Contract No. C-C20104P-WO03

Flood Protection Analysis for Broward County Water Preserve Areas C-11 and C-9 Impoundments (Final Report)

Submitted to

South Florida Water Management District

February, 2006

Project No. 36475

Burns & McDonnell
SINCE 1898

In Association With

Engineering & Applied Science, Inc.

Civil Services, Inc.

February 6, 2006

Mr. Jeffrey Needle, P.E.
CERP/ECP Department
South Florida Water Management District
3301 Gun Club Road
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South Florida Water Management District
Flood Protection Analysis for Broward County Water Preserve Areas
C-11 and C-9 Impoundments
SFWMD Contract No. C-C20104P-WO03
Burns & McDonnell Project No. 36475

Dear Mr. Needle:

Burns & McDonnell is pleased to submit this Final Report on our *Flood Protection Analysis for the Broward County Water Preserve Areas, C-11 and C-9 Impoundments*.

We wish to express our appreciation to you and to the other members of the District's and the Inter-Agency Modeling Center's staff who participated in this effort for your helpful direction, advice and assistance during the preparation of this report. We also gratefully acknowledge the contributions of our two subconsultants, Engineering and Applied Science, Inc. and Civil Services, Inc., to the successful completion of this analysis.

We trust the information presented herein will be of value to the District and to the Jacksonville District, U.S. Army Corps of Engineers in your continued efforts toward fulfilling the vision of the Comprehensive Everglades Restoration Plan. Please feel free to contact us if we may be of further assistance in those efforts, or if there are questions concerning the information, analyses, conclusions and recommendations presented in this report.

Sincerely,

Galen E. Miller, P.E.
Associate Vice President

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Index

Certification

I hereby certify, as a professional engineer in the State of Florida, that the information in this document was assembled under my direct personal charge. This report is not intended or represented to be suitable for reuse by the South Florida Water Management District or others without specific verification or adaptation by the Engineer. This certification is provided in accordance with the provisions of the Laws and Rules of the Florida Board of Professional Engineers under Chapter 61G15-29, Florida Administrative Code.

Galen E. Miller, P.E., Florida P.E. #40624
Date: _________________

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EXECUTIVE SUMMARY

The Broward County Water Preserve Areas project is comprised of three principal components:

- The C-11 Impoundment;
- The C-9 Impoundment;
- The WCA3A/3B Levee Seepage Management Projects.

Those components were recommended as a part of the *Central and Southern Florida Comprehensive Review Study Feasibility Report and Integrated EIS* in April of 1999 (the *Restudy*). In a related and complementary effort, the Water Preserve Area Feasibility Study was initiated. The Water Preserve Area (WPA) study region included multiple Comprehensive Everglades Restoration Plan (CERP) components in the area east of the Water Conservation Areas in Palm Beach, Broward, and Miami-Dade Counties. The WPA is proposed to consist of an interconnected series of marshlands, impoundments, conveyance and aquifer recharge areas. The findings and recommendations of the Feasibility Study are presented in the October, 2001 *Central and Southern Florida Project, Water Preserve Areas; Draft Integrated Feasibility Report and Supplemental Environmental Impact Statement* (the *Feasibility Study*), prepared by the Jacksonville District, U.S. Army Corps of Engineers and the South Florida Water Management District (SFWMD). The Broward County WPA Project, a separable element of the Comprehensive Everglades Restoration Plan (CERP) authorized under Section 601 of the Water Resources Development Act of 2000, includes buffer marsh areas, canals, levees, water control structures and above-ground improvements with a total storage capacity of approximately 6,000 acre-feet located in the western C-11 Canal basin and 6,600 acre-feet located in the western C-9 Canal basin in western Broward County.

Programmatic regulations developed for CERP responsive to the “savings clause” of WRDA 2000 include requirements that the level of flood protection following implementation of CERP projects not be reduced from that existing in December 2000. The purpose of the analyses summarized in this report, prepared by Burns & McDonnell Engineering Co., Inc. under SFWMD Work Order No. C-C20104P-WO03, is to assess the extent to which the level of flood protection in the western C-11 and C-9 drainage basins may be impacted by implementation of the Broward County WPA components. Those analyses generally include:

- Review and analysis of historic stage/discharge relationships;
- Review of existing groundwater model data and results;
- Hydraulic analysis of canal water surface profiles with and without the CERP project;
- Review of the conceptual design and planned operations of the project(s) for potential impacts on flood protection;
- Development of recommendations for operational and/or structural changes to the current plan necessary to mitigate any adverse impacts on flood protection under future "with project" conditions;
- Identification of such further operational and/or structural changes to the current plan as might be appropriate to minimize stormwater pumping from the western C-11 drainage basin into WCA-3A.

The analyses are specific to conditions anticipated to exist in 2010 upon completion of the presently authorized Broward County WPA projects. Future separable elements of CERP, such as the North and Central Lake Belt Storage Areas, are not considered herein. Specifically, the analyses consider the C-11 and C-9 Impoundments complete as described in the Feasibility Study, together with at least the following elements of the WCA-3A and WCA-3B Levee Seepage Management Projects necessary to transfer water between the two impoundments:

- Canal C-502A between Structure S-504 and the C-11 Canal;
- Canal C-502B between the C-11 Canal and Structure S-30;
- Siphon Structure S-502 and gated culvert S-502C.

It is assumed that those features of the Feasibility Study associated with flood protection at the Holly Lakes Mobile Home Community are completed concurrent with Canal C-502B. This analysis does not directly consider those facilities, which are to be designed and constructed specifically for the purpose of mitigating any flood protection impacts on the Holly Lakes Mobile Home Community associated with the Broward County WPA Project.

The "with project" condition considered herein also includes certain features authorized under Section 528 of the Water Resources Development Act of 1996 as the Western C-11 Water Quality Treatment Project, one of 34 "critical projects" authorized in that Act. The Western C-11 Water Quality Project includes the construction of Pumping Station S-9A and Structure S-381.

The findings of the analyses presented herein, given the design and operation of the project as presented in the Feasibility Study, are that:

1. Groundwater elevations in the vicinity of the impoundments and the WCA-3A and WCA3B Levee Seepage Management Areas will increase as a result of the project;
2. Under certain conditions, those increased groundwater elevations could be expected to result in increased volumes of runoff and groundwater flow to the primary canal system from adjacent areas;
3. In the C-9 Basin, the potential increased volume of runoff and groundwater flow from adjacent areas is more than offset by the reduction in runoff associated with removal of the C-9 Impoundment and adjacent wetlands mitigation area from the area tributary to the C-9 Canal;
4. In the C-11 West Basin, the reduction in runoff associated with removal of the C-11 Impoundment and adjacent wetlands mitigation area from the area tributary to the C-11 Canal does not fully (without mitigation, see next page) offset the potential increased volume of runoff and groundwater flow from adjacent areas;
5. The preliminary design of impoundment seepage collection and return systems presented in the Feasibility Study appears to meet defined criteria and can be expected to perform generally as described in the Feasibility Study, but would benefit greatly from additional subsurface investigations during the detailed design phase;
6. The seepage collection and return systems will capture relatively low percentages of the total seepage from the impoundments and adjacent wetlands mitigation areas, a conclusion consistent with those presented in the Feasibility Study;
7. The presence of new siphon Structure S-502 in the C-11 Canal upstream of Pump Station S-9 will introduce additional head loss in this reach of the canal;
8. It should be anticipated that, based on the record at S-9 and the results of the District's hydrologic simulations, there will be an occasional need to bypass the C-11 Impoundment and back pump basin runoff to WCA-3A at S-9. The magnitude of those diversion events can be sufficiently large as to require the use of the full capacity of S-9 for periods up to a full 24 hours;
9. For any diversion event requiring a discharge at S-9 approaching its capacity, stages in the C-11 Canal "with project" can be expected to increase above those which would exist prior to completion of the project, due almost exclusively to siphon Structure S-502. The

magnitude of that increase will vary with rate of diversion and beginning canal stage, but can exceed 1.0 foot, which is considered a significant increase requiring mitigation.

Principal conclusions of the analysis are that, given the design and operation of the project components as presented in the Feasibility Study:

- There should be no adverse flood protection impacts in the C-9 Basin due to either surface or subsurface inflows to the C-9 Canal;
- Adverse flood protection impacts can be expected in the C-11 West Basin, associated primarily with:
  - The potential for increased flood discharge volume and/or duration resulting from increased groundwater elevations. Those increased groundwater elevations result from the combined effects of the C-11 Impoundment and the control structure operations considered in the SFWMM simulation of the basin;
  - Increased head loss in the C-11 Canal between Pump Station S-9 and proposed C-11 Impoundment inflow pump station S-503, associated almost exclusively with proposed siphon Structure S-502, resulting in increased canal stages in the C-11 Canal upstream (east) of the project.

It was further concluded that those potential adverse impacts to flood protection can be mitigated through the following combination of structural and operational modifications to the project:

- A change in the wet-season operation of primary water control structures (in particular new Pump Station S-503) on the C-11 West Canal, in which pump start-stop elevations are lowered from those considered in the South Florida Water Management Model (SFWMM) simulation of the "with project" conditions, intended to generally reduce C-11 West Canal wet-season stages by roughly 0.3 foot from those reflected in the simulation;
- Conducting additional, more detailed analyses during the final design of the project to confirm the adequacy of the overall hydraulic capacity proposed for Pump Station S-503 (see text below);
- Elimination of proposed siphon Structure S-502;
- Modified operation of the project in which excess basin runoff that would otherwise be discharged by gravity to the headwater of S-9 through Structure S-381 is instead passed

through the C-11 Impoundment through continued operation of S-503 coupled with discharge through S-504 to the headwater pool of S-9 downstream (west) of S-381.

Implementation of the above modifications would require additional seepage control facilities along Griffin Road between U.S. Highway 27 and S-381 to avoid incremental groundwater elevation impacts in areas immediately south of Griffin Road in that reach of the C-11 Canal.

As presented in the Feasibility Study, Pump Station S-503 would have a total installed capacity of 2,575 cfs, which closely approximates the historic maximum mean daily pumping rate at Pump Stations S-9 and S-9A combined. Based on analyses included herein, that overall capacity can be distributed as:

- 2,100 cfs for normal drainage and flood protection needs on the C-11 West Canal (developed as a removal rate of 1-1/4" per day from the 40,000 acres of the basin that will remain tributary to S-503 and Structure S-381 following completion of the project);
- 175 cfs for return of seepage collected in the C-511 Canal to the C-11 Impoundment (provides a factor of safety of 5 applied to the estimated collection rate of 35 cfs);
- An allowance of 300 cfs for removal of increased seepage rates to the east resulting from both the C-11 Impoundment and the WCA-3A Seepage Management Area.

The adequacy of the above allowance of 300 cfs should be confirmed through additional, more detailed seepage analyses during final design of the project.

Additional modifications to the proposed operation of the project (as presented in the Feasibility Study) are suggested in the interest of further reducing the volume of water back pumped to WCA-3A at S-9 following completion of the project. Those operational modifications include:

- Maximizing discharge to the east through Structure S-13A during runoff events (within the available capacity of the C-11 Canal east of that point and Pump Station S-13) so as to reduce the total volume of inflow to the C-11 Impoundment;
- Drawing down the storage in the C-11 and C-9 impoundments during the wet season at the earliest practicable opportunity, within the available (non-damaging) capacity of the existing infrastructure east of the impoundments on both the C-9 and C-11 Canals;

- Modifying that part of the overall project operation associated with the transfer of water from the C-11 Impoundment to the C-9 Impoundment so as to maximize the effective use of the total available impoundment storage;
- During those times when both impoundments are full and there is a continuing need for removal of water from the C-11 Canal, employing the available storage capacity in the wetland mitigation areas adjacent to the impoundments, followed by employing available storage in the WCA-3A and WCA-3B Levee Seepage Management areas.

Maximum effectiveness in the final operational modification suggested above would result from the addition of a gated spillway in the C-11 Canal between S-9 and U.S. Highway 27, although that addition would not be required for any other purpose for those project elements to be completed prior to 2010. When other CERP components (such as the Central Lake Belt Storage Area and Canals C-500A and C-500B) are implemented, that structure would be required to maintain separation of basin runoff and water supply deliveries to the Everglades Protection Area.

The suggested operational modifications for further reducing backpumping at S-9 can be implemented without increasing the potential for adverse flood protection impacts resulting from the project. As implementation of those modifications would be expected to result in a lowering of the mean impoundment stages during the wet season, the suggested operational changes could instead be expected to provide additional assurance of no adverse impacts on flood protection. An additional South Florida Water Management Model (SFWMM) simulation is recommended to confirm the anticipated beneficial influence of the recommended adjustments on overall project operation.

Finally, given the staged implementation of the overall WCA-3A and WCA-3B Levee Seepage Management Project, in which certain features can be substantially delayed until subsequent separable CERP elements are instituted, additional suggestions are made to minimize required expenditures for the initial construction. That initial construction (those works to be completed before 2010) can be limited to those features necessary for the transfer of water from the C-11 Impoundment to the C-9 Impoundment. Additional adjustments to the project design are suggested to reduce the cost of required features. Most of those additional adjustments are made possible by implementation of the adjustments summarized above. Those additional suggestions include:

- Relocation of gated spillway S-504 and elimination of culvert S-504A at the C-11 Impoundment;
- Delaying enlargement of the C-9 Canal along the southern boundary of the C-9 Impoundment until such time as the North Lake Belt Storage Area (NLBSA) project may be implemented;
- Initially constructing S-510 for the significantly reduced capacity needed for operation of the C-9 Impoundment prior to implementation of the NLBSA project;
- Elimination of gated culvert S-502C;
- Delaying replacement of Structure S-30 until such time as the NLBSA project may be implemented;
- Initially constructing the C-502B Canal to the capacity necessary for planned transfer rates from the C-11 to C-9 Impoundment, delaying excavation for additional capacity (and the concrete-lined rectangular channel adjacent to the Holly Lakes Mobile Home Community) until such time as the NLBSA project may be implemented.

None of those additional suggestions would be expected to adversely impact flood protection in the C-11 West and C-9 basins.

The various adjustments to the project design and operation recommended herein would, if implemented, represent significant changes in project as it was presented in the Feasibility Study and originally simulated. It is recommended that additional long-term simulations be conducted to verify that those recommendations selected for implementation would perform as anticipated. Additional analyses will be required during the design phase to assure that the project as it is finally configured will not adversely impact flood protection in the C-9 and C-11 basins.

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Table of Contents

List of Figures

1. INTRODUCTION

The Broward County Water Preserve Areas project is comprised of three principal components:

- The C-11 Impoundment;
- The C-9 Impoundment;
- The WCA3A/3B Levee Seepage Management Projects.

Those components were recommended as a part of the *Central and Southern Florida Comprehensive Review Study Feasibility Report and Integrated EIS* in April of 1999 (the *Restudy*). In a related and complimentary effort, the Water Preserve Area Feasibility Study was initiated. The Water Preserve Area (WPA) study region included multiple Comprehensive Everglades Restoration Plan (CERP) components in the area east of the Water Conservation Areas in Palm Beach, Broward, and Dade Counties. The WPA is proposed to consist of an interconnected series of marshlands, impoundments, conveyance and aquifer recharge areas. The findings and recommendations of the Feasibility Study are presented in the October, 2001 *Central and Southern Florida Project, Water Preserve Areas; Draft Integrated Feasibility Report and Supplemental Environmental Impact Statement*. Descriptions of project components, structures, and planned operations presented in this report are taken from that document unless otherwise noted.

Section 601 of the Water Resources Development Act (WRDA) of 2000 authorized a framework and guide for modifications to the Central and Southern Florida (C&SF) Project to restore the South Florida ecosystem and to provide for other water-related needs of the region. That framework, based on the *Restudy*, is known as the Comprehensive Everglades Restoration Plan (CERP). The Broward County WPA Project, a separable element of CERP, includes buffer marsh areas, canals, levees, water control structures and above-ground improvements with a total storage capacity of approximately 6,000 acre-feet located in the western C-11 Canal basin and 6,600 acre-feet located in the western C-9 Canal basin in western Broward County. A general plan of the Broward WPA Project is presented in Figure 1.1.

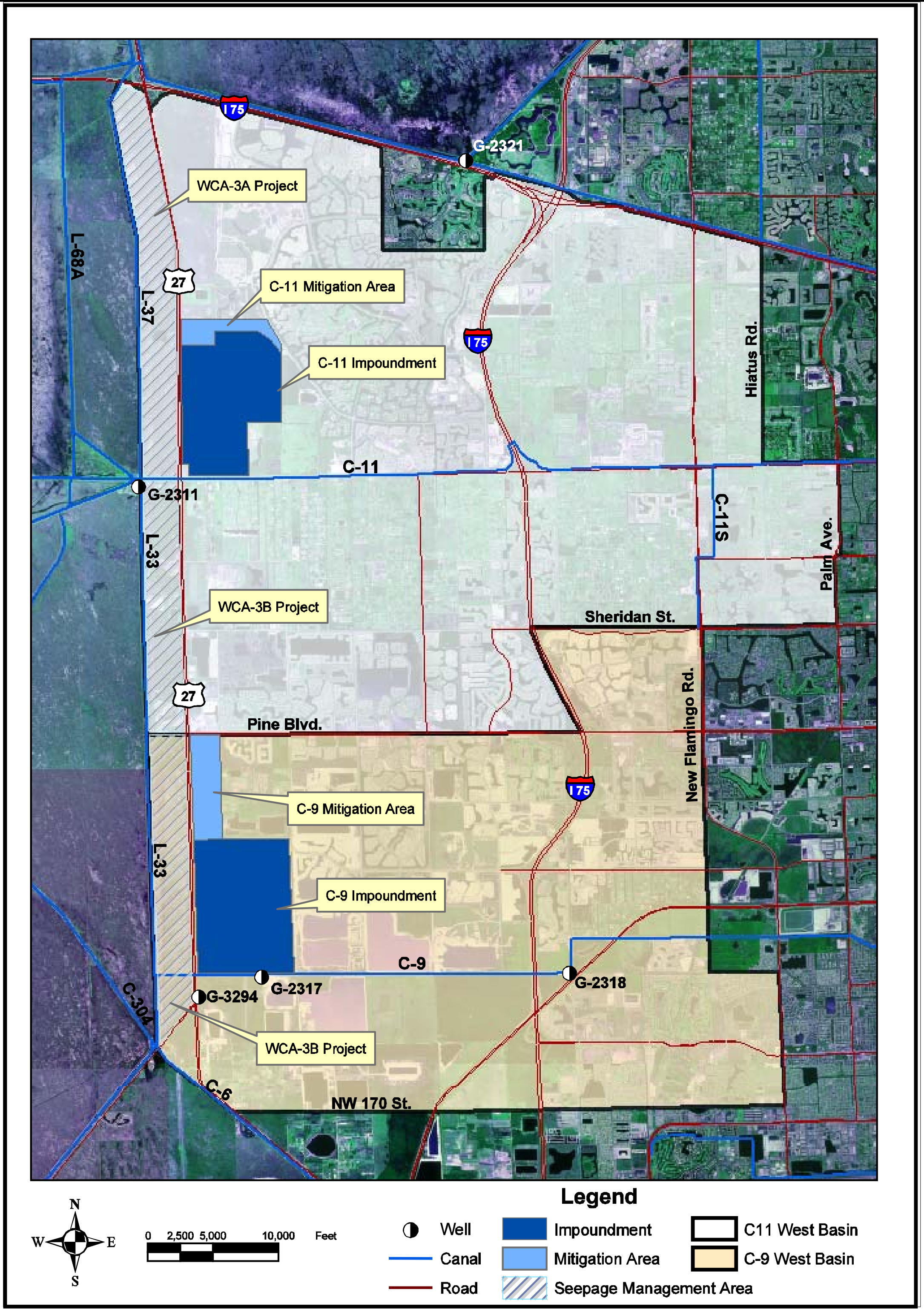

Figure 1.1 General Plan, Broward WPA Project

The WPA Project as a separable element of CERP is designed to direct runoff events from the western C-11 Canal drainage basin into the C-11 impoundment instead of pumping the untreated runoff into WCA-3A through the S-9 pump station. The purpose of the C-9 impoundment features is to pump storm events from the western C-9 drainage basin into the impoundment along with runoff transferred from the western C-11 basin. The impoundment pools are intended to assist in reducing seepage from the adjacent Water Conservation Areas (WCAs) 3A and 3B and the WCA-3A/3B Seepage Management areas; providing groundwater recharge; meeting the urban area water demands; and preventing saltwater intrusion in the surficial aquifer.

Programmatic regulations developed for CERP responsive to the “savings clause” of WRDA 2000 include requirements that the level of flood protection following implementation of CERP projects not be reduced from that existing in December 2000. The purpose of the analyses summarized in this report is to assess the extent to which the level of flood protection in the western C-11 and C-9 drainage basins may be impacted by implementation of the Broward County WPA components. Those analyses generally include:

- Review and analysis of historic stage/discharge relationships;
- Review of existing groundwater model data and results;
- Hydraulic analysis of canal water surface profiles with and without the CERP project;
- Review of the conceptual design and planned operations of the project(s) for potential impacts on flood protection;
- Development of recommendations for operational and/or structural changes to the current plan necessary to mitigate any adverse impacts on flood protection under future “with project” conditions;
- Identification of such further operational and/or structural changes to the current plan as might be appropriate to minimize stormwater pumping from the western C-11 drainage basin into WCA-3A.

The conduct of those analyses and preparation of this document was authorized by the South Florida Water Management District (SFWMD) through its issuance in May 2004 of Work Order No. C-C20104P-WO03 to Burns & McDonnell Engineering Co., Inc. Certain elements of the overall scope of work were prepared by Engineering and Applied Science, Inc. (EAS) of Tampa, Florida and Civil Services, Inc. (CSI) of Jacksonville, Florida under subcontract to Burns &

McDonnell. EAS prepared Parts 2 and 3 and Appendices B and C. CSI prepared Part 5 and Appendix D.

The analyses are specific to conditions anticipated to exist upon completion of the presently authorized Broward County WPA projects. Future separable elements of CERP, such as the North and Central Lake Belt Storage Areas, are not considered herein.

The "with project" condition considered herein does include certain features authorized under Section 528 of the Water Resources Development Act of 1996. Those features are included in the Western C-11 Water Quality Treatment Project, one of 34 "critical projects" authorized by WRDA 1996. The Western C-11 Water Quality Project includes the construction of Pumping Station S-9A and Structure S-381. Pumping Station S-9A was completed and placed into operation in 2002; Structure S-381 is presently under construction and nearing completion. For the purpose of the analyses presented herein, those features are considered as included in the system changes associated with the "with project" condition.

1.1. Existing Conditions

This section summarizes the structures in the western C-11 and C-9 drainage basins existing in December 2000, together with operational criteria then in effect. This information is directly relevant to the "without project" condition against which flood protection impacts associated with the Broward County WPA Project are to be evaluated.

1.1.1. C-11 West Basin

The C-11 West Basin covers an area of approximately 45,600 acres (71.2 square miles) in south-central Broward County generally bounded by the North New River Canal and Interstate Highway 75 on the north; Hiatus Road and Palm Avenue on the east; Pine Boulevard on the south; and WCA 3B and 3A on the west. Four primary canals exist in the basin, and include the C-11 West (that part of the C-11 Canal west of Structure S-13A) parallel and adjacent to Griffin Road; the C-11 South; that part of the L-33 Borrow Canal between Griffin Road and Pine Road; and the L-37 Borrow Canal. In addition, the C-11 South Canal extends southerly from the C-11 West Canal, generally following Flamingo Road, to the common boundary between the C-11 West and C-9 West basins at Sheridan Road. An overall map of the C-11 West Basin is presented in Figure 1.2.

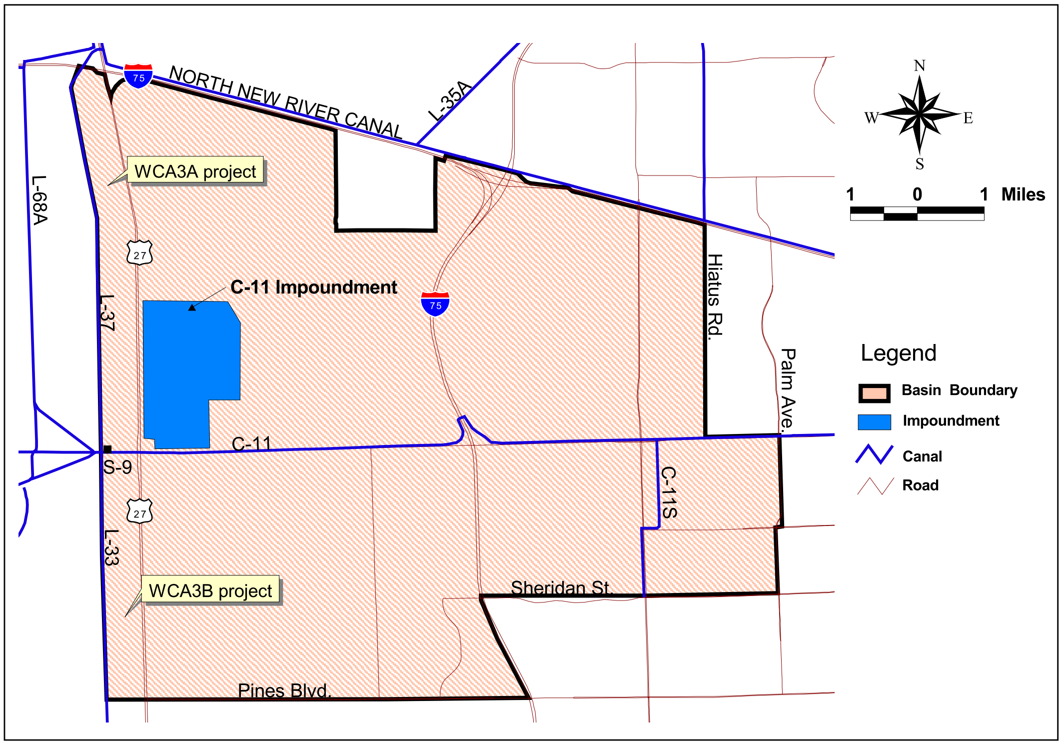

Figure 1.2 C-11 West Basin Boundary and Canals

In December 2000, principal water control structures in the C-11 West Basin included Structure S-13A; Pumping Station S-9; Culverts S-9XN and S-9XS; Culverts G-86N and G-86S; and Structure G-87.

Structure S-13A is located on the C-11 West Canal between Bright Road and Griffin Road roughly 4.5 miles west of State Road 7, just south of the intersection of Golden Shoe Road and Orange Drive. It consists of four 72-inch diameter corrugated metal pipe culverts, each 200 feet in length, controlled by manually operated sluice gates mounted on a steel frame head structure on each culvert. The current structure was constructed in 1988 in connection with a roadway widening project, replacing two 72-inch and two 54-inch diameter culverts having a design discharge capacity of 500 cfs. During flood conditions, this structure is operated to supplement S-9 pumping when capacity is available in the C-11 Canal east of S-13A. Discharges to the east at S-13A typically include irrigation supply to the C-11 basin east of the structure and seepage and minor flood runoff for gravity discharge at Pumping Station S-13. S-13 is a tidal control structure equipped with three 54-inch diameter flood control pumps and one gated spillway having a nominal capacity of approximately 500 cfs.

Pumping Station S-9 is located at the westerly end of the C-11 West Canal, roughly one-half mile west of U.S. Highway 27, at the junction of the C-11 West Canal with Levee L-37. The structure is intended to remove flood runoff from the C-11 West Basin, together with seepage from WCA-3A and WCA-3B collected in the L-33 and L-37 Borrow Canals. S-9 discharges to WCA-3A. It is equipped with three vertical-shaft axial flow pumps providing a design capacity of 2,880 cfs with headwater (C-11 Canal) stage of 4.0 feet NGVD and a tailwater stage (in WCA-3A) of 14.4 feet NGVD. In December 2000, operation of S-9 was to initiate when stages in the C-11 West Canal exceeded 4.0 ft. NGVD (measured at Structure S-13A), with the rate of pumping regulated to draw the C-11 West Canal down to elevation 1.0 ft. NGVD at the S-9 headwater. The maximum drawdown elevation in the C-11 West Canal at the S-9 headwater during pumping operations was established at elevation 0.0 ft. NGVD.

Culvert S-9XN is a double-barrel corrugated metal pipe culvert situated in the L-37 Borrow Canal immediately north of its confluence with the C-11 West Canal, and serves to discharge WCA-3A seepage collected in the L-37 Borrow Canal to the C-11 West Canal.

Culvert S-9XS is a double-barrel corrugated metal pipe culvert situated in the L-33 Borrow Canal immediately south of its confluence with the C-11 West Canal, and serves to discharge WCA-3B seepage collected in the L-33 Borrow Canal to the C-11 West Canal.

Culvert G-86N is a single-barrel corrugated metal pipe culvert located in the drainage ditch on the west side of U.S. Highway 27 north of the C-11 West Canal, and discharges to the C-11 West Canal. This structure acts in tandem with S-9XN to control water levels in the area between WCA-3A and U. S. Highway 27. The control stage in the G-86N headwater is 5.5 ft. NGVD.

Culvert G-86S is a single-barrel corrugated metal pipe culvert located in the drainage ditch on the west side of U.S. Highway 27 south of the C-11 West Canal, and discharges to the C-11 West Canal. This structure acts in tandem with S-9XS to control water levels in the area between WCA-3B and U. S. Highway 27. The control stage in the G-86S headwater is 5.5 ft. NGVD, measured at the West Hollywood Lakes development near the divide between the C-11 West and C-9 West basins.

Structure G-87 consists of a gate-controlled culvert at the south end of the C-11 South Canal, at the divide between the C-11 West and C-9 West basins, and is used to control inter-basin flows between the two basins. Normally closed, G-87 is opened only when stages are high in the C-9 West Basin and sufficiently low in the C-11 South and C-11 canals.

1.1.2. C-9 West Basin

The C-9 West Basin covers an area of approximately 31,400 acres (49 square miles) immediately south of the C-11 West Basin in south-central Broward County. The basin is generally bounded by Pines Boulevard and Sheridan Street on the north; New Flamingo Road on the east; N.W. 170th Street on the south; and WCA-3B on the west. Primary canals in the basin include the C-9 Canal and the L-33 Borrow Canal. An overall map of the C-9 West Basin is presented in Figure 1.3.

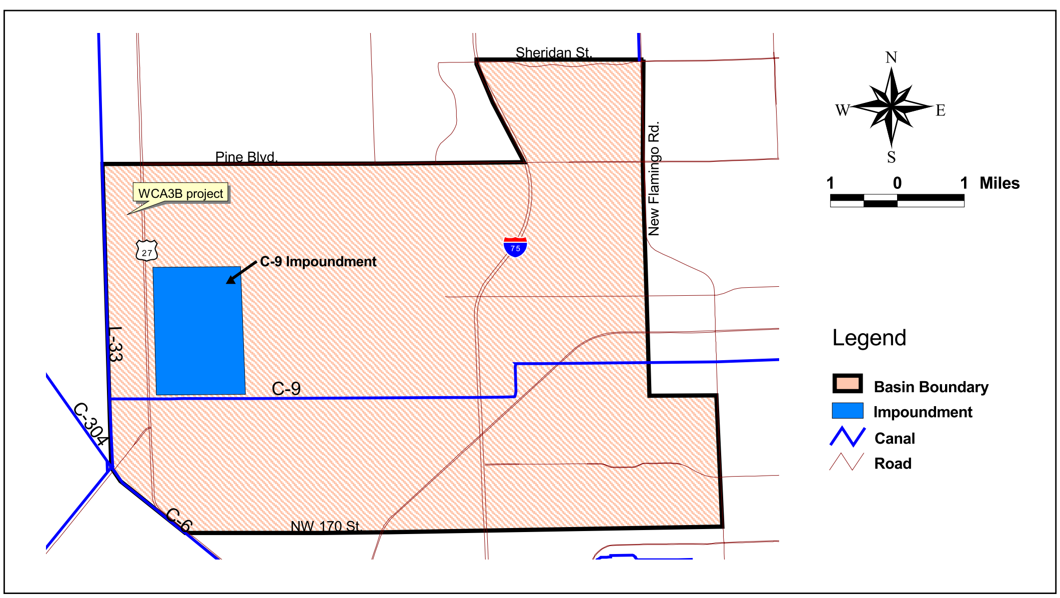

Figure 1.3 C-9 West Basin Boundary and Canals

In December 2000, the lone principal water control structure in the C-9 West Basin was Structure S-30. The C-9 West Canal is drained east into the C-9 East Basin; no control structure divides the two basins. Discharges from the entire C-9 Basin are directed east through tidal control structure S-29. Under favorable conditions, flood runoff from the C-9 West Basin can also be directed to the C-11 South Canal through G-87 as described above.

Structure S-29 is located in the C-9 Canal (Snake Creek) in North Miami Beach about 400 feet east of U.S. Highway 1. It is a reinforced concrete spillway equipped with four vertical lift gates; each gate opening is 22 feet wide and 15.5 feet high. S-29 is designed to pass 4,700 cfs under standard project flood conditions (headwater elevation west of the structure of 2.4 ft. NGVD, tailwater (tide) elevation east of the structure of 1.9 ft. NGVD). Under flood conditions, the gates are opened as necessary to prevent damaging stages in the C-9 Canal west of the structure. Otherwise, the structure is operated to maintain headwater stages between 1.5 and 2.5 feet NGVD, with an optimum C-9 Canal stage of 2.0 ft. NGVD.

Structure S-30 is located in the C-9 Canal at U.S. Highway 27. It consists of a gated, three barrel reinforced concrete pipe culvert. Each barrel is 84 inches in diameter and 288 feet in length at an invert elevation of -5.0 ft. NGVD. The current structure replaced the original S-30 (which was designed to pass 560 cfs with a headwater (westerly) elevation of 4.4 ft. NGVD and a tailwater elevation of 3.5 ft. NGVD) when U.S. Highway 27 was widened to four lanes. The purpose of this structure is to prevent excessive seepage losses from WCA-3A by permitting higher stages in the L-33 Borrow Canal west of U.S. Highway 27; it also supplies water from the L-33 Borrow Canal during dry periods to maintain stages in the C-9 Canal. The gates at S-30 are closed when releases from S-30 would aggravate downstream flood conditions (defined as the presence of a tailwater stage above 3.0 ft. NGVD). In the absence of a tailwater stage above 3.0 ft. NGVD, gates are opened as necessary when the headwater stage exceeds 6.0 ft. NGVD.

1.2. Western C-11 Water Quality Project

The Western C-11 Water Quality Project includes the construction of Pumping Station S-9A and Structure S-381. Pumping Station S-9A was completed and placed into operation in 2002; Structure S-381 is presently under construction and nearing completion. For the purpose of the analyses presented herein, those features are considered as included in the system changes associated with the "with project" condition. Structure descriptions and operations are taken from the May 2002 *Interim Water Control Plan for Pumping Station S-9A and Structure 381*, Jacksonville District, U.S. Army Corps of Engineers.

Structure S-381 is located in the C-11 West Canal approximately 4,700 feet east of the north bound lanes of U.S. Highway 27. It is an Obermyer gated spillway structure consisting of three 30 feet wide bays equipped with two gates per bay and one air bladder per gate. It was designed to pass 2,880 cfs with a headwater (east) elevation of 4.00 ft. NGVD and a tailwater (west) elevation of 3.97 ft. NGVD.

Pumping Station S-9A is located in the C-11 West Canal north of and adjacent to Pumping Station S-9, drawing from the S-9 headwater pool. It provides a nominal capacity of 500 cfs, distributed between two 175 cfs and two 75 cfs pumps, with headwater at 1.0 ft. NGVD and tailwater (WCA-3A) at 14.4 ft. NGVD.

S-9A is a seepage control pump station that replaces the existing S-9 role of pumping seepage from WCA-3A and WCA-3B collected in the L-37 and L-33 Borrow Canals, respectively, as well as seepage collected in the drainage ditch along the west side of U.S. Highway 27. Pumping seepage is the primary role of S-9A; however, S-9 can also perform this function if needed for larger forecasted storm events.

S-381 will act as a divide structure to separate seepage from the water conservation areas from the urban portions of the C-11 West Basin. The structure's gates will normally remain closed, which will allow S-9A to pump seepage back into WCA-3A at essentially the same rate it enters the 7,900 feet of C-11 West Canal between S-9 and S-381.

Upon completion of the C-11 West Water Quality Project, S-9A will pump seepage from the L-33 and L-27 Borrow Canals, as well as the drainage ditch west of U.S. Highway 27, into WCA-3A. The pumps will be operated to maintain an optimum headwater stage between 3.2 and 3.5 ft. NGVD in the canal reach between S-9 and S-381, with the gates closed at S-381. During runoff conditions, as headwater (east) elevations at S-381 rise above 3.5 ft. NGVD, the S-381 gates will be opened and the pumping at S-9A will be increased in order to draw down the canal stage. Under some conditions, the S-381 gates may be opened in advance of forecasted storm events to initiate a canal draw down, providing both increased canal conveyance capacity and additional canal storage for flood protection. To the extent that S-9A does not provide sufficient capacity to draw down the canal stage, Pumping Station S-9 may also be operated, although the combined discharge at S-9 and S-9A is limited to 2,880 cfs by the terms of the Florida Department of Environmental Protection (FDEP) operating permit for the project. The existing flood control regulation also requires that flood control pumping at S-9 and S-9A be conducted whenever canal stages at the S-13A headwater reaches 4.0 ft. NGVD. The normal draw down elevation at both S-9 and S-9A is limited to 1.0 ft. NGVD, which is also the maximum draw down elevation at S-9A. The maximum draw down elevation at S-9 is 0.0 ft. NGVD.

1.3. C-11 Impoundment

The following descriptive information on the presently formulated design of the C-11 Impoundment and associated features is taken from the October, 2001 *Central and Southern Florida Project, Water Preserve Areas; Draft Integrated Feasibility Report and Supplemental Environmental Impact Statement*, prepared by the Jacksonville District, U.S. Army Corps of Engineers and the South Florida Water Management District.

The C-11 Impoundment consists of a four-foot deep aboveground impoundment with two pump stations, one gated ogee spillway, one gated culvert, one ungated culvert, two fixed weir structures, an emergency overflow spillway, and perimeter seepage control canals. The design also includes a wetland buffer marsh area along the northern boundary of the Impoundment. The project is located immediately east of U.S. Highway 27 and north of the C-11 West Canal in Broward County; the northern boundary of the project is approximately 3.5 miles south of the Interstate 75/U.S. Highway 27 interchange.

The total impoundment footprint is approximately 1,850 acres with a storage area of approximately 1,490 acres and 205 acres in the wetland buffer marsh area. At normal pool (depth of four feet, elevation 10.0 ft. NGVD), the impoundment will store approximately 5,960 acre-feet; additional storage is available in the wetland buffer marsh area. An overall plan of the C-11 Impoundment and associated features is presented in Figure 1.4.

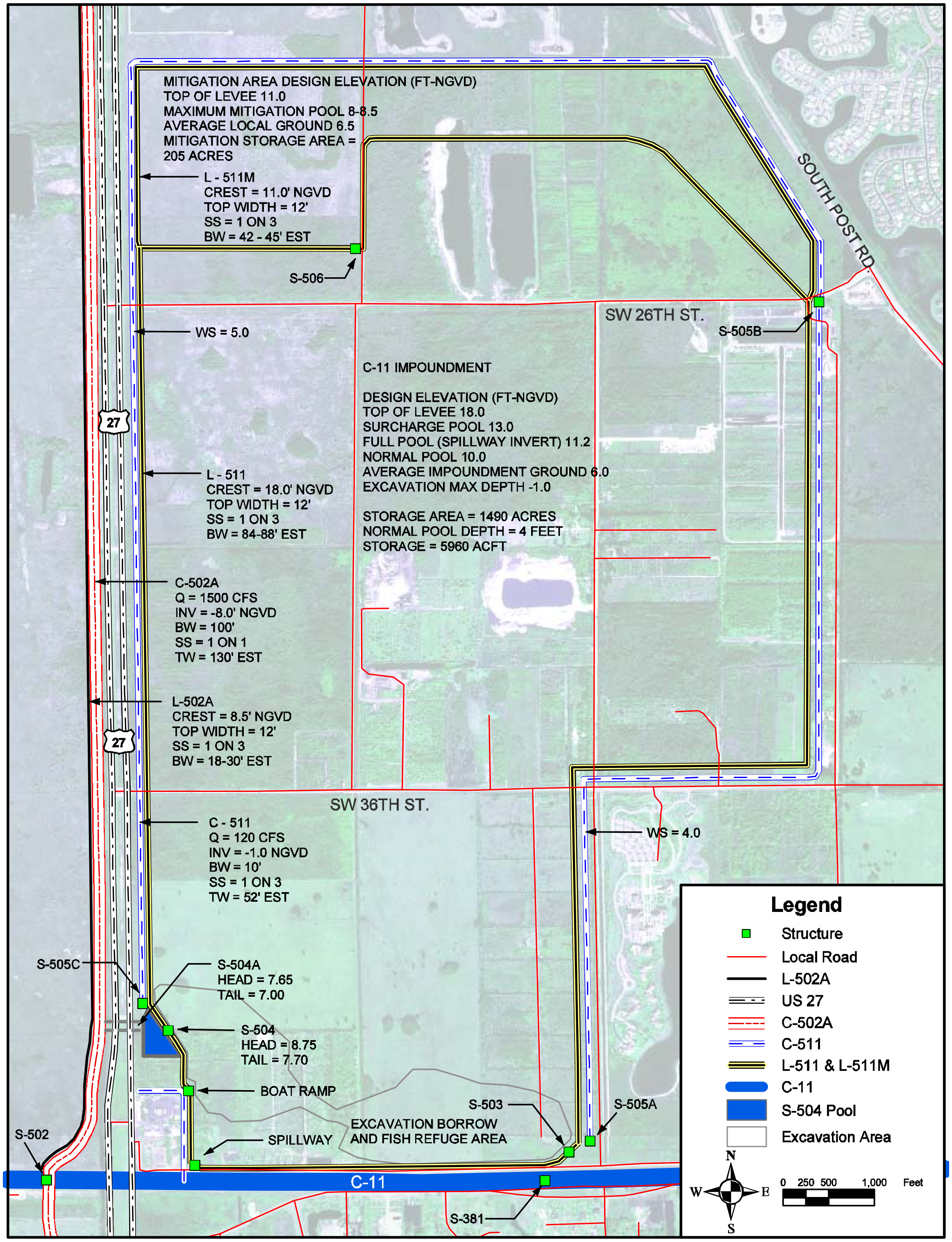

Figure 1.4 General Plan of C-11 Impoundment

The design storage of the impoundment ranges from elevation 6.0 to 10.0 ft. NGVD. The invert of the emergency overflow spillway is 11.2 ft. NGVD; full surcharge pool is 13.0 ft. NGVD.

1.3.1. Pumping Stations

Two new pumping stations (S-503 and S-505C) are features of the C-11 Impoundment.

S-503 will be the inflow pump station to the C-11 Impoundment, and is located on the C-11 West Canal at the southeast corner of the impoundment. It is designed to capture available storm runoff in the C-11 West Basin upstream (east) of Structure S-381 and to backpump seepage captured in the impoundments perimeter seepage collection canals that is returned to the C-11 West Canal. S-503 provides a nominal capacity of 2,575 cfs, with 75 cfs established as the seepage return capacity, and the balance (2,500 cfs) available for removal of C-11 West Basin runoff. It will begin pumping when the stage in the C-11 West Canal reaches 4.10 ft. NGVD. Normal drawdown elevations in the C-11 West Canal will be between 2.0 and 3.0 ft. NGVD; the minimum drawdown pumping elevation is 0.0 ft. NGVD. The maximum tailwater (impoundment) stage for design of the pumps is 12.0 ft. NGVD (two feet above normal pool and 0.8 feet above the emergency spillway crest elevation). While S-503 can pump against impoundment stages above 10.0 ft. NGVD, storage above that elevation is reserved for direct rainfall on the impoundment.

S-505C will be a seepage control pump station located near the southwest corner of the impoundment. It is designed to backpump seepage captured in the impoundment's perimeter seepage collection canals along the west, north, and a part of the east perimeter of the impoundment. S-505C will provide a nominal pumping capacity of 120 cfs. It will begin pumping when the stage in the seepage collection canal reaches 5.10 ft. NGVD. Normal drawdown elevations in the seepage collection canal will range from 3.5 to 5.0 ft. NGVD; the minimum drawdown pumping elevation is 3.5 ft. NGVD.

1.3.2. S-504 Gated Spillway

S-504 will consist of a three-bay gated spillway located near the southwest corner of the Impoundment. Its function is to draw down pool stages and transfer water (via the WCA-3A and WCA-3B Seepage Management Project) to the C-9 Impoundment (and, after its completion, the North Lake Belt Storage Area project). S-504 is designed to provide a capacity of 2,500 cfs (equal to the runoff removal capacity of S-503) with headwater (Impoundment pool) at elevation 8.75 ft. NGVD and tailwater at 7.75 ft. NGVD. Following completion of the C-11 and C-9 impoundments, but before completion of the North Lake Belt Storage Area, discharges through S-504 will be limited to 1,000 cfs (controlled by the capacity of new inflow pumps serving the C-9 Impoundment). The ability of this structure to draw down pool stages below 7.0 ft. NGVD will be limited by the control stage in the WCA-3A Seepage Management project canal intended to transfer flow to the C-9 Impoundment.

1.3.3. Culverts and Weirs

The C-11 Impoundment project includes two fixed weirs (S-505A and S-505B) for stage maintenance in the perimeter seepage collection canals, and two culverts (S-506 and S-504A); S-506 is a gated culvert, while S-504A is ungated.

S-505A will consist of an ungated fixed crest weir structure located near the southeast corner of the Impoundment in the seepage collection canal. It is designed to pass a discharge of 150 cfs with a hydraulic head of 0.25 feet. The weir crest is 100 feet in length at elevation 3.5 ft. NGVD. The function of S-505A is to prevent excessive drawdown of the seepage collection canal when S-503 is pumping.

S-505B will consist of an ungated fixed crest combination or notched weir structure located in the seepage collection canal at the northeast corner of the Impoundment, adjacent to SW 26th Street. The total length of the weir crest is 70 feet, with 38 feet of that length at elevation 4.90 ft. NGVD, and 32 feet at elevation 4.65 ft. NGVD. The purpose of this structure is to maintain a stage in the seepage collection canal upstream (north and west) of the structure at approximate elevation 5.0 ft. NGVD to benefit the wetland buffer area marsh hydroperiods and prevent deep soil dry-outs in the dry season. The total design flow for the structure is 75 cfs with a hydraulic head of 0.5 feet.

S-506 will be a two-barrel, gated culvert structure located at the northern boundary of the Impoundment in the levee that separates the Impoundment from the wetland buffer marsh area to the north. It is designed to pass 100 cfs with a hydraulic head of 1.0 feet. The function of S-506 is to transfer water from the Impoundment to the wetland buffer marsh area. S-506 tailwater (wetland buffer marsh stage) will normally be held at 7.5 ft. NGVD;

the maximum mitigation pool elevation in the marsh is 8.5 ft. NGVD (two feet above the estimated average ground surface elevation in the marsh). Under conditional circumstances the marsh can be used to store up to four feet of water (to elevation 10.5 ft. NGVD) for periods of time sufficiently short as to not endanger the marsh vegetation. No structure will exist to effect drawdown of the marsh area. The water surface elevation in the marsh will be dependent upon seepage and evapotranspiration losses and gains from direct rainfall and Impoundment releases through S-506.

S-504A will consist of ungated culverts beneath U.S. Highway 27 near the southwest corner of the Impoundment. The function of this structure is simply to pass discharges from S-504A to the WCA-3A Seepage Management Project for transfer to the C-9 Impoundment (and, after its completion, the North Lake Belt Storage Area). The design capacity of S-504A is 2,500 cfs with headwater (east side of U.S. Highway 27) at elevation 7.65 ft. NGVD, and tailwater elevation (west side of U.S. Highway 27) at elevation 7.00 ft. NGVD.

1.3.4. Overall Operation

The overall intent in the design is that all storm runoff from the C-11 West Basin be directed to the C-11 Impoundment through Pumping Station S-503. To facilitate that intent, Structure S-381 (a feature of the C-11 West Water Quality Project discussed earlier in this Part 1), located just west of S-503, will normally remain closed, allowing Pump Station S-9A to backpump seepage into WCA-3A at the same rate that it enters the C-11 West Canal between Pump Station S-9 and S-381.

Once the depth of storage in the C-11 Impoundment approaches the normal pool elevation (10.0 ft. NGVD), Structure S-504 would discharge from the Impoundment, and those discharges would be transferred to the C-9 Impoundment for storage. Hydrologic simulations conducted during development of the Feasibility Study indicated that there will be occasions where flooding events and conditions are such that no storage is available in the C-11 and C-9 Impoundments. Under those circumstances, the existing flood control system will be used to maintain flood protection in the C-11 West Basin.

When no storage remains in the two impoundments and there is a continuing need for removal of storm runoff from the C-11 West Canal, the operation of Pump Station S-503 would cease; the gates at Structure S-381 would be opened; and existing Pump Station S-9 would be used to remove storm runoff from the C-11 West Basin.

1.4. C-9 Impoundment

The following descriptive information on the presently formulated design of the C-9 Impoundment and associated features is taken from the October, 2001 *Central and Southern Florida Project, Water Preserve Areas; Draft Integrated Feasibility Report and Supplemental Environmental Impact Statement*, prepared by the Jacksonville District, U.S. Army Corps of Engineers and the South Florida Water Management District.

The C-9 Impoundment consists of a four-foot deep aboveground impoundment with two pump stations, one gated ogee spillway, two gated culvert structures, one fixed weir structure, an emergency overflow spillway, and perimeter seepage control canals. The design also includes a mitigated wetland buffer marsh area along the northern boundary of the Impoundment. The project is located immediately east of U.S. Highway 27 and north of the C-9 Canal in Broward County; the northern boundary of the project is approximately 10.7 miles south of the Interstate 75/U.S. Highway 27 interchange. The southern boundary of the project is approximately 0.4 miles north of the intersection of Krome Avenue and U.S. Highway 27.

The total impoundment footprint is approximately 1,800 acres with a storage area of approximately 1,650 acres; an additional 360 acres are included in the mitigated wetland buffer marsh area. At normal pool (depth of four feet, elevation 8.5 ft. NGVD), the impoundment will store approximately 6,650 acre-feet. The design storage of the impoundment ranges from elevation 4.5 to 8.5 ft. NGVD. The invert of the emergency overflow spillway is 9.7 ft. NGVD; full surcharge pool is 11.5 ft. NGVD.

An overall plan of the C-9 Impoundment and associated features is presented in Figure 1.5.

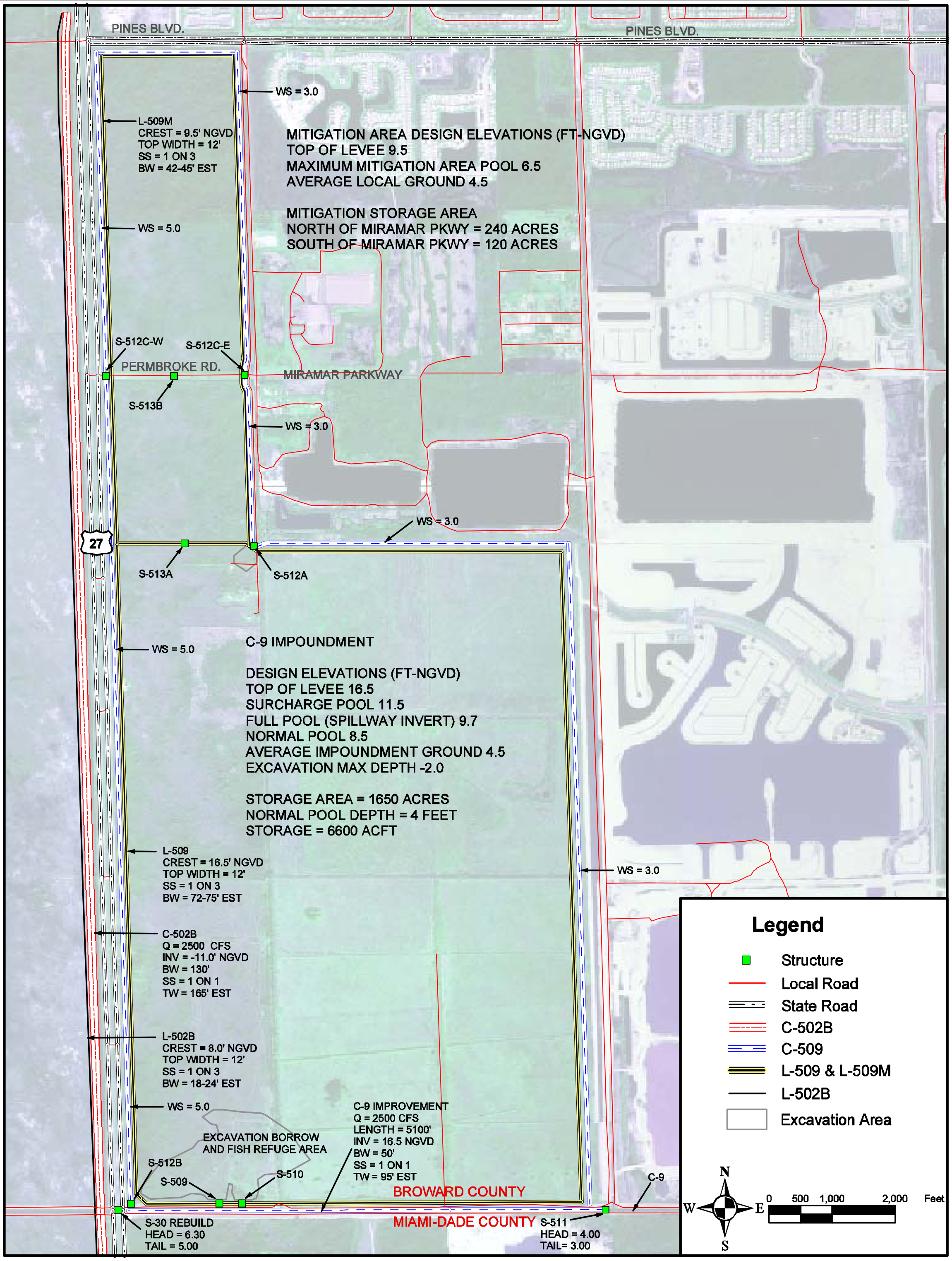

Figure 1.5 General Plan of C-9 Impoundment

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1.4.1. Pumping Stations

Two new pumping stations (S-503 and S-505C) are features of the C-11 Impoundment.

S-509 will be the inflow pumping station for the C-9 Impoundment, and is located on the southern boundary of the Impoundment, drawing from the C-9 Canal. It will provide a total pumping capacity of 1,075 cfs; 75 cfs is provided for the return of accumulated seepage to the Impoundment. The remaining 1,000 cfs capacity can be used either to capture available storm runoff in the C-9 West Basin or to lift discharges from the C-11 West Basin (released from the C-11 Impoundment) to the C-9 Impoundment. The following discussion of the intended operation of S-509 is specific to the period following completion of the C-11 and C-9 impoundments, but before completion of the North Lake Belt Storage Area.

S-509 would start pumping when stages in the C-9 Canal reach 3.5 ft. NGVD; normal drawdown elevations in the C-9 Canal would range from 2.0 to 3.0 ft. NGVD. The minimum draw down pumping elevation in the C-9 Canal would be 1.0 ft. NGVD. The maximum design tailwater (C-9 Impoundment pool) elevation during pump operations is established at 10.50 ft. NGVD, although the normal pool (maximum design storage) elevation in the Impoundment is 8.5 ft. NGVD.

S-512A will be a seepage control pump station with a total capacity of 225 cfs, located on the northern boundary of the C-9 Impoundment where the eastern boundary of the mitigated wetland buffer marsh area abuts to the northern levee of the Impoundment. Its function is to backpump seepage gathered in the perimeter seepage collection canals along the Impoundment's northern and eastern boundaries, as well as from the northern and eastern boundaries of the mitigation area, to the Impoundment. Of the total capacity of 225 cfs, 150 cfs is intended for control of stages in the seepage collection canals adjacent to the Impoundment. The remaining 75 cfs is intended for control of stages in the seepage collection canals adjacent to the mitigated wetland buffer marsh area. S-512A would start pumping when the stage in the seepage collection canal reaches 3.10 ft. NGVD; normal drawdown pumping elevations in the seepage collection canal would range from 2.5 to 3.0 ft. NGVD. The minimum drawdown pumping elevation would be 2.0 ft. NGVD.

1.4.2. Gated Spillway S-510

S-510 will consist of a two-bay gated spillway located on the southern boundary of the C-9 Impoundment, and will discharge from the Impoundment to the C-9 Canal. The design capacity of S-510 is 1,000 cfs with a headwater elevation (C-9 Impoundment) of 6.20 ft. NGVD and a tailwater elevation (C-9 Canal) of 5.70 ft. NGVD. S-510 is intended to control releases from the Impoundment for the combined purposes of pool drawdown and maintenance of optimal canal stages during dry periods. The capacity of S-510 is established in anticipation of the future development of the North Lake Belt Storage Area (NLBSA). Prior to development of the NLBSA, water supply releases from the Impoundment for maintenance of stages in the C-9 Canal are expected to be limited to approximately 150 cfs.

1.4.3. Culverts and Weirs

The C-9 Impoundment project includes two gated culverts (S-511 and S-513A) and one fixed crest weir (S-512B). Additional structures associated with control of stages in the mitigated wetland buffer marsh (S-513B, S-512C-E and S-512C-W) do not directly impact the design or operation of the Impoundment, and are not further discussed herein.

S-511 will be a two-barrel gated culvert located in the C-9 Canal approximately 200 feet east of the eastern boundary of the C-9 Impoundment. The design capacity of this structure is 500 cfs with a head differential of 0.90 feet. The design headwater elevation of S-511 (west side) is 4.40 ft. NGVD; the design tailwater elevation is 3.50 ft. NGVD. S-511 will function as a canal divide, with the gate settings dependent upon operational mode:

- When releases are being made from the C-9 Impoundment through S-510, the gates may be opened fully or partially to move water to the east, dependent upon stages in the C-9 Canal east of the structure. This operational mode would typically be for water supply releases to the C-9 Basin.
- When flows are being transferred from the C-11 Impoundment to the C-9 Impoundment, the gates would normally be closed.

- During runoff events, the gates may be adjusted to openings that would benefit pumping of runoff from the C-9 Basin or the direct discharge to the east of flows transferred from the C-11 Impoundment.

S-513A will be a two-barrel gated culvert structure located on the northern boundary of the Impoundment in the levee that separates the Impoundment from the mitigated wetland buffer marsh area to the north. It is designed to provide a capacity of 100 cfs with a head differential of 0.65 feet. The function of this structure is to release water from the Impoundment into the mitigated wetland area (when water is available) to enhance hydroperiods.

S-512B consists of two independent structures, an ungated fixed crest combination or notched weir and a two-barreled culvert structure with flap gates. It is located in the Impoundment's western seepage collection canal at the southwest corner of the Impoundment, near the confluence of the seepage collection canal with the C-9 Canal. The structure is intended to maintain an optimal stage of approximately 5.0 ft. NGVD in the seepage collection canal. The total length of the fixed crest weir is 95 feet; 86 feet of the crest length is at elevation 4.65 ft. NGVD, with the balance at elevation 4.80 ft. NGVD. The design discharge across the weir (from the seepage collection canal to the C-9 Canal) is 125 cfs with a hydraulic head of 0.60 feet. The flap gate structure allows flow from the seepage collection canal into the C-9 Canal. The flap gates prevent discharge from the C-9 Canal to the seepage collection canal (a condition likely to occur only when water supply deliveries are being made from the C-9 Impoundment to the C-6 Canal via the WCA-3A and WCA-3B Seepage Management Project.

1.4.4. Overall Operation

This discussion of the overall operation of the C-9 Impoundment is limited to regional hydrologic conditions expected to prevail in 2010 (e.g., the North Lake Belt Storage Area is not yet complete), and may be considered as applicable to a possible condition in which the North Lake Belt Storage Area is not constructed.

As stated in the Feasibility Study, the purpose of the C-9 Impoundment is two-fold:

- To pump excess storm runoff from the C-9 West Basin into the Impoundment and reduce the loss of runoff to tide;

- Impound flows released from the C-11 Impoundment in order to reduce (prevent if possible) the discharge of untreated water to WCA-3A at Pumping Station S-9.

Water stored in the C-9 Impoundment is meant to assist in meeting water supply demands and prevention of saltwater intrusion.

Inflow pumping station S-509 would operate (introduce water to the Impoundment) under either of the following conditions:

- When releases are being made from the C-11 Impoundment for transfer to the C-9 Impoundment;
- To pump a targeted amount of C-9 West Basin runoff to the Impoundment (requires the concurrent opening of gates at S-511).

The Feasibility Study is silent on the targeted amount of C-9 West Basin runoff to be delivered to the Impoundment, or on operating rules associated with that operation.

1.5. WCA-3A and WCA-3B Seepage Management Project

The following descriptive information on the presently formulated design of the WCA-3A and WCA-3B Seepage Management Project and associated features is taken from the Feasibility Study. This discussion is limited to those features of the overall project directly impacting the aggregate operations of the C-11 and C-9 Impoundments (and the existing flood protection works of the Central & Southern Florida Project, such as Pumping Station S-9) in 2010.

The overall project is designed in anticipation of the eventual construction of the following components of CERP, none of which are expected to be in place in 2010 (the focus of this analysis):

- North Lake Belt Storage Area (NLBSA);
- Central Lake Belt Storage Area (CLBSA);
- Diversion of excess waters from WCA-2B and WCA-3 to the Northeast Shark River Slough (NESRS) and CLBSA;
- Rerouting of Miami Canal and Miami River water supply deliveries to the South Dade Conveyance System (SDCS) to the North New River Canal, and decompartmentalization of WCA-3.

The following features of the WCA-3A and WCA-3B Seepage Management Project are considered directly applicable to this analysis, and are further discussed herein:

- The C-502A and C-502B Canals, particularly in that reach extending from Structure S-504A at the C-11 Impoundment south to the C-9 Canal;
- Gated spillway Structure S-508;
- Siphon structure S-502;
- Gated culvert structure S-502C;
- Rebuild of S-30.

C-502A and C-502B Canals will be constructed along the west side of U.S. Highway 27 north and south, respectively, of the C-11 Canal. They will replace the existing roadside drainage ditch adjacent to U.S. Highway 27. Two principal uses for these new canals are outlined in the Feasibility Study:

- Dry-season conveyance of water supply deliveries from Lake Okeechobee (1,500 cfs nominal delivery capacity);
- Transfer of waters released from the C-11 Impoundment to the C-9 Impoundment (through S-30).

The design conveyance capacity of the canals for the transfer of flow from the C-11 Impoundment to the C-9 Impoundment is 2,500 cfs. However, until such time as the North Lake Belt Storage Area (and other future CERP components) come on line, the design rate of transfer between the two impoundments is limited to 1,000 cfs. Stages in both canals are intended to be maintained by new Structure S-508.

S-508 will be a three-bay gated spillway constructed in the (new) C-502B Canal just west of U.S. Highway 27 at Krome Avenue. It is intended to, acting in combination with S-30, control stages in the C-502A and C-502B canals and direct flows from those canals selectively to the C-9 Impoundment and the NLBSA, or southerly to the C-6, C4/2, and SDCS. It is designed to pass 2,000 cfs with a headwater (north) elevation of 6.00 ft. NGVD and a tailwater (south) elevation of 5.75 ft. NGVD. It is not presently evident that this structure will be constructed in advance of the future CERP components described earlier.

S-502 will consist of an inverted siphon in the C-11 Canal immediately west of U.S. Highway 27. Its function is to carry C-11 flows beneath the new C-502A Canal (and a new C-500A Canal not otherwise discussed herein) to Pumping Station S-9. Those flows would typically occur only when no storage is available in the C-11 and C-9 impoundments, and overall operation of the project reverts to the existing flood control project (e.g., Pumping Station S-503 shut down, and Structure S-381 opened). In addition, S-502 will allow the transfer of seepage collected in C-502A and C-502B to Pumping Station S-9A during normal operation of the project. This structure is designed to pass the full capacity of S-9 (2,880 cfs) with a head loss of one foot (headwater elevation east of the structure at 3.00 ft. NGVD, tailwater elevation of 2.00 ft. NGVD). The calculated head loss for that discharge reported in the Feasibility Study is 0.87 feet. The structure is ungated, and is approximately 500 feet in length. The inverted barrel (minimum structure section) consists of two 18' wide by 12' high reinforced concrete box culverts constructed at an invert elevation varying from -25.0 to -25.5 ft. NGVD.

S-502C will consist of a two-barrel gated culvert located on the new C-502A Canal immediately north of S-502. Its function is to make dry-season water supply deliveries to the C-11 Canal, with the source of those deliveries being either Lake Okeechobee or releases from the C-11 Impoundment. It is designed to pass 300 cfs with a head differential of 1.4 feet.

S-30 is an existing structure described earlier in this Part 1. It is intended to be replaced as one feature of the WCA-3A and WCA-3B Seepage Management Project. The new structure will consist of a two-bay gated spillway designed to pass 2,500 cfs with a headwater elevation (in the new C-502B Canal) of 6.0 ft. NGVD and a tailwater elevation (in the C-9 Canal east of U.S. Highway 27) of 4.0 ft. NGVD. S-30 is intended to control the conveyance of flows diverted from the C-11 Impoundment to the C-9 Impoundment and (eventually) the North Lake Belt Storage Area. It may also be used to control Lake Okeechobee water supply deliveries to the C-9 Basin, and can under certain conditions be operated in reverse flow conditions to pass water supply releases from the C-9 Impoundment and NLBSA directed to the south.

1.6. Importance of Impoundment Draw Down in 2010 Operations

The modeling performed during development of the Feasibility Study determined that there will be occasions during which flooding events and conditions are such that no storage will be available in the C-11 and C-9 Impoundments. Under those circumstances, the existing flood control system will be used to maintain flood protection levels in the C-11 West Basin. This would require the S-381 gates to open and the S-9 and S-9A pumping stations be operated, with resultant discharge to WCA-3A.

As noted in the Feasibility Study, a key factor in minimizing that occurrence (and the potential negative impacts of backpumping to WCA-3A) is the ability to regain storage in the C-11 and C-9 Impoundments. Impoundment drawdown capability is noted to be critical prior to the construction of the North Lake Belt Storage Area. It will be necessary at times to convey excess water to tide when canal stages permit such conveyance without negatively impacting existing levels of flood protection.

TABLE OF CONTENTS

List of Tables

List of Figures

2. MODEL OUTPUT EVALUATION, REVIEW AND ANALYSIS

2.1. Objectives

Basinwide flood protection is one of the purposes of all Comprehensive Everglades Restoration Project (CERP) including the Broward County Water Preserve Area (BCWPA) CERP project. Adequate impoundment and seepage management system design is critical to flood protection. The main objective of Task 1 is to analyze the impact of the proposed reservoir and other CERP components on groundwater levels, which in turn may affect in flooding in the C-11 West and C-9 West basins and to evaluate the proposed impoundment seepage management system design. This analysis is to be performed for the worst-case scenarios of highest wet period rainfall, full impoundment water levels, and high groundwater levels to provide the highest level of flood protection.

2.2. Scope of Work

This report includes the scope of work outlined for Task 1 in the project work order. The following work was completed as part of this task.

- Task 1.0 – Compare South Florida Water Management Model (SFWMM) generated boundary conditions
- Task 1.1 – Delineate areas affected by increased levee seepage caused by project
- Task 1.2 – Determine pre-storm seepage into C11 and C9 canals under wet conditions
- Task 1.3 – Use the change in water tables to determine increase in storm runoff during design storm events
- Task 1.4 – Establish confidence bounds on increased inflows into C11 and C9 canals
- Task 1.5 – Define impoundment storage needed to be reserved for containing increases in inflows caused by the project

2.3. Methodology

To meet the objectives and scope of work outlined above, the following methodology was implemented. The Broward County groundwater model (referred to as BC MODFLOW in this report), which includes the C-11 West and C-9 West basins was used to simulate the surficial aquifer groundwater level in the Year 1995 base condition and Year 2010 with WPA project (hereinafter referred to as pre- and post- conditions, respectively). SFWMM simulated canal stages were used as the canal stage boundary conditions in the BC MODFLOW. Both these models were constructed by the South Florida Water

Management District (SFWMD) and the model input and output files were made available for this study. A comparison of the SFWMM computed canal stages for different base periods was made to identify any changes in the canal stages for use as robust estimate in the BC MODFLOW. Using the BC MODFLOW groundwater stage output for the pre and post CERP conditions, the area of increased groundwater stage was delineated and the corresponding area was designated as the impacted or affected area. The impoundment seepage and the capture of seepage by the seepage canals were estimated using the SEEP2D model and compared with the proposed seepage management system design. Pre-storm seepage flows for the basins for runoff analysis were derived from the BC MODFLOW results. The effect of groundwater stage increase on soil storage was estimated which in turn was used to develop runoff hydrographs using the Santa Barbara Unit Hydrograph (SBUH) procedure for storm events of 10-, 25-, and 100-year frequencies with and without seepage flows. A sensitivity analysis of the increased runoff and the required and available impoundment storage for flood protection were provided for the three storm events. The above analysis was performed separately for the C-11 and C-9 impoundments and their basins for the worst-case scenarios of highest wet period rainfall, full impoundment water level and high groundwater levels to provide the highest level of flood protection.

2.4. Source of Data

Available information and data collected and compiled during preparation of this report are listed below:

GIS Data:

- Drainage Basin Map (SFWMD)
- Land Use Map (SFWMD)
- Soil Series Map (SFWMD)

Reports:

- The electronic copies of the October 2001 Water Preserve Areas (WPA) Feasibility Study Report, including all appendices thereto
- SFWMM version 3.5 documentation report
- Broward County 2001 Groundwater Flow model technical report

- Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55), Soil Conservation Service (SCS), 2nd Edition, June 1986
- Environmental Resource Permit Information Manual Volume IV, SFWMD, 2000
- adICPR User's Manual, Version 2.0, September 1995

Modeling Data:

- Broward County MODFLOW model (BC MODFLOW) source code, input and output for 8-year simulation
- SFWMM version 3.5 output data for 31-year simulation

2.5. Study Area Description

The C-11 West basin and the C-9 West basin shown in Figure 2.1 are the areas of interest for the flood protection analysis. A brief description of these two basins and the Broward County Water Preserve Area (BCWPA) CERP Projects is stated below.

The C-11 Basin is structurally divided into two subbasins; C-11 West and C-11 East (Burns, 1982). The C-11 impoundment is located in the C-11 West basin and therefore the C-11West basin is analyzed. Although there is no structure dividing the basin into two subbasins, a surfacial ridge along Flamingo Road divides the basin into C-9 West and C-9 East basins (Burns, 1982). The C-9 West basin retains much of the runoff during normal runoff events due to the depressive topography. For higher rainfall events, the runoff from the C-9 West basin flows east to the C-9 East basin. However, The C-9 impoundment is located in the C-9 West Basin and is designed using the data from the C-9 West basin. The C-9 West Basin is alternatively referred to as the C-9 basin in the WPA Feasibility Study Report. This report refers the C-9 basin as the C-9 West basin because groundwater and flooding impacts are limited to the C-9 West Basin.

- **C-11 West Basin**

The C-11 West basin, as shown in Figure 2.2, encompasses a drainage area of approximately 45,600 acres (71.2 square miles) and is located in south central Broward County. The basin extends from I-75 to the north to Pines Boulevard and Sheridan Street to the south, and from L-37 and L-33 to the west to Hiatus Road and Palm Avenue to the east.

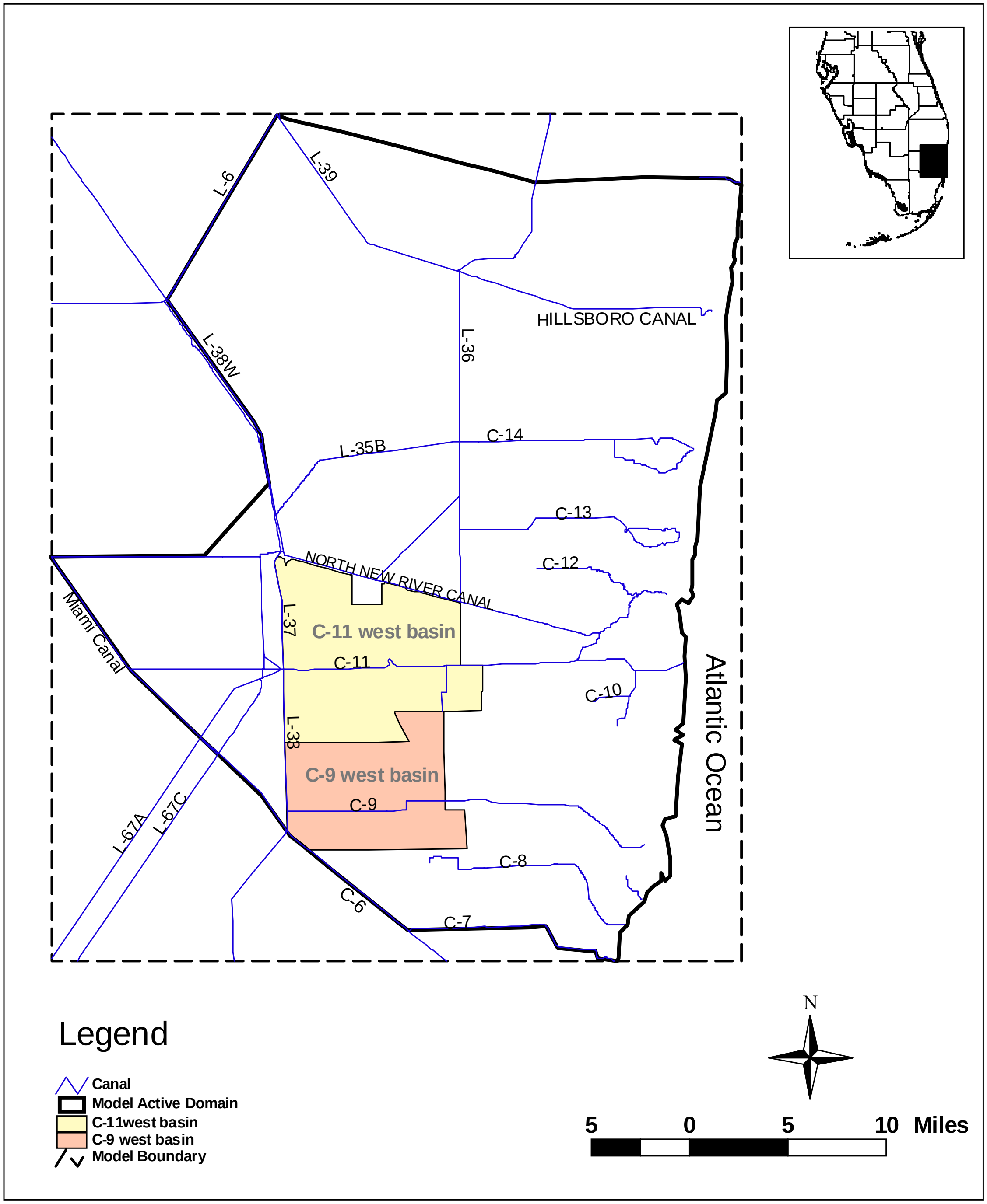


Figure 2.1 C-11 West Basin and C-9 West Basin in MODFLOW Domain

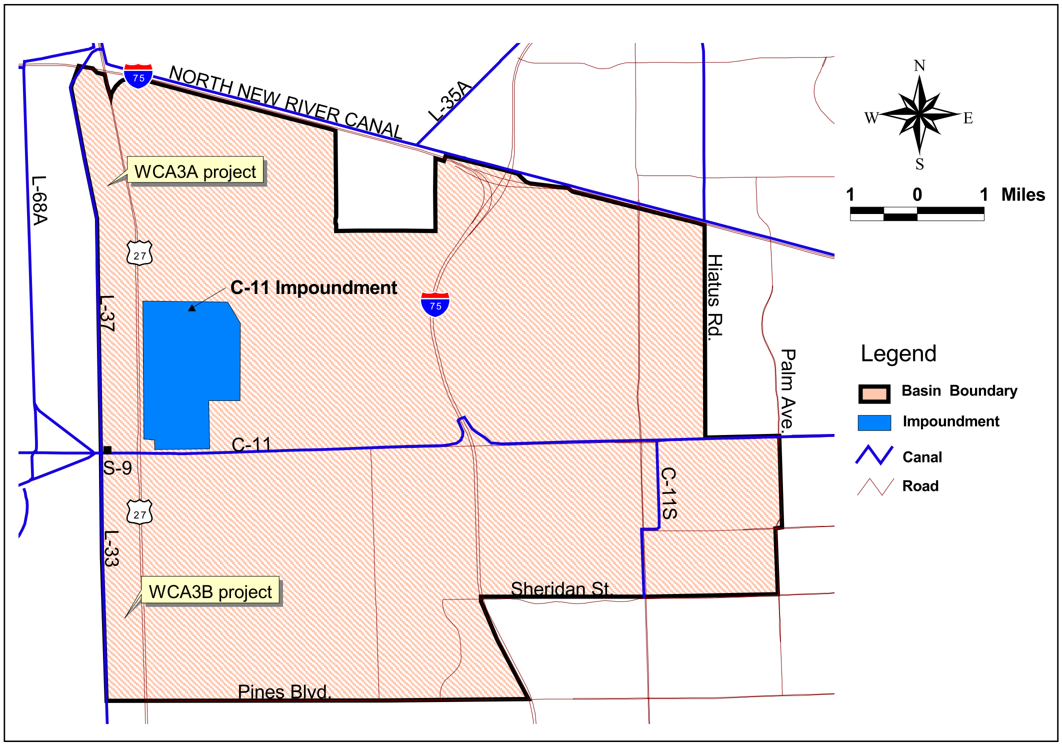


Figure 2.2 C-11 West Basin Boundary and Canals

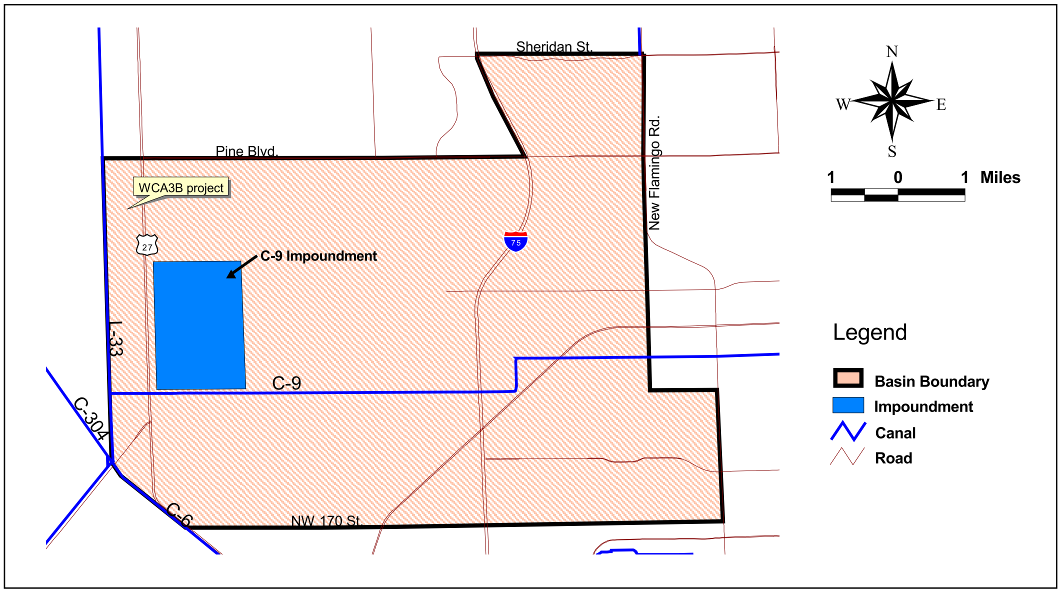


Figure 2.3 C-9 West Basin Boundary and Canals

The C-11 canal is the primary canal in the basin. The runoff from the C-11 West basin discharges to the C-11 canal through a number of lateral and equalizer canals, and is pumped west to Water Conservation Area (WCA) 3A through the primary discharge structure, S-9 pump station.

- **C-9 West Basin**

The C-9 West basin, as shown in Figure 2.3, is located south of the C-11 West basin and has a drainage area of approximately 31,400 acres (49 square miles). The basin extends from Pines Boulevard and Sheridan Street to the north to NW 170 Street to the south, and from L-33 to the west to New Flamingo Road to the east. The C-9 canal is the primary conveyance canal in the basin.

- **BCWPA CERP Project Description**

The BCWPA CERP project within the C-11 West and C-9 West basins is comprised of three components: the C-11 Impoundment, the C-9 Impoundment, and the Water Conservation Area 3A/3B Levee Seepage Management System.

The C-11 impoundment is located in the C-11 West basin, north of the C-11 canal and east of US 27. The storage area of the C-11 impoundment is approximately 1,490 acres and the wetland buffer marsh area on the north side is approximately 205 acres.

The C-9 impoundment is located in the C-9 West basin, north of the C-9 canal and east of US 27. The storage area of the C-9 impoundment is about 1,650 acres, and the wetland buffer marsh area on the north side is approximately 360 acres.

The C-11 and C-9 impoundments serve primarily six functions: (1) to aid in reducing seepage from the WCA 3A/3B Levee Seepage Management Area; (2) to provide groundwater recharge; (3) to provide adequate water supply to urban areas; (4) to prevent saltwater intrusion; (5) to provide flood protection capabilities; and (6) to aid in water quality improvement.

The purpose of the WCA 3A/3B Levee Seepage Management System is to reduce the seepage from WCA 3A/3B by holding higher water levels in the L-33 and L-37 borrow canals and marsh areas. The purpose of the C-11 impoundment is to provide storage for excess runoff from the C-11 West basin and prevent pumping the untreated runoff into the WCA 3A. If water is not available in the impoundment area, the S-381 gate will be opened to allow seepage water to recharge the C-11 West basin and prevent excessive dry outs. In addition, seepage will be collected and returned to the impoundment area.

The purpose of the C-9 impoundment is to provide storage for excess runoff from the western C-9 drainage basin and impound the runoff diverted from the C-11 West basin to prevent discharge of untreated runoff into WCA 3A. During the wet season, the S-511 gate, located in the C-9 canal about 200 ft east of the C-9 impoundment eastern boundary, will be opened, this will benefit pumping the runoff from the western C-9 basin and/or the diverted runoff from the C-11 West basin by the way of the C-502 canal. When water is released from the C-9 impoundment into the C-9 canal, S-511 can be either partially or fully opened to convey water to the east if desired, but would be closed if water was to move south by the way of the C-502 canal.

2.6. Comparison of SFWMM Generated Boundary Conditions (Task 1.0)

The boundary conditions taken from the SFWMM 1995 Base condition analysis were used in the subregional MODFLOW models employed in the WPA Feasibility Study. These models have not been updated to reflect potential changes to boundary conditions resulting from the SFWMM 2000 Base simulations which correspond to the pre-CERP baseline condition for flood protection assurance analyses prescribed by CERP regulatory requirements. In this task, the outputs of the 1995 Base, 2000 Base, and 2010 Base SFWMM simulations were reviewed and compared to identify changes in surface water profiles between these three simulations in the C-11 and C-9 canals. Details of this analysis are included in Appendix A.

The results of this review show no significant bias in boundary conditions (canal stages) was introduced to the subregional groundwater model by using the period 1988-1995 in lieu of the full 1965-1995 period of simulated data; and the boundary conditions (canal stages) for the subregional models taken from the 1995 Base simulations are approximately the same as those taken from the 2000 Base or 2010 Base simulations.

2.7. Delineation of Impacted Areas by Project (Task 1.1)

2.7.1. Methodology

The Broward County Groundwater Flow Model simulated two different conditions: pre- and post- conditions. The results for pre- and post- conditions are different in groundwater stage within the C-11 West basin and the C-9 West basin attributable to implementation of the C-11 impoundment, the C-9 impoundment, the WCA 3A/3B Levee Seepage Management System, and all other WPA CERP Project Components.

The model domain was discretized horizontally using a finite difference grid consisting of 456 rows, 371 columns, of 500-ft square cells, and the model has five layers in the vertical direction. The groundwater stage data in the top layer of the surficial aquifer was used for flood protection analysis in this report.

The model simulations were temporally discretized using a constant stress period and a time step length of one day. Each day from January 1, 1988 to December 31, 1995 was treated as one stress period. Thus, there are 2,920 daily values of the groundwater stage data for pre- and post- conditions.

For the C-11 West and C-9 West basins, the analysis procedure included:

- The grid cells within each subbasin from the BC MODFLOW domain (371 x 456 = 169,176 cells) were selected.
- For each day of 2,920 days in the simulation period, the groundwater stage difference values between post- and pre- conditions were added up for all selected cells, and the average value over the subbasin was calculated by dividing the summation by the amount of the selected cells.
- Among the total 2,920 daily average groundwater stage difference value, one peak day was selected as the "worst case."
- The groundwater stage difference grid map was plotted in ArcView GIS 3.2 for the "worst case" date selected above; the cells with groundwater stage difference of 0.1 ft or greater were then identified.

2.7.2. C-11 West Basin

The C-11 West basin shown in Figure 2.2 encompasses 7,944 grids cells, i.e., a drainage area of approximately 45,600 acres. In Figure 2.4, average groundwater stage for post- and pre- conditions and their difference are plotted. The groundwater stage for post-condition is higher than the pre-condition in wet season, over the period of simulation. The impact of higher groundwater stage on flood protection of the C-11 West basin will be evaluated in this report.

The peak value of average groundwater stage difference occurred on the 1035th day (November 1, 1990) among the 2,920-day simulation period. The area of impact is described as the area with the groundwater stage difference greater than 0.1 ft. The

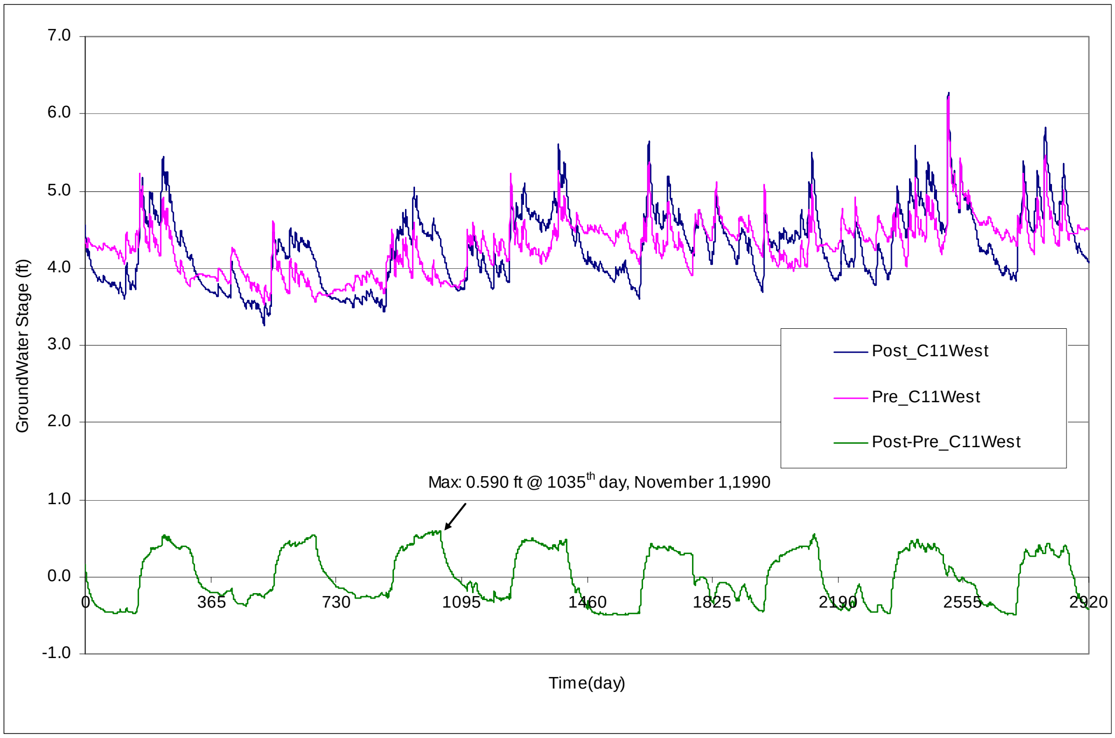
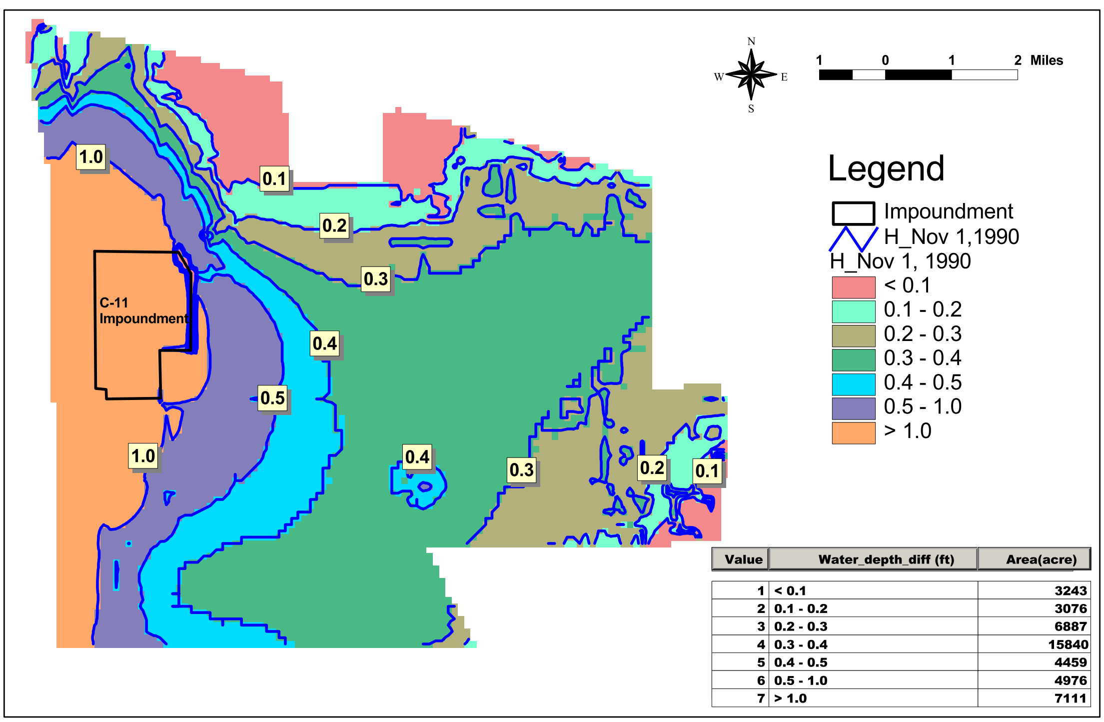


Figure 2.4 Daily Groundwater Stage in C-11 West Basin (2010WPA, 1995Base, and Their Difference) Average over the Basin



Value	Water_depth_diff (ft)	Area(acre)
1	< 0.1	3243
2	0.1 - 0.2	3076
3	0.2 - 0.3	6887
4	0.3 - 0.4	15840
5	0.4 - 0.5	4459
6	0.5 - 1.0	4976
7	> 1.0	7111

Figure 2.5 Groundwater Stage Difference of C-11 West Basin on November 1, 1990

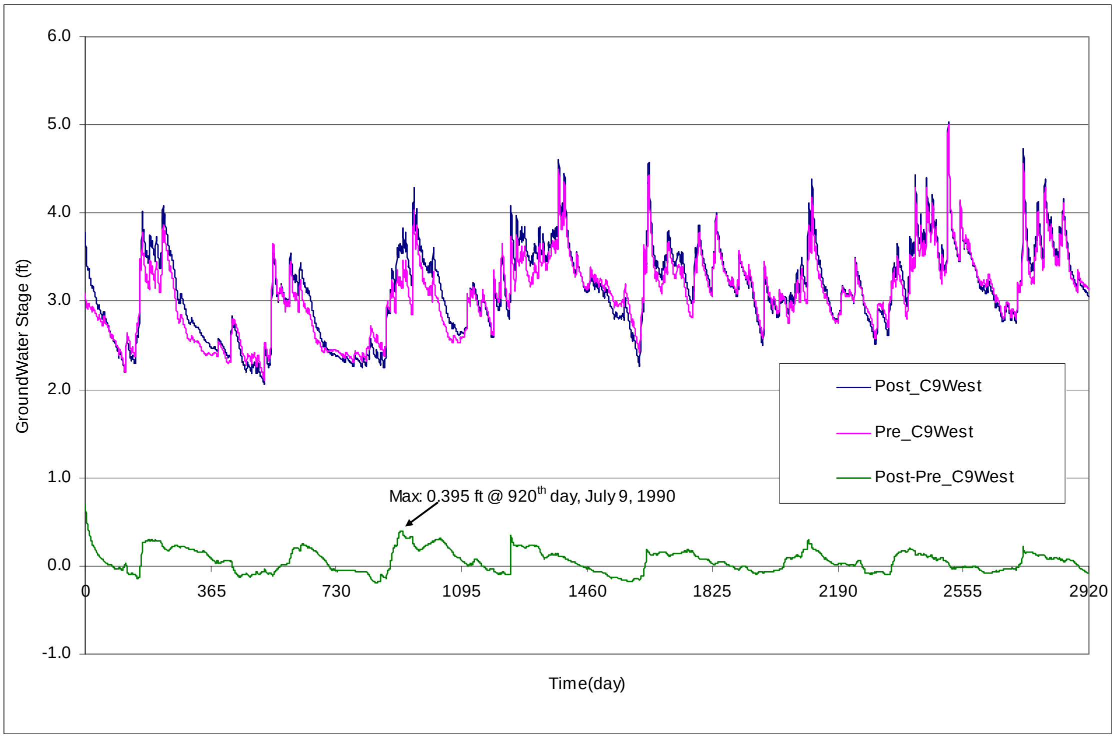
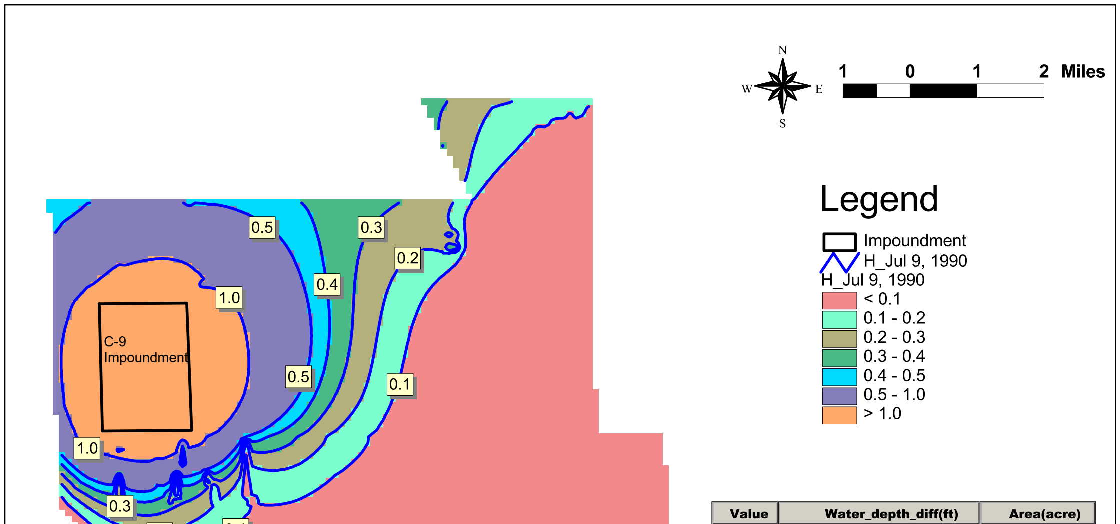


Figure 2.6 Daily Groundwater Stage in C-9 West Basin (2010WPA, 1995Base, and Their Difference) Average over the Basin



Value	Water_depth_diff(ft)	Area(acre)
1	< 0.1	14118
2	0.1 - 0.2	3042
3	0.2 - 0.3	2445
4	0.3 - 0.4	1567
5	0.4 - 0.5	1291
6	0.5 - 1.0	4603
7	> 1.0	4327

Figure 2.7 Groundwater Stage Difference of C-9 West Basin on July 9, 1990

contours of groundwater stage difference are plotted in Figure 2.5 and the total impacted area is estimated at 42,439 acres, which is approximately 93 % of total basin area.

2.7.3. C-9 West Basin

The C-9 West basin, as shown in Figure 2.3, includes 5,470 grids cells with a total drainage area of approximately 31,400 acres. In Figure 2.6, the average groundwater stage for post- and pre- conditions and their differences are plotted. It is seen that the groundwater stage for post-condition is higher than that for pre-condition in wet season, over the period of simulation. The impact of higher groundwater stage on flood protection of the C-9 West basin will be evaluated in this report.

