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Assessment of Flood Protection Level of Service for Current (2020) and Future (2070) Conditions, Upper Kissimmee Basin

I, Guillermo A. Regalado, P.E, hereby certify that the information contained in this evaluation is true and correct to the best of my knowledge, was based upon available information and the information provided and approved by the South Florida Water Management District and was prepared in accordance with sound engineering principles.

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Vice President

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Date: 06/05/2025

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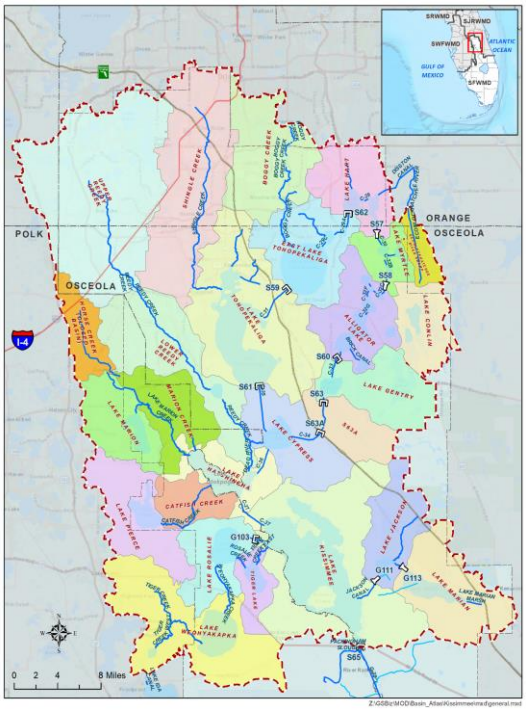

Upper Kissimmee Basin Flood Protection Level of Service

Task 6: Assessment of Flood Protection Level of Service for Current (2020) and Future (2070) Conditions

May 30, 2025

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List of Acronyms

Abbreviation	Definition
AG	Agriculture
APT	Aquifer Performance Test
AWC	Available Water Capacity
C&SF	Central and Southern Florida Flood Control
CERP	Central Everglades Restoration Project
CF	Controlled-Free flow regime
CII	Commercial / Industrial / Institutional
CFWI	Central Florida Water Initiative
COL	Chain of Lakes
CS	Controlled-Submerged flow regime
CSM	Cubic feet per second per square mile

Abbreviation	Definition
DCIA	Directly connected impervious areas
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
DSS	Domestic Self-Supply
DWM	Dispersed Water Management
ECFTX	East-Central Florida Transient Expanded
EMT	Emergency Modeling Team (SFWMD)
FEMA FIMA NFIP	Federal Emergency Management Agency Federal Insurance and Mitigation Administration National Flood Insurance Program
FAS	Floridan Aquifer System
FC	Field Capacity
FDOT	Florida Department of Transportation
FDACS	Florida Department of Agriculture and Consumer Services
FDEP	Florida Department of Environmental Protection
FLUCCS	Florida Land Use and Land Classification System
FPLOS	Flood Protection Level of Service
FWC	Fish and Wildlife Commission
GPCC	Global Precipitation Climatology Centre
GW	Ground Water
HAT	Hydrologic Analysis Team
H&H	Hydrology and Hydraulics
HRS	Headwaters Revitalization Schedule
IAS	Intermediate Aquifer System
ICU	Intermediate Confining Unit
KB	Kissimmee Basin
KBMOS	Kissimmee Basin Modeling and Operations Study
KCH or K-C-H	Lake Kissimmee, Cypress and Hatchineha
KCOL	Kissimmee Chain of Lakes

Abbreviation	Definition
KOE	Kissimmee-Okeechobee-Everglades
KRHRP	Headwaters Revitalization Project
KRRP	Kissimmee River Restoration Project
K_{Sat}	Hydraulic Conductivity at soil saturation
L/R	Landscape / Recreational
LCLU	Land Cover and Land Use
LKB	Lower Kissimmee Basin
LMA	Lake Management Area
LSM	Local-Scale Model
LTMP	Kissimmee Chain of Lakes Long-Term Management Plan
MA	Management Area
MAE	Mean Absolute Error
MAGO	Maximum Allowable Gate Opening
ME	Mean Error
MGD	million gallons per day
MGY	Million gallons per year
MPHD	Maximum Permissible Head Differential
NAVD88 or NAVD	North American Vertical Datum of 1988
NEXRAD	Next Generation Radar
NGVD29 or NGVD	National Geodetic Vertical Datum of 1929
NRCS	Natural Resources Conservation Service
NS	Nash-Sutcliffe
OC	Open Channel Flow
OL	Overland Flow
PAF	Paved Area Fraction
PET	Potential Evapotranspiration
PG	Power Generation

Abbreviation	Definition
POC	Period of Calibration
POS	Period of Simulation
PS	Public Supply
R	Correlation
RC	Runoff Coefficient
RCID	Reedy Creek Improvement District
RET	Reference Evapotranspiration
RIB	Rapid Infiltration Basin
RMSE	Root Mean Square Error
SA	Surficial Aquifer
SJRWMD	St. Johns River Water Management District
SFWMD	South Florida Water Management District
sq.mi	Square Miles
SWFWMD	Southwest Florida Water Management District
SD	Standard Deviation
SPT	Standard Penetration Test
SW	Surface Water
SZ	Saturated Zone
UFA	Upper Floridan Aquifer
UFA-upper	Upper Floridan Aquifer – upper permeable zone
UKB	Upper Kissimmee Basin
US	Uncontrolled-Submerged flow regime
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Survey
UZ	Unsaturated Zone
WMA	Water Management Area
WMD	Water Management District

Abbreviation	Definition
WP	Wilting Point
WRL	Water Reservation Line

Referenced Models

Model	Explanation
UKB FPLOS Model	Upper Kissimmee Basin Flood Protection Level of Service model – refers to any of the below versions
UKB FPLOS Calibration Model	Upper Kissimmee Basin Flood Protection Level of Service calibration model, representing Hurricane Irma
UKB FPLOS Validation Model	Upper Kissimmee Basin Flood Protection Level of Service validation model, representing October 2011 No-Name storm
UKB FPLOS Current Conditions Model	Upper Kissimmee Basin Flood Protection Level of Service base conditions model representing 2020 conditions
UKB FPLOS Future Conditions Model	Upper Kissimmee Basin Flood Protection Level of Service base conditions model representing 2070 conditions
ECFTX	East Coast Florida Transient Expanded model

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Executive Summary

The South Florida Water Management District (SFWMD) is conducting a review of their regional water management infrastructure to determine the Flood Protection Level of Service (FPLOS) provided by existing SFWMD operated infrastructure under current and future conditions in the Upper Kissimmee Basin FPLOS (UKB FPLOS). Modeling and analyses from this project can be used by SFWMD, Southwest Florida Water Management District (SWFWMD), St. Johns River Water Management District (SJRWMD), local, state, and federal agencies to identify areas where improvements to the design, retrofit, construction and/or operation of water management facilities are needed or would be beneficial.

The Kissimmee Basin (KB), consisting of UKB and Lower Kissimmee Basin (LKB) includes diverse ecosystems and the Kissimmee River and its floodplain. The basin forms the headwaters of Florida's Kissimmee-Okeechobee-Everglades (KOE) ecosystem. The Upper Kissimmee Basin (UKB) is comprised of the Kissimmee Chain of Lakes (KCOL) and the Econlockahatchee, Reedy, Boggy, Shingle and Catfish creek watersheds. KCOL discharges into the Kissimmee River through the S65 structure at the south end of Lake Kissimmee.

Performance Metrics: SFWMD's FPLOS Program developed six (6) performance metrics to quantify the LOS provided under the current and future conditions. The performance of the LMAs using a suite of six (6) performance metrics (PMs) were evaluated for each design storm simulation. The details of each PM are defined as follows:

- PM #1: Maximum stage in primary canals/lakes.
- PM #2: Maximum daily discharge capacity through the primary canals/lakes.
- PM #3: Structure flow performance
- PM #4-UKB: Peak storm runoff - maximum conveyance capacity of the watershed when limited by downstream flooding conditions
- PM #5: Frequency of flooding – stage-based LOS for LMAs.
- PM #6: Duration of flooding

UKB FPLOS PM #4 is different from other FPLOS PMs, since this is an inland watersheds assessment.

Software Selection: The process of model software selection in UKB FPLOS was comprehensive. The criteria used were applicability considering the hydrology of the Kissimmee Basin and the project objectives: software's proven track record, functionality, cost-effectiveness, SFWMD in-house skills, credibility and availability of technical support and training. Based on that MIKE SHE and MIKE HYDRO was selected as the software. The version of software used for UKB FPLOS is DHI 2022 Update 1.

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Storm Events: A review of the storm events that occurred between 2000 and 2020 was conducted and two of the most recent events were selected for calibration and validation: the calibration event is Hurricane Irma which occurred on September 9-11, 2017, and an additional validation storm event which occurred on October 8-11, 2011. The storm events selected for calibration and validation used a ranking methodology based on multiple criteria such as rainfall volume within 72-hr period, peak intensity, peak stages and flows within primary canals and structures, watershed inflows, uniformity over the study area, NOAA rainfall classification and chronology of the event relative to current conditions.

Model Development: The UKB FPLOS Model Study Area consists of 26 watersheds in the UKB and is approximately 1600 sq.mi. The UKB FPLOS Model domain is approximately 3560 sq.mi and establishes hydrology in the study area and reduces boundary effects into the study area by representing surface and subsurface exchanges of water that occur across the UKB watershed boundary. Conceptually, the model includes four (4) subdomains: OL – Two-dimensional Overland Flow, UZ – One dimensional flow in the unsaturated zone, SZ – Three-dimensional flow in the saturated zone and OC – Open Channel and Hydraulics (surface water network and structures). The model domain spans three (3) water management districts (WMD). Spatial datasets at 328 ft (100 m) resolution were developed to represent the distributed properties of the model. East Coast Florida Transient Expanded model (ECFTX) developed under Central Florida Water Initiative (CFWI) was the basis for the hydrostratigraphy and saturated zone properties of the Surficial and Upper Floridan aquifers (CFWI, Hydrologic Analysis Team (HAT), 2020).

Model Calibration and Validation: Statistical parameters were calculated based on defined primary and secondary calibration and validation targets. SFWMD FPLOS metrics require adequate match at surface water stages and flows, groundwater levels, storm peak elevations and discharges across the UKB FPLOS Study Area. For each monitoring station with observed data within this area, a set of statistical measures of goodness-of-fit were calculated. Primary calibration targets, in general, are located on the SFWMD primary conveyance system, have adequate observed data for the period of simulation, represent the regional hydrology and determine the outcome of the project objectives. Secondary stations guide the calibration but are not on SFWMD primary conveyance system and/or are more representative of local hydrology.

UKCOL hydrology is one-of-its-kind; historically, water flowed through the system freely by gravity. SFWMD management of this is unique and is best described by the concept of Lake Management Area (LMA) which has water control structures upstream and downstream of this drainage area. The model calibration was assessed in 3 different ways:

1. Regional Level assessment: Hydraulic gradient through the system
2. Sub-regional level assessment: Calibration of the LMA
3. Watershed level assessment: Individual stations

The first criterion assessed the performance of the model at the regional level – if the gradient through the system and the changes in the gradient as the storm progressed, is captured in the model, as in observed data.

The second criterion evaluated the model at sub-regional level. The following are the sub-criteria/guidelines that were used to assess if a LMA is reasonably calibrated:

1. Upstream structure flows are within the criterion (+/- 20% of observed flows) during at least 50% of the evaluation period
2. Downstream structure flows are within the criterion (+/- 20% of observed flows) during at least 50% of the evaluation period
3. Upstream structure headwater stage within 0.5 ft mean error (ME) during the evaluation period
4. Upstream structure tailwater stage within 0.5 ft ME during the evaluation period
5. Downstream structure headwater stage within 0.5 ft mean error (ME) during the evaluation period
6. Downstream structure tailwater stage within 0.5 ft ME during the evaluation period

The third criterion assessed the model performance at watershed level – statistics of stages, flows and cumulative flows at individual stations were used for this.

At the regional level, the gradient to Kissimmee-Cypress-Hatchineha system during Hurricane Irma was approximately 1.5 ft, and this was captured well in the calibration model. At the sub-regional level, 3 of the 6 sub-criteria were met at all LMAs except Gentry LMA. At the watershed level, 62% of surface water stations and 20% of groundwater stations met the criterion of +/- 0.5 ft and 87% of surface water stations and 57% of groundwater stations met the criterion of +/- 0.75 ft and +/- 1 ft respectively. For flows during Hurricane Irma, all structures except S60 and S63 simulate flows that are within 20% of the observed flows, more than 50% of the evaluation time. Although S63 does not meet the flows criterion, S63A, which is downstream of S63, does. S63 has the largest difference (2 ft) between observed and simulated stages at peak. As for flows, the largest difference is seen at Boggy Creek with a difference in magnitude of 2000 cfs, peaking 14 hours before the observed peak flows. The water budgets were checked for reasonableness.

