



Evaluation of Effectiveness of Abatement Strategies

*Compared Against Pre-Drainage and Existing Conditions in
the Lake Okeechobee Watershed*

Deliverable 3.1: Draft Abatement Scenarios Report

March 4, 2019

Soil and Water Engineering Technology, Inc.

Contents

1	Executive Summary	1
2	Introduction and Background.....	4
2.1	Project Description.....	6
2.2	Project Objectives	6
3	Advanced BMPs	8
3.1	Analyses for the Simulated BMP Scenarios	8
3.2	Advanced Fertility/Nutrient and Water Management BMPs	8
4	Lake Okeechobee Watershed Construction Projects (Scenario 3)	12
4.1	Introduction.....	12
4.1.1	Reach Cross-Sectional Profiles	14
4.1.2	Attenuation.....	15
4.1.3	Project Costs	16
4.2	Indian Prairie Lake Okeechobee Watershed Construction Projects	19
4.2.1	Introduction.....	19
4.2.2	Brighton Valley DWM.....	19
4.2.3	Istokpoga Marsh Watershed Improvement District (Phase 2) Retention Project.....	32
4.2.4	Summary of Indian Prairie LOWCP Results	38
4.2.5	K-05 Wetland Attenuation Feature.....	38
4.2.6	Kissimmee River Center Wetland Restoration.....	47
4.3	Taylor Creek/Nubbin Slough Lake Okeechobee Watershed Construction Projects.....	50
4.3.1	Lakeside Ranch STA and Brady Ranch STA	50
4.3.2	Summary of LRSTA/BRSTA Results	65
4.4	Fisheating Creek Lake Okeechobee Watershed Construction Projects.....	67
4.4.1	Floating Aquatic Vegetative Tilling (FAVT) Treatment Area.....	67
4.4.2	Nicodemus Slough Retention Area	77
4.4.3	Summary of Fisheating Creek LOWCP Results	86
4.5	Upper Kissimmee Lake Okeechobee Watershed Construction Projects.....	87
4.5.1	Rolling Meadows Wetlands Restoration Project.....	87
5	Kissimmee River Restoration (KRR) Project	95
5.1.1	Introduction and Background.....	95
5.1.2	Implementation in WAM.....	95
5.1.3	Simulation Results	103
6	Summary and Conclusions.....	106
7	References.....	109

8	Appendix A: Scenarios 1 and 2 P Load Reductions and Cost Effectiveness Factors for Individual BMPs Implemented on Three Representative Soils.....	112
---	---	-----

Figures

Figure 1. The nine sub-watersheds of the LOW showing the LOWCP, LOWRP, and KRR projects simulated for this study.	5
Figure 2. Example reach cross-sectional profile based on the data in Table 2.....	15
Figure 3. The Indian Prairie sub-watershed, with the IMWID, Brighton Valley DWM, the WAF, and major canals and structures labeled.	21
Figure 4. The Brighton Valley DWM impoundments and control structures.	22
Figure 5. The WAM setup for the Brighton Valley DWM.	25
Figure 6. The cross-sectional profile for WAM reach 162, a typical slough in the Brighton Valley DWM east impoundment, used for the simulation with the mean bottom elevation indicated.	26
Figure 7. Time series of pump inflow to the Brighton Valley DWM used for the WAM simulations.....	27
Figure 8. Annual simulated inflow volumes (ac-ft/y) to Brighton Valley DWM.....	29
Figure 9. Annual simulated outflow volumes (ac-ft/y) from Brighton Valley DWM at structure CS-O1, CS-O2, and CS-O3.....	29
Figure 10. Simulated TP concentration at the inflow (orange line) and outflow from CS-O1 (green line) from the Brighton Valley DWM.	30
Figure 11. Annual simulated inflow TP loads (mt/y) to Brighton Valley DWM.	30
Figure 12. Annual simulated outflow TP loads (mt/y) from Brighton Valley DWM at structures CS-O1, CS-O2, and CS-O3.....	31
Figure 13. Locations of discharge structures S-1 and S-9 from the IMWID to C-41.	33
Figure 14. Location of the IMWID impoundment and adjacent reaches.	34
Figure 15. IMWID impoundment profile (WAM reach 294) used for the simulation with the maximum stage and mean bottom elevation indicated.	35
Figure 16. Simulated flows out of (positive values) and out of (negative values) the IMWID impoundment (upper pane), and water depths within the impoundment (lower pane) during the 1993-2013 simulation period.	36
Figure 17. Simulated TP concentrations at IMWID structure S-1 comparing the base scenario (green line) and the scenario including the IMWID impoundment (orange line).	36
Figure 18. Simulated TP load discharging at IMWID structure S-1 comparing the base scenario (green bars) and the scenario including the IMWID impoundment (orange bars).....	37
Figure 19. Simulated TP concentrations at IMWID structure S-9 comparing the base scenario (green line) and the scenario including the IMWID impoundment (orange line).	37
Figure 20. Simulated TP loads discharging at IMWID structure S-9 comparing the base scenario (green bars) and the scenario including the IMWID impoundment (orange bars).....	38
Figure 21. The LOWRP Tentatively Selected Plan.....	39
Figure 22. The location of the proposed WAF and Paradise Run wetland restoration site in the Indian Prairie sub-watershed.....	41
Figure 23. Base scenario (left) and LOWCP WAF scenario (right).....	42
Figure 24. Simulated annual pumped inflow volumes to the K-05 WAF from C-41A over the 1993-2013 time frame.	45
Figure 25. Simulated annual discharge from the K-05 WAF over the 1993-2013 time frame.	45
Figure 26. Simulated TP concentration at the inflow (orange line) and outflow (green line) from the K-05 WAF.	46
Figure 27. Simulated annual pumped inflow TP loading to the K-05 WAF from C-41A over the 1993-2013 time frame.	46

Figure 28. Simulated annual TP discharge loading from the K-05 WAF into C-38 over the 1993-2013 time frame.	47
Figure 29. The Kissimmee River Center restored wetland scenario setup (right pane), compared with the base scenario (left pane).....	48
Figure 30. Comparison of simulated TP concentrations at the inflow point from the Kissimmee River to the Kissimmee River Center wetlands (orange line) and discharge concentrations back to the river from the wetlands (green line).	49
Figure 31. The WAM setup for the Taylor Creek/Nubbin Slough sub-watershed, with the location of the LRSTA and BRSTA shown.	51
Figure 32. Phase 1 and 2 of the LRSTA (Figure 1 in CDM Smith 2018).	53
Figure 33. LRSTA structures and canals. (Figure 4 in CDM Smith 2018).	54
Figure 34. STA cell layout and flow directions in LRSTA (Figure 5 in CDM Smith 2018).	55
Figure 35. WAM setup of the LRSTA and BRSTA. WAM reach numbers are shown for reach.	57
Figure 36. The cross-sectional profile for WAM reach 138 representing LRSTA-N used for the simulation with the mean bottom elevation indicated.	58
Figure 37. The WAM setup for LRSTA/BRSTA indicating the placement of the S-191A pump station and C-59.	60
Figure 38. Comparison of simulated TP concentrations between inflow to LRSTA-N (orange line) and outflow from LRSTA-S (green line).	62
Figure 39. Box-and-whisker plots comparing the distribution of simulated LRSTA inflow TP concentrations (left box) and LRSTA outflow concentrations (right box). Note that the y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.037 mg/L).	62
Figure 40. Simulated flows (upper pane) and water depths (lower pane) in LRSTA-N over the simulation period 1993-2013.	63
Figure 41. Simulated lows (upper pane) and water depths (lower pane) in LRSTA-S over the simulation period 1993-2013.	63
Figure 42. Comparison of simulated TP concentrations between inflow (orange line) and outflow (green line) from BRSTA.	64
Figure 43. Box-and-whisker plots comparing the distribution of simulated BRSTA inflow TP concentrations (left box) and BRSTA outflow concentrations (right box). Note that the y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.039 mg/L).	65
Figure 44. Simulated flows (upper pane) and water depths (lower pane) in BRSTA over the simulation period 1993-2013.	65
Figure 45. The WAM setup for the FEC sub-watershed, with the location of the FAVT and Nicodemus Slough projects shown.	68
Figure 46. Detailed FAVT system configuration (from Water & Soil Solutions 2018)	69
Figure 47. The WAM setup for the FAVT treatment area on Fisheating Creek. WAM reach numbers are indicated for each reach.	71
Figure 48. The cross-sectional profile for WAM reach 228 in the FAVT impoundment used for the simulation with the mean bottom elevation indicated.	72
Figure 49. Comparison of simulated inflow (orange line) and outflow (green line) TP concentrations from the FAVT system.	74
Figure 50. Box-and-whisker plots comparing the distribution of simulated inflow (left box) and outflow (right box) TP concentrations for the FAVT. The y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.105 mg/L).	75

Figure 51. Simulated flows (upper pane) and water depths (lower pane) in the FAVT cell 1 over the simulation period 1993-2013.	75
Figure 52. Simulated flows (upper pane) and water depths (lower pane) in the FAVT cell 2 over the simulation period 1993-2013.	76
Figure 53. Schematic diagram of the Nicodemus Slough project (from SFWMD 2014a).	78
Figure 54. The WAM setup for the Nicodemus Slough Structure.	79
Figure 55. The cross-sectional profile for WAM reach 201 in Nicodemus Slough used for the simulation with the mean bottom elevation indicated.	80
Figure 56. Simulated inflows to the Nicodemus Slough project area.	82
Figure 57. Comparison of simulated TP concentrations at the inflow to Nicodemus Slough to concentrations in the Southeastern basin.	83
Figure 58. Simulated outflows from C-5 (upper pane) and stages within the Southeastern basin. Open and close trigger elevations for the gate structure at C-5 are shown in the lower pane (green and magenta lines), together with the mean bottom elevation of the Southeastern basin (red line).	83
Figure 59. Simulated outflows from the Nicodemus Slough project area at Structure S-342.	84
Figure 60. Simulated outflow (upper pane) and water depths (lower pane) in the Western basin.	84
Figure 61. Simulated outflow (upper pane) and water depths (lower pane) in the Central basin.	85
Figure 62. Simulated outflow (upper pane) and water depths (lower pane) in the Eastern basin.	85
Figure 63. The WAM setup for the Upper Kissimmee sub-watershed, with the location of the RMWRP shown.	88
Figure 64. Location of Parcel B in the RMWRP (from SFWMD 2014b)	89
Figure 65. The layout of the flow system in parcel A. From SFWMD (2014b)	90
Figure 66. The WAM setup for RMWRP Parcel B.	91
Figure 67. The cross-sectional profile for WAM reach 532 in the RMWRP used for the simulation with the mean bottom elevation indicated.	92
Figure 68. Comparison of simulated TP concentrations between inflow to RMWRP Parcel B (orange line) and outflow to Lake Hatchineha (green line) over the 1993-2013 simulation period.	93
Figure 69. Simulated flows in RMWRP over the simulation period 1993-2013	94
Figure 70. Accumulated measured and HRS simulated volumes at S-65.	96
Figure 71. Location of the Kissimmee River in Florida with completed and projected construction phases indicated. Prior to restoration backfilling, the C-38 canal cut through the floodplain and intercepted the meandering river channel. Koebel et al. 2019.	98
Figure 72. The fully restored river channel and floodplain wetlands feature classes under the completed KRR project obtained from the SFWMD.	99
Figure 73. Comparison of land uses within the Lower Kissimmee base scenario (left pane, SWET 2017a) with those in the WAM setup for the full KRR project (right pane).	101
Figure 74. Comparison of the WAM reaches within the Lower Kissimmee base scenario (left pane, SWET 2017a) with those in the WAM setup for the KRR project (right pane).	102
Figure 75. Diagram of the cross-sectional profile for WAM reach 17. The main channel is indicated by the green shading, while the broad floodplain is denoted by the blue lines.	103
Figure 76. Comparison of simulated TP concentrations discharging at S-65E. The base scenario is indicated by the green line and the full KRR project is indicated by the orange line.	104
Figure 77. Comparison of simulated annual TP loads discharging at S-65E. The base scenario loads are shown by the green bars, while the full KRR project loads are shown by the orange bars.	104
Figure 78. Simulated annual discharge volumes at S-65E. The base scenario volumes are shown by the green bars, while the full KRR project volumes are shown by the orange bars.	105

Tables

Table 1. WAM Predicted N and P Load Reductions and Cost Effectiveness for Scenarios 1 and 2.....	11
Table 2. An example depth-to-width relationship for a WAM reach.	15
Table 3. Capital and O&M Costs for LOWPP and LOWRP Scenarios 3 and 4 Projects (SFWMD).	17
Table 4. Weir crest elevations and widths for the default pumping mode (SFWMD 2018b).	23
Table 5. Weir crest elevations for storm mode (SFWMD 2018b).....	23
Table 6. Weir crest elevations for off mode (SFWMD 2018b).	24
Table 7. Cross-sectional profile data for two representative reaches (162, a slough, and 167, a canal) in Brighton Valley DWM.	26
Table 8. Land use acreages in the Brighton Valley impoundment basins.	27
Table 9. Comparison of simulated annual average volumes and TP loads from the base and LOWCP scenarios over the 1993-2013 period.	28
Table 10. Cross-sectional profile of the IMWID impoundment (WAM reach 294).	32
Table 11. Pump parameters for drainage mode (SWET 2012).	32
Table 12. Simulated annual average volumes and TP loads discharging into C-41 from the IMWID from 1993-2013.	38
Table 13. Simulated annual average volumes and TP loads discharging from the Indian Prairie sub-watershed.	38
Table 14. Lake Okeechobee stages for diversions to and/or from the K-05 WAF.	43
Table 15. Weir crest elevations within the K-05 WAF. Default elevations are used during non-drainage phases.	43
Table 16. Pumping operations between the K-05 WAF and ASR.	44
Table 17. Parameters for WAM reaches representing the K-05 WAF.	44
Table 18. Annual average simulated volume and TP load results from the WAM K-05 WAF simulation.	47
Table 19. Reach parameters for LRSTA and BRSTA.	56
Table 20. Reach cross-sectional profiles for the LRSTA and BRSTA.	58
Table 21. LRSTA and BRSTA pump station parameters used in WAM (CDM Smith 2018). On/off triggers are stages in C-59.	59
Table 22. Weir parameters used in WAM for the LRSTA and BRSTA.	61
Table 23. S-191A pump station parameters used in WAM. On/off triggers are L-47 stages.	61
Table 24. Simulated combined LRSTA and BRSTA TP load reductions.	66
Table 25. Simulated annual average volumes and TP loads discharging from the Taylor Creek/Nubbin Slough sub-watershed.	66
Table 26. WAM reach parameters for cells within the FAVT.	70
Table 27. Reach cross-sectional profiles for the FAVT.	72
Table 28. FAVT pump station parameters used in WAM. On/off triggers are Fisheating Creek stages.	73
Table 29. FAVT weir parameters used in WAM.	73
Table 30. Reach parameters for WAM reaches in Nicodemus Slough.	80
Table 31. Reach cross-sectional profiles for Nicodemus Slough.	80
Table 32. Nicodemus Slough pump station #1 and #2 parameters (SFWMD 2014a).	81
Table 33. Parameters for weirs within the Nicodemus Slough project (SFWMD 2014a).	81
Table 34. Simulated annual average volumes and TP loads discharging from the Fisheating Creek sub-watershed. The “LOWCP Inflow” TP load indicates loading from the lake into Nicodemus Slough during pumping.	86
Table 35. Weir parameters in the RMWRP (updated in SFWMD 2018c).	88
Table 36. Parameters for WAM reaches that comprise Parcel B.	92
Table 37. Reach cross-sectional profiles for the RMWRP impoundments.	92
Table 38. Comparison of simulated Upper Kissimmee Base and RMWRP results.	94
Table 39. Restored wetland types given in the SFWMD floodplain delineation and the corresponding WAM land use type and ID.	100

Table 40. Land use acreage changes from the base scenario to the KRR project scenario.....	100
Table 41. Cross-sectional profiles for WAM reaches representing the restored Kissimmee River channel and floodplain. The main river channel is represented by the narrower inner part of the profile and the floodplain is represented by the wide (>1000 feet) part of the profile. All widths and depths are in feet.	102
Table 42. Simulation results comparing annual average volume (ac-ft/y) and TP loads (mt/y) between the WAM base and KRR simulations	105
Table 43. Estimated nutrient load reductions and costs for Scenarios 3 and 4.	108

List of Abbreviations

AHED	ArcHydro Enhanced Database
APPZ	Avon Park Permeable Zone
ASR	Aquifer Storage and Recovery
BLASRoute	Basin Land Area to Stream Routing
BMAP	Basin Management Action Plan
BMP	Best Management Practice
BRSTA	Brady Ranch Stormwater Treatment Area
BUCShell	Basin Unique Cell Shell (command line shell program)
C&SF	Central & Southern Florida
CERP	Comprehensive Everglades Restoration Plan
CFS	Cubic Feet per Second
DWM	Dispersed Water Management
EAA	Everglades Agricultural Area
EAAMod	Everglades Agricultural Area Model
EIS	Environmental Impact Statement
ESW	East Sub-Watershed (of the LOW)
FAVT	Floating Aquatic Vegetative Tilling
FDACS	Florida Department of Agriculture and Consumer Services
FDEP	Florida Department of Environmental Protection
FEC	Fisheating Creek
GIS	Geographic Information System
GLEAMS	Groundwater Loading Effects of Agricultural Management Systems
HRS	Headwaters Revitalization Schedule (for the Kissimmee River)
IFAS	Institute of Food and Agricultural Sciences
IMWID	Istokpoga Marsh Watershed Improvement District
KRR	Kissimmee River Restoration
KRREP	Kissimmee River Restoration Evaluation Program
LORS	Lake Okeechobee Regulation Schedule
LOW	Lake Okeechobee Watershed
LOWCP	Lake Okeechobee Watershed Construction Projects
LOWPP	Lake Okeechobee Watershed Protection Plan
LOWRP	Lake Okeechobee Watershed Restoration Project
LRSTA	Lakeside Ranch Stormwater Treatment Area

MT	Metric Tons
N	Nitrogen
NAVD	North American Vertical Datum of 1988
NGVD	National Geodetic Vertical Datum of 1929
O&M	Operation and Maintenance
P	Phosphorus
PIR	Project Implementation Report
POR	Period of Record
RMPBWRP	Rolling Meadows Parcel B Wetland Restoration Project
SFER	South Florida Environmental Report
SFWMD	South Florida Water Management District
SSW	South Sub-Watershed (of the LOW)
STA	Stormwater Treatment Area
SWET	Soil and Water Engineering Technology, Inc.
TSP	Tentatively Selected Plan
TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphorus
UBI	Upper Basin Initiatives
UF	University of Florida
UFA	Upper Floridan Aquifer
USACE	United States Army Corps of Engineers
WAF	Wetland Attenuation Feature
WAM	Watershed Assessment Model
WSW	West Sub-Watershed (of the LOW)

1 Executive Summary

The Watershed Assessment Model (WAM) was used by Soil and Water Engineering Technology, Inc. (SWET) to simulate the existing conditions as of 2013 (SWET 2017a, 2017b) and pre-drainage conditions (SWET 2018) for the Lake Okeechobee Watershed as designated in the Lake Okeechobee Basin Management Action Plan (BMAP) (FDEP 2014). The Lake Okeechobee Watershed includes six northern sub-watersheds (Taylor Creek/Nubbin Slough, Fisheating Creek, Indian Prairie, Lower Kissimmee River, Upper Kissimmee River, and Lake Istokpoga) and three southern sub-watersheds (East Lake Okeechobee, West Lake Okeechobee, and South Lake Okeechobee). The goal of the project is to evaluate the cost-effectiveness of various future abatement strategies for reducing nutrient loads to Lake Okeechobee as well as their impact to the drainage system in the Lake Okeechobee Watershed by implementing these strategies.

The objective of this deliverable is to assess the relative effectiveness of the most promising hydrological and nutrient abatement programs, such as Best Management Practices (BMPs) and other technologies, for meeting Lake Okeechobee Watershed Protection Plan (LOWPP) and BMAP targets. A forthcoming final comprehensive report will include a comparison against pre-drainage conditions. Specifically, this project estimated the flow and nutrient load reduction potential of the following four abatement scenarios by comparing their simulated flows, nutrient concentrations, and nutrient loads going into Lake Okeechobee to those for the existing base scenario for each sub-watershed:

1. Advanced fertility management BMPs for major crops and urban landscapes in all sub-watersheds (<https://www.freshfromflorida.com/Business-Services/Water/Agricultural-Best-Management-Practices>).
2. Advanced farm-level and urban stormwater water management BMPs, including additional on-farm and increased urban water retention in all sub-watersheds (<https://www.sfwmd.gov/our-work/water-storage-strategies>).
3. (a) The existing and approved Lake Okeechobee Watershed Construction Projects (LOWCP) and (b) the proposed Lake Okeechobee Watershed Restoration Program (LOWRP), which include reservoirs, wetland restoration, and other features. The evaluated projects in the LOWCP were Brighton Valley Dispersed Water Management (DWM), Istokpoga Marsh Watershed Improvement District (IMWID) (Phase 2), Lakeside Ranch and Brady Ranch Stormwater Treatment Areas (LRSTA and BRSTA), Fisheating Creek Floating Aquatic Vegetative Tilling (FAVT), Nicodemus Slough, Rolling Meadows Wetland Restoration Project (RMWRP). The evaluated project in the LOWRP was the planned LOWRP K-05 Wetland Attenuation Feature (WAF) with Aquifer Storage and Recovery (ASR) and the Paradise Run and Kissimmee River Center wetlands.
4. The fully implemented Kissimmee River Restoration (KRR) Project and Upper Basin Initiatives (UBI), which includes the Kissimmee River Headwaters Revitalization Schedule (HRS) and the Kissimmee River Restoration Evaluation Program (KRREP)

An economic assessment of the above BMP scenarios was developed to provide cost-effectiveness (defined as the cost per pound of nutrient reduction) estimates for each implemented BMP. Such information will allow water resource managers to better optimize the use of the limited funding resources as they come available. BMP cost effectiveness was developed by updating the values from the study entitled “Phosphorus Reduction Performance and Implementation Costs under BMPs and Technologies in the Lake Okeechobee Protection Plan Area” by SWET (2006) and later updated by SWET (2008) for the Caloosahatchee and St. Lucie River Watersheds. These BMP costs were updated by SWET based on inputs from University of Florida, a literature review of recent BMP studies including the Florida Department of Agriculture and Consumer Services’ (FDACS) BMPs literature database (FDACS

2014), and crop production data recently generated by SWET for crop damages assessments associated with the Sabal Trail pipeline.

The WAM simulated nitrogen (N) and phosphorus (P) load reductions of the four scenarios were calculated for each of the nine sub-watersheds. The predicted P reductions across all sub-watersheds were found to be higher than predicted N reductions, e.g., 19% for P versus 9% for N for scenario 1 (fertility BMPs) and 29% for P versus 12% for N for scenario 2 (water management BMPs). The lower N response is expected due to the naturally high organic N concentrations in the watershed versus the N as nitrates being anthropogenically generated, which are typically denitrified to the atmosphere as N₂ gas before reaching the Lake. The high denitrification of N is why N has not been the focus in the Okeechobee Watershed. The simulated phosphorus reductions for the fully implemented fertility (scenario 1) and water retention (scenario 2) BMPs indicate that they will not be sufficient to meet the phosphorus reduction target in the Lake Okeechobee BMAP, which is nearly an 80% reduction.

The water management BMPs (typically stormwater retention) provide a greater net P load reduction potential but at a higher cost than the fertility BMPs. Even though fertility BMPs (scenario 1) have lower net load P reduction potential, they were found to be more cost-effective for both agricultural and urban landscapes than water management BMPs. Therefore, fertility BMPs should be a high priority for implementation. The higher cost of the urban BMPs compared to agricultural BMPs resulted in urban sub-watersheds, such as Upper Kissimmee, having much higher net cost effectiveness values. The agricultural fertility BMPs were found to be about \$15 per pound of P removed compared to about \$150 per pound of P for water management BMPs. In contrast, urban fertility BMPs were found to be about \$250 per pound of P removed compared to about \$2500 per pound of P for water management BMPs. A combination of scenario 1 and 2 BMPs, which were not simulated, will increase net nutrient reductions for both N and P over what can be achieved by the individual scenarios. However, the resulting increase will only be marginally larger since the implementation of one BMP will typically reduce the effectiveness of other BMPs.

The next two scenarios evaluated were the LOWCP and LOWRP projects (scenario 3) and the fully implemented KRR Project and Upper Basin Initiatives (HRS and KRREP) (scenario 4). The P reductions for these projects were calculated by the difference between the pre- versus post-construction WAM simulations based on detailed design data and operational protocols provided the SFWMD. These P reductions were combined with cost data also provided by SFWMD to estimate the relative cost-effectiveness of the various projects.

The scenario 3 projects were estimated to provide a total of 37.9 mt/y of P reduction going into Lake Okeechobee, where the Lakeside Ranch (11.8 mt/y) and the planned LOWRP reservoir and ASR (9.8 mt/y) projects provided the two largest reductions. The cost-effectiveness in terms of dollars per pound of P removed ranged from \$6,500/lb-P for the LOWRP K-05 WAF and ASR project to a low of \$186/lb-P for the Brady Ranch project.

The scenario 4 KRR and UBI projects, which were run as a group, were estimated to provide a total of 31.8 mt/y of P reduction going into Lake Okeechobee with the KRR project representing most of this reduction. The cost-effective of these projects was estimated to be about \$990/lb-P.

The findings from the project hopefully will enable the Coordinating Agencies to better understand which projects and BMPs by land use will provide the largest, quickest, and most cost-effective benefits. In addition, WAM was used to assist in evaluating existing base conditions and evaluating the performance and impact on water quality of the proposed project features in future conditions that are under consideration in planning projects (such as the LOWRP). A variety of land management practices, source area treatment technologies, and regional treatment systems have been implemented or proposed to improve the hydrology and water quality of stormwater entering Lake Okeechobee. The effectiveness of these programs and treatment projects around Lake Okeechobee have been highly variable, but most have demonstrated that they can be effective in reducing nutrient loads to Lake Okeechobee (SFWMD 2014c).

95 However, as specified in the LOWPP and Lake Okeechobee BMAP, significant nutrient load reductions
96 (nearly 80% for P) will be needed to meet the Total Maximum Daily Load (TMDL) target for Lake
97 Okeechobee. This study's more detailed information on how to rank these various nutrient abatement
98 options as to their cost-effectiveness (dollars per pound of nutrient removed) will help water resource
99 managers to better optimize the limited resources for achieving the BMAP goals. The refined watershed
100 modeling results will also assist in the adaptive management and implementation of LOWRP.

101

2 Introduction and Background

The Watershed Assessment Model (WAM) was used to evaluate the impact of alternative land uses, implementation of BMPs, and nutrient load reduction projects for the LOW (Bottcher et al. 2012; SWET 2017a, 2017b). In addition to these applications, Task 2 of this study used WAM to simulate pre-drainage conditions (SWET, 2018). Task 2 estimated the native flow and nutrient load conditions that would have existed in the LOW prior to any anthropogenic activities in support of abatement programs, such as the LOWPP and the Lake Okeechobee BMAP, and in the federal planning process for the Comprehensive Everglades Restoration Plan (CERP). For the current task/deliverable, SWET used WAM to determine the effectiveness of several planned abatement strategies for reducing nutrient loads to Lake Okeechobee and the impact of those strategies on the drainage system in the LOW (**Figure 1**) and ultimately Lake Okeechobee.

WAM is a process-based model that can be used to perform hydrological and water quality analyses to:

- Simulate flows and nutrient loads for existing and pre-drainage land uses, soils, and land management practices.
- Analyze the hydrological and water quality impacts on streams and lakes for management scenarios, such as land use changes, implementation of BMPs, or the addition of regional reservoirs and stormwater treatment areas.
- View and analyze the simulated flow and concentrations for every stream reach in the sub-watershed using the ArcMap™ platform.
- Identify high total TP concentration area (hot spots) and prioritize these areas where BMP efforts are to be focused.
- Provide quantitative comparisons of flows and nutrient loads between pre-drainage (i.e., prior to development), existing, and future conditions.

The SFWMD, the Florida Department of Environmental Protection (FDEP), and FDACS, hereinafter collectively referred to as the Coordinating Agencies, have identified WAM as the best tool for performing such analyses in the LOW.

The LOW consists of nine sub-watersheds. Six of these are north and upstream of Lake Okeechobee, while the remaining three are located immediately east, south, and west of the lake and are generally downstream of the lake but in certain cases may discharge to the lake. In what follows, the three southernmost sub-watersheds will be referred to as the East sub-watershed (ESW), South sub-watershed (SSW), and West sub-watershed (WSW). A calibrated WAM simulation was developed to estimate water flows and nutrient loads across all the nine sub-watersheds in the LOW (SWET 2017a, 2017b), including the hydrologically connected southern part of the Everglades Agricultural Area (EAA) that does not lie within the Lake Okeechobee BMAP area. The simulation period of record (POR) was from 1975 through the end of 2013. The simulations for the northern six sub-watersheds were built upon earlier work (Bottcher et al. 2012), while the simulations of the three southern sub-watersheds were a new application. These simulations will serve as the reference estimates for existing conditions for the current project. After consultation with SFWMD staff, it was decided that this current project would use a POR from 1993 through the end of 2013 for all simulations because it is representative of the current land use condition in the LOW.

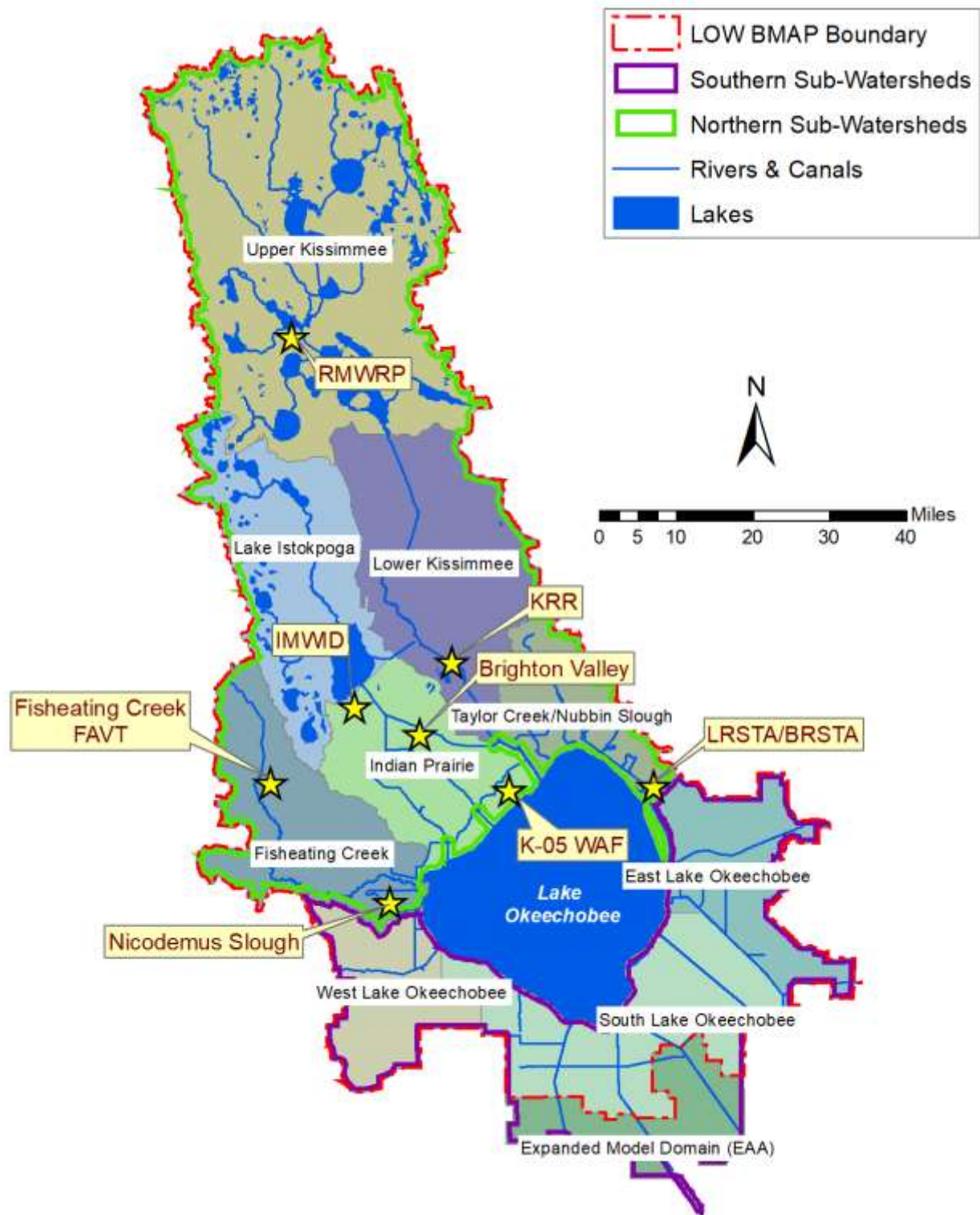


Figure 1. The nine sub-watersheds of the LOW showing the LOWCP, LOWRP, and KRR projects simulated for this study.

2.1 Project Description

This project used the calibrated WAM setup (SWET 2017a, 2017b) to evaluate the effectiveness of future abatement strategies for reducing nutrient loads to Lake Okeechobee and the impact of those strategies to the drainage system in the Lake Okeechobee Watershed. This task will provide the Coordinating Agencies additional information on how existing and proposed projects as well as more advanced BMPs can be prioritized to provide the largest, quickest, and most cost-effective benefits for meeting the BMAP targets. In addition, WAM was used to assist in evaluating existing base conditions and evaluating the performance and impact on water quality of the proposed project features in future conditions that are under consideration in planning projects, such as the LOWRP. A variety of land management practices, source area treatment technologies, and regional treatment systems have been implemented or proposed to improve the hydrology and water quality of stormwater entering Lake Okeechobee. The effectiveness of these programs and treatment projects around Lake Okeechobee have been highly variable, but most have demonstrated that they can be effective in reducing nutrient loads to Lake Okeechobee (SFWMD 2014c). However, as specified in the LOWPP and Lake Okeechobee BMAP, significant nutrient load reductions, particularly P, will be needed to meet the TMDL target for Lake Okeechobee. It is critical to obtain more detailed information on how to rank these various nutrient abatement options as to their cost-effectiveness, i.e., produce quantitative data for optimizing the limited resources. The refined watershed modeling results will also assist in the adaptive management and implementation of LOWRP.

2.2 Project Objectives

The objective of this project is to assess the relative effectiveness of the most promising hydrological and nutrient abatement programs and associated BMPs/technologies for meeting LOWPP/BMAP targets across the entire LOW (**Figure 1**) by comparing them to existing conditions. A final comprehensive report will include comparison against pre-drainage conditions. As an earlier task within this project SWET completed pre-drainage WAM simulations for the nine sub-watersheds. The current task is the determination of flow and nutrient load reduction potential of the following specific abatement scenarios, which are representative of the available technologies to the existing drainage conditions (the base WAM scenario) for each sub-watershed:

1. Advanced fertility management BMPs for major crops and urban landscapes in all sub-watersheds (<http://www.freshfromflorida.com/Business-Services/Water/Agricultural-Best-Management-Practices>).
2. Advanced farm-level and urban stormwater water management BMPs, including additional on-farm and increased urban water retention in all sub-watersheds (<https://www.sfwmd.gov/our-work/water-storage-strategies>).
3. (a) The existing and approved LOWCP (**Sections 4.2.2, 4.2.3, 4.3.1, 4.4.1, 4.4.2, and 4.5.1**) and (b) the proposed LOWRP K-05 WAF/ASR, including reservoirs, wetland restoration, and other features (**Section 4.2.5**).
4. The fully implemented KRR Project and UBI (HRS and KRREP) (**Section 5**).

An economic assessment of the above BMP scenarios was developed so that cost-effectiveness estimates can be generated in terms of dollars spent per pound of P removed. Such information will allow water resource managers to better optimize the use of the limited funding resources as they come available. BMP cost effectiveness was developed by updating the values from the study entitled “Phosphorus Reduction Performance and Implementation Costs under BMPs and Technologies in the Lake Okeechobee Protection Plan Area” by SWET (2006) and later updated by SWET (2008) for the Caloosahatchee and St. Lucie River Watersheds. These BMP costs were updated by SWET based on inputs from University of Florida Institute of Food and Agricultural Sciences, UF-IFAS crop experts,

190 literature review of recent BMP studies and literature database (FDACS 2014), and crop production data
191 recently generated by SWET for crop damages assessments associated with the Sabal Trail pipeline.

192 The costs of the phosphorus abatement scenarios are presented in terms of a total annual cost to the
193 landowner, which includes the amortized cost of any capital expenditures over the expected life of the
194 BMP or project plus its annual operation and maintenance (O&M) costs. These annual costs were divided
195 by the WAM predicted annual N and P reductions associated with the particular BMPs and projects,
196 which provides total cost per pound of P removed per year from that particular feature. The cost per
197 pound of P removal allows for comparison of the various evaluated scenarios.

3 Advanced BMPs

3.1 Analyses for the Simulated BMP Scenarios

The following assessment of an advanced BMP implementation program is intended to provide cost-effectiveness estimates based on available information. The BMP scenarios to be evaluated are for the full implementation of fertility (scenario 1) and water management BMPs (scenario 2) to pick up parcels where they have not already been implemented and for more advanced BMPs for parcels that already have a base level of BMPs implemented. These BMPs and their cost effectiveness values, as presented above, were built upon the previous work completed by Bottcher (2006) and SWET (2008) along with more recent information from FDACS (2014) and other SWET experiences.

3.2 Advanced Fertility/Nutrient and Water Management BMPs

Advanced fertility and water management BMPs were incorporated into the simulations through application of BMPs to the base land use feature class. In general, to simulate fertility management BMPs, the amount and/or timing of fertilizer application were changed to be consistent with recommendations provided in the FDACS BMP manuals for the various commodity groups. These manuals reference UF-IFAS fertility practices associated with proper soil and tissue testing. Similarly, the applied water management BMPs are consistent with the FDACS BMP manuals and SFWMD/FDEP dispersed water storage program, except for the EAA where practices presented by Bottcher and Izuno (1994) associated with the SFWMD EAA BMP regulatory program were applied. Fertility and water management BMPs were applied separately (i.e., in different simulations) to determine the influence of each BMP category. The set of fertility/nutrient BMPs applied across all land uses is referred to collectively as “Scenario 1” while the applied advanced water management BMPs are collectively referred to as “Scenario 2.” If future needs require it, extended simulations may be run to examine the results of combining different types of BMPs across the basin. Note that the P reductions for scenarios 1 and 2 cannot be added together to get their net effect because the effectiveness of one BMP can significantly lower the benefits of the second BMP.

The list of the land uses that have fertility/nutrient and water management BMPs developed for them is provided in **Table A.1** in **Appendix A** for three representative soils, i.e., a well-drained sandy soil (Archbold), a flatwood poorly drained soil (Immokalee), and an EAA muck soil (Terra Ceia). In this table the “base / existing conditions” scenario results are highlighted in orange while the fertility BMP simulations results are highlighted in green and the water management BMP simulations (“Scenario 2 – water”) are highlighted in blue. Only the BMPs applied for scenarios 1 and 2 are shown in **Table A.1**, but please note that parcels that had special BMPs implemented under the FDACS BMP program are represented in the base scenario so were not changed for these BMP scenarios. The advanced fertilizer/nutrient BMPs used for Scenario 1 represent a more aggressive fertilizer management program where tissue analyses were assumed to be routinely used to determine fertilizer needs and applications were done in a more “spoon fed” manner via fertigation in sync with the crop growth cycle. The labor and equipment associated with these advanced practices are represented in the costs shown in **Table A.1**. It should be noted that through efforts by the University of Florida Cooperative Extension and FDACS’ BMP program, particularly in the Okeechobee Watershed there has already been a significant adaption of the IFAS’ recommended fertility practices. Therefore, the marginal benefits for additional adaption of nutrient BMPs is much smaller today than it would have been twenty years ago. The advanced water management BMPs primarily involve the implementation of stormwater retention/detention systems for land uses that typically are not currently using them, such as improved pasture, or the expansion and inclusion of retention/detention systems with irrigation reuse for those that do currently have stormwater systems. Advanced irrigation practices are also implemented under the water management BMPs where

irrigation is being used. These retention systems are expensive but can provide substantially more nutrient reductions than the fertility BMPs.

Table A.1 provides example WAM predicted annual average N and P loads that will discharge from a source via surface and groundwater flow. These values are for a typical well drained soil (Archbold), poorly drained sandy soil (Immokalee), and muck soil (Terra Ceia) using one rain station (rain zone number 5) in the Lower Kissimmee River sub-watershed. Therefore, these values may vary for other soil types and rain zones but should be representative on a relative basis. Note that P values for the Archbold soil are not included because this well-drained soil will have little if any runoff, which means most of the reported discharge will be recharge to groundwater. Because little P will leach to groundwater, the P loading rates for the Archbold soil are extremely small, which means that percentage differences can vary significantly, although in reality there is little difference in measurable phosphorus. Therefore, P reduction data for the Archbold soil was considered too small and unreliable and therefore would not be meaningful to report. To review the parameter adjustments made for the various BMPs, the reader can compare the parameter sets via the WAM interface or do a direct comparison of the land use parameter files '*landuse.bnz*' and '*lueaa.bnz*' for the "base" run to the BMP scenario of interest.

Costs of the abatement BMPs presented in the BMP spreadsheet are in terms of a total annual cost to the landowner, which includes the amortized cost of any capital expenditures over the expected life of the BMP plus its annual O&M costs. The costs are provided in present-day (2018) value, so that appropriate comparisons can be made. This means all capital expenditures for the implementation of existing, new, or expanded BMPs were brought to a 2018 present-day value based on their remaining or expected life-expectancy and current interest rates. Life-expectancies ranged from 10 to 20 years using a current interest rate of 6%. Accounting for BMPs already in place was done based on inputs from FDACS associated with their BMP program. A recent study done by SWET (2016) provided a comprehensive summary of the status of FDACS cost-share funded BMPs in the Okeechobee basin, which were represented in the base scenario during its calibration. The existing BMPs versus future BMPs were handled by adjusting the base scenario (existing conditions) land use parameter datasets to reflect the percent of implementation based on the SWET (2016) study. These land use parameter datasets were then updated for the BMP scenarios 1 and 2 to reflect the full implementation of just the nutrient/fertility BMPs for scenario 1 and then just the water management BMPs for scenario 2 as required in the SOW. For land uses that do not have cropped components, such as cattle feeding operations, the BMPs are handled using the BMP factors in the Landuse.bnz file. However, land uses like row crops will have their fertilizer rates parameters in the Landuse.bnz (well-drained soils) and LUEaa.bnz (poorly drained soils) files reduced about 8% and 30% for N and P, respectively, and for the water management BMPs the retention volume increased by 50%.

The advanced BMPs identified for scenarios 1 and 2 are assumed to be applied to one hundred percent of the land parcels, which means that all parcels would be brought up to the new BMP level regardless if it had existing or no BMPs under the base scenario. The advanced BMPs are assumed to be quite aggressive and therefore parcels with existing BMPs are expected to also see significant nutrient reductions by their implementation, but at an additional cost. For example, for the Everglades Agricultural Area (EAA) the advanced BMPs are more robust than for the upper sub-watersheds because the EAA regulatory program already requires a high level of BMP implementation, which have achieved between 40% to 70% reductions in P going to the conservation areas. The most important advanced BMP is water retention and irrigation reuse, which can have dramatic benefits but at a very high cost.

The O&M costs were also based on 2018 present-day value, where it includes the additional labor, energy, and equipment maintenance costs associated with the BMPs. This type of economic analysis was included in the 2007 Lake Okeechobee Protection Plan Update (SFWMD et al. 2007). The cost of agricultural BMPs are mostly O&M related expenses, whereas urban BMPs and the larger restoration type projects have significantly more capital expenditures. These annual costs are then divided by the WAM predicted annual P reductions associated with the particular BMPs, which provides the total cost

per pound of N and P removed per year from that particular BMP or cost effectiveness (**Table A.1** in **Appendix A**). The cost per pound of nutrient removed allows for comparison of the various evaluated BMPs, but only for the three soil conditions and rain zone used for the spreadsheet. The assessment of the net nutrient reductions and cost benefits across the various sub-watersheds are provided next.

The annual nutrient load reductions and cost for scenarios 1 and 2 for each of the nine sub-watersheds are provided in **Table 1**. As can be seen, forcing the full implementation across all the land uses will provide significant nutrient load reductions to Lake Okeechobee but at a significant cost. The fertility/nutrient BMPs are more cost-effective on average than the water management BMPs but have a significantly less net reduction potential. The reason for the larger cost and cost-effectiveness differences between the different sub-watersheds is a ratio of the urban versus agricultural BMPs being implemented. In particular, urban water management BMPs, which are primarily enhanced retention systems, are significantly more costly than agricultural systems. In addition, urban BMPs will typically have lower P concentration entering them reducing their efficacy. The WSW cost-effectiveness value is very high because it is being calculated based on N and P load reductions going into Lake Okeechobee, which are very small because the majority of the sub-watershed's discharge is west to the Caloosahatchee River. The ESW and SSW sub-watersheds also have high cost-effectiveness values for the same reason. Clearly an optimized combination of the various BMPs based on their source cell cost-effectiveness values found in **Appendix A** and combined with their relative location within the Lake Okeechobee will provide the best overall cost efficiency for abatement programs.

312 **Table 1. WAM Predicted N and P Load Reductions and Cost Effectiveness for Scenarios 1 and 2.**

Sub-watershed	Base Scenario		Scenario 1 (Nutrients)					Scenario 2 (Retention)				
	N	P	N	P	Cost	Cost Effectiveness		N	P	Cost	Cost Effectiveness	
	(lbs/y)	(lbs/y)	(lbs/y)	(lbs/y)	(\$M/y)	(\$/lb-N)	(\$/lb-P)	(lbs/y)	(lbs/y)	(\$M/y)	(\$/lb-N)	(\$/lb-P)
Upper Kissimmee	2,515,625	177,770	2,331,576	170,635	34.48	\$187	\$4,833	2,228,767	151,325	267.8	\$934	\$10,127
Lower Kissimmee	3,900,192	328,176	3,886,863	317,316	0.54	\$41	\$50	3,824,982	296,298	31.0	\$412	\$973
Indian Prairie	2,062,514	235,339	1,666,908	116,530	0.93	\$2	\$8	1,617,644	96,235	23.8	\$54	\$171
Lake Istokpoga	1,368,807	108,516	1,318,518	76,379	0.97	\$19	\$30	1,299,359	66,168	138.6	\$1,995	\$3,272
Fisheating Creek.	1,001,617	133,977	982,858	126,407	0.47	\$25	\$62	917,952	96,399	17.5	\$209	\$466
Taylor Ck./Nubbin Slough	896,375	279,933	887,279	208,254	0.95	\$105	\$13	808,823	170,436	48.7	\$557	\$445
West Lake Okeechobee	99,960	6,182	84,024	4,921	2.10	\$132	\$1,663	78,457	4,592	27.3	\$1,270	\$17,174
East Lake Okeechobee	803,568	70,409	754,495	63,060	1.29	\$26	\$176	741,207	61,886	71.4	\$1,146	\$8,382
South Lake Okeechobee	2,471,796	106,870	1,888,859	81,248	16.15	\$28	\$630	1,903,649	69,363	78.8	\$139	\$2,100

313

4 Lake Okeechobee Watershed Construction Projects (Scenario 3)

4.1 Introduction

A total of seven projects in the LOWCP, distributed among five of the nine LOW sub-watersheds, were simulated using WAM for scenario 3. Six of these projects are either existing (i.e., have been constructed) or have been approved for construction (included in scenario 3a), while one is still in the planning process (included in scenario 3b).

The six constructed or approved projects in the LOWCP (scenario 3a) that were simulated for this report are:

1. The Brighton Valley DWM (**Section 4.2.2**). This is an 8,146-acre, two impoundment project that is planned for construction in the Indian Prairie sub-watershed (SFWMD 2018b). It is bounded on the north by the C-41A canal, just east of the S-83 structure and about 7 miles west of the S-84 structure. The primary operational objective is to divert as much regional water as allowed by the SFWMD while maintaining the allowable water levels inside the project and discharge rates (SFWMD 2018b). The water is pumped from C-41A into the first impoundment covering the eastern part of the project and subsequently allowed to flow across a broad swath into a second impoundment on the western side. The main discharge point is from the west impoundment into a canal that in turn discharges to the C-40 canal. Internal flows are controlled by weirs with crest elevations dependent on a set of operational modes defined in SFWMD (2018b). The volume of water that may be pumped into the project and the operation modes depend on the 2008 Lake Okeechobee Regulation Schedule (LORS).
2. IMWID Phase 2 (**Section 4.2.3**). This is a 306-acre impoundment project that has been constructed in the Indian Prairie sub-watershed (SWET 2012). This is located in the northwestern portion of the sub-watershed about a mile southeast of Lake Istokpoga. The primary purpose of the impoundment is to provide reduction in P loads discharging from the IMWID through water retention and reuse for irrigation. The impoundment is operated in two modes, “drainage” and “irrigation.” In drainage mode, stormwater is pumped into the impoundment from local drainage/irrigation canals when the stage in a canal adjacent to the impoundment (the IMWID “B canal”) is high enough (SWET 2012). When there is irrigation demand, as determined by the stage in the B canal, water is released from the impoundment into the B canal via a gate structure.
3. LRSTA and BRSTA (**Section 4.3.1**). These are two stormwater treatment areas (STAs) in the Taylor Creek/Nubbin Slough sub-watershed. They are intended to provide reductions in mass and concentration of TP entering Lake Okeechobee by treating stormwater runoff from the S-191 and portions of the S-135 Subbasins (CDM Smith 2018). The 1,762-acre LRSTA has been constructed in the S-135 Subbasin adjacent to the L-64 canal. It is divided into a total of eight cells across two main impoundments, LRSTA-North (LRSTA-N) and LRSTA-South (LRSTA-S). The planned BRSTA facility is a 1,661-acre STA in the S-191 Subbasin on the opposite side of the L-64 canal from LRSTA. The two STAs are designed to be operated in a similar fashion, with water pumped into the impoundments from the L-64 canal, with flow through the impoundment distributed across the cells comprising the STAs, and subsequently discharging into collection canals that in turn discharge into the L-47 canal adjacent to Lake Okeechobee. Pumping into the STAs is triggered by stages in the C-59 canal adjacent to the S-191 gate, which is located approximately 5 and 7 ½ miles from the inflow pumps to LRSTA and BRSTA, respectively. To maintain water balance between the S-191 and S-135 Subbasins the planned S-191A pump station is designed to pump water from the northwest end of the L-47 canal into the C-59 canal based on stages within the L-47 canal.