The peak value of average groundwater stage difference occurred on the 920th day (July 9, 1990) among the 2,920-day simulation period for the C-9 West basin. The contours of groundwater stage differences are plotted in Figure 2.7 and the total impacted area is 17,275 acres, which accounts for 55 % of total basin area.

2.7.4. Conclusions

The contours of groundwater stage differences for the C-11 West basin and the C-9 West basin are delineated in Figure 2.5 and Figure 2.7. About 93% of the C-11 West basin area and 55% of the C-9 West basin area were defined as the impacted area in which the groundwater stage increased over 0.1 ft.

2.8. Pre-storm Seepage Analysis (Task 1.2)

2.8.1. Seepage into C-11 and C-9 Canals

In the Broward County MODFLOW model, the Recharge package, River package, Drainage package, and Reinjection Drainflow package are used, as applicable, to interchange water between groundwater and surface water systems. These packages are briefly described in the WPA Feasibility Report, Appendix B: Engineering Design.

The increase of the groundwater stage affects the quantity and the direction of flow on the boundaries. The River and Drainage packages were applied in the BC MODFLOW to quantify the seepage rates into the C-11 and C-9 canals. The River package simulates groundwater interchanges with canals or rivers that either recharge or drain the aquifer. The Drainage package is essentially the same as the River package except that the canals can only drain the aquifer and water removed from the canals is removed permanently from the model.

The seepage rate of each subbasin was composed of the flow rates from the grid cells that were computed by two packages: the River and Drainage packages, for pre- and post-conditions, respectively. The groundwater stage data is available for each day of the simulation period of 1988-1995. However, the seepage rates calculated from the River and Drainage packages are available corresponding to the last day of each month for the same simulation period.

The C-11 Impoundment, the C-9 Impoundment and the WCA3A/3B Levee Seepage Management System are three components of the Broward County WPA project. The WCA3A/3B Levee Seepage Management System, which is located west of US 27 in the C-11 West basin and the C-9 West basin, is not considered significant in terms of flood protection. Therefore, the flood protection analysis was considered for the adjusted basin areas, in which the C-11 and C-9 impoundments are proposed to provide storage for excess runoff from the basin area east of US 27, as shown in Figure 2.2 and Figure 2.3.

- **C-11 West Basin**

The seepage rates for the C-11 West basin are plotted in Figure 2.8 for the pre-condition and in Figure 2.9 for the post-condition, respectively. Figure 2.10 presents the difference of seepage rates between post- and pre- conditions. The maximum and average differences of seepage rates for the C-11 West basin are approximately 4,242.79 cfs and 59.05 cfs, respectively, as shown in Figure 2.10.

- **C-9 West Basin**

The seepage rates for the C-9 West basin are plotted in Figure 2.11 for the pre-condition and in Figure 2.12 for the post-condition, respectively. Figure 2.13 shows the difference of seepage rates between post- and pre- conditions. The maximum and average differences of seepage rates for the C-9 West basin are approximately 1,815.41 cfs and 370.95 cfs, respectively, as shown in Figure 2.13.

- **Results**

The runoff analysis was performed with both increased seepage and increased groundwater stage. The date with the maximum seepage rate difference was selected as the “worst case,” i.e., the difference of runoff volume between post- and pre- conditions was predicted to be the largest. As shown in Figure 2.10 and Figure 2.13, the maximum value of the seepage rate difference into the C-11 and C-9 canals occurred at the end day of the 1st month in 8-year simulation period, at which the seepage rates were significantly affected by the initial condition for both pre- and post- conditions. Thus, 3,648.18 cfs at the end day of the 35th month (November 30, 1990) and 1,495.05 cfs at the end day of the 8th month (August 31, 1988) were selected as the maximum seepage rate difference values into the C-11 canal and the C-9 canal, respectively.

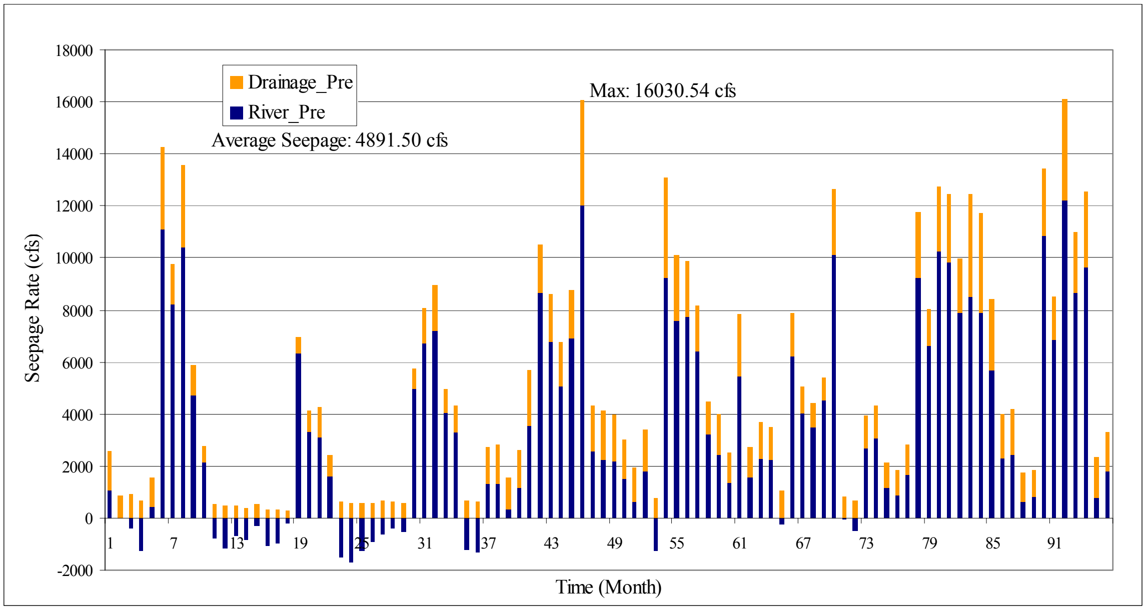


Figure 2.8 Seepage Rate into C-11 Canal for Pre-condition

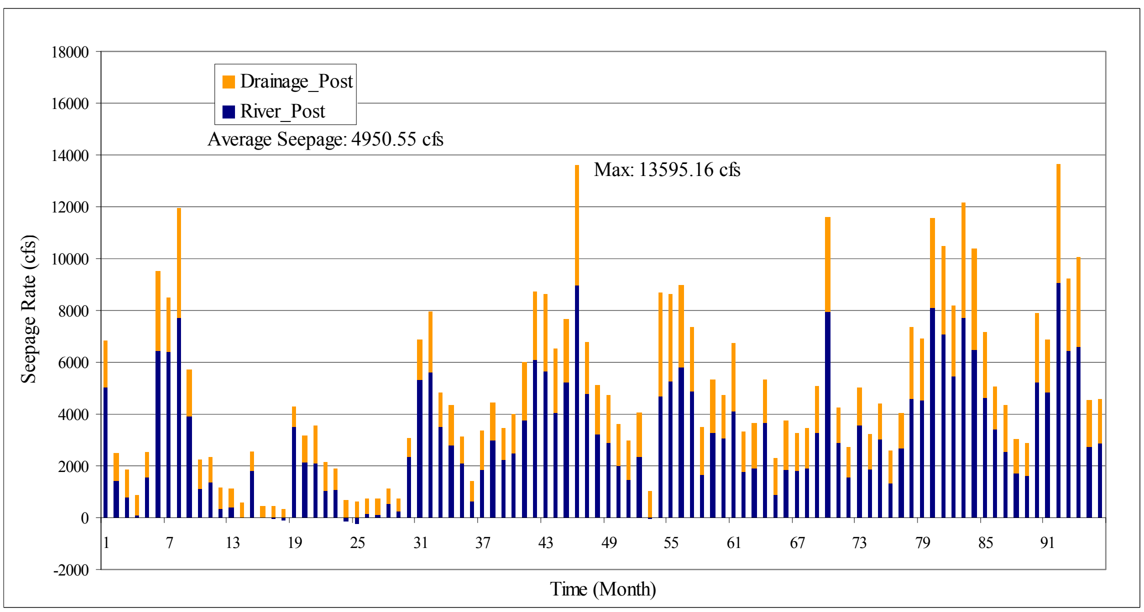


Figure 2.9 Seepage Rate into C-11 Canal for Post-condition

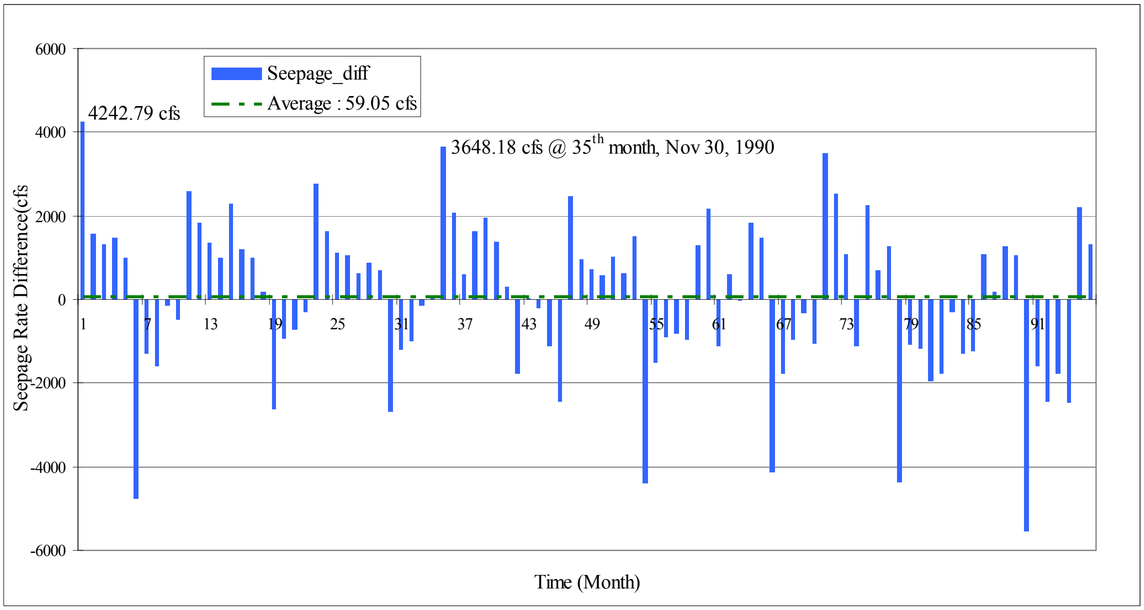


Figure 2.10 Seepage Rate Difference into C-11 Canal between Post- and Pre-conditions

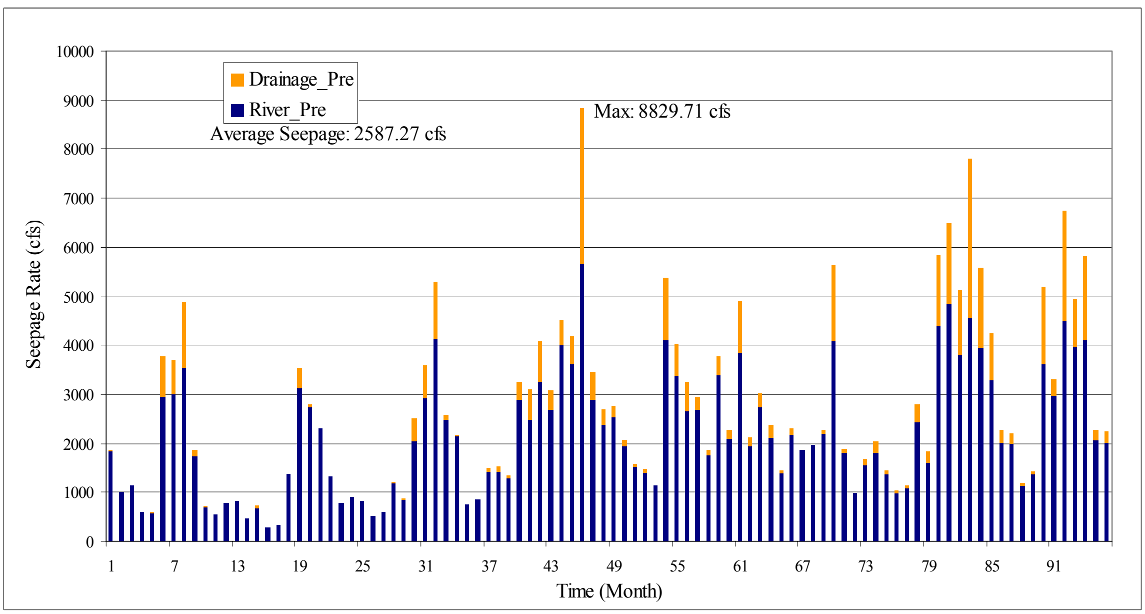


Figure 2.11 Seepage Rates into C-9 Canal for Pre-condition

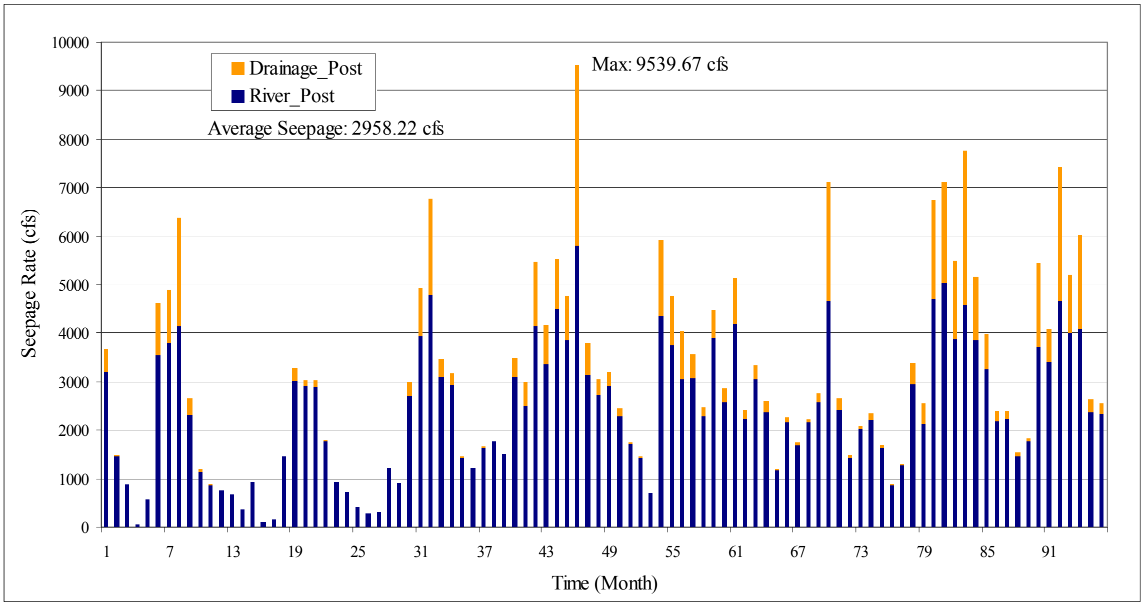


Figure 2.12 Seepage Rate into C-9 Canal for Post-condition

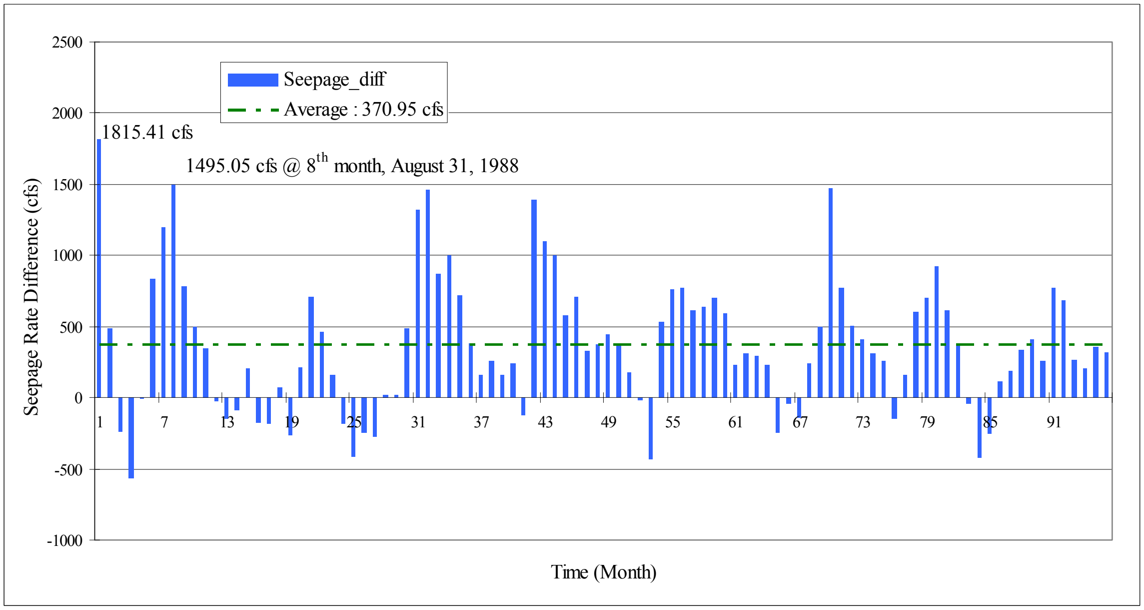


Figure 2.13 Seepage Rate Difference into C-9 Canal between Post- and Pre-conditions

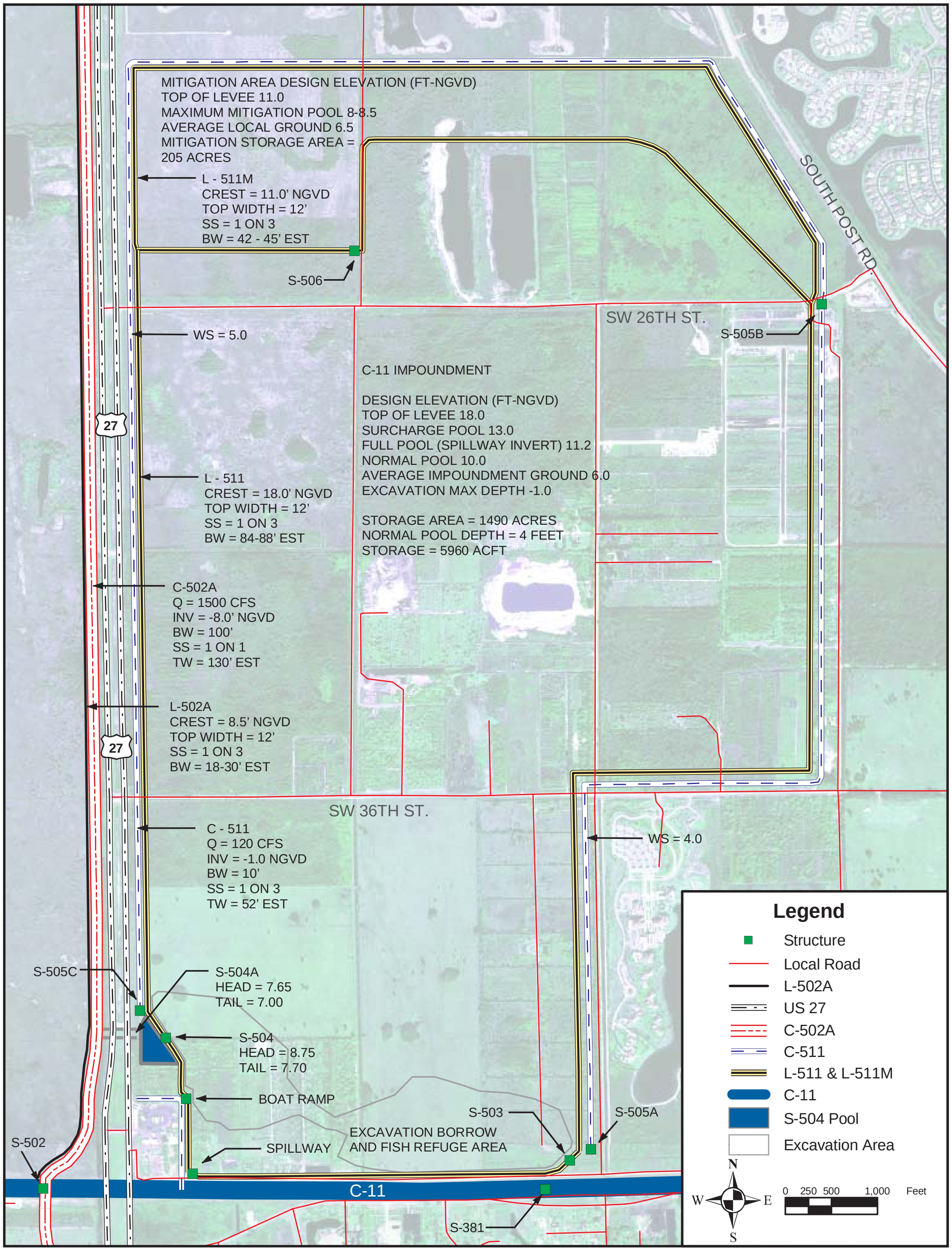


Figure 2.14 Structure Location Map of the C-11 Impoundment

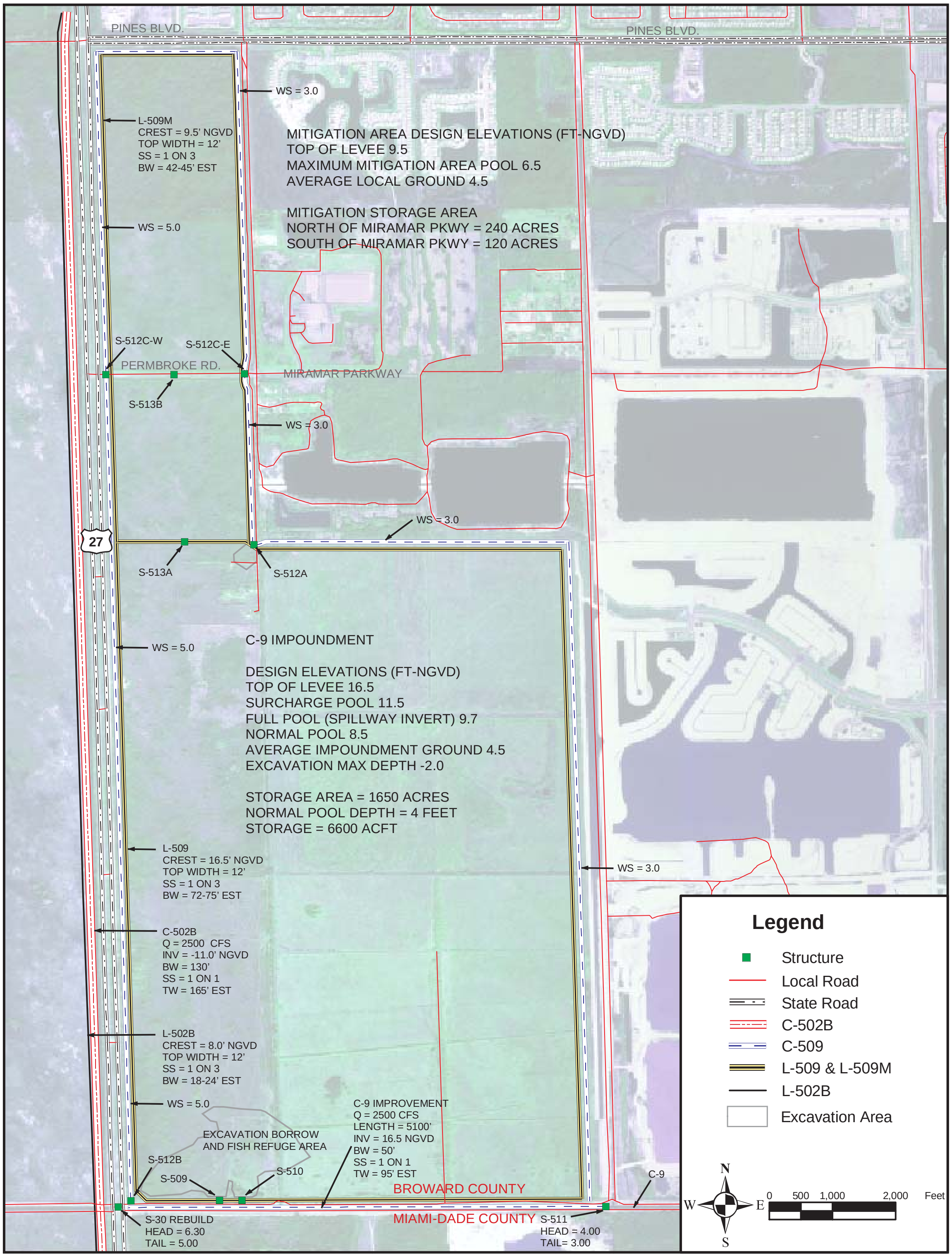


Figure 2.15 Structure Location Map of the C-9 Impoundment

2.8.2. Seepage Collection System Assessment

The C-11 and C-9 impoundments are both four-foot deep aboveground impoundments. The seepage canals, C-511 Canal and C-509 Canal, are designed to collect the seepage flow from the C-11 impoundment and the C-9 impoundment, respectively. The pump stations backpump seepage flow intercepted by seepage canals into the impoundments. Figure 2.14 and Figure 2.15 are the structure location maps of the C-11 impoundment and the C-9 impoundment, respectively.

- **SEEP2D Modeling**

The SEEP2D model, developed by the United States Army Corps of Engineer (USACE) Waterway Experiment Station, was selected for the WPA seepage modeling efforts. It is a two-dimensional steady-state finite element numeric model used to evaluate the seepage losses from impoundments and conveyance canals. More details about the model development and result evaluation are described in the WPA Feasibility Report, Appendix C: Geotechnical Appendix. The seepage rates captured by the C-511 canal from the SEEP2D modeling and the design capacity of the C-11 impoundment seepage collection system are provided in Table 2.1 and Table 2.2, respectively. The seepage rates captured by the C-509 canal from SEEP2D modeling and the design capacity for the C-9 impoundment seepage collection system are provided in Table 2.3 and Table 2.4, respectively.

By comparing the seepage rates by SEEP2D with the design seepage capacity, the seepage collection systems are capable of maintaining groundwater levels at pre-CERP baseline elevations. As noted in Table 2.1 and Table 2.3, SEEP2D model results were multiplied by a design safety factor of 5.0 to be consistent with the WPA Feasibility Report.

Table 2.1 Seepage Rate into C-511 Canal by SEEP2D

	Length (ft)	Seepage Rate (cfs/ft)*	Seepage Rate (cfs)
Northern Boundary	6,315	0.00178	11.24
Eastern Boundary			
North of S-505B	2,965	0.00786	23.30
South of S-505B	11,745	0.00786	92.31
Southern Boundary	1,390	0.00178	2.47
Western Boundary	10,290	0.00605	62.25
Totals	32,705		191.57

*Seepage rate into C-511 canal is based on the seepage rate into C-511 when the water surface maintains at normal pool (10 ft-NGVD), and multiples a safety factor of 5.0.

Table 2.2 Design Capacity of the C-11 Seepage Collection System

Structure	Design Capacity (cfs)*
S-505A Fixed Weir	150
S-505C Pump Station	120
Totals	270

*The design capacities of seepage collection structures are from the WPA Feasibility Report, Appendix B: Engineering Design, Section B.7.

Table 2.3 Seepage Rates into C-509 Canal by SEEP2D

	Length (ft)	Seepage Rate (cfs/ft)	Seepage Rate (cfs)
C-509 Western and Mitigated Wetland Boundary			
West Boundary	10,500	0.00205	21.53
Mitigated Wetland Boundary	2,250	0.00500	11.25
C-509 Northern and Eastern Boundary			
Northern Boundary	4,935	0.00500	24.68
Eastern Boundary	10,445	0.00648	67.68
Totals	28,130		125.14

*Seepage rate into C-509 canal is based on the seepage rate into C-509 when the water surface maintains at normal pool (8.5 ft-NGVD), and multiples a safety factor of 5.0.

Table 2.4 Design Capacity of the C-9 Seepage Collection System

Structure	Design Capacity (cfs)*
S-512A Pump Station	225
S-512B Fixed Weir	125
S-512B Gated Culvert	125
Totals	475

*The design capacities of seepage collection structures are from the WPA Feasibility Report, Appendix B: Engineering Design, Section B.8.

- **BC MODFLOW Modeling**

The seepage collection system was considered as one special boundary condition in the BC MODFLOW. The Reinjection Drainflow package, briefly described in the WPA Feasibility Report, was developed and assembled into the BC MODFLOW by SFWMD to simulate the backpumping of seepage into the impoundments by returning seepage collected by the perimeter canals to the impoundments. In the Reinjection Drainflow package, the surface water elevations of the impoundments and the seepage canals were maintained at fixed water elevations estimated in WPA Feasibility Study phase. The seepage canals did not recharge the aquifer, and the water captured by the seepage canals

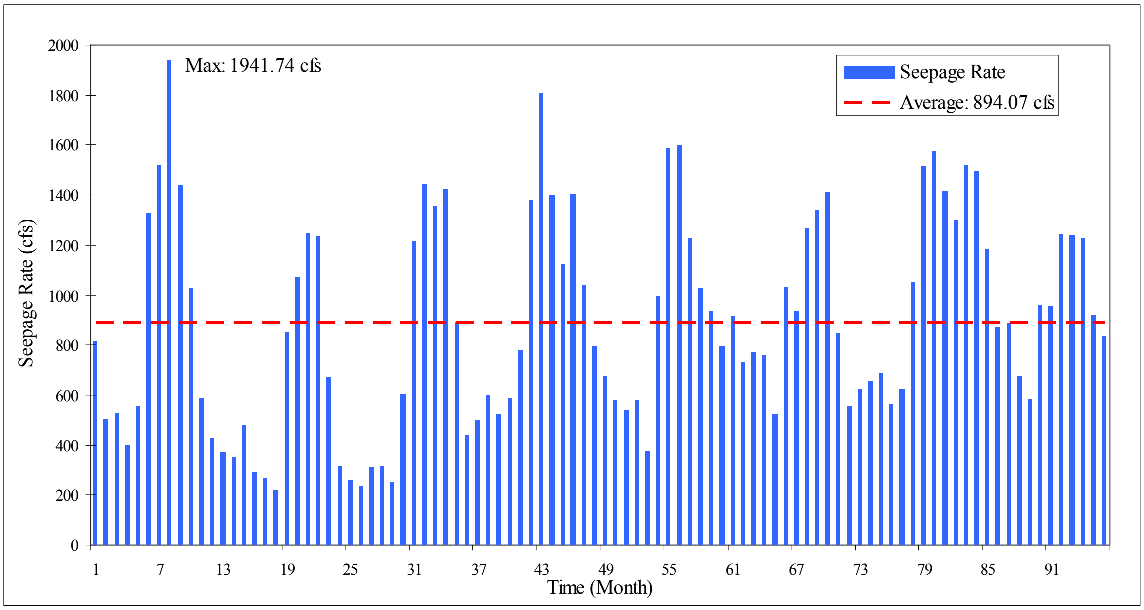


Figure 2.16 Seepage Rate into C-11 Seepage Collection System by MODFLOW

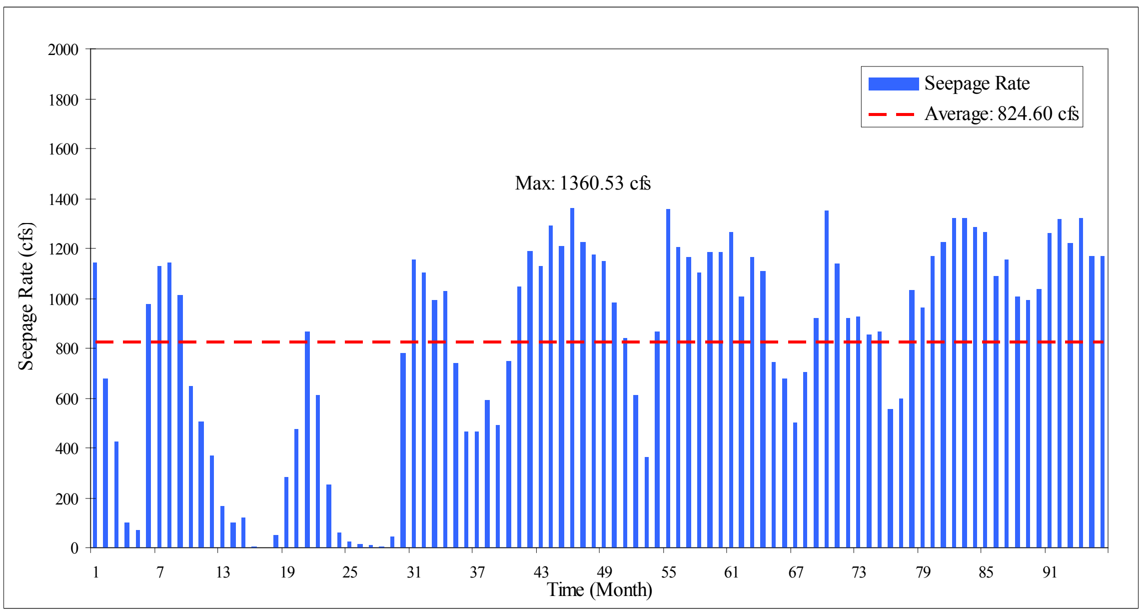


Figure 2.17 Seepage Rate into C-9 Seepage Collection System by MODFLOW

were assumed to be added (reinjected) to the aquifer below the impoundments within the same time step.

The available 96 daily seepage rates into the seepage canals for post-condition were used. Figure 2.16 and Figure 2.17 show the seepage rates into the C-511 canal and the C-509 canal, respectively. The maximum and average seepage rates into the C-511 canal are approximately 1,941.74 cfs and 894.07 cfs, respectively. The maximum and average seepage rates into the C-509 canal are approximately 1,360.53 cfs and 824.60 cfs, respectively.

2.8.3. Comparison of SEEP2D and MODFLOW Seepage Results

The seepage results by the BC MODFLOW are suspect due to the manner in which computation is made. The average seepage captured by the seepage canals using the BC MODFLOW was calculated based on the available 96 daily seepage results, but did not indicate the same magnitude of the seepage rates as the SEEP2D model or the design seepage capacity of the seepage collection system discussed above. The results in Table 2.5 indicate that the average seepage rates using the BC MODFLOW were about 20 times greater than the SEEP2D model results. A seepage factor of 20 was therefore used to revise the seepage rates into the C-11 and C-9 canals that were accumulated from the River and Drainage packages in the BC MODFLOW. The modified seepage rate differences of 182.41 cfs (3,648.18 cfs/20.0) on November 30, 1990 and 74.75 cfs (1,495.05 cfs/20.0) on August 31, 1988 were treated as the maximum seepage rate difference values into the C-11 canal and the C-9 canal, respectively, and were utilized in hydrograph computation in Task 1.3.

Table 2.5 Comparison of the Seepage Rate into Seepage Canals by SEEP2D and BC MODFLOW Models

Seepage Canal	Seepage Rate by SEEP2D (cfs)*	Seepage Rate by MODFLOW (cfs)**	Seepage Factor***
C-511	38.31	894.07	23.34
C-509	25.03	824.60	32.94

*Seepage rates into the C-511 and C-509 canals are based on the seepage rates when the impoundments water surface is maintain at normal pool without multiplying a safety factor of 5.0.
** Average seepage rates into the C-511 and C-509 canals are based on the 96 daily seepage rates from the BC MODFLOW (see Figures 2.16 and 2.17).
*** Seepage Factor = ratio of seepage rates computed using the BC MODFLOW and SEEP2D models.

2.8.4. Comparison of BC MODFLOW and SEEP2D Models

As discussed previously, the BC MODFLOW overestimates the seepage rates by a factor of 20 in comparison with the SEEP2D model. In the following, the limitation of the BC MODFLOW and the advantage of the SEEP2D model in computing seepage rates are discussed. It should be noted that most of the limitations of the BC MODFLOW arise by the way the model is constructed and should not be interpreted as applicable to MODFLOW model in general.

BC MODFLOW Limitations

- In Broward County MODFLOW model, the average groundwater head in a grid cell is utilized to calculate the head difference between the aquifer and the seepage canals. Also, the BC MODFLOW is a regional model with constant, large grid size (500 ft x 500 ft) that is applied to a large area. Consequently, the head difference is not accurately calculated and the seepage rate is therefore suspect. By refining the grid size around the seepage canals, more accurate adjacent groundwater head can be obtained from which a more accurate seepage rate can be obtained.
- The time step length of one day was modeled in the BC MODFLOW, while a pump used for seepage collection may be required to turn on/off several times per day. In the Reinjection Drainflow package, the seepage captured by the seepage canals was reinjected into the aquifer below the impoundments. The simulated groundwater levels in the aquifer below the impoundments may be raised substantially if a pumping facility continues to run during a full day time step. This modeling practice may overestimate the seepage collected by the seepage collection systems, and introduce inaccuracies in the computation of groundwater levels.
- Another factor that affects the seepage flow rate collected is the transmissivity of the interface between the aquifer and the canal, which is the intermediate result computed based on the hydraulic conductivity of the soil layer and other parameters. However, the transmissivity calculation is not available for review.
- Only 96 daily flow rate data corresponding to the end day of each month are available to review in the total 8-year simulation period.

SEEP2D Model Advantages

The SEEP2D model provides more accurate impoundment seepage and seepage canal collection computation than the BC MODFLOW for the following reasons:

- The SEEP2D model is a local, near field, 2D-vertical, finite element, variable grid model capable of accommodating specific impoundment, aquifer and seepage canal geometry and more accurate head computation whereas the BC MODFLOW is a regional, far field, 3D, finite difference, constant and large grid model, and may not accommodate the geometry and compute heads accurately.
- The SEEP2D model employs a more accurate nonlinear Dupuit equation than a Darcy equation used in the BC MODFLOW to compute heads in the aquifer
- The hydraulic conductivity values used in the SEEP2D model were derived from site specific geotechnical investigations at several sites in the vicinity of the impoundment as reported in the feasibility study report whereas the transmissivity values used in the BC MODFLOW are derived from sparse data
- The SEEP2D model calculates the total seepage from the impoundment, captured seepage in the seepage canals and underflow out of the model domain into the basin whereas the BC MODFLOW computes the seepage into the canals only.
- The model gives steady state heads and seepages and the time step is not specified and with no reinjection of seepage into the impoundment whereas the BC MODFLOW uses a one day time step with reinjection using seepage pumps. The pump operation schedule data was not available for review. If the pumps are assumed to operate continuously for each time step of one day used in the BC MODFLOW and if in practice the pumps are operated intermittently during a day, as is typically the practice, the BC MODFLOW overestimates the heads in the aquifer and seepage rates.
- Because of these reasons, SEEP2D has been extensively used over the past 25 years or more for impoundment seepage analysis rather than MODFLOW.

However, it must be noted that the SEEP2D model results were taken from the WPA Feasibility Report and no SEEP2D calibration results were available for review. Similarly, the BC MODFLOW calibration results were also not available for review.

2.8.5. Conclusions

Pre-storm seepage rates captured by the seepage collection systems were calculated using the available BC MODFLOW and SEEP2D model results. A comparison of the BC

MODFLOW and SEEP2D model results has indicated that the BC MODFLOW seepage results are approximately 20 times larger than the SEEP2D results. A number of reasons are provided for this in Section 2.8.4. The SEEP2D model results appear to be more accurate than BC MODFLOW results. The proposed capacities of the seepage collection system are adequate to remove the seepage captured by the seepage canals using the SEEP2D model. Note that SEEP2D results included in the WPA Feasibility Report in the geotechnical investigation of the C-11 and C-9 impoundments were used in this analysis. A new SEEP2D model was developed to more accurately estimate the seepage rates, which is addressed in the report of Task 2.

2.9. Analysis of Increased Storm Runoff Volume (Task 1.3)

2.9.1. Methodology

The "worst case" of the increased groundwater stage in impacted areas was determined in Task 1.1; and the analysis in Task 1.2 presented the amount of pre-storm seepage that enters into the C-11 and C-9 canals. Based on Task 1.1 and Task 1.2 results, runoff hydrographs were developed to estimate the increased runoff volume from the impacted area in the C-11 West basin and the C-9 West basin.

Each basin was discretized into 500-ft square cells. Each grid cell was treated as the basic computation unit. The hydrologic computation begins with a storm event distributed over each cell that generates runoff after the initial abstraction. The available soil moisture storage is calculated based on the "worst case" groundwater table depth (estimated in Task 1.1). The runoff fills the available storage in soil profile and then overflows or outflows to the outlet of the basin. The hydrograph rate and runoff volume of the cells in model domain are accumulated together.

The Santa Barbara Urban Hydrograph Method (SBUH), which has been modified by the SFWMD staff for consistent use with other procedures for stormwater system analysis, was selected to perform the runoff hydrograph computation. The basic modeling process using in the SBUH includes the following sequential steps:

- Select design storm events
- Provide basin parameters (model domain, discretization, topographic map, land use)
- Calculate impervious factor (based on the land use map)
- Calculate groundwater table depth
- Calculate soil moisture storage

- Calculate time of concentration
- Calculate runoff hydrograph by the SBUH method (without seepage)
- Calculate runoff hydrograph with seepage rate determined in Task 1.2

The modeling process above was repeated for both the C-11 West basin and the C-9 West basin for pre- and post- conditions. ArcView GIS 3.2 was utilized to perform the pre-processing and post-processing. A FORTRAN program was coded to simulate the SBUH program.

2.9.2. Design Storm Events

The design storms were identified as 10-, 25-, and 100-year, 72-hour storm events. The 24-hour (1-day) and 72-hour (3-day) duration maximum rainfalls are the most commonly considered storm events by the District's Regulation Department in the permit review process described in "Management and Storage of Surface Waters, Permit Information Manual, Volume IV" (SFWMD, 2000). The District is committed to maintain the most accurate and updated rainfall frequency data for use in evaluating the permit applications within its jurisdiction. In order to maintain such commitment, the District initially developed rainfall frequency curves for 24-hour through 120-hour durations (MacVicar, 1981). Based on the increased number of rainfall stations and rainfall measurement records, Trimble (1990) published revised rainfall frequency curves in the "Technical Memorandum, Frequency Analysis of One and Three-Day Rainfall Maxima for Central and Southern Florida". Since then the Regulation Department of the SFWMD has been using these new rainfall frequency curves as the basis of review for permit applications.

The 10-, 25-, and 100- year, 72-hour storm rainfall depths were obtained from the 3-day rainfall depth contours shown in the Figures C-7, C-8, and C-9 of the "Management and Storage of Surface Waters, Permit Information Manual, Volume IV" (SFWMD, 2000). Table 2.6 presents the estimated 72-hour storm event rainfall quantities for the C-11 West basin and the C-9 West basin, which were used for the current study. A single rainfall depth was applied over the C-11 West basin and the C-9 West basin, respectively. The 15-minute interval rainfall distribution consisting of the unit hydrograph and the cumulative percentage of 24-hour peak rainfall for a 72-hour storm event is presented in Appendix B of this report.

Table 2.6 72-hour Storm Rainfall Quantities for C-11 West Basin and C-9 West Basin

Storm Event	Storm Rainfall Depth (in)	
	C-11 West basin	C-9 West basin
10-year	10.2	10.3
25-year	12.0	12.5
100-year	15.0	16.0

2.9.3. C-11 West Basin

- **Basin Area and Topography**

The C-11 West basin (see Figure 2.2) has a drainage area of approximately 45,600 acres. The basin was discretized into 7,944 grid cells (500 ft x 500 ft) as a part of the BC MODFLOW domain. The area on the west of US 27 is included in the WCA3A/3B Seepage Management System and was therefore not considered in the flood protection analysis for the C-11 West basin. The resulting drainage area of the C-11 West basin was about 42,000 acres. The C-11 impoundment (1,980 acres) planned in the post-condition, does not contribute runoff to the C-11 canal and was also excluded from the hydrograph computation for post-condition. Therefore, the basin area used in the hydrograph computation was estimated at 42,000 acres for pre-condition and 40,000 acres for post-condition. The outlet of the basin was modeled at the intersection of the C-11 canal and US 27, since the C-11 West basin drains west a majority of the time.

The topographic maps of the C-11 West basin for pre- and post- conditions used in the Broward County MODFLOW model were utilized in estimating the time of concentration used in developing the hydrographs. The topographic map of the C-11 West basin for pre-condition is plotted in Figure 2.18. The topographic map for post-condition was a modification of that for pre-condition by adding the C-11 impoundment and other CERP components in the C-11 West basin.

- **Land Use and Impervious Factor**

The land use map was compiled from SFWMD GIS database of 1995. Figure 2.19 presents the land use distribution for pre-condition of the C-11 West basin. The land use map for post-condition was the modification of the pre-condition by adding the C-11 impoundment and other CERP components in the C-11 West basin. The land use types and their areas in each 500 ft x 500 ft grid cell were further estimated. The impervious factor was assigned to

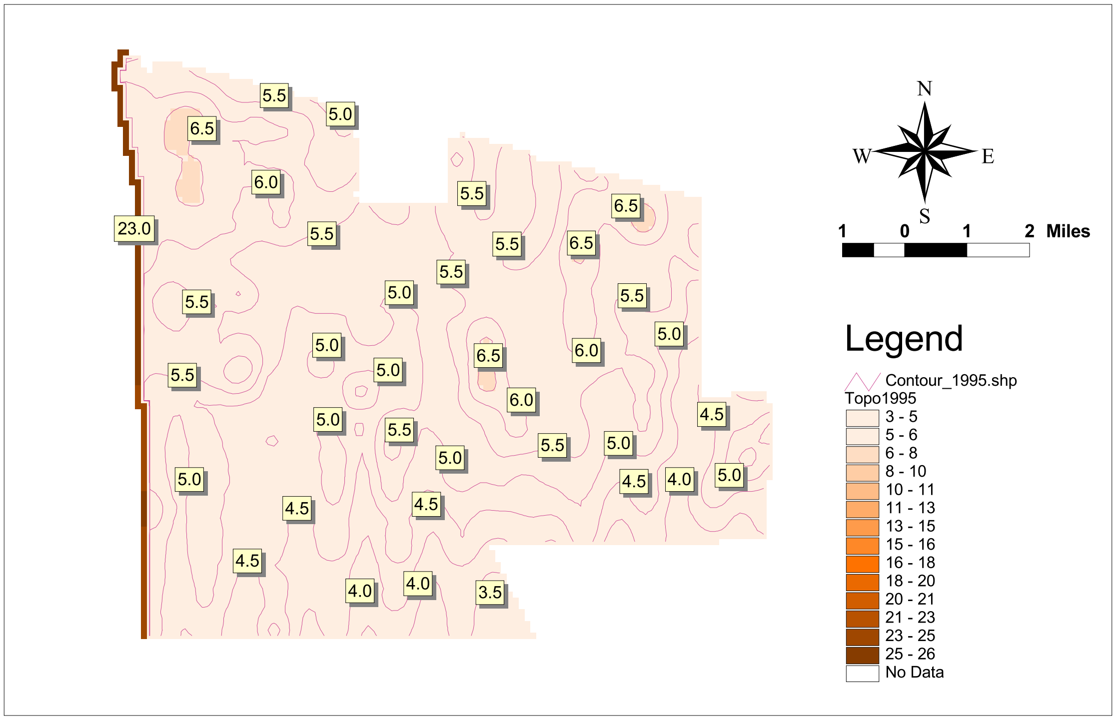


Figure 2.18 Topographic Map of C-11 West Basin for Pre-condition

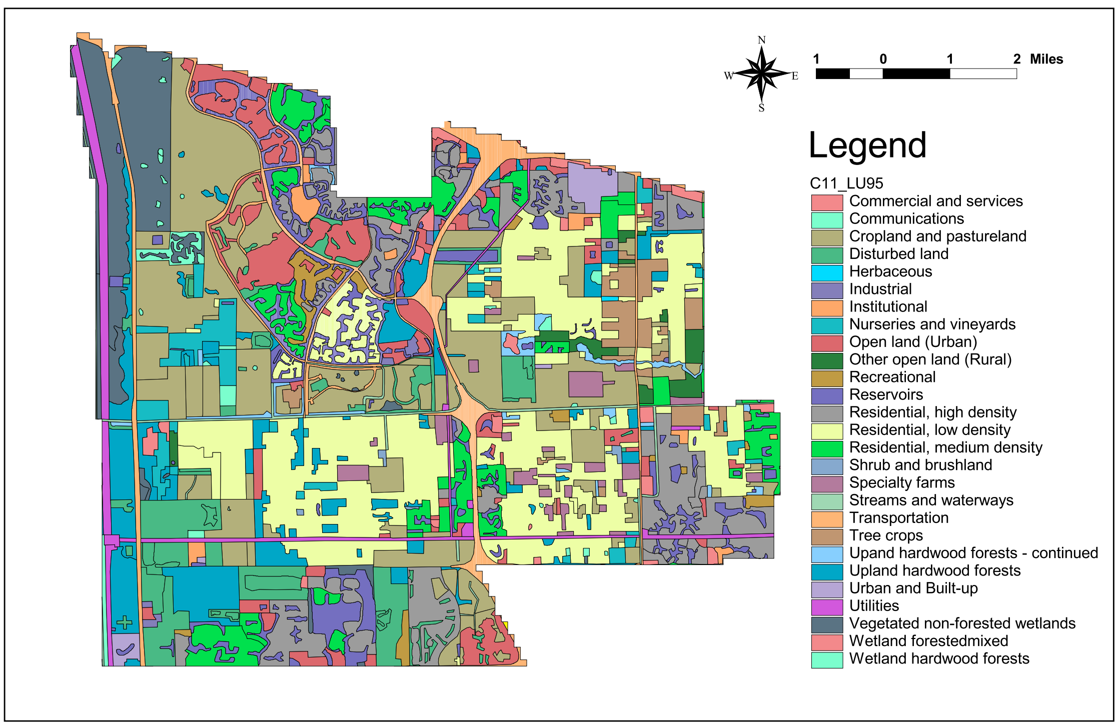


Figure 2.19 Land Use Map of C-11 West Basin for Pre-condition

Table 2.7 Land Use and Impervious Factors

ID	Land use Type*	Impervious Factor**
1000	Urban and Built-up	25%
1100	Residential, low density	12%
1200	Residential, medium density	25%
1300	Residential, high density	38%
1400	Commercial and services	85%
1500	Industrial	72%
1600	Extractive	100%
1700	Institutional	50%
1800	Recreational	0%
1900	Open land (Urban)	0%
2000	Agriculture	0%
2100	Cropland and pastureland	0%
2200	Tree crops	0%
2300	Feeding operations	0%
2400	Nurseries and vineyards	0%
2500	Specialty farms	0%
2600	Other open land (Rural)	0%
3000	Rangeland	0%
3200	Shrub and brushland	0%
3300	Mixed rangeland	0%
4000	Upland Forests	0%
4100	Upland coniferous forests	0%
4200	Upland hardwood forests	0%
4400	Tree plantations	0%
5000	Water	100%
5100	Streams and waterways	100%
5200	Lakes	100%
5300	Reservoirs	100%
5400	Bays and estuaries	100%
5600	Slough waters	100%
5700	Oceans, Seas, and Gulf's	100%
6000	Wetlands	100%
6100	Wetland hardwood forests	100%
6200	Wetland coniferous forests	100%
6300	Wetland forested mixed	100%
6400	Vegetated non-forested wetlands	100%
6500	Non-vegetated	100%
6900	Wetland shrub	100%
7000	Barren land	0%
7100	Beaches	0%
7200	Sand other than beaches	0%
7300	Exposed rocks	0%
7400	Disturbed land	0%
7500	Riverine sandbars	0%
8000	Transportation, communications, and utilities	100%
8100	Transportation	100%
8200	Communications	0%
8300	Utilities	0%

* From SFWMD GIS database.
** Modified from TR-55, Table 2-2a.

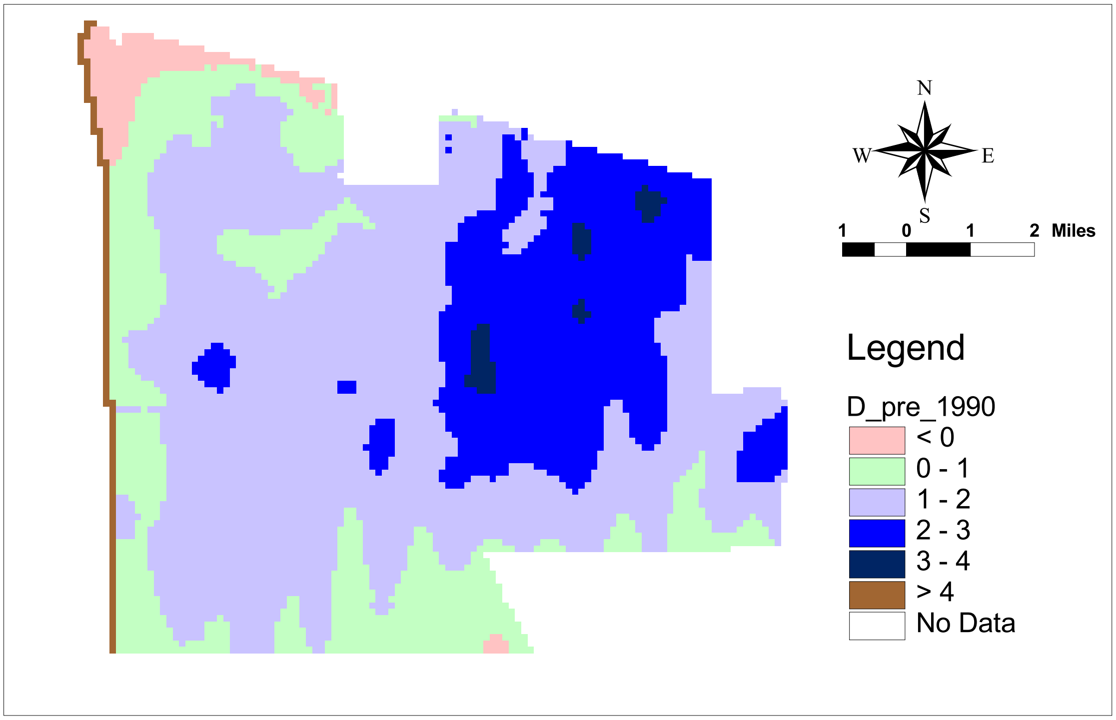


Figure 2.20 Water Table Depth of C-11 West Basin for Pre-condition on November 1, 1990

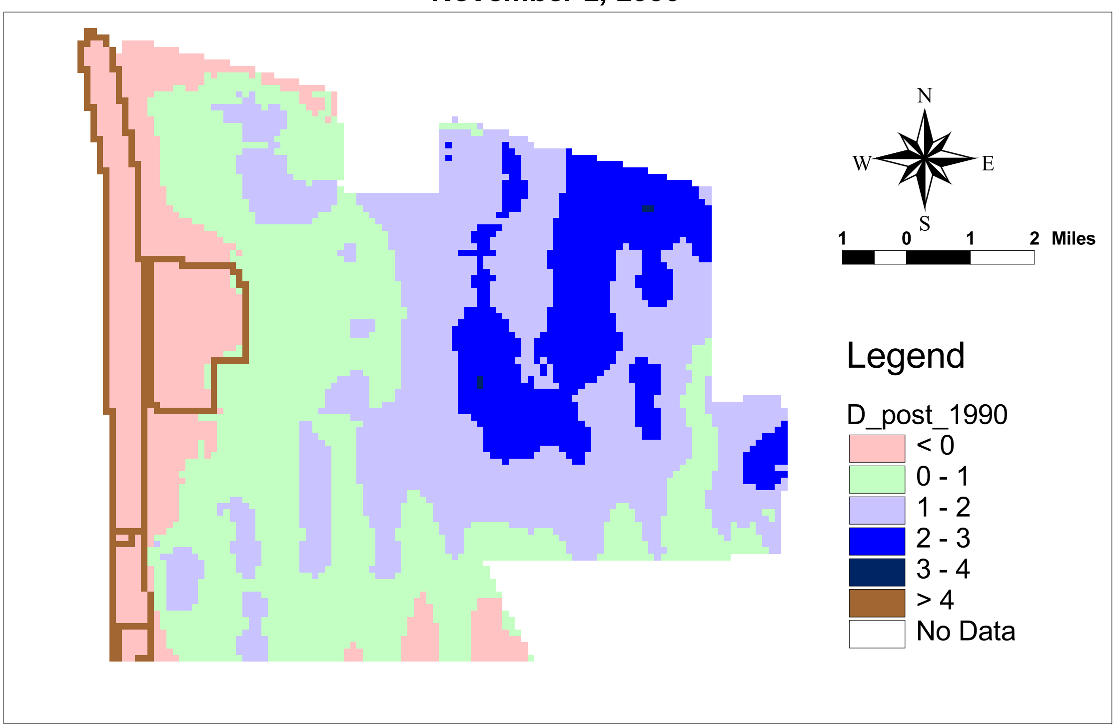


Figure 2.21 Water Table Depth of C-11 West Basin for Post-condition on November 1, 1990

each land use type, as listed in Table 2.7. The impervious factor was then calculated based on the land use in each grid cell, for pre- and post- conditions, respectively.

- **Groundwater Table Depth**

The depth to groundwater table is the difference between the ground surface elevation (topographic map) and the groundwater table elevation. From the analysis of previous Task 1.1, a "worst case" was determined for the C-11 West basin to be November 1, 1990, and the corresponding depth to groundwater table for pre- and post- conditions were used (see Figure 2.20 and Figure 2.21).

- **Soil Moisture Storage**

The moisture storage capability in the soil profile has been estimated by Natural Resources Conservation Service (NRCS, formerly, the Soil Conservation Service) for the normal sandy soils found within the South Florida Water Management District boundaries (SFWMD, 2000). The total amount of water, which can be stored in the soil profile expressed as a function of the depth to the water table for these soils, is listed in Table 2.8 (SFWMD, 2000).

Table 2.8 Soil Moisture Storage Value for Different Water Table Depth

Depth to Water Table (ft)	Cumulative Water Storage (in)	Compacted Water Storage (in)
1	0.6	0.45
2	2.5	1.88
3	6.6	4.95
4	10.9	8.18

The values in the third column of Table 2.8 represent the estimated amount of water, which can be stored in pervious areas after development. These values represent the cumulative water storage values reduced by 25% to account for the reduction in void spaces due to the compaction that occurred incidental to earthwork operations (SFWMD, 2000). The relationship of the compacted water storage and the depth to groundwater table is presented in Figure 2.22.

The impervious factor determined by the land use was introduced into Equation 2.1 below for calculating the weighted soil moisture storage.

$$S = S'(1 - f_i) \tag{2.1}$$

where,

S = weighted soil moisture storage, inches

S' = soil moisture storage for pervious areas after development, inches

f_i = impervious factor -- percentage of the impervious area, dimensionless (see Table 2.7)

Using the equation above, the soil moisture storage can be estimated for pre- and post-conditions of the C-11 West Basin.

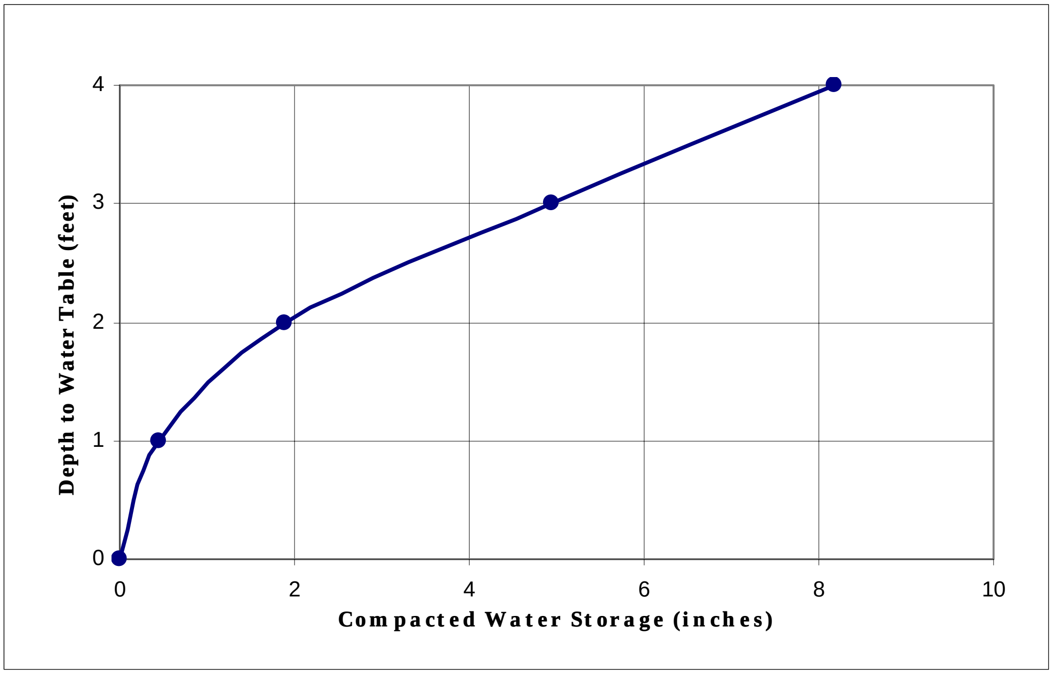


Figure 2.22 Compacted Water Storage and Water Table Depth Relationship

- **Runoff**

A method for estimating runoff from rainfall has been developed by NRCS. The runoff equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{2.2}$$

where,

Q = accumulated direct runoff, inches

P = accumulated rainfall, inches

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, inches

S = potential maximum retention, inches (See Equation 2.1)

The relationship between I_a and S was developed from experimental watershed data. The empirical relationship used in SCS runoff equation is

$$I_a = 0.2S \tag{2.3}$$

Substituting Equation 2.3 into Equation 2.2, it yields:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

(2.4)

- **Time of Concentration**

The time of concentration (T_c) is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed (TR-55, 2nd Edition, June 1986). This parameter controls the time in the storm event the entire basin is contributing the runoff. As storm water travels to the outlet of the basin, three types of runoff occur: sheet flow, shallow concentration flow, and open channel flow. These three components are then added up to get a total T_c. The time of concentration was calculated for all grid cells in the basin. The maximum T_c value was selected as the time of concentration of the basin, and assigned to all grid cells in the basin. The use of maximum T_c for each grid cell will preserve runoff volume but may underestimate the peak flow. However the method gave results similar to the lumped single basin method than the method employing separate T_c for each grid cell. This was verified early in the analysis. The grid-based approach better considers the impacted groundwater grid cells than a single, lumped basin approach. Also since in this study we are interested in the pre and post difference in runoff volume or unit runoff depth and not the design of flood protection system, the use of maximum T_c in a grid-based approach should be acceptable even if it somewhat underestimates the peak flow. The equations outlined in TR-55 are stated below.

1. Sheet Flow

Sheet flow is flow over plane surface. It usually occurs in the headwater of streams. For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows, 1976) to compute T_{t1}:

$$T_{t1} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \tag{2.5}$$

where,

T_{t1} = travel time for sheet flow (hr)

n = Manning's roughness coefficient

L = flow length (ft)

P_2 = 2-year, 24-hour rainfall (in)

s = slope of hydraulic grade line (land slope, ft/ft)

The following input data was used to compute the travel time of sheet flow for the C-11 West basin: Manning's n = 0.24 for dense grass (see Table 3-1, TR-55), flow length (L) = 200 ft (estimated using the procedure of TR-55), and 2-year, 24-hour rainfall (P_2) = 5.5 inches for C-11 West basin (see Figure B-3, TR-55). Based on the topographic map in Figure 2.18, the average slope (s) is 0.0002 ft/ft. Substituting the input data above into Equation 2.5, the travel time for sheet flow (T_{t1}) is calculated to be **2.0** hours for the C-11 West basin.

2. Shallow Concentrated Flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. For slopes less than 0.005 ft/ft, two equations were outlined in TR-55 based on the solution of Manning's equation.

$$V_1 = 16.1345\, s^{0.5} \text{ for unpaved area} \tag{2.6}$$

$$V_2 = 20.3282\, s^{0.5} \text{ for paved area} \tag{2.7}$$

where,

V_1 = average velocity for unpaved area (ft/s)

V_2 = average velocity for paved area (ft/s)

Therefore, the travel time for shallow concentrated flow is computed as below:

$$T_{t2} = \frac{2L}{3600(V_1 + V_2)} \tag{2.8}$$

The flow length (L) is assumed to be 800 feet (estimated using the procedure of TR-55), and the average slope (s) is 0.0002 ft/ft. Substituting this slope into Equations 2.6 and 2.7, V_1 = 0.228 ft/s and V_2 = 0.290 ft/s are obtained. Substituting these velocities into Equation 2.8, the travel time for shallow concentrated flow (T_{t2}) is calculated to be **0.85** hour for the C-11 West basin.

3. Open Channel Flow

Open channel flow is assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines appear on the USGS quadrangle sheets. Manning's equation can be used to estimate average velocity. Manning's equation is:

$$V_3 = \frac{1.49\, r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \tag{2.9}$$

where,

V_3 = average velocity for open channels (ft/s)

r = hydraulic radius (ft) and is equal to a/p_w

- a = cross section flow area (ft^2)
- p_w = wetted perimeter (ft)

s = slope of the hydraulic grade line (channel slope, ft/ft)

Then, the travel time for open channels is computed as below:

$$T_{t3} = \frac{L}{3600V_3} \tag{2.10}$$

where,

T_{t3} = travel time for open channels (hr)

The typical cross section area of the C-11 canal is 2,000 ft^2, and the wetted perimeter is 136.6 ft. The hydraulic radius (r) of the C-11 canal is then 14.6 ft. The hydraulic grade line slope (s) of the C-11 canal is 0.000012 ft/ft (s = ΔH/L, ΔH = 0.5 ft and L = 8 miles). Manning's n = 0.035 is used for the C-11 canal. From Equation 2.9, the average velocity in the C-11 canal (V_3) is calculated to be 0.88 ft/s.

The average velocity of 0.88 ft/s will be used for the C-11 canal and other secondary canals. The flow length of open channels is measured at 11.74 miles (62,000 ft), which is the longest distance from the farthest grid cell to outlet of the basin. From Equation 2.10, the travel time for open channel flow is calculated to be **19.57** hours for the C-11 West basin.

The time of concentration is the sum of the travel time for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t3} + T_{t3} \tag{2.11}$$

where,

T_c = time of concentration (hr)

Thus from Equation 2.11, the time of concentration (T_c) is calculated to be **22.42** hours, and assigned to all grid cells of the C-11 West basin.

- **Hydrographs**

The SBUH method was applied next based on the parameters from the previous steps. The main equations are listed here.

$$q_2 = q_1 + K(I_1 + I_2 - 2q_1) \tag{2.12}$$

$$K = \frac{\Delta t}{2T_c + \Delta t} \tag{2.13}$$

and

$$I_2 = \frac{(Q_2 - Q_1)A}{\Delta t} \quad (\text{since 1 acre-inch/hr} = 1\text{ cfs}) \tag{2.14}$$

where,

q_1 = hydrograph rate at time t-1, cfs

q_2 = hydrograph rate at time t, cfs

I_1 = instantaneous runoff rate at time t-1, cfs

I_2 = instantaneous runoff rate at time t, cfs

K = routing coefficient, dimensionless

Δt = routing intervals, hours

T_c = time of concentration, hours

Q_1 = accumulated direct runoff at time t-1, inches (see Equation 2.4)

Q_2 = accumulated direct runoff at time t, inches

A = area of basin (acres) – herein, one grid cell is treated as a computation unit.

- **Seepage Rate**

In Task 1.2, the maximum seepage difference between pre- and post- conditions was predicted to result in the "worst case" of the runoff volume increase in the basin. For the C-11 West basin, the seepage data on November 30, 1990 was selected. As discussed in Section 2.8.3, the BC MODFLOW appears to yield seepage rates approximately 20 times higher than the SEEP2D model and the results of SEEP2D appear to be more accurate. Thus, the seepage rates by the BC MODFLOW were divided by a factor of 20 for use in the hydrograph computation for the C-11 West basin.

2.9.4. C-9 West Basin

- **Basin Area and Topography**

The C-9 West basin (see Figure 2.3) has an area of approximately 31,400 acres. The basin was discretized into 5,470 grid cells (500 ft x 500 ft). The region west of US 27 is included in the WCA 3A/3B Levee Seepage Management System and was therefore not considered in flood protection analysis. The area of the C-9 impoundment was excluded from the hydrograph computation for post-condition. The basin area used in the hydrograph computation was estimated at 30,000 acres for pre-condition and 28,000 acres for post-condition. The outlet of the basin was modeled at the east boundary of the C-9 canal, since the C-9 West basin drains east a majority of the time.

The topographic maps of the C-9 West basin for pre- and post- conditions used in the Broward County MODFLOW model were utilized in the hydrograph computation. The topographic map of the C-9 West basin for pre-condition is plotted in Figure 2.23. The topographic map for post-condition was a modification of that for pre-condition by adding the C-9 impoundment and other CERP components in the C-9 West basin.

- **Land Use and Impervious Factor**

Figure 2.24 presents the land use distribution for pre-condition of the C-9 West basin. The impervious factor for each grid cell was then estimated for the C-9 West basin based on the land use map and Table 2.7.

- **Groundwater Table Depth**

  From the analysis of previous Task 1.1, a "worst case" was determined for the C-9 West basin to be July 9, 1990. The corresponding depth to groundwater table for pre- and post-conditions were used (see Figure 2.25 and Figure 2.26).

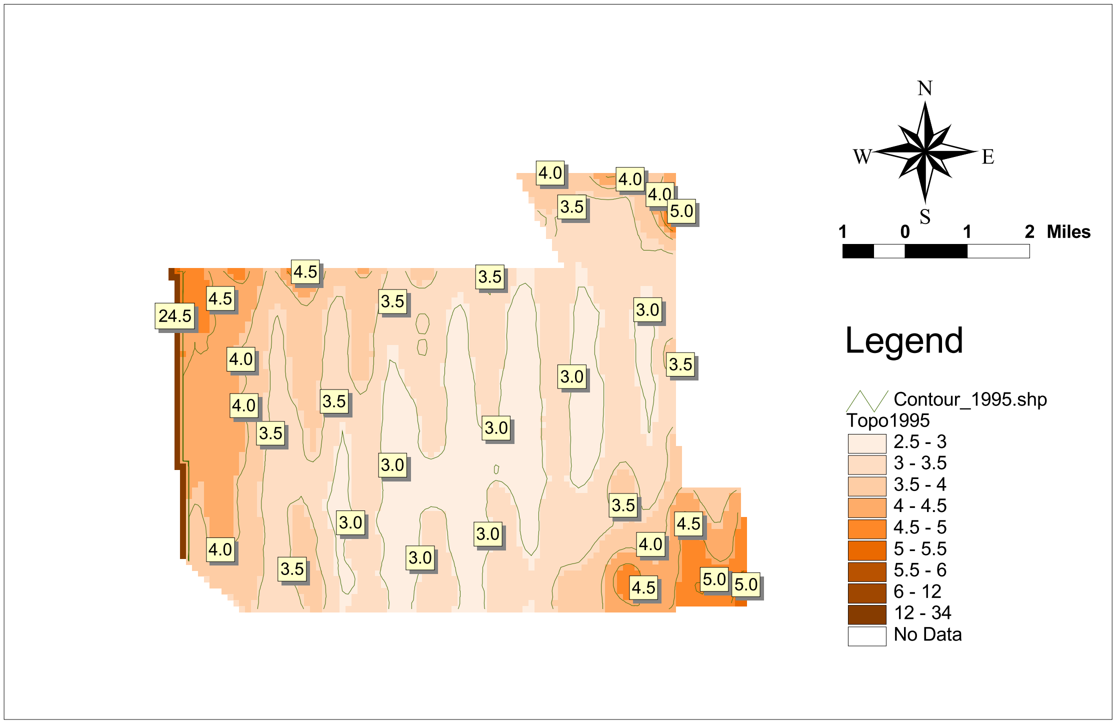


Figure 2.23 Topographic Map of C-9 West Basin for Pre-condition

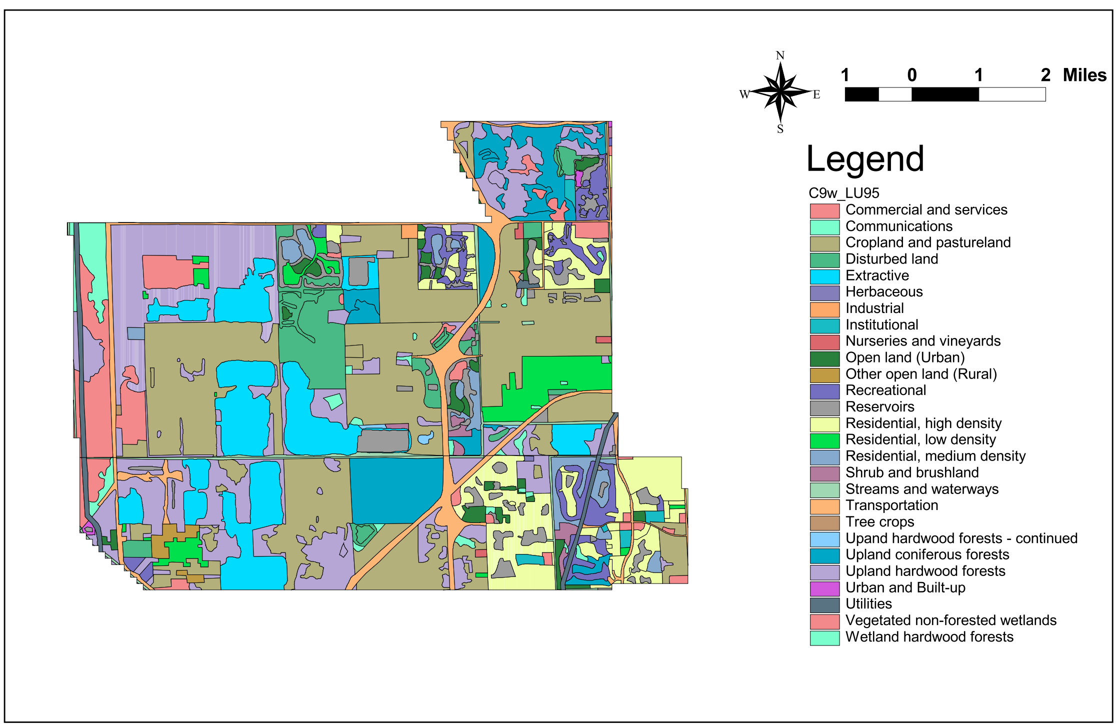


Figure 2.24 Land Use Map of C-9 West Basin for Pre-condition

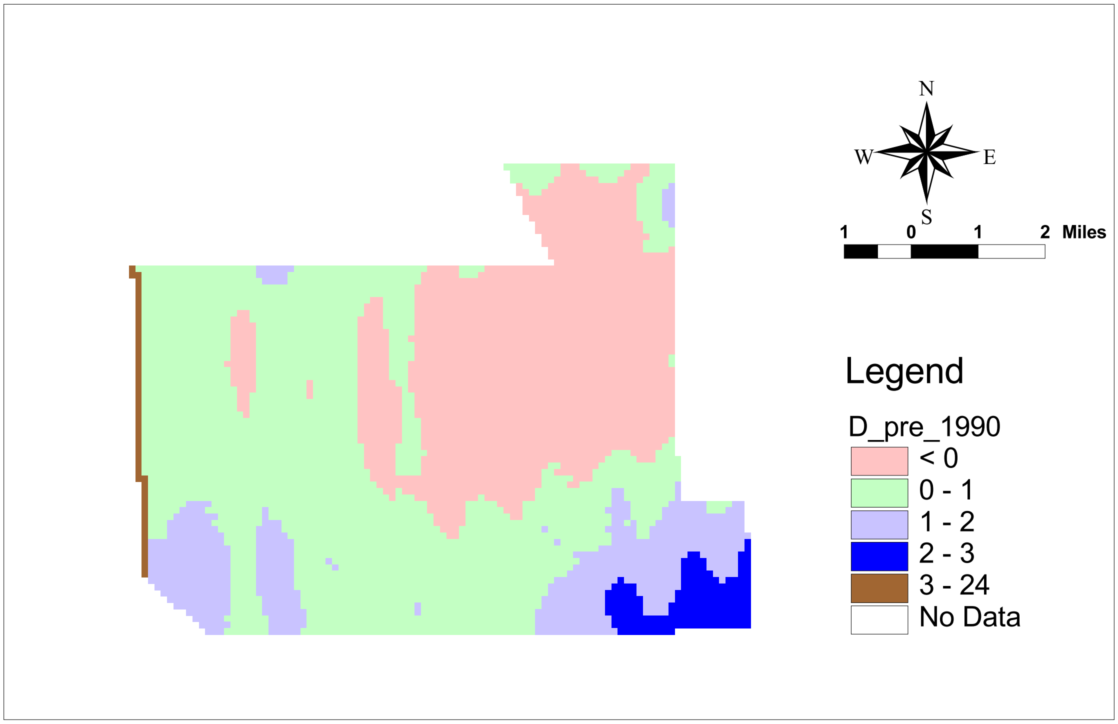


Figure 2.25 Water Table Depth of C-9 West Basin for Pre-condition on July 9, 1990

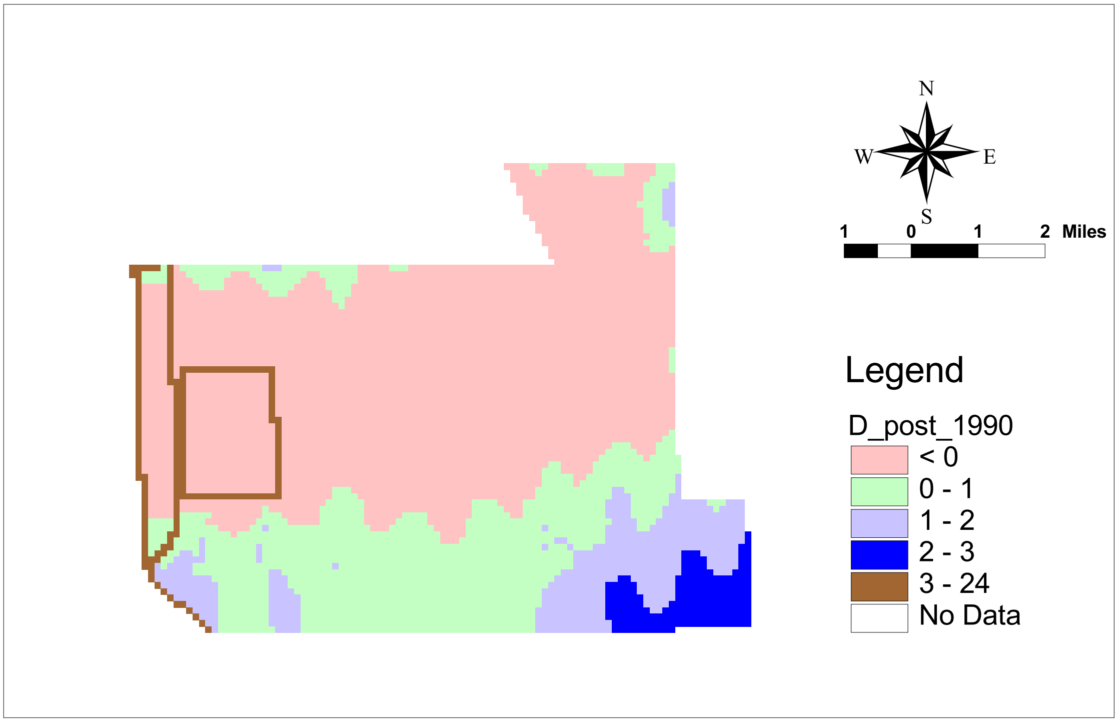


Figure 2.26 Water Table Depth of C-9 West Basin for Post-condition on July 9, 1990

- **Soil Moisture Storage**

  The soil moisture storage for the C-9 West basin was determined as described in Section 2.9.3.

- **Runoff**

  The same procedure used to compute runoff from rainfall described in Section 2.9.3 was applied for the C-9 West basin.

- **Time of Concentration**

  The equations and parameters, which were used in the computation of the time of concentration for the C-11 West basin, were used here. The travel times for sheet flow and shallow concentrated flow are calculated to be **2.0** hours and **0.85** hour, respectively. The average open channels velocity of 0.88 ft/s for C-11 West basin is also used in the canals and secondary canals of the C-9 West basin. The maximum flow length of open channels is about 10.42 miles (55,000 ft), and the travel time for open channels is calculated to be **17.36** hours from Equation 2.10. Adding these three T_c components yield a time of concentration of **20.21** hours. This T_c is assigned to each grid cell of the C-9 West basin.

- **Hydrographs**

  The procedure of hydrograph computation has been described in Section 2.9.3.

- **Seepage Rate**

  For the C-9 West basin, the seepage rats on August 31, 1988 (see Section 2.8.1) was selected as the "worst case" and divided by a factor of 20. The modified BC MODFLOW seepage rates were used into the hydrograph computation for the C-9 West basin.

2.9.5. Hydrograph Analysis

The basic hydrologic computation procedure and the basin parameters have previously been discussed for the C-11 West basin and the C-9 West basin, respectively. The hydrographs were developed under two basic cases: (1) the reduced soil moisture storage due to the increase in groundwater table elevation and with no seepage flow, and (2) the reduced soil moisture storage and with the seepage flow. The output evaluations for these two cases are discussed below.

- **Hydrograph Analysis without Seepage**

The runoff hydrographs were computed for the pre- and post- conditions to compare the runoff volume difference caused by the reduced soil moisture storage and without seepage flow, for 10-, 25-, and 100-year, 72-hour storm events, respectively.

C-11 West Basin

As shown in Table 2.9, the C-11 West basin area of 42,000 acres and 40,000 acres were used in the computation for pre-condition and post-condition, respectively. The basin area and runoff volumes are listed and classified based on the groundwater table depth difference values, as shown in Table 2.9 through Table 2.11, for the C-11 West basin for the "worst case" of November 1, 1990. The impacted area, as described in Task 1.1, include the areas of the basin with groundwater table depth difference of 0.1 ft, or greater. The graphic plots of the runoff volumes (acre-in) for the C-11 West basin are shown in Figure 2.27, Figure 2.29 and Figure 2.31 for 10-, 25-, and 100-year, 72-hour storm events. Dividing the runoff volumes by the basin area of 42,000 acres and 40,000 acres, the runoff depths (in) for pre- and post- conditions are plotted in Figure 2.28, Figure 2.30 and Figure 2.32, for 10-, 25-, and 100-year, 72-hour storm events.

The results were analyzed using runoff volume and depth, difference and percentage difference as shown in Table 2.9 through Table 2.11. Difference refers to the runoff volume difference between post- and pre- conditions, and percentage difference refers to the value of (post - pre)/pre x 100. The percentage difference values of runoff volumes in the impacted areas are -0.57%, -1.18% and -1.88%, which mean the runoff volumes for pre-condition are larger than those for post-condition, for 10-, 25-, and 100-year, 72-hour storm events, respectively. Typically, in hydrologic modeling, a percent difference of 5% ~ 10% is acceptable depending upon the quality and quantity of the data used and the methodology employed. Because the basin area for post-condition is less than the basin area for pre-condition, the runoff volume into the C-11 canal for post-condition does not exceed that for pre-condition even though the runoff depth for post-condition is greater than for pre-condition (see Figure 2.28, Figure 2.30, and Figure 2.32). The reduced basin area overrides the increased groundwater stage in runoff volume calculation for post-condition. It is therefore concluded that no increases on runoff volumes are introduced to the impacted areas by applying the reduced soil moisture storage and without seepage in the C-11 West basin.

C-9 West Basin

The basin area and runoff volumes are listed and classified based on the groundwater table depth difference, as shown in Table 2.12 through Table 2.14, for the C-9 West basin on the "worst case" of July 9, 1990. The graphic plots of the runoff volumes and runoff depths for the C-9 West basin are shown in Figure 2.33 through Figure 2.38 for 10-, 25-, and 100-year, 72-hour storm events. The percentage difference values of runoff volumes in the impacted areas are -11.56%, -11.65% and -11.74%, which mean the runoff volumes for pre-condition are greater than for post-condition, for 10-, 25-, and 100-year, 72-hour storm events. Because the basin area for post-condition is less than the basin area for pre-condition, the runoff volume into the C-9 canal for post-condition does not exceed that for pre-condition even though the runoff depth for post-condition is greater than for pre-condition (see Figure 2.34, Figure 2.36, and Figure 2.38). The reduced basin area overrides the increased groundwater stage in runoff volume calculation for post-condition. It is therefore concluded that no increases on runoff volumes are introduced to the impacted areas by applying the reduced soil moisture storage without seepage for the C-9 West basin.

