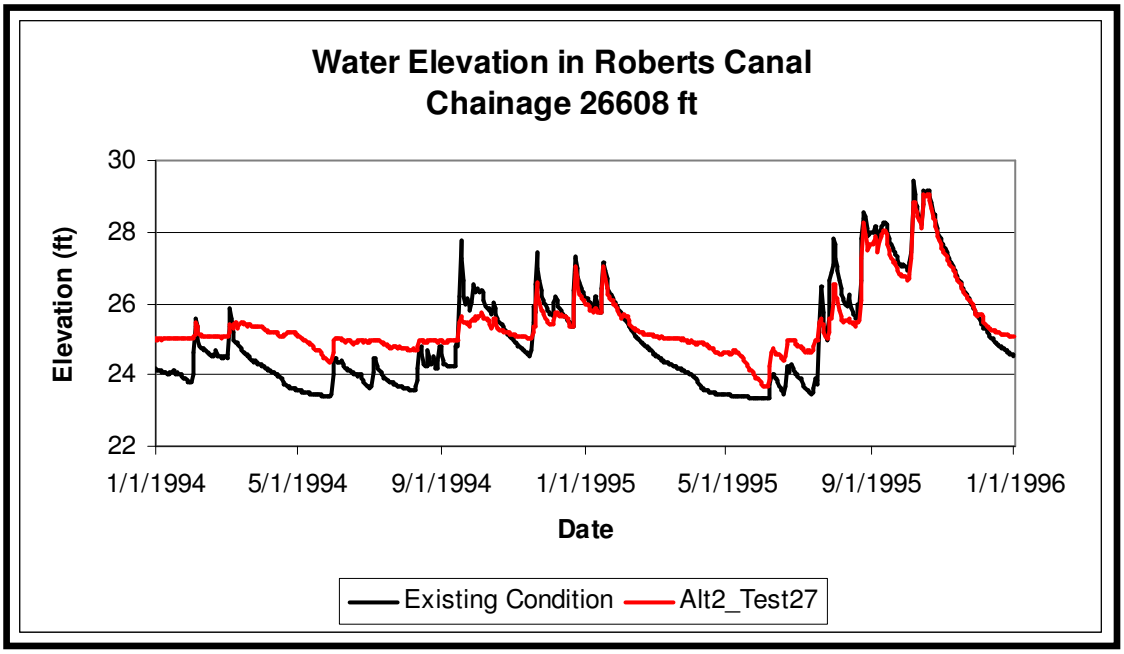


Figure 6D-14 – Roberts Canal Water Levels With and Without the Proposed Reservoir

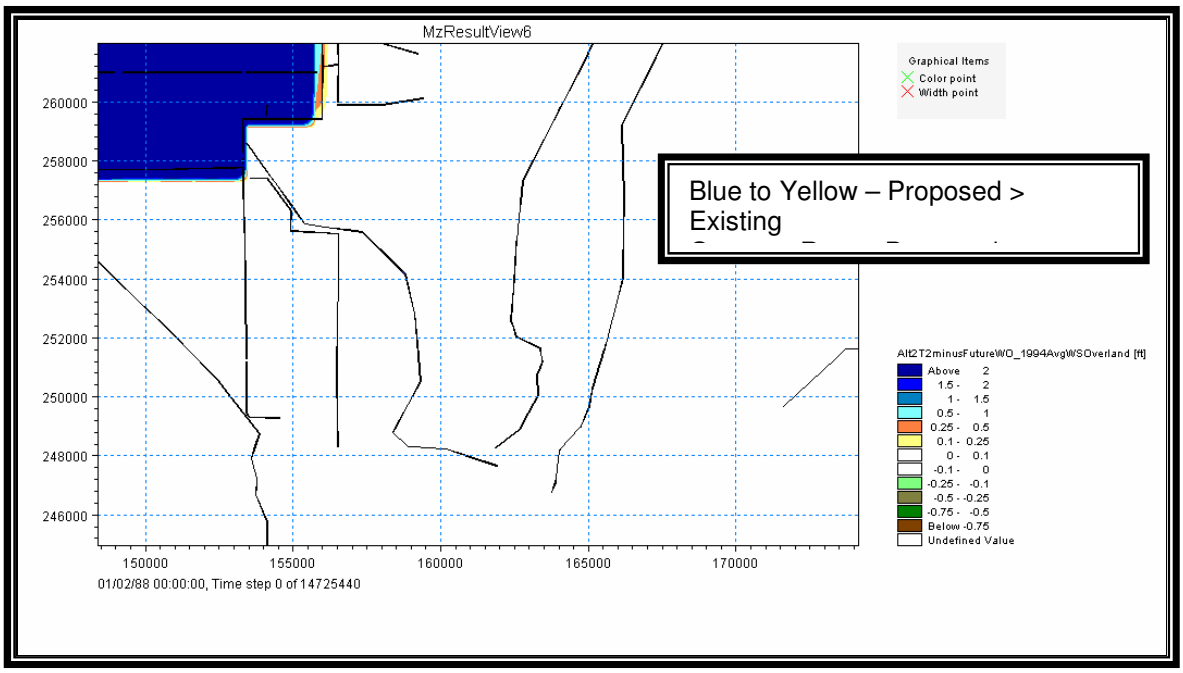


Figure 6D-15 – Map of Difference in Average Summer Wet Season Water Levels in the Vicinity of Roberts Canal for With and Without Project Conditions

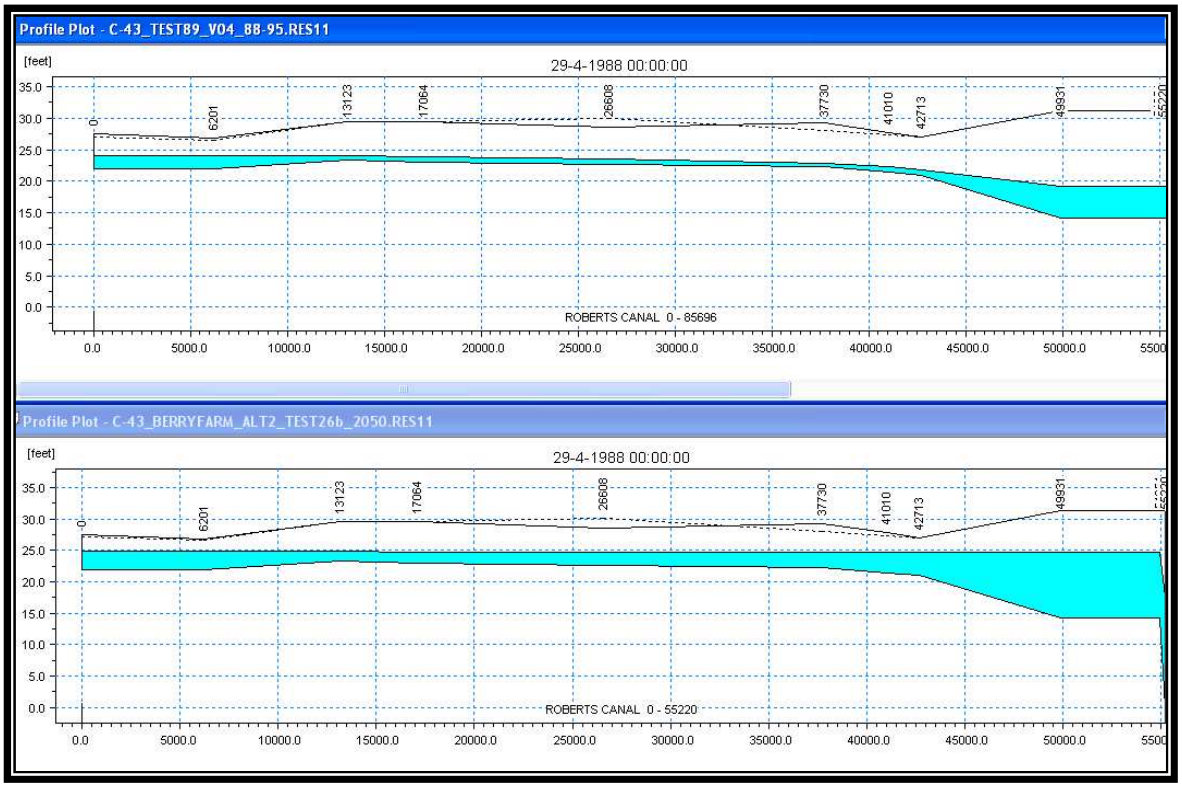


Figure 6D-16 – Profile of Roberts Canal Water Levels With (Bottom) and Without (Top) the Project

5) Impact of the Proposed Project on S.R. 29 South of Labelle

The proposed project includes a connecting channel (referred to as the LPDD Canal 3) from the S.R. 29 Ditch south of LaBelle to the Perimeter Canal. LPDD Canal 3 serves two purposes:

(a) Provide additional water to the Perimeter Canal that can be pumped into the Reservoir to augment low flows during the dry season.

(b) Provide additional conveyance capacity for flood flows to relieve flooding along S.R. 29 south of LaBelle.

Figure 6D-17 shows flows in the S.R. 29 Ditch for Existing Conditions, Future With Project Conditions, and the Future Without Project Conditions. It can be seen that peak flow in S.R. 29 Ditch for the With Project Conditions is less than 1/3 of the peak flow for either Existing Conditions or Future Without Project Conditions. **Figure 6D-18** illustrates that runoff that would otherwise continue north on the S.R. 29 Ditch toward LaBelle is diverted from the S.R. 29 Ditch into the LPDD Canal 3. This connection is expected to decrease flooding problems in the area along S.R. 29 south of LaBelle.

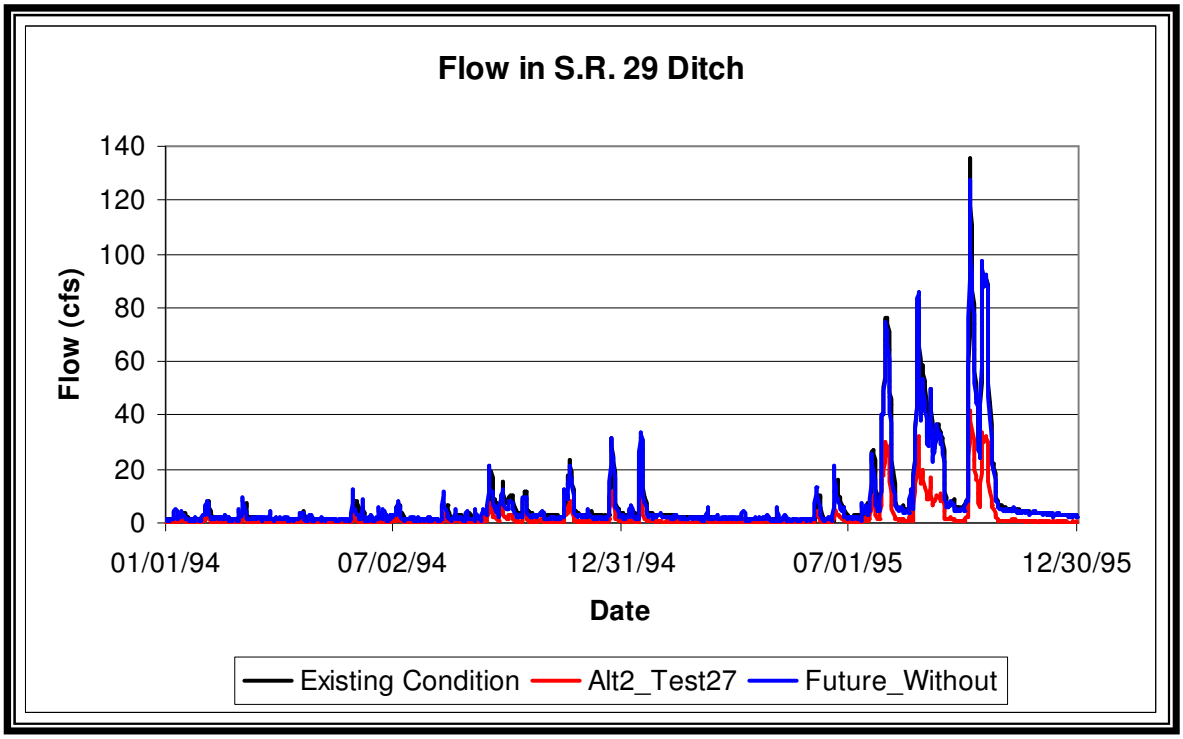


Figure 6D-17 – Flow in the S.R. 29 Ditch for With and Without the Project

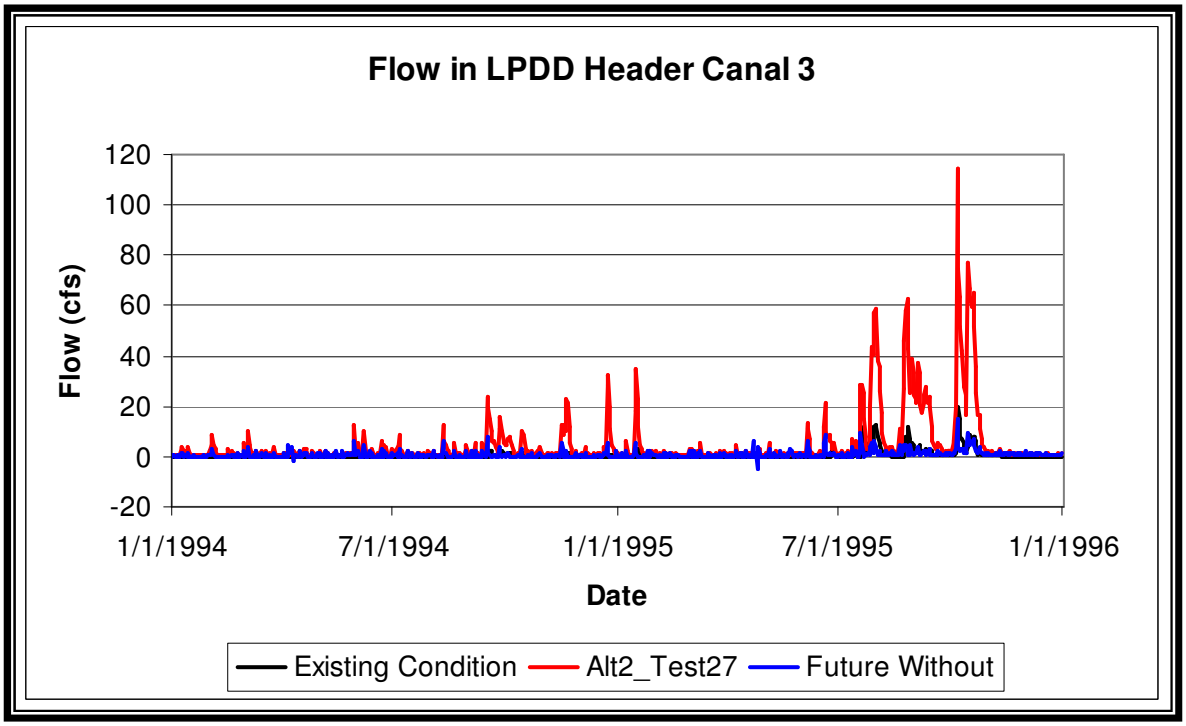


Figure 6D-18 – Flow in the LPDD Canal 3 for With and Without Project Conditions

The LPDD Canal 3 would have to demonstrate substantial environmental benefits in order to be considered as part of the Federal restoration project. However, the purpose of the connection to the SR 29 Ditch is to reduce local flooding east of SR 29 at the suggestion and request of Hendry County, and may be considered, a locally-preferred option depending on the recommendations in the PIR.

6) Reservoir Seepage and Evaporation Losses

The Reservoir will lose water due to both seepage and evaporation. A water budget analysis was conducted with a full Reservoir for a 3-month period with no rainfall and no Reservoir discharges (the existing rainfall file was altered to conduct this analysis). The water level in the Reservoir dropped 2 ft., with seepage equal to 16 inches and evaporation equal to 8 inches. Results from the future with project simulation using rainfall from 1994 with the Reservoir are presented in **Table 6D-01**.

Exist gradients on the face of the canal were checked as pat of the geotechnical analysis. The exit gradients were determined to be 0.3 or less.

The mass balance for Alternative B, Option 2 is similar to Option 1 except that Cell 3 PS-2 pumps approximately 50% of the inflow.

Components	Alt B Option 1
	Ac Ft/yr
Inflows	
Cell 1 PS-1	152,394
Cell 1 PS-2	
Seepage Pump S-8	14,469
Rainfall	20,265
Outflows	
Cell 2 Outflow S-8	3,194
Cell 3 Outflow S-5	38,658
Evaporation	23,508
Seepage	115,106
Sum In	187,128
Sum Out	180,465
Change in Storage	6,663

Table 6D-01 - Mass Balance for in and out flows

The seepage lost from the reservoir is approximately 69% of pumped inflow. The seepage rate reported by the geotechnical investigation is 6.2 cfs/mile of levee. This rate is equal to 88,500 ac ft/yr, which is 77% of the seepage

calculated by MIKE SHE. If seepage is as low as determined from the geotechnical investigations, the reservoir will perform better than reported by this section. Seepage is higher for the three-cell Alternative B than the two-cell Alternative A. If Alternate A is selected, the models will be adjusted to reflect these conditions. It is anticipated overall results will be very similar. The seepage volume will be significantly reduced with Alternative A.

E. Canals/Culverts/Flow-Ways

1) Water Surface Profiles

The water surface profile for Townsend Canal was presented above in **Figure 6B-05**. The existing capacity of Townsend Canal is sufficient for the project as long as the channel banks are maintained in a relatively smooth condition. This will require frequent removal of Brazilian Pepper. The water surface profile for Roberts Canal was presented previously in **Section D** in **Figure 6D-16**. Structure S-15 that is part of Option 2 will maintain higher water levels in Roberts Canal during baseflow conditions without causing higher peak flow water levels. This will be a benefit to wetlands adjacent to Roberts Canal near the southeast corner of the proposed reservoir. **Figure 6E-01** presents a profile of water levels for the Perimeter Canal. **Figure 6E-02** presents a profile of water levels for the connector canal between S.R. 29 and the Perimeter Canal.

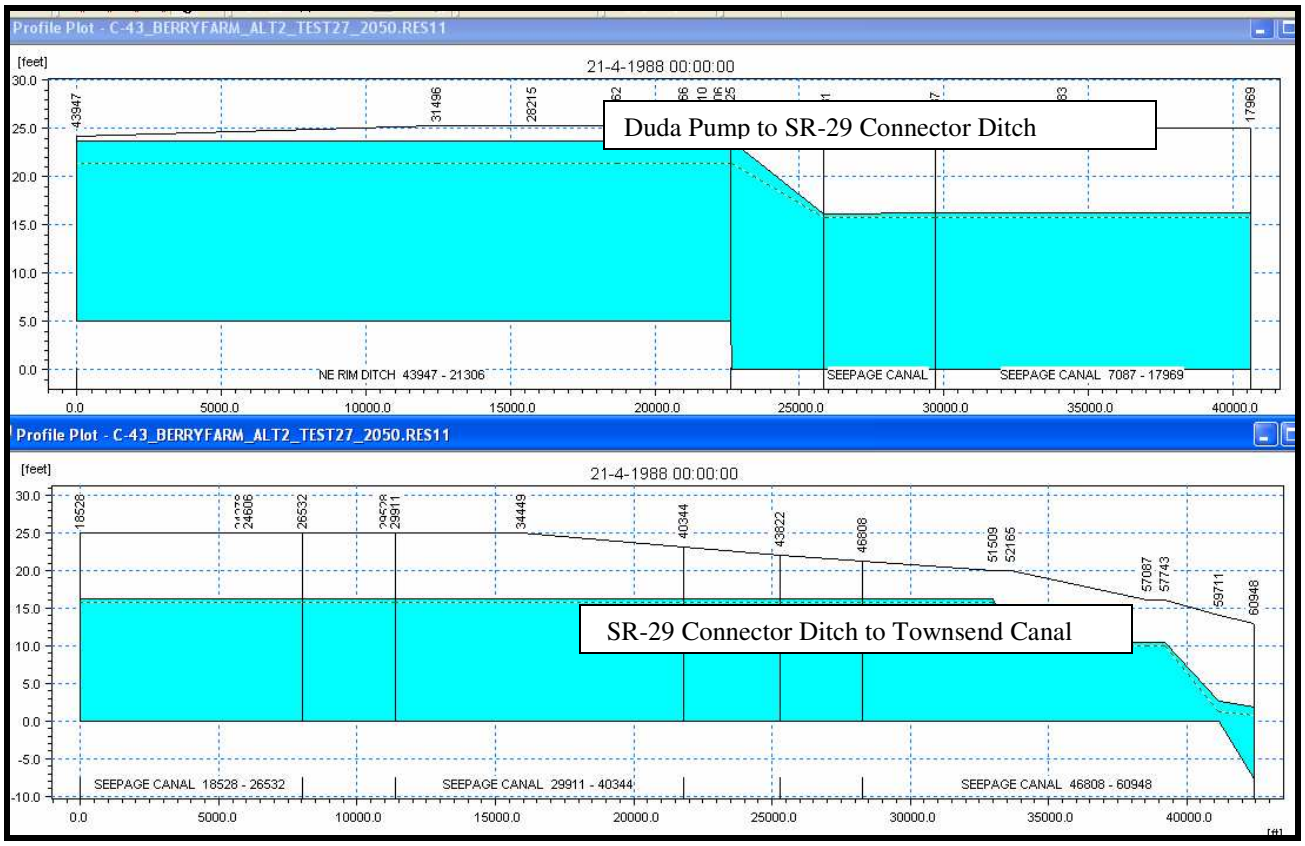


Figure 6E-01 – Water Surface Profiles for the Perimeter Canal

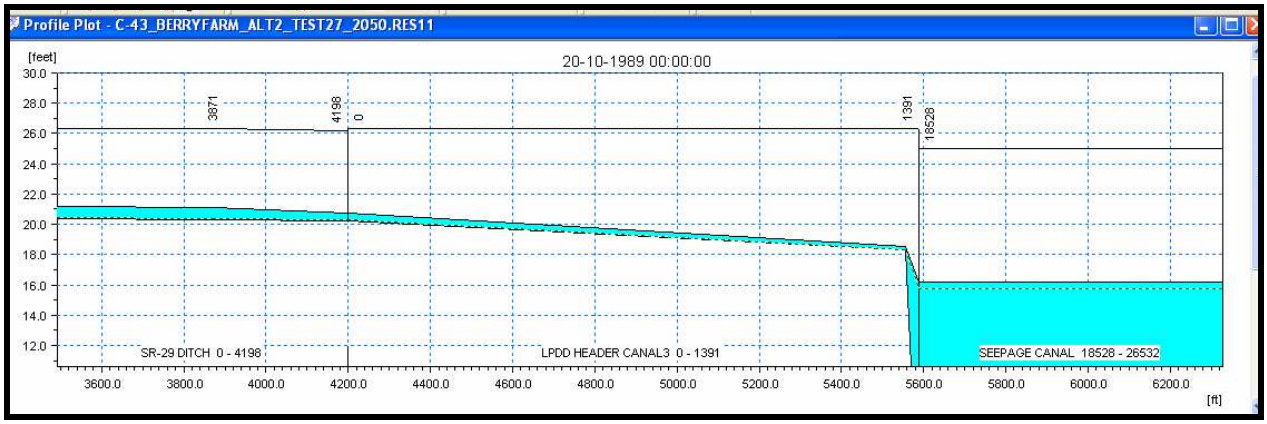


Figure 6E-02 – Water Surface Profile for the S.R. 29 Ditch, Connector Ditch, and Perimeter Canal

2) Head Loss

Manning's n values used for this project are presented in **Table 6E-01**.

Reach	Chainage	Manning's n
C-43	140734.91	0.0224
C-43	228553.15	0.0224
Townsend Canal	63494.09	0.0269
Townsend Canal	83907.48	0.0269
Roberts Canal	0.00	0.0673
Roberts Canal	48146.33	0.0673
Perimeter Canal	Entire Length	0.0336
Connector Canal to S.R. 29	Entire Length	0.0336

Table 6E-01 – Manning's n Values for Canals in the Vicinity of the Proposed Reservoir

3) Flows and Velocities

Flows in the project area have been discussed above in numerous sections and will not be discussed further. Velocities are generally low in the Perimeter Canal during the simulation period due to weirs that maintain relatively constant water surface elevations in the Perimeter Canal.

4) Energy Dissipation

The selection of the type and design details of the energy dissipators is largely depend upon the characteristics of the site, the magnitude of energy to be dissipated, and to a lesser extent upon the duration and frequency of spillway use.

The three most common types of energy dissipators are the stilling basin, the roller bucket and the flip bucket. The design discharge for a given spillway energy dissipator must be uniquely determined for each facility and should be dependent upon the damage consequences when the design discharge is exceeded. As a general rule, a spillway energy dissipater should be designed to operate at maximum efficiency and essentially damage-free with discharges at least equal to the magnitude of the standard project flood. Regardless of the type of dissipater selected, any spillway energy dissipater must operate safely at high discharges for extended periods of time without having to be shut down for emergency repairs (EM 1110-2-1603, January 1990). The design of energy dissipators is governed by USACE manual EM 1110-2-1603, January 1990 and summarized below.

The stilling basin employs the hydraulic jump for energy dissipation and is the most effective way of dissipating energy in flow over spillways. The basic parameters to be determined for design of a stilling basin are the apron length and elevation. The optimum stilling basin design would have an apron of sufficient length to confine the entire hydraulic jump. The designer must realize that conditions may occur that require unbalanced flow operation, e.g., development of fish attraction flows, operator error, or emergencies. Such conditions should be considered during evaluation of energy dissipation and stilling basin performance under conditions of non-uniform flow distribution.

The roller bucket energy dissipator consists of a circular arc bucket tangent to the spillway face terminating with an upward slope. This geometry when located at an appropriate depth below tailwater will produce hydraulic conditions consisting of a back roller having a horizontal axis above the bucket and a surge immediately downstream from the bucket. The design parameters for a roller bucket include bucket radius, maximum tailwater depth, bucket invert elevation, roller depth and surge height.

The purpose of the flip bucket is to direct high-velocity flow well away from the dam, spillway, powerhouse, and/or other appurtenances. A small amount of energy is dissipated by friction through the bucket. During the jet's trajectory to its impact location, the major portion of the jet's energy is dissipated by the interaction of the water and the air boundary resulting in considerable spray. The design parameters for a flip bucket include bucket radius, minimum bucket height, trajectory angle, and jet trajectory characteristics (horizontal distance and impact angle).

F. Water Quality

The water quality assessment utilized the site-specific model for the proposed Reservoir. The network of this model is shown below in **Figure 6F-01**, and details of the network within the reservoir are shown in **Figure 6F-02**.

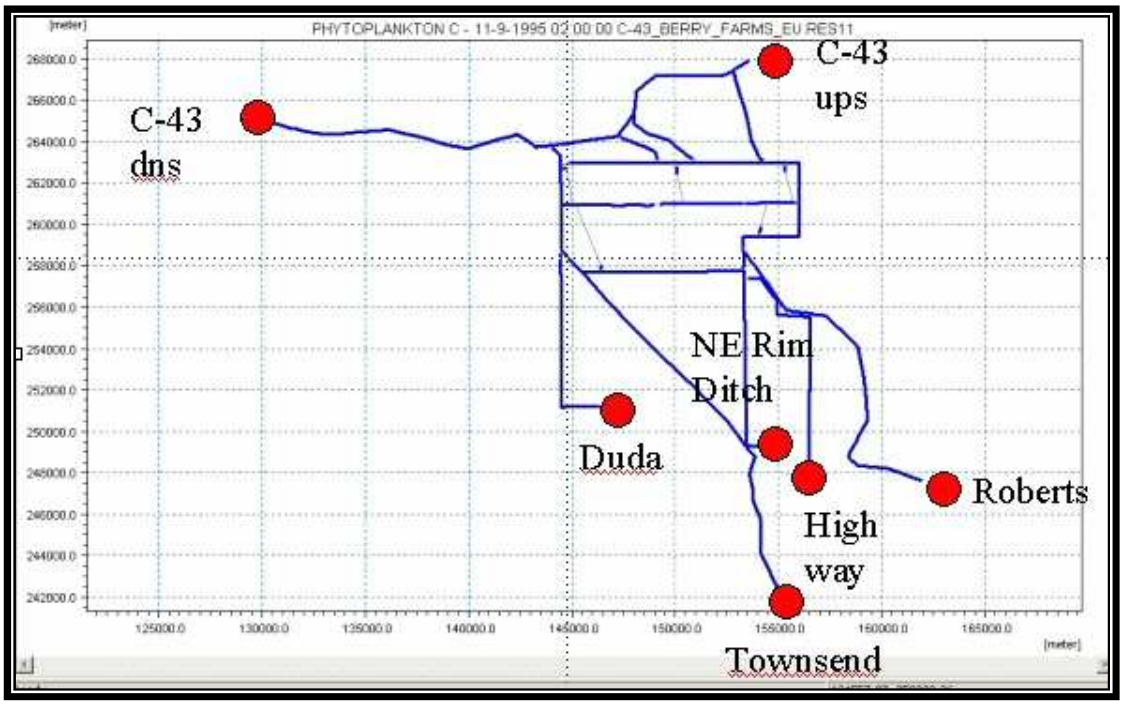


Figure 6.F-01 - Network for the Site-Specific Water Quality Model

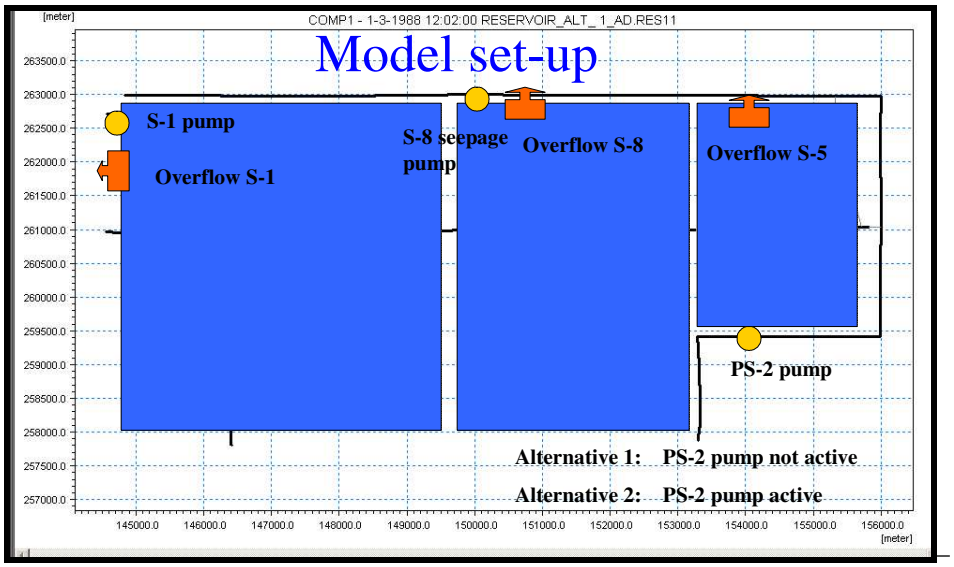


Figure 6F-02 - Reservoir Layout in the Site-Specific Water Quality Model

The red dots shown in Figure 6F-01 are boundaries in the site-specific model, and a time series of concentrations is required for each boundary point by the model. Measured concentrations were used for C-43 at S-78 and S-79. The inflow boundaries at the points labeled in Figure 6.F-01 as Duda, NE Rim Ditch Ext, Townsend, Highway Canal, and Roberts Canal were obtained from a WAMVIEW modeling study performed for FDEP by Tetra Tech, Inc. in association with Soil Water and Engineering Technologies, Inc. (SWET). Lateral inflows to C-43 from Jacks Branch, County Line Ditch, Carlos Waterway, Bedman Creek, Cypress Creek, and Hickey Creek were also obtained from the WAMVIEW result files. Minor tributaries were assumed to have an inflow concentration of 0.1 mg/l TP.

Precipitation data was received from Patricia Burke of SFWMD, and was found to be 18 mg/m2/yr, which is equal to 0.013 mg/l, assuming 55 inches of rainfall/year. Drainage flow is another inflow that enters the canals that feed into the reservoir. This flow is equivalent to shallow interception, and it was assumed that this concentration was 0.15 mg/l TP. Seepage out of the reservoir was assumed to have a TP concentration in the range of 0.091 mg/l, which was measured in seepage water from Stormwater Treatment Area 1W.

Three simulations were run for each Alternative. Simulations were completed to evaluate "no removal", "1st order removal", and "2nd order removal" processes within the reservoir.

The first order equation is defined as:

$$C_t = C_o e^{-kt}$$

The second order equation is defined as:

$$C_t = C_o e^{-(k^2)t}$$

The first order k value was assumed to be 10 m/yr, which is a typical value for south Florida aquatic systems. Values of 20 m/yr were found to be appropriate for wetlands TP removal. The second order k value assumed as 0.05 m^3/g/day.

The second order equation is used in the reservoir sub-model of the Dynamic Model for Stormwater Treatment Areas. First order decay is commonly used in simple models describing water quality dynamics. Neither equation is believed to represent the expected nutrient dynamics of the proposed reservoir, but each of these equations can be used for comparative analysis between scenarios. Furthermore, the end concentrations predicted by these two formulations can be expected to be in the range of possible reservoir behavior.

The “no removal” simulation was completed for each Alternative to verify that the mass balance is correct. **Table 6F-01** summarizes the mass balance for Alternative B, Option 1, and **Table 6F-02** summarizes the mass balance for Alternative B, Option 2.

Location	Structure ID	Alternative B Option 1 Volume Million M^3	1st Order Mass TP tons	1st Order Avg. Conc. mg/l	2nd Order Mass TP tons	2nd Order Avg. Conc. mg/l
Cell no. 1	Pump S-1	194.98	20.30	0.10	21.93	0.11
Cell no. 1	Overflow S-1	0.00	0.00	N.A.	0.00	N.A.
Cell no. 2	Pump S-8	17.85	1.95	0.11	2.16	0.12
Cell no. 2	Overflow S-8	1.66	0.04	0.02	0.08	0.05
Cell no. 3	Overflow S-5	46.37	1.46	0.03	2.15	0.05
Cell no. 3	Pump PS-2	0.00	0.00	N.A.	0.00	N.A.
Pump inflow		212.83	22.25		24.08	
Outflow		48.04	1.50		2.23	
Precipitation		24.27	0.25	0.01	0.25	0.01
Evaporation		28.75	0.00	0.00	0.00	0.00
Seepage		140.38	4.31	0.03	7.02	0.05
Volume Start		168.98	8.45	0.05	8.45	0.05
Volume End		187.91	4.44	0.02	9.49	0.05
				Reduction %		Reduction %
	In	237.10	30.95		32.78	
	Out	217.17	10.25	66.87	18.74	42.84
	Change in Storage	18.93				
	Error	-0.42				

Note: Metric tons x 1.1 = English tons, Million m3 x 810.61 = Acre-Feet

Table 6F-01 - Total Phosphorus Mass Balance Calculations for 1994 for Alternative B - Option 1

Location	Alternative B Option 2 Structure ID	Volume Million M^3	1st Order Mass TP Tons	1st Order Avg. Conc. mg/l	2nd Order Mass TP tons	2nd Order Avg. Conc. mg/l
Cell no. 1	Pump S-1	194.98	11.79	0.09	12.56	0.09
Cell no. 1	Overflow S-1	0.00	0.00	N.A.	0.00	N.A.
Cell no. 2	Pump S-8	17.85	1.46	0.08	1.67	0.09
Cell no. 2	Overflow S-8	1.66	0.00	N.A.	0.00	N.A.
Cell no. 3	Overflow S-5	46.37	4.32	0.05	5.80	0.06
Cell no. 3	Pump PS-2	0.00	11.98	0.06	13.63	0.07
Pump inflow		212.83	25.22		27.86	
Outflow		48.04	4.32		5.80	
Precipitation		24.27	0.24	0.01	0.24	0.01
Evaporation		28.75	0.00	0.00	0.00	0.00
Seepage		140.38	6.15	0.04	8.79	0.06
Volume Start		168.98	8.47	0.05	8.47	0.05
Volume End		187.91	8.18	0.04	13.13	0.07
				Reduction %		Reduction %
	In	237.10	33.93		36.57	
	Out	217.17	18.65	45.02	27.72	24.20
	Change in Storage	18.93				
	Error	-0.42				

Note: Metric tons x 1.1 = English tons, Million m3 x 810.61 = Acre-Feet

Table 6F-02 - Total Phosphorus Mass Balance Calculations for 1994 for Alternative B - Option 2

Table 6F-01 illustrates that the reservoir outflow TP load for Alternative B Option 1 ranges from 10.3 to 18.7 kg for first order and second order, respectively. The inflow load is 31 to 33 kg. The outflow TP load for Alternative B Option 2 ranges from 18.7 to 27.7 kg for first order and second order, respectively. The inflow TP load is 34 to 37 kg. Option 1 has a lower overall nutrient outflow load than Option 2. Option 1 also has a higher removal percentage than Option 2. Both Options have a significant removal effect for nutrients.

The water quality in C-43 ranges from good to poor depending on the time of the year, runoff from the contributing area, and distribution of rainfall. The reservoir outflow concentration could negatively affect C-43 water quality if high-concentration reservoir outflows reach C-43 during a period when C-43 concentrations are low. This is evaluated in **Figure 6F-03**. It can be seen that the reservoir outflows would not have a deleterious effect on C-43 water quality.

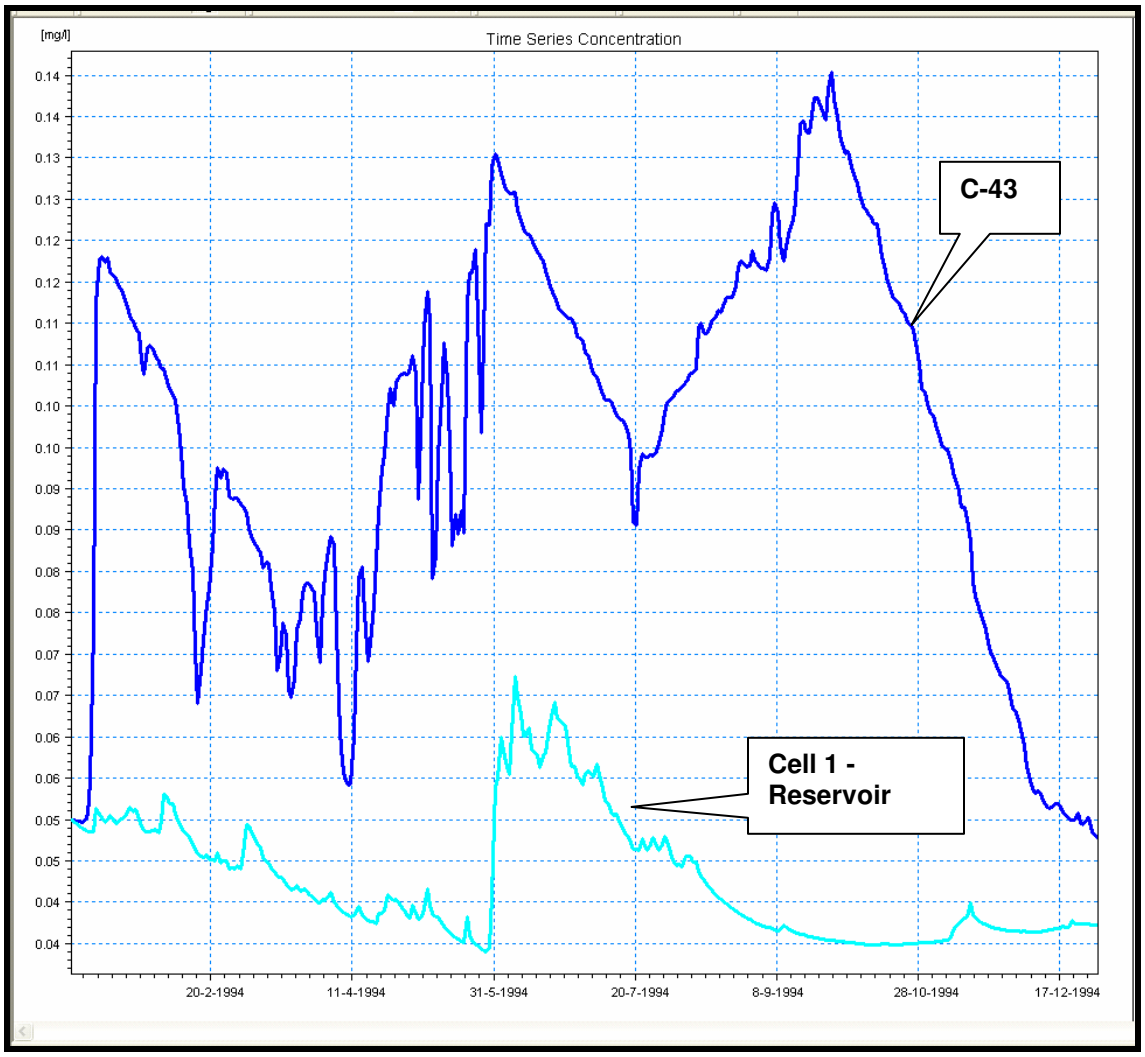


Figure 6F-03 – TP Concentration in Reservoir and C-43 for Simulated Conditions Using 1994 Rainfall

G. Erosion Control

As indicated in **Section 5 D 4) (e)** the internal dam and the upstream face of the perimeter dam should be armored using slope protection to limit the erosion potential due to wind generated wave. The most economic method for the Reservoir is the use of soil cement. Use of soil cement for erosion control is discussed in EM 1110-2-1614. The rough slope correction factor (r) was assumed to be 0.7 for the estimate of the wave run-up height from earlier estimates of run-up based on the monochromatic wave data and smooth slopes. Design details and guidance for typical practice on the Reservoir may be obtained in EM 1110-2-2302, EM 1110-2-2300, EM 1110-2-1414 and the USACE Shore Protection Manual (1984).

References:

- *DHI, Inc., 2004. (Draft) Updated Integrated Surface/Ground Water Model for the C-43 Basin, prepared for Stanley Consultants, Inc, and SFWMD.*

- *SFWMD, 2003. Existing Legal Sources for the Caloosahatchee Estuary at the Franklin Lock and Dam (S-79), Prepared by the Coast Ecosystems Division, Southern Restoration Department, May, 2003.*

- *Interagency Modeling Center, 2004. Regional Modeling for C43 PIR and Acceler8 Efforts, dated December 2, 2004.*

Section 7 – Subsurface Conditions

The reader should note that all tables and figures identified by the table or figure number in this section are located in Appendix Section 7. Embedded tables and figures are not given a numbering system.

A. Insitu Materials

1) Geophysical Setting

Refer to the discussion presented in **Section 2B**.

2) Geotechnical Exploration

A preliminary geotechnical investigation was completed (by others) to evaluate the suitability of the Berry Grove site for the intended project. The preliminary investigation included 31 core borings ranging in depth from 50 ft. to 150 ft. below existing ground surface. The core borings were located in and around the proposed site at approximately 4,000 ft. to 6,000 ft. spacings. Field permeability testing was performed in companion borings at 12 of the core boring locations. Disturbed and undisturbed samples from the core borings were laboratory analyzed for classification purposes and tested for compaction, consolidation and shear strength characteristics.

The results of the preliminary geotechnical investigation and evaluation concluded that, in general, the site soils and subsurface conditions are suitable for the proposed project. However, as recommended in the preliminary investigation report, additional investigation and analysis should be completed to improve the characterization of the site and to assist in final design of project features.

The additional geotechnical evaluations recommended for the project include the following:

- Additional subsurface exploration
- Additional laboratory testing
- Additional geotechnical engineering related to structure foundations, embankment stability and seepage control

Geotechnical engineering analysis is currently underway for structure foundation support, all as part of the 30% plans. Future work (i.e. 60% plans) will also include additional subsurface exploration, laboratory testing and geotechnical engineering analysis.

Sixty-two (62) additional core borings, 40 ft. deep, were drilled for this contract at closer spacings along the dam alignments to better characterize the subsurface conditions for embankment construction and analysis of stability, settlement and underseepage. The borings were drilled using mud rotary methods, and samples of the subsurface materials were obtained at frequent vertical intervals using Standard Penetration Test (SPT) procedures described in ASTM D 1586. Undisturbed (Shelby tube) samples of the clays were obtained using a fixed piston sampler, sealed and transported to the laboratory. SPT samples were obtained continuously in the upper 15 ft. of each borehole, and at 2.5-ft. vertical centers thereafter. The borings were sealed with cement bentonite grout upon their completion. Grouting was completed through the insertion of a one-inch diameter tremie pipe to the bottom of the boring, followed by pumping the cement bentonite through the tremie pipe, thus grouting the borehole in a bottom to top manner.

Locations of the borings are presented on **Figure 7A-2** in Section 7 of the Appendices. Generalized subsurface profiles for the borings are presented on **Figures 7A-3A through 7A-3E** in **Section 7** of the Appendices. Detailed subsurface profiles for the borings are presented on **Figures 7A-4A through 7A-4Z** in **Section 7** of the Appendices. The project surveyor established the ground surface elevation at the boring locations. The subsurface profiles are plotted to elevation using the North American Vertical Datum of 1988 (NAVD).

Disturbed and undisturbed samples collected from the core borings were classified using the Unified Soil Classification System (ASTM D 2487) and geologic nomenclature as appropriate. Representative samples from the borings were tested for index properties such as moisture content, organic content, grain size distribution and Atterberg Limits. Samples of the sands, clays and sand-clay mixtures were remolded to specific moisture-dry density conditions and tested for permeability using a triaxial compression device. These test results were used in the seepage analysis as representative of the permeability of the embankment materials.

Fifteen core boring locations along the perimeter dam were selected for field permeability testing to supplement the data collected in the preliminary investigation/evaluation. At each of the fifteen locations, a cluster of three piezometers was installed. Each piezometer consisted of a 5 to 10-ft. length of 2-inch diameter machine slotted PVC pipe (0.010-inch slot width) that was flush joint coupled to a blank riser of similar composition. Gravel pack surrounding the wellscreen consisted of clean 60/20 silica sand and is expected to have a permeability that is at least one order of magnitude higher than that of the formation materials tested. The piezometers had screened tips at nominal depths of 15, 25 and 50 ft., although these depths were modified somewhat to account for variations in the stratigraphy. The shallow, intermediate and deep piezometers were given subtitles A, B and C, respectively.

3) Stratigraphy

Subsurface materials found in the borings generally consisted of relatively clean sands, clayey sands, and sandy clays. Based upon the original borings by others and the additional borings drilled for this study, the following basic trends were observed in the subsurface profiles:

- A layer of relatively clean sands was found at the ground surface and extended to depths of 4 to 8 ft. In a few borings, the surficial clean sands extended to 20 ft.

- The surface layer of relatively clean sand is underlain by a thick deposit of clayey sand and sandy clay. While this deposit contains layers of clean sands and limestone at various depths, the clayey sand/sandy clay extends to a depth of 80 ft. below existing surface grades.

- Below the sandy clay/clayey sand layers, in some areas of the project, the borings disclosed clean sands and silty sands that continued to the terminal depths of the borings.

- Intermediate zones of clean sand were found in some of the borings within the sandy clay/clayey sand layer. These zones ranged between 5 and 25 ft. thick and the mid-points of the layers were found at depths of 20 to 30 ft. below surface grades.

- Limestone layers were found in the borings in layers 2 to 10 ft. thick. The tops of the limestone layers were found at depths of 3 to 15 ft. Borings drilled around the perimeter of Cell 3 showed considerably greater presence of limestone and correspondingly less thickness of clays and clayey sands. These anomalous ground conditions such as those depicted in Borings B-51 and B-67 will require further subsurface exploration in the 30% design phase to better define the limits of these conditions. The limestone materials found were generally weakly cemented, although some well cemented lenses and layers were found. It is not envisioned that blasting will be a requirement to facilitate seepage canal excavation. The excavated limestone materials can be utilized as random fill.

Within the upper 30 ft. of the subsurface profile, the sands are generally loose to medium dense whereas the clays are typically soft to medium stiff. The limestone layers are most often weakly cemented to moderately well cemented, except in some cases where the carbonate rock is hard and well cemented, especially within its upper zones.

The above subsurface profile description is purposefully generalized owing to the magnitude of the project site and variations in the subsurface profile conditions that occur across the site. However, geotechnical engineering evaluations and analyses for this project utilized average profile conditions in more discrete

zones. These zones include the north perimeter of Cells 1 and 2 (Zone A), the perimeter of Cell 3 (Zone B), the south perimeter of Cells 1 and 2 (Zone C), the west perimeter of Cell 1 (Zone D), and the boundary between Cells 2 and 3 (Zone E). These discrete zones and their subsurface profiles are discussed further in **Section 8**.

4) Laboratory Test Results

Representative samples of the materials recovered from the borings were tested for index properties including moisture content (ASTM D 2216), organic content (ASTM D 2974), grain size distribution (ASTM D 422), and Atterberg Limits (ASTM D 4318). Complete laboratory test results are presented in **Table 7A-1** in **Section 7** of the Appendices with each back-up as 7A - SheetsA1 through A28. Work sheets created to develop the lab results can be found in Section 7 of the Appendices. Ranges of the results of the tests are summarized in the following table:

Soil Type	Moisture Content (%)	Organic Content (%)	Atterberg Limits		Amount Passing Sieve Size (%)					
			LL	PI	#4	#10	#40	#60	#100	#200
SP, SP-SM	12.1-25.6	1.2-1.4	--	--	99-100	96-100	86-97	51-74	14-38	2-9
SC, SM	15.4-34.5	1.3-2.0	20	35	84-100	74-100	56-94	44-73	23-55	16-49
CL, CH	23.3-116.4	1.2	68-180	49-121	89-98	84-97	78-92	75-87	69-78	63-98
LS	13.6-28.6	--	--	--	61-92	51-84	39-66	27-56	17-48	12-44

Disturbed samples of the subsoils were obtained at four locations at various depths and each sample was blended to provide materials representative of those obtained from a borrow excavation at the same location. The blended materials were then tested for moisture-dry density relationships in accordance with Standard Proctor (ASTM D 698) and Modified Proctor (ASTM D 1557) procedures. These samples were also remolded to specific moisture-dry density conditions and tested in the laboratory for permeability. These test results are summarized in the following table and are also presented in **Table 7A-2** in **Section 7** of the Appendices.

Sample Location and Depth	Material Description	% Passing No. 200 Sieve	Optimum Moisture (%) - Maximum Dry Density (pcf)				Remolded Dry Density (pcf)	Permeability (feet/day)
			Standard		Modified			
			ω	γ	ω	γ		
CB-59 and CB-60, 0 to 10 feet deep	SAND with clay (SP-SC)	10.8	10.6	115.6	9.3	118.3	109.9 113.2 116.2	3.1 2.8 1.2
CB-68, 0 to 20 feet deep	Clayey, silty SAND (SC-SM)	37.2	9.4	126.4	8.1	133.2	119.5 123.7 128.2	0.19 0.05 0.03
CB-51, 0 to 20 feet deep	SAND with clay (SP-SC)	10.9	10.5	118.8	8.7	121.4	-- -- --	-- -- --
CB-47, 0 to 20 feet deep	SAND with silt (SP-SM)	9.3	10.8	114.3	9.4	115.8	-- -- --	-- -- --

Note: ω indicates optimum moisture content
γ indicates maximum dry density

The composite samples from Borings CB-59 and CB-60, and from CB-68 described previously were also remolded to specific moisture-dry density conditions and placed in a triaxial compression device for measurement of strength parameters. These tests included both consolidated drained and consolidated undrained with pore pressure measurements. The results of the tests are summarized in **Table 7A-3** in **Section 7** of the Appendices. Undisturbed Shelby tube samples of the clays were also tested for strength parameters using the consolidated undrained test method, and those test results are also summarized in **Table 7A-3** in **Section 7** of the Appendices.