The calibrated model was validated with an independent storm event to assess the parametrization and make sure the model is reasonable for application. The validation was assessed with the same criteria as the calibration model. Although the performance of the validation model was not as good as the calibration model, it did not show any bias in the watershed and channel parametrization. The differences were due to the storm event intensity and distribution being different from the one used for calibration.

The UKB FPLOS model was reasonably calibrated and validated for application of flood protection level of service assessment. The calibrated model was used to perform sensitivity to the performance measures. This sensitivity was used as a guide to how the current and future conditions models were formulated and applied.

For design rainfall data, NOAA volumes were used with SFWMD design rainfall pattern outlined in SFWMD ERP application manual. For structure operations, regulation schedules and the operating constraints were coded. Initial and boundary conditions were based on Hurricane Irma conditions; and adjusted according to the size of the design storms simulated. There were 4 current and 4 future conditions model implementations: 5-, 10-, 25- and 100-yr storm event models. Modifications for future conditions model also included implementation of change factor (range 1.13 to 1.26 depending on the design storm) for future rainfall, and future SFWMD projects.

FPLOS was determined for each LMA using design storm simulations for both current and future conditions. The LMAs were then evaluated based on the highest design storm event during the simulation that provided sufficient flood protection. The performance of the LMAs using a suite of six (6) performance metrics (PMs) were evaluated for each design storm simulation.

FPLOS rating was assigned to each LMA based on the evaluation. To summarize the overall findings of each LMA, the design storms identified as the LOS for all PMs were aggregated with additional weight given to PM #1, #5, and #6 since they provide a basin-level evaluation of flooding. The overall FPLOS rating is the following: less than or 5-yr FPLOS rating for all LMAs. The FPLOS rating was based not only on the main lake in the LMA, but all the water bodies within it. The tables below summarize the overall FPLOS in all of the UKB LMAs, and the FPLOS rating for PMs 1, 5 and 6.

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Overall FPLOS Rating

Conditions	Lake Alligator LMA	Lake Myrtle LMA	Lake Hart LMA	Lake Gentry LMA	Lake East Toho LMA	Lake Toho LMA	Lake Cypress LMA	Lake Hatchineha LMA	Lake Kissimmee LMA
Current	<5-yr	<5-yr	<5-yr	<5-yr	5-yr	<5-yr	<5-yr	<5-yr	5-yr
Future	<5-yr	<5-yr	<5-yr	<5-yr	5-yr	< 5-yr	< 5-yr	< 5-yr	5-yr

FPLOS Rating for PM 1, 5, 6

Conditions	Performance Metric	Lake Alligator LMA	Lake Myrtle LMA	Lake Hart LMA	Lake Gentry LMA	Lake East Toho LMA	Lake Toho LMA	Lake Cypress LMA	Lake Hatchineha LMA	Lake Kissimmee LMA
Current	1	<5-yr	<5-yr	<5-yr	< 5-yr	25-yr	< 5-yr	< 5-yr	< 5-yr	25-yr
	5	5-yr	< 5-yr	10-yr	5-yr	10-yr	10-yr	10-yr	10-yr	5-yr
	6	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr
Future	1	<5-yr	<5-yr	<5-yr	<5-yr	25-yr	< 5-yr	< 5-yr	< 5-yr	25-yr
	5	5-yr	< 5-yr	10-yr	5-yr	10-yr	10-yr	10-yr	10-yr	5-yr
	6	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr

The overall FPLOS rating of the entire UKB is < 5-yr. However, uncertainties of this estimate should be noted:

- Uncertainty in modeling: Flood protection assessments rely on modeling techniques, and every modeling software, equations used in the model have inherent uncertainties.
- Inaccurate data inputs: The accuracy of flood protection assessments heavily depends on the quality of data inputs, such as topographic information, land use, and infrastructure details. Inaccurate or incomplete data can lead to underestimating the actual flood protection levels. The data used were best available data, but imperfect data. This is particularly true of cross-sections and bathymetry used in the model.
- Changes in climate and land use: Flood protection assessments are based on current conditions and assumptions about future changes. However, climate change and land use developments can alter flood risk scenarios, potentially making the assessed protection levels inadequate in the future.
- Maintenance and deterioration: The effectiveness of flood protection infrastructure, such as levees and dams, can deteriorate over time due to lack of maintenance or changing environmental conditions. These factors were not considered in the assessment.
- Post-processing assumptions: Assumptions were made in post-processing, particularly about the edges of lakes. Assumptions about the actual edge of lakes, which varies depending on the season and other environmental factors, means the lands surrounding the lakes have uncertainty about what the land use is. This leads to assumptions about the significance of inundation depths and duration of the lands around the lakes. It is essential to thoroughly review and validate the assessment results to ensure their accuracy.

In addition to SFWMD FPLOS metrics, analysis was performed for water volume in lakes due to the design rainfall: cumulative and peak water volume (in acre-feet). The analysis was done for lakes under different design event (DE) scenarios (5-year, 10-year, 25-year, 100-year) under both current and future conditions. The analysis reveals a general trend of rising excess water volumes in lakes due to storm events, with future projections worsening this trend, as expected.

The modeling results for flood protection level of service in UKB provide valuable insights into the effectiveness of the existing flood protection infrastructure and the potential risks faced by the community. Through H&H modeling, this study evaluated the performance of the system under current and future conditions.

The modeling results reveal the areas that are most vulnerable to flooding in UKB, helping to identify the key areas where improvements in flood protection are needed. By simulating various flood scenarios, the modeling results also provide a comprehensive understanding of the potential impacts of different flood events on the community, including the extent of inundation, the depth of floodwaters, and the duration of flooding. This information is crucial for decision-makers and urban planners to develop effective strategies for mitigating flood risks and enhancing the resilience of the city. Additionally, modeling results can be evaluated to be used as boundary conditions for Lower Kissimmee Basin LOS study.

Addendum (May 2025):

The UKBFPLOS model calibration and design event models were based on DHI's 2022 Update 1. Following the completion of this report using the 'original run', a subsequent rerun yielded slightly different results; specifically, the variation in flows was less than 5% at peak for the largest structure (S65). The differences in stages were less than 0.1 ft. The investigation highlighted the following reasons for these differences:

- At one of the upstream structures (S61), the initial gate level in the 'original run' was set lower than the sill level, indicating that the gate was closed. In the new run, the initial gate level was set exactly at the sill level, still indicating the gate is closed to start the model run. This adjustment caused variations in flow when compared to the original run.
- The MAGO curves, which determine the gate opening, utilize a three-dimensional surface incorporating headwater, tailwater, and gate opening parameters. The SFWMD MAGO curves are a series of curves with no intermediate values between them. Even minor differences of less than 0.1 ft in large structures can result in slightly different flow calculations due to the sensitivity of the gate opening. Consequently, model runs on different machines may produce varying flow results owing to machine precision. Therefore, potential users of this model application can expect differences of less than 5% when running the same model on different machines.
- The software developer does not provide technical support of DHI 2022 version any longer, and therefore, do not have this version on their website for download any longer. So, future users of this model application will be using a later version, and so differences in the results should be expected.

It should be noted that the software developer DHI's letter discussing slight differences caused by versions, machine precision etc. has been sent to SFWMD.

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1. Introduction and Background

The South Florida Water Management District (SFWMD) is conducting a review of their regional water management infrastructure to determine the Flood Protection Level of Service (FPLOS) provided by existing SFWMD operated infrastructure under current and future conditions in the Upper Kissimmee Basin FPLOS (UKB FPLOS).

The Kissimmee Basin (KB), consisting of UKB and Lower Kissimmee Basin (LKB) includes diverse wetland and aquatic ecosystems, consisting of lakes and their tributary streams, the related marshes, and the Kissimmee River and its floodplain, shown in **Figure 1-1**. The basin forms the headwaters of Florida's Kissimmee-Okeechobee-Everglades (KOE) ecosystem.

The UKB is comprised of the Kissimmee Chain of Lakes (KCOL) and the Econlockhatchee watershed, Reedy, Boggy, Shingle and Catfish creek watersheds. KCOL is part of the Kissimmee River watershed and the headwaters of the Florida Everglades discharging into the Kissimmee River through the S-65 structure at the south end of Lake Kissimmee. The Kissimmee River flows south, and a large volume of the water goes into Lake Okeechobee and eventually into the Everglades.

In the 1960s, the Central and Southern Florida Flood Control (C&SF) Project modified these systems to provide flood control by construction of canals and water control structures in the KB, including the UKB. In the UKB, C&SF allowed the lake stages to be regulated at narrower ranges of fluctuation, altering the littoral zones around lakes, or eliminating them altogether. These adverse impacts and other environmental concerns led to the Kissimmee River Restoration Project (KRRP) and the Headwaters Revitalization Project (KRHRP). These projects involved land acquisition in the UKB to increase the water storage capacity of lakes in the Kissimmee Chain of Lakes (KCOL) and formulation of a new lake stage regulation to support the restoration efforts.

The Kissimmee Watershed Program of SFWMD was formed in the 1990s to coordinate and evaluate the restoration and other KB projects. One of the objectives of this program was to develop a long-term management plan for resolving water management issues in the KCOL and maintain the existing level of flood control in the KB. In addition to KRRP, some of the other major initiatives in the KB designed to meet these objectives were the Kissimmee Basin Modeling and Operations Study (KBMOS) and the KCOL Long-Term Management Plan (LTMP).

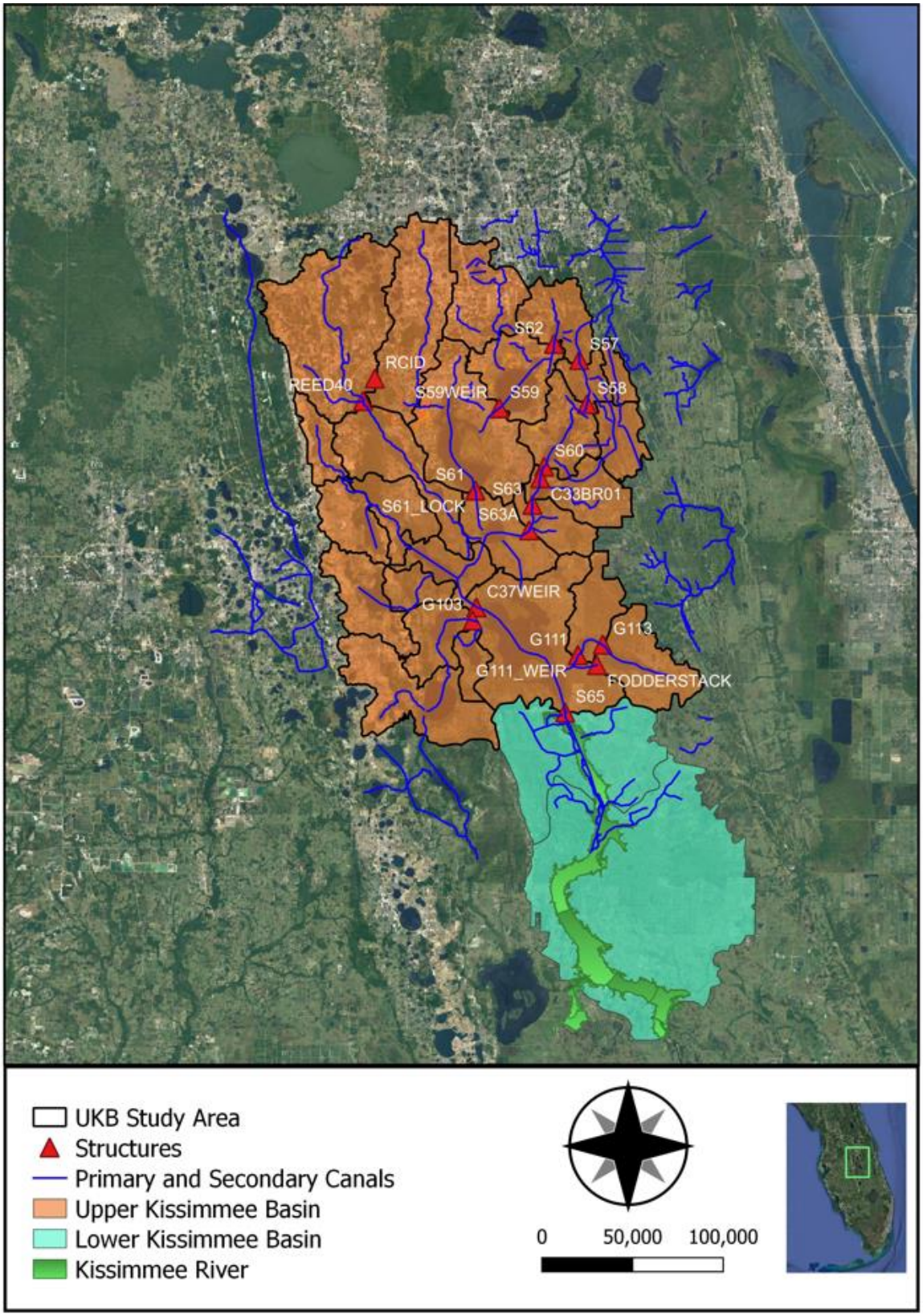

Figure 1-1: Kissimmee Basin

1.1 History of the Kissimmee Basin

UKB has interesting but complex hydrology and hydrogeology. Historically, the KCOL and the Kissimmee River comprised of a wetland system connected by creeks and large, shallow marshes (**Figure 1-2**). During the wet season, the KCOL lakes overflowed onto adjacent lands which provided continuous flow to the Kissimmee River and inundation of its floodplain (USFWS, 1959). Hurricanes prompted the state of Florida and the federal government to provide flood control, which resulted in the C&SF project. The C&SF Project was successful in meeting the flood control objective but had adverse impacts on the KB hydrology (Obeysekera and Loftin, 1990).