Table 2.9 Runoff Volume for 10-yr Storm without Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	30801.54	29964.56	-836.98	-2.72
0.1 - 0.2	2927	2927.00	27001.82	27453.45	451.63	1.67
0.2 - 0.3	6760.79	6760.79	60878.82	62898.86	2020.05	3.32
0.3 - 0.4	15765.61	15765.61	137232.48	144028.84	6796.36	4.95
0.4 - 0.5	4384.76	4384.76	40884.69	42758.18	1873.49	4.58
0.5 - 1.0	4734.85	4700.41	44746.09	46973.90	2227.82	4.98
> 1.0	4212.58	2364.55	39463.13	24084.39	-15378.74	-38.97
Total Area	41942.15	39962.12	381008.47	378160.12	-2848.34	-0.75
Impacted Area	38785.59	36903.12	350207.03	348197.62	-2009.41	-0.57

Table 2.10 Runoff Volume for 25-yr Storm without Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	36467.04	35456.98	-1010.06	-2.77
0.1 - 0.2	2927	2927.00	32218.85	32683.60	464.75	1.44
0.2 - 0.3	6760.79	6760.79	72881.51	74965.76	2084.25	2.86
0.3 - 0.4	15765.61	15765.61	165004.53	172067.00	7062.47	4.28
0.4 - 0.5	4384.76	4384.76	48730.14	50635.88	1905.73	3.91
0.5 - 1.0	4734.85	4700.41	53232.14	55430.25	2198.12	4.13
> 1.0	4212.58	2364.55	46999.74	28340.68	-18659.06	-39.70
Total Area	41942.15	39962.12	455534.97	449580.16	-5954.81	-1.31
Impacted Area	38785.59	36903.12	419066.91	414123.17	-4943.74	-1.18

Table 2.11 Runoff Volume for 100-yr Storm without Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	45917.62	44617.85	-1299.77	-2.83
0.1 - 0.2	2927	2927.00	40938.57	41419.16	480.59	1.17
0.2 - 0.3	6760.79	6760.79	92964.45	95126.67	2162.22	2.33
0.3 - 0.4	15765.61	15765.61	211570.41	218958.55	7388.14	3.49
0.4 - 0.5	4384.76	4384.76	61829.41	63772.86	1943.45	3.14
0.5 - 1.0	4734.85	4700.41	67394.03	69526.48	2132.45	3.16
> 1.0	4212.58	2364.55	59583.43	35434.21	-24149.22	-40.53
Total Area	41942.15	39962.12	580196.19	568851.62	-11344.56	-1.96
Impacted Area	38785.59	36903.12	534280.3	524237.93	-10042.37	-1.88

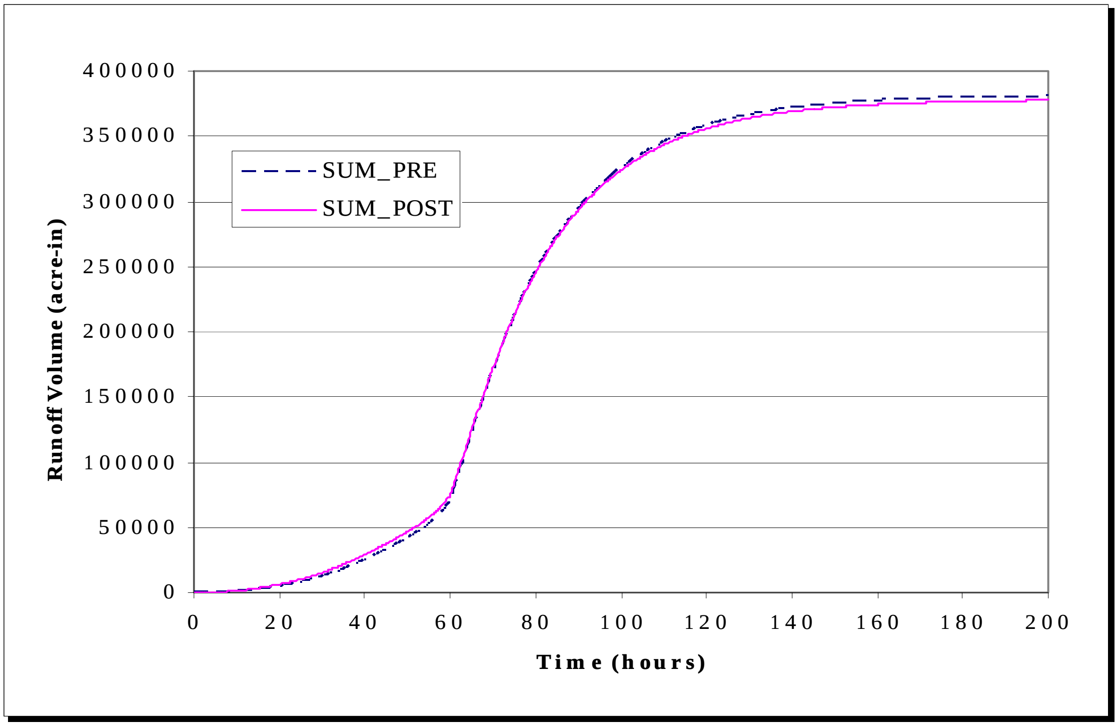


Figure 2.27 Accumulated Runoff Volume for 10-yr Storm of C-11 West Basin without Seepage

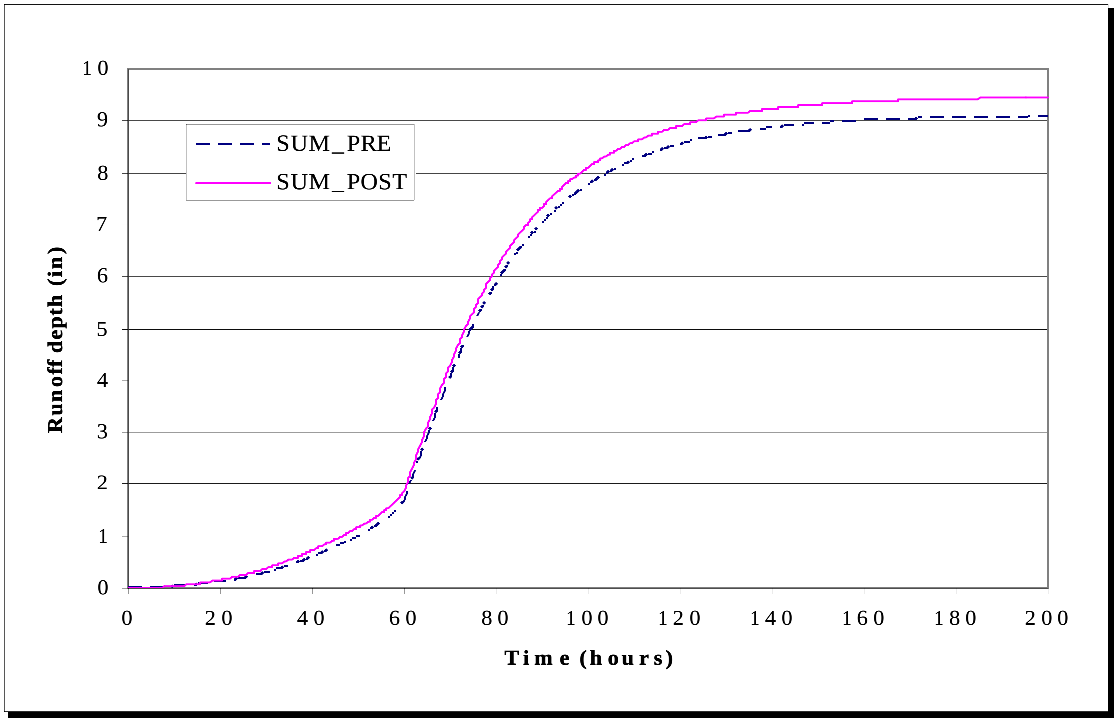


Figure 2.28 Accumulated Runoff Depth for 10-yr Storm of C-11 West Basin without Seepage

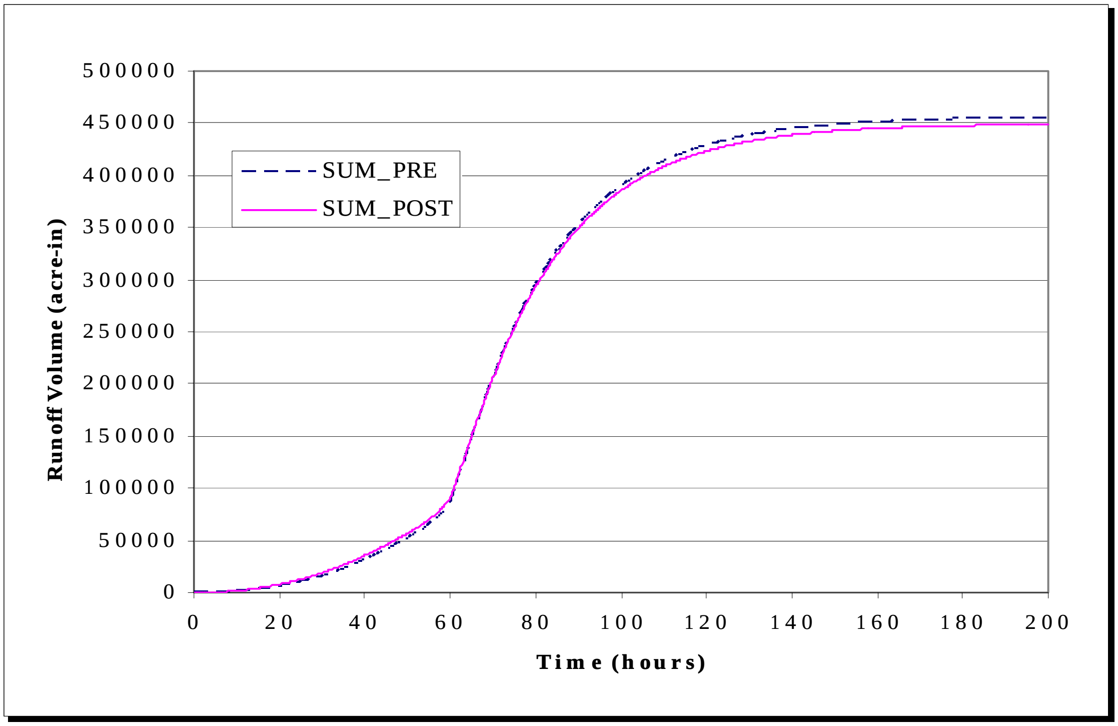


Figure 2.29 Accumulated Runoff Volume for 25-yr Storm of C-11 West Basin without Seepage

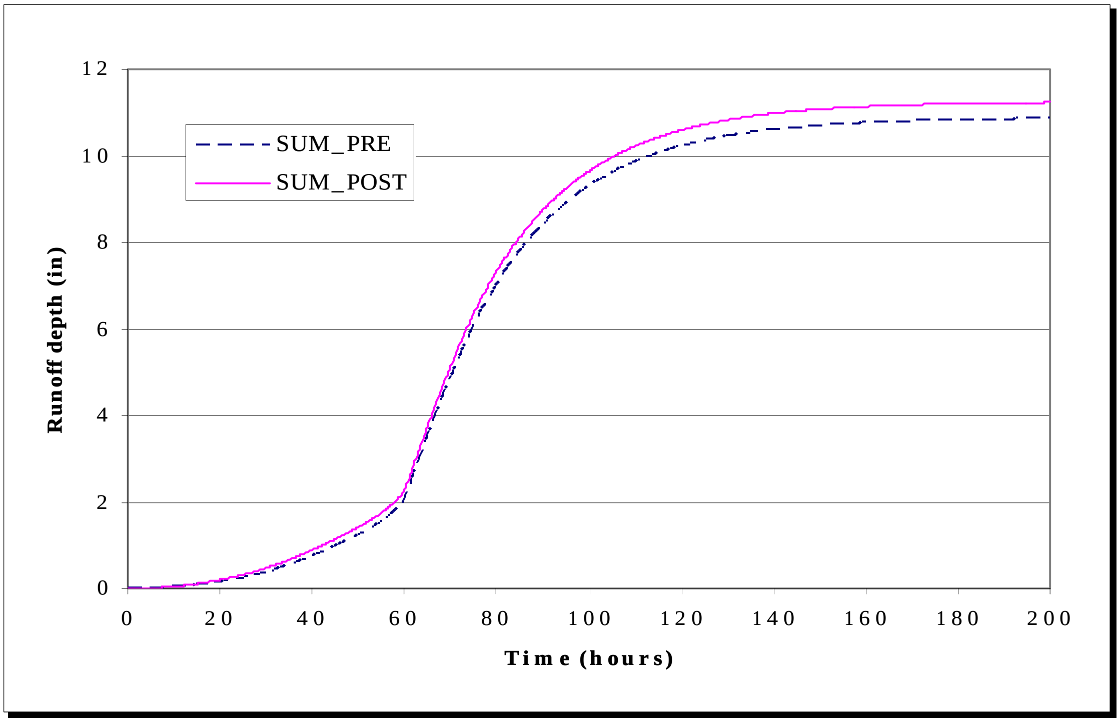


Figure 2.30 Accumulated Runoff Depth for 25-yr Storm of C-11 West Basin without Seepage

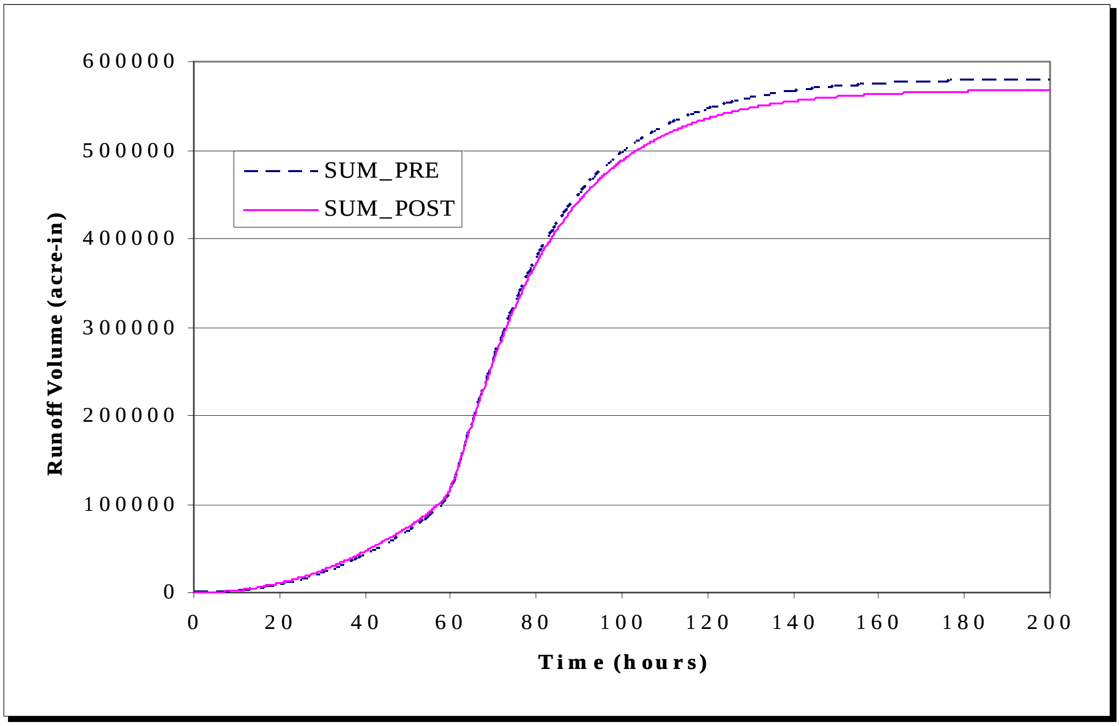


Figure 2.31 Accumulated Runoff Volume for 100-yr Storm of C-11 West Basin without Seepage

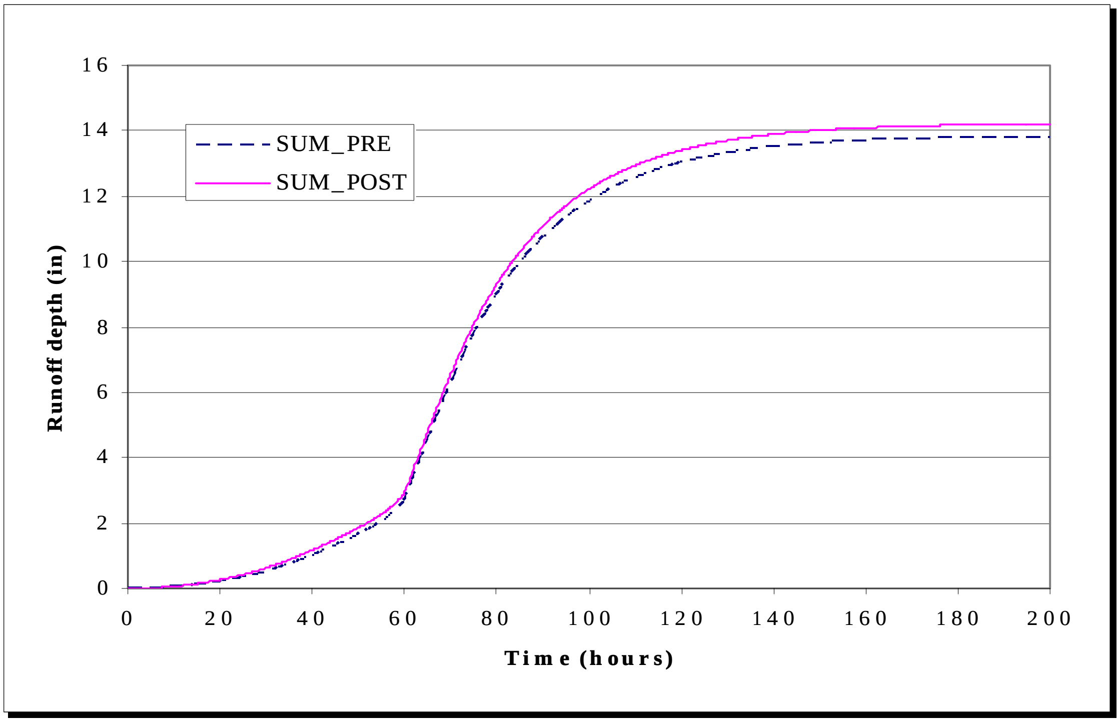


Figure 2.32 Accumulated Runoff Depth for 100-yr Storm of C-11 West Basin without Seepage

Table 2.12 Runoff Volume for 10-yr Storm without Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	140892.22	140629.30	-262.92	-0.19
0.1 - 0.2	3018.82	3018.82	30736.54	30834.96	98.42	0.32
0.2 - 0.3	2410.47	2410.47	24588.45	24684.47	96.02	0.39
0.3 - 0.4	1543.85	1543.85	15757.14	15834.57	77.44	0.49
0.4 - 0.5	1136.36	1136.36	11594.70	11671.43	76.74	0.66
0.5 - 1.0	3787.88	3787.88	38549.08	38966.21	417.12	1.08
> 1.0	3690.31	1807.85	37758.90	18620.56	-19138.34	-50.69
Total Area	29683.2	27800.73	299876.44	281241.09	-18635.34	-6.21
Impacted Area	15587.69	13705.23	158984.81	140612.2	-18372.61	-11.56

Table 2.13 Runoff Volume for 25-yr Storm without Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	171830.89	171554.42	-276.47	-0.16
0.1 - 0.2	3018.82	3018.82	37375.29	37474.83	99.54	0.27
0.2 - 0.3	2410.47	2410.47	29889.50	29986.71	97.21	0.33
0.3 - 0.4	1543.85	1543.85	19152.52	19230.75	78.23	0.41
0.4 - 0.5	1136.36	1136.36	14094.00	14171.33	77.34	0.55
0.5 - 1.0	3787.88	3787.88	46879.98	47299.34	419.36	0.89
> 1.0	3690.31	1807.85	45876.89	22597.84	-23279.05	-50.74
Total Area	29683.2	27800.73	365095.97	342310.94	-22785.03	-6.24
Impacted Area	15587.69	13705.23	193268.18	170760.8	-22507.38	-11.65

Table 2.14 Runoff Volume for 100-yr Storm without Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	221090.58	220798.81	-291.77	-0.13
0.1 - 0.2	3018.82	3018.82	47938.47	48039.18	100.71	0.21
0.2 - 0.3	2410.47	2410.47	38324.14	38422.60	98.46	0.26
0.3 - 0.4	1543.85	1543.85	24554.83	24633.90	79.07	0.32
0.4 - 0.5	1136.36	1136.36	18070.53	18148.45	77.93	0.43
0.5 - 1.0	3787.88	3787.88	60135.27	60557.31	422.04	0.70
> 1.0	3690.31	1807.85	58792.41	28925.36	-29867.05	-50.80
Total Area	29683.2	27800.73	468900.16	439520.19	-29379.97	-6.27
Impacted Area	15587.69	13705.23	247815.65	218726.8	-29088.85	-11.74

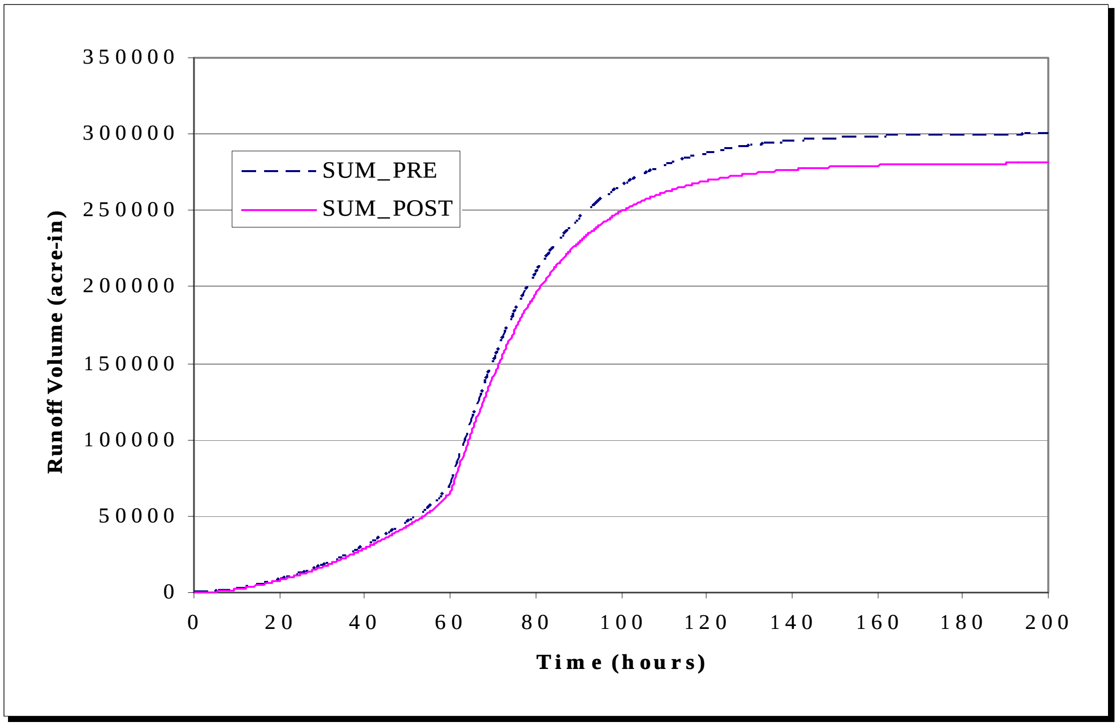


Figure 2.33 Accumulated Runoff Volume for 10-yr Storm of C-9 West Basin without Seepage

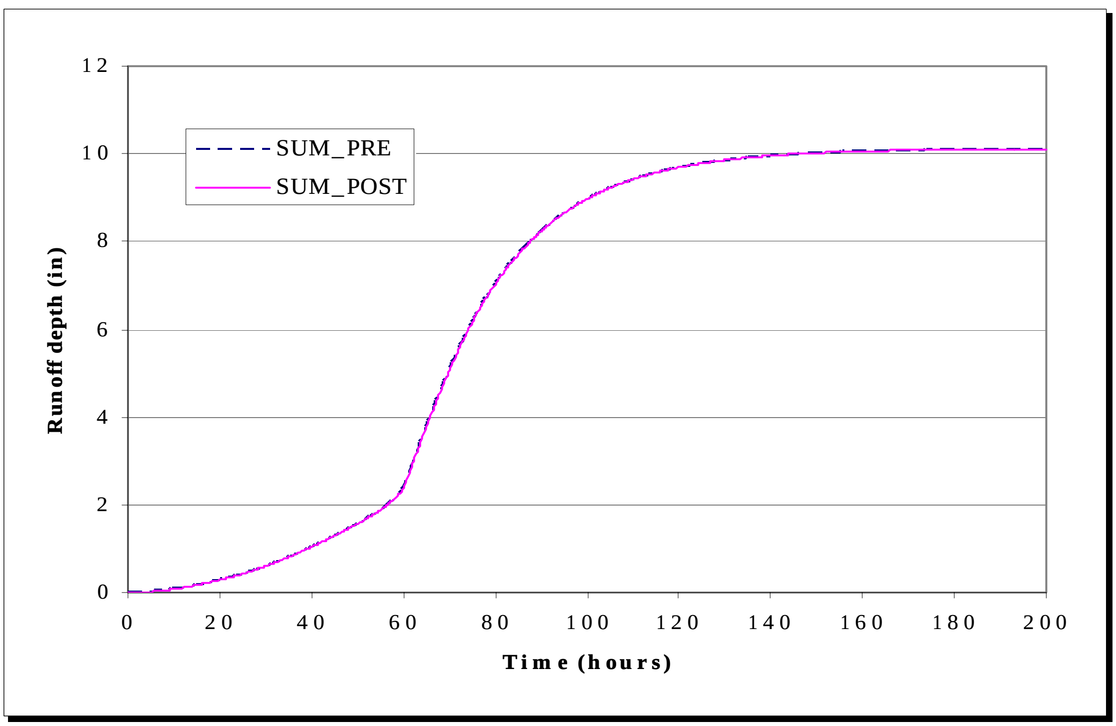


Figure 2.34 Accumulated Runoff Depth for 10-yr Storm of C-9 West Basin without Seepage

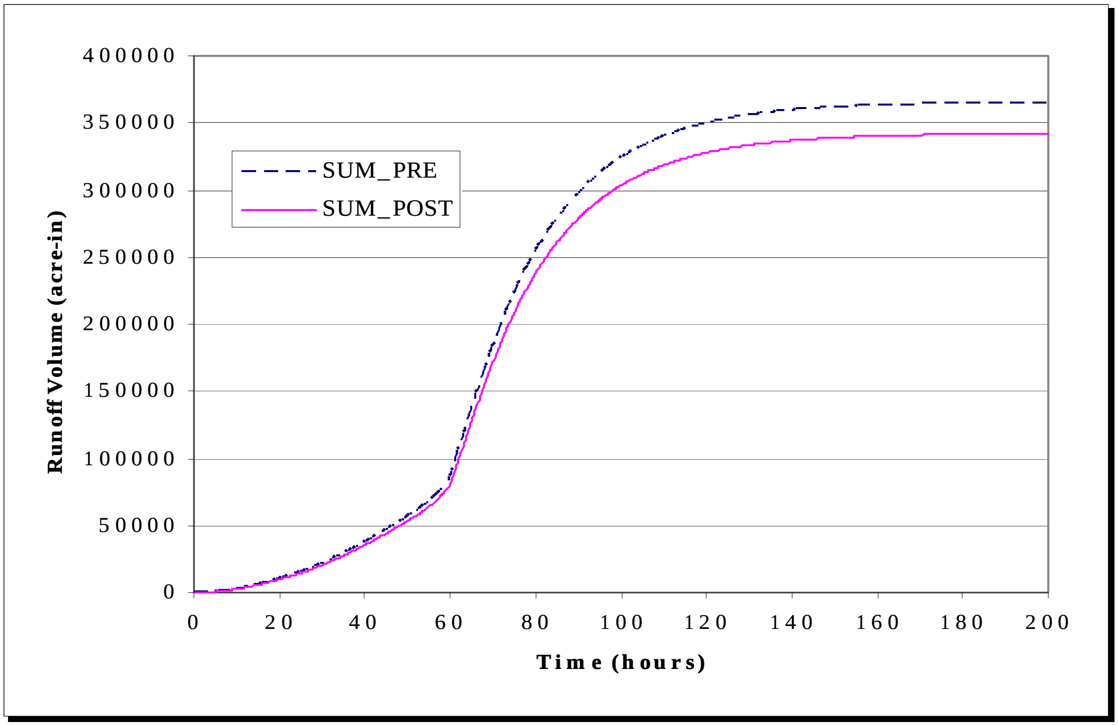


Figure 2.35 Accumulated Runoff Volume for 25-yr Storm of C-9 West Basin without Seepage

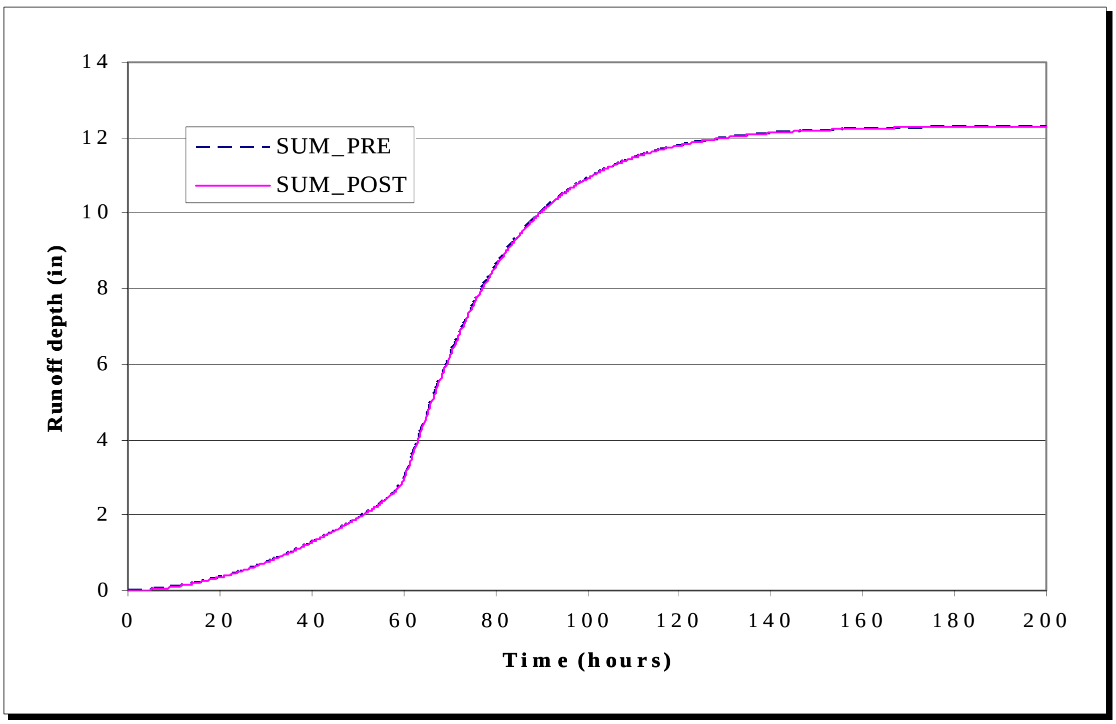


Figure 2.36 Accumulated Runoff Depth for 25-yr Storm of C-9 West Basin without Seepage

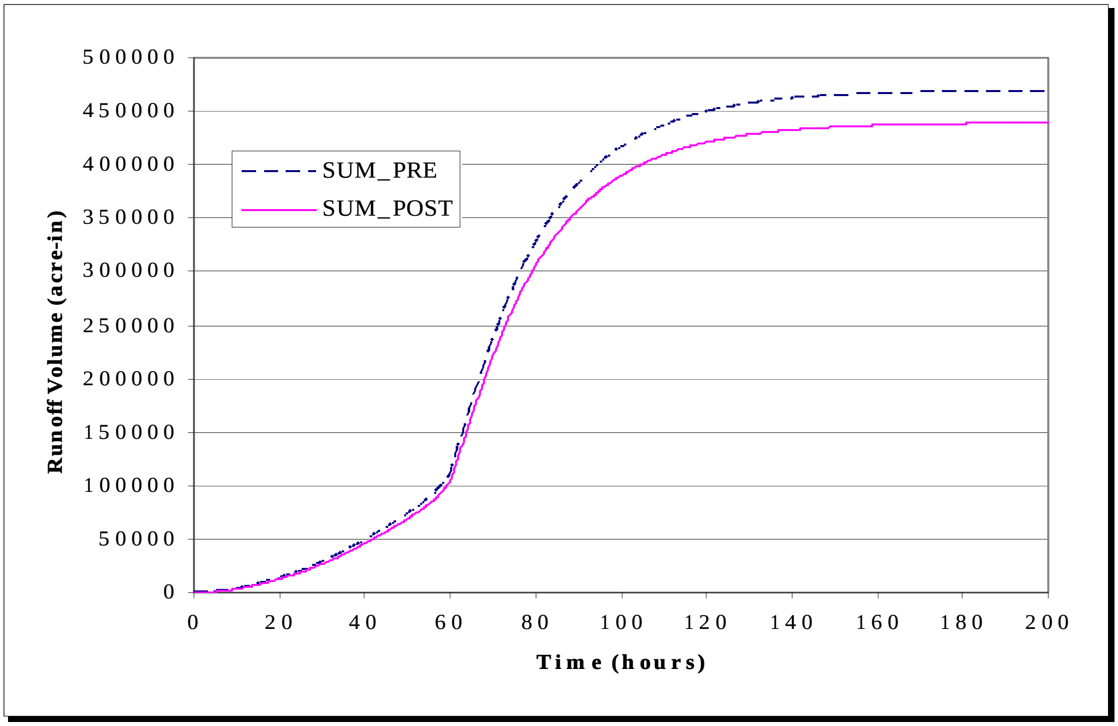


Figure 2.37 Accumulated Runoff Volume for 100-yr Storm of C-9 West Basin without Seepage

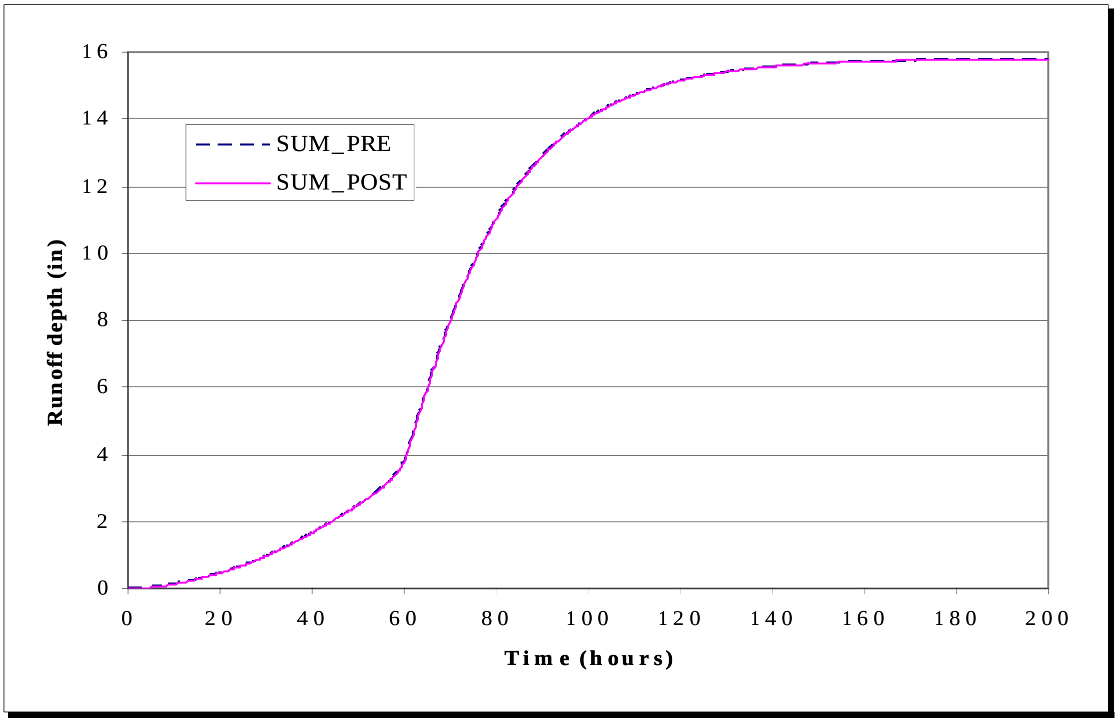


Figure 2.38 Accumulated Runoff Depth for 100-yr Storm of C-9 West Basin without Seepage

- **Hydrograph Analysis with Seepage**

The runoff hydrographs in impacted areas were computed for pre- and post- conditions to compare the runoff volume difference caused by the decreased soil moisture storage with seepage, for 10-, 25-, and 100-year, 72-hour storm events, respectively. The seepage rate was assumed to be constant throughout the hydrograph duration of 200 hours. As discussed in Section 2.8.3, the seepage rates were obtained by dividing the BC MODFLOW seepage rates by a factor of 20.

C-11 West Basin

As described in Section 2.9.3, the modified BC MODFLOW seepage rates on November 30, 1990 were selected as the "worst case" which results in the greatest increase in storm runoff volume along with the effects of groundwater stage increase on runoff. The seepage rates for pre- and post- conditions are listed in Table 2.15 for the C-11 West basin. Seepage difference refers to (post - pre), and percentage difference refers to (post - pre)/pre x 100, as shown in Table 2.15. Note that the seepage rate in the impact area for pre-condition is negative, which means the surface water leaked into the aquifer from the canal.

Table 2.15 Seepage Rates of C-11 West Basin on November 30, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	Seep_Pre (cfs)	Seep_Post (cfs)	Seep_Diff (cfs)	Seep_Diff (%)
< 0.1	3156.57	3059.00	37.53	42.14	4.61	12.28
0.1 - 0.2	2927	2927.00	0.10	2.15	2.05	2089.13
0.2 - 0.3	6760.79	6760.79	-19.96	-5.12	14.84	-
0.3 - 0.4	15765.61	15765.61	-62.19	25.90	88.10	-
0.4 - 0.5	4384.76	4384.76	0.04	35.80	35.76	90157.06
0.5 - 1.0	4734.85	4700.41	5.96	40.61	34.64	581.27
> 1.0	4212.58	2364.55	11.74	14.15	2.41	20.53
Total Area	41942.15	39962.12	-26.78	155.63	182.41	-
Impacted Area	38785.59	36903.12	-64.31	113.49	177.80	-

The total runoff volumes were updated with the seepage rates in Table 2.15, and classified based on the groundwater table depth difference values, as shown in Table 2.17 through Table 2.19, for the C-11 West basin on November 1, 1990 (November 1, 1990 corresponds to the maximum groundwater difference). The graphic plots of the runoff volumes and runoff depths for the C-11 West basin are shown in Figure 2.39 through Figure 2.44 for 10-, 25-, and 100-year, 72-hour storm events.

The results were analyzed using runoff volume, difference and percentage difference in Table 2.17 through Table 2.19. Difference refers to the runoff volume difference between post- and pre- conditions, and percentage difference refers to the value of (post – pre)/pre x 100. The percentage difference values of runoff volumes in the impacted areas are 9.86 %, 7.46%, and 4.84%, for 10-, 25-, and 100-year, 72-hour storm events, respectively. Comparing the results of the hydrograph computation without seepage, about 10% increases of runoff volumes are introduced to the impacted areas due to the increased seepage into the C-11 canal. The runoff volumes for post-condition exceeds that for pre-condition with seepage, but the percentage differences between post-condition and pre-condition are less than 10% and may be acceptable in term of the flood protection.

C-9 West Basin

For the C-9 West basin, the modified BC MODFLOW seepage rates on August 31, 1988 (see Section 2.9.4) were selected as the “worst case” and were used into the hydrograph computation. The seepage rates for pre- and post- conditions are listed in Table 2.16 for this basin.

Table 2.16 Seepage Rates of C-9 West Basin on August 31, 1988

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	Seep_Pre (cfs)	Seep_Post (cfs)	Seep_Diff (cfs)	Seep_Diff (%)
< 0.1	14095.5	14095.50	150.41	158.86	8.45	5.62
0.1 - 0.2	3018.82	3018.82	21.94	29.87	7.93	36.15
0.2 - 0.3	2410.47	2410.47	14.78	20.95	6.17	41.71
0.3 - 0.4	1543.85	1543.85	14.85	19.88	5.03	33.89
0.4 - 0.5	1136.36	1136.36	6.21	10.92	4.72	76.00
0.5 - 1.0	3787.88	3787.88	24.03	50.58	26.56	110.53
> 1.0	3690.31	1807.85	12.20	28.10	15.90	130.33
Total Area	29683.2	27800.73	244.41	319.16	74.75	30.58
Impacted Area	15587.69	13705.23	94.01	160.3	66.29	70.51

The updated runoff volumes are listed in Table 2.20 through Table 2.22, for the C-9 West basin on July 9, 1990. The graphic plots of the runoff volumes and runoff depths for the C-9 West basin are shown in Figure 2.45 through Figure 2.50, for 10-, 25-, and 100-year, 72-hour storm events, respectively.

The percentage difference values of runoff volumes with seepage in the impacted areas are -2.94%, -4.42% and -5.98%, which indicate that the runoff volumes for post-condition are less than for pre-condition, for 10-, 25-, and 100-year, 72-hour storm events, respectively.

2.9.6. Conclusions

Runoff hydrograph computations for the C-11 West and C-9 West basins were performed for three design storm frequencies (10-, 25-, and 100-year) using a widely accepted Santa Barbara Unit Hydrograph. Without seepage, no increases in runoff volumes occurred between pre- and post- conditions as a result of the groundwater stage increases. When seepage was applied, the changes in runoff volume (post - pre) were less than 10% for the C-11 West basin and may be considered insignificant given the uncertainties in the hydrologic data and the model employed. For the C-9 West basin, the changes in runoff volume were negative (post - pre) and indicates the changes are insignificant. It may be noted that the seepage rates used in runoff computations were derived by dividing the BC MODFLOW seepage values by a factor of 20. This factor was established by comparing the SEEP2D and BC MODFLOW seepage results and by establishing the accuracy of SEEP2D and the limitations of the BC MODFLOW.

Table 2.17 Runoff Volume for 10-yr Storm with Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	38245.32	38322.68	77.36	0.20
0.1 - 0.2	2927	2927.00	27021.31	27880.14	858.83	3.18
0.2 - 0.3	6760.79	6760.79	56920.76	61883.39	4962.62	8.72
0.3 - 0.4	15765.61	15765.61	124896.10	149166.27	24270.16	19.43
0.4 - 0.5	4384.76	4384.76	40892.57	49858.57	8966.00	21.93
0.5 - 1.0	4734.85	4700.41	45928.25	55027.79	9099.54	19.81
> 1.0	4212.58	2364.55	41791.72	26890.93	-14900.79	-35.65
Total Area	41942.15	39962.12	375696.09	409028.59	33332.50	8.87
Impacted Area	38785.59	36903.12	337450.71	370707.09	33256.38	9.86

Table 2.18 Runoff Volume for 25-yr Storm with Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	43910.83	43815.11	-95.72	-0.22
0.1 - 0.2	2927	2927.00	32238.34	33110.29	871.96	2.70
0.2 - 0.3	6760.79	6760.79	68923.48	73950.30	5026.82	7.29
0.3 - 0.4	15765.61	15765.61	152668.30	177204.36	24536.06	16.07
0.4 - 0.5	4384.76	4384.76	48738.03	57736.27	8998.24	18.46
0.5 - 1.0	4734.85	4700.41	54414.30	63484.16	9069.86	16.67
> 1.0	4212.58	2364.55	49328.36	31147.26	-18181.10	-36.86
Total Area	41942.15	39962.12	450222.72	480448.16	30225.44	6.71
Impacted Area	38785.59	36903.12	406310.81	436632.64	30321.83	7.46

Table 2.19 Runoff Volume for 100-yr Storm with Seepage of C-11 West Basin on November 1, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	3156.57	3059.00	53361.35	52975.97	-385.38	-0.72
0.1 - 0.2	2927	2927.00	40958.06	41845.85	887.79	2.17
0.2 - 0.3	6760.79	6760.79	89006.43	94111.17	5104.74	5.74
0.3 - 0.4	15765.61	15765.61	199234.30	224095.69	24861.39	12.48
0.4 - 0.5	4384.76	4384.76	61837.29	70873.27	9035.98	14.61
0.5 - 1.0	4734.85	4700.41	68576.22	77580.32	9004.10	13.13
> 1.0	4212.58	2364.55	61911.99	38240.83	-23671.16	-38.23
Total Area	41942.15	39962.12	574884.38	599721.44	24837.06	4.32
Impacted Area	38785.59	36903.12	521524.29	546747.13	25222.84	4.84

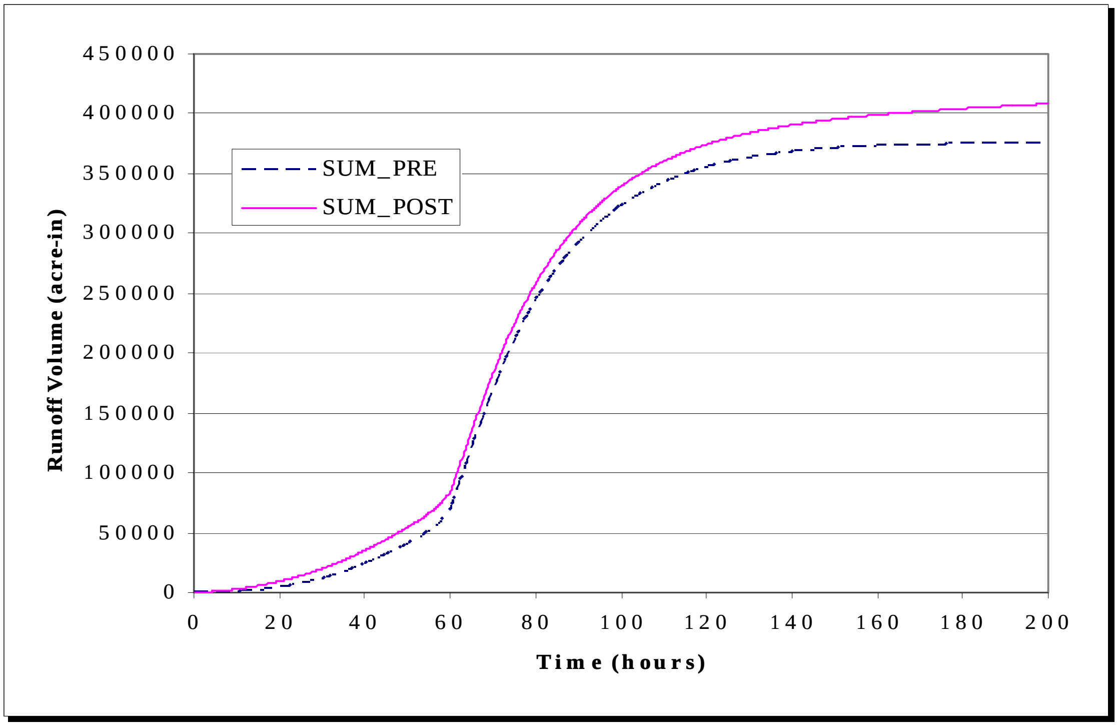


Figure 2.39 Accumulated Runoff Volume for 10-yr Storm of C-11 West Basin with Seepage

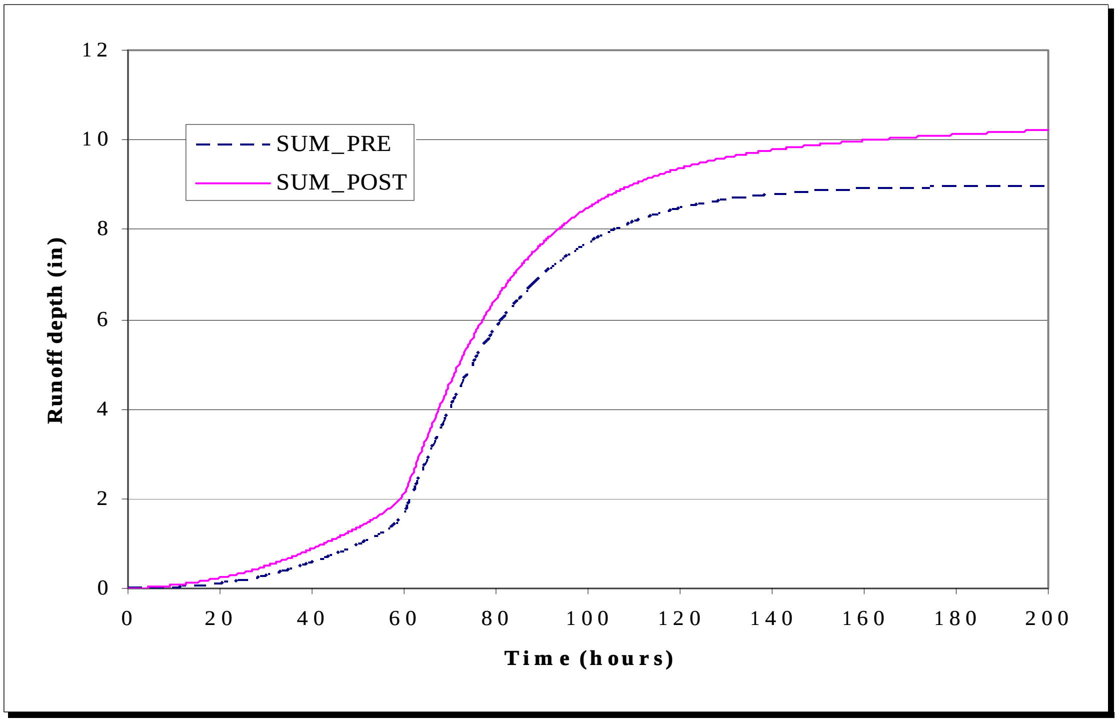


Figure 2.40 Accumulated Runoff Depth for 10-yr Storm of C-11 West Basin with Seepage

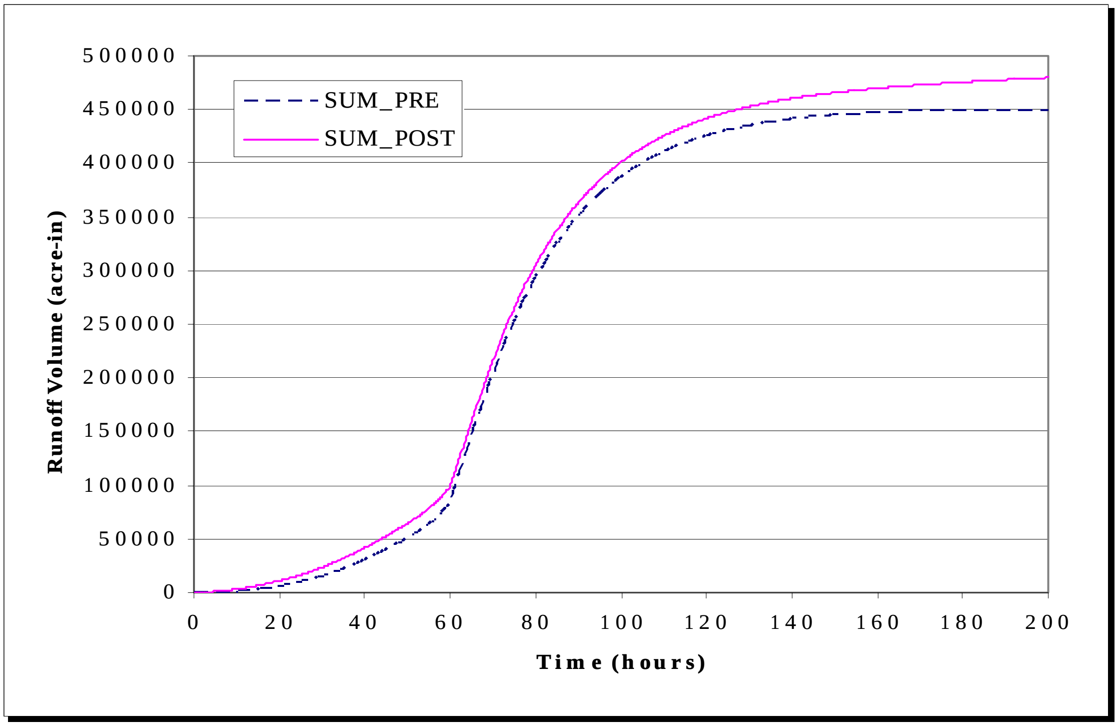


Figure 2.41 Accumulated Runoff Volume for 25-yr Storm of C-11 West Basin with Seepage

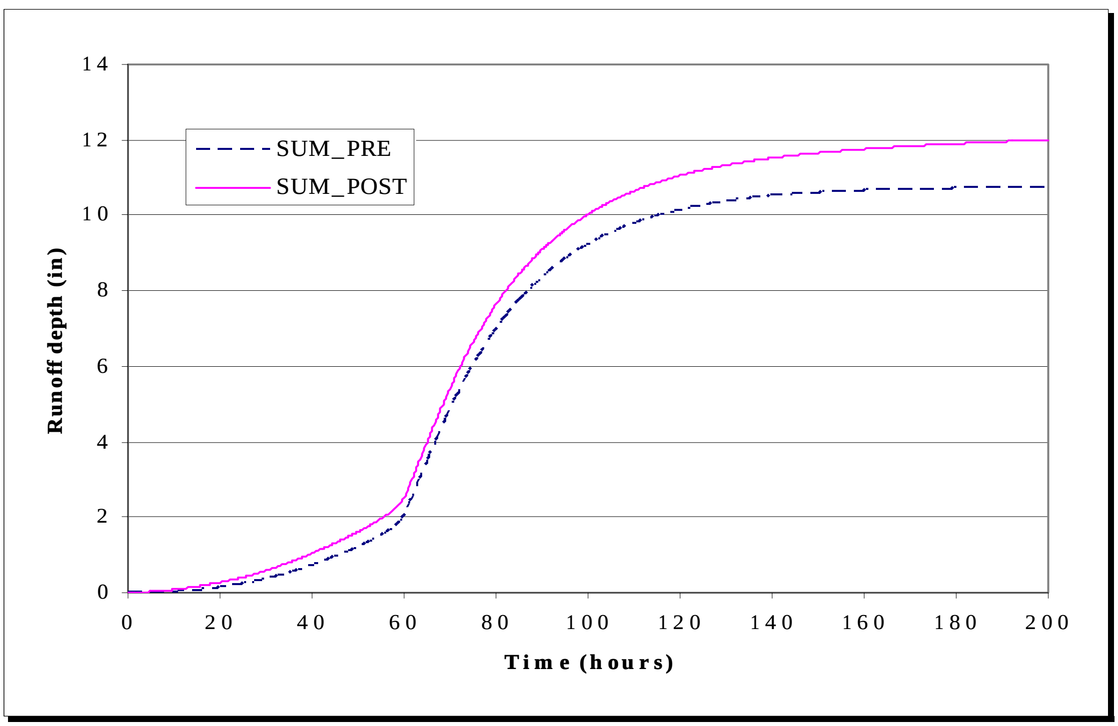


Figure 2.42 Accumulated Runoff Depth for 25-yr Storm of C-11 West Basin with Seepage

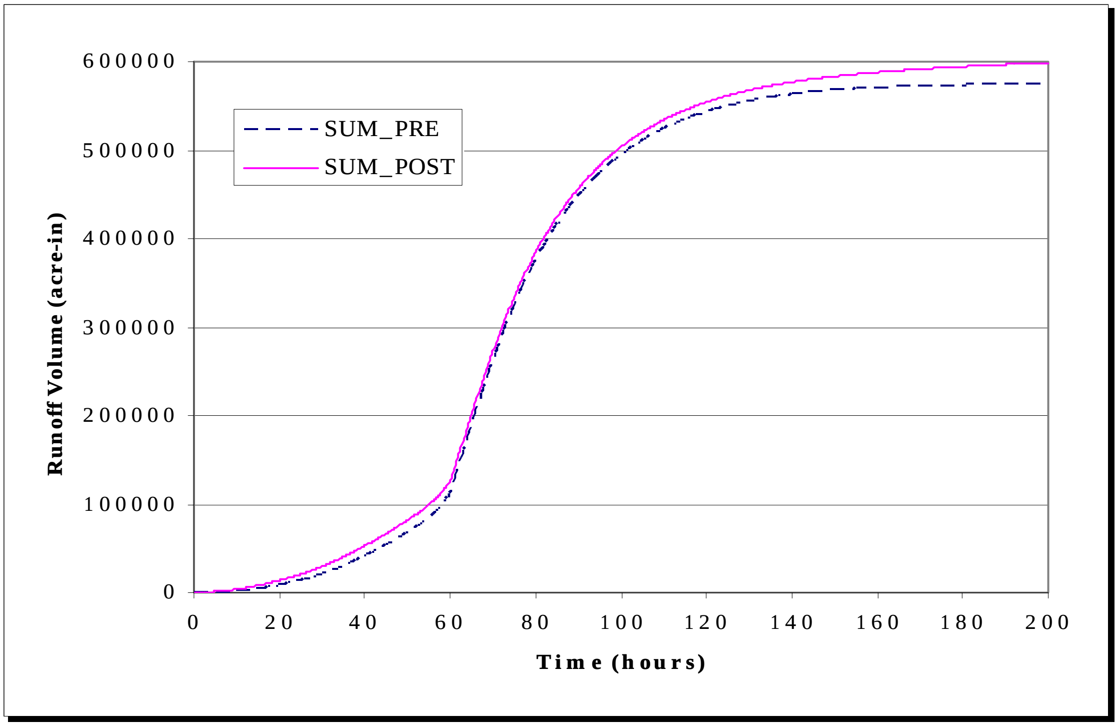


Figure 2.43 Accumulated Runoff Volume for 100-yr Storm of C-11 West Basin with Seepage

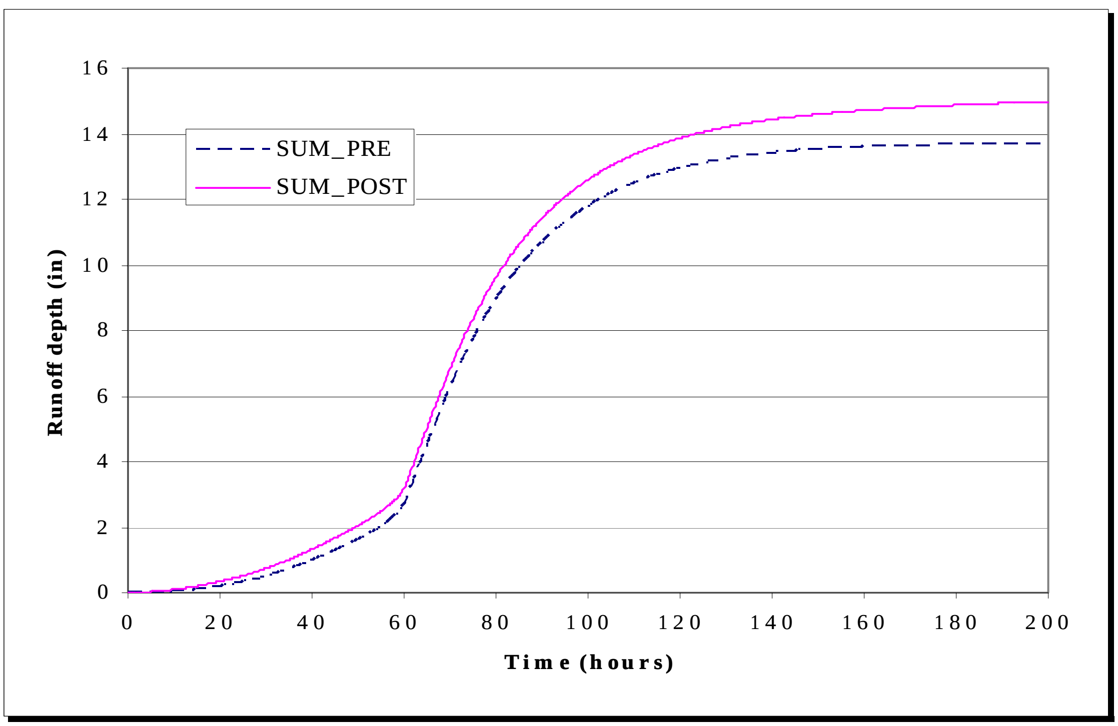


Figure 2.44 Accumulated Runoff Depth for 100-yr Storm of C-11 West Basin with Seepage

Table 2.20 Runoff Volume for 10-yr Storm with Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	170725.59	172139.23	1413.64	0.83
0.1 - 0.2	3018.82	3018.82	35088.59	36760.07	1671.48	4.76
0.2 - 0.3	2410.47	2410.47	27520.92	28840.00	1319.08	4.79
0.3 - 0.4	1543.85	1543.85	18701.81	19777.11	1075.30	5.75
0.4 - 0.5	1136.36	1136.36	12825.64	13837.86	1012.22	7.89
0.5 - 1.0	3787.88	3787.88	43314.65	48998.91	5684.27	13.12
> 1.0	3690.31	1807.85	40178.55	24193.68	-15984.87	-39.78
Total Area	29683.2	27800.73	348354.81	344545.84	-3808.97	-1.09
Impacted Area	15587.69	13705.23	177630.16	172407.63	-5222.53	-2.94

Table 2.21 Runoff Volume for 25-yr Storm with Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	201664.09	203064.59	1400.50	0.69
0.1 - 0.2	3018.82	3018.82	41727.31	43399.93	1672.62	4.01
0.2 - 0.3	2410.47	2410.47	32821.97	34142.25	1320.27	4.02
0.3 - 0.4	1543.85	1543.85	22097.17	23173.28	1076.10	4.87
0.4 - 0.5	1136.36	1136.36	15324.93	16337.75	1012.82	6.61
0.5 - 1.0	3787.88	3787.88	51645.50	57331.98	5686.47	11.01
> 1.0	3690.31	1807.85	48296.52	28170.98	-20125.54	-41.67
Total Area	29683.2	27800.73	413575.03	405617.81	-7957.22	-1.92
Impacted Area	15587.69	13705.23	211913.4	202556.17	-9357.23	-4.42

Table 2.22 Runoff Volume for 100-yr Storm with Seepage of C-9 West Basin on July 9, 1990

Water Depth Diff (ft)	Area_Pre (acres)	Area_Post (acres)	V_Pre (acre-in)	V_Post (acre-in)	V_Diff (acre-in)	V_Diff (%)
< 0.1	14095.5	14095.50	250923.41	252308.58	1385.17	0.55
0.1 - 0.2	3018.82	3018.82	52290.51	53964.31	1673.80	3.20
0.2 - 0.3	2410.47	2410.47	41256.62	42578.14	1321.52	3.20
0.3 - 0.4	1543.85	1543.85	27499.51	28576.45	1076.94	3.92
0.4 - 0.5	1136.36	1136.36	19301.47	20314.89	1013.43	5.25
0.5 - 1.0	3787.88	3787.88	64900.78	70589.98	5689.20	8.77
> 1.0	3690.31	1807.85	61212.05	34498.47	-26713.57	-43.64
Total Area	29683.2	27800.73	517378.00	502824.09	-14553.91	-2.81
Impacted Area	15587.69	13705.23	266460.94	250522.24	-15938.70	-5.98

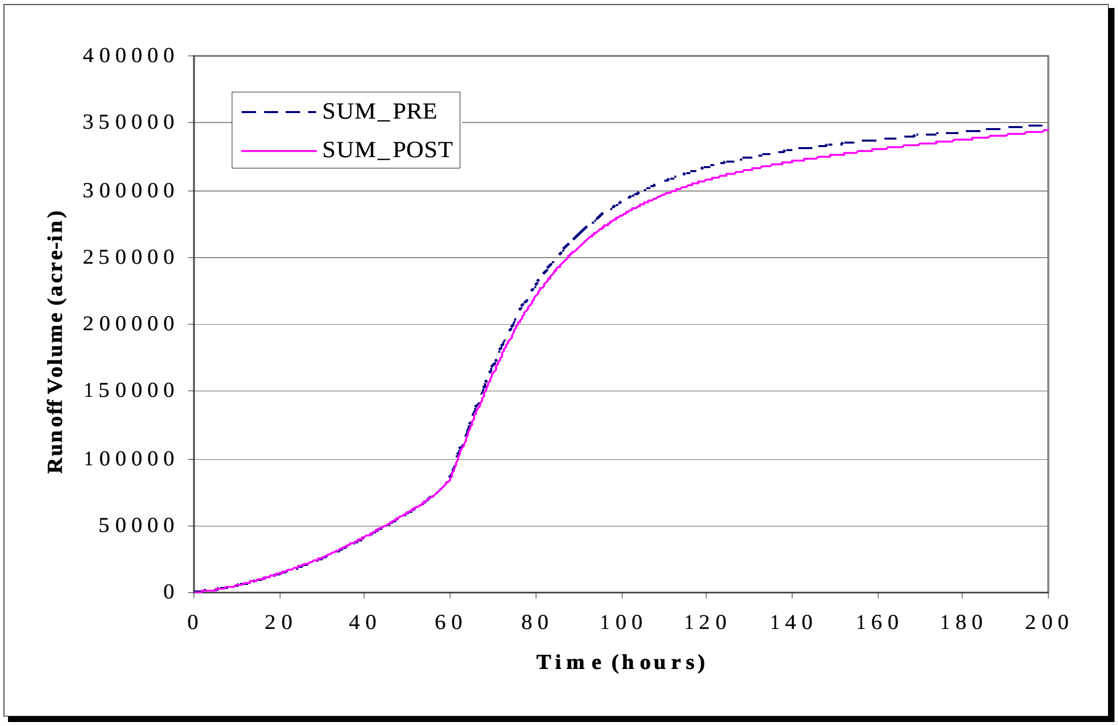


Figure 2.45 Accumulated Runoff Volume for 10-yr Storm of C-9 West Basin with Seepage

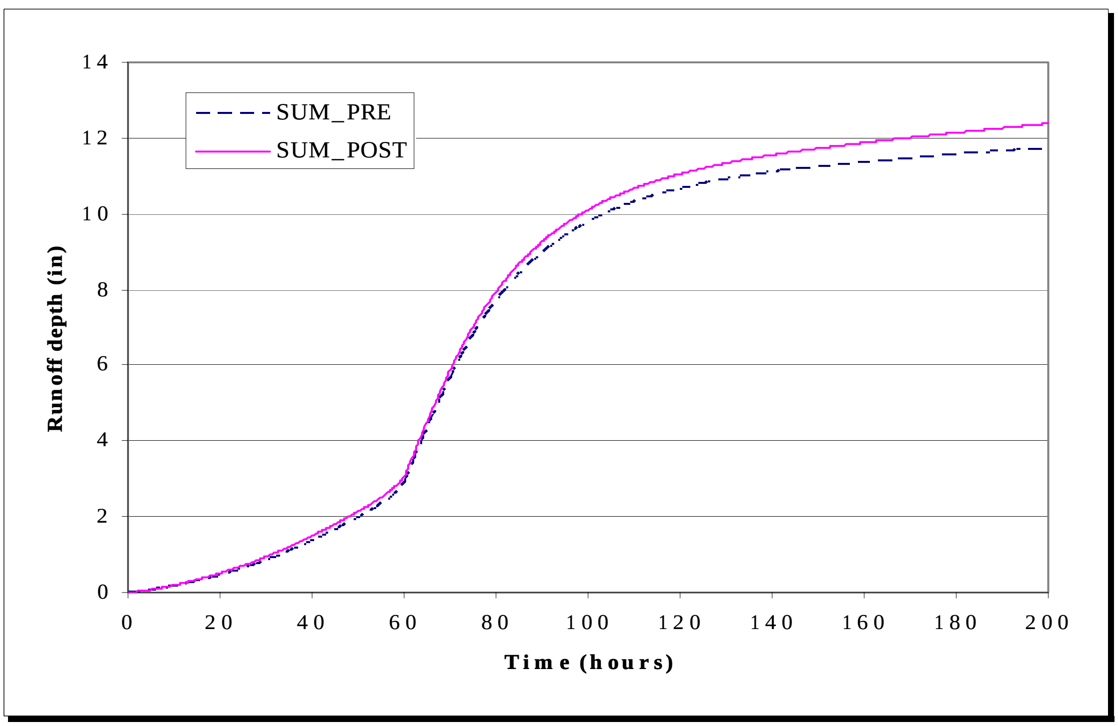


Figure 2.46 Accumulated Runoff Depth for 10-yr Storm of C-9 West Basin with Seepage

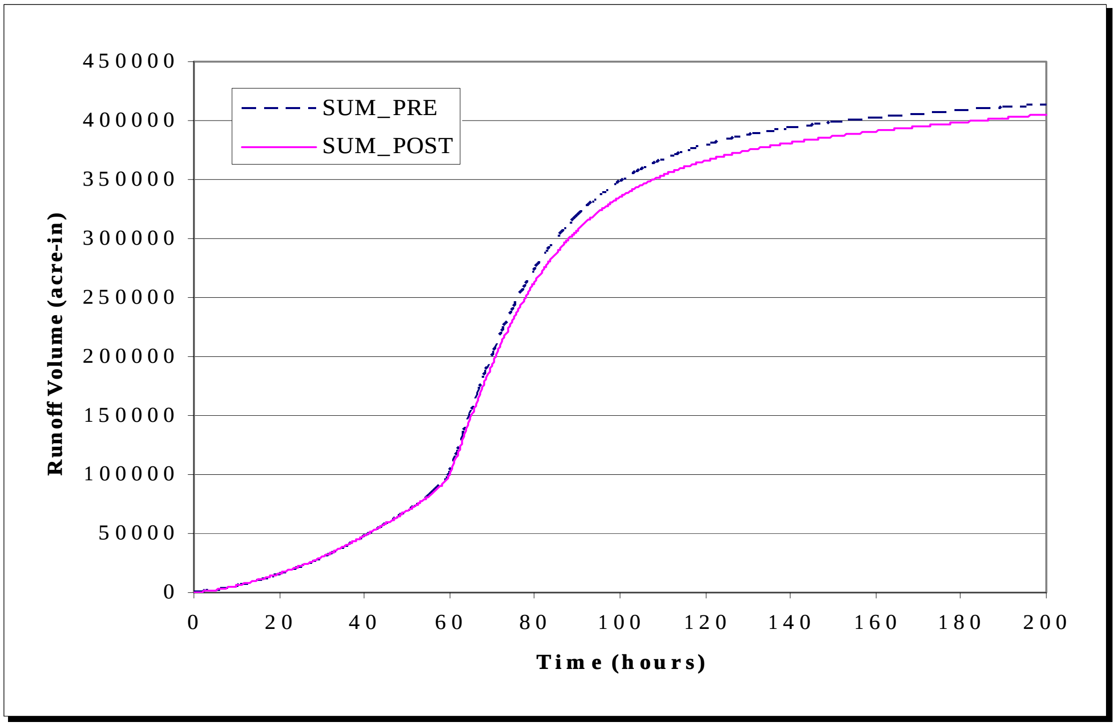


Figure 2.47 Accumulated Runoff Volume for 25-yr Storm of C-9 West Basin with Seepage

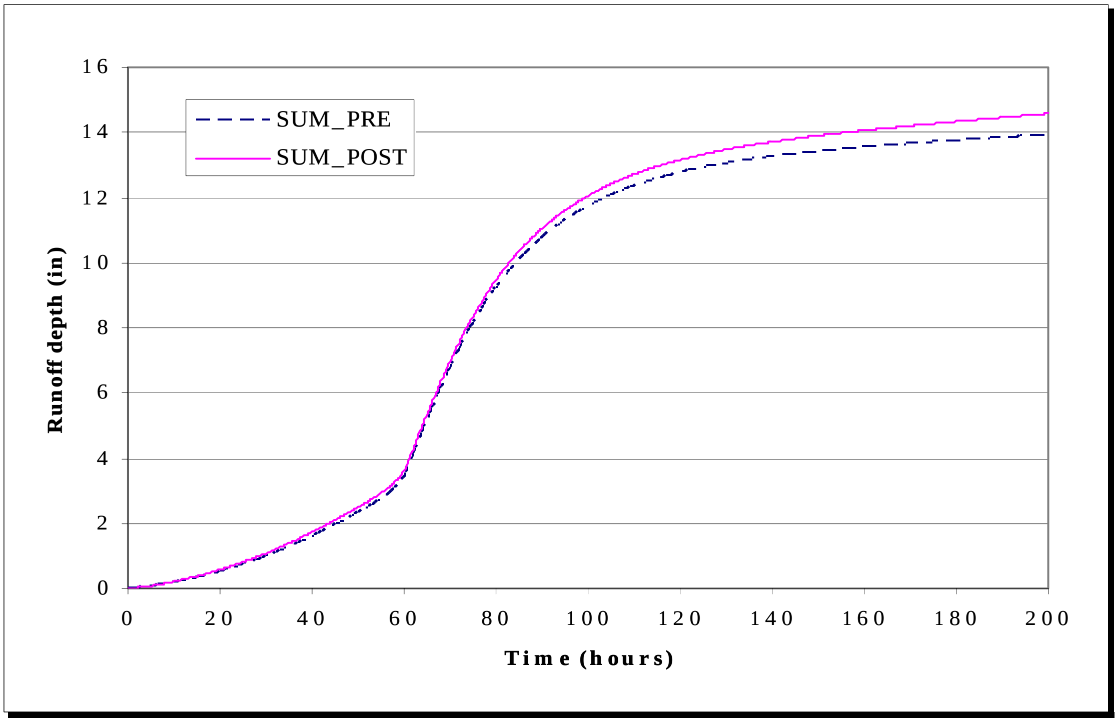


Figure 2.48 Accumulated Runoff Depth for 25-yr Storm of C-9 West Basin with Seepage

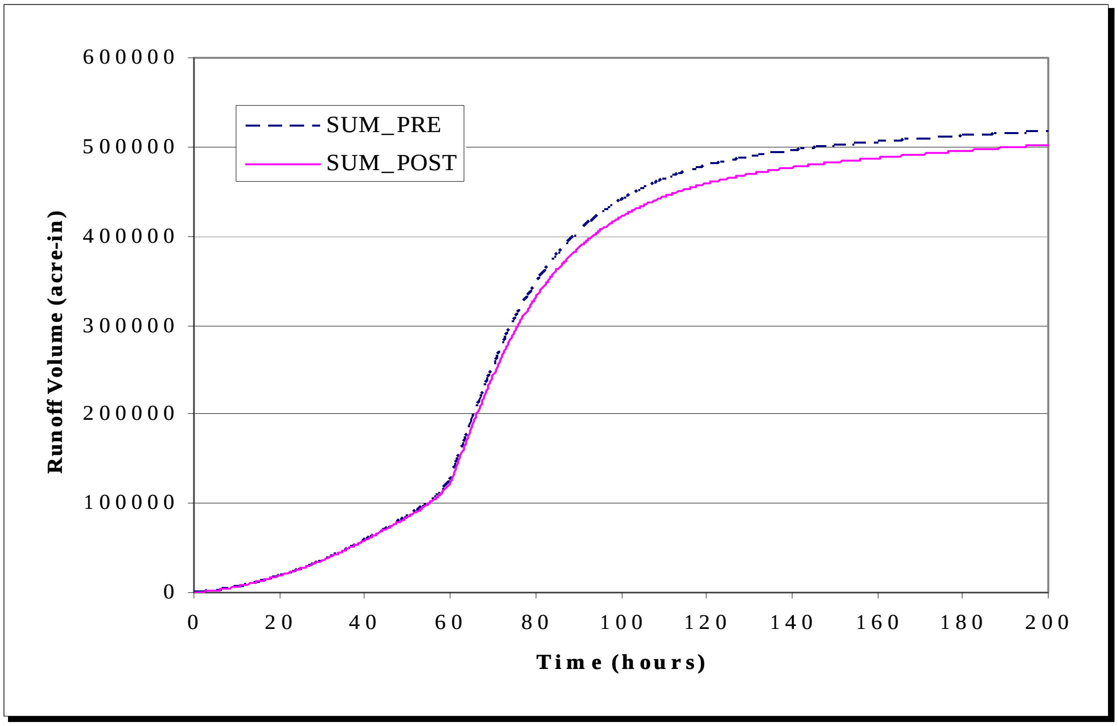


Figure 2.49 Accumulated Runoff Volume for 100-yr Storm of C-9 West Basin with Seepage

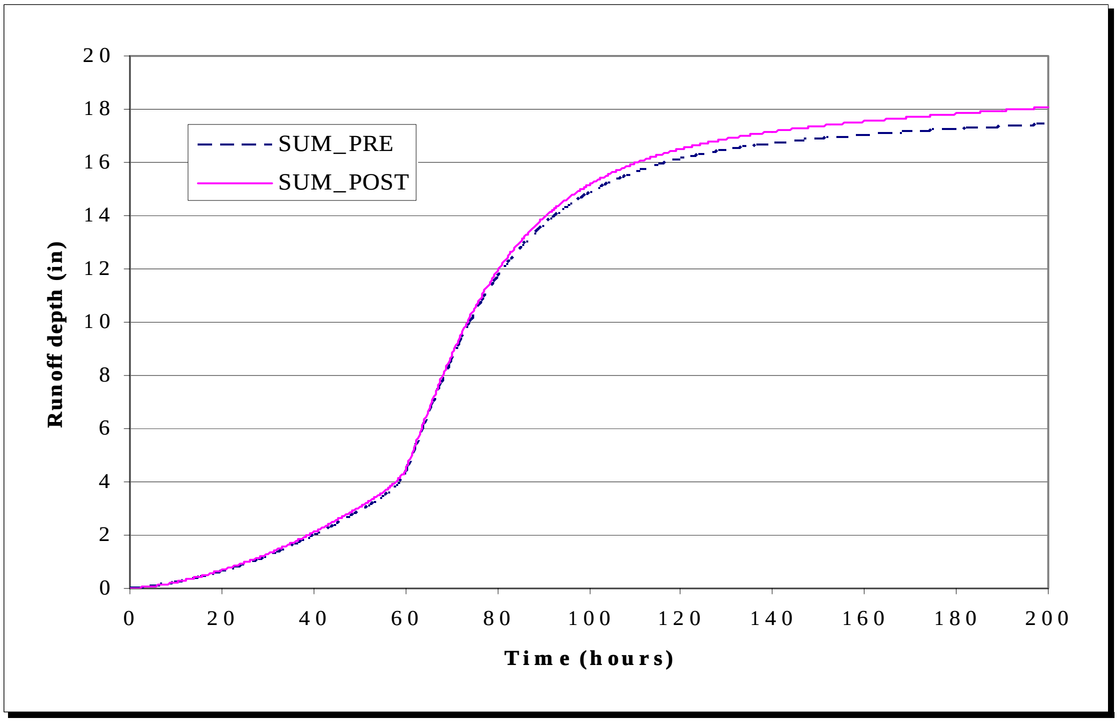


Figure 2.50 Accumulated Runoff Depth for 100-yr Storm of C-9 West Basin with Seepage

2.10. Sensitivity Analysis of Increased Groundwater Stage Elevation (Task 1.4)

2.10.1. Methodology

Task 1.3 included the hydrograph computation based on the "worst case" for post- and pre-conditions. In Task 1.4, a series of analyses were performed to determine the impact of varying groundwater stage increases on runoff volumes and runoff depths. The sensitivity of the runoff depths and runoff volumes to increases in groundwater stage was established in this task.

The "worst case" conditions of November 1, 1990 for the C-11 West basin and July 9, 1990 for the C-9 West basin, as described in Task 1.3, were used as the baseline of sensitivity analysis. Groundwater increases of 0.0, 0.1, 0.2, 0.3, 0.4 and 0.5 foot were added to the pre-condition groundwater stage. The land use map was modified to correspond with the land use of the C-11 and C-9 CERP projects. A series of hydrograph computations were performed in the impacted areas with and without considering the increased seepage rates. The relationships between groundwater stage increase and increased runoff depths and runoff volumes were then generated for the C-11 West basin and the C-9 West basin, respectively.

2.10.2. Sensitivity Analysis without Seepage

- **C-11 West Basin**

A number of hydrograph computations were performed without "seepage" as discussed above, and the percentage difference of runoff depths/volumes vs. the difference of groundwater stage values were computed as shown in Table 2.23 and Table 2.24, for different storm events in the impacted areas of the C-11 West basin. The percentage differences of runoff depths/volumes for the "worst case" on November 1, 1990 are listed in Table 2.23 and Table 2.24. The relationships of the increased runoff depths/volumes to the increased groundwater stage are presented in Figure 2.51 and Figure 2.52. The results in Table 2.23 and Table 2.24 indicate that the differences in runoff depths and runoff volumes both increase approximately 1% with each 0.1 foot increase of the groundwater stage in the C-11 West basin, and when the groundwater stage increases about 0.35 foot, the differences of the runoff depths/volumes are equivalent to the results of the "worst case" on November 1, 1990.

Table 2.23 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin

Storm Event	Impacted Area (acres)		Difference of Runoff Depth (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Nov 1, 1990
10-Year	38785.59	36903.12	-0.03	1.28	2.53	3.69	4.78	5.78	**4.50**
25-Year	38785.59	36903.12	-0.03	1.11	2.18	3.18	4.11	4.97	**3.86**
100-Year	38785.59	36903.12	-0.03	0.90	1.77	2.59	3.34	4.03	**3.13**

Table 2.24 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin

Storm Event	Impacted Area (acres)		Difference of Runoff Volume (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Nov 1, 1990
10-Year	38785.59	36903.12	-4.88	-3.63	-2.45	-1.34	-0.31	0.65	**-0.57**
25-Year	38785.59	36903.12	-4.88	-3.80	-2.78	-1.83	-0.94	-0.12	**-1.18**
100-Year	38785.59	36903.12	-4.88	-4.00	-3.17	-2.39	-1.68	-1.02	**-1.88**

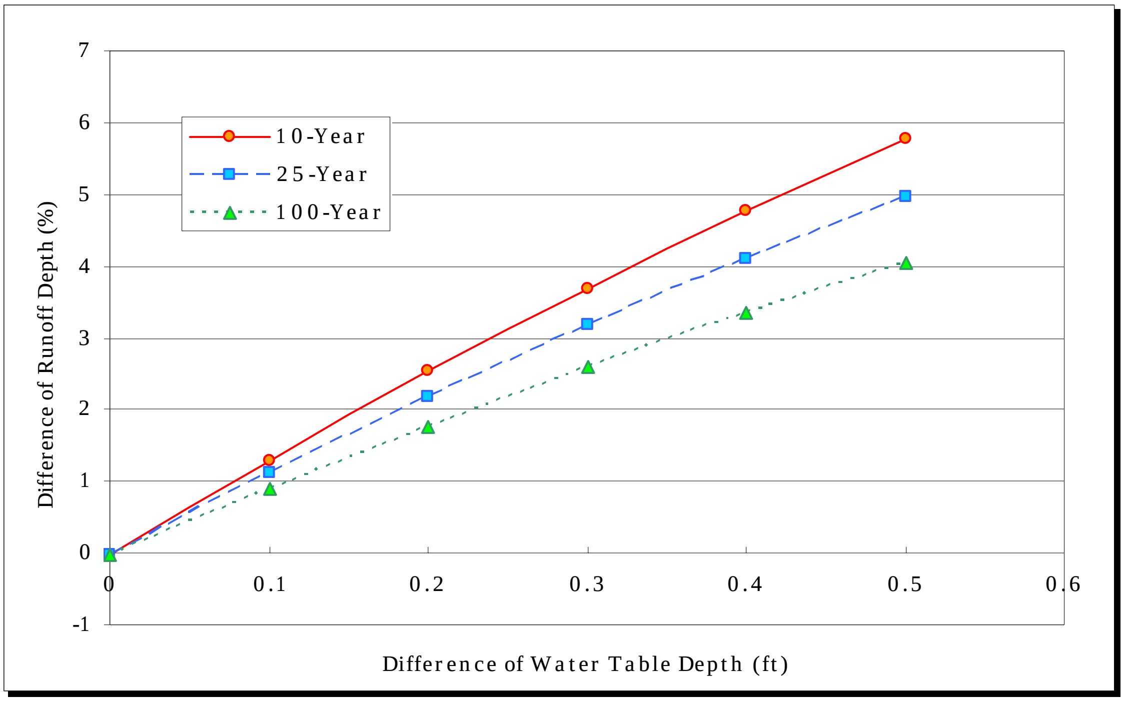


Figure 2.51 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin without Seepage

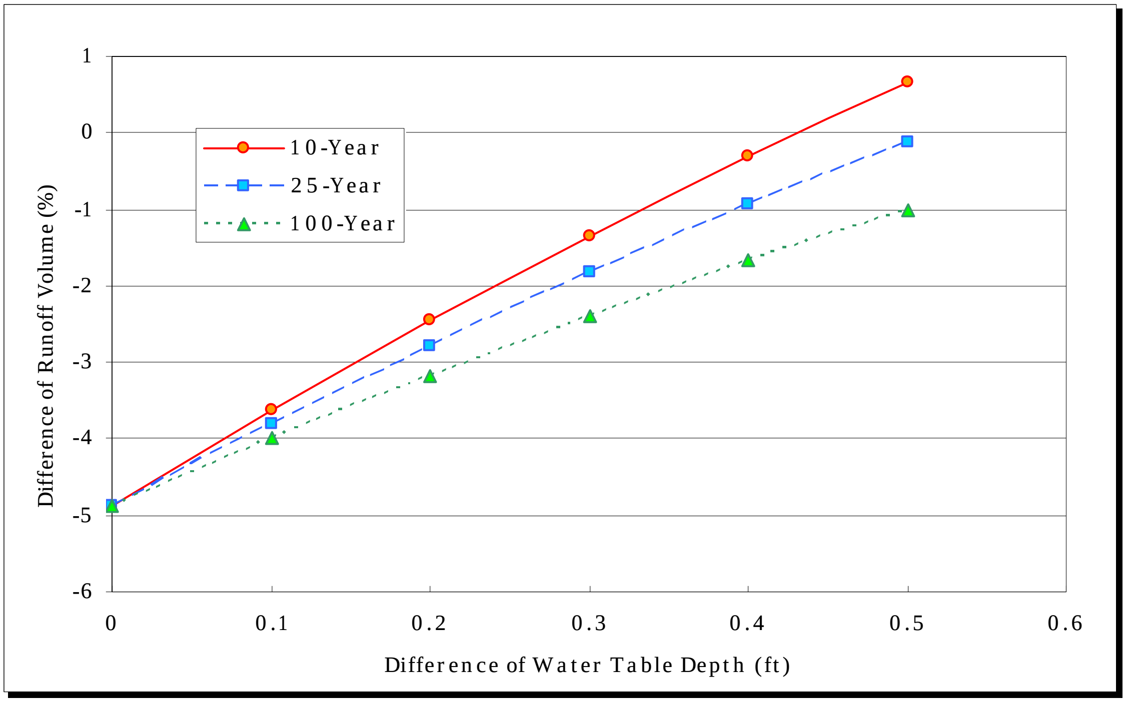


Figure 2.52 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin without Seepage

- **C-9 West Basin**

The difference of runoff depths/runoff volumes vs. the difference of groundwater stage values are listed in Table 2.25 and Table 2.26, for different storm events in the impacted areas of the C-9 West basin. The percentage differences of runoff depths/volumes for the "worst case" on November 1, 1990 are listed in Table 2.23 and Table 2.24. The relationships of the increased runoff depths/runoff volumes vs. the increased groundwater stage values can be found in Figure 2.53 and Figure 2.54, respectively. As is expected, the differences in runoff depths and runoff volumes increase with increases in groundwater stage. The differences in runoff depths and runoff volumes both increase approximately 0.1%, with each 0.1 foot increase of the groundwater stage in the C-9 West basin, and in the case of that the groundwater stage increases about 0.35 foot, the difference of the runoff depths/volumes are equivalent to the results of the "worst case" on July 9, 1990.

Table 2.25 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin

Storm Event	Impacted Area (acres)		Difference of Runoff Depth (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Jul 9, 1990
10-Year	15587.69	13705.23	-0.03	0.19	0.38	0.53	0.64	0.74	**0.59**
25-Year	15587.69	13705.23	-0.03	0.16	0.31	0.44	0.53	0.61	**0.49**
100-Year	15587.69	13705.23	-0.02	0.13	0.25	0.34	0.42	0.48	**0.38**

Table 2.26 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin

Storm Event	Impacted Area (acres)		Difference of Runoff Volume (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Jul 9, 1990
10-Year	15587.69	13705.23	-12.10	-11.91	-11.75	-11.61	-11.51	-11.43	**-11.56**
25-Year	15587.69	13705.23	-12.10	-11.94	-11.80	-11.69	-11.61	-11.54	**-11.65**
100-Year	15587.69	13705.23	-12.10	-11.97	-11.86	-11.78	-11.71	-11.65	**-11.74**

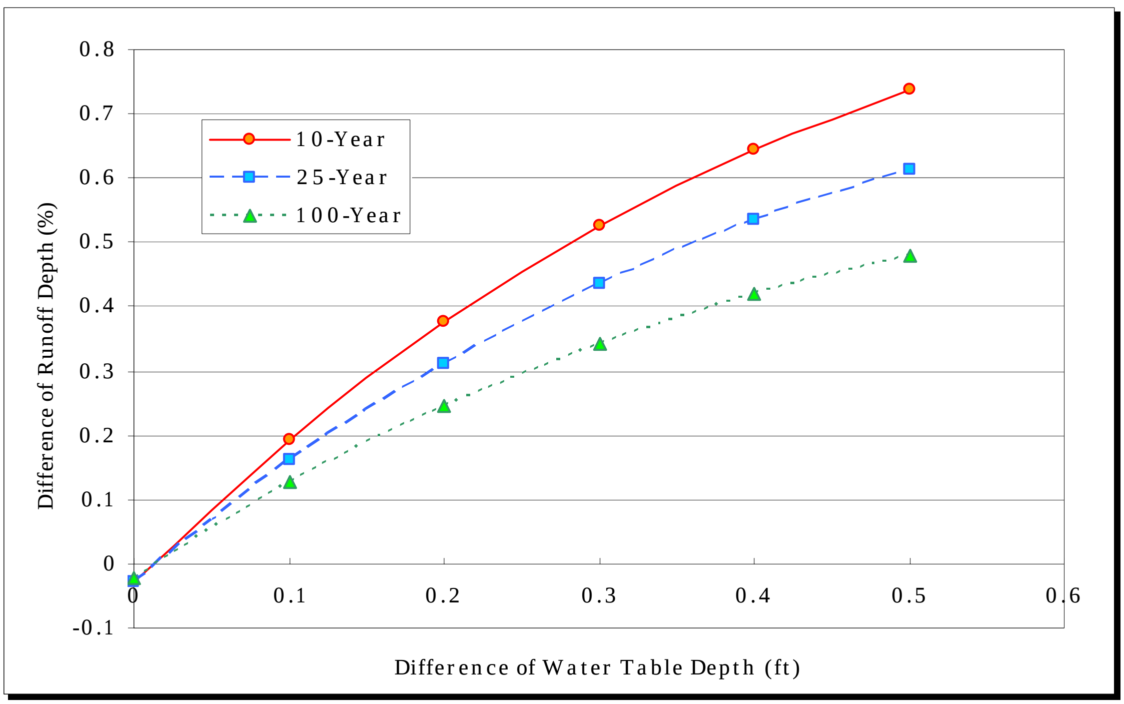


Figure 2.53 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin without Seepage

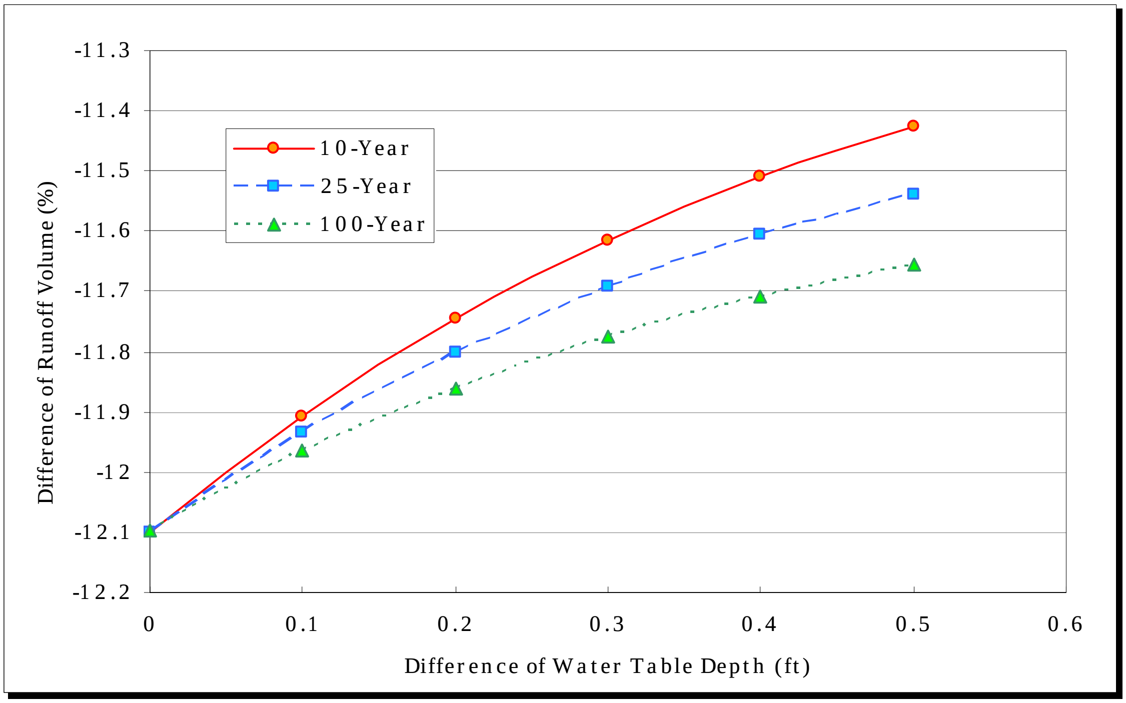


Figure 2.54 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin without Seepage

2.10.3. Sensitivity Analysis with Seepage

The increased seepage was introduced into the hydrograph computations to compare with the previous sensitivity analysis without seepage.

- **C-11 West Basin**

The results of sensitivity analysis with seepage are listed in Table 2.27 and Table 2.28, which can be represented by graphic plots of Figure 2.55 and Figure 2.56. The same results as the previous sensitivity analysis without seepage are obtained, in that the differences in runoff depths and runoff volumes both increase approximately 1% with each 0.1 foot increase of the groundwater stage in the C-11 West basin, and for a groundwater stage increase of about 0.35 foot, the differences in the runoff depths/volumes are equivalent to the results of the "worst case" on November 1, 1990.

Table 2.27 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin with Seepage

Storm Event	Impacted Area (acres)		Difference of Runoff Depth (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Nov 1, 1990
10-Year	38785.59	36903.12	10.76	12.12	13.41	14.62	15.75	16.79	**15.46**
25-Year	38785.59	36903.12	8.93	10.10	11.21	12.24	13.20	14.09	**12.94**
100-Year	38785.59	36903.12	6.96	7.90	8.80	9.63	10.40	11.11	**10.18**

Table 2.28 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin with Seepage

Storm Event	Impacted Area (acres)		Difference of Runoff Volume (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Nov 1, 1990
10-Year	38785.59	36903.12	5.39	6.68	7.91	9.06	10.13	11.12	**9.86**
25-Year	38785.59	36903.12	3.65	4.76	5.81	6.80	7.71	8.55	**7.46**
100-Year	38785.59	36903.12	1.77	2.67	3.52	4.31	5.04	5.72	**4.84**

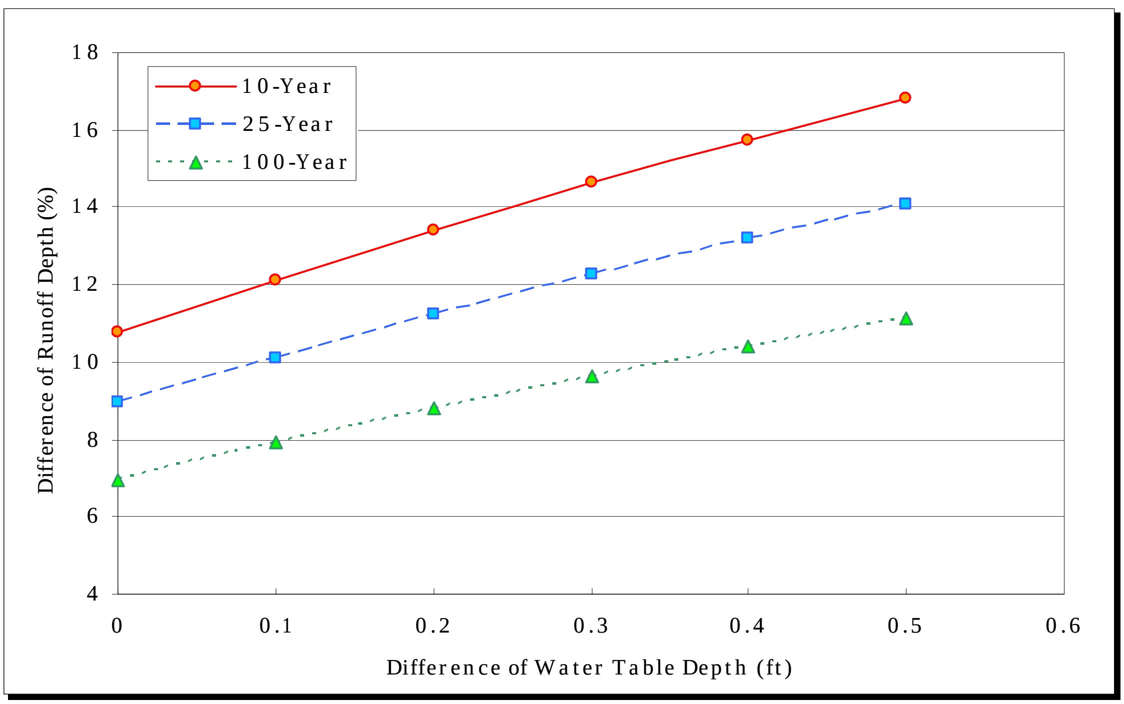


Figure 2.55 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin with Seepage

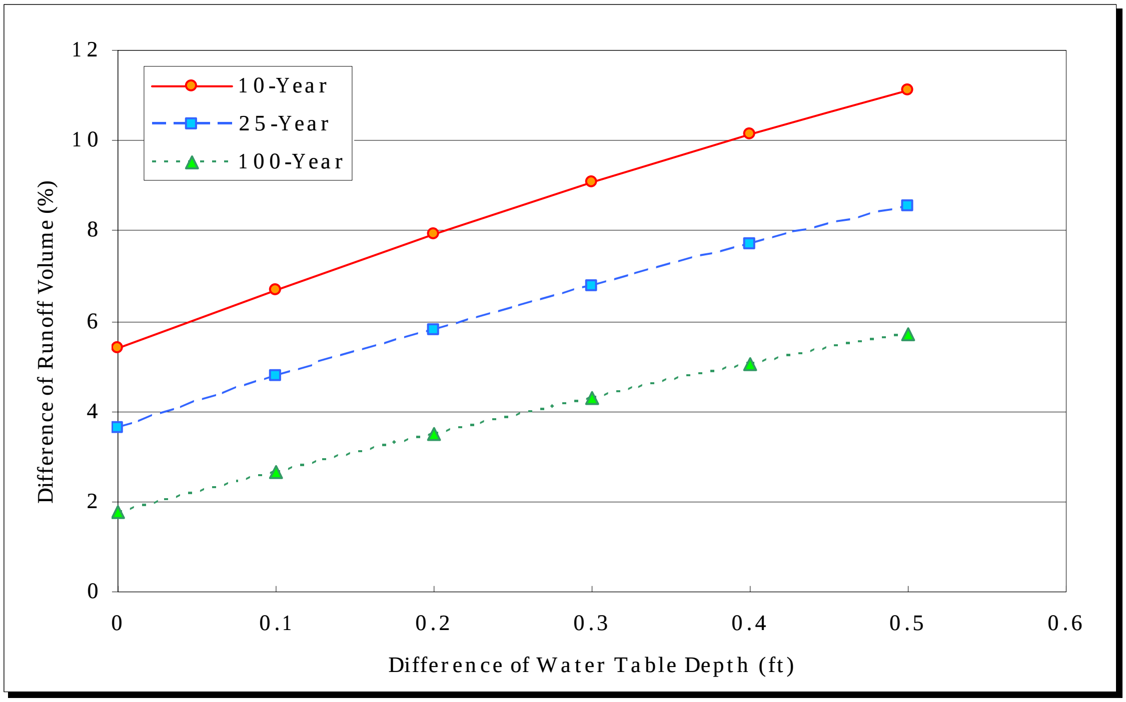


Figure 2.56 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-11 West Basin with Seepage

- **C-9 West Basin**

The results of sensitivity analysis with seepage are listed in Table 2.29 and Table 2.30, which are represented by graphic plots of Figure 2.57 and Figure 2.58. The same results as the previous sensitivity analysis without seepage for the C-9 West basin are obtained, in that the differences in runoff depths and runoff volumes both increase approximately 0.1% with each 0.1 foot increase of the groundwater stage in the C-9 West basin, and for a groundwater stage increase of about 0.35 foot, the differences in the runoff depths/volumes are equivalent to the results of the "worst case" on July 9, 1990.

Table 2.29 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin with Seepage

Storm Event	Impacted Area (acres)		Difference of Runoff Depth (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Jul 9, 1990
10-Year	15587.69	13705.23	9.84	10.04	10.20	10.33	10.44	10.52	**10.39**
25-Year	15587.69	13705.23	8.25	8.41	8.55	8.66	8.75	8.82	**8.71**
100-Year	15587.69	13705.23	6.56	6.69	6.80	6.89	6.96	7.02	**6.93**

Table 2.30 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin with Seepage

Storm Event	Impacted Area (acres)		Difference of Runoff Volume (%)						
	Area_Pre	Area_Post	0.00	0.10	0.20	0.30	0.40	0.50	Jul 9, 1990
10-Year	15587.69	13705.23	-3.43	-3.25	-3.11	-2.99	-2.90	-2.82	**-2.94**
25-Year	15587.69	13705.23	-4.83	-4.68	-4.56	-4.46	-4.38	-4.32	**-4.42**
100-Year	15587.69	13705.23	-6.31	-6.19	-6.10	-6.02	-5.95	-5.90	**-5.98**

2.10.4. Conclusions

The sensitivity of runoff depths and runoff volumes with increased groundwater levels was analyzed for the 10-, 25-, and 100-year, 72-hour storm events with and without seepage. The relationships between groundwater stage increase and increased runoff depths/runoff volumes were established in tabular and graphic forms.

For the case of the C-11 West basin sensitivity analysis with and without seepage, the differences in runoff depths and runoff volumes both increase approximately 1% for each 0.1 foot increase of the groundwater stage, and for a groundwater stage increase of about 0.35 foot, the difference of the runoff depths/volumes correspond with the "worst case" on November 1, 1990.

For the case of the C-9 West basin sensitivity analysis with and without seepage, the differences in runoff depths and runoff volumes both increase approximately 0.1% for each 0.1 foot increase of the groundwater stage in the C-9 West basin, and for a groundwater stage increase of about 0.35 foot, the difference of the runoff depths/volumes correspond with the "worst case" on July 9, 1990.

Establishing the statistical bounds or confident intervals by varying the model parameters is outside the scope of this study. Such an analysis will be complicated and extensive because of the many models employed in the analysis (SFWMM, BC MODFLOW, SEEP2D, and SBUH).

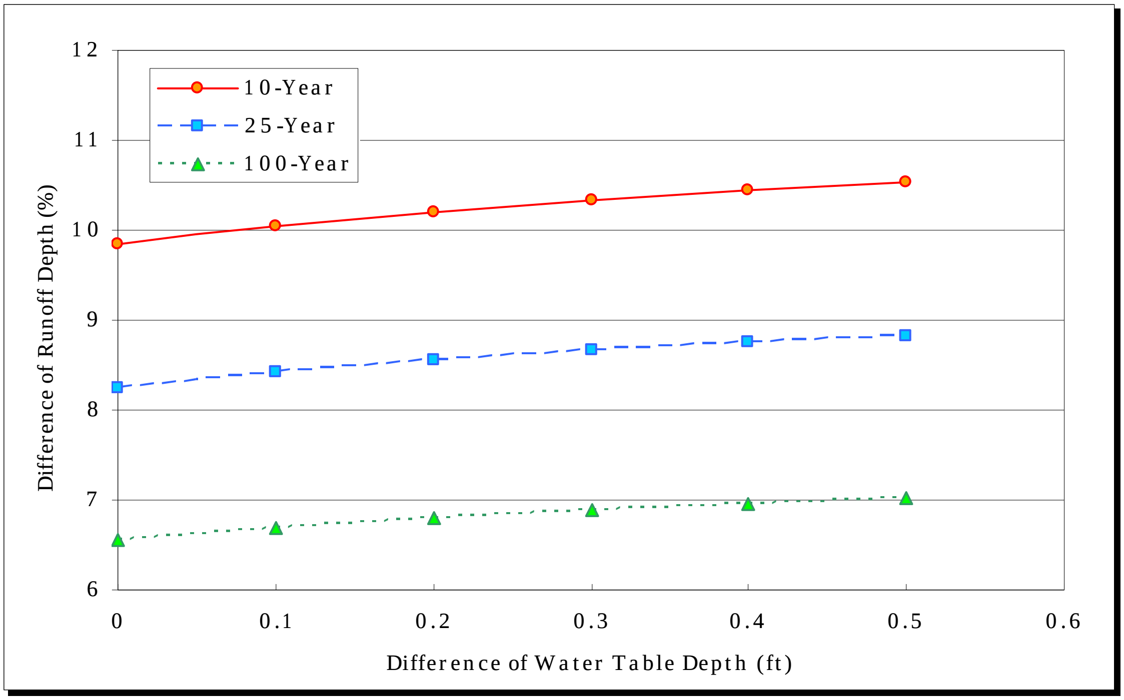


Figure 2.57 Difference of Runoff Depth vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin with Seepage

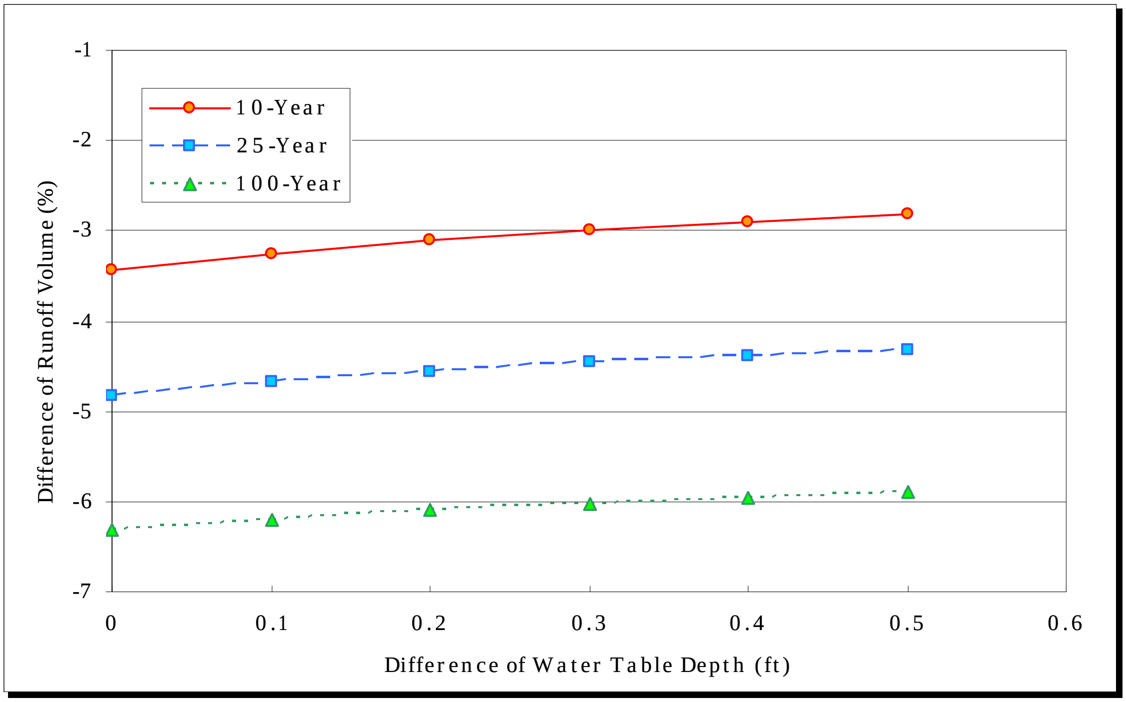


Figure 2.58 Difference of Runoff Volume vs. Difference of Groundwater Stage for the Impacted Areas of C-9 West Basin with Seepage

2.11. Available Impoundment Storage (Task 1.5)

2.11.1. C-11 Impoundment

The storage area of the C-11 impoundment is 1,490 acres. The surface water elevation is maintained at 10 ft-NGVD, which is the normal pool elevation of this impoundment throughout the simulation period of the BC MODFLOW modeling. The surcharge pool elevation is modeled at 13 ft. The storage space between the normal pool and the surcharge pool is the maximum available storage volume that might be reserved to contain the direct rainfall volume and the increases of runoff volume computed in Task 1.3. For the C-11 impoundment, the available storage volume is calculated to be 4,470 acre-ft (=1,490 acres x (13 ft - 10 ft)).

The volume of direct rainfall was obtained based on the storage area and the rainfall depth. The rainfall depths for the C-11 West basin can be found in Table 2.6 for 10-, 25-, and 100-year, 72-hour storm events, respectively. The runoff volume increment estimated in Task 1.3 is listed in Table 2.31, Table 2.32, for the case of without and with seepage, respectively. The total volume increment is the sum of the direct rainfall volume and the runoff volume increment. However, the runoff volume increment is added only when the runoff volume increment is positive. Thus, the total volume increments are calculated for the case of without and with seepage for the C-11 impoundment, and are listed in the sixth column of Table 2.31 and Table 2.32, respectively.