Samples of the clays were extruded and tested for one dimensional consolidation. Fifteen consolidation tests were performed. These will be used to refine settlement estimates for the embankment. Additional consolidation tests are planned for the structures. Settlement analyses for the structures will need to carefully consider the interaction and potential differential settlement between subsoils, embankments and structures. Consolidation test results are provided in **Table 7A-4** in **Section 7** of the Appendices.

Other samples of the clays were extruded from the Shelby tubes and measured for in-situ moisture-dry density conditions and these results are shown on **Table 7A-5** in **Section 7** of the Appendices. Laboratory test results are also provided in **Appendix 7A-A**.

Remolded samples of the clayey sand subsoils that were tested for strength parameters showed cohesion values and angles of internal friction as follows:

Sample Location	Material Description	% Passing No. 200 Sieve	Average Remolded Moisture-Density		Compaction (%)	Cohesion (ksf)	Phi (degrees)
			Moisture (%)	Dry Density (pcf)			
CB-59 and CB-60, 0 to 10 feet deep	Brown SAND with clay, trace gravel (SP-SC)	10.8	10.4 9.7	111.1 115.9	93.9 98.0	0.34 0.26	33.8 39.3
CB-68, 0 to 20 feet deep	Light gray to brown clayey, silty SAND, trace gravel (SC-SM)	37.2	10.5 9.8	119.9 125.9	90.0 94.5	0.72 0.96	23.3 34.9

Note: % compaction based upon Modified Proctor maximum dry density.

Undisturbed samples of the clays had cohesion values that ranged between 0.40 and 1.59 ksf and angles of internal friction of 15.8 to 36.5 degrees.

The stated properties of the underlying clay later were selected based on limited laboratory tests performed on clay samples. Additional laboratory tests are planned, and after obtaining those results, we will have better data to model the clay layer.

B. Groundwater

1) Groundwater

Groundwater was measured in the borings at elevations that ranged between +28.5 and +4.3 ft. with respect to the North American Vertical Datum of 1988 (NAVD). Groundwater elevation data was supplemented with periodic measurements of the groundwater surface at the piezometer locations. This was accomplished using a graduated measuring tape and an electronic water level indicator. The results of the groundwater measurements are summarized in **Table 7B-1** and on the graphical sheets that follow in **Section 7** of the Appendices. Groundwater levels generally follow variations in the ground surface topography. Based upon ground elevations for the boring locations, the site ground surface elevation ranges between +15 and +27 ft. NAVD, with the lowest elevation being near the site's northwest corner (i.e. nearest to the C-43 Canal). Groundwater gradients generally appear to slope from south to north and east to west across the site, and are influenced by local drainage practice and controls.

Groundwater elevation data is also presented on **Figures 7B-1 and 7B-2** in **Section 7** of the Appendices. The former plan shows the average groundwater elevation recorded in the "A" and "B" series piezometers between their installation and October 2004. The latter plan shows similar data for the "C" series piezometers, which have screened intervals at relatively deeper intervals

(when compared with the A and B series), and over a similar time period. Review of the referenced drawings indicates that the average groundwater elevations in the A and B series piezometers that are located in the western half of the site (and along the south boundary) are approximately 3 to 5 ft. higher than those at companion locations in the C series piezometers. Such a comparison for piezometers installed in the eastern portion of the site, particularly around the perimeter of Cell 3, reveals that the groundwater elevations in the shallow and deep piezometers are very similar. This is also readily observed in the graphical plots that follow **Table 7B-1** in **Section 7** of the Appendices.

Details regarding the piezometer installations are presented on **Figures 7B-3A through 7B-3O** in **Section 7** of the Appendices. These are plotted to elevation (NAVD) adjacent to the subsurface profiles for correlation of the piezometer's screened interval with the surrounding soil/rock formation materials. The permeability testing included the conduct of constant head tests by measuring the amount of water (inflow) required to maintain a constant head in the piezometer. At the conclusion of the constant head test, the falling head was recorded using a data logger.

Several of the deeper (C series) piezometers show a rise in the groundwater surface after their installation. This is likely due to the aquifer's response to the wet hydroperiod, which normally begins in the summer months and prevails through October. Groundwater levels in the more shallow piezometers (A and B series) are likely influenced by irrigation practice in the groves.

2) Field Permeability Tests

Soils at the piezometer locations were tested for field permeability using the constant head test, where each well was filled with water to the top of casing and maintained there using a measured volume of water over a finite period of time. Using the "packer" analogy, the data was then input to an equation developed by the U. S. Bureau of Reclamation, and presented by Harry Cedergren in his text "Seepage, Drainage and Flow Nets", published 1977, which is as follows:

$$K = \frac{q}{2\pi Lh} \log_e \frac{L}{R} \qquad \text{For L greater than or = 10r}$$

K = Permeability, feet/sec; q = flow, cfs

L = Screen length, feet; h = head, feet

r = Borehole radius, feet

The packer analogy equation has shown to provide good results in local practice. Permeability values predicted using constant head methods and the packer equation were very similar to those estimated using falling head data gathered with a data logger.

Following completion of the constant head permeability testing, the permeability of the screened interval for each piezometer was measured using the falling head test. The water levels in each well casing were continuously recorded using a downhole Hermit 3000 Data Logger (by In-Situ, Inc.). The field test data was then downloaded to a computer program (Aqtesolve) used to compute the coefficient of horizontal permeability.

Results of the field permeability tests are provided in **Table 7B-2** in **Section 7** of the Appendices. The field permeability test results indicate that the sands have an average permeability of 5 ft. per day, while the limestone averages approximately 10 ft. per day. The clays have an average permeability of somewhat less than 2 ft. per day. The magnitude of variability observed and ranges of hydraulic conductivity will be addressed and considered in the seepage analysis during the 30% design phase.

Some of the in-situ permeability tests for the clays were surprisingly high for CL and CH. Sand seams may exist in the clay layer. Additional discussion and clarification of the ranges of permeabilities measured for the various subsurface materials will be provided in the 30% design phase.

C. Excavations

Excavations for borrow and for canals can be made using large earth excavating equipment such as draglines and/or track-mounted hydraulic backhoes. Blasting to fracture and loosen the limestone layers may be necessary in a few isolated locations but does not appear to be a predominant requirement. Excavations will likely produce a mixture of sands, clays and some gravel in the form of broken limestone. The excavations will penetrate to depths below the water table, and therefore the materials from the cut will be wet and somewhat difficult to handle, place and compact without moisture conditioning to near the moisture content optimum for compaction. Moisture control of embankment fills will be critical to the successful construction and performance of perimeter and interior dams.

D. Borrow

Embankment construction should take the form of material placement in relatively thin lifts (i.e. 8-inch thick loose measure), with each lift compacted to relatively high dry density. The moisture content of the fills should be controlled within a range that extends below and above and that reduces the level of moisture control during fill placement, but still results in the engineering parameters used for design. It may be effective to establish a test fill wherein the materials are placed and compacted, and in-situ moisture-density test results are compared with laboratory Proctor. Fill samples will be tested for grain size distribution and remolded to field moisture density conditions, then tested for permeability. Such a test fill will likely enable the specified moisture range to be broadened. Some of the clays exhibited much higher permeabilties than expected.

Compaction equipment should consist of heavy, self-propelled vehicles, equipped with sheepsfoot rollers to promote kneading action of the sandy clays/clayey sands. Placement and compaction of the shell materials can be done in lift thicknesses greater that that of the core material. Suitable compaction can be achieved in the anticipated silty sand materials using a loose lift thickness ranging from 12 to 24-inches and a standard 10 ton vibratory roller. Placement and compactions requirements will be refined during the test cell program based on actual performance of the materials.

A Test Cell Project is planned for the C-43 West Storage Reservoir Project. It is anticipated that this Test Cell Project will be bid in August 2005 and testing will be completed in early 2006. Seepage, water quality, and on-site materials constructability are the primary reasons for the Test Cell Project. There will be two test cells. One of the test cells will be constructed with a clay core. The other test cell will be constructed with a slurry wall, eliminating the need for the clay core. The Test Cell Project will allow the design team to evaluate the performance of each of the two test cell options as well as the suitability of the on-site materials for the intended construction use. Processing of the clay core materials and drying times will certainly be evaluated and documented. If a clay core is to be used in final design of the C-43 West Storage Reservoir, test pits at the borrow sites will be considered to evaluate the moisture content and drying times of stockpiled materials and its potential impact on construction schedule.

It is expected that embankment construction materials will be generated from on-site excavations. These excavations will likely take the form of the seepage collection canal together with several strategically located borrow sites within the interior portion of the Reservoir. Using a required embankment volume of 100 cubic yards per foot of perimeter dam, the length of embankment (in miles) was estimated for various borrow pit footprint areas. The estimates are shown graphically on the left side of **Figure 7D-1** in **Section 7** of the Appendices.

The locations of eight borrow sites are presented on **Figure 7D-2** in **Section 7** of the Appendices. These were located so that the average one way haul distance between the center of the excavation and the embankment placement area was calculated to be less than 1 mile. The borrow features were positioned not less than 200 ft. from the interior toe of the embankment in order to lessen the potential for seepage. This was determined by calculating the magnitude of seepage at various and increasing distances between the embankment toe and the borrow site until no additional reduction in seepage was estimated.

Subsurface conditions that exist at the locations of these borrow features were used to evaluate the types of materials that would be generated from excavations of various cut depths. The cumulative fines content (i.e. amount passing the U.S. Standard No. 200 Sieve) was estimated as a function of depth of excavation. The estimated cumulative fines contents are plotted graphically on the right side of **Figure 7D-1** in **Section 7** of the Appendices. Further evaluation of borrow sources and materials will be completed in the 30% design phase to predict the availability of materials for construction.

Excavations will be required to produce materials for earth dam construction. Such materials will take the form of select borrow for construction of "impervious" zones, as well as for the upstream and downstream shells. The low permeability materials tested in the laboratory had 37 percent passing the U.S. Standard No. 200 Sieve. Review of **Figure 7D-1** reveals that cuts ranging between approximately 15 and 30 ft. deep will produce materials that have an estimated cumulative average fines content of 37 percent. However, selective excavation and stockpiling of subsurface materials will produce materials that are higher and/or lower in fines contents when compared with the overall cumulative blend from a borrow excavation. Additional clay materials can be obtained by excavation of additional borrow pits on the interior side of the perimeter dam.

Dewatering will be addressed in the 30% design phase. It is anticipated that some valuable dewatering information and experience will be gained from the Test Cell Project.

E. Foundations

Structure foundations have not been evaluated at this stage of the project. However, based upon generalized subsurface conditions disclosed in borings drilled for the perimeter dam, we expect that structures may be supported upon steel reinforced concrete mat foundations. More heavily loaded structures such as pumping stations may require a deep foundation system (i.e. piling) in order to reduce the risk of more than normal long term settlement. This evaluation will be completed in a subsequent phase of the work using structure specific borings and an appropriate level of geotechnical engineering.

F. Pipelines/Trenches

(Not Applicable)

G. Design Parameters

The Reservoir perimeter was subdivided in various "zones" as described in more detail in **Section 8A**. Based upon the results of the field and laboratory tests completed for this study, we have developed geotechnical parameters for design of the project components for the various zones. These are summarized in the following table.

Zone	Subsurface Component	Depth (feet)	Total Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion Intercept (psf)	Angle of Internal Friction (degrees)	Coefficient of Permeability (feet/day)
A1	Sand	0 – 5	105	110	0	30	20
	Silty sand	5 – 10	100	105	100	28	20
	Limestone	10 – 15	120	125	0	34	15
	Clay (CL)	15 – 20	90	95	400	0	5
	Clay (CH)	20 – 40	90	95	1000	0	5
A2	Sand	0 – 5	105	110	0	30	20
	Silty sand	5 – 10	100	105	100	28	20
	Limestone	10 – 15	120	125	0	34	15
	Clay (CL)	15 – 20	90	95	400	0	5
	Clay (CH)	20 – 40	90	95	1000	0	5
B	Sand	0 - 12	105	110	0	30	20
	Limestone	12 - 20	120	125	0	34	15
	Silty sand	20 - 40	100	105	100	28	20
C	Sand	0 – 5	105	110	0	30	20
	Limestone	5 – 18	120	125	0	34	15
	Clay (CL)	18 – 23	90	95	400	0	5
	Clay (CH)	23 – 40	90	95	1000	0	5
D	Sand	0 – 5	105	110	0	30	20
	Silty sand	5 – 15	100	105	100	28	10
	Clay (CL)	15 – 20	90	95	400	0	5
	Clay (CH)	20 – 40	90	95	1000	0	5
E	Sand	0 – 7	105	110	0	30	20
	Limestone	7 – 15	120	125	0	34	15
	Clay (CL)	15 – 20	90	95	400	0	5
	Clay (CH)	20 – 40	90	95	1000	0	5

Dam Component	Total Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion Intercept (psf)	Angle of Internal Friction (degrees)	Coefficient of Permeability (feet/day)
Impervious Core	125	130	800	30	0.1
Upstream Shell	115	120	200	32	1.0
Downstream Shell	110	115	0	32	10

It should be noted that the strength parameters (cohesion and angle of internal friction) selected for preliminary design of the embankments are conservative, however, additional

laboratory testing is planned and the results may modify the properties of the strength parameters.

The silty sand layer was input to several of the zones as a relatively thin stratum located below the upper clean sands. It was given strength and permeability parameters not greatly different from clean sands. Its presence is not expected to have a marked bearing on the results of the analyses.

Some of the in-situ permeability tests for the clays were surprising were surprisingly high. We expect that sand seams may exist in the clay layer. For this reason, the clay layer was assigned relatively high permeability.

A permeability of 15-ft/day for the limestone may be too low and therefore not conservative based on some of the in-situ testing. The limestone formation was somewhat silty and clayey, and therefore, was assigned a permeability lower than one might expect. Revised analyses will look at a range of permeability values for the limestone and check the sensitivity of the results to this input variable.

The six zones were sufficient for the reservoir optimization study to determine the reservoir footprint (i.e., two cell versus three cell options) based on geometry as well as geology, however, more discreet zones of similar geology will be included in the evaluation of seepage during the 30% preliminary design phase.

Section 8 – Dam & Reservoir Design

The reader should note that all tables and figures identified by the table or figure number in this section are located in Appendix Section 8. Embedded tables and figures are not given a numbering system.

A. Design Criteria

Design of perimeter and interior dams followed the guidelines presented in the U.S. Army Corps of Engineers (USACE) Engineering Manual (EM) 1110-2-2300, dated July 30, 2004, and titled "General Design and Construction Considerations for Earth and Rock-Fill Dams." Embankment slope stability evaluations followed procedures described in EM 1110-2-1902, "Slope Stability" dated October 31, 2003. The required minimum factors of safety presented in EM 1110-2-1902 are summarized as follows:

Analysis Condition	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)	1.3	Upstream and Downstream
Long-Term (steady seepage with maximum storage pool)	1.5	Downstream
Maximum Surcharge Pool	1.4	Downstream
Rapid Drawdown	1.1-1.3*	Upstream

*Note: Factor of Safety = 1.1 applies to drawdown from maximum surcharge pool.
Factor of Safety = 1.3 applies to drawdown from maximum storage pool.

For this analysis, rapid drawdown was assumed to be a 3-ft. drop in pool level, measured from the maximum storage pool elevation. This assumption was based on discussion with SFWMD staff who indicated it would be advantageous to lower the surface water level in the Reservoir rapidly in advance of a major storm event. The magnitude of the drawdown was designated to be 36-inches, which was considered to be the amount of rainfall resulting from such an event falling directly upon the reservoir footprint. Subsequent analysis will utilize the USACE criteria for design.

The geotechnical analysis for this study began with a set of assumptions regarding the external geometry of the perimeter earth dam. The embankments were determined to have sideslopes not steeper than 3:1 (horizontal:vertical) for maintenance mowing purposes. The crest width was established using a rule of thumb for earth dams of 10 ft. plus one-fifth of the embankment height. Using an average height of embankment of 25 ft., the resulting crest width was determined to be 15 ft. Ultimately, the crest width is dependant upon seepage and stability considerations.

A Perimeter Canal will be situated downstream of the embankment. The canal will be utilized to collect seepage, convey surface water and as a borrow source. The distance between the downstream toe of the embankment and the top of bank of the seepage collection canal (referred to as the bench) was determined (by seepage calculations) to be a minimum of 30 ft. wide. The analysis showed that seepage considerations controlled the design. However, subsequent analyses will be made that consider the implications of a canal slope failure on the embankment stability. This may control the bench width for the project.

In some areas, the bench width is required to be greater than 30 ft. to accommodate future Aquifer Storage and Recovery (ASR) systems that may be located within the bench. The western perimeter of Cell 1 may also include a future roadway along the bench. A preliminary embankment design for the perimeter dam was prepared and is presented on **Figure 8A-1** in **Section 8** of the Appendices.

Interior dams will separate Cells 1, 2 and 3. The typical section for these dams will be somewhat different than that for the perimeter dam. Since the consequence of overtopping the interior dams is not as significant as for the perimeter dams, the top of dam was established at five feet above the normal pool water surface elevation. However, the slopes of the interior dam are to be covered with soil-cement on both sides to protect them against wind generated wave attack and erosion. The typical section for the interior dam is presented on **Figure 8A-2** in **Section 8** of the Appendices.

An evaluation of the geotechnical issues (i.e. seepage, slope stability, embankment settlement) in the selection of the Reservoir footprint was completed. Specifically, the analysis was completed to provide a basis for selection of Alternate A (2-cell Reservoir, pool elevation of +39.4 feet NAVD) or Alternate B (3-cell Reservoir, pool elevation 37.4 feet NAVD) from a geotechnical perspective. This analysis was referred to as the Reservoir optimization study. For this analysis, we selected six (6) zones (sections) along various cell boundaries, and these are described as follows:

- Zone A - Northern perimeter of Cells 1 and 2. This zone was further subdivided into Zones A1 and A2 based upon differing tailwater conditions in the perimeter seepage canal.
- Zone B - Northern, eastern and southern perimeter of Cell 3.
- Zone C - Southern perimeter of Cells 1 and 2.
- Zone D - Western perimeter of Cell 1.
- Zone E - Eastern perimeter of Cell 2.

A plan showing the limits of each geographic zone is presented in **Figure 8A-3** in **Section 8** of the Appendices.

An average normal pool elevation of +39.4 ft. NAVD was assumed for the Alternative A analysis, whereas +37.4 ft. NAVD was assumed for Alternative B. Tailwater elevations in the seepage collection canals were based upon interaction with the H&H modeling team. (After this analysis the average pool elevations were revised slightly, but this would have no impact on the results.)

B. Slope Stability

Slope stability analysis was performed using the computer program STABL6H, which was developed at the Purdue University Engineering Experiment Section. The slope stability analysis evaluates stability of the dam based on subsurface profile, pool elevation, dam material properties, dam section, etc. The dam slope stability analysis was completed for three (3) alternative conditions, i.e. steady state seepage, end-of-construction and rapid drawdown.

The slope stability analysis was performed based on subsurface profiles found at each of the six (6) zones selected for the Reservoir optimization study. Initially, boundary conditions including crest width, embankment side slopes, bench width, location of core material within the embankment, seepage canal bottom elevation, and water elevations in the Reservoir and seepage canal were varied in order to evaluate the impact of each condition on the dam slope stability. In addition, boundary conditions required to provide adequate factor of safety for seepage were also taken into consideration for the slope stability analysis. For performing the slope stability analysis, we have restricted the initiation and termination points of failure surfaces to various sections of the seepage canal and dam section in order to simulate various failure conditions. For each case, we have performed three (3) runs with different initiation and termination points and they are as follows:

Run No.	Initiation Points Between	Termination Points Between
1	Left Edge of Canal Bottom and Right Top of Canal Bank	Middle of Downstream Slope of Dam and Upstream Edge of Crest of Dam
2	Left Edge of Canal Bottom and Right Top of Canal Bank	Middle of Downstream Slope of Dam and Upstream Toe of Dam
3	Right Edge of Canal Water and Downstream Toe of Dam	Middle of Downstream Slope of Dam and Upstream Toe of Dam

Note: The analyses were set with the dam located on the right side of the cross-section and the seepage canal on the left side of the cross-section.

Based on interaction between the seepage analysis and the dam slope stability analysis using various boundary conditions, a typical embankment section was developed. The stability analysis was also completed for both Alternative A (2-cell option) and

Alternative B (3-cell option) for three (3) analysis conditions, i.e. steady state seepage, end-of-construction and rapid drawdown.

Based on results obtained from the laboratory tests performed on representative borrow materials and soil samples, we have estimated the soil strength properties for embankment fill and subsurface layers for this analysis. These parameters are summarized in **Section 7G**.

The slope stability analysis results for steady state seepage along with soil layer types, thickness, various boundary conditions and analysis run options are detailed in **Table 8B-1** in **Section 8** of the Appendices. The detailed failure surfaces along with computer output for steady state seepage are presented in **Appendix 8B-A** in **Section 8** of the Appendices. **Tables 8B-2 and 8B-3** in **Section 8** of the Appendices present the analysis results for end-of-construction and rapid drawdown, respectively. Similarly, for end-of-construction and rapid drawdown, the detailed failure surfaces along with computer outputs are presented in **Appendix 8B-B and 8B-C** in **Section 8** of the Appendices, respectively.

Review of Engineering Manual EM 1110-2-1902 (October 31, 1993) - Slope Stability, Published by the U.S. Army Corps of Engineer (USACE) indicates that the required minimum factors of safety for long term (steady state seepage at maximum storage pool elevation), end-of-construction and rapid drawdown are 1.5, 1.3 and 1.3, respectively. Based on review of the analysis results presented in **Tables 8B-1, 2 and 3** in **Section 8** of the Appendices, the calculated factors of safety are achieved for all zones and for both Alternatives A (2-cell option) and B (3-cell option) for the end-of-construction and rapid drawdown analysis conditions. For long term steady state analysis condition, the calculated factors of safety are achieved for all zones with exception of Zone A and Zone E, wherein the factors of safety were between 1.34 and 1.44. While we recognize that these safety factors are below the required minimum, further refinement to the embankment design is required and the 30% preliminary design phase will address the factors of safety that are below 1.5. A post canal failure stability analysis of the embankment will be performed in order to evaluate if a canal failure impacts the factor of safety for the dam. We believe that the soil parameters utilized for the subsurface clay layer are conservative. Additional triaxial testing of the clay soils found in the borings will be performed in the 30% preliminary design phase. Based on additional laboratory test results, we will modify the properties of the clay layer as appropriate. We will also further evaluate the strength characteristics of the clay layer and the potential for strength gain resulting from increased overburden pressure which will cause the underlying soils to consolidate and gain strength. This strength gain may result in higher factors of safety.

The analysis of rapid drawdown to model operating conditions in the reservoir will be revised in the 30% preliminary design phase to include the evaluation of the upstream slope of dam following USACE Guidelines. Slip surfaces from RDD analyses are on the interior of the perimeter dam, not on the exterior as was incorrectly presented in the analysis provided. Rapid drawdown analysis will be required on both slopes of interior embankments/dams and will be performed in the 30% preliminary design phase.

We have also performed slope stability analysis on the typical interior dam section. The analysis results along with the soil layer types and thicknesses utilized are summarized in **Table 8B-4** in **Section 8** of the Appendices. The detailed failure surfaces along with computer output for steady state seepage are presented in **Appendix 8B-D** in **Section 8** of the Appendices. Based on review of the analysis results, the calculated factors of safety achieved satisfy the required minimum factor of safety for steady state seepage at maximum storage pool, i.e. 1.5.

The stability analysis for final design will utilize a water surface elevation of +40.7 feet NAVD (height of the maximum pool level after a major storm event).

C. Foundations/Bearing Capacity

See **Section 7E**.

D. Seepage Control

The zones and their stratigraphic profiles, permeabilities and boundary conditions are presented in the following table.

Zone	Depth (feet)	Subsurface Component	Permeability (feet/day)	Head Conditions (feet-NAVD)	
				Pool EL.	Seepage Canal EL.
A1	0 – 10	SAND	20	+37.4	+10
	10 – 15	LIMESTONE	15		
	15 – 60	CLAY	5		
A2	0 – 10	SAND	20	+37.4	+16
	10 – 15	LIMESTONE	15		
	15 – 60	CLAY	5		
B	0 – 12	SAND	20	+37.4	+16
	12 – 20	LIMESTONE	15		
	20 – 60	SAND	20		
C	0 – 5	SAND	20	+37.4	+23
	5 – 18	LIMESTONE	15		
	18 – 60	CLAY	5		
D	0 – 5	SAND	20	+37.4	+2
	5 – 15	SILTY SAND	10		
	15 – 60	CLAY	5		
E	0 – 7	SAND	20	+37.4	+16
	7 – 15	LIMESTONE	15		
	15 – 60	CLAY	5		

Note: Pool elevation shown in the table represents the 3-cell option with constant pool level. Add 2 feet to that elevation for the 2-cell option.

The boundary conditions (i.e. crest width and height, embankment sideslopes, pool elevation and seepage canal elevation) were used as input to an estimate of seepage

utilizing the computer program SEEP2D (GMS Model 4.0). SEEP2D is a two dimensional finite element, steady state flow model that was developed by Fred Tracy of the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi. A preliminary embankment design was prepared and presented on **Figure 8A-1** in **Section 8** of the Appendices. Elements of the preliminary design include a clay – clayey sand "impervious" core, a relatively low permeability upstream shell, a random fill downstream shell, a downstream filter blanket to collect seepage, and an upstream slope protection system consisting of soil cement. The embankment materials were given a range of permeabilities based upon the results of the laboratory testing. Various dam components of the embankment and their permeabilities are provided in the table that follows. The Reservoir optimization study also included an evaluation of the potential benefit of installing a slurry wall to various depths and in various zones.

Location	Dam Component	Coefficient of Permeability (feet/day)
Perimeter Dam	Upstream Shell	1.0
	"Impervious" Core	0.1
	Downstream "Random" Shell	10
	Filter Blanket	100
	Soil Cement	0.1
Interior Dam	Homogeneous Fill	10.0
	Soil Cement	None Assumed

The seepage analysis was completed for both the Alternative A (2-cell) and Alternative B (3-cell) Reservoir options. The Reservoir was subdivided into discrete zones, each with specific contributing lengths and subsurface profile conditions. Other variables in the seepage analysis include the inclusion of a slurry wall beneath the embankment, the downstream bench width, and the bottom elevation of the seepage collection canal. The analysis continued until the downstream seepage gradient produced a factor of safety of at least 3.0 against piping. The corresponding unit rate of seepage was multiplied by the zone's contributing length to estimate the seepage loss for the specific zone. Seepage losses for all the zones around the perimeter of the Reservoir were totaled for the two alternates. The results of the Reservoir optimization study from a seepage viewpoint are summarized in **Table 8D-1** in **Section 8** of the Appendices.

The results of the Reservoir optimization analyses indicate that Alternate A (2-cell option) is more efficient from an overall seepage loss standpoint. The 2-cell option has an average seepage rate of 4.68 cubic feet per second (cfs) per mile of perimeter compared with the 3-cell option which has an average seepage rate of 6.30 cfs per mile. These seepage rates are equivalent to approximately 147 acre-feet per day (Ac-Ft/Day) and 243 Ac-Ft/Day for the 2-Cell and 3-Cell options, respectively, at normal pool elevations.

The effects of change in the pool elevation with seepage losses was also evaluated. The 2-cell option for Zone A1 and the 3-cell option for Zone B were selected for this analysis. The water surface elevation was varied between the existing ground surface and the maximum pool elevation in approximately 2.5-ft. increments. Seepage losses varied between approximately 1.5 and 5 cfs/mile for Zone A1 and between approximately 5 and 13 cfs/mile for Zone B. The seepage rates were normalized on a per foot of head basis. The normalized seepage rates are slightly less than 0.6 and 0.2 cfs/mile/foot of head for Zones B and A1, respectively. This information is presented graphically on **Figure 8D-1** in **Section 8** of the Appendices.

The soil cement was modeled as tight, but will be revised as leaky as soil cement does crack and can have poor bond at lift joints. Subsequent modeling will include conditions assuming a crack through the soil cement.

The filter drain system does not have a marked impact on the phreatic surface unless we input a drain with extremely high permeability.

It should be noted that the Reservoir optimization study revealed that the piezometric surface in the embankments was near or coincident with the ground surface at the downstream toe of the embankment. For this reason, we recommend that the downstream bench be raised 4 ft. above the natural ground surface.

Flow nets that resulted from SEEP2D computer runs are presented in **Appendix 8D-A** in **Section 8** of the Appendices.

SEEP2 analyses results indicate Reservoir seepage will raise the surrounding groundwater levels on the property adjacent to the project. The potential for detrimental offsite impacts will be evaluated in the 30% preliminary design phase.

A sensitivity analysis of permeability values (i.e. conductivity) will be completed for the 30% preliminary design phase. The gradients were determined using the SEEP2D flownets. The factor of safety against piping in clean sands is normally required to be 3.0 or greater. For clean sands, the critical gradient is 1.0, and the factor of safety is simply the critical gradient divided by the actual gradient.

The seepage analysis for final design will utilize a water surface elevation of +40.7 feet NAVD (height of the maximum pool level after a major storm event).

E. Settlement

Settlement of the typical embankment section (explained later) was estimated based on existing subsurface soil profiles at each of the six (6) zones utilized for this study. Settlements were computed per design methodology detailed in FHWA Publication No. NH1-00-0045 “Soil and Foundation Workshop”.

The subsurface profiles were divided into layers with thickness varying between 3 and 6 ft. The soil index properties utilized are identical to those utilized for the global stability analysis. The stress increases at the center of each layer of the soil profile due to weight of the proposed dam is computed and hence the settlement of each layer is computed. The total settlement is the summation of the settlements of each of the individual soil layers. The estimated settlements along the centerline of the proposed dam at each of the six zones are presented below.

Zone	Estimated Settlement (inches)
A	10.0
B	4.0
C	10.0
D	9.5
E	9.5

Additional consolidation testing is planned and these will enable refinements to settlement estimates. Self-weight settlement of the embankment was not considered in the total settlements but will be evaluated in the 30% design phase.

F. Insitu Materials

See **Section 7A** – Subsurface Conditions, Insitu Materials

G. Borrow

See **Section 7D** – Subsurface Conditions, Borrow

H. Dam Design

1) Perimeter Dam

The results of the preliminary analysis indicate that a zoned embankment, after further design refinements, will provide adequate safety factors from the standpoint of seepage and stability. The dam should have a crest width of 15 ft. and upstream and downstream sideslopes of 3:1 (horizontal:vertical). The upstream face of the dam should be protected using soil cement. Seepage through and below the dam should be collected in a downstream drainage blanket. The seepage control drain is required to facilitate drainage at the toe of the perimeter dam. This is considered to be critical to the long term stability of the embankment. Details regarding slope protection and the drainage blanket are described in subsequent sections. The typical section for the perimeter dam is presented on **Figure 8A-1** in **Section 8** of the Appendices.

The zoned embankment section was selected in order to make use of materials that are available from on-site excavations. An alternative embankment section will be evaluated in the 30% design phase that includes a slurry wall in lieu of the compacted clay core.

The addition of a slurry wall does not appear to have an appreciable effect in reducing seepage. For this reason, its use was not recommended in the Pre-Final BODR. The Test Cell Project is currently underway to evaluate the performance of seepage, water quality and constructability of on-site materials. One of the test cells will be constructed with a clay core. The other test cell will be constructed with a slurry wall, eliminating the need for the clay core. The Test Cell Project will allow the design team to evaluate the performance of an embankment design that includes a slurry wall in lieu of a compacted clay core.

Zones within the embankment consist of a central "impervious" core made up of compacted clays and clayey sands. The upstream shell of the dam should consist of clayey/silty sands. Downstream portions of the dam may consist of random fill. The random fill may consist of clean sands or silty/clayey sands placed in a somewhat random (but compacted) manner.

The stability analysis was based upon laboratory test data that was completed at the time of writing the BODR. Additional analyses are forthcoming and will be used to refine the strength parameters used in the analysis. Stength gain in the clays through consolidation settlement is based upon the equation: tau = c + sigma*tan(phi), where tau is the shear strength, c is the soil's cohesion, sigma is the overburden pressure increases due to the weight of the embankment fill, the sheer strength increases as well. It would be conservative to discount this theory in future analyses and submittals. The reservoir optimization study looked at the potential benefit of constructing a slurry wall to a depth of 40 feet. The reduction in seepage was approximately 10 percent, indicating that the slurry wall would necessarily have to penetrate to great depth in order to significantly reduce seepage and make the cell option comparable in terms of seepage. Subsequent analyses will include an alternative embankment design that includes a slurry wall in lieu or a cemented clay core.

2) Drainage Blanket

The downstream section of the perimeter dam must be designed and constructed to perform the following functions:

- Relieve uplift pressure from seepage.
- Provide sufficient weight (buttress) to offset uplift pressures.
- Allow for collection and discharge of seepage water.

- Prevent piping of soil fines from embankment and/or the foundation.

These functions are accomplished using a downstream drainage blanket. Silica sand is the preferred material as limestone may dissolve over time in repeated wet-dry cycles however the on-site sand is generally quartz. We considered the use of a geonet in lieu of a sand or gravel drainage blanket for the downstream filter. The material was not considered to be industry standard for earth dam construction. However, placement of a geosynthetic filter fabric around the sand blanket is recommended to provide separation of drainage media from surrounding soils. The drainage filter requires additional analysis during future phases of the project.

For this analysis, gravel (i.e. FDOT No. 57 Stone), geosynthetic composites and sand were evaluated for potential usage for the construction of a downstream drainage blanket. A local (on-site) source of gravel is not readily available since the site does not have an abundant deposit of hard rock that could be crushed to meet the gradational requirements of FDOT No. 57 Stone. The use of geosynthetic composite drainage materials is not currently considered to be standard practice. For this reason, we recommend the drainage blanket consist of relatively clean sands meeting the requirements of ASTM C-33 Concrete Sand. Comparison of the grain size distribution of the relatively clean sands at the Reservoir Project site with the requirements for ASTM C-33 Concrete Sand indicates that the on-site materials will necessitate some screening and processing (i.e. washing) in order to meet the requirements of C-33 Sand. The thickness of the drainage blanket should be 2 ft based upon the volumetric flow rates of seepage, the permeability of the filter media (concrete sand), and the anticipated head conditions.

The drainage blanket design will be refined upon further analysis and may include finger drains and/or perforated header collection pipes with periodic discharge conduits routed to the seepage canal.

3) Interior Dam

Overtopping of interior dams was considered to be of lesser consequence when compared with the perimeter dams. For that reason, a freeboard of 5 ft. was assigned to these facilities. With a lesser total embankment height, the crest width was reduced to 14 ft. Side slopes were established (nominally) at 3:1 (horizontal:vertical). Considering that wave attack can be generated from either side of the embankment, both sides (and the top) of the interior dam should be slope protected.

The interior dam section was designed for the condition when one cell of the Reservoir is emptied for maintenance, creating an unbalanced hydrostatic pressure across the dam. The dam was considered to be constructed of materials that are locally available and as a homogeneous embankment. Considering that the full unbalanced hydrostatic condition will occur very infrequently, if ever, the

embankment was not designed with an internal drain. For this reason, the result of the seepage analyses indicates that a very flat slope (i.e. approximately 14%) of the phreatic surface will result for this embankment. In order to limit the potential for allowing the phreatic surface to daylight above the toe of the slope, the side slopes of the embankment should be flattened to 8:1 beginning at a level that is 9 ft. above the surrounding grade. Details regarding the typical section of the interior dam are shown on **Figure 8A-2** in **Section 8** of the Appendices.

I. Erosion Protection

The upstream face of the perimeter dam should be armored using slope protection to limit the potential for erosion due to wind, wind-generated waves and wave run-up. Methods of protecting slopes include stone or sand-cement bag rip-rap, concrete pavement, articulated block, rock-filled gabions, and soil-cement. The type of slope protection to be utilized is predicated upon locally available materials and economic considerations. The lack of an abundant, locally available, hard rock formation makes stone rip-rap and rock-filled gabions relatively expensive. The project site is somewhat remote, and for this reason, the use of articulated block was assumed to be relatively expensive based on historic costs.

While a number of methods exist for slope protection, the most appropriate for the Reservoir is the use of soil-cement. Soil cement consists of a mixture of soil, Portland cement and water that is placed in thin lifts and compacted to high dry density. Studies have shown that the water wave energy is dissipated more rapidly when the soil-cement is placed in a "stair-step" manner. Soil cement will be used for lining the upstream slope and top of the perimeter dam and for covering the interior dam slopes and top for erosion protection.

We recommend that the stair-step method be used for that portion of the embankment that is above the normal pool elevation (i.e. that portion subject to wave attack). Below the normal pool water elevation, the upstream face of the embankment should be surfaced with soil-cement using the flat plate method. Details regarding the use of soil-cement are presented on **Figures 8A-1 and 8I-1** in **Section 8** of the Appendices.

Design of the soil cement slope protection should follow the guidelines described in the USACE EM 1110-2-2300, General Design and Construction for Earth and Rock-Fill Dams. Soils should have a maximum size less than 2 inches, not more than 45 percent retained on the U.S. Standard No. 4 Sieve, and between 5 and 35 percent passing the U.S. Standard No. 200 Sieve. The soils should have a maximum plasticity index of 12 and not more than 2 percent (by weight) organic matter. Mix design for the soil-cement should be based upon laboratory tests that are completed at the onset of construction. Based upon discussions with the contractor for the Tampa Bay Reservoir, the cement content utilized for that project was 9 percent by weight. A site-specific soil cement design mix will be determined in the final design. The soil-cement should be compacted to at least 98 percent of the maximum dry density determined in accordance with Modified Proctor (ASTM D 1557). The flat plate portion of the slope protection should be 16 inches thick while the stair-step portion should consist of elements that are 8 ft. wide and 16 inches

thick. It is noted that maximum density of 98 percent modified proctor may be difficult to achieve and therefore, the final design may consider reducing to 98 percent standard proctor or perform “test strips” to determine “optimum” soil-cement density which can be obtained for site-specific soils.

Section 9 – Levees

(Not Applicable)

Section 10 – Canals

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 10 of the Appendices.

A. Stability

Six (6) zones (sections) were selected along the Reservoir perimeter for subsequent geotechnical analyses, and these zones are described as follows:

- Zone A - Northern perimeter of Cells 1 and 2. This zone was further split into Zones A1 and A2 based upon differing tailwater conditions in the perimeter seepage canal.
- Zone B - Northern, eastern and southern perimeter of Cell 3.
- Zone C - Southern perimeter of Cells 1 and 2.
- Zone D - Western perimeter of Cell 1.
- Zone E - Eastern perimeter of Cell 2.

The zones and their stratigraphic profiles, canal bottom elevations and hydraulic boundary conditions are presented in the following table.

Zone	Subsurface Component	Depth (feet)	Seepage Canal Bottom EL.	Head Conditions (feet-NAVD)	
				Pool EL.	Seepage Canal EL.
A1	SAND	0 – 5	0	+37.4	+10
	SILTY SAND	5 – 10			
	LIMESTONE	10 – 15			
	CLAY (CL)	15 – 20			
	CLAY (CH)	20 – 40			
A2	SAND	0 – 5	0	+37.4	+16
	SILTY SAND	5 – 10			
	LIMESTONE	10 – 15			
	CLAY (CL)	15 – 20			
	CLAY (CH)	20 – 40			
B	SAND	0 - 12	0	+37.4	+16
	LIMESTONE	12 - 20			
	SILTY SAND	20 - 40			
C	SAND	0 – 5	+5	+37.4	+23
	LIMESTONE	5 – 18			
	CLAY (CL)	18 – 23			
	CLAY (CH)	23 – 40			
D	SAND	0 – 5	-5	+37.4	+2
	SILTY SAND	5 – 15			
	CLAY (CL)	15 – 20			

Zone	Subsurface Component	Depth (feet)	Seepage Canal Bottom EL.	Head Conditions (feet-NAVD)	
				Pool EL.	Seepage Canal EL.
	CLAY (CH)	20 – 40			
E	SAND	0 – 7	0	+37.4	+16
	LIMESTONE	7 – 15			
	CLAY (CL)	15 – 20			
	CLAY (CH)	20 – 40			

Note: Pool elevation shown in the table above represents the 3-cell option with constant pool level.

Utilizing the subsurface profiles for each of the zones, we have selected a range of canal bottom elevations for the proposed seepage canal. The canal bottom elevations ranged between -5 and +5 feet NAVD. These were selected considering the top of the clay layer that underlies much of the project site and the seepage evaluation that was completed as part of the Reservoir optimization study. Please refer to **Section 8D**. The final selection of canal bottom level was made in order that the canal bottom elevation would coincide with the top of the underlying clay layer such that lateral seepage capture is optimized. The canal bottom level for Zone D was based upon review of existing cross sections of the Townsend Canal, given the understanding that this canal will not be modified as part of the Reservoir Project. The seepage evaluation is summarized in **Table 8D-1** in **Section 8** of the Appendices.

Slope stability analysis was performed using the computer program STABL6H, which was developed at the Purdue University Engineering Experiment Section. We have evaluated three (3) different canal slope scenarios for this study. They are presented as follows:

Seepage Canal Slope		Canal Slope Condition
Above Water Elevation	Below Water Elevation	
3:1	2:1	CS1
3:1	1.5:1	CS2
3:1	1:1	CS3

Based on the hydraulic head conditions provided in the table presented earlier in this section, Zone D has the maximum head difference between the pool water elevation and canal water elevation, and hence was chosen to evaluate the canal slope stability. In subsequent submittals, soil types will also be considered in selection of the analysis section. As discussed in **Section 8A** along the perimeter of the dam, the bench width varied between 30 and 150 ft. However, initially, in our slope stability analysis for canal slope evaluation with Zone D, we have used a bench width of 30 ft. The 30 ft side bench width was determined as that required to provide exit gradients that are 0.33 or less and resulting in a factor of safety of 3.0 or more against piping. However a more conservative bench width of 50 ft may be considered in subsequent analysis. The initial slope stability analysis included Zone D with a seepage canal water surface elevation of +2 ft. (NAVD), a pool elevation of +37.4 ft. (NAVD), a bench width of 30 ft. and three possible canal slope conditions. The phreatic surface was assumed to vary linearly between pool water elevation and canal water elevation.

The analysis results are as follows:

Zone	Pool EL. (feet)	Seepage Canal EL. (feet)	Bench Width (feet)	Canal Slope Condition	Factor of Safety
D	+37.4	+2	30	CS1	1.66
				CS2	1.67
				CS3	1.66

We also analyzed Zone D with a bench width of 150 ft. and the phreatic surface obtained from the seepage analysis, and the analyses results are provided below:

Zone	Pool EL. (feet)	Seepage Canal EL. (feet)	Bench Width (feet)	Canal Slope Condition	Factor of Safety
D	+37.4	+2	150	CS1	1.70
				CS2	1.70
				CS3	1.70

The Perimeter Canal was sized by designing the depth to intercept the clay layer, allow for a bottom width of 10 feet, provide side slopes below normal water levels of 2:1, and provide slopes above normal water levels of 3:1. These typical sections were used in the modeling to ensure that the sections were adequate for the high flows.

Based on review of the design criteria presented in **Section 8A** the above factors of safety values are acceptable. The results are graphically presented in **Appendix 10A-A**.

Zone C was also selected since it has the highest seepage canal water surface elevation [+23 ft. (NAVD)] with pool water elevation + 37.4 ft. (NAVD) and was analyzed with three (3) seepage canal slope conditions. The results of the analysis are as follows:

Zone	Pool EL. (feet)	Seepage Canal EL. (feet)	Bench Width (feet)	Canal Slope Condition	Factor of Safety
C	+37.4	+23	50	CS1	1.40
				CS2	1.33
				CS3	1.14

The above results are graphically presented in **Appendix 10A-A**. The calculated factors of safety for Zone D are remarkable close for the three conditions analyzed which is unlikely and requires the analysis to be reviewed and re-checked. A review of the factor of safety for Zone D will be completed prior to the submittal of the 30% preliminary design.

Zones B, A2 and A1 were also analyzed with three (3) seepage canal slope conditions. The results of the analysis are as follows:

Zone	Pool EL. (feet)	Seepage Canal EL. (feet)	Bench Width (feet)	Canal Slope Condition	Factor of Safety
B	+37.4	+16	30	CS1	1.40
				CS2	1.33
				CS3	1.21
A2	+37.4	+16	30	CS1	1.50
				CS2	1.35
				CS3	1.19
A1	+37.4	+10	50	CS1	1.47
				CS2	1.42
				CS3	1.42

The above results are graphically presented in **Appendix 10A-A**.

Based on review of EM 1110-2-1902 (Slope Stability), we understand that the required minimum factors of safety for maximum surcharge pool (pool thrust from maximum surcharge pool) and long term (steady state seepage and maximum storage pool) are 1.4 and 1.5, respectively. Based on review of analysis completed for this evaluation, we believe that seepage canal slope conditions CS2 and CS3 do not provide adequate factors of safety. Hence, the decision was made to use canal slope condition CS1, i.e. 3:1 (horizontal: vertical) sideslopes above the seepage canal water surface elevation and 2:1 (horizontal: vertical) sideslopes below the seepage canal water surface elevation for all subsequent analysis. However, we recognize that at Zones C and B, the factors of safety for canal slope condition CS1 are below 1.5. Final designs will address the factors of safety that are below 1.5. Seepage control measures will be evaluated to lower the phreatic surface adjacent to the canal bank, which should result in higher factors of safety. This is described in **Section 10B**.

Foundation conditions include a layer of low strength clays and therefore, a wedge analysis will be performed in the 30% preliminary design phase. The wedge analysis will include, at a minimum, starting in the bottom of the canal at the clay layer, extending upstream in the layer to a location approximately at the downstream side of the core and then extending to the upstream crest of the dam. A search routine for other wedge configurations will be performed.

In the 30% preliminary design phase, an evaluation will be made that includes the effects of a canal slope failure on the embankment stability. Seepage collars will likely be required around conduits that penetrate the perimeter embankment and sheetpile wingwalls may be required adjacent to and beneath pump stations and water control structures. The soil-structure interaction and differential settlement of embankment-structure interfaces also requires design. These types of engineering detailed will be analyzed during the final design. A large dollar amount was added for contingencies at this conceptual stage of design.

B. Seepage Control

Seepage control measures for the perimeter dam are described in **Section 8 H 2**). The downstream seepage will accumulate in the drainage blanket for eventual outfall to the seepage collection canal. The design of the outfall has not been completed. It will likely take the form of a downstream toe ditch, backfilled with concrete sand and including a perforated collection pipe. The pipe will have laterals which discharge to the canals at horizontal intervals determined by engineering calculations.

C. Erosion

The canal banks will consist primarily of sandy material and it is recommended to stabilize with sod to prevent erosion. The canal berm area and top of bank slope will be addressed in the 30% preliminary design submittal as part of the canal erosion protection measures.

Section 11 – Structural Design

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 11 of the Appendices.

A. Design Criteria

1) Loads and Forces

(a) Load Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in ASCE 7. If load combinations are prescribed in an applicable engineering manual, then the load combinations shown in the engineering manual takes precedence over ASCE 7.

(b) Wind Design

Wind loads acting on Highway Bridge structures will be calculated according to AASHTO LRFD Bridge Design Specifications and the FDOT Structures Manual.

Wind loads for the design of the main wind-force resisting system and for components and cladding of buildings and water control structures will be calculated according to the SFWMD Major Pumping Station Engineering Guidelines (November 2004), CERP Standard Design Manual (June 2003), Florida Building Code, and ASCE 7-98.

- Wind Speed (V)

  Basic sustained wind speed determined from Figure 6.1 of ASCE 7-98.

- Mean Recurrence Interval (MRI)

  For the control room of a pumping station or other facility that is manned during a hurricane, a MRI of 500 years (Importance Factor = 1.51) shall be applied. All structural elements shall be evaluated for effects of hazards, from elements within proximity of the control room, and should adopt the more stringent design criteria. For structures that are designated flood control structures and are required to operate during a hurricane, a MRI of 200 years (Importance Factor = 1.30) shall be applied. This includes the flood control structure's control building. For water control structures that

are not required to operate during a hurricane, which may include but not limited to seepage control and water supply structures, a MRI of 50 years (Importance Factor = 1.15) shall be applied.

- Wall openings

  For structures that have been designed for a MRI of 200 years or greater, rolling doors, fans, windows, ducts, and louvers shall be designed to meet the building envelope protection requirements of the Florida Building Code, Test Protocols for High Velocity Hurricane Zones. Additionally, all items shall meet this requirement for components and cladding. Single or double doors shall meet the requirements of FEMA 361. For structures that have been designed for a MRI of 50 years, fans, ducts, and louvers shall be designed to meet the requirements of the Florida Building Code. All exterior doors shall be steel.

(c) Seismic Design

Structures at this site need not be designed and constructed to resist stresses produced by seismic forces. Other loads/load combinations will govern by inspection. Bridge bearing seats will have to meet minimum seismic length requirements as specified by AASHTO specifications.

(d) Dead Loads

The loads resulting from the weight of all fixed construction and equipment, such as walls, partitions, floors, roofs, equipment bases, and all permanent non-removable stationary construction, will be considered dead loads. Dead loads shall be determined in accordance with applicable engineering manuals and ASCE 7.

(e) Live Loads

The loads resulting from equipment, pedestrian traffic, cranes and monorails, etc, and applied to offices, roofs, floors, stairways, catwalks, corridors, lobbies, decks, etc. Live loads shall be determined in accordance with applicable engineering manuals and ASCE 7.

Service Bridges will be designed for AASHTO HS-25 (ref. AASHTO Standard Specifications for Highway Bridges). Capacity of Service Bridges will also be checked for the following vehicles:

- SFWMD typical crane – Link-Belt HTC-1055, a 55-ton capacity crane with a total vehicle weight of 88 kips.

- Florida legal loads SU-3 and SU-4.

Highway Bridges will be designed for the HL-93 vehicular load of the AASHTO LRFD Bridge Design Specifications and criteria given in the Florida DOT's Bridge Design Manual.

(f) Live Load Surcharge

A minimum live load surcharge of 500 psf will be applied to all abutment walls and wingwalls of hydraulic structures and pump stations. The criteria of the AASHTO LRFD Bridge Design Specifications, Section 3.11.6.4 will be used to determine if higher live load surcharge loads should be used.

(G) Soil Loads

Structure base pressure will be considered and lateral soil loads will be analyzed for structure stability and strength.

(h) Hydrostatic Loads

Uplift pressure, structure stability and strength will be analyzed for hydrostatic loads.

(i) Rain Loads

Roofs shall be designed to preclude instability from ponding.

(j) Impact Loads

Supporting elements for machinery which produce impacts will have their operating loads increased as recommended by code or by manufacturer's recommendation. These include:

- Supports of elevators and elevator machinery.
- Supports of light machinery, shaft or motor driven, such as water pumps.
- Supports of reciprocating machinery or power driven units, such as generators and blowers.
- Hangers supporting floors and balconies.
- Traveling crane support girders and their connections.

Supporting elements for machinery which produce harmonic loads, such as generators and blowers, will be designed to provide natural dampening to avoid resonant vibration.

2) Stability

For the purpose of establishing safety factors for use in stability analyses, structures are designated as either critical or normal. Structures are designated as critical if their failure will result in loss of life. If the effects of a structural failure are unknown, then an inundation study may be required to classify the downstream hazard. Additional guidance on the classification of downstream hazards is provided in ACER Technical Memorandum No. 11.

Analyses will be made for stability of structures against overturning, sliding, flotation, and foundation pressure. Live loads, equipment weight, soil friction and cohesion on the walls should not be used to calculate the resisting dead load of the structure for flotation. The structure must be designed to resist the full uplift pressure from normal water level elevation while empty.