In the UKB, C&SF features constructed from 1964 to 1970 included dredging of canals between lakes to connect them and installation of nine water control structures to regulate the volume and timing of discharges between them (USACE, 1991). The S65 structure at the outlet of Lake Kissimmee also regulated the discharges to the LKB from UKB. Pre-C&SF lake level fluctuation was 2–10 ft, which was reduced to 2-4 ft after C&SF (Obeysekera and Loftin, 1990). The reduced water level fluctuation interfered with the natural flooding and drying cycle, and allowed residential, agricultural, commercial, and other land uses to encroach on lake flood zones (USACE, 1996).

SFWMD and the U.S. Army Corps of Engineers (USACE) have the primary authority over the water management in the UKB. There are other agencies involved in water management, some of them are: Valencia Water Control District, Reedy Creek Improvement District, Haines City Water Control District, as well as the local governments and municipalities.

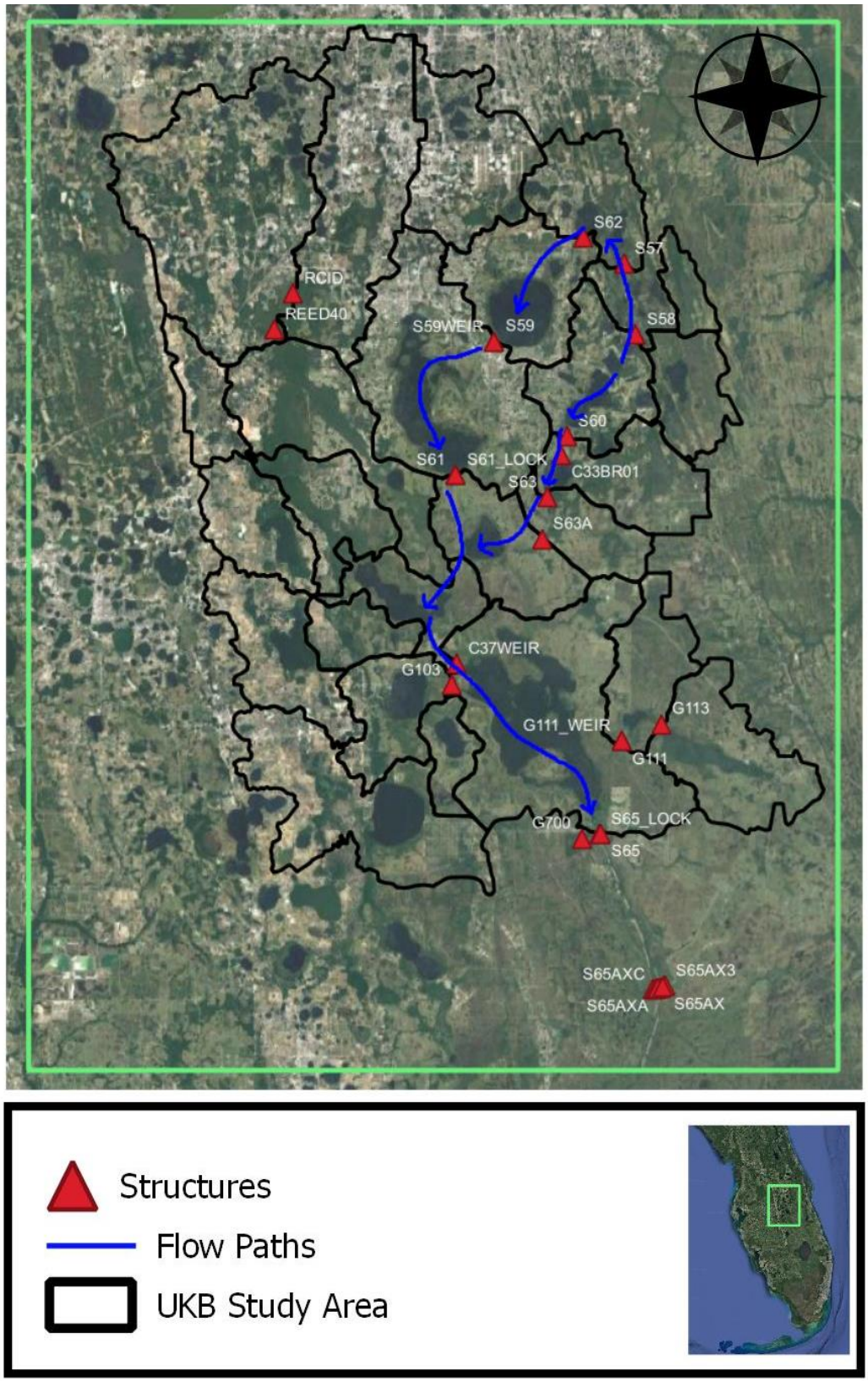

Figure 1-2: Upper Kissimmee Basin

1.2 Hydrology of the Study Area

In the past literature (reports, technical papers, presentations) the area of KB was documented as approximately 2300 square miles (sq. mi), with UKB at 1600 sq. mi and LKB at 700 sq. mi. The UKB FPLOS Study Area is a little over 1600 sq. mi but includes watersheds that are a little different from past literature, shown in **Table 1-1**. **Figure 1-3** shows the 26 UKB FPLOS Study Area watersheds.

The canals and associated C&SF water control structures in UKB primarily provide flood control but provide navigation and other recreational purposes as well. The C&SF structures in the UKB regulate the flow of water by discharging excess water from the lakes during flooding, maintaining optimum water levels balancing flood control and environmental needs.

The discussion in this section follows the flow of water in UKB FPLOS Study Area. For the purposes of this study, the surface water bodies (lakes, creeks) are grouped based on the hydrology in the following groups:

- The eastern chain of lakes consists of lakes that the Alligator Chain of Lakes flow to, or the lakes that flow from the Alligator Chain of Lakes. So, this grouping includes Alligator Lake, Lake Lizzie, Coon Lake, Trout Lake, Lake Joel, Lake Myrtle, and Lake Mary Jane, Econlockhatchee River Swamp and Lake Conlin, Cat Lake and Buck Lake, as well as Lake Gentry.
- The western chain begins with Lake Hart, continues to Lake Ajay, East Lake Tohopekaliga and Lake Tohopekaliga. This chain also includes Boggy Creek which flows into East Lake Tohopekaliga and Shingle Creek that flows into Lake Tohopekaliga.
- The southern chain consists of lakes that receive water from the eastern and western chain of Lakes. This group also includes Reedy Creek which is located to the west of the western chain but flows into lake Cypress and Hatchineha. Lake Cypress flows into Lake Hatchineha which flows into Lake Kissimmee. Lakes Jackson and Marian are located to the east of Lake Kissimmee, lakes Marion, Pierce, Rosalie, Tiger, Weohyakapka and Catfish and Marion creeks are also included in this group.
- Horse Creek is considered a closed drainage area and is discussed separately.

The terminology in this report refers to the drainage areas in **Figure 1-3** and **Table 1-1** as watersheds; a group of these watersheds is referred to as a basin. If and when appropriate, some of the discussion focuses on the waterbodies – lakes or canals, and others focus on the areas which drain to them. The polygon IDs in the table are specific to this project.

Elevations in discussions are in NAVD88 and NGVD29; the conversions are from SFWMD updated structure books, so the conversion is not the same everywhere in the UKB FPLOS Study Area.

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Table 1-1: UKB FPLOS Study Area Watersheds (SFWMD)

Polygon ID	Name of Watershed	Area (acres)	Area (sq. mi)
1	S63a	20,668	32.29
2	Tiger Lake	12,196	19.06
3	Lake Rosalie	25,872	40.43
4	Lake Kissimmee	103,653	161.96
5	Lake Jackson	26,919	42.06
6	Lake Pierce	35,712	55.8
7	Lake Gentry	32,441	50.69
8	Horse Creek (Closed Basin)	16,949	26.48
9	Lake Cypress	34,316	53.62
10	Lake Marian	42,036	65.68
11	Marion Creek	27,713	43.3
12	Catfish Creek	15,836	24.74
13	Lake Conlin	22,087	34.51
14	Lake Marion	27,194	42.49
15	Lake Hatchineha	23,706	37.04
16	Lower Reedy Creek	51,714	80.8
17	Upper Reedy Creek	113,856	177.9
18	Lake Tohopekaliga	83,841	131
19	Lake Myrtle	13,643	21.32
20	East Lake Tohopekaliga	39,598	61.87
21	Alligator Lake	33,699	52.66
22	Shingle Creek	75,416	117.84
23	Lake Hart	30,327	47.39
24	Lake Weohyakapka	64,879	101.37
25	Boggy Creek	45,447	71.01
26	Econlockhatchee River Swamp	9,035	14.12
	Total Area	**1,028,752**	**1607.4**