From Table 2.31 and Figure 2.59, it is seen that up to 41.67% of the total storage available in the C-11 impoundment should be reserved to store the direct rainfall, if only groundwater stage increases were considered in the hydrograph computation. In Table 2.32 and Figure 2.59, up to 90.33% of the total available storage volume is estimated to be reserved if both the seepage increases and the groundwater stage increases were considered.

Table 2.31 C-11 Impoundment Storage Availability Assessment without Seepage

Storm Event	Rainfall Depth (in)	Storage Area (acres)	Direct Rainfall Volume (acre-ft)	Runoff Volume Increment* (acre-ft)	Total Volume Increment (acre-ft)	Available Storage Volume (acre-ft)	% of Available Storage Volume
10-Year	10.2	1490	1266.50	-167.45	1266.50	4470.00	*28.33*
25-Year	12.0	1490	1490.00	-411.98	1490.00	4470.00	*33.33*
100-Year	15.0	1490	1862.50	-836.86	1862.50	4470.00	*41.67*

*The runoff volume increments of the impacted area are from Table 2.9 through Table 2.11 for 10-, 25-, and 100-year, 72-hour storm events, and then divided by 12 in/ft.

Table 2.32 C-11 Impoundment Storage Availability Assessment with Seepage

Storm Event	Rainfall Depth (in)	Storage Area (acres)	Direct Rainfall Volume (acre-ft)	Runoff Volume Increment* (acre-ft)	Total Volume Increment (acre-ft)	Available Storage Volume (acre-ft)	% of Available Storage Volume
10-Year	10.2	1490	1266.50	2771.36	4037.86	4470.00	*90.33*
25-Year	12.0	1490	1490.00	2526.82	4016.82	4470.00	*89.86*
100-Year	15.0	1490	1862.50	2101.90	3964.40	4470.00	*88.69*

*The runoff volume increments of the impacted area are from Table 2.17 through Table 2.19 for 10-, 25-, and 100-year, 72-hour storm events, and then divided by 12 in/ft.

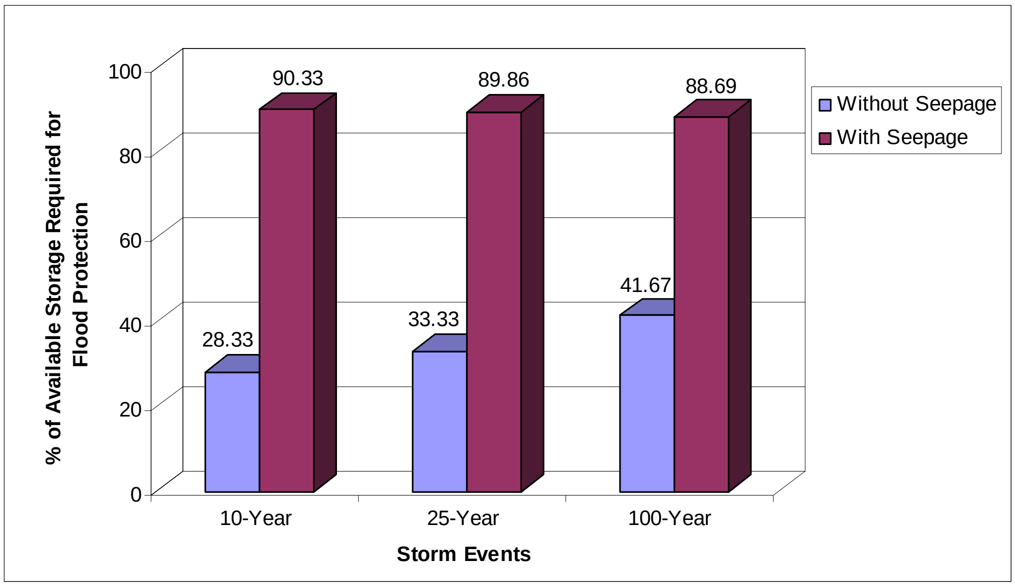


Figure 2.59 C-11 Impoundment Storage Availability Assessment with and without Seepage

2.11.2. C-9 Impoundment

The storage area of the C-9 impoundment is 1,650 acres. The normal and surcharge pool elevations are maintained at 8.5 ft and 11.5 ft, respectively. The available storage volume, as shown in Table 2.33, is calculated to be 4,950 acre-ft.

The direct rainfall volume was computed for different storm events. The increase of runoff volume estimated in Task 1.3 is listed in Table 2.33 and Table 2.34, for the case without and with seepage consideration, respectively. The results in Table 2.33 and Table 2.34 and in Figure 2.60 indicate that up to 44.44% of total available storage volume of the C-9 impoundment should be reserved to store the direct rainfall, since no increase of runoff volume was introduced into the C-9 West basin either with or without seepage consideration.

Table 2.33 C-9 Impoundment Storage Availability Assessment without Seepage

Storm Event	Rainfall Depth (in)	Storage Area (acres)	Direct Rainfall Volume (acre-ft)	Runoff Volume Increment* (acre-ft)	Total Volume Increment (acre-ft)	Available Storage Volume (acre-ft)	% of Available Storage Volume
10-Year	10.3	1650	1416.25	-1531.05	1416.25	4950.00	*28.61*
25-Year	12.0	1650	1650.00	-1875.61	1650.00	4950.00	*33.33*
100-Year	16.0	1650	2200.00	-2424.07	2200.00	4950.00	*44.44*

*The runoff volume increments of the impacted area are from Table 2.12 through Table 2.14 for 10-, 25-, and 100-year, 72-hour storm events, and then divided by 12 in/ft.

Table 2.34 C-9 Impoundment Storage Availability Assessment with Seepage

Storm Event	Rainfall Depth (in)	Storage Area (acres)	Direct Rainfall Volume (acre-ft)	Runoff Volume Increment* (acre-ft)	Total Volume Increment (acre-ft)	Available Storage Volume (acre-ft)	% of Available Storage Volume
10-Year	10.3	1650	1416.25	-435.21	1416.25	4950.00	*28.61*
25-Year	12.0	1650	1650.00	-779.77	1650.00	4950.00	*33.33*
100-Year	16.0	1650	2200.00	-1328.22	2200.00	4950.00	*44.44*

*The runoff volume increments of the impacted area are from Table 2.20 through Table 2.22 for 10-, 25-, and 100-year, 72-hour storm events, and then divided by 12 in/ft.

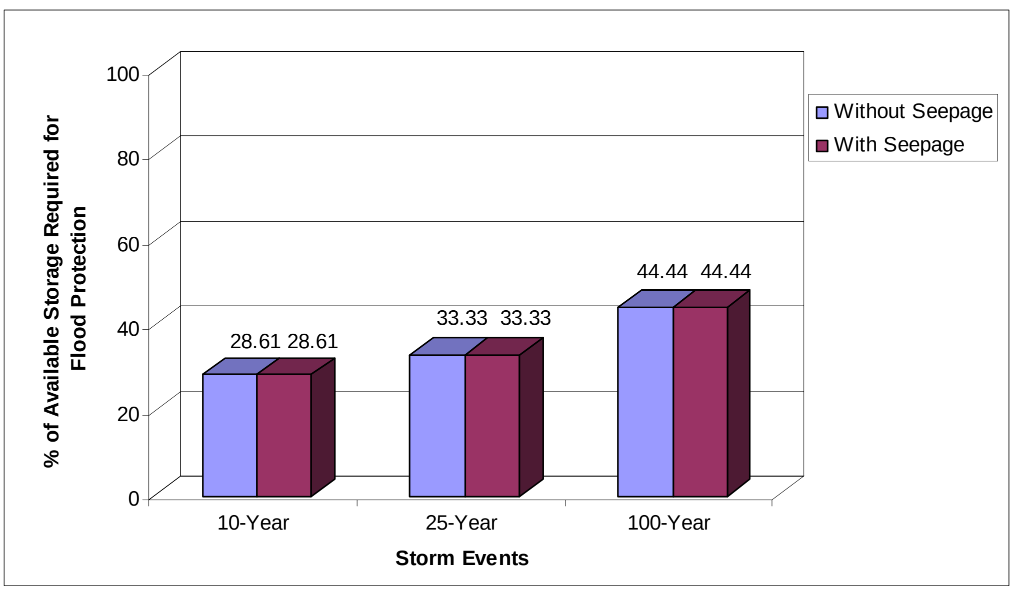


Figure 2.60 C-9 Impoundment Storage Availability Assessment with and without Seepage

2.11.3. Conclusions

Impoundment storage availability analysis was performed for 10-, 25-, and 100-year, 72-hour storm events with and without seepage consideration. As shown in Figure 2.59 and Figure 2.60, up to 90.33% of total storage volume of the C-11 impoundment was estimated to be reserved for flood protection of the C-11 West basin, and 44.44% of total storage volume of the C-9 impoundment should be reserved for flood protection of the C-9 West basin. It is therefore concluded that the C-11 impoundment and the C-9 impoundment have sufficient available storage volume to contain the increases of runoff volume caused by the CERP projects.

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TABLE OF CONTENTS

List of Tables

List of Figures

3. ESTABLISH THE BEHAVIOR OF THE C-11 AND C-9 IMPOUNDMENTS DURING DESIGN FLOOD CONDITIONS

3.1. Objectives

The objectives of Task 2 include the following: the limitations of the Broward County MODFLOW model (referred to as BC MODFLOW in this report) in computing seepage inflow rates to the seepage collection canals adjacent to the impoundments will be reviewed. If required as a result of that review, another approximate model will be developed. In addition, a hydraulic model to determine seepage canal stages will be developed. The engineering design of the C-11 and C-9 impoundments and their operations during wet seasons and their behavior during flood conditions will be assessed. The intent in the conduct of these analyses is to assess the capacity and design of the reservoirs seepage collection systems. These analyses will be conducted assuming the reservoirs to be full and high adjacent groundwater tables.

3.2. Scope of Work

This Part 3 presents the results of analyses conducted in connection with Task 2 as it is defined in SFWMD Contract No. C-20104P-WO03. Task 2 includes the following specific subtasks and primary activities:

- Task 2.1 – Review MODFLOW limitations and, if necessary, build model
- Task 2.2 – Quantify seepage from each impoundment as a function of impoundment storage – assume wet conditions in surrounding lands
- Task 2.3 – Establish confidence bounds on seepage rates from the C-11 and C-9 impoundments

3.3. Source of Data

Available information and data were collected and compiled during the preparation of this report. They are listed as follows:

GIS Data:

- Drainage Basin Map (South Florida Water Management District, SFWMD)
- Cross-section Drawings (U.S. Army Corps of Engineers, Jacksonville District)

Reports:

- The electronic copies of the October 2001 Water Preserve Areas Feasibility Study, including all appendices thereto
- River Analysis System (HEC-RAS) User's Manual, Version 3.1, November 2002
- River Analysis System (HEC-RAS) Hydraulic Reference Manual, Version 3.1, November 2002

Modeling Data:

- Broward County Groundwater flow model source code, input and output for 8-year simulation
- Planned seepage collection canal cross-sections and geometric data
- Planned seepage return pump capacities and operating logic

3.4. Study Area Description

The C-11 West basin and C-9 West basin, as shown in Figure 3.1, are the areas of interest for the seepage analysis. A brief description of these two basins and the CERP Projects has been discussed in Part 2.

The C-11 and C-9 impoundments serve primarily six functions: (1) to aid in reducing seepage from the WCA 3A/3B Levee Seepage Management Area; (2) to provide groundwater recharge; (3) to provide adequate water supply to urban areas; (4) to prevent saltwater intrusion; (5) to provide flood protection capabilities; and (6) to aid in water quality improvement.

The purpose of the WCA 3A/3B Levee Seepage Management System is to reduce the seepage from WCA 3A/3B by holding higher water levels in the L-33 and L-37 borrow canals and marsh areas. The purpose of the C-11 impoundment is to provide storage for excess runoff from the C-11 West basin and prevent pumping the untreated runoff into the WCA 3A. If water is not available in the impoundment area, the S-381 gate will be opened to allow seepage water to recharge the C-11 West basin and prevent excessive dry outs. In addition, seepage will be collected and returned to the impoundment area.

The purpose of the C-9 impoundment is to provide storage for excess runoff from the western C-9 drainage basin and impound the runoff diverted from the C-11 West basin to prevent discharge of untreated runoff into WCA 3A. During the wet season, the S-511

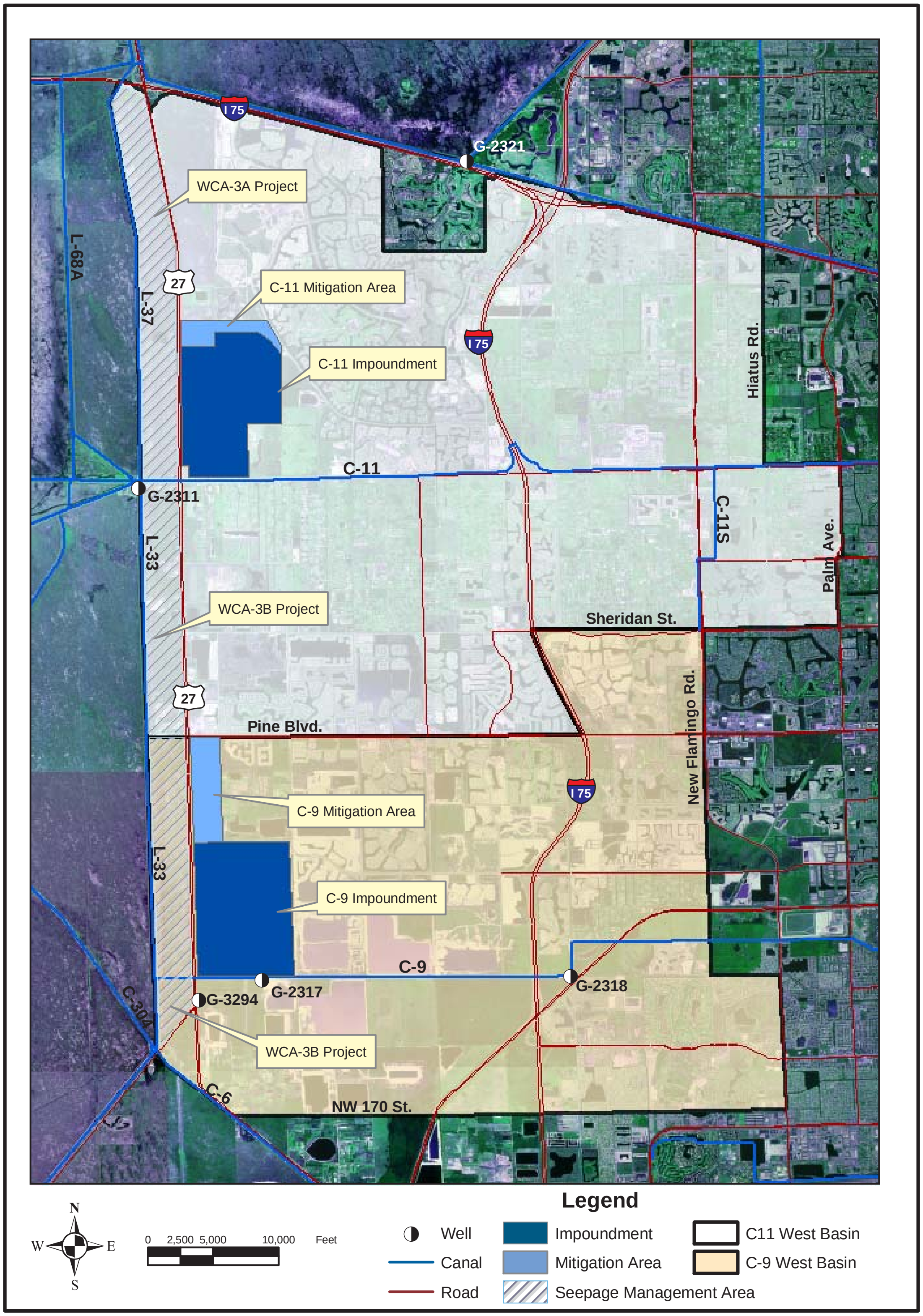

Figure 3.1 C-11 West Basin and C-9 West Basin Boundary and Canals

gate, located in the C-9 canal about 200 ft east of the C-9 impoundment eastern boundary, will be opened, this will benefit pumping the runoff from the western C-9 basin and/or the diverted runoff from the C-11 West basin by the way of the C-502 canal. When water is released from the C-9 impoundment into the C-9 canal, S-511 can be either partially or fully opened to convey water to the east if desired, but would be closed if water was to move south by the way of the C-502 canal.

3.5. Limitations of MODFLOW Seepage Modeling and Recommended Model

As part of the WPA Feasibility Study, both two-dimensional and three-dimensional groundwater models were prepared for the various design regions. Three separate sub-regional MODFLOW groundwater flow models were developed by the South Florida Water Management District, and were described in WPA Feasibility Report, Appendix B. Broward County groundwater flow model is a five-layer MODFLOW model to simulate water table elevations and seepage rate from the C-11 and C-9 impoundments as well. The model utilized a grid size of 500 ft x 500 ft. Canal boundary conditions, specifically the stages, for the model was derived from the 2 mi x 2 mi SFWMM model. The results of the BC MODFLOW should be interpreted keeping in mind the following limitations:

- Pumpages used in the BC MODFLOW are critical in the simulation of water table elevations. Individual pump types and withdraws used in the model were defined by SFWMD. Pumpages in the model were not reviewed for consistency with those defined in the WPA Feasibility Study.
- The BC MODFLOW is a 500 ft x 500 ft grid-based three dimensional model wherein each grid cell is treated as a homogeneous computation element without further discretization capability.
- The canal boundary conditions used in the BC MODFLOW were derived from the SFWMM with a grid size of 2 mi x 2 mi. The canal stages derived from the SFWMM may be approximate because of the large grid size used in SFWMM.
- In the BC MODFLOW, the average groundwater head in one grid cell is utilized to calculate the head difference between the aquifer and the seepage canals. Also, the BC MODFLOW is a regional model with constant, large grid size that is applied to a large area. Consequently, the head difference is not accurately calculated and the seepage rate is therefore suspect. By refining the grid size around the seepage canals, more

accurate adjacent groundwater head can be obtained from which a more accurate seepage rate can be obtained.

- Another factor that affects the seepage flow rate collected is the transmissivity of the interface between the aquifer and the canal, which is the intermediate result computed based on the hydraulic conductivity of the soil layer and other parameters. However, the transmissivity calculation is not available for review.
- Only 96 daily flow rate data (one day for each month) are available to review in the total 8-year simulation period.
- Seepage in MODFLOW is calculated using an equation that is simple and linearly proportional to the head difference between the canal and aquifer stages whereas a more accurate way is to use Dupuit approximation leading to a nonlinear equation involving the difference in the squares of the heads in canal and aquifer as is done in more accurate seepage models such as SEEP2D.
- The combination of inaccurate canal stages taken from the SFWMM model and the large grid size of SFWMM and BC MODFLOW in relation to the actual canal dimensions would render the canal as excessive drainage in that the canal removes water at a greater rate than the capacity of the seepage canal system.

As noted in Task 1.2, the SEEP2D model, originally developed by the U.S. Army Corps of Engineers (COE), was selected as part of the Feasibility Study for seepage calculations under Geotechnical investigations for the design of seepage collection system (the canals, pump stations) for the C-11, C-9 impoundments and SMA-3A/3B.

SEEP2D, a two-dimensional vertical steady state finite element numeric model, is used to evaluate the seepage losses from impoundments and conveyance canals. SEEP2D has been extensively used for over 20 years for dam seepage estimation. SEEP2D is considered more applicable than MODFLOW to the analysis of the seepage collection system as it is specifically a near field seepage model with more valid equations of flow and can be applied to either unconfined or confined aquifers. SEEP2D is also more accurate because it not only computes seepage into the seepage canal but also the so-called under flow or the flow below the canal bottom into the aquifer. However, there is no indication in the WPA Feasibility Study that this SEEP2D model was calibrated to seepage rates measured in the vicinity of the project.

Also in Task 1.2, it was noted that the BC MODFLOW seepage rates were approximately 20 times higher than the SEEP2D results and again BC MODFLOW calibration results were not available for our review. The SEEP2D results with a safety factor of five reported in the geotechnical investigations reported in the WPA Feasibility Study indicated that the proposed seepage collection system design is adequate.

The original SEEP2D model could not be applied in this study because the model documentation and input files could not be obtained. Therefore, a new SEEP2D model was constructed and applied to the seepage collection system performance analysis in Task 2.

3.6. Seepage Model Development

3.6.1. General Description and Background

After comparison of MODFLOW and SEEP2D models in Task 2.1, SEEP2D was selected for the WPA seepage modeling efforts. This model code was originally developed by Fred Tracy of the COE Waterway Experiment Station (WES) in the late 1980s. SEEP2D is a two-dimensional finite-element model code that is ideal for evaluating seepage losses from impoundments or conveyance canals.

Groundwater Modeling System (GMS), developed by the Brigham Young University Environmental Modeling Research Laboratory in cooperation with the Waterways Experiment Station, is a modeling system graphical user interface that both acts as a pre and post-processor of various numerical models including FEMWATER, MODFLOW, SEEP2D and others. GMS version 5.0 dated July 2004 was used to prepare the cross-section geometry and define the boundary conditions and material properties. Head, flow and Darcy velocity at each node were computed by SEEP2D and imported to GMS for display and analysis.

3.6.2. Hydrogeologic Conceptual Model

Before the detailed seepage model is built, it is necessary to delineate the hydrogeologic conceptual model first. Preparation of the hydrogeologic conceptual model was completed in the WPA Feasibility Report, Appendix C: Geotechnical Appendix. Sources of information included the SFWMD, Jacksonville District files, the United States Geological Survey (USGS), the Florida Geological Survey (FGS), the Florida Department of Environmental Protection (FDEP), local universities, and several engineering, consulting firms. Much of this data was complied into a database for further interpretation. The most pertinent data is shown and summarized in Table C.1.

After reviewing the data and evaluating the function of the various WPA project features, it was decided that the conceptual model development should concentrate on the Surficial Aquifer System (SAS). Generally, the SAS comprises all the rocks and sediments from land surface to the top of the intermediate confining unit and includes the Pleistocene Pamlico Sand, Anastasia Formation, Fort Thompson Formation, Miami Oolite, Key Largo Formation, Caloosahatchee Marl and the Pliocene Tamiami Formation. The SAS can be sub-divided into numerous different units depending on the location and individual unit properties.

The SAS conceptual model has been separated into five layers on the basis of the approximate hydraulic conductivity (K) ranges by COE Jacksonville District staffs. A sixth layer denotes the top of the Hawthorn Formation (base of the SAS). The following provides an explanation of the strata and characteristics of each layer.

- **Conceptual Model Layer 1**

The uppermost layer includes the undifferentiated soil and sand and portions of the Pamlico Sand, Miami Limestone and Fort Thompson in the east and central areas, and the uppermost Tamiami Formation in the west areas. Sediments in Layer 1 are composed primarily of surficial sands, peats, and clayey material and is generally on the order of a few feet to greater than 50 feet in Palm Beach and Hendry Counties. The K values for this layer are generally low to moderate (0.1 to <100 ft/d).

- **Conceptual Model Layer 2**

This layer is comprised primarily of the Quaternary age rocks above the Tamiami Formation, except in the western portion of the study area, where Layer 2 includes portions of the upper Tamiami Formation. Other units included are the Miami Limestone, Anastasia Formation, Key Largo Formation and the Fort Thompson Formation. Layer 2 deposits generally exhibit high to very high K (100 to >1000 ft/d) in the east, and moderate to high K (10 to 1000 ft/d) in the west. The thickness of Layer 2 ranges from 0 ft to 180 ft. The Biscayne Aquifer is included in this layer.

- **Conceptual Model Layer 3**

This layer primarily consists of the upper and middle Tamiami Formation, interbedded with some local early Quaternary deposits. The K values in this layer are generally low to moderate (1 to 100 ft/d), and the thickness ranges from 0 ft to 120 ft.

- **Conceptual Model Layer 4**

In general, this layer consists of Middle to Lower Tamiami Formation deposits, including the Gray Limestone. These deposits are characterized by predominately porous limestone, sandstone, shell & calcified beds with a wide range of solution cavities. The K values in this layer are generally moderate to high (10 to 1000 ft/d), and the thickness ranges from 0 ft to 140 ft.

- **Conceptual Model Layer 5**

This layer consists of Lower Tamiami Formation deposits composed of predominately sands, sandy limestone, micrites, and some limestone. These deposits are characterized by generally low K (0.1 to 10 ft/d), although local heterogeneity within this unit can lead to more highly conductive zones where K can be moderate to high. The thickness of Layer 5 ranges from 0 ft to 170 ft.

- **Conceptual Model Layer 6**

This layer consists of the uppermost deposits of the Hawthorn Group, which is characterized by clay, silt, clayey sand and clayey limestone. These deposits have a low to very low K (<0.1 to 10 ft/d). In the west, these deposits may locally include sand and sandy limestone with low to moderate K. For modeling purposes, this layer represents the bottom of the SAS.

3.6.3. SEEP2D Model Development Methodology

The most important model input for SEEP2D is the assumed hydrogeology of the two-dimensional cross-section. This information was developed by preparation of the hydrogeologic conceptual model discussed above. Most of the SEEP2D cross-section models adopted the conceptual model hydrogeology unless more detailed site specific data was available that allowed further layer refinement.

First of all, a finite element mesh was constructed to represent the region being modeled and the hydrogeologic conceptual layers (see Figure C.5). The material properties were then assigned to each element in the mesh. After the construction of the mesh and the assignment of material properties, boundary conditions were applied to the mesh. The boundaries represent sources or sinks for groundwater. The SEEP2D models mainly utilize constant head boundaries, flux boundaries, no-flow boundaries and flow boundaries. Constant head boundaries are typically used to represent standing bodies of water, such as impoundment and canal heads. However, flux, no-flow, and flow boundaries are also important for modeling certain situations. The section length corresponds to the location of a constant head boundary (seepage canal) or a groundwater divide (no-flow boundary).

The final product of the modeling was to determine seepage losses at the C-11 and C-9 impoundments. The seepage rate (in cubic feet per second per linear foot of section) was determined for a range of the impoundment water stage and the adjacent land water table to allow for more detailed design reviews. The seepage from the impoundment was partly collected or captured by the perimeter seepage canals. The model will also determine the quantity of seepage captured by the seepage canals and the quantity of seepage that bypasses the seepage canals and moves into the adjacent area. The following sections described the model results for each design region.

It is to be noted that the SEEP2D model developed here could not be calibrated because no field data was available for calibration.

3.6.4. C-11 Impoundment SEEP2D Model

To estimate seepage losses, nine cross-sections were modeled for the C-11 impoundment. The cross sections include five cross sections on the western, southern and eastern levees of the C-11 impoundment, and four cross sections on the western, northern and eastern levees of the C-11 mitigation area (see Figure C.1 and Figure C.2). Features included in the sections were as follows:

- Section 1 – C-11 Canal, located along the southern toe of the modeled levee segment, invert elevation at -15.0 feet. An excavation borrow area is located along the northern toe of the modeled levee segment, 800-foot bottom width, invert elevation at -1.0 feet;

- Section 2 – C-511, the seepage collection canal for the C-11 impoundment, located along the eastern toe of the modeled levee segment, 40-foot bottom width, invert elevation at -10.0 feet;
- Section 3A – C-511 located along the northeastern toe of the modeled levee segment of the mitigation area, 20-foot bottom width, invert elevation at -2.5 feet;
- Section 3B & 3C – C-511 located along the northern toe of the modeled levee segment of the mitigation area, 10-foot bottom width, invert elevation at -1.0 feet;
- Section 4 (from west to east) – a portion of the Seepage Management Area WCA-3A (SMA-3A), L-502A, C-502A (100-foot bottom width, invert elevation at -8.0 feet), US Hwy 27, C-511 (10-foot bottom width, invert elevation at -1.0 feet), the western impoundment levee and a portion of the main impoundment area itself;
- Section 5 (from west to east) – a portion of SMA-3A, L-502A, C-502A (100-foot bottom width, invert elevation at -8.0 feet), US Hwy 27, C-511 (10-foot bottom width, invert elevation at -1.0 feet), the western levee of the mitigation area and a portion of the mitigation area;
- Section 6 (from west to east) – a portion of SMA-3A, L-502A, C-502A (120-foot bottom width, invert elevation at -8.0 feet), US Hwy 27, S-504 discharge pool (200-foot average bottom width, invert elevation at -8.0 feet), the western impoundment levee and a portion of the main impoundment area;
- Section 7 (from west to east) – a portion of SMA-3A, L-502A, C-502A (120-foot bottom width, invert elevation at -8.0 feet), US Hwy 27, C-511 (10-foot bottom width, invert elevation at -1.0 feet), the western impoundment levee and a portion of the main impoundment area.

The perimeter levees, C-11 canal, seepage collection canals, and roads were modeled with side slopes, depths, and widths based on the cross-section information provided by Jacksonville District of U.S. Army COE and the WPA Feasibility Report, Geotechnical Appendix.

The vertical boundaries located at the ends of each cross section were chosen at distances where constant heads in the impoundment no longer influenced the groundwater seepage rate. The ends of the cross sections were modeled as open flow, constant head boundaries.

The seepage collection canals, pools, existing canals and impoundment and mitigation areas were treated as constant head boundaries. The detailed head values on the constant head boundaries are listed in Tables C.2 through C.15, for the C-11 impoundment.

The hydrogeology for the sections was based on data collected for the conceptual model. Specifically, wells numbered G-2311 and G-2321 provided data in close proximity to the impoundment (see Figure 3.1 and Table C.1). Interpretation of the data through the conceptual model development resulted in two pertinent soil layers.

Two soil layers were modeled below the impoundment levee template with bottom elevations at -13 and -70 feet, respectively. The hydraulic conductivity for each layer was 50 and 23,000 ft/d in the horizontal direction, and 5 and 2,300 ft/d in the vertical direction, respectively.

3.6.5. C-9 Impoundment SEEP2D Model

Seven cross-sections were modeled for the C-9 impoundment. The cross sections include five cross sections on the levee segments of the C-9 impoundment, and 2 cross sections on the levee segments of the C-9 mitigation storage area (see Figure C.3 and Figure C.4). Features included in the sections were as follows:

- Section 1A – the improved C-9 canal, located along the southern toe of the modeled levee segment, 50-foot bottom width, invert elevation at -16.5 feet, and an excavation borrow area located along the northern toe of the modeled levee segment, 800-foot bottom width, invert elevation at -2.0 feet;
- Section 1B – the existing C-9 canal, located along the southern toe of the modeled levee segment, 20-foot bottom width, invert elevation at -11.0 feet;
- Section 2 – C-509, the seepage collection canal of the C-9 impoundment, located along the eastern toe of the modeled levee segment, 20-foot bottom width, invert elevation at -4.5 feet;
- Section 3 – C-509 located along the northern toe of the modeled levee segment, 20-foot bottom width, invert elevation at -4.5 feet;
- Section 4 (from west to east) – a portion of the Seepage Management Area WCA-3B (SMA-3B), L-502B, C-502B (130-foot bottom width, invert elevation at -10.0 feet), US Hwy 27, C-509 (10-foot bottom width, invert elevation at -1.0 feet), the western impoundment levee and a portion of the main impoundment area itself;

- Section 5 (from north to south) – north land area, C-509 (10-foot bottom width, invert elevation at -2.0 feet), the northern levee of the mitigation area, the entire mitigation area, the levee between the impoundment and the mitigation area, and a portion of the main impoundment area itself;
- Section 6 (from west to east) – a portion of SMA-3B, L-502B, C-502B (130-foot bottom width, invert elevation at -10.0 feet), US Hwy 27, C-509 (10-foot bottom width, invert elevation at -1.0 feet), the western levee of the mitigation area, the mitigation area itself, C-509 (10-foot bottom width, invert elevation at -2.0 feet) and east land area.

The perimeter levees, improved C-9 canal, existing C-9 canal, seepage collection canals, and roads were modeled with side slopes, depths, and widths based on the cross-section information provided by Jacksonville District of U.S. Army COE and the WPA Feasibility Report, Geotechnical Appendix.

The hydrogeologic parameters for the cross sections were based on data used for the conceptual model. Specifically, well numbered G-2317 provided data in close proximity to the impoundment (see Figure 3.1 and Table C.1). Well G-3294 was also considered in developing the seepage models; however, using the appropriate data from G-3294 would have resulted in a less conservative estimate of seepage losses. Interpretation of the G-2317 data through the conceptual model development resulted in two pertinent soil layers.

Two soil layers were modeled below the impoundment levee template with bottom elevations at -11 and -70 feet, respectively. The hydraulic conductivity for each layer was 50 and 19,500 ft/d in the horizontal direction, and 5 and 1,950 ft/d in the vertical direction, respectively.

The vertical boundaries located at the ends of each cross section were chosen at distances where constant heads in the impoundment no longer influenced the groundwater seepage rate. The ends of the cross sections were modeled as open flow, constant head boundaries. The seepage collection canals, pools, existing canals and impoundment and mitigation areas were treated as constant head boundaries.

3.6.6. Scenarios of SEEP2D Model

The SEEP2D models for the C-11 impoundment and the C-9 impoundment were developed as discussed above. Two scenarios were designed to establish confidence bounds on the seepage rate from the impoundments. One scenario (Scenario A) was defined as the wet season condition (e.g. the adjacent water table at high wet season levels - about 2 feet below the ground surface), while in the second scenario (Scenario B), establish the adjacent water table at the ground surface. For both Scenario A and Scenario B, four different impoundment pool elevations were modeled for the C-11 impoundment and the C-9 impoundment, respectively. The water stage values for the impoundments,

Table 3.1 C-11 Impoundment Stage Summary

Cross Section	Simulated Water Level (ft NGVD)							
	Scenario A (in Wet Season)				Scenario B (Highest Water Table)			
Section 1								
C-11 Impoundment	10	11	12	13	10	11	12	13
C-11 Canal	4	4	4	4	4	4	4	4
South End of Model	4	4	4	4	6	6	6	6
Section 2								
C-11 Impoundment	10	11	12	13	10	11	12	13
C-511	4	4	4	4	4	4	4	4
East End of Model	3.5	3.5	3.5	3.5	5.5	5.5	5.5	5.5
Section 3A								
C-11 Impoundment	10	11	12	13	10	11	12	13
C-11 Mitigation Area	8.5	8.5	8.5	8.5	8.5	8.5	8.5	8.5
C-511	5	5	5	5	5	5	5	5
East End of Model	4	4	4	4	6	6	6	6
Section 3B/3C								
C-11 Impoundment	10	11	12	13	10	11	12	13
C-11 Mitigation Area	8.5	8.5	8.5	8.5	8.5	8.5	8.5	8.5
C-511	5	5	5	5	5	5	5	5
North End of Model	4.5	4.5	4.5	4.5	6.5	6.5	6.5	6.5
Section 4								
C-11 Impoundment	10	11	12	13				
C-511	5	5	5	5	Same as Scenario A			
C-502A	7	7	7	7				
SMA-3A	7.5	7.5	7.5	7.5				
Section 5								
C-11 Mitigation Area	8	8.5						
C-511	5	5			Same as Scenario A			
C-502A	7	7						
SMA-3A	7.5	7.5						
Section 6								
C-11 Impoundment	10	11	12	13				
S-504 Discharge Pool	7.5	7.5	7.5	7.5	Same as Scenario A			
C-502A	7	7	7	7				
SMA-3A	7.5	7.5	7.5	7.5				
Section 7								
C-11 Impoundment	10	11	12	13				
C-511	4	4	4	4	Same as Scenario A			
C-502A	7	7	7	7				
SMA-3A	7.5	7.5	7.5	7.5				

mitigation areas, canals and adjacent groundwater are listed in Table 3.1 and Table 3.2, for all cross sections of the C-11 and C-9 impoundments. In Table 3.1, Sections 1, 2, 3A, 3B, and 3C were modeled using different adjacent groundwater table elevations for both Scenarios A and Scenarios B. Sections 4, 5, 6, and 7 were modeled using the same adjacent groundwater table elevations for both scenarios. In Table 3.2, Sections 1A, 1B, 2, 3, and 5 were modeled using different adjacent groundwater table elevations for both Scenarios A and Scenarios B. Sections 4 and 6 were modeled using the same adjacent groundwater table elevations for both scenarios.

Table 3.2 C-9 Impoundment Stage Summary

Cross Section	Simulated Water Level (ft NGVD)							
	Scenario A (in Wet Season)				Scenario B (Highest Water Table)			
Section 1A/1B								
C-9 Impoundmen	8.5	9.5	10.5	11.5	8.5	9.5	10.5	11.5
C-9 Canal	4	4	4	4	4	4	4	4
South End of Model	2	2	2	2	4	4	4	4
Section 2								
C-9 Impoundmen	8.5	9.5	10.5	11.5	8.5	9.5	10.5	11.5
C-509	3	3	3	3	3	3	3	3
East End of Model	2	2	2	2	4	4	4	4
Section 3								
C-9 Impoundmen	8.5	9.5	10.5	11.5	8.5	9.5	10.5	11.5
C-509	3	3	3	3	3	3	3	3
North End of Model	2.5	2.5	2.5	2.5	4.5	4.5	4.5	4.5
Section 4								
C-9 Impoundment	8.5	9.5	10.5	11.5				
C-509	5	5	5	5	Same as Scenario A			
C-502B	6	6	6	6				
SMA-3B	6.5	6.5	6.5	6.5				
Section 5								
C-9 Impoundment	8.5	9.5	10.5	11.5	8.5	9.5	10.5	11.5
C-9 Mitigation Area	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
C-509	3	3	3	3	3	3	3	3
North End of Model	2.5	2.5	2.5	2.5	4.5	4.5	4.5	4.5
Section 6								
C-9 Mitigation Area	6	6.5						
C-509 (East)	3	3						
C-509 (West)	5	5			Same as Scenario A			
East End of Model	2.5	2.5						
C-502B	6	6						
SMA-3B	6.5	6.5						

3.7. Seepage Analysis Results

3.7.1. C-11 Impoundment

Seepage values were determined for both scenarios (A and B) for the nine cross sections modeled for the C-11 impoundment. As defined previously, the only difference between the two scenarios is the adjacent groundwater table elevation. The results of the seepage analysis are presented in Tables C.2 through C.10 for Scenario A, and Tables C.11 through C.15 for Scenario B. Note that only Sections 1, 2, 3A, 3B, and 3C were modeled and evaluated for Scenario A and Scenario B.

- **Scenario A**

The results of the seepage modeling for Scenario A (the adjacent water table at high wet season levels, e.g., 2 feet below the ground surface) are summarized in Table 3.3 based on the detailed results shown in Tables C.2 through C.10.

With the first Section, four different impoundment pool elevations were modeled starting with a normal pool elevation of 10.0 feet and ending with a surcharge pool elevation of 13.0 feet. In all four models, the water surface elevation in the C-11 canal was maintained at 4.0 feet and the adjacent groundwater elevation (to the south) was modeled at 4.0 feet (see Table C.2). The results from each model indicate the seepage losses from the impoundment range from a low of approximately 0.042 cubic feet per second per linear foot (cfs/ft) to a high of approximately 0.063 cfs/ft. The percentage of seepage loss captured by the C-11 canal is approximately 97 percent, and the remaining 3 percent will move south and out of the model. This split in seepage flow is reasonable given the modeled invert elevation of the C-11 canal being set below the top elevation of the second soil layer modeled. Compared with the seepage results from other cross sections, it is concluded that the C-11 canal is a very effective seepage collector.

With the second Section, the same impoundment pool elevations were modeled as in Section 1. In all four models, the water surface elevation in C-511 and the adjacent groundwater elevation (to the east) were modeled at 4.0 feet and 3.5 feet, respectively. The results shown in Table C.3 indicate that the seepage losses from the impoundment range from a low of approximately 0.027 cfs/ft to a high of approximately 0.039 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 11 percent, and the remaining 89 percent will move east and out of the model. In terms of seepage collection, C-511 in this cross section did not perform as well as the C-11 canal in Section 1. In Section 1, C-11 had a direct hydraulic connection to the deeper, more permeable soil layer due to its invert elevation at -15.0 feet. While in Section 2, C-511 did not have this direct hydraulic connection with an invert elevation of -10.0 feet, and as a result most of the seepage bypassed the canal and proceeded east and out of the model via the lower soil layer.

Sections 3A, 3B, and 3C, located along the northeast and north side of the impoundment and mitigation storage area, were modeled with the similar geometry meshes, boundary conditions and other parameters; they were therefore discussed below as a group. For these three cross sections, the same impoundment pool elevations were modeled as in Section 1. In all twelve models, the water surface elevations in the mitigation area and C-511were modeled at 8.5 feet and 5.0 feet, respectively. The adjacent groundwater elevations were kept at 4.0 feet in Section 3A, and 4.5 feet for Sections 3B and 3C. The results in Tables C.4 through C.6 indicate that the seepage losses from the impoundment and the mitigation area range from a low of approximately 0.019 cfs/ft to a high of approximately 0.030 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 3 percent, and the remaining 97 percent will move north and northeast and out of the model. As with Section 2, the invert elevation of the seepage canal was too high above the second, more permeable soil layer to be a factor in capturing any of the seepage flow leaving the impoundment and the mitigation area.

Section 4 and Section 7, located along the western levee segments of the impoundment, were modeled with the similar geometry meshes, boundary conditions and other parameters, and as a result they were grouped and discussed below. For these two cross sections, the same impoundment pool elevations were modeled as in Section 1. In all eight models, the water surface elevations in C-502A and SMA-3A were modeled at 7.0 feet, and 7.5 feet, respectively. The water surface elevations in C-511 were kept at 5.0 feet in Section 4, and 4.0 feet for Section 7. The results in Table C.7 and Table C.10 indicate that the seepage losses from the impoundment range from a low of approximately 0.010 cfs/ft to a high of approximately 0.023 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 12 percent, and the remaining 88 percent will move west and out of the model. As with Section 2, the invert elevation of the seepage canal was too high above the second, more permeable soil layer to be a factor in capturing any of the seepage flow leaving the impoundment. Also note that C-511 in Section 7 captured more seepage loss than C-511 in Section 4 because of the lower water surface elevation.

With Section 5, located along the western levee segments of the mitigation area, two mitigation area pool elevations (8.0 feet and 8.5 feet) were modeled. In these two models, the water surface elevations in C-511, C-502A and SMA-3A were modeled at 5.0 feet, 7.0 feet and 7.5 feet, respectively. The results shown in Table C.8 indicate that the seepage losses from the mitigation area are approximately 0.004 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 30 percent, and the remaining 70 percent will move west and out of the model. The seepage loss from this section of the mitigation area is less than other sections as the water elevation in the mitigation area is below the water elevation of the impoundment area. For further analysis on the total seepage loss of the impoundment and the seepage collection capability of the seepage collection canals, the results of the second model with higher water elevation modeled in the mitigation area were used in order to avoid the conservative seepage loss from the impoundment.

With Section 6, the same impoundment pool elevations were modeled as in Section 1. In all four models, the water surface elevations in C-502A and SMA-3A were modeled at 7.0 feet, and 7.5 feet, respectively. The water surface elevation in S-504 discharge pool was kept at 7.5 feet. The results in Table C.9 indicate that the seepage losses from the impoundment range from a low of approximately 0.010 cfs/ft to a high of approximately 0.021 cfs/ft. The percentage of seepage loss captured by S-504 discharge pool is approximately 20 percent, and the remaining 80 percent will move west and out of the model. The invert elevation of S-504 discharge pool was modeled at -8.0 feet, which means the discharge pool can capture more seepage loss than C-511 in Section 4 and Section 7 discussed previously. Also, further subsurface investigation is recommended to examine the site hydrogeology in more detail.

Total seepage losses (including the water captured by the seepage collection canal) from the C-11 impoundment (and mitigation storage area) estimated based on the seepage rate are presented in Figure 3.2. For this scenario, the seepage losses from the impoundment range from a low of approximately 834.24 cfs to a high of approximately 1,281.31 cfs.

Parts of seepage losses from the impoundment were captured by the perimeter seepage collection canal, and the total seepage rates into C-511 are summarized in Table 3.4 and also plotted in Figure 3.3. The seepage rates into C-511 range from a low of approximately 54.92 cfs to a high of approximately 80.24 cfs.

Table 3.3 Seepage Losses from C-11 Impoundment at Various Impoundment Stages for Scenario A

Section	Levee Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
		10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
1	5,000	0.04205	0.04906	0.05607	0.06308	210.24	245.29	280.34	315.40
2	11,745	0.02652	0.03065	0.03481	0.03899	311.52	359.98	408.83	457.93
3A	2,965	0.02196	0.02469	0.02743	0.03020	65.12	73.21	81.33	89.53
3B	3,740	0.02028	0.02298	0.02567	0.02839	75.86	85.93	96.00	106.18
3C	2,575	0.01857	0.02011	0.02165	0.02320	47.81	51.78	55.76	59.73
4	8,290	0.01082	0.01491	0.01899	0.02308	89.69	123.57	157.45	191.34
5	2,000	0.00474	0.00474	0.00474	0.00474	9.48	9.48	9.48	9.48
6	1,075	0.00968	0.01346	0.01724	0.02102	10.41	14.47	18.54	22.60
7	1,390	0.01015	0.01375	0.01734	0.02094	14.11	19.11	24.11	29.11
Total	**38,780**					**834.24**	**982.82**	**1,131.83**	**1,281.31**

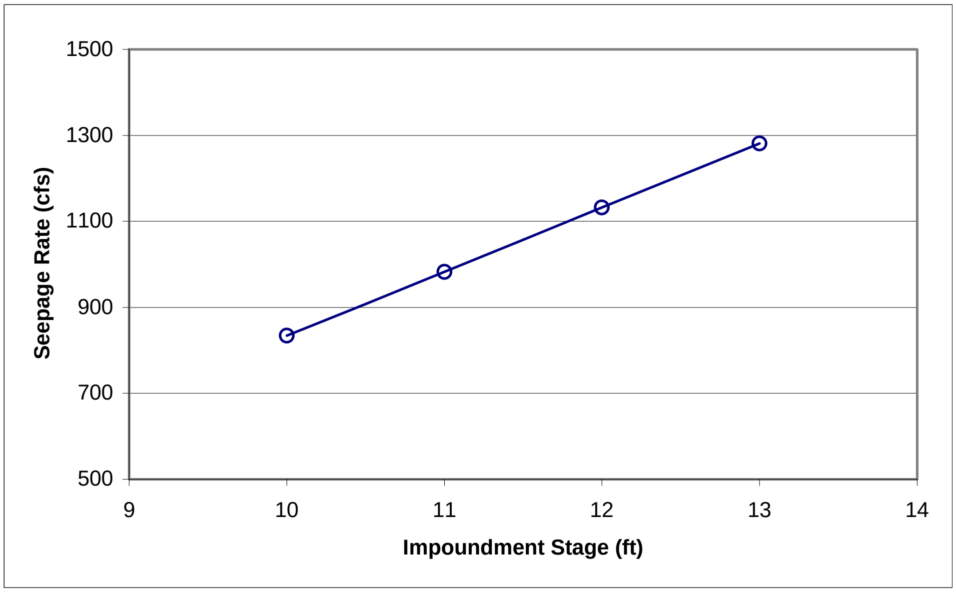

Figure 3.2 Seepage Losses from C-11 Impoundment on Various Impoundment Stages for Scenario A

Table 3.4 Seepage into C-511 at Various Impoundment Stages for Scenario A

Location	Section	Canal Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
			10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
C-11 Impoundment										
East C-511	2	11,745	0.00280	0.00335	0.00389	0.00442	32.87	39.37	45.71	51.93
West C-511	4	8,290	0.00134	0.00146	0.00158	0.00169	11.10	12.09	13.06	14.02
South C-511	7	1,390	0.00171	0.00182	0.00193	0.00204	2.38	2.53	2.69	2.84
C-11 Mitigation Storage Area										
East C-511	3A	2,965	0.00072	0.00088	0.00103	0.00117	2.15	2.60	3.04	3.47
North C-511	3B	3,740	0.00067	0.00077	0.00087	0.00097	2.50	2.88	3.26	3.62
North C-511	3C	2,575	0.00060	0.00066	0.00072	0.00078	1.56	1.71	1.85	2.00
West C-511	5	2,000	0.00118	0.00118	0.00118	0.00118	2.36	2.36	2.36	2.36
Total		**32,705**					**54.92**	**63.54**	**71.97**	**80.24**

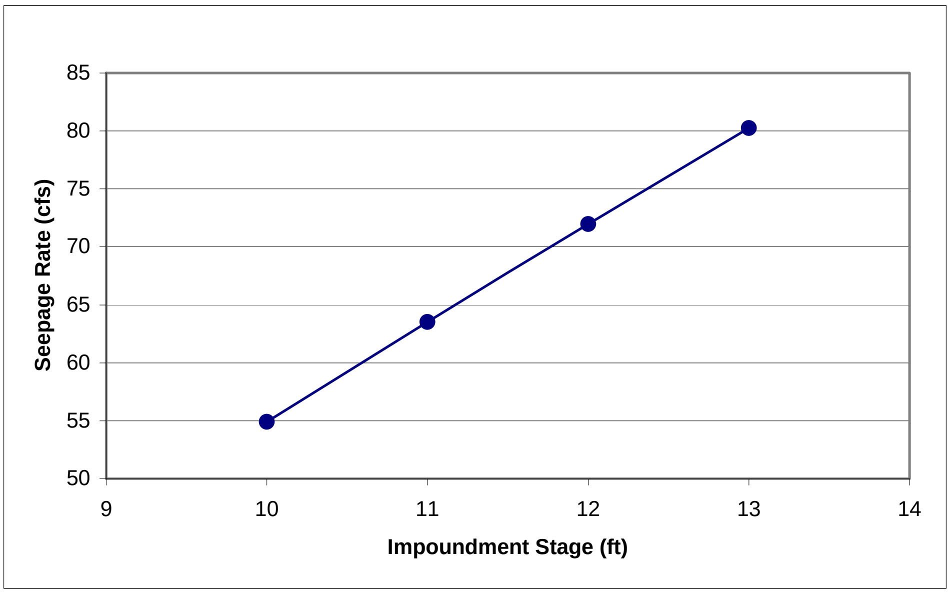

Figure 3.3 Seepage into C-511 at Various Impoundment Stages for Scenario A

- **Scenario B**

The results of the seepage analysis for Scenario B (the adjacent water table is at the ground surface) are summarized in Table 3.5 based on the detailed results shown in Tables C.11 through C.15. As discussed above, Sections 1, 2, 3A, 3B, and 3C were modeled again for Scenario B.

With the first Section, the water surface elevations in the impoundment and the C-11 canal were modeled as Scenario A. The only difference is that the adjacent groundwater table (to the south) was modeled at 6.0 feet, the ground surface elevation. The results from each model indicate the seepage losses from the impoundment range from a low of approximately 0.042 cfs/ft to a high of approximately 0.063 cfs/ft. The seepage losses from the impoundment were completely captured by the C-11 canal, and no water will move south and out of the model. Compared with the results of Scenario A, the increased adjacent groundwater table not only blocks the seepage loss from the impoundment into the south land, but also works as another seepage source for C-511.

For Section 2, the adjacent groundwater elevation (to the east) was modified from 3.5 feet of Scenario A to 5.5 feet. The results shown in Table C.12 indicate that the seepage losses from the impoundment range from a low of approximately 0.020 cfs/ft to a high of approximately 0.032 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 21 percent, and the remaining 79 percent will move east and out of the model. Due to the increase of the adjacent groundwater table, the seepage loss from the impoundment was reduced in quantity, but more seepage was captured by C-511 and less seepage bypassed the seepage canals to the adjacent area (see Table C.3 and Table C.12).

Sections 3A, 3B, and 3C were also remodeled based on the assumption of Scenario B. The adjacent groundwater elevations were kept at 6.0 feet in Section 3A, and 6.5 feet for Section 3B and Section 3C. The results in Tables C.13 through C.15 indicate that the seepage losses from the impoundment and the mitigation area range from a low of approximately 0.011 cfs/ft to a high of approximately 0.023 cfs/ft. The percentage of seepage loss captured by C-511 is approximately 13 percent, and the remaining 87 percent will move north and northeast and out of the model. For the same reason discussed in Section 2, the seepage loss from the impoundment was lower, while more seepage was collected by C-511.

Total seepage losses from the C-11 impoundment (and mitigation storage area) for Scenario A were estimated based on the seepage rate in Table 3.5 and then plotted in Figure 3.4. For this scenario, the seepage losses from the impoundment range from a low of approximately 685.09 cfs to a high of approximately 1,137.75 cfs. The total seepage losses from the impoundment for Scenario B were observed to be lower than those for Scenario A, at four different impoundment pool elevations.

The total seepage rates into C-511 are summarized in Table 3.6 and plotted in Figure 3.5. The seepage rates into C-511 range from a low of approximately 82.46 cfs to a high of approximately 106.02 cfs. The total seepage collected by C-511 significantly changed when the adjacent groundwater table elevation was increased to the ground surface in Sections 1, 2, 3A, 3B, and 3C.

Table 3.5 Seepage Losses from C-11 Impoundment at Various Impoundment Stages for Scenario B

Section	Levee Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
		10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
1	5,000	0.04169	0.04870	0.05571	0.06272	208.45	243.51	278.56	313.62
2	11,745	0.01977	0.02399	0.02822	0.03245	232.15	281.76	331.41	381.07
3A	2,965	0.01484	0.01766	0.02047	0.02329	44.00	52.35	60.70	69.06
3B	3,740	0.01289	0.01568	0.01847	0.02126	48.20	58.63	69.07	79.51
3C	2,575	0.01111	0.01270	0.01430	0.01590	28.60	32.72	36.83	40.95
4	8,290	0.01082	0.01491	0.01899	0.02308	89.69	123.57	157.45	191.34
5	2,000	0.00474	0.00474	0.00474	0.00474	9.48	9.48	9.48	9.48
6	1,075	0.00968	0.01346	0.01724	0.02102	10.41	14.47	18.54	22.60
7	1,390	0.01015	0.01375	0.01734	0.02094	14.11	19.11	24.11	29.11
Total	**38,780**					**685.09**	**835.59**	**986.16**	**1136.75**

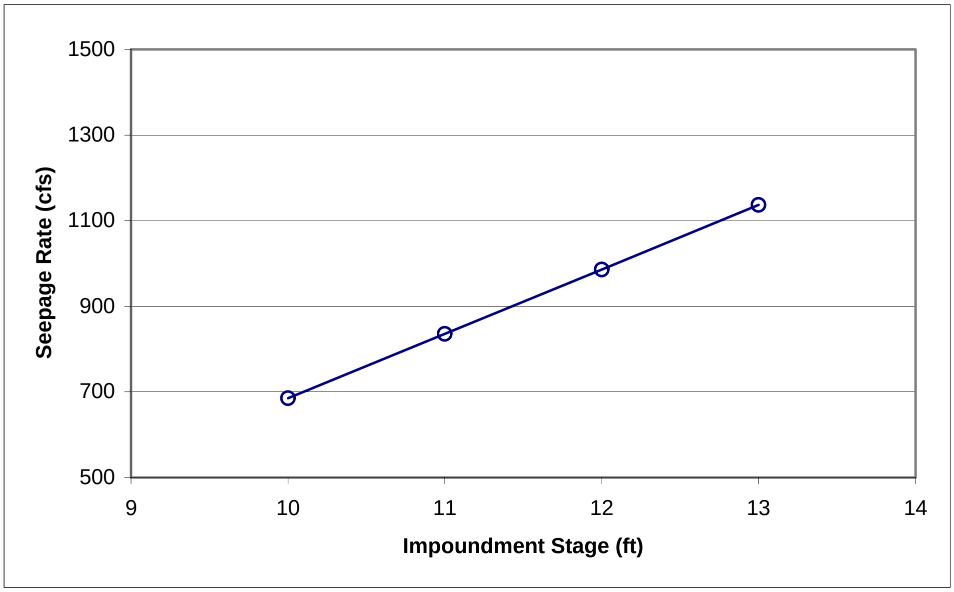

Figure 3.4 Seepage Losses from C-11 Impoundment on Various Impoundment Stages for Scenario B

Table 3.6 Seepage into C-511 at Various Impoundment Stages for Scenario B

Location	Section	Canal Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
			10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
C-11 Impoundment										
East C-511	2	11,745	0.00471	0.00522	0.00572	0.00621	55.27	61.28	67.13	72.98
West C-511	4	8,290	0.00134	0.00146	0.00158	0.00169	11.10	12.09	13.06	14.02
South C-511	7	1,390	0.00171	0.00182	0.00193	0.00204	2.38	2.53	2.69	2.84
C-11 Mitigation Storage Area										
East C-511	3A	2,965	0.00145	0.00158	0.00171	0.00184	4.29	4.68	5.07	5.46
North C-511	3B	3,740	0.00114	0.00123	0.00131	0.00139	4.27	4.58	4.89	5.18
North C-511	3C	2,575	0.00109	0.00113	0.00118	0.00123	2.79	2.92	3.05	3.17
West C-511	5	2,000	0.00118	0.00118	0.00118	0.00118	2.36	2.36	2.36	2.36
Total		**32,705**					**82.46**	**90.45**	**98.25**	**106.02**

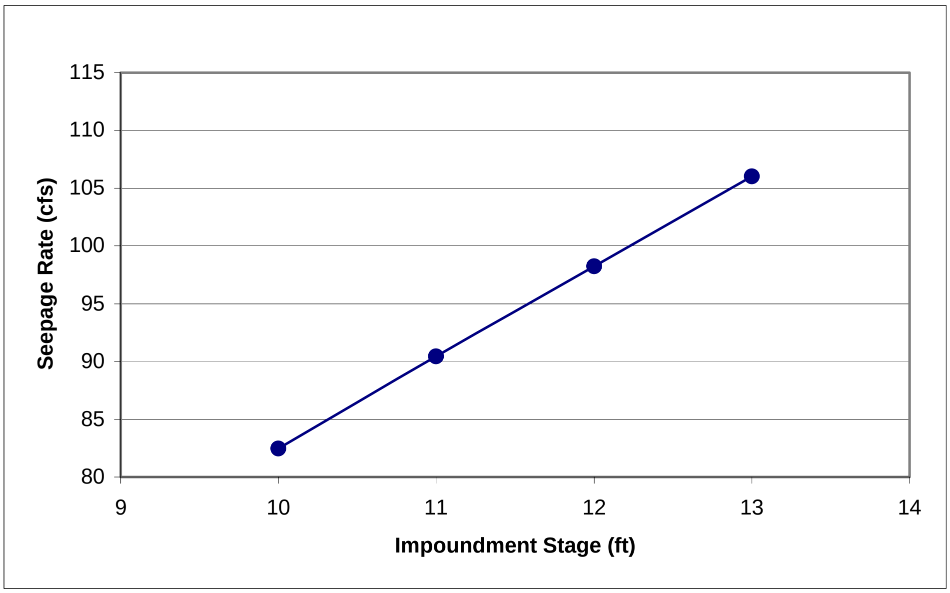

Figure 3.5 Seepage into C-511 at Various Impoundment Stages for Scenario B

3.7.2. C-9 Impoundment

Seepage values were determined for both scenarios (A and B) for the seven cross sections modeled for the C-9 impoundment. The only difference between the scenarios is the adjacent groundwater table elevation. The results of the seepage analysis are presented in Tables C.16 through C.22 for Scenario A, and Tables C.23 through C.28 for Scenario B. Note that the adjacent groundwater table were not modeled in all cross sections, and Sections 1A, 1B, 2, 3, 5, and 6 of the C-9 impoundment were modeled and evaluated for both Scenario A and Scenario B.

- **Scenario A**

The results of the seepage modeling for Scenario A are summarized in Table 3.7 based on the detailed results shown in Tables C.16 through C.22.

With Section 1A, four different impoundment pool elevations were modeled starting with a normal pool elevation of 8.5 feet and ending with a surcharge pool elevation of 11.5 feet. In all four models, the water surface elevation in the improved C-9 canal was maintained at 4.0 feet and the adjacent groundwater elevation (to the south) was modeled at 2.0 feet (see Table C.16). The results from each model indicate the seepage losses from the impoundment range from a low of approximately 0.034 cfs/ft to a high of approximately 0.057 cfs/ft. The percentage of seepage loss captured by the improved C-9 canal is approximately 65 percent, and the remaining 35 percent will move south and out of the model. This split in seepage flow is reasonable given the modeled invert elevation of the improved C-9 canal being set at -16.5 feet, which is below the top elevation of the second soil layer modeled of -11.0 feet. Compared with the seepage results from other cross sections, it is concluded that the improved C-9 Canal is a very effective seepage collector.

With Section 1B, using the same water elevations of the impoundment, the existing C-9 canal and the adjacent groundwater table were modeled as in Section 1A. In all four models, the only difference in Section 1B was the existing C-9 canal was modeled with narrower 20-foot bottom of and higher invert elevation of -11.0 feet. The results shown in Table C.17 indicate that the seepage losses from the impoundment range from a low of approximately 0.029 cfs/ft to a high of approximately 0.047 cfs/ft. The percentage of seepage loss captured by the existing C-9 canal is approximately 55 percent, and the remaining 45 percent will move south and out of the model. Due to the improvement project applied on the existing C-9 canal, the improved C-9 canal in Section 1A collected more seepage loss from the impoundment than the existing C-9 canal did, and as a result the seepage loss from the impoundment also increased.

With Section 2, the same impoundment pool elevations were modeled as in Section 1A. In all four models, the water surface elevation in C-509 and the adjacent groundwater elevation (in east land) were modeled at 3.0 feet and 2.0 feet, respectively. The results shown in Table C.18 indicate that the seepage losses from the impoundment range from a low of approximately 0.026 cfs/ft to a high of approximately 0.038 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 5 percent, and the remaining 95 percent will move east and out of the model. In terms of seepage collection, C-509 in Section 2 did not perform as well as the (improved/existing) C-9 canal in Sections 1A and 1B. In Sections 1A and 1B, the C-9 canal had a direct hydraulic connection to the deeper, more permeable soil layer. In Section 2, C-509 did not have such direct hydraulic connection with an invert elevation of -4.5 feet, and so most of the seepage bypassed the seepage canal and proceeded to east and out of the model via the lower soil layer.

With Section 3, the same impoundment pool elevations were modeled as in Section 1A. In all four models, the water surface elevation in C-509 and the adjacent groundwater elevation (to the east) were modeled at 3.0 feet and 2.5 feet, respectively. The results in Table C.19 indicate that the seepage losses from the impoundment range from a low of approximately 0.024 cfs/ft to a high of approximately 0.037 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 7 percent, and the remaining 93 percent will move north and out of the model. As with Section 2, the invert elevation of the seepage canal was too high above the second, more permeable soil layer to be a factor in capturing any of the seepage flow leaving the impoundment.

With Section 4, the same impoundment pool elevations were modeled as in Section 1A. The water surface elevations in C-509, C-502B and SMA-3B were modeled at 5.0 feet, 6.0 feet, and 6.5 feet, respectively. The results in Table C.20 indicate that the seepage losses from the impoundment range from a low of approximately 0.010 cfs/ft to a high of approximately 0.023 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 7 percent, and the remaining 93 percent will move west and out of the model. As with Section 2, the invert elevation of the seepage canal was too high above the second, more permeable soil layer to be a factor in capturing any of the seepage flow leaving the impoundment.

With Section 5, the same impoundment pool elevations were modeled as in Section 1A. In all four models, the water surface elevations in the mitigation area and C-509 were modeled at 6.5 feet and 3.0 feet, respectively, and the adjacent groundwater table elevation was kept at 2.5 feet. The results shown in Table C.21 indicate that the seepage losses from the impoundment are approximately 0.023 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 1 percent, and the remaining 99 percent will move north and out of the model. The impoundment with higher water elevation was located about 7,000 ft south of C-509 so that the change of the impoundment water elevation is not considered to be a factor in seepage analysis. The small head difference between the mitigation area and C-509 can explain why only 1 percent of seepage was captured by C-509.

With Section 6, two pool elevations of the mitigation area, 6.0 feet and 6.5 feet, were modeled. In these two models, the water surface elevations in the eastern C-509 canal, the western C-509 canal, C-502B and SMA-3B were modeled at 3.0 feet, 5.0 feet, 6.0 feet, and 6.5 feet, respectively, and the adjacent groundwater table (in east land) was kept at 2.5 feet. The results in Table C.22 indicate that the seepage losses from the impoundment range from a low of approximately 0.012 cfs/ft to a high of approximately 0.015 cfs/ft. The percentage of seepage loss captured by the eastern C-509 canal and the western C-509 canal are approximately 1.5 percent and 0.5 percent, respectively, and the remaining 98 percent will move east and out of the model. The SMA-3B area at high water stage pushed most of the seepage loss from the mitigation area to east land, and the seepage captured by the east and west C-509 canals can therefore be ignored. In addition, further subsurface investigation should be required to examine the site hydrogeology in more detail. For further analysis of the total seepage loss of the impoundment and the seepage into seepage collection canal, the results of the second model were used in order to be consistent with other cross sections.

Total seepage losses (including the water captured by the seepage collection canal) from the C-9 impoundment (and mitigation storage area) were estimated based on the seepage rate in Table 3.7 and then plotted in Figure 3.6. For this scenario, the seepage losses from the impoundment range from a low of approximately 880.40 cubic feet per second to a high of approximately 1,355.53 cfs.

The seepage losses captured by the perimeter seepage collection canal can be evaluated based on the results in Tables C.16 through C.22, and the total seepage rates into C-509 are summarized in Table 3.8 and also plotted in Figure 3.9. The seepage rates into C-509 range from a low of approximately 34.81 cfs to a high of approximately 52.33 cfs.

- **Scenario B**

The results of the seepage modeling for Scenario B (the adjacent water table is at the ground surface) are summarized in Table 3.9 based on the detailed results shown in Tables C.23 through C.28. Sections 1A, 1B, 2, 3, 5, and 6 were remodeled for Scenario B and discussed below.

With Section 1A, the same water surface elevations in the impoundment and the improved C-9 canal were modeled as in Scenario A. The only difference is that the adjacent groundwater elevation (to the south) was modeled at 4.0 feet (see Table C.23). The results from each model indicate the seepage losses from the impoundment range from a low of approximately 0.034 cfs/ft to a high of approximately 0.057 cfs/ft. The percentage of seepage loss captured by the improved C-9 canal is approximately 97 percent, and the remaining 3 percent will move south and out of the model. Compared with the results of Scenario A, most of the seepage loss from the impoundment was captured by the improved C-9 canal due to the increase of the adjacent groundwater table.

Table 3.7 Seepage Losses from C-9 Impoundment at Various Impoundment Stages for Scenario A

Section	Levee Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
		8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
1A	5,065	0.03435	0.04190	0.04945	0.05699	174.01	212.22	250.45	288.67
1B	1,840	0.02860	0.03476	0.04093	0.04709	52.61	63.96	75.30	86.65
2	10,445	0.02554	0.02960	0.03370	0.03782	266.74	309.15	351.96	395.00
3	4,935	0.02423	0.02836	0.03254	0.03675	119.59	139.97	160.60	181.36
4	10,425	0.00967	0.01401	0.01835	0.02269	100.86	146.09	191.33	236.58
5	2,270	0.02249	0.02259	0.02269	0.02279	51.06	51.28	51.51	51.74
6	7,670	0.01506	0.01506	0.01506	0.01506	115.53	115.53	115.53	115.53
Total	**42,650**					**880.40**	**1038.20**	**1196.68**	**1355.53**

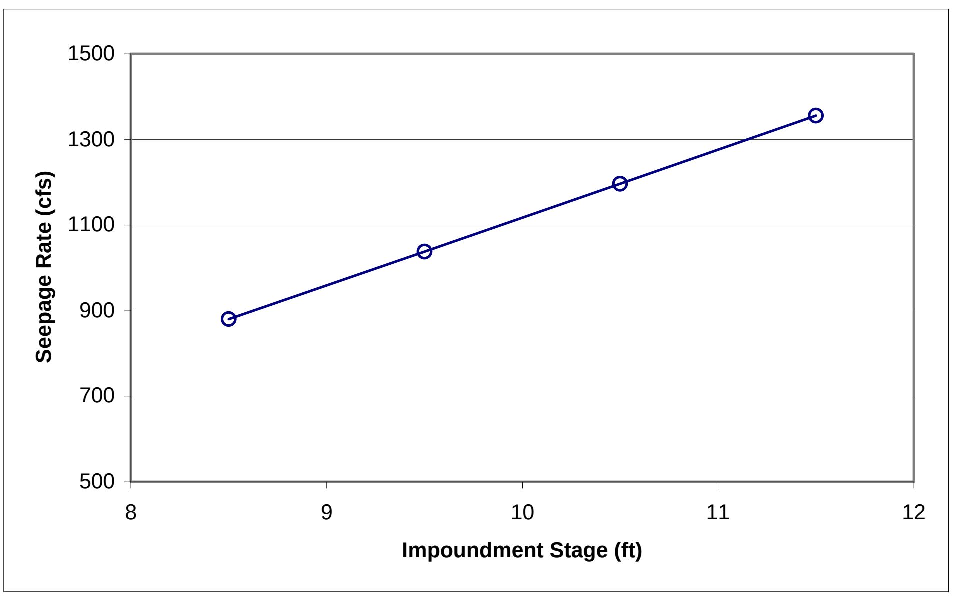

Figure 3.6 Seepage Losses from C-9 Impoundment on Various Impoundment Stages for Scenario A

Table 3.8 Seepage into C-509 at Various Impoundment Stages for Scenario A

Location	Section	Canal Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
			8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
C-9 Impoundment										
East C-509	2	10,445	0.00127	0.00157	0.00186	0.00213	13.25	16.42	19.40	22.26
North C-509	3	4,935	0.00157	0.00188	0.00217	0.00246	7.74	9.28	10.73	12.13
West C-509	4	10,425	0.00087	0.00100	0.00113	0.00126	9.05	10.43	11.79	13.16
C-9 Mitigation Storage Area										
East C-509	6	7,670	0.00022	0.00022	0.00022	0.00022	1.69	1.69	1.69	1.69
North C-509	5	2,270	0.00026	0.00026	0.00026	0.00026	0.58	0.58	0.59	0.59
West C-509	6	7,630	0.00033	0.00033	0.00033	0.00033	2.50	2.50	2.50	2.50
Total		**43,375**					**34.81**	**40.91**	**46.70**	**52.33**

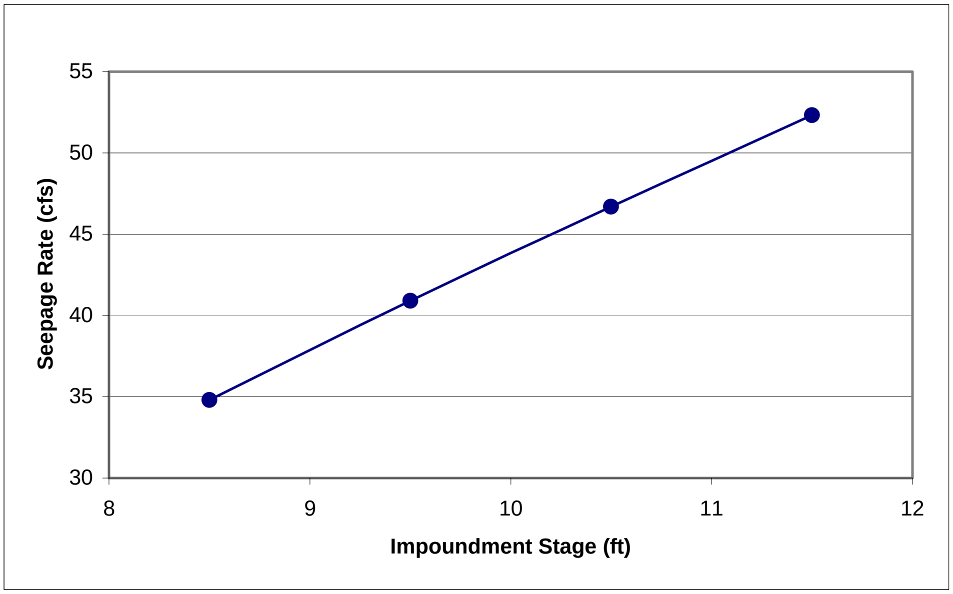

Figure 3.7 Seepage into C-509 at Various Impoundment Stages for Scenario A

With Section 1B, the same water elevations of the impoundment and the existing C-9 canal were modeled as in Scenario A. The only different is that the adjacent groundwater elevation (to the south) was remodeled at 4.0 feet. The results shown in Table C.24 indicate that the seepage losses from the impoundment range from a low of approximately 0.028 cfs/ft to a high of approximately 0.046 cfs/ft. The percentage of seepage loss captured by the existing C-9 canal is approximately 91 percent, and the remaining 9 percent will move south and out of the model. Compared with the results of Scenario A, more seepage loss from the impoundment was captured by the improved C-9 canal due to the same reason described for Section 1A.

With Section 2, the adjacent groundwater elevation was modified from 2.0 feet in Scenario A to 4.0 feet. The results shown in Table C.25 indicate that the seepage losses from the impoundment range from a low of approximately 0.021 cfs/ft to a high of approximately 0.029 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 10 percent, and the remaining 90 percent will move east and out of the model. Due to the increase of the adjacent groundwater table, the seepage loss from the impoundment was reduced in quantity, and more seepage water was forced into C-509.

With Section 3, the adjacent groundwater table was remodeled at 4.5 feet for Scenario B. The results in Table C.26 indicate that the seepage losses from the impoundment range from a low of approximately 0.017 cfs/ft to a high of approximately 0.030 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 12 percent, and the remaining 88 percent will move north and out of the model. Due to the increase of the adjacent groundwater table, the seepage loss from the impoundment was reduced in quantity, and more percentage of seepage was forced into C-509.

With Section 5, the adjacent groundwater table elevation was remodeled at 4.5 feet for Scenario B. The results shown in Table C.27 indicate that the seepage losses from the impoundment are approximately 0.012 cfs/ft. The percentage of seepage loss captured by C-509 is approximately 8 percent, and the remaining 92 percent will move north and out of the model. It is concluded that the seepage loss from the impoundment was reduced in quantity, and a higher percentage of seepage was forced into C-509, in comparison with the results of Scenario A.

With Section 6, the adjacent groundwater table was remodeled at 4.5 feet for Scenario B. The results in Table C.28 indicate that the seepage losses from the impoundment range from a low of approximately 0.0092 cfs/ft to a high of approximately 0.0096 cfs/ft. The percentage of seepage loss captured by the eastern C-509 canal and the western C-509 canal are approximately 15 percent and 1 percent, respectively, and the remaining 84 percent will move east and out of the model. Compared with the results of Scenario A, more seepage loss from the impoundment was captured by the eastern C-509 canal due to the increase of the adjacent groundwater table (to the east). Also the results of the second model (6.5 feet in the mitigation area) were used for further analysis in order to be consistent with other cross sections.

Total seepage losses (including the water captured by the seepage collection canal) from the C-9 impoundment (and mitigation storage area) were estimated based on the seepage rate in Table 3.9 and then plotted in Figure 3.8. For Scenario B, the seepage losses from the impoundment range from a low of approximately 695.71 cubic feet per second to a high of approximately 1,174.28 cfs.

The total seepage rates into C-509 are summarized in Table 3.10 and plotted in Figure 3.9. The seepage rates into C-509 range from a low of approximately 56.23 cfs to a high of approximately 72.51 cfs.

Table 3.9 Seepage Losses from C-9 Impoundment at Various Impoundment Stages for Scenario B

Section	Levee Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
		8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
1A	5,065	0.03395	0.04150	0.04905	0.05660	171.98	210.20	248.43	286.66
1B	1,840	0.02776	0.03393	0.04010	0.04628	51.08	62.44	73.79	85.15
2	10,445	0.01897	0.02313	0.02729	0.03145	198.09	241.55	285.02	328.50
3	4,935	0.01742	0.02168	0.02593	0.03019	85.99	106.98	127.99	148.99
4	10,425	0.00967	0.01401	0.01835	0.02269	100.86	146.09	191.33	236.58
5	2,270	0.01158	0.01168	0.01178	0.01188	26.29	26.52	26.75	26.98
6	7,670	0.00801	0.00801	0.00801	0.00801	61.42	61.42	61.42	61.42
Total	**42,650**					**695.71**	**855.20**	**1014.73**	**1174.28**

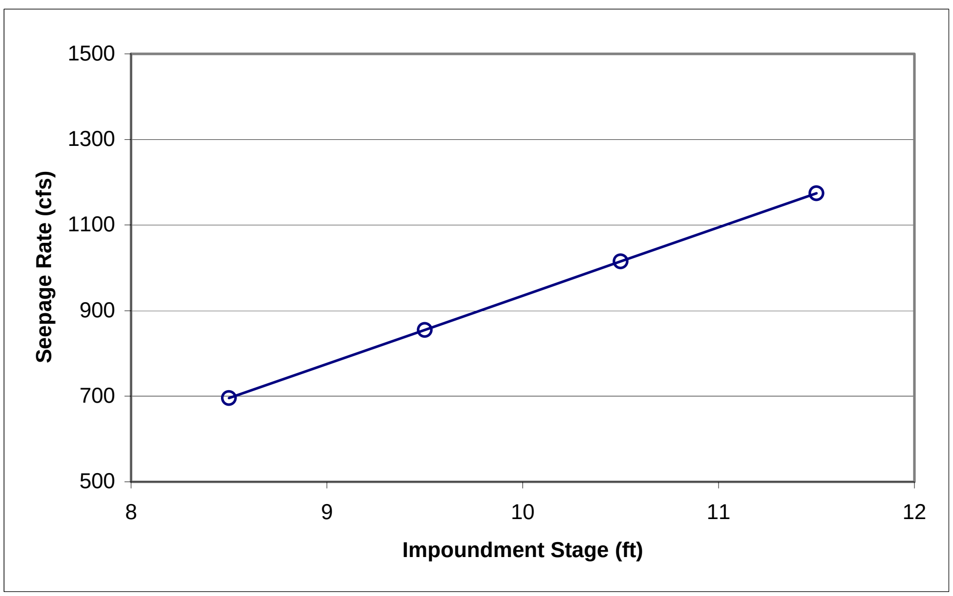

Figure 3.8 Seepage Losses from C-9 Impoundment on Various Impoundment Stages for Scenario B

Table 3.10 Seepage into C-509 at Various Impoundment Stages for Scenario B

Location	Section	Canal Length (ft)	Seepage Rate per Foot at different Impoundment Water Stage (cfs/ft)				Seepage Rate at different Impoundment Water Stage (cfs)			
			8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
C-9 Impoundment										
East C-509	2	10,445	0.00211	0.00237	0.00264	0.00290	22.05	24.77	27.54	30.26
North C-509	3	4,935	0.00242	0.00269	0.00296	0.00322	11.92	13.28	14.59	15.87
West C-509	4	10,425	0.00087	0.00100	0.00113	0.00126	9.05	10.43	11.79	13.16
C-9 Mitigation Storage Area										
East C-509	6	7,670	0.00096	0.00096	0.00096	0.00096	7.38	7.38	7.38	7.38
North C-509	5	2,270	0.00099	0.00099	0.00099	0.00099	2.24	2.24	2.24	2.25
West C-509	6	7,630	0.00047	0.00047	0.00047	0.00047	3.59	3.59	3.59	3.59
Total		**43,375**					**56.23**	**61.69**	**67.13**	**72.51**

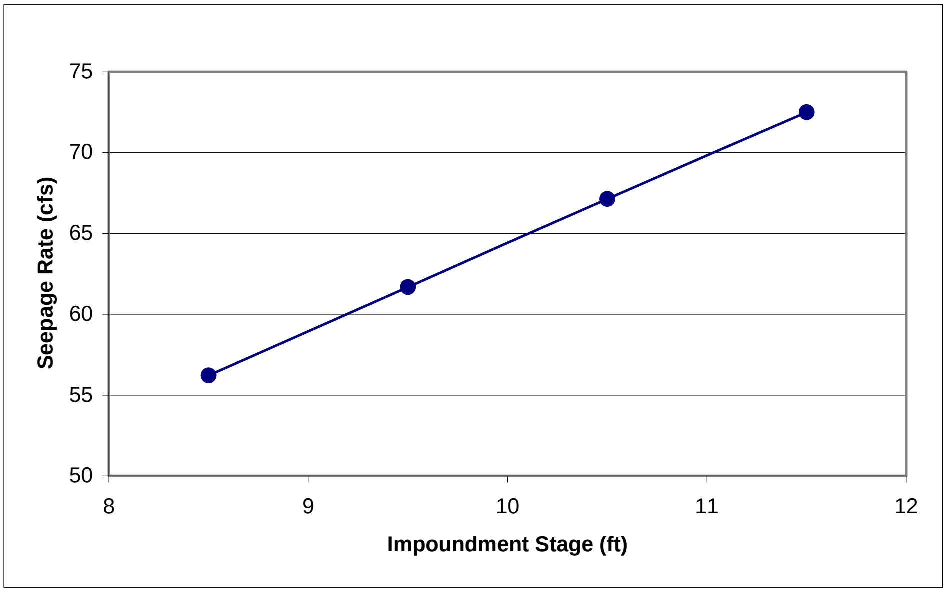

Figure 3.9 Seepage into C-509 at Various Impoundment Stages for Scenario B

3.8. Confidence Bounds on Seepage Rates into the Seepage Canals

3.8.1. C-511 Seepage Canal

The seepage canal C-511 is divided into three small reaches wherein the water elevations are controlled by the specific downstream structures. The seepage rates in three reaches of C-511 were summarized in Table 3.11 based on the seepage results shown in Table 3.4 and Table 3.6, for Scenario A and Scenario B under four impoundment stages. The design capacities of the control structures are also listed in Table 3.11, based on the design information established in the WPA Feasibility Report (see Tables C.29 through C.35). The total seepage rates of C-511 in both scenarios and the structure design capacity are presented in Figure 3.10. The results in Table 3.11 indicate that the seepage rates in each reach of C-511 are lower than the design capacity of the control structures or the conveyance capacity of the seepage canal. The maximum seepage rate, the seepage rate at the highest impoundment stage of Scenario B, was defined as the confidence bounds on seepage into this reach of C-511.

In addition, the factor of safety for each reach of C-511 was evaluated on the basis of the seepage results of Scenario A, and the results are summarized in Table 3.12. In the WPA Feasibility Report, Appendix B, a factor of safety of 5.0 was used for the structure capacity. The results in Table 3.12 indicate that except for S-505A weir, a factor of safety of 5.0 is achieved for all other structures. For S-505A fixed weir, a factor of safety of 4.28 is obtained for the normal pool elevation of 10.0 ft. Therefore a detailed HEC-RAS hydraulic model described in Section 3.8 was used to redesign S-505A fixed weir by increasing the weir width from 100 ft to 116 ft to provide a factor of safety of 5.0. For impoundment stages higher than normal pool elevation, even though the factor of safety is as low as 2.71, these value are still acceptable with the consideration that the structure design is only based on the normal pool elevation.

Table 3.11 Summary of Seepage Rates and Design Capacities of C-511

Canal	Reach	Control Structure	Design Capacity	Flow Rates in Seepage Canal for Scenario A (cfs)				Flow Rates in Seepage Canal for Scenario B (cfs)			
			cfs	10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
C-511	West	S-505C	120 *	17.52	19.04	20.54	22.01	20.53	21.96	23.36	24.74
C-511	East	S-505A	150 **	35.02	41.97	48.75	55.4	59.56	65.96	72.2	78.44
C-511	South	-	50 ***	2.38	2.54	2.69	2.84	2.38	2.54	2.69	2.84
Total			**320**	**54.92**	**63.54**	**71.97**	**80.24**	**82.46**	**90.45**	**98.25**	**106.02**

* Design capacity of S-505C pump station, see Table C.32.
** Design capacity of S-505A fixed weir, see Table C.29.
*** Design capacity of south C-511 seepage canal, see the WPA Feasibility Report, Appendix B.

Table 3.12 Factor of Safety for Each Reach of C-511 in Scenario A

Canal	Reach	Control Structure	Design Capacity	Flow Rates in Seepage Canal for Scenario A (cfs)				Factor of Safety for Scenario A			
			cfs	10 ft	11 ft	12 ft	13 ft	10 ft	11 ft	12 ft	13 ft
C-511	West	S-505C	120 *	17.52	19.04	20.54	22.01	*6.85*	*6.30*	*5.84*	*5.45*
C-511	East	S-505A	150 **	35.02	41.97	48.75	55.4	*4.28*	*3.57*	*3.08*	*2.71*
C-511	South	-	50 ***	2.38	2.54	2.69	2.84	*21.01*	*19.69*	*18.59*	*17.61*

* Design capacity of S-505C pump station, see Table C.32.
** Design capacity of S-505A fixed weir, see Table C.29.
*** Design capacity of south C-511 seepage canal, see the WPA Feasibility Report, Appendix B.

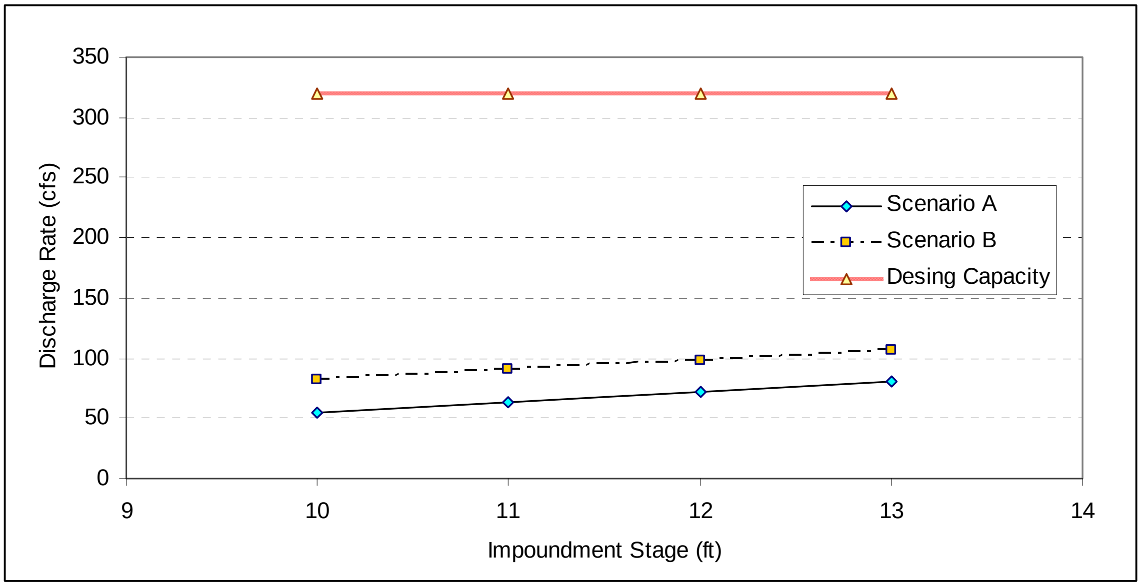

Figure 3.10 Design Capacity and Seepage Rates in C-511 for Scenarios A and B

3.8.2. C-509 Seepage Canal

The seepage canal C-509 is divided into three small reaches wherein the water elevations are controlled by the specific downstream structures. The seepage rates in three reaches of C-509 were summarized in Table 3.13 based on the seepage results shown in Table 3.8 and Table 3.10, for Scenario A and Scenario B under four impoundment stages. The design capacities of the control structures are also listed in Table 3.13, based on the design information established in the WPA Feasibility Report (see Tables C.29 through C.35). The total seepage rates of C-509 in both scenarios and the structure design capacity are presented in Figure 3.11. The results in Table 3.13 indicate that the seepage rates in each reach of C-509 are lower than the design capacity of the control structures or the conveyance capacity of the seepage canal. The maximum seepage rate, the seepage rate at the highest impoundment stage of Scenario B, was defined as the confidence bounds on seepage into this reach of C-509.

In addition, the factor of safety for each reach of C-509 was evaluated on the basis of the seepage results of Scenario A, and the results are summarized in Table 3.14. All the structures on C-509 are adequately designed to maintain the water elevations in the seepage canal with a factor of safety of 5.0 for the normal pool elevation of 8.5 ft. For impoundment stages higher than normal pool elevation, a factor of safety of only 4.36 is achieved at the east reach of C-509 for the stage of 11.5 ft. This value is less than 5.0; however, it is still acceptable with the consideration that the structure design is based on the normal pool elevation.

Table 3.13 Summary of Seepage Rates and Design Capacities of C-509

Canal	Reach	Control Structure	Design Capacity	Flow Rates in Seepage Canal for Scenario A (cfs)				Flow Rates in Seepage Canal for Scenario B (cfs)			
			cfs	8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
C-509	West	S-512B	125*	11.55	12.93	14.29	15.66	12.64	14.02	15.38	16.75
C-509	North	S-512A	75**	2.27	2.28	2.28	2.28	9.62	9.62	9.63	9.63
C-509	East	S-512A	150**	20.99	25.7	30.13	34.39	33.97	38.05	42.12	46.13
Total			**350**	**34.81**	**40.91**	**46.70**	**52.33**	**56.23**	**61.69**	**67.13**	**72.51**

* Design capacity of S-512B fixed weir, see Table C.34.
** Design capacity of S-512A pump station, see Table C.33.

Table 3.14 Factor of Safety for Each Reach of C-509 in Scenario A

Canal	Reach	Control Structure	Design Capacity	Flow Rates in Seepage Canal for Scenario A (cfs)				Factor of Safety for Scenario A			
			cfs	8.5 ft	9.5 ft	10.5 ft	11.5 ft	8.5 ft	9.5 ft	10.5 ft	11.5 ft
C-509	West	S-512B	125*	11.55	12.93	14.29	15.66	*10.82*	*9.67*	*8.75*	*7.98*
C-509	North	S-512A	75**	2.27	2.28	2.28	2.28	*33.04*	*32.89*	*32.89*	*32.89*
C-509	East	S-512A	150**	20.99	25.7	30.13	34.39	*7.15*	*5.84*	*4.98*	*4.36*

* Design capacity of S-512B fixed weir, see Table C.34.
** Design capacity of S-512A pump station, see Table C.33.

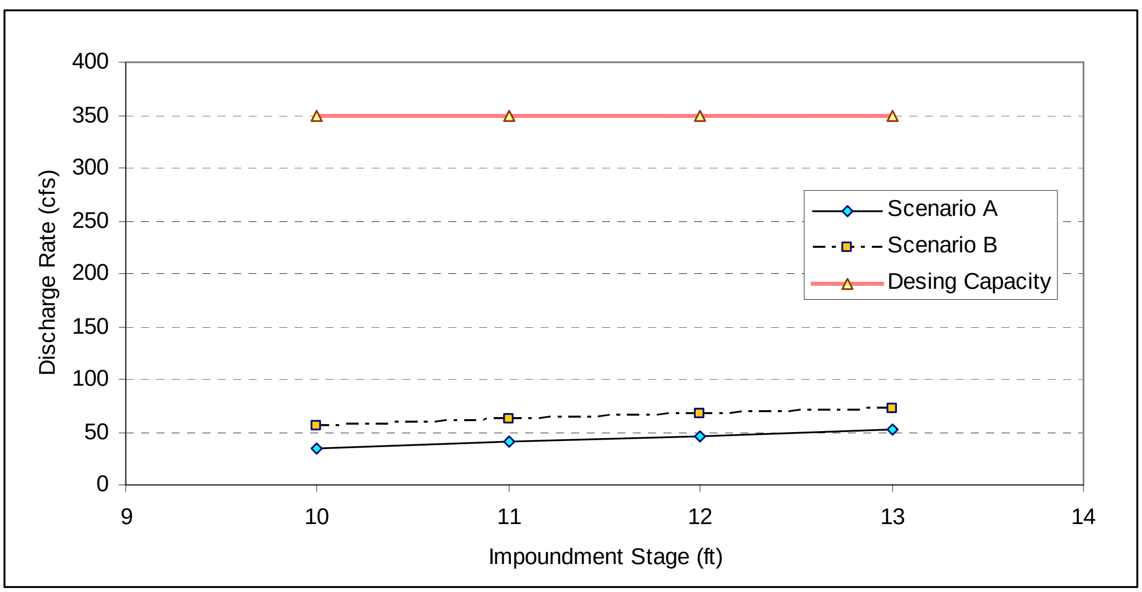

Figure 3.11 Design Capacity and Seepage Rates in C-509 for Scenarios A and B

3.9. Hydraulic Modeling in Seepage Collection Canal

3.9.1. General Description and Background

The seepage rates captured by the seepage collection canals were determined at each cross-section of the C-11 and C-9 impoundments, for two scenarios. A hydraulic model was designed to verify the conveyance capacity of the seepage collection canals for different designed conditions.

The HEC-RAS model, developed by the Hydrologic Engineering Center of U.S. Army COE, is an integrated system of software that was designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. Three hydraulic analysis components are included in HEC-RAS system: 1) steady flow water surface profile computations; 2) unsteady flow simulation; and 3) movable boundary sediment transport computation. A graphical user interface (GUI) is developed to assist user for more efficient construction of complex hydraulic models. The latest version of the HEC-RAS (version 3.1.2 with release date of April 2004) is employed in this Task.

3.9.2. Hydraulic Model Development Methodology

- **Reaches**

The seepage collection canals, C-511 and C-509, are included in the models as shown in Figure C.1 and Figure C.3. Based on the operation rules of the seepage canals and the control structures, C-511 and C509 were divided into six small reaches in which the water elevations are controlled by specific structures, such as the pump stations or fixed weirs. Each reach is treated as a simple constructed channel without junction, and then six hydraulic models were developed. The location information about the six reaches is summarized in Table 3.15.

Table 3.15 Summary of Information for Reaches Modeled in HEC-RAS

Canal	Reach	Location	Length (ft)	Note
C-511	West	North and West of impoundment	16,605	Pump station S-505C on downstream
	East	East of impoundment	14,720	Fixed weir S-505A on downstream Fixed weir S-505B on river station 11745
	South	Southwest of impoundment	1,390	
C-509	West	West of impoundment	18,135	Fixed weir S-512B on downstream
	North	North and east of mitigation area	9,940	Pump station S-512A on downstream
	East	North and east of impoundment	15,380	Pump station S-512A on downstream

- **Canal Cross-Sections**

Canal cross-sections are necessary to accurately simulate the stage in the conveyance system. In general, seven cross-sections shown in Table 3.16 were modeled in six reaches, on the basis of the design information that were utilized in SEEP2D models, and the remaining cross-sections were generated from interpolation at the 1,000-foot intervals. In addition, the cross-sections on the upstream or downstream of the structures, such as fixed weirs, were specified based on the design information for the structures.

Table 3.16 Summary of Information for Cross Sections Modeled in HEC-RAS

Section	Canal	Reach	Length	Inside Slope	Outside Slope	Bottom Width	Top of Bank*	Canal Invert	Canal Depth	Note
			feet	1V on ?H	1V on ?H	feet	ft-NGVD	ft-NGVD	feet	
1	C-511	West	16,605	3.0	3.0	10.0	6.0	-1.0	7.0	
2	C-511	East	2,965	3.0	3.0	20.0	6.5	-2.5	9.0	North of S-505B
3	C-511	East	11,745	2.0	2.0	40.0	5.5	-10.0	15.5	South of S-505B
4	C-511	South	1,390	3.0	3.0	10.0	6.0	-1.0	7.0	
5	C-509	West	18,055	3.0	3.0	10.0	5.0	-1.0	6.0	
6	C-509	North	9,940	3.0	3.0	10.0	4.5	-2.0	6.5	
7	C-509	East	15,380	3.0	3.0	20.0	4.5	-4.5	9.0	

* The data of the top of bank were kept at the same values that used in SEEP2D models.

- **Inline Structure**

The inline structures include culverts, weirs and gates that are located along the canal and directly control the flow along the conveyance system. There are three inline structures along the eastern reach of C-511. S-505A fixed weir is located at the southeast corner of the C-11 impoundment, the downstream of the eastern reach. S-505B fixed weir and S-505B culverts are located at the northeast corner of the C-11 impoundment; however, S-505B culvert is used to access the seepage canal to the impoundment, and as a result it was excluded from the hydraulic modeling for this reach. There are two inline structures along the western reach of C-509. S-512B fixed weir and S-512B gated culverts is located at the southwest corner of the C-9 impoundment, on the downstream of the western reach of C-509; however, S-512B gated culverts were excluded in the hydraulic computation in order to verify the capacity of the S-512 fixed weir. The detailed inline structures data are listed in Tables C.29 through C.35.

- **Pump Stations**

The pump stations are utilized to pump the seepage water collected by C-511 or C-509 back to the impoundments. S-505C pump station, located at the southwest corner of the C-11 impoundment, is used to control the water surface elevation in the west reach of C-511. S-512A pump station, located at the intersection of the southeast corner of the C-9 mitigation area and the C-9 impoundment, is utilized to control the water surface elevations of the north and east reaches of C-509 (see Table 3.15). In Table C.32 and Table C.33, the detailed design for these two pump stations are listed.

- **Manning's n**

In the WPA Feasibility Report, Appendix B, Manning's n 0.035 was used to design the main channel of C-511 and C-509. In our six hydraulic models, 0.035 and 0.05 were assigned as the Manning's coefficient of main channels and the flood plains, respectively.