3) Vibration

The natural frequency of the support structure must be significantly different from the frequency of the disturbing force. As the two frequencies approach each other, resonant vibration may occur in the support structure. To minimize the resonance, the ratio of the frequency of the structure to the frequency of the equipment should be either less than 0.5 or greater than 2.1, preferably the latter. With the stiffer support system, the machinery will not have to pass through the resonance frequency during start-up or shut down.

Assuming a single degree-of-freedom system, the natural frequency of a structural member can be calculated by:

$$\omega = \frac{\sqrt{\Delta st}}{3.13}$$

where ω = fundamental frequency
and Δst = static deflection due to its own weight (inches)

Refer to ACI 350R for more information on natural frequencies for beams and recommended maximum structural deflection for given equipment operating speeds.

The following guidelines are recommended for designing equipment supports:

- Use vibration isolators or dampeners on structural floors if appropriate
- Consult with the equipment manufacturer to obtain recommendations, frequencies, and unbalanced loads.
- Mount large rotating and reciprocating machinery on concrete foundations, and isolate from structure if possible.

- As a general rule of thumb, provide a concrete foundation equal in mass to 10 times the weight of the rotating parts of the equipment, or equal to 3 times the total weight of the equipment.

- Use embedded anchor bolts (with sleeves for adjustment) instead of drilled anchors.

- When the use of a structural steel support system is dictated by project requirements, carefully check the structure frequency for possible resonance. Add bracing as required to stiffen the structure to ensure a natural frequency greater than 2.1 times the equipment speed. Consider using vibration isolators or inertia pads, as recommended by the manufacturer. All bolted connections shall be slip-critical high strength bolted connections.

Vibration isolators may not eliminate natural frequency problems, but they may be appropriate for reducing induced frequencies and controlling the effects of short duration frequencies.

4) Reinforced Concrete Design

General criteria for concrete shall be in accordance with EM 1110-2-2000 and ACI 318.

(a) Load Factors

Load combinations and strength design factors for hydraulic concrete structures shall be in accordance with EM 1110-2-2104. Load combinations and strength design factors for all other concrete structures shall be in accordance with the most recent edition of ASCE 7.

(b) Design Values

Typical design values are as follows:

- Mass Concrete - f'_c = 2500 psi @ 28 days.

- Structural Concrete - f'_c = 4000 psi @ 28 days unless otherwise noted.

- CIP Concrete Box Culvert - f'_c = 5500 psi @ 28 days.

- Prestressed Concrete - f'_c = 5000 psi @ 28 days.

- Steel Reinforcement - f_y = 60 ksi.

5) Structural Steel Design

General criteria for steel shall be in accordance with EM 1110-2-2105 and AISC.

(a) Load Factors

Load combinations and strength design factors for hydraulic steel structures shall be in accordance with EM 1110-2-2105. Load combinations and strength design factors for all other steel structures shall be in accordance with the most recent edition of ASCE 7.

(b) Design Values

Typical design values are as follows:

- Structural Steel – A 572, Grade 50.
- Corrosion Resisting Steel – Type 304 (freshwater) and 316 or 317L (saltwater).
- Sheet Piles – Normally type A 328 or A 572 (grade 50), hot rolled for permanent structures. Cold rolled sections (A 328 or A572) may be used for temporary structures including bypass walls.

(c) Bolted Conditions

Structural steel connections normally type A325 Type I, or A490 Type I. Submerged connections (i.e., gate seals) normally CRES bolts with Armco Nitronic 60 nuts.

(d) Coatings

Normally, components that shall be exposed to the elements are either hot-dipped galvanized or painted with coal tar epoxy. Vertical lift gates and steel sheet pile structures may be painted with a vinyl paint system.

6) Aluminum Design

General criteria for aluminum shall be in accordance with the *Aluminum Design Manual*.

Aluminum alloy 6061-T6 will be used for the basic design of aluminum structures and members. Other alloys may be used for specific requirements. Design will be governed by the *Florida Building Code* and the *Aluminum Design Manual*.

B. Water Control Facilities

The design of hydraulic structures for this project is at a preliminary stage. While many details have been determined, complete hydraulic requirements have not yet been established. For example, the outlet structures and some of the spillway structures may require energy dissipaters, but the hydraulic design requirements for type and size of the dissipaters have not yet been developed. Also, exact existing and proposed ground elevations are not yet known, so estimated values are used for determination of structure dimensions. As these ground elevations are refined the structure dimensions will need to be revised. All structures will be designed to resist the flotation effects of dewatering.

Preliminary hydraulic modeling revealed that a single or multiple 12 ft. wide gates could be used at many of the water control structures. Therefore, to standardize the gate designs as much as possible, 12 ft. wide gates are used for the three primary outlet structures as well as for the gated spillway structures. SFWMD's design standards require all water control structure gates be stainless steel.

Geotechnical recommendations for the structures have not yet been developed. Therefore, the foundation types have not been finalized. The preliminary details show foundation types used to develop the preliminary cost estimate. The 30% preliminary plans will show recommended foundation types. If seepage along the bedding is a particular concern based on existing soil conditions, then a "seep shield" will be included in the 30% preliminary design.

1) Outlet Structures

There are a total of four outlet structures proposed. Structure S-1 is the primary outlet structure for Cell 1 of the Reservoir, Structure S-8 is the primary outlet structure for Cell 2 of the Reservoir, and Structure S-5 (Alternate B only) is the primary outlet structure for Cell 3 of the Reservoir. Structure S-2 serves as a discharge structure to provide water to the NE Rim Ditch. S-1 discharges from Cell 1 into the Townsend Canal. S-2 discharges from Cell 1 into the Perimeter Canal. These are gate culvert structures.

(a) Structures S-1, S-5 and S-8

These are the primary outlet structures to be used during normal operations of the Reservoir. Structure S-1, **Figure 11B-1**, provides an outlet to Cell 1 of the Reservoir; S-5, **Figure 11B-2**, is the outlet for Cell 3; and S-8, **Figure 11B-3**, is the outlet for Cell 2. **Figures 11B-1 through 11B-3** can be found in **Section 11** of the Appendices. See **Table 11B-1** for a summary of critical elevations and dimensions used to develop the probable structure construction costs.

The configuration of the gates used for the structure estimate is physically different but hydraulically similar to the gates used in the

hydraulic modeling (see **Section 6**). The hydraulic modeling uses gates at two elevations. The intent of this was to prevent discharge of sediment laden water from the lower levels of the reservoir by using only the upper gate for normal discharges. To minimize costs, the primary outlet structures have been sized with an underflow gate with an invert elevation at the bottom of the reservoir. They utilize bulkheads in front of the gates to prevent the flow of sediment laden water through the gate. The bulkheads are removable should a cell of the Reservoir need to be totally dewatered.

The primary outlet structures will consist of an intake area, a conduit to convey the water through the dam embankment, and an outlet area. The intake area will consist of wingwalls and intake bays, each with a 12 ft. wide vertical lift roller gate, removable bulkheads and a floating weed boom/trash rack debris control system. The conduit consists of a single or double barrel box culvert; each barrel has a 4 ft. high by 12 ft. wide opening. The outlet area consists of a headwall and wingwalls opening into a channel to carry the water to the Townsend Canal. Wingwalls for the intake and outlet areas are currently shown as cast-in-place concrete retaining walls. Steel sheet pile retaining walls will also be considered. The trash rack bars will be braced enough to minimize vibration.

Dewatering of individual bays can be accomplished by placing additional bulkhead panels above those required for normal operation. The structure's intake area concrete headwall and sluice type vertical roller gate will function as the Reservoir wall. At the top of the headwall in the intake area an emergency overflow opening is provided to allow excess water to be discharged through the conduit. The crest elevation of the emergency overflow will be 11 inches above the Reservoir "normal pool" water surface elevation.

A walkway will be provided from the top of dam elevation to the intake area, with a platform at the top of and behind the headwall. The dam will be widened to provide a minimum top width of 30 ft. in the vicinity of the structure. This will allow for full extension of the outriggers of the Link-Belt HTC-1055 crane being used for design, as well as providing an area to place the precast control building.

	Alternative A, Options 1 & 2		Alternative B, Options 1 & 2		
	S-1 (Cell 1)	S-8 (Cell 2)	S-1 (Cell 1)	S-8 (Cell 2)	S-5 (Cell 3)
Discharge (cfs)	1,000	500	1,000	500	500
Top of Dam Elev.	48.92	49.27	47.42	47.84	44.84
Full Pool Elev.	39.24	39.24	37.55	37.55	37.55
Gate Width	2 – 12’	1 – 12’	2 – 12’	1 – 12’	1 – 12’
Invert El. at Gate	16.0’ ±	20.0’ ±	16.0’ ±	20.0’ ±	25.0’ ±
Stop-log Elev.	20.0’ ±	25.0’ ±	20.0’ ±	25.0’ ±	30.0’ ±
Overflow Crest El.	40.16	40.16	38.47	38.47	38.47

Table 11B-1 – Primary Outlet Structures

For the 30% preliminary design, a more effective design will have gates that discharge water from the top of the water column rather than the bottom of the water column. This might require floating submerged weir gates or similar structures that follow the stage in Cell-1 resulting in a more expensive albeit a better discharge water quality result.

(b) Structure S-2

S-2 will be a discharge structure to provide water to the improved NE Rim Ditch and will be located within the dam alignment, along the southern edge of Cell 1. See **Figure 11B-4** in **Section 11** of the Appendices for a preliminary plan and elevation of Structure S-2, and **Table 11B-2** for a summary of critical elevations and dimensions. The hydraulic model is currently using a 7 ft. wide gate at this structure, however it is not required for hydraulics.

Discussions with the H/H modelers indicate that any gate width can be used – they can adjust the opening heights to achieve the required discharge. The final hydraulic model will be adjusted to match actual sizes proposed for use. No provision is made for emergency overflow discharges at S-2 since structure S-1 provides the emergency overflow for Cell 1. Stop-logs are used at the primary outlet structures to control release of sediment laden water into the Caloosahatchee River. Structure S-2 discharges water for irrigation use by Duda Farms. It was determined that these sediments would not be a detriment to irrigation use.

Therefore, for the structure estimate a 5 ft. wide gate has been assumed to allow use of standard gate sizes.

The configuration of S-2 is the same as for the primary outlet structures except for the following:

- no provision is made for emergency overflow discharges
- stop-logs are not provided; i.e., discharge of sediment laden water from the bottom of the Reservoir is not prevented
- dewatering for maintenance is achieved through the use of dewatering needles

	Alt. A	Alt. B
Top of Dam Elev.	48.92	47.42
Full Pool Elev.	39.24	37.55
Gate Width	1 – 5’	1 – 5’
Invert El. at Gate	23.0’ ±	23.0’ ±

Table 11B-2 – Structure S-2

In subsequent submittals, we will verify that the full pool level is modeled at the planned levels. The stop log elevations were selected based upon operating rules for the outflow gates. In the 3-cell configuration, releases occur first from Cell 3, then from Cell 2, and finally from Cell 1. Consider structure S-8 located in Cell 2, the ground surface at this location is approximately 20 feet. In the model, this gate will not operate if the Reservoir water level is greater than 32 feet. So a 5-feet high stoplog is approximately 42 percent of the maximum water column during periods of operation for this structure. A sediment loading evaluation will be conducted during implementation of the Test Cell Project.

The purpose of S-2 is to release water to the NE Rim Ditch when water is already being released to theC-43 and Duda starts their pumps. Essentially, under this condition, any additional water pulled from the C-43 will need to be replaced by flow from the Reservoir. Therefore, an amount equal to the amount Duda is pumping will need to be released to maintain minimum flows at S-79. If the Perimeter Canal is not separated from the NE Rim Ditch, these issues and others will need to be addressed in an agreement with Duda.

2) Spillways/Weirs

(a) Structures S-3, S-4, S-15

These structures are gated spillway structures which will conform to the general configuration shown in the SFWMD Standard Guideline Sheet GS-1. This standard shows the general criteria for the SFWMD standard water control structure with vertical lift roller gates. The structures will each have a precast control building. The ogee weir and vertical roller gates at these structures will maintain the desired water surface elevation.

The operating platforms will be elevated above the structure walls as necessary for proper gate operation. Each structure will also have a service bridge and dewatering capability to enable dry maintenance. The service bridge at S-3 is recommended to be wider than the minimum since it carries the Reservoir maintenance road over the Perimeter Canal, and the maintenance road will have 90-degree turns on both sides of the bridge. **Figures 11B-5 through 11B-7** found in **Section 11** of the Appendices show the preliminary details of these structures, and **Table 11B-3** summarizes the critical elevations and dimensions.

The current hydraulic model assumes three 12 ft. wide gates at S-3 and two 12 ft. wide gates at S-4. Since the flow is similar at these two structures, future runs will evaluate the possibility of using two 12 ft. wide gates at S-3. However, for the development of the opinion of cost, the current configuration of three 12 ft. wide gates has been used.

	Alts. A & B, Option 1		**Alternatives A & B, Option 2**		
	S-3	S-15	S-3	S-4	S-15
US WS Elev.	23.00	25.5	23.00	18.00	25.5
US Bottom Elev.	5.00	14.0±	5.00	0.00	14.0±
DS WS Elev.	16.00	16.00	18.00	16.00	18.00
DS Bottom Elev.	0.00	0.00	0.00	0.00	0.00
Spillway Crest Elev.	16.00	22.00	16.00	10.00	22.00
Max. Opening Elev.	26.00	28.00	26.00	18.00	28.00
Overflow Elev.	27.00	29.00	27.00	19.00	29.00
Number of Gates	3	2	3	2	2
Size of Gates (width x height)	12’ x 10’	12’ x 6’	12’ x 10’	12’ x 8’	12’ x 6’
Service Bridge Width (curb-curb)	22’	16’	22’	16’	16’

Table 11B-3 – Gated Spillway Structures

(b) Structures S-6, S-7 & S-9

These three structures are fixed weirs that serve to maintain water flow into the Crawford Canal (S-6), the Banana Branch (S-7), and the Ft. Simmons Branch (S-9). Typically, a channel will be built tying the Perimeter Canal to each of these existing canals. A reinforced concrete pipe culvert will carry the flow under the ASR and maintenance right-of-ways. North of the maintenance right-of-way a fixed weir will be constructed across the channel using steel sheet piling with a CIP

concrete cap (see **Figure 11B-8** in **Section 11** of the Appendices for details).

The exception to this is Structure S-6 in Alternate A, Options 1 & 2. For these options S-6 is moved south to be adjacent to the existing LPDD Canal. At this time it is not known whether or not a pipe culvert will be required to carry a maintenance access.

(c) Structure S-10

S-10 will be located on the Perimeter Canal just upstream from Structure S-11. It will maintain the water surface elevation in the Perimeter Canal upstream from S-10 at Elevation 16.00. This structure will consist of a fixed concrete weir across the Perimeter Canal at Elevation 13.00, with overflow slide gates to Elevation 16.00. The slide gates will operate downward.

Downward operation was used by hydraulic designers to achieve their needs. Issues related to potential for debris impact/accumulation on gate stem during discharges, operation and maintenance of the gate will be addressed in the 30% preliminary design phase.

The structure will have a service bridge to allow access to the structure for maintenance and removal of stop-logs. See **Figure 11B-9** in **Section 11** of the Appendices for details, and **Table 11B-4** for a summary of critical elevations and dimensions.

The configuration of the gates used for the structure estimate is physically different but hydraulically similar to the gates used in the hydraulic modeling (see **Section 6**). The hydraulic modeling uses five 5-ft. wide overflow gates. To improve the constructability of the structure, four 6-ft. wide overflow gates were assumed. This allows the placement of a center pier for support of the service bridge.

Structure S-10	
US WS Elev.	16.00
US Canal Bott. Elev.	0.00
DS Bottom Elev.	10.00
DS Canal Bott. Elev.	0.00
Fixed Spillway Crest Elev.	13.00
Stop-log Crest Elev.	16.00
Opening Widths	4 @ 6’
Service Bridge Width (curb-curb)	16’

Table 11B-4 – Structure S-10

(d) Structure S-11

S-11 will be located on the Perimeter Canal just upstream from the confluence of the Perimeter Canal with the Townsend Canal. It will maintain the water surface elevation in the Perimeter Canal upstream from S-11 at Elevation 10.00. This structure will consist of a fixed concrete ogee weir (no gates) across the Perimeter Canal. The structure will have a service bridge to carry the Reservoir maintenance roadway across the Perimeter Canal. See **Figure 11B-10** in **Section 11** of the Appendices for details, and **Table 11B-5** for a summary of critical elevations and dimensions.

Structure S-11	
US WS Elev.	10.00
US Canal Bott. Elev.	0.00
DS WS Elev.	3.00±
DS Canal Bott. Elev.	0.00 to -7.70
Fixed Spillway Crest Elev.	10.00
Opening Widths	3 @ 20’
Service Bridge Width (curb-curb)	22’

Table 11B-5 – Structure S-11

3) Gates/Valves

(a) Gates

A preliminary investigation of gate types was performed to determine applicability of different gate types. Tainter, Obermeyer and Vertical Lift gates were determined to be possibilities for the primary Reservoir outlet structures. These options were presented to several SFWMD staff in November 2004. The comment was made that while the SFWMD does have examples of all of these gate types in their system, the vertical lift roller gate is by far the most common. The consensus was that there would be significant advantage to using vertical lift roller gates for all locations. These advantages would be primarily in maintenance and repair of the gates; the SFWMD has an expertise in this type of gate, where significant training would be required for maintenance of other gate types.

Therefore, all gates are proposed to be vertical lift roller gates. The gated spillway structures in the Perimeter Canal will have gates similar to the gates the SFWMD regularly uses. The Reservoir control structures will require special attention during design since they must resist larger hydrostatic pressures than the typical SFWMD structure.

General criteria for steel shall be in accordance with EM 1110-2-2701 *Vertical Lift Gates*.

(b) Valves

See **Section 11** for a discussion of valves used in the pump stations.

4) Pump Stations

Structural design of the pump stations has been limited at this time to preliminary review of the conceptual drawings. Structural design will be further developed following selection of the pump station discharge arrangement and during preparation of 30% preliminary design drawings. Please refer to **Figures 13B-01 through 13B-08** in **Section 13** of the Appendices for conceptual drawings of the pump stations.

C. Culverts

1) Structures S-12 & S-13

The purpose of these structures is to balance the water surface elevations between the cells of the Reservoir. S-12 will be located in the separator dam between Cell 1 and Cell 2. S-13 will be located in the separator dam between Cell 2 and Cell 3, and is only required for Alternate B. Each of these structures consists of a two barrel box culvert similar to the SFWMD Standard Culvert Details, C6; the opening of each barrel is 10 ft. wide by 10 ft. high. A remotely controlled vertical lift roller gate will be provided at each end of the culvert to allow isolation of the Reservoir to either side of the separator dam. Dewatering for maintenance will be accommodated by use of dewatering needles.

A walkway will be provided from the top of dam elevation to the gate area, with a platform at the top of and behind the headwall. The dam will be widened to provide a minimum top width of 30 ft. in the vicinity of the structure. This will allow for full extension of the outriggers of the Link-Belt HTC-1055 crane being used for design, as well as providing an area to place the precast control building. See **Figure 11C-1** and **11C-2** in **Section 11** of the Appendices for details.

2) Structure S-14

This structure connects the existing LPDD Canal to the proposed Perimeter Canal. It allows for positive drainage of the S.R. 29 basin. At this time no weirs or gates are anticipated. The hydraulic analysis has not yet determined the size of culvert required.

D. Control/Operations Buildings

With the exception of the pump station structures discussed in **Section 11 B 4,** all of the water control structures with mechanized gates will have precast concrete control buildings in general conformance to the SFWMD Design Details S7, Control Building with Generator.

E. Miscellaneous Structures

1) S.R. 80 Bridge over the Townsend Canal

Hydraulic analysis of the capacity of the existing Townsend Canal cross section has not been completed. Preliminary runs indicate that it may be possible to provide adequate capacity in the canal by making relatively minor changes to the existing canal cross-section without lengthening the existing bridge. This type of change would require placement of some form of scour protection around the existing bridge piles, but would be more economical than lengthening or replacing the existing bridge.

The evaluation of options for the bridge has and will be on hold until final hydraulic evaluation of the canal requirements has been completed.

The existing bridge is a dual structure with approximately 30.9 ft. clear space between the bridges. The bridges consist of a reinforced concrete slab superstructure, supported on two end bents and four intermediate bents. Each of the end and intermediate bents consist of four plumb 18 inch square prestressed concrete piles with a reinforced concrete cap beam. The existing bridge was constructed in 2003 and is in excellent condition.

The final bridge configuration under any conditions should maintain a minimum of 2.0 ft. of freeboard above the 50-year DHW. Horizontal channel clearance should not be less than 25 ft. 9 in. provided by the existing bridge.

2) Monitoring Stations

The required number and locations of monitoring stations have not yet been established. Monitoring stations will be provided in general conformance to the SFWMD Design Details number S18.

Section 12 – Site Civil Design

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 12 of the Appendices.

A. Site Access and Roadways

Access to the project site is limited to S.R. 80. S.R. 80 is an east-west four-lane divided rural road that borders the northern boundary of the project. Currently there is access to the Berry Groves office and field operations from S.R. 80. This access is controlled by a guard stationed at the entrance just off of S.R. 80. It is anticipated that this will be the primary entrance road to be used by the contractor during construction. It will be recommended that control at this access be maintained during construction. When the construction is complete the entrance road will be rehabilitated, including milling and resurfacing, to provide access to the Reservoir site.

Access from the entrance road to the Reservoir during construction will be different from the access that will be provided when construction is complete. It is expected that there will be numerous perimeter/seepage canal crossings installed during construction to provide efficient access to the work areas. The construction crossings will be removed and the canal restored as construction is completed.

Permanent access through the site will include a perimeter road completely around the Reservoir, bridges to cross the perimeter/seepage canal, access along the top of the exterior and separator dams, ramps to the top of the dam, access roads and ramps to the pump station(s) and designated parking areas at the structures and pump station(s). Auto Turn software will be used to develop adequate auto and truck turning movements into and within the site. Vehicular ramps to the top of the dams will be designed to meet ADA standards for pedestrian access. A conceptual study of the access to and around Pump Station No. 1 (PS-1) has been developed and is shown on **Figure 12A-01** in **Section 12** of the Appendices. The plan will be further developed when details of PS-1 are further developed and adopted. Additionally, access to and around Pump Station No. 2 (PS-2) will be developed if Option 2 of either Alternative A or B is selected as the preferred alternative. Additionally, standard turnouts will be constructed along the dam top at 0.5 mile intervals. The dam top width will be widened at the control structures to accommodate maintenance equipment and a control building. Refer to **Figure 12A-02** in **Section 12** of the Appendices for the widening concept. Ramps at strategic locations along the Perimeter Canal for boat access will be considered in the design. Refer to **Figures 18B-01 and 18B-02** in **Section 18** of the Appendices for a conceptual plan of recreational features including the locations of proposed boat ramps.

B. Site Layout

1) General Discussion

At the beginning of the 30% preliminary design phase, the property to be considered for the Reservoir footprint included the Berry Groves, M.G. Enterprise and Griffin Properties. **Figure 12B-01** in **Section 12** of the Appendices indicates the approximate boundary of the property initially acquired by the SFWMD. It was recognized that this property and the exterior shape had some disadvantages to constructing an economical and functioning Reservoir, as a Reservoir with a rectangle shape is more economical than one that is irregularly shaped. There were also potential impacts to the community as the Reservoir property bordered S.R. 80. It was expressed to the SFWMD by local government officials and local property owners that it would be preferred if the Reservoir proper could be kept at a reasonable distance from S.R. 80. It was suggested that a one-mile buffer from S.R. 80 should be maintained, but a buffer of one-half mile was acceptable.

It was also recognized that property north of the existing FP&L overhead transmission line could only be developed if the power line was relocated or dams were placed parallel and on each side of the power line. These parcels north of the FP&L power line are noted as Parcels S1, S2, S3, S4, and S5 on **Figure 12B-01** in **Section 12** of the Appendices. It was agreed that these parcels would not be included in the development of the Reservoir.

Eliminating the acreage discussed above would greatly reduce the potential capacity of the Reservoir were it contained within the propertied owned by the SFWMD. As the 30% preliminary design progressed alternatives for the size and shape of the Reservoir were explored. The different alternatives would require additional land to allow the Reservoir to meet the goals of the project.

The SFWMD approached both Mr. Bryan Paul of Bryan Paul Citrus, and A. Duda and Sons to explore the possibility of land swaps or property purchases that would provide for a more rectangular shape of the Reservoir. Specific parcels will be identified as the alternate Reservoir configurations are discussed.

2) Alternate Reservoir Configurations

The Berry Groves Site Option 2 from the PIR process provided the first shape of the Reservoir (Refer to **Figure 12B-02**) in **Section 12** of the Appendices. Refinements such as locating the north Reservoir dam south of the FP&L easement and the east dam to encompass land parcels under consideration for purchase or land swap by the SFWMD were the basis for the "Point of Beginning".

Earthwork volumes and storage capacities were used as the primary indicators of evaluating various Reservoir footprints and interior cell layouts. Freeboard was calculated for each alternative used to determine dam heights and earthwork volumes. Alternate Reservoir configurations included one, two, three and four cells as well as constant and varying water surface elevations.

For this BODR the data pertaining to storage and earthwork volumes was developed using the best available topographic and DTM information. The storage and earthwork calculations were done utilizing excel spreadsheets. The results were checked against the LIDAR DTM (SFWMD point files) using Land Development Desktop software and the earthwork was found to be within 0.5% of the spreadsheet calculations. The survey and mapping products will be utilized in the final 30% preliminary design submittal and minor adjustments of earthwork and storage volumes is anticipated.

(a) Initial Reservoir Configuration

The initial configuration of the Reservoir required that the Bryan Paul parcels D and D1 and the Duda parcels A1, I, I1, and K become part of the SFWMD land holdings. The Basic Rectangle including parcels A, A1, B, C, D, D1, I, I1 and K would become the land area "Point of Beginning". See **Figure 12B-03** in **Section 12** of the Appendices for the land required for the Basic Rectangle**.**

In preparation for the May 7, 2004 kick off meeting the "Basic Rectangle" was divided into six cells with each cell approximately 1.5 miles by 2 miles. The Reservoir area was approximately 10,985 acres. Summary tables presenting Reservoir storage capacities and earthwork volumes for varying dam heights were prepared and presented. Several assumptions were made including: the existing ground was considered to be flat, the Reservoir water surface elevation would be the same in each of the 6 cells and both the perimeter and interior dam top elevations would be the same. The storage capacities and the earthwork volumes were estimated for average dam heights ranging from 18 ft. to 28 ft. and average water depths ranging from 8 ft. to 18 ft. Pumps and structures and flow directions were indicated on the initial figures but were not evaluated for the kick off meeting. The initial estimates for the "Basic Rectangle" indicated that a water depth of between 14 ft. and 16 ft. would satisfy the volume requirement. Freeboard was estimated to be 10 ft. **Table 12B-01** provides a summary of the estimated storage considering a constant Reservoir shape and six cells with water depths in all of the cells varying from 8 ft. to 18 ft. (*It is important to reiterate that this data was developed using basic assumptions and without the benefit of a DTM or topography. For the initial calculations the bottom of the Reservoir was considered to be level).*

Dam Height (ft)	18	20	22	24	26	28
Water Depth (ft)	8	10	12	14	16	18
Storage Volume (ac-ft)	82,900	103,300	123,700	143,900	164,000	184,000

Table 12B-01 - Basic Rectangle Impoundment at Varying Water Depths

To achieve the 160,000 ac-ft of storage the average water depth would be just less than 16 ft. with the average dam height just less than 26 ft.

As expected, the volume of material required for the perimeter and separator dams increased with the increase in the depth of water. Including a 20% shrinkage factor, the volumes ranged from 6,635,000 CY at the 8 ft. depth to 15,013,000 CY at the 28 ft. depth.

Figure 12B-04 in **Section 12** of the Appendices illustrates the cell layout for the initial Reservoir configuration.

(b) Basic Rectangle with Paul Acquisition

In an effort to reduce the average water depths and the dam heights expanding the land area to the east was explored. Acquisition of Paul parcel E1 to the east would increase the land area to 12,635 acres.

Using the same assumptions described above and treating the Paul property as a seventh cell it was determined that the 160,000 ac-ft impoundment could be met with an average water depth of approximately 14 ft. and an average dam height of 24 ft. The addition of the Paul property did not appear to incur any major additional costs beyond the cost of the land. Mr. Paul suggested that he would be willing to sell or trade for parcel E1. Refer to **Figure 12B-05** in **Section 12** of the Appendices for an illustration of the potential Paul acquisition and **Figure 12B-06** in **Section 12** of the Appendices for the associated storage volume summary.

(c) Basic Rectangle with Duda Acquisition

As an alternate to acquiring the additional Paul property the addition of Duda property along the southern limits of the basic rectangle was considered. By adding this land the Reservoir area would be approximately 14,493 acres. Using the same assumptions described above and expanding the size of the 6 cells it was determined that the 160,000 ac-ft impoundment could be met with an average water depth of approximately 12 ft. and an average dam height of 22 ft. The addition of the Duda property as part of the Reservoir would have a major impact on

the existing NE Rim Ditch which runs along Duda's northern boundary and serves as both an irrigation and drainage canal for the Duda operations. The NE Rim Ditch would have to be relocated outside the footprint of the Reservoir. Refer to **Figure 12B-07** in **Section 12** of the Appendices for an illustration of the potential Duda acquisition and **Figure 12B-08** in **Section 12** of the Appendices for the associated storage volume summary.

(d) Refinement of the Reservoir Land Configuration

With input from the SFWMD and refinements in the design criteria, the Basic Rectangle with the Duda Acquisition to the south was eliminated from further consideration as an option for the Reservoir land area. In anticipation of potential reductions in the Reservoir area due to Reservoir boundary changes, accommodating pump station areas, ASR injection well areas, evaporation and other unforeseen conditions 170,000 acre-feet was set as the Reservoir storage volume goal.

Negotiations with the land owners for land that could become part of the Reservoir have been an ongoing process. In an effort to build flexibility into the Reservoir shape, the design team evaluated the storage of the Basic Rectangle with the Paul Acquisition but without the 567 acres of land from parcels I and I1 at the southwest corner of the project and without the 230 acres of land from parcel K at the southeast corner of the project. With everything remaining equal (water surface elevations and dam elevations) the analysis indicated a reduction in storage of approximately 12,000 acre-feet. It was estimated that this loss in storage could be regained by raising the water elevation by approximately 12 inches.

During this process, it was determined that a Perimeter Canal was needed to encompass the Reservoir on the south, east and north sides. This Perimeter Canal could serve as a collector for a pump station at the southeast corner of the Reservoir. The addition of the Perimeter Canal made a pump station at the southeast corner of the Reservoir attractive. In a meeting with the USFWS it was agreed that the Duda parcel K at the southeast corner of the property had numerous environmental issues whose resolutions would have negatively impacted the schedule of the project. Based on that meeting Stanley was directed to eliminate the potential Duda parcel (parcel K) at the southeast corner of the project. Additionally, to minimize costs, parcel I1 was dropped from consideration to eliminate any costly impacts to the existing Duda pump station.

(e) Summary

It was determined that the Basic Rectangle, consisting of parcels A, B, C, D1 and I, would become one alternative land area and the Basic Rectangle parcels plus the additional Paul property, parcels D and E1 would become the second alternative land area. It was agreed that these two land area alternatives would be further evaluated in terms of cell configurations, water surface elevations and earthwork volumes. The water storage volume of 170,000 acre-feet was the goal for each of the options.

Appendix **Section 5** contains the calculations for each of the land area scenarios. After several iterations using different cell configurations, water surface elevations and top of dam elevations the most efficient choices were the two-cell, constant water surface elevation which has become Alternative A; and the three-cell, constant water surface elevation which has become Alternative B.

The Alternative A two cell/constant water elevation configuration resulted in an average water depth of 17.4 ft., an average dam height of 27.3 ft. and a dam embankment requirement of approximately 11,145,000 cubic yards of material. The Alternative B three-cell/constant water elevation configuration resulted in an average water depth of 14.3 ft., an average dam height of 23.4 ft. and a dam embankment requirement of approximately 11,165,000 cubic yards of material. It should be noted that the dam embankment quantities were developed to provide a tool for comparing alternative options. With the options narrowed a more refined analysis of the dam component quantities was made. Refer to **Section 22** for discussion on the dam component quantities.

Figures 12B-09 through 12B-41 in **Section 12** of the Appendices are "Cartoon" type drawings that present the elements of Alternatives A and B. The figures include dam cross sections for each alternative option.

3) Dam Geometry

(a) General

The Reservoir is intended to store water above ground. It is considered to be a "major impoundment" in accordance with the SFWMD guidelines. The levees or dams that will impound the water are considered to be and will be designed as earth dams. The dam cross section consists of three major components, the height, the crest width and the angle of the dam slope. Refer to **Section 8** and **Section 10** for in

depth geotechnical discussions regarding the dam components and their design.

(b) Dam Height

The height of the perimeter dam will be determined by the maximum water depth within the impounded cell plus the required freeboard. The freeboard requirements will also consider the post construction dam settlement. The average dam height for Alternative A is 27.3 ft. (average of Cells 1 and 2) and the average dam height for Alternative B is 23.4 ft. (average of Cells 1, 2 and 3). A discussion on freeboard requirements can be found in **Section 5**. The settlement is estimated to be less than 6 inches.

(c) Crest Width

The Natural Resources Conservation Service (NRCS) formula for determining the required dam top width is:

Crest Width (ft) = z/5 + 10 (where z= dam height)

Based on the average dam heights reported above the calculated crest width for Alternative A is 15.5 ft. and for Alternative B the calculated width is 14.68 ft. At the May 7, 2004 kick off meeting, attended by representatives from the SFWMD, it was agreed that the dam top width would be 14 ft. However, to simplify the alternative comparisons a dam width of 15 ft. was agreed to by the design team based upon embankment stability and seepage considerations. The interior slopes will be faced with a soil-cement protection. The soil-cement protection will not be an additional layer on the dam slope; therefore, it will not add more width to the dam. A discussion of the dam components and the design can be found in **Section 8**. Detailed typical sections for the Reservoir perimeter and interior dams are illustrated in **Figures 12B-42 and 12B-43** in **Section 12** of the Appendices.

(d) Angle of the Dam Slope

The SFWMD recommends that vegetated earth slopes not be steeper that 2.5 Horizontal: 1 Vertical (2.5H:1V) for external slopes and 3H:1V for interior slopes. At the May 7, 2004 kick off meeting, it was agreed that both the interior and exterior slopes would be 3H:1V with adequate slope protection provided. A bench on the exterior side of the dam will be provided for stability and the subsurface drainage blanket. The bench will have a minimum width of 30 ft.

4) Canals

(a) Canals are a key element to the success of this project. The canal construction will include new canals and improvements to existing canals. Side slopes of all new and improved canals above their normal water levels will be vegetated and will not be steeper than 3H:1V. The canal side slopes below their normal water levels will be graded at 2H:1V. Ground water analysis will be done to evaluate the impacts of the proposed canal improvements to the surrounding developments.

(i) Townsend Canal

Water from the Townsend Canal is the primary source of water for the Reservoir. Based on the Pre-Final results of the hydraulic models, the existing canal between the S.R. 80 bridge and the northwest corner of the Reservoir will need some improvements to accommodate the pumping requirements of PS-1. Additional improvements under and around the S.R. 80 bridges may be required to improve the flows from the Townsend Canal to the Reservoir. A new channel will be created between the Townsend Canal and the northwest corner of the Reservoir where PS-1 will be located. This new channel will provide the conveyance from the Townsend to PS-1. Improvements south of this point are not anticipated unless the demolition of the existing pump station that supplies water from the Townsend to the existing LPDD Canal requires modifications to the Townsend.

(ii) Roberts Canal

Water from the Roberts Canal will be routed around the Reservoir in the Perimeter Canal. The improvements to the Roberts Canal will be limited to the grading required for creating the connection to the Perimeter Canal. Under both Alternatives A and B, Option 2, flows from the Roberts Canal will be a secondary source of water for the Reservoir.

(iii) Perimeter Seepage Canal

The Perimeter Canal will be a new canal that (as discussed earlier) will serve several purposes including intercepting and collecting seepage from the Reservoir, collecting and conveying ground water, conveying water around the Reservoir, supplying water to the existing tributaries north of the Reservoir, mitigating existing flooding conditions and collecting flows from the Reservoir. In Option 2 of both Alternatives, PS-2 will lift the water from the Perimeter Canal into the Reservoir.

(iv) NE Rim Ditch

The NE Rim Ditch currently serves irrigation and drainage needs of the Duda properties to the south. Previously proposed in the Pre-Final BODR, the NE Rim Ditch would be widened and deepened to the north so that it becomes part of the Perimeter Canal. The improvements to the Rim Ditch Canal would be done without adverse impacts to the irrigation and drainage needs of the Duda property to the south. The project team is currently evaluating a design to completely separate Duda's operations from the operations of the C-43 WSR. A separate Perimeter/Seepage Canal would be constructed north and parallel to the NE Rim Ditch Canal. SFWMD is currently negotiating with Duda to determine what portion of parcel "I" (see Figure 12B-03, Volume III of the BODR Appendices) will be purchased. Regardless of the portion of parcel "I" to be purchased, the NE Rim Ditch will be re-connected to the Townsend Canal and the Duda Pump Station will remain intact. It is not anticipated that any modification to Duda's permit will be required.

(v) LPDD Canal

The LPDD Canal is an existing east-west canal that runs from just west of S.R. 29 west to the Townsend Canal. The primary function of the LPDD Canal is irrigation; however, it also provides conveyance of storm water runoff. It is anticipated that the LPDD Canal will remain in its current configuration and will serve as a "low point" for lowering the water surface elevations between the Reservoir cells. With the Reservoir in place there will remain a short portion of the LPDD Canal east of the Reservoir. It is proposed that the remaining LPDD Canal be extended east to S.R. 29 and connect to the new Perimeter Canal at the Reservoir's east side. It is also proposed that a new culvert be constructed under S.R. 29 at the proposed LPDD Canal intersection at S.R. 29. This will allow for continued conveyance of the flows in the LPDD Canal and also provide the opportunity for controlling the water surface elevation in the LPDD Canal east of the Reservoir and reducing the local flooding that is currently experienced east of S.R. 29.

(vi) Fort Simmons, Banana Branch and Crawford Canals

These are existing drainage canals that convey storm water runoff to the C-43 Canal. Construction of the Reservoir will bisect these canals leaving the remaining canals between the Reservoir and the Caloosahatchee without an upstream water

source. To overcome this, the new Perimeter Canal will serve as a means for distributing water to these canals. Control structures at each of these canals will maintain historical flows while mitigating flooding problems. Either option of Alternative A will impact only the Fort Simmons Branch and the Banana Branch and modifications to those canals will be as described above. In the case of Alternative A the Crawford Canal will not be severed from its current water source. However, a water control structure will be located in the canal at its intersection with the LPDD Canal to maintain historical flows.

(vii) West Rim Ditch, Hendry and Townsend Canals

Except for that portion of the Townsend between the C-43 Canal and the Reservoir there are no proposed improvements to these canals.

C. Stormwater Control/Site Drainage

1) During Construction

The size and nature of this project causes concern for managing the stormwater runoff during construction. A conceptual Stormwater Pollution Prevention Plan (SWPPP) will be required as part of the contract documents. The objectives of the SWPPP will be to prevent erosion where construction activities are occurring, prevent pollutants from mixing with stormwater and prevent pollutants from being discharged by trapping them on-site, before they can affect the receiving waters. The contractor(s) will be required to prepare and submit a comprehensive SWPPP that will be tailored to their sequence of construction. The contractor will be provided conceptual plans, guidelines and criteria so that he can prepare detailed drainage plans for all phases and sequences of construction. Unless required by permanent construction, all erosion and sediment control measures and all temporary drainage features will be removed from the project.

2) Permanent Construction

The site grading around the new facilities (pump stations, drainage structures, driveways and parking areas) will include provisions for capturing and treating the runoff before being allowed to discharge into the Reservoir, Perimeter Canal or other existing water bodies.

There will be a number of drainage structures that will be required to convey local surface drainage from driveways, parking lots, pump stations, and possibly

at the larger drainage structures. These systems will be evaluated using the rational method and a 3 year frequency storm for the Hendry County area.

D. Utilities

1) Power lines

(a) FP&L Overhead Transmission Lines

The location chosen for the north dam is now adjacent to, and south of, the FP&L right-of-way. Area transmission lines should not be disturbed by the Reservoir construction and operation.

(b) FP&L Overhead Primary Lines

Existing FP&L medium voltage (MV) overhead primary lines within the project footprint may be impacted by the project as described below. See **Figure 14A-01,** Area Electrical Plan in **Section 14** of the Appendices.

(i) Three-phase MV line along the Townsend Canal from S.R. 80 to the Duda Pump Stations at the southwest corner. This is the normal supply to pump stations and must remain in service during all phases of construction. Depending on how the west end dam is arranged, the southerly two miles may not have to be replaced. The northern one-half mile may be affected by changes to the Townsend Canal from the main pump station north to the C-43 Canal.

(ii) Three-phase MV line along the LPDD Canal and the Bob Paul Canal. This provides back-up to the pump stations along the Townsend Canal from the LaBelle Substation located at S.R. 29. The back-up supply capability must be replaced to maintain reliable supply to the Duda Pump Station at the southwest corner of the Reservoir. A replacement line is proposed along the north dam at the boundary with the FP&L transmission right-of-way.

(iii) Three-phase MV line east from the Townsend Canal for about one-half mile just north of the Duda Pump Station area. This line will be removed in its entirety.

(c) Stanley will coordinate with the utility owners and obtain input from them regarding utility relocation.

(i) Demolition of the existing utilities within the project work areas is understood to be the responsibility of the utility owner.

2) Local Irrigation System

The local irrigation system which consists of pump stations of varying sizes, canals and miles of irrigation pipe and tubing, will serve no purpose in the proposed Reservoir. The pump stations scattered throughout the project as well as the large pump station at the west end of the LPDD Canal will be removed and salvaged by the grove operators. Once construction begins all irrigation appurtenances that are remaining will become the property of the contractor and it will be the contractor's responsibility to dispose of the materials by an approved method.

Section 13 – Mechanical Design

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 13 of the Appendices.

A. Design Criteria

1) Pump Station 1 (PS-1)

   (a) Capacity: 1500 cfs.

   (b) Number of pumps: 2 – 250 cfs; 2 – 500 cfs. (Optional, with PS-2: 1-250 cfs; 2-500 cfs).

   (c) Design total dynamic head: 34.8 ft.

   (d) Pump arrangement: Vertical, mixed flow.

   (e) Pump intake: Suction bell.

   (f) Pump driver: Engine-driven with right-angle gear drive.

   (g) Engine fuel supply: Natural gas.

   (h) Discharge Arrangement: In-the-Embankment with Check Valves. See **Figures 13B-01 and 13B-02** in **Section 13** of the Appendices. The discharge chambers as shown allow maintenance of individual check valves. Head losses could be reduced through more careful hydraulic design and special discharge studies will be included into the model pump testing. Access walkways will be provided for discharge gates.

   (i) Check valves: Duckbill style check valve (Tideflex).

   (j) Trash racks: Bar racks with picker style traveling rake.

2) Pump Station 2 (PS-2) (optional)

   (a) Capacity: 500 cfs.

   (b) Number of Pumps: 2 – 250 cfs.

   (c) Design total dynamic head: 18.5 ft.

   (d) Pump intake: Suction bell.

(e) Pump arrangement: Vertical, mixed flow.

(f) Pump driver: Constant speed electric motors with soft start motor starters.

(g) Discharge Arrangement: In-the-Embankment with Check Valves. See **Figures 13B-07 and 13B-08** in **Section 13** of the Appendices show the arrangement with diesel engines.

(h) Check valves: Duckbill style check valve (Tideflex).

(i) Trash racks: Bar racks with picker style traveling rake.

3) Major Equipment

(a) Trash Rack

A trash rack is provided in each pump suction channel. Design headloss used for the pump design point will be 6 inches. Pump operation will be verified at maximum headloss of 12 inches based on the June 10, 2004 meeting with the SFWMD staff. Pump operation will also be verified at minimum headloss of 3 inches. Five feet of headloss will be used for structural calculations.

The trash removal system will use a rail mounted picker rake that can access each trash rack. Two rakes are provided and operate from either end for redundancy. The trash is disposed at a dump station located at either end of the rail. The disposal can be to a dumpster or to a truck.

The Bureau of Protected Species Management of the Florida Fish and Wildlife Conservation Commission must approve each specific project for compliance with manatee protection. Manatee protection was provided on past projects by using a temporary manatee barrier and manatee observer during construction; limiting rake descent speed to within acceptable limits; limiting gripper opening width; adding a solid bar to the front tines of the rake; stopping rake on automatic operations 3 ft above the bottom elevation; and using a manatee observer during manual rake operations that descend within 3 ft. of the bottom.

(b) Mixed-Flow Pumps

The mixed-flow pumps will be lubricated with softened groundwater provided from wells located on site. Enclosing tubes will separate the pump lubrication water from the river water being pumped. The mixed-flow pumps will have stainless steel impellers and shafts. Pump casing, column, discharge elbow, and shaft enclosing tube will be carbon steel construction.

(c) Engines

1. Drainage pumps shall be driven by engine drives suitable for continuous service. Performance, materials, and workmanship shall be in accordance with industry standard practices. The engine shall be a current production model with all auxiliaries normally furnished. The engine horsepower shall not be less than 110% of horsepower required for drainage pump operation, including all driveline losses. The engine shall be a vertical inline or v-type cylinder arrangement. The engine shall be naturally aspirated, supercharged, or turbocharged (with or without intercoolers). The engine shall be designed for and equipped with necessary components for compressed air starting. The engine shall include a PLC based control system and instrumentation that will interface with the station computer and software. Power shall be transferred to the pumps via a right angle gear reducer.

2. The engine shall include all necessary equipment for forced lubrication, including pumps, reservoirs, distribution system, and heat exchangers. The engine shall include a closed cycle cooling system for jacket water cooling. The cooling system shall include all necessary equipment, including a cooling water pump, expansion tank, and heat exchanger. Heat exchangers for jacket water, bypass fuel, and lube oil shall be designed to reject heat to a once-through river water cooling system. Heat exchangers shall meet the design requirements of HEI. Auxiliary components not mounted directly to engine shall be mounted on a single auxiliary skid.

3. Diesel engines shall be designed to operate on diesel fuel conforming to ASTM D 975. The fuel system shall include all necessary fuel system components, including a fuel transfer pump, injector pumps, a bypass system, and fuel filter, as necessary. The diesel fuel system associated with the engine drive pumps will be monitored and controlled through the pump manufacturer's package PLC System as a stand-alone system. All engine driven pump PLC packages will interface with the C-43 SCADA System. All control logic will however reside within packaged PLC©. These requirements are defined within the SFWMD Major Pumping Station Engineering Guidelines, date November 29, 2004.

   The bypass system shall include a heat exchanger for cooling recirculated fuel. The design of containment systems for all day

fuel tanks will be addressed in the 30% preliminary design phase and will comply with applicable codes and standards.

4. Natural Gas engine fuel train shall be design to NFPA standards

5. Diesel Engine Exhaust Emissions: Diesel engine shall be designed to minimize the discharge of gaseous exhaust pollutants to the atmosphere, and to comply with the Title V requirements of the Clean Air Act Amendments of 1990 for all operating ranges.

6. Maximum fuel consumption should be based on the required horsepower to drive the pumps not the rated horsepower of the engine:

   - 7908 BTU/bhp-hr for 1340 hp or greater
   - 7356 BTU/bhp-hr for 400-1339 hp

   Assumed Higher Heating Values (HHV) for fuels examined were 133,000 BTU/gal for diesel fuel and 1,000 BTU/scf for natural gas.

7. Engine exhaust shall include a stack and silencer, installed in accordance with the SFWMD Standard Mechanical Detail Drawing M-8. The stack outlet shall include a rain cap to prevent rain water from entering the stack. Necessary provisions shall be included for draining water in the exhaust system.

(d) Right Angle Gear Reducer:

A right angle gear reducer shall be provided for transferring power from the engine drives to the drainage pumps. The gear reducer shall be designed in accordance with AGMA 11 with a minimum service factor of 2. Bearings shall be rated for an L-10 life of 100,000 hours. The gear reducer shall be equipped with a self contained forced oil lubrication system, including a primary mechanically driven oil pump, an auxiliary electrically driven oil pump, and oil cooler. The oil cooler shall reject heat to a once-through river water cooling system. The oil cooler shall be designed in accordance with HEI. All auxiliaries shall be mounted directly to the gear reducer. The gear reducer shall include an anti-rotative device. The minimum transmission efficiency shall be 97%. A minimum tooth hardness of Rockwell 58 shall be required.

(e) Diesel Fuel System (optional):

1. Provide a fuel system suitable for unloading, storage, and transfer of supply and waste fuel. Provide all necessary storage tanks, pumps, piping, valves, controls and accessories. Exterior tanks and pumps shall be located in a concrete basin designed in accordance with state and local codes.

2. Fuel containment and monitoring shall be provided inside and outside of the station.

3. The diesel fuel oil system shall meet the requirements of the Fire Protection Code and Florida Department of Environmental Protection for fuel oil storage over water.

4. Fuel supply storage system shall have a minimum capacity of 10 days storage, based on full load operation of all drainage water pumps and emergency generators. A minimum of two storage tanks shall be provided. Storage tanks shall be aboveground double wall type and shall be designed and constructed in accordance with applicable industry codes, including API, UL, and NFPA, including applicable building set backs. Tanks shall be provided with level detection, leak detection, and overflow prevention devices.

5. Provide a packaged fuel day tank for each prime mover. The day tanks shall be provided with all appurtenances, controls, and instrumentation for level detection and automatic refilling. Day tank storage capacity shall be based on a minimum of 4 hours storage, based on full load fuel consumption of the prime mover. Regulations regarding the maximum allowable fuel storage inside the station will be considered when sizing the day tanks. Day tanks shall have an integral rupture basin with leak detection. Each day tank shall be provided with appropriate connections, including fill, drain, vent, overflow, supply, and return. The day tanks shall be designed and constructed in accordance with applicable sections of NFPA and UL codes.

6. System shall be designed for truck unloading of fuel and loading of fuel waste. Provide all necessary storage tanks, pumps, piping, valves, and accessories for unloading fuel and loading fuel waste. System shall be designed to minimize loading and unloading time. Minimum waste fuel oil system storage capacity shall be 150% of the largest day tank capacity.

7. Diesel fuel and waste fuel pumps shall be self-priming, positive displacement type.

8. Pumps shall be motor-driven and equipped with an integral internal relief valve. Provide 100% redundant pumping capacity on the diesel fuel supply transfer system from the main storage tanks to the day tanks.

9. Diesel fuel piping shall be ASTM A53 or A106 black steel piping. Minimum pipe wall thickness shall be based on ASME B31.3. Fuel piping and fittings shall be butt welded or socket welded. Butt weld fittings shall be in accordance with ASME B16.9 and socket weld fittings shall be in accordance with ASME B16.11. Valve construction and class shall be in accordance with ASTM B16.34. Underground piping shall be secondarily contained in a fiberglass reinforced plastic containment system, designed in accordance with NFPA 30. Containment piping shall be capable of withstanding H-20 highway loading, as defined by AASHTO HB-16.

(f) Lubrication Oil System

1. Provide a lube oil system suitable for unloading, storage, and transfer of supply and waste lube oil. Provide all necessary storage tanks, pumps, piping, valves, controls, and accessories.

2. Lube oil and waste lube oil storage systems shall have a minimum capacity of 30 days storage, based on equipment manufacturer recommended oil change capacity and intervals. Storage tanks shall be aboveground single wall type and shall be designed and constructed in accordance with applicable industry codes, including API and UL. Tanks shall be provided with level detection and overflow prevention devices.