4. Fisheating Creek FAVT (**Section 4.4.1**). This is an approximately 265-acre impoundment project that has been constructed in the Fisheating Creek sub-watershed, located adjacent to Fisheating Creek just south of SR-70. The primary purpose of the system is to remove nutrients from Fisheating Creek. The FAVT is divided into four cells. Water is pumped from Fisheating Creek, as available, into a canal that feeds two cells at the north end of the FAVT. Water subsequently flows through the two cells into a third and then a fourth cell. The treated water discharges at the southern end of the FAVT into an outflow canal which in turn discharges back into Fisheating Creek before entering Lake Okeechobee, approximately 40 miles downstream from the FAVT.
5. Nicodemus Slough retention area (**Section 4.4.2**). This is a 15,906-acre impoundment that has been constructed in the Fisheating Creek sub-watershed, at the southern end of the sub-watershed adjacent to Lake Okeechobee. The purpose of the impoundment is to provide water retention by removing water from Lake Okeechobee through the C-5 culvert and pumping it over Nicodemus Slough when stages in Lake Okeechobee would otherwise require discharges to the Caloosahatchee and St. Lucie Rivers (SFWMD 2014a). The project uses two pump stations in series to take water from Lake Okeechobee's rim canal via culvert C-5 and transport it through an on-site canal along the northern boundary of the project where it is delivered to the northwestern end of the project. The water is then dispersed through sheet flow across a series of internal impoundments eastward through the site until the internal impoundments reach their operating stages (SFWMD 2014a). The amount and timing of water that may be pumped into the project is dependent on the 2008 LORS and the Nicodemus Slough Modified Regulation Schedule.
6. RMWRP, Parcel B (**Section 4.5.1**). This is a 1,972-acre site that has been constructed in the Upper Kissimmee sub-watershed, adjacent to Lake Hatchineha and immediately southeast of Catfish Creek. The project is part of the Kissimmee Headwaters Revitalization Project. The overall objective of the project is to restore the natural littoral habitat of Parcel B by diverting some of the flow from Catfish Creek through the project area before discharging into Lake Hatchineha. A series of control structures (weirs) are used to divert water from Catfish Creek around the western edge of the project before directing water into either Parcel B or into Lake Hatchineha as needed for maintenance of Parcel B. The outflow structure from Parcel B to Lake Hatchineha is designed to optimize water levels in Parcel B and allow the inflow of lake flows when the lake stages above the design stages for utilization of the floodplain.
- The one project simulated for this report that is still in the planning process is part of the LOWRP and is included in scenario 3b:
7. The K-05 WAF, ASR, and restoration of the Paradise Run wetlands in the Indian Prairie sub-watershed (**Section 4.2.5**) and the Kissimmee River Center wetlands in the Lower Kissimmee sub-watershed (**Section 4.2.6**). The K-05 WAF is planned to be located about 1.5 miles west of the Kissimmee River about 7 miles southwest of the town of Okeechobee (USACE 2018a, 2018b). The WAF has an approximately 11,000-acre arcuate footprint that is bounded on the north by the C-41A canal immediately east of the S-84 structure and on the south by highway FL-78. The feature is intended to have a 4-foot storage depth. Two internal embankments will be located to divide the feature into three impoundments (as yet not finalized). Flow between the impoundments will be controlled by gated culverts. Water will be pumped into the feature at a pump station just downstream of S-84 on the C-41A canal. The outlet of the feature will be on the southeast edge of the project to a canal that will in turn discharge into the C-38 canal (USACE 2018b). A set of ASR wells will be integrated into the project to provide additional water storage with recovery from the wells during dry periods. Pumping into, and recovery from, the WAF and ASR wells will be governed by the Lake Okeechobee Diversion and Recover Schedule for K-05 and the Lake Okeechobee Diversion and Recover Schedule for ASR Wells (USACE 2018b).

The Paradise Run wetlands consist of 4,084 acres of the historic Kissimmee River channel and floodplains. The wetlands are located on the west bank of the Kissimmee River alongside the WAF. Flow will be controlled by pumping into the wetlands at the northern end with discharge to C-38 via a culvert through the Herbert Hoover Dike on C-38 or through the L-48 canal (USACE 2018b). The Kissimmee River Center wetlands will consist of 1,196 acres of lowland floodplains located in the Lower Kissimmee sub-watershed between the S-65D and S-65E structures. Water from the C-38 canal will enter the wetlands via a submerged weir and exit back into C-38 above the S-65E structure (USACE 2018b).

These projects were grouped into six separate WAM setups in scenarios 3a and 3b:

Scenario 3a:

Indian Prairie LOWCP: The Brighton Valley DWM and IMWID impoundment (items 1 and 2 above) were simulated together by modifying the Indian Prairie sub-watershed WAM base scenario.

Taylor Creek/Nubbin Slough LOWCP: The LRSTA and BRSTA (item 3) were simulated together by modifying the Taylor Creek/Nubbin Slough WAM base scenario.

Fisheating Creek LOWCP: The Fisheating Creek FAVT and Nicodemus Slough impoundment (items 4 and 5) were simulated together by modifying the Fisheating Creek sub-watershed WAM base scenario.

Upper Kissimmee LOWCP: The RMWRP (item 6) was simulated by modifying the Upper Kissimmee sub-watershed WAM base scenario.

Scenario 3b:

Indian Prairie LOWRP: The K-05 WAF, ASR, and Paradise Run wetlands (item 7) were simulated by a second modified Indian Prairie sub-watershed WAM scenario.

Lower Kissimmee LOWRP: The Kissimmee River Center wetlands (also item 7) was simulated by a modified Lower Kissimmee sub-watershed WAM scenario.

With the exception of the RMWRP and Paradise Run and Kissimmee River Center wetlands, these projects seek to provide water retention and subsequent treatment of nutrients by the construction of impoundments surrounded by berms. Water and associated nutrients are supplied to the system via hydraulic structures such as pumps or gates. These systems are simulated by WAM in a similar fashion where the area within an impoundment that is designed to be submerged is represented by one or more reaches in the WAM setup. In systems where the entire impoundment is designed to be submerged (e.g., STAs) the length and width of the WAM reaches are calculated so that the area of the water surface in the reach matches the impoundment area when the water stage is at the design stage. Other projects may only have a portion of the impoundment area that is designed to be submerged, i.e., some areas within the impoundment may have elevations above the highest design stage. In these cases, the length and width of the WAM reaches are calculated to represent the submerged portion. In addition to calculating the appropriate length and width of the reaches representing impoundments, two additional characteristics of WAM reaches must be determined: the cross-sectional profiles and the coefficients that control nutrient attenuation within the reaches. These are discussed in general below, with the specific details for each project discussed in the relevant project section.

4.1.1 Reach Cross-Sectional Profiles

During the WAM setup process, the contributing upstream area for each reach is calculated and an estimated cross-sectional profile for each reach is calculated as a function of the contributing area. The reach profiles are used by the WAM sub-program BLASRoute (“Basin Land Area to Stream Routing”) and are read in from the file “LocalInputs\streamprofile.in”. In many cases the profiles are subsequently altered when cross-sectional data are available, such as for canals, or most relevant for simulations of the projects here, reservoirs and STAs. The profiles are adjusted in the WAM interface in ArcMap. For these

latter cases, reach profiles are typically constructed with a broad bottom that slopes very slightly to the middle of the reach. A much narrower inner channel (or “notch”) is usually generated that is between about 1.5 and 3 feet deep (0.5-1 meter), with the inner channel being between 30 and 120 feet wide (approximately 10-40 meters). Defining a narrow inner channel is not required but is usually added to help computational performance.

WAM cross-sectional profiles are defined by a depth-to-width relationship (see **Table 2** for an example), e.g., when the water depth is 0, the width may be 0 feet, when the water is 1.5 feet deep, the width is 30 feet, etc. In this example, the narrow inner channel is the 0- to 1.5-foot deep portion, which spreads out to a width of 1,000 feet as the depth increases from 1.5 to 2 feet deep. Thus, the reach bottom changes from essentially dry (only the inner channel has water) to fully submerged as the water depth in the reach changes from 1.5 to 2 feet, i.e., 6 inches. The edge of the reach is the portion where the width is between 1,000 and 1,010 feet as the depth increases from 2 to 10 feet.

Table 2. An example depth-to-width relationship for a WAM reach.

Depth (ft)	Width (ft)
10.0	1,010
2.0	1,000
1.5	30
0.0	0

A diagram of the example profile (blue line) corresponding to the values in **Table 2** is shown in **Figure 2** where the distance from the centerline of the channel is measured along a line perpendicular to the length of the channel.

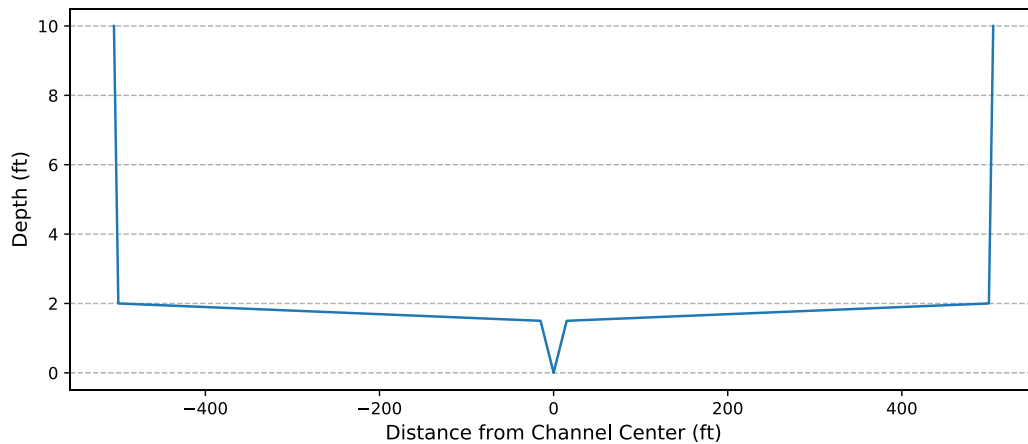


Figure 2. Example reach cross-sectional profile based on the data in Table 2.

4.1.2 Attenuation

WAM applies attenuation to nutrients within all reaches, not just those used to simulate reservoirs or STAs. The attenuation algorithm used by WAM assumes that the rate of constituent exchange between the water in the stream and the stream bottom sediment is proportional to the concentration difference between the stream and a user-defined background concentration. The rate of exchange is a function of

the contact area, i.e., the wetted surface of the stream. The algorithm used in WAM applies the following equation at each time step:

$$C_{t+\Delta t} = C_{BG} + (C_t - C_{BG}) \cdot \exp(-a \Delta t / R)$$

In the above equation C_t is the in-stream concentration at time t , C_{BG} is the background concentration, Δt is the time step, a is the first-order attenuation coefficient, and R is the hydraulic radius of the channel. Typical values of a are $1.0 \times 10^{-6} < a < 1.0$, although a can be any positive value. The higher the value of a , the “faster” (shorter residence time) the in-stream concentration approaches the background value. The values for a and C_{BG} are assigned through the WAM interface during the selection of the attenuation type for a given reach. In practice, reaches of general types are lumped together, i.e., canals, streams, lakes, etc., are assigned common attenuation coefficients, but it is possible to assign a different set of attenuation parameters (a set is called an “attenuation type”) for each reach in a stream network. Usually a specific attenuation type is developed for a specific impoundment within a WAM. As with the reach cross-sectional profiles, the attenuation coefficients are specified through the WAM ArcMap interface and used by the BLASRoute sub-program. BLASRoute reads the coefficients from the file “GlobalInputs\attenuate.in”. The attenuation parameters used for projects are specified in the appropriate section below.

The land use type assigned to impoundments is a special case land use type that passes rainfall directly through to the stream network, which then handles attenuation and evapotranspiration (ET), so unlike other land uses in WAM, there is no source-cell level simulation of hydrologic- or nutrient-related processes, since all processes are handled in-stream.

4.1.3 Project Costs

The SFWMD provided SWET with the capital and O&M costs for the LOWPP and LOWRP projects evaluated in this study (**Table 3**). In order to evaluate the cost-effectiveness values for these projects that are provided in **Section 6**, the capital costs were converted to an annual cost assuming a thirty-year life of the projects and an interest rate of 6%. These annual capital costs were then added to the O&M costs to obtain the total annual costs for the projects. Note that the O&M cost for the Istokpoga Marsh Watershed Improvement District (IMWID) project was not provided and therefore was estimated as 20% of the annual capital cost. The total annual costs were then divided by the WAM simulated annual average P load reductions presented in **Sections 4** and **5** to obtain their relative cost-effectiveness in terms of dollars per pound of P removed.

508 **Table 3. Capital and O&M Costs for LOWPP and LOWRP Scenarios 3 and 4 Projects (SFWMDD).**

Project Scenarios	Project Capital Cost (\$/yr)	Annual O&M Cost (\$/yr)	Notes
Scenario 3a - Existing LOWCP			
Fisheating Creek Floating Aquatic Vegetative Tilling (FAVT)	3,000,000	2,573,000	Both capital and O&M figures provided by FDACS.
Brady Ranch	30,367,737	254,307	Land acquisition costs only - Dec 2007 - Jun 2009; project in planning phase. Estimated O&M costs are extrapolated based on LR-STA.
Brighton Valley Dispersed Water Management (DWM)	12,182,509	3,050,000	Estimated construction 2018-2019; includes latest additional funds by FDACS of \$475K. O&M figure is average of contract amounts over ten years following certification of construction completion (est. 2019-2029).
Istokpoga Marsh Watershed Improvement District (IMWID) Phase I	14,837,146	N/A	Total cost per multi-agency agreement (Amendment 9), originally executed March 2009. Construction still ongoing. Annual O&M costs are not available (IMWID project).
Lakeside Ranch STA (includes Phases I & II as well as planned S191 Pump Station)	132,693,651	334,615	Includes actual costs for engineering, design, and construction from 2008 through Jan 2019, previous land acquisition costs, plus planned expenditures through 2020. O&M figure is average of projected costs FY2018-19 thru FY2022-23 as of 11/20/2018.
Nicodemus Slough	4,989,653	2,844,848	Construction Jan 2011-Jan 2015; O&M figure is average of contract amounts over eight years from 2/2015 - 1/2023.
Rolling Meadows Wetland Restoration Phase I	5,141,792	100,706	Capital costs include preliminary assessments, design, construction, and land management services. April 2010 - Jun 2017. O&M figure is average of five years expenditures FY2013-14 thru FY2017-18.

509

Table 3 (continued). Capital and O&M Costs for LOWPP and LOWRP Scenarios 3 and 4 Projects (SFWMD)

Project Scenarios	Project Capital Cost (\$/yr)	Annual O&M Cost (\$/yr)	Notes
Scenario 3b – LOWRP			
Planned Lake Okeechobee Watershed Restoration Project (LOWRP) (includes planned shallow reservoir, wetland restoration (2 wetland areas), and ASR features) ¹	1,437,000,000	15,401,000	The LOWRP cost estimate of \$1,437,000,000 includes construction and non-construction costs for ecosystem restoration activities. Non-construction costs generally include LERR (lands, easements, rights-of-way and relocations), Engineering During Construction, Pre-construction Engineering & Design (PED), and Supervision and Administration costs. Costs were estimated at FY2018 price levels and rounded to the nearest \$1,000. The federal discount rate of 2.75% and a 50-year economic period of analysis were used to amortize costs and determine the project investment costs. Total OMRR&R cost of \$15,401,000 includes LOWRP features (WAF, Kissimmee River Center wetland, Paradise Run wetland, and 80 ASR wells), but does not include monitoring costs, invasive species, or recreation. These are based on FY2018 price levels. Note that the above costs are based on the Draft Project Implementation Report and Environmental Impact Statement (PIR/EIS) (USACE and SFWMD, July 2018); these costs are currently being updated between the draft and final PIR/EIS. The total project cost that will be reported in the Final PIR/EIS is expected to be \$1,812,406,000. OMRR&R costs are currently under revision and are not available at this time but will increase from the estimate in the draft report, consistent with the overall total project cost increase.
Scenario 4 - Full KRR			
Fully implemented Kissimmee River Restoration Project and Upper Basin Initiatives	786,994,000	433,288	Total project capital cost (Federal + District) is based on USACE PB2a dated 3/29/18 (includes design, construction and real estate). <i>[Note: Currently being updated, which may be available for final reporting.]</i> O&M figure is average of projected costs FY2018-19 thru FY2022-23 as of 11/20/2018.

¹ - LOWRP will likely have an ancillary water quality benefit but is designed primarily for water storage; an estimate of cost-effectiveness for this project in terms of P removal is shown here for informational purposes only.

4.2 Indian Prairie Lake Okeechobee Watershed Construction Projects

4.2.1 Introduction

The WAM setup for Indian Prairie was developed as part of the WAM recalibration to estimate water flows and nutrient loads across all the nine sub-watersheds in the LOW (SWET 2017a, 2017b). The Indian Prairie sub-watershed was simulated as part of the northern LOW sub-watersheds (SWET 2017a). Three projects in the LOWCP were simulated within the Indian Prairie sub-watershed: Brighton Valley DWM, IMWID Phase 2, and a planned shallow reservoir, the K-05 WAF with wetland restoration (two 2 wetland areas) and aquifer storage and recovery (ASR) in the south-eastern part of the basin (**Figure 3**).

To incorporate these projects into WAM, two scenarios were created from the base WAM scenario for Indian Prairie. Each scenario was created by duplicating the base scenario, once to create a scenario with the Brighton Valley DWM and IMWID projects, and a second time to create a scenario that incorporated the K-05 WAF and restored wetlands. The Brighton Valley project is discussed first in **Section 4.2.2**, the Istokpoga Marsh project is discussed in **Section 4.2.3**, and the WAF is discussed in **Section 4.2.5**.

Shapefiles were obtained from the SFWMD that delineate the footprints for the features, along with design and operation documents (SFWMD 2018b; SWET 2012).

The Brighton Valley DWM is a planned project bordered on the north by the C-41A canal that is 8,146 acres in size. The primary purpose of the project is to accept as much regional water as allowed from the C-41A canal and allow it to flow through the project area while maintaining allowable depths, while maximizing discharge to the C-40 canal and minimizing the potential of recirculating water through the project.

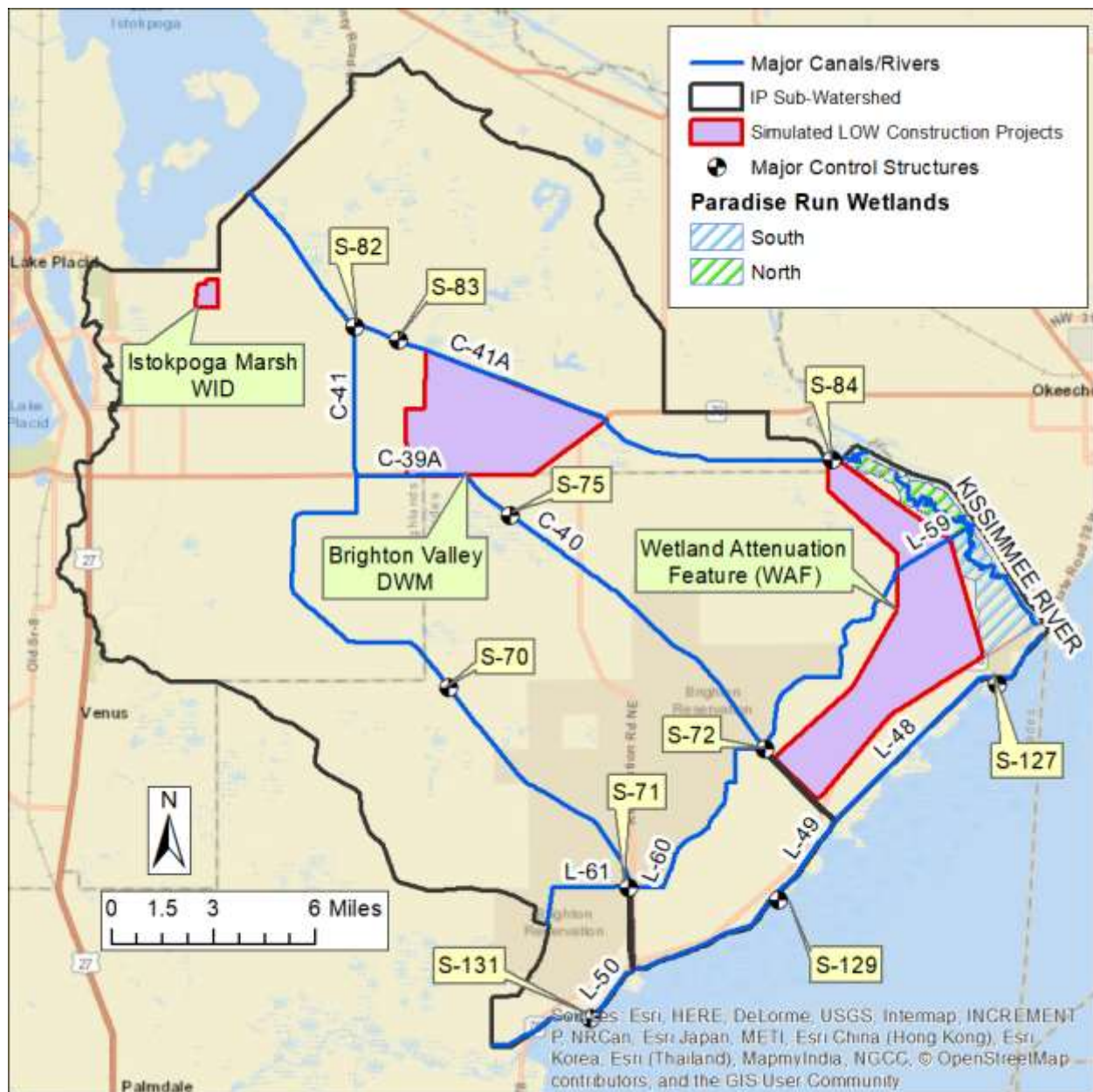
Note that in this and all following sections that elevations (e.g., stages and hydraulic structure data) are given in feet above mean sea level using National Geodetic Vertical Datum of 1929 (NGVD) unless otherwise noted.

4.2.2 Brighton Valley DWM

This section describes the addition of the Brighton Valley DWM project to the existing WAM model setup for the Indian Prairie sub-watershed. The primary operational objective is to accept as much regional water from the sub-watershed as allowed by the SFWMD while maintaining the water levels inside the project below the maximum allowable depths and to maintain the discharge rates at or below the maximum values (SFWMD 2018b).

The proposed project is 8,146 acres in size and lies within the Indian Prairie sub-watershed (**Figure 3**). The sub-watershed is bordered on the north by Lake Istokpoga, on the south by Lake Okeechobee, and the east by the C-38 Canal/Kissimmee River. The C-39A, C-40, C-41, and C-41A canals are the major canals in the sub-watershed, their primary purposes being drainage, maintenance of the Lake Istokpoga water regulation schedule, flood control, and irrigation. The C-41A Canal, also called the Stub Canal or the Brighton Canal, is the primary outlet for Lake Istokpoga with the S-68 structure located at the northern terminus of the canal serving as the primary outfall structure for the lake. The S-84 (**Figure 3**) and S-84X structures lie at the southern terminus of the canal and provide the controlled connection of the canal to the C-38 Canal/Kissimmee River, which ultimately discharges to Lake Okeechobee. The S-82 and S-83 structures lie within the C-41A Canal. The C-41 Canal, or Harney Pond Canal, is directly connected to the C-41A Canal at S-82 and extends south to its terminus at Lake Okeechobee. The S-70, S-71, and S-82 structures lie within the canal, while the G207 lies at its connection with the lake. The C-39A Canal connects the C-41 Canal in the west to the northern terminus of the C-40 Canal, or Indian Prairie Canal, in the east. The S72 and S75 structures lie within the C-40 Canal, which terminates at Lake Okeechobee with the G208 pump station.

The Brighton Valley DWM is divided into east (upstream) and west (downstream) impoundments (**Figure 4**). The east and west impoundments are 2,399 and 5,747 acres in extent, respectively. The project will divert excess water from the C-41A Canal via five intake culverts that will feed the pump station. The pump station will consist of multiple pumps totaling 560 cubic feet per second (CFS). The pump station will discharge into the eastern upstream impoundment. The flow within the eastern impoundment will be in the east to west direction. The eastern impoundment will discharge into the western downstream impoundment through several control structures located along the Interior Berm. The flow within the downstream impoundment will be predominantly in the east to west and towards the north and south outfalls. The project will discharge offsite through three control structures. Two of the outfalls (CS-O2, and CS-O3) will return water to the C-41A Canal through existing structures known as PC-16 and PC-19, respectively. The other outfall (CS-O1) will discharge to an existing canal that flows under SR 70 and connects to the C-40 Canal. An additional outfall (CS-O4 or overflow structure) will allow discharge to the C-41A Canal under internal high-water conditions. The water levels and discharge rates from the project will be controlled with pump operations and adjustment of the control structures.



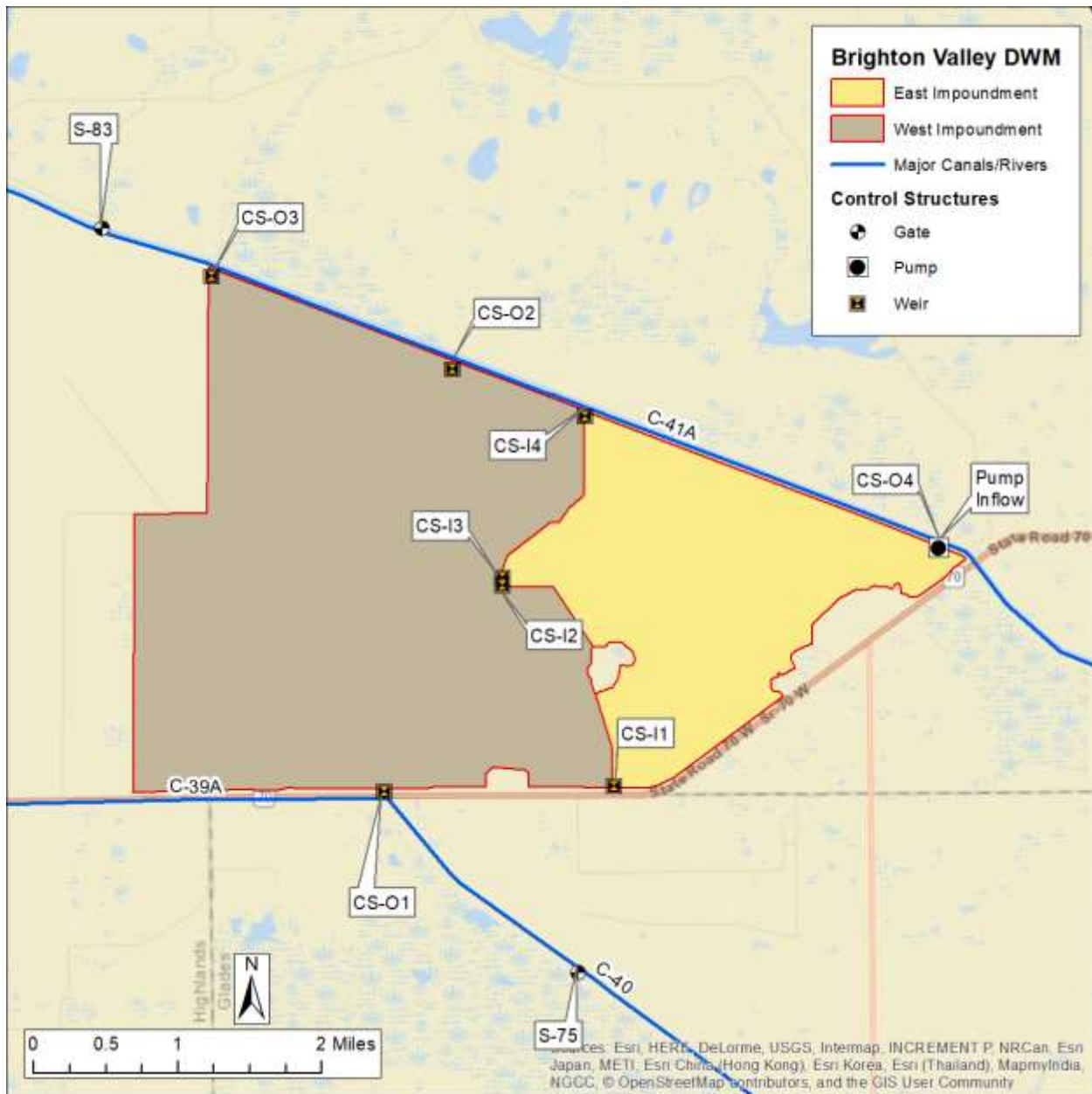


Figure 4. The Brighton Valley DWM impoundments and control structures.

4.2.2.1 Operational Modes

As described in the document provided to SWET by the SFWMD (SFWMD 2018b), the project flow intake facilities will only operate during periods when there is excess water. The definition of “excess water” is detailed in SFWMD (2018b) and relies primarily on the 2008 LORS. In addition to delineating the criteria for when there is excess water, SFWMD (2018b) defines the three operational modes for the Brighton Valley DWM:

- Default Pumping Mode
- Storm Mode
- Off Mode

A summary of each mode is provided below.

4.2.2.1.1 Default Pumping Mode

The default pumping mode will allow the project to intake and discharge up to the maximum pumping rate of 560 CFS from C-41A. This will be achieved by running the pump station and adjusting all weir crest elevations in the internal control structures and the gates at the south outfall to accommodate the flow (generally all internal structures completely open, **Table 4**). All weir crest elevations were obtained from SFWMD (2018b).

Table 4. Weir crest elevations and widths for the default pumping mode (SFWMD 2018b).

Structure ID	Top Elevation (ft)	Width (ft)
CS-O1	22.7	23
CS-O2	26.4	13
CS-O3	26.4	13
CS-I1	25.2	33
CS-I2	26.7	33
CS-I3	26.7	33
CS-I4	26.7	33
OVERFLOW (CS-O4)	31.2	33

4.2.2.1.2 Storm Mode

The pumps will not operate when the cone of probability of tropical storms, topical depressions, or hurricanes intersects with any part of peninsular Florida, or when the project is at maximum operating water levels and when the project daily rainfall exceeds 1.5 inches for the C-40 and C41A basins as determined by the SFWMD. The operational mode will be adjusted from storm mode back to the default mode three days after the condition that caused the activation of storm mode has passed. This will allow water levels to recede to a point that provides inflow capacity in the project. Weir crest elevations for storm mode are shown in **Table 5**. Note that the weir widths are the same as those for default mode.

Table 5. Weir crest elevations for storm mode (SFWMD 2018b).

Structure ID	Top Elevation (ft)
CS-O1	26.7
CS-O2	26.4
CS-O3	26.4
CS-I1	30.7
CS-I2	30.7
CS-I3	29.2
CS-I4	30.7
OVERFLOW (CS-O4)	29.6
Inflow Pump(s)	OFF

4.2.2.1.3 Off Mode

If at any time the default pumping mode is not in effect due to a lack of regional excess water available for the project, the inflow pumps shall be off, and the control structures shall be adjusted as recommended in **Table 6** to maintain the normal wet season pool elevations of 29.7 feet (NGVD) (eastern impoundment) and 27.6 feet (NGVD) (western impoundment). Note that the weir widths are the same as those for default mode.

Table 6. Weir crest elevations for off mode (SFWMD 2018b).

Structure ID	Top Elevation (ft)
CS-O1	27.6
CS-O2	27.6
CS-O3	27.6
CS-I1	29.7
CS-I2	29.7
CS-I3	29.7
CS-I4	29.7
OVERFLOW (CS-04)	29.7
Inflow Pump(s)	OFF

4.2.2.2 WAM Setup for the Brighton Valley DWM Project

The WAM setup for the Brighton Valley DWM project is shown in **Figure 5**. Reach numbers are indicated by the circled numbers on each reach. The C-41A canal runs adjacent to the project along the northern edge, with WAM reaches 239 – 243 comprising C-41A adjacent to the project. Structure S-83 is located on C-41A just northwest of the project between WAM reaches 245 and 244. WAM reaches 236 and 235 comprise the C-39A canal on the southwest edge of the project, with reaches 158 and 157 representing the C-40 canal just south of the project and structure S-75 located between WAM reach 157 and 156.

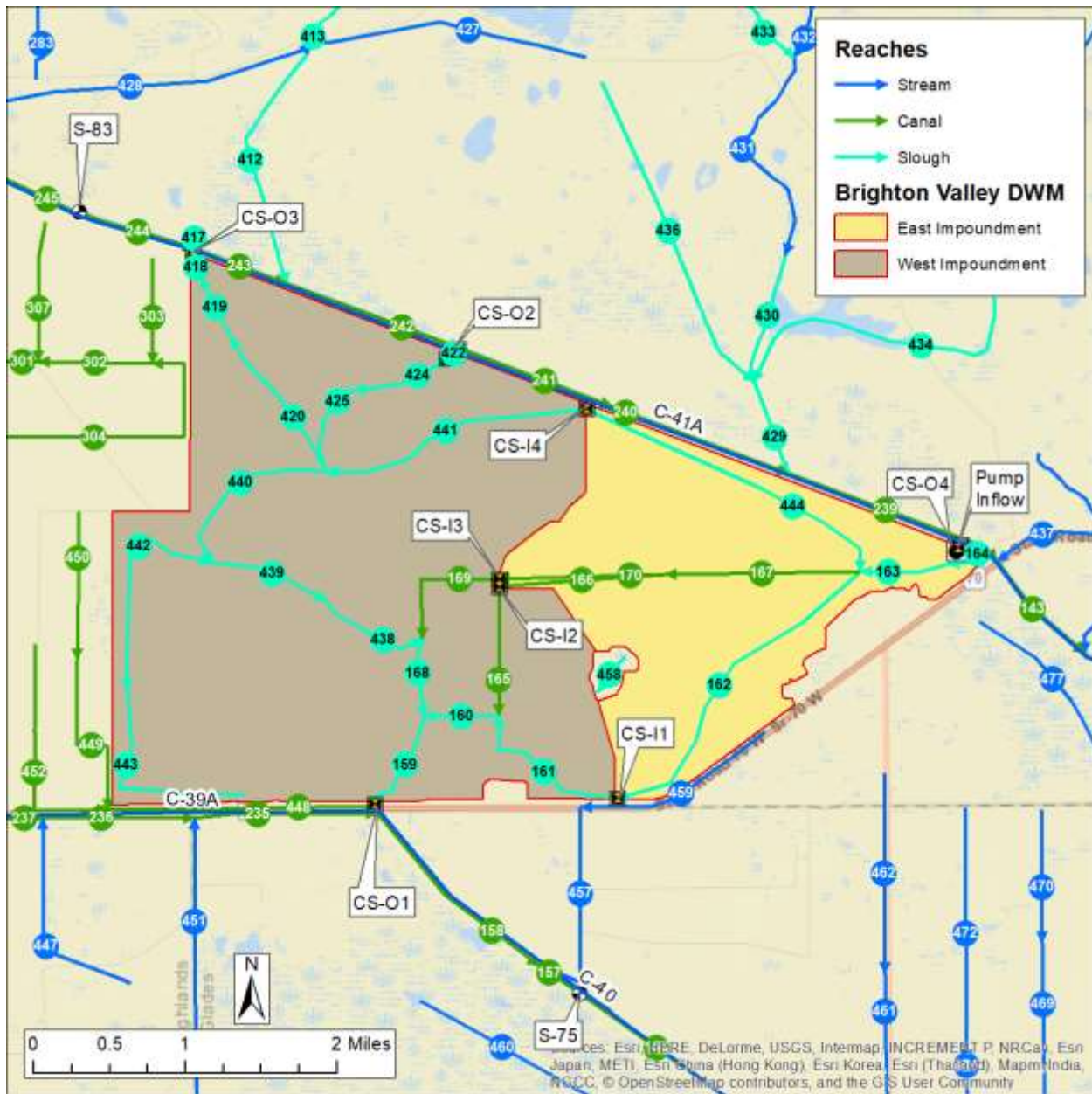


Figure 5. The WAM setup for the Brighton Valley DWM.

The Brighton Valley DWM is divided into east and west impoundments. The two types of WAM reaches within the impoundments are denoted as canals (dark green) and sloughs (light green) (**Figure 5**). The canals follow pre-existing canals while sloughs were added to reflect low areas in the topography that will control drainage. Since cross-sectional profiles can be assigned on a reach-by-reach basis, the distinction between canals and sloughs is important mainly for clarity when viewing the scenario in ArcMap. Water from C-41A is pumped into the east impoundment at the far eastern end at the point marked “Pump Inflow” in **Figure 5** and flows westward down WAM reach 163. The western end of that reach splits into three reaches: two sloughs that convey water along the northern (WAM reach 444) and southern (WAM reach 162) end of the impoundment, and a canal (WAM reach 167) that tracks across the middle of the impoundment before splitting into WAM reach 166 and 170. The four reaches discharge via weirs at the division between the western and eastern impoundments. From south to north, these weirs are denoted CS-I1, CS-I2, CS-I3, and CS-I4. In the eastern impoundment the flow is routed across the internal weirs

to the western impoundment, and then routed to the main outflow structure denoted as CS-O1 in **Figure 5**. Minor discharge can occur back to C-41 at structures CS-O2 and CS-O3 under high water conditions in either of the impoundments. Rain falling in the south-western end of the western impoundment discharges initially to the north part of the western impoundment via WAM reaches 443 and 442 before looping back around to the south to also discharge at CS-O1.

4.2.2.2.1 Profiles

The cross-sectional profiles of the WAM reaches representing sloughs are much wider than those representing canals to effectively simulate slower flow across the low areas within the Brighton Valley impoundments (see **Section 4.1.1**). Unlike a constructed STA, the implementation of slough profiles in the WAM setup for Brighton Valley DWM was not intended to match the total area within the impoundments, since many areas within the DWM area are not intended to be submerged for extended periods. Cross-sectional profile data for two representative reaches in the eastern impoundment are provided in Table 7. WAM reach 162 is a slough along the southern edge and reach 167 is a canal that runs through the middle of the eastern impoundment. A schematic diagram for the cross-sectional profile of WAM reach 162 (The east impoundment) is shown in **Figure 6**.

Table 7. Cross-sectional profile data for two representative reaches (162, a slough, and 167, a canal) in Brighton Valley DWM.

East Impoundment (Slough) WAM Reach 162		East Impoundment (Canal) WAM Reach 167	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
8.2	820		
4.3	656	9.8	656
3.6	492	5.9	98
3.3	39	3.3	49
0.0	0	0.0	26

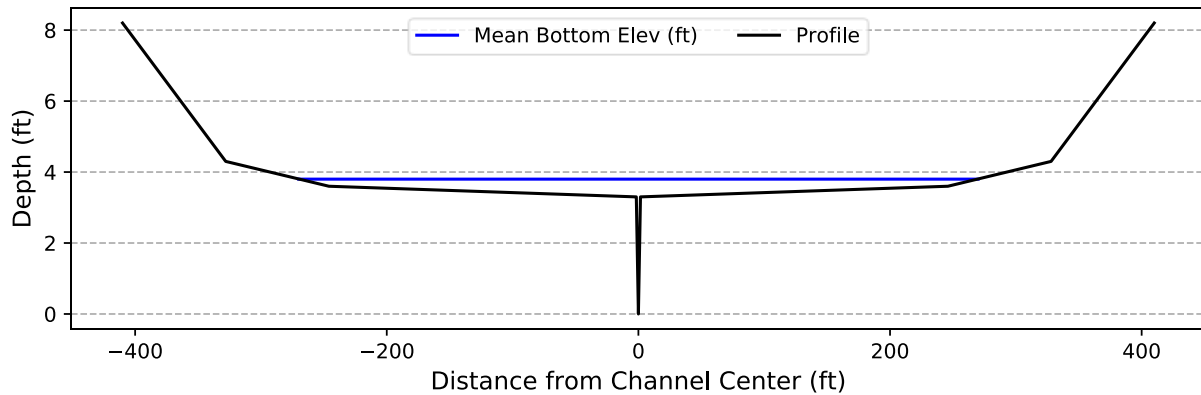


Figure 6. The cross-sectional profile for WAM reach 162, a typical slough in the Brighton Valley DWM east impoundment, used for the simulation with the mean bottom elevation indicated.

4.2.2.2.2 Land Use Changes

The land uses associated with the Brighton Valley impoundments were changed to reflect the anticipated conditions with the project area (**Table 8**). To develop the modified WAM Indian Prairie scenario, all pasturelands (improved, woodland, or unimproved) were reverted to scrub and brushland, or, in cases where pasture is underlain by muck soils, to freshwater marshes.

Table 8. Land use acreages in the Brighton Valley impoundment basins.

Land Use	Current (ac)	Modified (ac)
Barren Land	55	55
Freshwater Marshes	1,482	2,031
Improved Pasture	4,926	0
Mixed Wetland Hardwoods	291	291
Open Water	22	22
Scrub and Brushland	5	5,745
Unimproved Pasture	1,340	0
Woodland Pasture	22	0
Total	8,144	8,144

4.2.2.2.3 Pump Inflows

As discussed in **Section 4.2.2.1** above, the project flow intake facilities will only operate during periods when there is excess water (SFWMD 2018b). Time series of pump inflow rates and operational modes were developed by Lykes Bros. Inc., using SFWMD criteria. These time series data were in turn provided to SWET. The time series for pump inflow to Brighton Valley DWM is shown in **Figure 7**. The water volume added to the Brighton Valley DWM from pumping is 63,844 ac-ft/y.

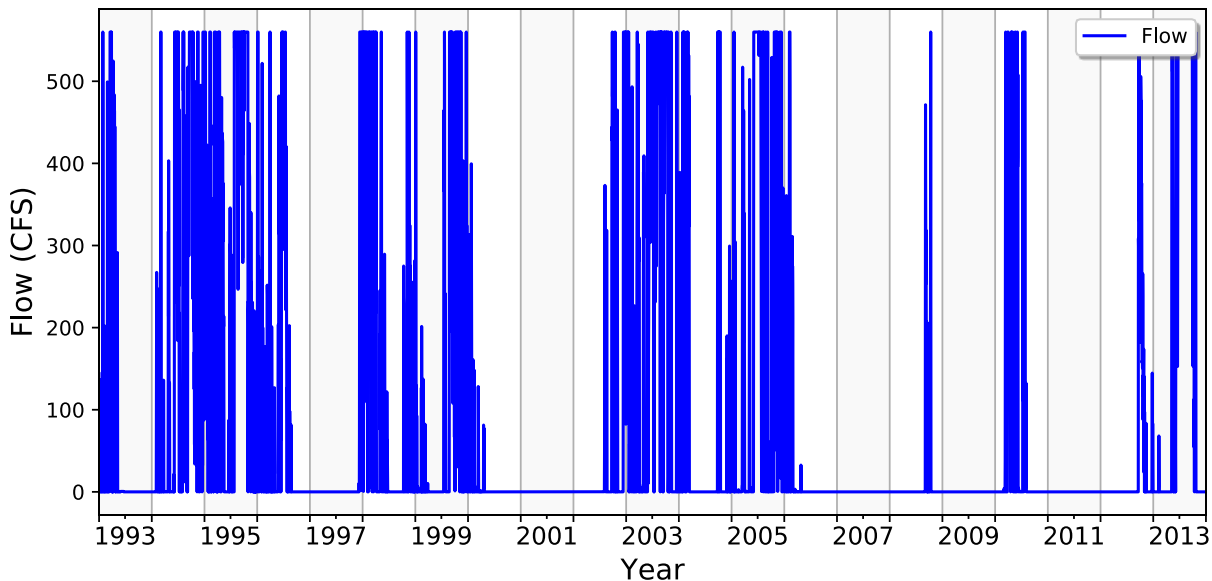


Figure 7. Time series of pump inflow to the Brighton Valley DWM used for the WAM simulations.

4.2.2.2.4 Weir Crest Elevations

The internal and outflow weirs were set to the elevations shown in **Tables 4 – 6** depending on which mode was operable. When pumping was occurring, the weir crests were set to the default mode elevations shown in **Table 4**. If daily rainfall exceeded 1.5 inches for the average of the stations near the project area, the weir crests were set to the storm mode elevations shown in **Table 5**. The crest elevations were set at the storm mode values until three days had passed since the last day for which rainfall exceeded 1.5 inches. Note that part of the criteria for determining storm mode relies on forecast tropical systems

impacting the Florida peninsula; these conditions were not incorporated into the WAM simulations. If neither the default nor the storm mode conditions were met the weir crests were set to the off mode elevations shown in **Table 6**.

4.2.2.3 Simulation Results

A summary of results is shown in **Table 9**, including a comparison with WAM simulations for the base conditions scenario (SWET 2017a). Simulated annual inflow volumes to the Brighton Valley dispersed water management (DWM) project are shown in **Figure 8**, and simulated annual discharge volumes are shown in **Figure 9**. Over the 21-year simulation period, the simulated average annual pump inflow volume is 63,844 ac-ft/y, while the annual average simulated discharge volume at CS-O1 (discharging to C-39A) is 53,170 ac-ft/y. Additional inflow from rain totals 29,289 ac-ft/y. Additional simulated discharges along the northern edge of the western impoundment back into C-41A are 6,374 ac-ft/y and 7,075 ac-ft/y at structures CS-O2 and CS-O3, respectively, and 1,895 ac-ft/y from the eastern impoundment at structure CS-O4. Comparison with the base scenario shows a decrease of simulated discharge volume at S-84 of 48,200 ac-ft/y (a decrease of 27%) and a corresponding increase at S-75 of 40,525 ac-ft/y (an increase of 112%).

Table 9. Comparison of simulated annual average volumes and TP loads from the base and LOWCP scenarios over the 1993-2013 period.

Location	Volume (ac-ft/y)			TP Load (mt/y)		
	Base	LOWCP (BV)	Change	Base	LOWCP (BV)	Change
Pump Inflow to East Impoundment	-	63,844	63,844	-	5.0	5.0
Rain	29,289	29,289	0	0.25	0.25	0.0
Outflow at CS-O1	-	53,170	53,170	-	1.95	1.95
Outflow at CS-O2	-	6,374	6,374	-	0.12	0.12
Outflow at CS-O3	-	7,075	7,075	-	0.14	0.14
Outflow at CS-O4	-	1,895	1,895	-	0.04	0.04
S-75 Outflow	36,134	76,659	40,525	12.98	13.74	0.76
S-84 Outflow	176,452	128,252	-48,200	20.47	15.11	-5.36

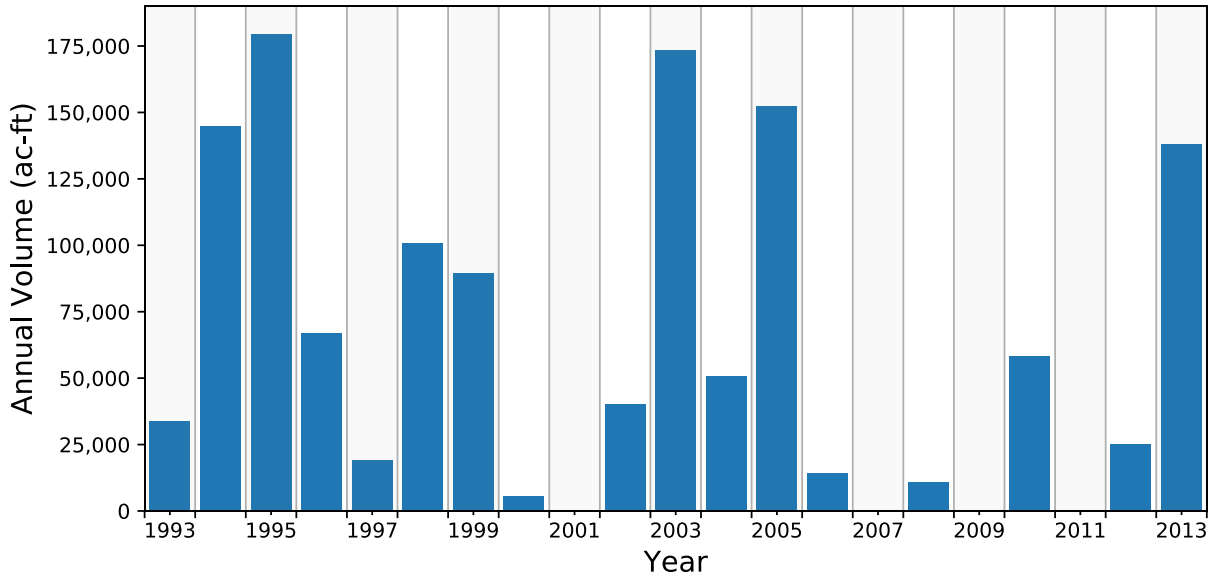


Figure 8. Annual simulated inflow volumes (ac-ft/y) to Brighton Valley DWM.