- **Boundary Conditions**

Table 3.17 lists the river station of the downstream end for each reach of C-511 and C-509, where the water surface elevation was modeled at a specific value for steady flow simulation as described below. For the east reach of C-511, the seepage water is discharged into the C-11 canal through S-505A fixed weir. The cross-section located about 100 feet downstream of S-505A, was maintained at 4.0 ft-NGVD, which is the optimum level of the C-11 canal. At the downstream of the west reach of C-509, S-512B fixed weir is utilized to control the water elevation, and the cross-section located 80 feet downstream of S-512B was modeled at the optimum level of the C-9 canal -- 4.0 ft-NGVD. As discussed in section 3.7, S-505C and S-512A pump stations are designed with efficient capacities so that the water elevations in the west reach of C-511, and the north and east reaches of C-509 were modeled at the designed water level as shown in Table 3.17. No control structure is located at the downstream end of the south reach of C-511; therefore the water surface at river station 0+00 was modeled at the optimum level of the C-11 canal, 4.0 ft-NGVD.

Table 3.17 Summary of Boundary Conditions in HEC-RAS

Canal	Reach	Downstream End Station	W.S (ft-NGVD)	Control Structure
C-511	West	0	5.0	S-505C Pump Station
	East	-100	4.0	S-505A Fixed Weir
	South	0	4.0	
C-509	West	-80	4.0	S-512B Fixed Weir
	North	0	3.0	S-512A Pump Station
	East	0	3.0	S-512A Pump Station

- **Flow Data**

In general, the flow rate varies gradually along the reach of the canals due to the seepage flow from the impoundments and the mitigation areas. As described previously, eight seepage inflow data (two scenarios of four impoundment stages) were determined using the SEEP2D models. These flow rates for each reach were accumulated section by section. The flow rate at the downstream end should be equal to the total seepage amount collected by this reach of the canal. The flow rate at each river station of the reach was calculated and assembled into the model for all eight profiles (see Tables C.36 through C.46).

3.9.3. Hydraulic Analysis Results

The HEC-RAS models simulated eight designed profiles that correspond with the seepage results of two scenarios and four impoundment stages in SEEP2D models. The confidence bounds on seepage, as defined in Section 3.7, refers to the eighth profile in which the adjacent groundwater table is at the ground surface and the impoundment stage at the surcharge pool elevation, therefore the hydraulic results of the eighth profile were used to evaluate the peak stage in the seepage canals. Figures C.6 through C.11 present the water surface for the eighth profile.

- **West Reach of C-511**

The downstream end of this reach is controlled by S-505C pump station. The maximum flow rate of 24.74 cfs at the downstream end is less than 120 cfs, which is the design capacity of S-505C. The peak stage along the west reach of C-511 is approximately 5.01 ft (see Table C.37), which means the design of S-505C pump station and C-511 west reach is adequate to maintain desired seepage canal elevation when the adjacent water table is at the ground surface.

- **East Reach of C-511**

The water surface of this reach is controlled by S-505A and S-505B fixed weir structures. The maximum flow rate at the downstream end was modeled at 78.44 cfs and the downstream water surface was assumed at 4.0 ft. The results in Table C.39 indicate that the peak stages are approximately 4.04 ft and 4.81 ft in the reach south of S-505B and north of S-505B, respectively. The seepage canals at south of S-505B and north of S-505B were modeled at 4.0 ft and 5.0 ft in SEEP2D. The results indicate that the design of S-505A, S-505B fixed weir structures and east reach of C-511 is adequate to maintain desired seepage canal elevation when the adjacent water table is at the ground surface.

- **South Reach of C-511**

The south reach of C-511 merges into the C-11 canal directly. The downstream water surface was modeled at 4.0 ft. Among all four profiles, the maximum flow rate at the downstream is 2.84 cfs. The results in Table C.41 indicate that the water surface in this reach is kept at approximately 4.00 ft, i.e., the conveyance capacity of south reach of C-511 is sufficient for seepage collection.

- **West Reach of C-509**

The water surface of this reach is controlled by S-512B fixed weir structure. The maximum flow rate at the downstream was modeled at 16.75 cfs and the downstream water surface was assumed at 4.0 ft. The results in Table C.43 indicate that the peak stage along the reach is approximately 4.83 ft. Regarding the designed seepage canal elevation of 5.0 ft, it is concluded that the design of S-512B fixed weir structure and west reach of C-509 is adequate to maintain desired seepage canal elevation when the adjacent water table is at the ground surface.

- **North Reach of C-509**

The downstream end of this reach is controlled by S-512A pump station. The maximum flow rate of 9.63 cfs at the downstream end is less than 75 cfs, which is the design pump rate used for north reach of C-509. The peak stage along the north reach of C-509 is approximately 3.00 ft (see Table C.45), which means the design of S-512A pump station and C-509 north reach is adequate to maintain desired seepage canal elevation when the adjacent water table is at the ground surface.

- **East Reach of C-509**

This reach is controlled by S-512A pump station also. The maximum flow rate of 46.13 cfs at the downstream end is less than 150 cfs, which is the design pump rate applied for north reach of C-509. The peak stage along the east reach of C-509 is approximately 3.01 ft (see Table C.47), which means the design of S-512A pump station and C-509 east reach is adequate to maintain desired seepage canal elevation when the adjacent water table is at the ground surface.

3.9.4. S-505A Weir

As described in section 3.7, the factor of safety for the east reach of C-511 is approximately 4.28 at normal pool in Scenario A (see Table 3.12). The designed discharge capacity of S-505A fixed weir is recommended to increase from 150 cfs to 175 cfs in order to keep the factor of safety at 5.0, which was used in the Feasibility Study Phase.

The original discharge rate of 150 cfs was used as the flow data for the east reach of C-511 in HEC-RAS modeling, and then the corresponding water surface elevation was obtained. The results in Table C.48 indicate the water elevation in seepage canal is maintained at 4.21 ft, which is assumed to be acceptable. The higher discharge rate of 175 cfs was used to update the flow data in HEC-RAS modeling, and the water surface elevation was higher than 4.21 ft. The length of S-505A fixed weir was then increased to keep the water elevation at 4.21 ft with the discharge rate of 175 cfs. When the length of S-505A fixed weir is changed from 110 ft to 116 ft, the water surface elevation is approximately 4.21 ft as shown in Table C.49, which is acceptable.

3.10. Conclusions

The following conclusions can be made based on the analysis performed to evaluate the C-11 and C-9 impoundments and the seepage management system performance under the design conditions:

- The Broward County MODFLOW model is inadequate in computing seepage losses from the impoundments and seepage canal flow rates mainly because of the large discretization scheme used in the model as well as the simplicity of the seepage calculation method.

- To adequately estimate seepages, SEEP2D models were developed to perform the seepage analysis for the impoundments as well as the associated seepage canals.
- Seepage losses from the impoundments and seepage canal flow rates were estimated using SEEP2D for two extreme condition scenarios, first, adjacent groundwater table at the high wet season level and second, with groundwater table at the ground surface. Both scenarios were evaluated for different water stages in the impoundments.
- As expected higher adjacent groundwater table prevents seepage losses from the impoundments and lower adjacent groundwater table reduces seepage amount into the seepage canals.
- The C-11 and C-9 canals function as more effective seepage collectors than the C-511 and C-509 canals, which is due to the direct hydraulic connection between the C-11/C-9 canals and the deeper, more permeable soil layer. The seepage collection capabilities of C-511 and C-509 could be improved by reducing the canal invert elevations, lowering the water levels and increasing the cross sectional areas.
- The results of the SEEP2D models indicate that approximate 10% of seepage losses from the impoundments are captured by the seepage collection canals and approximate 25% of seepage losses are captured by the C-11 and C-9 canals, and the remaining 65% of seepage losses will bypass the perimeter canals and move out to the adjacent land.
- The seepage flow into the C-11 west and C-9 west basins provides the groundwater recharge and as a result prevents saltwater intrusion. In addition, the seepage flow also helps to reduce the seepage from SMA-3A and SMA-3B.
- The seepage into the C-11 and C-9 canals is small in comparison with the basin-wide runoff into the canals. The canals, being large, appear to be adequate to handle both the seepage and basin-wide runoff. The seepage flow into the adjacent land increases the groundwater table and part of seepage flow is drained into the canals and drainage ponds on the east, south, and north side of the impoundments. With consideration of the seepage flow into the C-11 west and C-9 west basins, the hydrograph runoff computations were performed for pre-project and post-project conditions in Part 2, and the basin-wide runoff volumes or depths are not significantly increased for post-project condition.

- The seepage moving into the basin is about 65% of the total seepage losses from the impoundment. Part of this seepage will move out of the basin and a part will elevate the groundwater level and fill up the depressions and lakes in the basin. The latter part has the potential for increasing flooding in the basin. However this seepage is small in comparison with the stormwater flows and consequently will have a relatively small effect on flooding in the basin during storm events as demonstrated in Part 2.
- The seepage collection system capability can be improved, if reducing the flood runoff increment is required, by reducing the canal invert elevation, enlarging the canal bottom width, and operating the canals at lower water surface elevations. In addition, the design capacities of the fixed weirs and pump stations need to be redesigned in accord with the increased seepage flow in the seepage canals.
- Confidence bounds were developed for the seepage flow rates and compared with the seepage pump design capacities. Note that the design pump capacities were estimated in the WPA Feasibility Study Phase using a safety factor of five (5.0). The results indicate that the design pump capacities are adequate to accommodate the variation in the seepage rates under extreme groundwater conditions and impoundment stages.
- The design of the weirs provided in the seepage canals also provides a safety factor of 5.0, except for the weir S-505A in seepage canal C-511, for which the factor of safety is calculated as 4.28 instead of 5.0. This weir length needs to be changed from the current design of 100.0 ft to 116.0 ft to yield a safety factor of 5.0 and to keep the water stages of C-511 within acceptable ranges.
- The water surface profiles in the seepage canals were computed by developing the HEC-RAS models and the results indicate acceptable canal stages.
- The design of the seepage collection system appears to be adequate both in terms of seepage canal stages and flow rates except as noted for the weir S-505A above.
- Additional site specific hydrogeological and geotechnical investigations are recommended to provide improved or additional cross sections and soil properties, particularly in the SMA-3A/3B area, for a more refined modeling.

3.11. References

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Table of Contents

List of Tables

4. BASELINE SYSTEMS OPERATION

Task 3 of the Statement of Work for Work Order No. C-C20104P-WO03 requires:

- An assessment of the potential for adverse impact on flood protection along the C-9 Canal due to surface water operations, including consideration of the potential impact of increased seepage quantities determined to be entering the C-9 Canal east of the new S-511 divide structure;
- Definition of the operating rules of District structures affecting the C9 and C11W basins during large storm events;
- An assessment of the behavior of the C11W Impoundment under major storm events selected from the SFWMM simulations;
- An uncertainty analysis to demonstrate that the project will function as designed over the range of probable behaviors.

This analysis is specific to those conditions anticipated to exist upon completion of the presently authorized Broward County WPA projects. Future separable elements of CERP, such as the North and Central Lake Belt Storage Areas, are not considered herein. Part 1 of this document presents a full listing of all existing and proposed structures considered to influence the operation of the C-11 and C-9 impoundments during major storm events. The intended operation of those structures under major storm events, as taken from either Appendix B of the Feasibility Study or the Water Control Manual, is also included in Part 1.

4.1. Flood Protection on C-9 Canal

The following sections summarize the potential impact on flood protection on the C-9 Canal resulting from completion of the presently authorized Broward County WPA projects. Potential impacts due to surface water operations and those due to seepage from the C-9 Impoundment are considered separately.

4.1.1. Potential Impacts Due to Surface Water Operations

The preliminary design of the C-9 Impoundment as presented in the Feasibility Study includes new Structure S-511, a two-barrel gated culvert to be located in the C-9 Canal approximately 200 feet east of the easterly edge of the Impoundment (see Figure 1.5).

As presented in the Feasibility Study, S-511 will function as a canal divide, with gate position dependent on the operational mode.

- When water is released from the Impoundment into the C-9 Canal west of S-511, the gates may be opened, fully or partially, to move water to the east if desired;
- Under some conditions, the gates may be opened to allow reverse (e.g., east-to-west) flow, permitting the capture of excess C-9 Basin runoff through concurrent operation of Structure S-511 and Pumping Station S-509.

It is clear that rules governing operation of this structure can be established such that no potential impact on C-9 Canal stages east of S-511 would result from operation of project features west of S-511 (other than as might result from Impoundment seepage, which is separately discussed below).

The Feasibility Study does not present a clear and concise definition of the planned gate operations at S-511. The following is considered consistent with the apparent intent of the Feasibility Study and the SFWMM simulations of the project.

- Normal gate position – fully closed. Gates would be opened only for the purposes of delivering water to the east, or for removal of excess C-9 Basin runoff to the Impoundment. As presented in the Feasibility Study, the gates would remain closed at all times that flow is being delivered from the C-11 Impoundment to the C-9 Impoundment.
- When delivering water to the east, gate openings would be modulated such that the tailwater elevation east of S-511 remains at or below elevation 3.0 ft. NGVD (taken as indicative of potential flooding conditions on the C-9 Canal); during water supply deliveries, the tailwater elevation to be maintained would be 2.0 ft. NGVD;
- During periods of excess runoff in the C-9 Basin, the gates may be opened whenever S-509 is operating and canal stages east of S-511 are higher than those west of S-511.

Given only compliance with the above limitations on gate operation at S-511, it may be stated that no adverse impact on flood protection in the C-9 Canal east of S-511 would result from surface water operations.

4.1.2. Potential Impacts Due to Impoundment Seepage

A detailed evaluation of the potential impacts of increased groundwater elevations in the C-9 Basin on storm runoff potential following completion of the project is presented in Part 2 of this document. The following summary information is taken from Part 2:

- During an eight-year period of daily simulation (1988-1995), the maximum increase in overall groundwater elevations following completion of the project was predicted to have occurred on July 9, 1990;
- On that date, computed groundwater stages following project completion exceeded "existing condition" stages by more than 0.1 ft. over 55% of the C-9 West basin;
- Synthetic rainfall events having a duration of 72 hours and return periods of 10, 25 and 100 years were assumed to occur concurrent with the maximum difference between existing and with-project groundwater stages (e.g., beginning on July 9, 1990 when the reduction in soil storage as a result of the project would be at maximum);
- Hydrographs of storm runoff were computed for the above events. One set of hydrographs were developed considering only the impact of the increased groundwater stage on the conversion of rainfall to runoff (e.g., reduced ground storage or infiltration). A second set of hydrographs were computed in which the total discharge volume included seepage into the canal system, in addition to rainfall converted to runoff.
- The hydrographs of storm runoff were then compared to assess the impact of the project on the total volume of storm runoff.

For the development of "with project" storm runoff hydrographs, the C-9 West Basin area was reduced by the area to be occupied by the C-9 Impoundment. For consideration of the total discharge hydrographs with seepage into the canals, the hydrographs of rainfall converted to runoff were increased to include the estimated seepage into the canals. The estimated seepage used in that analysis was selected as that occurring on August 31, 1988, that date in the eight-year simulation on which the maximum difference in estimated seepage inflows rates as a result of the project were computed to occur. The coupling of the date on which the maximum difference in groundwater stages occurred with the maximum seepage rates to the canal (occurring on a substantially different date) is believed to result in a slightly conservative approach to the analysis.

As presented in Part 2, for that instance in which only the reduction in ground storage resulting from increased "with project" groundwater elevations is considered, the reduction in basin area resulting from conversion of lands to use in the C-9 Impoundment is more than adequate to offset increased runoff depth from the remainder of the basin. For each of the three events considered, the estimated total volume of runoff "with project" was just over 6% less than that estimated for existing conditions.

When increased seepage to the canals resulting from the higher "with project" groundwater elevations are added to the analysis, the results presented in Part 2 suggest that the total discharge volume "with project" would remain less than that for existing conditions, although the amount of the difference is reduced. The reduction in total discharge volume under that instance ranged from 1% for the 10-year event to 2.8% for the 100-year event.

4.1.3. Conclusions

Based on the above, it is concluded that, barring only inadvertent or improper opening of the gates at proposed Structure S-511, no adverse impact to flood protection on the C-9 Canal should result from completion of the presently authorized Broward County WPA components.

4.2. Flood Protection on C-11W Canal

The following sections summarize the potential impact on flood protection on the C-11W Canal resulting from completion of the presently authorized Broward County WPA projects. Potential impacts due to surface water operations and those due to seepage from the C-11W Impoundment are considered separately.

4.2.1. Potential Impacts Due to Impoundment Seepage

A detailed evaluation of the potential impacts of increased groundwater elevations in the C-11 West Basin on storm runoff potential following completion of the project is presented in Part 2 of this document. The following summary information is taken from Part 2:

- During an eight-year period of daily simulation (1988-1995), the maximum increase in overall groundwater elevations following completion of the project was predicted to have occurred on November 1, 1990;
- On that date, computed groundwater stages following project completion exceeded “existing condition” stages by more than 0.1 ft. over 93% of the C-11 West basin;
- Synthetic rainfall events having a duration of 72 hours and return periods of 10, 25 and 100 years were assumed to occur concurrent with the maximum difference between existing and with-project groundwater stages (e.g., beginning on November 1, 1990 when the reduction in soil storage as a result of the project would be at maximum);
- Hydrographs of storm runoff were computed for the above events. One set of hydrographs were developed considering only the impact of the increased groundwater stage on the conversion of rainfall to runoff (e.g., reduced ground storage or infiltration). A second set of hydrographs were computed in which the total discharge volume included seepage into the canal system, in addition to rainfall converted to runoff.
- The hydrographs of storm runoff were then compared to assess the impact of the project on the total volume of storm runoff.

For the development of “with project” storm runoff hydrographs, the C-11 West Basin area was reduced by the area to be occupied by the C-11 Impoundment. For consideration of the total discharge hydrographs with seepage into the canals, the hydrographs of rainfall converted to runoff were increased to include the estimated seepage into the canals. The estimated seepage used in that analysis was selected as that occurring on November 1, 1990, that date in the eight-year simulation on which the maximum difference in estimated seepage inflows rates as a result of the project were computed to occur. That date is consistent with the date on which the maximum difference in groundwater stage from existing to “with project” conditions was computed.

As presented in Part 2, for that instance in which only the reduction in ground storage resulting from increased “with project” groundwater elevations is considered, the reduction in basin area resulting from conversion of lands to use in the C-11 Impoundment is more than adequate to offset increased runoff depth from the remainder of the basin. For the three events considered, the estimated total volume of runoff “with project” was reduced as compared to existing conditions, with the amount of the reduction varying from 0.8% under the 10-year, 72-hour rainfall event to 2.0% under the 100-year, 72-hour rainfall event.

When increased seepage to the canals resulting from the higher “with project” groundwater elevations are added to the analysis, the results presented in Part 2 suggest that the total discharge volume “with project” would be greater than that for existing conditions, indicating that an increased removal capacity would be necessary to prevent an increase in either maximum canal stages or the duration of flooding events. Inspection of the information presented in Table 2.15 suggests that, for the C-11 West Basin as a whole, the increase in estimated seepage entering the primary canals resulting from the project was a maximum of approximately 180 cfs on November 1, 1990. As noted in Part 2, seepage rates considered in the analysis were reduced from those in the ModFlow analysis by a factor of 20, suggesting additional analysis would be appropriate to finalize any recommended increase in removal capacity intended to offset the impact of increased seepage during significant runoff events.

Part 3 of this document presents the results of two-dimensional groundwater modeling employing SEEP2D. As presented in Table 3.3, the total estimated seepage loss from the C-11 Impoundment to the east and north (sections 2, 3A, 3B and 3C) with an Impoundment stage of 10 ft. NGVD (maximum design storage elevation) is 500 cfs. Of that total estimated seepage loss, it was estimated that 50 cfs would be captured in the C-111 seepage collection canal and returned to the Impoundment (see Table 3.4), resulting in a loss to the areas north and east of the Impoundment of 450 cfs. While not directly presented in Part 3, similar information can be extrapolated for impoundment stages of 6.0 ft. NGVD (storage at ground surface) and 8.0 ft. NGVD (mid-point of active storage), leading to estimates of approximately 130 and 290 cfs, respectively, lost to areas east and north of the Impoundment.

Given the above, it is presently anticipated that an increase in the nominal capacity of Inflow Pump Station S-503 of between 130 cfs and 450 cfs would be necessary to offset the influence of increased seepage on flood volumes and durations. The detailed design phase of the project should include additional, more detailed analyses to finalize the desirable increase in capacity at S-503. It would not be considered necessary to concurrently increase the nominal capacity of other elements of the project.

4.2.2. Potential Impacts Due to Surface Water Operations

Review of the basic hydraulic design and planned operations presented in the Feasibility Study (and briefly summarized in Part 1 of this document) suggests that, barring only structure failure or improper operation, negative impacts on flood protection in the C-11 West Canal could only be projected to result from one or more of the following:

- Providing a capacity at Pump Station S-503 less than that historically employed at S-9 and S-9A for removal of C-11 West Basin runoff;
- Failure to include an appropriate allowance in the capacity of S-503 for increased seepage during significant flooding events resulting from the project (see discussion in the preceding section of this Part 4);
- An inability to operate S-503 at C-11 West Canal stages as low as those for which S-9 and S-9A can be operated (e.g., reduce draw-down capability);
- Perhaps most significantly, during prolonged runoff events resulting in exhaustion of available storage in the C-11 and C-9 Impoundments, increased C-11 West Canal stages as might result from the combination of the following:
    - Cessation of pump operations at S-503;
    - Opening of the gates at S-381 to direct flow in the C-11 West Canal to S-9 and/or S-9A, which would then lift discharges to WCA-3A.

Under the last of the possibilities listed above, all C-11 West canal flows (after the impoundments are "filled") would be carried through the C-11 West Canal west of S-381; proposed Siphon Structure S-502 would be in that reach of the C-11 West Canal. The presence of S-502 will result in hydraulic losses exceeding those now existing in this reach of the canal. The extent to which those additional losses would translate to an increased canal stage east of S-381 is dependent on the intensity and duration of the runoff event. Events for which the flow diverted through S-381 to S-9 approach the capacity of S-9 for a significant period of time would result in increased canal stages approximating the incremental hydraulic loss in S-502 (roughly one foot).

Part 5 of this document explores this possibility in greater detail, with specific analysis of historic runoff events for both "existing" and "with project" conditions. It is clear that the question of flood impacts due to increased canal stages is not a matter of whether or not they could actually result, but the frequency and intensity of events required to generate that result, and the extent to which that result can be mitigated.

Maximum Historic Removal Rates: Records of mean daily discharges at Pump Station S-9 were downloaded from SFWMD's DBHYDRO database (DBKEY K5483) for the entire available record (October 1, 1957 through September 30, 2004). In addition, records of mean daily discharges at Pump Station S-9A were downloaded from SFWMD's DBHYDRO database (DBKEY PC168) for its entire available record (April 25, 2003 through January 3, 2005). Those records were then summed to determine the total daily pumping at S-9 and S-9A combined.

Over the entire period of record, the greatest recorded mean daily discharge at Pump Station S-9 was 2,434 cfs, occurring on October 4, 2000. The greatest recorded mean daily discharge at Pump Station S-9A was 507 cfs, occurring on September 16, 2003 and again on October 4, 2003.The greatest recorded mean daily discharge at the two stations combined was 2,579 cfs, occurring on October 1, 2004.

No records of seepage inflows at S-9XS and S-9XN are available from the DBHYDRO database before February 22, 1990 or after April 3, 2002 (latest date of data at S-9XN is April 3, 2001). On October 4, 2000 the recorded mean daily discharges at those two structures (DBKEY P1027 and P1028) summed to but 4 cfs, with the result that, of the total pumped discharge of 2,434 cfs on that date, the removal from the basin was 2,430cfs. Given that result, it is <u>assumed</u> that the maximum historic rate of removal from the C-11 West Basin (e.g., excluding L-33 and L-37 seepage) was essentially equal to the maximum mean daily pumping rate of 2,579 cfs. Instantaneous peak pumping rates on that date were no doubt greater.

Over the 47 years of record at Pump Station S-9, the average annual volume pumped to WCA-3A was 154,070 acre-feet. Over the ten-year period beginning January 1, 1991 the average annual volume pumped to WCA-3A was 236,845 acre-feet.

Canal Draw-Down Elevation: As discussed in Part 1, the minimum C-11 West Canal elevation during pumping at S-9 (e.g., minimum draw-down) is 0.0 ft. NGVD. At S-9A, the minimum canal elevation during pumping is 1.0 ft. NGVD, which is also the station headwater elevation under design pumping conditions. Table B.10.5.2 in Appendix B of the Feasibility Study identifies a minimum draw-down elevation for proposed S-503 of 0.0 ft. NGVD, matching that of existing Pump Station S-9.

Seepage Discharge Allowance at S-503: As discussed earlier in this Part 4, it is presently anticipated that an increase in the nominal capacity of Inflow Pump Station S-503 of between 130 cfs and 450 cfs would be necessary to offset the influence of increased seepage on flood volumes and durations. The present design (as presented in Appendix B of the Feasibility Study) of S-503 results in a total installed capacity of 2,575 cfs, with a nominal flood control capacity of 2,500 cfs. For the present design, the maximum design rate of inflow to the C-11 Impoundment is limited to 2,500 cfs. It may be desirable to both increase the total installed capacity at S-503 by between 55 and 375 cfs, and to allow inflows to the Impoundment up to the total installed capacity of S-503, to offset the potential influence of increased "with project" seepage on flood protection in the C-11 West Basin.

4.2.3. Conclusions

Based on the above, it is concluded that:

- The design capacity of S-503 presented in the Feasibility Study closely approximates the maximum historic mean daily discharge at S-9 and S-9A; instantaneous pumping rates at those stations doubtless exceeded the mean daily rate;
- The maximum historic mean daily discharges at S-9 and S-9A do not appear to have included significant seepage inflows from the L-33 and L-37 borrow canals at S-9XS and S-9XN, respectively;

- The estimated increase in storm event runoff from the C-11 West Basin associated with increased groundwater stages is generally offset by the reduction in effective basin area resulting from the conversion of lands to use in the C-11 Impoundment. However, increased seepage volumes entering the primary canals during those flood events could result in some increase in canal stages, event durations, or both;
- An increased capacity at S-503 of between 55 and 375 cfs may be desirable to offset the influence of increased seepage into the primary canals during flood events.

4.3. Review of SFWMM Simulation for "With Project" Conditions

The results of a South Florida Water Management Model (SFWMM) simulation of the "with project" conditions were furnished to Burns & McDonnell by the South Florida Water Management District on disk. The specific SFWMM output file was dated May 17, 2004 and titled "O_2010WPA_REV_V3.5". That output file was reviewed for:

- Apparent consistency with the stated "with project" design conditions, specifically, those expected to prevail in 2010 after completion of the currently authorized projects but before completion of the North and Central Lake Belt Storage Projects;
- Identification of those events in the simulation which result in significant diversion of C-11 West Basin runoff past the Impoundment to S-9.

The above file included simulation results extending from January 1, 1965 through December 31, 1995.

4.3.1. Consistency with Design Conditions

The C11 Impoundment was simulated as a 1,730-acre impoundment having a land surface elevation of 6.2 ft. NGVD and a maximum storage elevation of 10.0 ft. NGVD (3.8 feet of storage depth). That model definition resulted from consideration of 1,521 acres of impoundment at a usable storage depth of 4.0 feet, and 209 acres of mitigation wetlands at a usable storage depth of 2.0 feet. The land surface elevation in the model was established at 6.2 ft. NGVD in lieu of 6.0 ft. NGVD to maintain the total estimated storage volume at an impoundment stage of 10.0 ft. NGVD. As simulated, the total volume of storage below elevation 10.0 ft. NGVD is 6,502 acre-feet. As described in Part 1, the total storage of the impoundment itself is 5,960 acre-feet below elevation 10.0 ft. NGVD; it is not clear from the Feasibility Study that firm usable storage is intended to be available in the mitigation area. It is therefore concluded that the usable storage volume in the C-11 Impoundment as simulated exceeds that available from the design as it is presented in the Feasibility Study.

Table 4.1 summarizes data for the C-11 West Basin taken from the 2010_WPA simulation, and compares peak discharges to those defined in the Feasibility Study (again, specific to 2010, before completion of the Central and North Lake Belt Storage Area projects).

Table 4.1 Summary of 2010 Simulation, C-11 West Basin

Title	Description	From 2010_WPA Simulation			Capacity from Feasibility Study (cfs)
		Max. Daily (cfs)	Min. Daily (cfs)	Ave. Annual (ac-ft/yr)	
S-13A	Discharge from C-11 West Basin through S-13A to East	269	0	54,342	N/A
C11RIN	Flood control pump (S-503) on C-11 West Canal to Impoundment	2,500	0	35,281	2,500
C11RO	Discharge from C-11 Imp. To C-502A Through S-504	696	0	22,766	1,000
C11WDV	Excess flow from C-11W through S-381 and S-9	2,933	0	4,366	2,880
WSC11W	Water supply discharge from C-11 Imp. To C-11W Canal	304	0	6,167	300
S9XS	C-11 Impoundment Discharge Pumped at S-9 due to lack of available storage in C-9 Impoundment	682	0	13,917	2,880
S9	Flow pumped from C-11W to WCA-3A (includes seepage to L-37 and L-33)	2,880	0	54,380	2,880

The results of the simulation indicate that an average annual volume of 4,366 acre-feet per year of C-11 West Basin runoff would bypass the C-11 Impoundment and be pumped at S-9. In addition, an average of 13,917 acre-feet per year would be discharged from the

C-11 Impoundment and pumped at S-9 due to lack of available storage capacity in the C-9 Impoundment. Of the total average annual volume of 54,380 acre-feet per year pumped at S-9, an average of 34,296 acre-feet per year would consist of seepage from WCA-3A and WCA-3B, and runoff from the WCA-3A/3B Levee Seepage Management Area.

The C9 Impoundment was simulated as a 1,739-acre impoundment having a land surface elevation of 4.5 ft. NGVD and a maximum storage elevation of 8.5 ft. NGVD (4.0 feet of storage depth). The total storage volume in the C-9 Impoundment as simulated is 6,956 acre-feet. As described in Part 1, the total storage of the impoundment itself is 6,650 acre-feet below elevation 8.5 ft. NGVD, with an effective impoundment area of 1,650 acres. In addition, the project footprint includes 360 acres in the mitigation area; it is not clear from the Feasibility Study that firm usable storage is intended to be available in the mitigation area. It is therefore concluded that the usable storage volume in the C-11 Impoundment as simulated exceeds that available from the design as it is presented in the Feasibility Study.

Table 4.2 summarizes data for the C-9 Basin taken from the 2010_WPA simulation, and compares peak discharges to those defined in the Feasibility Study (again, specific to 2010, before completion of the Central and North Lake Belt Storage Area projects).

Table 4.2 Summary of 2010 Simulation, C-9 Basin

Title	Description	From 2010_WPA Simulation			Capacity from Feasibility Study (cfs)
		Max. Daily (cfs)	Min. Daily (cfs)	Ave. Annual (ac-ft/yr)	
C9RC11	Pumped inflow to C-9 Imp. From C-502-B	767	0	22,007	1,000
C9TC9R	Flood control discharge from C-9 Basin to C-9 Imp.	1,000	0	9,026	1,000
C9RWS	Water supply discharge from C-9 Imp. To C-9 Canal	60	0	792	500
S29	C-9 Basin Discharge to East through S-29	8,219	0	186,101	4,700
S30	Total discharge from L-33 Borrow Canal to C-9 Canal	107	0	2,602	N/A

Stage and depth duration data for the C-11 and C-9 Impoundments taken from the 2010_WPA simulation is summarized in Table 4.3.

Table 4.3 C-11 and C-9 Impoundments, Stage and Depth Duration

Time Equalled or Exceeded	Stage in ft. NGVD		Depth Above (Below) Ground (ft.)	
	C-11 Reservoir	C-9 Reservoir	C-11 Reservoir	C-9 Reservoir
0% (Maximum)	11.19	9.22	5.19	4.72
2%	10.21	8.63	4.21	4.13
4.60%	10.00	---	4.00	---
5%	9.97	8.52	3.97	4.02
5.80%	---	8.50	---	4.00
10%	9.72	8.45	3.72	3.95
25%	9.60	8.42	3.60	3.92
50%	7.10	7.86	1.10	3.36
75%	6.42	6.25	0.42	1.75
90%	5.47	4.08	(0.53)	(0.42)
95%	4.97	3.21	(1.03)	(1.29)
98%	4.66	2.76	(1.34)	(1.74)
100% (Minimum)	3.93	1.86	(2.07)	(2.64)

For both impoundments, the maximum simulated stage is at or below the emergency spillway crest (11.2 ft. NGVD in the C-11 Impoundment, 9.7 ft. NGVD in the C-9 Impoundment). Also in both impoundments, the design normal storage elevation (4.0 feet above ground surface) is exceeded roughly 5% of the total time during the 31-year simulation. Given the data summarized in Table 4.3, it would appear that both impoundments are operated in the simulation to maximize storage (e.g., the impoundments are kept near full to the extent sufficient water is available to do so).

As noted in the Feasibility Study and in Part 1 of this document, a key factor in minimizing the diversion of C-11 West Basin runoff to S-9 and WCA-3A is the ability to regain storage in the C-11 and C-9 impoundments. Impoundment drawdown was noted to be critical to that end prior to completion of the North Lake Belt Storage Area. It is not apparent in the simulation data reviewed that impoundment drawdown was considered a significant management objective.

4.3.2. Significant Diversion Events

The mean daily discharge diverted through S-381 and pumped to WCA-3A at S-9 and S-9A was taken from the simulation results for the terms “C11WDV” and “S9XS” combined. The resulting daily time series was then analyzed to determine those events with the greatest volume diverted through S-381, for durations varying from 1 day to 30 days. A similar analysis was prepared for total pumped volume at S-9 (which would include both flows discharged through S-381 and seepage pumped from the L-33 and L-37 borrow canals). The results of that analysis are summarized in Table 4.4.

Table 4.4 Significant Diversion Events from 2010_WPA Simulation

Duration of Event (days)	C11WDV		S-9	
	Volume (cfs-days)	Ending Date of Event	Volume (cfs-days)	Ending Date of Event
1	2,933	04/25/1979	2,880	10/31/1969; 04/26/1979
2	3,093	11/17/1994	3,117	08/20/1981
5	5,317	11/03/1969	5,720	11/04/1969
10	6,430	11/06/1969	7,291	11/06/1969
15	7,120	11/04/1969	8,480	11/06/1969
30	9,122	08/18/1985	12,054	08/18/1985

Five different events are reflected in the above summary; it is apparent that the single most significant diversion event in the simulated 31-year period would have occurred in late October to early November, 1969. It is probable that, given that event as simulated, bypass of the C-11 Impoundment at the full design rate of siphon structure S-502 (2,880 cfs) would have occurred, which would be expected to lead to an incremental loss of head at S-502 and increased stages in the C-11 West Canal.

4.3.3. Conclusions

Based on the above, it is concluded that:

- The simulated usable storage volume in both the C-9 and C-11 impoundments exceeds that defined in the October 2001 Feasibility Study. The total simulated storage (below normal pool) is 13,458 acre-feet, as compared to the 12,610 acre-feet defined in the Feasibility Study;

- The normal pool elevations (e.g., storage depth of 4.0 feet in each impoundment) are routinely exceeded in the simulations, although no discharge through the overflow spillways is identified;

- Maximum transfer rates and controlling elevations reflected in the simulation appear to be consistent with those presented in the Feasibility Study, although the full design capacity of certain transfers (such as release rate from the C-11 Impoundment to the C-9 Impoundment) may not be reflected in the simulation results;

- Over the 31-year period of simulation (1965-1995), the most significant event with respect to discharge of C-11 West Basin runoff at S-9 would have occurred in October-November 1969, and the nature of that event would have been sufficiently severe as to have resulted in an increased C-11 West Canal stage "with project" due to the presence of proposed siphon structure S-502. The amount of that increase is not known, but is likely to be considered significant;

- It would appear that both impoundments are operated in the simulation to maximize storage (e.g., the impoundments are kept near full to the extent sufficient water is available to do so). It is not apparent in the simulation data reviewed that impoundment drawdown was considered a significant management objective, with the result that the volume of C-11 West Basin runoff delivered to WCA-3A at S-9 may not have been minimized.

Over the 31-year period of simulation, the volume of C-11 West Basin runoff that would have bypassed the C-11 Impoundment and been discharged to WCA-3A at S-9 was computed to average just under 4,400 acre-feet per year. In addition, an average of just over 13,900 acre-feet per year of C-11 West Basin runoff would have been discharged from the C-11 Impoundment and pumped at S-9 due to a lack of available storage in the C-9 Impoundment. The average annual volume of C-11 West Basin runoff back pumped to WCA-3A would have totaled 18,283 acre-feet per year.

4.4. Diversion Event of October-November 1969

As indicated above, the most significant event over the 31-year simulation period with respect to diversion of C-11 West Basin runoff to S-9 and WCA-3A would have occurred in late October to early November, 1969. The following is a summary of observations resulting from a more detailed review of the simulation results for that period.

On October 22, 1969 the simulated stages of the C-11 and C-9 impoundments were 9.60 and 8.43 ft. NGVD, respectively, leaving a total available storage in the two impoundments on that date of 814 acre-feet. Over the 26-day period ending on that date, the simulated stage in the C-11 Impoundment varied from 9.60 to 9.99, averaging 9.68 ft. NGVD; the simulated stage in the C-9 Impoundment varied from 8.42 to 8.55, averaging 8.45 ft. NGVD.

Over that same 26-day period, the simulated daily inflow to the C-11 Impoundment varied from 54 to 324 cfs, averaging 126 cfs; the simulated daily discharge through S-13A varied from 20 to 24 cfs, averaging 21 cfs. Daily discharges at S-13 varied from 68 to 126 cfs, averaging 93 cfs.

Over that 26-day period, a reduction in the volume stored in the C-11 Impoundment of 5,363 acre-feet would have been required to lower the simulated storage elevation from 9.60 to 6.5 ft. NGVD on October 22, equivalent to an average release rate of 104 cfs. Had reservoir drawdown been included as a principal management objective in the simulation and that average rate of release been effected, the daily discharges at S-13A would have increased to a range of 124 to 128 cfs, and at S-13 to a range of 172 to 230 cfs. Those ranges are well within the capacity of the structures. It would have been possible to increase the available storage at the C-11 Impoundment by at least 5,363 acre-feet on October 22.

Over that same 26-day period, a reduction in the volume stored in the C-9 Impoundment of 5,965 acre-feet would have been required to lower the simulated storage elevation from 8.43 to 5.0 ft. NGVD, equivalent to an average release rate of 116 cfs. During that period, simulated discharges at S-29 varied from 213 to 946 cfs, averaging 415 cfs. An average increase of 116 cfs would have been well within the capacity of S-29 and the C-9 Canal. It would have been possible to increase the available storage at the C-9 Impoundment by at least 5,965 acre-feet on October 22.

As simulated, between October 22 and November 21, 1969 (the end of the runoff event as simulated) a total of 2,901 acre-feet of C-11 Basin runoff was pumped into the Impoundment, while a total of 12,464 acre-feet bypassed the Impoundment and would have been pumped at S-9. Of the volume pumped into the C-11 Impoundment, roughly 96% (2,792 acre-feet) would have also been discharged at S-9 due to a lack of available storage in the C-9 Impoundment for those releases. Over that period, the total volume pumped at S-9 (21,066 acre feet) would have included 15,266 acre-feet of runoff from the C-11 West Basin east of S-381. Had impoundment drawdown been included as a principal management objective in the simulation, it would have been possible to reduce the volume of basin runoff pumped at S-9 by just over 11,000 acre-feet, or 74%.

Given that apparent result, it would appear desirable to, in any future project simulations, include impoundment drawdown as a principal management objective, at least through the end of the normal wet season (e.g., October 31 of any given year), in order to minimize future discharges of basin runoff to WCA-3A at S-9.

Table of Contents

List of Tables

List of Figures

5. HYDRAULIC ANALYSIS, C-11 WEST CANAL

Task 4 of the Statement of Work for Work Order C-C20104P-WO03 requires an assessment of the impact of the project on historic flood stages in the C11W canal. Canal water surface elevations for “with Project” conditions must be compared to those for Baseline conditions (defined as those conditions existing in December 2000). This requires development and calibration of a hydraulic model for the affected canal reach. This Part 5 presents the results of analyses conducted to satisfy that element of the overall Statement of Work.

The analyses are specific to conditions anticipated to exist upon completion of the presently authorized Broward County WPA projects. Future separable elements of the Comprehensive Everglades Restoration Plan (CERP), such as the North and Central Lake Belt Storage Areas, are not considered herein. For the purpose of this analysis, the Broward County Water Preserve Areas project is composed of three principal components:

- The C-11 Impoundment;
- The C-9 Impoundment;
- The WCA-3A and WCA-3B Levee Seepage Management Projects.

It is anticipated that only those elements of the WCA-3A and WCA-3B Levee Seepage Management Projects necessary to support the transfer of water from the C-11 Impoundment to the C-9 Impoundment will be included in the features to be completed by 2010. It is assumed that other features of that component (such as the C-500A and C-500B canals, and the majority of the C-502A canal north of the C-11 Canal) will be included with future separable elements of CERP.

The “with project” condition considered herein does include certain features authorized under Section 528 of the Water Resources Development Act of 1996. Those features are included in the Western C-11 Water Quality Treatment Project, one of 34 “critical projects” authorized by WRDA 1996. The Western C-11 Water Quality Project includes the construction of Pumping Station S-9A and Structure S-381. Pumping Station S-9A was completed and placed into operation in 2002; Structure S-381 has recently been completed. For the purpose of the analyses presented herein, those features are considered as included in the system changes associated with the “with project” condition.

5.1. Introduction

Part 1 of this document presents a full listing of all existing and proposed structures considered to influence the operation of the C-11 and C-9 impoundments during major storms. The intended operation of those structures under major storm events, as taken from either Appendix B of the Feasibility Study or the Central & Southern Florida Project Water Control Manual, is also included in Part 1. Part 4 of this document further defines the anticipated operating rules of District structures affecting the C-11 West and C-9 basins, and presents an assessment of the behavior of the C-11 Impoundment and associated features under major storm events. The objective of the analyses summarized in this Part 5 is to assess the impact of the proposed structures and operations on canal water surface elevations in the C-11 Canal immediately upstream of all modifications presented in the Feasibility Study (e.g., immediately upstream of the southeasterly corner of the C-11 Impoundment).

5.1.1. Objective

The Statement of Work was developed upon the basic assumption that a hydraulic analysis of the C11W canal is needed only in the reach of the canal affected by the Project. It is not needed for the entire basin if it can be demonstrated that stages at the upstream end of the model domain under "with Project" conditions are not significantly higher than they would have been under Baseline (December 2000) conditions. For this analysis, the model domain extends along the C-11 West Canal from Pumping Station S-9 to a point 2,000 feet east of the southeasterly corner of the C-11 impoundment, or 10,300 feet upstream of S-9. The objective of this analysis is to define changes resulting from the project in the C-11 Canal water surface elevation at that point.

5.1.2. Original Scope of Work

The Statement of Work required that the following specific tasks should be performed in order to achieve the objectives of Task 4:

- Task 4.1: Development of a HEC-RAS model of the C-11 Canal encompassing the entire reach of interest. Geometry files were to be established using District-furnished cross section surveys, supplemented as necessary by record drawings of the S-9 site and US Highway 27 bridges. Boundary conditions were to consist of S-9 headwater stages and discharges obtained from the District-furnished "break-point" data. Discharge files were to be established on the basis of the S-9 discharges, modified upstream of S-9 as necessary to account for inflows to the C-11 Canal at S-9XN, S-9XS, G-86S, and G-86N.

- Task 4.2: The review and analysis of historic stage and discharge relationships to characterize the current level of flood protection. Upstream inflows to the hydraulic model under selected design events were to be defined based on the "break point" data (these were to be the same for both existing and "with project" conditions, and were to be based on the results of Task 4.1). Lateral inflows to the hydraulic model under selected design events were to be defined, so that changes in discharge through the reach of interest resulting from the project (such as eventual elimination of inflows from G-86S and G-86N, and elimination of runoff from the area occupied by the C-11 Impoundment) could be determined. It was intended that the District-furnished data and information be analyzed to:

  - Define a minimum of five specific events that are significantly different (from the perspective of total precipitation) during the period of analysis for more detailed hydraulic analysis, and subsequent acquisition of detailed flow and stage data during those events. Those events were to be selected from the ten-year period 1990-2000, as earlier data was not expected to properly reflect current system management and hydrographic influences such as land use.
  - Document system performance throughout those five selected events, such that specific determination of the impact of system modifications due to CERP and the WPA components might subsequently be determined.

  The end point of this analysis was intended to be, for each of the five events, specific delineation of discharges by canal segment for the reach of interest, together with such record stage data as is available from the District-furnished information and data.

- Task 4.3: The model was to be calibrated to measured stages and discharges in the reach of interest during a variety of runoff events over the 10-year period

1990-2000. Specific analysis of basin rainfall and runoff would not be considered, with no resultant need for meteorological records or basin simulations. The measured daily discharges to parallel estimates for the same periods of time taken from the District's Base2000 SFWMM simulation were then to be compared, and any significant discrepancies identified, together with recommendations for such modifications to the simulation results as may be appropriate for subsequent analysis. Inflows were to be divided into "upstream" and "local" inflows. Local inflows were to be defined as those developed in Task 4.2 for December 2000 conditions. Upstream inflows were to be defined as pumped discharges minus local inflows. The end point of this task was to be, for each of the five events, specific definition of the computed water surface profile under existing conditions.

- Task 4.4: The calibrated model simulations of task 4.3 were to be used to represent December 2000 conditions. The model was then to be modified to simulate the "Recommended CERP Alternative" and water surface profiles calculated for each of the five events. The model was to be modified to reflect:

  - Physical changes to the canal geometry files to reflect proposed project features, including, but not necessarily limited to:

    - Canal modifications
    - All structures included in the C-11, C-9 impoundments and WCA 3A/3B Levee Seepage Management components that affect the canal stage in the western C-11 Canal, including S-381 and S-502.

  - Changes to the model's boundary conditions to reflect modified discharges caused by the impoundments (from Task 4.2)

  - Changes in model boundary conditions caused by the new operations of:

    - S-9
    - S-9A
    - Pump Station S-503

The end point of this analysis was to be, for each of the five events, specific delineation of computed water surface elevations at the upstream end of the reach of interest under “with project” conditions together with a comparison of those elevations to those previously determined for December 2000 conditions;

- Task 4.5: Submit a draft report presenting the results of all analyses conducted under Task 4 for review and comment.

5.1.3. Influence of Earlier Findings on the Analysis

The specific approach defined in the original Statement of Work, reflected in the above listing of subtasks under Task 4, was developed upon the central assumption that the need for operation (opening) of S-381 and pumping at S-9 would occur on the receding limb of the inflow hydrograph of any given major runoff event. It was anticipated that the majority of the runoff event would have been introduced to and stored in the C-11 and C-9 impoundments prior to the need for that diversion, with the result that the comparison would concern itself with potential increases in canal stage under less than maximum pumping at S-9. It was anticipated that at least some of the more significant diversion events would have been necessary during the most recent ten years of record, given the recent urbanization of much of the C-11 West and C-9 basins. It was further anticipated that the total inflows to S-9 during the diversion would be at least slightly reduced (as compared to existing conditions) by the elimination of lateral inflows at G-86S, G-86N, S-9XN and S-9XS that would result from full implementation of the WCA-3A and WCA-3B Levee Seepage Management project.

Inspection of the information presented in Table 4.4 of this document reveals that, based on the results of the SFWMD’s South Florida Water Management Model (SFWMM) simulation of “with project” (2010) conditions, the five most significant diversion events at the C-11 Impoundment would have occurred in 1969, 1979, 1981, 1985 and 1994. It could not be reasonably anticipated that basin hydrography and system operation at those times could closely parallel that existing in December 2000. That presumption is reinforced by the observation that the greatest recorded mean daily discharge in the period of record at S-9 analyzed occurred in October, 2000 (see Part 4). As a result, detailed comparison of recorded to simulated discharges during the 1967-1985 events would not be expected to be representative of recent basin hydrography and system operations. The single event of the five falling within the 1990-2000 period considered representative of current basin hydrography and operations is the 1994 event, ending November 17, 1994 (which resulted in the highest simulated two-day volume of bypass at S-381); the highest simulated two-day volume of pumping at S-9 ended August 20, 1981.

It was further concluded in Part 4 that, over the 31-year period of simulation (1965-1995), the most significant event with respect to discharge of C-11 West Basin runoff at S-9 would have occurred in October-November of 1969. The nature of that event would have been sufficiently severe as to have resulted in a mean daily discharge through S-381 and S-9 equal to the capacity of S-9. As a result, the central assumption in the development of the original Statement of Work for this task (essentially, that diversion of the full capacity of S-9 would not be required "with project") was shown to be incorrect. It should instead be anticipated that, given simply a repetition of historic rainfall patterns, at some future point in operation of the project as it is presented in the Feasibility Study it will be necessary to divert flows around the C-11 Impoundment at the full discharge capacity of S-9.

It was therefore considered appropriate to quantify the impact of the project on canal stages at the point of interest (e.g., upstream end of the model domain) for any combination of headwater elevation at S-9 and diversion discharge rate.

5.1.4. Influence of Data Availability on the Analysis

In certain instances, the scope of Task 4 as defined in the Statement of Work was developed considering that certain data would be available which could not be provided. The following is a summary of those data:

- No recent surveyed cross sections of the C-11 Canal were available within the limits of the model domain. It was therefore necessary to use data taken from record drawings for the analysis. The extent to which those record drawings reflect current conditions along the C-11 Canal is unknown. If the record data does not accurately represent the physical properties of the canal, the accuracy of the hydraulic analysis can be compromised. However, as the focus of this analysis is the change in hydraulic characteristics introduced by the proposed project, that limitation is not considered serious;

- It was originally anticipated that the results of the SFWMM simulation furnished for use in the analysis would extend through calendar year 2000. The simulation data furnished (“O_2010WPA_REV_3.5” dated May 17, 2004) extended only through calendar year 1995, limiting the availability of simulation results for direct comparison to recorded data for recent basin hydrography and system operation.
- No recorded canal water surface elevation data was available at or near the upstream end of the model domain, with the result that it was not possible to calibrate the “existing condition” hydraulic model to record data.

5.2. Methodology

5.2.1. Model Assumptions

The C11W canal was analyzed utilizing the steady state flow routines in HEC-RAS for the given profiles. The flow regime was considered to be subcritical for the reach and the flow profiles examined. The possible exceptions to these conditions are at the pump intakes and under partial flow conditions within the siphon, where unsteady flow conditions might be anticipated. The analysis specifically excluded hydraulic conditions in the immediate vicinity of the pump intakes. The development of low flow conditions within the siphon would require water surface elevations well below those which could result from maximum drawdown pumping at S-9.

5.3. Reference Data

The following comprises a list of references and documents utilized to develop input to the HEC-RAS model and analysis. The list includes, but is not necessarily limited to District provided documents, as well as other reference materials:

- *C&SF Project Water Preserve Areas Feasibility Study, Engineering Appendix B (USACE, 2001)*
- *Western C-11 Basin Phase 2, Spillway 381 Drawings (HDR /USACE, 2003)*
- *Canal 11 C-11-XS Cross Section Drawings (SFWMD, 1977)*

- *Canal 11 Improvements Cross Section Drawings (SFWMD, 1994)*
- *West End Canal 11 Drainage Area Profile Drawings (SFWMD, 1975, 1979)*
- *Canal 11 Plans for Enlargement (SFWMD, 1978)*
- *C-11 Improvements Drawings (SFWMD, 1989)*
- *HEC-RAS River Analysis System V.3.1.2 – Applications Guide (USACE, 1998, 2004)*
- *HEC-RAS River Analysis System V3.1.2 – Hydraulic Reference Manual (USACE, 1998, 2004)*
- *HEC-RAS River Analysis System V.3.1.2 – User's Manual (USACE 1998, 2004)*
- *State of Florida Department of Transportation – Broward County State Road 25 Project No. 86060-3515 Plan Drawings (FDOT, 1977)*
- *Design of Small Canal Structures (USBR, 1983, 1991)*
- *Design of Small Dams (USBR, 1987)*

5.4. Analytical Methods

The principal tool employed in analysis of the C-11 Canal in the reach of interest was the Hydrologic Engineering Center River Analysis System (HEC-RAS), Version 3.1.2, promulgated by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers at Davis, California. The general extent of the HEC-RAS model domain is shown in Figure 5.1.

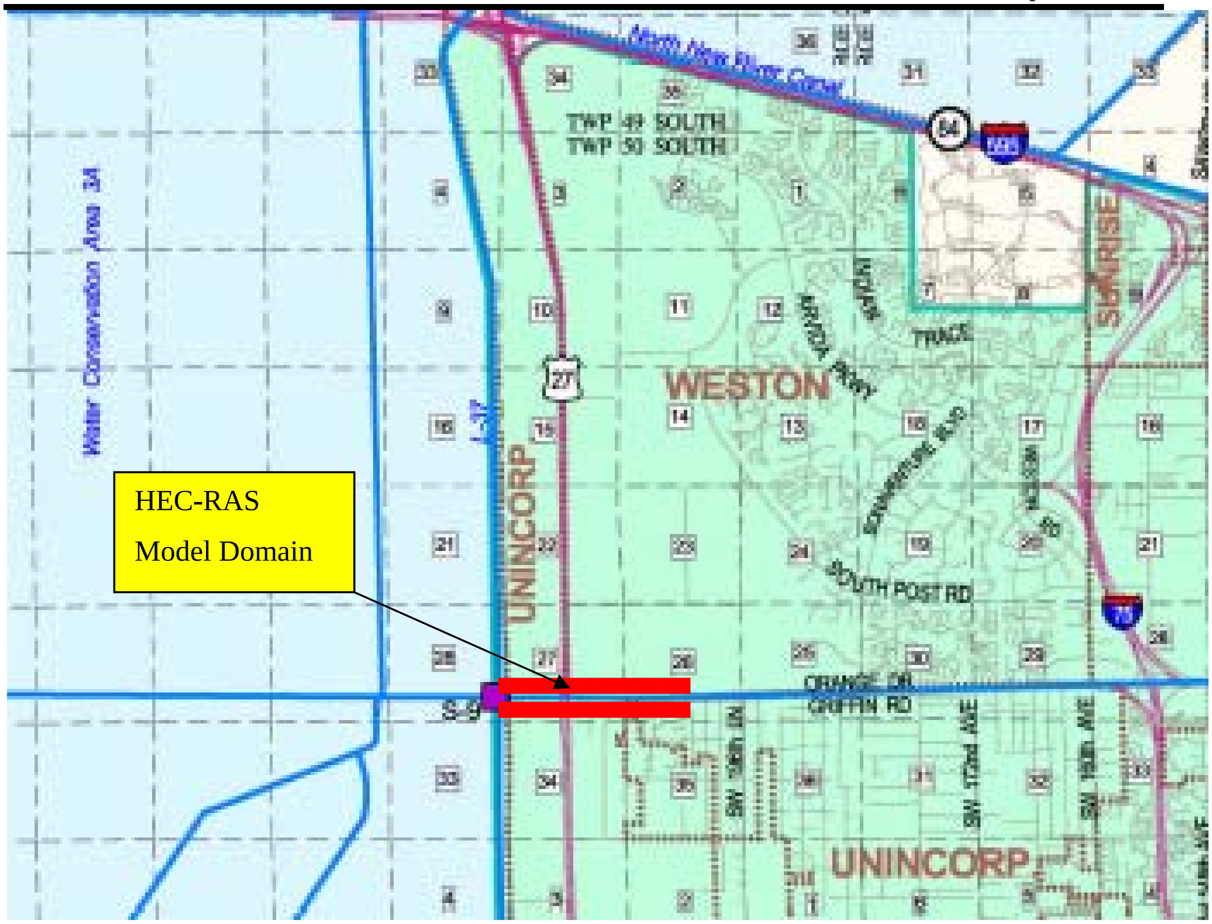

Figure 5.1 HEC-RAS Model Domain

5.4.1. HEC-RAS Geometry

Three basic geometry files were set up to analyze the canal and structures for existing baseline conditions and with project conditions. The geometry files are defined for the existing conditions without project; the "with project" conditions exclusive of the inverted siphon; and a separate geometry file for the siphon. The siphon was analyzed separate from the canal reach in order to account more accurately for all losses across the structure as well as to minimize inconsistencies due to limitations of the HEC-RAS model to accurately calculate backwater analysis through such a structure.

The canal geometry was taken from canal improvement drawings and the alignment was developed on a relative coordinate system in Microstation Geopak for input in the reach schematic in HEC-RAS. The alignment and stationing were taken from the improvement drawings with the river stationing equated to the existing canal stationing.

For the inverted siphon, geometry was taken from Plate 3A-7 of the *Water Preserve Areas Feasibility Study (USACE, 2001)*. Stationing and locations were approximated and or scaled from reference drawings.

5.4.2. HEC-RAS Flow Regime, Profiles and Boundary Conditions

Although it is anticipated that the predominant flow regime will be subcritical for the cases analyzed, the initial plan runs were calculated utilizing both the mixed flow regime and the subcritical flow regime routines. Boundary conditions were set at the upstream and downstream reach extents and included known water surface elevations representing the average stage of 3.5' NGVD and maximum stage of 6.0' NGVD for the canal as well as the downstream maximum drawdown stage of 1.0' NGVD at the headwater of the pump stations.

Flow profiles were developed for flood event flows based on maximum pumping rates at S-9 and S-9A. S-9 maximum pumping rate is 2,880 cfs, while an additional available capacity at S-9A of 500 cfs aggregates to 3,380 cfs of maximum available capacity. The analysis was limited to a maximum discharge of 2,880 cfs, consistent with established operating rules for S-9 and S-9A. A flow profile under a discharge of 1,050 cfs was developed in order to compare hydraulic performance of the canal and structures under a lesser event flow; that discharge was selected as it is approximately the maximum capacity of a single pump (design capacity of 960 cfs) under normal static head differentials at Pumping Station S-9.

5.4.3. Roughness Coefficients

The canal main channel is considered for the purposes of this analysis to be an aged canal, an earthen channel with moderate vegetative growth in good alignment with a fairly large, constant section. A Manning's *n* value for the main channel of 0.030 was assigned. That value is consistent with that reported in Appendix B of the Feasibility Study for "aged" canals in this geographic region, and is considered appropriate for this analysis. For the overbank areas and upper canal slopes, where vegetative growth may be increased and the section may typically be cleared but not maintained, a value of 0.035 was assigned for the roughness coefficient. For this analysis, the roughness coefficient assigned to the overbank has little influence on the results, as little or no overbank flow exists for the conditions considered.

Modifying values for irregularities, channel section changes and variations in alignment such as meanders, were considered of negligible effect in the canal reach considered. For concrete sections, a Manning's *n* of 0.013 was typically assigned, however, it may be noted that a coefficient of 0.015 was utilized in the *Water Preserve Areas Feasibility Study, Engineering Appendix B (USACE, 2001)* for analysis of the inverted siphon losses. As discussed herein, the siphon was analyzed separately.

In those limited areas where revetment is to be installed, a coefficient of 0.040 was utilized, however, friction losses due to reveted sections were considered negligible to other losses at structures. The upstream and downstream revetment at the S-381 spillway were the only areas of revetment included in the analysis.

5.4.4. Expansion and Contraction Losses

Flow contractions and expansion coefficients were assigned for subcritical flow in all design cases considered. For gradual transitions, in the relatively uniform and straight canal reaches, a contraction coefficient of 0.1 was assigned with an expansion coefficient of 0.3 assigned to account for minor losses. Contraction coefficients in the range of 0.1 to 0.3 represent a degree of flow constriction between 0.25 and 0.50.

For the transitions at structures, typical values for subcritical flow of 0.3 for contractions and 0.5 for expansions were utilized.

The above expansion and contraction coefficients are taken from guidance contained in HEC-RAS River Analysis System V3.1.2 – Hydraulic Reference Manual (USACE, 1998, 2004)

As discussed above, the siphon structure was analyzed and modeled separately and is the only case where abrupt transitioning of the channel was examined. For the expansion coefficients in abrupt transitions, a value of 0.8 was assigned.

This value was checked against the following empirical equation taken from *Design of Small Canal Structures (USBR, 1983, 1991)*:

$$Ce = -0.09 + 0.570\,[D_x / D_c] + 0.075\,[F_{c2} / F_{c1}]$$

Where:

Ce = maximum value of expansion coefficient.

D_x / D_c = the ratio of the hydraulic depth at the fully expanded flow section to the hydraulic depth in the channel section.

F_{c2} / F_{c1} = the ratio of the Froude number in the upstream section of the expansion reach to the Froude number in the fully expanded flow section.

For an initial water surface elevation of 3.5 ft. NGVD at the headwater of S-9, application of the above empirical equation would result in a maximum expansion loss coefficient of 0.88.

5.5. Structures

5.5.1. Pump Station S-9 and S-9A

Pump Stations S-9 and S-9A were not directly modeled; flow profiles were developed to account for the operation of the pump stations during a flood event. Pump Station S-9 has a full pump capacity of 2,880 cfs. Pump Station S-9A has a full pump capacity of 500 cfs. A combined pump capacity for both stations operating at peak capacity is 3,380 cfs, although the combined operation of the two stations for removal of flood water from the basin is prohibited under the established operating rules. For the purposes of the model, a maximum allowable drawdown elevation of 1.0 NGVD was assigned at the headwaters of the pump stations, with an average canal stage of 3.5 NGVD assigned, with a potential maximum stage of 6.0 NGVD considered. For the analysis case where flow will be to the WCA during flood events, pump station S-9 was considered to be in full operation backpumping to WCA-3A.

5.5.2. Siphon Structure S-502

As shown in the Feasibility Study, Siphon Structure S-502 would be situated in the C-11 West Canal between Pump Station S-9 and U.S. Highway 27. The basic definition of this structure was taken directly from the Feasibility Study. Figures 5.2, 5.3 and 5.4, excerpted from the Feasibility Study, present a general plan of the area in which the siphon would be located; a detailed site plan; and structure dimensional data for the siphon, respectively.

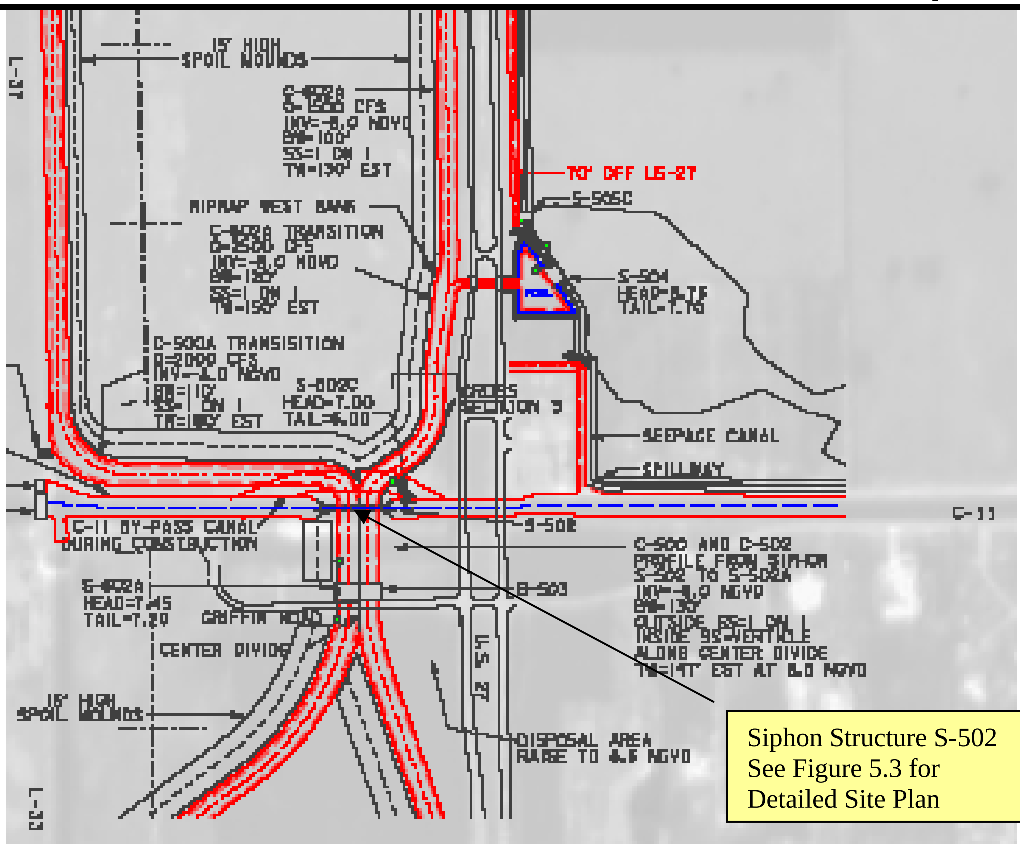

Figure 5.2 General Plan, Siphon Structure S-502

The throat section of Siphon Structure S-502 would consist of two 18' wide, 12' high reinforced concrete box culverts at an invert elevation ranging from -25.0 to -25.5 ft. NGVD, 260 feet in length. The overall length of the enclosed section of the structure is 376 feet; water surface elevations above 0.25 ft. NGVD would completely submerge the enclosed section of the culvert (with the exception of a part of the length of the upstream transition section, in which the maximum crown elevation is 5.5 ft. NGVD).

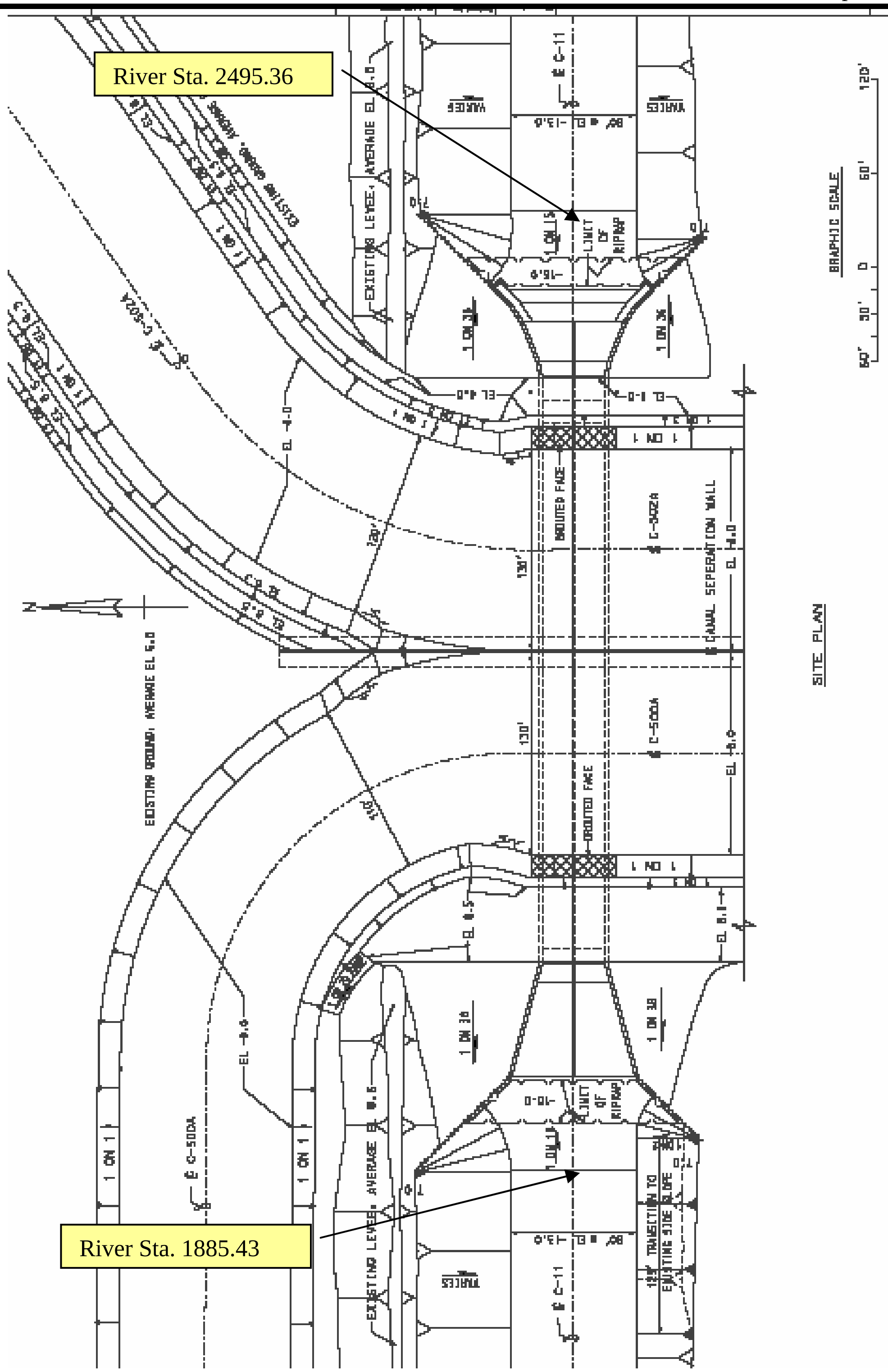

Figure 5.3 Site Plan, Siphon Structure S-502

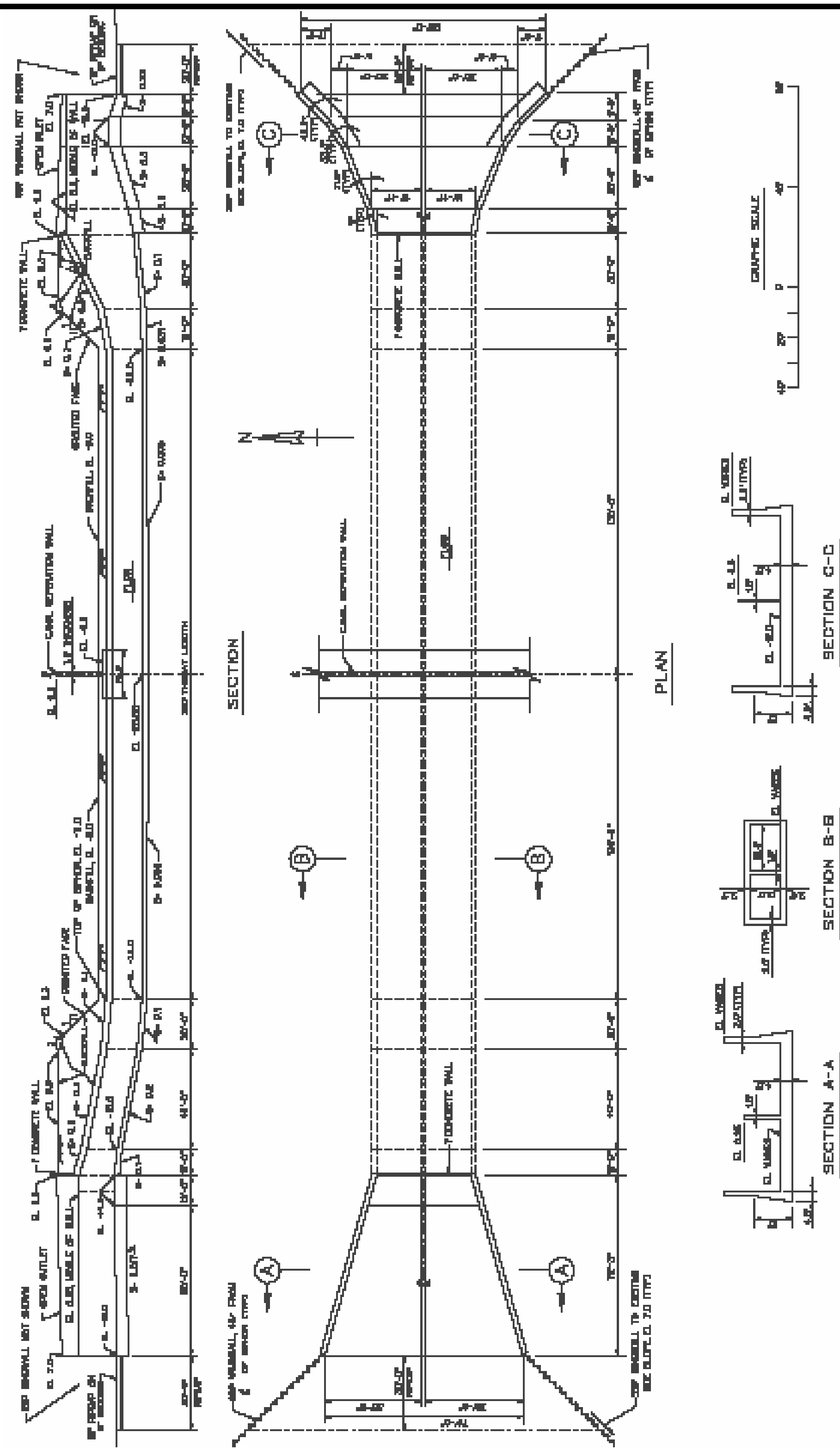

Figure 5.4 Siphon Structure S-502 Details

The inverted siphon was modeled by inclusion of a rating curve boundary at the location of the siphon. A separate geometry file was developed for the structure and the flow plans were applied to the siphon reach of the canal in order to cross reference hydraulic performance data for the structure and verify the loss rating, as well as to compare the backwater calculation through the structure. HEC-RAS does not readily support an inverted siphon or typical closed conduit structure, however, the culvert and bridge routines can be utilized to approximate flow and losses at the structure, as well as to examine performance under partial and/or low flow conditions. Adequate maintainable surcharge / driving head at the structure is a concern and should design and installation of the inverted siphon be developed, additionally more detailed analysis of its performance under all anticipated conditions is recommended.

5.5.3. U.S. Highway 27 Bridges

The U.S. Highway 27 / State Road 25 bridges are each five-span bridges having an overall length of 140.0 ft., with each span 28.0 feet in length. The lowest chord elevation of the bridges is 10.63 ft. NGVD. Each of the four intermediate bents are founded on six 18”x18” concrete piles, yielding a clear opening between bents of 2.5 feet. The bridges’ design was based on a design cross section in the C-11 West Canal consisting of a trapezoidal channel having a bottom width of 60’; an invert elevation of -13.0 ft. NGVD; and canal bank slopes of 1.5H:1V.

The bridges were modeled with the typical bridge routines within HEC-RAS. Some head loss and flow attenuation is exhibited in the area immediately upstream of these bridges; however, due to the low canal velocities, the backwater effects are inconsequential in nature. The hydraulic regime downstream of the bridges does not increase or induce an additional available surcharge head for the siphon operation under gravity flow conditions.

5.5.4. Spillway Structure S-381

Structure S-381 consists of a three-bay Obermeyer bascule gate spillway. Controlling dimensions for Structure S-381 are presented in Table 5.1.

Table 5.1 Structure S-381 Dimensions

Description	Units	Value
Number of Gate Bays	Ea.	3
Net Width of Each Gate Bay	Ft.	30.0
Top of Gate Elevation, Fully Up	Ft. NGVD	5.5
Top of Gate Elevation, Fully Down (Approximate)	Ft. NGVD	-10.0
Service Bridge Low Chord Elevation	Ft. NGVD	7.0
Training Wall Top Elevation	Ft. NGVD	7.5
Overall Gate Well Structure Length	Ft.	48.5
Downstream Structure Sill Elevation	Ft. NGVD	-13.05
Upstream Structure Sill Elevation	Ft. NGVD	-11.5
Upstream and Downstream Transitions to Canal		
Length (All Revetted)	Ft.	75
Canal Revetment Top Elevation	Ft. NGVD	5.0
Upstream and Downstream Canal Section		
Bottom Width	Ft.	59.0
Invert Elevation	Ft. NGVD	-15.0
Canal Bank Side Slope	H:V	2.5

The Obermeyer bascule gate spillway structure was modeled with gates full open (e.g., lowered to minimum gate panel elevation). As an operation rule for the structure, no interim position was considered in either design case. The basis for analysis was the flow case with the canal under flood control flow in which Pump Station S-9 is backpumping to WCA-3A at full capacity of 2,880 cfs and gates at S-381 are full open.

5.6. Hydraulic Design Considerations

5.6.1. Inverted Siphon

The inverted siphon was analyzed separately, as noted, as a closed pressure conduit. At design capacity (2,880 cfs), the structure should ideally operate without excess head. Allowable velocity, available head and other economic considerations influence the design feasibility of the structure.

The computed velocity within the siphon is 6.67 fps at the design case. This falls within the range of 3.5 to 10 fps typically recommended for siphons. For the basic design presented, the maximum allowable velocity would typically be limited to 10 fps or less for a longer siphon with a concrete transition. Head losses across the structure include:

the convergence loss at the inlet transition; friction and bend losses within the conduits; divergence losses at the outlet transition section; and other minor form losses associated with convergence and divergence at transition sections upstream and downstream with the canal. Normal design guidance for such structures would include a 1.10 factor of safety applied to the total computed head loss to assure that headwater is not excessive upstream of the structure. Further, the canal bank elevation upstream of the siphon would normally be increased to insure adequate freeboard to accommodate any excessive headwater conditions.

Friction losses within the conduit were computed utilizing: Manning's equation, which assumes that energy losses are dependent on the velocity, conduit size and roughness coefficient; and with check calculations against the Darcy-Weisbach equation, which is typically applied to large conduits flowing under pressure, considering the velocity, conduit size, fluid viscosity and density (Reynolds Number) and the relative roughness of the conduit walls. The Manning's equation was applied to develop an envelope loss rating for the structure yielding a bracketed range of losses where minimum losses equates to maximum flow and is utilized to examine the downstream effects of the siphon; and conversely, maximum losses equate the minimum flow through the structure and is used to determine the upstream, headwater effects of the siphon.

The Darcy-Weisbach formula utilized for closed conduit flow is as follows:

$h_f = (f\,L / D)*(v^2 / 2g)$, where

h_f = friction loss in the closed conduit (ft.);
f = friction factor in the Darcy-Weisbach equation;
L = length of conduit in feet;
v = velocity in feet per second, and;
g = acceleration due to gravity (32.2 feet per sec^2).

The Mannings formula used for closed conduit flow is as follows:

$h_f = 29.1\, n^2\, (L / r^{1.33})*(v^2 / 2g)$, where

h_f = friction loss in the closed conduit (ft.);
n = Manning's roughness coefficient;
r = hydraulic radius of conduit =cross sectional area divided by wetted perimeter, (ft.);

L = length of conduit in feet;

v = velocity in feet per second, and;

g = acceleration due to gravity (32.2 feet per sec^2).

The above equations for the computation of head loss due to closed conduit flow are each taken from Part 10 of *Design of Small Dams (USBR, 1987)*, equations (9) for the Darcy-Weisbach formula and (10) for the Manning's formula.

For minimum losses and maximum flow, a Manning's *n* value of 0.013 was assigned for the concrete conduit to account for friction losses within the structure. For the design case analyzing maximum losses and minimum flow, an *n* value of 0.015 was assigned.

A summary of the various head loss calculations for the inverted siphon Structure S-502 is presented in Table 5.2.

Table 5.2 Inverted Siphon Loss Rating

ENERGY LOSSES Head (ft.)	ELEMENT	MANNING'S Minimum Losses / Maximum Flow $n = 0.013$	MANNING'S Maximum Losses / Minimum Flow $n = 0.015$	DARCY WEISBACH
h_e	Approach [Convergence]	0.138	0.138	0.138
h_c	Transition [Entrance]	0.069	0.069	0.069
h_f	Conduit [Friction]	**0.463**	**0.615**	**0.556**
h_b	Bend [Form]	0.048	0.048	0.048
h_v	Exit	0.276	0.276	0.276
h_{ex}	Transition [Expansion]	0.207	0.207	0.207
	TOTAL	**1.202**	**1.354**	**1.295**
	F.S. = 1.1	**1.322**	**1.489**	**1.424**

The entrance loss coefficients aggregate to an approximate Ke = 0.30 with an exit loss coefficient combined to Kex = 0.70 (summation of entrance and exit form losses taken from Table 5.1, as applied to the barrel velocity head of (0.69 ft.). The velocity head in the siphon barrel is calculated to be 0.6908 ft. at a conduit velocity of 6.67 fps, based on the maximum design flow of 2,880 cfs and a barrel cross sectional area of 432 square feet (two 18'Wx12'H rectangular barrels). The loss estimated through use of the Darcy-Weisbach friction loss formula falls intermediate to the ranges computed using the Manning's friction loss formula, and was selected as representative of the losses associated with the siphon structure under maximum design flow conditions.

For establishing ratings of the siphon, the actual computed loss of 1.30 feet using the Darcy-Weisbach friction loss formula is recommended. The resultant rating for Siphon Structure S-502 under given rate of discharge can then represented by the following formula, applicable to all cases in which the siphon crown is submerged (e.g., full flow):

$H_T = 1.876*(Q/432)^2/2g$, where

H_T = total head loss across S-502 (ft.)

Q = total discharge (cfs)

g = acceleration due to gravity (32.2 ft/sec^2)

5.7. Summary of Water Surface Profile Computations

Table 5.3 presents a summary of the HEC-RAS analysis of water surface profiles in the C-11 West Canal in that reach extending east from the headwater pool of Pumping Station S-9 to a point immediately upstream (east) of the southeasterly corner of the C-11 Impoundment. More detailed information is included in Appendix D. The data summarized in Table 5.3 was computed for an assigned S-9 headwater elevation of 3.50 ft. NGVD. The upstream end of the reach of interest is at River Station 8355 in the HEC-RAS output, located approximately 8,410 feet east of S-9.

Table 5.3 Summary of HEC-RAS Results

Discharge (cfs)	Computed Water Surface Elevation			Change from Existing	
	Existing	With S-381	With Project	With S-381	With Project
1,050	3.55	3.57	3.74	+0.02	+0.19
2,880	3.88	3.89	5.11	+0.01	+1.23

Little increase in water surface elevation results from addition of S-381 to the existing canal model. However, the increase associated with addition of the inverted siphon structure S-502 is clearly significant and, should S-502 be constructed, would clearly require mitigation.

5.8. Conclusions and Recommendations

Based on the analyses summarized above, it is concluded that:

1. The occurrence of one or more bypass events at the C-11 Impoundment of sufficient magnitude to require operation at the full capacity of S-9 should be considered probable given only a repetition of basin rainfall experienced over the period 1965-1995. The results of the District's simulations indicate that at least one event in that period would have required a mean daily discharge equal to the full capacity of S-9 (2,880 cfs).
2. The construction of Structure S-502 would, under that maximum discharge of 2,880 cfs, introduce an additional head loss of 1.42 feet (with the normally recommended factor of safety of 1.1), and 1.30 feet without that factor of safety in the reach of interest.
3. The hydraulic gradient of the C-11 Canal within the reach of interest is insufficient to measurably attenuate that induced head loss between the siphon inlet and the upstream end of the model domain (e.g., the induced head loss propagates to and beyond the upstream end of the model domain).
4. An incremental head loss of over one foot must be considered significant in this system, where substantial upstream areas discharge to the C-11 Canal by gravity, and should be mitigated in some manner.

Recommendations for mitigating that incremental head loss are included in Part 6 of this document.

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Table of Contents

6. RECOMMENDED OPERATIONS AND DESIGN REFINEMENTS

The purpose of analyses summarized in this Part 6 is to refine the conceptual operational plans developed during CERP and the WPA Feasibility Study, to more efficiently move water between the Broward County WPA components and regain storage in the impoundments in order to capture subsequent events to minimize C-11 Western Basin storm water from being pumped to WCA-3A.

These analyses are comprised of the following primary elements:

- Identification of those operational and/or structural changes to the Selected Plan necessary to offset any flood protection impacts identified in earlier Parts of this document;
- Identification of those operational and/or structural changes to the Selected Plan necessary for improved conformance with defined design criteria;
- Identification of such further operational and/or structural changes to the Selected Plan as might be appropriate to minimize stormwater pumping from the C-11 West Basin into WCA-3A;
- Identification of other operational and/or structural changes as might be appropriate in the interest of reducing project costs.

The principal focus of this Part 6 is on those elements of the overall plan of improvement presently intended to be complete and operational in 2010. However, a discussion of the compatibility of structural and/or operational changes recommended herein with the ultimate plan of improvement in the area (e.g., following completion of the North and Central Lake Storage Areas and other related components of CERP) is included at the end of this Part 6.

6.1. Recommendations to Offset Flood Protection Impacts

Potential negative impacts on flood protection identified in earlier parts of this document are generally limited to:

- Potential for increased volume and/or duration of flooding events on the C-11 West Canal resulting primarily from increased seepage into the primary canals associated with higher groundwater stages;
- Probability under extreme runoff events (e.g., events requiring diversion of C-11 Basin runoff to S-9) of an increased stage in the C-11 West Canal resulting from hydraulic losses associated primarily with siphon Structure S-502.

The following recommendations are offered in the interest of mitigating the above potential flood protection impacts. With respect to the potential for increased volume and/or duration of flooding events on the C-11 West, mitigating measures could include both a modified operation of the C-11 West Canal during the wet season, and/or provision of sufficient pumping capacity at S-503 to accommodate the increased runoff volumes. Each of those two mitigation measures are discussed below, followed by a discussion of a suggested approach to mitigating increased canal stages resulting from hydraulic losses associated with siphon Structure S-502.

6.1.1. Modify Wet Season Operations in C-11 West Canal

As discussed in Part 2, the maximum overall increase in groundwater stage in the C-11 West Basin resulting from implementation of the project as it is presented in the Feasibility Study was computed to occur on November 1, 1990. Additional analyses summarized in Part 2 concluded that, upon the assumption that a major rainfall event would have occurred on that date, the overall volume of runoff from the C-11 West Basin would have been increased from that which would have been expected for hydrologic conditions in the watershed as they would have existed in 2000 (e.g., pre-project conditions). The overall increase in runoff volume for 72-hour events having return periods of 10, 25 and 100 years was estimated as follows (taken from Tables 2.17, 2.18 and 2.19, respectively):

- 10-year event: increased runoff volume of 33,333 acre-inches (2,778 acre-feet);
- 25-year event: increased runoff volume of 30,225 acre-inches (2,519 acre-feet);
- 100-year event: increased runoff volume of 24,837 acre-inches (2,070 acre-feet).

The above values were computed inclusive of additional seepage into the canals resulting from the higher groundwater stages in the basin. Without consideration of the additional seepage associated with the higher groundwater stages, it was concluded in Part 2 (see Tables 2.9, 2.10 and 2.11) that the volume of runoff "with project" would have been reduced from the baseline condition for each of the three hypothetical storm events considered as follows:

- 10-year event: decreased runoff volume of 2,848 acre-inches (237 acre-feet);
- 25-year event: decreased runoff volume of 5,955 acre-inches (4,96 acre-feet);
- 100-year event: decreased runoff volume of 11,345 acre-inches (945 acre-feet).

It can be concluded from the above that the entire increased volume of runoff is associated with the increased seepage into the canals resulting from the higher groundwater stages in the basin. The volume associated with that increased seepage is 36,180 acre-inches (3,105 acre-feet) for each of the three events analyzed, computed as as an increased seepage inflow rate of 182 cfs applied to the entire runoff hydrograph time base of 200 hours.

Those higher groundwater stages result from a combination of increased seepage induced by storage in the C-11 Impoundment and WCA-3A/3B Seepage Management Areas, and modified wet-season operations of primary water control structures controlling stages in the C-11 West Canal.

In the "1995 Base" SFWMM simulation, wet-season operations at Pumping Station S-9 (at the west end of the C-11 West Canal) were initiated when canal stages reached 4.0 ft. NGVD, and terminated when canal stages fell to 3.1 ft. NGVD. Structure S-13A (at the east end of the C-11 West Canal) was simulated to open when the canal stage reached 4.0 ft. NGVD, and to close when the canal stage fell to 3.1 ft. NGVD. The operations logic embedded in the "1995 Base" simulation are taken as representative of actual operations in December, 2000. In the "2010 WPA" SFWMM simulation, wet-season operations at Pumping Station S-503 (at the west end of the remaining C-11 West Canal) were initiated when canal stages reached 3.9 ft. NGVD, and terminated when canal stages fell to 3.5 ft. NGVD. Structure S-13A (at the east end of the C-11 West Canal) was simulated to open when the canal stage reached 4.0 ft. NGVD, and to close when the canal stage fell to 3.3 ft. NGVD. It is concluded from the above that the maximum daily groundwater stage difference summarized in Figure 2.5 is heavily influenced by the change in wet-season operations of the C-11 West Canal between the "1995 Base" and "2010 WPA" simulations. The change in simulated canal operating elevations during the wet season can be expected to account for an average increase of between 0.2 and 0.4 feet in groundwater elevation in the C-11 West Basin as a whole. That change in and of itself explains much of the increased ground water stages on November 1, 1990 summarized in Figure 2.5.

One means of mitigating those increased runoff volumes would be to modify the “with project” operation of the primary water control structures on the C-11 West Canal. A change in operation of Pump Station S-503, in which the pump operation range is changed from 3.9-3.5 ft. NGVD to 3.6-3.1 ft. NGVD, could be expected to reduce the ground water stage differential by roughly 0.3 feet, thereby removing the bulk of the C-11 West Basin from the area potentially impacted by the project.

Figure 2.22 in Part 2 of this document presents the estimated basin-wide average relationship between soil moisture storage and depth to groundwater table in the C-11 Basin. For water table depths greater than two feet, the relationship shown in Figure 2.22 may be closely approximated as 1/4” of soil moisture storage per inch of change in water table depth. For water table depths between 18” and 24”, the relationship may be approximated as 1/6” of soil moisture storage for each inch of change in water table depth. Simulated pre-and-post-condition water table depths in the C-11 West Basin on November 1, 1990 (date of maximum simulated difference in water table depth) are shown in Figures 2.20 and 2.21 in Part 2 of this document.

For water table depths of 24” and greater, a reduction of 0.3 feet (3.6”) in the groundwater stage at the beginning of the rainfall events can be expected to increase the available soil storage by roughly 0.9 inches of water, reducing the storm runoff volume by that same amount. Over an area of 40,000 acres, the resultant reduction in runoff volume would be 36,000 acre-inches, adequate to completely offset the increased runoff volumes for each of the three rainfall events considered (see Tables 2.17, 2.18 and 2.19 in Part 2 of this document). For a change (reduction) in mean water table depth of 0.3 feet in a range of water table depths from 18” to 24”, the estimated reduction in runoff volume would be 24,000 acre-inches, closely approximating the estimated increase in storm runoff volume from a synthetic 100-year event applied to simulated basin conditions on November 1, 1990 (see Table 2.19).

6.1.2. Assure Adequate Capacity of Pump Station S-503

In the above section, it was concluded that the potential for increased flood runoff volumes in the C-11 West Basin could be mitigated through modified wet-season operation of the primary water control structures on the C-11 West Canal. Should it be determined that those modified operations cannot be implemented, an alternative would be to assure provision of adequate pump capacity at Pump Station S-503 to accommodate the increased runoff volumes.

As presented in the Feasibility Study, Pump Station S-503 (inflow pump station to the C-11 Impoundment) will provide a nominal capacity of 2,575 cfs, and is meant to essentially replace the drainage and flood control function of existing Pump Station S-9 in the C-11 Basin. S-9 provides a capacity of 2,880 cfs (see description in Part 1). The capacity of S-503 is distributed as 2,500 cfs to flood control and drainage on the C-11 Canal, and 75 cfs to return of seepage collected in the seepage collection canal C-511.

The lone discussion of the determination of flood control pump capacity at S-503 in the Feasibility Study is found at Table B.10.5.2, which states “Pump Station Capacity Criteria: Maintain flood protection of the Western C-11 basin at existing levels that are currently provided by the S-9 pump station. The Western C-11 Critical Project provides a pump station S-9A for seepage collected in the reach between S-9 and critical project spillway S-381. The 2,500-cfs pump capacity provided by S-503 combined with the additional 500 cfs provided by S-9A is greater than the 2880-cfs pump capacity of S-9”.

As stated in the C&SF Project Master Water Control Manual, S-9 was originally designed to remove ¾” per day of water from a 71-square mile tributary area (1,410 cfs) plus a maximum expected seepage rate collected in the L-33 and L-37 borrow canals of 1,460 cfs. The SFWMD subsequently determined that the seepage allowance of 1,460 cfs was excessive, thereby making available additional capacity for drainage of the C-11 West Basin. The design capacity of Pumping Station S-9A (500 cfs) was established to provide an adequate rate of removal for seepage accumulated in L-33 and L-37, and for removal of runoff from those areas of the basin lying west of U.S. Highway 27.

At present, the Indian Trace Development District and the South Broward Drainage District in the C-11 West Basin are each permitted to discharge at a removal rate of 1-1/4” per day. For this analysis, a minimum flood control and drainage capacity of 1-1/4” per day from the entire C-11 West Basin is assumed.

The total area of the C-11 West Basin as it existed in December, 2000 was approximately 45,600 acres (71.2 square miles). Following completion of the WCA-3A and WCA-3B Seepage Management Project and the C-11 Impoundment, a total of approximately 40,000 acres will remain tributary to the C-11 West Canal at Structure S-381 and Pump Station S-503. A removal rate of 1-1/4" per day from a 40,000-acre tributary area yields a required capacity at S-503 of 2,100 cfs, to which should be added an allowance for seepage inflows to the C-11 West Canal and other primary canals east of S-503.

As discussed in Part 4, analysis of the District's recorded data at Pump Stations S-9 and S-9A indicates that the maximum mean daily rate of removal from the C-11 West Basin (exclusive of seepage inflows along the L-33 and L-37 borrow canals) was approximately 2,579 cfs (occurring on October 1, 2004). That discharge is equivalent to a removal rate of 1.35" per day over the 45,600-acre area of the C-11 West Basin, and included removal of any seepage entering the primary canals. Applying that removal rate to the 40,000-acre basin area remaining after completion of the project, it is concluded that a capacity of not less than 2,290 cfs at S-503 would be necessary to maintain the maximum historic rate of removal for both drainage and seepage inflows to the primary canals.

In Part 3, it was estimated that the maximum rate of pumped seepage return from the C-511 Seepage Canal along the east side of the C-11 Impoundment would be 35.0 cfs (see Table 3.11 for an impoundment stage of 10.0 ft. NGVD, Scenario A). In order to maintain a factor of safety of five with respect to seepage flows (criteria defined in the Feasibility Study), the desired pump capacity for return of seepage captured in C-511 is 175 cfs.

Also in Part 3, it was estimated that the total seepage lost from the C-11 Impoundment to the north and east of the impoundment was 500 cfs, again taken from Table 3.11 for an impoundment stage of 10.0 ft. NGVD and Scenario A. That total includes estimated seepage along sections 2, 3A, 3B and 3C as they are identified in Part 3. Of that total, an estimated 35 cfs would be captured in the C-511 Seepage Canal (see above), leaving an estimated 465 cfs passing beneath the seepage canal and into areas east and north of the impoundment. It should here be noted that the seepage passing beneath the seepage collection canal includes not only seepage induced by the impoundment, but also that seepage passing east which would occur prior to construction of the project.

The nominal total capacity of 2,575 cfs for Pumping Station S-503 presented in the Feasibility Study closely approximates the maximum historic mean daily rate of pumping at S-9 and S-9A combined (2,579 cfs), and can be considered to include the following for conditions expected to exist following completion of the project:

- A capacity of 2,100 cfs for removal of runoff from the C-11 West Basin (1-1/4" per day over a remaining tributary area of 40,000 acres);
- A capacity of 175 cfs for the return of seepage collected in the C-511 seepage canal (established with a factor of safety of 5 applied to the estimated collection rate of 35 cfs);
- A remaining capacity of 300 cfs for increased seepage to the east from the C-11 Impoundment and the WCA-3A Seepage Management Project combined.

The adequacy of the final capacity listed above (300 cfs for increased seepage to the east) is presently uncertain. It is recommended that more detailed analyses be conducted during the final design phase, in which the total seepage to the east is compared for pre-and-post project conditions, using analytical methods such as SEEP2D in lieu of MODFLOW, and reflecting the results of additional, more detailed subsurface investigations.

6.1.3. Eliminate Siphon Structure S-502

The single purpose of the proposed siphon Structure S-502 is to maintain a complete separation of C-11 West Basin runoff from flows to be carried in Canals C-500 (A and B) and C-502 (A and B). Until implementation of the North Lake Belt Storage Area (NLBSA) or Central Lake Belt Storage Area (CLBSA), Canal C-500 will not exist, and Canal C-502 will exist only as needed for the transfer of flows from the C-11 Impoundment to the C-9 Impoundment. During that interim period, it would appear that the single operational benefit accruing to the presence of S-502 would be the capacity to maintain stages in the C-502 Canal (and, presumably, the WCA 3A and WCA 3B seepage management regions) above those in the C-11 Canal west of S-381. Stages in the C-11 Canal west of S-381 would need to be lowered whenever it would become necessary to bypass C-11 West Basin runoff around the C-11 Impoundment to Pump Station S-9.