3. System shall be designed for truck unloading of lube oil and loading of lube oil waste. Provide all necessary storage tanks, pumps, piping, valves, and accessories for unloading lube oil and loading lube oil waste. System shall be designed to minimize loading and unloading time.

4. Lube oil and waste lube oil pumps shall be self-priming, positive displacement type.

5. Pumps shall be motor-driven and equipped with an integral internal relief valve.

6. Lube oil piping shall be ASTM A53 or A106 black steel piping. Minimum pipe wall thickness shall be based on ASME B31.3. Lube oil piping and fittings shall be butt welded or socket welded. Butt weld fittings shall be in accordance with ASME

B16.9 and socket weld fittings shall be in accordance with ASME B16.11. Valve construction and class shall be in accordance with ASTM B16.34. Underground piping shall be secondarily contained in a fiberglass reinforced plastic containment system. Containment piping shall be capable of withstanding H-20 highway loading, as defined by AASHTO HB-16.

(g) Cooling System

1. Provide a once-through cooling water system for cooling engine jacket water, engine lube oil, recirculated fuel, and gear reducer lube oil. System shall include all necessary equipment, including heat exchangers, filters, pumps, piping, valves, instrumentation, controls, and accessories.

2. The cooling system shall be designed to pump river water through the various heat exchangers and return water to the basin reservoir. River water pumps shall be motor-driven, vertical type pumps. Each drainage pump, drive, and gear reducer shall have an associated cooling water pump. One additional backup cooling water pump shall be provided for redundancy. Provide self-cleaning strainers on the discharge of each cooling water pump, designed to remove suspended solids from the cooling water. Self-cleaning strainer cleaning cycle shall be initiated by high differential pressure or field adjustable timer setting.

3. All cooling water piping shall be ASTM A53 or A106 black steel piping. Minimum pipe wall thickness shall be based on ASME B31.3. Fittings shall be in accordance with ASME B16.9 and B16.11. Valve construction and class shall be in accordance with ASTM B16.34.

(h) Compressed Air System

1. Provide a compressed air system suitable for air starting drainage pump engine drives, instrument air, and service air. Compressed air system shall include all necessary equipment and accessories, including compressors, receivers, dryers, filters, motors, piping, instrumentation, and controls.

2. Provide a minimum of two duplex reciprocating air compressors. Air compressors shall be air cooled and oil lubricated. Compressors shall be motor-driven with the motor and compressor mounted on a common baseplate. Compressor shall utilize a belt drive with a spring-loaded tensioner. Discharge pressure shall be as required for engine starting with a minimum

pressure of 150 psig. Accessories shall include oil filter, air filter, silencers, and vibration isolators.

3. Each drainage pump engine drive shall have an associated receiver for air starting. Provide an additional receiver for instrument and service air. Receivers shall be ASME code stamped vessels and shall include a pressure relief valve and automatic condensate drain.

4. Provide pressure regulators for instrument air and service air. Provide refrigerated dryer for instrument air suitable for continuous operation. Dryer shall be equipped with inlet and outlet filters and an automatic condensate drain.

5. Compressed air piping shall be ASTM A53 or A106 black steel piping. Minimum pipe wall thickness shall be based on ASME B31.3. Fittings shall be in accordance with ASME B16.9 and B16.11. Valve construction and class shall be in accordance with ASTM B16.34.

(i) Vertical Gates

Enclosed grease lubricating systems will be provided on submerged moving parts.

(j) Bridge Crane

Remote controlled bridge crane will be sized to lift the heaviest component in the station. Sufficient overhead clearance will be provided so that the pump can be removed. A service platform will be built on top of the control room to bridge crane.

B. Pump Station Discussion and Alternatives Comparison

1) Overview

The primary pump station (PS-1) will be located at the northwest corner of the Reservoir. This location is closest to the C-43 Canal and requires the shortest length of Townsend Canal widening. PS-1 will have a capacity of 1,500 cfs. One additional pump station (PS-2) may be located at the southeast corner of the site to manage the water flow from south of the project site. PS-2 (if implemented) would have a capacity of 500 cfs while reducing the required capacity of PS-1 to 1,250 cfs.

Three different discharge arrangements have been considered for PS-1:

- **Figures 13B-01 and 13B-02** in **Section 13** of the Appendices show the In-the-Embankment with Check Valve Arrangement.

- **Figures 13B-03 and 13B-04** in **Section 13** of the Appendices show the In-the-Embankment with Siphon Arrangement.

- **Figures 13B-05 and 13B-06** in **Section 13** of the Appendices show the Outside-the-Embankment with Over-the-Embankment Arrangement.

- **Figures 13B-07 and 13B-08** in **Section 13** of the Appendices show the arrangement for PS-2 with diesel engines.

The elevations shown in **Figures 13B-01** through **13B-08** in **Section 13** of the Appendices are approximate and will be finalized following selection of cell configuration and freeboard requirements.

The pump stations consists of a supply canal, suction channels, trash racks, pumps, engine drives, right angle gear reducers, and discharge conduits. Electric motors are recommended in lieu of engine drives and right angle gear reducers at PS-2.

2) Discharge Arrangement

Three discharge arrangements are commonly used on pump stations that must pump over an embankment. The first arrangement is illustrated in **Figures 13B-01 and 13B-02** in **Section 13** of the Appendices. The pump station is located in the embankment with discharge pipes and check valves. The second arrangement is illustrated in **Figures 13B-03 and 13B-04** in **Section 13** of the Appendices. The pump station is located in the embankment with a siphon discharge. The third arrangement is illustrated in **Figures 13B-05 and 13B-06** in **Section 13** of the Appendices. The pump station is located at the landside toe of the embankment and utilizes discharge pipes up and over the embankment.

Many design features would remain essentially the same for the three arrangements:

- Suction elevation.

- Trash rake system.

- Pump intake.

- Pump flow capacity and number of pumps.

- Size of the pump station building.

(a) In-The-Embankment with Check Valve Arrangement

The first arrangement utilizes discharge pipes to convey the water from the pumps through a concrete wall to the discharge chamber. The pipes will be circular steel pipe. The concrete wall functions as the Reservoir wall through the pump station. The discharge pipe elevation is near the bottom of the Reservoir and no siphon recovery system is required.

Two means of preventing backflow through the pumps are required for redundancy. The primary means will be check valves. A rubber Tideflex duckbill style check valve is recommended. This valve offers high reliability, the ability to seal itself around debris, low maintenance and long life. The pump station is protected by a trash rack system so debris should not be a major concern. The size of the Tideflex valve is larger than a traditional flap gate so the discharge chamber dimensions will have to be increased to accommodate the valves. Large Tideflex valves are available in the size of the discharge pipe, but are not common. The vendor has been contacted for references and installations using similar sized valves. The references will be contacted to verify performance of valves on existing installations.

As an alternative to a Tideflex duckbill style valve, a cast iron flap gate could be used. This type of traditional flap gate is susceptible to corrosion, trapping debris and sticking open. The flap gate has moving parts that require maintenance and seats that require replacement. A flexible neoprene flap valve is available from Rodney Hunt that reduces problems associated with the flap gate sticking open. The flexible neoprene flap valve would not seal as tight as the duckbill style valve.

An automated gate or a sluice gate has been suggested as an alternate to check valves or flap gates. The operation of a sluice gate would be coordinated with operation of the pump. The gate would be opened in conjunction with or immediately following pump start up. During pump shut down, the gates will have to be closed. Some backflow of water through the pumps will occur during the relatively slow operation of the gates. Sluice gates require motor operators and a control system, which increases cost. A closed sluice gate also creates the potential to deadhead the pump and cause damage.

The duckbill style check valves are recommended for PS-1 and PS-2.

Each discharge chamber will have a vertical sluice gate as a secondary means of dewatering for maintenance of the pump discharge chamber and check valve. A less expensive option would be to use stop logs rather than a motorized gate. The stop logs require less maintenance, but cannot be deployed as quickly. It is recommended that motorized gates be used on the discharge chambers as a means of quickly isolating the pump and check valve.

(b) In-The-Embankment with Siphon Arrangement

The second arrangement is commonly used on the SFWMD large capacity pump stations. Specially formed concrete is used to create a siphon conduit above the high water level and back down to a submerged discharge. The rectangular concrete pipes are gradually expanded to reduce headloss in the system. This provides a reliable method of preventing backflow through the pump station. When the pumps start, a vacuum priming system is used to remove air and establish the siphon. The pumps then operate based on the difference in water surface elevations and do not "see" the high siphon elevation. This results in the lowest energy use. When the pumps stop, air is introduced to the siphon, the siphon is broken, and water cannot backflow.

At the Reservoir site, this arrangement requires a tall structure, as the siphon must be above the high water level. The arrangement requires a submerged outlet to establish the siphon.

(c) Outside-The-Embankment with Over-The-Embankment Arrangement

The third arrangement uses discharge pipes that are routed up and over the embankment. The discharge pipes would be steel with flexible couplings due to settlement that can occur in the embankment. The invert of the pipe is required to be placed at the normal top elevation of the embankment for the project. The pipe is then covered with earth for protection. The buried pipes will require corrosion protection.

The pipes are routed down the water side of the embankment as a siphon recovery method of recovering head. The location of the pipes above the typical Reservoir embankment do not increase the potential pathways for seepage or piping. Either a submerged outlet or an upturned discharge (saxophone discharge) is required to develop a siphon over the top of the embankment. The pumps are required to develop the head necessary to self prime the siphon without a vacuum assist system. A vacuum assist system is recommended for normal operation to reduce the typical operating range of the pump. A vacuum assist system requires a submerged discharge so that adequate water is available to draw up the pipe during vacuum operation.

A siphon breaker and vent is required at the top of the embankment to prevent reverse flow and allow the downside pipe to drain when the pumps are stopped and to bleed air from the pipes while the discharge line is filling. The siphon breaker consists of a valve that opens to allow air to escape during filling, closes to establish a siphon, and opens when the pumps turn off to allow air in, break the siphon, and prevent reverse flow.

(d) Comparison

The selection of pump station arrangement will be driven primarily by cost. However, some non-monetary factors influence the decision making.

Table 13B-01 summarizes the advantages, disadvantages, and other issues of the three discharge arrangements.

Arrangement	Advantages	Disadvantages	Other Issues
In-the-Embankment with Check Valve	• Simplest Construction. • Lower operating floor than siphon arrangement. • Smallest footprint. • No vacuum system required.	• Submerged gates. • Higher energy use than siphon. • Higher operating floor than over-the-embankment. • Debris can interfere with check valve.	• Reservoir embankment stability. • Maintenance of check valves. • Appearance (tall structure). • Flap gate alternatives. • Vertical gate alternatives.
In-the-Embankment with Siphon	• Similar arrangement as other SFWMD pump stations. • Minimizes potential for backflow. • Lowest energy use.	• Complex concrete construction. • Tallest structure. • Requires vacuum system to prime siphon. • Large footprint. • Longest pump column and more pump bearings.	• Appearance (very tall structure). • COE requirement to prime without aid of vacuum system. • Complex construction. • Reservoir embankment stability.
Over-the-Embankment	• Shortest pump station structure. • Shortest pump column and fewest bearings. • No pathways for increased seepage or piping.	• Highest energy requirement. • Extra embankment height over pipes. • Requires vacuum system to prime.	• Corrosion protection of buried pipes. • Discharge must be submerged for vacuum priming. • COE requirement to prime without aid of vacuum system.

Table 13B-01 - Arrangement Alternatives Non-Monetary Comparison

Table 13B-02 shows a cost comparison for the three alternative arrangements. Labor expense is estimated for 4 operators and 1 laborer. Maintenance expense is based on 2% of the mechanical equipment capital cost. A contract service is an estimate of the expense for hiring outside services such as janitorial, general upkeep, etc.

Item	Check Valve	Siphon	Over-the-Embankment
Construction Cost	$25,700,000	$28,600,000	$25,800,000
Annualized Construction Cost (6%, 50 years)	$1,630,000	$1,810,000	$1,640,000
Operating Cost			
Fuel	$881,000	$875,000	$898,000
Lube Oil	$12,000	$12,000	$12,000
Electrical	$49,000	$49,000	$49,000
Labor	$210,000	$210,000	$210,000
Maintenance (2% of Equipment Capital)	$194,000	$196,000	$184,000
Contract Services	$25,000	$25,000	$25,000
Total Annual O&M	$1,371,000	$1,367,000	$1,378,000
Total Annual Cost	$3,000,000	$3,180,000	$3,020,000
Equivalent Present Worth (6%, 50 years)	$47,300,000	$50,100,000	$47,600,000

Table 13B-02 - Pump Station Arrangement Cost Difference Comparison

The cost comparison shows that all three options are within approximately 10% of each other on a construction cost and total annual cost basis. Since the cost comparison does not indicate a clear choice, the non-monetary considerations lead to the recommendation. Based on the relatively short structure height, small footprint, and simple construction, the In-the-Embankment with Check Valves arrangement is recommended.

3) Suction Channel

The suction channel serves as an approach for the pump intake, a location for the trash rack, and a means of isolating and dewatering a pump.

The depth of the suction channel is determined by considering water surface elevation in the supply canal, trash rack headloss, minimum submergence over the pump intake, and minimum vertical clearance between the pump intake and the floor of the sump. Minimum submergence was determined using Hydraulic Institute Standard 9.8 (HI 9.8). Minimum submergence may be adjusted based on the physical model test that will be conducted.

Table 13B-03 shows the depth dimensions for PS-1 and PS-2 for both a suction bell and a Formed Suction Intake (FSI). Using HI 9.8, there is less than 1 ft difference between the suction bell and FSI depth.

	PS-1, Suction Bell	PS-1, FSI	PS-2, Suction Bell	PS-2, FSI
Supply canal, minimum water surface elevation, ft.	0.1	0.1	16.5	16.5
Trash rack, maximum headloss, ft.	1.0	1.0	1.0	1.0
Minimum pump submergence, ft.	18.2	19.5	15.0	15.0
Vertical clearance below pump intake, ft.	4.0	2.2	3.0	2.1
Suction channel floor elevation, ft.	-23.1	-22.6	-2.5	-1.6

Table 13B-03 - PS-1 and PS-2 Suction Channel Depth Comparison

The width of the suction channel is determined by looking at three variables: maximum velocity in the channel, equipment spacing in the pump station, and pump intake width. The design approach velocity will be 1.5 ft/s maximum in accordance with Hydraulic Institute Standard 9.8 (HI 9.8). Proper equipment spacing requires access around the engines and gear drives for operation and maintenance. Pump intake width dimensions are 2 times the bell diameter for a suction bell or the width of the Formed Suction Intake.

Table 13B-04 shows the width dimensions for PS-1 and PS-2. A channel wall thickness of 2 ft. was used to compare channel widths with required equipment spacing. In each case, the controlling dimension is based on equipment spacing.

Basis of channel width	PS-1, 500 cfs	PS-1, 250 cfs	PS-2, 300 cfs
Maximum velocity + channel wall	17’	10’	13’
Equipment spacing	30’	20’	20’
2 x suction bell diameter + channel wall	22’	15’	18’
FSI width + channel wall	14’	11’	13’
Controlling dimension	30’	20’	20’

Table 13B-04 - PS-1 and PS-2 Suction Channel Width Comparison

The length of the suction channel is determined by considering service bridge width, HI 9.8 dimensional requirements for suction bell intakes, and length of FSI. The SFWMD recommends a minimum of 20’ width for the service bridge to adequately service the trash racks and allow use of their mobile cranes. HI 9.8 requires 5 suction bell diameters between the trash rack and the centerline of the pump. The FSI length is based on 3.3 characteristic diameters for the FSI and an estimated 2 characteristic diameters of open channel between the FSI and the trash rack. It should be noted that the FSI characteristic diameter recommended by the pump manufacturers is much smaller than an equivalent suction bell diameter.

Table 13B-05 shows the length dimensions for PS-1 and PS-2. The overall length based on a FSI will be shorter than the length based on a suction bell intake. The dimensions for PS-1 are based on the 500 cfs pump. The same length will be used for the 250 cfs pumps.

Basis of channel length	PS-1	PS-2
5 x suction bell diameter	50’	38’
3.3 x FSI diameter plus open channel of 2 x FSI diameter (estimated)	26’	25’
Service bridge width plus minimum pump spacing from outside wall of pump station	35’	35’
Controlling length, FSI	35’	35’
Controlling length, suction bell	50’	50’

Table 13B-05 - PS-1 and PS-2 Suction Channel Length Comparison

4) Pump Intake

Two types of pump intakes have been considered: suction bell intake and Formed Suction Intake (FSI). The FSI is generally thought to reduce the minimum submergence and minimize pump station excavation and associated shoring and dewatering. This may prove to be true through model testing of the intake. At this point in the design, following HI 9.8 standards, there is very little difference between the calculated submergence of the two intake types. As shown in **Table 13B-03**, there is less than 1 ft of difference in suction channel floor elevation at PS-1 and PS-2.

Suction channel length required is different for the two intake types. As shown in **Table 13B-05**, there may be a 15 ft difference at PS-1 and a 3 ft difference at PS-2.

Construction costs are different for the two intake types. Assuming either the suction bell or FSI is provided by the pump manufacturer, the suction bell has a lower purchase price. The suction bell requires forming of concrete anti-rotation fillets and baffles. The FSI requires concrete cast around the intake.

Headloss and energy costs are different for the two intake types. The headloss through the FSI is based on a friction factor of K=0.15. The headloss through the suction bell is based on a friction factor of K=0.04.

Table 13B-06 summarizes the advantages and disadvantages of the intake types.

	Advantages	Disadvantages
Formed Suction Intake (FSI)	- Shorter length for approach channel. - Possible lower submergence requirement after model testing.	- Difficult concrete forming. - Higher headloss. - Higher energy costs.
Bell Intake	- Simpler construction. - Lower headloss. - Lower energy costs.	- Possible deeper suction bays. - Forming of anti-rotation baffles and fillets. - Longer length of approach channel.

Table 13B-06 - Intake Non-Monetary Comparison

Table 13B-07 summarizes the cost differences between the two intake types for the entire pump station (all pumps). Energy cost is based on the headloss through the intake using an average annual pumping rate. Construction cost captures only captures the differences between the two options and does not cover the cost to construct the entire intake structure. Further construction cost savings may be realized if model testing allows reducing the sump depth. On the basis of lower construction cost and operating costs, the suction bell intake is recommended at PS-1 and PS-2.

	PS-1	PS-2
Formed Suction Intake (FSI)		
Annual Energy Cost	$7,300	$1,600
Construction Cost	$440,000	$208,000
Annualized Construction Cost	$28,100	$12,000
Total Annual Cost	$35,400	$13,200
Suction Bell		
Annual Energy Cost	$1,900	$400
Construction Cost	$405,000	$127,000
Annualized Construction Cost	$25,700	$8,000
Total Annual Cost	$27,700	$8,500

Table 13-07 - Intake Cost Comparison

5) Pumps

Based on the latest hydrologic modeling, pump station capacity at PS-1 will be 1,500 cfs. The pump station will consist of 2 pumps of 250 cfs and 2 pumps of 500 cfs. Pumps will be vertical mixed-flow pumps. No redundant pumps will be provided for the mixed flow pumps. Redundant pumps will be provided for auxiliary pumps (cooling, lube, potable water).

Tables 13B-08 and 13B-09 summarize pumping conditions.

	Design	**Minimum pumping head (high suction pool, low reservoir pool)**	**Maximum pumping head (low suction pool, high reservoir pool)**
Townsend Canal water surface elevation, ft.	1.2	3.3	0.1
Trash rack headloss, ft.	0.5	0.3	1.0
Pump Suction elevation, ft.	0.7	3.0	-0.9
Reservoir pool elevation, ft.	32.5	17.0	37.4
Static head, ft.	31.8	14.0	38.3
System headloss, ft.	3.0	3.0	3.0
TDH, ft	34.8	17.0	41.3

Table 13B-08 - PS-1 Pumping Conditions

	Design Point	**Minimum pumping head (high suction pool, low reservoir pool)**	**Maximum pumping head (low suction pool, high reservoir pool)**
Perimeter Canal water surface elevation, ft.	16.5	20.0	16.5
Trash rack headloss, ft.	0.5	0.3	1.0
Pump Suction elevation, ft.	16.0	19.7	15.5
Reservoir pool elevation, ft.	32.5	26.0	37.4
Static head, ft.	16.5	6.3	21.9
System headloss, ft.	2.0	2.0	2.0
TDH, ft	18.5	8.3	23.9

Table 13B-09 - PS-2 Pumping Conditions

The Reservoir water surface elevation will vary widely over the course of the year. During the wet season, the pump station will fill the Reservoir from near empty to full. This represents a range in elevation of 20 ft. This wide range in elevation translates into a wide range of static head that the pump could operate under. The concern with the range of head is finding a pump that can reliably operate over the range. Included in this evaluation is the system headloss i.e., the dynamic headloss associated with the intake, pump column, pump discharge elbow, discharge pipe and check valves. Axial and mixed flow pumps have unsteady regions in their curves at higher heads and lower flows. Axial and mixed flow pumps also develop high net positive suction head required (NPSHR) at high flows and low head (runout condition).

With the recommended discharge arrangement shown in **Figure 13B-01** in **Section 13** of the Appendices, the maximum head that can be developed on the pump is limited by the top elevation of the discharge chamber. Even if the pump operates against a closed vertical gate, water will overflow the discharge chamber. This is an inherent safety that prevents the pump from operating at too high of head.

6) Net Positive Suction Head

Pump manufacturers have been consulted to confirm that the proposed pumping range can be handled with their pump designs. Initial feedback has been that the proposed operating points for design, minimum, and maximum head conditions should not be a concern as long as NPSHR is met.

Net positive suction head available (NPSHA) is 48 ft at PS-1, based on the HI 9.8 requirements for submergence. It is anticipated that model testing will allow the suction channel floor elevation to be raised slightly. While this will reduce construction cost, it will also reduce NPSHA and limit the range over which the pump should operate. If the sump floor elevation is raised to the level allowed by Corps guidance for a FSI, NPSHA would drop to 36 ft.

Table 13B-10 shows the minimum Total Dynamic Head (TDH) to limit NPSHR for pump curves obtained from three different pump manufacturers. The limit is based on keeping NPSHR to 3 ft. less than NPSHA.

Manufacturer	**NPSHA = 48'**	**NPSHA=36'**
Flowserve	15'	30'
Fairbanks Morse	22'	28'
ITT-AC	18'	22'

Table 13B-10. Minimum TDH to limit NPSHR

There are several options available to limit NPSHR: variable pitch impellers, variable speed pumps, and raising the discharge elevation of the pump discharge.

(a) Variable Pitch Impellers

The first option is variable pitch impellers. Variable pitch impellers require additional mechanical equipment to vary the angle of the impellers. This represents an additional maintenance item for each pump. Variable pitch impellers also reduce the number of manufacturers that can bid, as not all pump manufacturers build variable pitch impellers. For these reasons, variable pitch impellers are not recommended.

(b) Variable Speed Pumps

The second option is variable speed pumps. The speed of the pump is varied to better match the pump performance with the system head curve. There is inherent speed variation available with an engine drive, typically 80-100% of full speed. Speed variation can also be accomplished with a hydraulic speed reducer. However, a hydraulic speed reducer introduces an additional maintenance item and reduces the reliability of the system.

Figure 13B-09 in **Section 13** of the Appendices shows a family of pump curves at PS-1 adjusted for 80%, 90%, and 100% of full speed. The system head curve is also shown for minimum and maximum static head conditions at PS-1. With speed variation, the entire range of static head can be met. At full speed, the runout portion of the curve may not be available due to high NPSHR, as previously discussed.

At 90% speed, the low static condition can be reached. At the lower speed, NPSHR drops and may not present a problem. The high static condition can also be met, but the efficiency drops off significantly, and the capacity is much less than 500 cfs.

At 80% speed, the low static condition can be met, and NPSHR should not be a problem. The capacity at low static condition is close to design capacity of 500 cfs. However, the high static condition cannot be met. Operating the pump under the high static condition at 80% speed may result in damage to the pump.

Figure 13B-10 in **Section 13** of the Appendices shows a family of pump curves at PS-2 adjusted for 80%, 90%, and 100% of full speed. This pump curve is shown as an example only and is not the final pump selection for PS-2. The system head curve is also shown for minimum and maximum static head conditions at PS-2. With speed variation, the entire range of static head can be met.

If speed variation is used to meet the wide range of static head, there are several dangers. There is a danger that if the pump is operated at a lower speed when the Reservoir is full, the pump could be forced back into the unstable region of its curve and damage the pump. There is a second danger that if the pump is operated at full speed when the Reservoir is empty, the pump could cavitate due to high NPSHR. These dangers could be mitigated by operator training and alarms or safeties built into the control system. Alarms would be based on a comparison of Reservoir and suction levels and pump speed. The alarm points would be based on model testing results where cavitation begins plus a safety margin that would be added.

(c) Pump Discharge Elevation

The third option to deal with the wide range of static head is to raise the pump discharge elevation. This effectively reduces the range over which the pump is required to operate. When the Reservoir level is at or below the pump discharge elevation, the pump operates at the static head associated with the discharge elevation. In the extreme case, the discharge elevation could be raised to maximum pool elevation and the pump would always operate at the same point. The drawback to raising the discharge elevation is that energy is wasted while the Reservoir level

is below the pump discharge elevation. The benefit of this arrangement is that it is a simple, reliable system to operate.

Figure 13B-11 in **Section 13** of the Appendices illustrates how raising the pump discharge limits the NPSHR.

(d) Recommended Approach

The recommended approach is to raise the discharge elevation to limit the NPSHR and prevent cavitation at full speed. This approach allows the pump to be operated at full speed over the entire range of Reservoir levels. It is a fail-safe approach that does not depend on mechanical equipment, operator input, or control systems. The discharge elevation that meets this requirement varies between manufacturers and depends on NPSHA. The final pump discharge elevation will be determined following bidding, model testing, and determination of NPSHA.

In addition to raising the discharge elevation, speed variation will also be available for the engine driven pumps. The speed variation can be used by operators to make modest adjustments to pump capacity and improve the pump efficiency for a given Reservoir level. The minimum allowable speed of the pump should be set in the control system to prevent the pump from operating in an unstable region of the curve.

Electric motors with VFDs could also provide speed adjustment to deal with varying head, however, the extra cost of VFDs is not justified because the range of head at PS-2 was much smaller due to higher ground elevation. With a smaller range of head, it is possible to select a pump that can meet the full range of head conditions without varying speed. For PS-2, with the electric motor option, no speed variation will be used. The discharge elevation will be raised as required to ensure that the pump can operate under the full range of head conditions at full speed.

7) Startup

The pumps and right-angle gear drives will be direct coupled to the engine shaft; no clutching arrangement will be used. Engines are typically started and allowed to warm up at 50% speed for a period of 3-5 minutes. Flowserve says that this is typical and is not a cause for concern with the pump. At 50% speed, the pump will not be moving water under the range of head at PS-1. The engines will be operating under a relatively small load, which will not cause problems with the engine, right-angle gear drive, or the pump.

8) Testing

(a) Model and Intake Test

Physical modeling of the pumps and intake conditions will be required. Each pump station and each pump size will be modeled. Results of the modeling will be used to finalize the suction channel floor elevation, pump elevation, and discharge elevation.

(b) Field Operating Vibration Test

A separate equipment vibration specification section will be developed that details the test method and sets allowable vibration limits. This provision is used to confirm that the SFWMD obtains high quality equipment.

9) Other Pump Issues

The pumps will be lubricated with water in an enclosing tube. The source of lubrication water will be groundwater wells. The groundwater will be softened.

The SFWMD technical specification for axial flow pumps and the Corps guide specification for mixed flow pumps will be used to develop the pump specification.

10) Engines

(a) Dual Fuel Engines

The use of dual fuel (95% natural gas and 5% diesel pilot) engine drivers combines many of the advantages of both diesel and natural gas engine drives since the engine is started as a diesel and switches over to natural gas at higher loads. A small amount of diesel fuel is always required since it operates as a "liquid spark plug". A dual fuel engine is also capable of operating at full load on diesel fuel. A dual fuel engine would offer the fuel supply reliability of a diesel fuel engine with the potential fuel cost savings of a natural gas engine. Fairbanks Morse and Wartsila are the only two manufacturers that currently offer a dual fuel engine in the size range required.

During discussions with a Fairbanks Morse (FM) Representative it was learned that Fairbanks Morse does not recommend dual fuel engines for pumping applications. The load curve associated with a pumping application does not match up well with the operation characteristics of their dual fuel engine. FM stated that they would typically recommend dual fuel engines for electrical generation applications. Wartsila

declined to participate. For these reasons dual fuel engine drivers will not be considered a viable option.

(b) PS -1 Pump Driver Analysis

Two different pump driver systems were evaluated for PS-1: diesel, and natural gas (spark ignited).

Table 13B-11 summarizes the advantages and disadvantages of each engine type. Comments from the Draft DDR requested that on-site natural gas storage be considered. On-site natural gas storage was determined to be infeasible due to the large tank volumes required to store gas at low pressure; the lack of high pressure gas in the vicinity; and the undesirability of operating and maintaining gas compressors at the site.

	Advantages	Disadvantages
Diesel Engines	• Maintenance and operations personnel are already familiar with this type of pump driver. • Spare parts and consumables may be common with other SFWMD pump stations if same manufacturer of pump driver is selected. • On-site diesel storage supply is the most reliable source of fuel for the pump driver. • Diesel engine pump drivers are a mature and proven technology. • Generally more efficient than a natural gas engine.	• Diesel engine pollution emissions are greater than a natural gas engine. • Creates the potential for diesel fuel release to the environment. • Fuel costs are potentially higher than a natural gas engine.
Natural Gas Engines	• Natural gas engine pollution emissions are less than a diesel engine. • Fuel costs are potentially lower than a diesel engine. • Eliminates the need to store fuel oil onsite. • Maintenance costs are less than diesel engine.	• Maintenance and operations personnel are not as familiar with this type of pump driver. • Natural gas engine fuel supply is less reliable than onsite fuel oil storage. Note: The storage of natural gas is possible if there is access to a high pressure gas line. Initial conversations with the local gas company indicate that no such line is available. • Due to lower BTU content in natural gas, a larger engine will generally need to be purchased for the same output. • Greater explosion hazard. • Non-competitive pricing (i.e., CAT is the only vendor).

Table 13B-11 - PS-1 Engine Drive Non-Monetary Comparison

Table 13B-12 summarizes a life cycle cost analysis for the two different engine drives. The analysis was done using fuel oil price of $1.50/gallon and natural gas prices varying from $5/MMBTU to $9/MMBTU.

	Diesel	**Natural Gas (NG)**				
NG Cost ($/MMBTU)	--	5	6	7	8	9
Total Capital ($)	6,360,000	8,460,000	8,460,000	8,460,000	8,460,000	8,460,000
Annualized Capital, 50 yrs, 6% ($)	404,000	537,000	537,000	537,000	537,000	537,000
Annual Fuel Cost ($)	881,000	471,000	566,000	660,000	754,000	849,000
Annual Maintenance ($)	64,000	57,000	57,000	57,000	57,000	57,000
Total O & M ($)	945,000	528,000	623,000	717,000	811,000	906,000
Total Cost ($)	1,349,000	1,065,000	1,160,000	1,254,000	1,348,000	1,443,000
NG Savings ($/yr)	--	284,000	189,000	95,000	1,000	-94,000
Energy Cost ($/kWhr)	0.101	0.057	0.067	0.077	0.087	0.097

Table 13B-12 - PS-1 Engine Drive Life Cycle Cost Comparison

As cost analysis indicates, there exists the potential for significant cost savings when natural gas prices are below $8/MMBTU when compared to diesel fuel at $1.50/gal. The local gas company (TECO Energy's Peoples Gas) has indicated willingness to extend a pipeline to PS-1 from a proposed pipeline extension along S.R. 80, therefore the cost of gas supply line was not included in the life cycle cost comparison.

The reliability of natural gas could potentially be an issue. A TECO representative stated that in the last five years gas users were curtailed once, for one day, due to a pipeline rupture. The representative also stated that TECO offers uninterruptible gas service, but typically the higher fuel cost associated with uninterruptible gas service would eliminate the fuel cost benefit over diesel fuel. The interruption of gas supply for several days should not significantly impact the seasonal operation of the pump station. Based on the cost analysis, natural gas engines are recommended at PS-1.

The recommendation for natural gas engines is subject to the following considerations by the SFWMD:

1. More formal discussions should be conducted with TECO to confirm that they will extend a pipeline to PS-1 and commit to a natural gas price within the range to make favorable compared to diesel fuel.

2. Of the two major natural gas engine vendors who have experience in this size (Wartsila and Caterpillar), Wartsila has declined to participate. This leaves Caterpillar as potentially a sole source for the engines.

(c) PS -2 Pump Driver Analysis

Three different pump driver systems were evaluated for PS-2: diesel, natural gas (spark ignited), and electrical.

Table 13B-13 summarizes the advantages and disadvantages of each engine type.

	Advantages	Disadvantages
Diesel Engines	• Maintenance and operations personnel are already familiar with this type of pump driver. • Spare parts and consumables may be common if same manufacturer of pump driver is selected. • On-site diesel storage supply is the most reliable source of fuel for the pump driver. • Diesel engine pump drivers are a mature and proven technology. • Generally more efficient than a natural gas engine.	• Diesel engine pollution emissions are greater than a natural gas engine. • Creates the potential for diesel fuel release to the environment. • Fuel costs are potentially higher than a natural gas engine.
Natural Gas Engines	• Natural gas engine pollution emissions are less than a diesel engine. • Fuel costs are potentially lower than a diesel engine. • Eliminates the need to store fuel oil onsite. • Maintenance costs are less than diesel engine.	• Maintenance and operations personal are not as familiar with this type of pump driver. • Service interruptions (e.g., due to hurricanes) are less prevalent and of a shorter duration than electrical interruptions. • Natural gas engine fuel supply is less reliable than onsite fuel oil storage. Note: The storage of natural gas is possible if there is access to a high pressure gas line. Initial conversations with the local gas company indicate that is no such line is available. • Due to lower BTU content in natural gas, a larger engine will generally need to be purchased for the same output.
Electric Motors	• Significant reduction in capital cost. • Potential for remote operability. • Smaller pump station.	• Least reliable.

Table 13B-13 - PS-2 Engine Drive Non-Monetary Comparison

Table 13B-14 summarizes a life cycle cost analysis for the three different pump drives. The analysis was done using a fuel oil price of $1.50/gallon, a natural gas price of $6/MMBTU, and an electrical demand charge of $8.06/kW and an energy charge of $0.0426/kWh. The electric rates are based on the SFWMD experience elsewhere and have not been confirmed for the Reservoir site.

	Diesel	Natural Gas	Electrical
Total Capital ($)	3,010,000	5,500,000	900,000
Annualized Capital, 50 yrs, 6% ($)	191,000	349,000	57,000
Annual Energy Cost ($)	301,000	193,000	290,000
Annual Maintenance ($)	22,000	20,000	--
Total O & M ($)	323,000	213,000	290,000
Total Cost ($)	514,000	562,000	347,000
Energy Cost ($/kWhr)	0.101	0.067	0.091

Table 13B-14 - PS-2 Engine Drive Life Cycle Cost Comparison

The potential energy cost savings present at PS-1 are offset by the higher capital cost of natural gas engines and the natural gas line to PS-2. While the energy costs are comparable between the diesel and the electrical option, the much lower capital cost makes electric motors the best option based on the cost analysis. The reliability of electrical service is potentially an issue. However, PS-1 would be available to backup PS-2 if electrical service was lost for a period and should not significantly impact the seasonal operation of the pump stations. Based on the cost analysis, electric motors are recommended at PS-2.

The recommendation for electric motors is subject to the following considerations by the SFWMD:

1. More formal discussions should be conducted with FP&L to confirm that they are willing to provide the capacity to PS-2 and that electric rates are within the range that make electric motors the most favorable alternative. If the utility company or provider is unable or unwilling to provide a sufficient quantity of power to drive the pumps, diesel engine drives would be recommended.

(d) Common Engine Requirements:

Common engine requirements and rationale that would apply to either diesel or natural gas engines are:

1. 4-cycle engine design – Modern design.

2. Computer controlled engine – Increases flexibility in engine operation.

3. Platform access for engine and gear drive – Improves access for maintenance.

4. No clutch will be required. Discussions with both engine and pump vendors have indicated that neither are aware of an operation condition that would require a clutch for pump engine sets of similar size.

(e) Fuel System

A natural gas fuel system will be designed with 100% redundant gas pressure control stations for the pump station. Redundant valve stations will increase reliability by reducing the effects of a single failure.

(f) Cooling System

Cooling system requirements and rationale are:

1. The cooling system will be designed to pump river water through the various heat exchangers and return water to the basin Reservoir. River water offers the lowest temperature heat sink, which will reduce size of cooling water equipment.

2. Each drainage pump, drive, and gear reducer will have an associated cooling water pump. One additional backup cooling water pump will be provided for redundancy. This will increase system reliability by reducing the effects of a single failure.

3. The discharge from each cooling water pump will be combined into a single supply header. This will increase operation flexibility of the cooling water system.

4. Provide self-cleaning strainers on the discharge of each cooling water pump, designed to remove suspended solids from the cooling water. Cleaning of the river water will decrease the likelihood of fouling the heat exchangers.

(g) Compressed Air System

Compressed Air System requirements and rationale are:

1. Provide a minimum of two duplex reciprocating air compressors. This will increase system reliability by reducing the effects of a single failure.

2. Each drainage pump diesel engine drive will have an associated receiver with an air starting feed from a common air receiver. This will increase system reliability by reducing the effects of a single failure.

(h) Combustion Air System:

Combustion air will be ducted from the outside based on discussions with SFWMD in an effort to reduce the size of the building ventilation system for the pump station. The 30% design phase shall further address the criteria for the ventilation system design.

11) Discharge Apron

The discharge apron is a concrete sill that receives the flow from the discharge chambers. The discharge apron allows the water velocity to slow and prevents erosion at the outlet. The discharge apron is followed by a riprap apron for further protection. The dimensions of the discharge apron will be based on an analysis of the pump station flow and velocity leaving the discharge chambers. The siphon and over-the-embankment arrangements utilize a riprap apron for erosion control at the outlet.

12) Dewatering System

A dewatering system will be provided to dewater the pump suction area behind the dewatering needles and the discharge chambers. Dewatering needles will follow the SFWMD Standards for Construction of Water Resource Facilities Design Details, Drawing S3.

13) Comparison of Options 1 and 2

Option 1 includes a single pump station for the project site, PS-1. Option 2 includes PS-1 and a second pump station, PS-2. The primary purpose for the project is to maintain flows in the C-43 Canal within established limits. Therefore, both pump stations exist to fill the Reservoir with water to reduce the flow in the C-43 Canal.

The benefit of a second pump station is to take advantage of the higher water surface elevation available at the southeast corner of the site to reduce pumping costs. In keeping with the purpose of the project, the second pump station should be operated when the flow in the C-43 Canal is in a range where pumping is desired. In addition, the constraint is placed that sufficient water must be available at PS-2 to prevent adverse impact to the citrus operations during pumping.

The highest capacity of water available for pumping is from the C-43 Canal through PS-1.

PS-2 provides a second pumping location and adds flexibility to the system by pumping water into Cell 2 so Cell 1 could be taken offline. Each pump station discharges to a different cell so there is flexibility in the operation of the system. Option 1 with PS-1 can only pump into Cell 1 so there is a lack of flexibility in the system.

Table 13B-15 summarizes a non-monetary comparison of Options 1 and 2.

	Advantages	**Disadvantages**
Option 1 PS-1 only	• Lower total construction cost. • Lower labor requirement. • Closest for fuel (diesel delivery or natural gas pipe). • Highest capacity available for pumping from C-43 Canal.	• No flexibility. Reservoir must be filled from PS-1. • Cell 1 is the only cell that can receive pump discharge.
Option 2 PS-1 and PS-2	• More flexibility. • Perceived environmental benefit. • PS-2 lower head – lower energy cost to pump. • Pumps into Cell 2 so could take Cell 1 off-line.	• More expensive construction cost. • Longer distance for fuel (diesel delivery or natural gas pipe). • More O&M labor. • PS-2 not always available. Must meet flow criteria in Caloosahatchee and have excess water available in Perimeter Canal. • Smaller capacity available at PS-1 if needed. • Due to lower BTU content in natural gas, a larger engine will generally need to be purchased for the same output.

Table 13B-15 - Options 1 and 2 Non-Monetary Comparison

Table 13B-16 summarizes a cost comparison of Options 1 and 2.

Item	Option 1	Option 2
Construction Cost	$27,800,000	$35,500,000
Annualized Construction Cost (6%, 50 years)	$1,760,000	$2,250,000
Operating Cost		
Fuel	$566,000	$275,000
Lube Oil	$12,000	$12,000
Electrical	$49,000	$312,000
Labor	$210,000	$255,000
Maintenance (2% of Equipment Capital)	$194,000	$240,000
Contract Services	$25,000	$25,000
Total Annual O&M	$1,031,000	$1,119,000
Total Annual Cost	$3,000,000	$3,369,000
Equivalent Present Worth (6%, 50 years)	$47,300,000	$53,100,000

Table 13B-16 - Option 1 and 2 Cost Comparison

Section 14 – Electrical Design

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 14 of the Appendices.

A. Design Criteria

1) Project Electrical Features

(a) Utility Power Lines

Existing Florida Power and Light (FP&L) overhead power lines within the site development area are shown in **Figure 14A-01** in **Section 14** of the Appendices. Existing transmission lines crossing the site will not be disturbed with the proposed dam arrangement. The section of primary distribution line providing back-up to existing irrigation pump stations will be inundated. A replacement line by FP&L along the north dam will restore back-up and provide service to new water control structures and PS-1.

(b) Pump Station 1 (PS-1), 1500 cfs capacity

The SFWMD provided design documents for Pump Station G370, and the design team toured the facility. It is intended that pump station facilities for the Reservoir will substantially provide the features most desirable for operation and maintenance at the G370 facility. Key features:

- Three-phase, 480 volt power supply from FP&L transformer to the station switchboard through transfer switches for each of two bus sections.
- Redundant standby generators connected through main-tie-main switching and the two transfer switches to station switchboard.
- Motor control centers for each engine-driven pump assembly and for station auxiliary systems; utilization transformers, panelboards and power wiring; control system input/output wiring to all equipment; exterior and interior lighting; fuel and lube oil storage; trash rakes; ventilation system; fire detection and security monitoring systems.

PS-1 shall be used as the centralized site for operations and maintenance of the remaining pump stations, spillways, and culverts throughout the Reservoir. This station shall be the site for the microwave terminal for communications to the SFWMD Control Center. See **Section 15**.

(c) Pump Station 2 (PS-2), 500 cfs capacity

PS-2 requires the following three-phase, 480-volt power supply from FP&L transformers through single standby generator transfer switch to station switchboard; motor control centers for each engine-driven pump assembly and for station auxiliary systems; utilization transformers, panelboards and power wiring; control system input/output wiring to all equipment; exterior and interior lighting; fuel and lube oil storage; trash rakes; ventilation system; fire detection and security monitoring systems.

(d) Gated Spillways and Gated Culverts (as for cell isolation)

The Gated Spillways require the following single-phase 120/240 volt power supply from FP&L transformer through standby generator transfer switch to station service panel, gate control panel in control building, power wiring, exterior and interior lighting, control system input/output wiring, ventilation equipment, and security monitoring.

(e) Monitoring Stations

Monitoring Stations will require the following single-phase 120/240 volt power supply from FP&L or solar panel to service equipment, power wiring, control system input wiring, including level transmitters, and perhaps water sampler.

2) General Electrical Design Requirements

(a) Personnel Safety and Preservation of Property

The most important factors in the design and installation of a power system are the safety of personnel and preservation of property. The objective of a protection scheme in a power system is to minimize hazards to personnel and equipment while causing the least disruption of power service. Coordination studies are required for 480 volt systems to select or verify the clearing characteristics of protective devices. These studies are also required to determine the protective device settings that will provide selective fault isolation. The following shall be evaluated and considered in order to provide safe working conditions for personnel:

- Interrupting devices must be able to function safely and properly under the most severe duty to which they may be exposed.
- Protection must be provided against accidental contact with energized equipment, buses and/or conductors
- Isolating switches must not be operated while they are carrying current, unless designed to do so.

- The power system shall be arranged so that maintenance work can be accomplished with the particular circuits and equipment de-energized and grounded. System design shall provide for locking out circuits or equipment for maintenance.

- Clearances and exits shall be maintained per code requirements and access to equipment should be limited to electrical maintenance or operating personnel.

- Warning signs shall be installed on electrical equipment, on doors giving access to electrical equipment rooms and conduits containing circuits rated above 600 volts.

(b) Maintenance Program

Electrical equipment installation must be evaluated considering adequacy of performance, safety and reliability. The maintenance program will be tailored to the type of equipment and the details of the particular installation.

(c) Installation Requirements

As a minimum, the power system installation will provide the following:

- Adequate working space.

- Easy access for inspection and testing.

- Ability for emergency disconnect for the protection of personnel.

- Warning signs.

- Circuit lock-out features

(d) Environmental Influences

Equipment selection and building plan will consider the environmental influences listed below. Should variable frequency drives become a part of the construction, it is recommended that such power electronic equipment be enclosed in a room. Benefits include the following:

- Cleaner ventilation and cooling.

- Noise isolation.

- Moisture and corrosion control

(e) Future Load Growth

Interior electrical distribution systems will be designed to allow for a future load growth of 20%, with a minimum of 20% spare breakers in all panelboards.

(f) Ambient Conditions

Equipment will be installed such that rated operating temperature limits are met. If necessary this will require the housing of equipment in a heated, ventilated and air-conditioned enclosure.

- Temperature ranges for control enclosure mounted devices will be limited from 30 to 105 degrees Fahrenheit before heating or cooling is required. Relative humidity will be limited to 95% non-condensing.

- Pump station control room temperatures will range from 50 to 90 degrees Fahrenheit. Relative humidity will be limited to 50%.

3) Lighting Design

(a) Exterior - IES Handbook recommendations, including cutoff fixtures and shields, will be used to minimize light trespass. Lighting will be provided for parking area and the exterior of the building. Fixtures will be visually compatible with the SFWMD standards.

- Light poles in parking areas shall be on a 3 ft. tall concrete pedestal. Parking area lighting will be pole top mounted, rectilinear shaped fixtures with cutoffs; high-pressure sodium lamps, 2 FC average with a uniformity ratio of 3:1.

- Provide wall-mounted lighting fixtures with 100-watt high pressure sodium lamps at each personnel door and overhead door, and around the entire building. Exterior lights mounted above personnel doorways and the building exterior will be photocell controlled. Exterior light mounted above bay doors will be controlled by photocell and turn on/off with respective bay lights.

(b) Interior - Ceiling mounted fixtures will be provided for all rooms and interior areas. Lighting control will be from wall-mounted switches, except for selected lights in egress routes and night lights which will be controlled by circuit breaker only.

- Fixtures and Lamps: Light fixtures, exit signs, egress light fixtures, etc., shall be as shown on fixture schedules. Fluorescent fixtures shall have energy saving electronic ballasts. Lamps will be high

efficiency, reduced wattage. Metal halide light fixtures will have instant restrike ballasts.

- Egress and Emergency Lighting: Emergency egress lighting and exit signs will be provided as required by NFPA 101. The emergency power will be obtained from emergency battery packs. The emergency lighting fixtures will be on the same circuit as the room lights. Exit signs will have LED type lamps with emergency battery packs.

(c) Lighting level requirements (foot candles)

- Exterior forebay and pump discharge: To be determined (TBD)
- Vehicle parking: (TBD)
- Pump room: (TBD)
- Electrical room: (TBD)
- Maintenance area: (TBD)
- Control room: (TBD)
- Locker room: (TBD)

4) Pumping Station Engineering Guidelines

The SFWMD is in the final stages of publishing a manual documenting the preferred Pumping Station Engineering Guidelines for typical SFWMD low-head applications. Preliminary copies have been made available for design team comparison with proposed design requirements herein. The guidelines are also based on the G370 pump stations, and electrical requirements appear to be substantially the same as this BODR for high-head pumping applications. The final copy will be examined in detail during the design stage.

B. New Overhead Primary Lines

The proposed project location is southwest of LaBelle in Hendry County, west of S.R. 29 and south of S.R. 80 as indicated on **Figure 15D-01** in **Section 15** of the Appendices. Most existing facilities in the proposed construction zone are fed from the Alva Substation to the west. Backup power is available through a normally-open tie to the LaBelle Substation just east of the project boundary. The reliability of power supply to local irrigation district pump stations is high. The duration of outages to irrigation pump stations rarely exceeds two to three hours. Existing and new overhead lines are shown

in **Figure 14A-01** in **Section 14** of the Appendices. Existing power lines and their disposition are described in **Section 2** and **Section 12**, respectively.

1) Utility Service by FP&L

It is expected that overhead primary lines and service transformers for new facilities will be provided by FP&L. Additional substation capacity may be necessary, for which FP&L will be responsible.

(a) The replacement three-phase backup feeder along the north dam will provide service to several new structures associated with the Perimeter Canal routed along the south, east and north perimeter of the new dam.

(b) Gated culverts in the cell separation dams will be served via single-phase overhead primary lines from the north.

(c) Gated structures in the south and east perimeter dams will likely be served via single-phase overhead primary lines from closest FP&L lines.

2) Final Routing of New Lines will be Determined by FP&L

Routings shall consider future service to Aquifer Storage and Recovery (ASR) wells that are proposed along the perimeter of the Reservoir. A reference to the ASR development will be provided on site plans with contact at the SFWMD for FP&L line design coordination regarding ASR development at the time of construction.

3) Design Coordination

Design coordination with FP&L will include establishing easements and construction scheduling. The project will clear and prepare proposed on-site power line easements for FP&L work.

(a) A letter dated January 6, 2005 has been sent to FP&L at the address below to update them regarding this on-going design development associated with the Reservoir Project:

Mr. Randy Camp, Supervisor, Power Systems,

Florida Power and Light Company,

15834 Winkler Road, Ft. Myers, FL 33908

(b) The anticipated construction start date at the time of this report is June, 2007.

C. Backup Power

Backup power for normal operation of pump stations with engine-driven pumps and gate structures will be provided by engine-generator. For PS-2, backup generation need only be minimum size for station security. The FP&L feeder length would be approximately four miles, and is expected to have high reliability.

1) PS-1

PS-1 will include redundant backup generators as shown in **Figure 14C-02** in **Section 14** of the Appendices, PS-1, One-Line Diagram, Engine-Driven Pumps, similar to Pump Station G370. Fuel will be the same as used by the engine-driven pumps (diesel or natural gas). Through automatic transfer switches, backup power will be provided to all essential and life safety loads, including lighting, ventilation and required safety equipment for full plant operation. Preliminary generator rating is estimated to be 500 kW each for continuous service. Generators will provide backup power at 480 volts and be located within the pump building structure. For diesel engines, fuel will be provided from the same fuel storage system used for the diesel pumps.

2) PS-2 with Engine-Driven Pumps

PS-2 would include a single backup generator as shown in **Figure 14C-03** in **Section 14** of the Appendices, PS-2, One-Line Diagram, Engine-Driven Pumps. Fuel would be diesel as used by the pump engines. Through one automatic transfer switch, backup power will be provided to all essential and life safety loads, including lighting, ventilation and required safety equipment for full plant operation. Preliminary rating is estimated to be 250 kW for continuous service. The generator will provide backup power at 480 volts and be located within the pump building structure.