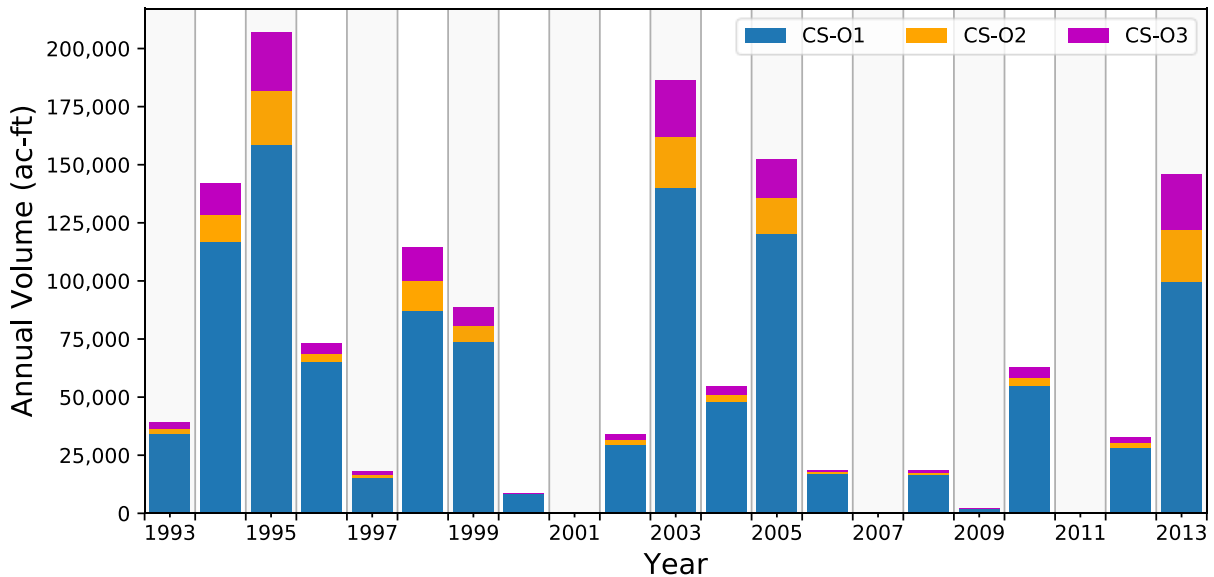


Figure 9. Annual simulated outflow volumes (ac-ft/y) from Brighton Valley DWM at structure CS-O1, CS-O2, and CS-O3.

Figure 10 shows time series of the inflow TP concentrations to Brighton Valley DWM (orange line) and the outflow concentrations at CS-O1 (green line). Note that while concentrations in the inflow spikes to high values in 2007 and 2009, inflow volumes are near zero during those periods with consequently low TP loadings into the impoundment. Also note that concentrations show fewer spikes in 2004-2005, but are consistently elevated during that time frame, with consequently higher loadings during the higher inflow volumes in 2005.

Simulated annual average TP loading to Brighton Valley DVM and simulated annual average discharge loads are shown in **Figures 11** and **12**. Over the 21-year simulation period, the simulated annual average

TP load into the impoundment is 5.0 mt/y, while the annual average discharge TP load at CS-O1 (discharging to C-39A) is 1.95 mt/y. An additional 0.25 mt/y of TP is added due to rainfall. Additional discharges of TP loads along the northern edge of the western impoundment back into C-41A are 0.12 mt/y and 0.14 mt/y from CS-O2 and CS-O3, respectively, and 0.04 mt/y from the eastern impoundment at CS-O4 for a total TP discharge load of 2.25 mt/y. This corresponds to a reduction of 2.75 mt/y, or 55%. Comparison with the base scenario shows a decrease of 5.36 mt/y TP at S-84 (26% decrease) and a corresponding increase of 0.76 mt/y TP at S-75 (increase of about 6%). The increased attenuation of TP within the impoundment is responsible for the overall decrease of 4.6 mt/y of TP loading to the sub-watershed, which is the sum of loading changes at S-84 and S-75.

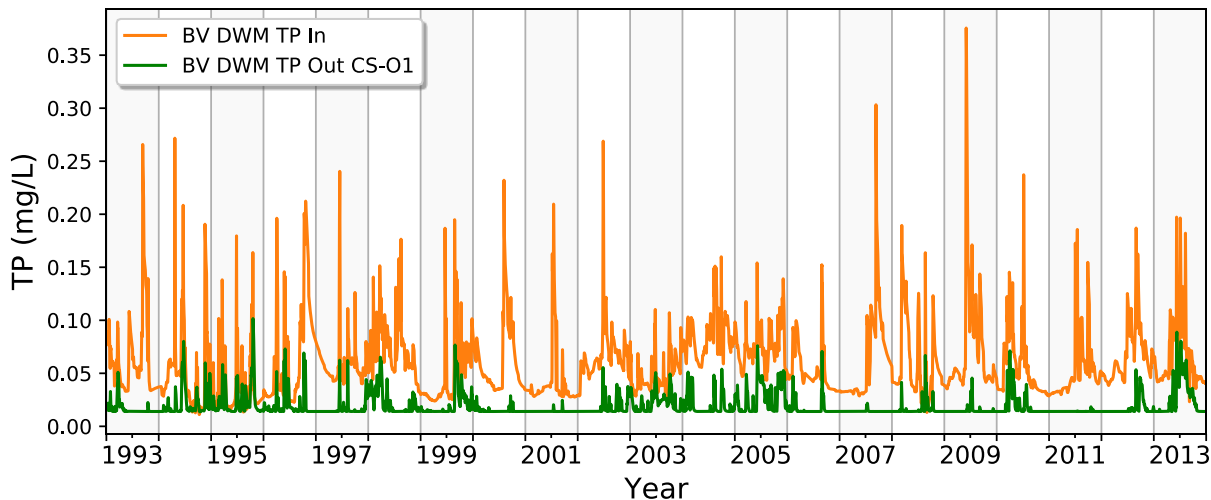


Figure 10. Simulated TP concentration at the inflow (orange line) and outflow from CS-O1 (green line) from the Brighton Valley DWM.

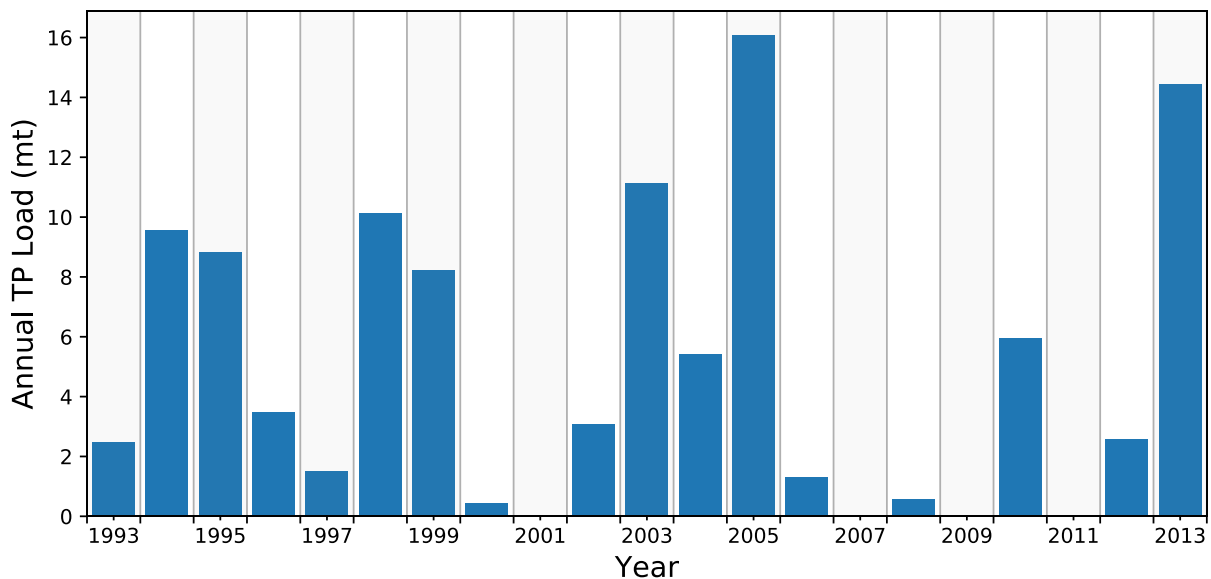


Figure 11. Annual simulated inflow TP loads (mt/y) to Brighton Valley DWM.

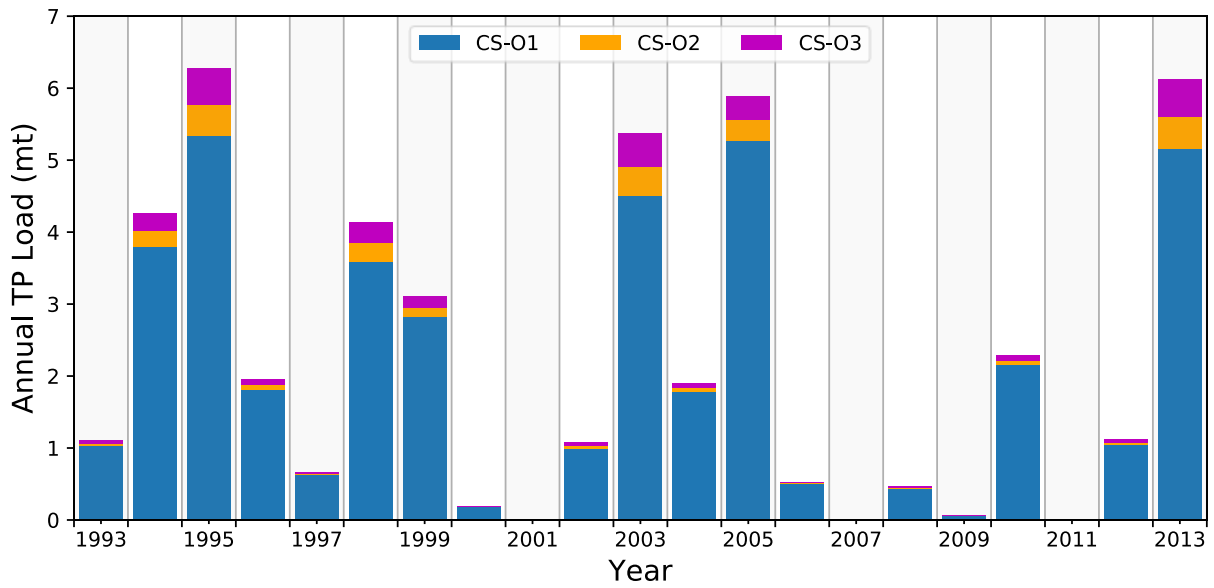


Figure 12. Annual simulated outflow TP loads (mt/y) from Brighton Valley DWM at structures CS-O1, CS-O2, and CS-O3.

4.2.3 Istokpoga Marsh Watershed Improvement District (Phase 2) Retention Project

The IMWID (Phase 2) retention and tailwater recovery project (SWET 2012) is a 306-acre project located in the northwest part of the Indian Prairie sub-watershed (**Figure 3**). The primary objective of this project is to store water within the impoundment when stormwater drainage from the surrounding area is required (“drainage mode”). Releases of stored water from the impoundment will occur to meet irrigation demand within the IMWID when required (“irrigation mode”). A set of three pumps are used to pump water into the impoundment from the canal immediately south of the project area (“Canal B”). A single gate is used to discharge water from the impoundment to Canal B when required. Discharge from the IMWID to C-41 is via the structures designated by IMWID as “S-1” and “S-9” on WAM reaches 271 and 265, respectively (**Figure 13**).

4.2.3.1 WAM Setup for the IMWID Project

Figure 14 shows the location of the IMWID impoundment (dark blue polygon), immediately southeast of Lake Istokpoga. The impoundment is represented in WAM by reach 294, which is assigned a length of 3,960 and an average width of 3,365 feet which gives it an area of 306 acres to match the project area. The bottom elevation of WAM reach 294 is 23.6 feet. The cross-sectional profile has a relatively narrow inner channel that is 3.3 feet deep and the impoundment is almost fully submerged when water is 3.9 feet deep (**Table 10**). The “mean bottom elevation” is calculated as the half-way point between 3.3 and 3.9 feet, i.e., when roughly half of the impoundment is submerged and half is dry; this is 3.6 feet above the reach bottom, i.e., at an elevation of 27.2 feet.

Table 10. Cross-sectional profile of the IMWID impoundment (WAM reach 294).

IMWID Impoundment WAM Reach 294	
Depth (ft)	Width (ft)
16.4	3,380
3.9	3,350
3.3	164
0.0	33

The pump and gate structures are located at the bottom of reach 294. Canal B is represented by WAM reaches 273-275. Pumping into the IMWID impoundment is controlled by water levels in Canal B (**Table 11**) (SWET 2012). Pumping terminates either when stages in Canal B drop below the off elevation for pump 1 (31.3 feet) or when the water depth relative to the mean bottom elevation in the impoundment exceeds 11 feet, which occurs when the stage within the impoundment is 28.2 feet (**Figure 15**).

Table 11. Pump parameters for drainage mode (SWET 2012).

Pump	Capacity (CFS)	On Elevation (ft)	Off Elevation (ft)
1	11.1	31.6	31.3
2	33.4	32.0	31.7
3	33.4	32.0	31.8

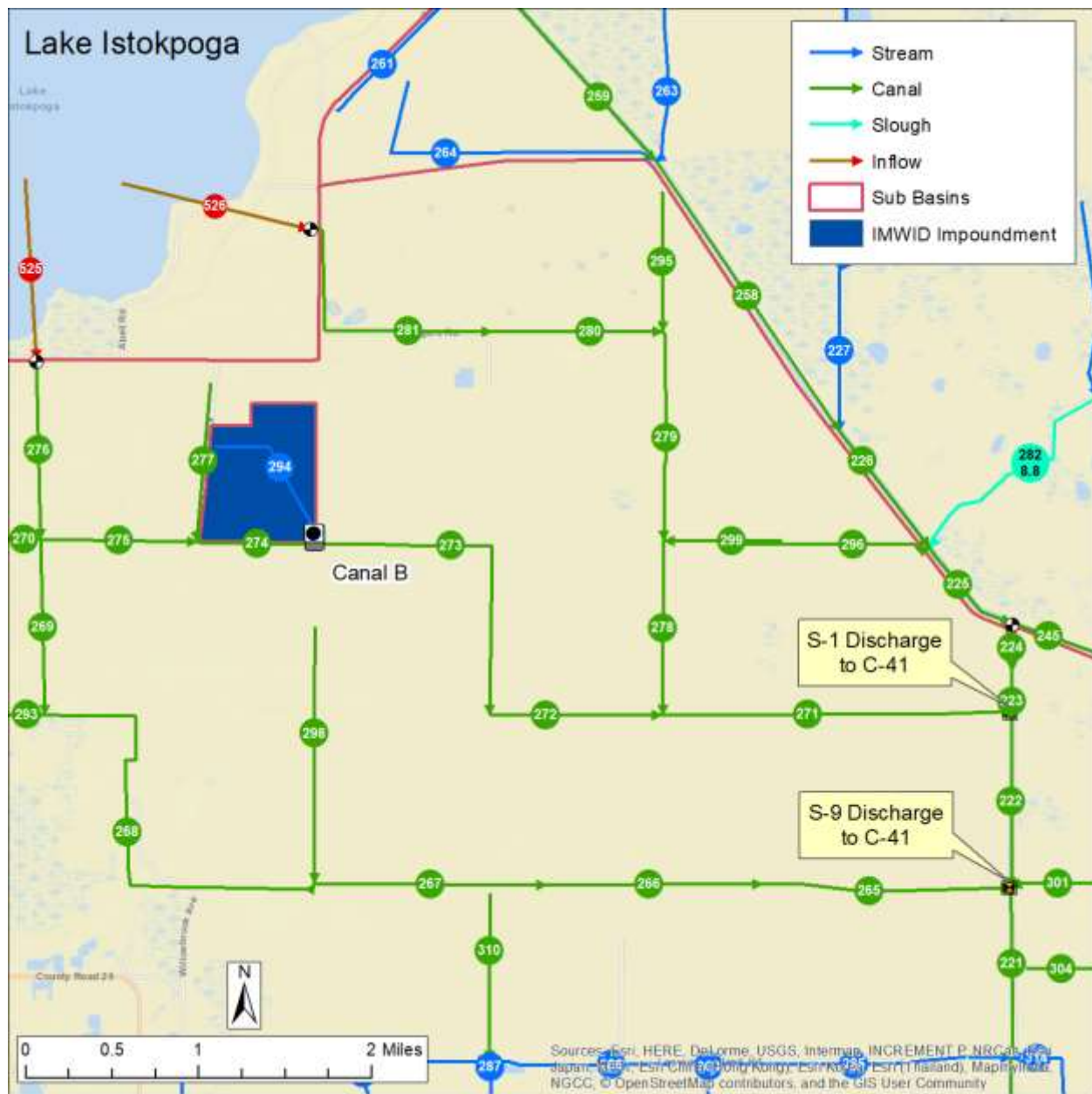


Figure 13. Locations of discharge structures S-1 and S-9 from the IMWID to C-41.

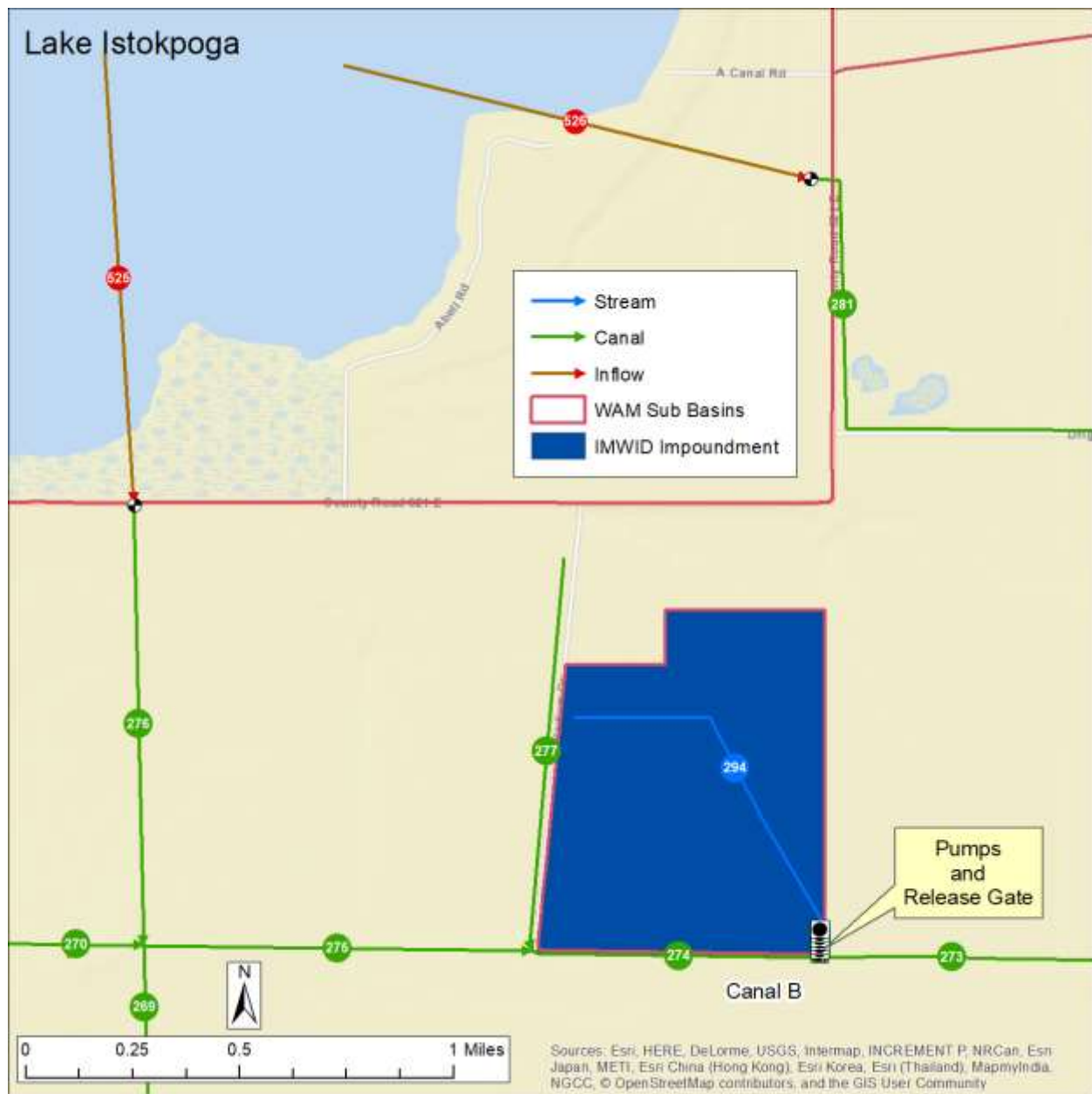


Figure 14. Location of the IMWID impoundment and adjacent reaches.

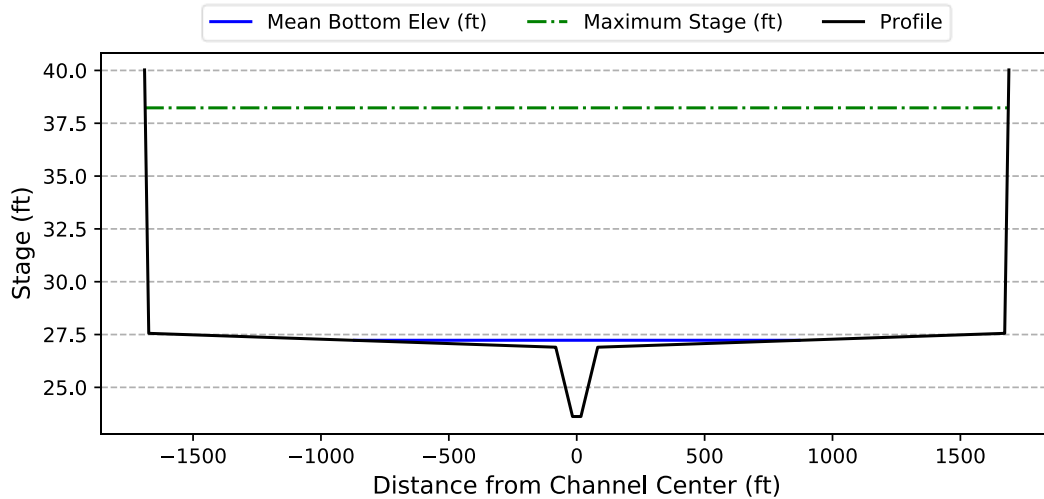


Figure 15. IMWID impoundment profile (WAM reach 294) used for the simulation with the maximum stage and mean bottom elevation indicated.

When irrigation is required in the IMWID water is released from Lake Istokpoga (represented by WAM inflow reaches 525 and 526) and enters the IMWID via gates at the top of WAM reaches 276 and 281, with operations of the gates controlled by stages within their respective reaches. The gates open when stages in the canals drop below 30.8 feet, and close when stages are above 31.5 feet (SWET 2012).

Drainage mode is activated when stages in Canal B drop below 30.8 feet, at which point the discharge gate from the impoundment opens. The gate will close when the stage in Canal B reaches 31.5 feet (SWET 2012). At no point will the pumps be operating when the gate is open.

4.2.3.2 Simulation Results

Simulated inflows out of and into the IMWID impoundment are shown in the upper pane of **Figure 16** with positive values indicating discharges out of the impoundment and negative values indicating pumping into the impoundment from Canal B. The lower pane of **Figure 16** shows simulated water depths within the impoundment.

Simulation results using the IMWID impoundment were compared with WAM simulations from the base conditions scenario (SWET 2017a). Volumes and TP loads from two IMWID canals that discharge into C-41A at IMWID structures S-1 and S-9 were examined for the comparison (**Figure 13**). **Figure 17** shows simulated TP concentrations for the base scenario (green line) and the IMWID impoundment (orange line) at the S-1 structure. Note that the concentrations with the IMWID impoundment are higher at times as water is released from the impoundment during dry periods. **Figure 18** shows simulated annual TP loads at S-1. The first two years are considerably lower due to the filling of the impoundment at the outset of the simulation but loads with the impoundment are lower for most years. Similar but reduced effects are shown for simulated TP concentrations at loads at the S-9 structure in **Figures 19** and **20**, respectively.

A summary of results is shown in **Table 12**, including a comparison with WAM simulations for the base conditions scenario (SWET 2017a). The results indicate a slight decrease of simulated discharge volume to C-41 as a result of the impoundment, with reductions of 6% and 2% at S-1 and S-9, respectively. Simulated TP load reductions to C-41 are 4% and 1% at S-1 and S-9, respectively.

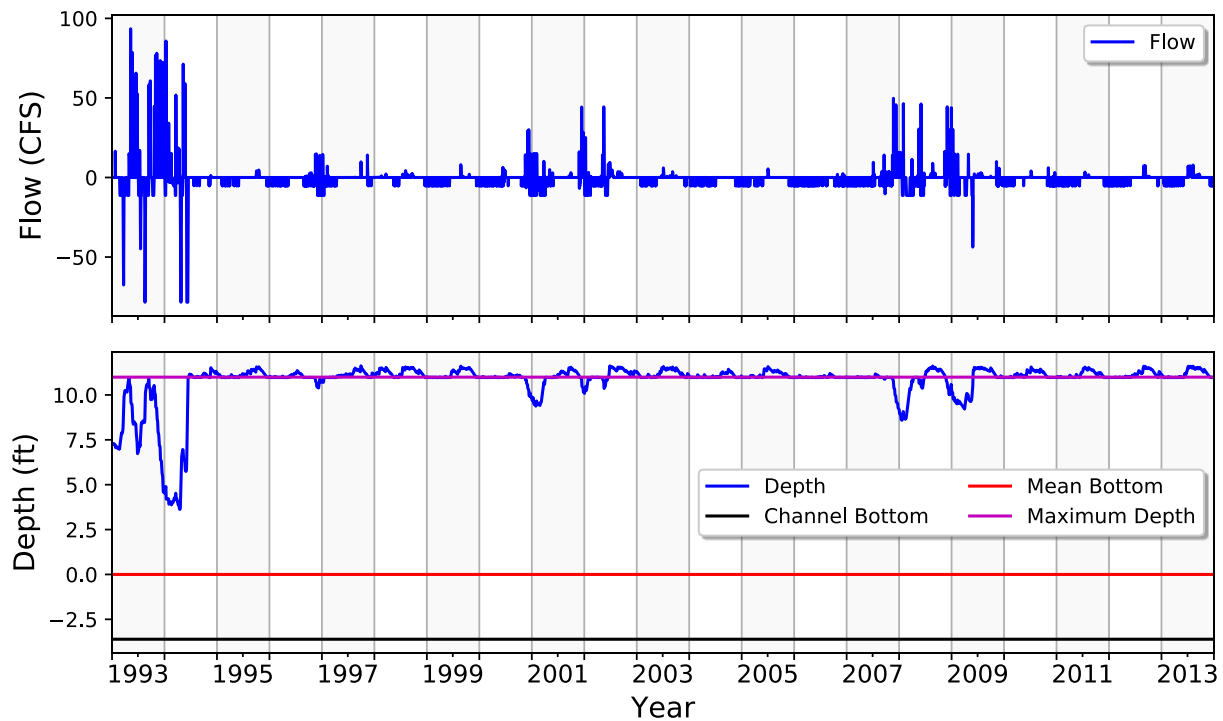


Figure 16. Simulated flows out of (positive values) and out of (negative values) the IMWID impoundment (upper pane), and water depths within the impoundment (lower pane) during the 1993-2013 simulation period.

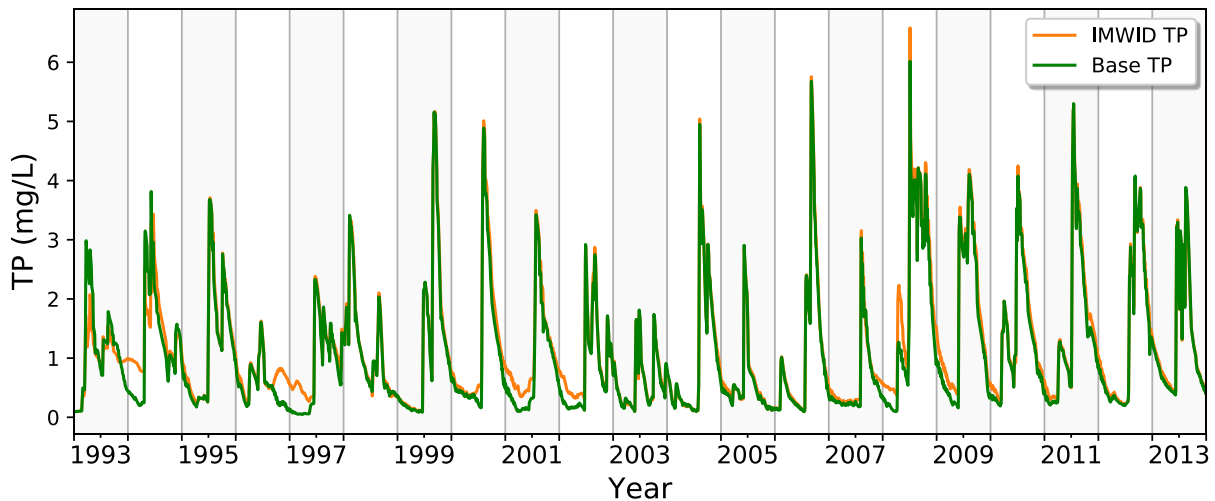


Figure 17. Simulated TP concentrations at IMWID structure S-1 comparing the base scenario (green line) and the scenario including the IMWID impoundment (orange line).

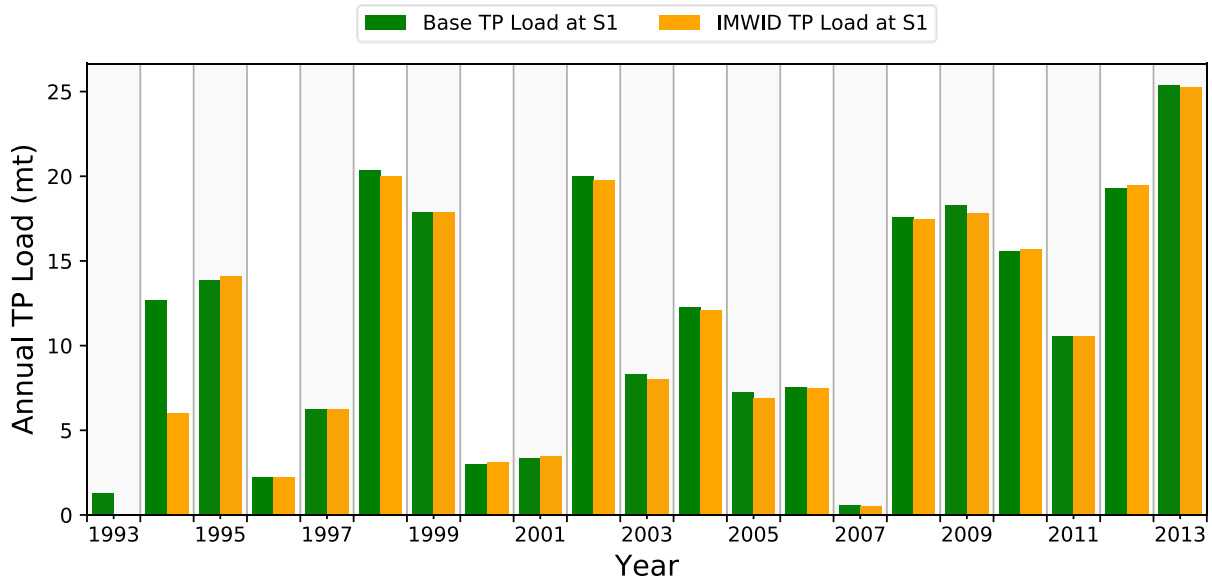


Figure 18. Simulated TP load discharging at IMWID structure S-1 comparing the base scenario (green bars) and the scenario including the IMWID impoundment (orange bars).

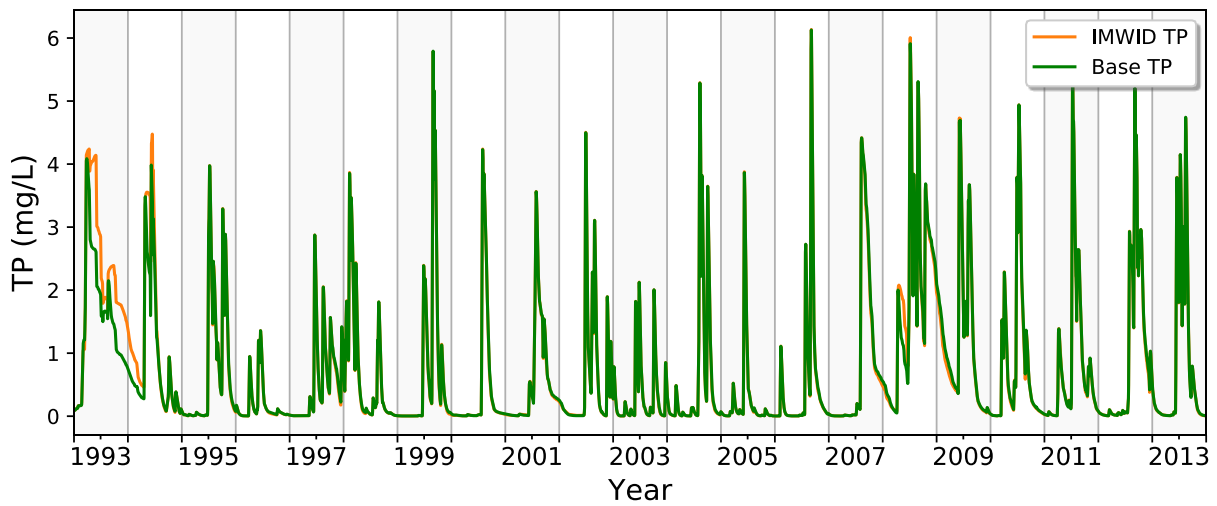


Figure 19. Simulated TP concentrations at IMWID structure S-9 comparing the base scenario (green line) and the scenario including the IMWID impoundment (orange line).

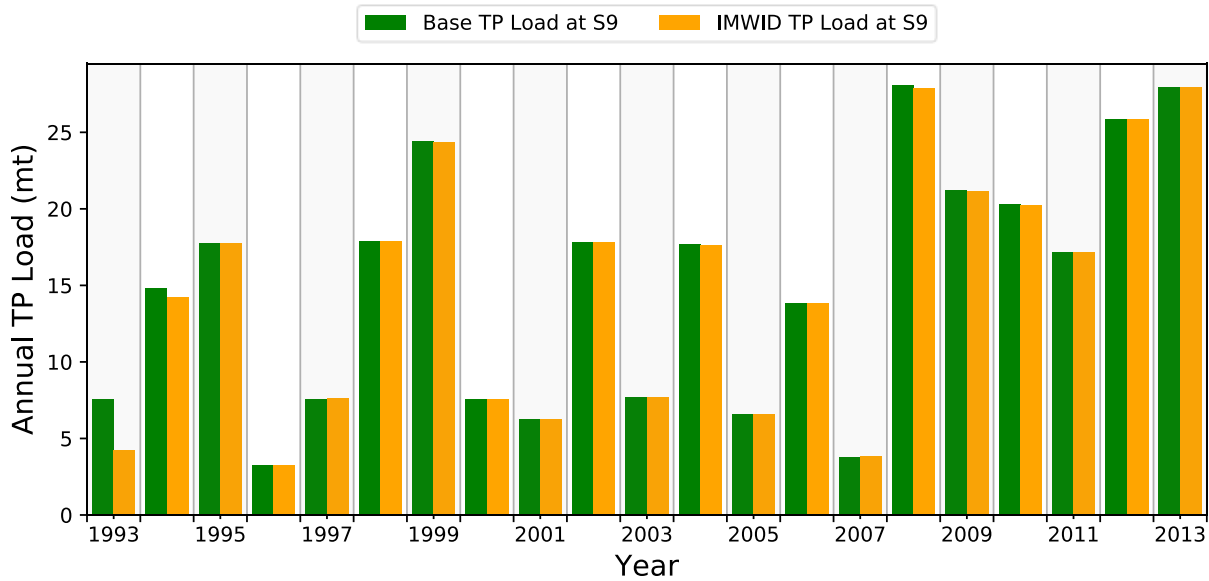


Figure 20. Simulated TP loads discharging at IMWID structure S-9 comparing the base scenario (green bars) and the scenario including the IMWID impoundment (orange bars).

Table 12. Simulated annual average volumes and TP loads discharging into C-41 from the IMWID from 1993-2013.

Location	Volume (ac-ft/y)			TP Load (mt/y)		
	Base	LOWCP (BV)	Change	Base	LOWCP (BV)	Change
S-1	4,922	4,628	-294	11.57	11.14	-0.43
S-9	10,189	10,023	-167	14.93	14.78	-0.15
Total	15,111	14,651	-461	26.50	25.92	-0.58

4.2.4 Summary of Indian Prairie LOWCP Results

The simulated annual average total volumes and TP loads discharging from the entire Indian Prairie sub-watershed are shown in **Table 13**. The simulated volume discharging from the sub-watershed decreased by 0.5%, while the simulated TP load decreased by about 4.7%.

Table 13. Simulated annual average volumes and TP loads discharging from the Indian Prairie sub-watershed.

	Volume (ac-ft/y)			TP Load (mt/y)		
	Base	LOWCP (BV & IMWID)	Change	Base	LOWCP (BV & IMWID)	Change
Sub-watershed Total	430,701	428,682	-2,019	107.0	101.9	-5.1

4.2.5 K-05 Wetland Attenuation Feature

This section describes the addition of components of the Tentatively Selected Plan (TSP), alternative 1BW of the LOWRP (USACE 2018a, b). The three components of the TSP are

1. The K-05 wetland attenuation feature (or WAF) (dark blue polygon) with a footprint of approximately 11,000 acres and a storage volume of approximately 43,000 acre-feet (ac-ft);
2. Aquifer storage and recovery (ASR) wells, including watershed ASR wells (red circles) and wetland attenuation ASR wells (yellow circles) with a storage volume of approximately 448,000 ac-ft per year;
3. The Paradise Run (approximately 4,084 acres) and Kissimmee River Center wetland restoration sites (approximately 1,196 acres) (green polygons);



Figure 21. The LOWRP Tentatively Selected Plan

The K-05 WAF is located within the Indian Prairie sub-watershed west of the C-38 canal, north of SR 78, east of the Brighton Reservation, and south of the C-41A canal. From the Draft Integrated Project Implementation/Environmental Impact Statement (PIR/EIS) (USACE 2018a):

“The WAF is to be used primarily for surface water storage to attenuate peak flows into Lake Okeechobee from the Kissimmee River Basin. The secondary purpose is to encourage habitat utilization through growth of wetland vegetation by maintaining water levels at 0.5–3 ft. during non-flow attenuation periods. Wetland attenuation ASR wells will rehydrate habitat during dry times to ensure that wetland conditions are maintained within the WAF footprint without sacrificing desired regional benefits for Lake Okeechobee and the northern estuaries. The WAF footprint, including the embankments, seepage canal, and other perimeter features, is approximately 12,500 acres with a storage capacity of approximately 43,000 ac-ft. A pump station located downstream of the existing S-84 structure on the C-41A canal serves as the water source for the proposed WAF. The pump draws water from the downstream area considered to be part of Lake Okeechobee.”

4.2.5.1 WAM Sub-Basin and Reach Setup

The K-05 WAF and the wetland restoration sites were incorporated into the WAM setup for the Indian Prairie sub-watershed, together with those ASR wells used for water supply for the WAF and the wetland restoration sites. The WAM simulation was developed as a separate scenario from the scenario discussed above that incorporated both the Brighton Valley impoundment and the IMWID project. This setup was done in this way because the K-05 WAF and Paradise Run wetland restoration features take water from, and return water to the C-38 canal. The footprint of the WAF covers portions of two basins within the sub-watershed, L-48 and L-59E (**Figure 22**). It splits the L-48 and L-59E basins of the base WAM scenario (“sub-basins” in WAM parlance; the existing boundaries are shown by the dashed green outlines) and will require alteration of the WAM sub-basin feature class, as well as alteration of the reach network. The suggested changes within WAM are to make the WAF a separate WAM sub-basin (K-05, shown in green in **Figure 22**) and split the areas of both the L-48 and L-59E basins that do not lie within the WAF into two parts. For the current L-48 basin, one part will comprise the east and south-east part of the basin (keeping the L-48 designation), while the second contains the north-west areas of the current basin, designated within the WAM setup as L-48-NW in **Figure 22**. This proposed L-48-NW WAM sub-basin is located adjacent to the L-59 canal and must either discharge into the L-59 canal at some point or into the C-40 canal along the south-westernmost part of the basin. Similarly, the L-59E basin is split by the WAF, the north-easternmost part of the current basin is labeled in the WAM setup as L-59E-NE, although most of this area lies within the Paradise Run wetland restoration site.

The WAM base scenario stream network and the stream network for the proposed WAF scenario are shown in **Figure 23**. The stream network in WAM was modified by removing the existing streams and canals within the WAF footprint and adding reaches (WAM reaches 191, 85, and 86) within the WAF to simulate flow within the WAF. Each of these reaches that represent an impoundment within the WAF was assigned an appropriate width and cross-sectional profile so that the area of each reach segment matches the corresponding area in the WAF. Reaches within the proposed L-48-NW basin that currently enter the footprint of the proposed WAF were rerouted to drain to either C-40 or L-59. According to the TSP (USACE 2018b), the section of the L-59 canal that bisects the WAF will be removed during construction and the canal will be blocked at the west side of the WAF. The eastern part of L-59 will serve as a discharge point from the WAF. These were added to the WAM scenario for the WAF as WAM reaches 81 and 84. In addition, a restored wetland feature (the Paradise Run wetland) was incorporated into the WAM setup immediately to the east of the WAF. Flow within the Paradise Run wetland was simulated by the addition of WAM reaches 53 – 55.

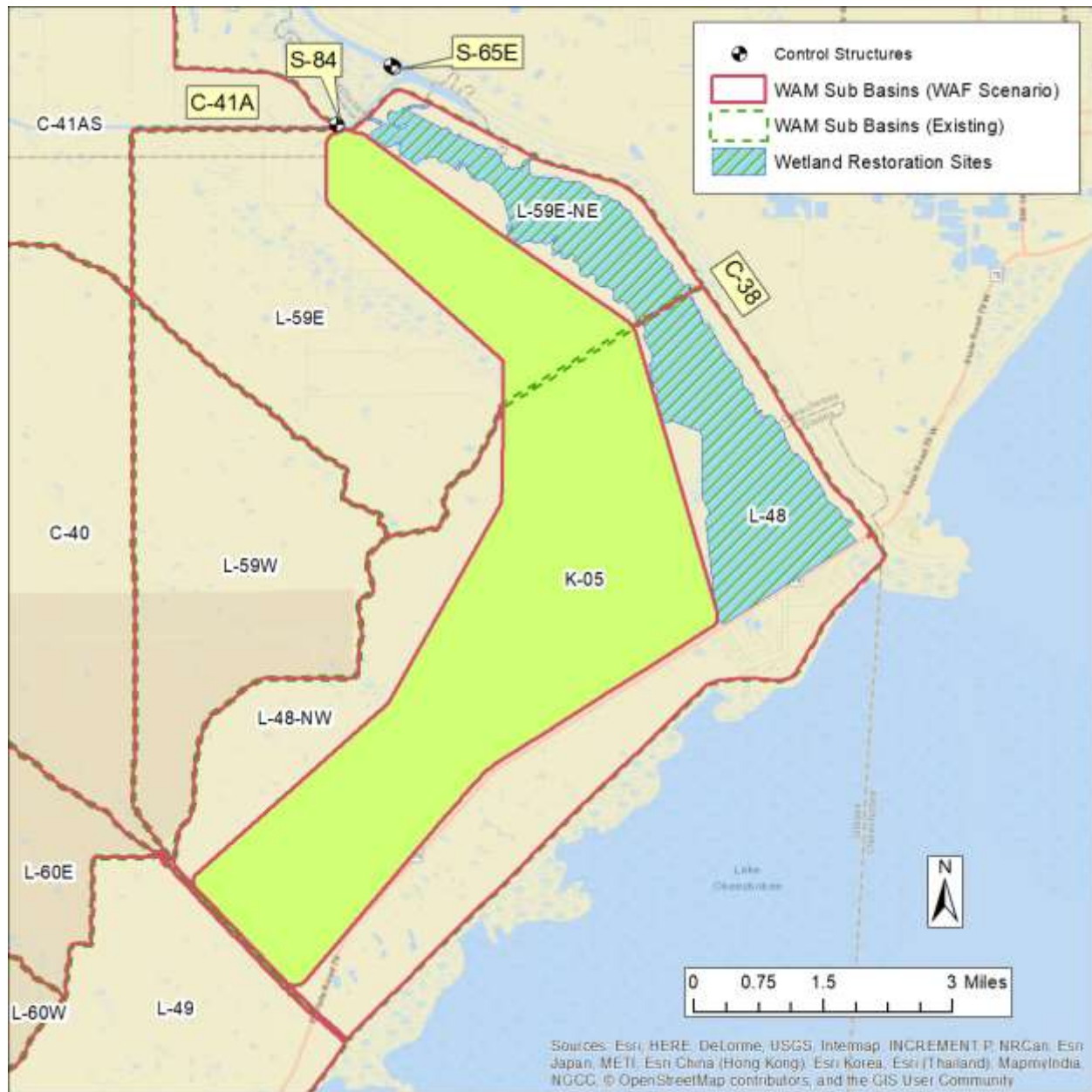


Figure 22. The location of the proposed WAF and Paradise Run wetland restoration site in the Indian Prairie sub-watershed.

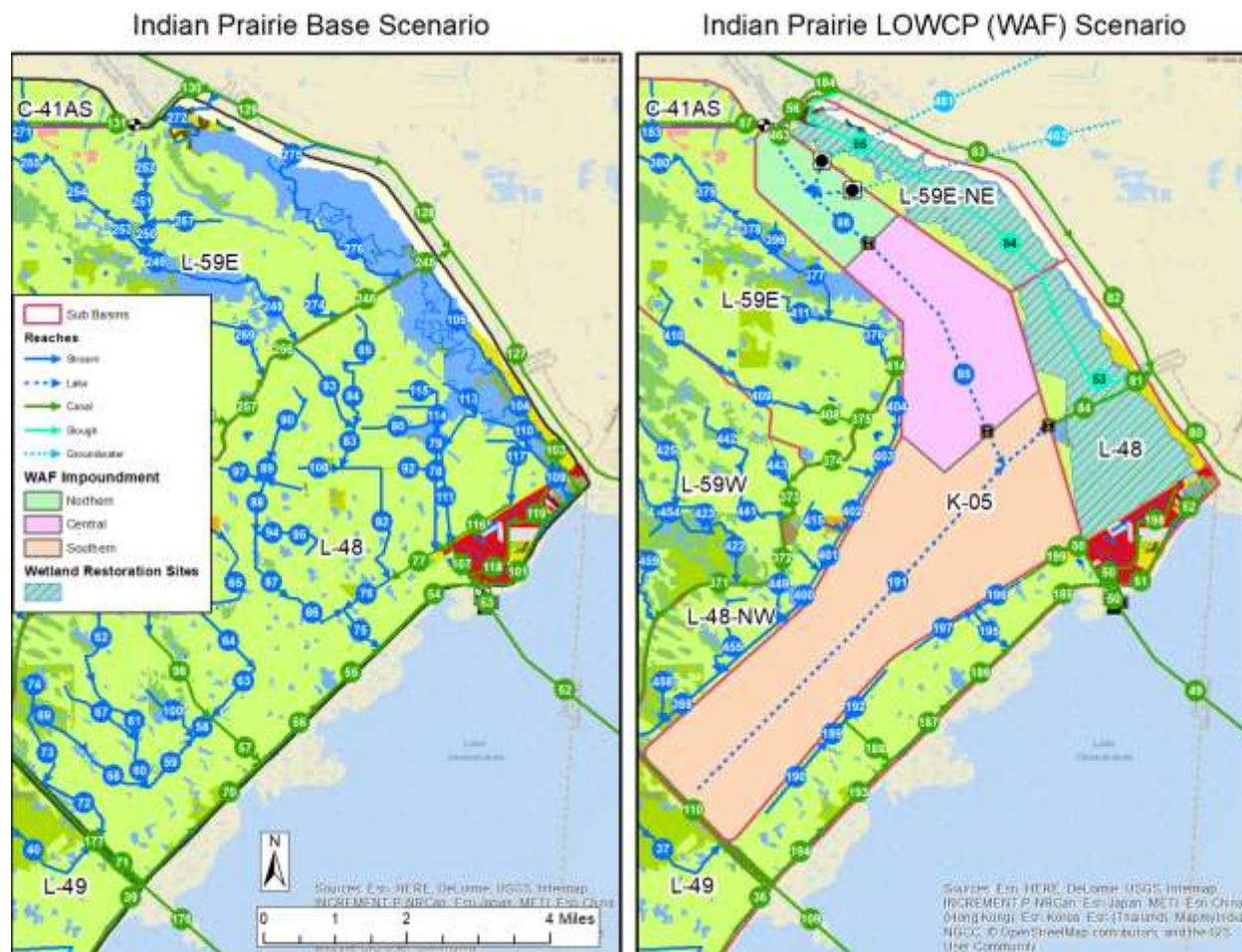


Figure 23. Base scenario (left) and LOWCP WAF scenario (right).