That need for bypass (and associated need for lowering of the C-11 Canal west of S-381) can be eliminated through an operational change in which bypass of the C-11 Impoundment through S-381 is replaced by continued operation of S-503, coupled with operation of S-504, when it is necessary to operate S-9 (e.g., when both the C-11 and C-9 impoundments are full). A disadvantage of this operational change is the resultant double pumping of C-11 West Basin runoff when the two impoundments are full. As noted in Part 4, the District's SFWMM simulation for 2010 conditions with the WPA project resulted in an estimated average annual bypass of the C-11 Impoundment of roughly 5,000 acre-feet per year. The savings associated with elimination of S-502 should readily exceed the additional operating costs for double-pumping of that average annual volume of water.

Should the above operational change not be implemented, it would then continue to be necessary to bypass excess C-11 West Basin runoff through Structure S-381 to Pump Station S-9. Hydraulic analyses presented in Appendix D and summarized in Part 5 of this document include an analysis of the incremental head loss in the C-11 Canal for a condition wherein S-502 is not constructed and S-381 is in place and operational. Those analyses conclude that the incremental head loss resulting from that condition (e.g., change in canal stage immediately east of the location of S-381, as compared to pre-project conditions) of 0.02 feet or less, dependent upon discharge. That estimated increase in canal stage is considered insignificant.

6.1.4. Add Seepage Control Facilities on C-11 Canal West of S-381

Given the recommended elimination of Siphon Structure S-502, stages in the C-11 Canal between U.S. Highway 27 and S-381 will parallel those in the WCA-3A Seepage Management Area, and thus can be expected to generally vary between 5.5 and 7-7.5 ft. NGVD. This reach of the C-11 Canal is at present normally held at or below elevation 4.0 ft. NGVD. It is therefore probable that it will be necessary to add seepage control facilities along that reach of the C-11 Canal to the project in order to avoid impacts to areas south of and adjacent to the C-11 Canal.

6.2. Recommendations for Minimizing Pumping to WCA-3A

Recommendations directed to minimizing the frequency and volume of stormwater pumping from the C-11 West Canal to WCA-3A generally focus on:

- Minimizing the volume of basin runoff directed to the C-11 Impoundment through increased operational emphasis on maximizing discharges to the east at S-13A;
- Maximizing recovery of storage in the C-11 and C-9 Impoundments through increased operational emphasis on impoundment drawdown;
- Maximizing the availability and use of storage for C-11 West Basin runoff in the C-9 Impoundment;
- Maximizing the availability and use of other potentially available system storage in the project area.

6.2.1. Maximize Discharge Through S-13A During Runoff Events

During the wet season (e.g., roughly May 1 through October 31 of any given year), it will be desirable to maximize the quantity of C-11 West Basin runoff discharged to the east through Structure S-13A, so that the proportion of the remaining runoff from the C-11 West Basin that can be captured in the C-11 and C-9 impoundments is maximized. It appears that the need for this operational strategy has been recognized by the SFWMD in its simulation of "with project" conditions. The SFWMM simulation of 2010 WPA conditions results in an average daily discharge of 75 cfs (54,340 acre-feet per year) through S-13A over the 31-year period simulated (calendar years 1965-1995). Recorded data at S-13A taken from the District's DBHYDRO database (DBKEY P0955) for calendar years 1990-2000 indicate an actual average daily discharge through S-13A of 36 cfs (25,720 acre-feet per year). Maximizing the quantity of C-11 West basin discharges through S-13A will require that the structure be automated such that it may be controlled and monitored from the District's Operations Center in West Palm Beach.

6.2.2. Discharge Through S-13A to Draw Down C-11 Impoundment

In Part 4 it was concluded that it would be desirable to consider impoundment drawdown as a principal management objective, at least during the wet season in order to minimize future discharges of basin runoff to WCA-3A at S-9. Given the automation of S-13A recommended above, it should be possible to effectively implement reservoir drawdown through concurrent operation of S-504 (discharging from the C-11 Impoundment), S-381 (directing those discharges to the east) and S-13A.

6.2.3. Discharge through S-510 and S-511 to Draw Down C-9 Impoundment

In Part 4 it was concluded that it would be desirable to consider impoundment drawdown as a principal management objective, at least during the wet season, in order to minimize future discharges of basin runoff to WCA-3A at S-9. That objective can be accomplished through concurrent operation of S-510 and S-511 (when capacity is available in the C-9 Canal east of S-511) without the need for additional structures or capital investment.

6.2.4. Modify Operation of S-504 for Increased Transfer to C-9 Impoundment

Structure S-504 is expected to consist of a three-bay gated spillway designed for a nominal capacity of 2,500 cfs with a headwater elevation (C-11 Impoundment) of 8.75 ft. NGVD and a tailwater elevation (Canal C-502A) of 7.70 ft. NGVD (head differential of 1.05 feet). As contemplated herein, S-504 can be expected to serve three functions until such time as the NLBSA and/or CLBSA components of CERP are implemented:

- The transfer of up to 1,000 cfs from the C-11 Impoundment to the C-9 impoundment;
- The release of stored waters from the C-11 Impoundment intended to be sent east through S-381, either as water supply or for the purpose of impoundment drawdown;
- The transfer of flow from the C-11 Impoundment to the headwater pool of Pump Stations S-9 and S-9A when all project storage has been exhausted and it becomes necessary to discharge to WCA-3A. In that instance, the maximum discharge from S-504 could be roughly equal to the capacity of inflow pump station S-503 (taken as 2,930 cfs for this analysis).

The current SFWMM simulation is structured such that the C-11 Impoundment is filled to roughly 90% of its volume at normal pool before transfers are made to the C-9 Impoundment. It is recommended that those transfers be initiated at the lowest practicable stage in the C-11 Impoundment so that maximum utilization may be made of the available storage in both impoundments. In addition, that adjustment can be expected to contribute to an improved drawdown capability at the C-11 Impoundment. During those transfers, the operation of S-504 should be driven based on its tailwater stage; the tailwater at S-504 should be maintained at or below elevation 7.0 ft. NGVD to minimize the potential for unintended flood impacts at the Holly Lakes Mobile Home Community.

Given the structure geometry defined in the Feasibility Study, for a tailwater elevation of 7.0 ft. NGVD, a discharge of 1,000 cfs, all gates open equally, and submerged uncontrolled flow, the headwater elevation would be just below 7.2 ft. NGVD. Greater reservoir stages would require that submerged, controlled flow be employed to limit the discharge to 1,000 cfs and/or the tailwater elevation to 7.0 ft. NGVD.

6.2.5. New Gated Spillway in C-11 Canal West of US-27

Given the reconfiguration and modified operation contemplated herein, the headwater pool of Pump Station S-9A would, with no other change, be directly connected to the upper end of Canal C-502B. As a result, whenever flows are transferred from the C-11 Impoundment to the C-9 Impoundment, it would be necessary to interrupt operation of S-9A to prevent the possible unintended discharge of basin runoff to WCA-3A. The cessation of pumping at S-9A would result in seepage along the L-33 and L-37 borrow canals (which would otherwise be pumped to WCA-3A) adding to the basin runoff volume to be stored, in essence reducing the effective storage of the project.

That concern can be allayed through the addition of a new gated spillway in the C-11 Canal between its confluence with Canal C-502B west of U.S. Highway 27 and Pump Stations S-9 and S-9A. It is anticipated that the new gated spillway would be similar in design and capacity to S-381. Given the cost of such a structure (expected to be in excess of $4 million), the benefit associated with the structure should be carefully considered prior to a determination to include it in the project prior to implementation of the NLBSA component of CERP.

6.2.6. Employ Available Storage in Wetland Mitigation Areas

Available storage in the wetlands mitigation areas adjacent to the two impoundments should be exhausted prior to initiation of pumping to WCA-3A at S-9. On the assumption that the stages in the mitigation areas are normally held at or below a depth of roughly 0.5 feet, and that a maximum mitigation pool depth of 2.0 feet is acceptable. The result of that assumption is the general availability of approximately 1.5 feet of storage depth in the mitigation areas, yielding an available storage volume of just over 300 acre-feet in the C-11 mitigation area, and approximately 540 acre-feet in the C-9 mitigation area.

6.2.7. Employ Available Storage in WCA-3A & WCA-3B Seepage Management Areas

Until such time as the NLBSA and/or CLBSA components are constructed and future Canal C-500A and Canal C-500B are constructed, it may be practicable to utilize available surface storage area in the WCA-3A and WCA-3B Seepage Management Areas to minimize the need for pumping to WCA-3A at S-9. As presented in the Feasibility Study, the future wet-season design stage in the WCA-3A Seepage Management Area is approximately 7.5 ft. NGVD; the future wet-season design stage in the WCA-3B Seepage Management Area is approximately 6.5 ft. NGVD. As considered herein, the controlling stages during the transfer of water from the C-11 Impoundment to the C-9 Impoundment are 6.0 ft. NGVD at the S-30 headwater, and 7.0 ft. NGVD at the S-504 tailwater.

It therefore appears that a storage depth of roughly 0.5 feet may be available throughout the extent of the seepage management areas for use as short-term storage. The available surface area in those seepage management areas is over 4,000 acres, with the result that over 2,000 acre-feet of water could possibly be stored on those areas prior to the need for pumping at S-9.

6.3. Recommendations for Reducing Cost

It is recommended that the SFWMD consider the following additional adjustments to the Selected Plan in the interest of reducing project cost, to the extent that implementation of the adjustments would not impair project functionality. Certain of these recommendations are directed primarily to a delay in construction and/or reduction in capacity of project features until such time as they are needed in connection with future CERP projects.

- At the C-11 Impoundment,
    - Relocation of S-504, coupled with elimination of S-504A;

- At the C-9 Impoundment,
  - Delay enlargement of the C-9 Canal;
  - Design S-510 for reduced initial capacity requirements.
- On the WCA-3A and 3B Levee Seepage Management Project,
  - Elimination of Structure S-502C;
  - Delay in replacement of existing Structure S-30;
  - Design initial construction of Canal C-502B for interim condition conveyance of 1,000 cfs in lieu of 2,500 cfs ultimate design conveyance.

6.3.1. Relocate S-504 and Eliminate S-504A

The location of Structure S-504 as presented in the Feasibility Study was established to permit the release of flows from the C-11 Impoundment destined for the C-9 Impoundment to a point upstream (north) of Siphon Structure S-502. With the recommended elimination of S-502, it would appear practicable to relocate S-504 to the southwesterly corner of the Impoundment, discharging directly to the C-11 Canal. This relocation of S-504 then permits the elimination of the S-504A culverts, originally intended to carry S-504 releases beneath U.S. Highway 27.

6.3.2. Delay Enlargement of C-9 Canal

The Feasibility Study includes enlargement of the C-9 Canal between S-30 and the eventual inflow structure for the North Lake Belt Storage Area (NLBSA) south of the C-9 Canal. That enlargement is associated with the eventual increase in discharge at S-30 to 2,500 cfs in connection with the NLBSA component of CERP. The enlarged canal is shown to consist of a trapezoidal channel having a bottom width of 50 feet at invert elevation -16.5 ft. NGVD, and side slopes of 1H:1V.

Until such time as the NLBSA component is implemented, the maximum design discharge through S-30 to this reach of the C-9 Canal is 1,000 cfs. The existing canal in this reach is estimated to consist of a trapezoidal channel having a bottom width of roughly 20 feet at an approximate invert elevation of -11 ft. NGVD, and side slopes of roughly 1H:1V. That cross section provides a waterway area of approximately 430 square feet below elevation 2.0 ft. NGVD (the lower end of the normal drawdown range for operation of inflow pump station S-509). At a discharge of 1,000 cfs, the average velocity in the channel would be roughly 2.3 fps, slightly below a presumptive non-scouring value of 2.5 fps for channels founded in medium sands.

Given the above, it would appear practicable to delay enlargement of the C-9 Canal until such time as the NLBSA component is implemented.

6.3.3. Design S-510 for Reduced Initial Capacity Requirements

As presented in the Feasibility Study, S-510 is to consist of a two-bay gated spillway providing a design capacity of 1,000 cfs with a headwater elevation (in the C-9 Impoundment) of 6.25 ft. NGVD and a tailwater elevation (in the C-9 Canal) of 5.75 ft. NGVD. Its principal function as contemplated in the Feasibility Study is to discharge water stored in the C-9 Impoundment to the south; that functionality will not be needed until such time as the NLBSA and/or CLBSA components are implemented. However, as recommended earlier in this Part 6, S-510 should be used to effect drawdown of the C-9 Impoundment whenever capacity exists in the C-9 Canal east of S-511. S-511 is designed to provide a capacity of 500 cfs with headwater at 4.4 ft. NGVD, and tailwater (C-9 Canal east of the structure) at 3.5 ft. NGVD.

It may desirable to initially construct S-510 to satisfy only its interim function (C-9 Impoundment drawdown) prior to completion of the NLBSA and/or CLBSA components. Upon the assumption that the basic character of the interim S-510 is similar to that proposed for S-511 (e.g., two 8-ft. diameter gated CAP culverts), it would appear practicable to effect a drawdown rate of 500 cfs (more if capacity exists in the C-9 Canal east of S-511) for all C-9 Impoundment stages at or above elevation 5.5 ft. NGVD. The available drawdown rate would be reduced for Impoundment stages below that elevation, to a minimum of approximately 350 cfs at an Impoundment stage of 4.5 ft. NGVD.

6.3.4. Eliminate S-502C

Structure S-502C consists of a culvert intended to discharge from Canal C-502A to the C-11 Canal. Its function would be to permit water supply deliveries, either from the north (following implementation of the NLBSA and/or CLBSA components), or from the C-11 Impoundment (via S-504). With the recommended elimination of S-502, the water supply function of S-502C can be replaced through operation (opening) of S-381 to permit discharges to the east. As a result, S-502C would no longer be needed and could be eliminated.

6.3.5. Delay Replacement of Existing S-30

Existing Structure S-30 is located in the C-9 Canal at U.S. Highway 27. It consists of a gated, three barrel reinforced concrete pipe culvert. Each barrel is 84 inches in diameter and 288 feet in length at an invert elevation of -5.0 ft. NGVD. The current structure replaced the original S-30 (which was designed to pass 560 cfs with a headwater (westerly) elevation of 4.4 ft. NGVD and a tailwater elevation of 3.5 ft. NGVD) when U.S. Highway 27 was widened to four lanes. The purpose of this structure is to prevent excessive seepage losses from WCA-3A by permitting higher stages in the L-33 Borrow Canal west of U.S. Highway 27; it also supplies water from the L-33 Borrow Canal during dry periods to maintain stages in the C-9 Canal. The gates at S-30 are closed when releases from S-30 would aggravate downstream flood conditions (defined as the presence of a tailwater stage above 3.0 ft. NGVD). In the absence of a tailwater stage above 3.0 ft. NGVD, gates are opened as necessary when the headwater stage exceeds 6.0 ft. NGVD.

It is intended that the existing Structure S-30 be replaced as one feature of the WCA-3A and WCA-3B Seepage Management Project. The new structure will consist of a two-bay gated spillway designed to pass 2,500 cfs with a headwater elevation (in the new C-502B Canal) of 6.0 ft. NGVD and a tailwater elevation (in the C-9 Canal east of U.S. Highway 27) of 4.0 ft. NGVD. S-30 is intended to control the conveyance of flows diverted from the C-11 Impoundment to the C-9 Impoundment and (eventually) the North Lake Belt Storage Area (NLBSA). It may also be used to control Lake Okeechobee water supply deliveries to the C-9 Basin, and can under certain conditions be operated in reverse flow conditions to pass water supply releases from the C-9 Impoundment and NLBSA directed to the south.

Until such time as the NLBSA project is implemented, the maximum design rate of delivery from the C-11 Impoundment to the C-9 Impoundment is 1,000 cfs. It may therefore be desirable to consider delaying the replacement of S-30 until the NLBSA and/or the Central Lake Belt Storage Area (CLBSA) projects are implemented.

Under a total discharge of 1,000 cfs, the estimated head loss through the existing Structure S-30 is 3.2 feet. That loss is computed assuming all three gates full open; an entrance loss coefficient of 0.9 velocity head; an exit loss coefficient of 1.0 velocity head; and a Manning's roughness coefficient of 0.015 for friction losses in the culvert barrels.

For a design headwater elevation of 6.0 ft. NGVD, the maximum tailwater elevation would then be 2.8 ft. NGVD, 0.7 feet below the currently defined "start pumping" elevation of the C-9 Canal at Pump Station S-509, but within the normal drawdown range of 2.0-3.0 ft. NGVD defined in Table B.10.6.1 of the Feasibility Study. It is concluded that the existing structure could serve to maintain headwater elevations (e.g., stage in the C-502B Canal) at 6.0 ft. NGVD given a discharge of 1,000 cfs and a tailwater elevation (Pump Station S-509 headwater) of 2.8 ft. NGVD or below. It should therefore be possible to delay replacement of S-30 until the NLBSA or CLBSA projects are implemented. In the interim, it would be necessary to, at a minimum, automate the operation of S-30; add telemetric monitoring and control capability; and in all probability improve revetment protection of the canal banks and invert at the structure entrance and exit.

6.3.6. Reduce Initial Design Capacity of C-502B Canal

As presented in the Feasibility Study, Canal C-502B is a new canal to be designed for a capacity of 2,500 cfs between the C-11 Canal and the C-9 Canal, and for a capacity of 2,000 cfs between the C-9 Canal and the C-6 (Miami) Canal. The reach between the C-9 Canal and the Miami Canal is not of interest to this analysis, as no improvement is contemplated in that reach until such time as the NLBSA or CLBSA components of CERP are implemented.

Between the C-11 and C-9 canals, the design capacity of 2,500 cfs will be needed only when the NLBSA and/or CLBSA components are implemented. Until that time, the maximum design rate of discharge in that reach of the C-502B Canal is 1,000 cfs. It may therefore be practicable to initially construct C-502B to a design capacity of 1,000 cfs and delay additional expansion to a capacity of 2,500 cfs to that point in time at which the future CERP components are implemented.

The preliminary design of C-502B presented in the Feasibility Study includes three distinct reaches:

- From the C-11 Canal south to the Holly Lakes Mobile Home Community (approximate length of 16,200 feet), the canal would be a trapezoidal channel having a bottom width of 130 feet, an invert elevation of -10 ft. NGVD, and side slopes of 1H:1V. The estimated average ground elevation in this reach is 5.6 ft. NGVD.
- Along the Holly Lakes Mobile Home Community (approximate length of 2,200 feet), the canal would be a "U" shaped concrete channel having a width of 75 feet and an invert elevation of -11 ft. NGVD. The estimated ground elevation in this reach is 7.0 ft. NGVD.
- Between the Holly Lakes Mobile Home Community and Structure S30 (approximate length of 18,700 feet), the canal would be a trapezoidal channel having a bottom width of 130 feet, an invert elevation of -11 ft. NGVD, and side slopes of 1H:1V. The estimated average ground elevation in this reach is 6.1 ft. NGVD.

The above segments are connected by transition zones having an apparent length of roughly 450 feet both north and south of the Holly Lakes Mobile Home Community. The overall length of the C-502B Canal between S-30 and the C-11 Canal is approximately 38,000 feet.

It would appear practicable to initially construct the entire 38,000-ft. length of the C-502B Canal as a trapezoidal channel having a bottom width of 40 feet; an invert elevation of -11 ft. NGVD; and side slopes of 1H:1V (simply assigned equal to those presented in the Feasibility Study). Assigning a water surface elevation at the upstream end of the canal (at its confluence with the C-11 Canal) of 7.0 ft. NGVD, the estimated water surface elevation at the downstream end of the canal (e.g., S-30 headwater) would vary from approximately 6.2 ft. NGVD immediately following construction (for Manning's "n" = 0.035) to 6.4 ft. NGVD following "aging" of the canal (Manning's "n" = 0.030). The above values of "n" are taken from the Feasibility Study.

The top width of the canal adjacent to the Holly Lakes Mobile Home Community would be 76 feet, closely approximating the eventual net top width of the "U" shaped concrete channel presented in the Feasibility Study. It would therefore appear practicable to delay construction of the "U" shaped concrete channel until future implementation of the NLBSA and CLBSA components of CERP.

6.4. Compatibility with Future CERP Components

This section provides additional suggestions and observations relative to the compatibility of the recommendations made herein with future CERP components, in particular the construction of Canals C-500A and C-500B and the eventual implementation of the NLBSA and CLBSA.

Certain of the earlier recommendations in this Part 6 contemplate a simple delay in construction of individual features until such time as they are needed for the future CERP components, and are not further discussed in this section.

6.4.1. Future Construction of C-500A and C-500B Canals

Upon implementation of the NLBSA and CLBSA components, including construction of the C-500A and C-500B canals (actually elements of the WCA-3A Decompartmentalization component of CERP) it will be necessary to maintain separation of the accumulated seepage in those canals (destined for delivery to the Everglades Protection Area) and runoff from the C-11 and C-9 basins. The addition of a new gated spillway in the C-11 Canal between Canal C-502B and Pump Station S-9 as recommended earlier in this Part 6 permits maintenance of that separation at all times other than when it becomes necessary to discharge basin runoff through S-9. The frequency and duration of that need are intended to be minimized as a central objective of the C-11 and C-9 Impoundments. It is suggested that the rather minor operational limitation imposed by the interruption of seepage delivery to the EPA when it is necessary to operate S-9 can be accepted in lieu of the substantial additional expense necessary to completely avoid such interruptions. The complete avoidance of such interruptions would in all probability require the construction of an inverted siphon to carry C-500 beneath the C-11 Canal.

6.4.2. Design of S-510 for Interim Conditions

The recommendation that S-510 (at the C-9 Impoundment) be initially designed and constructed for interim conditions is the only instance in which the recommendations herein would result in the construction of a feature not meeting eventual needs. That recommendation is made strictly in the interest of reducing the initial capital cost of the C-9 Impoundment. The District may wish to evaluate the suitability of that recommendation in light of any updated projections for the scheduled completion of future CERP components.

6.4.3. Staged Construction of Canal C-502B

It was recommended earlier in this Part 6 that Canal C-502B be initially constructed for an interim condition in which its required conveyance capacity is 1,000 cfs, with additional excavation necessary to meet the eventual design capacity of 2,500 cfs delayed until that capacity is needed. It is suggested that the initial excavation C-502B be made along the east bank of the eventual larger canal, and that the spoil resulting from that excavation be placed along the westerly toe of the spoil mound necessary for accommodating materials excavated from the eventual larger canal. While this will result in an increased cost for materials handling in the initial construction of C-502B, it will facilitate a cost-effective earthwork operation in it subsequent enlargement.

6.4.4. Design of Access Bridge B-501

Access Bridge B-501 will cross Canal C-502B at the northeast corner of the Holly Lakes Mobile Home Community, providing access to the Community from U.S. Highway 27. This bridge (and in particular its abutments) should be designed for compatibility with the eventual “U”-shaped concrete channel that will in the future replace the initially constructed C-502B along the Holly Lakes Mobile Home Community.

6.5. Summary of Recommendations

The following sections summarize recommended adjustments to the design and operation of the project. It is also recommended (see 6.5.3) that an additional SFWMM simulation be conducted. Additional detailed analyses will be necessary during the design phase to assure that flood protection is maintained upon implementation of these, and any other, recommended adjustments to the design and operation of the project.

6.5.1. Recommended Design Adjustments

The following adjustments to the design presented in the Feasibility Study are recommended for consideration by the South Florida Water Management District and the U.S. Army Corps of Engineers as they proceed with planning and the detailed design of the C-11 and C-9 Impoundments.

1. Confirm the suitability of the proposed 2,575 cfs capacity of S-503 through the conduct of more detailed subsurface investigations and seepage analyses during the final design of the project. Specifically, confirm that increases in total seepage to the east from the C-11 Impoundment and the WCA-3A Seepage Management Project, as compared to pre-project conditions, do not exceed 300 cfs;
2. Eliminate Siphon Structure S-502;
3. Eliminate Culvert S-502C;
4. Relocate Structure S-504 to the southwest corner of the C-11 Impoundment, discharging directly to the C-11 Canal;
5. Eliminate Culvert S-504A;
6. Add new seepage control facilities along the C-11 Canal east of U.S. Highway 27 and west of Structure S-381;
7. Add a new gated spillway, anticipated to be similar in design and capacity to Structure S-381, in the C-11 Canal between its confluence with Canal C-502B and Pump Station S-9;
8. Automate Structure S-13A so that it may be remotely monitored and operated;
9. Delay replacement of Structure S-30 until implementation of the North Lake Belt Storage Area (NLBSA) and/or Central Lake Belt Storage Area (CLBSA) components of CERP;
10. Automate existing Structure S-30 so that it may be remotely monitored and operated; add riprap or other erosion protection suitable for its anticipated interim operations;
11. Initially construct Canal C-502B for a conveyance capacity of 1,000 cfs between S-30 and the C-11 Canal; subsequent enlargement to its fully intended capacity

of 2,500 cfs would be delayed until implementation of the NLBSA and/or CLBSA components;

12. Delay enlargement of the C-9 Canal east of S-30 until implementation of the NLBSA and/or CLBSA components;

13. Initially construct Structure S-510 (C-9 Impoundment release structure) for an interim function of C-9 Impoundment drawdown (approximate capacity of 500 cfs at a head differential of 1.1 feet), delaying construction of the eventual 2-bay gated spillway recommended in the Feasibility Study until implementation of the NLBSA and/or CLBSA components.

In Part 3, it was noted that the capacity of Weir Structure S-505A (in the C-511 seepage canal near the southeast corner of the C-11 Impoundment) does not provide a factor of safety of five with respect to its design discharge; a factor of safety of 4.28 is reported. The District may wish to consider increasing the length of the weir as suggested in Part 3, although the increased head associated with the higher anticipated seepage flow is nominal.

6.5.2. Recommended Operations Adjustments

It is recommended that consideration be given to modifying the wet-season operations of Pump Station S-503 from that embedded in the 2010 WPA simulation. Specifically, it is recommended that the operations be modified to yield roughly a 0.3-foot average decrease in the operating range during the wet season. The purpose in this change would be to reduce the potential for increased runoff from the C-11 West Basin during storm events. Should a modified wet-season operational schedule not be implemented, it would be necessary to consider the potential need for additional pump capacity at S-503.

A significant operations adjustment recommended herein is to continue operation of Pump Station S-503 during those times when it may be necessary to discharge basin runoff to WCA-3A at S-9, in lieu of opening S-381. That operational change, coupled with the recommended addition of a gated spillway east of S-9, affords the opportunity for elimination of Siphon Structure S-502.

The following hierarchy for addressing C-11 West Basin runoff, at least during the wet season, would be consistent with the earlier recommendations in this Part 6. This operational sequence is structured upon the assumption that the runoff event is of adequate intensity and duration to require full utilization of all available storage, leading eventually to the required use of S-9, and is intended to minimize the required frequency and duration of pumping at S-9.

1. The first destination for C-11 West Basin runoff would be east through S-13A. Those discharges should be made up to the maximum available capacity in the C-11 East Canal and at S-13;

2. The next step would be to operate S-503 to introduce runoff to the C-11 Impoundment;

3. When the C-11 Impoundment reaches a stage of roughly 7.0 ft. NGVD, operate S-504, S-30 and S-509 to transfer inflows at S-503 to the C-9 Impoundment (maximum transfer rate of 1,000 cfs);

4. Continue the above operation until either or both the C-9 and C-11 Impoundments reach normal storage depth (approximately four feet above ground surface). At the C-9 Impoundment, once storage reaches a depth of approximately 2.5 feet, initiate releases to the C-9 wetlands mitigation area through S-513A (limit maximum tailwater elevation to 6.5 ft. NGVD). At the C-11 Impoundment, once storage reaches a depth of approximately 3.5 feet, initiate releases to the C-11 wetlands mitigation area through S-506 (limit maximum tailwater elevation to 8.5 ft. NGVD);

5. Once the C-9 Impoundment is at normal storage depth (stage of 8.5 ft. NGVD), close S-30 and S-504 if additional storage is available in the C-11 Impoundment; if additional storage is not available in the C-11 Impoundment, close S-30 only;

6. Once the C-11 Impoundment is at normal storage depth (stage of 10.0 ft. NGVD), reopen S-504, permitting up to approximately 2,000 acre-feet of releases from the C-11 Impoundment to exhaust available surface storage in the WCA-3A and WCA-3B Seepage Management Areas. Discontinue operation (if any) of Pump Station S-9A. There should be no use of S-9A during this operation; the use of S-9A would resume only following evacuation of this storage.

7. When stages in the Seepage Management Areas reach the desirable maximum, initiate operation of S-9.

At all times in the above sequence, every opportunity should be taken to minimize inflows at S-503 (through operation of S-13A). In addition, as any given wet season event recedes, drawdown of both reservoirs should commence at the earliest practicable time. The lone exception would be for those events (anticipated to be infrequent) in which it is necessary to store water in the Seepage Management Areas. Priority should be given to drawdown of those areas (through S-30 and S-381) so that the use of S-9A for seepage pumping can be resumed.

It should also be noted that the introduction of C-9 Basin runoff to the C-9 Impoundment can be expected to occupy storage volume, potentially reducing the extent to which discharges from the C-11 Impoundment may be transferred to the C-9 Impoundment. To the extent that the primary objective of the C-9 and C-11 Impoundments project is to minimize or eliminate the discharge of basin runoff to WCA-3A at S-9, the storage of C-9 Basin runoff during the wet season should not be considered a management objective of the project.

6.5.3. Additional SFWMM Simulation Recommended

It is recommended that the SFWMD conduct an additional SFWMM simulation in which the 2010 WPA simulation is modified in accordance with the information contained herein. The purpose of that simulation would be to quantify the amount and timing of remaining discharges to WCA-3A at S-9 so that additional options for addressing those discharges can be developed. In that simulation, it would be desirable to correct the modeled storage volumes available in the impoundments.

* * * * *

APPENDIX A

EVALUATION OF SFWMM GENERATED BOUNDARY CONDITIONS

Burns & McDonnell
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September 9, 2004

Mr. Jeffrey Needle, P.E.
Lead Project Manager
Office of Ecosystem Restoration
South Florida Water Management District
3301 Gun Club Road
West Palm Beach, FL 33406

South Florida Water Management District
Contract No. C-C20104P-WO03
Flood Protection Analysis for Broward County Water Preserve Areas
Task 1.0 – Compare SFWMM Generated Boundary Conditions
B&McD Project No. 36475

Dear Mr. Needle:

We have completed our analysis of SFWMM-generated boundary conditions as required under Task 1.0 of Work Order No. C-C20104P-WO03 (added to the original scope of work through the District's issuance of Revision 1 on June 14, 2004). The results of our analysis are summarized in this letter.

BACKGROUND

The Broward County Water Preserve Areas project is comprised of three components (C-11 Impoundment, C-9 Impoundment and WCA3A/3B Levee Seepage Management Projects) that were recommended as part of the Central and Southern Florida Comprehensive Review Study Feasibility Report and Integrated EIS Plan in April of 1999 (CERP). In a related and complementary effort, the Water Preserve Area Feasibility Study (WPA) was initiated. The WPA study region included multiple CERP components in the area east of the Water Conservation Areas in West Palm Beach, Broward and Miami-Dade Counties. The WPA consisted of an interconnected series of marshlands, impoundments, conveyance, and aquifer recharge areas.

CERP regulatory requirements stipulate the development of Flood Protection Assurances requiring a comparison of the flood protection as it existed in 'December 2000' against and the flood protection offered by 'Recommended CERP Alternative'. Task 1 "Groundwater Model Output Evaluation, Review and Analysis" of the Scope of Work of C-C20104P-WO03 required that the effect of CERP Projects on watertables be determined and that the resulting change in inflow into the C11W and C9 Basins during design runoff events be defined.

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It was intended that computed water levels and seepages of groundwater model simulations for 'December 2000' (aka pre-CERP Baseline) and 'Recommended CERP Alternative' be reviewed to compare conditions during wet periods, and to identify significant impacts resulting from implementation of the 'Recommended CERP Alternative. Any significant impact will have to be reduced by proper seepage management control and operation.

The WPA Feasibility Study used three subregional MODFLOW groundwater models, with each model covering a different geographic region within the overall planning area. Each model was run for a period of record from 1988-1995 (eight years), and was temporally discretized using constant stress period and time step lengths of one day. The 1988-1995 time period was selected for use in the subregional models as it encompasses relatively wet years (1994-1995), average years (1992-1993), and relatively dry years (1988-1989). The subregional model consists of 500 feet by 500 feet spatial cells and up to seven vertical layers. Boundary conditions, ET, rainfall, canal stages and other details for the models were derived from the South Florida Water Management Model (SFWMM) simulations for the same time period. The SFWMM simulation employed in the analyses conducted for the WPA Feasibility Study was for 1995 Base conditions.

PURPOSE AND OBJECTIVE OF TASK 1.0

As noted above, the subregional MODFLOW groundwater models employed in the WPA Feasibility Study were developed using boundary conditions taken from the SFWMM "1995 Base" condition analysis. Those models have not to date been updated to reflect potential changes to boundary conditions resulting from the SFWMM "2000 Base" simulation. The purpose of Task 1.0 is to complete a review and comparison of the output of the "1995 Base", "2000 Base", and "2010 Base" SFWMM simulations. The objective of that review and comparison is to identify and quantify significant changes (between those three simulations) in surface water profiles within the C-11W and C-9 Canals. The overall intent of this analysis is to assess the extent to which use of the boundary conditions resulting from the "1995 Base" SFWMM simulation (used in the available MODFLOW models) can be considered representative of boundary conditions associated with hydrologic conditions existing in December 2000 (the pre-CERP baseline condition for Flood Protection Assurance analyses prescribed by CERP regulatory requirements).

ANALYSIS

For this analysis, the boundary conditions subject to change between the three SFWMM simulations (e.g., "1995 Base", "2000 Base", and "2010 Base") were considered to be directly reflected in the simulated stages in the C-11W and C-9 canals. Accordingly, the analysis focused on the simulated stages in those canals reflected in the SFWMM simulations; the metric selected for evaluation is canal stage-duration data. The analyses were developed to permit two separate evaluations:

- The extent to which the 1988-1995 period directly considered in the subregional groundwater models can be considered representative of a longer period of "record" (e.g., the full 31-year period 1965-1995 available in the simulation output).
- The extent to which the boundary conditions resulting from the "1995 Base" simulation may be considered generally comparable to December 2000 pre-CERP baseline conditions (taken as those associated with the "2000 Base" simulation) for both:
  - All-seasons data (e.g., all available data, both wet and dry seasons);
  - Wet season only (e.g., data only for the period May 1 through October 31 in each calendar year).

Source of Data

Simulated mean daily canal stages at "C9" and "C11W" were taken from the following SFWMM output files (furnished to Burns & McDonnell on compact disk by the District):

- "1995 Base": out_95BS_V3.5;
- "2000 Base": 2000B2_WMM5.4_033104_out;
- "2010 Base": O2010B_REV_V3.5.

Each of the above files is dated May 17, 2004.

Methods

Simulated mean daily canal stages were copied from the SFWMM output files into a single Microsoft Excel workbook. Canal stage-duration data (e.g., identification of canal stages equalled or exceeded a given percentage of the time) were then developed for the following three "records" for each of the three simulations:

- All-season data for the period encompassing calendar years 1965-1995, inclusive.
- All-season data for the period encompassing calendar years 1988-1995, inclusive.
- Wet-season (May-October) data for water years 1988-1995.

A compact disk containing the Excel workbook developed in the analysis is being separately furnished to the District.

Results

The results of the stage-duration analyses are presented in Tables 1-6 attached to this letter. Tables 1 and 2 present "all-season" stage-duration data for the C-9 and C-11W canals, respectively, for the period 1965-1995. Tables 3 and 4 present "all-season" data for the C-9 and C-11W canals, respectively, for the period 1988-1995. Tables 4 and 5 present "wet-season" data for the C-9 and C-11W canals, respectively, for the period 1988-1995.

EVALUATION

Tables 7 and 8 present a comparison of stage duration data taken from the 1965-1995 simulation period (all available data) to that taken from the 1988-1995 simulation period (eight year period considered in subregional groundwater modeling) for the C9 and C11W Canals, respectively. With the exception of the maximum daily stage during the simulation periods (particularly in the C11W Canal), the data taken from the 8-year simulation closely approximates that taken from the 31-year simulation.

- In the C11W Canal, the maximum simulated daily stage over the 31-year simulation, 1995 Base was 5.95 ft. NGVD, occurring on April 25, 1979. The next highest daily stage in that simulation was 4.19 ft. NGVD. Excluding only that one day, the difference in maximum daily stages in the two simulation periods was 0.17 ft. A difference of but 0.01 ft. was computed for the canal stage exceeded 0.5% of the time.
- Also in the C11W Canal, the maximum simulated daily stage over the 31-year simulation, 2010 Base was 5.34 ft. NGVD, again occurring on April 25, 1979. The next highest daily stage in that simulation was 4.83 ft. NGVD. Excluding only that one day, the difference in maximum daily stages in the two simulation periods was 0.87 ft. No difference in maximum daily stages was computed for the canal stage exceeded 0.5% of the time.

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Tables 9 and 10 present a comparison of stage duration data taken from the "1995 Base" simulation to that taken from the "2000 Base" simulation period (eight year period considered in subregional groundwater modeling) for the C9 and C11W Canals, respectively for each of the three periods under consideration. Again with the exception of the maximum daily stage during the simulation periods, the data taken from the "1995 Base" simulation closely approximates that taken from the "2000 Base" simulation. Other than the maximum daily stage (see above discussion), the maximum difference in the computed stages for any given exceedance frequency is roughly 3 inches; a lesser difference is shown for the average daily stages.

CONCLUSIONS

On the basis of the analyses summarized in this letter and the attached tables, we conclude that:

1. No significant bias in boundary conditions (canal stages) was introduced to the subregional groundwater model analyses by using the period 1988-1995 for analysis in lieu of the full 1965-1995 period of simulated data.
2. Boundary conditions (canal stages) for the subregional groundwater models taken from the "1995 Base" simulation closely approximate those which would be taken from the "2000 Base" simulation, and may reasonably be substituted therefore.

Please feel free to contact me should you have any questions or desire additional information.

Sincerely,

Galen E. Miller, P.E.
Associate Vice President
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Table 1
Stage-Duration Data at C9 Canal
All Seasons, Calendar Years 1965-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	3.91	2.88	2.86
0.5%	2.60	2.70	2.31
1%	2.47	2.56	2.25
2%	2.38	2.41	2.20
5%	2.28	2.33	2.16
10%	2.21	2.29	2.13
20%	2.17	2.25	2.10
30%	2.14	2.22	2.08
40%	2.12	2.19	2.07
50%	2.10	2.16	2.06
60%	2.08	2.12	2.05
70%	2.05	2.06	2.04
80%	2.02	1.93	2.03
90%	1.80	1.88	2.01
95%	1.79	1.70	1.96
100%	1.10	1.16	1.39
Summary Data			
Max. Stage	3.91	2.88	2.86
Min. Stage	1.10	1.16	1.39
Ave. Stage	2.07	2.11	2.06
Median Stage	2.10	2.16	2.06

Table 2
Stage-Duration Data at C11W Canal
All Seasons, Calendar Years 1965-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	5.95	4.03	5.34
0.5%	3.86	3.84	3.78
1%	3.84	3.84	3.72
2%	3.83	3.84	3.66
5%	3.82	3.83	3.61
10%	3.82	3.83	3.56
20%	3.81	3.81	3.49
30%	3.81	3.80	3.46
40%	3.80	3.79	3.43
50%	3.80	3.78	3.39
60%	3.17	2.97	3.35
70%	3.13	2.94	3.33
80%	3.13	2.93	3.31
90%	3.12	2.91	3.23
95%	3.12	2.90	3.08
100%	2.93	2.88	2.52
Summary Data			
Max. Stage	5.95	4.03	5.34
Min. Stage	2.93	2.88	2.52
Ave. Stage	3.51	3.44	3.38
Median Stage	3.80	3.78	3.39

Table 3
Stage-Duration Data at C9 Canal
All Seasons, Calendar Years 1988-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	3.61	2.88	2.53
0.5%	2.54	2.77	2.33
1%	2.44	2.69	2.25
2%	2.36	2.57	2.21
5%	2.26	2.36	2.16
10%	2.21	2.31	2.13
20%	2.17	2.26	2.10
30%	2.14	2.23	2.08
40%	2.12	2.20	2.07
50%	2.09	2.16	2.06
60%	2.07	2.12	2.05
70%	2.05	2.06	2.04
80%	2.01	1.93	2.03
90%	1.82	1.88	2.01
95%	1.80	1.73	1.97
100%	1.52	1.45	1.81
Summary Data			
Max. Stage	3.61	2.88	2.53
Min. Stage	1.52	1.45	1.81
Ave. Stage	2.08	2.13	2.06
Median Stage	2.09	2.16	2.06

Table 4
Stage-Duration Data at C11W Canal
All Seasons, Calendar Years 1988-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	4.02	3.86	3.96
0.5%	3.85	3.84	3.78
1%	3.84	3.84	3.73
2%	3.83	3.83	3.66
5%	3.82	3.83	3.62
10%	3.82	3.83	3.57
20%	3.81	3.81	3.49
30%	3.81	3.80	3.46
40%	3.80	3.79	3.43
50%	3.52	3.78	3.38
60%	3.18	2.97	3.35
70%	3.14	2.94	3.33
80%	3.13	2.93	3.32
90%	3.12	2.91	3.26
95%	3.12	2.90	3.17
100%	3.11	2.88	2.99
Summary Data			
Max. Stage	4.02	3.86	3.96
Min. Stage	3.11	2.88	2.99
Ave. Stage	3.49	3.44	3.40
Median Stage	3.52	3.78	3.38

Table 5
Stage-Duration Data at C9 Canal
Wet Seasons (May-Oct), Calendar Years 1988-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	3.61	2.88	2.53
0.5%	2.77	2.77	2.36
1%	2.47	2.67	2.28
2%	2.42	2.51	2.24
5%	2.30	2.36	2.18
10%	2.25	2.31	2.15
20%	2.20	2.28	2.12
30%	2.17	2.25	2.10
40%	2.16	2.23	2.09
50%	2.14	2.21	2.08
60%	2.12	2.17	2.07
70%	2.11	2.12	2.06
80%	2.07	1.96	2.05
90%	2.02	1.74	2.03
95%	1.85	1.67	2.02
100%	1.68	1.45	1.92
Summary Data			
Max. Stage	3.61	2.88	2.53
Min. Stage	1.68	1.45	1.92
Ave. Stage	2.13	2.14	2.09
Median Stage	2.14	2.21	2.08

Table 6
Stage-Duration Data at C11W Canal
Wet Seasons (May-Oct), Calendar Years 1988-1995

Time Equalled or Exceeded	Mean Daily Canal Stage in ft. NGVD from Simulation		
	1995 Base	2000 Base	2010 Base
Max.	4.02	3.83	3.89
0.5%	3.89	3.83	3.78
1%	3.83	3.83	3.75
2%	3.82	3.83	3.68
5%	3.81	3.81	3.63
10%	3.80	3.80	3.59
20%	3.20	2.97	3.53
30%	3.14	2.95	3.49
40%	3.14	2.94	3.48
50%	3.13	2.94	3.46
60%	3.13	2.93	3.44
70%	3.13	2.92	3.42
80%	3.12	2.91	3.37
90%	3.12	2.90	3.32
95%	3.12	2.90	3.30
100%	3.11	2.88	3.07
Summary Data			
Max. Stage	4.02	3.83	3.89
Min. Stage	3.11	2.88	3.07
Ave. Stage	3.24	3.08	3.46
Median Stage	3.13	2.94	3.46

Table 7
Stage-Duration Data at C9 Canal
Comparison of 1965-1995 to 1988-1995 Simulations

Time Equalled or Exceeded	Difference in Stage (31 Yr. - 8 Yr.), ft.		
	1995 Base	2000 Base	2010 Base
Max.	0.30	0.00	0.33
0.5%	0.06	-0.07	-0.02
1%	0.03	-0.13	0.00
2%	0.02	-0.16	-0.01
5%	0.02	-0.03	0.00
10%	0.00	-0.02	0.00
20%	0.00	-0.01	0.00
30%	0.00	-0.01	0.00
40%	0.00	-0.01	0.00
50%	0.01	0.00	0.00
60%	0.01	0.00	0.00
70%	0.00	0.00	0.00
80%	0.01	0.00	0.00
90%	-0.02	0.00	0.00
95%	-0.01	-0.03	-0.01
100%	-0.42	-0.29	-0.42
Ave.Stage	-0.01	-0.01	0.00

Table 8
Stage-Duration Data at C11W Canal
Comparison of 1965-1995 to 1988-1995 Simulations

Time Equalled or Exceeded	Difference in Stage (31 Yr. - 8 Yr.), ft.		
	1995 Base	2000 Base	2010 Base
Max.	1.93	0.17	1.38
0.5%	0.01	0.00	0.00
1%	0.00	0.00	-0.01
2%	0.00	0.01	0.00
5%	0.00	0.00	-0.01
10%	0.00	0.00	-0.01
20%	0.00	0.00	0.00
30%	0.00	0.00	0.00
40%	0.00	0.00	0.00
50%	0.28	0.00	0.01
60%	-0.01	0.00	0.00
70%	-0.01	0.00	0.00
80%	0.00	0.00	-0.01
90%	0.00	0.00	-0.03
95%	0.00	0.00	-0.09
100%	-0.18	0.00	-0.47
Ave.Stage	0.02	0.00	-0.01

Table 9
Stage-Duration Data at C9 Canal
Comparison of 1995 Base and 2000 Base Simulations

Time Equalled or Exceeded	Difference in Stage (1995 Base - 2000 Base), ft.		
	1965-1995 All Seasons	1988-1995 All Seasons	1988-1995 Wet Seasons Only
Max.	1.03	0.73	0.73
0.5%	-0.10	-0.23	0.00
1%	-0.09	-0.25	-0.20
2%	-0.03	-0.21	-0.09
5%	-0.05	-0.10	-0.06
10%	-0.08	-0.10	-0.06
20%	-0.08	-0.09	-0.08
30%	-0.08	-0.09	-0.08
40%	-0.07	-0.08	-0.07
50%	-0.06	-0.07	-0.07
60%	-0.04	-0.05	-0.05
70%	-0.01	-0.01	-0.01
80%	0.09	0.08	0.11
90%	-0.08	-0.06	0.28
95%	0.09	0.07	0.18
100%	-0.06	0.07	0.23
Ave.Stage	-0.04	0.05	0.00

Table 10
Stage-Duration Data at C11W Canal
Comparison of 1995 Base and 2000 Base Simulations

Time Equalled or Exceeded	Difference in Stage (1995 Base - 2000 Base), ft.		
	1965-1995 All Seasons	1988-1995 All Seasons	1988-1995 Wet Seasons Only
Max.	1.92	0.16	0.19
0.5%	0.02	0.01	0.06
1%	0.00	0.00	0.00
2%	-0.01	0.00	-0.01
5%	-0.01	-0.01	0.00
10%	-0.01	-0.01	0.00
20%	0.00	0.00	0.23
30%	0.01	0.01	0.19
40%	0.01	0.01	0.20
50%	0.02	-0.26	0.19
60%	0.20	0.21	0.20
70%	0.19	0.20	0.21
80%	0.20	0.20	0.21
90%	0.21	0.21	0.22
95%	0.22	0.22	0.22
100%	0.05	0.23	0.23
Ave.Stage	0.07	0.05	0.17

APPENDIX B

DESIGN STORM EVENTS FOR HYDROLOGIC MODELING

List of Tables

List of Figures

Table B.1 Design Storm Events for Hydrologic Modeling

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
0	0	0	0	0	0	0	0	0
0.25	0.2	0.0011	0.0112	0.0132	0.0165	0.0113	0.0138	0.0176
0.5	0.3	0.0022	0.0224	0.0264	0.0330	0.0227	0.0275	0.0352
0.75	0.5	0.0034	0.0347	0.0408	0.0510	0.0350	0.0425	0.0544
1	0.6	0.0045	0.0459	0.0540	0.0675	0.0464	0.0563	0.0720
1.25	0.8	0.0056	0.0571	0.0672	0.0840	0.0577	0.0700	0.0896
1.5	0.9	0.0067	0.0683	0.0804	0.1005	0.0690	0.0838	0.1072
1.75	1.1	0.0078	0.0796	0.0936	0.1170	0.0803	0.0975	0.1248
2	1.2	0.0089	0.0908	0.1068	0.1335	0.0917	0.1113	0.1424
2.25	1.4	0.0101	0.1030	0.1212	0.1515	0.1040	0.1263	0.1616
2.5	1.5	0.0112	0.1142	0.1344	0.1680	0.1154	0.1400	0.1792
2.75	1.7	0.0123	0.1255	0.1476	0.1845	0.1267	0.1538	0.1968
3	1.8	0.0134	0.1367	0.1608	0.2010	0.1380	0.1675	0.2144
3.25	2	0.0145	0.1479	0.1740	0.2175	0.1494	0.1813	0.2320
3.5	2.1	0.0157	0.1601	0.1884	0.2355	0.1617	0.1963	0.2512
3.75	2.3	0.0168	0.1714	0.2016	0.2520	0.1730	0.2100	0.2688
4	2.4	0.0179	0.1826	0.2148	0.2685	0.1844	0.2238	0.2864
4.25	2.6	0.019	0.1938	0.2280	0.2850	0.1957	0.2375	0.3040
4.5	2.7	0.0201	0.2050	0.2412	0.3015	0.2070	0.2513	0.3216
4.75	2.9	0.0213	0.2173	0.2556	0.3195	0.2194	0.2663	0.3408
5	3	0.0224	0.2285	0.2688	0.3360	0.2307	0.2800	0.3584
5.25	3.2	0.0235	0.2397	0.2820	0.3525	0.2421	0.2938	0.3760
5.5	3.3	0.0246	0.2509	0.2952	0.3690	0.2534	0.3075	0.3936
5.75	3.5	0.0257	0.2621	0.3084	0.3855	0.2647	0.3213	0.4112
6	3.6	0.0268	0.2734	0.3216	0.4020	0.2760	0.3350	0.4288
6.25	3.8	0.028	0.2856	0.3360	0.4200	0.2884	0.3500	0.4480
6.5	4	0.0291	0.2968	0.3492	0.4365	0.2997	0.3638	0.4656
6.75	4.1	0.0302	0.3080	0.3624	0.4530	0.3111	0.3775	0.4832
7	4.3	0.0313	0.3193	0.3756	0.4695	0.3224	0.3913	0.5008
7.25	4.4	0.0324	0.3305	0.3888	0.4860	0.3337	0.4050	0.5184
7.5	4.6	0.0336	0.3427	0.4032	0.5040	0.3461	0.4200	0.5376
7.75	4.7	0.0347	0.3539	0.4164	0.5205	0.3574	0.4338	0.5552
8	4.9	0.0358	0.3652	0.4296	0.5370	0.3687	0.4475	0.5728
8.25	5	0.0369	0.3764	0.4428	0.5535	0.3801	0.4613	0.5904
8.5	5.2	0.038	0.3876	0.4560	0.5700	0.3914	0.4750	0.6080
8.75	5.3	0.0391	0.3988	0.4692	0.5865	0.4027	0.4888	0.6256
9	5.5	0.0403	0.4111	0.4836	0.6045	0.4151	0.5038	0.6448
9.25	5.6	0.0414	0.4223	0.4968	0.6210	0.4264	0.5175	0.6624
9.5	5.8	0.0425	0.4335	0.5100	0.6375	0.4378	0.5313	0.6800
9.75	5.9	0.0436	0.4447	0.5232	0.6540	0.4491	0.5450	0.6976
10	6.1	0.0447	0.4559	0.5364	0.6705	0.4604	0.5588	0.7152
10.25	6.2	0.0459	0.4682	0.5508	0.6885	0.4728	0.5738	0.7344
10.5	6.4	0.047	0.4794	0.5640	0.7050	0.4841	0.5875	0.7520
10.75	6.5	0.0481	0.4906	0.5772	0.7215	0.4954	0.6013	0.7696
11	6.7	0.0492	0.5018	0.5904	0.7380	0.5068	0.6150	0.7872
11.25	6.8	0.0503	0.5131	0.6036	0.7545	0.5181	0.6288	0.8048
11.5	7	0.0515	0.5253	0.6180	0.7725	0.5305	0.6438	0.8240
11.75	7.1	0.0526	0.5365	0.6312	0.7890	0.5418	0.6575	0.8416
12	7.3	0.0537	0.5477	0.6444	0.8055	0.5531	0.6713	0.8592
12.25	7.4	0.0548	0.5590	0.6576	0.8220	0.5644	0.6850	0.8768
12.5	7.6	0.0559	0.5702	0.6708	0.8385	0.5758	0.6988	0.8944
12.75	7.8	0.057	0.5814	0.6840	0.8550	0.5871	0.7125	0.9120
13	7.9	0.0582	0.5936	0.6984	0.8730	0.5995	0.7275	0.9312
13.25	8.1	0.0593	0.6049	0.7116	0.8895	0.6108	0.7413	0.9488
13.5	8.2	0.0604	0.6161	0.7248	0.9060	0.6221	0.7550	0.9664
13.75	8.4	0.0615	0.6273	0.7380	0.9225	0.6335	0.7688	0.9840

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
14	8.5	0.0626	0.6385	0.7512	0.9390	0.6448	0.7825	1.0016
14.25	8.7	0.0638	0.6508	0.7656	0.9570	0.6571	0.7975	1.0208
14.5	8.8	0.0649	0.6620	0.7788	0.9735	0.6685	0.8113	1.0384
14.75	9	0.066	0.6732	0.7920	0.9900	0.6798	0.8250	1.0560
15	9.1	0.0671	0.6844	0.8052	1.0065	0.6911	0.8388	1.0736
15.25	9.3	0.0682	0.6956	0.8184	1.0230	0.7025	0.8525	1.0912
15.5	9.4	0.0693	0.7069	0.8316	1.0395	0.7138	0.8663	1.1088
15.75	9.6	0.0705	0.7191	0.8460	1.0575	0.7262	0.8813	1.1280
16	9.7	0.0716	0.7303	0.8592	1.0740	0.7375	0.8950	1.1456
16.25	9.9	0.0727	0.7415	0.8724	1.0905	0.7488	0.9088	1.1632
16.5	10	0.0738	0.7528	0.8856	1.1070	0.7601	0.9225	1.1808
16.75	10.2	0.0749	0.7640	0.8988	1.1235	0.7715	0.9363	1.1984
17	10.3	0.0761	0.7762	0.9132	1.1415	0.7838	0.9513	1.2176
17.25	10.5	0.0772	0.7874	0.9264	1.1580	0.7952	0.9650	1.2352
17.5	10.6	0.0783	0.7987	0.9396	1.1745	0.8065	0.9788	1.2528
17.75	10.8	0.0794	0.8099	0.9528	1.1910	0.8178	0.9925	1.2704
18	10.9	0.0805	0.8211	0.9660	1.2075	0.8292	1.0063	1.2880
18.25	11.1	0.0816	0.8323	0.9792	1.2240	0.8405	1.0200	1.3056
18.5	11.2	0.0828	0.8446	0.9936	1.2420	0.8528	1.0350	1.3248
18.75	11.4	0.0839	0.8558	1.0068	1.2585	0.8642	1.0488	1.3424
19	11.6	0.085	0.8670	1.0200	1.2750	0.8755	1.0625	1.3600
19.25	11.7	0.0861	0.8782	1.0332	1.2915	0.8868	1.0763	1.3776
19.5	11.9	0.0872	0.8894	1.0464	1.3080	0.8982	1.0900	1.3952
19.75	12	0.0884	0.9017	1.0608	1.3260	0.9105	1.1050	1.4144
20	12.2	0.0895	0.9129	1.0740	1.3425	0.9219	1.1188	1.4320
20.25	12.3	0.0906	0.9241	1.0872	1.3590	0.9332	1.1325	1.4496
20.5	12.5	0.0917	0.9353	1.1004	1.3755	0.9445	1.1463	1.4672
20.75	12.6	0.0928	0.9466	1.1136	1.3920	0.9558	1.1600	1.4848
21	12.8	0.094	0.9588	1.1280	1.4100	0.9682	1.1750	1.5040
21.25	12.9	0.0951	0.9700	1.1412	1.4265	0.9795	1.1888	1.5216
21.5	13.1	0.0962	0.9812	1.1544	1.4430	0.9909	1.2025	1.5392
21.75	13.2	0.0973	0.9925	1.1676	1.4595	1.0022	1.2163	1.5568
22	13.4	0.0984	1.0037	1.1808	1.4760	1.0135	1.2300	1.5744
22.25	13.5	0.0995	1.0149	1.1940	1.4925	1.0249	1.2438	1.5920
22.5	13.7	0.1007	1.0271	1.2084	1.5105	1.0372	1.2588	1.6112
22.75	13.8	0.1018	1.0384	1.2216	1.5270	1.0485	1.2725	1.6288
23	14	0.1029	1.0496	1.2348	1.5435	1.0599	1.2863	1.6464
23.25	14.1	0.104	1.0608	1.2480	1.5600	1.0712	1.3000	1.6640
23.5	14.3	0.1051	1.0720	1.2612	1.5765	1.0825	1.3138	1.6816
23.75	14.4	0.1063	1.0843	1.2756	1.5945	1.0949	1.3288	1.7008
24	14.6	0.1074	1.0955	1.2888	1.6110	1.1062	1.3425	1.7184
24.25	14.8	0.1091	1.1128	1.3092	1.6365	1.1237	1.3638	1.7456
24.5	15	0.1107	1.1291	1.3284	1.6605	1.1402	1.3838	1.7712
24.75	15.3	0.1123	1.1455	1.3476	1.6845	1.1567	1.4038	1.7968
25	15.5	0.1139	1.1618	1.3668	1.7085	1.1732	1.4238	1.8224
25.25	15.7	0.1155	1.1781	1.3860	1.7325	1.1897	1.4438	1.8480
25.5	15.9	0.1171	1.1944	1.4052	1.7565	1.2061	1.4638	1.8736
25.75	16.1	0.1188	1.2118	1.4256	1.7820	1.2236	1.4850	1.9008
26	16.4	0.1204	1.2281	1.4448	1.8060	1.2401	1.5050	1.9264
26.25	16.6	0.122	1.2444	1.4640	1.8300	1.2566	1.5250	1.9520
26.5	16.8	0.1236	1.2607	1.4832	1.8540	1.2731	1.5450	1.9776
26.75	17	0.1252	1.2770	1.5024	1.8780	1.2896	1.5650	2.0032
27	17.2	0.1269	1.2944	1.5228	1.9035	1.3071	1.5863	2.0304
27.25	17.5	0.1285	1.3107	1.5420	1.9275	1.3236	1.6063	2.0560
27.5	17.7	0.1301	1.3270	1.5612	1.9515	1.3400	1.6263	2.0816
27.75	17.9	0.1317	1.3433	1.5804	1.9755	1.3565	1.6463	2.1072
28	18.1	0.1333	1.3597	1.5996	1.9995	1.3730	1.6663	2.1328

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
28.25	18.3	0.135	1.3770	1.6200	2.0250	1.3905	1.6875	2.1600
28.5	18.6	0.1366	1.3933	1.6392	2.0490	1.4070	1.7075	2.1856
28.75	18.8	0.1382	1.4096	1.6584	2.0730	1.4235	1.7275	2.2112
29	19	0.1398	1.4260	1.6776	2.0970	1.4399	1.7475	2.2368
29.25	19.2	0.1414	1.4423	1.6968	2.1210	1.4564	1.7675	2.2624
29.5	19.4	0.143	1.4586	1.7160	2.1450	1.4729	1.7875	2.2880
29.75	19.7	0.1447	1.4759	1.7364	2.1705	1.4904	1.8088	2.3152
30	19.9	0.1463	1.4923	1.7556	2.1945	1.5069	1.8288	2.3408
30.25	20.1	0.1479	1.5086	1.7748	2.2185	1.5234	1.8488	2.3664
30.5	20.3	0.1495	1.5249	1.7940	2.2425	1.5399	1.8688	2.3920
30.75	20.5	0.1511	1.5412	1.8132	2.2665	1.5563	1.8888	2.4176
31	20.8	0.1528	1.5586	1.8336	2.2920	1.5738	1.9100	2.4448
31.25	21	0.1544	1.5749	1.8528	2.3160	1.5903	1.9300	2.4704
31.5	21.2	0.156	1.5912	1.8720	2.3400	1.6068	1.9500	2.4960
31.75	21.4	0.1576	1.6075	1.8912	2.3640	1.6233	1.9700	2.5216
32	21.6	0.1592	1.6238	1.9104	2.3880	1.6398	1.9900	2.5472
32.25	21.9	0.1609	1.6412	1.9308	2.4135	1.6573	2.0113	2.5744
32.5	22.1	0.1625	1.6575	1.9500	2.4375	1.6738	2.0313	2.6000
32.75	22.3	0.1641	1.6738	1.9692	2.4615	1.6902	2.0513	2.6256
33	22.5	0.1657	1.6901	1.9884	2.4855	1.7067	2.0713	2.6512
33.25	22.7	0.1673	1.7065	2.0076	2.5095	1.7232	2.0913	2.6768
33.5	23	0.1689	1.7228	2.0268	2.5335	1.7397	2.1113	2.7024
33.75	23.2	0.1706	1.7401	2.0472	2.5590	1.7572	2.1325	2.7296
34	23.4	0.1722	1.7564	2.0664	2.5830	1.7737	2.1525	2.7552
34.25	23.6	0.1738	1.7728	2.0856	2.6070	1.7901	2.1725	2.7808
34.5	23.8	0.1754	1.7891	2.1048	2.6310	1.8066	2.1925	2.8064
34.75	24.1	0.177	1.8054	2.1240	2.6550	1.8231	2.2125	2.8320
35	24.3	0.1787	1.8227	2.1444	2.6805	1.8406	2.2338	2.8592
35.25	24.5	0.1803	1.8391	2.1636	2.7045	1.8571	2.2538	2.8848
35.5	24.7	0.1819	1.8554	2.1828	2.7285	1.8736	2.2738	2.9104
35.75	24.9	0.1835	1.8717	2.2020	2.7525	1.8901	2.2938	2.9360
36	25.2	0.1854	1.8911	2.2248	2.7810	1.9096	2.3175	2.9664
36.25	25.4	0.187	1.9074	2.2440	2.8050	1.9261	2.3375	2.9920
36.5	25.6	0.1887	1.9247	2.2644	2.8305	1.9436	2.3588	3.0192
36.75	25.9	0.1903	1.9411	2.2836	2.8545	1.9601	2.3788	3.0448
37	26.1	0.1919	1.9574	2.3028	2.8785	1.9766	2.3988	3.0704
37.25	26.3	0.1935	1.9737	2.3220	2.9025	1.9931	2.4188	3.0960
37.5	26.5	0.1951	1.9900	2.3412	2.9265	2.0095	2.4388	3.1216
37.75	26.7	0.1968	2.0074	2.3616	2.9520	2.0270	2.4600	3.1488
38	27	0.1984	2.0237	2.3808	2.9760	2.0435	2.4800	3.1744
38.25	27.2	0.2	2.0400	2.4000	3.0000	2.0600	2.5000	3.2000
38.5	27.4	0.2016	2.0563	2.4192	3.0240	2.0765	2.5200	3.2256
38.75	27.6	0.2032	2.0726	2.4384	3.0480	2.0930	2.5400	3.2512
39	27.8	0.2049	2.0900	2.4588	3.0735	2.1105	2.5613	3.2784
39.25	28.1	0.2065	2.1063	2.4780	3.0975	2.1270	2.5813	3.3040
39.5	28.3	0.2081	2.1226	2.4972	3.1215	2.1434	2.6013	3.3296
39.75	28.5	0.2097	2.1389	2.5164	3.1455	2.1599	2.6213	3.3552
40	28.7	0.2113	2.1553	2.5356	3.1695	2.1764	2.6413	3.3808
40.25	28.9	0.213	2.1726	2.5560	3.1950	2.1939	2.6625	3.4080
40.5	29.2	0.2146	2.1889	2.5752	3.2190	2.2104	2.6825	3.4336
40.75	29.4	0.2162	2.2052	2.5944	3.2430	2.2269	2.7025	3.4592
41	29.6	0.2178	2.2216	2.6136	3.2670	2.2433	2.7225	3.4848
41.25	29.8	0.2194	2.2379	2.6328	3.2910	2.2598	2.7425	3.5104
41.5	30	0.221	2.2542	2.6520	3.3150	2.2763	2.7625	3.5360
41.75	30.3	0.2227	2.2715	2.6724	3.3405	2.2938	2.7838	3.5632
42	30.5	0.2243	2.2879	2.6916	3.3645	2.3103	2.8038	3.5888
42.25	30.7	0.2259	2.3042	2.7108	3.3885	2.3268	2.8238	3.6144

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
42.5	30.9	0.2275	2.3205	2.7300	3.4125	2.3433	2.8438	3.6400
42.75	31.1	0.2291	2.3368	2.7492	3.4365	2.3597	2.8638	3.6656
43	31.4	0.2308	2.3542	2.7696	3.4620	2.3772	2.8850	3.6928
43.25	31.6	0.2324	2.3705	2.7888	3.4860	2.3937	2.9050	3.7184
43.5	31.8	0.234	2.3868	2.8080	3.5100	2.4102	2.9250	3.7440
43.75	32	0.2356	2.4031	2.8272	3.5340	2.4267	2.9450	3.7696
44	32.2	0.2372	2.4194	2.8464	3.5580	2.4432	2.9650	3.7952
44.25	32.5	0.2389	2.4368	2.8668	3.5835	2.4607	2.9863	3.8224
44.5	32.7	0.2405	2.4531	2.8860	3.6075	2.4772	3.0063	3.8480
44.75	32.9	0.2421	2.4694	2.9052	3.6315	2.4936	3.0263	3.8736
45	33.1	0.2437	2.4857	2.9244	3.6555	2.5101	3.0463	3.8992
45.25	33.3	0.2453	2.5021	2.9436	3.6795	2.5266	3.0663	3.9248
45.5	33.6	0.2469	2.5184	2.9628	3.7035	2.5431	3.0863	3.9504
45.75	33.8	0.2486	2.5357	2.9832	3.7290	2.5606	3.1075	3.9776
46	34	0.2502	2.5520	3.0024	3.7530	2.5771	3.1275	4.0032
46.25	34.2	0.2518	2.5684	3.0216	3.7770	2.5935	3.1475	4.0288
46.5	34.4	0.2534	2.5847	3.0408	3.8010	2.6100	3.1675	4.0544
46.75	34.7	0.255	2.6010	3.0600	3.8250	2.6265	3.1875	4.0800
47	34.9	0.2567	2.6183	3.0804	3.8505	2.6440	3.2088	4.1072
47.25	35.1	0.2583	2.6347	3.0996	3.8745	2.6605	3.2288	4.1328
47.5	35.3	0.2599	2.6510	3.1188	3.8985	2.6770	3.2488	4.1584
47.75	35.5	0.2615	2.6673	3.1380	3.9225	2.6935	3.2688	4.1840
48	35.9	0.2642	2.6948	3.1704	3.9630	2.7213	3.3025	4.2272
48.25	36.2	0.266	2.7132	3.1920	3.9900	2.7398	3.3250	4.2560
48.5	36.4	0.2678	2.7316	3.2136	4.0170	2.7583	3.3475	4.2848
48.75	36.7	0.2697	2.7509	3.2364	4.0455	2.7779	3.3713	4.3152
49	36.9	0.2715	2.7693	3.2580	4.0725	2.7965	3.3938	4.3440
49.25	37.2	0.2734	2.7887	3.2808	4.1010	2.8160	3.4175	4.3744
49.5	37.4	0.2752	2.8070	3.3024	4.1280	2.8346	3.4400	4.4032
49.75	37.7	0.277	2.8254	3.3240	4.1550	2.8531	3.4625	4.4320
50	37.9	0.2789	2.8448	3.3468	4.1835	2.8727	3.4863	4.4624
50.25	38.2	0.2811	2.8672	3.3732	4.2165	2.8953	3.5138	4.4976
50.5	38.5	0.2833	2.8897	3.3996	4.2495	2.9180	3.5413	4.5328
50.75	38.8	0.2855	2.9121	3.4260	4.2825	2.9407	3.5688	4.5680
51	39.1	0.2877	2.9345	3.4524	4.3155	2.9633	3.5963	4.6032
51.25	39.4	0.2901	2.9590	3.4812	4.3515	2.9880	3.6263	4.6416
51.5	39.8	0.2925	2.9835	3.5100	4.3875	3.0128	3.6563	4.6800
51.75	40.1	0.2949	3.0080	3.5388	4.4235	3.0375	3.6863	4.7184
52	40.4	0.2973	3.0325	3.5676	4.4595	3.0622	3.7163	4.7568
52.25	40.8	0.3004	3.0641	3.6048	4.5060	3.0941	3.7550	4.8064
52.5	41.3	0.3035	3.0957	3.6420	4.5525	3.1261	3.7938	4.8560
52.75	41.7	0.3067	3.1283	3.6804	4.6005	3.1590	3.8338	4.9072
53	42.1	0.3098	3.1600	3.7176	4.6470	3.1909	3.8725	4.9568
53.25	42.6	0.3136	3.1987	3.7632	4.7040	3.2301	3.9200	5.0176
53.5	43.2	0.3175	3.2385	3.8100	4.7625	3.2703	3.9688	5.0800
53.75	43.7	0.3214	3.2783	3.8568	4.8210	3.3104	4.0175	5.1424
54	44.2	0.3252	3.3170	3.9024	4.8780	3.3496	4.0650	5.2032
54.25	44.8	0.3298	3.3640	3.9576	4.9470	3.3969	4.1225	5.2768
54.5	45.5	0.3344	3.4109	4.0128	5.0160	3.4443	4.1800	5.3504
54.75	46.1	0.339	3.4578	4.0680	5.0850	3.4917	4.2375	5.4240
55	46.7	0.3436	3.5047	4.1232	5.1540	3.5391	4.2950	5.4976
55.25	47.4	0.349	3.5598	4.1880	5.2350	3.5947	4.3625	5.5840
55.5	48.2	0.3543	3.6139	4.2516	5.3145	3.6493	4.4288	5.6688
55.75	48.9	0.3596	3.6679	4.3152	5.3940	3.7039	4.4950	5.7536
56	49.6	0.365	3.7230	4.3800	5.4750	3.7595	4.5625	5.8400
56.25	50.5	0.3712	3.7862	4.4544	5.5680	3.8234	4.6400	5.9392
56.5	51.3	0.3775	3.8505	4.5300	5.6625	3.8883	4.7188	6.0400

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
56.75	52.2	0.3837	3.9137	4.6044	5.7555	3.9521	4.7963	6.1392
57	53	0.39	3.9780	4.6800	5.8500	4.0170	4.8750	6.2400
57.25	54	0.3974	4.0535	4.7688	5.9610	4.0932	4.9675	6.3584
57.5	55	0.4047	4.1279	4.8564	6.0705	4.1684	5.0588	6.4752
57.75	56.1	0.4128	4.2106	4.9536	6.1920	4.2518	5.1600	6.6048
58	57.2	0.4209	4.2932	5.0508	6.3135	4.3353	5.2613	6.7344
58.25	58.4	0.4297	4.3829	5.1564	6.4455	4.4259	5.3713	6.8752
58.5	59.6	0.4386	4.4737	5.2632	6.5790	4.5176	5.4825	7.0176
58.75	61.2	0.4503	4.5931	5.4036	6.7545	4.6381	5.6288	7.2048
59	62.8	0.4621	4.7134	5.5452	6.9315	4.7596	5.7763	7.3936
59.25	65.3	0.4805	4.9011	5.7660	7.2075	4.9492	6.0063	7.6880
59.5	67.8	0.4989	5.0888	5.9868	7.4835	5.1387	6.2363	7.9824
59.75	82.8	0.6093	6.2149	7.3116	9.1395	6.2758	7.6163	9.7488
60	101.5	0.7469	7.6184	8.9628	11.2035	7.6931	9.3363	11.9504
60.25	105.2	0.7737	7.8917	9.2844	11.6055	7.9691	9.6713	12.3792
60.5	108.8	0.8006	8.1661	9.6072	12.0090	8.2462	10.0075	12.8096
60.75	110.7	0.8146	8.3089	9.7752	12.2190	8.3904	10.1825	13.0336
61	112.6	0.8286	8.4517	9.9432	12.4290	8.5346	10.3575	13.2576
61.25	114	0.8389	8.5568	10.0668	12.5835	8.6407	10.4863	13.4224
61.5	115.4	0.8492	8.6618	10.1904	12.7380	8.7468	10.6150	13.5872
61.75	116.6	0.8576	8.7475	10.2912	12.8640	8.8333	10.7200	13.7216
62	117.7	0.8661	8.8342	10.3932	12.9915	8.9208	10.8263	13.8576
62.25	118.6	0.8723	8.8975	10.4676	13.0845	8.9847	10.9038	13.9568
62.5	119.4	0.8786	8.9617	10.5432	13.1790	9.0496	10.9825	14.0576
62.75	120.2	0.8841	9.0178	10.6092	13.2615	9.1062	11.0513	14.1456
63	120.9	0.8896	9.0739	10.6752	13.3440	9.1629	11.1200	14.2336
63.25	121.7	0.8951	9.1300	10.7412	13.4265	9.2195	11.1888	14.3216
63.5	122.4	0.9007	9.1871	10.8084	13.5105	9.2772	11.2588	14.4112
63.75	123.2	0.9062	9.2432	10.8744	13.5930	9.3339	11.3275	14.4992
64	123.9	0.9117	9.2993	10.9404	13.6755	9.3905	11.3963	14.5872
64.25	124.4	0.915	9.3330	10.9800	13.7250	9.4245	11.4375	14.6400
64.5	124.8	0.9183	9.3667	11.0196	13.7745	9.4585	11.4788	14.6928
64.75	125.3	0.9216	9.4003	11.0592	13.8240	9.4925	11.5200	14.7456
65	125.7	0.9249	9.4340	11.0988	13.8735	9.5265	11.5613	14.7984
65.25	126.2	0.9283	9.4687	11.1396	13.9245	9.5615	11.6038	14.8528
65.5	126.6	0.9316	9.5023	11.1792	13.9740	9.5955	11.6450	14.9056
65.75	127.1	0.9349	9.5360	11.2188	14.0235	9.6295	11.6863	14.9584
66	127.5	0.9382	9.5696	11.2584	14.0730	9.6635	11.7275	15.0112
66.25	128	0.9415	9.6033	11.2980	14.1225	9.6975	11.7688	15.0640
66.5	128.4	0.9448	9.6370	11.3376	14.1720	9.7314	11.8100	15.1168
66.75	128.9	0.9481	9.6706	11.3772	14.2215	9.7654	11.8513	15.1696
67	129.3	0.9514	9.7043	11.4168	14.2710	9.7994	11.8925	15.2224
67.25	129.8	0.9547	9.7379	11.4564	14.3205	9.8334	11.9338	15.2752
67.5	130.2	0.9581	9.7726	11.4972	14.3715	9.8684	11.9763	15.3296
67.75	130.7	0.9614	9.8063	11.5368	14.4210	9.9024	12.0175	15.3824
68	131.1	0.9647	9.8399	11.5764	14.4705	9.9364	12.0588	15.4352
68.25	131.4	0.9669	9.8624	11.6028	14.5035	9.9591	12.0863	15.4704
68.5	131.7	0.9691	9.8848	11.6292	14.5365	9.9817	12.1138	15.5056
68.75	132	0.9713	9.9073	11.6556	14.5695	10.0044	12.1413	15.5408
69	132.3	0.9735	9.9297	11.6820	14.6025	10.0271	12.1688	15.5760
69.25	132.6	0.9757	9.9521	11.7084	14.6355	10.0497	12.1963	15.6112
69.5	132.9	0.9779	9.9746	11.7348	14.6685	10.0724	12.2238	15.6464
69.75	133.2	0.9801	9.9970	11.7612	14.7015	10.0950	12.2513	15.6816
70	133.5	0.9823	10.0195	11.7876	14.7345	10.1177	12.2788	15.7168
70.25	133.8	0.9845	10.0419	11.8140	14.7675	10.1404	12.3063	15.7520
70.5	134.1	0.9868	10.0654	11.8416	14.8020	10.1640	12.3350	15.7888
70.75	134.4	0.989	10.0878	11.8680	14.8350	10.1867	12.3625	15.8240

Time (hr)	Cumulative percentage of Peak one day rainfall (%)	Ordinate for Unit Hydrograph (%)	C-11 West Basin			C-9 West Basin		
			Ordinate for 10-yr Storm (in) (10.2 in)	Ordinate for 25-yr Storm (in) (12 in)	Ordinate for 100-yr Storm (in) (15 in)	Ordinate for 10-yr Storm (in) (10.3 in)	Ordinate for 25-yr Storm (in) (12.5 in)	Ordinate for 100-yr Storm (in) (16 in)
71	134.7	0.9912	10.1102	11.8944	14.8680	10.2094	12.3900	15.8592
71.25	135	0.9934	10.1327	11.9208	14.9010	10.2320	12.4175	15.8944
71.5	135.3	0.9956	10.1551	11.9472	14.9340	10.2547	12.4450	15.9296
71.75	135.6	0.9978	10.1776	11.9736	14.9670	10.2773	12.4725	15.9648
72	135.9	1	10.2	12	15	10.3	12.5	16

Data obtained from the table included in Section 8.2(a) of *the Basis of Review for Environmental Resource Permit Application (09/2003).* The values in column "Cumulative Percentage of Peak One Day Rainfall" were interpolated in order to obtain the respective values for each 15 minute interval, and later were divided by 135.9 to get a unitary distribution for 72-hour period.

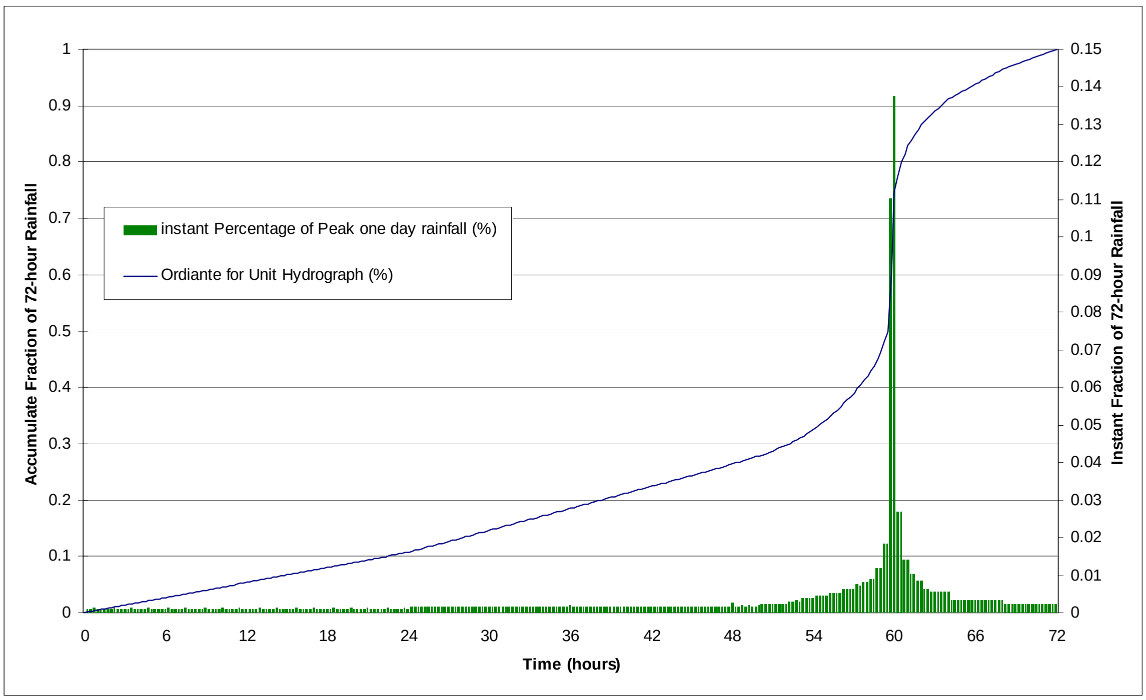

Figure B.1 SFWMD 72-hour Rainfall Distribution

APPENDIX C

TABLES AND FIGURES

List of Tables

List of Figures

Table C.1 Regional Hydrogeology Information

				FL St. 85 NAD				Model Layers (top elev)						Based on USGS (Fish, et al) reports for Broward and Dade Counties					
								1	2	3	4	5	6	Hydraulic Conductivity (ft/d)					
County	Well	Latitude	Longitude	Northing	Easting	Total Depth	Ground Surface Elev.	Peat, Sands, Cap Rock	Biscayne, shallow aquifer	Gray lm, generally Lower K	Tamiami aquifer	Predom, Sand, Lower K	Top Hawthorn	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
Broward	G-2311	26 03 35	80 26 37	627786.622	838821.990		10e	10	-14	-63	-127	-158	-177	10-100	22,500	0.1-10	10-100	0.1-10	≤0.1
	G-2312	26 13 47	80 27 37	689554.002	833093.949		14e	14	-2	-47	-92	-125	-201	0.1-10	300	10-100	100-1000	5-50	≤0.1
	G-2313	26 19 58	80 41 06	726769.069	759335.563		9e	9	6	-17	-32	-134	-197	0.1-10	1800	0.1-10	500	0.1-10	≤0.1
	G-2314	26 19 52	80 50 02	726072.462	710572.087		21e	21	6	-9	-21	-129	-185	10-100	10-100	0.1-10	100-1000	5-50	≤0.1
	G-2315	26 19 58	80 50 02	726678.251	710571.308		20e	20	4	-32	-96	-136	-220	5-50	10-100	0.1-10	100-1000	5-50	≤0.1
	G-2316	25 57 32	80 32 56	591003.939	804386.693		12e	12	-14	-62	-87	-161	-194	0.1-10	26000e	0.5-5	100-1000	0.1-10	≤0.1
	G-2317	25 57 22	80 24 55	590168.024	848292.185		6e	6	-11	-80	-104	-155	-197e	5-50	19,500	0.5-5	0.5-1	0.1-10	≤0.1
	G-2318	25 57 24	80 20 36	590482.043	871930.976		5e	5	-20	-101	-123	-155	-199	10-100	100-1000	5-50	10-100	5-50	≤0.1
	G-2319	26 08 43	80 28 39	658837.486	827571.732		10e	10	-14	-39	-102	-156	-200	10-100	23,000	5-50	590	0.1-10	0.5-5
	G-2320	26 08 46	80 35 42	659002.895	789024.699		11e	11	1	-46	-73	-155	-200	10-100	2600	5-50	910	5-50	≤0.1
	G-2321	26 07 42	80 22 00	652840.195	863961.206		8e	8	-12	-80	-135	-156	-273	10-100	24,000	0.1-10	10-100	0.1-10	.1-10
	G-2322	26 06 17	80 16 12	644424.223	895726.338		13	13	-25	-119	-154	-186	-210	10-100	13,000	10-100	5-50	0.5-5	≤0.1
	G-2323	26 19 38	80 12 15	725426.888	916835.002		26e	26	-76		-148	-178	-263	5-50	500-5000		10-100	5-50	≤0.1
	G-2325	26 19 38	80 07 52	725581.076	940764.792		14e	14	-63		-187	-209	-312	10-100	100-1000		10-100	5-50	≤0.1
	G-2327	25 58 29	80 14 48	597215.830	903656.517		6e	6	-45	-120	-134	-250	-267	36	50-500	0.1-10	10-100	5-50	≤0.1
	G-2328	25 59 18	80 09 18	602347.174	933740.748		9e	9	-20		-138	-279		10-100	500-≥1000		10-100	≤0.1	
	G-2329	26 10 14	80 51 22	667706.775	703358.581		15e	15		6	-68	-124	-295e	100-1000	0.1-100	10	5-50	≤0.1	
	G-2330	26 08 44	80 41 59	658707.796	754671.386		13e	13	-2	-41	-56	-154	-189	10-100	41,800	0.1-10	930	5-50	
	G-2338	26 05 32	80 50 36	639240.354	707583.662		16e	16	6	-34	-85	-142	-152	5-50	680	≤0.1	890	0.1-10	≤0.1
	G-2340	26 14 58	80 49 47	696390.759	711975.846		13e	13	12	-6	-47	-128	-172	0.1-10	10-100	0.1-10	5-50	≤0.1	≤0.1
	G-2341	26 13 43	80 17 58	689402.358	885820.614		12e	12	-8	-91	-104	-111	-160	5-50	10-100	10-100	100-1000	0.1-10	
	G-2342	26 13 48	80 12 20	690084.684	916597.023		14e	14	-66	-136	-143	-204	-285	10-100	500-5000	0.-10	75	10-100	≤0.1
	G-2344	26 14 23	80 07 15	693797.949	944347.490		17e	17	-34	-195	-244	-317	-370	5-50	100-1000	10-100	100-1000	10-100	0.1-10
	G-2345	26 06 41	80 12 35	646962.923	915493.366		11e	11	-50	-124	-135	-188	-306	10-100	≥1000	10-100	50-500	0.1-10	≤0.1
	G-2346	25 59 58	80 52 22	605508.960	697953.051		12e	12	5	-8	-48	-120	-153	0.1-10	10-100	≤0.1	100-1000	5-50	≤0.1
	G-2347	26 05 07	80 08 56	637597.779	935518.497		5e	5	-57		-215		-332	10-100	≥1000		100-1000		0.1-10
Dade	G-3294	25 57 07	80 25 48	588632.253	843461.313		8e	8	-1	-78	-133	-166	-200	10-100	15000e	10-100	100-1000	5-50	≤0.1
	G-3295	25 52 49	80 50 44	562207.069	706945.324	220	12e	12		4	-48	-127	-215	0.5-5		10-100	100-1000	5-50	≤0.1
	G-3296	25 52 24	80 38 05	559820.313	776271.109		12e	12	0	-33	-75	-168	-181	0.1-10	40000e	10-100	100-1000	0.1-10	≤0.1
	G-3297	25 50 58	80 29 03	551303.820	825809.043		10e	10	-3	-51	-112	-130	-161	0.1-10	29000e	470e	100-1000	10-100	0.1-10
	G-3298	25 50 20	80 23 10	547605.912	858075.254		8e	8	4	-88	-137	-157	-173	10-100	≥1000	5-50	100-1000	0.5-5	≤0.1
	G-3299	25 50 22	80 16 30	547993.951	894619.805		5e	5	3		-110	-169	-226	10-100	500-≥1000		100-1000	10-100	0.1-10
	G-3300	25 51 48	80 11 07	556849.707	924076.857		10e	10	-10			-187	-211	10-100	100-1000			10-100	0.1-10
	G-3301	25 45 37	80 49 36	518601.113	713213.141		12e	12	6	-5	-67	-139	-200	0.1-10	100-1000	530e	780e	5-50	≤0.1
	G-3302	25 45 42	80 42 17	519177.229	753345.845	210	9e	9	-2	-9	-68	-130	-192	0.1-10	100-1000	450e	420e	5-50	≤0.1
	G-3303	25 45 45	80 36 17	519566.305	786256.327		10e	10	-5	-26	-110	-151	-172	10-100	40000e	94e	430e	0.1-10	≤0.1
	G-3304	25 45 39	80 30 06	519075.513	820175.657		10e	10	3	-37	-110	-136	-182	5-50	≥1000	100-1000	240e	10-100	≤0.1
	G-3305	25 45 36	80 23 03	518936.055	858848.933	200	6e	6	4	-75	-100	-128	-170	0.1-10	0.1-10	10-100	100-1000	33e	≤0.1
	G-3306	25 46 00	80 17 37	521508.595	888640.360	230	10e	10	6			-111	-250	10-100	500-≥1000			10-100	≤0.1
	G-3307	25 45 38	80 14 00	519398.290	908491.503		12e	12	8			-131	-235	500-500	100-≥1000			0.1-10	0.1-10
	G-3308	25 39 27	80 45 59	481277.879	733117.548		4e	4	-1	-14	-104	-149	-197	0.1-10	10-100	0.1-10	100-1000	5-50	0.1-10
	G-3309	25 39 54	80 40 25	484068.364	763671.756	210	8e	8	2	-12	-100	-135	-199	10-100	≥1000	0.1-10	100-1000	5-50	≤0.1
	G-3310	25 37 14	80 34 59	467999.109	793549.865	250	9e	9	7	-37	-149	-176	-159	50-500	29000e	0.1-10	100-1000	10-100	≤0.1
	G-3311	25 37 46	80 29 50	471327.895	821820.107	240	13e	13	9	-40	-124	-156	-180	50-500	≥1000	10-100	210e	0.1-10	≤0.1
	G-3312	25 38 42	80 22 58	477140.993	859501.339	167	13e	13	9	-96		-112		50-500	3300e	10-100		0.1-10	≤0.1
	G-3313	25 38 31	80 18 02	476165.145	886594.891	213	16e	16	14			-118	-145	0.1-10	8700e			10-1000	≤0.1
	G-3314	25 30 18	80 33 35	426026.410	801377.549	260	8e	8	5	-52	-176	-208	-238	0.1-10	37000e	0.1-10	100-1000	10-100	0.1-10
	G-3315	25 31 19	80 17 48	432557.697	888107.297	190	18e	18	15	-72		-160	-185	0.1-10	27000e	3.7e		10-100	≤0.1
	G-3316	25 30 10	80 22 50	425454.074	860474.682	180	13e	13	5			-83	-143	50-500	≥1000			0.1-10	0.1-10
	G-3317	25 23 26	80 47 57	384243.679	722467.134	210	6e	6	3	-23	-81	-148	-173	10-100	36000e	10-100	100-1000	10-100	0.1-10
	G-3318	25 22 56	80 36 35	381353.426	785017.135		5e	5	2	-35	-128	-163	-209	0.1-10	≥1000	5-50	100-1000	5-50	≤0.1

Table C.1 Regional Hydrogeology Information (continue)

County	Well	Latitude	Longitude	FL St. 85 NAD Northing	FL St. 85 NAD Easting	Total Depth	Ground Surface Elev.	Model Layers (top elev) 1 Peat, Sands, Cap Rock	2 Biscayne, shallow aquifer	3 Gray lm, generally Lower K	4 Tamiami aquifer	5 Predom, Sand, Lower K	6 Top Hawthorn	Based on USGS (Fish, et al) reports for Broward and Dade Counties – Hydraulic Conductivity (ft/d) Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
Dade	G-3319	25 25 07	80 34 27	394614.152	796713.796	240	14e	14	3	-42	-162	-190	-229	10-100	55000e	0.1-10	100-1000	10-100	0.1-10
	G-3320	25 25 55	80 28 10	399583.802	831258.872	86	7e	7	3	-80			-136	50-500	21000e	0.1-10			≤0.1
	G-3321	25 25 06	80 21 28	394798.936	868135.442	200	6e	6	0	-98			-169	0.1-10	≥1000			10-100	0.1-10
	G-3322	26 15 12	80 47 53	697819.162	722352.536		10e	10	0	-15	-80	-151	-200	10-100	≥1000	10-100	100-1000	10-100	0.1-10
	G-3323	25 19 02	80 31 24	357822.896	813623.010	212	9e	9	1	-62	-138	-152	-172	10-100	≥1000	0.1-100	100-1000	10-100	0.1-10
	G-3324	26 19 48	80 27 18	726010.182	834670.588		8e	8	1	-60		-160	-172	0.1-10	24000e	0.1-10		0.5-5	0.1-10
	G-3344	25 23 20	80 27 54	383941.709	832788.339	58	3e	3	0	-70				0.1-10	≥1000	5-50			
	G-3394	25 29 44	80 39 51	422493.510	766938.419	210	7e	7	4	-28	-105	-146	-177	0.1-10	≥1000	5-50	400e	5-50	≤0.1
	G-3395	25 14 10	80 26 04	328458.818	843110.307	220	0e	0	-6	-88		-150	-226e	0.1-10	≥1000	0.1-10		0.1-10	

Table C.2 Seepage Model Results of C-11 Impoundment – Section 1 for Scenario A

Section1 of L-511: Southern Levee of C-11 Impoundment (Length: 5,000 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-11 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-11 Canal (cfs/ft)	% of Total Flow into C-11 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_10	10	4	4	3632.870	0.04205	0.04098	97.46	0.00107	2.54
xsect1_R_11	11	4	4	4238.592	0.04906	0.04781	97.46	0.00125	2.54
xsect1_R_12	12	4	4	4844.357	0.05607	0.05464	97.46	0.00143	2.54
xsect1_R_13	13	4	4	5450.169	0.06308	0.06166	97.74	0.00143	2.26

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	7.0 to 5.5	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.3 Seepage Model Results of C-11 Impoundment – Section 2 for Scenario A

Section 2 of L-511: Eastern Levee of C-11 Impoundment (Length: 11,745ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-511 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect2_R_10	10	4	3.5	2291.673	0.02652	0.00280	10.55	0.02373	89.45
xsect2_R_11	11	4	3.5	2648.116	0.03065	0.00335	10.94	0.02730	89.06
xsect2_R_12	12	4	3.5	3007.465	0.03481	0.00389	11.18	0.03092	88.82
xsect2_R_13	13	4	3.5	3368.700	0.03899	0.00442	11.34	0.03457	88.66

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	5.5	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.4 Seepage Model Results of C-11 Impoundment – Section 3A for Scenario A

Section 3A of L-511M: Eastern Levee of C-11 Impoundment Mitigation Area (Length: 2,965 ft)

4000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect3A_R_10	10	8.5	5	4	1897.570	0.02196	0.00072	3.30	0.02124	96.70
xsect3A_R_11	11	8.5	5	4	2133.465	0.02469	0.00088	3.55	0.02382	96.45
xsect3A_R_12	12	8.5	5	4	2370.020	0.02743	0.00103	3.74	0.02640	96.26
xsect3A_R_13	13	8.5	5	4	2608.906	0.03020	0.00117	3.87	0.02903	96.13

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.5 Seepage Model Results of C-11 Impoundment – Section 3B for Scenario A

Section 3B of L-511M: East Part of Northern Levee of C-11 Impoundment Mitigation Area (Length: 3,740 ft)

5000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect3B_R_10	10	8.5	5	4.5	1752.449	0.02028	0.00067	3.30	0.01961	96.70
xsect3B_R_11	11	8.5	5	4.5	1985.051	0.02298	0.00077	3.35	0.02220	96.65
xsect3B_R_12	12	8.5	5	4.5	2217.701	0.02567	0.00087	3.40	0.02480	96.60
xsect3B_R_13	13	8.5	5	4.5	2452.924	0.02839	0.00097	3.41	0.02742	96.59

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.6 Seepage Model Results of C-11 Impoundment – Section 3C for Scenario A

Section 3C of L-511M: West Part of Northern Levee of C-11 Impoundment Mitigation Area (Length:2,575 ft)

5000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect3C_R_10	10	8.5	5	4.5	1604.024	0.01857	0.00060	3.25	0.01796	96.75
xsect3C_R_11	11	8.5	5	4.5	1737.430	0.02011	0.00066	3.29	0.01945	96.71
xsect3C_R_12	12	8.5	5	4.5	1870.843	0.02165	0.00072	3.33	0.02093	96.67
xsect3C_R_13	13	8.5	5	4.5	2004.264	0.02320	0.00078	3.36	0.02242	96.64

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.7 Seepage Model Results of C-11 Impoundment – Section 4 for Scenario A and Scenario B

Section 4 of L-511: Western Levee of C-11 Impoundment (Length: 8,290 ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-511(ft)	WSE in C-502A (ft)	WSE in SMA-3A (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into C-502A (cfs/ft)	% of Total Flow into C-502A	Flowrate into the area between C-511 and C-502A (cfs/ft)	% of Total Flow into the area between C-511 and C-502A	Flowrate into SMA-3A (cfs/ft)	% of Total Flow into SMA-3A	Flowrate into West End of Model (cfs/ft)	% of Total Flow into West End of Model
xsect4_R_10	10	5	7	7.5	934.785	0.01082	0.00134	12.35	0.00177	16.39	0.00042	3.84	0.00042	3.84	0.00688	63.58
xsect4_R_11	11	5	7	7.5	1287.820	0.01491	0.00146	9.77	0.00216	14.48	0.00052	3.47	0.00062	4.18	0.01015	68.09
xsect4_R_12	12	5	7	7.5	1640.966	0.01899	0.00158	8.29	0.00254	13.39	0.00063	3.29	0.00083	4.37	0.01342	70.65
xsect4_R_13	13	5	7	7.5	1994.171	0.02308	0.00169	7.33	0.00293	12.68	0.00074	3.19	0.00104	4.50	0.01669	72.30

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	12 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.8 Seepage Model Results of C-11 Impoundment – Section 5 for Scenario A and Scenario B

Section 5 of L-511M: Western Levee of Mitigation Area (Length: 2,000 ft)

3500ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Mitigation Area (ft)	WSE in C-511 (ft)	WSE in C-502A (ft)	WSE in SMA-3A (ft)	Total FlowRate from Mitigation Area (cfd/ft)	Total FlowRate from Mitigation Area (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into C-502A (cfs/ft)	% of Total Flow into C-502A	Flowrate into the area between C-511 and C-502A (cfs/ft)	% of Total Flow into the area between C-511 and C-502A	Flowrate into SMA-3A (cfs/ft)	% of Total Flow into SMA-3A	Flowrate into West End of Model (cfs/ft)	% of Total Flow into West End of Model
xsect5_R_8	8	5	7	7.5	233.616	0.00270	0.00110	40.74	0.00099	36.65	0.00024	8.90	0.00002	0.76	0.00035	12.96
xsect5_R_85	8.5	5	7	7.5	409.710	0.00474	0.00118	24.80	0.00120	25.26	0.00029	6.12	0.00012	2.62	0.00195	41.20