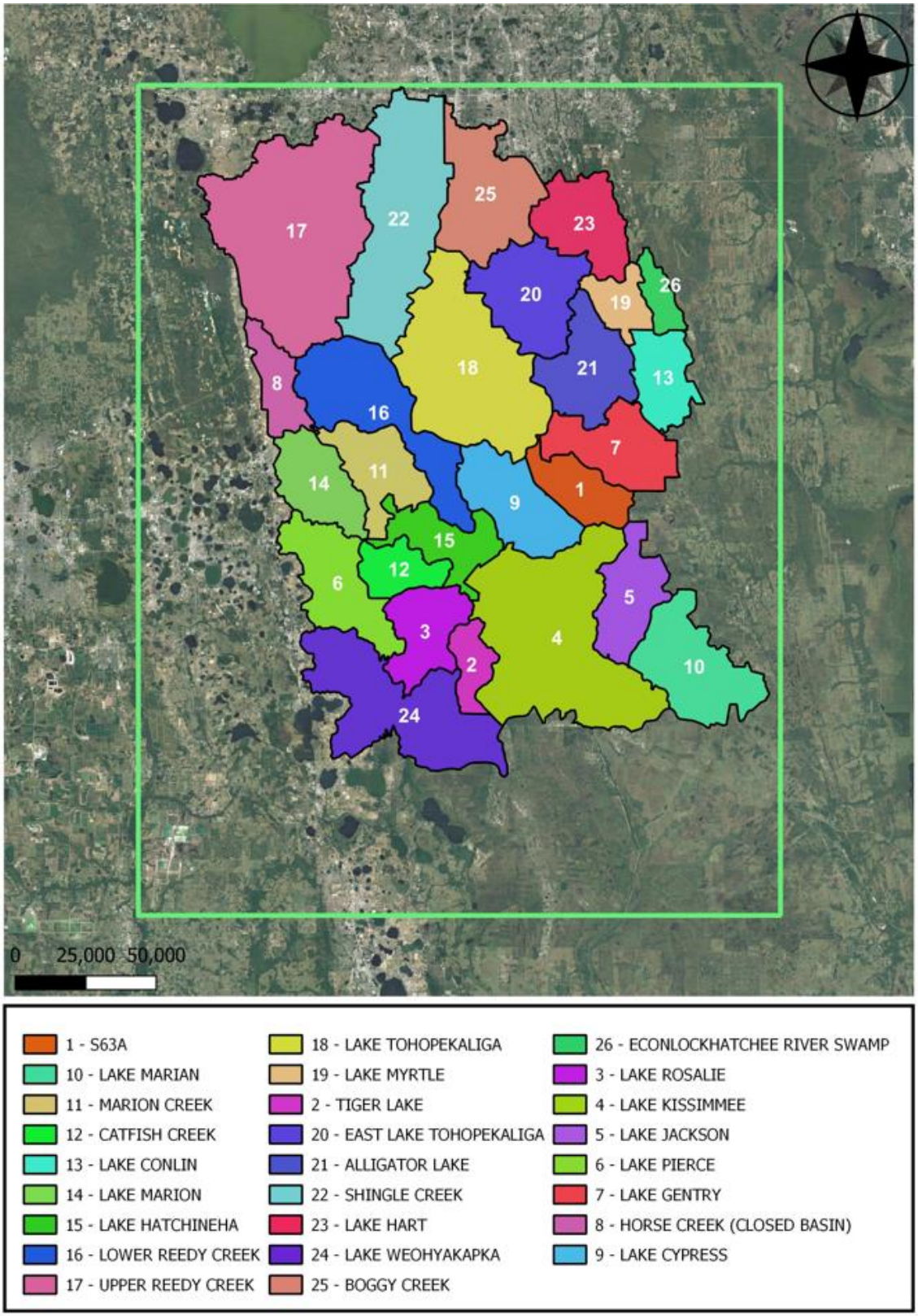

Figure 1-3: UKB FPLOS Study Area watersheds

1.2.1 Eastern Chain of Lakes

1.2.1.1 Lake Alligator Watershed

Lake Alligator watershed (also known as Alligator Chain of Lakes) includes Alligator Lake and many small lakes; some of them are Lake Lizzie, Trout Lake, Coon Lake, Sardine Lake, Live Buck Lake, Oak Lake, Bay Lake, and Lake Center. Location of Alligator COL in UKB is shown in **Figure 1-4**. Alligator Lake is the uppermost of the KCOL and is controlled by structures S58 on the north through C-32 canal and S60 on the south through C-33 canal, both paths take water to Lake Cypress. Both structures together are operated in accordance with the Lake Alligator Regulation Schedule which ranges between elevations 60.91 ft NAVD88 (62.0 ft NGVD29) to 62.91 ft NAVD88 (64 ft NGVD29).

For flood control operations, S58 and S60 are operated to return the lake to regulation schedule when the water level in Alligator Lake exceeds regulation schedule and is within 0.5 feet above the regulation schedule. **Figure 1-5** and **Figure 1-6** show the regulation schedules for the S58 and S60 structures. The releases are made with maximum releases subject to Maximum Allowable Gate Opening (MAGO) and Maximum Permissible Head Differential (MPHD) limitations (SFWMD, 2022c).

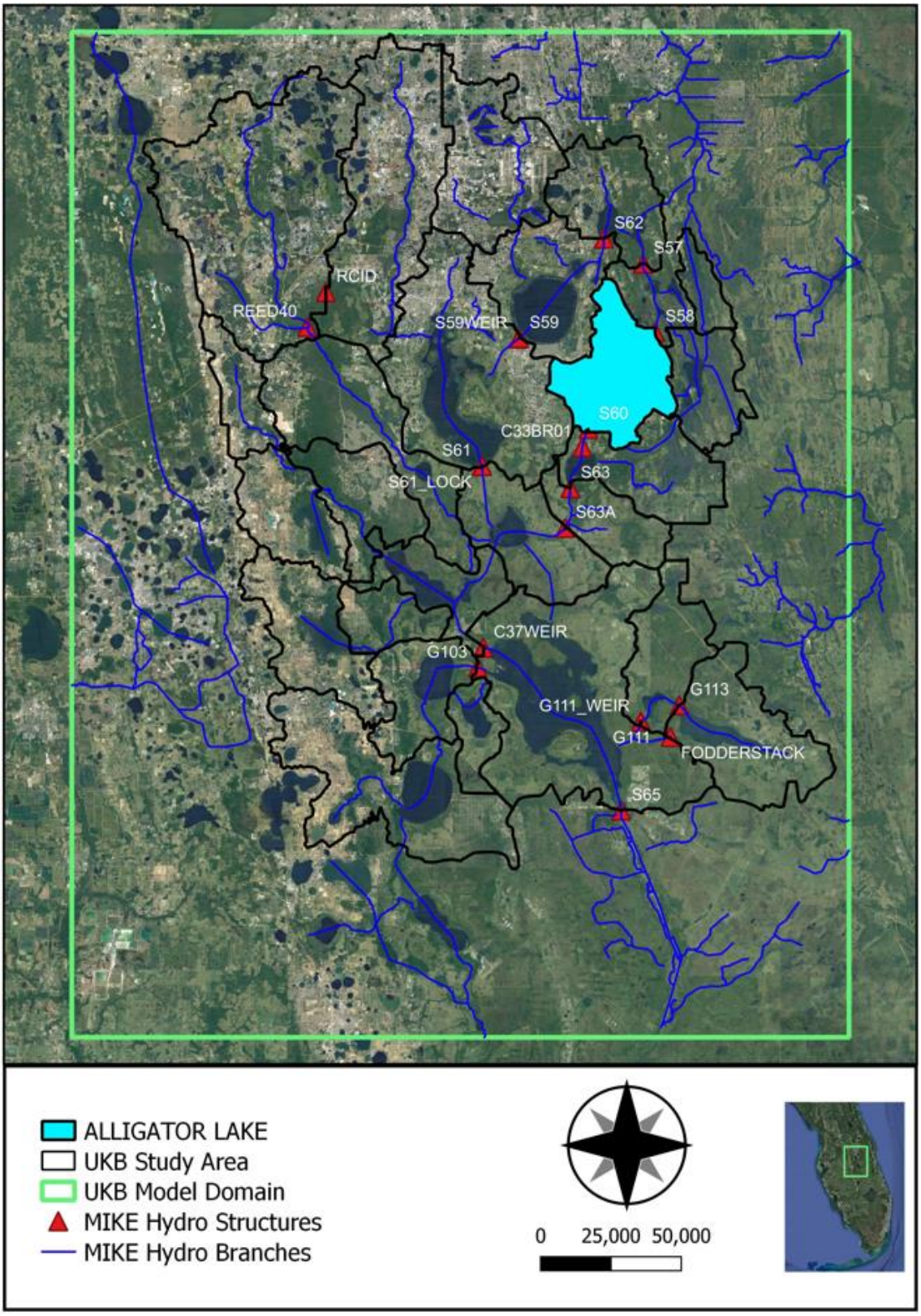

Figure 1-4: Alligator Chain of Lakes Watershed in UKB

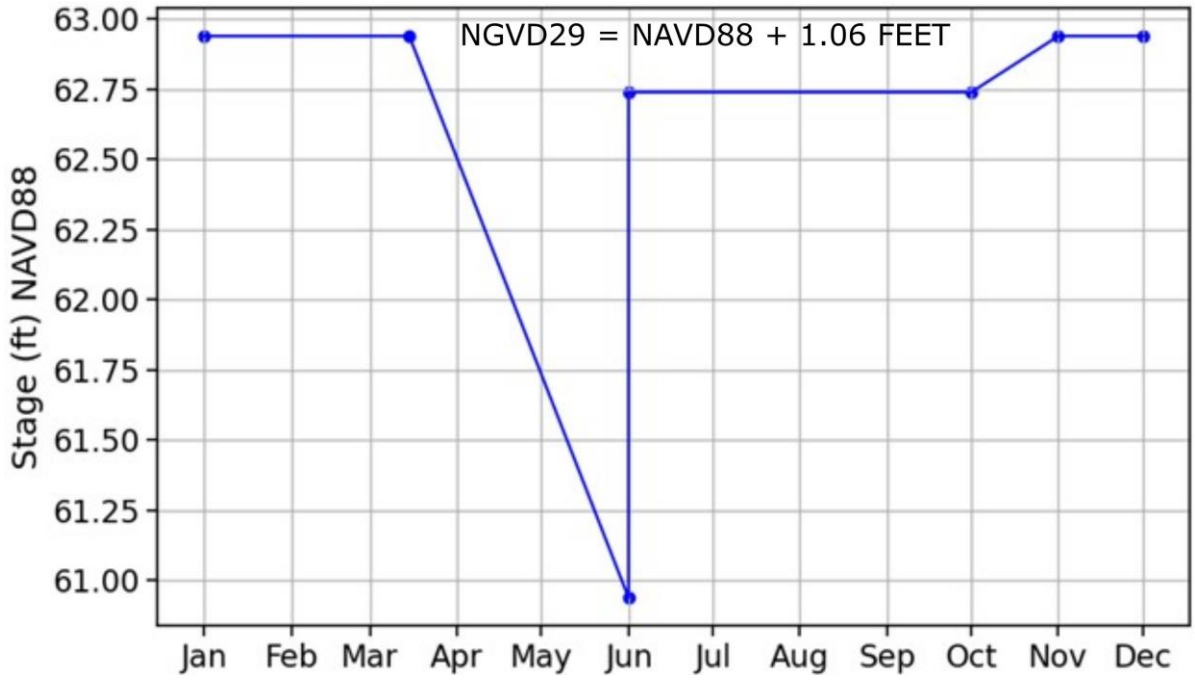

Figure 1-5: Lake Alligator, Brick, Lizzie, Coon, Center, and Trout Regulation Schedule at S58

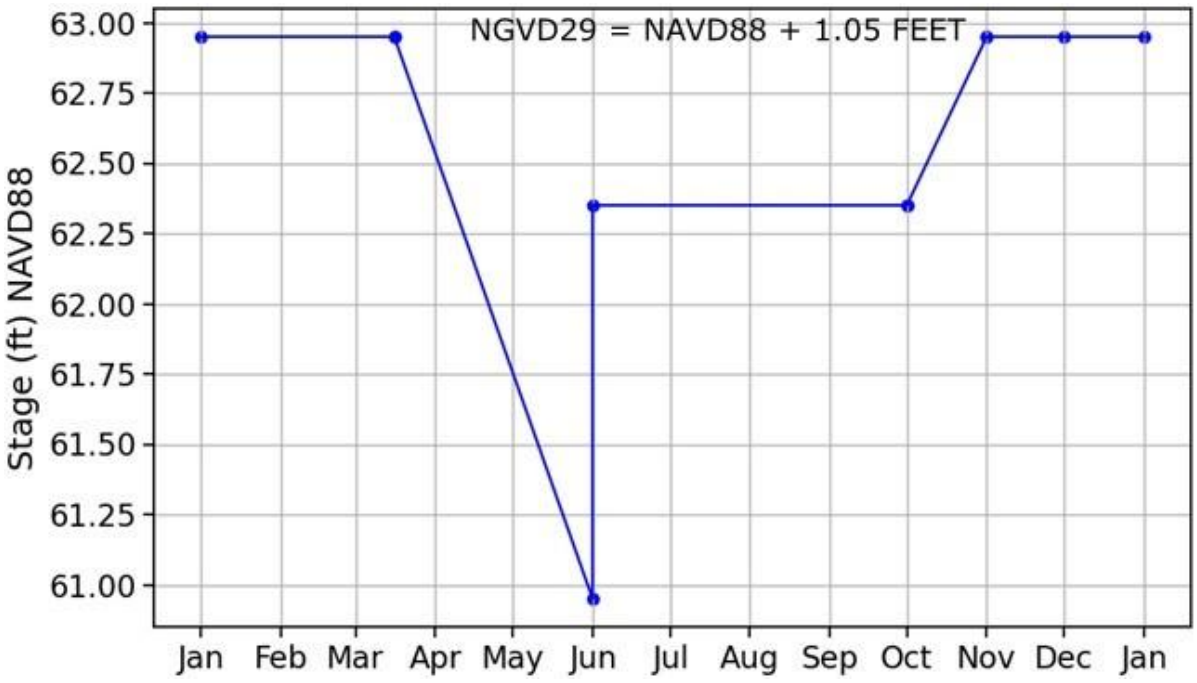

Figure 1-6: Lake Alligator, Brick, Lizzie, Coon, Center, and Trout Regulation Schedule at S60

1.2.1.2 Lake Myrtle Watershed

Lake Myrtle Watershed, shown in **Figure 1-7** receives flow from Alligator Lake through Lake Lizzie when S60 structure flowing to the eastern chain (Lake Gentry) is at maximum capacity. Trout Lake (Lake Alligator Watershed) discharges to Lake Joel through the S58 structure. Also discharging into Lake Myrtle are Cat Lake, Lake Conlin, and Lake Preston, which are part of the same watershed. C-30 canal conveys flows from Lake Myrtle Watershed to Lake Mary Jane in Lake Hart Watershed through the S57 structure.