4.2.5.2 Operations

4.2.5.2.1 Lake Okeechobee/WAF Operations

Flow into the WAM is simulated by pumping from the C-41A canal just downstream of the tailwater (east) end of the S-84 control structure into the northernmost edge of the WAF. This is represented in the simulation by WAM reach 56 (the reach east of S-84) and reach 463 (inflow canal to the WAF). The pump station will be capable of pumping approximately 1,500 CFS. Since there are no control structures between S-84 and Lake Okeechobee the water will contain a significant fraction of water from the lake, in addition to water discharged from S-65E. The number and capacity of each pump has not yet been determined (USACE 2018b), but a single pump with the maximum capacity of 1,500 CFS was added to the WAM setup. Additional inlets into the K-05 will include 15 ASR wells from the Upper Floridan Aquifer (UFA) at 7.7 CFS each (116 CFS total flow) and 10 ASR wells from the Avon Park Permeable Zone (APPZ) also at 7.7 CFS each (77 CFS total flow).

The operational intent of using K-05 WAF is to receive and store water at periods of high lake stages and return water back during periods of low flows. Note that the accounting period has not yet been determined at the time of this report (USACE 2018b). The K-05 WAF feature was modeled, and is intended to be operated as follows (USACE 2018b):

- K-05 will accept water (“filling phase”) when the official Lake Okeechobee elevation is above the stages listed in **Table 14** in the column “Inflow to K-05 WAF” **and** the depth in the feature is at or below 4.0 feet.
- K-05 will return water to the lake (“drainage phase”) when the lake elevation is below the stages listed in **Table 14** in the column “Outflow from K-05 WAF” **or** when the depth in the feature is above 4.
- K-05 will deliver water to Paradise Run wetland but the operations for this have not yet been determined or modeled.

The operational criteria in **Table 14** are given by the “Lake Okeechobee Diversion and Recover Schedule for K-05” (USACE 2018b).

Table 14. Lake Okeechobee stages for diversions to and/or from the K-05 WAF.

Lake Okeechobee Stage (ft NGVD)		
Date	Inflow to K-05 WAF	Outflow from K-05 WAF
Jan 1	14.79	14.4
Jan 15	14.78	14.39
Feb 15	14.51	14.23
Mar 15	14.14	13.94
Apr 15	13.75	13.55
May 15	13.36	13.16
Jun 15	13.05	12.85
Jul 15	12.88	12.68
Aug 15	12.9	12.7
Sep 15	13.14	12.89
Oct 15	13.74	13.21
Nov 15	14.32	13.6
Dec 15	14.7	13.97
Dec 31	14.79	14.13

For the WAM simulations, when the schedule indicates that water can be pumped into the K-05 WAF, diversion into the WAF from C-41A will occur at the rate of 1,500 CFS. Outflow from the K-05 WAF is controlled by adjusting the weir crest elevations at the outflow point (the bottom of WAM reach 191) and at the two weirs within the K-05 WAF (the bottom of WAM reaches 85 and 86) (**Table 15**). When Lake Okeechobee stages are low enough to permit discharge from the WAF (**Table 14**, “Outflow”), the weir crest elevations are set to the drainage phase elevations. Otherwise they are set to the default elevations. No weir crest elevations were specified in the documentation provided (USACE 2018b) other than that the K-05 WAF will only accept water if the depth within the impoundment is less than 4 feet, and water will be released from the K-05 WAF if the depth is greater than that. Accordingly, the default crest elevations are set to maintain a mean water depth of 4 feet and during drainage elevations the crest elevations are set so that some water remains within the K-05 WAF impoundments.

Table 15. Weir crest elevations within the K-05 WAF. Default elevations are used during non-drainage phases.

Location	Crest Elevation (default, feet NGVD)	Crest Elevation (drainage, feet NGVD)
Northern WAF (WAM Reach 86)	27.2	22

Central WAF (WAM Reach 85)	22.8	17.6
Southern WAF Outflow (WAM Reach 191)	21.4	16.2

4.2.5.2.2 ASR/WAF Operations

The UFA and APPZ are represented by WAM groundwater reaches (WAM reaches 461 and 462, respectively). Groundwater reaches in WAM are not subject to evaporation, and in this specific instance, do not simulate nutrient attenuation. Simulated diversions and recoveries between ASR and the K-05 WAF occur at the stages in the WAF impoundment (WAM reach 86) listed in **Table 16**.

Table 16. Pumping operations between the K-05 WAF and ASR.

Pump Action	Pump on stage (ft NGVD)	Pump off stage (ft NGVD)	Capacity (CFS)
WAF to UFA Pump (diversion)	26.25	26.08	116
WAF to APPZ Pump (diversion)	26.25	26.08	77
UFA to WAF Pump (recovery)	23.75	23.92	116
APPZ to WAF Pump (recovery)	23.75	23.92	77

4.2.5.2.3 K-05 WAF WAM Profiles

The cross-sectional profiles of the three WAM reaches that simulate the K-05 WAF were set so that the length and width of each reach correctly represent the area of each impoundment (**Table 17**). The simulated total area of the K-05 WAF is 10,834 acres.

Table 17. Parameters for WAM reaches representing the K-05 WAF.

Reach	Length (ft)	Width (ft)	Area (ac)
Northern WAF (WAM 86)	8,215	5,102	962
Central WAF (WAM 85)	14,193	8,809	2,870
Southern WAF (WAM 191)	22,159	13,763	7,001

4.2.5.3 Simulation Results

The simulation period for WAM was from 1993 through the end of 2013. Simulated annual pumped volumes (ac-ft/y) into the K-05 WAF and subsequent discharge from the K-05 WAF are shown in **Figures 24** and **25**, respectively. The simulated annual average pumped inflow volume to the K-05 WAF was 115,097 ac-ft/y while the simulated annual average discharge volume from the WAF was 61,775 ac-ft/y (**Table 18**). Rain accounted for the addition of 43,824 ac-ft/y directly into the K-05 WAF.

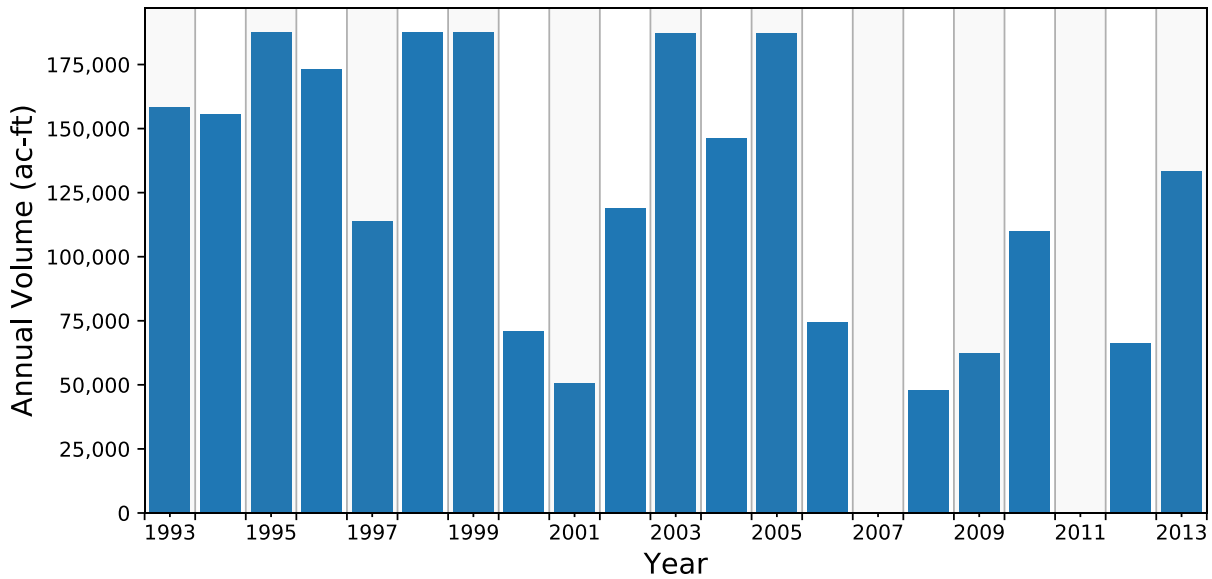


Figure 24. Simulated annual pumped inflow volumes to the K-05 WAF from C-41A over the 1993-2013 time frame.

Figure 26 shows time series of the pumped inflow TP concentrations to the K-05 WAF (orange line) and the discharge concentrations at the bottom or WAM reach 191 (green line). Note that the spikes in inflow concentrations in 1995 and 1999 correspond to two of the years with the highest loadings to the impoundment.

Simulated annual TP loadings pumped into the WAF and discharged via the weir at the bottom of WAM reach 191 are shown in **Figures 27 and 28**, respectively. The simulated annual average TP load pumped into the K-05 WAF was 13.2 mt/y while the simulated annual discharge TP load was 3.4 mt/y (**Table 18**) for a simulated reduction of 9.8 mt/y. Rain accounted for the direct addition of 0.4 mt/y of TP.

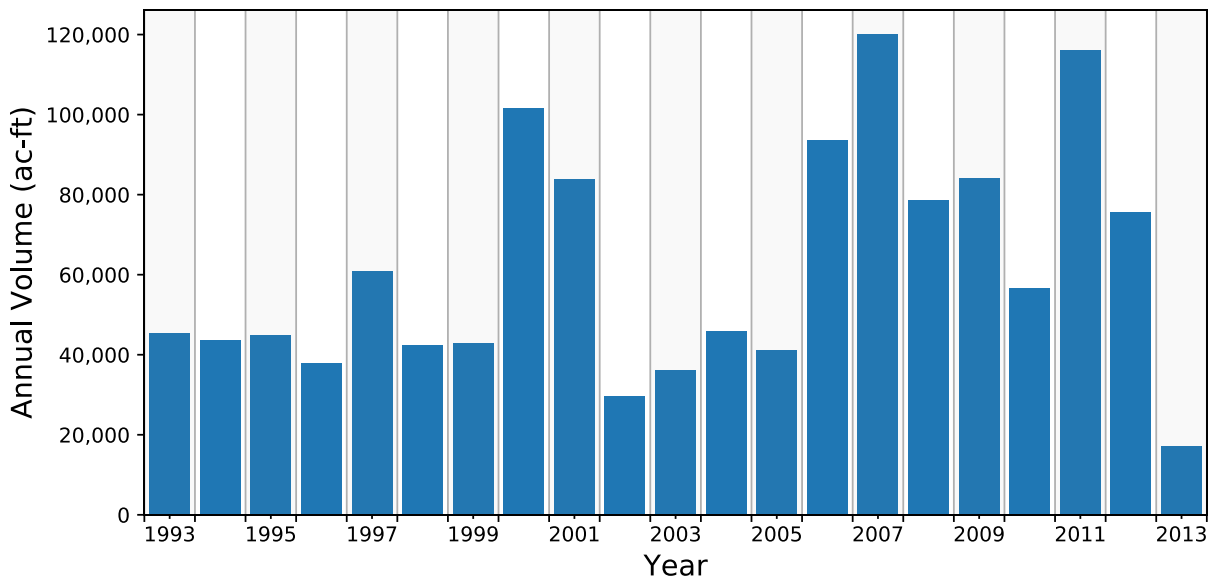


Figure 25. Simulated annual discharge from the K-05 WAF over the 1993-2013 time frame.

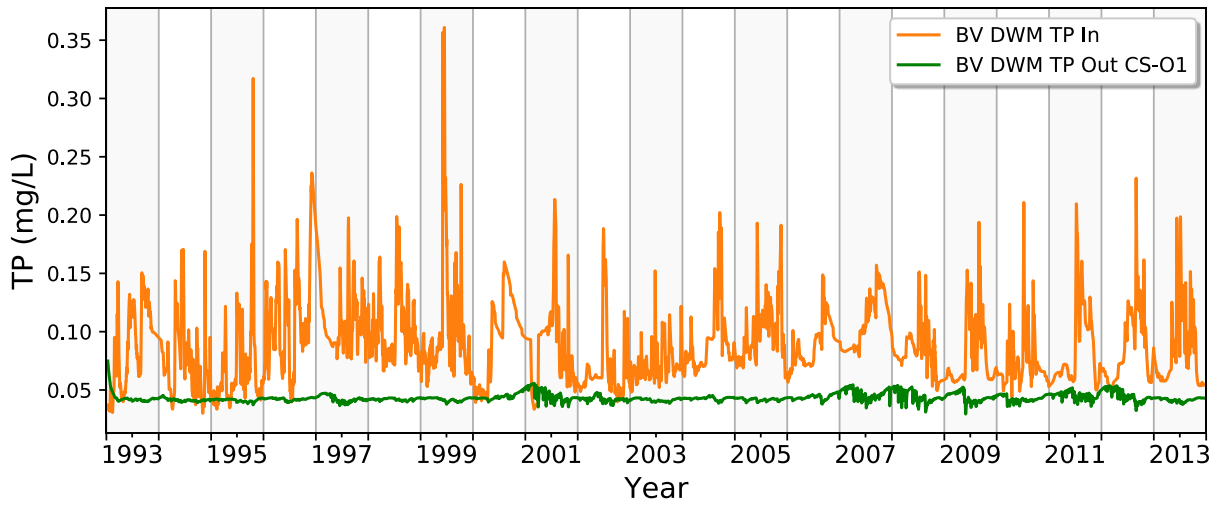


Figure 26. Simulated TP concentration at the inflow (orange line) and outflow (green line) from the K-05 WAF.

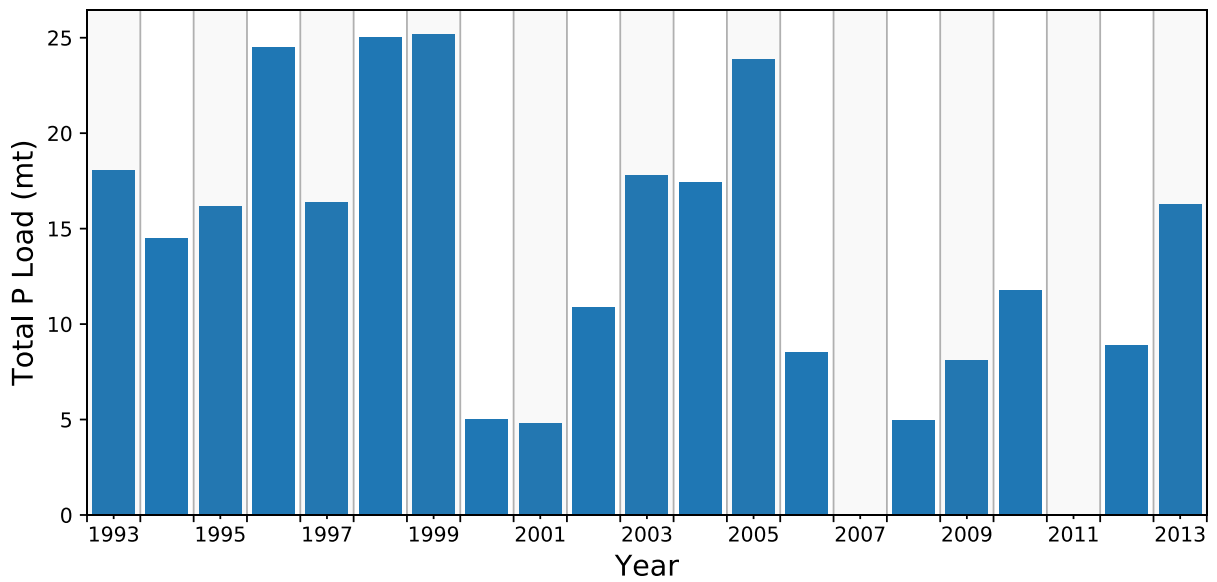


Figure 27. Simulated annual pumped inflow TP loading to the K-05 WAF from C-41A over the 1993-2013 time frame.

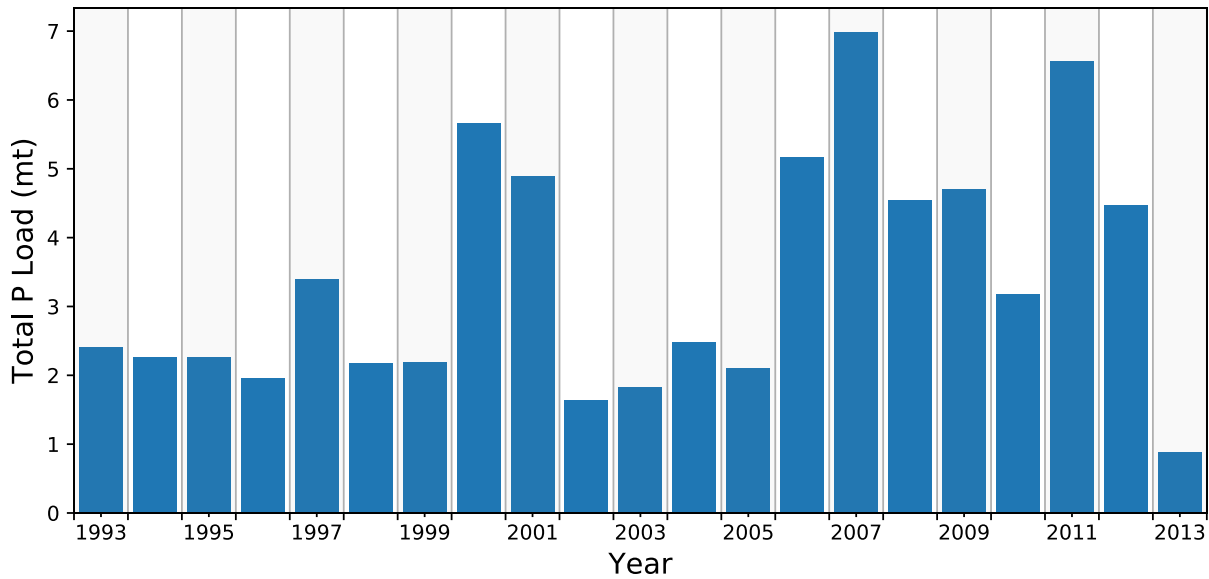


Figure 28. Simulated annual TP discharge loading from the K-05 WAF into C-38 over the 1993-2013 time frame.

Simulated annual average inflow volume and TP load to the UFA over the 1993-2013 time frame were 51,375 ac-ft/y and 5.1 mt/y, respectively, while 22,450 ac-ft/y and 2.5 mt/y were recovered from the UFA during the same period. Simulated annual average inflow volume and TP load to the APPZ over the 1993-2013 time frame were 34,093 ac-ft/y and 3.4 mt/y, respectively, while 14,899 ac-ft/y and 1.7 mt/y were recovered from the AAPZ during the same period (**Table 18**). The TP load reductions from the Paradise Run wetland are approximately 0.12 mt/y removed between the entry (WAM reach 55) and the exit to C-38 (WAM reach 53), although this is very uncertain since the details of structure emplacement to control flow into the wetlands from C-38 are not yet fully determined (USACE 2018b).

Table 18. Annual average simulated volume and TP load results from the WAM K-05 WAF simulation.

Location	WAF ac-ft/y	WAF TP mt/y
C-41A Inflow	115,097	13.2
Rainfall	43,824	0.4
WAF to UFA	51,375	5.1
WAF to APPZ	34,093	3.4
UFA to WAF	22,450	2.5
APPZ to WAF	14,899	1.7
WAF Outflow	61,775	3.4

4.2.6 Kissimmee River Center Wetland Restoration

Additionally, the TSP includes the Kissimmee River Center restored wetland. This wetland lies within the Lower Kissimmee sub-watershed. The base WAM scenario for Lower Kissimmee (SWET 2017a) was modified to include this wetland by altering the existing drainage through the wetland so that water is able to flow into the wetland via a weir on C-38 (**Figure 29**). Water enters the wetland (WAM reach 335 and 346) and flows across the restored wetland before discharging over a weir into WAM reach 344, which in

turn discharges into C-38. No weir crest elevations were specified in the planning documents. Other than these changes no other alterations were done to the base scenario.

Figure 30 shows simulated TP concentrations at the inflow point from the Kissimmee River to the Kissimmee River Center wetlands (orange line) and the outflow from the wetlands back to the river (green line). The outflow concentrations show attenuation of the peak values as water flows through the wetland. Note, however, that unlike the impoundments discussed for previous scenarios, water from upland areas to the west of the wetlands also flows into the wetlands and provides additional loading to the wetland. This overland flow would otherwise discharge directly into the Kissimmee River. Comparison of simulated TP loadings between the base scenario and the modified scenario with the Kissimmee River Center wetlands at the S-65E structure indicate that the simulate TP load is reduced by approximately 0.15 mt/y.

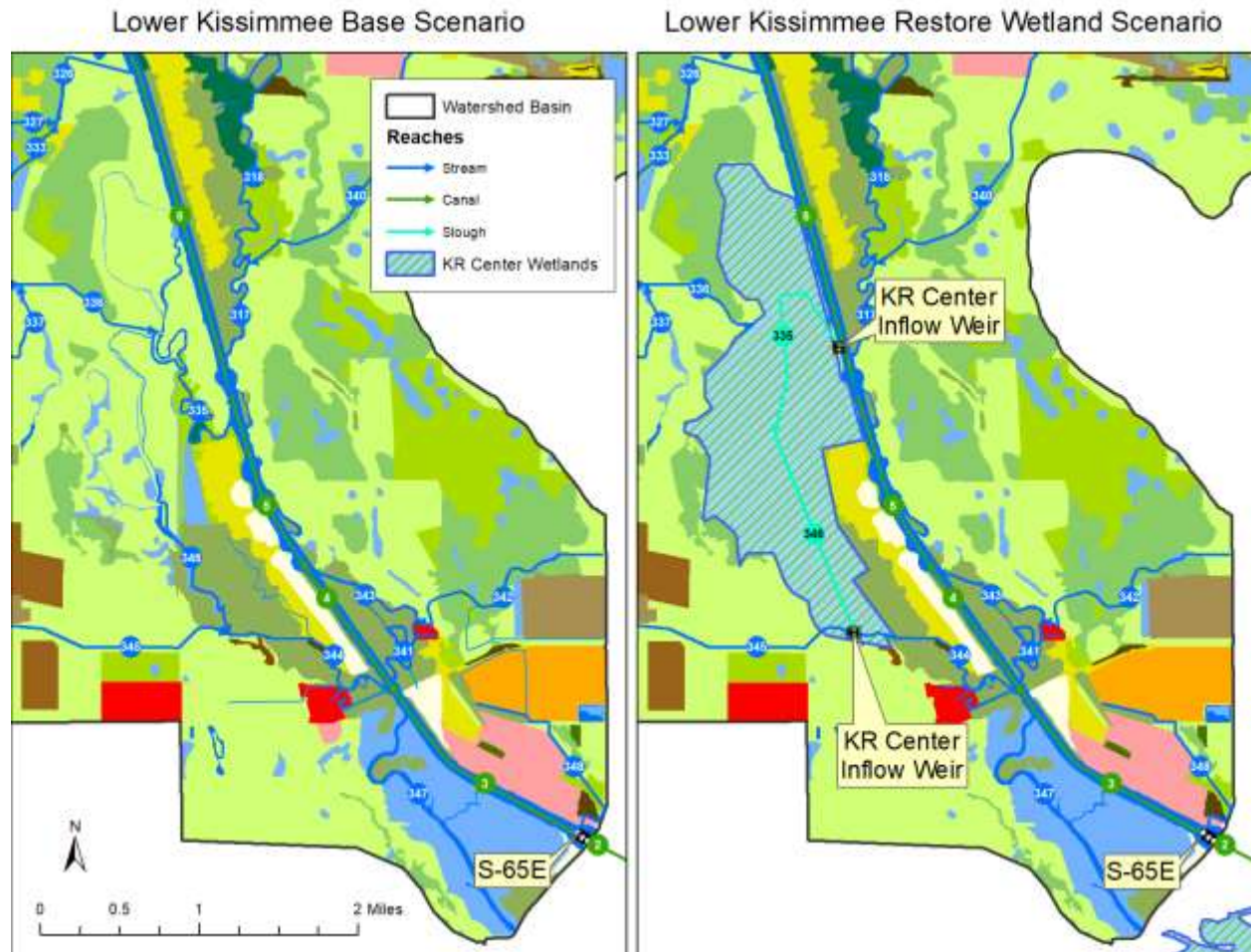


Figure 29. The Kissimmee River Center restored wetland scenario setup (right pane), compared with the base scenario (left pane).

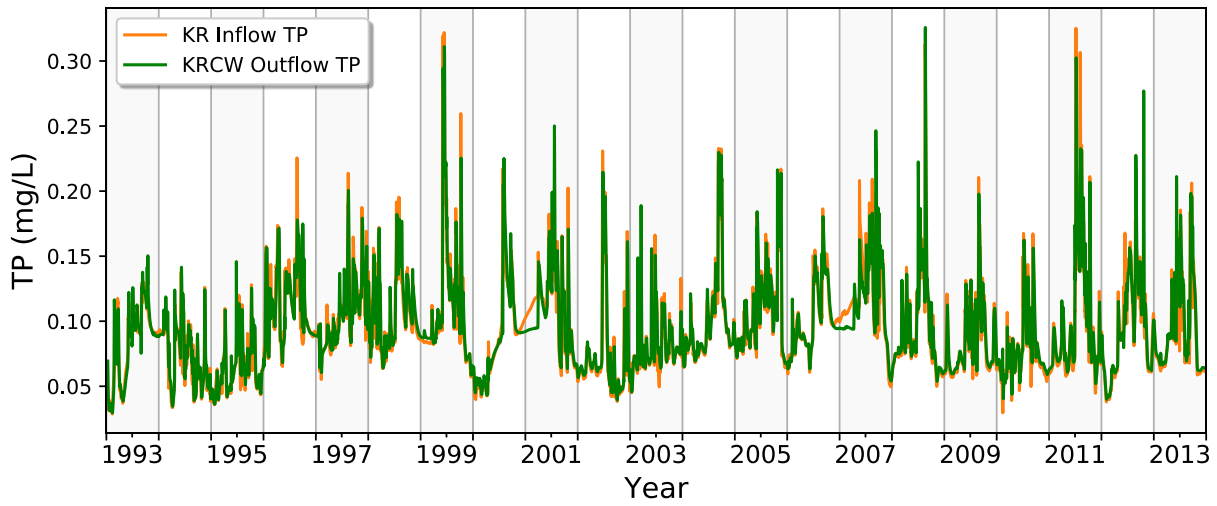


Figure 30. Comparison of simulated TP concentrations at the inflow point from the Kissimmee River to the Kissimmee River Center wetlands (orange line) and discharge concentrations back to the river from the wetlands (green line).

4.3 Taylor Creek/Nubbin Slough Lake Okeechobee Watershed Construction Projects

4.3.1 Lakeside Ranch STA and Brady Ranch STA

This section describes the addition of the LRSTA and BRSTA to the WAM model for Taylor Creek/Nubbin Slough. The WAM setup for Taylor Creek/Nubbin Slough (**Figure 31**) was developed as part of the WAM recalibration to estimate water flows and nutrient loads across all the nine sub-watersheds in the LOW (SWET 2017a, 2017b). The Taylor Creek/Nubbin Slough sub-watershed was simulated as part of the northern LOW sub-watersheds (SWET 2017a) and included the Taylor Creek STA and Nubbin Slough STA, but the LRSTA was not simulated within WAM since it was not fully operational at the end of the simulation period (1975-2013). In addition, the Brady Ranch STA (BRSTA) is an additional STA proposed for the south-east part of the Taylor Creek/Nubbin Slough sub-watershed, east of the L-48 canal. The BRSTA was also added to the WAM simulation. Shapefiles were obtained from the SFWMD that contain polygons outlining these STAs, as well as a document describing the operations of the LRSTA (CDM Smith 2018). Operations for the BRSTA were obtained from the SFWMD (J. Zhang, personal communication, December 18th, 2018).

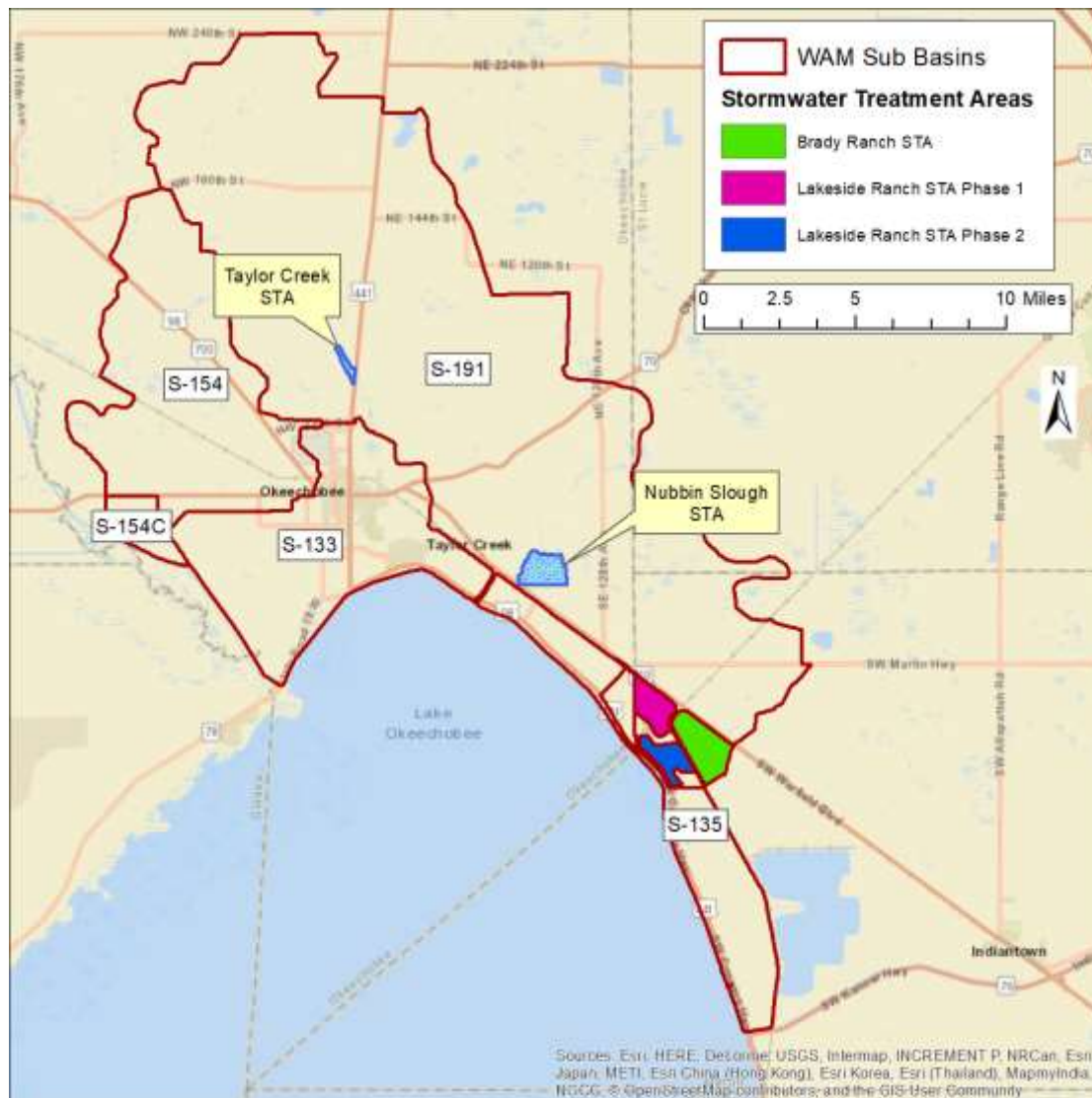


Figure 31. The WAM setup for the Taylor Creek/Nubbin Slough sub-watershed, with the location of the LRSTA and BRSTA shown.

In south Florida, STAs are constructed treatment wetlands designed to remove nutrients (P in particular) from agricultural stormwater runoff. The objective of the LRSTA is to reduce the mass and concentration of TP entering Lake Okeechobee. The LRSTA serves as a large regional facility that provides treatment of stormwater from the S-191 and portions of the S-135 subbasins in the Taylor Creek/Nubbin Slough sub-watershed (**Figure 45**). Model results (CDM Smith 2018) indicate a TP removal rate of approximately 19 to 25 metric tons per year (mt/y) for the overall LRSTA system. To maximize TP removal rates, the design is based on the management of water depths throughout the LRSTA to be an optimum of 1.5 ft (18 in) for vegetation health and uptake with an allowable minimum depth of 6 in and a maximum allowable depth of 28 in with mature wetland vegetation (CDM Smith 2018).

4.3.1.1 LRSTA Design

The design of the northern and southern LRSTAs is given in detail in CDM Smith (2018). The section describing the flow patterns is quoted below.

“Within the STA, water moves through parallel flow-ways, from the S-650 Pump Station along the north boundary Distribution Canal C-650 (**Figure 32**). Each flow-way is subdivided by internal levees. The seepage collection ditches are located along all four sides of the LRSTA-N and along the east, west and north boundary of the LRSTA-S. The west and south seepage collection ditches of the LRSTA-N discharge into a seepage ditch running southwest and connect with existing ditches north of Cell 4 and then through an existing outlet to the L-47 Canal. The north seepage collection ditch in LRSTA-S continues to the west boundary and connect with the west seepage collection ditch. The east seepage collection ditch from LRSTA-N continues along the east boundary of LRSTA-S and discharge to the Outlet Canal along the south boundary, as shown in **Figure 33**. It should be noted that the Outlet Canal also serves as means to control seepage along the south perimeter.

“Within the STA, the vegetative communities will provide water quality treatment. The intent of the vegetation management is to sustain an emergent macrophyte community that will likely be dominated by cattails. The water quality improvement in the STA is not solely attributed to direct uptake of TP by the vegetation, but also to physical (i.e., settling and burial) and chemical (i.e., adsorption/ precipitation) processes for TP removal.

“Flows travel in a southward direction through the treatment cells and are released to the LRSTA-N Outlet Canals through control structures. Flow is then transferred to the LRSTA-S distribution canal along the north boundary of the LRSTA-S. The stormwater in the LRSTA-S flows in a southward direction as well. It then is collected in the outlet structures from Cells 6 through 8 and discharged into the outlet canal. The water then flows to the west to be discharged to the L-47 Canal via culvert S-675 under U.S. Highway 98. The flow pattern for the LRSTA-N&S is shown in the schematic in **Figure 34**.”





Figure 34. STA cell layout and flow directions in LRSTA (Figure 5 in CDM Smith 2018).

Water is delivered to LRSTA via the S-650 pump station, which pumps from the L-64 canal at the north-western edge of LRSTA-N and pumps into a distribution canal that runs along the northern end of LRSTA-N. Water flows south through the three cells comprising LRSTA-N. Discharge out of the cells

into the collection canal that runs along the southern edge of LRSTA-N is controlled by a set of weirs. From the collection canal, water flows via the canal connecting LRSTA-N to LRSTA-S and subsequently into a distribution canal along the northern edge of LRSTA-S where flow splits to the west and east. From the western part of the distribution canal flow cascades through the three cells comprising the western side of LRSTA-S to the discharge point at the collection canal. Flow from one cell to the next is controlled by weirs. Flow from the eastern part of the distribution canal moves through the two cells of the central and eastern part of LRSTA-S, with discharge to the collection canal controlled by weirs. Water discharges from the collection canal to the L-47 canal via culvert S-675 (**Figure 33**).

The operation of the LRSTA takes water from the S-191 basin and discharges into the S-135 basin. To balance the distribution of water between the two basins, the S-191A pump station is being constructed to move water back into the S-191 basin from the S-135 basin. The S-191A pump station is located at the northern end of the L-47 canal and discharges into the C-59 canal.

4.3.1.2 WAM Implementation

The Taylor Creek/Nubbin Slough WAM setup was modified to incorporate the major features of the LRSTA as described by CDM Smith (2018). To simulate the operation of the LRSTA in WAM, the two LRSTAs were modeled as two single impoundments that were not divided into cells. This was done for computational efficiency, since the detail required to simulate each cell independently would greatly increase simulation times without having significantly different overall results.

4.3.1.2.1 Reaches

LRSTA-N and LRSTA-S were each simulated in WAM as a single reach, with the two reaches connected by a canal (**Figure 35**). LRSTA-N is represented by WAM reach 138, LRSTA-S is represented by WAM reach 291, and BRSTA is represented by WAM reach 239. The canal connecting LRSTA-N and LRSTA-S is WAM reach 137. LRSTA-S discharges into a canal (WAM reach 290) that in turn discharges into one of the reaches representing L-47 (WAM reach 125). BRSTA discharges into WAM reach 292 which in turn discharges into L-47 at the same point as WAM reach 290.

The length and cross-sectional profile of each WAM reach representing an STA were calculated so that the total area of the reach under optimal depth conditions matched the area given in Table 1 in CDM Smith (2018). The parameter values used for WAM reaches are shown in **Table 19** below.

Table 19. Reach parameters for LRSTA and BRSTA.

Treatment Area	WAM Reach	Area (ac)	Length (ft)	Width (ft)	Bottom Elevation (ft)	Mean Bottom (ft)
LRSTA-N	138	929	11,381	3,560	24.3	26.1
LRSTA-S	291	788	9,803	3,504	18.0	19.8
BRSTA	239	1,661	12,044	6,007	24.1	25.9

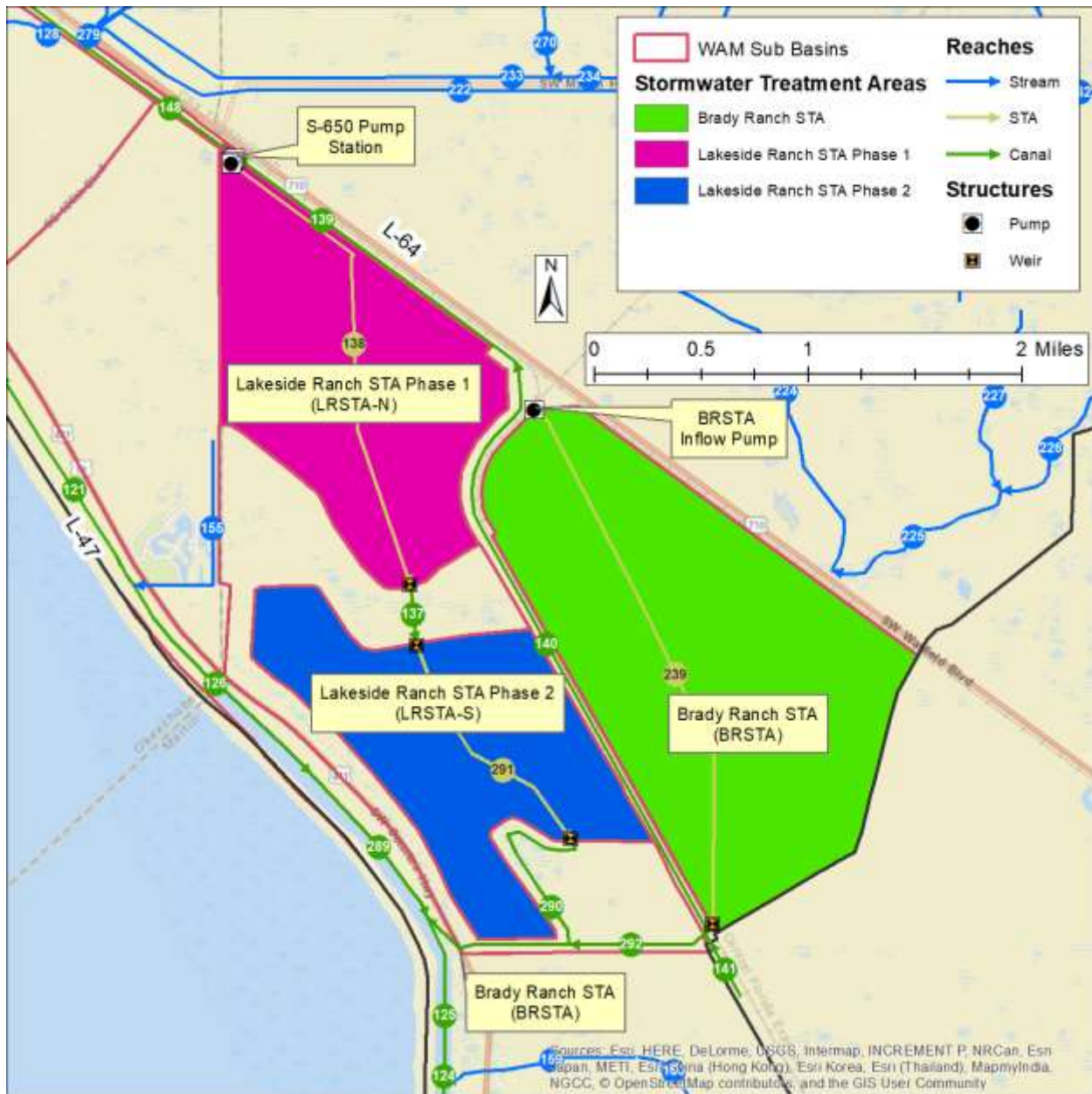


Figure 35. WAM setup of the LRSTA and BRSTA. WAM reach numbers are shown for reach.

4.3.1.2.2 Profiles

The general setup of cross-sectional profiles in WAM is discussed in **Section 4.1.1**. **Table 20** shows the cross-sectional profiles assigned for the WAM reaches that make up the LRSTA and BRSTA. Each profile has a narrow channel that is 33 feet wide when the water depth is 1.6 feet deep. The profile spreads out to a width of between 3,500 and 6,000 feet (depending on the STA) at a depth of 2 feet. Thus, the reach bottom changes from essentially dry (only the inner channel has water) to fully submerged as the water depth in the reach changes from 1.6 to 2 feet, i.e., 4.8 inches. The optimal (average) depth for each STA was determined to be 18 inches deep (CDM Smith 2018). For the STAs simulated here, this is 18 inches over the midpoint between 1.6 and 2 feet, i.e., 1.8 feet, or 21.6 inches. This implies that the optimum water surface in each STA is 39.6 inches (~1 meter) above the bottom of the reach, including the depth of the narrow inner channel. The reach bottom elevations and the mean bottom elevation (1.8 feet above the

bottom) for each WAM reach representing the LRSTA and BRSTA are shown in **Table 19**, and a schematic diagram for the cross-sectional profile of WAM reach 138 (LRSTA-N) is shown in **Figure 36**.

Table 20. Reach cross-sectional profiles for the LRSTA and BRSTA.

LRSTA-N WAM Reach 138		LRSTA-S WAM Reach 291		BRSTA WAM Reach 239	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
13.1	3,576	13.1	3,520	13.1	6,037
2	3,556	2	3,500	2	6,000
1.6	33	1.6	33	1.6	33
0	0	0	0	0	0

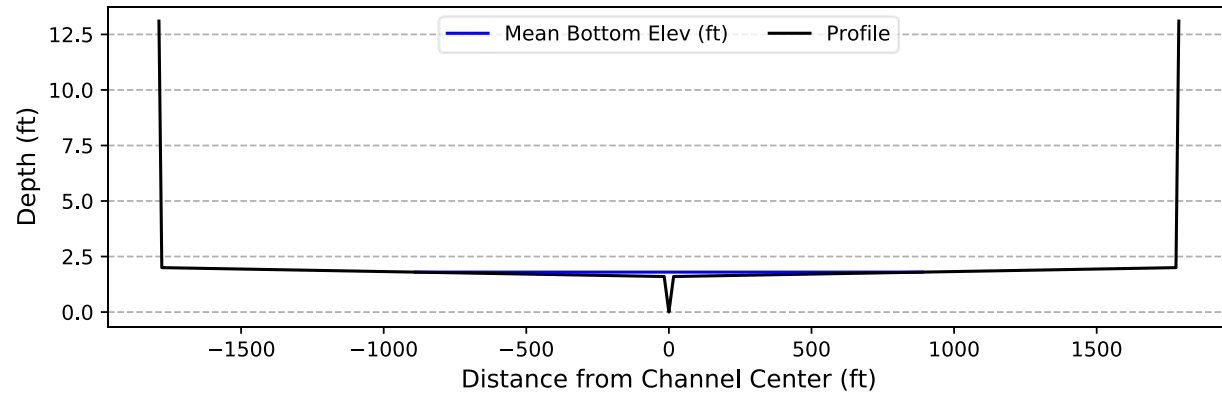


Figure 36. The cross-sectional profile for WAM reach 138 representing LRSTA-N used for the simulation with the mean bottom elevation indicated.

4.3.1.2.3 Attenuation

A general description of the nutrient attenuation process in WAM is given in **Section 4.1.2**. For the LRSTA and BRSTA, attenuation coefficients were selected to target the estimated average removal rates of 19 – 25 metric tons per year (mt/y) of TP. The attenuation coefficients used for both LRSTA impoundments and the BRSTA are $a = 0.4$ and $C_{BG} = 0.015$ mg/L for dissolved P and 0.005 mg/L for particulate P (i.e., $C_{BG} = 0.02$ mg/L for TP).

4.3.1.3 Structures

4.3.1.3.1 LRSTA/BRSTA Inflow Pumps

Water is pumped from the L-64 canal (WAM reaches 139, 140, 148) into LRSTA-N via the S-650 pump station. The S-650 pump station is comprised of three pumps, two of which have capacities of 75 CFS and one that has a capacity of 100 CFS, for a total capacity of 250 CFS. The WAM setup also includes a 150 CFS pump station to pump water from the L-64 canal into the BRSTA. The on/off controls for the pumps are determined by the stage in the C-59 canal (WAM reach 150) (**Figure 37**). The control elevations are shown in **Table 21**. Note that the pump on stages have been slightly lowered from those presented in Table 7 of CDM Smith (2018) (J. Zhang, personal communication, January 23rd, 2019). The control elevations are staged by a few inches, with each pump triggering on at a slightly higher elevation. This is done to provide smoother flow time profiles and improve run times within WAM. Besides the stage controls in C-59, if the water depth in either of the impoundments in the LRSTA exceeds 28 inches, the pumps are turned off. The stage controls in C-59 for pumping into the BRSTA are identical to those for

1131

1132

1133

1134

LRSTA; in a similar fashion, the BRSTA pump is shut off if the BRSTA depth exceeds 28 inches (but is independent of water depths in the LRSTA).

Table 21. LRSTA and BRSTA pump station parameters used in WAM (CDM Smith 2018). On/off triggers are stages in C-59.

	On (ft)	Off (ft)	Capacity (CFS)
LRSTA (S-650)			
Pump 1	18.0	17.6	75
Pump 2	18.4	17.9	75
Pump 3	18.7	18.2	100
BRSTA			
Pump 1	18.0	17.6	150

1135

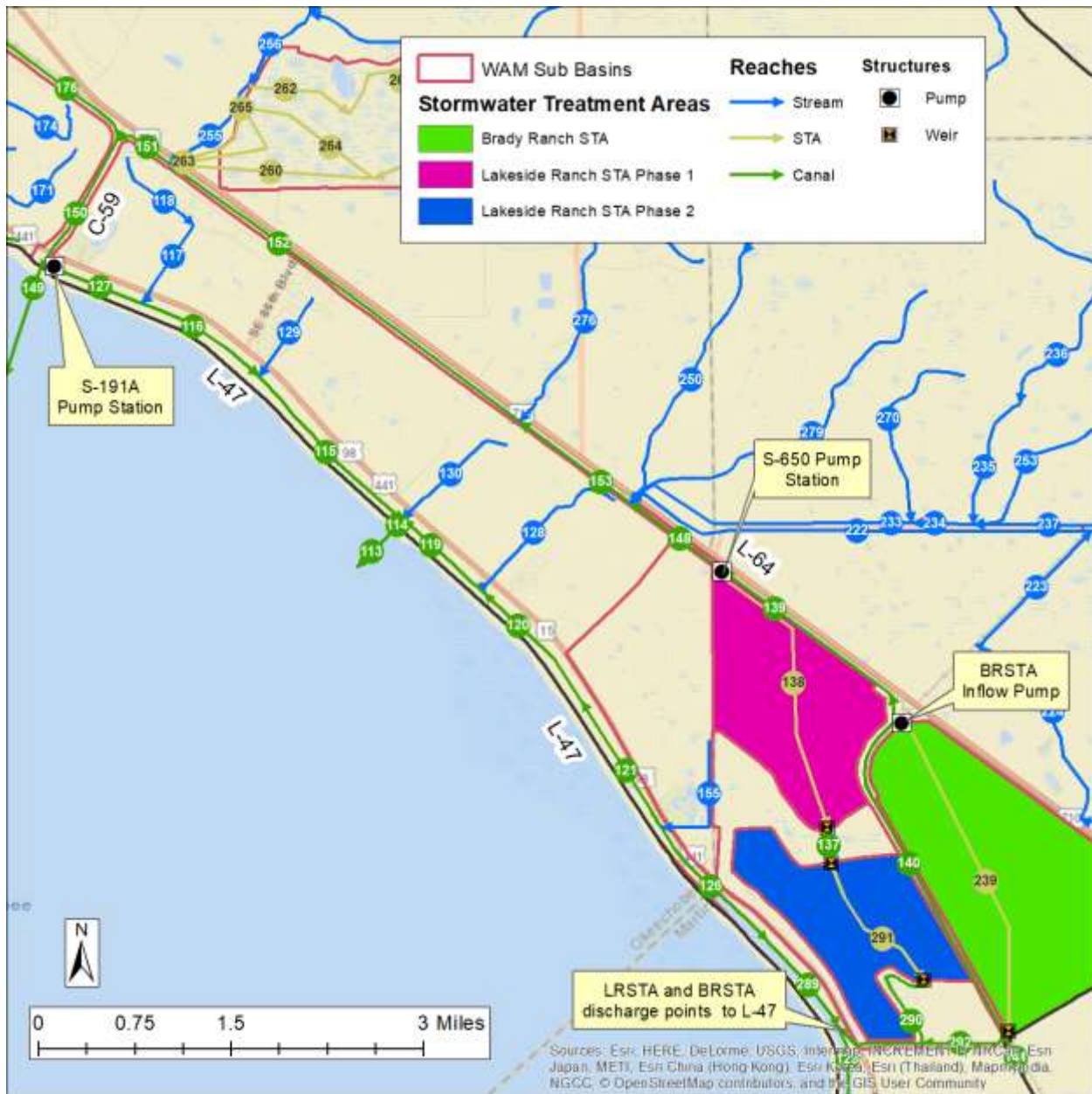


Figure 37. The WAM setup for LRSTA/BRSTA indicating the placement of the S-191A pump station and C-59.