3) PS-2 with Motor-Driven Pumps

PS-2 equipped with electric motor pumps would have a single backup generator as shown in **Figure 14C-04** in **Section 14** of the Appendices, Pump Station 2, One-Line Diagram, Motor-Driven Pumps. However, fuel would be propane. Through one automatic transfer switch, backup power will be provided to essential and life safety loads, including lighting, security systems, and ventilation to maintain plant security during an outage. Preliminary rating is estimated to be 50 kW for continuous service. The generator will provide backup power at 480 volts and be located within the pump building structure.

D. Facility Electrical Systems

1) Power Distribution

(a) Fault and Overcurrent Protection

Line-side and load-side overcurrent and fault protection devices will be coordinated to isolate any electrical fault or overload from the rest of the system.

(b) Service Entrance

The building will be fed from the transformer via secondary conductors sized for 150 percent of the connected plus future load of the building. The service entrance conductors will not be larger than 500 kcmil. If the ampacity of the total load exceeds the ampacity of 500 kcmil wire, then parallel runs of conductors will be used. Parallel runs will be installed as required by the NEC.

(b) Sizing Feeders and Transformers

Sizing calculations will be provided for all feeders and transformers. Transformers and feeders will be sized for 150 percent of the connected load plus future load.

(d) Motor Control Centers (MCC)

Power will be distributed to plant auxiliary loads through MCC employing feeder monitoring equipment with single point connection to the plant monitoring system. Feeder performance data can be used to monitor motor condition, time of operation, and detect abnormal behavior to enhance maintenance and improve reliability.

For control of drainage pump drive and engine auxiliary motors and equipment, MCCs will be furnished as part of each equipment package, wired and fully tested prior to delivery.

(e) Panelboards

Secondary and local panelboards will be sized for a minimum of 150 percent of the demand load they serve. They will be fully rated for the available fault current, and furnished with main circuit breakers, full sized bolt-on branch breakers, insulated neutral buses, and bonded equipment grounding buses. Panelboards will be surface mounted with flush fronts and hinged doors. The Contractor shall furnish and install printed labels in the panelboard for all installed circuits.

- Panelboard Feeders: Panelboard feeders from the service entrance equipment will be sized to supply the full load rating of the panel they serve, plus minimum 50 percent capacity.

- Separate Circuits: Lighting and receptacles will be on separate branch circuits.

2) Motors and Controllers

(a) Motors

Energy efficient motor designs will be used for the Reservoir pump station(s). All motors will be high-efficiency squirrel cage induction type. Unless the utility has stricter requirements, all constant speed motor applications rated 50 hp and above will employ reduced voltage starters to minimize inrush transients and help control hydraulic surges. Motors will be specified and applied in accordance with NEMA Standard MG-1.

(b) Reduced Voltage Starters

Reduced voltage solid-state (RVSS) starters will be used to control starting and stopping of all pumping applications, and all constant speed motor applications rated 100 hp and above. Starting inrush currents can be reduced from a typical 600% of full load to about 250% using power semiconductors. RVSS technology permits precise control of voltage applied to the motor during starting and stopping. For greater reduction in inrush and surge control, variable frequency drives can be similarly used, but at higher cost.

3) Wiring and Wiring Devices

(a) Conductors

All conductors will be copper (aluminum is not allowed).

- Power feeder conductors shall not be smaller than #12 AWG. Conductors #12 will be solid. Conductors #10 and larger shall be stranded. All conductors will be installed in metallic conduit. Nonmetallic electrical conduit (smurf tube) is not allowed. Wire types XHHW, or THHN/THWN will be used. Metal-clad cable will be allowed only for light fixture connections with 5 ft maximum cable lengths.

- For instrumentation, stranded conductors shall be #14 maximum and #16 will be permitted for shorter runs of 24 vdc discrete and analog I/O wiring.

(b) Receptacles

Branch Circuits, Receptacles, and Outlets: General receptacle and lighting circuits will be 20-ampere circuits, fed by 20-ampere circuit breakers. All branch circuits required by the NEC will be provided.

- Isolated ground receptacles to serve computer loads will be provided.
- Exterior receptacles will be GFCI and will have weatherproof type covers, which are rated NEMA 3R with a plug cord attached.
- Exposed receptacles will be installed in cast metal boxes located in the web of building columns, or in another location where the receptacle is protected from damage. Exposed sheet metal boxes are not allowed.

(c) Installation Details

Only galvanized rigid steel conduit will be installed exposed within 20 ft of the floor. Minimum conduit size is 3/4 inch.

480/277-volt circuits will not be installed in the same raceway with 208/120-volt circuits.

Labeling: All materials, equipment, fixtures and appurtenances will be labeled by Underwriters Laboratories, Inc., or a similar acceptable organization. Electrical warning signs should be installed on electrical equipment, doors, and conduits for circuits rated 480 VAC and above.

Areas: Unpainted areas will be considered as an unfinished area.

4) Fire Detection System

Facilities with engine-driven equipment and associated fuel systems should employ a means of fire detection for monitoring and alarm.

(a) For the pump stations, the system would include a fire alarm control panel (FACP) with a full complement of sensors and signaling equipment:

- Monitoring inputs would include smoke and heat detectors, and manual pull stations.

- Alarm outputs would include horn/stobes, fan control shut-down, and the station master control panel for remote alarm reporting.

(b) For spillways, a smoke or heat detector can be wired as an input to the SCADA unit for remote alarming.

5) Lightning Protection

Lightning protection for pump stations will provide roof mounted air terminal system bonded to ground mounted earth electrodes. Earth electrodes will be bonded to the buried grounding system.

6) Grounding

The grounding electrode will be a grounding counterpoise which will consist of a buried grid interconnecting the waterway structures on the site. The grid shall consist of #4/0 bare copper cable which will be connected to the building service entrance neutral/grounding bonding point with a minimum of two #4/0 copper wire grounding electrode conductors and per other requirements in the NEC.

The counterpoise will be connected to the building grounding systems with #4/0 copper cables where required by NFPA 78 for lightning protection systems. A minimum of four connections will be provided. Additional electrode conductors shall be physically protected, and will be bonded to the electrode with exothermic welds. Building steel and slab rebar will be bonded to the grounding system.

The final design will insure that the system will have no ground loops for protection of the equipment. The final design shall provide ground bars located throughout the facility for connection of equipment.

Circuits will be equipped with a green ground no matter what type of load is connected. The ground will be connected to the panel grounding bus per the NEC.

E. Spillway, Culvert and Monitoring Station Electrical Systems

Design will be in accordance with the SFWMD Standard Design Details. Final construction drawings will be based on drawing files provided by the SFWMD. Changes required to adapt to the site application will be kept to a minimum. Pump station electrical features common among all facilities will be implemented accordingly.

F. Seepage/Ground Water Control (Pumps and Wells)

Specific collection systems are not provided for seepage or ground water control. The Perimeter Canal will intercept seepage from the Reservoir. Should it be necessary to return excess seepage to the Reservoir, the pump stations will be used to periodically draw down the Perimeter Canal.

Section 15 – Instrumentation and Controls

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 15 of the Appendices.

A. Design Criteria

1) Instrumentation and Control Features

Instrumentation and control work for the Reservoir will include pump stations, gated spillways, gated culverts, and water monitoring sites.

(a) PS-1

- Packaged systems requiring complex control will be provided with stand-alone Allen Bradley SLC-5/04 programmable logic controllers (PLC). These controllers will be networked with the main redundant Allen Bradley ControlLogix 1756 System PLC. Non-packaged controlled devices will hardwire to the ControlLogix I/O modules.

- Monitoring and control of remote sites within the project, including smaller pump stations, gated spillways, gated culverts, and monitoring stations will share data over a network of spread spectrum radios. Signals to control gate movement will be controlled from the main PLC and fed to the local PLC or RTU.

- Pumps will be capable of operation from the District Control Center in West Palm Beach, the Pump Station SCADA System, or locally at the pump location. Gated spillways and gated culverts will be likewise controllable.

- Any dual-walled fuel storage tanks used to store diesel fuel for the engine-generators or engine drive pumps will be monitored for continuous level and level alarms. The interstitial area between the walls will be monitored for leakage and alarmed.

(b) PS-2

- If PS-2 is implemented, Motorola MOSCAD-L RTUs will be provided at each pump station. Control programming shall reside within these RTUs, with setpoints being changeable from the

SCADA System at the PS-1, as well as at PS-2. Setpoint data will reside within the local RTU. Thus, any communication failure will not affect operation of PS-2.

- Communication with PS-1 will be via spread spectrum radio telemetry.

(c) Gated Spillways

- Monitoring and control of gated spillway applications will be through Motorola MOSCAD-L RTU.

- Water level on both sides of the dam near the spillway will be monitored locally and sent to the SCADA System. Decisions to control the level in the cell by adjusting the positioning of the outlet structure gate shall be made manually. The desired positioning signal will then be sent to the RTU through the local telemetry system.

(d) Gated Culverts

- Monitoring and control of gated culvert applications will be through Campbell Scientific RTUs and Motorola MOSCAD-L RTUs. Both of these systems will need to communicate over the DataRadio Spread Spectrum radio network to PS-1. The detailed design phase will determine whether this is a single radio network or two.

- Water level on both sides of the interior embankments near the gated culverts will be monitored locally and sent to the SCADA System. Decisions to control the differential level between the two adjoining cells by adjusting the positioning of the culvert gate will be made manually. The desired positioning signal will then be sent to the RTU through the local telemetry system.

(e) Monitoring Systems

- Monitoring of water level or other site conditions will be through Campbell Scientific RTUs.

- Water level in a cell not associated with a gated spillway or gated culvert applications will be monitored locally. This level signal will be transmitted to the SCADA System for display.

2) Ambient Conditions

PLC, RTU, and telemetry equipment will be installed such that rated operating temperature limits are met. If necessary this may require the housing of equipment in a heated, ventilated and/or air-conditioned enclosure.

(a) Temperature ranges for control enclosure mounted devices will be limited from 10 to 105 degrees Fahrenheit before heating or cooling is required. Relative humidity will be limited to 95% non-condensing.

(b) Control room temperatures will range from 50 to 90 degree Fahrenheit. Relative humidity will be limited to 50%.

3) Power Supplies

Available sources will be 120-volts ac, 60 Hz.

(a) Control room SCADA equipment will operate at 120-volt ac, backed up with a reliable uninterruptible power source (UPS). Communication equipment operating on low voltage dc supply will be furnished through an authorized power supply or vendor-furnished power adapter.

(b) Remote sites will be powered from 120-volt ac, 60 Hz source with enclosure mounted UPS backup unit.

4) Control Signals

(a) Analog signals will be provided as 24-volt dc, 4-20 mA. In as much as possible, these will be 2-wire signals. Impedance loading will be 0-750 ohms, minimum.

(b) Discrete signals will interface with the PLCs and RTUs as 24-volt dc signals. As required, interposing relays will be used to provide isolation for converting different signal voltages to 24-volt dc.

(c) Discrete points from PLCs and RTUs will interface with field devices through interposing relays.

(d) Transient voltage surge suppression will be provided for all instrumentation. The SFWMD Design Details will be followed. It is understood that these standards provide the latest means that the District employs for existing telemetry and instrumentation systems, and that

they are periodically reviewed, evaluated and improved with lessons learned.

5) Level Monitoring in Deep Stilling Wells

This Reservoir will involve higher heads at structures than are typically found at other SFWMD water control structures. The typical method of level monitoring used is the linear potentiometer in a stilling well. The SFWMD has standardized equipment furnished by Balluff, Inc. That equipment is limited to a range of measurement of 10 ft.

(a) For the pump stations and discharge structures, the range of water level sensing will be more than 20 ft. For levels exceeding 10 ft., Rittmeyer pressure transducer type level sensors should provide the same accuracy over the greater range of operation.

(b) Stilling wells will be located in accordance with the SFWMD Design Details. The in-bank type is the preferred installation.

6) Embankment Monitoring

Instrumentation of the exterior dam embankment and perimeter seepage collection canal shall be implemented where feasible. The telemetry system shall include monitoring of these instruments where feasible.

(a) Measurement of seepage flow will not be possible because seepage collection is not isolated from other water sources. The Perimeter Canal will be receiving water from the upstream watershed. Separating seepage water from natural runoff would require an additional canal around the perimeter of the Reservoir.

(b) If embankment monitoring is necessary, the practical method is understood to be monitoring wells (piezometers) in the embankment. The SFWMD is developing standards for Embankment instrumentation, including the following:

- Piezometers water levels and pore pressure transducers.
- Inclinometers and surface movement monuments.
- Flumes to measure flow in seepage collection channels, where applicable.

(c) Embankment instrumentation is different than conventional SCADA systems. Instruments trends to be passive, and do not require "real-time" SCADA-like control data reporting. Monitoring intervals are typically less frequent. Changes in readings and associated trends are used as an indication of developing problems.

7) Water Quality

It is expected that discharges will have to be monitored for water quality. The SFWMD is finalizing plans for all Reservoirs. Sampling equipment will be provided in accordance with the SFWMD Design Details.

8) References

(a) SFWMD Standards for Construction of Water Resource Facilities, Design Details for vertical lift gate structures and associated controls, SCADA interface and instruments.

(b) Structure G370/G372 Design Documents.

(c) The Pumping Station Engineering Guidelines Manual which is in the final review process by the SFWMD.

B. Control Systems

1) Pump Stations (PS-1 and PS-2)

(a) Control and monitoring of major pieces of equipment, such as Drainage Pumps, water systems, fuels storage, trash rakes, Engine-Generators, ventilation system, fire and security systems will be performed using an Allen Bradley ControlLogix 1756 PLC System. This PLC will be provided with redundant Logix5550 processors, but will utilize non-redundant I/O. I/O distribution will be as dictated by the SFWMD Design Guidelines, Design Details, and Pump Station G-370 design.

(b) The main Allen Bradley ControlLogix PLC will interface with other PLC's and power monitoring devices provided with individual systems via serial communication protocols. Interface of the ControlLogix system with the local SCADA System shall be via 100 MHz Ethernet communication protocol.

(c) If PS-2 is implemented it will use Motorola MOSCAD-L RTUs for monitoring and control.

(d) Data obtained and received signals required for control will be transmitted via the local spread spectrum radio telemetry system.

2) Gated Spillways and Gated Culverts

(a) Depending on site specific applications, a control building will be provided to house local control enclosures and standby generator. The control enclosure will include the RTU, radio, battery backup power and assorted control devices. All control enclosures will be grounded and provided with circuit breakers on the ac and dc supply sides and surge suppressors on the load side feeding the analog signal inputs. Internal powered components will be connected to an internal ground bus connected to earth ground.

- For application requiring remote positioning of gates and monitoring, Motorola MOSCAD-L RTUs will be used.

- For culvert applications where only gate monitoring is required, Campbell Scientific RTUs will be used. Sites without 120-volt available supply power will be powered using solar modules and batteries for power storage.

(b) Water level monitoring on both sides of the gate will be measured to determine positioning requirements.

(c) Depending on the application, some walkways will be equipped with automatic water samplers. In applications were walkways are not provided, in-ground stilling wells with feeder pipes extending in the water will be used.

(d) Stevens Water Monitoring staff gauges will be provided near the water level measurement sensing locations.

(e) Data obtained and received signals required for control will be transmitted via local radio telemetry. See below for details.

C. Monitoring Stations

Although not specifically identified in the scope of work for the Reservoir construction, it is anticipated that monitoring and/or sampling stations will be added at far reaches of the cells where no control structures are required. Campbell Scientific RTU dataloggers will be used for monitoring applications.

1) Stilling Wells

Where walkways off of the embankment are provided, stilling wells will be installed at the far end of the walkway to house a level measuring instrument.

2) Control Enclosure

Sites will be powered using BP Solarex solar modules and batteries for power storage. The control enclosure will include the datalogger, radio, battery and charging system, and assorted control devices. All control enclosures will be grounded and provided with fuses to protect the batteries. Internal powered components will be connected to an internal ground bus connected to earth ground.

3) Water Level Monitoring

Level sensor types will be similar to walkway installed stilling well applications. Stevens Water Monitoring staff gauges will be provided near the water level measurement sensing locations.

4) Instruments

Various meteorological instruments will be used and the measured value input into the datalogger. Depending on the site these will include instruments manufactured by:

(a) Campbell Scientific instruments

- Barometric pressure barometer
- Ambient temperature,
- Air temperature and relative humidity

- Net radiometer
- Quantum radiometer
- Pyranometer.

(b) All weather instruments

- Rain fall

(c) Viasala instrument

- Wind speed.

5) Data Transmission

Data obtained will be transmitted via local radio telemetry.

D. Remote Telemetry – Microwave Radio

1) General

The existing Microwave Telemetry System will be used for transmission of data from PS-1 to the SFWMD Control Center B-1 Control Room. Capabilities will be available within the existing SCADA System at the SFWMD Control Center to transmit control parameters and control instructions to the centralized PS-1 control system.

(a) No new telemetry system components will be added at the existing Control Center. The new intercept point at North Haven will provide the means to add the Reservoir data to the existing SCADA System.

(b) Configuration modifications to the SCADA System database and graphic screens to support the Reservoir applications will be performed by the SFWMD personnel

2) Existing System

The existing District Communication Microwave System consists of two loops; north and south. The dual path loops split at Clewiston Field Station as shown on

Figure 15D-01 in **Section 15** of the Appendices. The north loop passes through a tower at S-47B Moore Haven Lock, which is the closest station to the Reservoir location. Because of the distance from the Reservoir to the Moore Haven microwave site, a microwave repeater station will be required.

3) Radio Path Study

A radio path study has been performed to confirm the repeater site and determine the tower height and antenna mounting height. Mr. Wayne Lutz (SFWMD) coordinated that effort and provided summary sheets from the path surveys that can be found in Appendix 15. Excerpts for each path are as follows:

(a) Reservoir (Berry Groves) to S-78 Ortona Lock

The path is 17.08 miles in length. A new 190 ft. tall, self-supporting tower will be needed at both ends of the path. The recommended radio is the Megastar at a frequency of 6.175 GHz with spacing diversity. The recommended centerlines are 175 ft. for the main antenna and 145 ft. for the diversity antennae at the West Reservoir pump station, and 175 ft. for the main antenna and 145 ft. for the diversity antenna at S-78.

(b) S-78 Ortona Lock to S-47B Moore Haven

The path is 11.27 miles in length. S-47B already has a 190 ft. tall, self-supporting tower. The recommended radio is the Megastar at a frequency of 6.175 GHz with spacing diversity. The recommended centerlines are 175 ft. for the main antenna and 135 ft. for the diversity antennae at S-78, and 140 ft. for the main antenna and 100 ft. for the diversity antenna at S-47B.

4) Linking

The SFWMD is interested in extending the existing Microwave Telemetry System west and south to link District operations on to the Naples, Florida area. This extension of the microwave system to the Reservoir will complete about half of that link.

5) Telemetry System Interface

Interface equipment between the Reservoir PLC System and existing North Haven telemetry site will be designed around existing SFWMD applications.

(a) PS-1 will be used as the central location for data collection and the interface point with the SFWMD Control Center SCADA System. Process parameters measured at PS-2, including the gated spillways, gated culverts, and monitoring stations will be stored at the PS-1 PLC using the redundant Allen Bradley ControlLogix PLC's. The Microwave Telemetry System will obtain the data from the PLC data tables and transmit that information to the existing SCADA System. Control requirements issued from the existing SFWMD Control Central SCADA System will be input into the redundant Allen Bradley ControlLogix PLC's via the Microwave Telemetry System for redistribution as necessary through the local spread spectrum telemetry system.

- Equipment provided at this location will include, but not be limited to, the modem to interface with the Allen Bradley ControlLogix PLC's, control enclosure, power supplies and voltage conditioners, coaxial signal cables, voltage surge and lightning protection systems, tower and microwave antennas.

- A listing of data points to be transmitted between the Reservoir sites and the SFWMD Control Center SCADA System will be identified. The signal types, analog or discrete, will be defined as well as the calibration for the analog signals and the functional description of each point.

(b) Radio System Extension

A new microwave system interface to the existing SFWMD microwave north loop will be added at Moore Haven. Interface equipment will include a modem, control enclosure, power supplies and voltage conditioners, coaxial signal cables, voltage surge and lightning protection systems, and microwave antennas. The new control enclosure will be located in the existing site control structure.

(c) New Repeater Site

A new Microwave Telemetry System repeater station will be added at S-78 Ortona Lock. Repeater equipment will include one set of common control components such as a control enclosure, power supplies and voltage conditioners, and two sets of digital communication equipment components such as coaxial signal cables, voltage surge and lightning protection systems, and microwave antennas. A single microwave tower will be used to mount both antennas. A climate controlled site structure will be used to house the equipment.

(d) Microwave Telemetry Building

Telemetry building structures for microwave radio equipment will be of prefabricated concrete construction in accordance with the SFWMD Design Details.

E. Reservoir Telemetry – Spread Spectrum Radio

A local, intra-project telemetry system using DataRadio Spread Spectrum radios will be used to provide the means to transmit data between PS-1 and PS-2, gated spillways, gated culverts and water monitoring sites within the Reservoir site as shown in **Figure 15E-01** in **Section 15** of the Appendices.

1) Data Radio Spread Spectrum Radios

Depending on data requirements and connectivity of PC and RTU manufacturers to the radio system, it may be possible to DataRadio Spread Spectrum radios. Through the use of this technology, data transfer shall be available without delays associated with polled radio systems.

2) Complementary Equipment

Because of multiple manufacturers used for monitoring and control at PS-2, gated spillways, gated culverts, and monitoring stations, complementary equipment may be required at both ends of the data path for each manufacturer to comply with communication protocols used.

3) Antennas

An omni-directional antenna will be installed at PS-1 to allow communication with the remote devices. Each of the remote locations will be provided with a directional Yagi antenna for data transfer. Antenna cabling shall be provided with Polyphaser surge suppressors.

4) SCADA Interface

Monitoring and control sites associated with the Reservoir will interface with the SCADA System at PS-1.

F. Design Documentation

The design engineer shall provide a design package for Instrumentation and Controls based on the SFWMD Design Guidelines, Design Details, and construction documents similar to Pump Station G-370. Documents within this package are expected to include:

1) Control System Operating Description

This shall include control sequences and operating philosophies for automated operation of the fuel, cooling, and pumping control at the pumping stations, gated spillways, and gated culverts. A description of the operation of the proposed local spread-spectrum radio telemetry system will be provided. Location of proposed level monitoring stations will be defined.

2) Graphic Screen Requirements

This document will establish guideline requirements for configuration of SCADA and HMI screens. The SFWMD personnel will be contacted to provide sample screens and to provide input into the creation of this document.

3) Process and Instrumentation Diagrams

These diagrams will identify major pieces of equipment, associated instrumentation, and PLC/RTU I/O points for all systems interfaced with the Pump Station SCADA System.

4) Control Schematics

Schematics will define hardwired control logic requirements. Software programming will be provided by the System Integrator based on the Control System Operational Description.

5) I/O listing for each PLC/RTU

I/O lists will identify the appropriate tag number, description, wiring point, calibration range and/or setpoint, and energization configuration. The SFWMD personnel will be contacted to coordinate a tagging scheme that is consistent with the SFWMD Design Guidelines.

6) Construction Specifications

Specifications using the SFWMD Design Details and Guide Specifications will be prepared for hardware instrumentation, PLC/RTU components, SCADA/HMI System components, software, the telemetry system, and control panels.

Section 16 – Architectural

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 16 in the Appendices.

A. Design Criteria

1) Introduction

There will be two types of architectural buildings on site; Pump Stations and Control Buildings.

Pump Stations: Pump stations are of two categories in this project: one large (PS-1) and one medium capacity (PS-2). The pump station buildings will be constructed to accommodate the pumps, motors, generators, and ancillary systems. In addition, adequate area will be provided for a control room, offices, break room, toilet, locker/shower and mechanical equipment.

Control Buildings: Control buildings will be designed and constructed in accordance with the SFWMD's Standard for Construction of Water Resource Facilities Design Guidelines, dated April, 2003.

2) Design Requirements

Codes and Standards: Design and specifications of all work will be in accordance with latest laws and regulations of the Federal Government, applicable state and local codes and ordinances, and applicable industry standards.

Other recommended standards will be used where required to serve as guidelines for design, fabrication and construction when not in conflict with the above standards.

All buildings will be designed in accordance with Florida Accessibility Code and Americans with Disabilities Act Accessibility Guidelines (ADAAG).

There is no requirement for Anti-Terrorism/Force Protection measures for the buildings.

There is no requirement to incorporate the principles of Sustainable Design and Development for the buildings.

3) Life Safety

The buildings will be designed to meet or exceed the minimum construction and life safety requirements as required by the applicable codes and criteria. Appropriate type, size and quantity of fire extinguishers will be provided in equipment rooms in compliance with all applicable fire and life safety codes and per mechanical design criteria. In addition, automatic sprinkler system will be provided in all office areas and break room as per NFPA 13.

4) Material and Life Cycle

The buildings shall be designed to minimize life cycle cost, energy consumption, and maintenance through proper selection of mass, form, materials, and construction standards. Integrally colored materials shall be used as much as possible to eliminate painting. The design life of the building shall be a minimum of 50 years. All buildings shall be designed to withstand seismic events and wind loads in accordance with applicable local codes. Service Life Span: Same as building service life, except:

(a) Load-Bearing Structural Members: Minimum 100 years.

(b) Protective Elements: Minimum 25 years.

(c) Wall Primary Weather-Barrier Elements: Minimum 50 years functional and aesthetic service life, excluding joint sealers.

(d) Joint Sealers: Minimum 20 years before replacement.

(e) Surfaces Exposed to View: Minimum 20 years aesthetic service life. There will be no color fading, crazing and delamination of applied coatings during that period.

(f) Roof Covering Weather-Barriers: Minimum 20 years, fully functional.

B. Exterior Architectural Features

1) Shell

The elements forming usable enclosed space and separating that space from the external environment comprise the shell and consist of:

(a) Superstructure:

All elements forming floors and roofs above grade, and the elements required for their support, insulation, fireproofing and fire stopping.

There are two major structural systems suitable for the superstructure: steel or reinforced poured concrete. Although steel framing may cost less than reinforced poured concrete, the latter will be fire rated and suitable for permanent construction in a damp environment. Therefore, reinforced concrete construction is recommended. All exposed interior and exterior concrete surfaces will have an application of latex block filler and an acrylic coating.

(b) Exterior Enclosure:

All essentially vertical elements forming the separation between exterior and interior conditioned space, including exterior skin, components supporting weather barriers, and jointing and interfacing components; not including the interior skin unless an integral part of the enclosure.

Pre-cast concrete panels will be evaluated for the exterior enclosure during the 30% design phase. Thermal performance is not applicable to main equipment rooms. Exterior enclosures will be insulated for all air-conditioned spaces.

All exterior doors will be painted hollow metal doors with painted hollow metal frames. Insulated doors will provide for air-conditioned spaces. Overhead doors shall be roll-formed galvanized steel construction, electrically operated and shall be sized to fit the largest equipment for the building. Louvers will be designed as required for ventilation of the spaces and equipments. All doors and louvers will be hurricane impact resistant.

(c) Roofing:

All elements forming weather and thermal barriers at horizontal roofs, decks and roof fixtures.

Built-up roofing membrane systems with asphalt bitumen and modified bitumen granule cap sheet will be used over the reinforced poured concrete roof deck. Stainless steel sheet metal water collectors and conductors will be used for roof drainage. All flashing, trim, and accessories will be of stainless steel sheet metal. Access to roof will be provided by a roof hatch and will be controlled for authorized personnel only.

C. Interior Architectural Features

1) Floor

All floor slabs will be sealed reinforced poured concrete with floor drains.

2) Partitions

Partitions provided for physical separation between spaces will be constructed to achieve fire ratings required by code, appropriate security between adjacent spaces, and visual, acoustical, olfactory, and atmospheric isolation as necessary to maintain desirable conditions in each space. Partitions will comprise the following elements: Fixed partitions of fully-grouted, reinforced, full-height CMU; and partial height partitions of fixed, solid, opaque visual barriers, for toilet compartments. Acoustical isolation for fixed partitions from the equipment area will be Sound Transmission Coefficient (STC) of not less than 45.

3) Interior Doors and Windows

All interior doors shall be painted hollow metal doors with painted hollow metal frames. Interior windows will be provided between adjacent spaces. Fixed interior windows and operable interior windows, when closed, will function as partition elements and will not degrade performance of partitions below the levels specified. Sound doors and windows will be provided to meet the Sound Transmission Coefficient (STC) of not less than 45.

4) Interior Finishes

(a) Offices/Control Room/Break Room

Wall: Painted

Floor: VCT

Ceiling: Suspended acoustical ceiling tiles

(b) Toilets/Showers

Wall: Ceramic tiles

Floor: Non-skid ceramic tiles

Ceiling: Moisture resistant gypsum board

(c) Equipment Room

Wall: Painted

Floor: Sealed concrete

Ceiling: None. All exposed concrete will be painted

5) Vertical Circulation

One stairwell will be provided for access to all unoccupied roofs, mechanical spaces, and equipment mezzanines.

6) Interior Fixtures

Interior fixtures permanently attached to interior walls, ceilings, and floors, except for equipment items will be provided and comprise the following elements:

(a) Identifying Devices: Informational accessories, including room numbers, signage, and directories.

(b) Storage Fixtures: Non-furniture items intended primarily for storing or securing objects, materials, and supplies, including cabinets, casework, and shelving.

(c) Accessory Fixtures: Specialty items intended to provide service or amenity to building interiors, including toilet and bath accessories, visual display surfaces, and telecommunications fixtures.

(d) Other Interior Fixtures: Other items fixed to interior construction that enhance comfort or amenity in building spaces.

Section 17 – HVAC, Plumbing and Fire Suppression

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 17 in the Appendices.

A. Design Criteria

1) Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal Government, with applicable local codes and ordinances, and with codes and industry standards referenced herein.

2) Recommended and recognized standards from other organizations shall be used where required to serve as guidelines for design, fabrication, and construction when not in conflict with standards referenced herein.

3) Codes and industry standards used for design, fabrication, and construction shall be codes and industry standards, including all addenda, in effect as stated in contract documents.

4) Mechanical and electrical elements, including equipment, piping, and their supports, shall be designed to withstand seismic events in accordance with local codes.

5) Comply with code requirements and manufacturer's guidelines for clearances. Minimum clearance under overhead equipment, piping, and ductwork shall be 7'.

6) District will review all design assumptions, criteria, and calculations.

7) Site Criteria: Design of HVAC systems shall be as stipulated herein and based on Ft, Myers, Florida, ASHRAE climatic design information. Following design temperatures and ventilation rate criteria shall be used for system design parameters.

   (a) Temperature conditions at Ft. Myers (nearest station of record) range from +97°F in July to +34°F in January. Summer dehumidification design is approximately 138 gr/lb.

   (b) Project Location: 5 miles southwest of Labelle, Florida.

   (c) Latitude: Approximately 26°45'41"N.

   (d) Longitude: Approximately 81°26'19"W.

   (e) Elevation: 16 ft. to 27 ft.

(f) Prevailing Wind: No data.

8) Outside Design Conditions:

(a) Heating: 37ºF dry bulb (TM-5-785, Engineering Weather Data, Ft. Myers, FL).

(b) Cooling: 93ºF dry bulb, 77ºF wet bulb (ASHRAE 2001 1% condition).

(c) Dehumidification: 83°F dry bulb, 78°F wet bulb (ASHRAE 2001 1% condition).

(d) Air-Cooled Condenser Ambient Temperature: 100ºF.

9) Inside Design Conditions:

Area	Heating	Cooling
Control room, break rooms	68°F	78°F, 50% RH
Locker and toilet facilities, janitor closets	68°F	95°F
Mechanical and electrical rooms	68°F	95°F
Engine pump bays	40°F	103°F
Generator rooms	40°F	103°F

10) HVAC load calculations and equipment sizing shall be based on procedures outlined in ASHRAE handbooks. Detailed room-by-room heat loss and heat gain (sensible and latent) calculations shall include the following:

(a) Accurate determination of glass U-values and shading coefficients, and wall/roof U-values from architectural design.

(b) Building envelope transmission and solar loads. Separate winter and summer infiltration loads through doors, windows, skylights, walls, roofs, and any other building openings shall be calculated.

(c) Interior heat gains from people, lights, and other electrical equipment.

(d) Outside air ventilation loads for indoor air quality (IAQ) and/or exhaust fan make-up.

(e) Air handling unit fan motor heat gain.

(f) As permitted by energy code after unoccupied periods, allowances for warm-up or cool-down periods.

B. HVAC

1) Work includes, but is not limited to, providing a complete design and construction for:

   (a) Pump stations: Heating, ventilation, and air condition all pump stations control rooms and ventilate pump engine bays.

   (b) Gated Spillways and Gated Culverts: Ventilate all gated spillways and gated culvert control structures.

   (c) Instrumentation and controls for heating, ventilating and air conditioning systems.

   (d) Testing, adjusting, and balancing.

2) Ventilation for Control Rooms and Engine Pump Bays, shall be done to meet NFPA 820 requirements. Also, comfort ventilation shall be provided for these spaces during all times of the year to remove space heat gains based on a 5°F temperature rise, or six air changes per hour, whichever is greater. Ventilation for the Generator Rooms shall be sized to accommodate the heat gain to the space while the generator is running, or radiator airflow, whichever is greater. If natural gas is used for engine power, the HVAC system design will comply with additional hazard requirements, where applicable.

3) Final equipment selection shall be based on a 10% safety factor over design loads to account for additional capacity and system changes. Installation of all equipment shall comply with the manufacturer's instructions to accommodate operation and maintenance. Provide all required accessories to meet insulation, vibration, and sound requirements.

4) The air distribution system shall be a galvanized, metal ductwork system. The system shall be sized using equal friction. The maximum friction drop will be equal to 0.08 inches of water per 100 ft. of duct. Maximum duct velocities shall not exceed 2,000 ft. per minute (FPM). To help minimize the pressure drop, aerodynamic fittings, long radius elbows and conical-type take-offs shall be used

5) Insulation shall be provided where required or allowed by code, and to eliminate condensation. (This includes piping and ductwork). In general, external insulation is preferred. For sound attenuation, double-wall ductwork may be used for short distances. Approved insulation liners shall be covered with a perforated metal liner.

6) Provide intake hurricane type louvers, ventilation designed and sized with a

minimum of 400 FPM of air velocity forced across engine bays. Exhaust louvers shall be sized for a maximum velocity through free area of 1,000 FPM.

7) Use low-leakage, motorized dampers for automatic control. Gravity-type dampers shall not be used. Fire, smoke, and combination fire/smoke dampers shall be used in accordance with the local code. Balancing and volume dampers shall be installed to comply with AABC or NEBB standards for system balancing.

8) All fan inlets and outlets shall have approved flexible connections. Access doors shall be provided for cleaning, inspection, and service of filters, heating and cooling coils, and dampers. Provide access doors at frequent and maximum intervals allowed by the local codes.

9) Flexible duct shall be limited to a maximum length of six feet. Flexible duct shall be used for concealed connections to diffusers, registers, and grilles.

10) When available, furnish prefabricated roof curbs to match the roofing system. The roofing contractor shall conduct the installation.

11) Diffusers, registers, and grilles at air inlets and outlets shall be as uniform as possible throughout the facility. The location of these devices shall be coordinated with the architectural features of the space, equipment layout, and lighting layout. Throw and noise criteria (NC) levels shall meet the space requirements.

12) Provide filters with efficiency required to meet space indoor air quality. Filters shall be sized for a velocity of 300 FPM, with the maximum velocity permissible at 400 FPM.

13) A stand-alone control system with an electric source shall be provided for the HVAC system. Include all thermostats/temperature sensors, control dampers/actuators, control valves/actuators, wiring, and other required components for safety and operating control sequences.

14) To maintain the desired room conditions, the capacity of the equipment shall be controlled automatically according to space heating or cooling load. To that end, the control system shall have the capability to anticipate demand and provide automatic compensation so that the system capacity will change according to the load.

15) For small, simple systems, such as unit heaters that can not be adapted to modulating control, two-position controls shall be used for limit or safety control.

16) Modulating controls shall be used for varying equipment capacity in response to load changes and for simultaneous control of more than one variable.

C. Potable Water

1) A well shall be drilled on the property to supply the potable water to the facility. This may require obtaining a permit as well as conducting water quality tests. Subsequent to this, verification of the well capacity shall be done. Based on the results of the water quality tests, then an appropriate water purification system may be included as part of the potable system. A backflow prevention device may be required between the potable system and the well.

2) Piping shall be installed from the well to all fixtures requiring potable water within the facility. Piping shall include all required accessories, including isolation valves, insulation, sleeves, valves, strainers, arrestors, and supports. Wall hydrants shall be located on the exterior perimeter of the building at each entry/exit, and at every 100 ft. around the building.

3) Potable water shall be supplied to the electric-powered domestic water heater. Hot water shall be supplied at 120°F to the lavatories, sinks, and showers. The hot water system may utilize a re-circulation system to provide continuous hot water to all fixtures. Mixing valves shall be used to prevent scalding at each fixture or group of fixtures.

D. Sanitary System

1) Sanitary drainage shall be installed on all fixtures and equipment requiring drainage. Sanitary drainage from the building will be collected in a septic tank. Soil tests shall be conducted to determine if the area can accommodate a leaching field. Based on the results of the soil tests, the septic tank may have to be pumped regularly.

E. Storm Water System

1) Storm drainage shall be collected from the roof drains and leaders. All storm drainage at the pump stations shall be routed to the Forebays.

F. Fire Suppression System

1) Fire protection and detection system shall be provided for the entire facility. See detection system in **Section14**.

2) The District shall review all design assumptions, criteria, and calculations. Verification with the District and the District's insurance underwriter shall be done for the fire protection system type.

3) Portable fire extinguishers shall be installed in accordance with NFPA 10. Water fire extinguishers shall be used in the Pump Bay area, and CO2 fire extinguishers shall be used in the Control Room. Recessed or non-recessed types will be shown and specified under architectural sections and drawings.

4) A dry-pipe sprinkler system shall be installed in all occupied areas in accordance with NFPA 13.

5) Locations of extinguishers shall be shown on the architectural plans, and detailed as necessary for construct ability.

Section 18 – Special Considerations

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 18 of the Appendices.

The Reservoir Project includes special design considerations that provide opportunities for recreational and public access, while maintaining safety and security requirements for facility operations. This section discusses proposed special use considerations that may be integrated into subsequent phases of design development that will provide facilities for public recreational opportunities, opportunities for wildlife enhancement, access requirements, fire protection, and site security.

A. Environmental

The final design development for the Reservoir will strive to include wildlife enhancements and improvements to project features. As design development progresses and additional criteria is developed for the project features, the inclusion of the wildlife enhancements will be considered. The key criteria that will be utilized when evaluating the use of wildlife enhancements will be the following:

- Does enhancement diminish the function of the project feature?
- Does the enhancement create a maintenance nuisance?
- Does the enhancement create a safety concern?

Given this criteria will be applied, some of the potential wildlife enhancements that have been considered include:

- Inclusion of littoral zones along the seepage/Perimeter Canals
- Integration of existing habitat lands with project areas

Littoral Zones: The use of narrow, elongated littoral zones in a variety of places along the perimeter seepage canals is considered as a potential enhancement that might add to the wildlife viewing opportunities for the Reservoir. Coordination of this proposed enhancement will be required with the Reservoir design and the proposed future Aquifer Storage and Recovery (ASR) project to make sure that project performance is not impacted. The placement of littoral zones will yield to the engineering needs such that even once established, littoral zones may need to be filled moved or elongated. The seepage canals and resulting water depths will be operated to fulfill the project's intended purpose. The littoral zones are intended to vary and provide different areas ranging from 6 to 18 inches of water. These may become deeper or dry depending on the seasons and available water with in the seepage canals. The design for littoral zones will anticipate a wide variety of changes in water depth so as to benefit from a variety of depths creating the many edges between water and land that can benefit fish and wildlife.

Habitat Areas: Properties that are contiguous to the Reservoir property are generally recognized as prime habitat for several species of animals. Consideration will be given to extending those habitat areas into unused Reservoir areas.

B. Recreation

While the Reservoir is a civil works project that will be designed to improve the estuaries in the C-43 Canal, there is potential for incorporating recreational features into the project.

The current list of recreational opportunities proposed for the Reservoir generally consists of passive recreation opportunities with the possibility of some restricted boating opportunities. Final recreational design considerations will evolve further during the final phases of design as the CERP Master Recreational Plan and the SFWMD Public Recreational Access and Use Policy are woven into the design process. The following are opportunities that might be included as recreational opportunities for the Reservoir:

- Wildlife Viewing
- Birding
- Hiking
- Biking
- Equestrian Activities
- Picnic
- Fishing
- Canoeing
- Restricted Motorized Vehicles and Boats
- Hunting

Consideration for boat access and use will require additional evaluation as the design progresses for the Reservoir. The bottom gradient of the Reservoir generally slopes downward toward the northwest of the site. The current hydrological and hydraulic analysis for the Reservoir indicates that more than three feet of water will be retained in Cell 1 over a modeled eight-year period of record. Therefore, it appears feasible to

provide public access facilities for low-horsepower boating on the Reservoir in Cell 1. Refer to **Section 18 C** for discussion regarding public access.

For the 30% preliminary design, a boat ramp can also be added I Cell 2 for the primary use to provide motorized (air boat and motor boat) access for SFWMD staff. Duign periods when there is adequate water depth in Cell 2, the boat ramp could also be available for public use. A summary of water depths and durations in Cell 2 will be prepared for review and will be included in the Final BODR. Centralizing ramps and facilities will be considered as we move into the 30% preliminary design.

Areas within the project boundary will need to be identified and reserved to accommodate facilities necessary to support the potential activities. The facilities will need to be ADA compliant and would include:

- Visitor Kiosk
- Viewing Tower
- Porta Potties
- Parking
- Trail Heads
- Picnic Shelters and Tables
- Motorized Boat Ramps
- Non-Motorized Boat Ramps

Figures 18B-01 (Alternative A) and **Figure 18B-02** (Alternative B) in **Section 18** of the Appendices were developed to illustrate how the recreation activities and features might be incorporated into the Reservoir Project boundaries. With safety as a major concern, the water control structures and pump station operations were given much consideration. The activities and features illustrated in these two figures were identified based on presenting active recreation outside the footprint of the Reservoir dam and presenting the passive activities along the top of the dam or within the proposed cells.

Refinements to the design of the perimeter/seepage canal such as providing littoral shelves to promote certain plant and animal species could be considered for positive impacts to recreational activities.

While there is interest in incorporating recreation features into the project, it is critical that the activities and facilities are coordinated with the purpose, elements and operations of the Reservoir.

In June 2004 a CERP Recreational Conceptual Plan was prepared by the SFWMD. Please refer to **Figure 18B-01** in **Section 18** of the Appendices. The plan presents locations for activities and facilities related to active and passive recreation. The conceptual plan was developed without the benefit of knowing the current Reservoir footprint alternatives, the optional cell configurations, and the knowledge of the pump station and the drainage structure locations and operational plans or an understanding of the proposed scope of the canal improvements.

C. Access

Access to and within the Reservoir has been discussed earlier in **Section 12**. When recreational features are finalized and become part of the project, the access will need to be reevaluated to consider public pedestrian and vehicular recreational access. Because of safety issues related to the Reservoir operations it will be important to make the recreational access user friendly without compromising the user's safety.

Provision of public access to the Reservoir facility requires minimum parameters be considered in the design process. These parameters include:

- Upgraded access roadways allowing two-way traffic
- Parking area large enough park vehicles with trailers (boats and equestrian)
- Parking area for cars, pickups, SUVs, etc.
- Access paths to top of dams
- Handicapped access to top of dams
- Possible boardwalks
- Portable restroom facilities
- Informational kiosks
- Security gating, chains, and other deterrents to limit access to restricted areas
- Occasional turnouts along dams
- Occasional turnouts for picnic tables and shelter for public
- Water control structures will be designed to allow easy passage of pedestrian, bicycle and equestrian traffic such that structures can operate uninhibited and maintenance activities can be conducted simultaneously.
- Upgraded service ramp to facilitate public boat launch

D. Security

1) Fences and Gates

Security at and around the elements of the Reservoir will be a function of safety, liability and need for access. The Reservoir site is located in a remote rural area and will initially be surrounded by undeveloped agriculture land. As recreation features become part of the project, access to and around the site may not be limited to just the SFWMD operators and personnel. Although the Reservoir site will be accessible to the general public, entry to the site will be discouraged (until such time as recreational features are accessible) through the use of signs.

It is not anticipated that the entire site will be surrounded by a perimeter fence; however, for safety and liability reasons, it is recommended that the pump stations and drainage structures have controlled access through the use of fencing and access gates. The fencing will be vinyl coated chain link, 6 ft. high with a barbed wire outrigger on top. The gates will be vinyl coated chain link sized to accommodate the required access. The structure gates will typically be locked and controlled by the staff at the pump station. The access to the pump stations could be controlled by electric gates with a keyless entry system.

2) Site Monitoring, Pump Stations

A closed circuit television (CCTV) will be provided to monitor the building exterior. At a minimum two cameras will be positioned on opposite corners of the building to cover two sides each. Included will be pan-tilt-zoom (PTZ) capability to monitor doorways and all access drive gates, trash rake mechanism(s), antennae tower and Reservoir level gages. Additional camera(s) may be required to examine the intake and suction level gage. If remote monitoring is required, cameras will be provided inside. Segmented color monitors for simultaneous viewing of multiple images, image recorder, switching and remote control capability will be provided.

3) Building Access, All Buildings

Items that will be considered when controlled access to buildings will include:

(a) Door Position Switches

(b) Beam detectors across main floor, or interior motion sensors with movement detections and alarms to SCADA.

(c) Keypad access with timed alarm override

All security features and elements will be coordinated with the SFWMD prior to final design.

Section 19 – Public Impacts and Issues

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 19 in the Appendices.

Implementation of the Reservoir Project will have impacts on the local communities and on southwest Florida in general. In coordination with the ongoing PIR effort, the proposed design development for the Reservoir will contemplate public impacts that will result from the project.

During subsequent design development and construction of the Reservoir Project the SFWMD will work proactively with stakeholders to discuss project development, key milestones, and issues. The use of stakeholder workshops has been integrated into the design development process for this project. These stakeholder forums will provide information regarding project development.

Stakeholders on the project include:

- Recreation
- Agricultural (Agri-Business)
- IMPACT
- Watershed Council
- Local Government
- Environmental

Information and comments received from the stakeholder briefings will be integrated with the technical design review comments and used for the formal Water Resources Advisory Committee (WRAC) briefing. All comments will be carried forward for evaluation in subsequent design development for the project.

Section 20 – Operation & Maintenance

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 20 in the Appendices.

Development of the Reservoir Project will require continuous coordination with the SFWMD Operations and Maintenance Department. As the SFWMD O&M is the ultimate owner and keeper of the project, it is imperative that all design development be vetted through O&M representatives that will provide feedback on the projects. It is planned to use workshop formats to review design concepts with O&M staff to optimize review cycles and minimize schedule impacts to key SFWMD staff. In addition to design concepts, the following are other key items that will be addressed in the final design phases.

A. Startup and Operation

The project design team needs to work with O&M to develop a strategy for project start-up and turn over from the contractor to the SFWMD. This critical step in project implementation often gets overlooked and can lead to serious resource constraints as several projects might be brought on line at the same time. Similarly, making sure that the SFWMD O&M staff is provided with the appropriate training and instruction prior to start-up will reduce the learning curve necessary to become fully operational. Information included in the contract documents will dictate the start-up requirements for the project, including:

- Start-Up Objectives
- Comprehensive Start-up Plan
- Work Tests
- Delivery Acceptance Tests
- Installation Verification Inspections
- Pre-Start-Up Testing
- Performance Testing
- Start-Up and Hand-Over
- Interim O&M Plan

B. O&M Manuals

During the final design development phase, provisions for O&M manual development will be specified. The use of innovative methods for assembly and delivery of maintenance manuals will help to ensure that these products are delivered before the project is turned over. Operations and maintenance manuals for this project will be prepared by the design consultant, and shall include the following:

- Process descriptions, design criteria and process component listings;
- System descriptions with schematic line diagrams and photographs;
- Process evaluation and control descriptions;
- Start up, normal running, shutdown and emergency procedures;
- Control settings with daily/ seasonal change indicators and alarms;
- Safety, health and emergency procedures
- Record Drawing Requirements
- Spare Parts Inventory

C. Coordination Objectives

The following is a brief discussion of coordination efforts that have been implemented to date with the SFWMD O&M staff. Also included is a tentative timeline for future briefings that will be presented to O&M staff regarding the project design development. The goal of coordination efforts will be to obtain as much feedback as possible from O&M regarding the proposed project development with the primary focus being on delivering the product that O&M is looking for.

An initial meeting was held with O&M staff in May 2003 to discuss the proposed operational requirements for the facility. Issues talked about at the meeting included the potential for pre-storm drawdowns.

Three copies of the Conceptual Design Report (Draft DDR) were forwarded to the O&M Department for review and comment in August 2003.

Future briefings will be provided to O&M staff for the following stages of design development:

- Pre-Final Basis of Design Report (February 2005)
- Preliminary Design Development (June 2005)

- Preliminary Design Report (June 2006)
- Final Design Development (April 2007)

Section 21 – Value Engineering Reviews

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 21 in the Appendices.

The SFWMD strongly supports the use of value engineering (VE) to bring the knowledge of experience into the design process. It is contemplated that the C-43 West Storage Reservoir Project will go through VE following the Preliminary Engineering (30%) design development. At this stage of development the basic design features and concepts will be determined such that a VE team can perform a meaningful VE session. The results of the VE session will be measured against other design drivers to determine the feasibility of implementing VE recommendations.

It is contemplated that the U.S. Army Corps of Engineers will be involved in the VE process for this project.

Section 22 – Opinion of Probable Cost

The reader should note that all tables, figures and supporting information that are not "embedded" in this Section are contained in Section 22 of the Appendices.

A. Introduction

1) Perimeter Dam

The Reservoir is being designed to store water during the periods of high flow in the C-43 Canal, which will then be released during periods of low flows in the C-43 Canal to maintain flows within desirable ranges for estuarine health. The present concept of the Reservoir is to store up to 170,000 acre-feet of water within a perimeter earth dam which will enclose approximately 10,200 acres. The Reservoir will be split into two interconnected cells (Alternative A). An alternative configuration, also being considered, is to increase the size of the Reservoir to approximately 12,200 acres by adding a third interconnected cell (Alternative B). This would lower the perimeter dam height while maintaining the same storage capacity. The final area of the Reservoir will depend on the selection of the design configuration, two-cells or three-cells.

The unit prices for Dam embankment related work were developed using data from the 2004 National Construction Cost Estimator's Florida Heavy Construction Cost Estimator Guide. This information is available from Get-A-Quote.net. See the Appendix **Section 22** for detailed cost estimates and cost data.

2) Pump Stations

The Reservoir will be filled by utilizing either one Pump Station (Option 1) with a 1,500 CFS capacity located at the northwest corner of the Reservoir, or by two Pump Stations (Option 2), the first with a 1,250 CFS capacity located at the northwest corner of the Reservoir and the second with a 500 CFS capacity located near the southeast corner of the Reservoir. The costs of these pump stations is also a function of the fuel used to operate the large motors needed to power the pumps.

The unit prices for the pump stations were developed using 2004 editions of Richardson - Process Plant Construction Estimating Standards; Means Cost Data - Building Construction Cost Data; and Means Cost Data - Sitework & Landscaping Construction Cost Data.

3) Control Structures

As currently planned, the Reservoir will have approximately 15 control structures. The actual number of structures involved will depend on the alternative selected for the final configuration. Gated control structures will be utilized to release water from the Reservoir, control water elevations in the Perimeter Canal (seepage canal) and control water elevations in the Roberts Canal. Gated culverts will be utilized to release water into the NE Rim Ditch and maintain flows between the Reservoir cells. A standard culvert will allow water to flow under the access road at the intersection of the LPDD Canal and the Perimeter Canal. Weir structures are proposed to maintain and control flows into Crawford Canal, Banana Branch, and Fort Simmons Branch as well as separate water in the Perimeter Canal from the Townsend Canal.