S57 is operated to maintain the stages in Lakes Preston-Myrtle-Joel (PMJ) between 58.9 ft NAVD88 (60.0 ft NGVD29) and 60.9 ft NAVD88 (62.0 ft NGVD29), subject to MAGO and MPHD criteria (SFWMD, 2022c). When PMJ water level exceeds regulation schedule and is within 0.5 feet above the regulation schedule, which is shown in **Figure 1-8**, releases will be made to return lakes levels to the regulation schedule within 15 days.

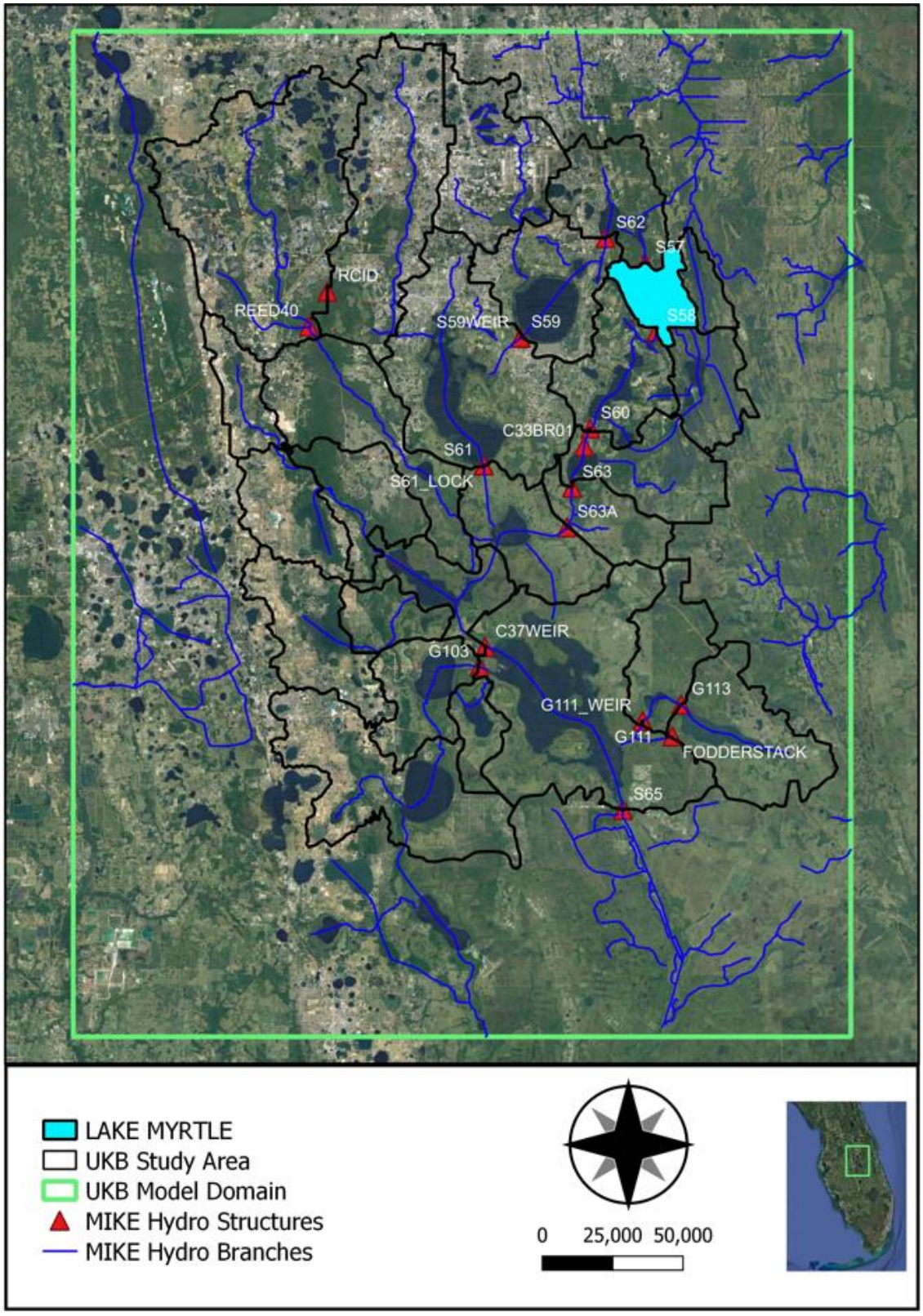

Figure 1-7: Lake Myrtle Watershed in UKB

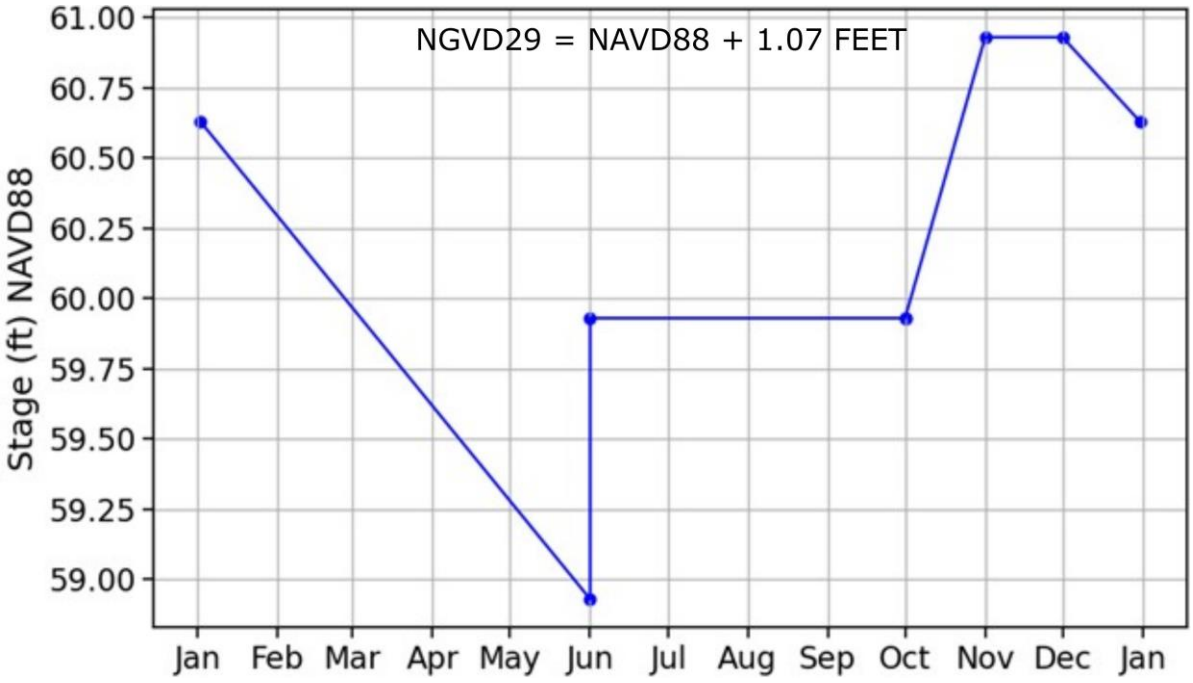

Figure 1-8: Lake Joel, Myrtle, and Preston Regulations Schedule at S57

1.2.1.3 Econlockahatchee River Watershed

The Econlockahatchee Watershed is located to the east in the UKB FPLOS Study Area (**see Figure 1-9**) and is in the St. Johns River Water Management District. Located within the UKB FPLOS Model domain, it includes part of St. Johns River Basin, specifically parts of upper and middle St. Johns River watersheds. Interestingly, this watershed shares its hydrologic characteristics with the KB, although St. Johns River flows north, and Kissimmee River flows south. The hydrologic connection has been documented during helicopter flyovers and field reconnaissance after hurricanes (Konyha, 2011).

1.2.1.4 Lake Gentry

Figure 1-10 shows the location of Lake Gentry in UKB. Alligator Lake discharges into Lake Gentry through the S60 structure on the C-33 canal. Lake Gentry receives water through S60 and sheet flow from Brick Lake, Pearl Lake and Big Bend Swamp. The S63 structure on C-34 canal (Canoe Creek Canal) is the outlet of Lake Gentry. Water level in C-34 canal, downstream of the S63 structure, is further stepped down by S63A and then discharged to Lake Cypress.

The S63 structure is operated in accordance with the Lake Gentry regulation schedule shown in **Figure 1-11**, with the normal range of 58.41 ft NAVD88 (59.5 ft NGVD29) to 60.41 ft NAVD88 (61.5 ft NGVD29). For flood control operations, the S63 structure is operated to return the lake to regulation schedule when the water level in Alligator Lake exceeds regulation schedule and is within 0.5 feet above the regulation schedule. The releases are made with maximum releases subject to MAGO and MPHD limitations (SFWMD, 2022c).