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	12 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.9 Seepage Model Results of C-11 Impoundment – Section 6 for Scenario A and Scenario B

Section 6 of L-511: S-504 Discharge Pool Levee of C-11 Impoundment (Length: 1,075 ft)

2500ft in the impoundment and 1750ft outside of the impoundment

FileName	WSE in the Impoundment (ft)	WSE in S-504 Discharge Pool (ft)	WSE in C-502A (ft)	WSE in SMA-3A (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into S-504 Dicharge Pool (cfs/ft)	% of Total Flow into S-504 Dicharge Pool	Flowrate into C-502A (cfs/ft)	% of Total Flow into C-502A	Flowrate into the area between S-504 and C-502A (cfs/ft)	% of Total Flow into the area between S-504 and C-502A	Flowrate into SMA-3A (cfs/ft)	% of Total Flow into SMA-3A	Flowrate into West End of Model (cfs/ft)	% of Total Flow into West End of Model
xsect6_R_10	10	7.5	7	7.5	836.397	0.00968	0.00186	19.20	0.00160	16.50	0.00063	6.50	0.00032	3.33	0.00527	54.47
xsect6_R_11	11	7.5	7	7.5	1163.033	0.01346	0.00267	19.82	0.00189	14.04	0.00072	5.33	0.00048	3.56	0.00771	57.25
xsect6_R_12	12	7.5	7	7.5	1489.707	0.01724	0.00348	20.20	0.00218	12.65	0.00080	4.64	0.00064	3.69	0.01014	58.82
xsect6_R_13	13	7.5	7	7.5	1816.421	0.02102	0.00430	20.45	0.00247	11.76	0.00088	4.20	0.00080	3.78	0.01257	59.81

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	12 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.10 Seepage Model Results of C-11 Impoundment – Section 7 for Scenario A and Scenario B

Section 7 of L-511: Southern Levee of C-11 Impoundment (Length: 1,390 ft)

2500ft in the impoundment and 1750ft outside of the impoundment

FileName	WSE in the Impoundment (ft)	WSE in C-511 (ft)	WSE in C-502A (ft)	WSE in SMA-3A (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into C-502A (cfs/ft)	% of Total Flow into C-502A	Flowrate into the area between C-511 and C-502A (cfs/ft)	% of Total Flow into the area between C-511 and C-502A	Flowrate into SMA-3A (cfs/ft)	% of Total Flow into SMA-3A	Flowrate into West End of Model (cfs/ft)	% of Total Flow into West End of Model
xsect6_R_10	10	4	7	7.5	877.081	0.01015	0.00171	16.81	0.00114	11.21	0.00335	33.01	0.00002	0.19	0.00394	38.78
xsect6_R_11	11	4	7	7.5	1187.718	0.01375	0.00182	13.25	0.00128	9.29	0.00386	28.10	0.00004	0.26	0.00675	49.10
xsect6_R_12	12	4	7	7.5	1498.383	0.01734	0.00192	11.09	0.00142	8.16	0.00439	25.30	0.00005	0.31	0.00956	55.14
xsect6_R_13	13	4	7	7.5	1809.143	0.02094	0.00204	9.75	0.00155	7.41	0.00490	23.40	0.00007	0.34	0.01237	59.10

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	12 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.11 Seepage Model Results of C-11 Impoundment – Section 1 for Scenario B

Section1 of L-511: Southern Levee of C-11 Impoundment (Length: 5,000 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-11 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-11 Canal (cfs/ft)	% of Total Flow into C-11 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_10	10	4	6	3602.089	0.04169	0.04169	100.00	0.00000	0.00
xsect1_R_11	11	4	6	4207.810	0.04870	0.04870	100.00	0.00000	0.00
xsect1_R_12	12	4	6	4813.575	0.05571	0.05571	100.00	0.00000	0.00
xsect1_R_13	13	4	6	5419.386	0.06272	0.06272	100.00	0.00000	0.00

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	7.0 to 5.5	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.12 Seepage Model Results of C-11 Impoundment – Section 2 for Scenario B

Section 2 of L-511: Eastern Levee of C-11 Impoundment (Length: 11,745ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-511 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect2_R_10	10	4	5.5	1707.763	0.01977	0.00471	23.81	0.01506	76.19
xsect2_R_11	11	4	5.5	2072.703	0.02399	0.00522	21.75	0.01877	78.25
xsect2_R_12	12	4	5.5	2437.974	0.02822	0.00572	20.25	0.02250	79.75
xsect2_R_13	13	4	5.5	2803.276	0.03245	0.00621	19.15	0.02623	80.85

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	5.5	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.13 Seepage Model Results of C-11 Impoundment – Section 3A for Scenario B

Section 3A of L-511M: Eastern Levee of C-11 Impoundment Mitigation Area (Length: 2,965 ft)

4000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect3A_R_10	10	8.5	5	6	1282.138	0.01484	0.00144	9.71	0.01340	90.29
xsect3A_R_11	11	8.5	5	6	1525.505	0.01766	0.00157	8.90	0.01608	91.10
xsect3A_R_12	12	8.5	5	6	1768.906	0.02047	0.00170	8.32	0.01877	91.68
xsect3A_R_13	13	8.5	5	6	2012.409	0.02329	0.00183	7.87	0.02146	92.13

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.14 Seepage Model Results of C-11 Impoundment – Section 3B for Scenario B

Section 3B of L-511M: East Part of Northern Levee of C-11 Impoundment Mitigation Area (Length: 3,740 ft)

5000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into North End of Model (cfs/ft)	% of Total Flow into Nroth End of Model
xsect3B_R_10	10	8.5	5	6.5	1113.431	0.01289	0.00113	8.81	0.01175	91.19
xsect3B_R_11	11	8.5	5	6.5	1354.422	0.01568	0.00122	7.80	0.01445	92.20
xsect3B_R_12	12	8.5	5	6.5	1595.609	0.01847	0.00130	7.03	0.01717	92.97
xsect3B_R_13	13	8.5	5	6.5	1836.849	0.02126	0.00138	6.49	0.01988	93.51

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.15 Seepage Model Results of C-11 Impoundment – Section 3C for Scenario B

Section 3C of L-511M: West Part of Northern Levee of C-11 Impoundment Mitigation Area (Length:2,575 ft)

5000ft in the mitigation area and 1500ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-511 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-511 (cfs/ft)	% of Total Flow into C-511	Flowrate into North End of Model (cfs/ft)	% of Total Flow into Nroth End of Model
xsect3C_R_10	10	8.5	5	6.5	959.527	0.01111	0.00108	9.70	0.01003	90.30
xsect3C_R_11	11	8.5	5	6.5	1097.707	0.01270	0.00113	8.90	0.01157	91.10
xsect3C_R_12	12	8.5	5	6.5	1235.873	0.01430	0.00118	8.24	0.01312	91.76
xsect3C_R_13	13	8.5	5	6.5	1374.168	0.01590	0.00123	7.71	0.01468	92.29

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2311 and G-2321)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	6.5 to 6.0	-13	50	5
Layer 2	-13	-70	23000	2300

Table C.16 Seepage Model Results of C-9 Impoundment – Section 1A for Scenario A

Section1A of L-509: Southern Levee of C-9 Impoundment at Improved C-9 Canal(Length: 5,065 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-9 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-9 Canal (cfs/ft)	% of Total Flow into C-9 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_85	8.5	4	2	2968.231	0.03435	0.01952	56.81	0.01484	43.19
xsect1_R_95	9.5	4	2	3620.172	0.04190	0.02674	63.81	0.01516	36.19
xsect1_R_105	10.5	4	2	4272.158	0.04945	0.03408	68.92	0.01537	31.08
xsect1_R_115	11.5	4	2	4924.193	0.05699	0.04142	72.67	0.01557	27.33

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.17 Seepage Model Results of C-9 Impoundment – Section 1B for Scenario A

Section1B of L-509: Southern Levee of C-9 Impoundment at Existing C-9 Canal (Length: 1,840 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-9 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-9 Canal (cfs/ft)	% of Total Flow into C-9 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_85	8.5	4	2	2470.617	0.02860	0.01322	46.23	0.01538	53.77
xsect1_R_95	9.5	4	2	3003.274	0.03476	0.01894	54.48	0.01582	45.52
xsect1_R_105	10.5	4	2	3536.022	0.04093	0.02465	60.22	0.01628	39.78
xsect1_R_115	11.5	4	2	4068.856	0.04709	0.03035	64.45	0.01674	35.55

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.18 Seepage Model Results of C-9 Impoundment – Section 2 for Scenario A

Section 2 of L-509: Eastern Levee of C-9 Impoundment (Length: 10,445 ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-509 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect2_R_85	8.5	3	2	2206.462	0.02554	0.00127	4.97	0.02427	95.03
xsect2_R_95	9.5	3	2	2557.250	0.02960	0.00157	5.31	0.02803	94.69
xsect2_R_105	10.5	3	2	2911.391	0.03370	0.00186	5.51	0.03184	94.49
xsect2_R_115	11.5	3	2	3267.380	0.03782	0.00213	5.63	0.03569	94.37

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.19 Seepage Model Results of C-9 Impoundment – Section 3 for Scenario A

Section 3 of L-509: Northern Levee of C-9 Impoundment (Length: 4,935 ft)

5000ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-509 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect3_R_85	8.5	3	2.5	2093.778	0.02423	0.00157	6.47	0.02267	93.53
xsect3_R_95	9.5	3	2.5	2450.474	0.02836	0.00187	6.59	0.02649	93.41
xsect3_R_105	10.5	3	2.5	2811.752	0.03254	0.00216	6.63	0.03039	93.37
xsect3_R_115	11.5	3	2.5	3175.221	0.03675	0.00245	6.67	0.03430	93.33

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.20 Seepage Model Results of C-9 Impoundment – Section 4 for Scenario A and Scenario B

Section 4 of L-509: Western Levee of C-9 Impoundment (Length: 10,425 ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-509(ft)	WSE in C-502B (ft)	WSE in SMA-3B (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into C-502B (cfs/ft)	% of Total Flow into C-502B	Flowrate into the area between C-509 and C-502B (cfs/ft)	% of Total Flow into the area between C-509 and C-502B	Flowrate into SMA-3B (cfs/ft)	% of Total Flow into SMA-3B	Flowrate into West End of Model (cfs/ft)	% of Total Flow into West End of Model
xsect4_R_85	8.5	5	6	6.5	835.914	0.00967	0.00086	8.93	0.00566	58.48	0.00039	4.04	0.00019	1.99	0.00257	26.56
xsect4_R_95	9.5	5	6	6.5	1210.787	0.01401	0.00100	7.11	0.00711	50.73	0.00047	3.38	0.00039	2.78	0.00505	36.01
xsect4_R_105	10.5	5	6	6.5	1585.717	0.01835	0.00113	6.16	0.00856	46.62	0.00055	3.02	0.00059	3.21	0.00752	40.98
xsect4_R_115	11.5	5	6	6.5	1960.715	0.02269	0.00126	5.55	0.01000	44.07	0.00064	2.84	0.00079	3.48	0.01000	44.06

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	11.5 to 4.5	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.21 Seepage Model Results of C-9 Impoundment – Section 5 for Scenario A

Section 5 of L-509M: Northern Levee of C-9 Impoundment Mitigation Area (Length: 2,270 ft)

5000ft in the impoundment, 7600 in the mitigation area and 600ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-509 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment/ Mitigation Area (cfd/ft)	Total FlowRate from Impoundment/ Mitigation Area (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect5_R_85	8.5	6.5	3	2.5	1943.307	0.02249	0.00026	1.14	0.02224	98.86
xsect5_R_95	9.5	6.5	3	2.5	1951.812	0.02259	0.00026	1.14	0.02233	98.86
xsect5_R_105	10.5	6.5	3	2.5	1960.494	0.02269	0.00026	1.14	0.02243	98.86
xsect5_R_115	11.5	6.5	3	2.5	1969.230	0.02279	0.00026	1.14	0.02253	98.86

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.22 Seepage Model Results of C-9 Impoundment – Section 6 for Scenario A

Section 6 of L-509M: Eastern and Western Levee of C-9 Impoundment Mitigation Area(Length: 7,670 ft)

2100ft in the mitigation area, 500ft east of the mitigation area, and 1500ft west of the mitigation area

FileName	WSE in the Mitigation Area (ft)	WSE in East C-509 (ft)	WSE in East End of Model* (ft)	WSE in West C-509 (ft)	WSE in C-502B (ft)	WSE in SMA-3B (ft)	Total FlowRate from Mitigation Area (cfd/ft)	Total FlowRate from Mitigation Area (cfs/ft)	Flowrate into East C-509 (cfs/ft)	% of Total Flow into East C-509	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model	Flowrate into West C-509 (cfs/ft)	% of Total Flow into West C-509
xsect6_R_6	6	3	2.5	5	6	6.5	1035.698	0.01199	0.00018	1.47	0.01177	98.18	0.00004	0.35
xsect6_R_65	6.5	3	2.5	5	6	6.5	1301.376	0.01506	0.00022	1.47	0.01475	97.91	0.00009	0.62

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is 2 feet below the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	11.5 to 4.5	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.23 Seepage Model Results of C-9 Impoundment – Section 1A for Scenario B

Section1A of L-509: Southern Levee of C-9 Impoundment at Improved C-9 Canal(Length: 5,065 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-9 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-9 Canal (cfs/ft)	% of Total Flow into C-9 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_85	8.5	4	4	2933.606	0.03395	0.03280	96.61	0.00115	3.39
xsect1_R_95	9.5	4	4	3585.671	0.04150	0.04009	96.61	0.00141	3.39
xsect1_R_105	10.5	4	4	4237.782	0.04905	0.04738	96.61	0.00166	3.39
xsect1_R_115	11.5	4	4	4889.937	0.05660	0.05468	96.61	0.00192	3.39

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.24 Seepage Model Results of C-9 Impoundment – Section 1B for Scenario B

Section1B of L-509: Southern Levee of C-9 Impoundment at Existing C-9 Canal (Length: 1,840 ft)

5000ft in the impoundment and 2000ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-9 Canal (ft)	WSE in South End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-9 Canal (cfs/ft)	% of Total Flow into C-9 Canal	Flowrate into South End of Model (cfs/ft)	% of Total Flow into South End of Model
xsect1_R_85	8.5	4	4	2398.639	0.02776	0.02532	91.20	0.00244	8.80
xsect1_R_95	9.5	4	4	2931.823	0.03393	0.03095	91.20	0.00299	8.80
xsect1_R_105	10.5	4	4	3465.053	0.04010	0.03658	91.20	0.00353	8.80
xsect1_R_115	11.5	4	4	3998.328	0.04628	0.04220	91.20	0.00407	8.80

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.25 Seepage Model Results of C-9 Impoundment – Section 2 for Scenario B

Section 2 of L-509: Eastern Levee of C-9 Impoundment (Length: 10,445 ft)

3500ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-509 (ft)	WSE in East End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model
xsect2_R_85	8.5	3	4	1638.602	0.01897	0.00211	11.13	0.01685	88.87
xsect2_R_95	9.5	3	4	1998.056	0.02313	0.00237	10.26	0.02075	89.75
xsect2_R_105	10.5	3	4	2357.666	0.02729	0.00264	9.66	0.02465	90.34
xsect2_R_115	11.5	3	4	2717.345	0.03145	0.00290	9.21	0.02855	90.79

Note:

WSE - Water Surface Elevation

SMA - Seepage Management Area

Elev. - Elevation

**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.26 Seepage Model Results of C-9 Impoundment – Section 3 for Scenario B

Section 3 of L-509: Northern Levee of C-9 Impoundment (Length: 4,935 ft)

5000ft in the impoundment and 1500ft outside of the impoundment.

FileName	WSE in the Impoundment (ft)	WSE in C-509 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment (cfd/ft)	Total FlowRate from Impoundment (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect3_R_85	8.5	3	4.5	1505.472	0.01742	0.00241	13.81	0.01502	86.20
xsect3_R_95	9.5	3	4.5	1873.041	0.02168	0.00268	12.38	0.01900	87.62
xsect3_R_105	10.5	3	4.5	2240.729	0.02593	0.00295	11.38	0.02298	88.62
xsect3_R_115	11.5	3	4.5	2608.492	0.03019	0.00321	10.64	0.02698	89.36

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5 to 4	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.27 Seepage Model Results of C-9 Impoundment – Section 5 for Scenario B

Section 5 of L-509M: Northern Levee of C-9 Impoundment Mitigation Area (Length: 2,270 ft)

5000ft in the impoundment, 7600 in the mitigation area and 600ft outside of the mitigation area

FileName	WSE in the Impoundment (ft)	WSE in Mitigation Area (ft)	WSE in C-509 (ft)	WSE in North End of Model* (ft)	Total FlowRate from Impoundment/ Mitigation Area (cfd/ft)	Total FlowRate from Impoundment/ Mitigation Area (cfs/ft)	Flowrate into C-509 (cfs/ft)	% of Total Flow into C-509	Flowrate into North End of Model (cfs/ft)	% of Total Flow into North End of Model
xsect5_R_85	8.5	6.5	3	4.5	1000.826	0.01158	0.00098	8.47	0.01060	91.53
xsect5_R_95	9.5	6.5	3	4.5	1009.360	0.01168	0.00098	8.42	0.01070	91.59
xsect5_R_105	10.5	6.5	3	4.5	1018.059	0.01178	0.00098	8.36	0.01080	91.65
xsect5_R_115	11.5	6.5	3	4.5	1026.821	0.01188	0.00099	8.31	0.01090	91.69

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	4.5	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.28 Seepage Model Results of C-9 Impoundment – Section 6 for Scenario B

Section 6 of L-509M: Eastern and Western Levee of C-9 Impoundment Mitigation Area(Length: 7,670 ft)

2100ft in the mitigation area, 500ft east of the mitigation area, and 1500ft west of the mitigation area

FileName	WSE in the Mitigation Area (ft)	WSE in East C-509 (ft)	WSE in East End of Model* (ft)	WSE in West C-509 (ft)	WSE in C-502B (ft)	WSE in SMA-3B (ft)	Total FlowRate from Mitigation Area (cfd/ft)	Total FlowRate from Mitigation Area (cfs/ft)	Flowrate into East C-509 (cfs/ft)	% of Total Flow into East C-509	Flowrate into East End of Model (cfs/ft)	% of Total Flow into East End of Model	Flowrate into West C-509 (cfs/ft)	% of Total Flow into West C-509
xsect6_R_6	6	3	4.5	5	6	6.5	425.768	0.00493	0.00092	18.64	0.00397	80.65	0.00003	0.71
xsect6_R_65	6.5	3	4.5	5	6	6.5	691.841	0.00801	0.00096	12.01	0.00697	87.01	0.00008	0.97

Note:
WSE - Water Surface Elevation
SMA - Seepage Management Area
Elev. - Elevation
**** The water table is at the ground surface.***

Soil Information: (based on the data from well number G-2317)

Layer Name	Top Elevation (ft)	Bottom Elevation (ft)	Kh(ft/d)	Kv(ft/d)
Embankment Fill			1	0.1
Layer 1	11.5 to 4.5	-11	50	5
Layer 2	-11	-70	19500	1950

Table C.29 S-505A Fixed Weir – Ungated CIT Weir Structure

Revision	6 December 2000 - Original Submission		
XY Coord	847200 628910		
Location	SE corner of C-11 Impoundment, on C-511 (C-11Canal Extension).		
Purpose	Prevent excessive drawdown of C-511 when backpumping C-11 Basin.		
Notes	1. Riprap requirements have not been verified with Geotech.		
Design Conditions			
	Discharge (CFS)	150	cfs
	Headwater Elevation	4.25	ft,NGVD
	Tailwater Elevation	4.00	ft,NGVD
Maximum Expected Stages			
	Headwater Elevation	5.50	ft,NGVD
	Tailwater Elevation	5.00	ft,NGVD
Maximum Head Difference			
	Maximum Headwater Elevation	4.50	ft,NGVD
	Minimum Tailwater Elevation	1.00	ft,NGVD
Weir Data			
	Weir Type	Broad Crested	
	Weir Breadth	1.00	feet
	Crest Length	100.0	feet
	Crest Elevation	3.50	ft,NGVD
	Maximum Head on Crest	1.75	feet
	Upstream Weir Height (Crest El - Apron El)	1.50	feet
	Minimum Tieback Wall Elevation	7.00	ft, NGVD
	Weir Control	Not Gated	
Downstream Stilling Basin			
	Apron Width	100.0	feet
	Apron Elevation	-0.50	ft,NGVD
	Length (feet)	5.00	feet
	Minimum Sidewall Elevation	5.50	ft,NGVD
	End Sill Elevation	0.00	ft,NGVD
Canal Data			
	Side Slopes Cotangent	3	
	Upstream Bottom Width	40.00	feet
	Upstream Bottom Elevation	-10.00	ft,NGVD
	Downstream Bottom Width	35.00	feet
	Downstream Bottom Elevation	-12.00	ft,NGVD
Riprap Requirements			
	Upstream Velocity at crest	1.00	fps
	Downstream Velocity Over the End Sill	2.31	fps
	Design Riprap Velocity	4.00	fps
	Riprap Protected Area	1,000	sq-ft
	Riprap Thickness	1.5	feet
	Riprap Bedding Thickness	1.0	feet
Control Protection Elevation		6.50	ft,NGVD

Table C.30 S-505B Fixed Weir – Ungated CIT Weir Structure

Revision 5 September 2001 - Original Submission

XY Coord 849720 638050
Location NE corner of C-11 Impoundment, on C-511 (C-11Canal Extension).
Purpose Control perimeter mitigation seepage canal stage at 5.00 ft, NGVD.
Notes 1. Weir is a combination or notched weir.
2. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	75	cfs
Headwater Elevation	5.15	ft,NGVD
Tailwater Elevation	4.00	ft,NGVD
Maximum Expected Stages		
Headwater Elevation	7.00	ft,NGVD
Tailwater Elevation	6.00	ft,NGVD
Maximum Head Difference		
Maximum Headwater Elevation	5.40	ft,NGVD
Minimum Tailwater Elevation	3.50	ft,NGVD
Weir Data		
Weir Type	Broad Combination	
Weir Breadth	1.00	feet
Lower Weir Crest Length	32.0	feet
Lower Weir Crest Elevation	4.65	ft,NGVD
Overall Crest Length	70.0	feet
Upper Weir Crest Elevation	4.90	ft,NGVD
Upstream Weir Height, P (Crest El - Apron El)	2.15	feet
Minimum Tieback Wall Elevation	7.25	ft, NGVD
Weir Control	Not Gated	
Downstream Stilling Basin		
Apron Width	70.0	feet
Apron Elevation	2.50	ft,NGVD
Length (feet)	3.50	feet
Minimum Sidewall Elevation	6.25	ft,NGVD
End Sill Elevation	2.75	ft,NGVD
Canal Data		
Side Slopes Cotangent	3	
Upstream Bottom Width	Pool	feet
Upstream Bottom Elevation	-3.50	ft,NGVD
Downstream Bottom Width	40.00	feet
Downstream Bottom Elevation	-10.00	ft,NGVD
Riprap Requirements		
Riprap Protected Area	800	sq-ft
Riprap Thickness	1.5	feet
Riprap Bedding Thickness	1.0	feet
Control Protection Elevation	7.50	ft,NGVD

Table C.31 S-505B Ungated Round Culverts

Revision	5 September 2001 - Original Submission	
XY Coord	849720 638050	
Location	Junction of C-11 mitigation seepage canal and C-11 Extension	
Purpose	Provides access to C-11 Impoundment on eastern boundary	
Notes	1. Riprap requirements have not been verified with Geotech.	
Design Conditions		
	Discharge (CFS)	75 cfs
	Headwater Elevation	4.65 ft,NGVD
	Tailwater Elevation	4.55 ft,NGVD
Maximum Expected Stages		
	Headwater Elevation	7.00 ft,NGVD
	Tailwater Elevation	7.00 ft,NGVD
Maximum Head Difference		
	Maximum Headwater Elevation	4.65 ft,NGVD
	Minimum Tailwater Elevation	4.45 ft,NGVD
Culvert Data		
	Number of Barrels	3
	Barrel Type	CAP
	Barrel Diameter	6.0 feet
	Barrel Length	60.0 feet
	Barrel Invert Elevation	-3.00 ft, NGVD
	Type of Control	None
Canal Data		
	Side Slopes Cotangent	1
	Upstream Bottom Width	20.00 feet
	Upstream Bottom Elevation	-2.50 ft,NGVD
	Downstream Bottom Width	Pool feet
	Downstream Bottom Elevation	-3.50 ft,NGVD
Riprap Requirements		
	Design Barrel Velocity	1.00 fps
	Design Riprap Velocity	2.00 fps
	Riprap Protected Area	800 sq-ft
	Riprap Thickness	2.00 feet
	Riprap Bedding Thickness	1.00 feet
Control Protection Elevation		7.50 ft,NGVD

Table C.32 S-505C Pump Station

Revisions:

- 12 December 2000 – Original submission.

XY Coordinate[1] – 842300 630480
Location: Southwestern corner of C-11 Impoundment, north of Truck Stop.

Purpose/Operational Intent: Seepage Control

- Control water surface elevation in seepage collection canal C-511 for the C-11 Impoundment.

Design Condition:	Seepage Control	120 cfs

Pump Station Capacity Criteria:

- The design pump rate was determined by multiplying the seepage rate (0.0015 cfs/linear ft) times seepage canal length (16,000 ft) times a safety factor (5).

Number of Pumps	2	
Pump Mix Type and Size		
	Electric	2 @ 60 cfs

Mix Criteria:

- The pump station will have two bays; two identical 60-cfs pumps.
- The pump mix allows for an intermediate flow value of half capacity for lower seepage rates corresponding with lower impoundment stages.

Control: Manned & Remote by SCADA

Design Heads (ft.)		
Normal (4.50 HW to 12.00 TW)	7.50	feet
Maximum (3.50 HW to 12.00 TW)	8.50	feet

Intake Water Surface Elevations		
Maximum Non-Pumping	8.00	ft-NGVD
Maximum Pumping	7.00	ft-NGVD
Start Pumping	5.10	ft-NGVD
Normal Drawdown	3.5 to 5.0 ft-NGVD	
Minimum Drawdown	3.50	ft-NGVD
Minimum Non-Pumping	3.50	ft-NGVD
Channel Invert	-1.00	ft-NGVD

Discharge Water Surface Elevations		
Maximum Non-Pumping	15.0	ft-NGVD
Maximum Pumping	12.0	ft-NGVD
Normal Pumping 12.0	ft-NGVD	
Minimum Pumping	3.50	ft-NGVD
Minimum Non-Pumping	3.50	ft-NGVD
Channel Invert -1.00	ft-NGVD	

Notes:

- [1] XY coordinates system used is NAD 83, Florida east, state plane
- All elevations are in feet, NGVD (National Geodetic Vertical Datum of 1929)
- Diesel generator is required for control station operations and electric pumps in cases of power outage.

Data Compiled from:

- Selected Plan features.

Table C.33 S-512A Pump Station

Revisions:
12 December 2000 – Original submission

XY Coordinate[1] – 845250 600920
Location: At the intersection of the SE corner of the mitigation area and the C-9 Impoundment.

Purpose/Operational Intent: Flood Control
- Control WSE in two divided seepage collection canals: (1) mitigation northern/eastern reach, and (2) C-9 Impoundment's northern/eastern reach.

Design Condition: Seepage Control 225 cfs

Pump Station Capacity Criteria:
- The design pump rate was determined by seepage rate analysis and incorporating a safety factor of 5.

Number of Pumps	3	
Pump Mix Type and Size		
	Electric	3 @ 75 cfs

Mix Criteria:
- The pump station will have three identical 75 cfs pumps.
- One pump is utilized for the mitigation northern/eastern sides seepage canal.
- Two pumps are utilized for C-9 Impoundment's northern/eastern sides seepage canal.

Control: Remote by SCADA or Local

<u>Design Heads</u>		
Normal (2.5 HW to 10.5 TW)	8.00	feet
Maximum (2.0 HW to 10.5 TW)	8.50	feet

Intake Water Surface Elevations		
Maximum Non-Pumping	6.50	ft-NGVD
Maximum Pumping	6.50	ft-NGVD
Start Pumping	3.10	ft-NGVD
Normal Pumping	2.5 to 3.0	ft-NGVD
Minimum Drawdown Pumping	2.00	ft-NGVD
Minimum Non-Pumping	2.00	ft-NGVD
Channel Invert	-4.50	ft-NGVD

<u>Discharge Water Surface Elevations</u>		
Maximum Non-Pumping	12.50	ft-NGVD
Maximum Pumping	10.50	ft-NGVD
Normal Pumping	10.50	ft-NGVD
Minimum Pumping	3.00	ft-NGVD
Minimum Non-Pumping	3.00	ft-NGVD
Channel Invert	0.00	ft-NGVD

Notes:
- [1] XY coordinates system used is NAD 83, Florida east, state plane.
- All elevations are in feet, NGVD (National Geodetic Vertical Datum of 1929)

Data Compiled from:
- Selected Plan parameters.

Table C.34 S-512B Fixed Weir – Ungated CIT Weir Structure

Revision 5 September 2001 - Original Submission

XY Coord 843310 590500
Location SW corner of C-9 Impoundment on C-509 seepage canal
Purpose Control perimeter mitigation seepage canal stage at 5.00 ft, NGVD.
Notes
1. Weir is a combination or notched weir.
2. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	125	cfs
Headwater Elevation	5.20	ft,NGVD
Tailwater Elevation	4.00	ft,NGVD
Maximum Expected Stages		
Headwater Elevation	7.00	ft,NGVD
Tailwater Elevation	6.00	ft,NGVD
Maximum Head Difference		
Maximum Headwater Elevation	5.40	ft,NGVD
Minimum Tailwater Elevation	3.50	ft,NGVD
Weir Data		
Weir Type	Broad Combination	
Weir Breadth	1.00	feet
Lower Weir Crest Length	86.0	feet
Lower Weir Crest Elevation	4.65	ft,NGVD
Overall Crest Length	95.0	feet
Upper Weir Crest Elevation	4.80	ft,NGVD
Upstream Weir Height, P (Crest El - Apron El)	2.15	feet
Minimum Tieback Wall Elevation	7.25	ft, NGVD
Weir Control	Not Gated	
Downstream Stilling Basin		
Apron Width	95.0	feet
Apron Elevation	2.50	ft,NGVD
Length (feet)	3.50	feet
Minimum Sidewall Elevation	6.25	ft,NGVD
End Sill Elevation	2.75	ft,NGVD
Canal Data		
Side Slopes Cotangent	3	
Upstream Bottom Width	15.00	feet
Upstream Bottom Elevation	-1.00	ft,NGVD
Downstream Bottom Width	Pool	feet
Downstream Bottom Elevation	-1.00	ft,NGVD
Riprap Requirements		
Riprap Protected Area	800	sq-ft
Riprap Thickness	1.5	feet
Riprap Bedding Thickness	1.0	feet
Control Protection Elevation	7.25	ft,NGVD

Table C.35 S-512B Gated Round Culverts

Revision	5 September 2001 - Original Submission
XY Coord	843310 590500
Location	SW corner of C-9 Impoundment on C-509 perimeter seepage canal
Purpose	Allows excess seepage water to drain to C-9 Canal. Blocks higher C-9 Canal stages from passing into seepage canal.
Notes	1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	125	cfs
Headwater Elevation	4.65	ft,NGVD
Tailwater Elevation	4.35	ft,NGVD
Maximum Expected Stages		
Headwater Elevation	6.00	ft,NGVD
Tailwater Elevation	7.00	ft,NGVD
Maximum Head Difference		
Maximum Headwater Elevation	4.50	ft,NGVD
Minimum Tailwater Elevation	3.50	ft,NGVD
Culvert Data		
Number of Barrels	2	
Barrel Type	CAP	
Barrel Diameter	6.0	feet
Barrel Length	60.0	feet
Barrel Invert Elevation	-4.50	ft, NGVD
Type of Control	Flap Gates	
Canal Data		
Side Slopes Cotangent	1	
Upstream Bottom Width	Pool	feet
Upstream Bottom Elevation	-1.00	ft,NGVD
Downstream Bottom Width	Pool	feet
Downstream Bottom Elevation	-16.50	ft,NGVD
Riprap Requirements		
Design Barrel Velocity	2.21	fps
Design Riprap Velocity	4.00	fps
Riprap Protected Area	800	sq-ft
Riprap Thickness	2.00	feet
Riprap Bedding Thickness	1.00	feet
Control Protection Elevation	8.00	ft,NGVD

Table C.36 HEC-RAS Input – Flow Rates Along West Reach of C-511

				Flow Rates at Each Reach Station for Various Profiles (cfs)							
	River	Reach	RS	10 ft Base	11 ft Base	12 ft Base	13 ft Base	10 ft Max	11 ft Max	12 ft Max	13 ft Max
1	C-511	West	16605	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
2	C-511	West	15605	0.67	0.77	0.87	0.97	1.14	1.23	1.31	1.39
3	C-511	West	14605	1.34	1.54	1.74	1.94	2.28	2.45	2.61	2.77
4	C-511	West	13605	2.01	2.31	2.62	2.9	3.42	3.68	3.92	4.16
5	C-511	West	12865	2.5	2.88	3.26	3.62	4.27	4.58	4.89	5.18
6	C-511	West	11865	3.11	3.54	3.98	4.4	5.35	5.72	6.07	6.42
7	C-511	West	10865	3.71	4.21	4.7	5.18	6.44	6.85	7.26	7.65
8	C-511	West	10290	4.06	4.59	5.12	5.62	7.06	7.51	7.94	8.36
9	C-511	West	9290	5.24	5.77	6.3	6.81	8.24	8.69	9.12	9.54
10	C-511	West	8290	6.42	6.95	7.48	7.99	9.43	9.87	10.3	10.72
11	C-511	West	7290	7.76	8.41	9.05	9.68	10.76	11.33	11.88	12.41
12	C-511	West	6290	9.1	9.87	10.63	11.37	12.1	12.78	13.45	14.1
13	C-511	West	5290	10.44	11.32	12.2	13.06	13.44	14.24	15.03	15.79
14	C-511	West	4290	11.78	12.78	13.78	14.75	14.78	15.7	16.6	17.48
15	C-511	West	3290	13.12	14.24	15.35	16.44	16.12	17.16	18.18	19.18
16	C-511	West	2290	14.46	15.7	16.93	18.13	17.46	18.62	19.75	20.87
17	C-511	West	1290	15.8	17.15	18.5	19.83	18.8	20.07	21.33	22.56
18	C-511	West	290	17.14	18.61	20.08	21.52	20.14	21.53	22.9	24.25
19	C-511	West	0	17.52	19.04	20.54	22.01	20.53	21.96	23.36	24.74

Table C.37 West Reach of C-511 – HEC-RAS Output for Profile "13 ft Max"

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
West	16605	13 ft Max	0.00	-1.00	5.01		5.01	0.000000	0.00	168.54	46.07	0.00
West	15605	13 ft Max	1.39	-1.00	5.01		5.01	0.000000	0.01	168.54	46.07	0.00
West	14605	13 ft Max	2.77	-1.00	5.01		5.01	0.000000	0.02	168.54	46.07	0.00
West	13605	13 ft Max	4.16	-1.00	5.01		5.01	0.000000	0.02	168.53	46.07	0.00
West	12865	13 ft Max	5.18	-1.00	5.01		5.01	0.000000	0.03	168.53	46.07	0.00
West	11865	13 ft Max	6.42	-1.00	5.01		5.01	0.000000	0.04	168.53	46.07	0.00
West	10865	13 ft Max	7.65	-1.00	5.01		5.01	0.000000	0.05	168.52	46.07	0.00
West	10290	13 ft Max	8.36	-1.00	5.01		5.01	0.000000	0.05	168.51	46.07	0.00
West	9290	13 ft Max	9.54	-1.00	5.01		5.01	0.000000	0.06	168.50	46.06	0.01
West	8290	13 ft Max	10.72	-1.00	5.01		5.01	0.000000	0.06	168.48	46.06	0.01
West	7290	13 ft Max	12.41	-1.00	5.01		5.01	0.000001	0.07	168.45	46.06	0.01
West	6290	13 ft Max	14.10	-1.00	5.01		5.01	0.000001	0.08	168.42	46.06	0.01
West	5290	13 ft Max	15.79	-1.00	5.01		5.01	0.000001	0.09	168.38	46.05	0.01
West	4290	13 ft Max	17.48	-1.00	5.01		5.01	0.000001	0.10	168.34	46.04	0.01
West	3290	13 ft Max	19.18	-1.00	5.01		5.01	0.000001	0.11	168.28	46.04	0.01
West	2290	13 ft Max	20.87	-1.00	5.00		5.00	0.000002	0.12	168.21	46.03	0.01
West	1290	13 ft Max	22.56	-1.00	5.00		5.00	0.000002	0.13	168.13	46.02	0.01
West	290	13 ft Max	24.25	-1.00	5.00		5.00	0.000002	0.14	168.03	46.00	0.01
West	0	13 ft Max	24.74	-1.00	5.00	-0.46	5.00	0.000002	0.15	168.00	46.00	0.01

Table C.38 HEC-RAS Input – Flow Rates Along East Reach of C-511

	River	Reach	RS	Flow Rates at Each Reach Station for Various Profiles (cfs)							
				10 ft Base	11 ft Base	12 ft Base	13 ft Base	10 ft Max	11 ft Max	12 ft Max	13 ft Max
1	C-511	East	14710	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
2	C-511	East	13710	0.72	0.88	1.03	1.17	1.45	1.58	1.71	1.84
3	C-511	East	12710	1.45	1.75	2.05	2.34	2.89	3.16	3.42	3.68
4	C-511	East	11820	2.09	2.53	2.97	3.38	4.18	4.56	4.95	5.32
5	C-511	East	11775	2.15	2.6	3.04	3.47	4.29	4.68	5.07	5.46
6	C-511	East	11700	2.27	2.75	3.22	3.66	4.5	4.92	5.33	5.74
7	C-511	East	11600	2.55	3.08	3.61	4.11	4.97	5.44	5.9	6.36
8	C-511	East	10745	4.95	5.95	6.93	7.89	8.99	9.9	10.79	11.67
9	C-511	East	9745	7.74	9.3	10.83	12.31	13.7	15.12	16.5	17.89
10	C-511	East	8745	10.54	12.65	14.72	16.73	18.41	20.33	22.22	24.1
11	C-511	East	7745	13.34	16.01	18.61	21.15	23.11	25.55	27.94	30.31
12	C-511	East	6745	16.14	19.36	22.5	25.57	27.82	30.77	33.65	36.53
13	C-511	East	5745	18.94	22.71	26.39	29.99	32.52	35.99	39.37	42.74
14	C-511	East	4745	21.74	26.06	30.28	34.42	37.23	41.2	45.08	48.95
15	C-511	East	3745	24.54	29.41	34.18	38.84	41.93	46.42	50.8	55.17
16	C-511	East	2745	27.33	32.77	38.07	43.26	46.64	51.64	56.51	61.38
17	C-511	East	1745	30.13	36.12	41.96	47.68	51.35	56.86	62.23	67.59
18	C-511	East	745	32.93	39.47	45.85	52.1	56.05	62.07	67.94	73.81
19	C-511	East	100	34.74	41.63	48.36	54.95	59.09	65.44	71.63	77.82
20	C-511	East	40	35.02	41.97	48.75	55.4	59.56	65.96	72.2	78.44
21	C-511	East	-40	35.02	41.97	48.75	55.4	59.56	65.96	72.2	78.44
22	C-511	East	-100	35.02	41.97	48.75	55.4	59.56	65.96	72.2	78.44

Table C.39 East Reach of C-511 – HEC-RAS Output for Profile "13 ft Max"

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
East	14710	13 ft Max	0.00	-2.50	4.81		4.81	0.000000	0.00	360.93	93.87	0.00
East	13710	13 ft Max	1.84	-2.50	4.81		4.81	0.000000	0.01	360.93	93.87	0.00
East	12710	13 ft Max	3.68	-2.50	4.81		4.81	0.000000	0.01	360.93	93.87	0.00
East	11820	13 ft Max	5.32	-2.50	4.81		4.81	0.000000	0.02	360.93	93.87	0.00
East	11775	13 ft Max	5.46	-3.50	4.81	-3.44	4.81	0.000000	0.01	685.41	119.87	0.00
East	11745		Inl Struct									
East	11700	13 ft Max	5.74	-10.00	4.04	-9.91	4.04	0.000000	0.01	869.95	127.18	0.00
East	11600	13 ft Max	6.36	-10.00	4.04		4.04	0.000000	0.01	986.95	127.18	0.00
East	10745	13 ft Max	11.67	-10.00	4.04		4.04	0.000000	0.01	986.95	127.18	0.00
East	9745	13 ft Max	17.89	-10.00	4.04		4.04	0.000000	0.02	986.95	127.18	0.00
East	8745	13 ft Max	24.10	-10.00	4.03		4.04	0.000000	0.03	986.95	127.18	0.00
East	7745	13 ft Max	30.31	-10.00	4.03		4.03	0.000000	0.03	986.94	127.17	0.00
East	6745	13 ft Max	36.53	-10.00	4.03		4.03	0.000000	0.04	986.94	127.17	0.00
East	5745	13 ft Max	42.74	-10.00	4.03		4.03	0.000000	0.04	986.93	127.17	0.00
East	4745	13 ft Max	48.95	-10.00	4.03		4.03	0.000000	0.05	986.92	127.17	0.00
East	3745	13 ft Max	55.17	-10.00	4.03		4.03	0.000000	0.06	986.91	127.17	0.00
East	2745	13 ft Max	61.38	-10.00	4.03		4.03	0.000000	0.06	986.90	127.17	0.00
East	1745	13 ft Max	67.59	-10.00	4.03		4.03	0.000000	0.07	986.88	127.17	0.00
East	745	13 ft Max	73.81	-10.00	4.03		4.03	0.000000	0.08	986.86	127.17	0.00
East	100	13 ft Max	77.82	-10.00	4.03		4.03	0.000000	0.08	986.84	127.17	0.00
East	40	13 ft Max	78.44	-10.00	4.03	-9.51	4.03	0.000000	0.08	986.84	127.17	0.00
East	0		Inl Struct									
East	-40	13 ft Max	78.44	-12.00	4.00	-11.47	4.00	0.000000	0.06	1328.00	131.00	0.00
East	-100	13 ft Max	78.44	-12.00	4.00	-11.47	4.00	0.000000	0.06	1328.00	131.00	0.00

Table C.40 HEC-RAS Input – Flow Rates Along South Reach of C-511

	River	Reach	RS	Flow Rates at Each Reach Station for Various Profiles (cfs)			
				10 ft Base	11 ft Base	12 ft Base	13 ft Base
1	C-511	South	1390	0.001	0.001	0.001	0.001
2	C-511	South	890	0.86	0.91	0.97	1.02
3	C-511	South	390	1.71	1.82	1.94	2.05
4	C-511	South	0	2.38	2.54	2.69	2.84

Table C.41 South Reach of C-511 – HEC-RAS Output for Profile "13 ft Base"

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
South	1390	13 Base	0.00	-1.00	4.00		4.00	0.000000	0.00	125.00	40.00	0.00
South	890	13 Base	1.02	-1.00	4.00		4.00	0.000000	0.01	125.00	40.00	0.00
South	390	13 Base	2.05	-1.00	4.00		4.00	0.000000	0.02	125.00	40.00	0.00
South	0	13 Base	2.84	-1.00	4.00	-0.87	4.00	0.000000	0.02	125.00	40.00	0.00

Table C.42 HEC-RAS Input – Flow Rates Along West Reach of C-509

	River	Reach	RS	Flow Rates at Each Reach Station for Various Profiles (cfs)							
				8.5 ft Base	9.5 ft Base	10.5 ft Base	11.5 ft Base	8.5 ft Max	9.5 ft Max	10.5 ft Max	11.5 ft Max
1	C-509	West	18055	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
2	C-509	West	17055	0.33	0.33	0.33	0.33	0.47	0.47	0.47	0.47
3	C-509	West	16055	0.66	0.66	0.66	0.66	0.94	0.94	0.94	0.94
4	C-509	West	15055	0.98	0.98	0.98	0.98	1.41	1.41	1.41	1.41
5	C-509	West	14055	1.31	1.31	1.31	1.31	1.88	1.88	1.88	1.88
6	C-509	West	13055	1.64	1.64	1.64	1.64	2.35	2.35	2.35	2.35
7	C-509	West	12055	1.97	1.97	1.97	1.97	2.82	2.82	2.82	2.82
8	C-509	West	11055	2.29	2.29	2.29	2.29	3.29	3.29	3.29	3.29
9	C-509	West	10455	2.49	2.49	2.49	2.49	3.58	3.58	3.58	3.58
10	C-509	West	10425	2.5	2.5	2.5	2.5	3.59	3.59	3.59	3.59
11	C-509	West	9425	3.37	3.5	3.63	3.76	4.46	4.59	4.72	4.85
12	C-509	West	8425	4.24	4.5	4.76	5.03	5.33	5.59	5.85	6.12
13	C-509	West	7425	5.1	5.5	5.89	6.29	6.19	6.59	6.98	7.38
14	C-509	West	6425	5.97	6.5	7.03	7.55	7.06	7.59	8.12	8.64
15	C-509	West	5425	6.84	7.5	8.16	8.81	7.93	8.59	9.25	9.9
16	C-509	West	4425	7.71	8.5	9.29	10.08	8.8	9.59	10.38	11.17
17	C-509	West	3425	8.58	9.5	10.42	11.34	9.67	10.59	11.51	12.43
18	C-509	West	2425	9.45	10.5	11.55	12.6	10.54	11.59	12.64	13.69
19	C-509	West	1425	10.31	11.5	12.68	13.87	11.4	12.59	13.77	14.95
20	C-509	West	425	11.18	12.51	13.81	15.13	12.27	13.6	14.9	16.22
21	C-509	West	100	11.46	12.83	14.18	15.54	12.55	13.92	15.27	16.63
22	C-509	West	45	11.55	12.93	14.29	15.66	12.64	14.02	15.38	16.75
23	C-509	West	-35	11.55	12.93	14.29	15.66	12.64	14.02	15.38	16.75
24	C-509	West	-80	11.55	12.93	14.29	15.66	12.64	14.02	15.38	16.75

Table C.43 West Reach of C-509 – HEC-RAS Output for Profile "11.5 ft Max"

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
West	18055	11.5ft Max	0.00	-1.00	4.83		4.83	0.000000	0.00	160.39	45.00	0.00
West	17055	11.5ft Max	0.47	-1.00	4.83		4.83	0.000000	0.00	160.39	45.00	0.00
West	16055	11.5ft Max	0.94	-1.00	4.83		4.83	0.000000	0.01	160.39	45.00	0.00
West	15055	11.5ft Max	1.41	-1.00	4.83		4.83	0.000000	0.01	160.39	45.00	0.00
West	14055	11.5ft Max	1.88	-1.00	4.83		4.83	0.000000	0.01	160.39	45.00	0.00
West	13055	11.5ft Max	2.35	-1.00	4.83		4.83	0.000000	0.01	160.39	45.00	0.00
West	12055	11.5ft Max	2.82	-1.00	4.83		4.83	0.000000	0.02	160.39	45.00	0.00
West	11055	11.5ft Max	3.29	-1.00	4.83		4.83	0.000000	0.02	160.39	45.00	0.00
West	10455	11.5ft Max	3.58	-1.00	4.83		4.83	0.000000	0.02	160.39	45.00	0.00
West	10425	11.5ft Max	3.59	-1.00	4.83		4.83	0.000000	0.02	160.39	45.00	0.00
West	9425	11.5ft Max	4.85	-1.00	4.83		4.83	0.000000	0.03	160.38	45.00	0.00
West	8425	11.5ft Max	6.12	-1.00	4.83		4.83	0.000000	0.04	160.38	44.99	0.00
West	7425	11.5ft Max	7.38	-1.00	4.83		4.83	0.000000	0.05	160.37	44.99	0.00
West	6425	11.5ft Max	8.64	-1.00	4.83		4.83	0.000000	0.05	160.36	44.99	0.01
West	5425	11.5ft Max	9.90	-1.00	4.83		4.83	0.000000	0.06	160.34	44.99	0.01
West	4425	11.5ft Max	11.17	-1.00	4.83		4.83	0.000001	0.07	160.32	44.99	0.01
West	3425	11.5ft Max	12.43	-1.00	4.83		4.83	0.000001	0.08	160.29	44.98	0.01
West	2425	11.5ft Max	13.69	-1.00	4.83		4.83	0.000001	0.09	160.26	44.98	0.01
West	1425	11.5ft Max	14.95	-1.00	4.83		4.83	0.000001	0.09	160.22	44.97	0.01
West	425	11.5ft Max	16.22	-1.00	4.83		4.83	0.000001	0.10	160.17	44.97	0.01
West	100	11.5ft Max	16.63	-1.00	4.83		4.83	0.000001	0.10	160.15	44.96	0.01
West	45	11.5ft Max	16.75	-1.00	4.83	-0.67	4.83	0.000001	0.09	204.36	95.00	0.01
West	0		Inl Struct									
West	-35	11.5ft Max	16.75	-1.00	4.00		4.00	0.000000	0.04	475.00	95.00	0.00
West	-80	11.5ft Max	16.75	-1.00	4.00	-0.91	4.00	0.000000	0.04	475.00	95.00	0.00

Table C.44 HEC-RAS Input – Flow Rates Along North Reach of C-509

	River	Reach	RS	Flow Rates at Each Reach Station for Various Profiles (cfs)							
				8.5 ft Base	9.5 ft Base	10.5 ft Base	11.5 ft Base	8.5 ft Max	9.5 ft Max	10.5 ft Max	11.5 ft Max
1	C-509	North	9940	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
2	C-509	North	8940	0.26	0.26	0.26	0.26	0.99	0.99	0.99	0.99
3	C-509	North	7940	0.51	0.52	0.52	0.52	1.97	1.97	1.98	1.98
4	C-509	North	7670	0.58	0.58	0.59	0.59	2.24	2.24	2.24	2.25
5	C-509	North	6940	0.74	0.75	0.75	0.75	2.94	2.94	2.95	2.95
6	C-509	North	5940	0.96	0.97	0.97	0.97	3.9	3.91	3.91	3.91
7	C-509	North	4940	1.18	1.19	1.19	1.19	4.87	4.87	4.87	4.88
8	C-509	North	3940	1.4	1.41	1.41	1.41	5.83	5.83	5.83	5.84
9	C-509	North	2940	1.62	1.63	1.63	1.64	6.79	6.79	6.8	6.8
10	C-509	North	1940	1.85	1.85	1.85	1.86	7.75	7.76	7.76	7.76
11	C-509	North	940	2.07	2.07	2.07	2.08	8.72	8.72	8.72	8.72
12	C-509	North	0	2.27	2.28	2.28	2.28	9.62	9.62	9.63	9.63

Table C.45 North Reach of C-509 – HEC-RAS Output for Profile "11.5 ft Max"

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
North	9940	11.5ft Max	0.00	-2.00	3.00		3.00	0.000000	0.00	125.11	40.02	0.00
North	8940	11.5ft Max	0.99	-2.00	3.00		3.00	0.000000	0.01	125.10	40.02	0.00
North	7940	11.5ft Max	1.98	-2.00	3.00		3.00	0.000000	0.02	125.10	40.02	0.00
North	7670	11.5ft Max	2.25	-2.00	3.00		3.00	0.000000	0.02	125.10	40.02	0.00
North	6940	11.5ft Max	2.95	-2.00	3.00		3.00	0.000000	0.02	125.10	40.02	0.00
North	5940	11.5ft Max	3.91	-2.00	3.00		3.00	0.000000	0.03	125.10	40.01	0.00
North	4940	11.5ft Max	4.88	-2.00	3.00		3.00	0.000000	0.04	125.09	40.01	0.00
North	3940	11.5ft Max	5.84	-2.00	3.00		3.00	0.000000	0.05	125.08	40.01	0.00
North	2940	11.5ft Max	6.80	-2.00	3.00		3.00	0.000000	0.05	125.07	40.01	0.01
North	1940	11.5ft Max	7.76	-2.00	3.00		3.00	0.000000	0.06	125.05	40.01	0.01
North	940	11.5ft Max	8.72	-2.00	3.00		3.00	0.000001	0.07	125.03	40.00	0.01
North	0	11.5ft Max	9.63	-2.00	3.00	-1.70	3.00	0.000001	0.08	125.00	40.00	0.01

Table C.46 HEC-RAS Input – Flow Rates along East Reach of C-509

	River	Reach	RS	Flow Rates at Each Reach Station for Various Profiles (cfs)							
				8.5 ft Base	9.5 ft Base	10.5 ft Base	11.5 ft Base	8.5 ft Max	9.5 ft Max	10.5 ft Max	11.5 ft Max
1	C-509	East	15380	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
2	C-509	East	14380	1.27	1.57	1.86	2.13	2.11	2.37	2.64	2.9
3	C-509	East	13380	2.54	3.15	3.71	4.26	4.22	4.74	5.27	5.79
4	C-509	East	12380	3.81	4.72	5.57	6.39	6.33	7.12	7.91	8.69
5	C-509	East	11380	5.08	6.29	7.43	8.52	8.44	9.49	10.55	11.59
6	C-509	East	10380	6.34	7.86	9.29	10.65	10.56	11.86	13.18	14.48
7	C-509	East	9380	7.61	9.43	11.14	12.79	12.67	14.23	15.82	17.38
8	C-509	East	8380	8.88	11.01	13	14.92	14.78	16.6	18.45	20.28
9	C-509	East	7380	10.15	12.58	14.86	17.05	16.89	18.97	21.09	23.17
10	C-509	East	6380	11.42	14.15	16.71	19.18	19	21.34	23.73	26.07
11	C-509	East	5380	12.69	15.72	18.57	21.31	21.11	23.72	26.36	28.97
12	C-509	East	4935	13.25	16.42	19.4	22.26	22.05	24.77	27.54	30.26
13	C-509	East	4380	14.12	17.47	20.6	23.62	23.39	26.27	29.18	32.04
14	C-509	East	3380	15.69	19.35	22.78	26.08	25.8	28.96	32.13	35.26
15	C-509	East	2380	17.26	21.23	24.95	28.54	28.22	31.65	35.09	38.47
16	C-509	East	1380	18.83	23.11	27.13	30.99	30.64	34.34	38.05	41.69
17	C-509	East	380	20.39	24.99	29.3	33.45	33.05	37.03	41	44.91
18	C-509	East	0	20.99	25.7	30.13	34.39	33.97	38.05	42.12	46.13

Table C.47 East Reach of C-509 – HEC-RAS Output for Profile “11.5 ft Max”

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
East	15380	11.5ft Max	0.00	-4.50	3.01		3.01	0.000000	0.00	349.40	95.04	0.00
East	14380	11.5ft Max	2.90	-4.50	3.01		3.01	0.000000	0.01	349.40	95.04	0.00
East	13380	11.5ft Max	5.79	-4.50	3.01		3.01	0.000000	0.02	349.40	95.04	0.00
East	12380	11.5ft Max	8.69	-4.50	3.01		3.01	0.000000	0.03	349.40	95.04	0.00
East	11380	11.5ft Max	11.59	-4.50	3.01		3.01	0.000000	0.04	349.39	95.04	0.00
East	10380	11.5ft Max	14.48	-4.50	3.01		3.01	0.000000	0.04	349.38	95.04	0.00
East	9380	11.5ft Max	17.38	-4.50	3.01		3.01	0.000000	0.05	349.36	95.04	0.00
East	8380	11.5ft Max	20.28	-4.50	3.01		3.01	0.000000	0.06	349.34	95.04	0.00
East	7380	11.5ft Max	23.17	-4.50	3.01		3.01	0.000000	0.07	349.31	95.04	0.01
East	6380	11.5ft Max	26.07	-4.50	3.01		3.01	0.000000	0.08	349.27	95.03	0.01
East	5380	11.5ft Max	28.97	-4.50	3.01		3.01	0.000001	0.09	349.23	95.03	0.01
East	4935	11.5ft Max	30.26	-4.50	3.00		3.00	0.000001	0.09	349.20	95.03	0.01
East	4380	11.5ft Max	32.04	-4.50	3.00		3.00	0.000001	0.10	349.17	95.03	0.01
East	3380	11.5ft Max	35.26	-4.50	3.00		3.00	0.000001	0.11	349.10	95.02	0.01
East	2380	11.5ft Max	38.47	-4.50	3.00		3.00	0.000001	0.12	349.01	95.02	0.01
East	1380	11.5ft Max	41.69	-4.50	3.00		3.00	0.000001	0.13	348.91	95.01	0.01
East	380	11.5ft Max	44.91	-4.50	3.00		3.00	0.000001	0.14	348.80	95.00	0.01
East	0	11.5ft Max	46.13	-4.50	3.00	-3.97	3.00	0.000001	0.14	348.75	95.00	0.01

Table C.48 East Reach of C-511 – HEC-RAS Output for Profile "Design" With Original Weir Structure

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
East	14710	Design	0.00	-2.50	4.88		4.88	0.000000	0.00	367.57	94.29	0.00
East	13710	Design	3.08	-2.50	4.88		4.88	0.000000	0.01	367.57	94.29	0.00
East	12710	Design	6.21	-2.50	4.88		4.88	0.000000	0.02	367.57	94.29	0.00
East	11820	Design	8.95	-2.50	4.88		4.88	0.000000	0.03	367.56	94.29	0.00
East	11775	Design	9.21	-3.50	4.88	-3.41	4.88	0.000000	0.01	692.11	120.29	0.00
East	11745		Inl Struct									
East	11700	Design	9.72	-10.00	4.21	-9.88	4.21	0.000000	0.01	882.23	128.05	0.00
East	11600	Design	10.92	-10.00	4.21		4.21	0.000000	0.01	1009.34	128.05	0.00
East	10745	Design	21.20	-10.00	4.21		4.21	0.000000	0.02	1009.34	128.05	0.00
East	9745	Design	33.15	-10.00	4.21		4.21	0.000000	0.03	1009.33	128.05	0.00
East	8745	Design	45.15	-10.00	4.21		4.21	0.000000	0.05	1009.32	128.05	0.00
East	7745	Design	57.14	-10.00	4.21		4.21	0.000000	0.06	1009.31	128.05	0.00
East	6745	Design	69.13	-10.00	4.21		4.21	0.000000	0.07	1009.29	128.05	0.00
East	5745	Design	81.13	-10.00	4.21		4.21	0.000000	0.08	1009.27	128.05	0.00
East	4745	Design	93.12	-10.00	4.21		4.21	0.000000	0.10	1009.24	128.05	0.01
East	3745	Design	105.11	-10.00	4.21		4.21	0.000000	0.11	1009.20	128.05	0.01
East	2745	Design	117.06	-10.00	4.21		4.21	0.000000	0.12	1009.14	128.04	0.01
East	1745	Design	129.05	-10.00	4.21		4.21	0.000000	0.13	1009.08	128.04	0.01
East	745	Design	141.05	-10.00	4.21		4.21	0.000001	0.14	1009.01	128.04	0.01
East	100	Design	148.80	-10.00	4.21		4.21	0.000001	0.15	1008.95	128.04	0.01
East	40	Design	150.00	-10.00	4.21	-9.25	4.21	0.000001	0.15	1008.95	128.04	0.01
East	0		Inl Struct									
East	-40	Design	150.00	-12.00	4.00	-11.19	4.00	0.000000	0.11	1328.00	131.00	0.01
East	-100	Design	150.00	-12.00	4.00	-11.19	4.00	0.000000	0.11	1328.00	131.00	0.01

Table C.49 East Reach of C-511 – HEC-RAS Output for Profile "10 ft Base x5" With Updated Weir Structure

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
East	14710	10 ft Base x5	0.00	-2.50	4.91		4.91	0.000000	0.00	369.85	94.44	0.00
East	13710	10 ft Base x5	3.62	-2.50	4.91		4.91	0.000000	0.01	369.85	94.44	0.00
East	12710	10 ft Base x5	7.24	-2.50	4.91		4.91	0.000000	0.02	369.85	94.44	0.00
East	11820	10 ft Base x5	10.47	-2.50	4.91		4.91	0.000000	0.03	369.84	94.44	0.00
East	11775	10 ft Base x5	10.74	-3.50	4.91	-3.41	4.91	0.000000	0.01	800.40	120.44	0.00
East	11745		Inl Struct									
East	11700	10 ft Base x5	11.37	-10.00	4.21	-9.87	4.21	0.000000	0.01	882.37	128.06	0.00
East	11600	10 ft Base x5	12.77	-10.00	4.21		4.21	0.000000	0.01	1009.58	128.06	0.00
East	10745	10 ft Base x5	24.73	-10.00	4.21		4.21	0.000000	0.03	1009.58	128.06	0.00
East	9745	10 ft Base x5	38.72	-10.00	4.21		4.21	0.000000	0.04	1009.58	128.06	0.00
East	8745	10 ft Base x5	52.72	-10.00	4.21		4.21	0.000000	0.05	1009.56	128.06	0.00
East	7745	10 ft Base x5	66.71	-10.00	4.21		4.21	0.000000	0.07	1009.55	128.06	0.00
East	6745	10 ft Base x5	80.70	-10.00	4.21		4.21	0.000000	0.08	1009.52	128.06	0.00
East	5745	10 ft Base x5	94.69	-10.00	4.21		4.21	0.000000	0.10	1009.49	128.06	0.01
East	4745	10 ft Base x5	108.69	-10.00	4.21		4.21	0.000000	0.11	1009.45	128.06	0.01
East	3745	10 ft Base x5	122.68	-10.00	4.21		4.21	0.000000	0.13	1009.39	128.05	0.01
East	2745	10 ft Base x5	136.67	-10.00	4.21		4.21	0.000001	0.14	1009.32	128.05	0.01
East	1745	10 ft Base x5	150.66	-10.00	4.21		4.21	0.000001	0.15	1009.24	128.05	0.01
East	745	10 ft Base x5	164.65	-10.00	4.21		4.21	0.000001	0.17	1009.14	128.04	0.01
East	100	10 ft Base x5	173.68	-10.00	4.21		4.21	0.000001	0.18	1009.06	128.04	0.01
East	40	10 ft Base x5	175.08	-10.00	4.21	-9.17	4.21	0.000001	0.18	1009.05	128.04	0.01
East	0		Inl Struct									
East	-40	10 ft Base x5	175.08	-12.00	4.00	-11.10	4.00	0.000000	0.13	1328.00	131.00	0.01
East	-100	10 ft Base x5	175.08	-12.00	4.00	-11.11	4.00	0.000000	0.13	1328.00	131.00	0.01