The unit prices for the control structures were developed using data from the 2004 National Construction Cost Estimator's Florida Heavy Construction Cost Estimator Guide. This information is available from Get-A-Quote.net

4) Mobilization and Contingencies

The cost to mobilize enough construction equipment to efficiently construct the Reservoir will be significant. The screening and soil cement plants will be relocated several times during the construction of the project. We have added 10% to the probable cost of the Reservoir earthworks, control structures and pump stations for the mobilization of construction equipment and shipping of materials.

The nature and size of this project is such that there will be some unanticipated items. This is particularly true at this stage of design. Some of the factors considered in setting the contingency are:

- S-78 Tower Site
- Sodding Exterior Embankment
- Site Remediation
- Access Roadway Paving
- Costs for Clearing & Grubbing
- Manufactor Costs of Filter Sand
- Recreational Features
- Environmental Mitigation

- Fluctuation of Cement Costs

We have added 40% of the probable cost of the Reservoir earthworks, control structures and pump stations for contingencies.

Table 22G-01 will have the Mobilization and Contingency amounts combined in one column that adds 50% to the estimated probable costs.

B. Cost as Function of Dam Height

1) General

The largest factor in the cost of the Reservoir is the construction of the earth perimeter dam. The cost of the dam will depend on the cross-section and the height. The height is a function of area, storage capacity, design water surface elevation (normal pool) and the freeboard needed for safety during wind storm events. Considering these factors is there an optimum dam height that will give the lowest cost per acre-foot of storage capacity?

There are many factors that come into play in deciding the optimum height of a dam: political, environmental, safety and cost. The storage capacity of the Reservoir using a perimeter dam will increase nearly on a straight line function vs. the dam height. The construction cost vs. the dam height is also nearly a straight line function but the slopes of the two functions are not the same. Therefore the function of the unit cost per acre-foot of storage vs. dam height is a curvelinear function similar to a logarithmic curve. We have developed the following chart shown as **Figure 22B-01** to show the Cost Per Acre-Foot of Storage as a function of normal pool elevation. The preliminary design and the costs of the two-cell option are based on a normal pool elevation of 39.2 ft. Only the two-cell option was analyzed as a similar curve would result from the three-cell option.

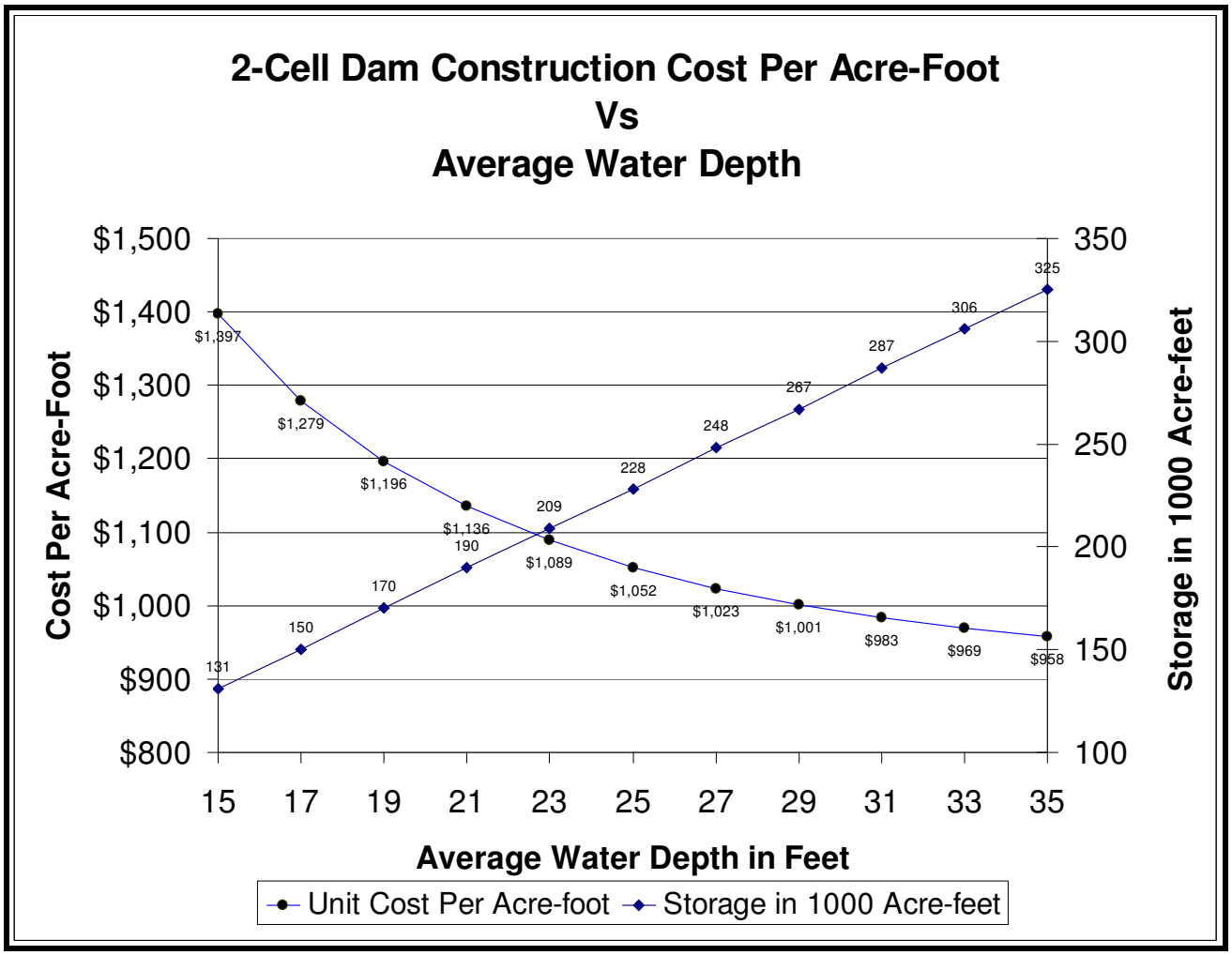


Figure 22B-01 – Average Water Depth vs. Cost Per Acre-Foot and Storage Cost Comparison

2) Chart Criteria

It is necessary to note that the data used in this chart is based on the following criteria which could skew the results.

(a) The assumption was made that the basic typical cross section of the dam would not change with height. Dam stability calculations may require changes in the typical sections for a higher structure.

(b) The top of dam elevation is approximately 10 ft. above the normal pool elevation based on freeboard calculations.

(c) The centerline of the perimeter dam was maintained at the same location as the design water surface elevations were changed. The additional height might require adjustment of the centerline location that would reduce the storage capacity and increase the unit price for storage.

(d) The cost does not take into account the difference in operation costs for pumping against a higher head to fill the impoundment.

(e) The cost does not take into consideration the cost of the water lost from the impoundment due to increased seepage that will occur as the water depth is increased.

(f) The cost does not take into account the added costs of increased structure height for control structures and pump stations.

(g) The only costs considered were the estimated construction costs for the perimeter and separator dam embankment.

The purpose of **Figure 22B-01** is to get a general "visual" idea of how storage cost varies with dam height (water surface elevation) for a large Reservoir. It was prepared to determine if there is an optimum dam height. While it is not apparent what the "optimal" height really is, it is apparent that it is economical to increase the dam height above the current design and thus reduce the cost per acre-foot of storage.

Again, only the Reservoir "earthwork" was used in this comparison as the earthwork cost would certainly be the significant incremental cost. It is recognized that costs discussed above would increase with dam height, but the incremental cost increase would be insignificant when compared to the earthwork costs.

Figure 22B-02 shows the relation of the storage capacity of the 2-Cell Alternative A, Option 2 Reservoir to the total estimated capital costs. The total capital costs increase dramatically with the increase in storage capacity.

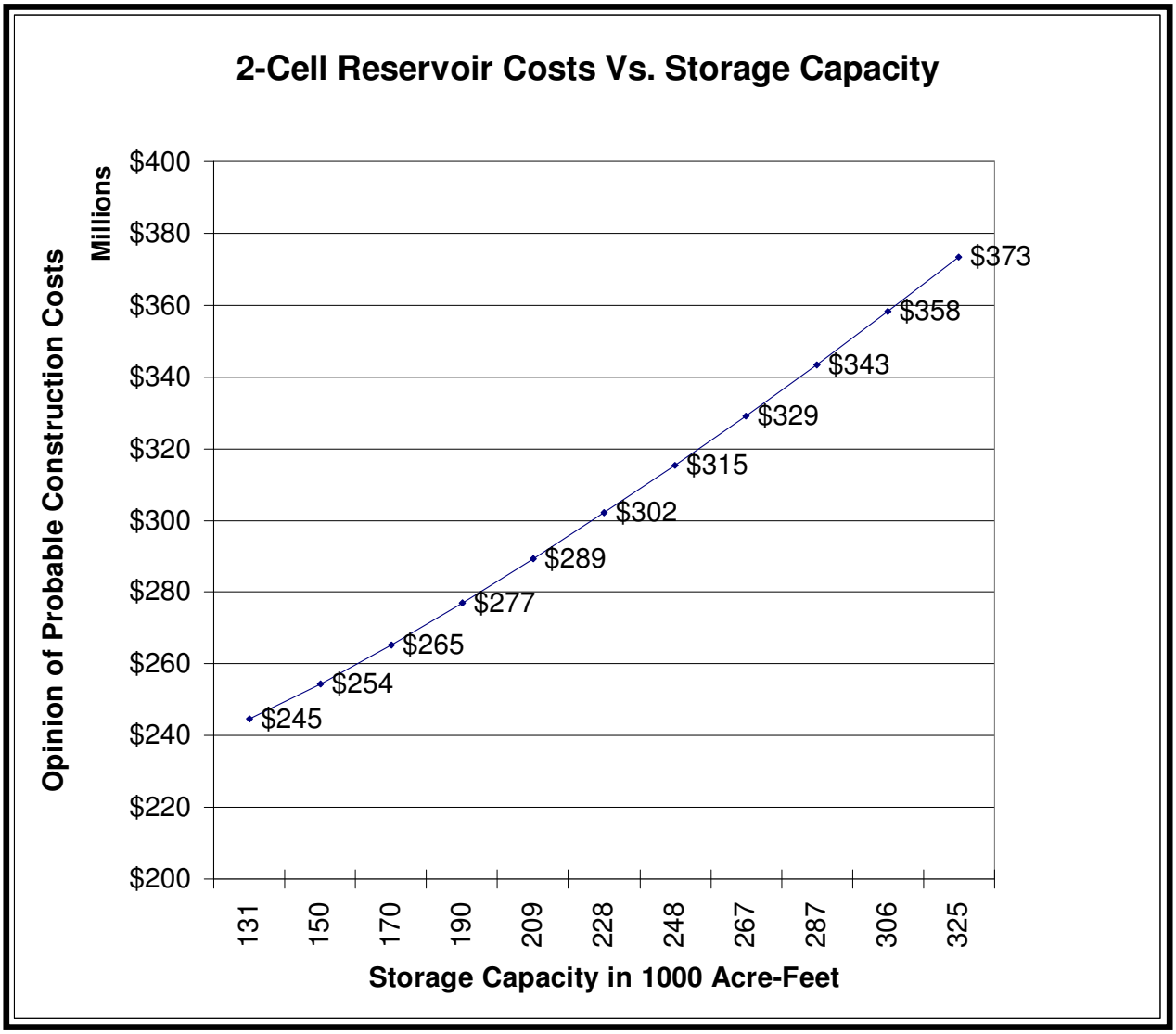


Figure 22B-02 – Total Project Cost vs. Storage Capacity

The final decision on the height of the dam will be based on a storage requirement that takes into account the PIR findings, the total capital costs and the stakeholder needs and concerns.

C. Cost Model Development

1) Perimeter Dam

(a) General

The perimeter dam for the Reservoir will have a variable cross section; however certain components are consistent throughout. See **Figures 8A-1 and 8A-2** in **Section 8** of the Appendices.

The cost was developed making the assumption, based on the soil borings taken at the site, that all basic soil materials would be available on site from borrow pits. Naturally occurring materials can be processed by screening when necessary to provide specific gradation requirements. Soil cement can be processed on site by adding cement to natural materials and mixing through a pug mill.

(b) Cross Section

The perimeter dam will have the same relatively impervious clay core throughout its length. On the exterior side of the dam there will be a sand drainage blanket system in place to lower the hydraulic profile through the dam to ensure stability. This sand drainage blanket will be enveloped in a geo-textile filter to prevent infiltration of clay and silt particles that would reduce the systems flow capacity. The impervious core with its related drainage blanket will be overlaid with random fill consisting of sand, siltly/sand, clayey/sand or sandy/clay. On the exterior side the horizontal section of the drainage blanket is overlaid by a random fill bench to an elevation that is 4 ft. about the existing ground. The width of this bench and the drainage blanket it covers varies between 30 ft. and 150 ft. based on soil and land use conditions.

The interior portion of the perimeter dam is protected from wave action by a soil cement plate covering the face of the dam below the design normal pool elevation and by stepped soil cement layers above that elevation. The soil cement plate is underlain by a drainage blanket that will allow swift drainage of the dam's embankment during periods of drawdown, thus preventing possible failure of the interior dam face.

It has been determined from subsoil investigation that the relatively impervious clay, the random fill, the soil for the soil cement and the sand for the drainage blanket can all be obtained from borrow pits located within the perimeter dam. The sand will require screening to obtain material with the correct gradation for the drainage blanket. The soil cement will be processed at the borrow pit sites.

(c) Drainage Blanket

The cost of the sand drainage blanket includes processing the sand material from the borrow pits to ensure proper gradation and the ability to maintain the desired flows, geo-textile filter fabric to encase the sand blanket, transportation and placement of the sand. This was compared to using a crushed stone drainage blanket and found to be much more cost

effective than the crushed stone. A cost comparison was also made between using the sand drainage blanket and using a geo-textile drainage net that would allow the similar flows in a thin layer. It was determined that using the state of the art geo-textile drainage net would increase the cost of the dam embankment by nearly 10%.

(d) Soil Cement

The cost of the soil cement includes processing the soil material obtained from the borrow pits through a pug-mill to add 7% cement uniformly distributed through the mixture, transportation and placement on the embankment. The use of stone rip rap was also considered. The base material and placement cost to cover one square yard of the dam surface with a 1.33 ft. thickness of soil cement is approximately $36.00, while the cost to place riprap would include a geo-textile filter blanket at $2.71 per square yard, a one foot thick layer of bedding stone (10 pound to 99 pound pieces) at approximate $12.00 per square yard and a two foot thick layer of armor stone (50 pound to 1000 pound pieces) at approximately $68.00 per square yard for a total cost of approximately $83.00 per square yard.

(e) Separator Dams

The interior of the impoundment will be divided into cells by separator dams. The separator dams are to be constructed of random fill and are protected by encasing the entire exposed surface of the separator dams with a soil cement plate.

(f) Clearing and Grubbing

The existing land use is primarily citrus groves. This will require that the entire Reservoir site be cleared and grubbed and the citrus trees be removed and disposed of. It would not be necessary to grub out all of the roots except in the areas to be stripped for construction of the perimeter and separator dams.

(g) Stripping

Because of the existing land use it was determined that the entire area for the construction of the perimeter dam along with the perimeter seepage canal, the perimeter road and the separator dams would need to be stripped to an average depth of 18-inches to eliminate the bulk of organics. This is also true of the borrow pit areas. The stripped materials would be spread and not used in the construction of the perimeter dam nor the separator dams.

(h) Calculation Methods

An EXCEL program was written to calculate the areas and volumes of the various components in the perimeter features and the separator dams. This program used the various typical sections for the dams in conjunction with the existing ground elevations and the water table elevations from the subsoil investigation to calculate end areas at the major profile break points along the perimeter dam centerline profile and the separator dams centerline profiles.

The EXCEL program also assigned sections of the dam to the various borrow pits and determined the area for each borrow pit needed to provide the impervious clay for the core and the total embankment materials to construct the project. These calculations were based on subsoil investigation of material depths found in each borrow pit area. This information was then utilized to compute the cubic yard haul required to move the material from the borrow pit to the final location in the dams. Double handling of sand material that overlaid the clay in the borrow pit was taken into consideration since the impervious core would have to be constructed prior to the random fill.

(i) Power Distribution Line

There is a 15KV 3-phase distribution line that traverses the site from west to east. This line will need to be relocated to the north side of the site. The work will require the construction of approximately 6 miles of new line that can be placed in service prior to the removal of the existing facility. This replacement line will be 4/0 ACSR-3PH and will cost $45,000 per mile.

(j) Seeding, Fertilizer and Watering

The outside slope of the dam structure will need to be stabilized to prevent erosion. This surface will need to be shaped and seeded. The seeding should include two types of seed. Part of the seeding needs to be quick growing annual grasses that will quickly establish a layer of vegetation. Slower germinating perennial grasses also need to be included in the seeding to establish a permanent ground cover that will resist erosion. In order to get a healthy stand of grass the areas should be watered and fertilized during the early stages depending on the time of year the plantings are made. It would be preferable to get the ground cover established prior to the start of heavy rains.

An alternate solution would be to sod the outside slope of the dam structure. This would be a more effective solution and would take less time to become stabilized. The cost of using sod however is approximately 20 times the cost of seeding and would add about

$20,000,000 dollars to the cost of the reservoir. We have not included the cost of sod in the opinion of cost. The use of sod in certain areas that may be more susceptible to erosion may be cost effective. Such areas would be around structures, access roads, recreation facilities and pump houses.

2) Control Structures

There are several control structures required to allow for discharge from the various cells included in the impoundment design. The surrounding area includes irrigated agricultural use lands. Control structures are also included to maintain flow in the various irrigation canals around the perimeter of the impoundment and to provide flood control during high flow periods.

The preliminary plans were used to determine quantities for the various components of the structures involved for each alternative and option. Unit prices were applied to these quantities to develop the costs. See the Appendix **Section 22** for detailed cost estimates of the Control Structures.

3) Pump Stations

(a) General

The operation of the Reservoir will depend on the utilization of either one large pump station (Pump Station 1) located at the northwest corner of the impoundment (referred to as Option 1 below) or with one large Pump station located at the northwest corner of the impoundment and a second smaller pump station (Pump Station 2) located near the southeast corner of the impoundment (referred to as Option 2 below).

(b) Fuel Options

There are various discharge arrangements that can be considered for Pump Station 1. The various arrangements have differing characteristics that may make one arrangement more desirable than the others. The three discharge arrangements under consideration is a through the embankment flap gate arrangement (FGA), a through the embankment siphon arrangement (SA) and an over the embankment arrangement (OEA). The estimated cost for the Pump Station is dependent upon the discharge arrangement, the type of fuel and the pumping capacity. **Table 22C-01** provides a summary of costs for the various combinations of discharge and fuel used.

Option	Pump Station	Discharge Arrangement	Capacity	Fuel		
				Diesel	Natural Gas	Electric
1	1	FGA	1,500 CFS	$25,410,000	$26,330,000	N/A
1	1	SA	1,500 CFS	$28,220,000	$29,140,000	N/A
1	1	OEA	1,500 CFS	$25,780,000	$26,700,000	N/A
2	1	FGA	1,250 CFS	$22,510,000	$23,430,000	N/A
2	1	SA	1,250 CFS	$25,320,000	$36,240,000	N/A
2	1	OEA	1,250 CFS	$22,880,000	$23,800,000	N/A
2	2	FGA	500 CFS	$12,820,000	$13,070,000	$10,520,000

Table 22C-01 - Pump Station Fuel Option Cost Comparisons

(c) Probable Cost Option

Our probable cost study for Option 1 is based on our recommendation to use the through the embankment flap gate discharge arrangement using natural gas for fuel. Our probable cost for Option 2 is based on using the through the embankment flap gate arrangement using natural gas for fuel at PS-1 and using electric motors for PS-2.

The quantities takeoff from the preliminary design of the pump stations have been used along with unit prices to estimate the probable cost of the various pump station options. See Appendix **Section 22** for detailed cost estimates of the Pump Stations

D. Alternative A, Two-Cell Alternative Costs

1) Dam Embankment and Seepage Canal Costs

In the two-cell alternative the perimeter dam is approximately 16.35 miles in length and there is only one separator dam that is approximately 2.88 miles in length. The normal pool elevation of 39.4 ft. was set to maintain the design storage capacity. A freeboard requirement above the normal pool elevation of 10 ft. was used to determine the top of dam elevation for the perimeter dam. The separator dams have been designed with a 5 ft. freeboard. **Table 22D-01** shows the earthwork costs.

Two-Cell Earthworks Costs

Item	Unit Prices***	Quantities	Cost
Seepage Canal Dry Excavation per CY	$1.22	452,135	$551,604
Seepage Canal Wet Excavation per CY	$1.88	2,507,473	$4,714,050
Borrow Material Sand Excavation per CY	$1.22	5,203,769	$6,348,598
Borrow Material Clay Excavation per CY	$1.40	5,792,486	$8,109,481
Borrow Material Limestone Excavation per CY	$1.88	1,378,682	$2,591,922
Haul CY-Mile	$1.11	7,975,353	$8,852,642
Perimeter Road Fill Spread Compaction per CY	$1.19	600,606	$714,721
Drainage Blanket Screening Costs per CY	$2.96	1,277,844	$3,782,417
Drainage Blanket Spread per CY	$0.87	1,277,844	$1,111,724
Drainage Blanket Compact per CY	$1.71	1,277,844	$2,185,113
Total Impervious (Core)* Spread Compact per CY	$1.19	3,608,059	$4,293,590
Total Random Fill Spread and Compact per CY	$0.90	2,736,592	$2,462,933
Total Silty Sand Spread and Compact per CY	$0.90	2,971,377	$2,674,239
Total Soil Cement** per CY	$81.70	710,209	$58,024,060
Rehandle costs Sand and Canal Excavation per CY	$0.67	8,163,377	$5,469,462
Strip Volume (Waste?) Excavate per CY	$0.48	2,810,346	$1,348,966
Strip Volume (Waste?) Haul and disposal per CY-mile	$1.11	2,810,346	$3,119,484
Clearing and Grubbing per acre	$739.00	10,166	$7,512,674
Filter Fabric Area to encase Drainage Blanket per SY	$2.71	3,931,322	$10,653,882
Replace three phase power line per mile	$45,000	6	$270,000
Seeding, fertilizer and water per acre	$1,012.82	812	$822,670
Estimated Earthworks Cost			$135,614,231

Notes: * Includes core and key trench
** Includes stair step and plate
*** See Pricing Data Sheets

Table 22D-01 - Alternative A - Earthwork Costs

2) Control Structures

(a) Alternative A, Option 1

Option 1 requires a total of 12 control structures, including 3 gated outlets, 2 gated spillways, 4 weirs, 1 controlled weir, 1 culvert and 1 equalization culvert.

(b) Alternative A, Option 2

Option 2 requires a total of 13 control structures, including 3 gated outlets, 3 gated spillways, 4 weirs, 1 controlled weir, 1 culvert and 1 equalization culvert.

Table 22D-02 summarizes control structure costs for Alternative A.

ALTERNATE "A" CONTROL STRUCTURE COSTS		
Description	**Option 1**	**Option 2**
Sitework	$1,533,474	$1,928,016
Concrete	$2,076,139	$2,393,317
Metals	$276,666	$301,474
Doors and frames	$33,499	$36,562
Finishes	$118,800	$158,761
Gates	$1,258,462	$1,451,761
Special construction	$90,000	$105,000
Electrical	$750,000	$875,000
TOTAL	$6,157,290	$7,249,401

Table 22D-02 - Alternative A - Control Structure Costs

3) Pump Stations

(a) Alternative A, Option 1

Option 1 consists of one pump station with a capacity of 1,500 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP natural gas fueled engines and 2 each 250 CFS pumps powered by 1,600 HP natural gas fueled engines. The pump station discharge arrangement is through the dam discharge pipes with flap gates to prevent backflow.

(b) Alternative A, Option 2

Option 2 consists of two pump stations. PS-1 has a capacity of 1,250 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP natural gas fueled engines and 1 each 250 CFS pump powered by a 1,600 HP natural gas fueled engine. The PS-2 has a capacity of 500 CFS produced by 2 each 250 CFS pumps powered by 1,000 HP electric motors. The pump station discharge arrangements are through the dam discharge pipes with flap gates to prevent backflow.

Table 22D-03 provides Alternate A pump station cost comparisons.

ALTERNATE "A" PUMP STATION COSTS		
Description	**Option 1**	**Option 2**
Sitework	$189,627	$273,500
Building	$7,818,664	$12,734,195
Electrical	$932,495	$1,315,094
Mechanical	$17,389,205	$19,621,887
TOTAL	$26,329,991	$33,944,676

Table 22D-03 - Alternative A - Pump Station Costs

E. Alternative B, Three-Cell Alternative Costs

1) Dam Embankment and Seepage Canal Costs

In the three-cell alternative the perimeter dam is approximately 19.20 miles in length and there is one separator dam that is 2.88 miles in length and one separator dam that is approximately 1.81 miles in length. The main difference in these two alternatives is the addition of a third cell which is completely outside of the perimeter of the two-cell alternative. This added area means that the normal pool elevation can be lowered and still maintain the design storage capacity. The normal pool elevation used was 37.4 ft. The freeboard on the perimeter dam was 10.3 ft., calculated based on the lesser water depth. The freeboard requirement above the normal pool elevation is used to determine the top of dam elevation for the perimeter dam. The separator dams have been designed with a 5 ft. freeboard.

Table 22E-01 shows the Alternative B earthwork costs.

Three-Cell Earthworks Costs			
Item	Unit Prices***	Quantities	Cost
Seepage Canal Dry Excavation per CY	$1.22	507,018	$618,561
Seepage Canal Wet Excavation per CY	$1.88	3,281,739	$6,169,669
Borrow Material Sand Excavation per CY	$1.22	3,823,790	$4,665,024
Borrow Material Clay Excavation per CY	$1.40	5,675,773	$7,946,082
Borrow Material Limestone Excavation per CY	$1.88	2,144,087	$4,030,883
Haul CY-Mile	$1.11	8,459,404	$9,389,938
Perimeter Road Fill Spread Compaction per CY	$1.19	769,257	$915,416
Drainage Blanket Screening Costs per CY	$2.96	1,333,142	$3,946,101
Drainage Blanket Spread per CY	$0.87	1,333,142	$1,159,834
Drainage Blanket Compact per CY	$1.71	1,333,142	$2,279,673
Total Impervious (Core)* Spread Compact per CY	$1.19	3,516,917	$4,185,132
Total Random Fill Spread and Compact per CY	$0.90	2,652,774	$2,387,497
Total Silty Sand Spread and Compact per CY	$0.90	3,425,686	$3,083,117
Total Soil Cement** per CY	$81.70	828,455	$67,684,739
Rehandle costs Sand and Canal Excavation per CY	$0.67	7,612,547	$5,100,406
Strip Volume (Waste?) Excavate per CY	$0.48	3,076,706	$1,476,819
Strip Volume (Waste?) Haul and disposal per CY-mile	$1.11	3,076,706	$3,415,143
Clearing and Grubbing per acre	$739.00	12,260	$9,060,140
Filter Fabric Area to encase Drainage Blanket per SY	$2.71	4,567,604	$12,378,208
Replace three phase power line per mile	$45,000	6	$270,000
Seeding, fertilizer and water per acre	$1,012.82	944	$995,818
Estimated Cost Earthworks Cost			$151,168,202

Notes: * Includes core and key trench
** Includes stair step and plate
*** See Pricing Data Sheets

Table 22E-01 - Alternative B Earthwork Costs

2) Control Structures

(a) Alternative B, Option 1

Alternative B, Option 1 requires a total of 14 control structures, including 4 gated outlets, 2 gated spillways, 4 weirs, 1 controlled weir, 1 culvert and 2 equalization culverts.

(b) Option 2

Alternative B, Option 2 requires a total of 15 control structures, including 4 gated outlets, 3 gated spillways, 4 weirs, 1 controlled weir, 1 culvert and 2 equalization culverts.

Table 22E-02 provides Alternative B control structure cost comparisons.

ALTERNATE "B" CONTROL STRUCTURE COSTS		
Description	**Option 1**	**Option 2**
Sitework	$1,450,899	$1,717,935
Concrete	$2,271,437	$2,543,787
Metals	$306,511	$331,320
Doors and frames	$39,374	$42,187
Finishes	$118,800	$158,271
Gates	$1,772,272	$1,965,570
Special construction	$120,000	$135,000
Electrical	$1,000,000	$750,000
TOTAL	$7,079,293	$7,644,071

Table 22E-02 - Alternative B - Control Structure Costs

3) Pump Stations

(a) Alternative B, Option 1

Option 1 consists of one pump station with a capacity of 1,500 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP natural gas fueled engines and 2 each 250 CFS pumps powered by 1,600 HP natural gas fueled engines. The pump station discharge arrangement is through the dam discharge pipes with flap gates to prevent backflow.

(b) Alternative B, Option 2

Option 2 consists of two pump stations. PS-1 has a capacity of 1,250 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP natural gas fueled engines and 1 each 250 CFS pump powered by a 1,600 HP natural gas fueled engine. PS-2 has a capacity of 500 CFS produced by 2 each 250 CFS pumps powered by 1,000 HP electric motors. The pump station discharge arrangements are through the dam discharge pipes with flap gates to prevent backflow.

Table 22E-03 provides Alternative B pump station comparisons.

ALTERNATE "B" PUMP STATION COSTS		
Description	**Option 1**	**Option 2**
Sitework	$189,627	$273,500
Building	$7,818,664	$12,734,195
Electrical	$932,495	$1,315,094
Mechanical	$17,389,205	$19,621,887
TOTAL	$26,329,991	$33,944,676

Table 22E-03 - Alternative B - Pump Station Costs

F. Seepage

1) General

The Reservoir will experience seepage through the soil below the dam structure. The quantity of seepage will vary according to the pool elevation, the soil conditions and the perimeter length of the dam.

During the C-43 Canal low flow periods water is to be released to maintain the desired flow in the C-43 Canal. During those periods the release rate through the control structures is reduced by the seepage rate. This is accounted for in the model.

During high flow periods when water is being pumped into the reservoir the pumping duration can be extended to compensate for the seepage with little effect on the operation of the reservoir.

2) 2-Cell Option

At normal pool the seepage rate estimated for the 2-cell option is 147 acre-feet per day. At this rate in a year's time approximately 54,000 acre-feet would seep out of the 2-Cell Reservoir (this assumes normal pool throughout the year). This is 32% of the Reservoir's capacity. If it is desired to make up for the seepage on a daily basis it would require pumping 6,400,000 cubic feet of water per day from the seepage canal back into the Reservoir. Utilizing one of the electric powered 250 CFS pumps at PS-2 it would require approximately 7 ¼ hours of pumping per day. The cost of the electrical power to do this would be approximately $3.50 per acre-foot or $515 per day.

3) 3-Cell Option

For the 3-Cell Reservoir the perimeter dam length is increased by approximately 4 miles and the subsoil in the third cell has a higher permeability rate than the subsoil in the other two cells. These two factors would increase the seepage at normal pool to 243 acre-feet per day for the 3-Cell option. In a years time over 50% of the reservoir's capacity would seep out. The daily pumping costs would increase to $850.50, a 65% increase over the pumping cost for the 2-Cell option.

4) Slurry wall

The seepage for the 3-Cell option could be reduced to be comparable to the 2-Cell option by constructing a slurry wall along the exterior portion of the third cell's perimeter to a bottom elevation of 0 ft. The cost to construct this slurry wall is estimated to be $2,600,000. It would take approximately 20 years of daily pumping savings to recapture the capital investment for the slurry wall.

G. Summary

The probable cost of the Reservoir will ultimately depend on the total storage capacity desired, the configuration of the perimeter dam and the diversity of pumping options selected. It will also depend to a lesser extent on environmental and land use options including public access. These numbers can also change because of land costs if additional land needs to be acquired for construction. An important and rather unpredictable factor will be the inflation of construction costs between now and the actual construction date. The past year has seen large cost increases in construction materials because of the changes in the world economy and the demand for cement and steel as more countries become part of the industrial base. The following **Table 22G-01** summaries the current estimated probable costs developed for each of the alternatives and options as described above:

Alternative	Description	Earthwork Costs	Pump Station Costs	Control Structure Costs	10% Mobilization and 40% Contingencies	Total Costs
A1	2-Cell, 1 Pump Station	$135.6	$26.3	$6.2	$84.1	$252.2
A2	2-Cell, 2 Pump Station	$135.6	$33.9	$7.3	$88.4	$265.2
B1	3-Cell, 1 Pump Station	$151.2	$26.3	$7.1	$92.3	$276.9
B2	3-Cell, 2 Pump Station	$151.2	$33.9	$7.6	$96.4	$289.1

Table 22G-01 - Opinion of Probable Cost Summary (All Values in $ Millions)

Note: Refer to Appendix 22 for Detailed Cost Estimate.

Section 23 – Construction Coordination

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 23 in the Appendices.

A. Introduction

1) General

Construction of the Reservoir will be coordinated with the SFWMD Acceler8 Program Construction Manager. The construction management approach for the Acceler8 Program will involve a group of construction specialists that will focus on project bidding and delivery requirements for the project. During subsequent phases of design development the Reservoir design team will consult with the Construction Manager on issues related to project cost estimating and budgeting (including market research), constructability, quality assurance (including materials testing), partnering, construction scheduling, risk management, and construction permitting. The following sections provide an overview of each construction management coordination requirement.

The Reservoir Project limits will encompass between 10,000 and 12,500 acres. The project is not immediately adjacent to any state or county owned highway. The existing land use is primarily citrus groves within which there is a system of irrigation canals. The project will include up to 15 control structures of varying size and 1 or 2 pump stations. The perimeter dam will require in excess of 14,000,000 cubic yards of excavation to construct approximately 12,000,000 cubic yards of embankment. The project is a large scale construction operation with many facets that need to be coordinated; maintaining irrigation for adjacent agricultural lands, access, staging areas, material handling, security, environmental protection, schedule, etc. The list will be long and the coordination will be critical to having a successful project.

Separate bid packages will be considered in preparing the bid documents. Construction may be broken into 4 bid packages for pump equipment, pump station construction, earthwork and structures to distribute the work among large and smaller contractors.

2) Perimeter Dam

The Reservoir is being designed to store up to 170,000 acre-feet of water within a perimeter earth dam. The perimeter dam will be constructed of locally available material. A relatively impervious clay core is to be constructed of clay materials which underlay the site. This material will have to be accessed by removing a sand or silty/sand overburden. A portion of the overburden material will be

processed to provide approximately 1,500,000 cubic yards of clean graded sand to be used in the construction of drainage blankets placed in the dam to control seepage. A portion of the overburden will be mixed in a pug mill to provide approximately 900,000 cubic yards of soil cement to be used as slope protection on the interior surface of the dam. The remainder of the overburden and the excavation for the perimeter (seepage) canal will be used to construct the flattened slopes outside of the clay core to provide stability and maintenance access. For more information on the perimeter dam cross-section see **Figure 8A-1** in **Section 8** of the Appendices.

The perimeter dam is 16.35 miles long in alternative A, the two-cell alternative, with 2.88 miles of separator dams and 19.20 miles long in alternative B, the three-cell alternative, with 4.69 miles of separator dams. It is presently planned to have 7 borrow pit locations in alternative A and 8 in alternative B. For more information on the separator dam cross-section see **Figure 8A-2** in **Section 8** of the Appendices.

3) Pump Stations

The Reservoir will be filled by utilizing either one Pump Station (Option 1) or two pump stations (Option 2). The Option 1 Pump Station has a capacity of 1,500 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP motors and 2 each 250 CFS pumps powered by 1,600 HP motors. The first pump station in Option 2 has a capacity of 1,250 CFS produced by 2 each 500 CFS pumps powered by 3,200 HP motors and 1 each 250 CFS pump powered by a 1,600 HP motor. The second pump station in Option 2 has a capacity of 500 CFS produces by 2 each 250 CFS pumps powered by 1,000 HP motors. The pump station(s) will require large concrete structures to house the pumps and motors as well as to form the pump intake and discharge features.

4) Control Structures

As currently planned, the Reservoir will have approximately 15 control structures. The actual number of structures involved will depend on the alternative selected for the final configuration. Gated control structures will be utilized to release water from the Reservoir, control water elevations in the Perimeter Canal (seepage canal) and control water elevations in the Roberts Canal. A gated culvert will be utilized to release water into the NE Rim Ditch and maintain flows between the Reservoir cells. A standard culvert will allow water to flow under the access road at the intersection of the LPDD Canal and the Perimeter Canal. Weir structures are proposed to maintain and control flows into Crawford Canal, Banana Branch, and Fort Simmons Branch as well as separate water in the Perimeter Canal from the Townsend Canal.

The gates being utilized on the control structures will be special order because of size and head requirements. The gated control structures will require electric power to operate the large gates and some form of backup power will need to be available to operate the gates during storm periods when normal power feeds might be disrupted.

B. Cost Estimating and Budgeting

Cost estimates prepared during subsequent design phases will require coordination with the Construction Manager to evaluate market conditions, resource and material availability, Federal bidding requirements, and overall construction management costs. The project manager must coordinate with the Construction Manager to refine the project budgetary estimates and refine the baseline project schedule.

C. Constructability

Following the preliminary design development (30% design), the project team will convene a meeting of construction industry experts to evaluate the proposed construction methodology for the Reservoir. The experts will be asked to provide feedback related to the reasonableness of the proposed construction approach and proposed alternatives that might improve the efficiency or cost of the project. These ideas will be captured and if reasonable, integrated into the final design phases.

D. Quality Assurance

The Acceler8 Program will develop guidance for project designers related to the quality assurance (QA) program that will be utilized during construction of the project. The use of a standardized QA program will be necessary to ensure consistency across QA programs. The QA program will essentially provide a mechanism to inspect the contractor's work and verify that it maintains compliance with the project construction documents. Integration of the Program's QA initiative into the project design will help carry this initiative into the construction phase. The QA program will integrate material testing requirements that will be required during construction of the project.

E. Partnering

The use of partnering will be recommended for inclusion in the final contract documents that will be prepared for this project.

F. Construction Scheduling

The Construction Manager will be consulted during development of the proposed construction schedule to provide continuity between the design and construction phases. The Construction Manager will evaluate durations for long lead items, availability of resources, seasonal impacts, general economic conditions, and Acceler8 programmatic constraints. The goal will be to include in the Contract Documents a workable schedule that provides work products in a timely manner without adding undue cost for premium time. It has been previously noted that the critical path to the earthwork schedule is based on the moisture content of the clay core, especially during the rainy season.

G. Risk Management

During subsequent phases of design development for the project, the design team will develop risk profiles for the key components of the projects. These risk templates will consider critical risk factors, identify high risk project features, define impact to the overall project, and define possible mitigation measures. A risk management plan will be developed that details these factors and assigns potential contingency budgets to each. This tool will allow the Acceler8 Program to better manager risk on this project with respect to all projects in the Program.

H. Construction Permitting

During subsequent phases of design development for the project, the design team will coordinate with the Construction Manager on necessary construction permits for the project. This coordination will assist in identifying any special permitting requirements that need to be established prior to contracting for the project.

I. Construction Sequencing

1) General

The size, complexity and remoteness of the project allows for a great flexibility in actual construction sequencing. There are certain general statements of sequencing that will hold but most specific details will be based on the construction plan of the successful construction contract bidder.

In general one of the first aspects of each operation is to provide environmental protection. This will include setting up proper staging areas with double containment of fuels, lubricants and other hazardous materials. The use of silt fencing, dewatering settlement basins, floating turbidity barriers and erosion control features will need to be in place prior to operations that could cause contamination, pollution or silting of the existing canals.

Irrigation systems in the area are used for agriculture. A supply of water will have to be available to these irrigation facilities that will provide the minimum flow in the canals to satisfy the irrigation needs. The water flowing in these canals also replenishes the ground water supply to local wells. The construction of the Reservoir is not to adversely effect the irrigation or the groundwater during or after completion. The LPDD and the Robert canals for instance can not be cut off until enough of the Perimeter Canal is constructed to supply water to the Banana Branch. This however does not mean the first feature to be constructed has to be the Perimeter Canal.

There is an electric distribution line that traverses the project site. This facility will have to be relocated at some point during the construction. A new facility will need to be built and put into service prior to removing the existing line. The construction of the replacement line along the north side of the project site will need to be coordinated with the perimeter dam construction.

2) Clearing and Grubbing

One of the first steps in any project includes site preparation. The site for the Reservoir is no exception. The existing citrus groves will need to be cleared. The footprint for the perimeter dam, the interior levees, the perimeter road and the Perimeter Canal will need to be grubbed to remove roots and stripped to a depth of 18 inches to remove organics. Work areas along the dam should be cleared, grubbed and stripped as needed to keep ahead of the embankment construction, however, this process should not get too far ahead of the embankment construction or new vegetation growth will require additional stripping. The clearing of the Reservoir areas can proceed in parallel with other work items with the exception of the borrow pit areas that are needed to construct the dam embankment. Borrow pit areas will need to be cleared, grubbed and stripped to a depth of 18 inches the same as the construction footprint.

3) Borrow Pits

Borrow pits will be the source of the clay for the relatively impervious core, the clean graded sand for the drainage blanket, the material to be mixed for soil cement and the extra random fill for the dam slopes. The excavation of the perimeter (seepage) canal will be used for the perimeter road and the random fill on the exterior and top of the dam structure, but the quantity is inadequate for all of the random fill.

For each section of dam to be constructed the borrow pit should be cleared, grubbed and stripped. A screening and/or wash plant needs to be set up to process the sand overburden needed for the drainage blanket. The clean graded sand material produced should be stockpiled for later use. The remainder of the overburden can be excavated and stockpiled to be used for soil cement and

random fill. As the overburden is removed access to the exposed clay material will allow the construction of the relatively impervious clay dam core to begin.

4) Perimeter Dam

The perimeter dam is a very large earthworks project. As in any earthwork project it is necessary to clear, grub and strip the construction footprint. Once the existing sub grade is compacted, construction of the Dam structure can commence. The horizontal 2 ft. thick, filter fabric enveloped, sand drainage blanket on the outside of the dam can be completed in conjunction with the first two feet of the impervious clay core. The placement of the clay core would then continue. As the clay core is built the sloped drainage blanket on the outside of the core can be placed as the slope is dressed. The placement of the fill on the inside dam slope can proceed as the clay core is constructed. Care will need to be taken to prevent contamination of the clay core. Excavation of the perimeter (seepage) canal can begin to supply material for the placement of the random fill on the outside slope and can begin after the horizontal drainage blanket is in place. Excess excavation from the Perimeter Canal can be used in the placement of the fill on the inside of the dam or in construction of the separator dams. Completion of the fill on the inside of the dam will be followed by the placement of the drainage blanket on the inside slope and the placement of the soil cement slope protection.

The construction of the dam can proceed at several locations around the perimeter of the dam at the same time if the contractor has sufficient plant capacity. Some coordination will be necessary at the sites where the control structures and the pump stations will be built in order to support the schedule for those structures.

5) Pump Stations

The pumps and large motors required to power them will be long lead time items. Once the design of these components is completed an equipment contract should be let separate from the pump station construction contract. The construction of the pump station can then be coordinated with the earthworks and the equipment delivery schedule. This will allow for the most efficient schedule possible for the pump station construction.

6) Control Structures

The construction of the control structures should be coordinated with the earthworks and the perimeter (seepage) canal. The lead time for the large lift gates, operators and standby power units should be determined during the design

stage. If the lead time is too long the use of equipment contracts should be considered.

J. Staging Areas

1) Introduction

The staging area concept for the Reservoir includes a central staging area and site specific staging areas. The existing access to the Reservoir site from S.R. 80 will also serve as the access during construction and after construction is complete. The cleared area around the existing Berry Groves administrative, storage and maintenance offices will make an excellent location for accepting, storing and controlling the materials and equipment that will be required to do the work. As discussed earlier in this report, the access to the project area is controlled by a guard stationed at the entrance. It will be recommended that the existing guard shack remain and be manned during construction. This would allow the contractor to control access and inventory.

2) Perimeter Dam

The perimeter dam will be constructed from up to 8 borrow areas where material will be processed for the sand drainage blanket and soil cement. Each borrow pit area will become a staging area for processing equipment and the excavating, hauling and compacting equipment. Each of the areas would probably have fuel supplies and other maintenance support facilities.

3) Pump Stations

The pump stations would each have their own staging and laydown areas. Materials and equipment could be transported from the central staging area to the site specific areas as needed.

4) Control Structures

The major gated control structures would have their own staging and laydown areas. Control structure S-1 would share a staging and laydown area with pump station PS-1. The staging and laydown areas for some of the minor structure will be small.

K. Construction Access and Traffic Routing

Access to the site will be from S.R. 80. The existing entrance road will be used for all construction traffic entering and leaving the site. Access from the entrance road to the Reservoir during construction will be different from the access that will be provided when construction is complete. If the Perimeter Canal is completed prior to completing all of the other work including the dam, pump station(s) and drainage structures then it is expected that there will be numerous canal crossings installed during construction to provide direct and efficient access to those work areas. The construction crossings will be removed and the canal restored as construction is completed.

The contractor will be required to prepare, institute and maintain a traffic control plan within the project site during construction. Traffic control signing, barricades and barriers within the S.R. 80 right of way during construction will meet the current Florida Department of Transportation (FDOT) current design standards. Since the project site is occupied by citrus trees, it will be important that the traffic in and out of the site during construction is aware of the procedures used in the prevention of spreading citrus canker disease. Vehicle and pedestrian rinsing stations will be required until the potential for contamination is eliminated.

L. Disposal

The current use of the property within the project's construction limits is agriculture and consists primarily of citrus groves. It is anticipated that all materials, machinery and equipment that is to be removed or salvaged will be done so by the current citrus grove operators. Once the construction begins, unless otherwise noted in the construction documents, the contractor will take possession of all of the materials and the removal and disposal of the materials will be the responsibility of the contractor. Once all erosion and sedimentation controls are in place and the appropriate permits have been obtained the land will be cleared and grubbed. The citrus trees could be stockpiled and burned on site. The disposition of the irrigation pipes and tubing is yet to be determined.

The stripping of the construction footprint and the borrow pits to a depth of 18 inches to eliminate organic contaminated material will result in nearly 3,000,000 cubic yards of soil that will need to be disposed of. During the stripping operation this material will probably be stockpiled at various locations within the project limits. Once the borrow pits have been depleted or at the point additional material is not needed the organically contaminated soil can be used to fill portions of the borrow pits. The organically contaminated soil could also be used on the outside slope of the perimeter dams.

Section 24 – Emergency Action Plan

The reader should note that all tables, figures and supporting information are "embedded" in this Section and there is no Section 24 in the Appendices.

A. Dam Break Analysis for the Reservoir

A dam break analysis was conducted to determine the potential effects on adjacent properties. A number of possible modeling tools were considered for this analysis. Models include the National Weather Service Model DAMBRK, the U.S. Army Corps of Engineers Program HEC-RAS, and the DHI models MIKE FLOOD and MIKE SHE. Because MIKE SHE has been used for the reservoir design, it was initially decided to use this tool for the dam break analysis. MIKE SHE and MIKE 11 are also acceptable methods for dam break analysis according to Steven Partney of FDEP's Dam Safety Division (Personal Communication, 1/11/2005). MIKE FLOOD (which includes MIKE 11) is widely used by the U.S. Bureau of Reclamation for dam break analysis. MIKE FLOOD is a linking of the MIKE 11 river hydraulics model with MIKE 21, which is a 2D overland flow model used for fully dynamic overland flow and estuarine conditions. MIKE FLOOD is similar to MIKE SHE, however the overland flow routine in MIKE SHE is calculated using dynamic wave principles, while the overland flow in MIKE 21 is fully dynamic. The initial tests indicated that the MIKE SHE overland flow routine was not able to handle the highly dynamic nature of a dam break flood wave. Accordingly, it was decided to use MIKE FLOOD for this analysis. .

The MIKE 11 hydraulic control structure Dambreak was used to simulate the dam break. This hydraulic control structure within MIKE 11 allows for a number of failure methods to be modeled, and MIKE 11 opens up the Dambreak structure that is essentially similar to a gate. The dam break has a trapezoid shape that increases in size, and the water flow through the gap increases as the gap widens. Based on a discussion with Mr. Partney, the dam break has been assumed to originate as a piping failure which then evolves into a full breach of the dam embankment. The details are explained below.

The location of the dam break is shown in **Figure 24A-01**. The dam break has been assumed to occur in Cell 1 of the Reservoir, and it is assumed that only Cell 1 will contribute flow to the breach. This assumption is reasonable because flow between the cells is controlled by gates that can be shut in the event of a dam break. Water levels are monitored continuously at the Operations Center of the SFWMD, and there will be an alarm that will sound in the event of a significant water level drop in the reservoir. Operators are on-site during most of the day and will be on-call at night. Therefore, it is reasonable to assume that the gates can be shut. The Dambreak Structure menu in MIKE 11 is presented in **Figures 24A-02 and 24A-03**. It can be seen from **Figure 24A-03**, that the piping failure starts at approximate elevation 23 ft., and has an initial diameter of 3.3 ft. The final bottom level of the breach was assumed to be at elevation 20 ft. and 328 ft. wide, with a breach slope of 1:1. The crest elevation is 48.5 ft., so the top width at full failure will be 385 ft. [(48.5 – 20) x 2 +328 = 385 ft]. Dam material properties

(presented in **Figure 24A-03**) were provided by the geotechnical engineering consultant and were based on geotechnical testing of materials expected to be used for the reservoir. The mean grain size is assumed to be 0.15 mm, the specific gravity is 2.7, porosity is 0.3, and the shear stress is 800 psf for the core, and 200 psf for the downstream toe.

The flow through the breach is presented in **Figure 24A-04**. The initial flow is through a piping failure of the reservoir. At approximately 30 minutes after the beginning of the piping, the embankment fails, and the breach flow begins. The evolution of the breach is presented in **Figure 24A-05**.

Peak stages in the Perimeter Canal exceed the top-of-bank elevation, and flooding is expected to occur in residential lands north of the proposed reservoir and south of S.R. 80. A map of water depths at the point of maximum depth in the lands north of the reservoir is shown in **Figure 24A-06** and a plot of the change in water level in key locations is presented in **Figure 24A-07**.

Water depth in Fort Simmons Branch is shown in **Figure 24A-08**, and it can be seen that the maximum elevation in this Branch is approximately 1 ft. higher than the top of bank elevation. Water depth in the Perimeter Canal does exceed the top of bank elevation as shown in **Figure 24A-09**.

The water depths shown in **Figure 24A-06** could occur at any location because the dam failure location was assumed and cannot be predicted. Therefore, safety to residents living near the reservoir may be compromised if a dam break is experienced.