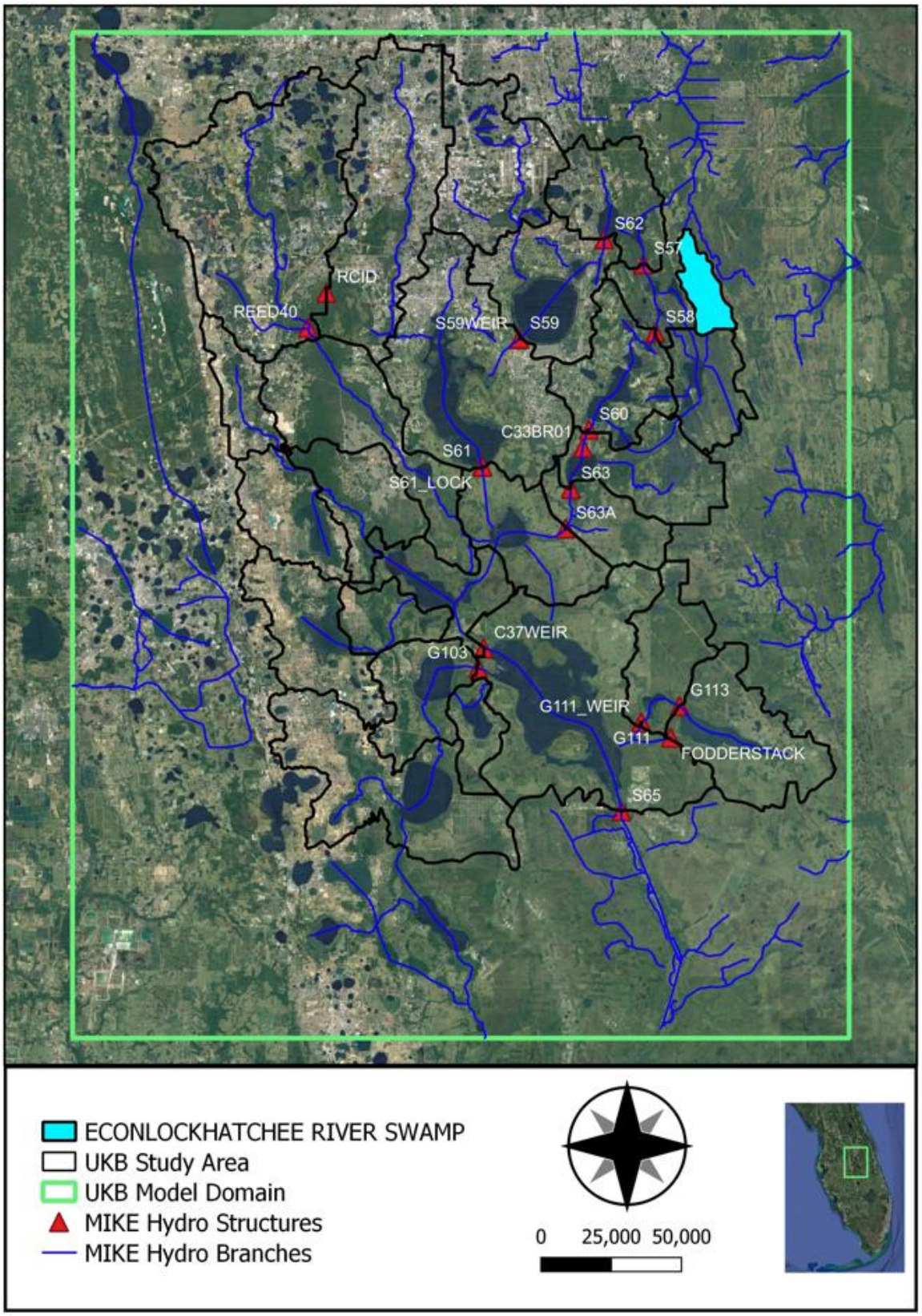

Figure 1-9: Econlockhatchee River Watershed in UKB

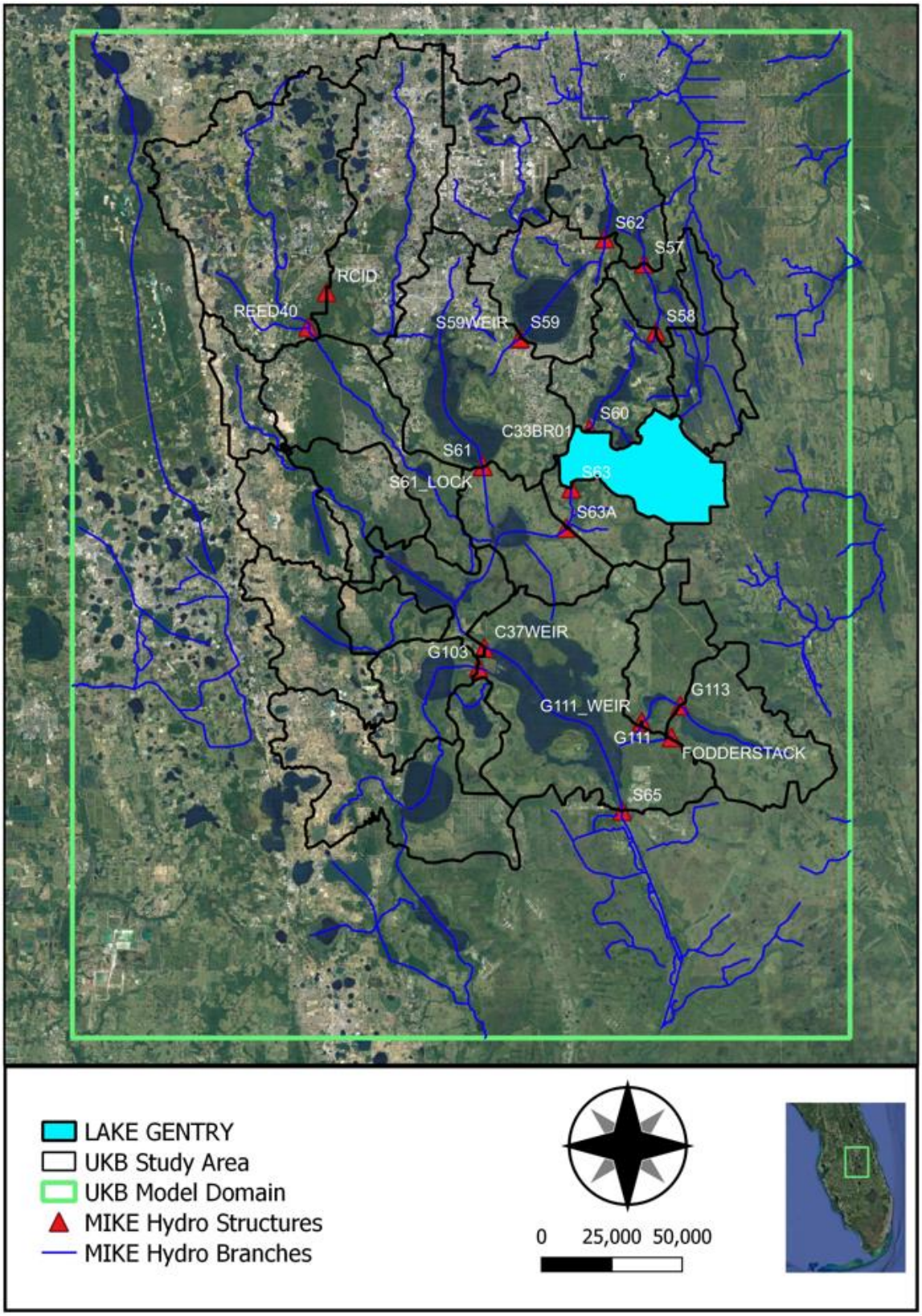

Figure 1-10: Lake Gentry Watershed in UKB

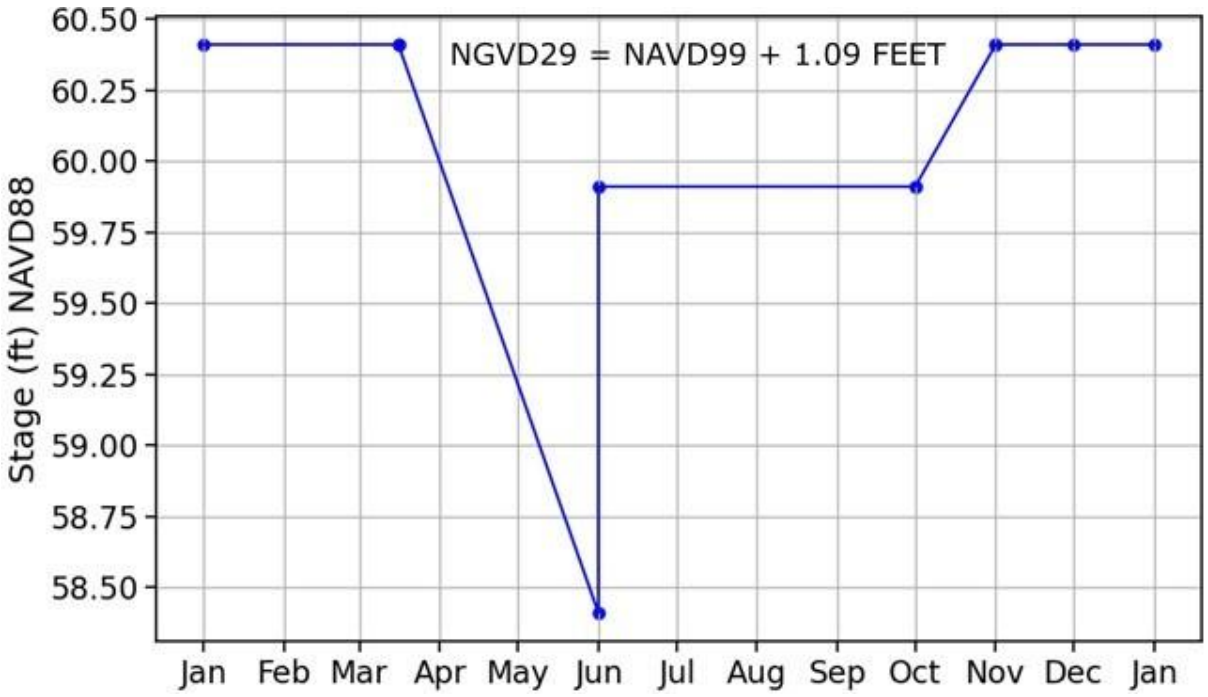

Figure 1-11: Lake Gentry Regulation Schedule at S63

1.2.2 Western Chain of Lakes

1.2.2.1 Lake Hart Watershed

Lake Hart Watershed, shown in **Figure 1-12**, is downstream of Lake Myrtle Watershed. There are several lakes in this watershed connected by canals: C-29 connects lakes Hart and Mary Jane, C-29A connects Ajay Lake with Lake Hart, Myrtle Bay connects a chain of Nona-Red Lake-Buck Lake to Lake Hart and an unnamed short channel connects Barton Lake to Lake Hart. Lake Hart discharges through the S62 structure on C-29B canal to Ajay Lake and then flows into East Lake Tohopekaliga (Fells Cove).

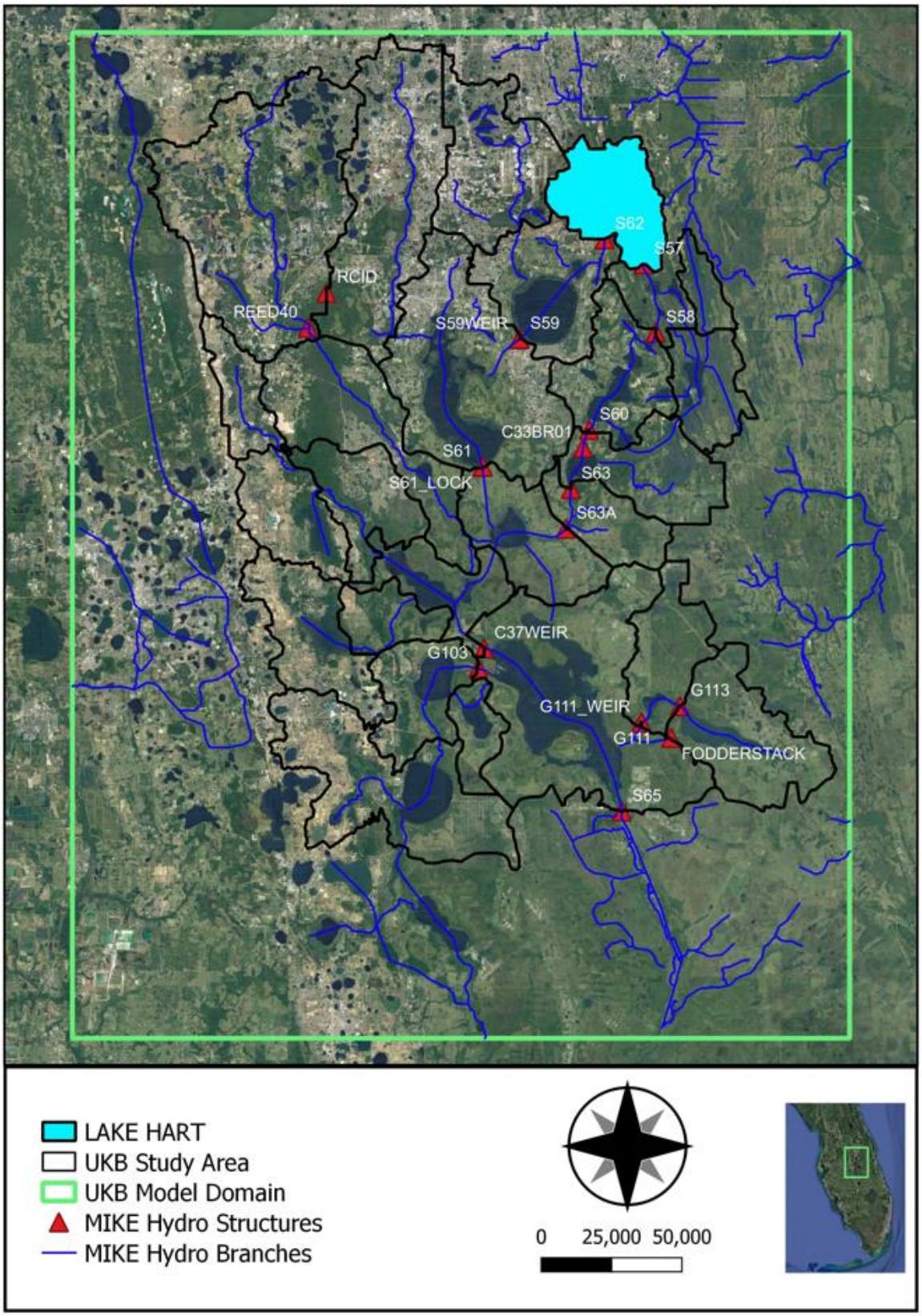

Figure 1-12: Lake Hart Watershed in UKB

S62 is operated to maintain the stages in Lakes Hart and Mary Jane between 58.47 ft NAVD88 (59.5 ft NGVD29) and 59.97 ft NAVD88 (61.0 ft NGVD29), subject to MAGO and MPHD criteria (SFWMD, 2022c). When water levels at lakes Hart and Mary Jane exceed 0.5 foot of the regulation schedule shown in **Figure 1-13**, releases will be made to return lakes levels to the regulation schedule within 15 days.