4.3.1.3.2 Weirs

To control depths within the LRSTA and BRSTA, weirs were emplaced at the downstream end of the LRSTA-N (at the bottom of WAM reach 138), at the end of the LRSTA-N outlet canal connecting it to LRSTA-S (WAM reach 137), and at the bottom of LRSTA-S (WAM reach 291). A weir was also placed at the discharge point of BRSTA (WAM reach 239). Weir crest elevations were not specified in CDM Smith (2018) so the crest elevations were adjusted to maintain the target depth of 18 inches in the impoundments. **Table 22** shows the crest elevation and width determined for each weir.

Table 22. Weir parameters used in WAM for the LRSTA and BRSTA.

Weir	Width (ft)	Crest Elevation (ft)
LRSTA-N Outlet	72.2	27.4
N. Outlet Canal	24.0	23.0
LRSTA-S Outlet	72.2	21.1

4.3.1.3.3 Outflow Pumps

The S-191A pump station is located at the north-west end of the L-47 canal, immediately adjacent to the C-59 canal. This pump station is comprised of three pumps, each with a capacity of 150 CFS. The pumps will lift water from the L-47 Canal to the C-59 Canal which, in turn, discharges to Lake Okeechobee through the S-191 Gate Structure. **Table 23** shows the control elevations for the S-191A pumps (see Table 8 in CDM Smith, 2018).

Table 23. S-191A pump station parameters used in WAM. On/off triggers are L-47 stages.

	On (ft)	Off (ft)	Capacity (CFS)
S-191A Pump Station			
Pump 1	13.1	11.6	150
Pump 2	13.6	12.6	150

4.3.1.4 Simulation Results

4.3.1.4.1 LRSTA Results

With the attenuation coefficients described above, the simulated total TP removal rate from the full LRSTA system (LRSTA-N and LRSTA-S combined) was 8.6 mt/y. The simulated inflow TP loading was 9.85 mt/y (at the inflow to LRSTA-N, i.e., TP entering WAM reach 138) while the TP outflow load from LRSTA-S (leaving WAM reach 291) was 1.26 mt/y, giving a reduction percentage of 87%.

Figure 38 shows time series of the inflow TP concentrations to LRSTA-N (orange line) and the outflow concentrations from LRSTA-S (green line). The concentration spike in mid-2001 corresponds to the re-initiation of pumping after the dry period from late 2000 through early- to mid-2001. **Figure 39** shows box-and-whisker plots comparing the distribution of inflow TP concentrations to LRSTA-N with those at the outflow from LRSTA-S. Each box shows the interquartile range of TP concentrations (top and bottom of each box), the median TP concentration (line within the box), the 95th and 5th percentiles of TP concentrations (upper and lower whisker, respectively), and outliers above and below the 95th and 5th percentiles (open black circles). The median TP concentration was lowered from 0.091 to 0.021 mg/L, while over 95% of all outflow TP concentrations were below all inflow concentrations (indicated by the red dashed line).

Flows (upper pane) and water depths (lower pane) of water within LRSTA-N and LRSTA-S are shown in **Figures 40** and **41**, respectively. Depths within LRSTA are shown in inches above the mean bottom, where the mean bottom is defined as halfway between the top of the narrow inner channel and the full width. In both LRSTA impoundments the mean bottom is 1.8 feet (21.6 in) above the bottom of the inner channel (see **Section 4.4.2.2.1**). In both figures, the bottom of the inner channel is shown by the solid black line, the mean bottom elevation is shown by the red line, the optimal depth is (18 in above mean bottom) is shown by the green line, and the weir crest elevation for the controlling outflow structure is shown by the magenta line. These values are all relative to the mean bottom.

The depths fall below the weir crest elevations during late 2000 through mid-2001, and again during the first several months of 2007, 2008, 2010, and 2011. These periods correspond to times no pumping was occurring due to low stages in C-59. Only during 2001 and briefly in 2007 does the water level in LRSTA-S drop below the mean bottom elevation, indicating that the LRSTA-S experiences some drying out during these times. At no time does the water level in LRSTA-N drop below the mean bottom elevation.

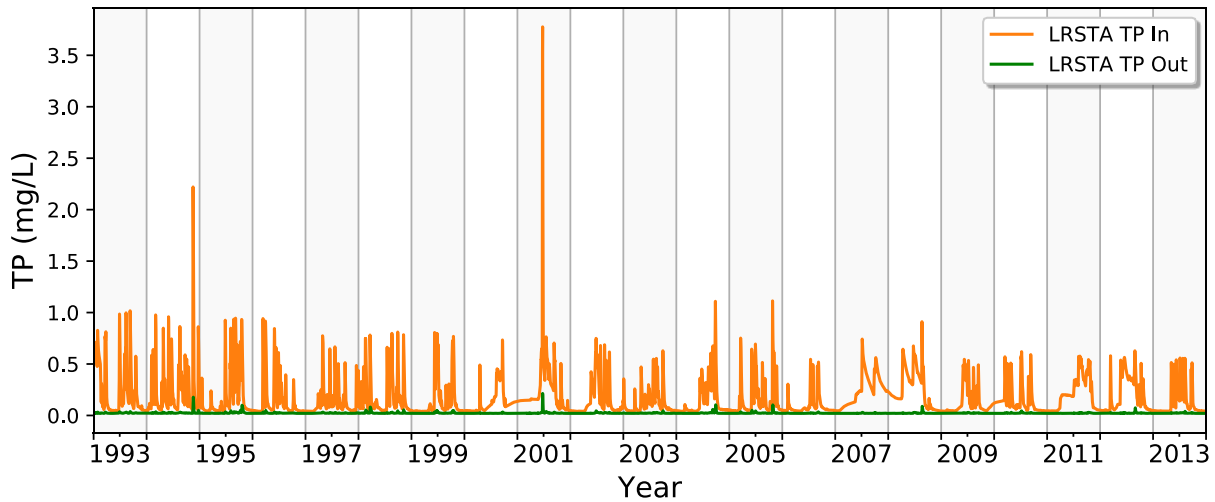


Figure 38. Comparison of simulated TP concentrations between inflow to LRSTA-N (orange line) and outflow from LRSTA-S (green line).

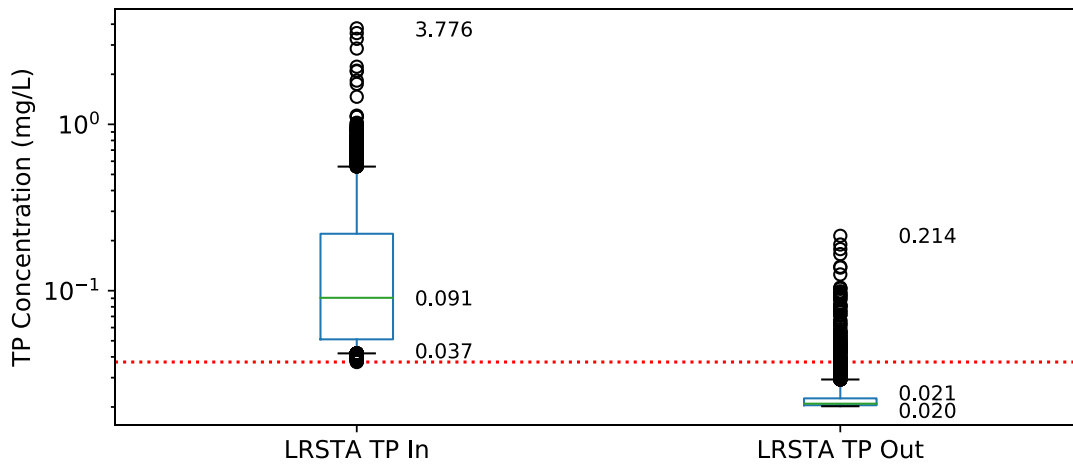


Figure 39. Box-and-whisker plots comparing the distribution of simulated LRSTA inflow TP concentrations (left box) and LRSTA outflow concentrations (right box). Note that the y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.037 mg/L).

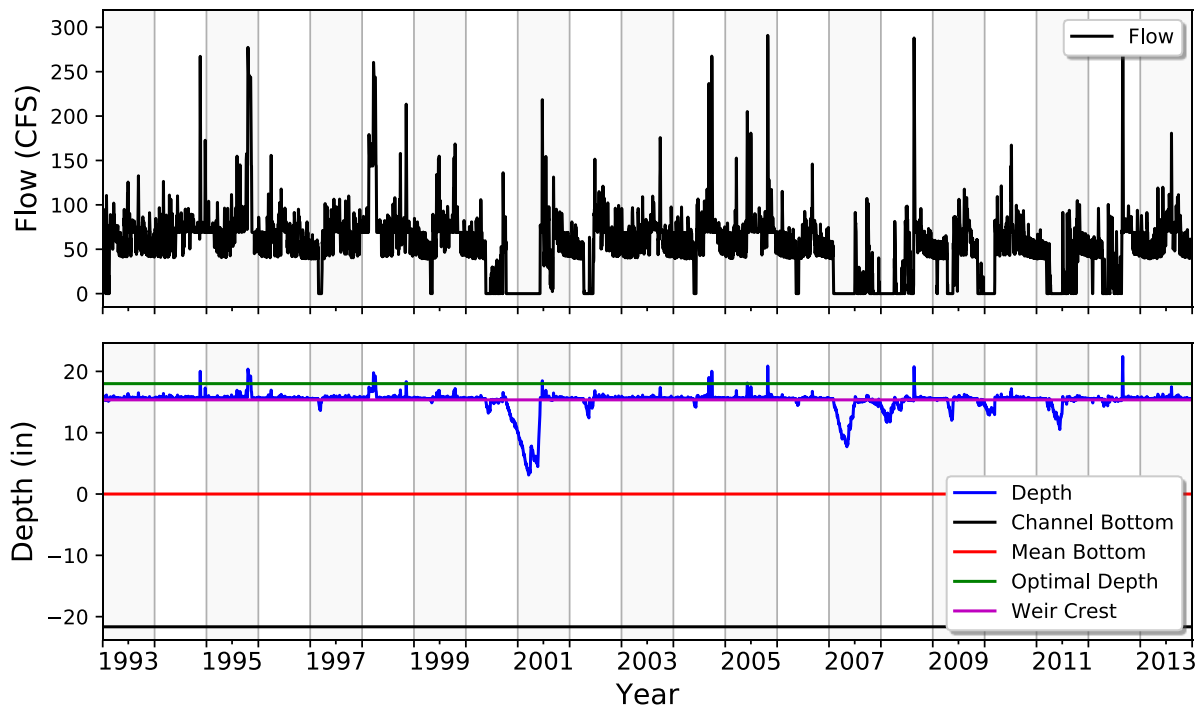


Figure 40. Simulated flows (upper pane) and water depths (lower pane) in LRSTA-N over the simulation period 1993-2013.

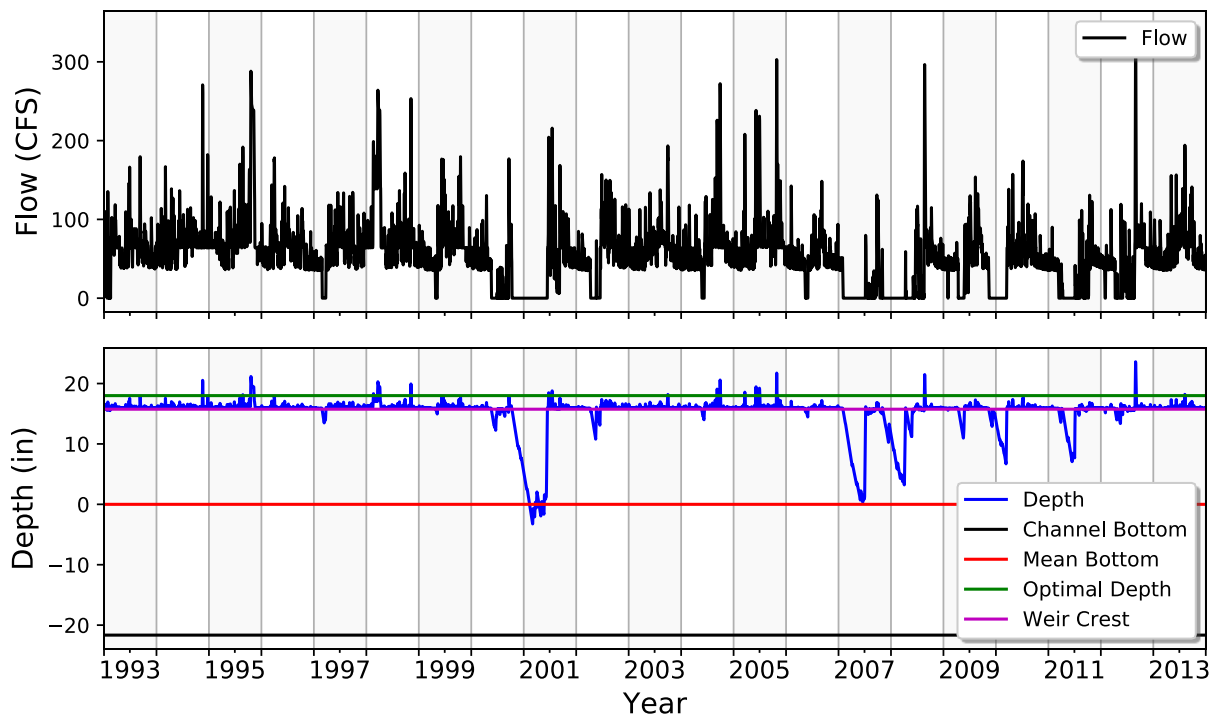


Figure 41. Simulated lows (upper pane) and water depths (lower pane) in LRSTA-S over the simulation period 1993-2013.

4.3.1.4.2 BRSTA Results

The same attenuation coefficients were used for the BRSTA as those used the LRSTA. The simulated total TP removal rate from the BRSTA was 13.5 mt/y. The simulated inflow TP loading to the BRSTA was 17.3 mt/y (i.e., TP entering WAM reach 239) while the TP outflow load from BRSTA (leaving WAM reach 239) was 3.8 mt/y, for a 78% reduction percentage.

Figure 42 shows time series of the inflow TP concentrations to (orange line) and outflow concentrations (green line) from BRSTA. The inflow concentrations are nearly similar to those to LRSTA since both STAs draw water from similar locations in L-64. **Figure 43** shows box-and-whisker plots comparing the distribution of inflow TP concentrations to outflow concentrations from BRSTA. Each box shows the interquartile range of TP concentrations (top and bottom of each box), the median TP concentration (line within the box), the 95th and 5th percentiles of TP concentrations (upper and lower whisker, respectively), and outliers above and below the 95th and 5th percentiles (open black circles). The median TP concentration was lowered from 0.092 to 0.025 mg/L, while roughly three quarters of all outflow TP concentrations were below all inflow concentrations (indicated by the red dashed line).

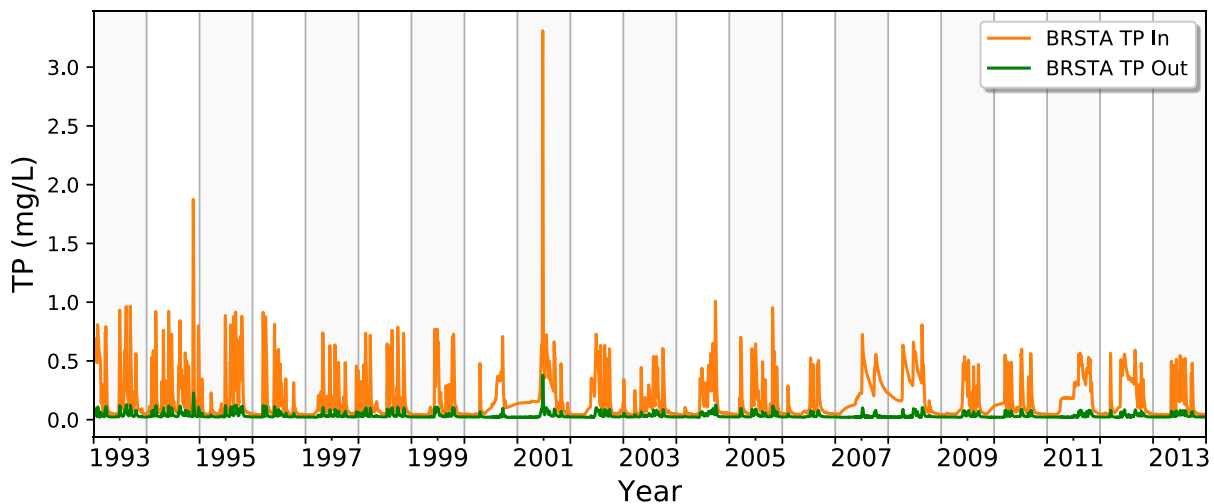


Figure 42. Comparison of simulated TP concentrations between inflow (orange line) and outflow (green line) from BRSTA.

Flows (upper pane) and water depths (lower pane) of water within BRSTA are shown in **Figure 44**. As with results from LRSTA, depths within BRSTA are shown in inches above the mean bottom, where again the mean bottom is defined as halfway between the top of the narrow inner channel and the full width. In **Figure 44** the bottom of the inner channel is shown by the solid black line, the mean bottom elevation is shown by the red line, the optimal depth is (18 inches above mean bottom) is shown by the green line, and the weir crest elevation for the controlling outflow structure is shown by the magenta line. Again, these values are all relative to the mean bottom.

The depths fall below the weir crest elevations during late 2000 through mid-2001, and again during brief periods over the 2007 – 2011 time frame. These periods correspond to times no pumping was occurring due to low stages in C-59. Unlike LRSTA-S, at no time does the water level in BRSTA drop below the mean bottom elevation, and only during 2001 does the water level drop below 10 inches.

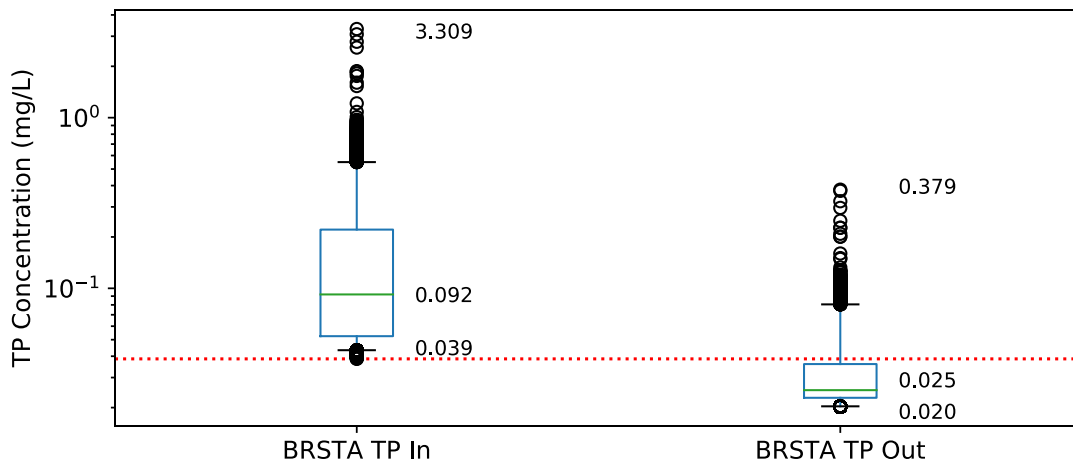


Figure 43. Box-and-whisker plots comparing the distribution of simulated BRSTA inflow TP concentrations (left box) and BRSTA outflow concentrations (right box). Note that the y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.039 mg/L).

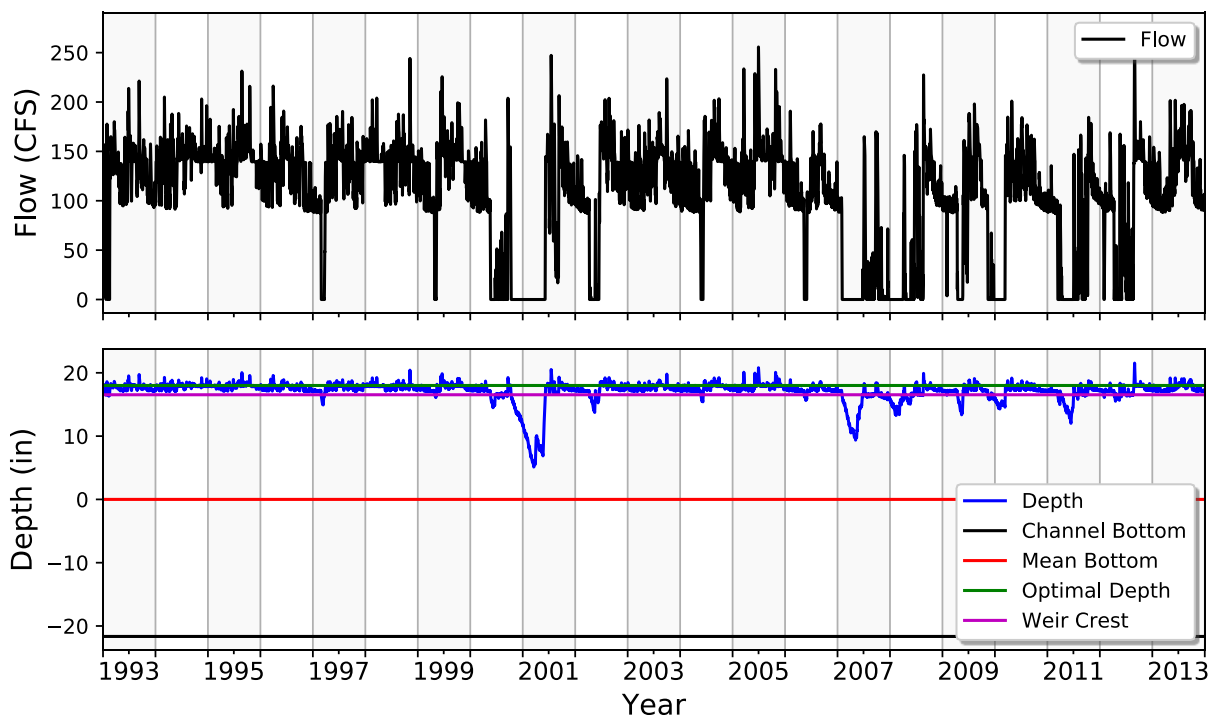


Figure 44. Simulated flows (upper pane) and water depths (lower pane) in BRSTA over the simulation period 1993-2013.

4.3.2 Summary of LRSTA/BRSTA Results

The combined simulated TP removed from LRSTA and BRSTA was 22.1 mt/y (**Table 24**). For the entire Taylor Creek/Nubbin Slough sub-watershed, the simulated annual average total discharge volumes and TP

loads are shown in **Table 25**. The simulated volume discharging from the sub-watershed decreased by about 2.9%, while the simulated TP load decreased by about 16.5%. The total change in the sub-watershed discharged TP is about 1.6 mt/y less than the 22.1 mt/y removed from the STAs, this is due to slightly less attenuation within the overall reach network as well as some additional backflow from Lake Okeechobee and seepage through the Herbert Hoover dike due to the system-wide changes. Note that the SFWMD has indicated to SWET that the gate controls for the S-191 structure have been altered so that the gate opens when the stage in C-59 is at 18.1 feet, which is just above the S-650 pump-on stage of 18.0 ft (J. Zhang, personal communication, January 23rd, 2019). In the base WAM scenario, the S-191 gate opens at when the stage in C-59 is above 19.0 feet. These changes were made to ensure that the water balance between the S-191 and S-135 basins.

Table 24. Simulated combined LRSTA and BRSTA TP load reductions.

Location	TP Load (mt/y)		
	Inflow	Outflow	Change
LRSTA (N&S)	9.9	1.3	-8.6
BRSTA	17.3	3.8	-13.5
Total	27.2	5.1	-22.1

Table 25. Simulated annual average volumes and TP loads discharging from the Taylor Creek/Nubbin Slough sub-watershed.

	Volume (ac-ft/y)			TP Load (mt/y)		
	Base	LRSTA & BRSTA	Change	Base	LRSTA & BRSTA	Change
Sub-watershed Total	153,537	149,082	-4,455	123.8	103.3	-20.5

4.4 Fisheating Creek Lake Okeechobee Watershed Construction Projects

The WAM setup for FEC (**Figure 45**) was developed as part of the WAM recalibration to estimate water flows and nutrient loads across all the nine sub-watersheds in the LOW (SWET 2017a, 2017b). The FEC sub-watershed was simulated as part of the northern LOW sub-watersheds (SWET 2017a). Two projects in the LOWCP were simulated within the FEC sub-watershed: Nicodemus Slough and the FAVT system. The base WAM scenario for FEC was duplicated and modified to incorporate both of these construction projects. The FAVT project is discussed first in **Section 4.4.1**, while the Nicodemus Slough project is discussed in **Section 4.4.2**.

Shapefiles were obtained from the SFWMD that delineate the FAVT treatment area and the Nicodemus Slough impoundment area. In addition, design and operation documents for the FAVT (Water & Soil Solutions 2018) and Nicodemus Slough (SFWMD 2014a) were obtained from the SFWMD.

4.4.1 Floating Aquatic Vegetative Tilling (FAVT) Treatment Area

This section describes the addition of the FAVT to the existing WAM model for FEC. The goal of the FEC FAVT system is to remove P and N from the upper reaches of Fisheating Creek. The FEC FAVT system was deployed on approximately 265 acres of privately owned land east of and adjacent to Fisheating Creek in Highlands County, Florida (Water & Soil Solutions 2018). The facility receives water from Fisheating Creek that is destined for Lake Okeechobee. A schematic depicting the general location and flow path of the FAVT system with respect to Fisheating Creek is depicted in **Figure 46** (Figure 1 in Water & Soil Solutions 2018).

The draft South Florida Environmental Report (SFER) for 2019 (SFWMD 2018) indicates that during Water Year (WY) 2018, 10,555 million gallons of water were treated, and 1.5 tons of TP were removed from Fisheating Creek. Although WY 2018 was not covered by the calibration period (SWET, 2017a) and was not simulated for this project, these values were used as targets to develop model setup and attenuation parameters. As with all the WAM simulations for the current project, the simulation period was over the calendar years 1993-2013.

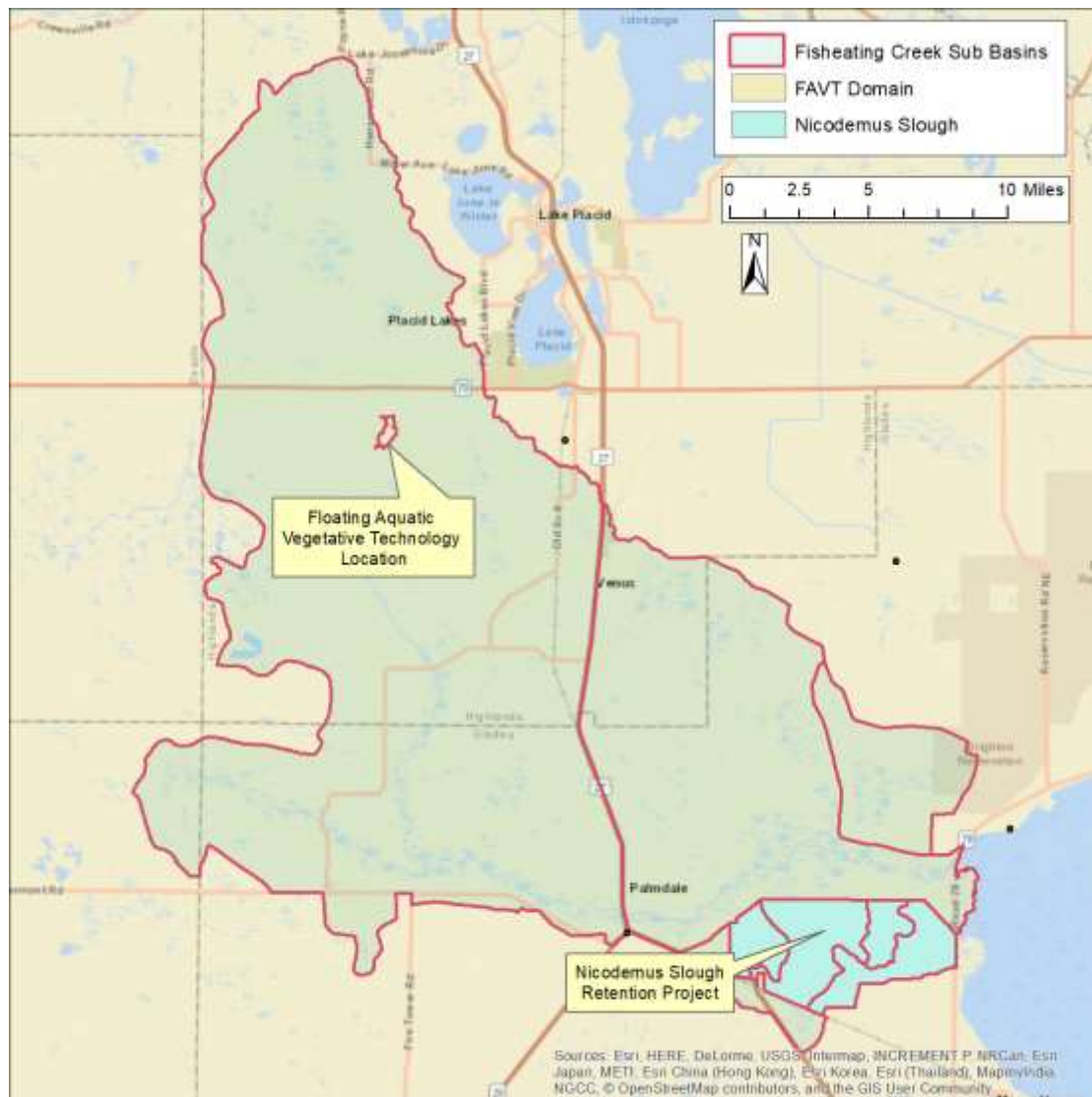


Figure 45. The WAM setup for the FEC sub-watershed, with the location of the FAVT and Nicodemus Slough projects shown.

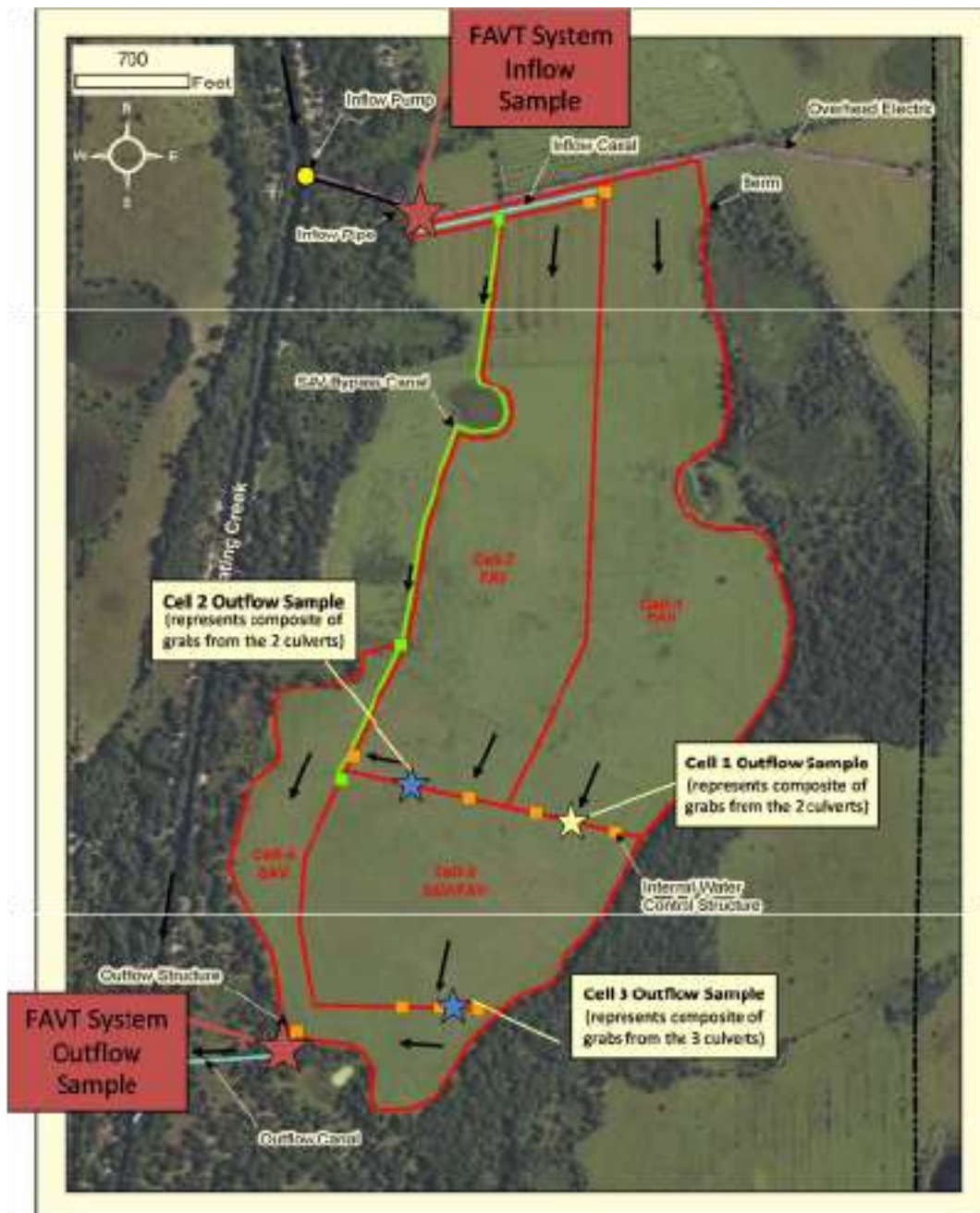


Figure 46. Detailed FAVT system configuration (from Water & Soil Solutions 2018)

4.4.1.1 WAM Setup for the FAVT

The WAM setup for the FAVT system is shown in **Figure 47**. Reach numbers are indicated by the circled numbers on each reach. The section of Fisheating Creek adjacent to the FAVT system is WAM reach 26; water for treatment is pumped out of Fisheating Creek at the northern end of reach 26 by a series of four pumps that are triggered on/off depending on the stage in Fisheating Creek at the inflow point. Water is pumped into the distribution canal (WAM reach 226) at the north end of the FAVT.

Water enters cell 1 and cell 2 via the distribution canal and is controlled by two weirs at the northern end of the cells. WAM reaches 228 and 230 represent cell 1 and cell 2, respectively, and are assigned lengths and cross-sectional profiles that match the treatment area of each cell. Water also flows from reach 226

southward along the SAV bypass canal (Water & Soil Solutions 2018) on the western perimeter of the FAVT, which in turn discharges into cell 4 (WAM reach 232). Discharges from cells 1 and 2 into cell 3 (WAM reach 233) are controlled by weirs at the southern end of 228 and 230. Discharge from cell 3 into cell 4 is controlled by a weir at the southern end of 233. Discharge from the FAVT occurs from WAM reaches 229 and 232 into the FAVT outflow canal (WAM reach 227) at the southern end of the treatment area. The outflow canal in turn discharges into Fisheating Creek at the top of WAM reach 25.

The length and cross-sectional profile of each WAM reach representing a cell within the FAVT were calculated so that the total area of the reach matched the area in the FAVT shapefile provided by the SFWMD. The WAM parameters for reaches representing FAVT cells are shown in **Table 26**. Bottom elevations were estimated from the elevation DEM using the ArcMap tool “Zonal Statistics.” Note that cell 4 is represented by two reaches, 232 and 233, that total 36 acres in area.

Table 26. WAM reach parameters for cells within the FAVT.

Treatment Area	WAM Reach	Area (ac)	Length (ft)	Width (ft)	Bottom elevation (ft NGVD)	Mean Bottom (ft NGVD)
Cell 1	228	90	3,789	1,033	80.38	82.35
Cell 2	230	80	3,556	984	80.38	82.35
Cell 3	233	57	2,625	951	78.08	80.05
Cell 4	232	19	2,940	285	78.08	80.05
Cell 4	233	17	2,625	285	78.08	80.05

4.4.1.1.1 Profiles

The general setup of cross-sectional profiles in WAM is discussed in **Section 4.1.1**. **Table 27** shows the cross-sectional profiles assigned for the WAM reaches that make up the FAVT. Each profile has a narrow channel that is 33 feet wide when the water depth is 1.6 feet deep. The profile spreads out to a width of between 285 and 1,033 feet (depending on the cell) at a depth of 2.3 feet. Thus, the reach bottom changes from essentially dry (only the inner channel has water) to fully submerged as the water depth in the reach changes from 1.6 to 2.3 feet, i.e., about 8 inches. An optimal depth for each cell was not indicated in the documentation, but based on other treatment areas (e.g., the Lakeside Ranch STA in the Taylor Creek/Nubbin Slough sub-watershed), it was assumed to be about 18 inches in depth. For the STAs simulated here, this is 18 inches over the midpoint between 1.6 and 2.3 feet, i.e., 1.95 feet, or 23.4 inches. This implies that the optimum water surface in each STA is 41.4 inches (~1.05 meters) above the bottom of the reach, including the depth of the narrow inner channel. The reach bottom elevations and the mean bottom elevation (1.95 feet above the bottom) for each WAM reach representing a cell in the FAVT are shown in **Table 26**, and a schematic diagram for the cross-sectional profile of WAM reach 228 (Cell 1) is shown in **Figure 48**.

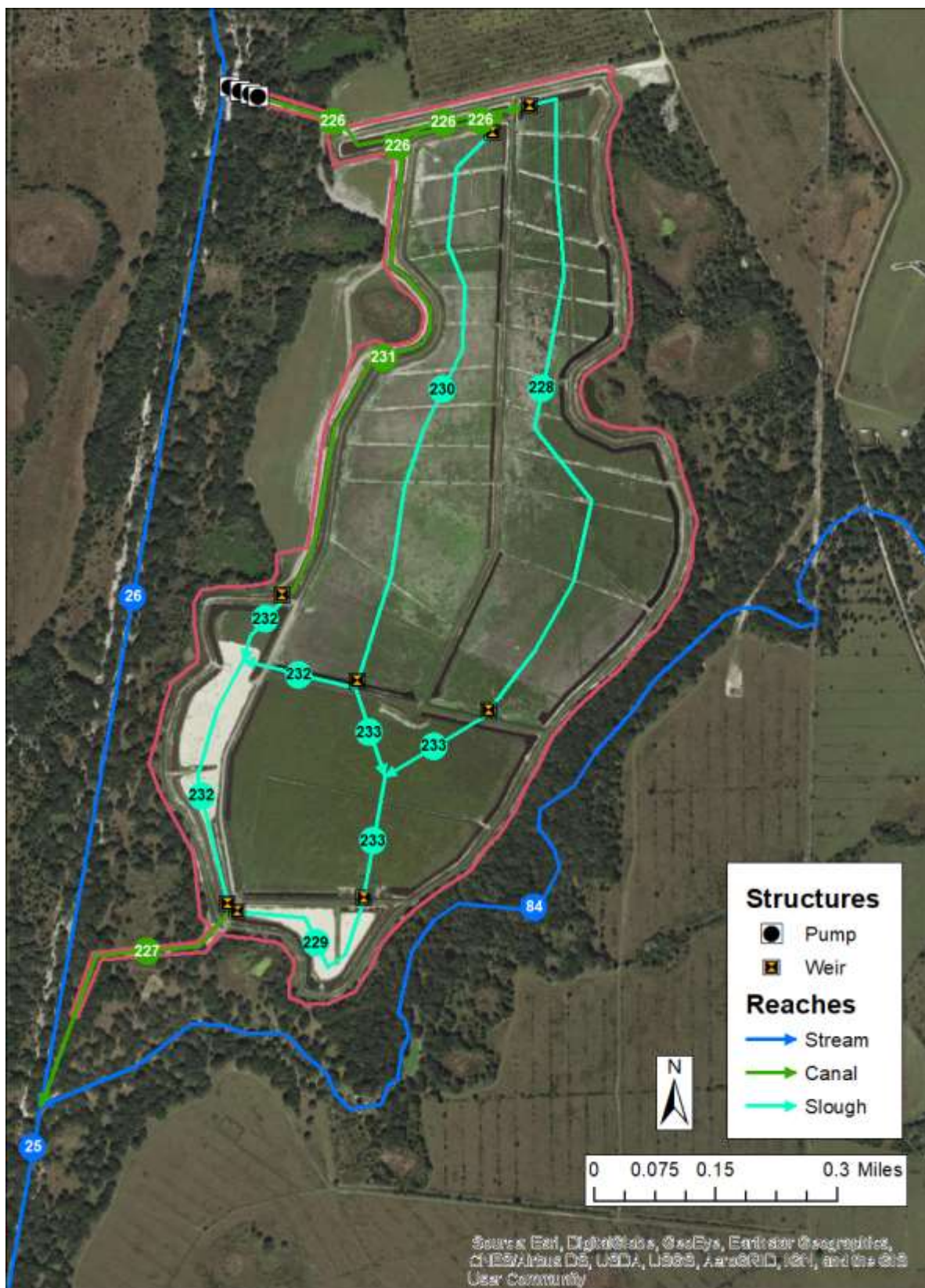


Figure 47. The WAM setup for the FAVT treatment area on Fisheating Creek. WAM reach numbers are indicated for each reach.

Table 27. Reach cross-sectional profiles for the FAVT

Cell 1 WAM Reach 228		Cell 2 WAM Reach 230		Cell 3 WAM Reach 233		Cell 4 WAM Reach 229		Cell 4 WAM Reach 232	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
10	1,066	10	1,017	10	984	10	302	10	302
2.3	1,033	2.3	984	2.3	951	2.3	285	2.3	285
1.6	33	1.6	33	1.6	33	1.6	33	1.6	33
0.0	7	0.0	7	0.0	7	0.0	7	0.0	7

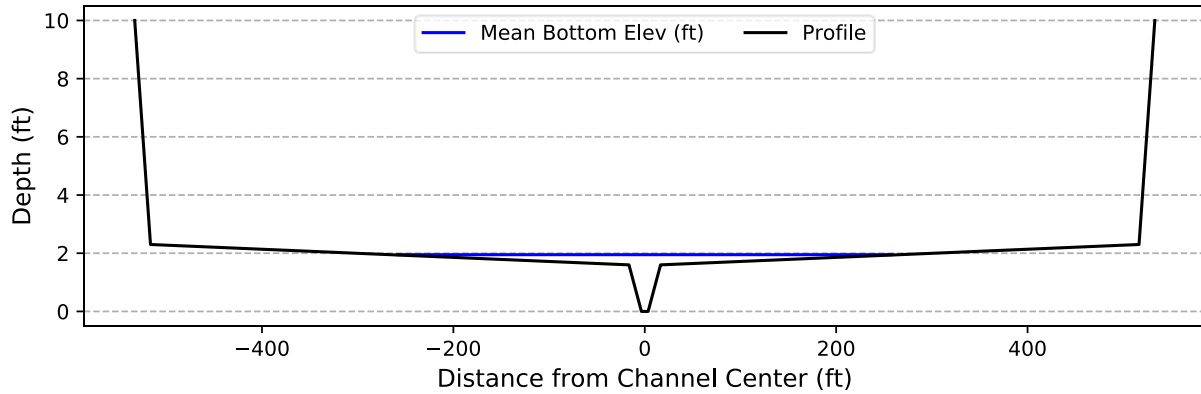


Figure 48. The cross-sectional profile for WAM reach 228 in the FAVT impoundment used for the simulation with the mean bottom elevation indicated.

4.4.1.1.2 Attenuation

A general description of the nutrient attenuation process in WAM is given in **Section 4.1.2**. For the FAVT, attenuation coefficients were selected to target the estimated average removal rates of 1.5 metric tons per year (mt/y) of TP. The attenuation coefficients used for all cells within the FAVT are $a = 0.4$ and $C_{BG} = 0.015$ mg/L for dissolved P and 0.005 mg/L for particulate P (i.e., $C_{BG} = 0.02$ mg/L for TP). Note that during the July 2016 through June 2017 time frame for which performance data were obtained from the SFWMD, there was no discharge from the FAVT back to Fisheating Creek, so attenuation coefficients could not be calibrated with existing data. The attenuation coefficient values used here are the same as those estimated from another LOWCP, the LRSTA in the Taylor Creek/Nubbin Slough sub-watershed. If and/or when TP concentrations in water discharging from the FAVT become available, it will be possible to refine the attenuation coefficients.

The land use type assigned to the impoundments is a special case land use type that passes rainfall directly through to the stream network, which then handles attenuation and ET. In other words, unlike other land uses in WAM, there is no source-cell level simulation of hydrologic- or nutrient-related processes, since all processes are handled in-stream.

4.4.1.1.3 Structures

4.4.1.1.3.1 FAVT Inflow Pumps

Water is pumped from Fisheating Creek into the FAVT through a pump station where the inflow canal (WAM reach 226) intersects Fisheating Creek. Neither the number of pumps nor the trigger stage(s) of Fisheating Creek for operating the pumps were specified in the documentation SWET received from the SFWMD, but it was specified that the pump capacity ranges from 5 to 100 CFS. Further communication from FDACS (via the SFWMD) indicated that pumping will only activate when stages in Fisheating

Creek rise above 73 ft NGVD and remain above 72.7 ft NGVD (FDACS, personal communication, January 10th 2019). Consequently, four pumps were simulated in WAM with varying capacities so that the total capacity (when Fisheating Creek is high) is 100 CFS and the minimum capacity is 5 CFS. The pump station parameters are shown in **Table 28**. Note that the control elevations are staged, with each pump triggering on at a slightly higher elevation.

Table 28. FAVT pump station parameters used in WAM. On/off triggers are Fisheating Creek stages.

	On (ft)	Off (ft)	Capacity (CFS)
Fisheating Creek			
Pump 1	73.0	72.7	5
Pump 2	73.8	73.5	20
Pump 3	74.8	74.5	30
Pump 4	75.8	75.5	45

4.4.1.1.3.2 Weirs

To control depths within the FAVT, weirs were placed at the downstream end of each cell in the FAVT, as well as at the inflow canals to control flow into the FAVT. No weir crest elevations were provided in the documentation received, but a target depth of 18 inches above the mean bottom elevation for each cell was assumed. Weir crest elevations and widths were then adjusted to fit this target depth. **Table 29** shows the estimated crest elevation and width for each weir in the FAVT.

Table 29. FAVT weir parameters used in WAM.

Weir	Width (ft)	Crest Elevation (ft)
Inflow canal to reach 228 (cell 1)	50	86.9
Inflow canal to reach 230 (cell 2)	50	86.9
Bottom of reach 228 (cell 1 to cell 3)	50	83.6
Bottom of reach 230 (cell 2 to cell 3)	50	83.6
Bypass canal (reach 231) to reach 232 (cell 4)	50	86.9
Bottom of reach 233 (cell 3 to cell 4)	50	81.3
Bottom of cell 4 to outflow canal	50	81.3

4.4.1.2 WAM Simulation

4.4.1.2.1 FAVT Simulation Results

With the attenuation coefficients described above, the simulated total TP removal rate from all cells of the full FAVT system was 1.81 mt/y. The simulated inflow TP loading was 2.98 mt/y (at the inflow to FAVT, i.e., TP entering WAM reach 226) while the TP outflow load from the FAVT (leaving WAM reach 227) was 1.17 mt/y, giving a reduction percentage of 61%.

Figure 49 shows time series of the inflow TP concentrations to the FAVT (orange line) and the outflow concentrations (green line). The concentration spike in mid-2001 corresponds to the re-initiation of pumping after the dry period from late 2000 through early- to mid-2001. **Figure 50** shows box-and-whisker plots comparing the distribution of inflow TP concentrations to the FAVT with those at the outflow. Each box shows the interquartile range of TP concentrations (top and bottom of each box), the median TP concentration (line within the box), the 95th and 5th percentiles of TP concentrations (upper and lower whisker, respectively), and outliers above and below the 95th and 5th percentiles (open black

circles). The median TP concentration was lowered from 0.234 to 0.02 mg/L, while nearly 95% of all outflow TP concentrations were below all inflow concentrations (indicated by the red dashed line).

Figures 51 and 52 show flows (upper pane) and water depths (lower pane) of water within cells 1 and 2 of the FAVT, respectively. Depths within FAVT are shown in inches above the mean bottom, where the mean bottom is defined as halfway between the top of the narrow inner channel and the full width. In all cells the mean bottom is 1.95 feet (23.4 in) above the bottom of the inner channel (see **Section 4.4.2.2.1**). In both figures, the bottom of the inner channel is shown by the solid black line, the mean bottom elevation is shown by the red line, the optimal depth is (18 in above mean bottom) is shown by the green line, and the weir crest elevation for the controlling outflow structure is shown by the magenta line. These values are all relative to the mean bottom.