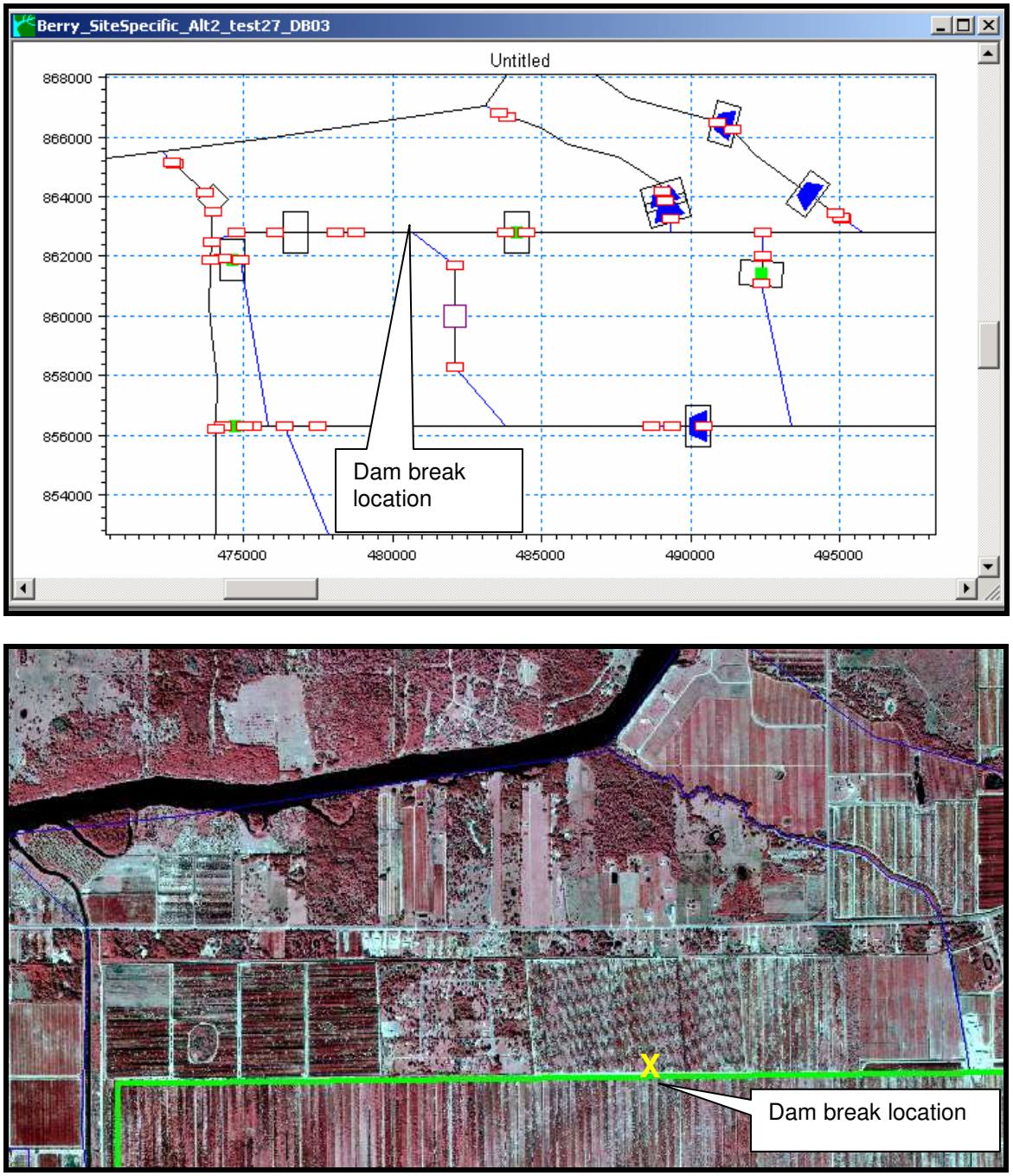


Figure 24A-01 - Map of Dam Break Location (MIKE 11 Network and Aerial Photo Views)

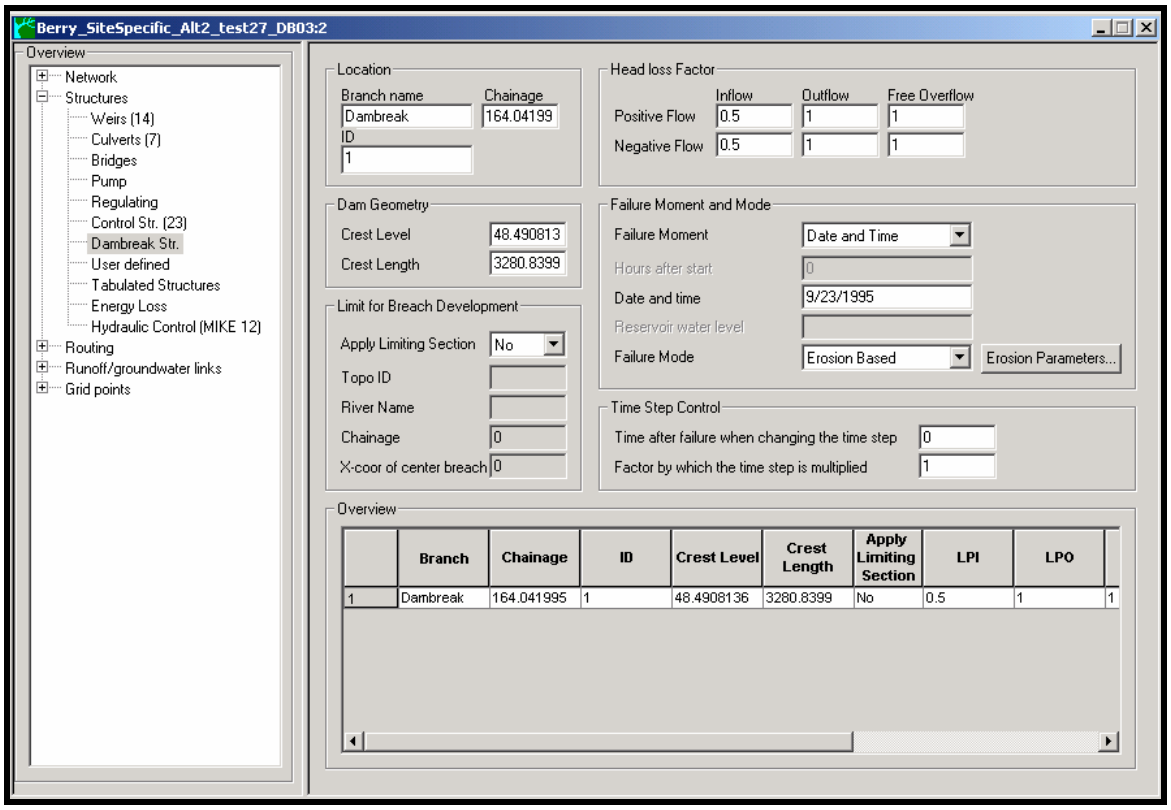


Figure 24A-02 – MIKE 11 Dambreak Menu

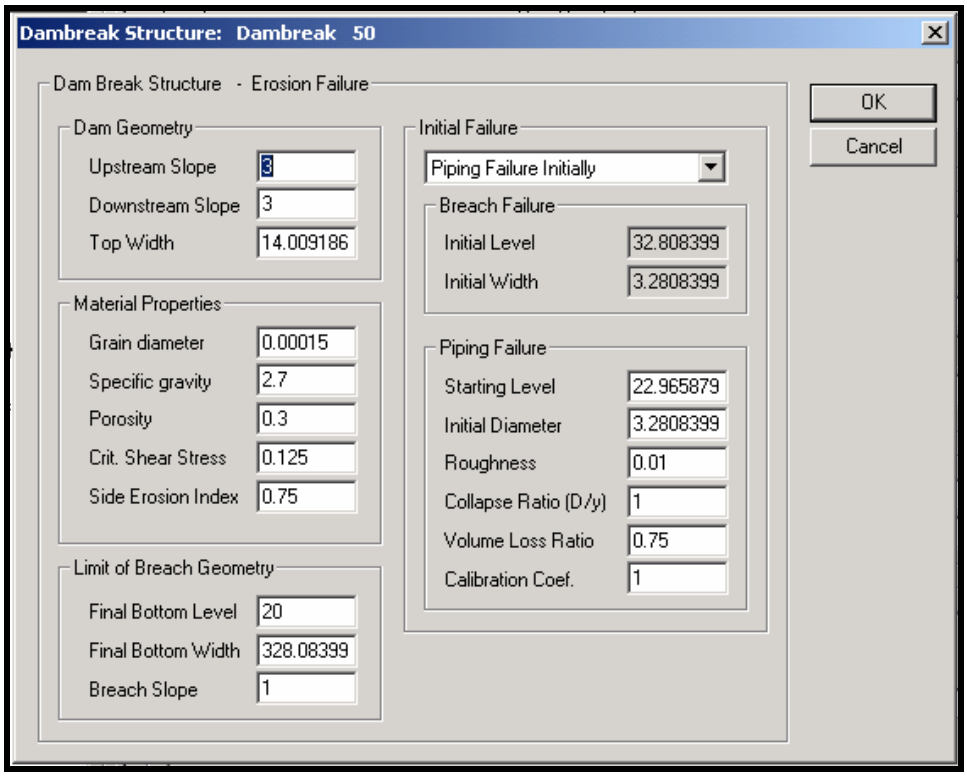


Figure 24A-03 – MIKE 11 Dam Breach Menu for Erosion Parameters

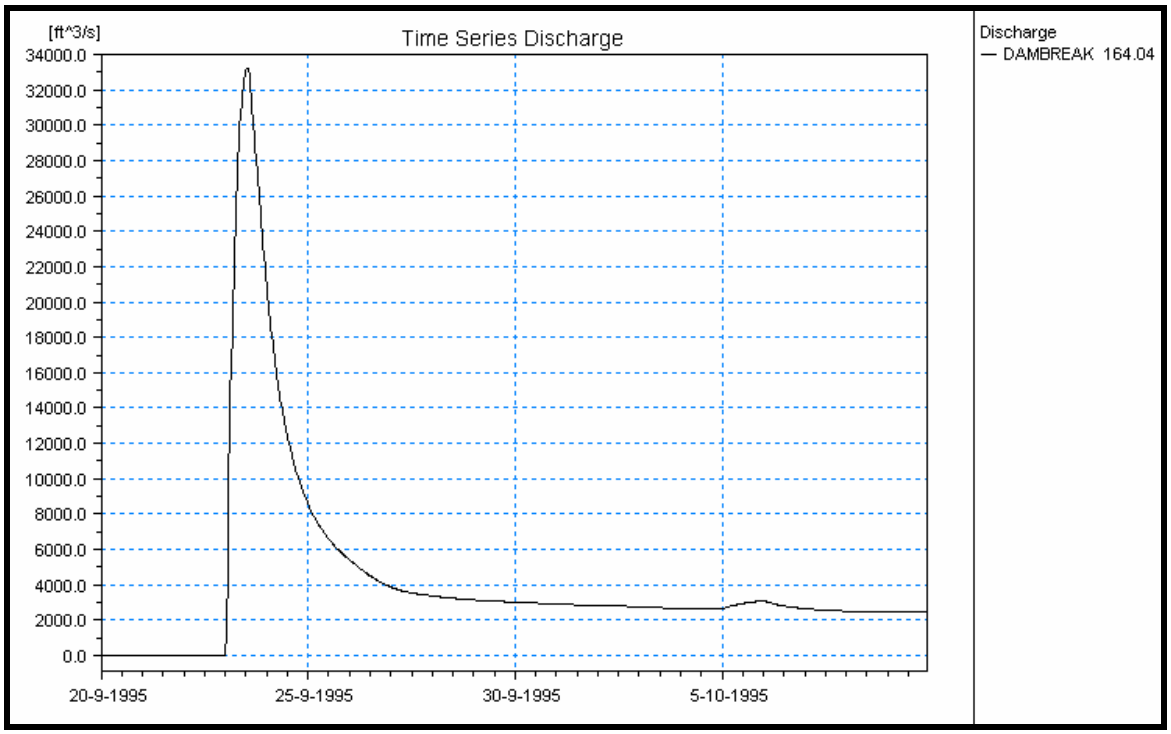


Figure 24A-04 – Flow through the breach for the C-43 West Storage Reservoir

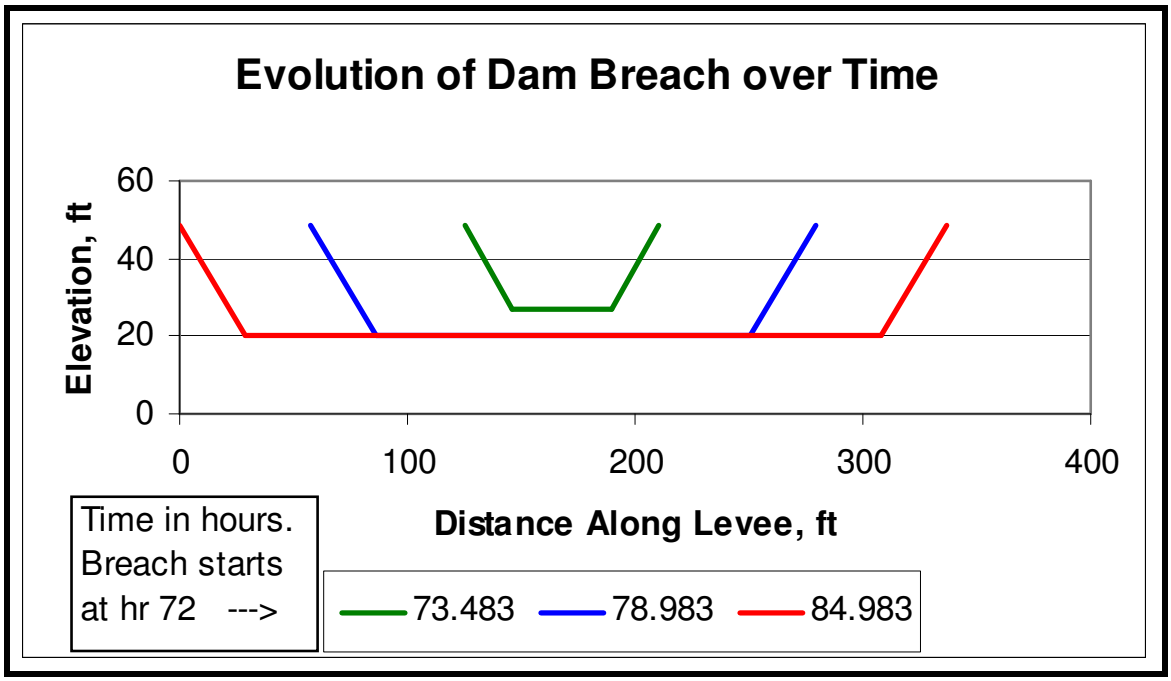


Figure 24A-05 – Evolution of Dam Breach over Time

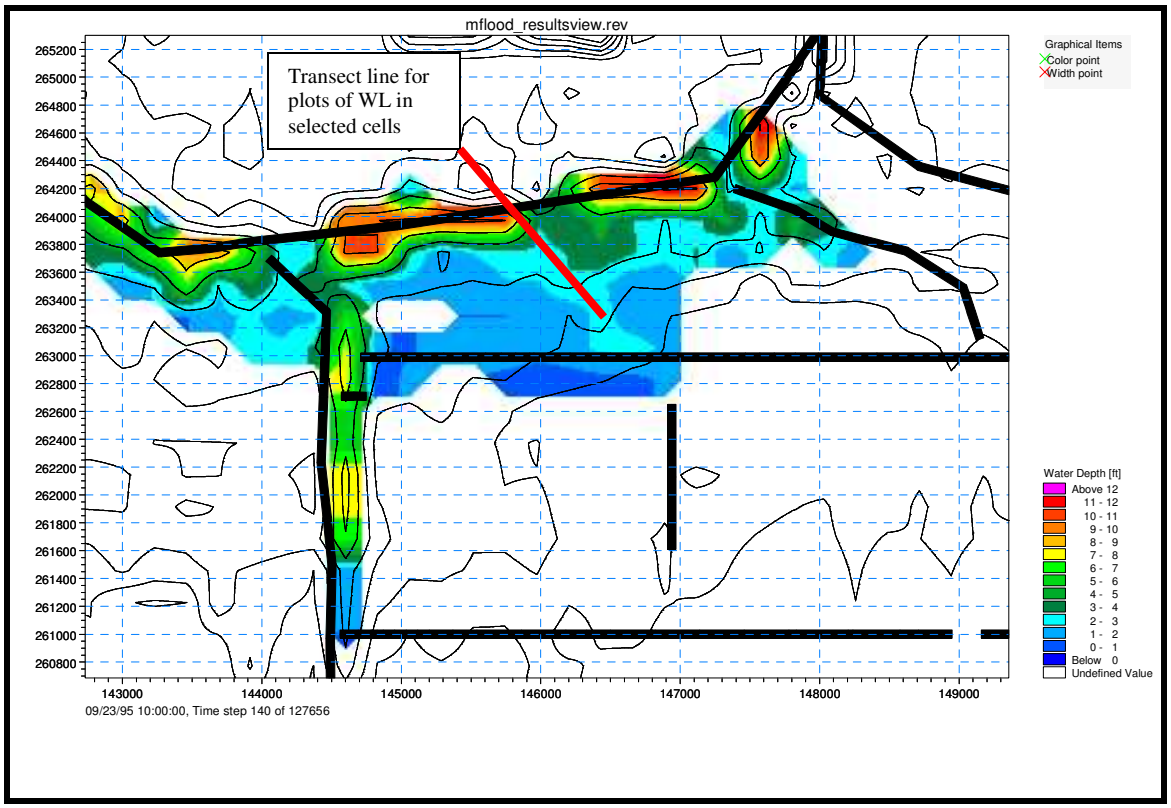


Figure 24A-06 – Plot of Maximum Water Depths During a Simulated Dam Break

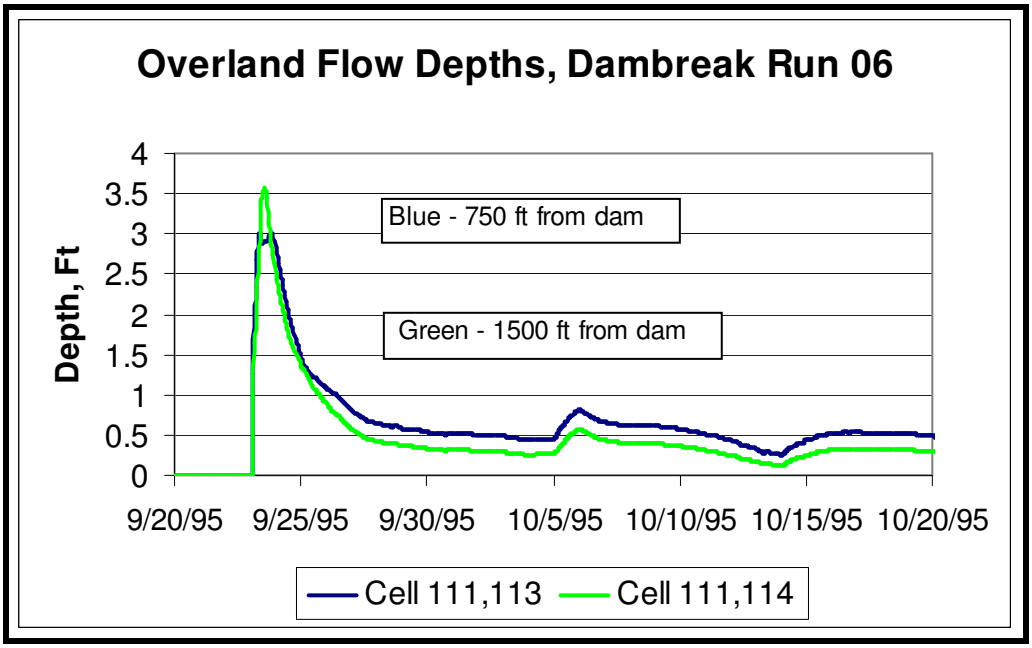


Figure 24A-07 – Plot of Water Levels During a Dam Break at Key Locations North of the Proposed Reservoir (See Figure 4 for Location of cells)

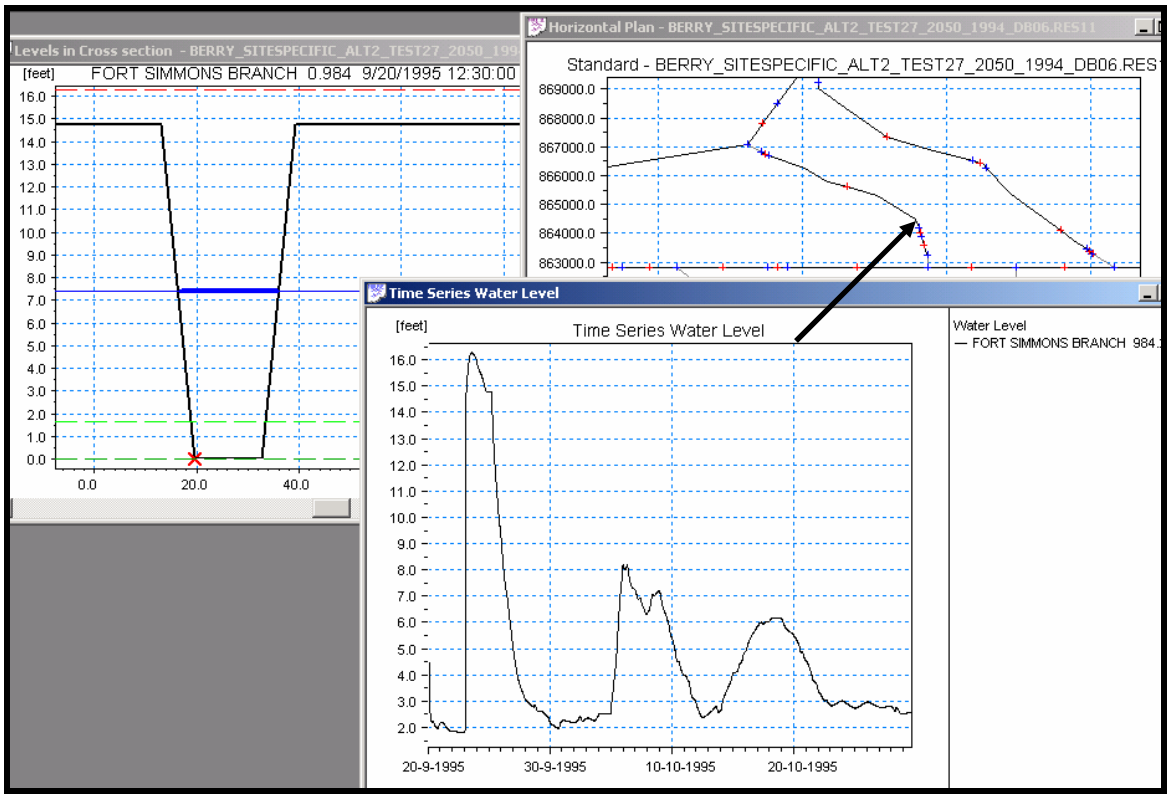


Figure 24A-08 – Water Depth in the Fort Simmons Branch during a Dambreak

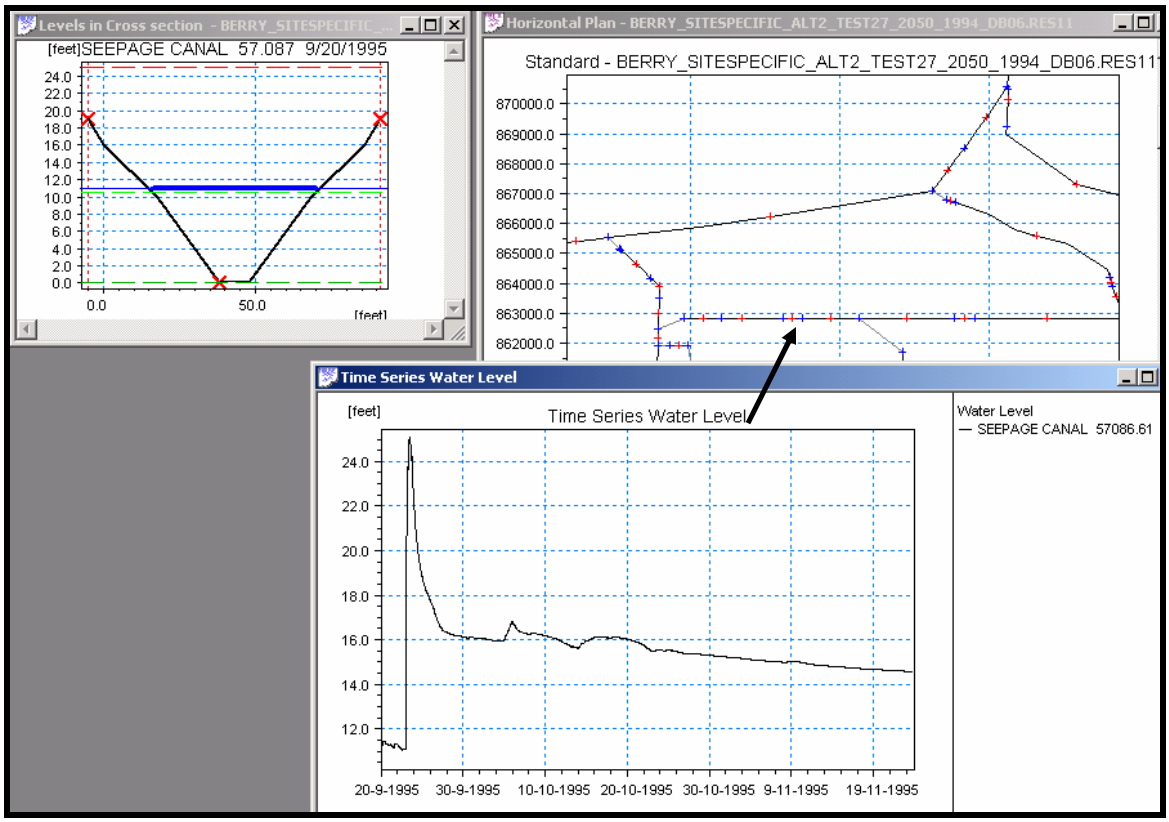


Figure 24A-09 – Water Depth in the Perimeter Canal during a Dambreak

The dam break location was selected to evaluate the impact of a dam break into a low-lying area that, in the future, is predicted to be highly urbanized. A dam break is possible at any location. The design team assumed a dam break at the location where the impact would likely cause the most damage. The design team completed only one dam break analysis using the assumptions described in the BODR.

The model was not calibrated using DAMBRK or FLDWA. The methodology for conducting the dam break analysis for this project has not been finalized. The SFWMD is expected to release a draft final approach to issues related to dams in the near future. The project team will review the information and modify the dam break modeling approach accordingly.

Section 25 – Draft DDR Comments and Responses

The following are the Draft DDR comments with responses.

C-43 WEST STORAGE RESERVOIR 30% DESIGN
SFWMD CONTRACT: C-C10401P (Work Orders No. 2 through 7)

DRAFT DESIGN DOCUMENTATION REPORT (DDR)
RESPONSES TO COMMENTS
OCTOBER 12, 2004

GENERAL

Comment: 1G
The determination of the design flood and freeboard is incomplete. USACE references and guidelines by the Federal Emergency Management Agency describe the criteria and procedures that should be used. For example: a) Safety Design Standards for a High Hazard Potential Structure would be "Standard 1" and designed to "safely pass an IDF computed from probable maximum precipitation (PMP) occurring over the watershed above the dam site."; b) the PMP is defined as "the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage area at a certain time of year"; and c) freeboard is generally based on the maximum probable wind conditions when the reference elevation is the normal operating level. When estimating the freeboard to be used with the probable maximum reservoir level, a lesser wind condition is used because it is improbable that maximum wind conditions will occur with maximum flood level.

In addition, design of the reservoir should consider conditions "…to assure that failure of the dam will not result from wind setup, wave action, uncertainties in analytical procedures, and uncertainties in project function in combination with the most critical pool elevation." The DDR does not cover conditions that could reasonably expect to occur in determining the most critical pool elevation. For example, Hurricane Easy occurred in 1950 in Central Florida. Hurricane Easy resulted on 38.7 inches of rainfall in 24 hours, and with a maximum reported wind speed of 125 mph. The design conditions reported in the DDR consist PMP of 36 inches and the FBC wind speed of __ mph.

Based on the definition of PMP, the reported PMP is too low. The method used in the DDR to calculate the PMP was based on Technical Paper No. 40. TP No. 40 presents data for 1 year to 100 year events. Probable Maximum Precipitation for Florida is covered by Hydrometeorlogical Report No. 51, "Probable Maximum Precipitation Estimates for the United States East of the 105th Meridian" published by the National Oceanic and Atmospheric Administration.

Response: The required freeboard for a dam is defined as the vertical distance between the top of a dam and some specified pool level, usually the normal operating level or the maximum flood level. The draft DDR did consider the C-43 West Storage Reservoir as a High Hazard Potential Structure and is designed to "safely pass an IDF computed from probable maximum precipitation (PMP) occurring over the watershed above the dam site." The method used in the draft DDR to calculate the PMP will be revised and based on NOAA's Hydrometeorological Report No. 51. The all-season PMP of 55.7 inches for 72 hours and the rainfall depth of the PMP for 12, 24 , and 48 hours will be adopted and added to the normal pool level for the condition of maximum flood level .

The freeboard was determined using a probable maximum wind condition with the reference elevation as the normal operating level. As indicated in the last page of Appendix 2I-1, the

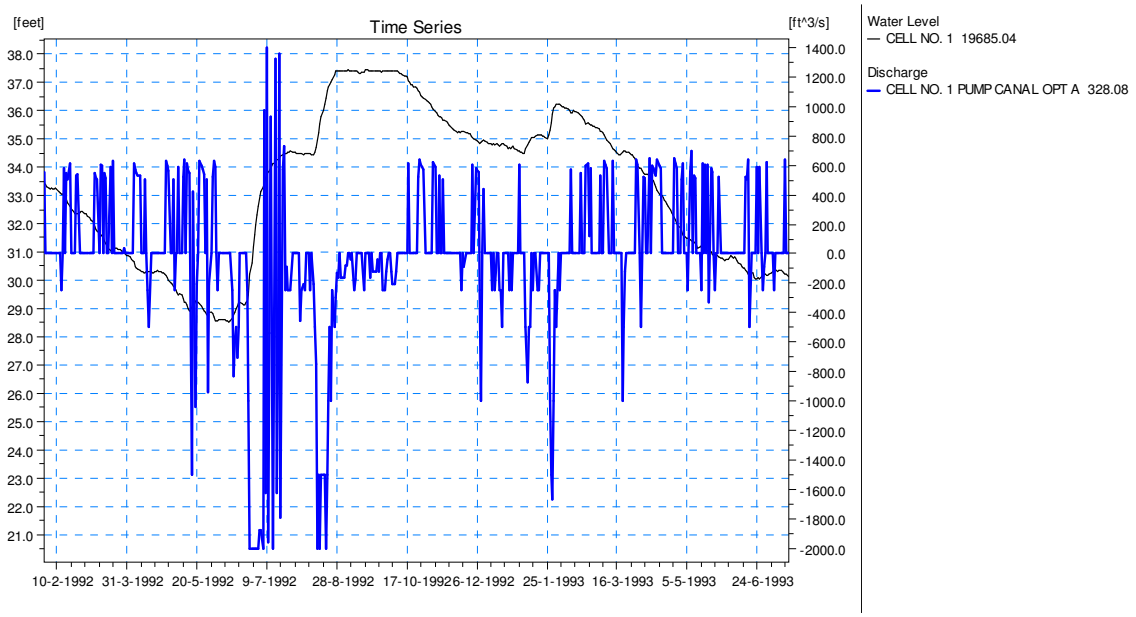
adjusted wind speed of 146 – 156 mph was used for the wind setup and wave run up analysis. When estimating the freeboard to be used with the probable maximum reservoir level, the 120 mph wind speed for the region from the Florida Building Code or the maximum recorded wind speed will be used for the revision. According to the criteria published in Section 15-1 Chapter 15 of USACE's EM 1110-2-1420, a less wind condition, slower than the probable maximum wind conditions, is used to estimate the freeboard with the maximum reservoir level because it is improbable that maximum wind conditions will occur simultaneously with the maximum flood level.

Based on EM 1110-2-1420, the effective fetch adjusts radial lines from the embankment to various points on the reservoir boundary. The radials spanning 45 degrees on each side of the central radial are adjusted by the cosines of their angles to the central radial to estimate an average effective fetch. The wave run-up will be corrected for the embankment slopes (3H: 1V, 4H: 1V and 5H: 1V) and slope surface roughness (soil cement and riprap). The top-of-dam elevation will be selected as the greater of the freeboards obtained under the conditions of the normal operating level and the maximum flood level. A memo to describe the details of estimating the freeboard will be prepared and submitted in the future and included in the appendix of the Pre-Final DDR.

Comment: 2G

Operating criteria for the C-43 Basin Reservoir is needed for design. Modeling of reservoir levels, supply and release data should be summarized in the report. Reservoir levels and operating criteria will be needed to established critical structure elevations, sizes and operating conditions.

Response: This information is readily available and will be provided in the next draft. An example plot for one year of the simulation is provided below:



Comment: 3G

The report would be greatly enhanced by adding summary tables in several areas of the report that present the important features of the alternatives being evaluated. It is difficult to identify what the recommended features are from the report. An Executive Summary is also needed.

Response: Summary tables will be provided in the next submittal, the Pre-Final DDR. An Executive Summary was included in this submittal. The Executive Summary will be updated for the Pre-Final DDR.

Comment: 4G
Should begin using "C-43 West Storage Reservoir" as the project name

Response: Noted.

Comment: 5G
Replace all reference to completion of or as a result of the PIR process with completion of or as a result of the Site Screening Process. The PIR has not been completed.

Response: This correction will be made.

Comment: 6G
A discussion should be included somewhere in the report about the dependence of the landowners to the north of the project on existing agricultural seepage for water supply. This will become a constraint on the amount of reservoir seepage that should be mitigated.

Response: Groundwater elevation maps showing the difference between existing and future conditions will be prepared and provided in the next draft.

EXECUTIVE SUMMARY

Comment: 1ES Page 1, Para. 1
Delete sentence "It is also assumed that depending upon the location of the facility and the pollutant loading conditions in the watershed, the facilities could be designed to achieve water quality improvements." Sentence is leading.

Response: This sentence will be deleted. The point that we were trying to make is that the other C-43 reservoir site(s) may contain higher local pollutant loading and may offer a better opportunity for treatment in conjunction with storage.

Comment: 2ES Page 1, Para. 2
"The PIR will formulate, evaluate & compare suites of alternatives. Early efforts identified many sites to be evaluated for use in the development of alternatives. The sites were considered in an initial site screening process. The final sites selected could cost effectively meet the project goals and objectives."

Response: This wording will be incorporated in the next submittal.

Comment: 3ES Page 1, Para. 2
Delete sentence "Some alternative sites may offer localized water quality improvements as well as storage." Sentence is leading.

Response: This sentence will be deleted. See response to 1ES.

Comment: 4ES Page 1, Para. 3

In October 2003 South Florida Water Management District decided to proceed with the design and implementation of several CERP projects to provide early benefits to the Everglades Ecosystem. The expedited process will be performed on a dual track with the PIR process. The PIR will be lead by the Corp of Engineers (the Corp-SFWMD partnership will be maintained). SFWMD will proceed with the design and implementation of a reservoir on the parcels previously acquired by SFWMD, the Florida Department of Environmental Protection and the US Department of the Interior. These parcels include Berry Groves, Griffin Groves and MG Enterprise Groves and ranked first among the sites evaluated during the site screening process. SFWMD is currently performing parcel exchanges and acquisitions that will provide a footprint for the reservoir that will avoid impacts to Florida Power & Light and potential economic development as well as improvements in site conditions (geotechnical and topographic). Stanley Consultants was directed to perform the 30% design services for the C-43 West Basin Storage Reservoir.

Response: This wording will be incorporated in the next submittal.

Comment: 5ES Page 2, Para. 1

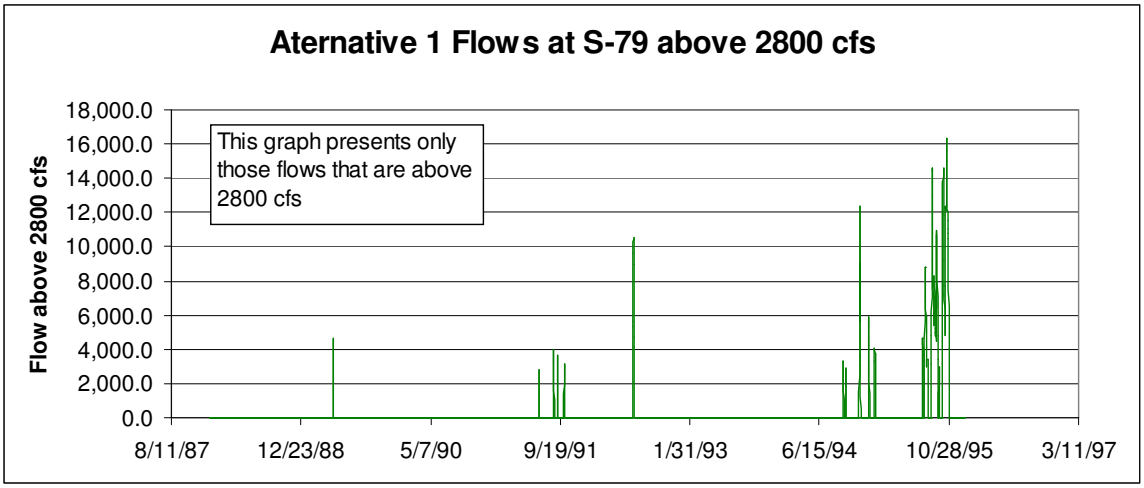
The conceptual alternatives presented in this report are based on the information from the mapping, survey, geotechnical and hydraulic modeling efforts that have been completed to date. It is anticipated that minor adjustments in flows, typical sections and locations of structures may be required when the final existing data is collected and analyzed and the hydraulic modeling reaches final conclusions.

Response: This wording will be incorporated in the next submittal.

Comment: 6ES Page 3, Bullet 1

How many times and how long will excessive flows (>2,800 cfs) pass over S-79? This should be identified for the period of record and annually, not only in mean monthly averages. Identify how quickly the reservoir will fill using the modeling data and these pump sizes.

Response: This information can be easily provided. An example is presented below:



Comment: 7ES Page 1, Bullet 2
… In addition to seepage collection, the perimeter canal will also convey water, including primary and emergency discharges from the reservoir, improve drainage for S.R. 29, provide flows to the Crawford Canal, Banana Branch and Fort Simmons Branch, and maintain the surface water elevation necessary for irrigation and drainage for the A. Duda and Sons (Duda) citrus operations.

Response: This wording will be incorporated in the next submittal.

Comment: 8ES General
Executive Summary could be enhanced if typical section with elevations were provided.

Response: We will consider referring to drawings or adding typical sections for the next submittal.

DESIGN REPORT

1A - GENERAL

Comment: 1A.1 General
Include a table that identifies each Work Order and describe the major work effort(s) of each. This would help to clarify some of the discussions related to Work Orders in subsequent sections.

Response: We will consider this in the next submittal. A Work Order description was provided on page 1 of the Executive Summary.

1B - INTRODUCTION

Comment: 1B.1 Page 6
Suggest that sections from the C-43 Basin Storage Reservoir Project - Part 1 Project Management Plan be cut and pasted into this section for identification of the Project Goals and the approval/authorization. The text is more succinct and relevant to this project. The PIR has not been completed; this is a very important piece of information as Stanley could not participate in detailed design if they had been involved in the completion of the PIR which is essentially a feasibility study. Need to separate the section on the Option 2 plan components into a separate sub-section of the Introduction.

Response: We will review and incorporate your suggestions in the next submittal.

1C - CURRENT RESERVOIR ALTERNATIVES

Comment: 1C.1 Pg. 9, Para. 1
… the Berry Groves Site Option 2, which was one of the site options developed early in Step 3, Development of Alternatives, of the PIR process…. Need to be very specific about the PIR process, which has not been completed. Alternative development, evaluation, comparison and selection are continuing in the PIR process being lead by the COE.

Response: We will expand on this in the next submittal.

Comment: 1C.2 Pg. 9

The last bullet describes structure S-1 is to control "releases prior to and during a storm event". It should be noted that releases prior to a storm event are not typically considered when routing of the inflow design flood for sizing of the reservoir and establishing the design freeboard. If releases were part of the design operating criteria, operating controls would require release of water in advance of a storm. Would the loss of water be acceptable? What if rainfall subsequent to release was insufficient to meet the project goals? If flows in the Caloosahatchee River are too high from rainfall in other parts of the region are too high to allow releases from C-43, then how would the operational release required be ensured?

> ***Response: An "operational release" prior to a storm event is not being considered as part of the reservoir design. Additional modeling will determine if we should consider any releases prior to or during a storm. We may determine that there will be no releases when a release exceeds some maximum flow over S-79. Additional modeling is required. Again, the reservoir will be designed without any releases being considered prior to or during a storm event.***

Comment: 1C.3 Pg. 9

Have held approximately 10,000 acre feet of storage as "contingency" in the design. This is appropriate at this level of design. However, might be useful to provide either graphical or tabular indication of water levels for both 160,000 and 170,000 acre feet. This provides understanding of how much depth the contingency adds.

> ***Response: It was stated in item 2 on page 4 of the Executive Summary that "A one-foot reduction in "storage height" reduces storage capacity by approximately 10,000 acre-feet". We will include a table in the next submittal that makes it easier to see footprint/volume/height relationships.***

Comment: 1C.4

Has a structure been considered to provide flows both to and from the remaining Paul property on the east side of the reservoir? This is one of the considerations given to adjacent landowners during the acquisition of the Berry property. Paul is a member of the LaBelle Private Drainage District that was dependent upon flows to and from the Berry Header Canal.

> ***Response: Flows to the Paul property are to be provided by Structure S-6 in the Crawford Canal. If additional water is required, a structure(s) can be added in the LPDD extension between the Perimeter Canal and SR 29.***

Comment: 1C.5

The last bullet describes that the emergency spillway crest elevation will be based on the 25 year, 72 hour storm event. The report later describes that this emergency spillway crest elevation will be used as the reference basis for establishing the design freeboard. The criteria in this report should include a description of how the normal operating pool will be maintained without encroaching on the 25 year, 72 hour flood pool.

> ***Response: The discussion of the emergency spillway, PMP, wave run up and freeboard will be expanded and a detailed analysis provided in the Appendix.***

Comment: 1C.6 Pg. 10
The second bullet describes the Rim Ditch Water level at el. 23 and bullet six describes a canal water level at el. 16. The seepage and stability conditions will be substantially different for this range of perimeter water levels. The analysis will need to cover this full range.

Response: Agreed.

Comment: 1C.7
The sixth bullet describes that the water level in the Townsend canal cannot be maintained at El. 16 without flooding the adjacent property. It is not clear how this potential flooding is going to be controlled. More detail on how to control the flooding and impacts, is needed.

Response: Essentially, we need to determine that the high water level in the Perimeter Canal will always be maintained at an elevation that allows a minimum freeboard to the canal bank. We will be able to accomplish this with additional modeling and with the contour mapping that we will be receiving in the near future. More detail will be provided with the next submittal.

Comment: 1C.8 Pg. 11
The last paragraph describes that when operation costs are included, the two pump station alternative in unattractive. The cost estimate data to support this conclusion is not included. Please provide the cost detail backup. Also, as described in the comments below, I am not sure that the cost estimate has been developed sufficiently at this stage to make a conclusive decision at this stage.

Response: Additional analysis and a more detailed cost estimate will be provided with the next submittal. The cost estimate for this submittal was for conceptual design and provided to give an approximate overall project cost. It is agreed that not enough cost backup was provided to support a conclusive decision.

Comment: 1C.9 Figures 1c-xx
Please provide an "approximate" scale with the drawings. It is difficult to develop some general dimensions without a good reference.

Response: Agree.

1D – RESERVOIR FOOTPRINT

Comment: 1D.1 Pg. 12
Please provide the approximate reservoir depth that corresponds with the storage reduction in Parcels C, E and F.

Response: A table will be provided in the next submittal.

Comment: 1D.2
The revised reservoir footprint is a significant improvement over the initial PIR layout. Further evaluation of Parcels F and G are recommended, however. The shape efficiency of the reservoir limit shown on Figure 1D-03 is about 1.39 (comparing the perimeter shown to the perimeter of a circle which would have the minimum perimeter length for the same surface area). Two alternatives that should be considered are: 1) eliminating parcel F and part of C would reduce the perimeter embankment length by about 28,000 lineal feet (over 25 %) – and the same storage

volume could be provided by increasing the reservoir water depth by about 3 feet; and 2) eliminating parcel F and part of C and expanding to the south in Parcel G (in the portion east of the Townsend canal) with little change in reservoir water depth.

Response: 1) A more detailed analysis of this option will be considered in the next submittal. 2) Expanding to the south in Parcel G is not an option as per the SFWMD's direction. The decision has been made to minimize impacts to the Duda property.

Comment: 1D.3 Figures 1D-xx
Please provide an "approximate" scale with the drawings. It is difficult to develop some general dimensions without a good reference.

Response: Agee.

1E - CELL CONFIGURATION

Comment: 1E.1 Pg. 14
A more detailed review of the evaluation in Appendix 2I should be performed to confirm that the cell configuration selected is the most economic choice. Experience with other projects has shown that more detailed analysis, including economic factors, is needed to make an informed selection.

Response: A more detailed analysis will be completed and provided in the pre-final DDR.

2A - SURVEY AND MAPPING

Comment: 2A.1 Page 17, Para. 1
Replace the first sentence with …"The survey and mapping effort was completed under a separate Work Order (W.O.) from the W.O.s related to preparing the 30% design package." …

Response: Will replace the first sentence.

Comment: 2A.2 Page 17, Para. 1
Strike the sentence …"The survey and mapping work was completed for the SFWMD, Stanley simply facilitated its coordination and production." … This throws the reader off balance.

Response: Will delete the sentence.

Comment: 2A.3 Page 17, Para. 1
Disagree with statement …"It was intended that the survey and mapping be completed and utilized for the final design." It has always been contemplated that production of drawings and figures for the 30% design deliverable would be based on the new mapping data.

Response: During the initial scope development and fee negotiations when the schedule required the 30% submittal be complete by mid summer of 2004 it was agreed that the South West Florida Feasibility Study Topographic Data and Mapping was the most current and accurate data to use for the site/civil design. The Topographic Data and Mapping would be supplemented by "ground surveying". This is how Subtask 7.1 of the Scope of Work of Work Order No. 7 was written. However, due to the dynamics of the project and priorities of concurrent SFWMD projects it became evident that the

survey and mapping products from Work Order No. 2 would be available for use in the 30% design of this project. The sentence could either be eliminated without confusing the reader or detracting from the substance of the report or the text could be expanded to provide an explanation similar to the above.

Comment: 2A.4 Pg. 17, Para. 2
"who's" should be "whose".

Response: The word "who's" will be replaced.

2B - GEOTECHNICAL

Comment: 2B.1 Figure 2B-01
Are borings to be completed along the other proposed interior levee (not indicated on Figure 2B-01)? This is not clear.

Response: Yes. As soon as the location of the interior levee that will separate Cells 1 and 2 is fixed, borings will be drilled along that alignment.

Comment: 2B.2 Figure 2B-01
Please provide an "approximate" scale with the drawings. It is difficult to develop some general dimensions without a good reference.

Response: Figure 2B-01 has a scale of 1"=3000'. Figures 2B-02 through 2B-06 are drawn to 1"=10' vertical with no horizontal scale. Stability and seepage analyses (Figures 2B-07 through 2B-12) are drawn to 1"=40' while Figure 2B-13 is drawn to 1"=20'. This information is shown in the title block on the right hand side of the drawing. Scales will be presented more clearly on follow-up submittals.

Comment: 2B.3 Pg. 22
The permeabilities shown are based on a depth interval. Are the tests representative of each of the subsurface materials encountered? Field permeabilities need to be characterized for the each subsurface materials encountered. Subsequent design will require evaluation of different thicknesses/layering and cutoff configurations, when developing design sections along segments of the embankment alignment.

Response: The screened intervals were selected primarily after drilling the SPT boring at the piezometer locations. Screened intervals were established in order to characterize the permeability values of various soil/rock strata. This information will be presented more clearly in a subsequent submittal

Comment: 2B.4 Pg. 23
The second table summarizes laboratory compaction test results. I recommend reporting the optimum moisture content (which is typically included) for each of the tests.

Response: Optimum moisture contents were recorded for the moisture-density relationship tests and will be reported in the next submittal.

Comment: 2B.5 Pg. 23
Item 6, describes a core trench as a method of seepage mitigation. A clay core cutoff trench beneath a dam is the most positive methods to reduce underseepage. Subsurface conditions are

conducive to construction of a cutoff trench beneath the dam. However, other methods, such as a soil bentonite slurry wall, should be considered for portions of the dam alignment where a good cutoff layer is very deep or inconsistent (e.g. the eastern reservoir portion around parcel F, see summary logs on Figure 2B-06).

Response: Agree. This will be evaluated during the next phase of the work.

Comment: 2B.6 Pg. 24, Para. 1
Be very, very careful when using the term "normal" to describe stripping, clearing and grubbing of existing vegetation. One man's normal might be another's piping path. Setting the right requirements early on can aid in the development of the appropriate specifications later. Some might consider matted vegetation the same as a 6" diameter tree, so if we mean complete vegetation removal for the base of the embankment, let's set that requirement now.

Response: Agree and appreciate this helpful comment. Grubbing and stripping is to include complete removal of the root zone.

Comment: 2B.7 Pg. 24
Item 7, 2nd paragraph. The dimensions of the perimeter canal may be larger than needed as a seepage collection ditch. Is the perimeter canal for seepage collection and conveyance, or for water distribution? A typical Earthfill dam design consists of an impervious zone and underseepage cutoff, as shown on Figure 2-12. A "large" 20 ft deep collection trench would not normally be needed. The design shown on Figure 2-12 provides seepage stability by reducing the potential for excess seepage forces exiting at the downstream (exterior) portion of the embankment, and also reduces the amount of underseepage loss. Since water will be pumped to fill the C-43 reservoir, the "cost of water lost" by underseepage should be considered as a design factor in developing the embankment section and foundation cutoff. The perimeter canal level was reported as 15 feet NAVD, which is slightly different than stated earlier in the report.

Response: The perimeter canal is to be used for seepage collection, conveyance, water distribution and borrow. An evaluation of the "cost of water lost" by under-seepage will be made prior to the next submittal. The surface water elevation in the perimeter canal varies throughout the project. Elevation +15 feet NGVD was selected as an average condition. Our next analysis will utilize a more rigorous evaluation of perimeter canal water surface elevations.

Comment: 2B.8 Pg. 24, Item 7
Crest width (15 feet) matches discussion in Section 2-I, 6b. Good.

Response: No response necessary.

Comment: 2B.9 Pg. 25, Item 8
Paragraph includes a sentence, "We also understand that in order to avoid a piping failure due to seepage through the dam (explain later), …". The (explain later) should be replaced with (explained in Section 2-B, 9, para. 4.).

Response: Agree.

Comment: 2B.10 Pg. 25
Where is the "piping failure due to seepage through the dam" referenced explained?

> ***Response: The reference to "piping failure due to seepage through the dam" is explained on p. 26, in the paragraph at the bottom beginning with "Regardless of the dam section utilized,".***

Comment: 2B.11 Pg. 26, 2nd Table
Is it possible to add the existing case and utilize minimum canal elevations for the Berry Header Canal and the Duda NE Rim Ditch (maintained at 23 feet.) to determine the current rate of seepage available to the north for water supply?

> ***Response: During the Pond Seepage evaluation, it will be possible for us to estimate the existing rate of seepage to the north, and to determine the impacts of the project on water supply to the north.***

Comment: 2B.12 Pg. 26
The paragraph below the second table states that the seepage rates are relatively low for each of the sections. What is the basis for this comparison? Also, what is the cost of the "lost water"? The C-43 reservoir will require pumping to be filled. The "yield" of the reservoir (water supplied less evaporative and seepage losses) should be evaluated and included in selecting design features.

> ***Response: The basis for comparatively low rates of seepage is the EAA, where seepage rates are documented to be relatively high. The "cost of water lost" will be evaluated during the next phase of the work.***

Comment: 2B.13 Pg. 27, Item 10
Can controlling the water elevation in the Berry Header Canal during construction aid in controlling the water table (groundwater) impact on excavation and placement for areas near and north of the Header Canal? How would this impact Duda's Main Canal?