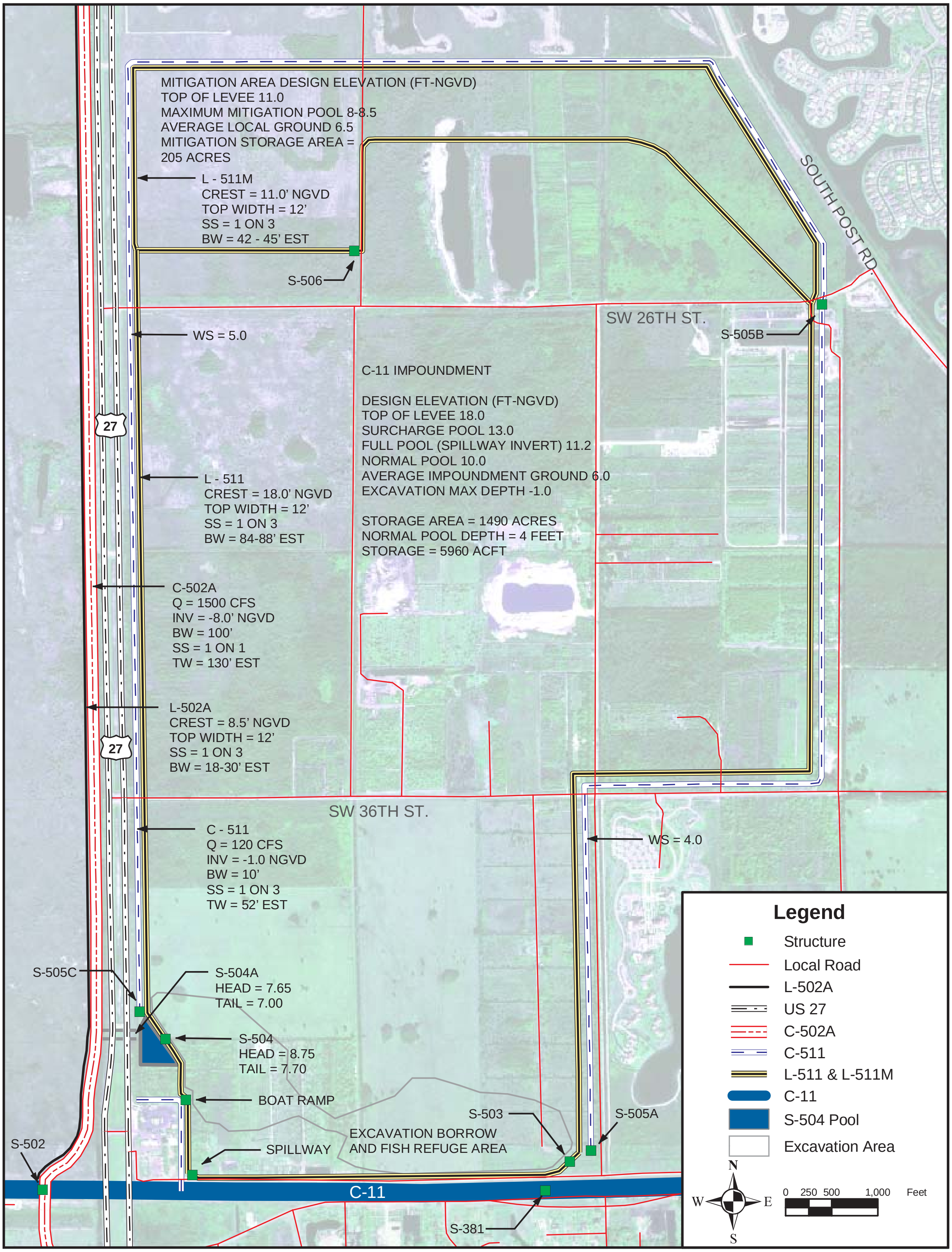

Figure C.1 C-11 Impoundment Structure Layout Map

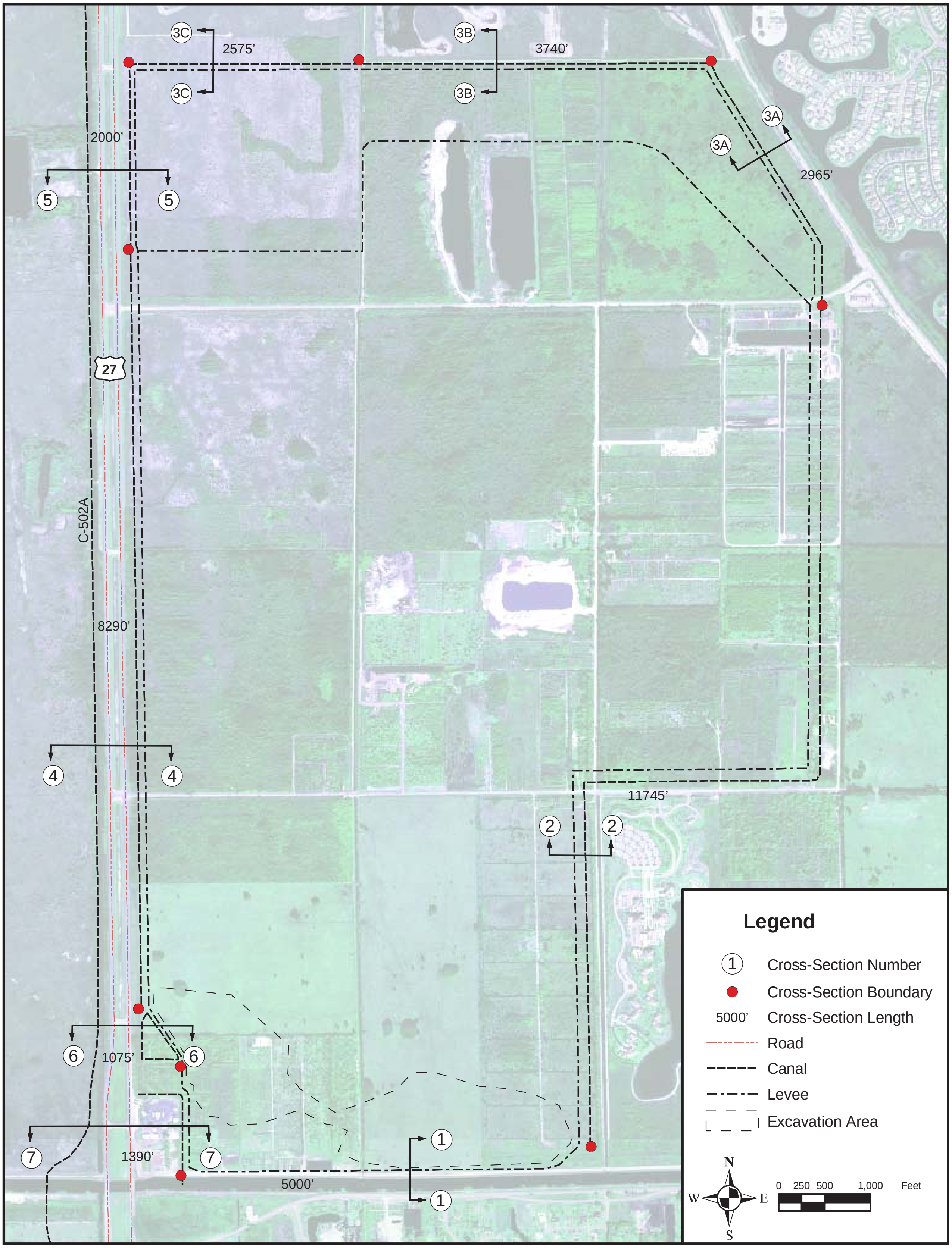

Figure C.2 C-11 Impoundment Cross Sections in SEEP2D Modeling

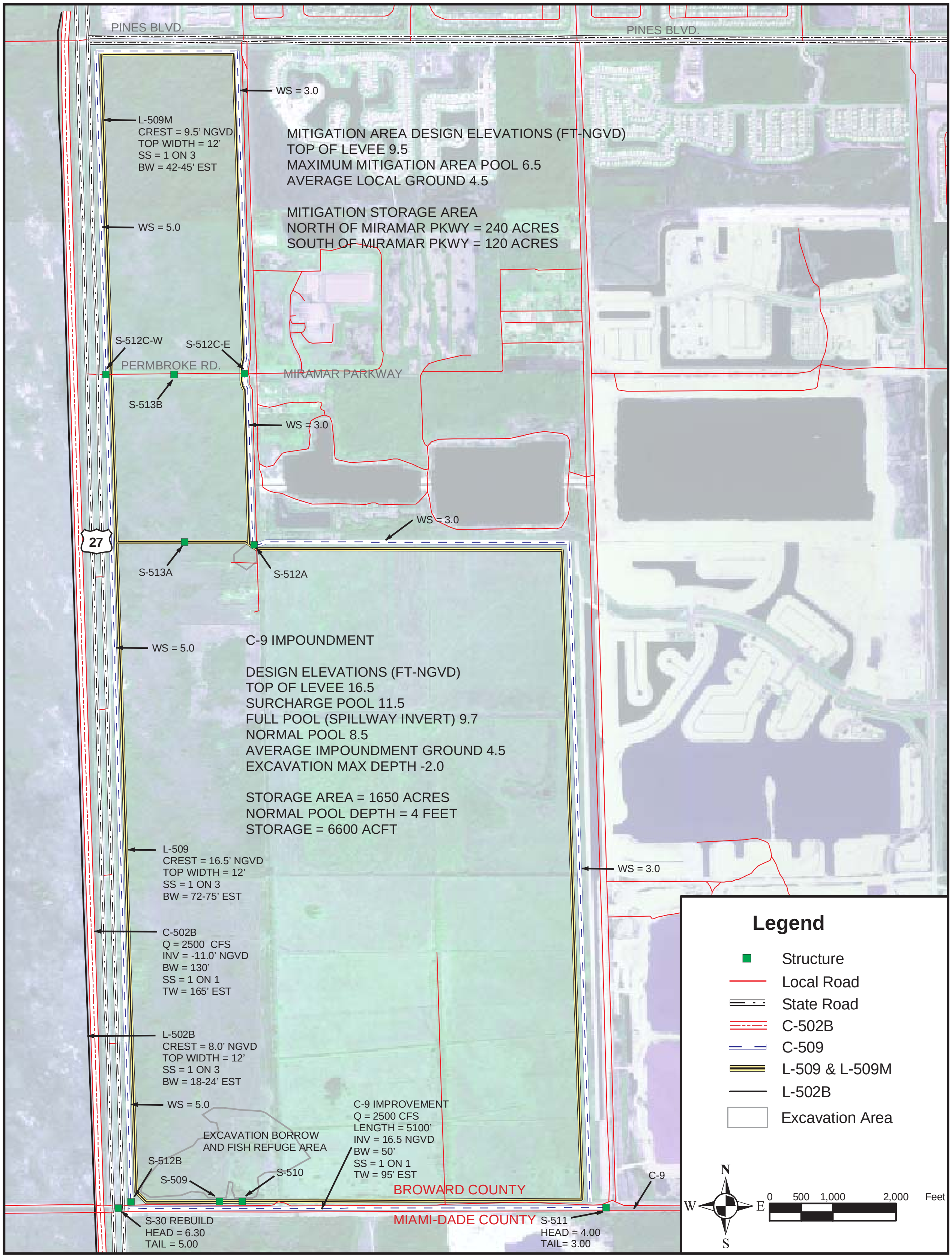

Figure C.3 C-9 Impoundment Structure Layout Map

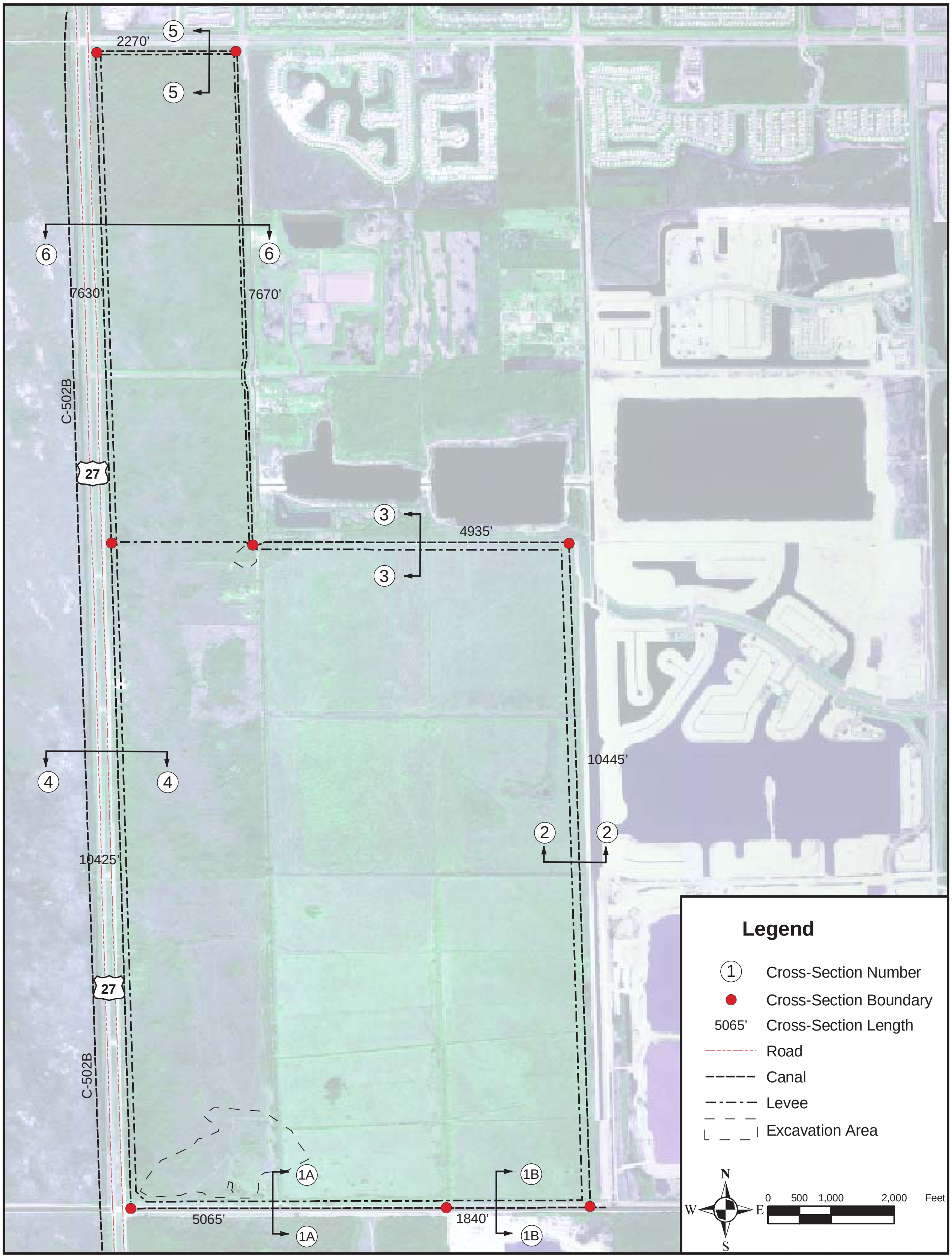

Figure C.4 C-9 Impoundment Cross Sections in SEEP2D Modeling

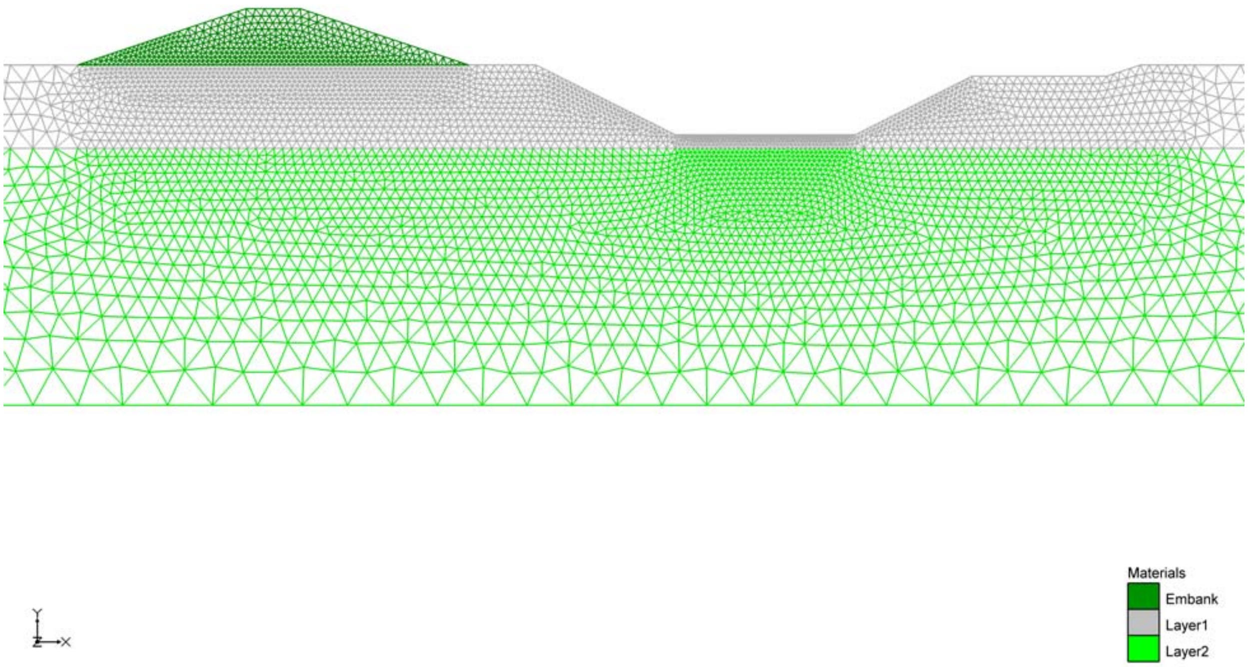

Figure C.5 Geometry Mesh Example in SEEP2D Modeling

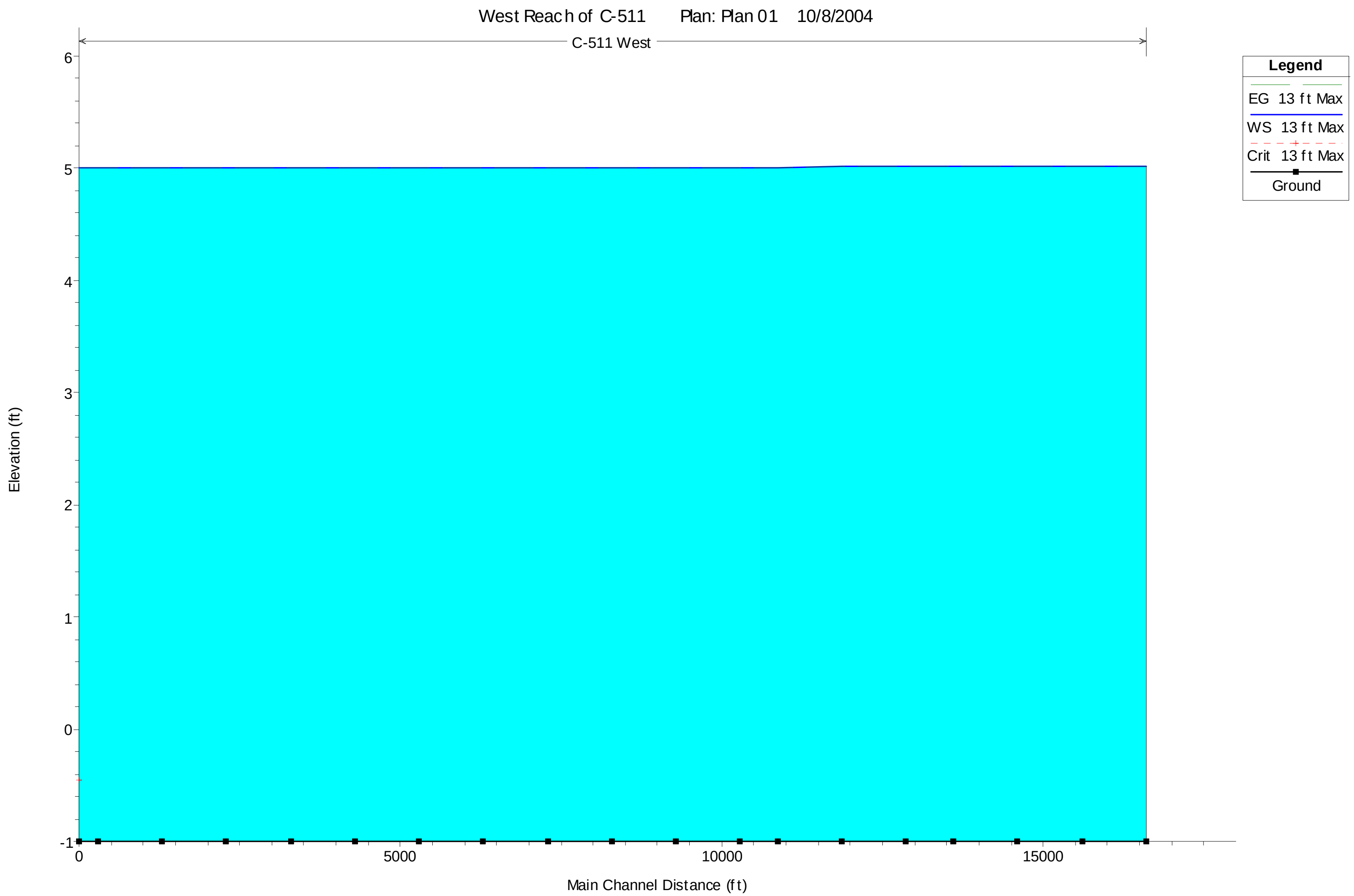

Figure C.6 West Reach of C-511 – HEC-RAS Water Surface Profile Plot

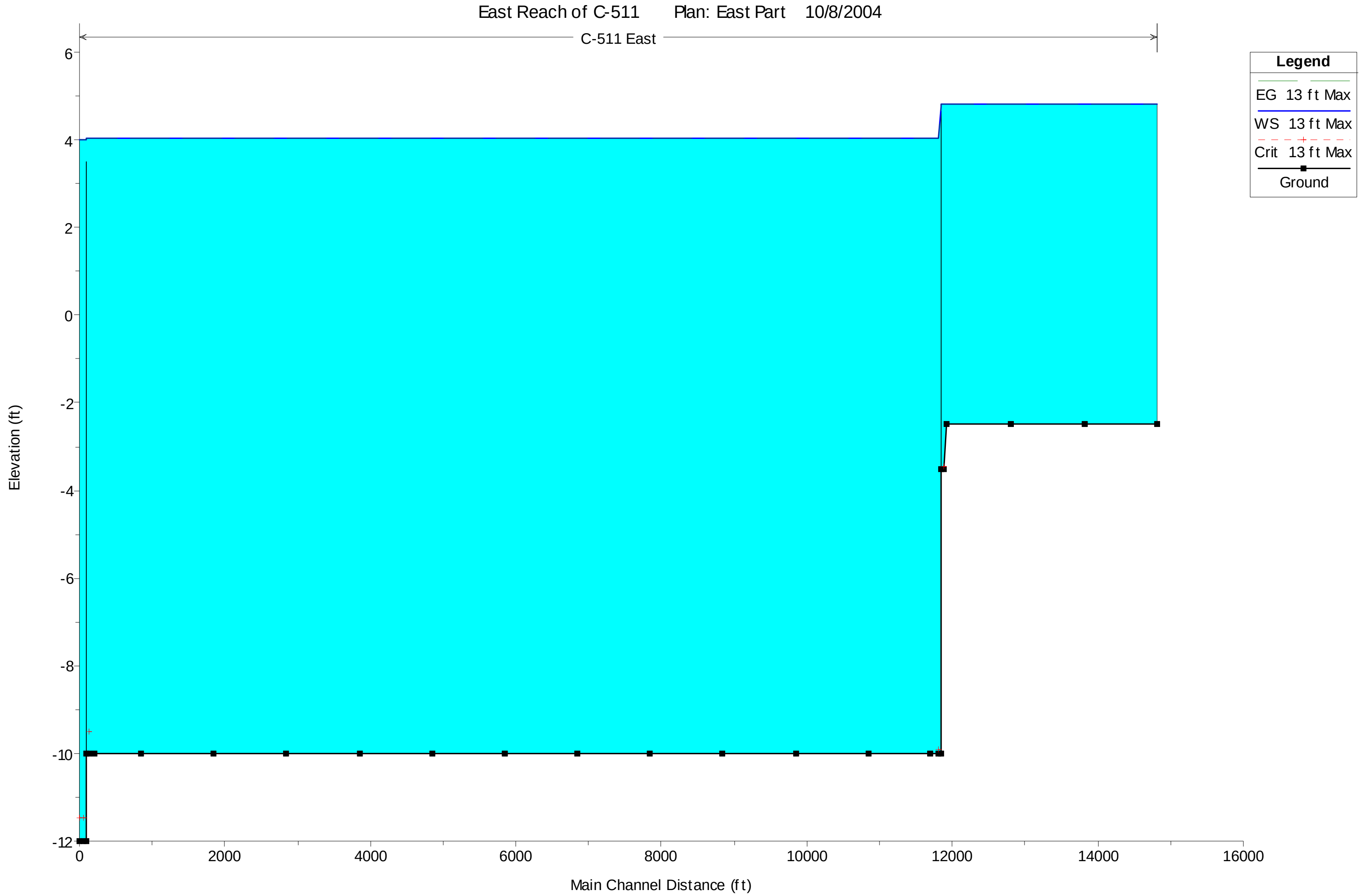

Figure C.7 East Reach of C-511 – HEC-RAS Water Surface Profile Plot

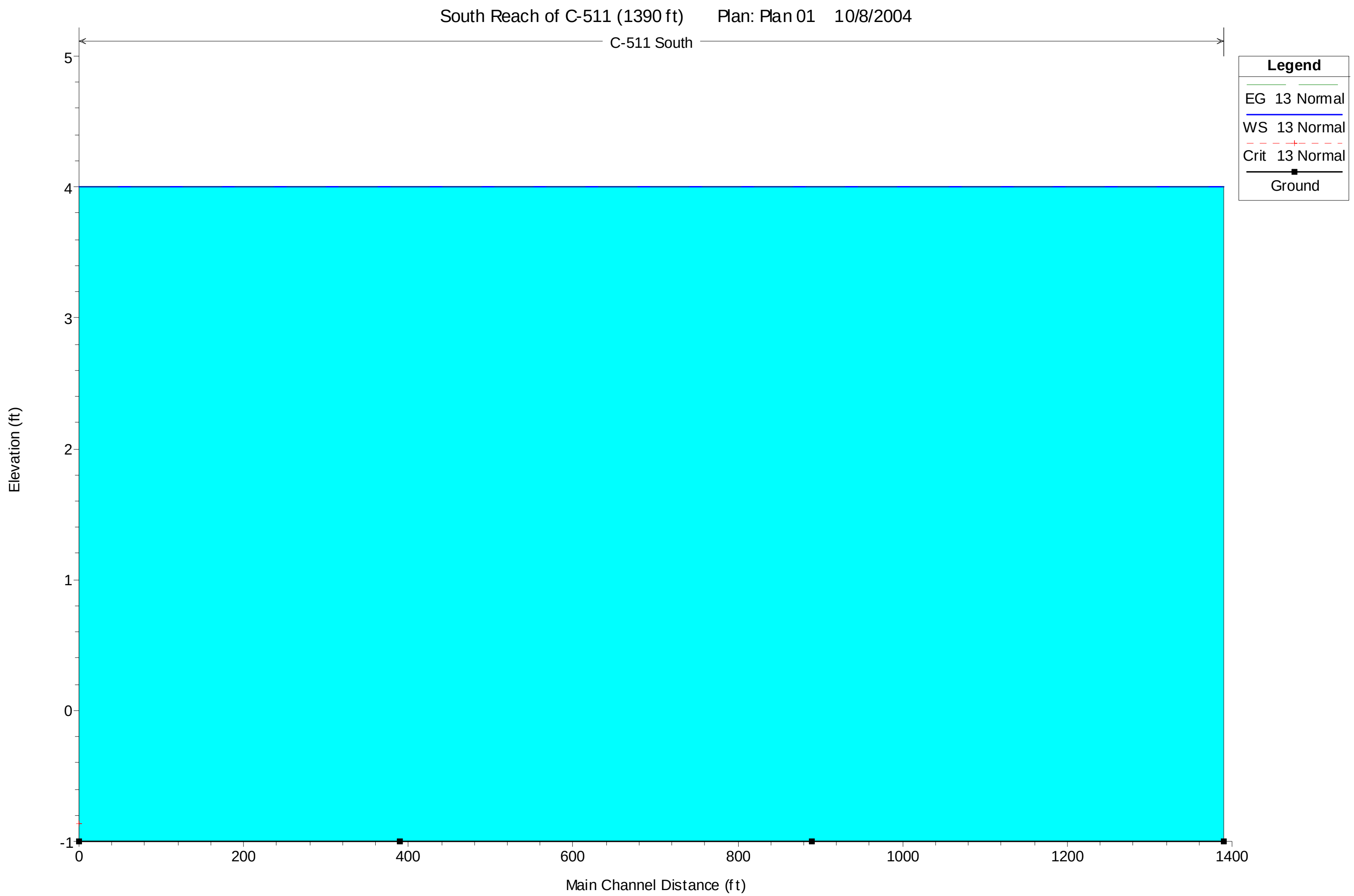

Figure C.8 South Reach of C-511 – HEC-RAS Water Surface Profile Plot

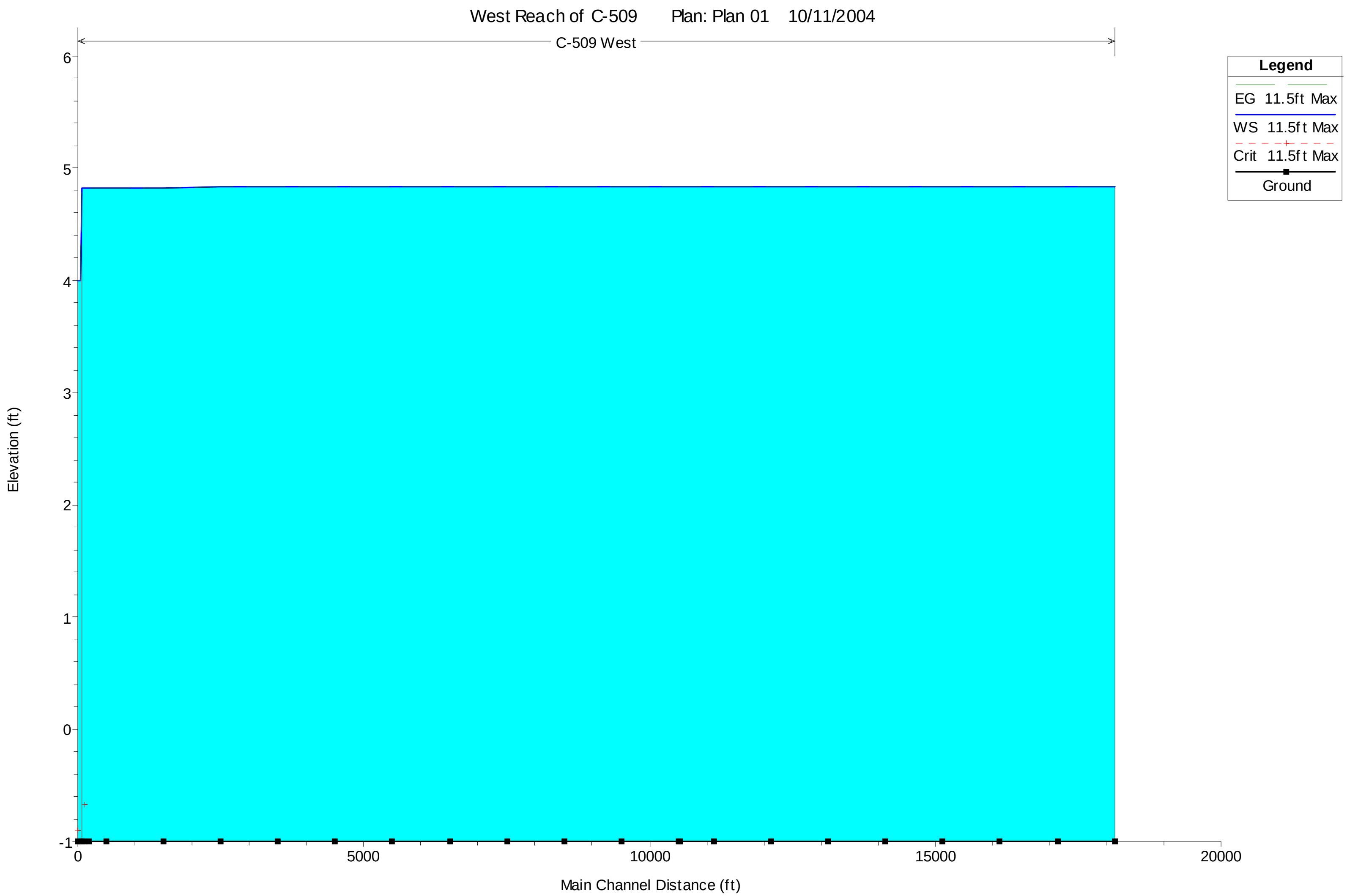

Figure C.9 West Reach of C-509 – HEC-RAS Water Surface Profile Plot

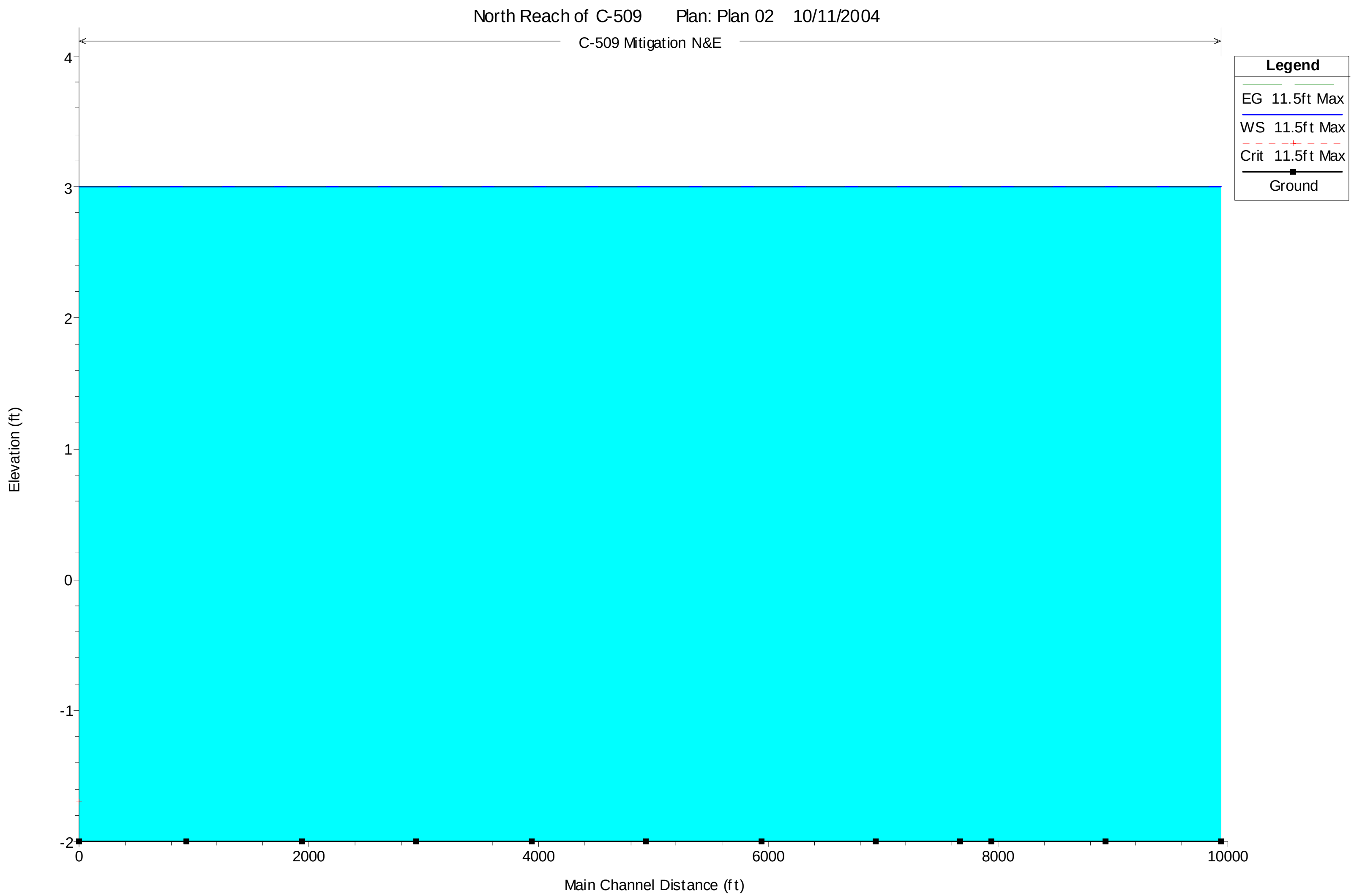

Figure C.10 North Reach of C-509 – HEC-RAS Water Surface Profile Plot

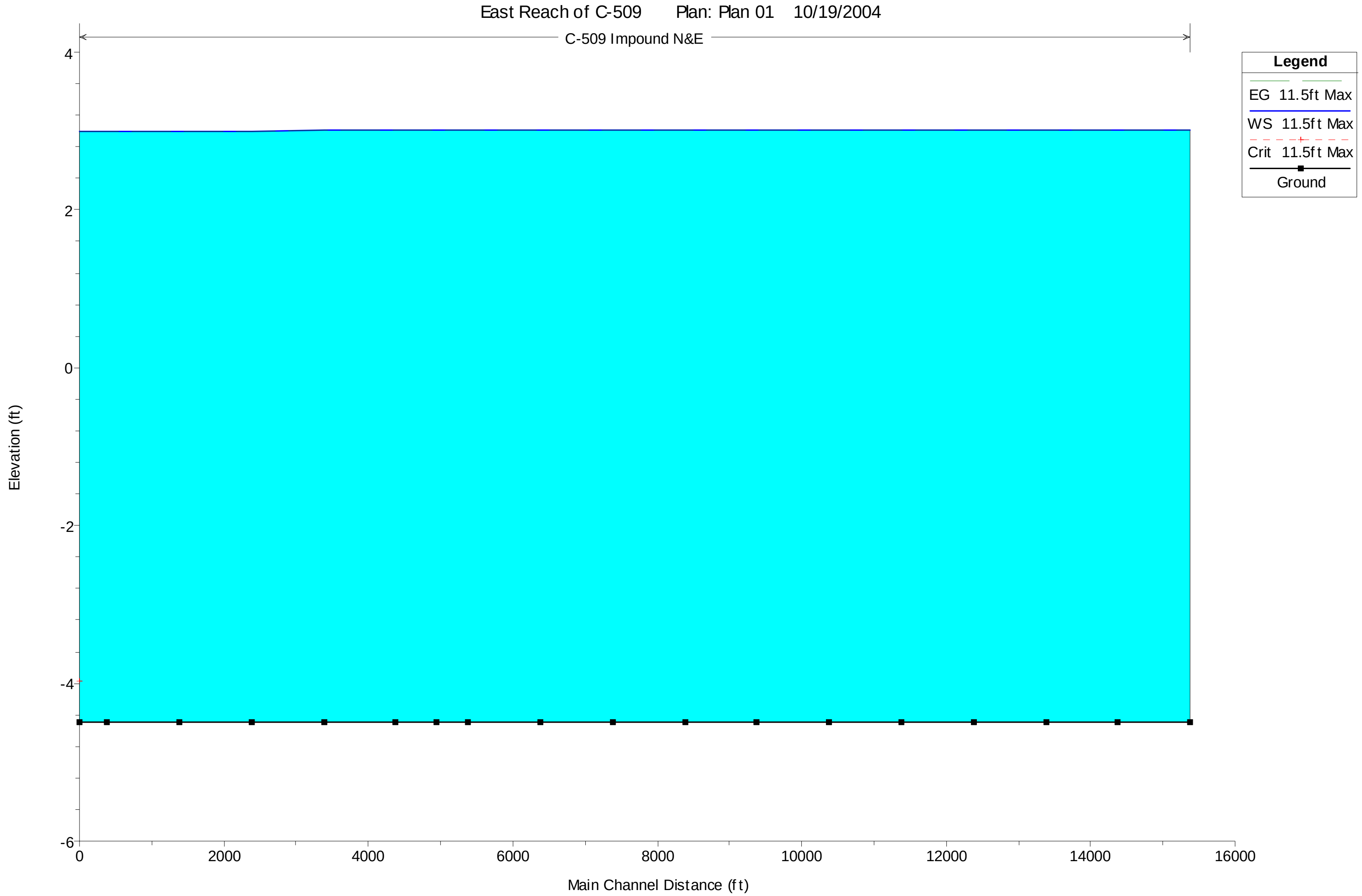

Figure C.11 East Reach of C-509 – HEC-RAS Water Surface Profile Plot

APPENDIX D

HEC-RAS MODEL OF C-11 WEST CANAL

Appendix D
HEC-RAS Model of C-11 West Canal
Table of Contents

List of Figures

List of Tables

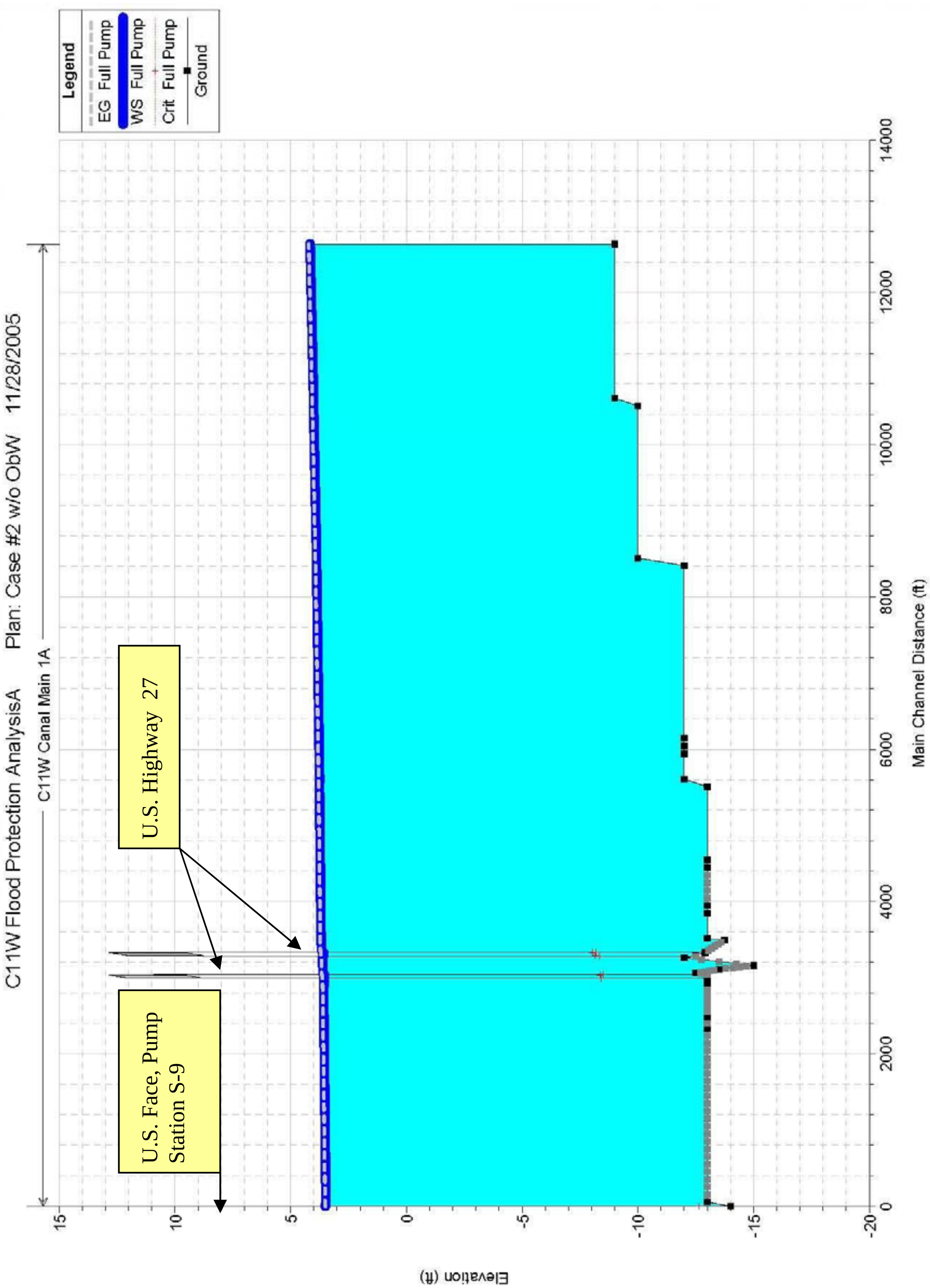

Figure D.1 Water Surface Profile, Base (December 2000) Condition, Q=2,880 cfs

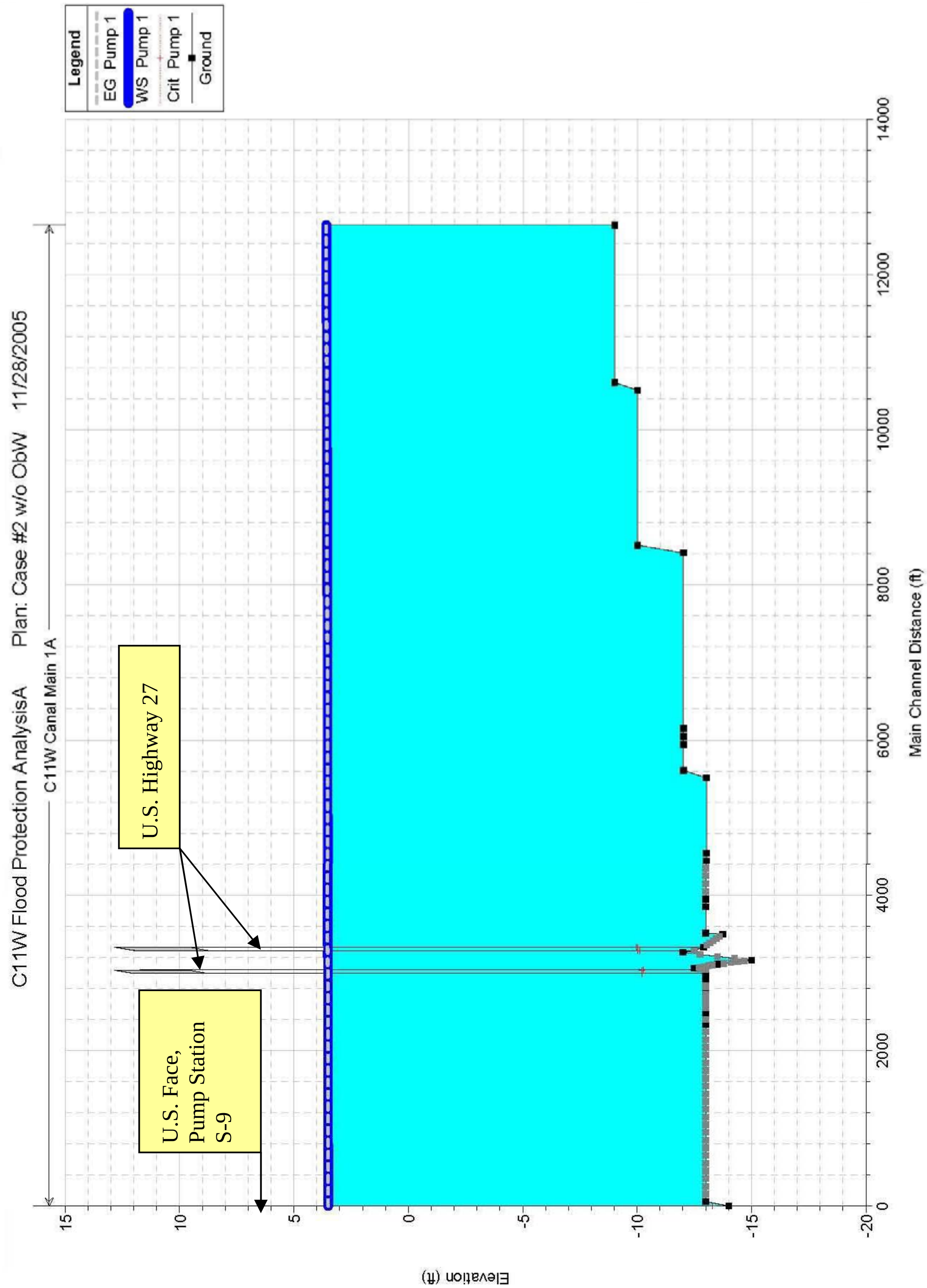

Figure D.2 Water Surface Profile, Base (December 2000) Condition, Q=1,050 cfs

Table D.1 Water Surface Profile Data, Base (December 2000) Condition

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	12582.44	Full Pump	2880.00	-9.00	4.16		4.22	0.000071	1.97	1464.54	137.63	0.11
Main 1A	12582.44	Pump 1	1050.00	-9.00	3.59		3.60	0.000011	0.76	1387.58	135.37	0.04
Main 1A	10555.00	Full Pump	2880.00	-9.00	4.01		4.07	0.000074	1.99	1443.98	137.03	0.11
Main 1A	10555.00	Pump 1	1050.00	-9.00	3.57		3.58	0.000011	0.76	1384.51	135.28	0.04
Main 1A	10455.00	Full Pump	2880.00	-10.00	4.00		4.06	0.000063	1.90	1512.56	136.02	0.10
Main 1A	10455.00	Pump 1	1050.00	-10.00	3.57		3.58	0.000009	0.72	1453.88	134.28	0.04
Main 1A	8455.00	Full Pump	2880.00	-10.00	3.87		3.93	0.000066	1.93	1494.85	135.49	0.10
Main 1A	8455.00	Pump 1	1050.00	-10.00	3.55		3.56	0.000009	0.72	1451.34	134.20	0.04
Main 1A	8355.00	Full Pump	2880.00	-12.00	3.88		3.92	0.000045	1.71	1688.93	137.75	0.09
Main 1A	8355.00	Pump 1	1050.00	-12.00	3.55		3.56	0.000006	0.64	1644.30	136.47	0.03
Main 1A	6094.38	Full Pump	2880.00	-12.00	3.77		3.82	0.000046	1.72	1674.64	137.34	0.09
Main 1A	6094.38	Pump 1	1050.00	-12.00	3.54		3.54	0.000006	0.64	1642.30	136.41	0.03
Main 1A	5990.53	Full Pump	2880.00	-12.00	3.77		3.81	0.000046	1.72	1673.98	137.32	0.09
Main 1A	5990.53	Pump 1	1050.00	-12.00	3.54		3.54	0.000006	0.64	1642.21	136.40	0.03
Main 1A	5890.53	Full Pump	2880.00	-12.00	3.76		3.81	0.000046	1.72	1673.34	137.30	0.09
Main 1A	5890.53	Pump 1	1050.00	-12.00	3.54		3.54	0.000006	0.64	1642.12	136.40	0.03
Main 1A	5555.00	Full Pump	2880.00	-12.00	3.75		3.79	0.000046	1.72	1671.19	137.24	0.09
Main 1A	5555.00	Pump 1	1050.00	-12.00	3.53		3.54	0.000006	0.64	1641.82	136.39	0.03
Main 1A	5455.00	Full Pump	2880.00	-13.00	3.75		3.79	0.000033	1.51	1901.33	147.01	0.07
Main 1A	5455.00	Pump 1	1050.00	-13.00	3.53		3.54	0.000005	0.56	1869.44	146.14	0.03
Main 1A	4490.53	Full Pump	2880.00	-13.00	3.72		3.76	0.000033	1.52	1896.61	146.88	0.07
Main 1A	4490.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.79	146.12	0.03
Main 1A	4390.53	Full Pump	2880.00	-13.00	3.72		3.75	0.000033	1.52	1896.12	146.86	0.07
Main 1A	4390.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.72	146.12	0.03
Main 1A	4290.53	Full Pump	2880.00	-13.00	3.71		3.75	0.000033	1.52	1895.62	146.85	0.07
Main 1A	4290.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.65	146.11	0.03
Main 1A	4190.53	Full Pump	2880.00	-13.00	3.71		3.75	0.000033	1.52	1895.13	146.84	0.07
Main 1A	4190.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.58	146.11	0.03
Main 1A	4090.53	Full Pump	2880.00	-13.00	3.71		3.74	0.000033	1.52	1894.64	146.82	0.07
Main 1A	4090.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.52	146.11	0.03
Main 1A	3990.53	Full Pump	2880.00	-13.00	3.70		3.74	0.000033	1.52	1894.15	146.81	0.07
Main 1A	3990.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.45	146.11	0.03
Main 1A	3890.53	Full Pump	2880.00	-13.00	3.70		3.74	0.000033	1.52	1893.65	146.80	0.07
Main 1A	3890.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.38	146.11	0.03
Main 1A	3790.53	Full Pump	2880.00	-13.00	3.70		3.73	0.000033	1.52	1893.16	146.78	0.07
Main 1A	3790.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.31	146.10	0.03
Main 1A	3455.00	Full Pump	2880.00	-13.00	3.68		3.72	0.000034	1.52	1891.50	146.74	0.07
Main 1A	3455.00	Pump 1	1050.00	-13.00	3.52		3.53	0.000005	0.56	1868.09	146.10	0.03
Main 1A	3440.22	Full Pump	2880.00	-13.74	3.68		3.72	0.000032	1.50	1914.35	146.74	0.07
Main 1A	3440.22	Pump 1	1050.00	-13.74	3.52		3.53	0.000004	0.56	1890.93	146.10	0.03
Main 1A	3412.68	Full Pump	2880.00	-13.60	3.68		3.72	0.000036	1.58	1826.57	140.76	0.08
Main 1A	3412.68	Pump 1	1050.00	-13.60	3.52		3.53	0.000005	0.58	1804.67	140.16	0.03

(Continued Next Page)

Table D.1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	3385.14	Full Pump	2880.00	-13.46	3.67		3.72	0.00004	1.65	1743.75	134.91	0.08
Main 1A	3385.14	Pump 1	1050.00	-13.46	3.52		3.53	0.000005	0.61	1723.36	134.35	0.03
Main 1A	3357.61	Full Pump	2880.00	-13.32	3.67		3.72	0.000044	1.73	1665.88	129.17	0.08
Main 1A	3357.61	Pump 1	1050.00	-13.32	3.52		3.53	0.000006	0.64	1646.98	128.67	0.03
Main 1A	3330.07	Full Pump	2880.00	-13.18	3.66		3.71	0.000049	1.81	1592.97	123.62	0.09
Main 1A	3330.07	Pump 1	1050.00	-13.18	3.52		3.53	0.000007	0.67	1575.52	123.16	0.03
Main 1A	3302.53	Full Pump	2880.00	-13.04	3.66		3.71	0.000055	1.89	1524.68	118.28	0.09
Main 1A	3302.53	Pump 1	1050.00	-13.04	3.52		3.53	0.000007	0.70	1508.65	117.87	0.03
Main 1A	3275.00	Full Pump	2880.00	-12.90	3.60	-8.20	3.70	0.000075	2.66	1084.08	113.03	0.12
Main 1A	3275.00	Pump 1	1050.00	-12.90	3.51	-10.04	3.53	0.00001	0.97	1078.33	112.81	0.04
Main 1A	3273.38		Bridge									
Main 1A	3224.28	Full Pump	2880.00	-12.50	3.59	-8.33	3.69	0.000069	2.56	1125.42	118.60	0.11
Main 1A	3224.28	Pump 1	1050.00	-12.50	3.51	-10.13	3.53	0.000009	0.94	1120.04	118.33	0.04
Main 1A	3205.00	Full Pump	2880.00	-12.00	3.62		3.67	0.000049	1.81	1589.08	121.56	0.09
Main 1A	3205.00	Pump 1	1050.00	-12.00	3.52		3.52	0.000007	0.67	1576.43	121.07	0.03
Main 1A	3180.00	Full Pump	2880.00	-12.75	3.62		3.67	0.000047	1.80	1598.06	119.41	0.09
Main 1A	3180.00	Pump 1	1050.00	-12.75	3.52		3.52	0.000006	0.66	1585.71	119.26	0.03
Main 1A	3155.00	Full Pump	2880.00	-13.50	3.62		3.67	0.000046	1.80	1603.57	117.82	0.09
Main 1A	3155.00	Pump 1	1050.00	-13.50	3.52		3.52	0.000006	0.66	1591.47	117.68	0.03
Main 1A	3130.00	Full Pump	2880.00	-14.25	3.62		3.67	0.000046	1.79	1605.52	116.39	0.09
Main 1A	3130.00	Pump 1	1050.00	-14.25	3.52		3.52	0.000006	0.66	1593.68	116.27	0.03
Main 1A	3105.00	Full Pump	2880.00	-15.00	3.62		3.67	0.000046	1.80	1604.12	115.13	0.08
Main 1A	3105.00	Pump 1	1050.00	-15.00	3.52		3.52	0.000006	0.66	1592.52	115.02	0.03
Main 1A	3095.00	Full Pump	2880.00	-14.70	3.61		3.67	0.000049	1.84	1564.21	114.47	0.09
Main 1A	3095.00	Pump 1	1050.00	-14.70	3.51		3.52	0.000007	0.68	1553.02	114.27	0.03
Main 1A	3085.00	Full Pump	2880.00	-14.40	3.61		3.66	0.000052	1.89	1526.24	113.60	0.09
Main 1A	3085.00	Pump 1	1050.00	-14.40	3.51		3.52	0.000007	0.69	1515.47	113.38	0.03
Main 1A	3075.00	Full Pump	2880.00	-14.10	3.61		3.66	0.000055	1.93	1490.04	112.56	0.09
Main 1A	3075.00	Pump 1	1050.00	-14.10	3.51		3.52	0.000007	0.71	1479.71	112.33	0.03
Main 1A	3065.00	Full Pump	2880.00	-13.80	3.60		3.66	0.000058	1.98	1455.87	111.31	0.10
Main 1A	3065.00	Pump 1	1050.00	-13.80	3.51		3.52	0.000008	0.73	1446.00	111.07	0.04
Main 1A	3055.00	Full Pump	2880.00	-13.50	3.60		3.66	0.000061	2.02	1424.20	109.80	0.10
Main 1A	3055.00	Pump 1	1050.00	-13.50	3.51		3.52	0.000008	0.74	1414.81	109.54	0.04
Main 1A	3045.00	Full Pump	2880.00	-13.30	3.60		3.66	0.000062	2.01	1431.09	110.68	0.10
Main 1A	3045.00	Pump 1	1050.00	-13.30	3.51		3.52	0.000008	0.74	1421.64	110.43	0.04
Main 1A	3035.00	Full Pump	2880.00	-13.10	3.60		3.66	0.000061	2.00	1441.58	111.73	0.10
Main 1A	3035.00	Pump 1	1050.00	-13.10	3.51		3.52	0.000008	0.73	1432.04	111.49	0.04
Main 1A	3025.00	Full Pump	2880.00	-12.90	3.60		3.66	0.000061	1.98	1455.43	113.04	0.10
Main 1A	3025.00	Pump 1	1050.00	-12.90	3.51		3.52	0.000008	0.73	1445.76	112.81	0.04
Main 1A	3015.00	Full Pump	2880.00	-12.70	3.60		3.66	0.000061	1.96	1471.92	114.74	0.10
Main 1A	3015.00	Pump 1	1050.00	-12.70	3.51		3.52	0.000008	0.72	1462.07	114.53	0.04

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Table D.1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	3005.00	Full Pump	2880.00	-12.50	3.60		3.66	0.000061	1.93	1490.15	117.21	0.10
Main 1A	3005.00	Pump 1	1050.00	-12.50	3.51		3.52	0.000008	0.71	1480.04	117.03	0.04
Main 1A	2995.64	Full Pump	2880.00	-12.75	3.60		3.66	0.000061	1.96	1467.47	114.27	0.10
Main 1A	2995.64	Pump 1	1050.00	-12.75	3.51		3.52	0.000008	0.72	1457.87	114.06	0.04
Main 1A	2986.29	Full Pump	2880.00	-13.00	3.55	-8.49	3.65	0.000066	2.52	1144.23	112.22	0.11
Main 1A	2986.29	Pump 1	1050.00	-13.00	3.51	-10.28	3.52	0.000009	0.92	1140.82	112.09	0.04
Main 1A	2977.38		Bridge									
Main 1A	2934.53	Full Pump	2880.00	-13.00	3.57		3.63	0.000062	1.99	1444.46	112.25	0.10
Main 1A	2934.53	Pump 1	1050.00	-13.00	3.51		3.52	0.000008	0.73	1438.03	112.09	0.04
Main 1A	2881.45	Full Pump	2880.00	-13.00	3.56		3.62	0.000064	2.05	1404.83	109.68	0.10
Main 1A	2881.45	Pump 1	1050.00	-13.00	3.51		3.52	0.000009	0.75	1399.23	109.52	0.04
Main 1A	2855.00	Full Pump	2880.00	-13.00	3.56		3.62	0.000064	2.05	1404.64	109.67	0.10
Main 1A	2855.00	Pump 1	1050.00	-13.00	3.51		3.52	0.000009	0.75	1399.21	109.52	0.04
Main 1A	2790.22	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.35	146.30	0.08
Main 1A	2790.22	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.94	146.04	0.03
Main 1A	2781.41	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.30	146.30	0.08
Main 1A	2781.41	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.94	146.04	0.03
Main 1A	2772.61	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.26	146.29	0.08
Main 1A	2772.61	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.93	146.04	0.03
Main 1A	2763.80	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.21	146.29	0.08
Main 1A	2763.80	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.92	146.04	0.03
Main 1A	2755.00	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.17	146.29	0.08
Main 1A	2755.00	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.92	146.04	0.03
Main 1A	2668.45	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1874.73	146.28	0.08
Main 1A	2668.45	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.86	146.04	0.03
Main 1A	2581.91	Full Pump	2880.00	-13.00	3.57		3.60	0.000034	1.54	1874.29	146.27	0.08
Main 1A	2581.91	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.80	146.04	0.03
Main 1A	2495.36	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.86	146.26	0.08
Main 1A	2495.36	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.74	146.03	0.03
Main 1A	2408.82	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.42	146.24	0.08
Main 1A	2408.82	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.68	146.03	0.03
Main 1A	2345.59	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.10	146.24	0.08
Main 1A	2345.59	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.64	146.03	0.03
Main 1A	2282.37	Full Pump	2880.00	-13.00	3.56		3.59	0.000034	1.54	1872.78	146.23	0.08
Main 1A	2282.37	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.60	146.03	0.03
Main 1A	2183.13	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1872.27	146.21	0.08
Main 1A	2183.13	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.53	146.03	0.03
Main 1A	2083.90	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1871.77	146.20	0.08
Main 1A	2083.90	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.46	146.03	0.03
Main 1A	1984.67	Full Pump	2880.00	-13.00	3.55		3.58	0.000035	1.54	1871.26	146.19	0.08
Main 1A	1984.67	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.40	146.02	0.03
Main 1A	1885.43	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.76	146.17	0.08
Main 1A	1885.43	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.33	146.02	0.03

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Table D.1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	1786.20	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.25	146.16	0.08
Main 1A	1786.20	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.26	146.02	0.03
Main 1A	1686.96	Full Pump	2880.00	-13.00	3.54		3.57	0.000035	1.54	1869.75	146.14	0.08
Main 1A	1686.96	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.19	146.02	0.03
Main 1A	1587.73	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1869.24	146.13	0.08
Main 1A	1587.73	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.13	146.02	0.03
Main 1A	1488.50	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1868.74	146.12	0.08
Main 1A	1488.50	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.06	146.02	0.03
Main 1A	1389.26	Full Pump	2880.00	-13.00	3.53		3.56	0.000035	1.54	1868.23	146.10	0.08
Main 1A	1389.26	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.99	146.01	0.03
Main 1A	1290.03	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.72	146.09	0.08
Main 1A	1290.03	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.93	146.01	0.03
Main 1A	1190.80	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.22	146.07	0.08
Main 1A	1190.80	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.86	146.01	0.03
Main 1A	1091.56	Full Pump	2880.00	-13.00	3.52		3.55	0.000035	1.54	1866.71	146.06	0.08
Main 1A	1091.56	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.79	146.01	0.03
Main 1A	992.33	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1866.20	146.05	0.08
Main 1A	992.33	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.72	146.01	0.03
Main 1A	893.10	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1865.69	146.03	0.08
Main 1A	893.10	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.66	146.00	0.03
Main 1A	793.87	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1865.18	146.02	0.08
Main 1A	793.87	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.59	146.00	0.03
Main 1A	694.63	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1864.68	146.00	0.08
Main 1A	694.63	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.52	146.00	0.03
Main 1A	595.40	Full Pump	2880.00	-13.00	3.50		3.53	0.000035	1.54	1864.17	145.99	0.08
Main 1A	595.40	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.45	146.00	0.03
Main 1A	496.17	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.66	145.98	0.08
Main 1A	496.17	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.39	146.00	0.03
Main 1A	396.93	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.15	145.96	0.08
Main 1A	396.93	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.32	146.00	0.03
Main 1A	297.70	Full Pump	2880.00	-13.00	3.49		3.52	0.000035	1.55	1862.64	145.95	0.08
Main 1A	297.70	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.25	145.99	0.03
Main 1A	198.47	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1862.13	145.93	0.08
Main 1A	198.47	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.19	145.99	0.03
Main 1A	99.23	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1861.62	145.92	0.08
Main 1A	99.23	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.12	145.99	0.03
Main 1A	0.00	Full Pump	2880.00	-13.00	3.48		3.51	0.000035	1.55	1861.11	145.91	0.08
Main 1A	0.00	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.05	145.99	0.03
Main 1A	-55.00	Full Pump	2880.00	-14.00	3.50	-12.65	3.50	0.000002	0.47	6066.03	373.26	0.02
Main 1A	-55.00	Pump 1	1050.00	-14.00	3.50	-13.31	3.50	0	0.17	6066.03	373.26	0.01

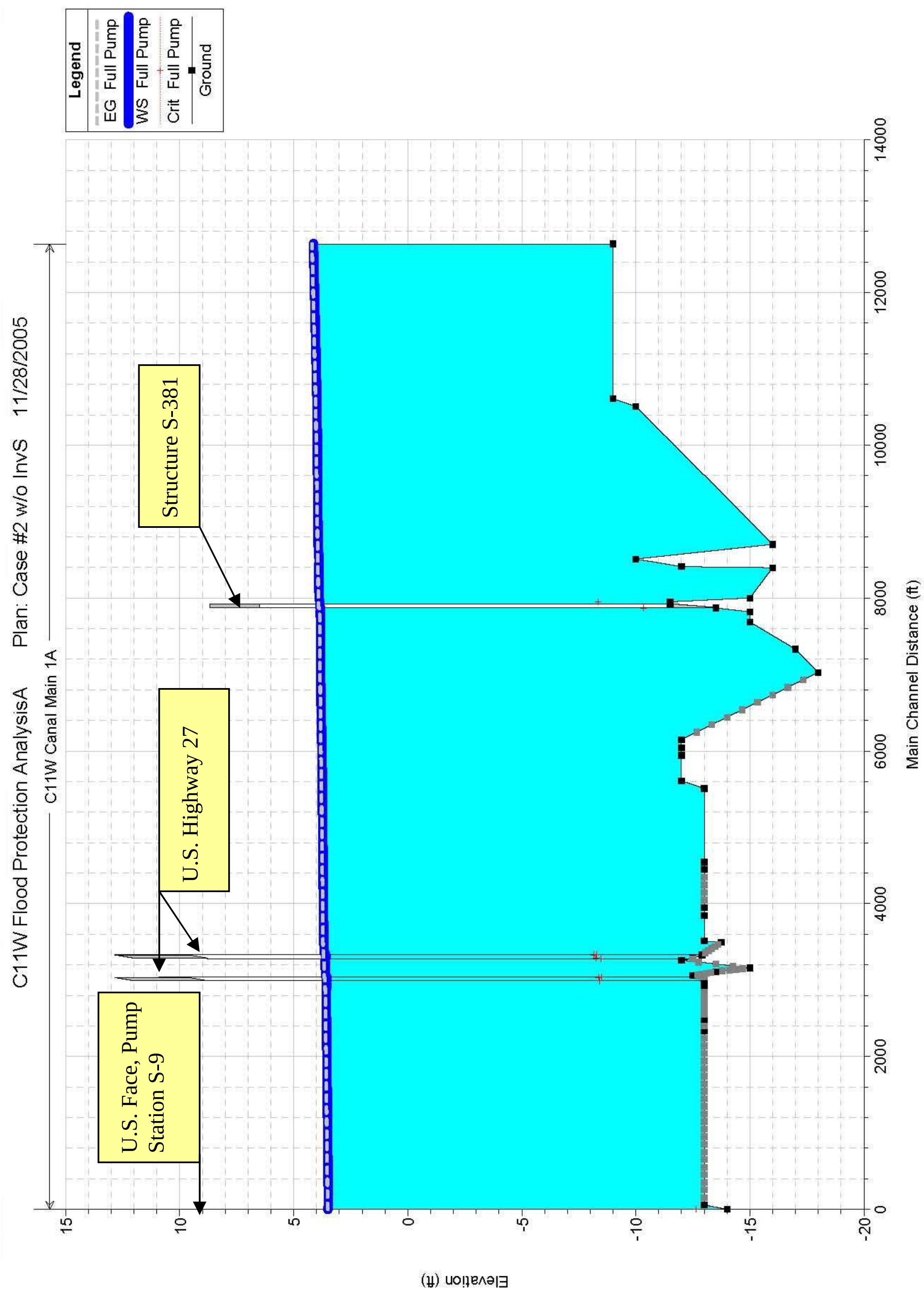

Figure D.3 Water Surface Profile with S-381, Q=2,880 cfs

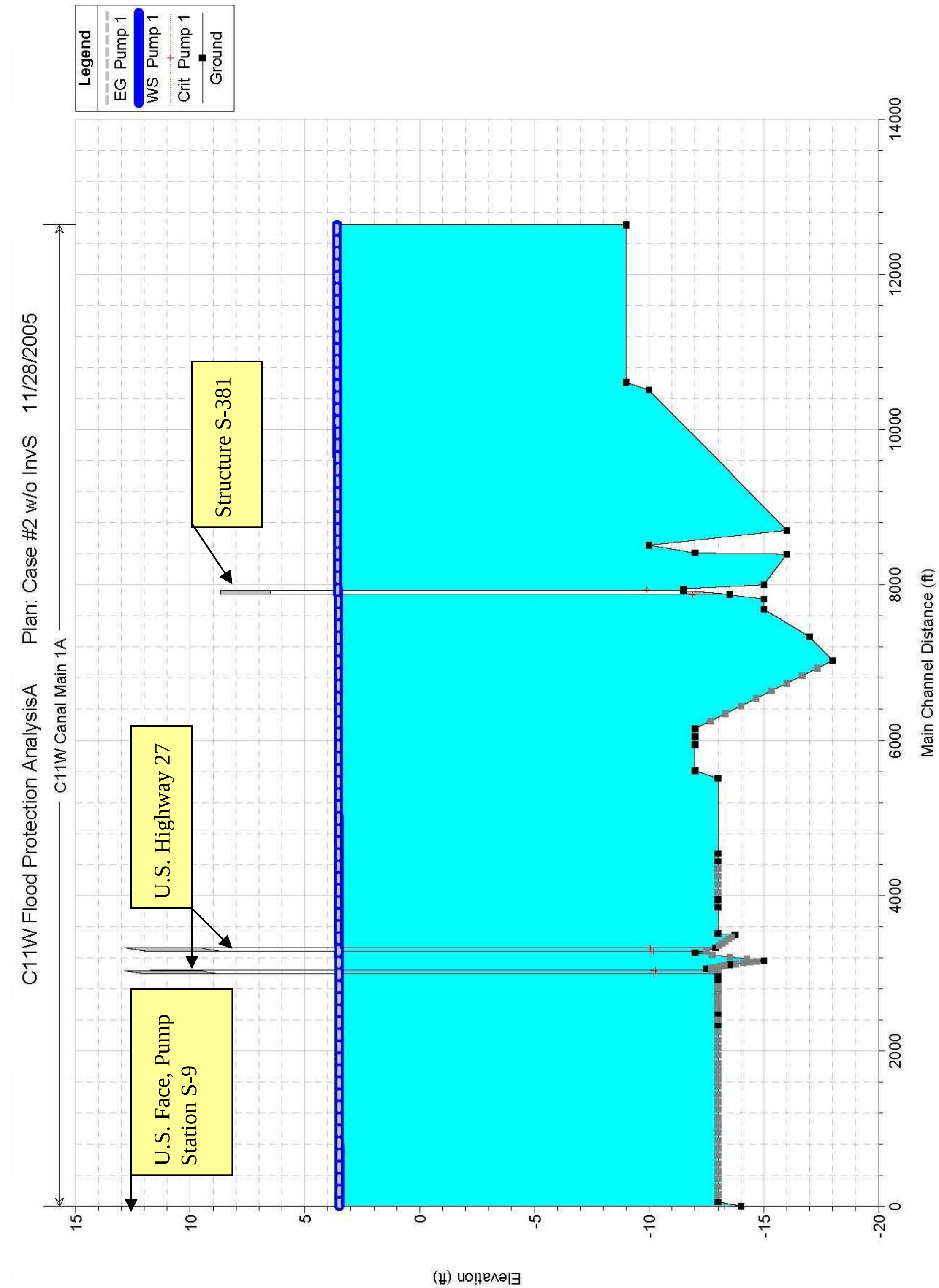

Figure D.4 Water Surface Profile with S-381, Q=1,050 cfs

Table D.2 Water Surface Profile Data with S-381

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	12582.44	Full Pump	2880.00	-9.00	4.13		4.19	0.000072	1.97	1460.92	137.52	0.11
Main 1A	12582.44	Pump 1	1050.00	-9.00	3.60		3.61	0.000011	0.76	1388.95	135.41	0.04
Main 1A	10555.00	Full Pump	2880.00	-9.00	3.98		4.04	0.000075	2.00	1440.22	136.92	0.11
Main 1A	10555.00	Pump 1	1050.00	-9.00	3.58		3.59	0.000011	0.76	1385.89	135.32	0.04
Main 1A	10455.00	Full Pump	2880.00	-10.00	3.98		4.03	0.000064	1.91	1508.83	135.91	0.10
Main 1A	10455.00	Pump 1	1050.00	-10.00	3.58		3.59	0.000009	0.72	1455.25	134.32	0.04
Main 1A	8647.21	Full Pump	2880.00	-16.00	3.93		3.96	0.000026	1.40	2057.00	147.47	0.07
Main 1A	8647.21	Pump 1	1050.00	-16.00	3.57		3.58	0.000004	0.52	2005.26	145.90	0.02
Main 1A	8455.00	Full Pump	2880.00	-10.00	3.89		3.95	0.000065	1.92	1496.83	135.55	0.10
Main 1A	8455.00	Pump 1	1050.00	-10.00	3.57		3.58	0.000009	0.72	1453.55	134.27	0.04
Main 1A	8355.00	Full Pump	2880.00	-12.00	3.89		3.94	0.000045	1.70	1690.95	137.81	0.09
Main 1A	8355.00	Pump 1	1050.00	-12.00	3.57		3.57	0.000006	0.64	1646.55	136.53	0.03
Main 1A	8340.96	Full Pump	2880.00	-16.00	3.90		3.93	0.000026	1.40	2053.44	147.36	0.07
Main 1A	8340.96	Pump 1	1050.00	-16.00	3.57		3.57	0.000004	0.52	2004.76	145.89	0.02
Main 1A	7945.50	Full Pump	2880.00	-15.00	3.89		3.92	0.000031	1.49	1939.11	146.35	0.07
Main 1A	7945.50	Pump 1	1050.00	-15.00	3.57		3.57	0.000004	0.55	1892.68	144.87	0.03
Main 1A	7895.00	Full Pump	2880.00	-11.50	3.85	-8.34	3.91	0.000093	2.09	1380.61	90.00	0.09
Main 1A	7895.00	Pump 1	1050.00	-11.50	3.56	-9.89	3.57	0.000013	0.77	1354.96	90.00	0.04
Main 1A			Inl Struct									
Main 1A	7815.00	Full Pump	2880.00	-13.50	3.82	-10.33	3.87	0.000086	1.85	1558.64	90.00	0.08
Main 1A	7815.00	Pump 1	1050.00	-13.50	3.54	-11.88	3.55	0.000012	0.68	1533.89	90.00	0.03
Main 1A	7763.30	Full Pump	2880.00	-15.00	3.83		3.86	0.000031	1.49	1930.71	146.08	0.07
Main 1A	7763.30	Pump 1	1050.00	-15.00	3.54		3.55	0.000004	0.56	1889.41	144.77	0.03
Main 1A	7631.08	Full Pump	2880.00	-15.00	3.82		3.86	0.000031	1.49	1930.10	146.06	0.07
Main 1A	7631.08	Pump 1	1050.00	-15.00	3.54		3.55	0.000004	0.56	1889.32	144.77	0.03
Main 1A	7276.48	Full Pump	2880.00	-17.00	3.82		3.85	0.000022	1.34	2153.67	147.89	0.06
Main 1A	7276.48	Pump 1	1050.00	-17.00	3.54		3.55	0.000003	0.50	2112.93	146.70	0.02
Main 1A	6971.88	Full Pump	2880.00	-18.00	3.82		3.84	0.000019	1.27	2265.40	148.69	0.06
Main 1A	6971.88	Pump 1	1050.00	-18.00	3.54		3.55	0.000003	0.47	2224.99	147.56	0.02
Main 1A	6874.38	Full Pump	2880.00	-17.33	3.81		3.84	0.000021	1.31	2200.95	147.43	0.06
Main 1A	6874.38	Pump 1	1050.00	-17.33	3.54		3.55	0.000003	0.49	2161.34	146.32	0.02
Main 1A	6776.88	Full Pump	2880.00	-16.67	3.81		3.84	0.000023	1.35	2136.81	146.13	0.06
Main 1A	6776.88	Pump 1	1050.00	-16.67	3.54		3.55	0.000003	0.50	2098.03	145.04	0.02
Main 1A	6679.38	Full Pump	2880.00	-16.00	3.80		3.83	0.000025	1.39	2071.38	144.85	0.06
Main 1A	6679.38	Pump 1	1050.00	-16.00	3.54		3.55	0.000003	0.52	2033.47	143.78	0.02
Main 1A	6581.88	Full Pump	2880.00	-15.33	3.80		3.83	0.000027	1.44	2005.91	143.60	0.07
Main 1A	6581.88	Pump 1	1050.00	-15.33	3.54		3.54	0.000004	0.53	1968.90	142.56	0.03
Main 1A	6484.38	Full Pump	2880.00	-14.67	3.79		3.83	0.00003	1.48	1940.53	142.32	0.07
Main 1A	6484.38	Pump 1	1050.00	-14.67	3.54		3.54	0.000004	0.55	1904.49	141.29	0.03
Main 1A	6386.88	Full Pump	2880.00	-14.00	3.79		3.82	0.000033	1.54	1874.32	141.08	0.07
Main 1A	6386.88	Pump 1	1050.00	-14.00	3.54		3.54	0.000005	0.57	1839.28	140.08	0.03

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Table D.2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	6289.38	Full Pump	2880.00	-13.33	3.78		3.82	0.000037	1.59	1807.44	139.81	0.08
Main 1A	6289.38	Pump 1	1050.00	-13.33	3.54		3.54	0.000005	0.59	1773.48	138.84	0.03
Main 1A	6191.38	Full Pump	2880.00	-12.67	3.77		3.82	0.000041	1.65	1741.44	138.57	0.08
Main 1A	6191.38	Pump 1	1050.00	-12.67	3.54		3.54	0.000006	0.61	1708.62	137.63	0.03
Main 1A	6094.38	Full Pump	2880.00	-12.00	3.77		3.81	0.000046	1.72	1673.75	137.32	0.09
Main 1A	6094.38	Pump 1	1050.00	-12.00	3.54		3.54	0.000006	0.64	1642.18	136.40	0.03
Main 1A	5990.53	Full Pump	2880.00	-12.00	3.76		3.81	0.000046	1.72	1673.09	137.30	0.09
Main 1A	5990.53	Pump 1	1050.00	-12.00	3.54		3.54	0.000006	0.64	1642.09	136.40	0.03
Main 1A	5890.53	Full Pump	2880.00	-12.00	3.76		3.80	0.000046	1.72	1672.45	137.28	0.09
Main 1A	5890.53	Pump 1	1050.00	-12.00	3.53		3.54	0.000006	0.64	1642.00	136.40	0.03
Main 1A	5555.00	Full Pump	2880.00	-12.00	3.74		3.79	0.000046	1.72	1670.30	137.22	0.09
Main 1A	5555.00	Pump 1	1050.00	-12.00	3.53		3.54	0.000006	0.64	1641.70	136.39	0.03
Main 1A	5455.00	Full Pump	2880.00	-13.00	3.74		3.78	0.000033	1.52	1900.38	146.98	0.07
Main 1A	5455.00	Pump 1	1050.00	-13.00	3.53		3.54	0.000005	0.56	1869.31	146.13	0.03
Main 1A	4490.53	Full Pump	2880.00	-13.00	3.71		3.75	0.000033	1.52	1895.64	146.85	0.07
Main 1A	4490.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.66	146.11	0.03
Main 1A	4390.53	Full Pump	2880.00	-13.00	3.71		3.75	0.000033	1.52	1895.15	146.84	0.07
Main 1A	4390.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.59	146.11	0.03
Main 1A	4290.53	Full Pump	2880.00	-13.00	3.71		3.74	0.000033	1.52	1894.66	146.82	0.07
Main 1A	4290.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.52	146.11	0.03
Main 1A	4190.53	Full Pump	2880.00	-13.00	3.70		3.74	0.000033	1.52	1894.17	146.81	0.07
Main 1A	4190.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.46	146.11	0.03
Main 1A	4090.53	Full Pump	2880.00	-13.00	3.70		3.74	0.000033	1.52	1893.67	146.80	0.07
Main 1A	4090.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.39	146.11	0.03
Main 1A	3990.53	Full Pump	2880.00	-13.00	3.70		3.73	0.000033	1.52	1893.18	146.78	0.07
Main 1A	3990.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.32	146.10	0.03
Main 1A	3890.53	Full Pump	2880.00	-13.00	3.69		3.73	0.000033	1.52	1892.68	146.77	0.07
Main 1A	3890.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.25	146.10	0.03
Main 1A	3790.53	Full Pump	2880.00	-13.00	3.69		3.73	0.000033	1.52	1892.19	146.76	0.07
Main 1A	3790.53	Pump 1	1050.00	-13.00	3.53		3.53	0.000005	0.56	1868.19	146.10	0.03
Main 1A	3455.00	Full Pump	2880.00	-13.00	3.68		3.71	0.000034	1.52	1890.53	146.71	0.07
Main 1A	3455.00	Pump 1	1050.00	-13.00	3.52		3.53	0.000005	0.56	1867.96	146.09	0.03
Main 1A	3440.22	Full Pump	2880.00	-13.74	3.68		3.71	0.000032	1.51	1913.38	146.71	0.07
Main 1A	3440.22	Pump 1	1050.00	-13.74	3.52		3.53	0.000004	0.56	1890.80	146.09	0.03
Main 1A	3412.68	Full Pump	2880.00	-13.60	3.67		3.71	0.000036	1.58	1825.64	140.73	0.08
Main 1A	3412.68	Pump 1	1050.00	-13.60	3.52		3.53	0.000005	0.58	1804.55	140.16	0.03
Main 1A	3386.14	Full Pump	2880.00	-13.46	3.67		3.71	0.00004	1.65	1742.86	134.88	0.08
Main 1A	3386.14	Pump 1	1050.00	-13.46	3.52		3.53	0.000005	0.61	1723.25	134.35	0.03
Main 1A	3357.61	Full Pump	2880.00	-13.32	3.66		3.71	0.000044	1.73	1665.02	129.15	0.08
Main 1A	3357.61	Pump 1	1050.00	-13.32	3.52		3.53	0.000006	0.64	1646.87	128.67	0.03
Main 1A	3330.07	Full Pump	2880.00	-13.18	3.66		3.71	0.000049	1.81	1592.15	123.60	0.09
Main 1A	3330.07	Pump 1	1050.00	-13.18	3.52		3.53	0.000007	0.67	1575.42	123.16	0.03

(Continued Next Page)

Table D.2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	3302.53	Full Pump	2880.00	-13.04	3.65		3.71	0.000055	1.89	1523.89	118.26	0.09
Main 1A	3302.53	Pump 1	1050.00	-13.04	3.52		3.53	0.000007	0.70	1508.55	117.87	0.03
Main 1A	3275.00	Full Pump	2880.00	-12.90	3.61	-8.29	3.70	0.000064	2.44	1182.37	113.06	0.11
Main 1A	3275.00	Pump 1	1050.00	-12.90	3.51	-10.07	3.53	0.000009	0.89	1175.20	112.81	0.04
Main 1A	3273.38		Bridge									
Main 1A	3224.28	Full Pump	2880.00	-12.50	3.60	-8.47	3.68	0.00006	2.39	1207.50	118.64	0.11
Main 1A	3224.28	Pump 1	1050.00	-12.50	3.51	-10.18	3.52	0.000008	0.87	1201.10	118.34	0.04
Main 1A	3205.00	Full Pump	2880.00	-12.00	3.62		3.67	0.000049	1.81	1589.08	121.56	0.09
Main 1A	3205.00	Pump 1	1050.00	-12.00	3.52		3.52	0.000007	0.67	1576.43	121.07	0.03
Main 1A	3180.00	Full Pump	2880.00	-12.75	3.62		3.67	0.000047	1.80	1598.06	119.41	0.09
Main 1A	3180.00	Pump 1	1050.00	-12.75	3.52		3.52	0.000006	0.66	1585.71	119.26	0.03
Main 1A	3155.00	Full Pump	2880.00	-13.50	3.62		3.67	0.000046	1.80	1603.57	117.82	0.09
Main 1A	3155.00	Pump 1	1050.00	-13.50	3.52		3.52	0.000006	0.66	1591.47	117.68	0.03
Main 1A	3130.00	Full Pump	2880.00	-14.25	3.62		3.67	0.000046	1.79	1605.52	116.39	0.09
Main 1A	3130.00	Pump 1	1050.00	-14.25	3.52		3.52	0.000006	0.66	1593.68	116.27	0.03
Main 1A	3105.00	Full Pump	2880.00	-15.00	3.62		3.67	0.000046	1.80	1604.12	115.13	0.08
Main 1A	3105.00	Pump 1	1050.00	-15.00	3.52		3.52	0.000006	0.66	1592.52	115.02	0.03
Main 1A	3095.00	Full Pump	2880.00	-14.70	3.61		3.67	0.000049	1.84	1564.21	114.47	0.09
Main 1A	3095.00	Pump 1	1050.00	-14.70	3.51		3.52	0.000007	0.68	1553.02	114.27	0.03
Main 1A	3085.00	Full Pump	2880.00	-14.40	3.61		3.66	0.000052	1.89	1526.24	113.60	0.09
Main 1A	3085.00	Pump 1	1050.00	-14.40	3.51		3.52	0.000007	0.69	1515.47	113.38	0.03
Main 1A	3075.00	Full Pump	2880.00	-14.10	3.61		3.66	0.000055	1.93	1490.04	112.56	0.09
Main 1A	3075.00	Pump 1	1050.00	-14.10	3.51		3.52	0.000007	0.71	1479.71	112.33	0.03
Main 1A	3065.00	Full Pump	2880.00	-13.80	3.60		3.66	0.000058	1.98	1455.87	111.31	0.10
Main 1A	3065.00	Pump 1	1050.00	-13.80	3.51		3.52	0.000008	0.73	1446.00	111.07	0.04
Main 1A	3055.00	Full Pump	2880.00	-13.50	3.60		3.66	0.000061	2.02	1424.20	109.80	0.10
Main 1A	3055.00	Pump 1	1050.00	-13.50	3.51		3.52	0.000008	0.74	1414.81	109.54	0.04
Main 1A	3045.00	Full Pump	2880.00	-13.30	3.60		3.66	0.000062	2.01	1431.09	110.68	0.10
Main 1A	3045.00	Pump 1	1050.00	-13.30	3.51		3.52	0.000008	0.74	1421.64	110.43	0.04
Main 1A	3035.00	Full Pump	2880.00	-13.10	3.60		3.66	0.000061	2.00	1441.58	111.73	0.10
Main 1A	3035.00	Pump 1	1050.00	-13.10	3.51		3.52	0.000008	0.73	1432.04	111.49	0.04
Main 1A	3025.00	Full Pump	2880.00	-12.90	3.60		3.66	0.000061	1.98	1455.43	113.04	0.10
Main 1A	3025.00	Pump 1	1050.00	-12.90	3.51		3.52	0.000008	0.73	1445.76	112.81	0.04
Main 1A	3015.00	Full Pump	2880.00	-12.70	3.60		3.66	0.000061	1.96	1471.92	114.74	0.10
Main 1A	3015.00	Pump 1	1050.00	-12.70	3.51		3.52	0.000008	0.72	1462.07	114.53	0.04
Main 1A	3005.00	Full Pump	2880.00	-12.50	3.60		3.66	0.000061	1.93	1490.15	117.21	0.10
Main 1A	3005.00	Pump 1	1050.00	-12.50	3.51		3.52	0.000008	0.71	1480.04	117.03	0.04
Main 1A	2995.64	Full Pump	2880.00	-12.75	3.60		3.66	0.000061	1.96	1467.47	114.27	0.10
Main 1A	2995.64	Pump 1	1050.00	-12.75	3.51		3.52	0.000008	0.72	1457.87	114.06	0.04
Main 1A	2986.29	Full Pump	2880.00	-13.00	3.55	-8.49	3.65	0.000066	2.52	1144.23	112.22	0.11
Main 1A	2986.29	Pump 1	1050.00	-13.00	3.51	-10.28	3.52	0.000009	0.92	1140.82	112.09	0.04
Main 1A	2977.38		Bridge									

(Continued Next Page)

Table D.2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	2934.53	Full Pump	2880.00	-13.00	3.57		3.63	0.000062	1.99	1444.46	112.25	0.10
Main 1A	2934.53	Pump 1	1050.00	-13.00	3.51		3.52	0.000008	0.73	1438.03	112.09	0.04
Main 1A	2881.45	Full Pump	2880.00	-13.00	3.56		3.62	0.000064	2.05	1404.83	109.68	0.10
Main 1A	2881.45	Pump 1	1050.00	-13.00	3.51		3.52	0.000009	0.75	1399.23	109.52	0.04
Main 1A	2855.00	Full Pump	2880.00	-13.00	3.56		3.62	0.000064	2.05	1404.64	109.67	0.10
Main 1A	2855.00	Pump 1	1050.00	-13.00	3.51		3.52	0.000009	0.75	1399.21	109.52	0.04
Main 1A	2790.22	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.35	146.30	0.08
Main 1A	2790.22	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.94	146.04	0.03
Main 1A	2781.41	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.30	146.30	0.08
Main 1A	2781.41	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.94	146.04	0.03
Main 1A	2772.61	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.26	146.29	0.08
Main 1A	2772.61	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.93	146.04	0.03
Main 1A	2763.80	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.21	146.29	0.08
Main 1A	2763.80	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.92	146.04	0.03
Main 1A	2755.00	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1875.17	146.29	0.08
Main 1A	2755.00	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.92	146.04	0.03
Main 1A	2668.45	Full Pump	2880.00	-13.00	3.57		3.61	0.000034	1.54	1874.73	146.28	0.08
Main 1A	2668.45	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.86	146.04	0.03
Main 1A	2581.91	Full Pump	2880.00	-13.00	3.57		3.60	0.000034	1.54	1874.29	146.27	0.08
Main 1A	2581.91	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.80	146.04	0.03
Main 1A	2495.36	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.86	146.26	0.08
Main 1A	2495.36	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.74	146.03	0.03
Main 1A	2408.82	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.42	146.24	0.08
Main 1A	2408.82	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.68	146.03	0.03
Main 1A	2345.59	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.10	146.24	0.08
Main 1A	2345.59	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.64	146.03	0.03
Main 1A	2282.37	Full Pump	2880.00	-13.00	3.56		3.59	0.000034	1.54	1872.78	146.23	0.08
Main 1A	2282.37	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.60	146.03	0.03
Main 1A	2183.13	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1872.27	146.21	0.08
Main 1A	2183.13	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.53	146.03	0.03
Main 1A	2083.90	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1871.77	146.20	0.08
Main 1A	2083.90	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.46	146.03	0.03
Main 1A	1984.67	Full Pump	2880.00	-13.00	3.55		3.58	0.000035	1.54	1871.26	146.19	0.08
Main 1A	1984.67	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.40	146.02	0.03
Main 1A	1885.43	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.76	146.17	0.08
Main 1A	1885.43	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.33	146.02	0.03
Main 1A	1786.20	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.25	146.16	0.08
Main 1A	1786.20	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.26	146.02	0.03
Main 1A	1686.96	Full Pump	2880.00	-13.00	3.54		3.57	0.000035	1.54	1869.75	146.14	0.08
Main 1A	1686.96	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.19	146.02	0.03
Main 1A	1587.73	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1869.24	146.13	0.08
Main 1A	1587.73	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.13	146.02	0.03

(Continued Next Page)

Table D.2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	1488.50	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1868.74	146.12	0.08
Main 1A	1488.50	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.06	146.02	0.03
Main 1A	1389.26	Full Pump	2880.00	-13.00	3.53		3.56	0.000035	1.54	1868.23	146.10	0.08
Main 1A	1389.26	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.99	146.01	0.03
Main 1A	1290.03	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.72	146.09	0.08
Main 1A	1290.03	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.93	146.01	0.03
Main 1A	1190.80	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.22	146.07	0.08
Main 1A	1190.80	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.86	146.01	0.03
Main 1A	1091.56	Full Pump	2880.00	-13.00	3.52		3.55	0.000035	1.54	1866.71	146.06	0.08
Main 1A	1091.56	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.79	146.01	0.03
Main 1A	992.33	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1866.20	146.05	0.08
Main 1A	992.33	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.72	146.01	0.03
Main 1A	893.10	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1865.69	146.03	0.08
Main 1A	893.10	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.66	146.00	0.03
Main 1A	793.87	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1865.18	146.02	0.08
Main 1A	793.87	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.59	146.00	0.03
Main 1A	694.63	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1864.68	146.00	0.08
Main 1A	694.63	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.52	146.00	0.03
Main 1A	595.40	Full Pump	2880.00	-13.00	3.50		3.53	0.000035	1.54	1864.17	145.99	0.08
Main 1A	595.40	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.45	146.00	0.03
Main 1A	496.17	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.66	145.98	0.08
Main 1A	496.17	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.39	146.00	0.03
Main 1A	396.93	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.15	145.96	0.08
Main 1A	396.93	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.32	146.00	0.03
Main 1A	297.70	Full Pump	2880.00	-13.00	3.49		3.52	0.000035	1.55	1862.64	145.95	0.08
Main 1A	297.70	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.25	145.99	0.03
Main 1A	198.47	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1862.13	145.93	0.08
Main 1A	198.47	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.19	145.99	0.03
Main 1A	99.23	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1861.62	145.92	0.08
Main 1A	99.23	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.12	145.99	0.03
Main 1A	0.00	Full Pump	2880.00	-13.00	3.48		3.51	0.000035	1.55	1861.11	145.91	0.08
Main 1A	0.00	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.05	145.99	0.03
Main 1A	-55.00	Full Pump	2880.00	-14.00	3.50	-12.65	3.50	0.000002	0.47	6066.03	373.26	0.02
Main 1A	-55.00	Pump 1	1050.00	-14.00	3.50	-13.31	3.50	0	0.17	6066.03	373.26	0.01

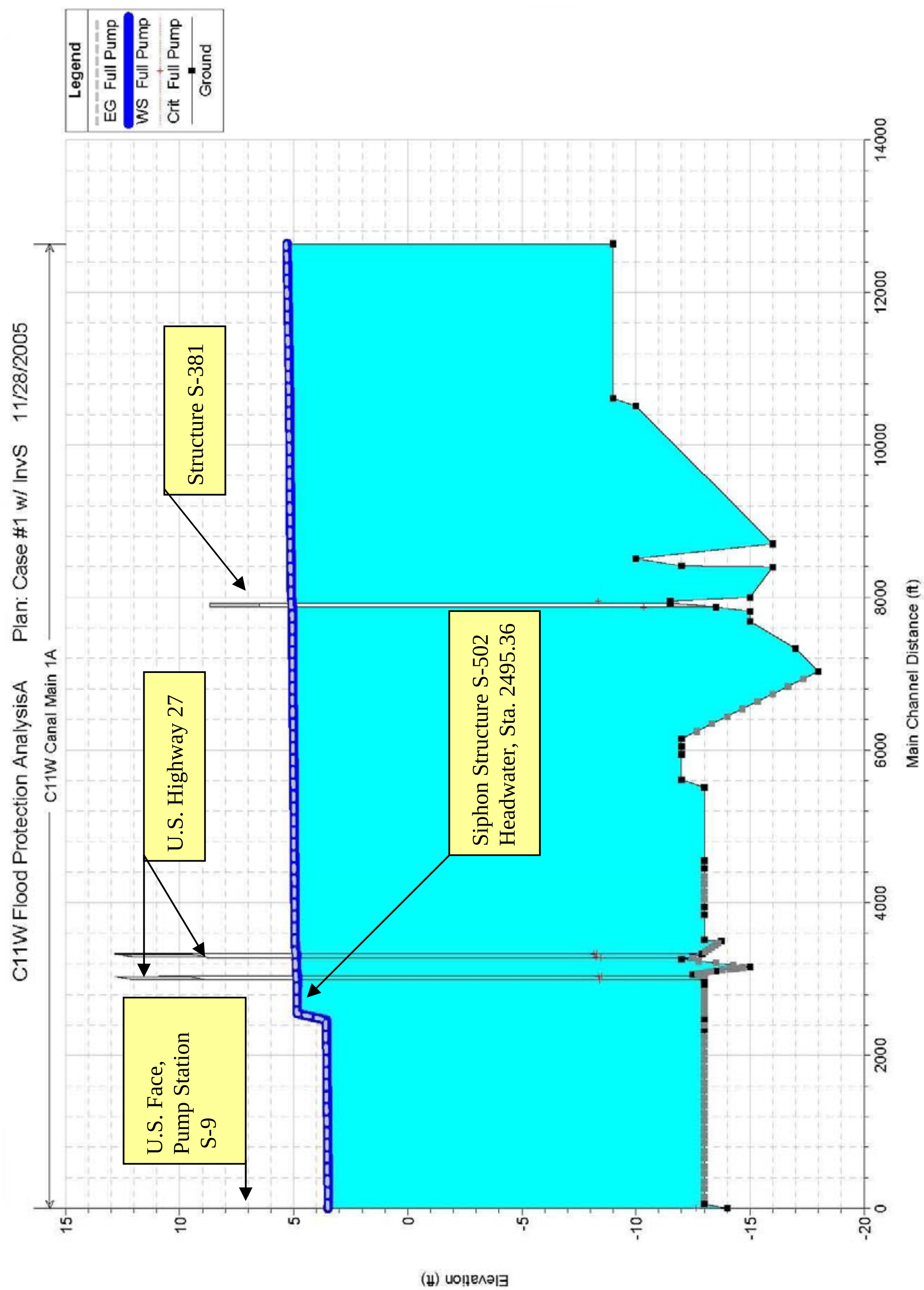

Figure D.5 Water Surface Profile with S-381 and S-502, Q=2,880 cfs

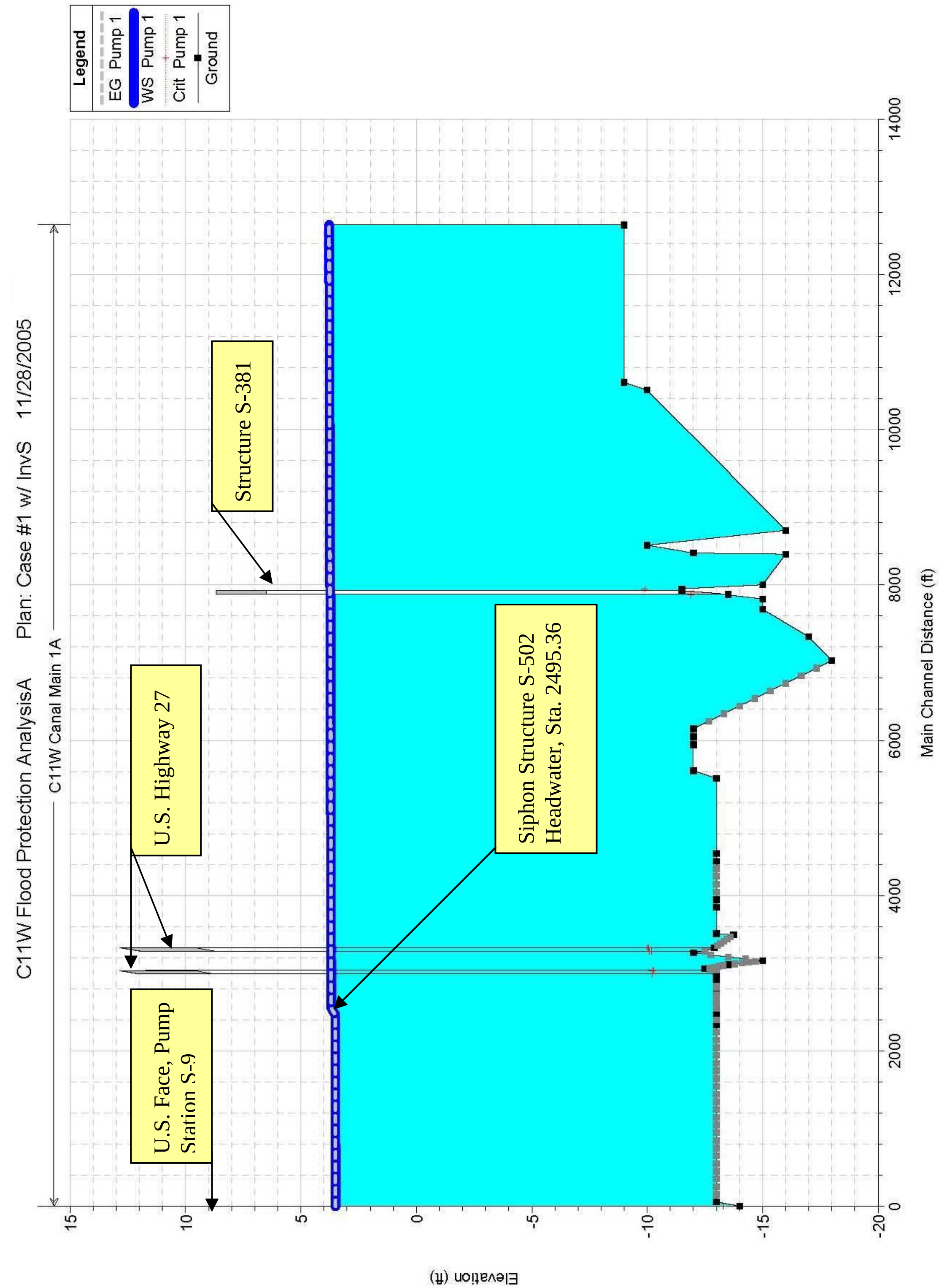

Figure D.6 Water Surface Profile with S-381 and S-502, Q=1,050 cfs

Table D.3 Water Surface Profile Data with S-381 and S-502

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	12582.44	Full Pump	2880.00	-9.00	5.29		5.33	0.000053	1.78	1622.35	142.14	0.09
Main 1A	12582.44	Pump 1	1050.00	-9.00	3.77		3.78	0.000011	0.74	1411.80	136.09	0.04
Main 1A	10555.00	Full Pump	2880.00	-9.00	5.17		5.22	0.000055	1.79	1606.68	141.70	0.09
Main 1A	10555.00	Pump 1	1050.00	-9.00	3.75		3.76	0.000011	0.75	1408.87	136.00	0.04
Main 1A	10455.00	Full Pump	2880.00	-10.00	5.17		5.22	0.000047	1.72	1674.18	140.69	0.09
Main 1A	10455.00	Pump 1	1050.00	-10.00	3.75		3.76	0.000009	0.71	1478.06	135.00	0.04
Main 1A	8647.21	Full Pump	2880.00	-16.00	5.13		5.16	0.00002	1.29	2238.27	152.83	0.06
Main 1A	8647.21	Pump 1	1050.00	-16.00	3.74		3.75	0.000004	0.52	2030.08	146.66	0.02
Main 1A	8455.00	Full Pump	2880.00	-10.00	5.10		5.15	0.000048	1.73	1664.61	140.42	0.09
Main 1A	8455.00	Pump 1	1050.00	-10.00	3.74		3.75	0.000009	0.71	1476.42	134.95	0.04
Main 1A	8355.00	Full Pump	2880.00	-12.00	5.11		5.14	0.000034	1.55	1861.31	142.61	0.08
Main 1A	8355.00	Pump 1	1050.00	-12.00	3.74		3.74	0.000006	0.63	1669.80	137.20	0.03
Main 1A	8340.96	Full Pump	2880.00	-16.00	5.11		5.14	0.00002	1.29	2235.45	152.75	0.06
Main 1A	8340.96	Pump 1	1050.00	-16.00	3.74		3.74	0.000004	0.52	2029.60	146.64	0.02
Main 1A	7945.50	Full Pump	2880.00	-15.00	5.10		5.13	0.000024	1.36	2120.53	151.97	0.06
Main 1A	7945.50	Pump 1	1050.00	-15.00	3.74		3.74	0.000004	0.55	1917.35	145.66	0.03
Main 1A	7895.00	Full Pump	2880.00	-11.50	5.07	-8.34	5.13	0.000075	1.93	1490.47	90.00	0.08
Main 1A	7895.00	Pump 1	1050.00	-11.50	3.73	-9.89	3.74	0.000013	0.77	1370.26	90.00	0.03
Main 1A	7855.00		Inl Struct									
Main 1A	7815.00	Full Pump	2880.00	-13.50	5.04	-10.33	5.09	0.000072	1.73	1668.54	90.00	0.07
Main 1A	7815.00	Pump 1	1050.00	-13.50	3.71	-11.88	3.72	0.000012	0.68	1549.22	90.00	0.03
Main 1A	7763.30	Full Pump	2880.00	-15.00	5.05		5.08	0.000024	1.36	2112.51	151.73	0.06
Main 1A	7763.30	Pump 1	1050.00	-15.00	3.71		3.72	0.000004	0.55	1914.13	145.56	0.03
Main 1A	7631.08	Full Pump	2880.00	-15.00	5.05		5.08	0.000024	1.36	2112.02	151.71	0.06
Main 1A	7631.08	Pump 1	1050.00	-15.00	3.71		3.72	0.000004	0.55	1914.05	145.55	0.03
Main 1A	7276.48	Full Pump	2880.00	-17.00	5.04		5.07	0.000018	1.23	2337.69	153.11	0.06
Main 1A	7276.48	Pump 1	1050.00	-17.00	3.71		3.72	0.000003	0.49	2137.99	147.43	0.02
Main 1A	6971.88	Full Pump	2880.00	-18.00	5.04		5.06	0.000015	1.18	2450.41	153.72	0.05
Main 1A	6971.88	Pump 1	1050.00	-18.00	3.71		3.72	0.000003	0.47	2250.19	148.27	0.02
Main 1A	6874.38	Full Pump	2880.00	-17.33	5.04		5.06	0.000017	1.21	2384.52	152.44	0.05
Main 1A	6874.38	Pump 1	1050.00	-17.33	3.71		3.72	0.000003	0.48	2186.33	147.02	0.02
Main 1A	6776.88	Full Pump	2880.00	-16.67	5.03		5.06	0.000018	1.24	2318.90	151.13	0.06
Main 1A	6776.88	Pump 1	1050.00	-16.67	3.71		3.72	0.000003	0.49	2122.80	145.74	0.02
Main 1A	6679.38	Full Pump	2880.00	-16.00	5.03		5.06	0.00002	1.28	2252.01	149.83	0.06
Main 1A	6679.38	Pump 1	1050.00	-16.00	3.71		3.72	0.000003	0.51	2058.03	144.47	0.02
Main 1A	6581.88	Full Pump	2880.00	-15.33	5.03		5.05	0.000021	1.32	2185.15	148.57	0.06
Main 1A	6581.88	Pump 1	1050.00	-15.33	3.71		3.72	0.000004	0.53	1993.26	143.25	0.02
Main 1A	6484.38	Full Pump	2880.00	-14.67	5.02		5.05	0.000023	1.36	2118.34	147.27	0.06
Main 1A	6484.38	Pump 1	1050.00	-14.67	3.71		3.71	0.000004	0.54	1928.63	141.98	0.03
Main 1A	6386.88	Full Pump	2880.00	-14.00	5.02		5.05	0.000026	1.40	2050.78	146.01	0.07
Main 1A	6386.88	Pump 1	1050.00	-14.00	3.71		3.71	0.000004	0.56	1863.22	140.76	0.03

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Table D.3 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	6289.38	Full Pump	2880.00	-13.33	5.01		5.05	0.000028	1.45	1982.53	144.73	0.07
Main 1A	6289.38	Pump 1	1050.00	-13.33	3.71		3.71	0.000005	0.58	1797.21	139.52	0.03
Main 1A	6191.88	Full Pump	2880.00	-12.67	5.01		5.04	0.000031	1.50	1915.22	143.47	0.07
Main 1A	6191.88	Pump 1	1050.00	-12.67	3.71		3.71	0.000006	0.61	1732.15	138.31	0.03
Main 1A	6094.38	Full Pump	2880.00	-12.00	5.00		5.04	0.000035	1.56	1846.25	142.19	0.08
Main 1A	6094.38	Pump 1	1050.00	-12.00	3.71		3.71	0.000006	0.63	1665.51	137.08	0.03
Main 1A	5990.53	Full Pump	2880.00	-12.00	5.00		5.04	0.000035	1.56	1845.73	142.18	0.08
Main 1A	5990.53	Pump 1	1050.00	-12.00	3.71		3.71	0.000006	0.63	1665.42	137.08	0.03
Main 1A	5890.53	Full Pump	2880.00	-12.00	4.99		5.03	0.000035	1.56	1845.22	142.17	0.08
Main 1A	5890.53	Pump 1	1050.00	-12.00	3.71		3.71	0.000006	0.63	1665.34	137.07	0.03
Main 1A	5555.00	Full Pump	2880.00	-12.00	4.98		5.02	0.000035	1.56	1843.54	142.12	0.08
Main 1A	5555.00	Pump 1	1050.00	-12.00	3.70		3.71	0.000006	0.63	1665.05	137.07	0.03
Main 1A	5455.00	Full Pump	2880.00	-13.00	4.98		5.01	0.000025	1.38	2085.67	151.94	0.07
Main 1A	5455.00	Pump 1	1050.00	-13.00	3.70		3.71	0.000004	0.55	1894.32	146.81	0.03
Main 1A	4490.53	Full Pump	2880.00	-13.00	4.96		4.99	0.000026	1.38	2081.92	151.84	0.07
Main 1A	4490.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.69	146.80	0.03
Main 1A	4390.53	Full Pump	2880.00	-13.00	4.96		4.99	0.000026	1.38	2081.53	151.83	0.07
Main 1A	4390.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.62	146.80	0.03
Main 1A	4290.53	Full Pump	2880.00	-13.00	4.95		4.98	0.000026	1.38	2081.14	151.82	0.07
Main 1A	4290.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.56	146.79	0.03
Main 1A	4190.53	Full Pump	2880.00	-13.00	4.95		4.98	0.000026	1.38	2080.75	151.81	0.07
Main 1A	4190.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.49	146.79	0.03
Main 1A	4090.53	Full Pump	2880.00	-13.00	4.95		4.98	0.000026	1.38	2080.36	151.80	0.07
Main 1A	4090.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.43	146.79	0.03
Main 1A	3990.53	Full Pump	2880.00	-13.00	4.95		4.98	0.000026	1.38	2079.97	151.79	0.07
Main 1A	3990.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.36	146.79	0.03
Main 1A	3890.53	Full Pump	2880.00	-13.00	4.94		4.97	0.000026	1.38	2079.58	151.78	0.07
Main 1A	3890.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.30	146.79	0.03
Main 1A	3790.53	Full Pump	2880.00	-13.00	4.94		4.97	0.000026	1.39	2079.19	151.77	0.07
Main 1A	3790.53	Pump 1	1050.00	-13.00	3.70		3.70	0.000004	0.55	1893.23	146.79	0.03
Main 1A	3455.00	Full Pump	2880.00	-13.00	4.93		4.96	0.000026	1.39	2077.88	151.73	0.07
Main 1A	3455.00	Pump 1	1050.00	-13.00	3.69		3.70	0.000004	0.55	1893.01	146.78	0.03
Main 1A	3440.22	Full Pump	2880.00	-13.74	4.93		4.96	0.000025	1.37	2100.73	151.73	0.06
Main 1A	3440.22	Pump 1	1050.00	-13.74	3.69		3.70	0.000004	0.55	1915.85	146.78	0.03
Main 1A	3412.68	Full Pump	2880.00	-13.60	4.93		4.96	0.000028	1.44	2005.47	145.55	0.07
Main 1A	3412.68	Pump 1	1050.00	-13.60	3.69		3.70	0.000005	0.57	1828.58	140.81	0.03
Main 1A	3385.14	Full Pump	2880.00	-13.46	4.93		4.96	0.000031	1.50	1915.33	139.47	0.07
Main 1A	3385.14	Pump 1	1050.00	-13.46	3.69		3.70	0.000005	0.60	1746.29	134.97	0.03
Main 1A	3357.61	Full Pump	2880.00	-13.32	4.92		4.96	0.000034	1.57	1830.27	133.48	0.07
Main 1A	3357.61	Pump 1	1050.00	-13.32	3.69		3.70	0.000006	0.63	1668.94	129.26	0.03
Main 1A	3330.07	Full Pump	2880.00	-13.18	4.92		4.96	0.000038	1.65	1750.37	127.65	0.08
Main 1A	3330.07	Pump 1	1050.00	-13.18	3.69		3.70	0.000007	0.66	1596.54	123.71	0.03

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Table D.3 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	3302.53	Full Pump	2880.00	-13.04	4.91		4.96	0.000042	1.72	1675.34	121.99	0.08
Main 1A	3302.53	Pump 1	1050.00	-13.04	3.69		3.70	0.000007	0.69	1528.77	118.38	0.03
Main 1A	3275.00	Full Pump	2880.00	-12.90	4.87	-8.29	4.95	0.000049	2.25	1279.34	116.44	0.10
Main 1A	3275.00	Pump 1	1050.00	-12.90	3.69	-10.07	3.70	0.000008	0.88	1188.33	113.27	0.04
Main 1A	3273.38		Bridge									
Main 1A	3224.28	Full Pump	2880.00	-12.50	4.86	-8.47	4.94	0.000046	2.21	1305.85	123.06	0.09
Main 1A	3224.28	Pump 1	1050.00	-12.50	3.68	-10.18	3.70	0.000008	0.86	1214.39	118.96	0.04
Main 1A	3205.00	Full Pump	2880.00	-12.00	4.89		4.93	0.000038	1.65	1746.59	127.46	0.08
Main 1A	3205.00	Pump 1	1050.00	-12.00	3.69		3.69	0.000006	0.66	1597.24	121.87	0.03
Main 1A	3180.00	Full Pump	2880.00	-12.75	4.88		4.93	0.000036	1.65	1750.35	121.71	0.08
Main 1A	3180.00	Pump 1	1050.00	-12.75	3.69		3.69	0.000006	0.65	1606.16	119.51	0.03
Main 1A	3155.00	Full Pump	2880.00	-13.50	4.88		4.93	0.000035	1.64	1753.71	119.54	0.08
Main 1A	3155.00	Pump 1	1050.00	-13.50	3.69		3.69	0.000006	0.65	1611.65	117.91	0.03
Main 1A	3130.00	Full Pump	2880.00	-14.25	4.88		4.92	0.000035	1.64	1753.78	117.96	0.08
Main 1A	3130.00	Pump 1	1050.00	-14.25	3.69		3.69	0.000006	0.65	1613.61	116.48	0.03
Main 1A	3105.00	Full Pump	2880.00	-15.00	4.88		4.92	0.000035	1.65	1750.72	116.55	0.07
Main 1A	3105.00	Pump 1	1050.00	-15.00	3.69		3.69	0.000006	0.65	1612.24	115.21	0.03
Main 1A	3095.00	Full Pump	2880.00	-14.70	4.88		4.92	0.000037	1.68	1710.48	116.35	0.08
Main 1A	3095.00	Pump 1	1050.00	-14.70	3.69		3.69	0.000006	0.67	1572.62	114.61	0.03
Main 1A	3085.00	Full Pump	2880.00	-14.40	4.88		4.92	0.000039	1.72	1671.89	116.11	0.08
Main 1A	3085.00	Pump 1	1050.00	-14.40	3.69		3.69	0.000007	0.68	1534.93	113.77	0.03
Main 1A	3075.00	Full Pump	2880.00	-14.10	4.87		4.92	0.000042	1.76	1634.69	115.62	0.08
Main 1A	3075.00	Pump 1	1050.00	-14.10	3.69		3.69	0.000007	0.70	1498.99	112.75	0.03
Main 1A	3065.00	Full Pump	2880.00	-13.80	4.87		4.92	0.000044	1.80	1599.25	114.72	0.08
Main 1A	3065.00	Pump 1	1050.00	-13.80	3.68		3.69	0.000008	0.72	1465.08	111.54	0.03
Main 1A	3055.00	Full Pump	2880.00	-13.50	4.87		4.92	0.000047	1.84	1565.99	113.60	0.09
Main 1A	3055.00	Pump 1	1050.00	-13.50	3.68		3.69	0.000008	0.73	1433.62	110.05	0.04
Main 1A	3045.00	Full Pump	2880.00	-13.30	4.87		4.92	0.000047	1.83	1573.95	114.39	0.09
Main 1A	3045.00	Pump 1	1050.00	-13.30	3.68		3.69	0.000008	0.73	1440.61	110.93	0.04
Main 1A	3035.00	Full Pump	2880.00	-13.10	4.87		4.92	0.000047	1.82	1585.69	115.30	0.09
Main 1A	3035.00	Pump 1	1050.00	-13.10	3.68		3.69	0.000008	0.72	1451.19	111.97	0.04
Main 1A	3025.00	Full Pump	2880.00	-12.90	4.87		4.92	0.000047	1.80	1601.09	116.43	0.09
Main 1A	3025.00	Pump 1	1050.00	-12.90	3.68		3.69	0.000008	0.72	1465.13	113.27	0.04
Main 1A	3015.00	Full Pump	2880.00	-12.70	4.87		4.92	0.000046	1.78	1619.59	117.88	0.08
Main 1A	3015.00	Pump 1	1050.00	-12.70	3.68		3.69	0.000008	0.71	1481.73	114.96	0.03
Main 1A	3005.00	Full Pump	2880.00	-12.50	4.87		4.92	0.000046	1.76	1640.70	119.95	0.08
Main 1A	3005.00	Pump 1	1050.00	-12.50	3.68		3.69	0.000008	0.70	1500.13	117.40	0.03
Main 1A	2995.64	Full Pump	2880.00	-12.75	4.87		4.92	0.000046	1.78	1614.63	117.48	0.08
Main 1A	2995.64	Pump 1	1050.00	-12.75	3.68		3.69	0.000008	0.71	1477.45	114.49	0.03
Main 1A	2986.29	Full Pump	2880.00	-13.00	4.83	-8.49	4.91	0.000051	2.33	1237.15	115.72	0.10
Main 1A	2986.29	Pump 1	1050.00	-13.00	3.68	-10.28	3.69	0.000009	0.91	1153.32	112.56	0.04
Main 1A	2977.38		Bridge									

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Table D.3 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	2934.53	Full Pump	2880.00	-13.00	4.84		4.89	0.000047	1.81	1589.74	115.76	0.09
Main 1A	2934.53	Pump 1	1050.00	-13.00	3.68		3.69	0.000008	0.72	1457.29	112.57	0.04
Main 1A	2881.45	Full Pump	2880.00	-13.00	4.83		4.89	0.000049	1.86	1547.21	113.50	0.09
Main 1A	2881.45	Pump 1	1050.00	-13.00	3.68		3.69	0.000008	0.74	1418.06	110.04	0.04
Main 1A	2855.00	Full Pump	2880.00	-13.00	4.83		4.89	0.000049	1.86	1547.06	113.50	0.09
Main 1A	2855.00	Pump 1	1050.00	-13.00	3.68		3.69	0.000008	0.74	1418.03	110.04	0.04
Main 1A	2790.22	Full Pump	2880.00	-13.00	4.85		4.88	0.000026	1.39	2064.91	151.39	0.07
Main 1A	2790.22	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1891.04	146.73	0.03
Main 1A	2781.41	Full Pump	2880.00	-13.00	4.85		4.88	0.000026	1.39	2064.87	151.39	0.07
Main 1A	2781.41	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1891.03	146.73	0.03
Main 1A	2772.61	Full Pump	2880.00	-13.00	4.85		4.88	0.000026	1.39	2064.84	151.39	0.07
Main 1A	2772.61	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1891.03	146.72	0.03
Main 1A	2763.80	Full Pump	2880.00	-13.00	4.85		4.88	0.000026	1.39	2064.80	151.39	0.07
Main 1A	2763.80	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1891.02	146.72	0.03
Main 1A	2755.00	Full Pump	2880.00	-13.00	4.85		4.88	0.000026	1.39	2064.77	151.39	0.07
Main 1A	2755.00	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1891.02	146.72	0.03
Main 1A	2668.45	Full Pump	2880.00	-13.00	4.84		4.87	0.000026	1.40	2064.42	151.38	0.07
Main 1A	2668.45	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1890.96	146.72	0.03
Main 1A	2581.91	Full Pump	2880.00	-13.00	4.84		4.87	0.000026	1.40	2064.08	151.37	0.07
Main 1A	2581.91	Pump 1	1050.00	-13.00	3.68		3.69	0.000004	0.56	1890.90	146.72	0.03
Main 1A	2495.36	Full Pump	2880.00	-13.00	4.84		4.87	0.000026	1.40	2063.73	151.36	0.07
Main 1A	2495.36	Pump 1	1050.00	-13.00	3.68		3.68	0.000004	0.56	1890.85	146.72	0.03
Main 1A	2408.82	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.42	146.24	0.08
Main 1A	2408.82	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.68	146.03	0.03
Main 1A	2345.59	Full Pump	2880.00	-13.00	3.56		3.60	0.000034	1.54	1873.10	146.24	0.08
Main 1A	2345.59	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.64	146.03	0.03
Main 1A	2282.37	Full Pump	2880.00	-13.00	3.56		3.59	0.000034	1.54	1872.78	146.23	0.08
Main 1A	2282.37	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.60	146.03	0.03
Main 1A	2183.13	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1872.27	146.21	0.08
Main 1A	2183.13	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.53	146.03	0.03
Main 1A	2083.90	Full Pump	2880.00	-13.00	3.55		3.59	0.000035	1.54	1871.77	146.20	0.08
Main 1A	2083.90	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.46	146.03	0.03
Main 1A	1984.67	Full Pump	2880.00	-13.00	3.55		3.58	0.000035	1.54	1871.26	146.19	0.08
Main 1A	1984.67	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.40	146.02	0.03
Main 1A	1885.43	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.76	146.17	0.08
Main 1A	1885.43	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.33	146.02	0.03
Main 1A	1786.20	Full Pump	2880.00	-13.00	3.54		3.58	0.000035	1.54	1870.25	146.16	0.08
Main 1A	1786.20	Pump 1	1050.00	-13.00	3.51		3.51	0.000005	0.56	1865.26	146.02	0.03
Main 1A	1686.96	Full Pump	2880.00	-13.00	3.54		3.57	0.000035	1.54	1869.75	146.14	0.08
Main 1A	1686.96	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.19	146.02	0.03
Main 1A	1587.73	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1869.24	146.13	0.08
Main 1A	1587.73	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.13	146.02	0.03

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Table D.3 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main 1A	1488.50	Full Pump	2880.00	-13.00	3.53		3.57	0.000035	1.54	1868.74	146.12	0.08
Main 1A	1488.50	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1865.06	146.02	0.03
Main 1A	1389.26	Full Pump	2880.00	-13.00	3.53		3.56	0.000035	1.54	1868.23	146.10	0.08
Main 1A	1389.26	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.99	146.01	0.03
Main 1A	1290.03	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.72	146.09	0.08
Main 1A	1290.03	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.93	146.01	0.03
Main 1A	1190.80	Full Pump	2880.00	-13.00	3.52		3.56	0.000035	1.54	1867.22	146.07	0.08
Main 1A	1190.80	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.86	146.01	0.03
Main 1A	1091.56	Full Pump	2880.00	-13.00	3.52		3.55	0.000035	1.54	1866.71	146.06	0.08
Main 1A	1091.56	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.79	146.01	0.03
Main 1A	992.33	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1866.20	146.05	0.08
Main 1A	992.33	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.72	146.01	0.03
Main 1A	893.10	Full Pump	2880.00	-13.00	3.51		3.55	0.000035	1.54	1865.69	146.03	0.08
Main 1A	893.10	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.66	146.00	0.03
Main 1A	793.87	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1865.18	146.02	0.08
Main 1A	793.87	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.59	146.00	0.03
Main 1A	694.63	Full Pump	2880.00	-13.00	3.50		3.54	0.000035	1.54	1864.68	146.00	0.08
Main 1A	694.63	Pump 1	1050.00	-13.00	3.50		3.51	0.000005	0.56	1864.52	146.00	0.03
Main 1A	595.40	Full Pump	2880.00	-13.00	3.50		3.53	0.000035	1.54	1864.17	145.99	0.08
Main 1A	595.40	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.45	146.00	0.03
Main 1A	496.17	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.66	145.98	0.08
Main 1A	496.17	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.39	146.00	0.03
Main 1A	396.93	Full Pump	2880.00	-13.00	3.49		3.53	0.000035	1.55	1863.15	145.96	0.08
Main 1A	396.93	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.32	146.00	0.03
Main 1A	297.70	Full Pump	2880.00	-13.00	3.49		3.52	0.000035	1.55	1862.64	145.95	0.08
Main 1A	297.70	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.25	145.99	0.03
Main 1A	198.47	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1862.13	145.93	0.08
Main 1A	198.47	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.19	145.99	0.03
Main 1A	99.23	Full Pump	2880.00	-13.00	3.48		3.52	0.000035	1.55	1861.62	145.92	0.08
Main 1A	99.23	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.12	145.99	0.03
Main 1A	0.00	Full Pump	2880.00	-13.00	3.48		3.51	0.000035	1.55	1861.11	145.91	0.08
Main 1A	0.00	Pump 1	1050.00	-13.00	3.50		3.50	0.000005	0.56	1864.05	145.99	0.03
Main 1A	-55.00	Full Pump	2880.00	-14.00	3.50	-12.65	3.50	0.000002	0.47	6066.03	373.26	0.02
Main 1A	-55.00	Pump 1	1050.00	-14.00	3.50	-13.31	3.50	0	0.17	6066.03	373.26	0.01

Table D.4 Summary Comparison of Water Surface Profiles, Q=2,880 cfs

River Station (ft.)	Water Surface Elevation (ft. NGVD)			Change from Base (ft.)		Remarks
	Existing (Base)	With S-381 Only	With S-381 & S-502	With S-381 Only	With S-381 & S-502	
(55.00)	3.50	3.50	3.50	0.00	0.00	Upstream Face, Pump Station S-9
0.00	3.48	3.48	3.48	0.00	0.00	Begin Normal Canal Section
992.33	3.51	3.51	3.51	0.00	0.00	
1885.43	3.54	3.54	3.54	0.00	0.00	Downstream End, S-502 Wingwalls
2495.36	3.56	3.56	4.84	0.00	1.28	Upstream End, S-502 Wingwalls
2581.91	3.57	3.57	4.84	0.00	1.27	
2881.45	3.56	3.56	4.83	0.00	1.27	
2934.53	3.57	3.57	4.84	0.00	1.27	Downstream Face, U.S. 27 Southbound
2986.29	3.55	3.55	4.83	0.00	1.28	Upstream Face, U.S. 27 Southbound
2995.64	3.60	3.60	4.87	0.00	1.27	
3205.00	3.62	3.62	4.89	0.00	1.27	
3224.28	3.59	3.60	4.86	0.01	1.27	Downstream Face, U.S. 27 Northbound
3275.00	3.60	3.61	4.87	0.01	1.27	Upstream Face, U.S. 27 Northbound
3302.53	3.66	3.65	4.91	(0.01)	1.25	
3330.07	3.66	3.66	4.92	0.00	1.26	
4290.53	3.71	3.71	4.95	0.00	1.24	
5455.00	3.75	3.74	4.98	(0.01)	1.23	
6094.38	3.77	3.77	5.00	0.00	1.23	
7631.08		3.82	5.05			
7763.30		3.83	5.05			S-381 Tailwater
7855.00						Approx. Centerline, Structure S-381
7945.50		3.89	5.10			S-381 Headwater
8355.00	3.88	3.89	5.11	0.01	1.23	End, Approx. East R/W C-11 Impoundment

Table D.5 Summary Comparison of Water Surface Profiles, Q=1,050 cfs

River Station (ft.)	Water Surface Elevation (ft. NGVD)			Change from Base (ft.)		Remarks
	Existing (Base)	With S-381 Only	With S-381 & S-502	With S-381 Only	With S-381 & S-502	
(55.00)	3.50	3.50	3.50	0.00	0.00	Upstream Face, Pump Station S-9
0.00	3.50	3.50	3.50	0.00	0.00	Begin Normal Canal Section
992.33	3.50	3.50	3.50	0.00	0.00	
1885.43	3.51	3.51	3.51	0.00	0.00	Downstream End, S-502 Wingwalls
2495.36	3.51	3.51	3.68	0.00	0.17	Upstream End, S-502 Wingwalls
2581.91	3.51	3.51	3.68	0.00	0.17	
2881.45	3.51	3.51	3.68	0.00	0.17	
2934.53	3.51	3.51	3.68	0.00	0.17	Downstream Face, U.S. 27 Southbound
2986.29	3.51	3.51	3.68	0.00	0.17	Upstream Face, U.S. 27 Southbound
2995.64	3.51	3.51	3.68	0.00	0.17	
3205.00	3.52	3.52	3.69	0.00	0.17	
3224.28	3.51	3.51	3.68	0.00	0.17	Downstream Face, U.S. 27 Northbound
3275.00	3.51	3.51	3.69	0.00	0.18	Upstream Face, U.S. 27 Northbound
3302.53	3.52	3.52	3.69	0.00	0.17	
3330.07	3.52	3.52	3.69	0.00	0.17	
4290.53	3.53	3.53	3.70	0.00	0.17	
5455.00	3.53	3.53	3.70	0.00	0.17	
6094.38	3.54	3.54	3.71	0.00	0.17	
7631.08		3.54	3.71			
7763.30		3.54	3.71			S-381 Tailwater
7855.00						Approx. Centerline, Structure S-381
7945.50		3.57	3.74			S-381 Headwater
8355.00	3.55	3.57	3.74	0.02	0.19	End, Approx. East R/W C-11 Impoundment