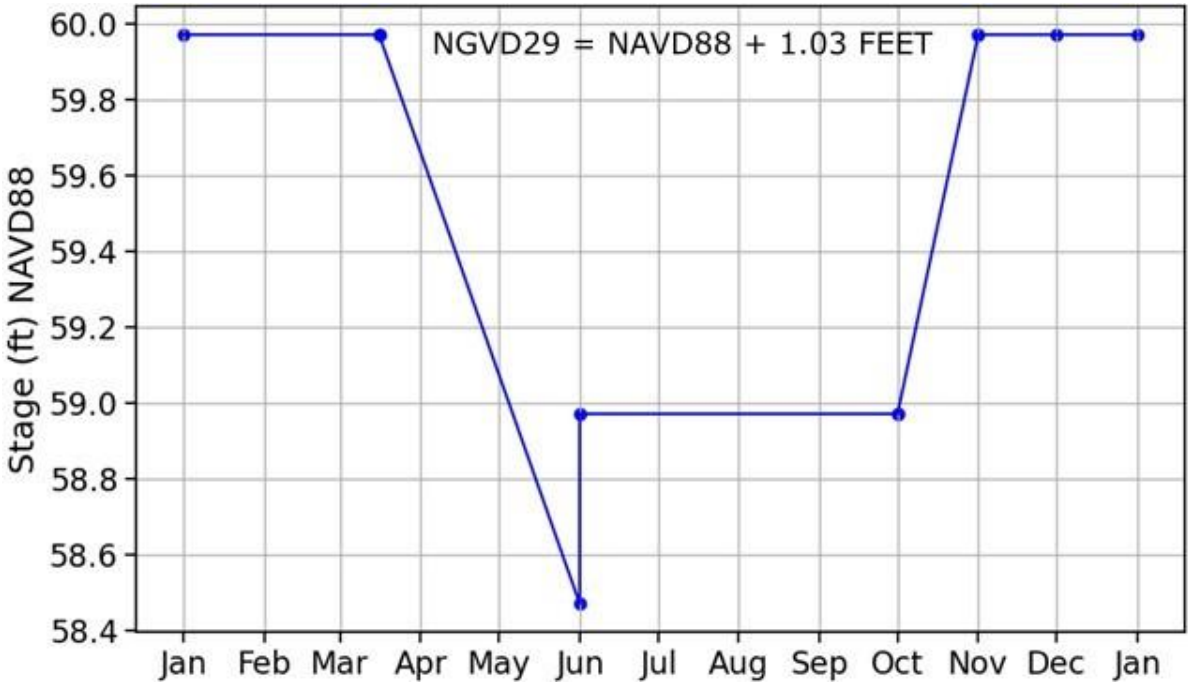

Figure 1-13: Lakes Hart and Mary Jane Regulation Schedule at S62

1.2.2.2 East Lake Tohopekaliga (E. Toho) Watershed

Boggy Creek, unmanaged by SFWMD, discharges into the northwest side of East Lake Tohopekaliga; **Figure 1-14** shows Lake E. Toho. Lake Hart discharges into the E. Toho watershed at Fells Cove through the S62 structure on the C-29B canal from Ajay Lake. Boggy Creek and Lake Hart basins contribute most of the surface flow to E. Toho. Flows are released from E. Toho to Lake Tohopekaliga (Toho) through the S59 structure on the C-31 canal (also known as St. Cloud Canal).

The S59 structure is operated to maintain the stages at Lake E. Toho between 54.03 ft NAVD88 (55.0 ft NGVD29) and 57.03 ft NAVD88 (58.0 feet NGVD29). When the lake level exceeds regulation schedule and is within 0.5 feet above the regulation schedule shown in **Figure 1-15**, releases are made at S59 to return the lake to regulation schedule within 15 days, subject to MAGO and MPHD limitations (SFWMD, 2022c).

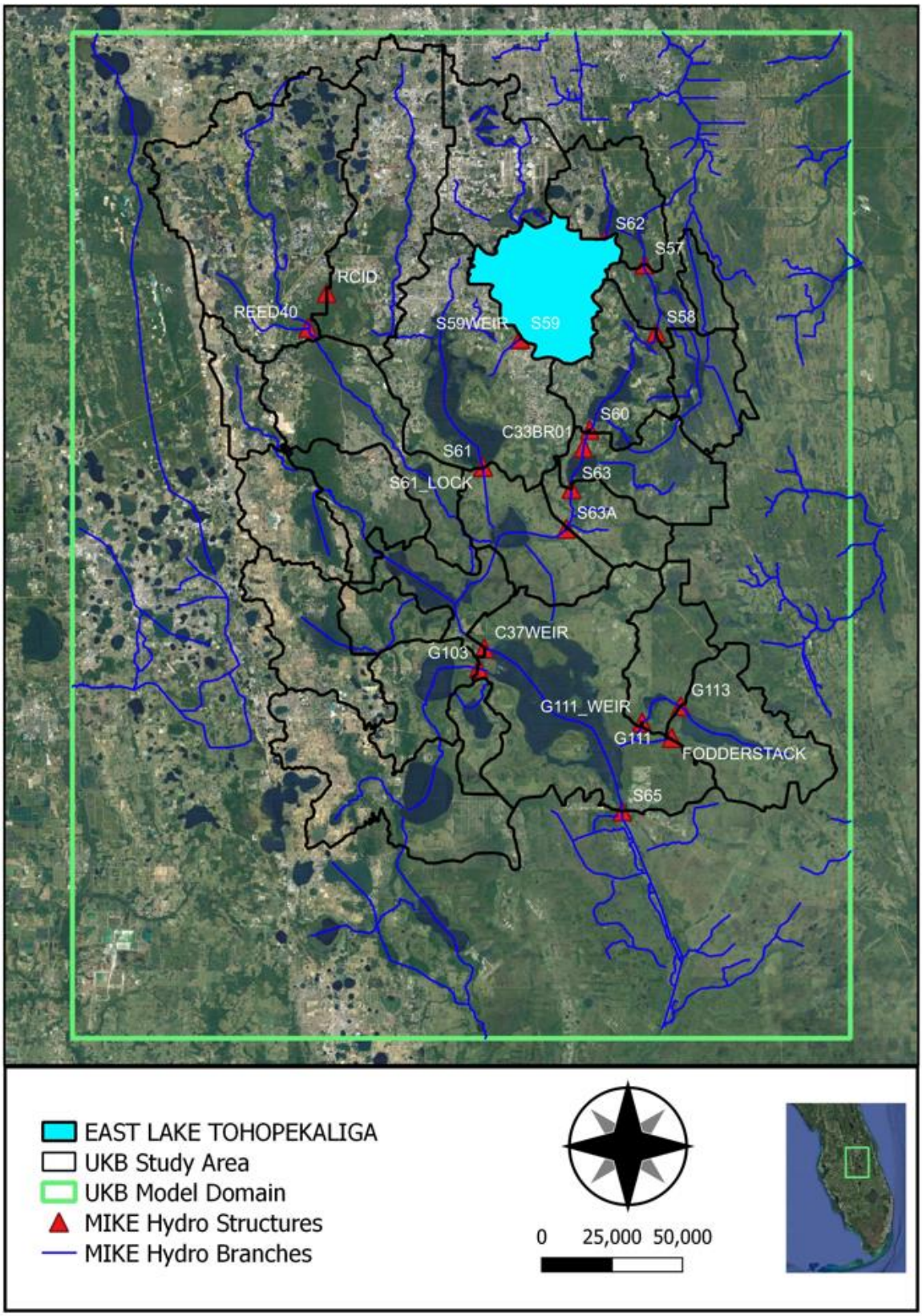

Figure 1-14: East Lake Tohopekaliga Watershed in UKB

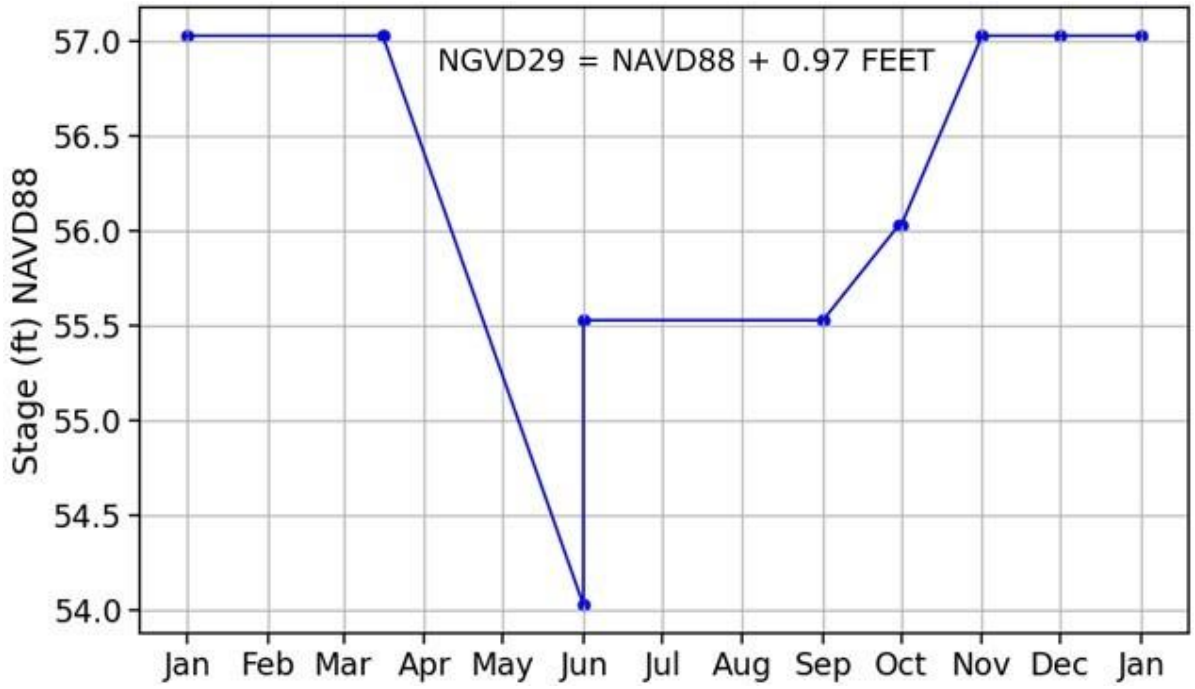

Figure 1-15: East Lake Tohopekaliga Regulation Schedule at S59

1.2.2.3 Lake Tohopekaliga (Toho) Watershed

Lake Tohopekaliga (Lake Toho) receives water from Lake E. Toho through the C-31 canal and from Shingle Creek. There are other smaller lakes in this watershed; the lakes together occupy almost half of the watershed area. Structure S61 is the outlet of Lake Toho and is operated to maintain Lake Toho stages between 50.95 and 53.95 NAVD88 (52 and 55 ft NGVD), subject to MAGO and MPHD limitations (SFWMD, 2022c). **Figure 1-16** shows the location of Lake Toho in UKB, operated according to the schedule shown in **Figure 1-17**.