The depths fall below the weir crest elevations numerous times, and below the mean bottom elevation over 10 times during the simulation period. Depths below the mean bottom elevation indicate that the FAVT is drying out, which is expected to occur periodically (Water & Soil Solutions 2018). The FAVT is designed so that during the dry-down periods the FAV and SAV that have accumulated during flooded periods will be tilled into the soil.

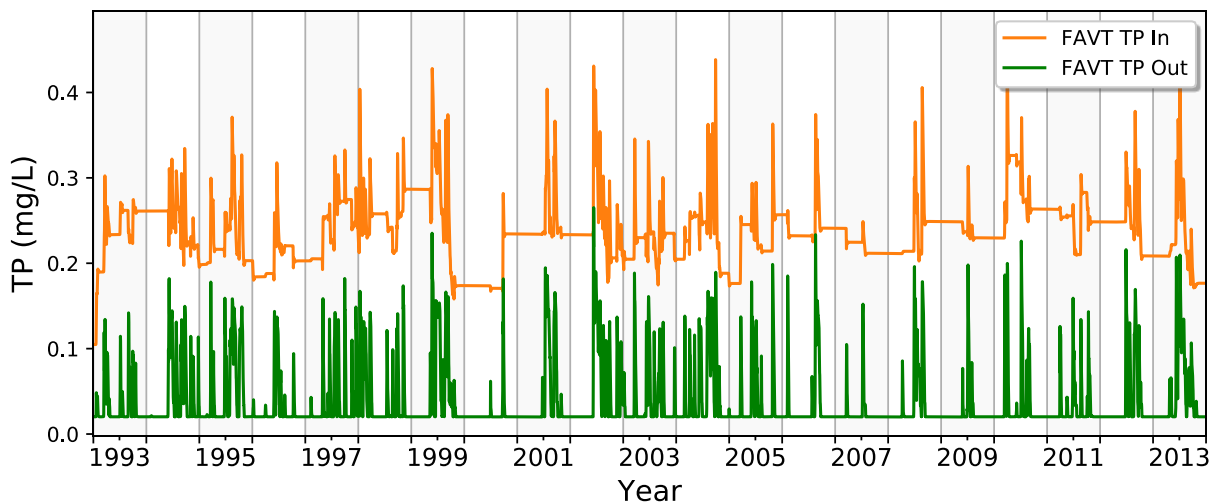


Figure 49. Comparison of simulated inflow (orange line) and outflow (green line) TP concentrations from the FAVT system.

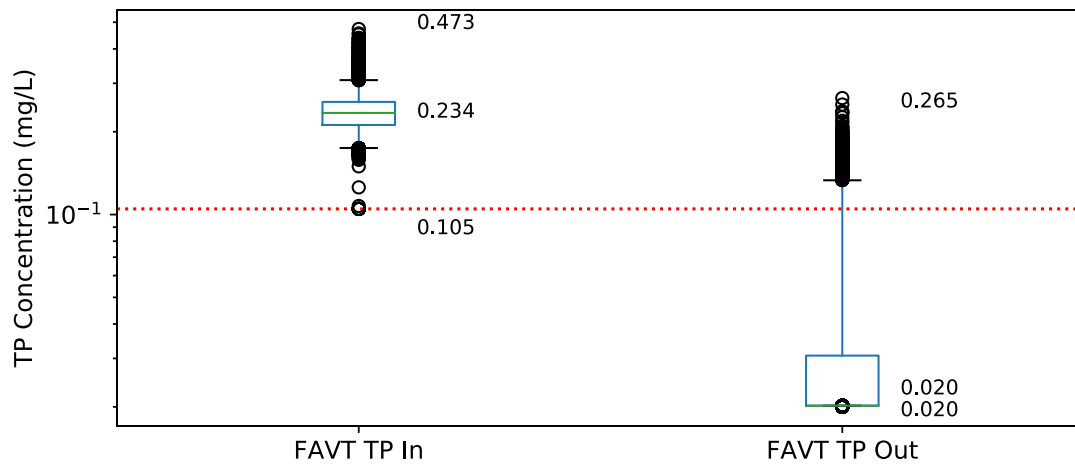


Figure 50. Box-and-whisker plots comparing the distribution of simulated inflow (left box) and outflow (right box) TP concentrations for the FAVT. The y-axis is scaled logarithmically. Numbers to the right of each box indicate (from top to bottom) the maximum, median, and minimum TP concentrations. The red dashed line is the minimum inflow TP concentration (0.105 mg/L).

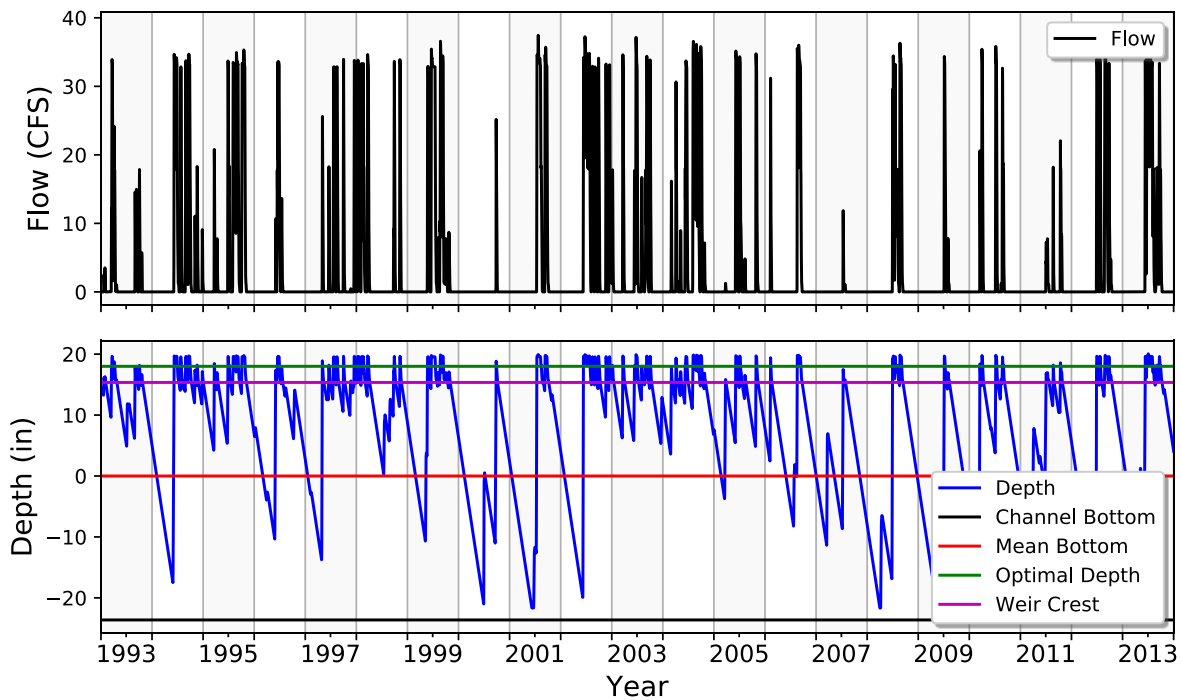


Figure 51. Simulated flows (upper pane) and water depths (lower pane) in the FAVT cell 1 over the simulation period 1993-2013.

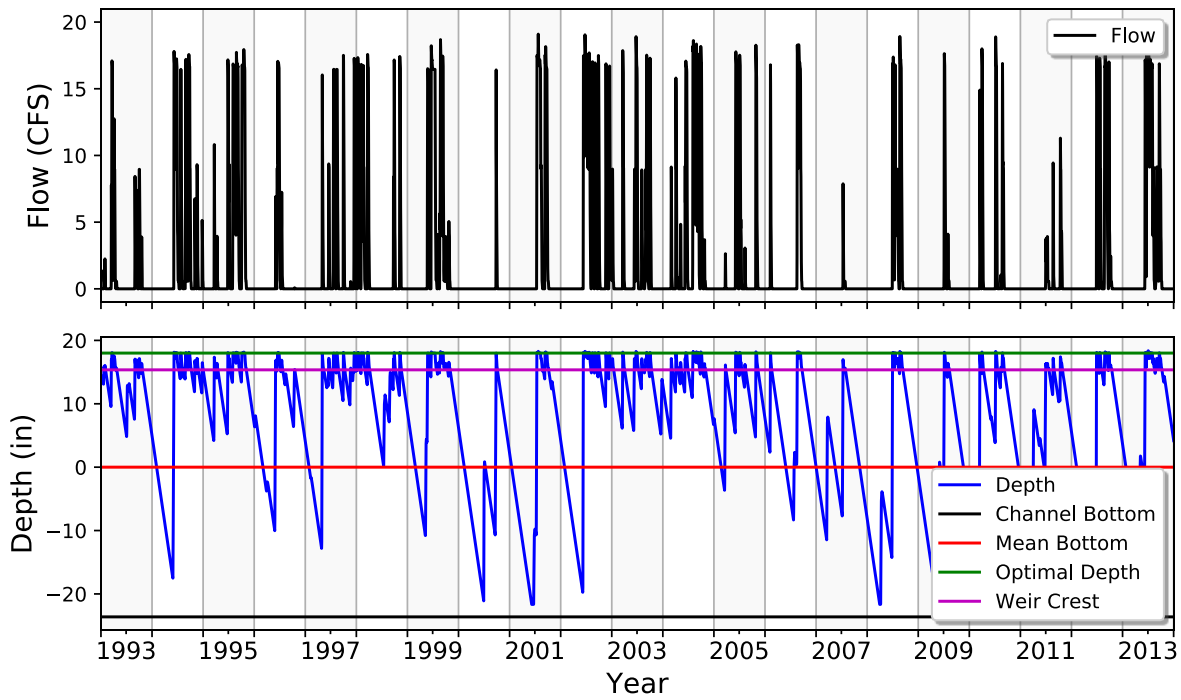


Figure 52. Simulated flows (upper pane) and water depths (lower pane) in the FAVT cell 2 over the simulation period 1993-2013

4.4.2 Nicodemus Slough Retention Area

4.4.2.1 Nicodemus Slough Project Objectives and Operations

The purpose of the Nicodemus Slough Water Management Project (**Figure 45**) is to provide water retention on the 15,906-acre site in the Northern Everglades and Estuaries Protection Area (SFWMD 2014a). The project area is located in Glades County just south of the portion of the Herbert Hoover dike along Fisheating Creek and west of County Road 78.

According to the document provided by the SFWMD (2014a), the primary objective of the water management project is to remove water from Lake Okeechobee through the C-5 culvert and pump it over Nicodemus Slough when Lake Okeechobee levels require discharges to the estuaries in excess of the environmentally desirable salinity envelope. The project will operate during periods when excess water in the Lake Okeechobee system is being discharged to the estuaries due to the lake's regulation schedule. The Nicodemus Slough Project will provide land to retain Lake Okeechobee water that would otherwise be sent to the Estuaries which are negatively impacted by these flows. The Nicodemus Slough Project will pump excess water into the project area which will serve to re-hydrate the slough system and to lessen the impact of excess water on Lake Okeechobee and the Caloosahatchee and St. Lucie Rivers (SFWMD 2014a).

The project will take water from Lake Okeechobee's rim canal via culvert C-5 and transport the water in an on-site ditch along the northern boundary of the project using two pumps in series (**Figure 53**). Water will enter the project through the C-5 culvert and be lifted and directed north and then west through Pump #1 to an elevation of 21.0 feet (all elevations are in feet NGVD). The water is lifted again as it is delivered to the Western basin at elevation 28.0 feet. The water is then dispersed in the Western basin to sheet flow eastward through the site. In addition to the Southeastern basin which includes the 2,000-acre Nicodemus Slough Flowage Easement, a series of four project retention basins were created to retain the water in shallow storage over the land surface. As the downstream basin reaches design operating stage, boards are placed in the series of culverts stopping flow to that basin from upstream (SFWMD 2014a).

Schematic Diagram Nicodemus Slough Project

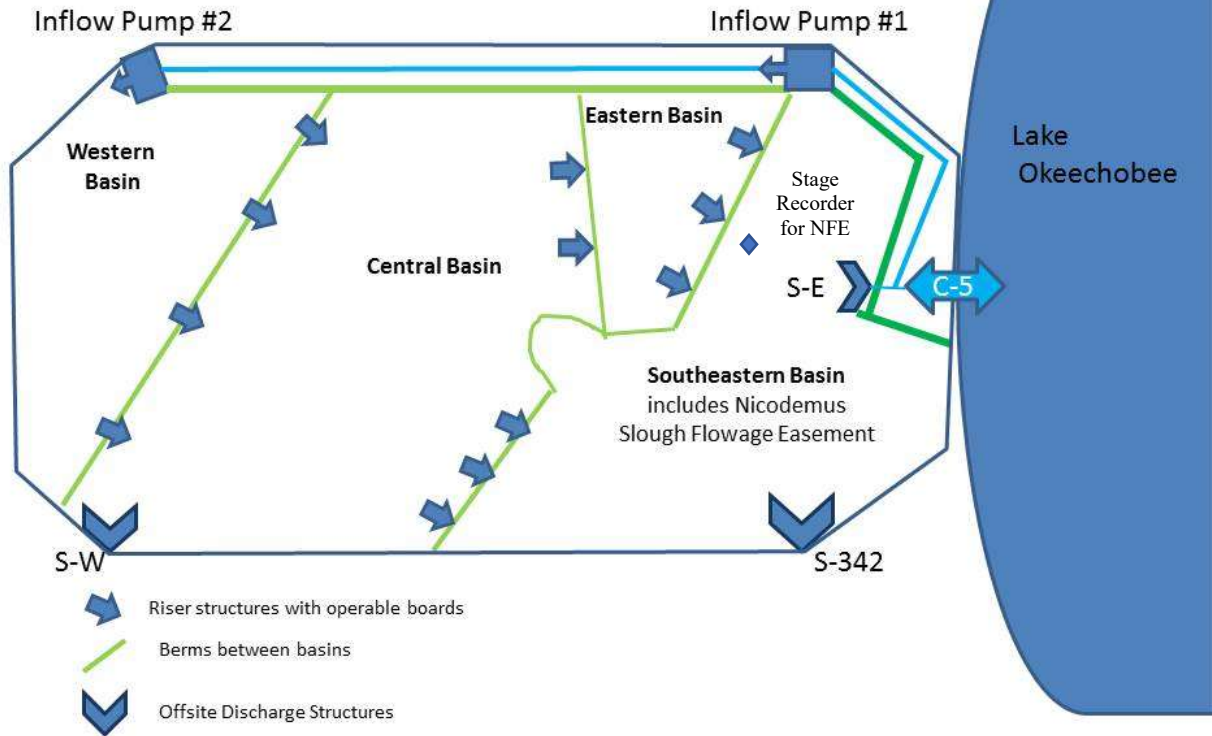


Figure 53. Schematic diagram of the Nicodemus Slough project (from SFWMD 2014a).

4.4.2.2 WAM Setup for Nicodemus Slough

The WAM setup representing the Nicodemus Slough Project is shown in **Figure 54**. The FEC setup was modified to incorporate the major features as described in SFWMD (2014a) discussed in the **Section 4.4.2.1** above. Each Nicodemus Slough basin (Western, Central, Eastern, and Southeastern) are represented in WAM by one (or more) reaches. The Western, Central, and Eastern basins are represented by a single reach, while the Southeastern basin is represented by five. The Western basin is represented by WAM reach 201, the Central basin by WAM reach 217, the Eastern basin by WAM reach 224, and the Southeastern basin by WAM reaches 197, 215, and 216, with reaches 198 and 235 parameterized as narrow reaches to represent discharge points. The Southeastern basin is divided into multiple reaches since multiple structures are placed at the inflow points (weirs) and at the outflow points (weirs and the gate representing the C-5 structure.) The inflow canals along the northern edge of the project are represented by WAM reaches 199, 200, and 218. Water is pumped from the Lake Okeechobee rim canal (WAM reach 196) into the short WAM reach 218 via pump station #1 (at the C-5 structure) and flows westward through WAM reaches 199 and 200 and is then pumped into Nicodemus Slough proper at inflow pump #2 at the western terminus of WAM reach 200. Water discharging from pump station #2 is dispersed across the Western basin (WAM reach 201), flowing eastward where it discharges into the Central basin via a broad weir. In a similar fashion, water flows across the broad Central basin, discharging into the Eastern and Southeastern basins. Flow across the Eastern basin in turn discharges into the Southeastern basin as well. When the stage in the Southeastern basin (i.e., in WAM reach 235) is above its maximum design operating stage the inflow pumps will cease inflow. Water may discharge back

into Lake Okeechobee from the Southeastern basin via C-5 if stages in the Southeastern basin are above the design stage and the pumps are not operating.

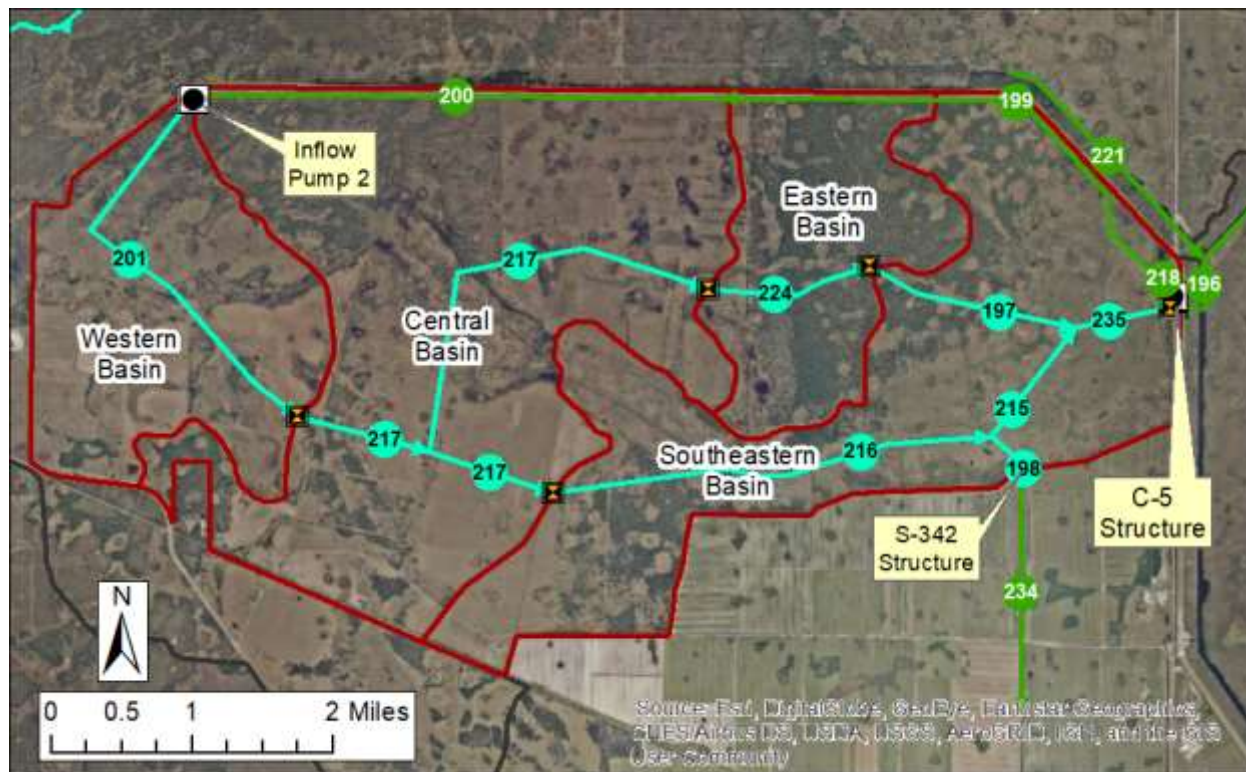


Figure 54. The WAM setup for the Nicodemus Slough Structure.

The length and cross-sectional profile of each WAM reach representing one of the Nicodemus Slough project basins were calculated so that the total area of the reach (length times width) matched the area of the basin calculated from the shapefile in ArcMap. For the Southeastern basin, the areas of the three WAM reaches sum to the area of the shapefile. The parameter values used for WAM reaches are shown in **Table 30** below.

4.4.2.2.1 Profiles

As with the setup of the WAM reaches used to simulate flow within impoundments such as the FAVT discussed above, the reach profiles within Nicodemus Slough are constructed with a broad bottom that slopes very slightly to the middle of the reach, with a narrow inner channel (see **Section 4.1.1**). **Table 31** shows the cross-sectional profiles assigned for the WAM reaches that make up the Nicodemus Slough basins. Each profile has a narrow channel that is 39 feet wide when the water depth is 3 feet deep. The profile spreads out to a width of between about 4,700 and 16,700 feet (depending on the reach and basin) at a depth of 3.6 feet. Thus, the reach bottom changes from essentially dry (only the inner channel has water) to fully submerged as the water depth in the reach changes from 3 to 3.6 feet, i.e., 7.2 inches. The design depth for each basin is approximately 2 feet deep (see Table 1 in SFWMD 2014a). Here, this is 24 inches above the midpoint between 3 and 3.6 feet, i.e., 3.3 feet, or 39.6 inches (The mean bottom elevation in **Table 30** is the elevation of this midpoint.) This implies that the target water surface in each basin is 63.6 inches above the bottom of the reach, including the depth of the narrow inner channel. The reach bottom elevations and the mean bottom elevation (3.3 feet above the bottom) for each WAM reach representing the Nicodemus Slough are shown in **Table 30**, and a schematic diagram for the cross-sectional profile of WAM reach 201 (Western Basin) is shown in **Figure 55**.

1487 Table 30. Reach parameters for WAM reaches in Nicodemus Slough

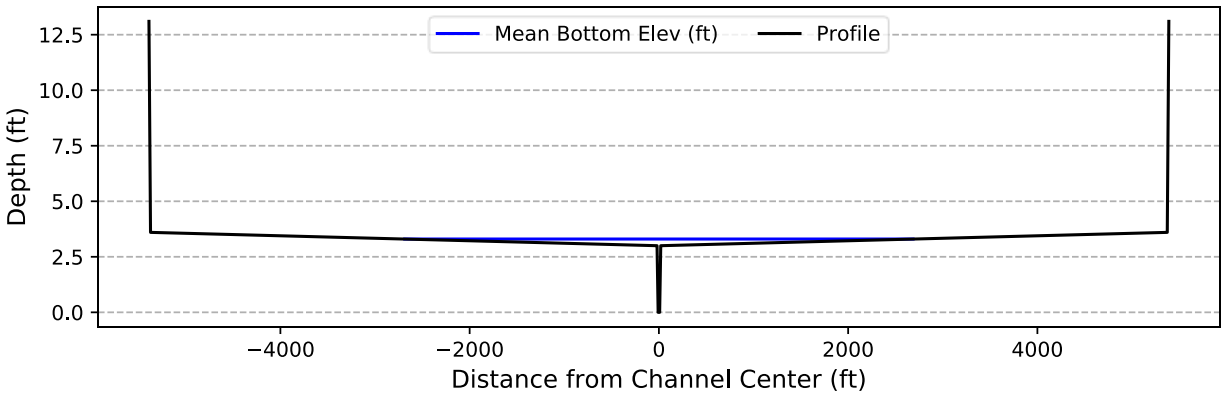
Treatment Area	WAM Reach	Area (ac)	Length (ft)	Width (ft)	Bottom Elevation (ft)	Mean Bottom (ft)
Western Basin	201	2,652	10,745	10,759	22.72	26
Central Basin	217	6,413	12,992	16,715	19.72	23
Eastern Basin	224	1,927	9,154	9170	15.72	19
Southeastern Basin		4,942	-	-	-	-
	197		7,766	7,200	10.72	14
	215		9,094	4,739	10.72	14
	216		12,795	6,545	10.72	14

1488

1489 Table 31. Reach cross-sectional profiles for Nicodemus Slough

Western Basin WAM Reach 201		Central Basin WAM Reach 217		Eastern Basin WAM Reach 224	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
13.1	10,778	13.1	16,732	13.1	9,186
3.6	10,745	3.6	16,699	3.6	9,154
3.0	39	3.0	39	3.0	39
0.0	16	0.0	16	0.0	16
Southeastern Basin WAM Reach 197		Southeastern Basin WAM Reach 215		Southeastern Basin WAM Reach 216	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
13.1	7,218	13.1	4,757	13.1	6,562
3.6	7,185	3.6	4,724	3.6	6,529
3.0	39	3.0	39	3.0	39
0.0	16	0.0	16	0.0	16

1490



1491 Figure 55. The cross-sectional profile for WAM reach 201 in Nicodemus Slough used for the simulation with
1492 the mean bottom elevation indicated.
1493

4.4.2.3 Structures

4.4.2.3.1 Nicodemus Slough Inflow Pumps

In the WAM setup, inflow pumps are placed to pump water from the Lake Okeechobee rim canal (WAM reach 196) into the easternmost distribution canal (WAM reach 218) inside the Nicodemus Slough project area. A second pump station is located at the westernmost end of reach 200 that pumps water from the distribution canal into the Western basin. Each simulated pump station has three pumps, with the trigger on/off elevations shown in Table 32. Note that WAM reach 196 is connected to reach 218 at the location of the C-5 structure. Each pump has a capacity of 200 CFS (SFWMD 2014a). The pump trigger elevations are staged to smooth out the simulated flow response. If the stage in the Southeastern basin (WAM reach 235 is used) exceeds 16 feet, inflow pumping operations are terminated.

Table 32. Nicodemus Slough pump station #1 and #2 parameters (SFWMD 2014a).

	On (ft)	Off (ft)	Trigger Reach
Pump Station #1			
Pump 1	13.8	13.5	196
Pump 2	14.0	13.7	196
Pump 3	14.2	13.9	196
Pump Station #2			
Pump 1	21.0	20.5	200
Pump 2	21.1	20.6	200
Pump 3	21.2	20.7	200

4.4.2.3.2 Weirs

Weirs are located at the internal berms that divide the Nicodemus Slough basins to hold stages within each basin and control flow from one basin to the next. The locations, crest elevations, and total widths are shown in **Table 33**. The crest elevations were obtained from SFWMD (2014a, Table 1). Note that weirs are placed at the top of WAM reach 224 to control inflow to the Eastern basin from the Central basin and at the bottom of the same reach to control outflow from the Eastern basin into the Southeastern basin. The weir at the top of WAM reach 216 controls flow into the Southeastern basin from the Central basin.

Table 33. Parameters for weirs within the Nicodemus Slough project (SFWMD 2014a).

Basin Location	WAM Reach	Reach Location	Crest Elevation (ft)	Width (ft)
Western	201	Bottom	27.2	100
Central	216, 224	Top	24.2	100
Eastern	224	Bottom	20.2	100

4.4.2.3.3 Outflow Gate at C-5

A gate to control outflows from Nicodemus Slough is located at the bottom of WAM reach 235, i.e., the location of structure C-5. This gate will open if stages within the Southeastern basin are above 16.0 feet (at which point the inflows at pump station #1 will cease operation) and close when the stage falls to 15.5 feet; however, the gate will not open (or will close if already open) if the stage in Lake Okeechobee is above 16.0 feet.

4.4.2.3.4 S-342 Outflow Structure

When stages within the Southeastern basin exceed 16.0 feet, water discharges via the S-342 structure in the southeastern corner of the Southeastern basin to the C-19 canal. If Lake Okeechobee stages are also above 16 feet, releases via S-342 are the only discharges from the project area, however, flow from the C-19 canal back into Nicodemus Slough is not allowed during irrigation operations south of Nicodemus Slough (SFWMD, personal communication, February 21st, 2019). To simplify the simulation, the discharge structure is simulated as a weir with a crest elevation of 16.0 feet and a width of 49 feet, located at the bottom of WAM reach 198. If overflow occurs, the structure will discharge to the C-19 canal (WAM reach 234), which is simulated as a single reach with a fixed-stage boundary condition set at 12.5 feet to ensure that flow from C-19 back into Nicodemus Slough does not occur.

4.4.2.4 WAM Simulation

4.4.2.4.1 Nicodemus Slough Simulation Results

Flows into Nicodemus Slough via pump station #2 (WAM reach 200) are shown in **Figure 56**. Note that this reflects the three pumps (200 CFS each) triggering at slightly different levels. Over the 21-year simulation period from 1993-2013, an average of 60,400 ac-ft/y were pumped into the project area. During the simulation period, an average of 18,026 ac-ft/y were released back into Lake Okeechobee via either C-5 (**Figure 58**) or the S-342 (**Figure 59**) structure to C-19. Because the pumps will cease operations when water levels in the Southeastern basin reach 16 feet, under the current simulation setup the only time water is released from the project is when rainfall causes stages to rise above the trigger elevation. **Figure 58** shows both outflows from the Southeastern basin to Lake Okeechobee via the C-5 structure (upper pane) and stages within the Southeastern basin (lower pane). In addition to the stages, the mean bottom elevation of the Southeastern basin is shown by the red line, which as discussed above is 3.3 feet above the bottom of the narrow inner channel. In addition, the open and close trigger elevations for the gate structure at C-5 are shown by the green and magenta lines, respectively. Note that since the S-342 structure is modeled as a weir, the discharge from this structure is much less than that from the C-5 gate, since once triggered open, it will remain open until the stage in the Southeastern basin falls below 15.5 feet.

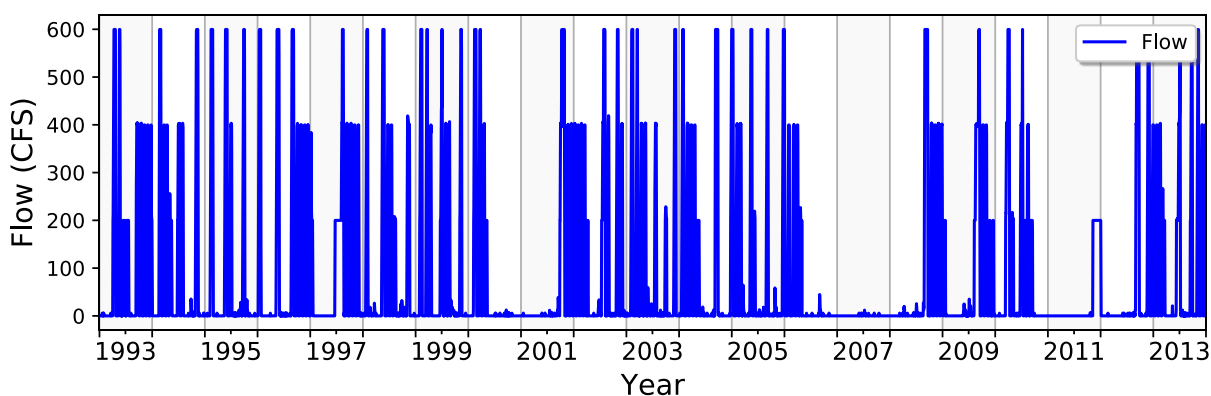


Figure 56. Simulated inflows to the Nicodemus Slough project area.

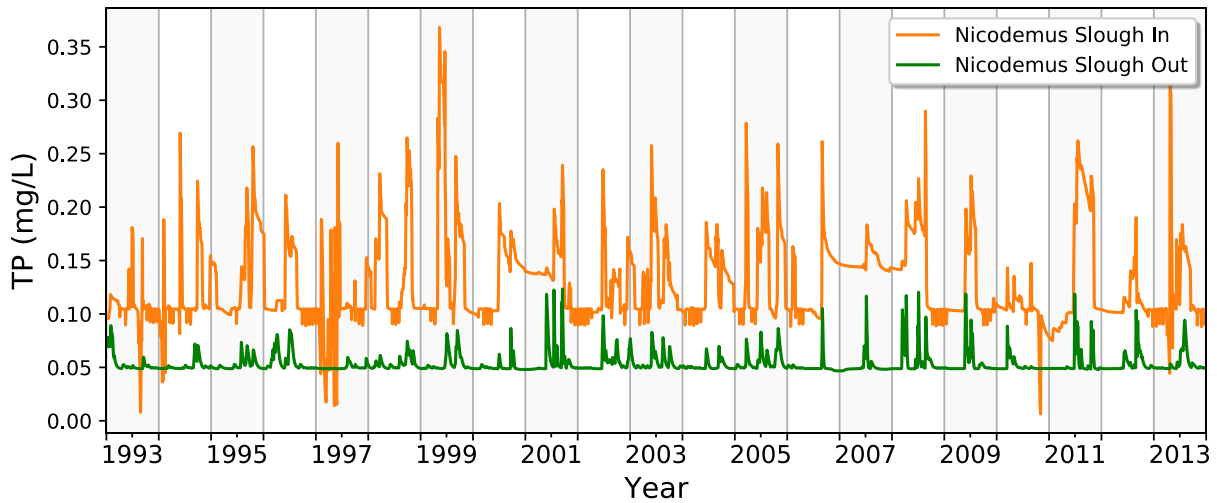


Figure 57. Comparison of simulated TP concentrations at the inflow to Nicodemus Slough to concentrations in the Southeastern basin.

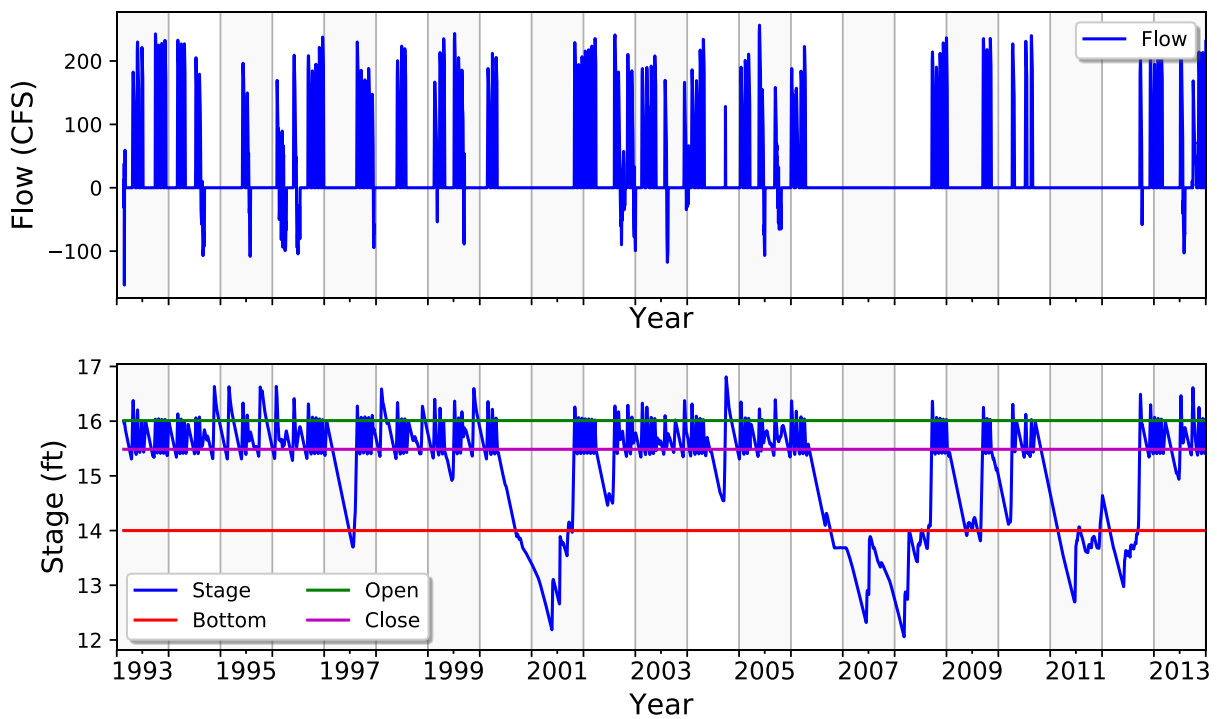


Figure 58. Simulated outflows from C-5 (upper pane) and stages within the Southeastern basin. Open and close trigger elevations for the gate structure at C-5 are shown in the lower pane (green and magenta lines), together with the mean bottom elevation of the Southeastern basin (red line).

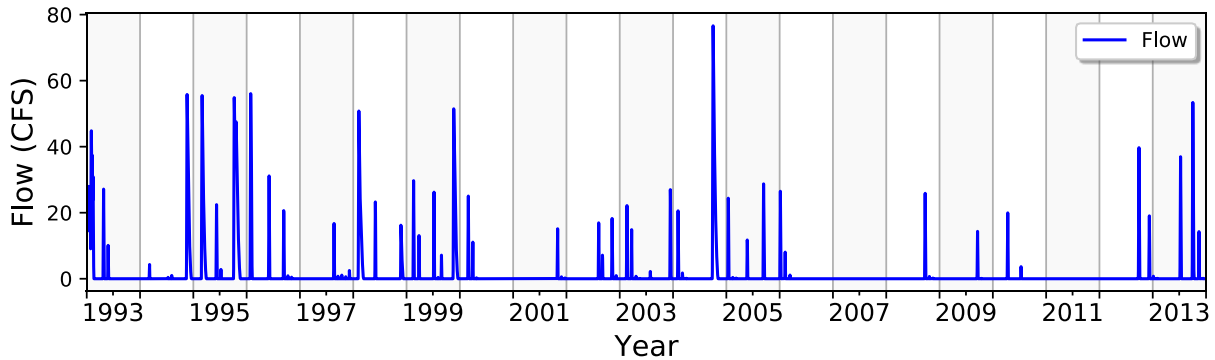


Figure 59. Simulated outflows from the Nicodemus Slough project area at Structure S-342.

Flows (upper pane) and water depths (lower pane) of water within the Western, Central, and Eastern Nicodemus Slough basins are shown in **Figures 60 – 62**. Depths are shown in inches above the mean bottom, where the mean bottom is defined as halfway between the top of the narrow inner channel and the full width. In all cells the mean bottom is 3.3 feet (39.6 in) above the bottom of the inner channel (see **Section 4.4.2.2.1.**) In all figures, the bottom of the inner channel is shown by the solid black line, the mean bottom elevation is shown by the red line, the design depth (24 inches above mean bottom) is shown by the green line, and the weir crest elevation for the controlling outflow structure is shown by the magenta line. These values are all relative to the mean bottom.

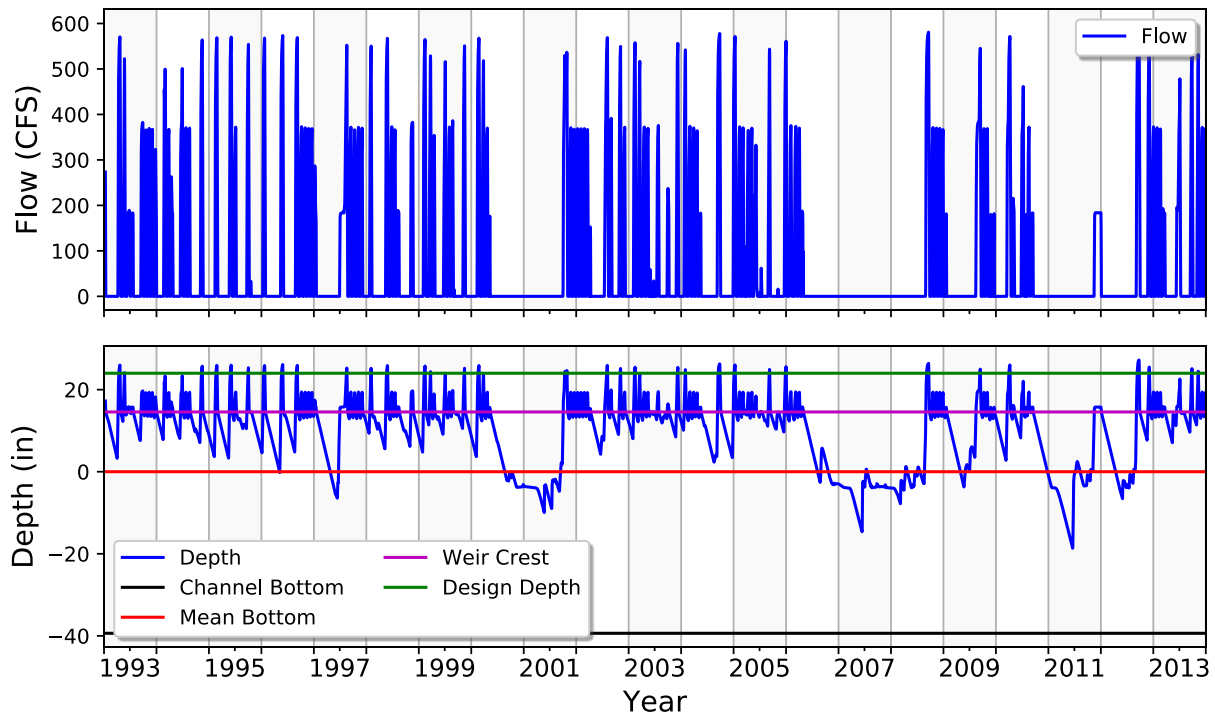
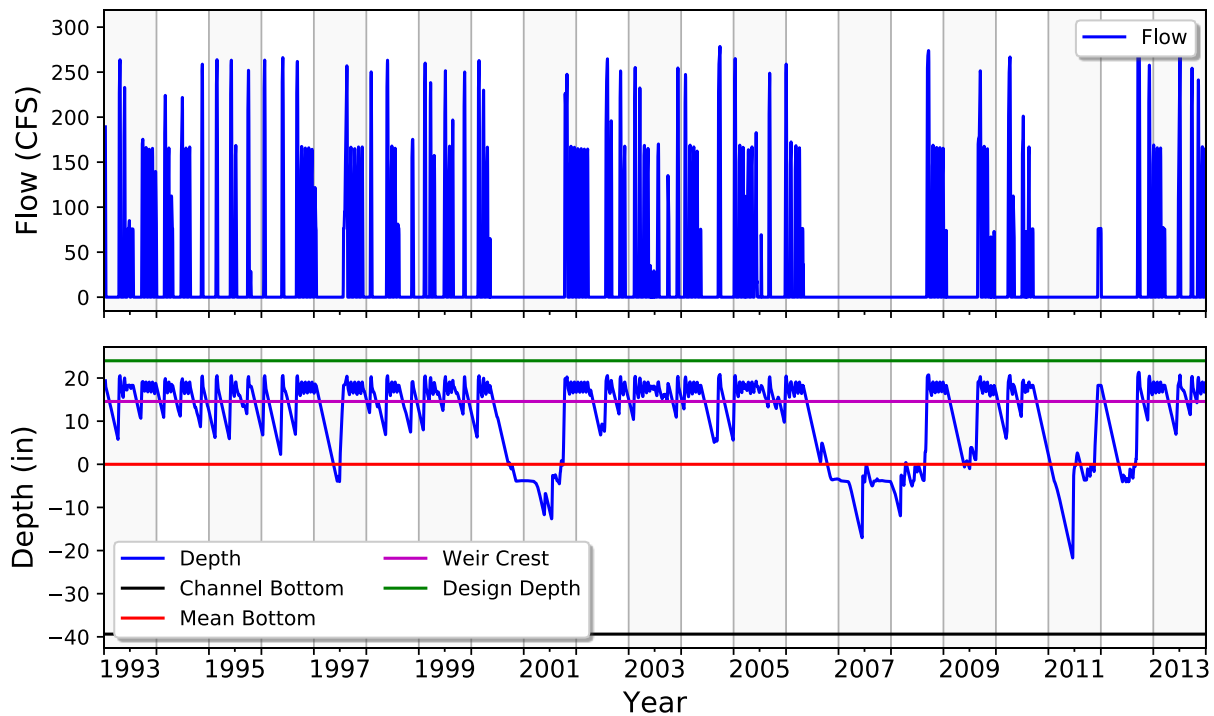
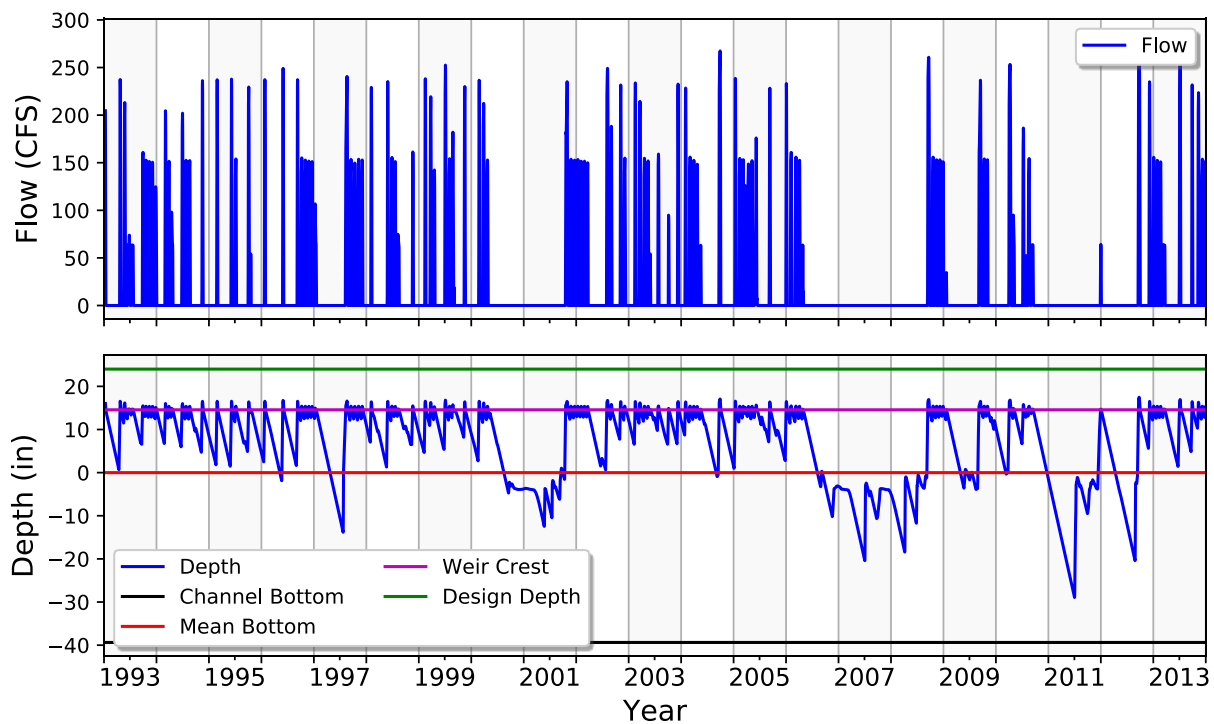


Figure 60. Simulated outflow (upper pane) and water depths (lower pane) in the Western basin.



1571

1572 **Figure 61. Simulated outflow (upper pane) and water depths (lower pane) in the Central basin.**



1573

1574 **Figure 62. Simulated outflow (upper pane) and water depths (lower pane) in the Eastern basin.**

4.4.3 Summary of Fisheating Creek LOWCP Results

The simulated annual average total volumes and TP loads discharging from Fisheating Creek into Lake Okeechobee and from Nicodemus Slough into the lake are shown in **Table 34**. The simulated volume discharging from Fisheating Creek was virtually unchanged (0.02% lower), while the simulated TP load decreased by about 2.8%. Nicodemus Slough was accounted separately here since the primary objective was water storage, not necessarily nutrient reduction (SFWMD 2014a). In addition, Nicodemus Slough only receives water from Lake Okeechobee and nutrient reductions should not necessarily be counted as a reduction from the Fisheating Creek sub-watershed. Overall a net total of 39,917 ac-ft/y were pumped into the slough, compared with 13,812 ac-ft/y discharging from the slough in the base scenario. The total discharge TP load into Lake Okeechobee decreased from 2.9 mt/y to 2.1 mt/y, however, an additional 5.1 mt/y of TP was pumped into Nicodemus Slough from Lake Okeechobee due to operations of the project. Thus, the overall reduction was from 8 mt/y (5.1 + 2.9 mt/y) to 2.1 mt/y.

Table 34. Simulated annual average volumes and TP loads discharging from the Fisheating Creek sub-watershed. The “LOWCP Inflow” TP load indicates loading from the lake into Nicodemus Slough during pumping.

Location	Volume (ac-ft/y)			TP Load (mt/y)			
	Base	LOWCP	Change	Base	LOWCP Inflow	LOWCP Discharge	Change
Fisheating Creek	215,511	215,477	-35	58.0	-	56.4	-1.7
Nicodemus Slough	13,812	-39,917	-53,729	2.9	5.1	2.1	-5.9

4.5 Upper Kissimmee Lake Okeechobee Watershed Construction Projects

The WAM setup for Upper Kissimmee (**Figure 63**) was developed as part of the WAM recalibration to estimate water flows and nutrient loads across all the nine sub-watersheds in the LOW (SWET 2017a, 2017b). The Upper Kissimmee sub-watershed was simulated as part of the northern LOW sub-watersheds (SWET 2017a). One project in the LOWCP was simulated within the Upper Kissimmee sub-watershed, the RMWRP. The base WAM scenario for Upper Kissimmee was duplicated and modified to incorporate this construction project.

A shapefile was obtained from the SFWMD that partially delineates (only Parcel B, not Parcel A) the RMWRP (**Figure 63**). In addition, two documents outlining the operational plan for the RMWRP were provided by the SFWMD (SFWMD 2014b; SFWMD 2018c).