> ***Response: Controlling the water surface elevation in the Berry Header Canal may be advantageous from the viewpoint of constructability. The potential advantages and impacts on offsite areas will be evaluated during the Pond Seepage portion of the work and in Task 5.3.1.1 Site Specific Modeling.***

Comment: 2B.14 Pg. 27, Item 11
Since a recommendation is being made on a specific method of slope protection, might want to beef up the rationale for the use of soil-cement. Also, identify other methods and a brief description of their pros and cons.

> ***Response: Agree.***

Comment: 2B.15 Pg. 27
Item 10) 2nd paragraph. Describes placement in relatively thin lifts (i.e. 12 inch thick loose measure). A 12 inch thick loose lift is actually relatively thick for rolled earthfill dam. I would expect the final lift thickness specification to be about 8 inch loose lift for the clay core zone, and no more than a 10 inch loose lift for the exterior sand shell zone.

> ***Response: Agree.***

Comment: 2B.16 Fig 2B-06
The subsurface logs indicate that a good "clay cutoff" zone is not consistent or shallow in this area. Significant underseepage loss could occur in this area without some special treatment (deep cutoff). The underseepage loss potential, and seepage control measures, should be evaluated in detail for all segments in the next stage of design, as well as reconfiguration of the dam to avoid this area.

Response: This evaluation shall be done in the next phase of the work.

Comment: 2B.17 Fig. 2B-07 through 2B-12
The seepage and stability analyses models do not appear to reflect the drain system (graded filter and lateral drains) described in the text and shown on Figure 2B-13. The internal phreatic level within the embankment is too high for the drain system shown. This should be checked and revised as needed in the subsequent analysis. In addition, the core zone should be included in the analysis since it will effect the stability calculations. The stability analyses should also include analysis of "shallower" shear surfaces. The shear surfaces shown are deeper than would normally be expected. Infinite slope type failure surfaces (say shallower than 15 feet deep) should not be included.

Response: Agree. Subsequent seepage and stability analyses will include a revised phreatic surface that accounts for the presence of the graded filler.

Comment: 2B.18 Fig. 2B-07 through 2B-10
What is the basis of the shear strength parameters? Laboratory testing?

Response: Shear strength parameters for the embankment materials were based on our experience with compacted materials similar to those expected to be generated from on-site excavations, and for the subsurface materials from SPT N-values and standard correlations found in the geotechnical literature. The shear strength parameters will be refined during a subsequent analysis using field and laboratory test results.

Comment: 2B.19 Fig. 2B-13
Shouldn't the upstream slope protection (soil-cement is being recommended) be identified on the Conceptual Embankment Section?

Response: Yes, and it will be on subsequent submittals.

Comments: 2B.20 General
Please show core borings from previous investigation in your report. Please depict the all borings (current and previous investigations) in the fence diagram. All borings should be shown in Gint format as required under the scope.

Response: Law borings will be included in fence diagrams for subsequent submittals. Presentation of subsurface profiles in Gint format is not appropriate for this or subsequent submittals. Such information will be provided in the Report of Geotechnical Investigation.

Comment: 2B.21 General
Core borings CB-51 and CB-74 do not show clay although the adjacent borings have thick clay bands in them. Please confirm the non existence of the clays in the said borings. Suggest installing intermediate borings between the borings with clays and no clay.

Response: Agree.

Comment: 2B.22 General
The fence diagram (Fig 2B-02) is a good depiction of the subsurface along profile lines. The n values, PL, organic content and other info collected needs to be shown.

Response: Presentation of SPT N-values and laboratory test data is not appropriate for this submittal. That information will be included in the Report of Geotechnical Investigation.

Comment: 2B.23 General
QA/QC: Check the top of boring elevations as shown in Figure 2B-02 and 2B-04. CB-17 (elev.19.87 feet) looks to be higher than CB-38 (elev. 19.87 feet). Similarly, check CB-72 with CB-39 and CB-58 with CB-57.

Response: Agree, the top of Borings CB-38, CB-39 and CB-58 are plotted too low and CB-57 is plotted too high. We will revise accordingly.

Comment: 2B.24 Page 22, (4)
Packer test is very approximate. It is not recommended for purpose of planning for reservoirs. The packer test has limited effect on the adjacent formation and therefore does not provide a good understanding of aquifer properties. Its limitation is apparent from the range of values obtained for hydraulic conductivity. Please identify the borings where the "packer test" was conducted.

It is recommended that an APT test be performed and transmissivity value obtained using methods for partially penetrating wells in unconfined aquifer.

Response: Packer test is utilized extensively by the U.S. Bureau of Reclamation for planning and design of reservoirs throughout the United States. The next submittal will show the screened intervals and permeability values measured for each. We are currently considering the implementation of an APT for the project utilizing some of the piezometers that have been installed already for the project.

Comment: 2B.25 General
Geotechnical overview: Please discuss from where the construction material will be obtained. In case, a clay core is implemented it will be expensive and may pose constructability problems. It will probably be ideal to leave the clay layer as it is without disturbing it. Please identify the type of clay present and how long it needs to be dried before it can be (if) reused for the clay core.

Response: The materials utilized for embankment construction will be obtained from on-site excavations. The low permeability core materials will be obtained from deeper portions of the perimeter canal cut, which for the most part, contains increasing amounts of soils fines. This was taken into account during the conceptual embankment design stage. Handling, spreading, disking and compacting the clay materials will be a critical aspect to the embankment construction.

Comment: 2B.26 General
Why do you feel that dam settlement will be minimum (less than 6 inches)?

> ***Response: Dam settlement of 6 inches is an estimate that will be refined in a subsequent analysis.***

Comment: 2B.27 General
There is no discussion on removal of clays or foundation issues at the pump station/gate locations? The existence of clay may warrant the use of piles at the pump station locations.

> ***Response: This is a good comment and an important consideration that will be addressed later in the project sequence. Pump station locations have not been identified; therefore, this discussion is premature at this time.***

Comment: 2B.28 General
Dam Stability studies: STABL6H was run on assumption that levees will be constructed from embankment fill however if other materials are used the shear surfaces are not valid. Please clarify.

> ***Response: The most practical and likely source of embankment materials is from an adjacent and parallel cut in the form of the perimeter canal. The slope stability analysis follows this logic.***

Comment: 2B.29 General
Slope protection from waves: Terracing and then plastering a cement mix will have a significant cost implication. Consultant needs to look at other alternatives like riprap, articulated blocks or a combination.

> ***Response: Agreed.***

Comment: 2B.30 Page 25
Slope stability: Please indicate where the subsurface profiles are from.

> ***Response: The subsurface profile for the site was subdivided into four generalized profiles for the purpose of conceptual embankment stability evaluation. These generalizations will be further refined during a subsequent dam stability analysis.***

2C - HTRW

Comment: 2C.1
Might want to add that existing Phase I & II assessments are being reviewed. Additional reviews will occur as reports by others are provided by the SFWMD.

> ***Response: This will be added as suggested above. SEA will include a paragraph describing the status of the existing effort.***

2D - ARCHAEOLOGICAL

Comment: 2D.1
Might want to mention the date that the other studies will be available.

> ***Response: A completion date will be determined and this section will be updated accordingly.***

Comment: 2D.2
A map would be nice to show what work has been completed.

> ***Response: A map will be included showing the former and current site boundaries and the new area requiring Phase I update work.***

2E - ENVIRONMENTAL

Comment: 2E.1 Pg. 30, Para. (b)
As previously noted, "16 acres of wetlands" is closer to "200" acres of wetlands

> ***Response: The acreage impact is 190.7 acres and this will be reflected in the next submittal of the DDR.***

Comments: 2E.2
Does not appear that much effort has been put forth toward this effort since Work Order 05 was executed in April 2004. Expect to see significant improvement in this next report.

> ***Response: The draft Wetland Evaluation Report was completed and submitted on 9-24-04 and the protected species assessment will be submitted on 10-6-04. Agency coordination will be continued in October-January for the project. A tentative meeting with the USACE is scheduled for 10-15-04. A multi-agency coordination meeting is planned for early December***

2F - TELEMETRY

Comment: 2F.1
Instrumentation of the embankment and perimeter seepage collection canal will be required. The telemetry should include provisions for this instrumentation as well.

> ***Response: Need input from SFWMD on what is required for embankment instrumentation. Wes spoke with Bob Urban, SFWMD, on how he interprets instrumentation of the embankment. Bob knows of no other instrumentation other than the standard drawings. This probably refers to the use of embankment mounted wells rather than ones on a platform or dock. With the potential for a lot of wave action, less structure exposed is better.***

2H - HYDROLOGY AND HYDRAULIES

Comment: 2H.1 Pg. 37
Item 1) 2nd paragraph A 3 to 5 foot design freeboard on top of the surcharge pool is not the standard practice for freeboard design of a high hazard structure. A calculation of rainfall and wind conditions from the two recent hurricanes (Charley and Frances) would likely show that the dam would have been overtopped, or nearly overtopped by rainfall within the reservoir and wind generated waves with only 3 to 5 feet of freeboard. As a high Hazard structure (C-43 is located adjacent to a major highway as well as some agriculture and light industry), the dam and spillway must be designed to protect the public. Development of the PMP event, inflow design flood and freeboard should be revised as noted in the General comments, above.

Response: Development of the PMP event and freeboard will be revised as indicated in the responses of general comment 1G. The procedure in Section 9c of ER 1110-8-2(FR) indicates that reservoir with surcharge pool elevations within three feet of the maximum pool level for a substantial period of time, 36 hours or longer, have increased probability that high winds in the critical fetch may coincide with this level. Therefore, when the inflow design flood pool hydrograph is within three feet of the maximum pool for 36 hours or longer, the minimum freeboard will be five feet of embankment dams and three feet for concrete dams.

Comment: 2H.2 Pg. 37
Item 1) 1st paragraph. "Selecting the top elevation of a reservoir dam requires an accurate estimate of wind set up and water wave run up."

Response: Yes, agree

Comment: 2H.3 Pg. 37/38
Item 1) (a). In addition to evaluating fetch length normal to the embankment, two additional scenarios must be evaluated. They include: 1) the maximum fetch distance in any direction, with wave run-up corrected for the angle to the slope, and 2) fetch distance in the direction of the maximum recorded wind speed. Wave run-up is then based on the worst case scenario. Wave height calculations should be included in the DDR.

Response: The wave run-up estimate will include not only the maximum fetch distance in any direction, but also include the correction for the upstream embankment slope. The estimate was based on the fetch distance in the direction of the maximum wind speed and the worst case scenario. Wave height calculations are already included in the Appendix 2I-1 of the draft DDR. Wave height and wave period in the revised calculations will be shown also.

Comment: 2H.4 Pg. 39 Item (e)
The soil cement slope protection is described as 6-inch thick by 8-ft wide horizontal lifts. A thicker lift thickness (up to 9 inches) may be suitable and a stair step height of 18 inches to 2 feet should be considered. Contractors generally prefer to have a higher step height, and a lower slope roughness coefficient could be used with a higher step.

Response: The design of the slope protection requires additional refinement which will be completed during the next phase of the work. It will involve comparison of the stair-step method and the plating method, with various thicknesses in order to develop a cost effective design.

Comment: 2H.5 Pg. 40
1st paragraph. The reference to an example presented in Table 2H-01 does not match the Table which shows the ratio of wind overland to wind over water.

> ***Response: Table 2H-01 should be Table 2H-02 which was inadvertently left out in the draft DDR. The reference of Table 2H-01 is the USACE engineering manuals EM 1110-2-1420.***

Comment: 2H.6
Item (g). Technical Paper No. 40 reports data for 1 year to 100 years events. The appropriate references that are used in engineering practice to determine the PMF are the Hydrometeorlogical Report Nos. 51, 52 and 53. The Inflow Design Flood should be determined in accordance with the procedures described in those reports.

> ***Response: As indicated in the response of comment 1G, the Inflow Design Flood will be determined in accordance with the procedures described in Hydrometeorological Report Nos. 51.***

Comment: 2H.7
Item (g) describes the overflow spillway crest will be set 14 inches above the normal pool elevation. More detail is needed to show how the spillway will be designed to avoid overfilling of the reservoir above the normal pool elevation.

> ***Response: The overflow spillway crest will be set 1.2 feet above the normal pool elevation based on SFWMD permitting requirement to store a 25-year, 72-hour rainfall event that occurs at the time the reservoir is at the normal pool elevation. More details to show how the spillway is designed to avoid overfilling of the reservoir above the normal pool elevation will be provided in the site-specific model simulation.***

Comment: 2H.8
It is not clear how the freeboard requirement of 5 to 7 reported was arrived at. The stated IDF value of 36 inches plus 14 inches equals 4.1 ft. and adding 3 to 5 feet from page 37 does not match the reported number. Freeboard calculations reported in item (f) ranges from 6.2 to 15.6 feet. Freeboard design needs to be more thoroughly explained and the analysis revised as described in the comments above.

> ***Response: Freeboard calculations will be revised as indicated in the response of comment 1G and the results will be given in the appendix of the Pre-Final DDR.***

Comment: 2H.9 Pg. 55
Perimeter Canal. The perimeter canal does provide a ready location to obtain borrow for embankment fill. However, as described elsewhere in the report, borrow from much of the perimeter canal will be quite wet and will require processing for embankment fill. Design of the dam with a core in the embankment and a core cutoff beneath the dam will significantly reduce the quantity of underseepage. Therefore, a full depth perimeter canal would not be needed. The next phase of design should include seepage analyses to size the depth and location of the perimeter canal to satisfy the design requirements of the design section. In addition, locating economical sources of clay need to be confirmed and included in the cost estimate.

> ***Response: The most economical location for borrow for embankment construction is from an adjacent and parallel cut. Generally speaking, the deeper the cut, the more***

clayey the subsoils become. The next phase of the study will determine the optimum depth and location for the perimeter canal.

Comment: 2H.10 Pg. 55
Reservoir seepage and Evaporation losses. More information is needed on the method of analysis and water balance computations. How is the evaporation component determined and how does it compare to the evaporation records for the area? What is the basis of the seepage component? Seepage losses will be tied to the embankment and foundation cutoff design. Seepage results from the SEEP/2D modeling would be expected to represent potential seepage losses. What Model scenario was used, how was the loss calculated?

Response: Reservoir seepage and evaporation were calculated by the integrated surface/ground water model MIKE SHE. Evaporation was determined using a Potential Evaporation Rate file provided by SFWMD Hydrologic Systems Modeling and converted to actual evaporation by MIKE SHE. The equations for this model have been reviewed in detail by SFWMD and accepted as a model for use in CERP. Seepage results are based on a 3D groundwater model MIKE SHE that uses the same equations as MODFLOW and was calibrated for use in the C-43 watershed. Seepage rates were also calculated by DET using SEEP2D. The rates of seepage for MIKE SHE and SEEP2D (by DET) were similar.

Comment: 2H.11 General H&H
Majority of the Hydrology and Hydraulics Report is a presentation of general method followed and then a quick statement of decisions or findings. Very little in discussion or presentation of proof of concept or results. Very difficult to check findings without knowing what parameters were selected for various subjects.

Response: A separate report has been provided that documents the calibration of the C-43 integrated surface/ground water model. The reference is: DHI, 2004 (Draft). Updated Integrated Surface/Ground Water Model for the C-43 Basin, SFWMD.

Comment: 2H.12 General H&H
Portions of the hydrology and hydraulics sections are counter stated in the hydraulic modeling section.

Response: This will be checked and revised as appropriate

Comment: 2H.13 General H&H
There is no analysis toward separation of cleaner seepage water and "dirtier" bypass water around the reservoir. The amounts may be too small to consider trying to keep that waters separate, but it should be addressed and presented as being analyzed.

Response: A separate section will be added to the report that describes water quality.

Comment: 2H.14 General H&H
Without having MIKE model as a submittal with the report, there is no way to technically review the actual modeling effort.

Response: See the calibration report cited above in the response to comment 2H.11.

Comments: 2H.15 General H&H
Submittal of computations would allow a more complete technical review or more presentation of computational results within the body of the report would make report more defendable. The review consisted taking the results that are presented on "faith".

Response: Appendices and model files will be provided.

Comment: 2H.16 General H &H
There is no discussion or presentation of pre-conditional modeling to compare the proposed model against. Similarly, there is no discussion in the modeling section of how there various options either improve the existing condition or prove the CERP initiatives change the existing conditions.

Response: Figures 2H-11, -15, -16, and tables 2H-03, -05, and -06 all show results from the calibrated model, which is the same as a "pre-conditional model". Additional plots will be provided in the next report.

Comment: 2H.17 General H & H
Several statements regarding assumptions, possibilities, and maybes. General feel of the modeling write up should be more assertive and definitive.

Response: There are large data gaps in the watershed that do not allow for precise definition of hydrology and hydraulics in the C-43 watershed. There is no definitive information on the actual amounts of irrigation water used by the farming community. There are many hydraulic structures in the watershed that have the ability to pump at least the average flow for the entire C-43 Basin, however it was not possible to precisely define the size of these pumps. The Stanley Consultants design team would greatly appreciate it if additional information was supplied that could reduce uncertainties. However, the data gaps remain, and it should be recognized as they could affect the response of the watershed to the proposed Berry Groves Reservoir.

Comment: 2H.18 Page 37, Para. 3
A verbal listing of the key parameters to determine wind setup and wave run up are verbally presented, but the final numbers associated with these parameters are never clearly presented. There is no way to check the calculations that lead to the majority of the decisions presented: emergency spillway elevation, final max fetch length, etc.

Response: Values of parameters were already presented in the Appendix 2I-1 and Table 2H-02 of the draft DDR.

Comment: 2H.19 Page 38, Para. (a)
A description of the approach is verbally described on the finding of the fetch lengths, but there is never a presentation of the final decisions or lengths determined. Is there a detail available that shows this work and the final determination and what was the final length(s) that were determined?

Response: The final values of the fetch lengths were presented in the Appendix 2I-1 of the draft DDR.

Comment: 2H.20 Page 38, Para. (b)
Lots of description of the method and formulas used to determine wind velocities, but there is never a final presentation of the various parameters or the final wind velocity determined.

Response: Values of wind velocities and a final presentation of the various parameters were presented in the Appendix 2I-1 of the draft DDR.

Comment: 2H.21 Page 38, Para. (c)
Minor point: The Site/Civil Section mentions that the maximum average is going to be 18.75 ft and the hydraulics section calls it 18.7 ft.

Response: Will use the same number.

Comment: 2H.22 Page 39, Para. (d)
The dam slope discussion talks about the slopes that most dams are built with and then makes the statement " all interior slopes of dam were assumed to be 3H:1V." This should be a more forceful statement since it is definitively defined to be 3:1 on page 62 of Section 2-I, Site-Civil.

Response: The interior and exterior slopes of 3H: 1V were determined to be used in the kick-off meeting on May 7, 2004 (Appendix 1B-1). Freeboard calculations for slopes 4H: 1V and 5H: 1V were conducted and will be presented in the Pre-Final DDR.

Comment: 2H.23 Page 39, Para. (e)
All slope roughness discussion is centered on soil cement being the primary slope protection mechanism. Is soil cement going to be the most favored approach? During this early 30% design effort, it is expected that a compare and contrast exercise would have occurred to determine the most helpful method for control of wave runup. There is no comparison analysis presented, simply a statement of the roughness associated with soil cement slopes.

Response: Freeboard estimate and comparison analysis for riprap and soil cement were performed and the results will be presented in the Pre-Final DDR.

Comment: 2H.24 Page 40, Para. (f)
Table 2H-01 is presented as the procedure of wind setup and wave run up analysis, but this chart is more of a presentation of results. There is no proof of concept discussion as to how the numbers were developed, so they must be taken on faith. More discussion of the computations undertaken would be more helpful during technical review.

Response: The wind velocity over water is higher than that over land surfaces because of smoother and more uniform surface conditions. According to EM 1110-2-1420, wind velocity over water was adjusted based on Table 2H-01. More discussion of the computations undertaken for Table 2H-02 will be included in the Pre-Final DDR.

Comment: 2H.25 Page 40, Para. (g)
The following statement is made, "Since the Berry Groves reservoir isn't a classic dam with a contributing watershed above it, the contributing watershed area used to compute the IDF was simply the 11,500-acre area of the impoundment itself". Agree with this statement; however, others probably will not and there needs to be evidence presented to quantify the decision. Some sort of drainage mapping proving that the classic drainage pattern does not exist is needed. Others will argue that the options that utilize PS-2 gathers runoff from the southern side of the

catchment and the PMP should be applied to that portion. Bottom line, there needs to be a much deeper defense presented in the report to defend the minimal area decision.

> ***Response: Since the C-43 West Storage Reservoir isn't a classic dam with a contributing watershed above it, the contributing watershed area used to compute the IDF was simply the 11,500-acre area of the impoundment itself. This is a worst case scenario, but will verify the catchment area and determine if the PMP should be applied and adjusted.***

Comment: 2H.26 Page 40, Para. (g)
The overflow spillway crest is stated that it will be set 14 inches above the normal pool elevation. There is no presentation of the dimensions or functionality of this emergency spillway. Taking the freeboard requirements presented for the various options in Figures 2H-02 and 2H-03, the notch in the embankment wall would be at least 10 feet deep. More description toward the configuration and functionality of the spillway is needed along with diagrams.

> ***Response: More description of the spillway will be provided by the site-specific model simulation along with diagram. More details to show how the spillway is designed to avoid overfilling of the reservoir above the normal pool elevation will be provided in the site-specific model simulation.***

Comment: 2H.27 Page 41, Para. 2
No discussion in this section toward pre-condition analysis. It is necessary to open this section with a quantification of the existing condition hydrology and hydraulics aspect of the site so that a compare and contrast analysis can be done. The section does not present the typical 30% design analysis of "here is the existing condition", here is the functionality of how the proposed condition meets the CERP objective and finally here is how the existing condition is impacted in order to meet those CERP objectives. This section simply presents the functionality on a high level of the proposed project toward meeting CERP objectives.

> ***Response: Section H. 2) (b) was intended to address this. Further information will be needed from the commenter so that this section can be revised to address the commenter's concerns.***

Comment: 2H.28 Page 41, Para. 2
A sub-basin map is presented which contradicts the IDF discussion that the PMP storm will only impact the reservoir footprint. This map shows that the catchment to the south flow through the reservoir site. There is no discussion stating that this flow is routed around the reservoir, so other reviews would argue that the PMP storm should be analyzed over a larger area. Defense of the PMP area need to be addressed in this modeling section as well.

> ***Response: There was significant discussion regarding how the PMP analysis would be conducted during the two months prior to submission of the 30% Design Report. An approved approach to analyzing the potential impacts of the PMP was not obtained until two weeks prior to the submittal of the 30% Design Report. In addition, surveyed cross sections for tributaries north of the proposed reservoir were not received in time to complete these simulations. The cross sections have been incorporated into the model and the PMP analysis has been conducted. The PMP storm has been modeled for the Berry Groves Catchment and shows the impact for the "With and Without" Project Condition. This information will be provided in the next submittal.***

Comment: 2H.29 Page 42, Para. 2 (b)
There is no presentation of the calibration results. Was the model taken through a verification process as well? What were the dates associated with the calibration (and/if) verification period? There is no way to ensure that extreme wet and dry periods were covered during calibration/verification. Without presentation of calibration results, there is no way to check model reliability.

Response: See response to comment 2H.11.

Comment: 2H.30 Page 43, Para. 2(b)
A site-specific model is mentioned with a finer grid, but no images showing the details of this model are presented. There is no presentation of calibration/verification and no presentation of details of this model. Pre and post condition modeling conducted with this finer grid?

Response: Additional information will be provided in the next submittal.

Comment: 2H.31 Page 45, First Paragraph
The large loss of water between the S-77, S-78 and S-79 gages needs to be explained much more definitively then it is. There explanation given is "possibly due to high irrigation withdrawals in the sub-watersheds…". These flows form the basis of the entire analysis and the explanation should be more concrete for the "missing" water Figure 2H-07. We are quite sure that it is flow by-passing around the gage through Canals 1, 2 and 3, we should concretely state so.

Response: There are no as-built cross sections or measured flows for Canals 1, 2, and 3, and there were additional data gaps cited above in the response to comment 2H.17. We feel that the statement of probability is more appropriate and that presenting the text as a statement of fact would be misleading to the reader.

Comment: 2H.32 Page 47, First Paragraph
The statement is made, "It should therefore be expected that the reservoir may not be able to augment flows at S-79 sufficiently to meet low-flow requirements during dry years, and that it will probably not be able to control peak flows during the wet years..." This statement needs to be quantified with pros and cons toward CERP objectives and impacts on the 30% design of the reservoir. Quantification of the impact of this statement is need early in the design.

Response: This project is a Phase I reservoir. In addition, the Berry Groves reservoir is one element of an overall Project Implementation Report for the C-43 Basin. Accordingly, the reservoir is not expected to meet all CERP objectives for the C-43 Basin. The reservoir appears to do a very good job at meeting many of the objectives even though a prior study (DHI, 1999) concluded that over 400,000 acre-feet of reservoir storage would be needed to meet the multiple objectives of storage reservoirs in the C-43 Basin that had been established in 1999.

Comment: 2H.33 Page 55, First Paragraph
The perimeter canal findings are presented as a statement of "fact", but there is no proof of concept presented in order to conduct a review of design effort.

Response: There has been no detailed sensitivity analysis conducted on the perimeter canal dimensions. During the initial modeling of the scenarios, simulated water levels in portions of the perimeter canal north of the proposed reservoir were higher than

desired, and modifications were made to the proposed perimeter canal dimensions. Additional details can be provided to document this work.

Comment: 2H.34 Page 55, Third Paragraph
Reservoir seepage and evaporation losses are presented as a statement of findings, but there is little in proof of concept presented in the report to allow a technical review. The last paragraph mentions "The low water levels during normal conditions is due to elevation of the proposed weir..." This statement is not following by analysis outlining whether this impact is manageable by local AG industry, residential wells, and environment. How far and wide is this impact? Can the local community adjust to it? There needs to be much more analysis toward the impact of this during this 30% design phase. A pre-condition model would help to quantify the changes due to the reservoir.

Response: Figures 2H-15 and -16 show existing conditions (which can also be called pre-condition analysis or Calibration Model). There is additional water available for irrigation with the reservoir when compared to existing conditions. Additional information will be provided to augment the discussion presented on page 55.

2I - SITE/CIVIL

Comment: 2I.1 General
As described above for Section 1E – Cell Configuration, further work is recommended to optimize the design of the C-43 Reservoir. In particular the subsurface investigations shows that the depth to a clay layer for a cutoff to reduce under seepage losses is variable and/or deep, in particular on the east side of the site. Re-configuration of the reservoir to reduce this "inefficient" portion of the reservoir with the potential for higher seepage losses needs to be evaluated further.

Response: Based on the water depths that the design is attempting to maintain in the reservoir, the eastern portion of land is necessary to achieve the programmed storage volume. The issue of acceptable water depths will be re-evaluated. The seepage losses will be evaluated in more detail prior to the next submittal.

Comment: 2I.2 Pg. 58
The Table number is missing.

Response: The table number will be added.

Comment: 2I.3 Pg. 61
1st paragraph, line 7, describes that it was determined that a perimeter canal was needed to encompass the reservoir on the south, east and north sides. It is not clear what this need is based on, or why a canal would not be needed on the west. The conclusion should be explained further, and the rationale provided to justify the need.

Response: The perimeter canal is intended to serve several purposes including seepage interception, providing conveyance around the reservoir and a source of water for hydrating the remaining tributaries north of the reservoir. The west side is occupied by the Townsend Canal and it is felt that a perimeter ditch on that side will be redundant. The DDR will be revised to include this explanation.

Comment: 2I.4 Pg. 61, para (5)
Second sentence "They are dated July 21, 2004". Not sure what context of this statement is.

Response: The date is referring to the two alternatives mentioned in the preceding sentence. The statement will be revised to clarify.

Comment: 2I.5 Pg. 62
The average dam height will need to be revised based on revisions to the Inflow Design Flood and freeboard calculations.

Response: The dam height will be revised as required.

Comment: 2I.6 Pg. 62
Item (c). Is the bench for ASR wells for access or for actual well locations? If it is intended for ASR wells, what is the depth of the injection zone and the potential impact of ASR wells in this location should be discussed.

Response: The bench is intended to serve as both the location of and access to the ASR wells. Further discussion regarding the impacts of the ASR wells to the reservoir will be provided. Input from the SFWMD is needed regarding the ASR wells. Also, see response to 2I.15.

Comment: 2I.7 Pg. 62, para (b)
Listed that Soil-Cement protection will be used on slopes. No analysis or justification has been provided that this is the most cost-effective method of shore protection for this project. An analysis should be performed that considers other options such as rip-rap, concrete block-mats, geocells.

Response: The analysis that considers other options such as rip-rap, concrete block mats, geocells, will be provided in the next submittal.

Comment: 2I.8 Pg. 64, Item 8)
What areas were identified for borrow for the low permeability core material, or the graded filters? How was the depth and location factored in to the cost estimate in section 3?

Response: The low permeability core materials come from the lower reaches of the perimeter canal cut. The graded filter requires design, but is expected to consist of clean sand and gravel materials. The clean sands may be obtained from relatively shallow on-site excavations. The gravels may require import from an offsite source, or may be provided by crushing and screening materials produced from on-site excavations in the limestone formation.

Comment: 2I.9 Pg. 64, Item 9)
Vehicular access to the top of the dams should be restricted. While access is needed for O & M, the type of traffic should be restricted, and security considerations should be included in the design.

Response: Access will be restricted to O & M vehicles. Security measures will be researched and identified in consultations with all involved agencies.

Comment: 2I.10 Pg.67 Item J. 2)
The criteria describes that there is "no requirement for Anti-Terrorism/Force Protection measures for the buildings". As a high hazard structure storing a large volume of water, security of the reservoir should be considered in the design and access to the facility.

Response: Noted. This will be discussed in SFWMD/Design team meetings.

Comment: 2I.11 Page 57, paragraph 2 (a)
Freeboard was estimated to be 10'. What was the basis for this? Were calculations done for wave run up and wind tide?

Response: The freeboard estimate was based on the wind setup and wave run-up analysis.

Comment: 2I.12 Page 62, paragraph 6
Top of page. There is a statement that the dam cross section consists of three major components - height, crests width, and angle of slope. There are a number of other major components that also need consideration such as key core, seepage control, bank stabilization, compaction, materials, etc.

(b) The top levee width should be 14' wide minimum.

Response: (a) All applicable components will be considered in designing the dam cross section. (b) Top width of dam is 15' as shown on the typical sections.

Comment: 2I.13 Figure 2I-12
The Typical Section – West Perimeter Dam indicates 84' for the Townsend Canal. Will this width be the same for the two proposed alternates?

Response: Final improved cross section for the Townsend Canal will be determined based on more accurate flow data as it becomes available. The bottom width has not been determined for either alternative.

Comment: 2I.14 Figure 2I-13
Typical perimeter canal section indicated at 3:1 slopes. May consider variable slopes (3:1 transitioning to 2:1) to save space. At 3:1 the perimeter canal consumes almost 200 acres of property.

Response: Other side slopes will be considered in coordination with the geotechnical analysis. Figure 2I-13 does not exist.

Comment: 2I.15 Figure 2I-14
Coordination with the ASR will be required prior to finalization of this report. Need to make sure that adequate space is provided around the project perimeter to accommodate ASR wells, piping, and treatment systems. May consider moving ASR wells and facilities to outboard side of perimeter canal to minimize dependencies.

Response: Agree that it may be better to move the ASR wells and infrastructure associated with the wells to the outside of the Perimeter Canal. We have discussed this internally and feel that adequate width already exists in the typical sections. It is suggested that a meeting be set in the near future with the ASR staff at SFWMD to

make sure that the ASR needs are being met. It is suggested that ASR staff develop a typical plan and elevation indicating space requirements that can be provided at the meeting.

Comment: 2I.16 General
It appears that volumetric calculations are based on centerline of berms. This gives a water volume storage quantity error of approximately 2,000 acre feet (1%) for the site. The error factor should be noted or corrected for in the calculations presented in Appendix 2I-1.

Response: The calculations for water storage in each cell were based on the centerline of the dam. The resulting water storage for the reservoir was adjusted for the amount of fill of both the perimeter and the separation (Internal) dam as shown on the excel spreadsheets.

Comment: 2I.17 General
Subsequent analysis of storage quantities should utilize site mapping as being prepared under this contract. Subsequent analysis should include reference figures to the new basemaps to validate elevation data.

Response: Elevations used in the calculations were based on the point data from the SFWMD and some available survey points. Final alternatives will be analyzed based on the Digital Terrain Model derived from 3D-line (Mapping) prepared by Spectrum Florida, LLC. The spread sheet used to evaluate the alternatives will be calibrated using the current mapping.

2K - STRUCTURAL

Comment: 2K.1 Page 71, Para. 1 (a)
What capacity of pump station is proposed for each alternate? How many pumps? What will the pumping cost be? How will this pump station be operated and controlled? (This may be covered elsewhere) Manatees are in the C-43. The trash removal equipment will be required to be "manatee friendly". For the 30% design, will the type of foundation been determined? Spread footings, pile supported, etc.

Response: Pump station capacity and number of pumps is addressed in Section 2-O, Special Mechanical Equipment. Pump station costs and pumping costs will be addressed in the Prefinal-DDR. Control of the pump station is addressed under Section 2-P, Instrumentation and Control. Operation of the pump station will be addressed more adequately in the Prefinal DDR. We understand that the "Manatee friendly" designation is determined on a site by site basis, but the trash removal equipment we are proposing has qualified at several other installations. All that is usually required is a slower descent rate on the rake.

Recommendation as to the foundation type will be made in the final DDR, when the 30% structure design has been completed.

Comment: 2K.2 Page 71, Para. 1 (b)
PS-2 is called out to be a "medium size pump station". Define medium. What is the capacity and number of pumps? Define the requirements for back up emergency power, telemetry, automation and control.

Response: The definitions used in the CERP Standard Design Manual were followed in preparation of this report. A medium size pump station is listed as 250 cfs to 1,000 cfs. This will be clarified in the Prefinal DDR, along with recommended size and number of pumps. The modeling utilized 600 cfs capacity for PS-2. If Alternative 2 is selected, then more detail on PS-2 will be provided in the Prefinal DDR. We plan to follow the District Design Guidelines for Medium Capacity Type 2 Pump Station in developing PS-2.

Comment: 2K.3 Page 72, Para. 1 (b)
Define more clearly the load requirements for the service bridge. The bridge must have the capacity for the District's hydro crane. The surcharge for the wing walls should be 500psf minimum. The overhead doors should be a minimum of 7' 9" W by 10" H.

Response: The P&H Crane (440-TC) will be used for the bridge live load as recommended by the District for a recent design. Surcharge for the wingwalls will be 500 psf, although SFWMD Design Guidelines G-S1, sheet 1 of 2, recommends 2 ft of traffic surcharge loading for water control structure abutment walls. AASHTO LRFD Bridge Design Specifications recommends between 2 ft to 5 ft of traffic surcharge loading on retaining walls depending upon wall height. The overhead doors will be a minimum of 7'-9" W by 10'-0" H.

Comment: 2K.4 Page 72, Para. 1 (c - o)
Has any calculation been made regarding the size or flow capacity for the structures? With head differentials as great as what is proposed, will energy dissipaters be needed? Will vegetation barriers be needed?

Response: H&H will determine the required size and flow capacity of the structures, and if energy dissipaters or vegetation barriers are required. This information will be provided in the next submittal.

Comment: 2K.5 Page 79, Para. 4 (h)
Wind loads for CERP projects need to conform to the CERP Design manuals (500-yr mean recurrence interval or I = 1.51).

Response: All structures for the project will be designed as Critical Facilities (500-yr mean recurrence interval).

Comments:2K.6 Page 79, Para. 5 (a & d)
Surcharge should be 500 psf.

Response: Concur.

Comment: 2K.7 Appendix 2K-1, Page 4, Para. 3.1
I thought the purpose of this study was sort out and make recommendations regarding which option to use (Option 1 or 2). Therefore, I assume that the selected option whether to lengthen the existing bridge or make a complete replacement would be based on whether one or two pump stations are built since the flow in the Townsend Canal would be different.

Response: It is correct that this report will ultimately recommend which option to construct and that the recommendation will be tied to the required cross-section of the Townsend Canal. However, at the time this draft DDR was published, the H&H

requirements for flow in the Townsend Canal were not developed enough for a final required canal cross-section to be available.

Comments:2K.8 General Comment
The civil and structural information presented is very general. For the 30% design, I would expect to see more specific information such as number of bays for the structures, various key dimension and elevations such as length of fore bays, gate operating ranges, approximate platform heights, building locations, access routes, identification of monitoring sites, more detailed cross sections etc.

Response: This information will be included.

2O - SPECIAL MECHANICAL EQUIPMENT

Comment: 2O.1 Page 92, Item 6
Formed Suction Intake. Formed suction intakes may not be necessary with favorable suction conditions and may add significantly to the capital cost. There also may be an increase in headloss per HI. Please review the recommendation to use formed suction intakes.

Response: The use of FSI's will be evaluated further and a recommendation given in the Prefinal DDR. Note that the CERP Standard Design Manual indicates that FSIs are more efficient than suction bell intakes.

Comment: 2O.2 Page 93
Second to last paragraph, variable speed pumps. Look at controlling the engine speed as an alternative to adding a hydraulic speed reducer. This is one of the inherent advantages of using an engine driver (See EM 1110-2-3105). Pump speed could be controlled based on reservoir level. Automatic controls could also be used to prevent operation of the pump in an unstable range.

Response: Agree. Controlling pump speed with engine speed will be evaluated in the Prefinal DDR.

Comment: 2O.3 Page 91, Item 5
Trash Rack. Please clarify the exact meaning of the 2.5 ft/sec design parameter. Clarify whether the 12 inch headloss assumption will be for the purpose of system head calculations, or for structural design. Add EM 1110-2-3104 to the list of Design References. This reference indicates a minimum design differential head of 6 inches for system head calculations and 5 feet for structural calculations.

Response: Design velocities and headlosses will be clarified in the Prefinal DDR. Will add design reference.

Comment: 2O.4 Page 91, Item 4
second paragraph, last sentence. Define or provide reference for "dewatering needle".

Response: Will define in Prefinal DDR.

Comment: 2O.5 Page 92, Item 8
Pumps. It is a little unusual not to have a redundant pump. Please explain the logic for this recommendation. The total cost of the facility should be considered in looking at the possibility of it's not being able to fully accomplish its mission to collect design flows.

> ***Response: Our understanding is that SFWMD does not provide redundancy on the main drainage pumps. See meeting notes dated June 10, 2004 with SFWMD staff. Example pump stations G-370 and C-44 (S401) did not utilize redundant pumps. Need direction from SFWMD if redundancy on the main drainage pumps is required. Redundancy is provided on auxiliary pumps (domestic, cooling, lube water).***

Comment: 2O.6 Page 94, Item 9
Consider the alternative of electric motor drives to reduce capital costs and energy costs. If necessary, engine-generator sets could be provided for partial or complete back-up power. Also, the engine-generator sets could be equipped to sell power to the utility when not being used for pumping service, if economically viable.

> ***Response: It is our opinion that electric motor drives would be cost prohibitive due to the high demand charge by the electric utility. The CERP Standard Design Manual for large capacity pump stations (>1,000 cfs) says that pumps shall be driven by full diesel engines. The scope of work for this project does not currently include evaluation of electric motor drives. Need direction from SFWMD if this additional evaluation is required.***

Comments: 2O.7 Page 94, Item 9 Engines
Consider the alternative of using hydraulic turbines to generate electric power when emptying the reservoir in order to reduce net power costs by selling power back to the utility.

> ***Response: The scope of work for this project does not currently include evaluation of hydroelectric power generation. Need direction from SFWMD if this additional evaluation is required.***

Comment: 2O.8 Page 95, Item 9(c)
Natural Gas Engines. It should be noted that natural gas can also be stored on site in compressed form (see EM 1110-2-3105).

> ***Response: Noted. This will be addressed in next revision of DDR.***

Comment: 2O.9 Page 97, Item 9(j)
Cooling Water System. Trashracks should be provided at the suction inlet for the cooling water pumps.

> ***Response: Agree. Cooling water pumps will be provided with trashracks. Current plan is to provide an additional bay with trash rack for cooling water pumps.***

Comment: 2O.10 Page 99, Item 10
Discharge Arrangement. It should be noted that the discharge arrangement does have an effect on the height of the building (overall structure). Perhaps the term "size of the building" could be clarified.

> ***Response: Agree. This will be clarified in Pre-final DDR.***

Comment: 2O.11 Page 94, Item 8
Second to last paragraph. Please clarify the statement about the pumps possibly not running reliably at low head without vacuum priming. It would seem that the pump head is lower with the vacuum priming system operating. Vacuum assist may be needed for starting. If so, perhaps a work-around could be developed for the Corps requirement.

Page 100, Table 2O-02
Each alternative presented appears to be feasible. The economics will factor into the final selection for the pumping station configuration.

> ***Response: Will clarify comment in prefinal DDR. When the pump first starts, the water in the pump column is very low, which would cause the pump to operate in a run-out condition, which can be unstable. We have since verified with a pump manufacturer that the pump column fills very quickly and does not cause problems. Vacuum assist is required to establish the siphon, but not to start the pump or fill the pump column. If the vacuum assist were to fail, the pump could still operate, but the siphon may not be established. We will complete the economic comparison (Table 2O-02) for prefinal DDR.***

Comment: 2O.12 Pumps General
I would not exclude variable speed pumps. Engine rpms should be variable to match required horsepower for varying head conditions.

Require pump manufacturer to conduct physical scale modeling of each station and pump size

All critical pump parts should be made of stainless steel

Bearing lubrication system should be water

> ***Response: Agree. Will be addressed in prefinal DDR.***

Comment: 2O.13 Engines General
Include requirement for the engines to meet title 5 air quality standards for all operating ranges

Require a minimum fuel consumption rate

Allow for turbo charging

Allow variable speed. With a diesel engine you are only looking at a 300 to 500 rpm range

Engine computer system should be able to interface with station computer system and software (i.e. Allen Bradley)

> ***Response: Noted. Will be addressed in prefinal DDR***

Comment: 2O.14 Gears
Require AGMA 11 Quality number

Require anti-reverse rotation device (i.e. Backstop)

Require minimum transmission efficiency of 97%

Require minimum tooth hardness of Rockwell 58

Require bearing L-10 rating of 100,000 hours

Require self contained lube system

Response: Noted. Will be addressed in prefinal DDR.

Comment: 2O.15 Fuel System
Fuel system must meet the requirements of the National Fire Protection Code and FDEP for fuel stored over water.

Minimum tank to building set backs

Maximum capacity of fuel that can be stored inside a building

Provide fuel containment and monitoring inside and outside the building

Response: Noted. Will be addressed in prefinal DDR.

Comment: 2O.16 Pump Station Drawings
Enclosed grease lubricating systems will be required for all submersed moving appurtenances such as flap gates and backflow gate roller wheels.

Trash rake monorail should be straight with no curves. This is a lesions learned from G310 and G335, as they have had continuous problems with misalignment.

Overhead bridge cranes should be provided capable of lifting the heaviest component in the station. Sufficient overhead clearance must be provided so that the pump can be removed.

Station service bridges should be a minimum of 16 feet wide.

Garage doors need to be sized for removal of largest piece of equipment.

Response: Noted. Will be addressed in prefinal DDR.

2P - INSTRUMENTATION AND CONTROL

Comment: 2P.1 Pg. 103
Embankment Instrumentation should also be described here. Grounding and lightening protection will also be an important consideration in the instrumentation selection and design.

Response: Need input from SFWMD on what is required for embankment instrumentation. Grounding and lightning protection will be addressed in prefinal DDR.

Comment: 2P.2
Item (d) describes gated spillway controls. Provisions in the design of the spillways to ensure that the reservoir cannot be overfilled, assuming failure/inoperability of the gate controls should be included in the design.

> ***Response: Emergency overflow spillways will be provided in the reservoir and will be addressed in the Pre-final DDR.***

Comment: 2P.3 Pg. 103, Item P.1.d
Gated Spillways. Balluff level sensors are listed as the instrument for water level monitoring. Be aware that the Balluff level probe is limited to a 120" measurement range. Consider using a pressure transducer type sensor for greater measurement depths.

> ***Response: Noted. Will be addressed in Pre-final DDR.***

2Q - INSTRUMENTATION AND CONTROL

Comment: 2Q.1 Page 111, Item 2a
Electrical warning signs should be installed on electrical equipment, doors, and conduits for circuits rated 480 VAC and above.

> ***Response: Agree. Safety signage as required by NEC and lock-out, tag-out materials will be specified.***

Comment: 2Q.2 Page 116, Item 5J
Conductors. The statement is made that conductors will not be smaller than #12 AWG. You may want to consider using #14 AWG stranded for control and instrument circuits. Numerous #12 AWG conductors can become crowded in control and instrument panels. Also use solid #12 AWG only for lighting circuits. Use stranded conductors for instrument and control.

> ***Response: Agree. The #12 minimum is intended to apply to power circuits only. Stranded conductors sized #14 maximum to be used for instrumentation, and #16 is permitted for shorter runs of 24 vdc discrete and any analog I/O wiring.***

SECTION 3 – COST ESTIMATES AND APPENDIX 21

Comment: 3.1 General
Several major items appear to be missing. The cost estimate should include: a) drain material (an offsite source needs to be identified for this item so that the transportation cost can be included in the estimate), b) stripping beneath the embankment and for the perimeter canal, c) underseepage cutoff trench (volume as well as the cost to develop the required borrow), d) Mobilization, bonds, insurance etc., and e) the cost of cement for slope protection (there is currently a shortage in cement supply which is expected to abate over the next few years. The price used is probably too low considering the current price of cement and using a separate line item will allow the change in supply cost to be easily incorporated in subsequent phases).

> ***Response: A more detailed cost estimate will be provided with the next submittal. The cost estimate for this submittal was for conceptual design and provided to give an approximate overall project cost. The project was not developed to a level that would allow for a detailed cost estimate.***

It is the intent of the Design Team to provide enough cost information in the Pre-final submittal be able to recommend a final alternative and design options. This cost information will be provided so that the review team can evaluate the recommendations. However, a 30% detailed cost estimated can not be provided until the 30% design plans are complete. The plans will not be completed for the Pre-final submittal.

Comment: 3.2 General
It would be easier to review and evaluate if the cost of the pump station, bridge replacement, power line and pump station service and microwave repeater station were separated from the dam so that the estimated cost for different elements can be more readily evaluated.

Response: Agree. See response to Comment 3.1.

Comment: 3.3 General
No basis or cost build-up provided for the unit costs utilized in this report.

Response: Agree. See response to Comment 3.1.

Comment: 3.4 Pump Station
The unit cost for the pump station (approx. $22,000/cfs) appears to be substantially higher than the SFWMD experience with similar size pump stations. I would suggest using the SFWMD experience with pump stations be used as the basis for the unit cost at this conceptual level.

Response: Agree with some additional cost included for the head requirements.

Comment: 3.5 Relocate 3 Ph Transmission line
The unit price used appears to be low. The unit cost should be evaluated more to develop a suitable rate.

Response: Will investigate unit prices for prefinal DDR for overhead lines. The impact of future ASR wells is outside this scope but will be much more significant than the pump station and water control structures.

Comment: 3.6 Borrow on site
What is included in this cost? A unit cost of $5.00 per cubic yard seems high. Is the 1-mile haul one way or round trip? Why such a long haul distance? A Contractor will look to use the minimum haul length and would be expected to develop a moving borrow pit parallel to the embankment to keep the shortest haul possible.

Response: Good questions. The unit cost will be refined and a detailed analysis provided with the next submittal.

Comment: 3.7 Compaction
A unit rate of $1.84 per cubic yard seems quite high for the compaction, on top of the excavation/borrow items.

Response: This will be reviewed prior to the next submittal.

Comment: 3.8 Embankment Quantities
What embankment section was used for the cost estimate? How were the different requirements for different embankment zones (sand shell, clay core and drain/filter material) accounted for in developing the cost estimate?

Response: The embankment section was considered as a homogenous section for this initial conceptual cost estimate. It will be refined into individual components for the next submittal.

Comment: 3.9 Final Design
At the Conceptual Design level the estimated number seems low. I recommend using a cost of the 6 percent of construction cost, less the 30 % design cost.

Response: Agree.

Comment: 3.10 Added Items
Embankment Drain, Stripping, Borrow for Core Material, Mobilization

Response: Agree. See response to Comment 3.1.

Comment: 3.11 Un-estimated Items
The Conceptual Cost estimate with the clarifications/changes noted above will still not represent all of the items that will be required to construct the project. A line item should be included to account for these "unestimated items". The unestimated items at the Conceptual Design level typically ranges between 10 and 25 percent of the construction items. The factor used should be based on an analysis of the items estimated for the project versus comparison with reservoir bid projects and/or direct cost/crew based detailed cost estimates. This different than the Contingency which should still be used. The Contingency is used to account for unknowns due the level of investigation, design definition, bidding climate, and other issues and does not account for the un-estimated items at this level of estimate.

Response: This will be considered and the percent used for the "un-estimated" items will be dependent upon the level of the design and the level of detail provided in the estimate.

Comment: 3.12 General
Only two cost estimates were presented in the report. Additional detailed cost estimates for various alternatives should be included to optimize the design.

Response: Agree. This will be provided in the next submittal. See response to Comment 3.1.

Comment: 3.13
Key quantities, comparative costs should be presented in a summary table to assist in evaluation of the data.

Response: Agree.

Comment: 3.14
Comparative costs of various alternatives needs to be developed to "optimize the design. Preliminary evaluation using the quantities provided in the Appendix, and unit costs for

embankment construction prepared for a similar project, indicate that the cost for a reservoir without Parcel D could be up to 10 percent lower in cost than the same storage volume in the reservoir alignment with Parcel D.

Response: Agree with the need to provide additional details and costs to "optimize the design". Parcel D was not included in either of the two alternatives presented. Parcel D was eliminated due to environmental impacts. You may be referring to parcel F. A more detailed analysis will be made comparing two and three cells.

Comment: 3.15 General
The estimate needs to clarify if estimates are escalated or un-escalated.

Response: The estimate is presented as present day dollars.

Comment: 3.16 General
At this level may wish to call these numbers "Order of Magnitude" rather than "Probable Construction Cost"

Response: The next submittal will provide an estimate with the details and analysis necessary for a Probable Construction Cost estimate.

Comment: 3.17 General
The earthwork quantities indicate a balanced excavation to embankment project. No swell and compaction factors have been applied. Future estimates should have shrink/swell factors applied

Response: A compaction factor of 1.2 was used.

Comment: 3.18 General
General on the costs:

a) Soil cements seems low
b) Pump stations seem high
c) Final Design at 3% seems low

Response: Comment noted. These items will be reviewed prior to the next submittal.

Comment: 3.19 General
Cost analysis between reservoir footprints should consider the incremental cost of building the dams higher using additional material versus building more embankment (which requires additional length of clay-core embankment). Would be useful to know at what point the design height for the reservoir embankment might be optimized versus additional footprint in order to help guide the acquisition of additional properties.

Response: A more detailed analysis will be provided with the next submittal comparing two and three cells. At the direction of the SFWMD, we have generally tried to keep the water depth in the 14 foot range while trying to provide at least 160,000 acre feet of storage.

APPENDICES

Comments: Page 3-1
Indicated procedure is to provide project information, preparer, checker, etc. on all computations. Note that in Appendix 2I-1 no such information is provided. Future submittals must comply with the QA/QC processes put forth

Response: Agree.

Comments: General
Is an independent QA/QC manager assigned to this project at a corporate level?

Response: Bennett Reischauer, Regional Manager, is our Corporate QA officer. David Grounds and Jon Ahlschwede coordinated the QC reviews.