4.5.1 Rolling Meadows Wetlands Restoration Project

The Rolling Meadows property encompasses approximately 5,787 acres and was jointly purchased in 2002 by the FDEP and the SFWMD as part of the Kissimmee Headwaters Revitalization Project. The property is divided into two parcels, which are identified as Parcel A & Parcel B (**Figure 64**). Parcel A is the southern parcel and encompasses approximately 3,815 acres. This parcel has historically been utilized as a tomato and sod farm and is currently being managed by FDEP as part of the Allen David Broussard Catfish Creek Preserve State Park. Parcel B is the northern parcel and encompasses approximately 1,972 acres. This parcel has historically been utilized as a sod farm and for cattle grazing and is currently being managed by the SFWMD (SFWMD 2018c).

The goal of the RMWRP was to restore the natural littoral habitat of Parcel B. The restoration was to be accomplished through re-establishing connectivity between Parcel B and Lake Hatchineha and the diversion of some of the flows from the historically diverted Catfish Creek (SFWMD 2014b). Catfish Creek flows east out of Lake Pierce until reaching a system of agricultural ditches used to divert and manipulate water flow and levels. The flow continues north and east through the project to Lake Hatchineha. The operation plan documents seven locations within the system that contain hydraulic structures (weirs and culverts) to be operated to facilitate the restoration project (SFWMD 2014b). Two main flow paths through Parcel A (that join and discharge to Parcel B) are described (**Figure 65**). One is delineated by the blue line along the east side of the former sod fields in Parcel A in **Figure 65**. The second is described as flowing north from location 1 along the west side of the former sod fields and bypasses locations 2, 4, and 5. Flow then splits to the south and east at location 6, with southward directed water entering Parcel B and eastward directed water discharging over a weir into Lake Hatchineha. Water discharges from Parcel B into Lake Hatchineha over a weir at the eastern end of the parcel at location 11 (**Figure 66**).

A weir structure (location 11) has been designed to optimize water levels in Parcel B for wetland restoration. This is primarily accomplished by keeping weir levels at location 6 below weir levels at location 7. This strategy directs all base flows to Parcel B. The weir configuration and elevation will also allow the inflow of flows from Lake Hatchineha when the lake stages are above the design elevation. The weir crest elevations at location 1 determines if base flows will be directed towards historic Catfish Creek or through the project. The structures used within the project are shown in **Table 35**. Note that elevations were originally given in the NAVD 88 datum; to be consistent with the Upper Kissimmee setup, these have been altered to NGVD 29 by adding a shift of 1.16 feet; unless otherwise noted elevations are in NGVD. Structures used in the WAM setup are highlighted in green.

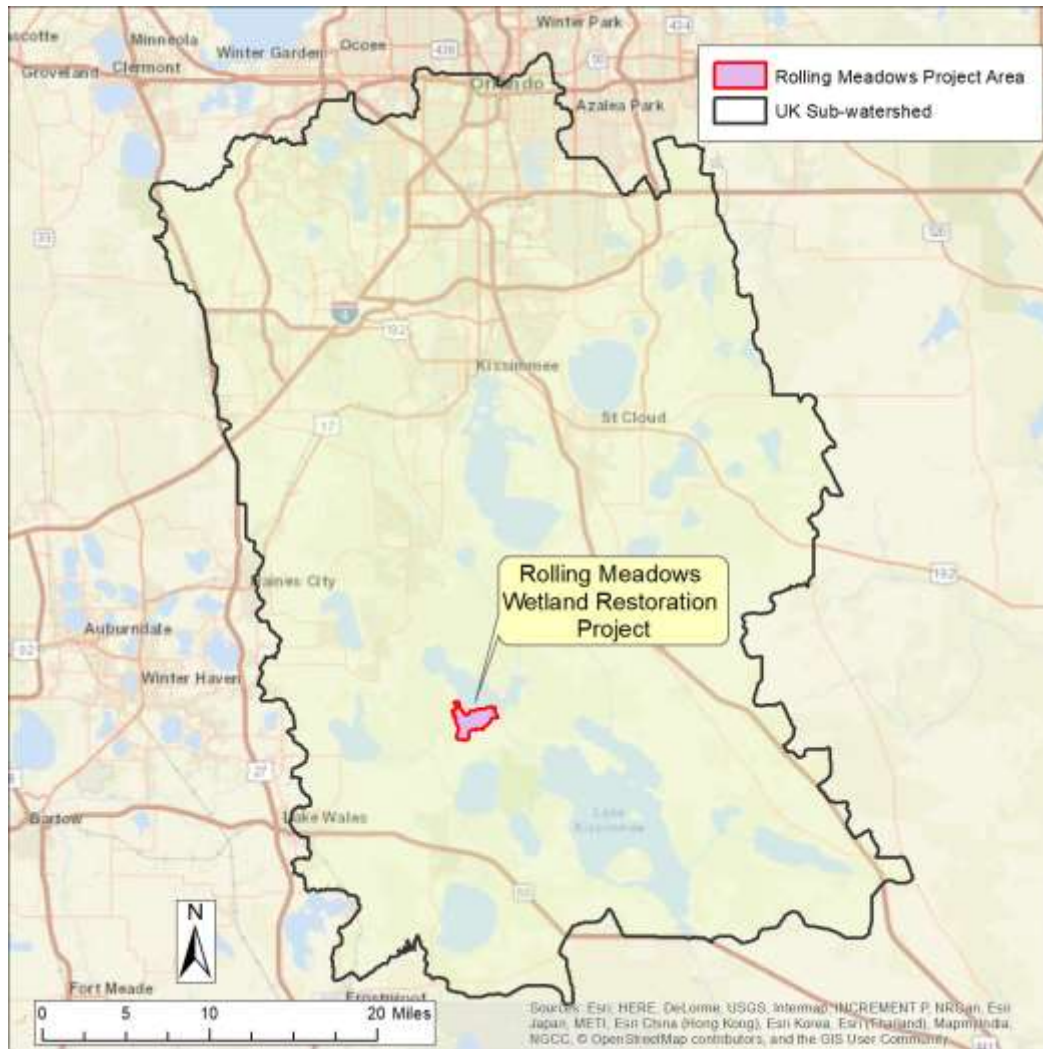


Figure 63. The WAM setup for the Upper Kissimmee sub-watershed, with the location of the RMWRP shown.

Table 35. Weir parameters in the RMWRP (updated in SFWMD 2018c).

Location	Structure Number	Weir Elevation (ft NAVD 88)	Weir Elevation (ft NGVD 29)	Simulated Weir Width	Used in WAM
1	1W	Open			No
1	1N	55	56.16	33	Yes
2	2	56.8	57.96	N/A	No
4	4W	Open		N/A	No
4	4N	55.5	56.66	N/A	No
4	4S	56.35	57.51	N/A	No
5	5S	56.4	57.56	N/A	No
5	5N	Open		N/A	Yes
6	6	Open		N/A	Yes
7	7	53.8	54.96	33	Yes
11	11	51.5	52.66	50	Yes

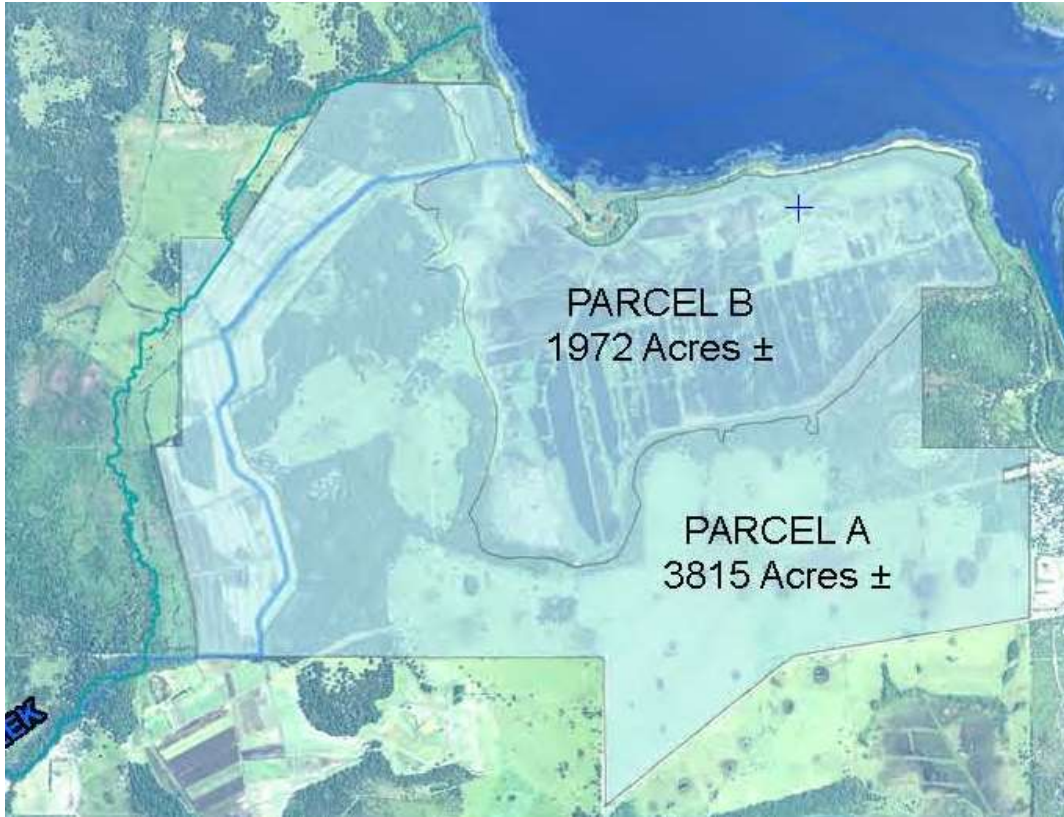


Figure 64. Location of Parcel B in the RMWRP (from SFWMD 2014b)

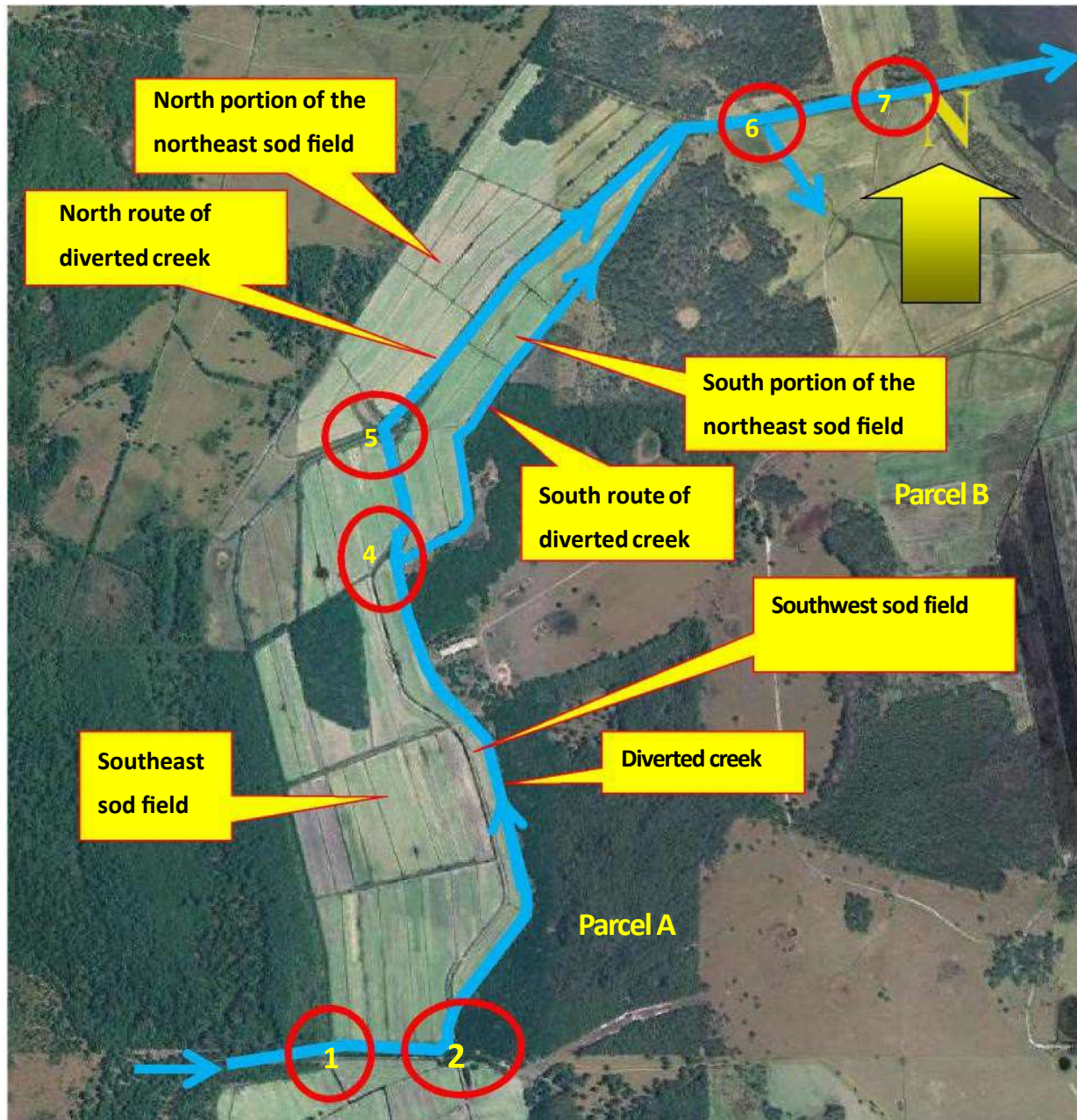


Figure 65. The layout of the flow system in parcel A. From SFWMD (2014b)

4.5.1.1 WAM Setup for RMWRP.

The WAM setup is shown in **Figure 66**. Because the primary goal is to simulate Parcel B, the WAM scenario treats the flow through the former sod fields in Parcel A as a single reach (WAM reach 519) with a weir controlling inflow from Catfish Creek. This WAM reach follows the route through location 1W (open), 1N (weir crest 56.16 ft NGVD), and 5N (open) until it reaches the north edge of Parcel B. Since the 1W and 5N weirs are open, flow through the canals on the east side of the sod fields was not simulated and only the single reach (519) was used. At the point where the reach is adjacent to Parcel B, two reaches split off – one to the east (WAM reach 568) to enter Lake Hatchineha (represented by WAM reach 5), and the second comprises Parcel B, which is represented by two WAM reaches, 532 and 529. Reach 529 discharges into Lake Hatchineha (WAM reach 5) over the outflow weir (location 11). The

structures highlighted in green in **Table 35** were implemented in the WAM simulations; the others were not included.

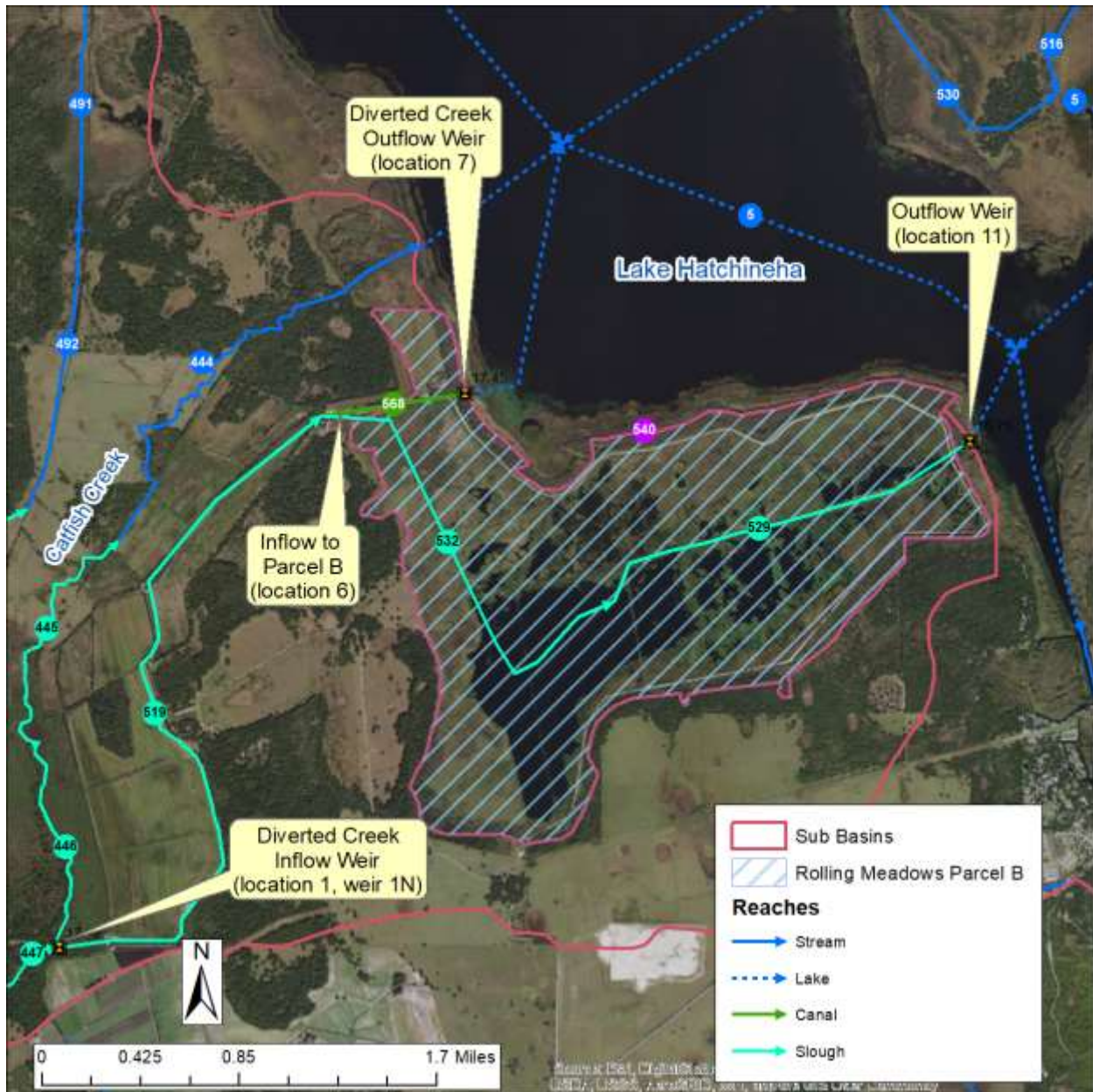


Figure 66. The WAM setup for RMWRP Parcel B.

4.5.1.1.1 Parcel B Reach Parameters

The two WAM reaches representing Parcel B (reaches 532 and 529) were set to have lengths and cross sections that match the area of Parcel B (**Table 36**). The area covered by the footprints of the two reaches total 1,972 acres. As with reach cross-sectional profiles for other retention areas simulated, the profiles contain a narrow “notch” that is about 1.6 feet deep and 66 feet wide, with the flow width reaching full width at a depth of about 3.3 feet (1 meter) (**Table 37**). See **Section 4.1.1** for a general discussion on the setup of cross-sectional profiles in WAM. The bottoms of the two reaches (at the bottom of the “notch”) are 46 feet. A schematic diagram of the cross-sectional profile for WAM reach 532 is shown in **Figure 67**. Therefore, the stage at which the impoundment is at the threshold of being fully submerged is 49.3 feet.

Table 36. Parameters for WAM reaches that comprise Parcel B.

WAM Reach	Length (ft)	Width (ft)	Acres	Bottom Elev (ft)
529	8,858	5,249	1,067	45.9
532	10,499	3,753	905	45.9

Table 37. Reach cross-sectional profiles for the RMWRP impoundments.

RMWRP WAM Reach 532		RMWRP WAM Reach 532	
Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
13.1	3,773	13.1	5,282
3.3	3,740	3.3	5,249
1.6	66	1.6	66
0.0	16	0.0	16

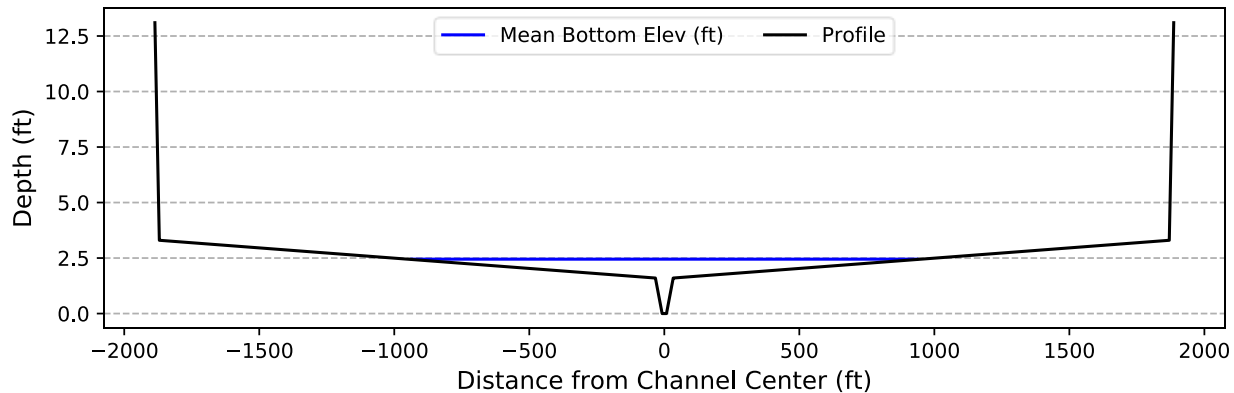


Figure 67. The cross-sectional profile for WAM reach 532 in the RMWRP used for the simulation with the mean bottom elevation indicated.

4.5.1.1.2 Attenuation Coefficients

As discussed in **Section 4.1.2**, WAM applies attenuation to nutrients within all reaches, including those simulating STAs. For RMWRP Parcel B, in the absence of performance data for nutrient attenuation, the attenuation coefficients were set to lie within the range of those used for other impoundments constructed or planned as part of the LOWCP. The attenuation coefficients used for the two reaches within Parcel B are $a = 0.1$ and $C_{BG} = 0.025$ mg/L for dissolved P and 0.015 mg/L for particulate P (i.e., $C_{BG} = 0.04$ mg/L for TP). If and/or when TP concentrations in water discharging from Parcel B become available, it will be possible to refine the attenuation coefficients.

The land use type assigned to the impoundment was split between a special case land use type that passes rainfall directly through to the stream network, which then handles attenuation and ET, and a freshwater marsh wetland type.

4.5.1.1.3 Weir controls

The WAM setup included weirs at locations 1N, 7, and 11, and an open connection at location 6 (**Figure 66**). The simulated weir widths are shown in **Table 36**. The reported weir crest elevations at locations 7

and 11 are 54.96 and 52.66 (NGVD), respectively (SFWMD 2018c), with the higher crest elevation at location 7 designed to pass most water through Parcel B and discharging over the lower weir at location 11. Because the WAM simulated stages in Lake Hatchineha are higher than the observed stages (the stage for this lake was not a calibrated parameter), the simulated weir crest elevations at locations 7 and 11 were raised above the reported values (SFWMD 2018c), being set to 57.25 and 54.96, respectively. This was done to prevent excessive backflow into Parcel B from the lake, although there is some degree of flow back into the parcel.

4.5.1.2 WAM Simulation Results

The WAM simulation for the RMWRP was run over the 21-year period from 1993 through the end of 2013. The WAM base simulation was rerun over the same period to obtain comparable results. Simulated daily inflow and outflow TP concentrations are shown in **Figure 68** by the orange and green lines, respectively, while simulated daily flows are shown in **Figure 69**. Note that the negative flows indicate occasional backflow from Lake Hatchineha into the RMWRP, which is responsible for the outflow concentration values being less than inflow concentrations during the brief period in 2005.

Summaries of changes in simulated water volume and TP loadings are shown in **Table 38**. In the table, location 11 is the discharge from Parcel B to Lake Hatchineha and location 7 is the discharge from the bypass canal to the lake. Discharge to the lake from Catfish Creek is included since water (and nutrients) are diverted to pass through Parcel B. Also included is the change in the Upper Kissimmee sub-watershed discharge as measured at the discharge point from the S-65 structure. The subtotal shows an annual average decrease of about 10% per year in water volume entering Lake Hatchineha and an annual average decrease in TP of about 58% (1.73 mt/y). About 11% of the water in Catfish Creek is diverted into the RMWRP; however, discharge from the RMWRP also includes rainfall within both parcels. Note that most of the water diverted from Catfish Creek enters Parcel B and discharges via the weir at location 11 with only a small amount (25 ac-ft/y) discharging to the lake at location 7. Overall, the simulated project indicates about a 0.4% decrease in water volume leaving the sub-watershed and about a 1.9% reduction in TP (about 1.5 mt).

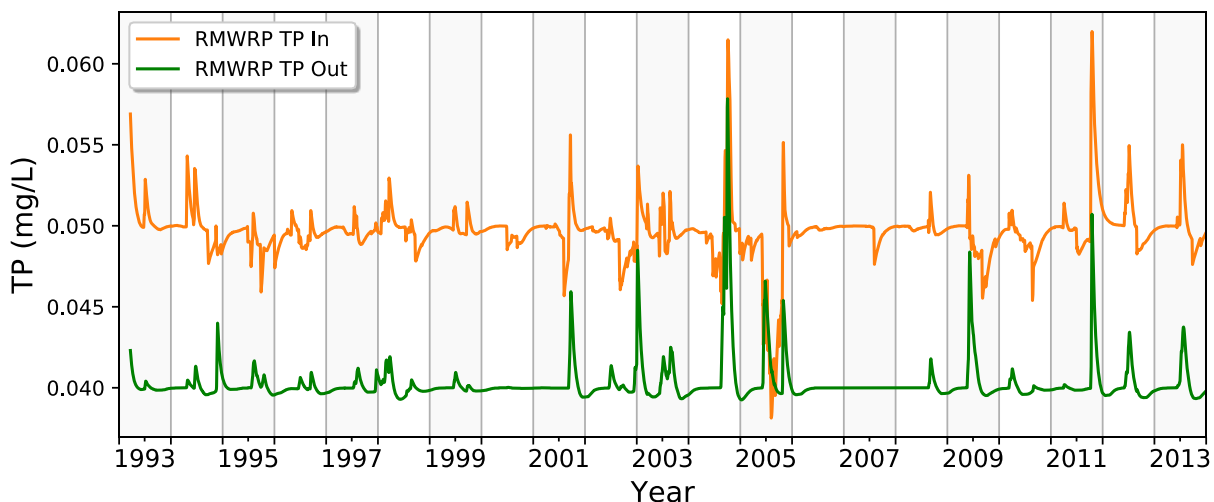


Figure 68. Comparison of simulated TP concentrations between inflow to RMWRP Parcel B (orange line) and outflow to Lake Hatchineha (green line) over the 1993-2013 simulation period.

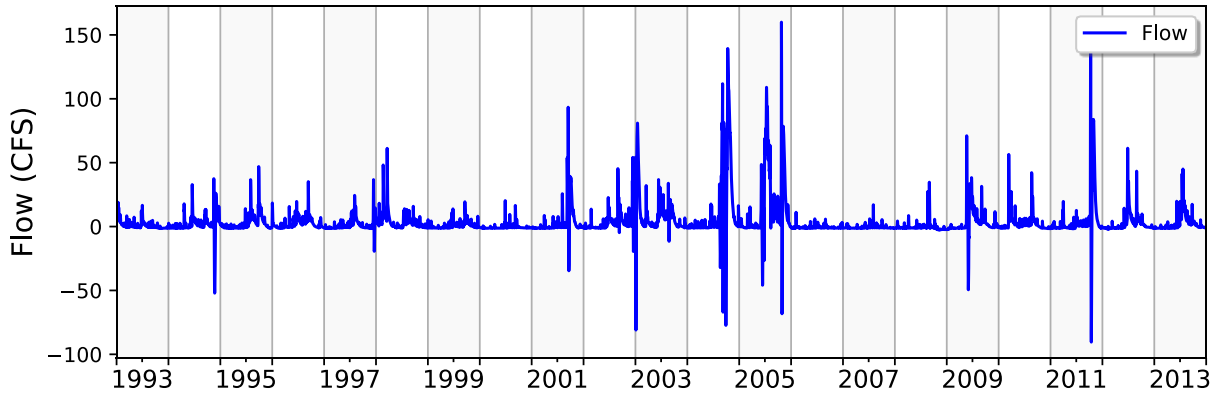


Figure 69. Simulated flows in RMWRP over the simulation period 1993-2013

Table 38. Comparison of simulated Upper Kissimmee Base and RMWRP results.

Location	Base ac-ft/y	RMWRP ac-ft/y	Change	Percent	Base TP mt/y	RMWRP TP mt/y	Change	Percent
RMWRP Area Discharge								
RMWRP Location 11	1,728	3,579	1,851	107%	0.65	0.23	-0.42	-64%
RMWRP Location 7	1,769	25	-1,744	-99%	0.33	0.00	-0.33	-99%
Catfish Creek	26,804	23,805	-2,999	-11%	1.99	1.01	-0.98	-49%
Subtotals	30,301	27,409	-2,893	-10%	2.98	1.25	-1.73	-58%
Upper Kissimmee Sub-Watershed Discharge								
UK S-65 Discharge	728,831	726,038	-2,793	-0.4%	81.0	79.5	-1.5	-1.9%

5 Kissimmee River Restoration (KRR) Project

5.1.1 Introduction and Background

The restoration of the Kissimmee River in central Florida is one of the largest such restoration projects in the world (**Figure 71**). The goal of the project is to restore ecological function and biological communities to much of the river and floodplain, primarily by restoring lost hydrologic drivers (Bousquin et al. 2005). When complete in 2020 (projected), the project will restore nearly 25,000 acres of floodplain wetlands and over 40 miles of meandering river channel (Bousquin et al. 2005).

As part of the Central & Southern Florida (C&SF) flood control project, the C-38 canal was excavated along the entire length of the Kissimmee River and floodplain. This construction was completed in 1971. The canal was designed to provide a high level of flood protection for surrounding communities and agricultural interests after several hurricanes and floods in the early half of the 20th century. The primary flood protection strategy in the design of C-38 was to contain all flow that the river and floodplain had formerly carried south to Lake Okeechobee. The canal effectively eliminated flow in the river and ended seasonal overbank flow that historically had caused inundation of the floodplain (Bousquin et al. 2005). However, the project resulted in the loss of most floodplain wetlands with subsequent declines in wildlife populations and reductions in water quality.

The primary strategy of the restoration project is to backfill a substantial portion of C-38 and to re-carve and reconnect remnant river channels where necessary to restore the original continuity to the river channel. Water enters the river channel from the canal at the north end of backfilling, resulting in reestablished flow in the river channel, and, when flow exceeds channel bankfull, inundation of the floodplain. The Kissimmee River Restoration Evaluation Program (KRREP) has published several reports, including “*Establishing a Baseline: Pre-restoration Studies of the Channelized Kissimmee River*” (SFWMD 2005), which includes a wealth of information regarding the history, impacts, and overall plan to restore the Kissimmee River. **Figures 71** and **72** show the boundaries of the restoration areas, broken down into the four phases of construction. Phase I construction (Phase I) was completed in 2001, while construction in Phases IVa and IVb was completed in 2007 and 2010, respectively. Phases II and III are scheduled to be completed in 2020. The restoration effort also included removal of the S-65B structure in 2001 and the removal of the S-65C structure in 2017.

As part of the current task to examine abatement strategies within the LOW, the fully implemented KRR project as designed was incorporated into the hydrography, land use, and hydraulic structure feature classes used in the WAM setup to simulate the Lower Kissimmee sub-watershed.

5.1.2 Implementation in WAM

The Lower Kissimmee sub-watershed was simulated with WAM in 2017 (SWET 2017a). That simulation included portions of the restored river channel and floodplain in Phases I, IVa, and IVb that were in place by the end of the simulation period in 2013. The land use dataset utilized in the 2017 simulations was obtained from the SFWMD in 2016 (SFWMD 2016), however, most of the feature data were from 2009 with some updates added in 2013; this land use dataset included portions of the restored floodplain wetlands in Phases I and IVa, with some in Phase IVb as well. The backfilled section of canal in Phases I and IV were present in the SFWMD ArcHydro Enhanced Database (AHED) included in the 2017 WAM setup. Consequently, it is important to understand that the 2017 WAM setup simulated a partially-completed restoration project with three phases complete. Results from that simulation already reflect some degree of hydrologic and water quality benefits resulting from the completed portions of the KRR Project.

In this report, that setup with the partially restored river is referred to as the “base” condition. The WAM implementation described here with the fully restored Kissimmee River channel and floodplain will be

referred to as the “Full KRR project”. Note that to determine the full effect of all phases of the KRR project would require simulations of the intact C&SF C-38 and non-restored floodplain.

To represent the full KRR project in WAM, the 2017 setup for the Lower Kissimmee sub-watershed was altered to include data obtained from the SFWMD. The two GIS datasets obtained were:

1. A shapefile representing the fully restored river.
2. A shapefile with the expected post-restoration land cover in the floodplain.

These datasets are shown in **Figure 72** as the feature classes “Restored River Path” and “Planned Wetlands”, respectively. The restored river path dataset was used as the basis for altering the WAM hydrography feature class used in the 2017 simulations of the Lower Kissimmee sub-watershed to include reaches that represent the restored river. The features from the shapefile representing the post-restoration land cover in the floodplain were incorporated into the land use feature class that was used in the 2017 setup for the Lower Kissimmee sub-watershed. The S-65B structure had been removed from the 2017 WAM setup and for the full KRR project the S-65C water control structures were removed from the structures feature class.

Time series data representing discharges from the S-65 structure (the Lake Kissimmee outflow) were obtained from the SFWMD. These data were generated by model simulations conducted by the SFWMD with the Headwaters Revitalization Schedule (HRS) in place. The HRS will be implemented upon completion of project construction and will therefore be an important driver of floodplain inundation and nutrient retention in the future condition. These data were incorporated into the Lower Kissimmee sub-watershed WAM setup as the inflow boundary condition at the northern end of the sub-watershed. Over the simulation period, the accumulated simulated HRS discharge volume from S-65 is about 1.5% lower than the measured volume (measured data were obtained from DBHydro for the 2017 WAM simulations [SWET 2017a]). A comparison of the accumulated measured and HRS simulated volumes are shown in **Figure 70**.

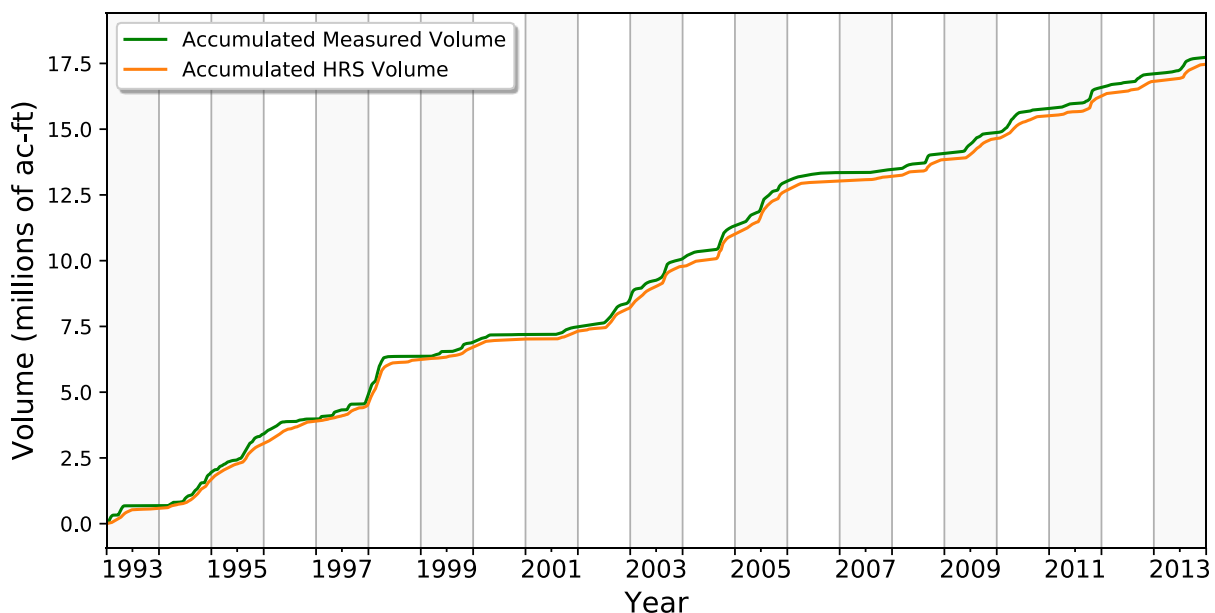


Figure 70. Accumulated measured and HRS simulated volumes at S-65.

As mentioned above, for these reasons, the scenarios compared here contrast a partially-completed restoration project (the base scenario with three construction phases complete) with the fully constructed

KRR that uses model simulations of S-65 releases based on a new water regulation schedule, the HRS, to estimate inflow. The analysis presented here is therefore conservative and represents the benefits of the completed KRR compared only to the partially-restored Base condition and thus may not reflect the full effects of the KRR project on nutrient retention.

5.1.2.1 Land Use Changes

The shapefile of the expected post-restoration land cover in the floodplain that was obtained from the SFWMD used a different classification system for the vegetation within the floodplain than that used by WAM. **Table 39** shows the lookup table used to reclassify the vegetation types in the fully restored KRR land cover dataset (i.e., **Figure 72**) to WAM land use types and IDs (i.e., **Figure 73**). The expected changes in land use acreages from the base scenario to the fully restored KRR scenario are listed in **Table 39**. Note that the base scenario already contained significant acreages of restored wetlands in the Kissimmee River floodplain since it utilized the 2009 SFWMD land use/land cover GIS dataset (updated in 2013), which contained floodplain land use changes in Phases I and IVa of the restoration project, and some of those in Phase IVb as well. Most of these wetlands were categorized in WAM as “Freshwater Marshes”, while fully restored KRR land cover dataset provided by the SFWMD used a more refined distribution of wetland categories. Consequently, the reclassification resulted in the relatively large acreage changes from Freshwater Marshes to other wetland types (e.g., “Mixed Wetland Hardwoods”). Changes from Improved Pasture and other agricultural (and urban) land uses to wetland categories reflects the restoration of non-native land uses within the final two phases of the restoration project in the floodplain. About 5,000 acres were restored to wetland (and open water) land uses between the land use dataset used in the base scenario (the 2017 WAM simulations) and the fully restored KRR floodplain dataset.

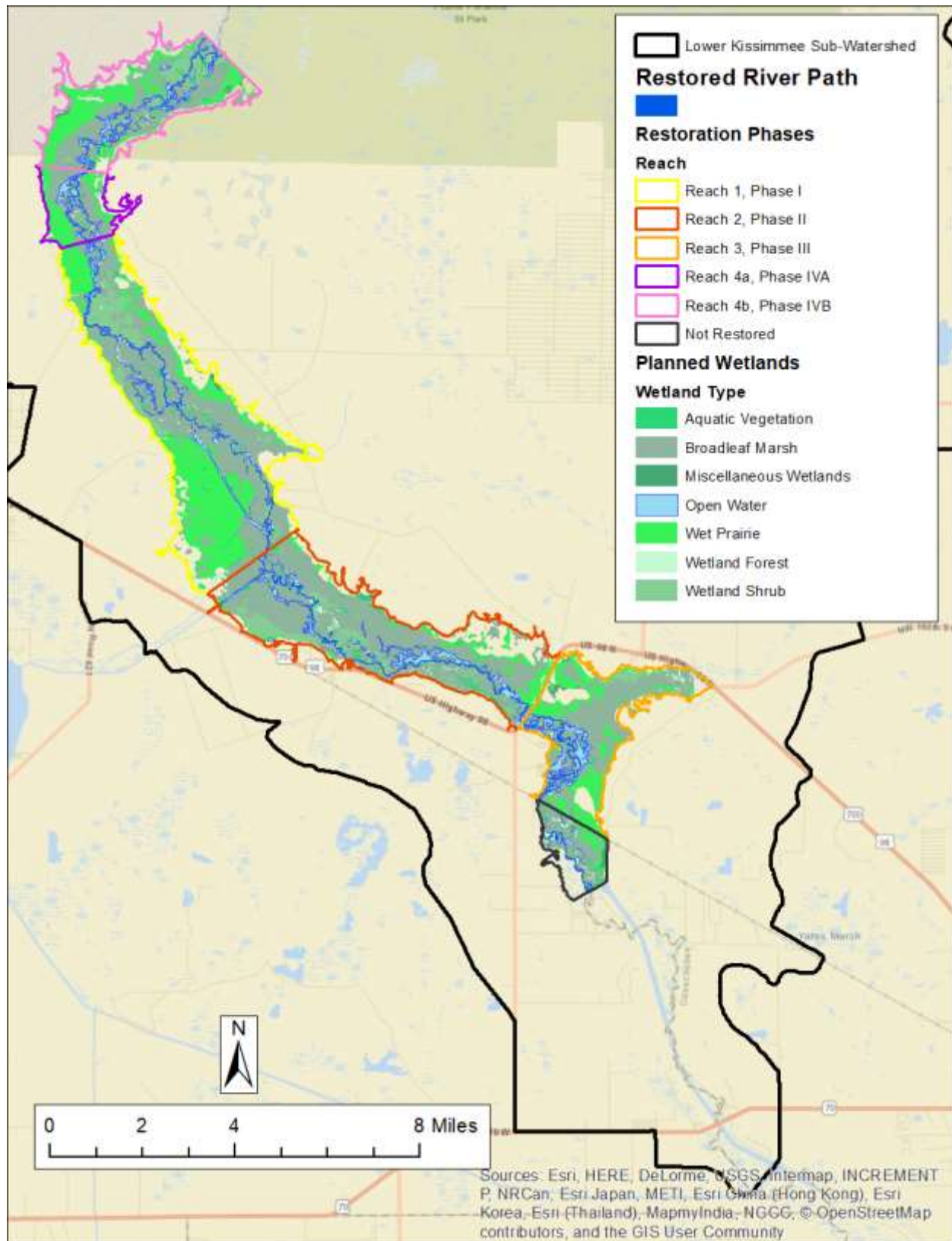


Figure 72. The fully restored river channel and floodplain wetlands feature classes under the completed KRR project obtained from the SFWMD.

1826 **Table 39. Restored wetland types given in the SFWMD floodplain delineation and the corresponding WAM**
 1827 **land use type and ID.**

Wetland Type	WAM Land Use	WAM Land Use ID
Aquatic Vegetation	Freshwater Marshes	16
Broadleaf Marsh	Mixed Wetland Hardwoods	12
Miscellaneous Wetlands	Freshwater Marshes	16
Open Water	Open Water	9
Wet Prairie	Freshwater Marshes	16
Wetland Forest	Cypress	14
Wetland Shrub	Wetland Forested Mixed	15

1828

1829 **Table 40. Land use acreage changes from the base scenario to the KRR project scenario.**

Land Use	Base (ac)	Full KRR (ac)	Change (ac)
Mixed Wetland Hardwoods	17,197	25,408	8,211
Wetland Forested Mixed	1,877	3,788	1,911
Open Water	3,432	4,187	755
High Density Residential	112	111	-1
Cypress	1,387	1,387	-1
Medium Density Residential	193	191	-2
Commercial and Services	1,944	1,935	-9
Hardwoods	440	386	-55
Low Density Residential	6,571	6,387	-184
Hardwood Conifer Mixed	7,620	7,422	-198
Unimproved Pasture	45,593	45,334	-259
Woodland Pasture	11,408	11,117	-291
Scrub and Brushland	90,312	89,889	-423
Barren Land	1,594	1,089	-505
Improved Pasture	115,722	112,617	-3,105
Freshwater Marshes	69,140	63,296	-5,844
Unchanged	54,633	54,633	0
Total in sub-watershed	429,175	429,176	0

1830

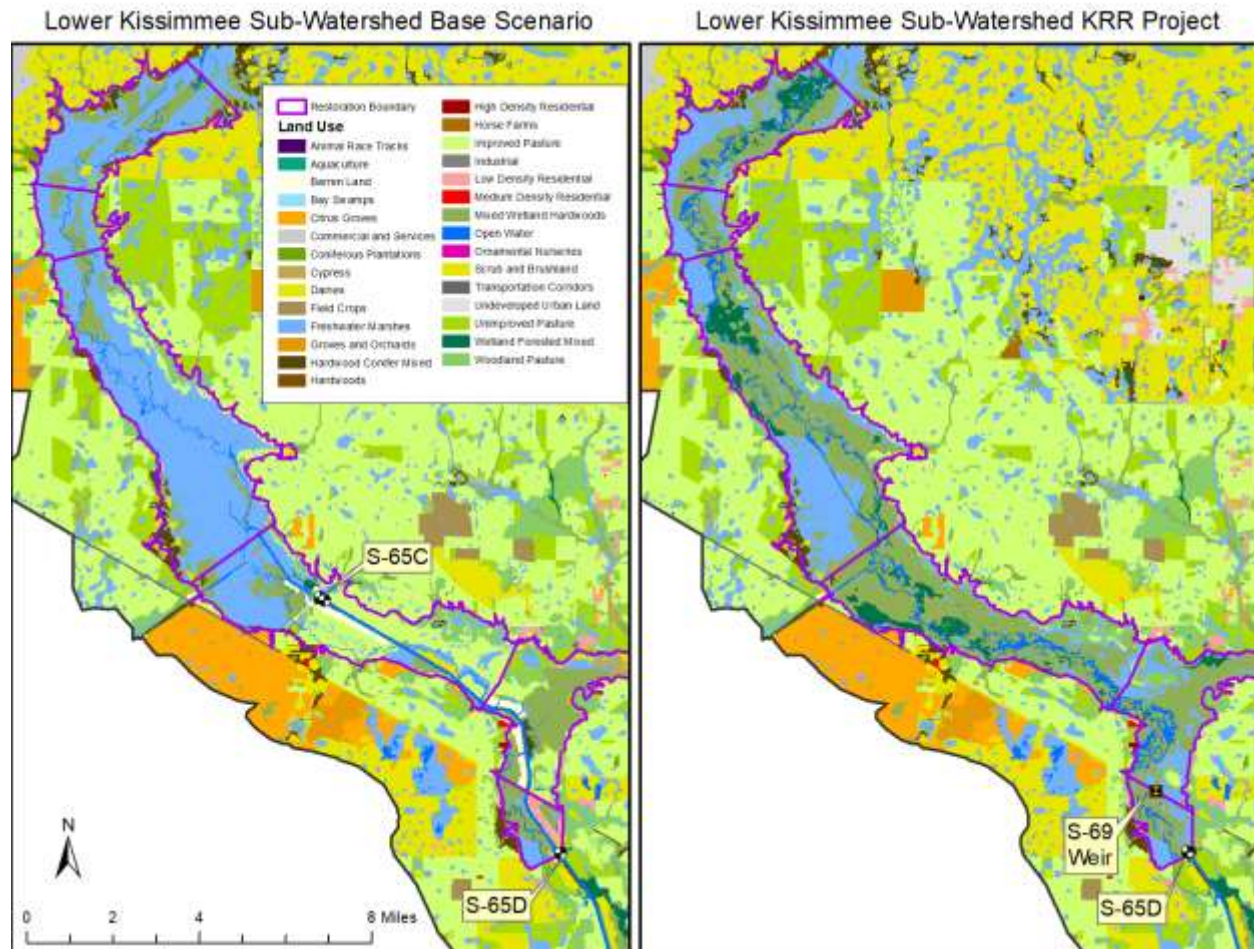


Figure 73. Comparison of land uses within the Lower Kissimmee base scenario (left pane, SWET 2017a) with those in the WAM setup for the full KRR project (right pane).

5.1.2.2 Hydrography Changes

As discussed above, the hydrography in the 2017 WAM setup was modified using the restored Kissimmee River shapefile obtained from the SFWMD (**Figure 74**). The WAM reaches (numbered 9, 10, and 11) in the left pane of **Figure 74** that represent the sections of C-38 within Phases II and III were removed and reaches were added to represent the reestablished, meandering main river channel within these areas. In addition, reach 293 (representing the MacArthur Ditch) was removed. In the full KRR WAM project scenario the historic Kissimmee River channel throughout the restoration area is represented by WAM reaches 10 – 21, 188, 288, and 289.

Construction data, i.e., as-builts for the river sections that have already been restored and plans for areas in progress were used to develop cross-sectional profiles. Reach cross-sectional profiles within the floodplain were modified so that each WAM reach representing the restored Kissimmee River and floodplain had an inner section to represent the main channel of the river (with widths and depths reflecting data from as-built drawings) and a broad floodplain that encompassed the width of the restored floodplain (**Table 41**). The reach cross-sectional profiles sloped very gently from the edge of the floodplain to the main channel. A schematic diagram of the cross-sectional profile of WAM reach 17 is shown in **Figure 75**, with the main channel of the Kissimmee River indicated by the green shading and the extent of the floodplain indicated by the blue arrows on either side of the main channel.

The water control structure S-65C in the base scenario were removed from the WAM setup for the KRR project, and the S-69 weir (north of S-65D) was added to the setup (**Figure 74**). The S-69 weir structure is designed as a 2,560-foot-long U-shaped weir at the bottom of WAM reach 9. In the WAM setup the weir crest elevation was set to 31.5 feet. When the stage in WAM reach 9 is at this elevation, the floodplain for WAM reach 9 is just fully submerged. Stages lower than the crest elevation will cause water to bypass the structure and flow down WAM reaches 289 and 288, which represent the historic channel of the Kissimmee River.

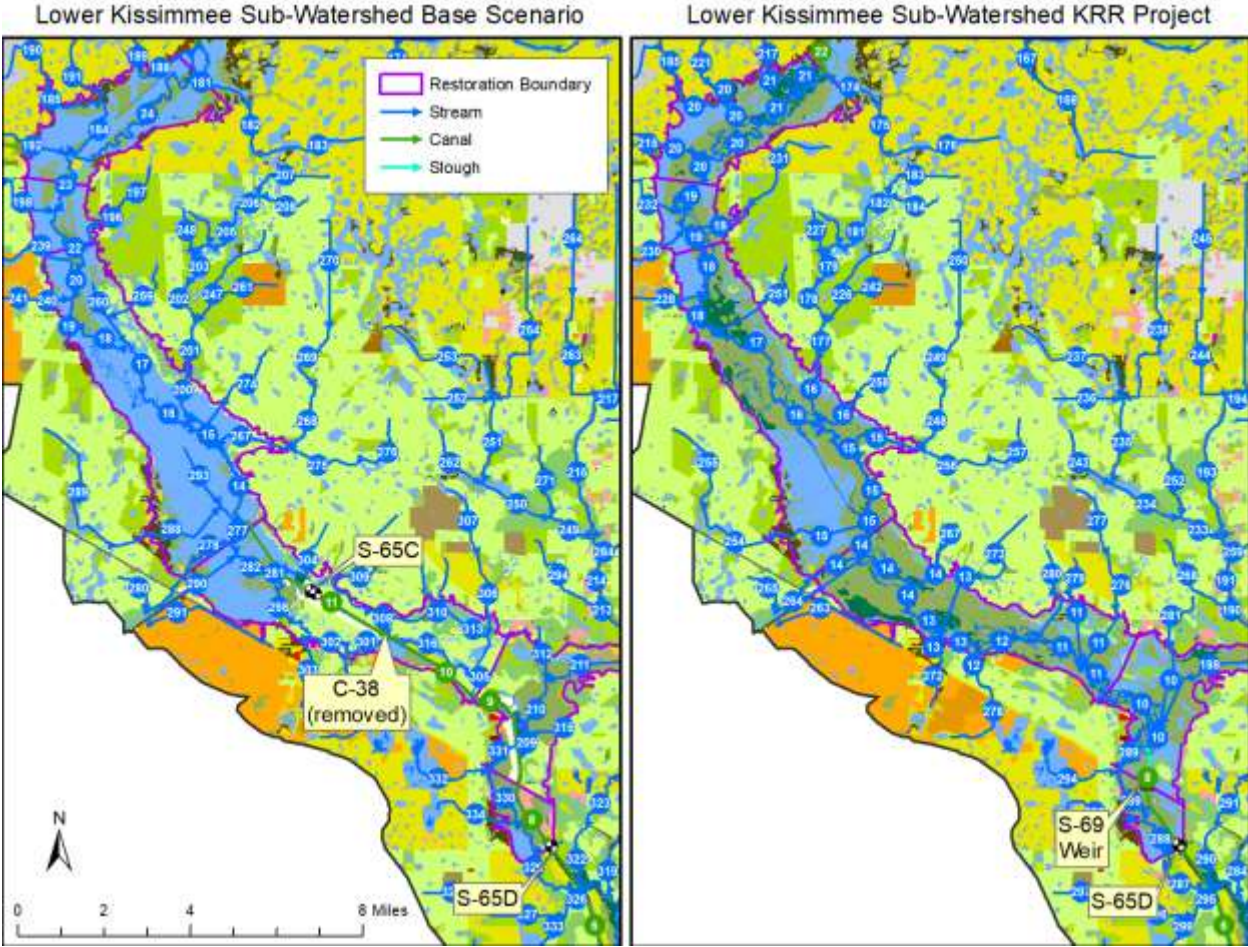


Figure 74. Comparison of the WAM reaches within the Lower Kissimmee base scenario (left pane, SWET 2017a) with those in the WAM setup for the KRR project (right pane).

Table 41. Cross-sectional profiles for WAM reaches representing the restored Kissimmee River channel and floodplain. The main river channel is represented by the narrower inner part of the profile and the floodplain is represented by the wide (>1000 feet) part of the profile. All widths and depths are in feet.

Reach 10		Reach 13		Reach 16		Reach 19		Reach 188	
Depth	Width	Depth	Width	Depth	Width	Depth	Width	Depth	Width
16.4	9,022	16.4	6,818	16.4	8,504	16.4	6,762	16.4	5,430
11.8	8,990	11.8	6,785	11.8	8,471	11.8	6,729	13.1	5,397
11.2	131	10.8	131	10.8	131	10.8	131	11.8	112

0.0	59	0.0	59	0.0	59	0.0	59	0.0	39
Reach 11		Reach 14		Reach 17		Reach 20		Reach 288	
16.4	8,484	16.4	9,337	16.4	7,930	16.4	9,022	16.4	4,094
11.8	8,451	11.8	9,304	11.8	7,897	11.8	8,990	11.8	4,062
10.8	131	10.8	131	10.8	131	10.8	131	10.8	131
0.0	59	0.0	59	0.0	59	0.0	59	0.0	59
Reach 12		Reach 15		Reach 18		Reach 21		Reach 289	
16.4	4,944	16.4	10,994	16.4	7,211	16.4	7,585	16.4	4,580
11.8	4,911	11.8	10,961	11.8	7,178	11.8	7,552	11.8	4,547
10.8	131	10.8	131	10.8	131	10.8	131	10.8	131
0.0	59	0.0	59	0.0	59	0.0	59	0.0	59

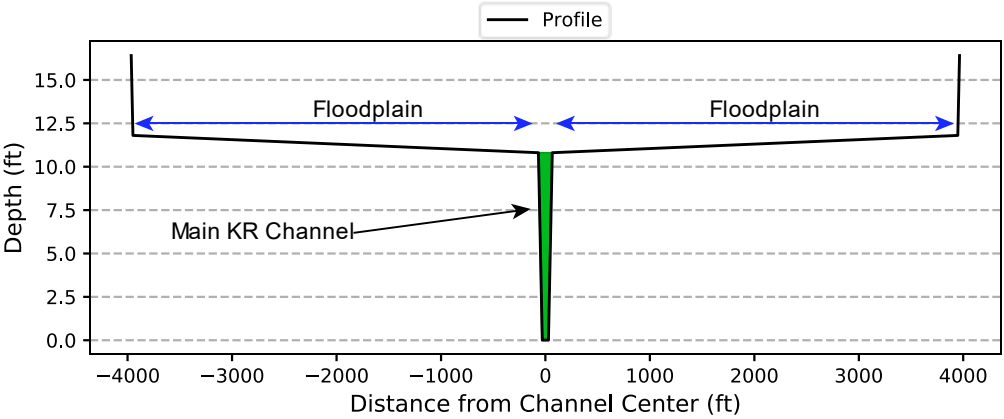


Figure 75. Diagram of the cross-sectional profile for WAM reach 17. The main channel is indicated by the green shading, while the broad floodplain is denoted by the blue lines.

5.1.3 Simulation Results

The simulation period was the 21-year time frame from 1993 through the end of 2013. The only changes from the 2017 WAM simulations (SWET 2017a) were:

- Changes to the reach network within the restored floodplain
- Changes to the land use within the restored floodplain
- Use of the modified inflows at north end of the Kissimmee River at the S-65 structure
- Removal of the S-65C structure and emplacement of the S-69 weir.

The downstream boundary conditions, rainfall, and land use outside of the restored floodplain were the same as the 2017 simulations. The downstream boundary condition is applied to WAM reach 1 and is the S-65E tailwater elevations obtained from DBHydro.

A comparison between simulated TP concentrations in the base scenario (green line) and full KRR scenario (orange line) at S-65E is shown in **Figure 76**. Simulated concentrations in the full KRR are less than the base scenario for most of the simulation period. The average simulated outflow concentrations for the base scenario are 0.094 mg/L, while the average simulated outflow concentration for the full KRR scenario is 0.74 mg/L. Simulated annual TP loads at S-65E are shown in **Figure 77**. Except for 1996, simulated TP loads under the full KRR project (green bars) are lower than the corresponding years in the

base scenario (orange bars). Note that the higher load in 1996 corresponds to the spike in simulated concentrations in mid-1996 in **Figure 76**.

Simulated annual volume discharges at S-65E are shown in **Figure 78**. In some years (e.g., 1993-1995) discharge volumes in the base scenario (green bars) are higher than the corresponding years in the full KRR scenario (orange bars), while in some other years (1996, 1998) the simulated discharge volumes in the KRR scenario are higher than the base scenario.

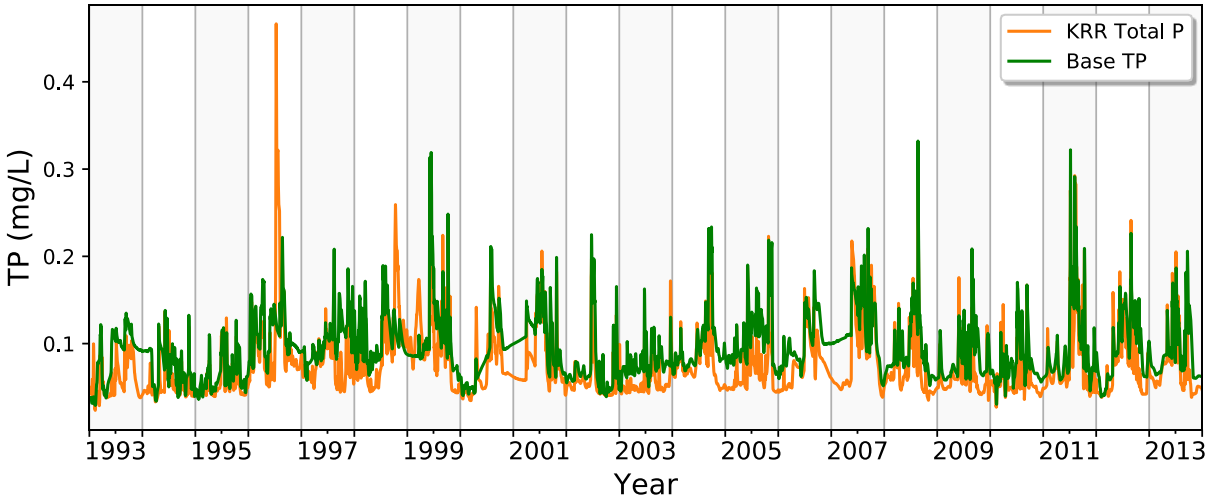


Figure 76. Comparison of simulated TP concentrations discharging at S-65E. The base scenario is indicated by the green line and the full KRR project is indicated by the orange line.

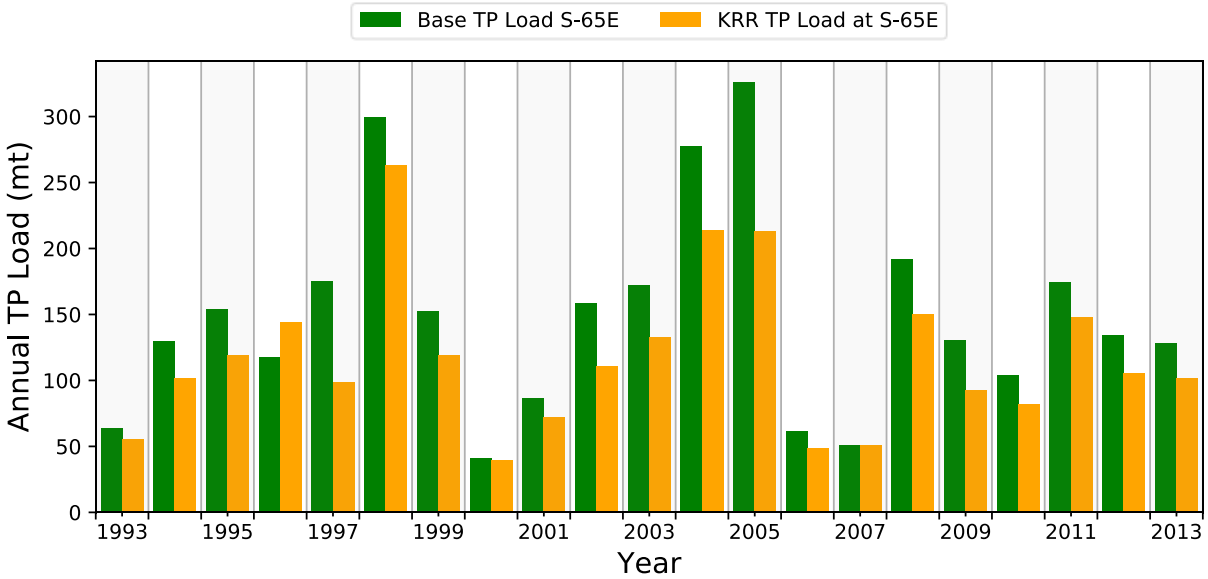


Figure 77. Comparison of simulated annual TP loads discharging at S-65E. The base scenario loads are shown by the green bars, while the full KRR project loads are shown by the orange bars.

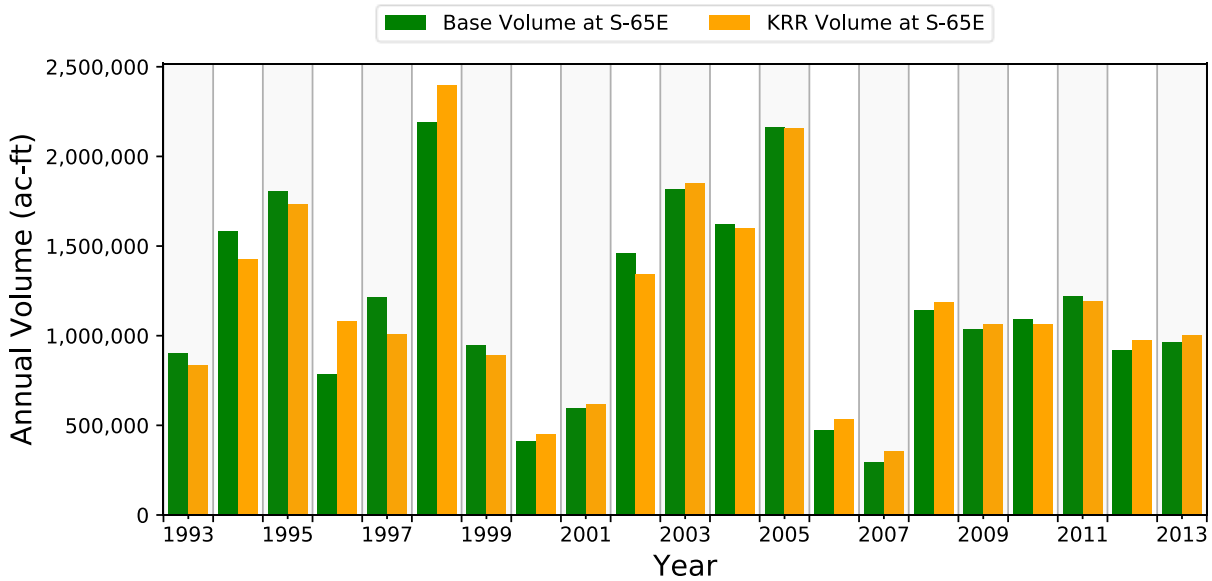


Figure 78. Simulated annual discharge volumes at S-65E. The base scenario volumes are shown by the green bars, while the full KRR project volumes are shown by the orange bars.

Using the inflow volumes calculated by simulations conducted by the SFWMD using the HRS, the full KRR project scenario shows a volume decrease of 11,506 ac-ft/y (1.4%) and a decrease in TP load of 0.9 mt/y (1.1%) at the inflow. At the S-65E structure, the exit point of the sub-watershed, the full KRR project scenario shows a minor increase in volume of slightly over 5,592 ac-ft/y (0.5%), but a decrease in TP loading of almost 32 mt/y (21.3%) when compared with the base (partial KRR) scenario. The slight increase in volume is probably due to removal of the small amount of land use within the floodplain that utilizes irrigation.

Table 42. Simulation results comparing annual average volume (ac-ft/y) and TP loads (mt/y) between the WAM base and KRR simulations

Location	Base ac-ft/y	KRR ac-ft/y	Change ac-ft/y	Base TP mt/y	KRR TP mt/y	Change mt/y
S-65	843,047	831,541	-11,506	82.9	82.0	-0.9
S-65E	1,171,893	1,177,845	5,592	149.1	117.3	-31.8

6 Summary and Conclusions

A series of four nutrient abatement scenarios were evaluated by comparing their WAM predicted N and P loads to the predicted N and P loads for the existing conditions/base scenario (SWET 2017a, 2017b).

The four scenarios evaluated were:

1. Advanced fertility management BMPs for major crops and urban landscapes in all sub-watersheds.
2. Advanced farm-level and urban water management BMPs, including additional on-farm and urban water retention and on-farm structural controls for irrigation/drainage in all sub-watersheds.
3. (a) Seven existing and approved LOWCP and (b) the proposed LOWRP K-05 WAF/ASR.
4. The fully implemented KRR Project and Upper Basin Initiatives, including HRS and KRREP.

It was assumed for the first two BMP scenarios that all existing BMPs within the LOW were represented in the existing conditions base scenario so that scenarios 1 and 2 would only represent additional or new more advanced BMPs not currently in place. Therefore, presented BMP costs would be future costs.

Table A.1 in **Appendix A** shows simulated N and P reductions and associated cost-effectiveness values for the individual fertility and water management BMPs for three representative soils. The fertilizer BMPs on average were found to be more cost-effective than the water management BMPs, i.e., have lower average cost per pound of N and P removed. However, the fertility BMPs are more limited in the amount of total N and P reduction that can be achieved as compared to the water management BMPs. In conclusion the fertilizer BMP should be a higher initial priority but the full implement of both fertility and water management BMPs plus additional N and P abatement projects will be needed to achieve meet BMAP targets.

The annual nutrient load reductions and cost for scenarios 1 and 2 for each of the nine sub-watersheds are provided in **Table 1** in **Section 3.2**. These results reflect the same relative N and P reduction and cost-effectiveness trends as described above for the individual BMPs. However, these sub-watershed results show the importance of the land use distributions within the sub-watersheds on the total N and P reductions and relative cost-effectiveness. For example, the results show that Indian Prairie should be priority sub-watershed for future BMPs efforts due to its lowest net cost-effectiveness value. In contrast, the WSW and Upper Kissimmee sub-watersheds will be significantly costlier to achieve similar N and P reductions for Lake Okeechobee.

The Upper Kissimmee cost-effectiveness value is high because its relative percentage of urban lands to agricultural lands is high, i.e., urban BMPs are costlier and typically have lower P concentrations entering them reducing their efficacy, plus Lake Kissimmee and the other upper chain of lakes buffer the N and P concentrations toward their background levels as water passes through them. This buffering effect reduces nutrient load reduction benefits downstream of the lakes including Lake Okeechobee. The WSW cost-effectiveness value is high because it is being calculated based on N and P load reductions going into Lake Okeechobee, which are small since the majority of discharge from the sub-watershed is west to the Caloosahatchee River. The ESW and SSW sub-watersheds also have high cost-effectiveness values for the same reason, but not to the extent seen for WSW. Therefore, it is recommended that future BMPs programs for these three southern sub-watersheds be developed based on their net nutrient discharges, not just their impact on Lake Okeechobee. Clearly an optimized combination of the various BMPs based on their source cell cost-effectiveness values found in **Appendix A** and combined with their relative location within the Lake Okeechobee will provide the best overall cost efficiency for BMP abatement programs.

WAM estimated nutrient loads reductions resulting from implementation of the LOWCP and LOWRP projects (scenarios 3a and 3b) and KKR (scenario 4) at representative discharge points from the projects as well as their costs and cost-effectiveness values are provided in **Table 43**. The P load reductions at

Lake Okeechobee for the projects will depend on the flow distance and canal conditions between the projects' outlet and Lake Okeechobee. Scenario 3a includes seven LOWCP projects (counting LRSTA and BRSTA separately) across four sub-watersheds, while scenario 3b includes the LOWRP WAF/ASR and Paradise Run wetlands projects in the Indian Prairie sub-watershed and Kissimmee River Center wetlands project in the Lower Kissimmee River sub-watershed. With the exception of the RMWRP project in the Upper Kissimmee sub-watershed, there are either large canals connecting these projects to the Lake Okeechobee or they are located next to the lake resulting in minimal additional attenuation of the reported P load reduction in **Table 43**. The P load reduction for RMWRP is presented for the reduction at the S65 structure (outflow of Lake Kissimmee) and therefore its P load reductions at Lake Okeechobee would likely be considerably less due to the KRR project. Even though the UBI projects are also located above S65, their influence at Lake Okeechobee were represented because they were included in the KRR simulations. Note the UBI are represented in WAM by boundary inflow data provided by the SFWMD with the flows at the S-65 structure.

As can be seen in **Table 43**, the net P load reductions for the LOWCP were found to be about 36 mt/yr while the LOWRP K-05 WAF/ASR/wetlands projects were estimated to provide about 10.1 mt/y P load reduction. The cost-effectiveness of these projects ranges from \$98/lb-P (BRSTA) to \$6,525/lb-P (K-05 WAF/ASR). Collectively the KRR and UBI projects were estimated to provide about 31.8 mt/y at a cost-effectiveness of \$987/lb-P.

In conclusion, from a cost-effectiveness standpoint, the priority for the implementation of the abatement strategies would be in the following order: agricultural BMPs, LOWCP, KRR, urban BMPs, and the LOWRP (K-05 WAF/ASR). However, due to the level of P load reduction required to meet the BMAP target, all of the evaluated scenarios will need to be implemented. It is fortunate that many of the projects are either already in place or are planned to be constructed in the near future. Unfortunately, even additional projects will be needed.

1981 **Table 43. Estimated nutrient load reductions and costs for Scenarios 3 and 4.**

Scenarios/Project Name	P Load Reduction Project Outlet (mt/y)	Project Capital Cost (\$)	Amortized Annual Capital Cost ¹ (\$/y)	Annual O&M Cost (\$/y)	Cost Benefit P (\$/lb)
Scenario 3a - LOWCP Projects					
Fisheating Creek (FAVT)	1.7	3,000,000	261,554	2,573,000	758 ²
BRSTA	13.5	30,368,000	2,647,621	254,307	98
Brighton Valley (DWM)	4.6	12,182,509	1,062,127	3,050,000	406
Istokpoga Marsh (Phase 2)	0.58	14,837,146	1,293,570	258,714 ³	1,217
LRSTA	8.6	132,693,651	11,568,837	334,615	629
Nicodemus Slough	5.9	4,989,653	435,021	2,844,848	253
Rolling Meadows	1.5	5,141,792 ⁴	448,285	100,706	166
Scenario 3b - LOWRP Projects					
K-05 WAF/ASR/Wetlands	10.1	1,437,000,000	125,284,208	15,401,000	6,350
Scenario 4 - Kissimmee River Restoration					
Restored River Hydrography					
Restored Wetlands	31.8	786,994,000	68,613,723	433,288	987
Upper Kissimmee Initiatives					
¹ Amortized annual capital cost assumed 20-year project life and 6% interest. ² Reported as \$136/lb-P in project report. ³ O&M cost not reported, so was assumed to be 20% of annual capital cost. ⁴ Reduction reported at S-65, the reduction at Lake Okeechobee would likely be less.					

1982

1983

1984

7 References

- 1985
- 1986 Bottcher, A.B. and F.T. Izuno, 1994. Everglades Agricultural Area (EAA) – Water, Soil, Crops, and
1987 Environmental Management. University Press of Florida. Gainesville, FL
- 1988 Bottcher, A.B., B.J. Whiteley, A.I. James, and J.G. Hiscock, 2012. Watershed assessment model (WAM):
1989 model use, calibration, and validation. Transactions of the ASABE 55, 1367–1383.
- 1990 Bousquin, S. G., D. H. Anderson, G. E. Williams, and D. J. Colangelo, 2005. Establishing a Baseline: Pre-
1991 restoration Studies of the Channelized Kissimmee River. SFWMD Technical Publication ERA 432.
1992 South Florida Water Management District, West Palm Beach, FL.
- 1993 CDM Smith, 2018. Operations Plan. Lakeside Ranch Stormwater Treatment Area. Work order No.
1994 CN9600003007-WO11. Prepared for the South Florida Water Management District.
- 1995 FDACS. 2014. BMP Research Publication Spreadsheet Summary. Florida Department of Agriculture
1996 and Consumer Services. Office of Agricultural Water Policy. Tallahassee, FL.
- 1997 FDEP, 2014. Basin Management Action Plan for the Implementation of Total Maximum Daily Loads for
1998 Total Phosphorus by the Florida Department of Environmental Protection in Lake Okeechobee.
1999 FDEP. Tallahassee, FL.
- 2000 Jones, B.L., 2005. Water Quality in the Channelized Kissimmee River. Chapter 5 in Bousquin et al.,
2001 *Establishing a Baseline: Pre-restoration Studies of the Channelized Kissimmee River*. SFWMD
2002 Technical Publication ERA 432. South Florida Water Management District, West Palm Beach, FL.
- 2003 Koebel, J.W. and S.G. Bousquin, 2014. The Kissimmee River Restoration Project and Evaluation
2004 Program, Florida, U.S.A. Restoration Ecology – The Journal of the Society for Ecological
2005 Restoration.
- 2006 Koebel, J.W., S. G. Bousquin, D. H. Anderson, M. D. Cheek, C. Carroll, H. Chen, C. Hanlon, Z. Welch,
2007 B. C. Anderson, L. Spencer, T. Beck, and A. Brunell, 2018. Kissimmee River Restoration and Basin
2008 Initiatives. 2019 South Florida Environmental Report, Volume I, Chapter 9.
- 2009 SFWMD, 2006. Kissimmee River Restoration Studies: Evaluation Program Executive Summary. South
2010 Florida Water Management District, West Palm Beach, FL.
- 2011 SFWMD, 2014a. Draft Nicodemus Slough Project Preliminary Operations and Monitoring Plan. 2014-08-
2012 27.
- 2013 SFWMD, 2014b. Operational Plan for Rolling Meadows Parcel B Wetland Restoration. December 2014.
- 2014 SFWMD, 2014c. Chapter 8: Lake Okeechobee Watershed Protection Program Annual and Three-Year
2015 Update. South Florida Environmental Report. SFWMD. West Palm Beach, FL
- 2016 SFWMD, 2016. Land Feature Class. South Florida Water Management District Geospatial Services, 2016
- 2017 SFWMD, 2018a. Draft 2019 SFER. Volume 1, Chapter 8A.
- 2018 SFWMD, 2018b. APPENDIX I Brighton Valley Draft Operations Plan.

2019 SFWMD, 2018c. Operational Plan for Rolling Meadows Parcel B Wetland Restoration. December 2014.
2020 Updated March 2018.

2021 SFWMD, FDEP, and FDACS, 2007. Lake Okeechobee Protection Program. Lake Okeechobee Protection
2022 Plan Evaluation Report. South Florida Water Management District, West Palm Beach, FL; Florida
2023 Department of Environmental Protection, Tallahassee, FL; and Florida Department of Agriculture and
2024 Consumer Services, Tallahassee, FL.

2025 SWET, 2006. Letter report entitled: Phosphorus reduction performance and implementation costs under
2026 BMPs and technologies in the Lake Okeechobee protection plan area, Soil and Water Engineering
2027 Technology, Inc., Gainesville, FL

2028 SWET, 2008. Final report: Nutrient loading rates, reduction factors and implementation costs associated
2029 with BMPs and technologies, Soil and Water Engineering Technology, Inc., Gainesville, FL.

2030 SWET, 2012. Operational Protocols for Istokpoga Marsh Watershed Improvement District (IMWID)
2031 Water Retention and Tailwater Recovery Project. Submitted to Chastain Skillman.

2032 SWET, 2016. Estimation of Total Phosphorus and Nitrogen Loads Reductions Associated with FDACS
2033 Lake Okeechobee Cost-Share BMP Program. Tasks 1 and 2. Prepared for the Florida Department of
2034 Agriculture and Consumer Services, Tallahassee, FL.

2035 SWET, 2017a. Watershed Assessment Model (WAM): Recalibration of the Northern Lake Okeechobee
2036 Basins. Deliverable 1: WAM Recalibration Report. Prepared for the Florida Department of
2037 Agriculture and Consumer Services, Tallahassee, FL. Contract No. 024010.

2038 SWET, 2017b. WAM Calibration for the Lake Okeechobee Watershed. Deliverable #2: Southern Sub-
2039 watersheds Calibration; Deliverable #3: Southern Sub-watersheds Verification; Deliverable #4:
2040 Southern Sub-watersheds Goodness of Fit. Prepared for the Florida Department of Agriculture and
2041 Consumer Services, Tallahassee, FL. Contract No. 024010.

2042 SWET, 2018. Final Pre-Drainage Characterization Report. Deliverable 2.2. Prepared for the South
2043 Florida Water Management District. West Palm Beach, FL. Contract No. Department of Agriculture
2044 and Consumer Services, Tallahassee, FL. Contract No. 4600003809.U.S. Army Corps of Engineers,
2045 2016. Central and Southern Florida Project. Water Control Plan for S-67, S-67X, S-68X, S-83X, S-
2046 84X, S-65C, S-65D, S-65DX1, S-65DX2, and S-65EX1. Jacksonville District, USACE.

2047 U.S. Army Corps of Engineers, 2018a. Lake Okeechobee Watershed Restoration Project. Draft Integrated
2048 Project Implementation Report and Environmental Impact. Jacksonville District, USACE.

2049 U.S. Army Corps of Engineers, 2018b. Lake Okeechobee Watershed Restoration Project. Draft Integrated
2050 Project Implementation Report and Environmental Impact. Annex C, Draft Project Operating Manual.
2051 Jacksonville District, USACE.

2052 Water & Soil Solutions, LLC, 2018. Implementation of Floating Aquatic Vegetative Tilling Treatment
2053 Technology. Deliverable #3: Operations, Optimization & Maintenance Plans. Fisheating Creek &
2054 North Feeder Canal Sites 2 & 3. Prepared for Florida Department of Agricultural and Consumer
2055 Services. Contract # 025321 FY 2018/19.

2056 Williams, G., J.W. Koebel, D.H. Anderson, and J. Jorge. 2005. Kissimmee River Restoration and Upper
2057 Basin Initiatives. Chapter 11 in South Florida Environmental Report Volume I: The South Florida
2058 Environment - Water Year 2004. South Florida Water Management District, West Palm Beach, FL.

2059 **8 Appendix A: Scenarios 1 and 2 P Load Reductions and**
2060 **Cost Effectiveness Factors for Individual BMPs**
2061 **Implemented on Three Representative Soils**

2062 Table A.1. BMP Land Use Description, LUCodes, Implementation Costs, N&P Reductions at Edge of Parcel, and Cost Effectiveness for Scenarios 1 and 2

2063

Land Use Description	LUCode	Implementation	Well Drained - Ridge 'Archbold' Soil ²			Poorly Drained - Flatwood 'Immokalee' Soil						Poorly Drained - Muck 'Terra Ceia' Soil					
Special Note: Orange Represent the "Base" Run, Green Represents Scenario 1 - Nutrient BMPs, and Blue Represents Scenario 2 - Water BMPs		Cost ¹ \$/ac/yr	Nitrogen Load	Nitrogen Reduction	Cost Effectiveness	Nitrogen Load	Phosphorus Load	Nitrogen Reduction	Phosphorus Reduction	Cost Effectiveness		Nitrogen Load	Phosphorus Load	Nitrogen Reduction	Phosphorus Reduction	Cost Effectiveness	
			lbs/ac/yr	%	Nitrogen \$/lb-removed					Nitrogen \$/lb-removed	Phosphorus \$/lb-removed					Nitrogen \$/lb-removed	Phosphorus \$/lb-removed
Low Density Residential [Low Density Residential]	2	-	20.1	-	-	29.2	0.44	-	-	-	-	27.0	2.46	-	-	-	-
Low Density Residential (storm retention BMP) [Low Density Residential]	102	2500	20.0	0.76	16459	29.0	0.44	0.42	0.92	20325	611423	26.6	2.33	1.41	5.02	6578	20253
Low Density Residential (fertility BMP) [Low Density Residential]	302	10	18.8	6.61	8	26.9	0.41	7.74	8.79	4	256	25.8	1.74	4.22	29.05	9	14
Commercial and Services [Commercial and Services]	3	-	11.6	-	-	14.9	1.05	-	-	-	-	14.1	1.79	-	-	-	-
Commercial and Services (storm retention BMP) [Commercial and Services]	103	2500	7.6	34.01	635	14.1	1.04	5.48	1.51	3061	157650	13.2	1.72	6.50	4.08	2729	34176
Coniferous Plantations [Coniferous Plantations]	8	-	2.5	-	-	6.5	0.47	-	-	-	-	7.5	2.89	-	-	-	-
Coniferous Plantations (fertility) [Coniferous Plantations]	108	5	1.8	27.73	7	6.3	0.26	1.93	45.38	40	23	7.4	2.48	0.78	14.08	86	12
Coniferous Plantations (non-clear cut BMP) [Coniferous Plantations]	208	100	0.1	95.28	41	6.1	0.34	6.09	27.77	254	764	7.1	2.55	5.44	11.70	246	296
Barren Land [Barren Land]	17	-	0.6	-	-	2.2	1.18	-	-	-	-	14.2	2.22	-	-	-	-
Barren Land (retention) [Barren Land]	117	2500	0.1	76.71	5168	1.9	1.08	15.08	8.07	7415	26265	12.5	1.95	12.27	12.20	1436	9230
Transportation Corridors [Transportation Corridors]	18	-	3.0	-	-	8.2	0.09	-	-	-	-	7.5	0.93	-	-	-	-
Transportation Corridors (storm retention BMP) [Transportation Corridors]	118	2000	1.5	49.24	1376	7.9	0.08	4.73	10.98	5132	210684	6.9	0.84	7.86	9.90	3414	21766
Medium Density Residential [Medium Density Residential]	19	-	21.7	-	-	22.7	1.35	-	-	-	-	20.9	3.05	-	-	-	-
Medium Density Residential (storm retention BMP) [Medium Density Residential]	119	3000	20.8	4.18	3305	22.5	1.34	1.24	0.55	10663	401020	20.4	2.93	2.40	3.87	5981	25464
Medium Density Residential (fertility BMP) [Medium Density Residential]	419	3000	16.4	24.19	572	22.1	1.32	3.00	2.47	4401	89909	20.5	2.45	1.73	19.73	8310	4990
High Density Residential (Offsite Sewage Deposal) [High Density Residential]	20	-	13.5	-	-	10.0	0.12	-	-	-	-	8.4	1.60	-	-	-	-
High Density Residential (Offsite Tertiary and storm retention BMP) [High Density Residential]	120	-	12.1	10.64	-	9.6	0.11	3.97	8.30	-	-	7.8	1.49	7.06	7.18	-	-
High Density Residential (Offsite Tertiary and fertility BMP) [High Density Residential]	520	100	7.5	44.52	17	8.5	0.09	15.20	24.02	65	3448	7.7	1.08	9.17	32.79	129	190
Multiple Dwelling Units (Offsite Sewage Treatment) [Multiple Dwelling Units]	21	-	13.1	-	-	9.6	0.15	-	-	-	-	8.4	1.21	-	-	-	-
Multiple Dwelling Units (Offsite Sewage Treatment and Fertility/storm retention BMPs)	121	3000	6.5	50.32	456	9.0	0.14	6.12	8.72	5123	232148	7.7	1.12	8.64	7.51	4130	32954
Industrial [Industrial]	22	-	11.4	-	-	14.9	1.05	-	-	-	-	14.1	1.79	-	-	-	-
Industrial (storm retention BMP) [Industrial]	122	3000	7.4	34.57	762	14.1	1.04	5.48	1.51	3673	189180	13.2	1.72	6.50	4.08	3275	41012
Row Crops [Row Crops]	25	-	73.2	-	-	92.5	2.84	-	-	-	-	83.8	13.65	-	-	-	-
Row Crops (fertility BMP) [Row Crops]	125	15	54.4	25.69	1	73.8	2.46	20.18	13.46	0.8	39	67.0	12.14	20.00	11.05	0.9	10
Row Crops (Increased retention BMP) [Row Crops]	225	150	73.0	0.24	844	91.3	2.64	1.29	7.09	126	745	82.5	12.42	1.56	9.02	114	122
Improved Pasture [Improved Pasture]	26	-	33.1	-	-	8.4	1.37	-	-	-	-	9.2	4.75	-	-	-	-
Improved Pasture (fertility BMP) [Improved Pasture]	126	2	27.7	16.21	0	7.5	1.27	10.81	7.21	2	20	8.8	4.65	4.19	2.17	5	19
Improved Pasture (Retention BMP) [Improved Pasture]	426	25	31.6	4.40	17	4.3	0.53	48.41	61.30	6	30	3.8	1.21	59.20	74.48	5	7
Unimproved Pasture [Unimproved Pasture]	27	-	1.2	-	-	6.9	0.84	-	-	-	-	8.8	3.00	-	-	-	-
Unimproved Pasture (fertility BMP) [Unimproved Pasture]	127	1	1.0	20.04	4	6.5	0.83	6.40	1.22	2	97	8.5	2.99	2.81	0.35	4	97
FDACS Unimproved Pasture - Drainage and Water Control Structures (2 and 16) [Unimproved Pasture]	427	125	1.0	19.12	532	5.5	0.66	20.89	21.51	86	689	6.4	2.27	26.81	24.28	53	172
Woodland Pasture [Woodland Pasture]	28	-	0.8	-	-	6.4	0.75	-	-	-	-	8.3	2.75	-	-	-	-
Woodland Pasture (fertility BMP) [Woodland Pasture]	128	1	0.5	35.73	3	5.5	0.74	14.79	1.57	1	85	7.8	2.73	5.94	0.65	2	56
Woodland Pasture - Drain and Water Control Structures (2 and 16) [Woodland Pasture]	528	25	0.7	19.02	156	5.0	0.45	21.84	39.34	18	85	6.0	1.90	27.76	30.88	11	29
Peach and Pecan Orchards [Groves and Orchards]	30	-	24.4	-	-	23.5	7.02	-	-	-	-	26.6	17.52	-	-	-	-
Peach and Pecan Orchards (fertility BMP) [Groves and Orchards]	130	10	11.2	54.07	1	21.2	3.55	10.00	49.42	4	3	24.6	10.76	7.60	38.60	5	1
Blueberries [Blueberries]	31	-	4.0	-	-	0.5	0.15	-	-	-	-	1.2	1.93	-	-	-	-
Blueberries (fertility BMP) [Blueberries]	131	10	0.8	80.37	3	0.1	0.08	71.24	46.16	28	143	#N/A	#N/A	#N/A	#N/A	-	-
Cattle Feeding Operations [Cattle Feeding Operations]	32	-	100.7	-	-	18.7	9.79	-	-	-	-	24.2	25.03	-	-	-	-
Cattle Feeding Operations (waste balance BMP) [Cattle Feeding Operations]	132	250	80.5	20.00	12	14.9	7.84	20.00	20.00	67	128	19.4	20.02	20.00	20.00	52	50
FDACS Cattle Feeding Operations with Water Control Structures - 16 [Cattle Feeding Operations]	332	1100	90.5	10.13	108	16.7	7.29	10.82	25.60	544	439	21.0	19.29	13.04	22.91	349	192
Poultry Feeding Operations [Poultry Feeding Operations]	33	-	86.0	-	-	7.3	8.94	-	-	-	-	2.0	20.28	-	-	-	-
Poultry Feeding Operations (fertility BMP) [Poultry Feeding Operations]	133	15	77.4	10.00	2	6.5	8.05	10.00	10.00	21	17	1.8	18.25	10.00	10.00	74	7
Poultry Feeding Operations (fertility and waste balance BMP) [Poultry Feeding Operations]	233	250	68.8	20.00	15	5.8	7.15	20.00	20.00	172	140	1.6	16.22	20.01	20.00	613	62
Swine Feeding Operations [Swine Feeding Operations]	34	-	277.7	-	-	50.6	8.12	-	-	-	-	49.6	32.45	-	-	-	-
Swine Feeding Operations (waste balance BMP) [Swine Feeding Operations]	134	15	205.0	26.17	0	37.5	6.30	25.94	22.36	1	8	38.7	24.61	21.87	24.17	1	2
Swine Feeding Operations (low animal density BMP) [Swine Feeding Operations]	234	250	135.8	51.08	2	25.0	4.56	50.70	43.76	10	70	28.4	16.93	42.74	47.81	12	16

2064 Table A.1. Cont'. BMP Land Use Description, LUCodes, Implementation Costs, N&P Reductions at Edge of Parcel, and Cost Effectiveness for Scenarios 1 and 2

Land Use Description	LUCode	Implementation Cost ¹ \$/ac/yr	Well Drained - Ridge 'Archbold' Soil ²			Poorly Drained - Flatwood 'Immokalee' Soil						Poorly Drained - Muck 'Terra Ceia' Soil					
			Nitrogen Load lbs/ac/yr	Nitrogen Reduction %	Cost Effectiveness Nitrogen \$/lb-removed	Nitrogen Load lbs/ac/yr	Phosphorus Load lbs/ac/yr	Nitrogen Reduction %	Phosphorus Reduction %	Cost Effectiveness		Nitrogen Load lbs/ac/yr	Phosphorus Load lbs/ac/yr	Nitrogen Reduction %	Phosphorus Reduction %	Cost Effectiveness	
										Nitrogen \$/lb-removed	Phosphorus \$/lb-removed					Nitrogen \$/lb-removed	Phosphorus \$/lb-removed
Tree Nurseries [Tree Nurseries] ¹	35	-	23.1	-	-	26.9	2.84	-	-	-	-	24.5	8.98	-	-	-	-
Tree Nurseries (Fertility BMP) [Tree Nurseries] ¹	135	20	12.9	44.15	2	24.7	1.27	8.12	55.29	9	13	22.8	5.82	7.19	35.19	11	6
Tree Nurseries (Fertility and Storm Retention BMPs) [Tree Nurseries] ¹	235	250	12.9	44.15	25	1.1	0.09	96.02	96.78	10	91	0.6	1.84	97.59	79.51	10	35
Sod Farms [Sod Farms] ¹	36	-	103.4	-	-	5.0	3.43	-	-	-	-	7.4	10.52	-	-	-	-
Sod Farms (Fertility BMP) [Sod Farms] ¹	136	30	64.7	37.45	1	5.0	1.83	0.01	46.71	51437	19	7.4	8.06	0.00	23.35	403443	12
Sod Farms (Retention BMP) [Sod Farms] ¹	236	45	103.4	0.01	4351	4.9	3.29	2.36	4.22	378	311	6.8	10.00	8.57	4.95	71	86
Ornamental Nurseries [Ornamental Nurseries] ¹	37	-	62.9	-	-	9.2	0.74	-	-	-	-	8.7	1.63	-	-	-	-
Ornamental Nurseries (Fertility BMP) [Ornamental Nurseries] ¹	137	15	43.5	30.92	1	8.5	0.47	7.76	36.73	21	55	8.3	1.12	5.20	31.32	33	29
Ornamental Nurseries (Retention BMP) [Ornamental Nurseries] ¹	237	100	59.9	4.78	33	6.1	0.53	34.46	28.39	31	475	6.2	0.83	29.52	49.32	39	124
Horse Farms [Horse Farms] ¹	38	-	68.8	-	-	13.7	0.76	-	-	-	-	11.8	4.64	-	-	-	-
Horse Farms (Fertility BMP) [Horse Farms] ¹	138	5	56.1	18.47	0	12.1	0.71	11.85	6.07	3	108	11.0	4.59	6.93	1.08	6	99
FDACS Horse Farms with Water Control Structures -16 [Horse Farms] ¹	338	45	61.9	10.00	7	11.8	0.59	13.86	21.74	24	272	9.5	3.52	19.56	24.07	20	40
Dairies [Dairies] ¹	39	-	146.4	-	-	15.4	1.89	-	-	-	-	13.4	12.99	-	-	-	-
Dairies (Fertility BMP) [Dairies] ¹	139	15	114.0	22.14	0	12.3	1.51	20.00	20.01	5	40	10.7	10.39	20.00	20.00	6	6
FDACS Dairy with Drainage and Water Control Structures (2 and 16) [Dairies] ¹	739	350	118.5	19.00	13	12.1	1.35	21.41	28.82	106	641	10.5	9.12	21.65	29.82	121	90
Kennels [Kennels] ¹	40	-	166.8	-	-	21.7	1.00	-	-	-	-	11.1	3.35	-	-	-	-
Kennels (waste balance BMP) [Kennels] ¹	140	50	108.4	35.01	1	12.8	0.81	41.03	19.32	6	258	5.3	2.57	52.75	23.38	9	64
Kennels (low animal density BMP) [Kennels] ¹	240	50	69.9	58.10	1	6.1	0.63	71.73	37.47	3	133	3.2	1.81	70.99	46.01	6	32
Field Crops [Field Crops] ¹	62	-	28.5	-	-	15.4	1.23	-	-	-	-	15.0	6.04	-	-	-	-
Field Crops - Fertility BMP [Field Crops] ¹	162	5	19.8	30.65	1	13.3	0.95	13.15	23.09	2	18	13.9	5.56	7.87	7.96	4	10
Field Crops - (Retention BMP) [Field Crops] ¹	262	75	28.4	0.45	582	15.1	1.19	1.52	3.00	321	2028	14.0	5.71	6.88	5.51	73	225
Sugar Cane [Sugar Cane] ¹	68	-	14.8	-	-	8.7	1.10	-	-	-	-	9.7	5.12	-	-	-	-
FDACS Sugar Cane with Water Control Structures -16 [Sugar Cane] ¹	168	50	13.3	10.00	34	7.5	0.80	13.97	27.19	41	167	7.8	3.38	19.72	33.98	26	29
Sugar Cane (Fertility BMP) [Sugar Cane] ¹	268	30	6.4	56.91	4	8.7	0.70	0.03	36.52	12851	74	9.7	4.25	0.00	17.04	3089670	34
Ranchettes [Ranchettes] ¹	71	-	3.5	-	-	3.6	0.48	-	-	-	-	4.4	1.77	-	-	-	-
Ranchettes Fertility BMP [Ranchettes] ¹	171	2	3.5	1.38	41	3.6	0.50	0.28	-6.04	198	-70	4.4	1.41	0.00	20.68	45526	5
Prisons [Prisons] ¹	72	-	16.4	-	-	15.4	2.50	-	-	-	-	15.6	3.44	-	-	-	-
Prisons (storm retention BMP) [Prisons] ¹	172	2000	15.8	3.70	3304	15.2	2.50	1.24	0.25	10463	325747	15.3	3.43	1.65	0.49	7777	117418
Citrus [Citrus Groves] ¹	84	-	59.8	-	-	24.7	1.96	-	-	-	-	27.2	9.15	-	-	-	-
Citrus (Fertility BMP) [Citrus Groves] ¹	184	15	57.3	4.21	6	22.4	0.72	9.13	63.23	7	12	24.9	6.28	8.28	31.37	7	5
Citrus (Retention and Irrigation BMP) [Citrus Groves] ¹	484	150	59.1	1.14	220	24.5	1.91	0.91	2.91	670	2624	26.0	8.84	4.31	3.43	128	478
Intensive Pasture [Intensive Pasture] ¹	85	-	104.1	-	-	28.1	1.87	-	-	-	-	33.6	9.04	-	-	-	-
Intensive Pasture Fertilizer BMP [Intensive Pasture] ¹	185	10	57.8	44.50	0	22.5	1.24	20.00	33.55	2	16	26.9	6.97	20.00	22.84	1	5
FDACS Intensive Pasture with Dairy and Drainage Improvements -3 [Intensive Pasture] ¹	585	150	65.6	37.01	4	17.0	1.01	39.47	46.09	14	174	19.5	4.73	42.03	47.62	11	35
Field Crops - Dairy Sprayfield [Spray Fields] ¹	86	-	17.3	-	-	22.8	2.84	-	-	-	-	22.9	13.58	-	-	-	-
Field Crops - Dairy Sprayfield BMP [Nutrient Balance] ¹	186	35	13.9	20.00	10	18.3	2.27	20.00	20.00	8	62	18.3	10.87	20.00	20.00	8	13
FDACS Field Crops (Dairy Sprayfield) Drain and Water Control Structures -18 [Spray Fields] ¹	586	350	14.0	19.39	104	17.7	1.96	22.31	31.15	69	396	16.8	9.11	26.51	32.94	58	78
Dairy High Intensity Area - Untreated [Dairies HIA - Untreated] ¹	87	-	359.6	-	-	45.7	20.23	-	-	-	-	15.8	33.25	-	-	-	-
FDACS Dairy High Intensity Area - Dairy and Drainage Improvements -3 [Dairies HIA - Untreated] ¹	487	350	226.4	37.02	3	28.5	9.98	37.70	50.68	20	34	8.9	16.91	43.86	49.16	50	21
Abandoned Dairies [Abandoned Dairies] ¹	89	-	1.8	-	-	10.9	1.98	-	-	-	-	15.2	8.93	-	-	-	-
FDACS Abandoned Dairies with Drainage and Water Control Structures -18 [Abandoned Dairies] ¹	389	300	1.2	32.76	505	8.4	1.37	23.50	30.96	117	488	10.8	5.95	29.30	33.32	67	101
Outer Pasture [Dairy Boundary Pastures] ¹	90	-	170.0	-	-	37.8	13.99	-	-	-	-	35.4	28.19	-	-	-	-
Intensive Pasture Fertilizer BMP [Dairy Boundary Pastures] ¹	190	20	77.3	54.56	0	30.3	11.19	20.00	20.00	3	7	28.4	22.55	20.00	20.00	3	4
FDACS Outer Pasture with Dairy and Drainage Improvements -3 [Dairy Boundary Pastures] ¹	490	250	35.0	79.39	2	22.9	7.48	39.43	46.50	17	38	20.6	14.74	41.77	47.72	17	19

¹ The annual cost of the BMP is the sum of its amortized annual capital cost calculated using its life expectancy (ranging from 10 to 30 years) using an interest rate of 6%, which is then added to its annual O&M cost.