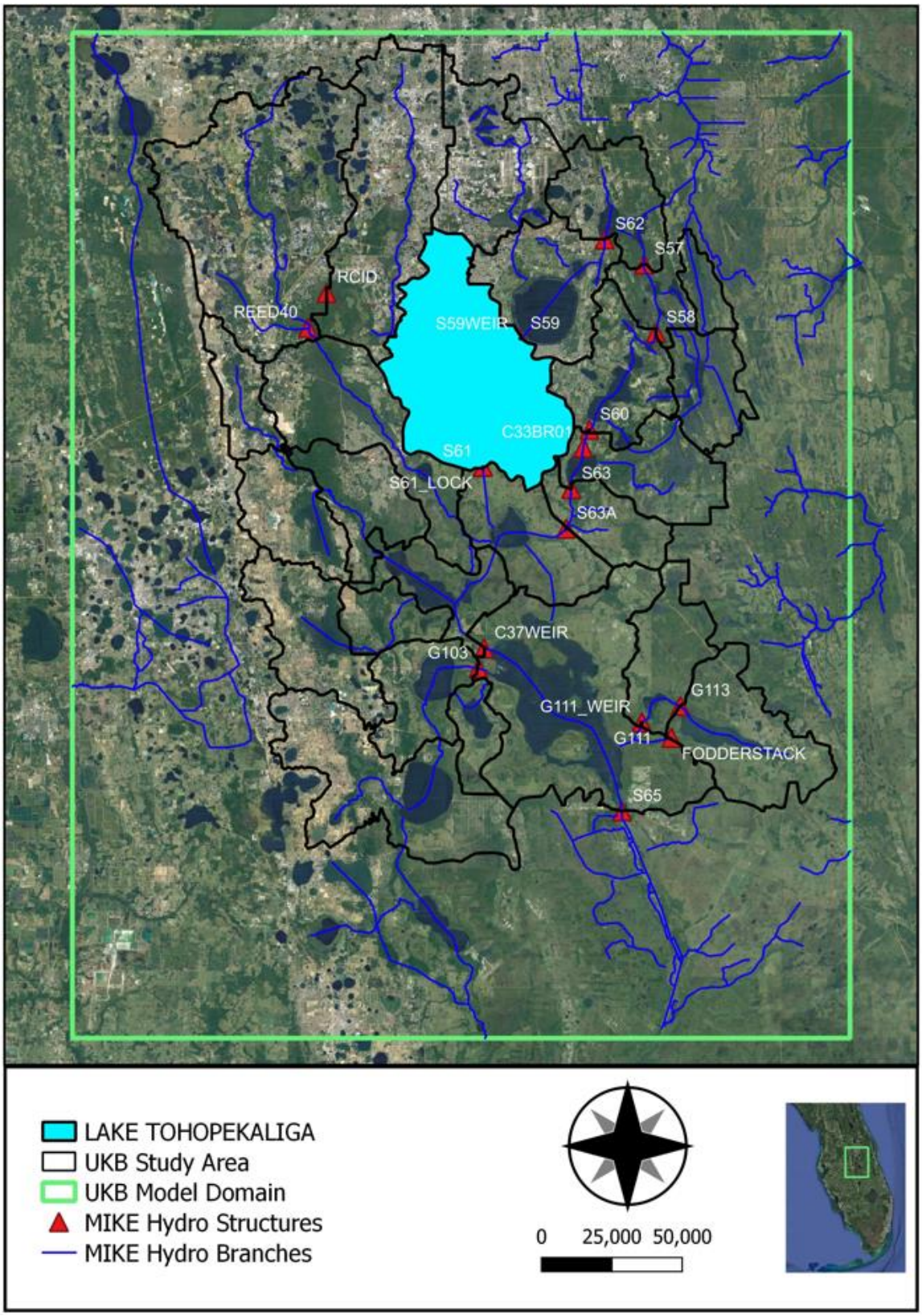

Figure 1-16: Lake Tohopekaliga Watershed in UKB

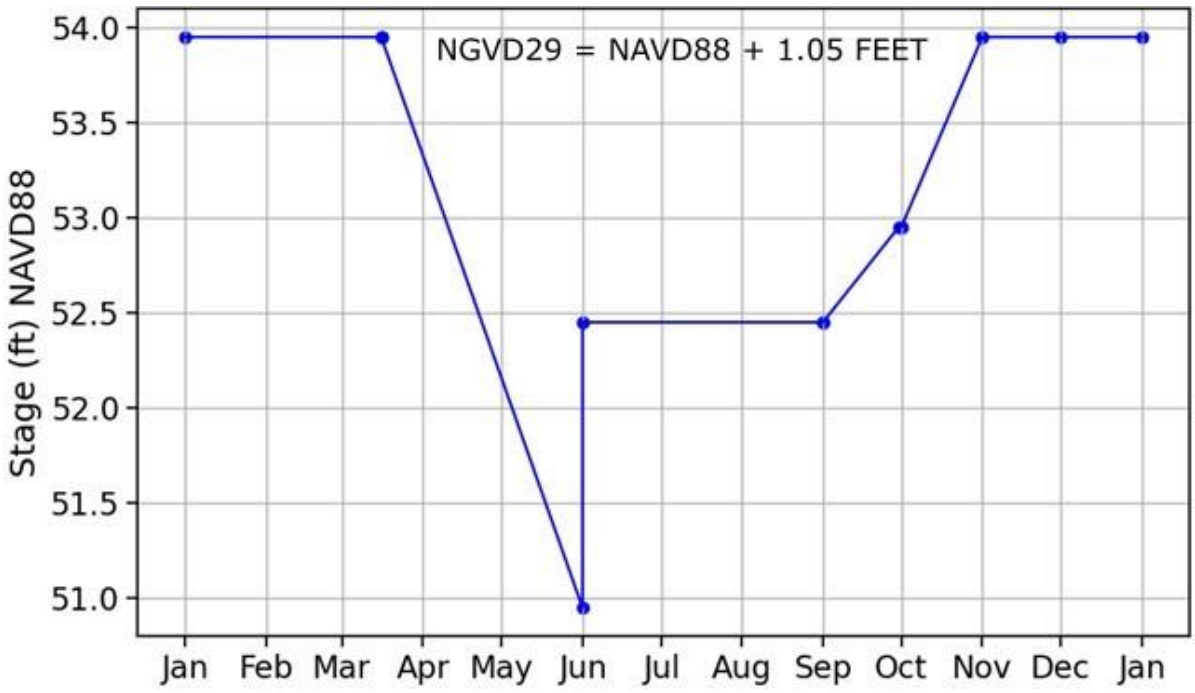

Figure 1-17: Lake Tohopekaliga Regulation Schedule at S61

1.2.2.4 Boggy Creek Watershed

The Boggy Creek Watershed shown in **Figure 1-18**, makes up the largest inflow to Lake E. Toho. This watershed has over 20 lakes, and the entire watershed drains to the approximately 12 mile long Boggy Creek. The upper portion of Boggy Creek is channelized and is surrounded by urban lands including Orlando International Airport; the lower portion is natural and drains many sloughs and swamps. This watershed does not contain any water control structures.

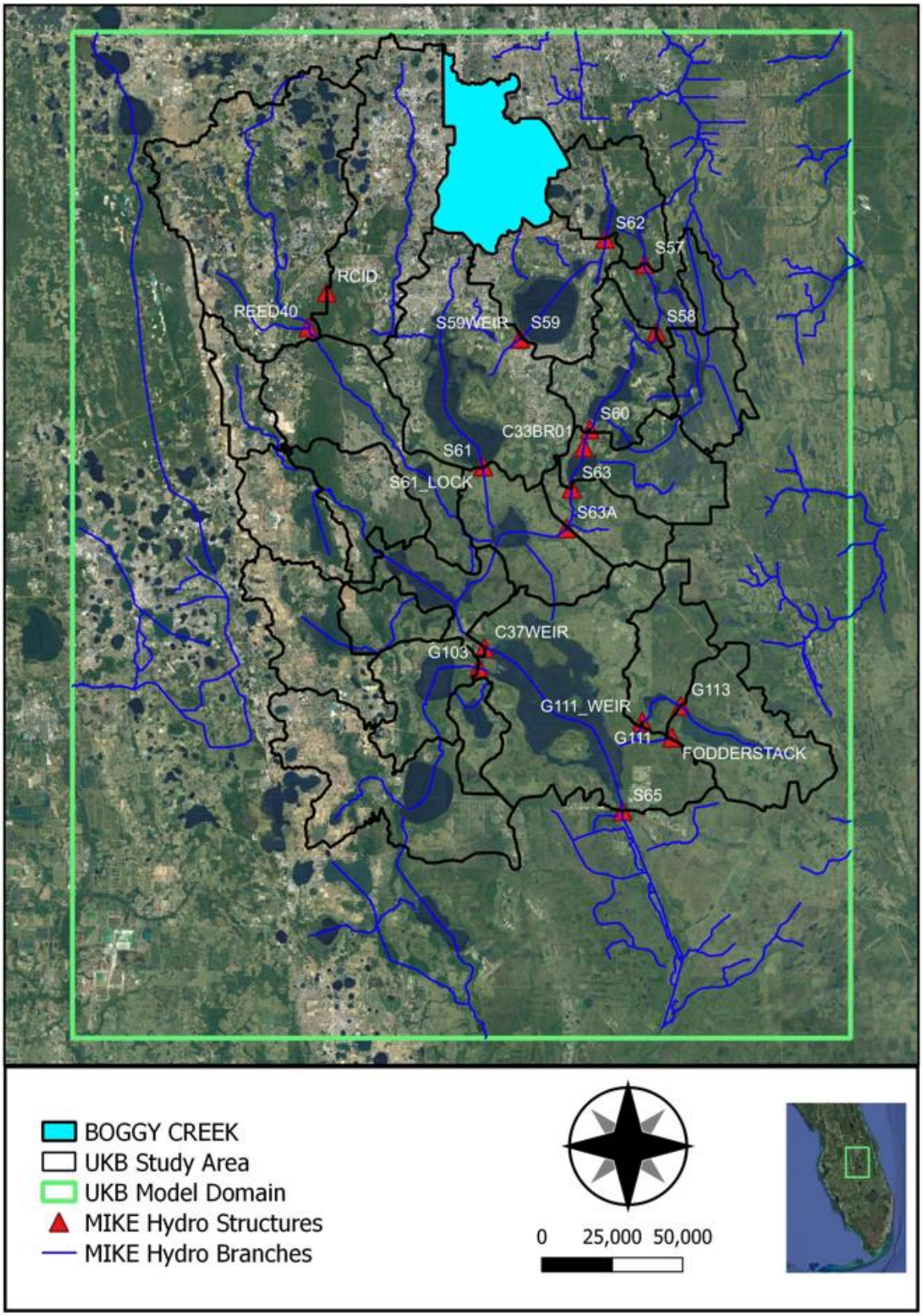

Figure 1-18: Boggy Creek Watershed in UKB

1.2.2.5 Shingle Creek Watershed

Shingle Creek is a major source of inflows to Lake Toho; the location of this watershed is shown in **Figure 1-19**. The headwaters of Shingle Creek are in the residential area east of Florida's Turnpike and are channelized. The southern part of the watershed has large wetlands and is natural in its flow. There are over 20 lakes in this watershed, some connected directly to Shingle Creek. Included in the Shingle Creek drainage area is a portion of Reedy Creek swamp and surface flow that discharges to Shingle Creek. When the outflow from the Reedy Creek Improvement District (RCID) exceeds 800 cfs, water will flow east to Shingle Creek through a culvert (Guardo, 1992). This watershed does not contain any water control structures.

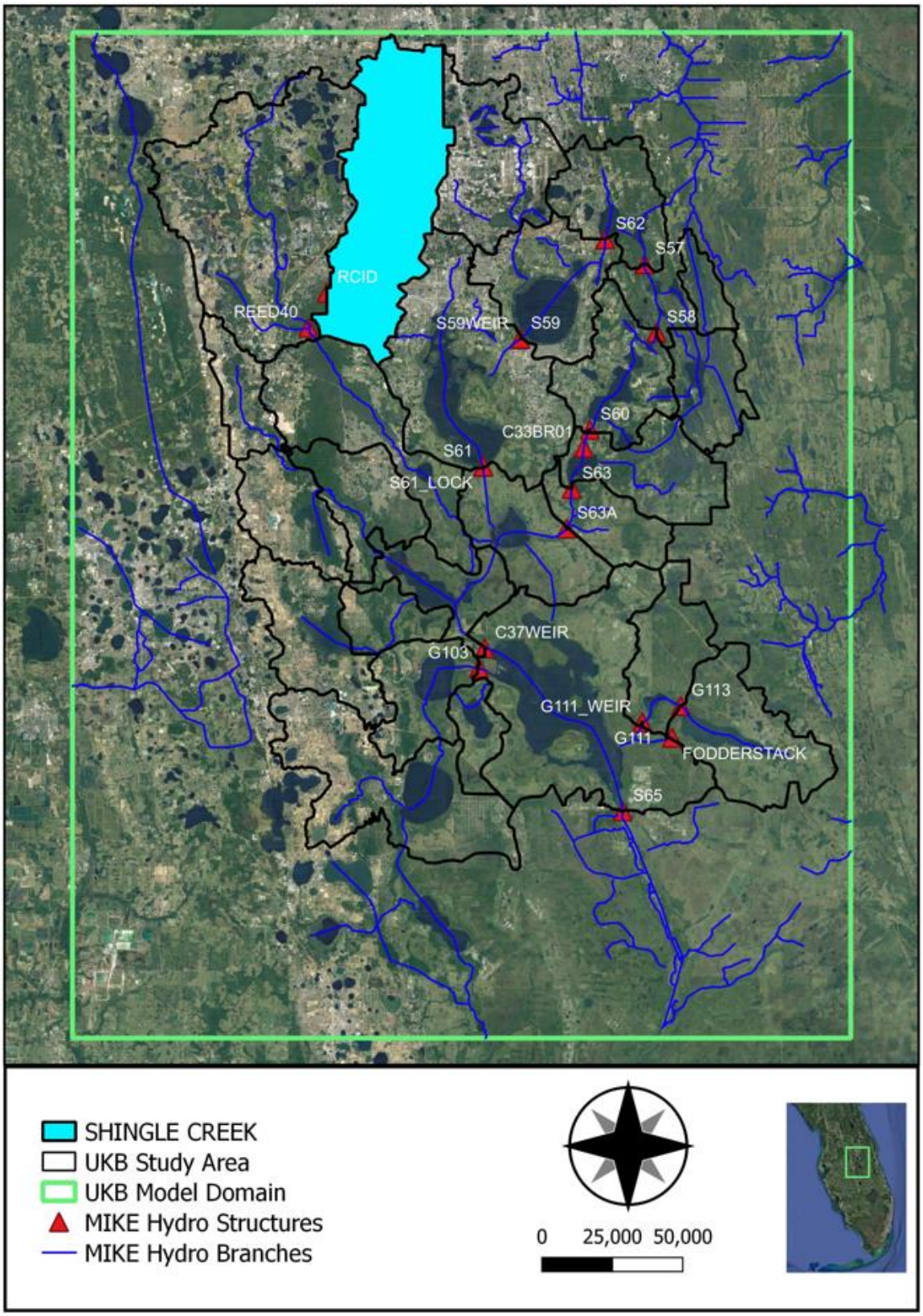

Figure 1-19: Shingle Creek Watershed in UKB

1.2.3 Horse Creek Watershed (Closed drainage area)

Horse Creek watershed which is in the western part of the UKB FPLOS Study Area has an approximate area of 26.5 square miles. Horse Creek flows into Peace River. The location of this watershed is shown in **Figure 1-20**. This watershed does not contain any water control structures. All the literature to date suggested this to be a closed drainage area; however, this has not been conclusively demonstrated.

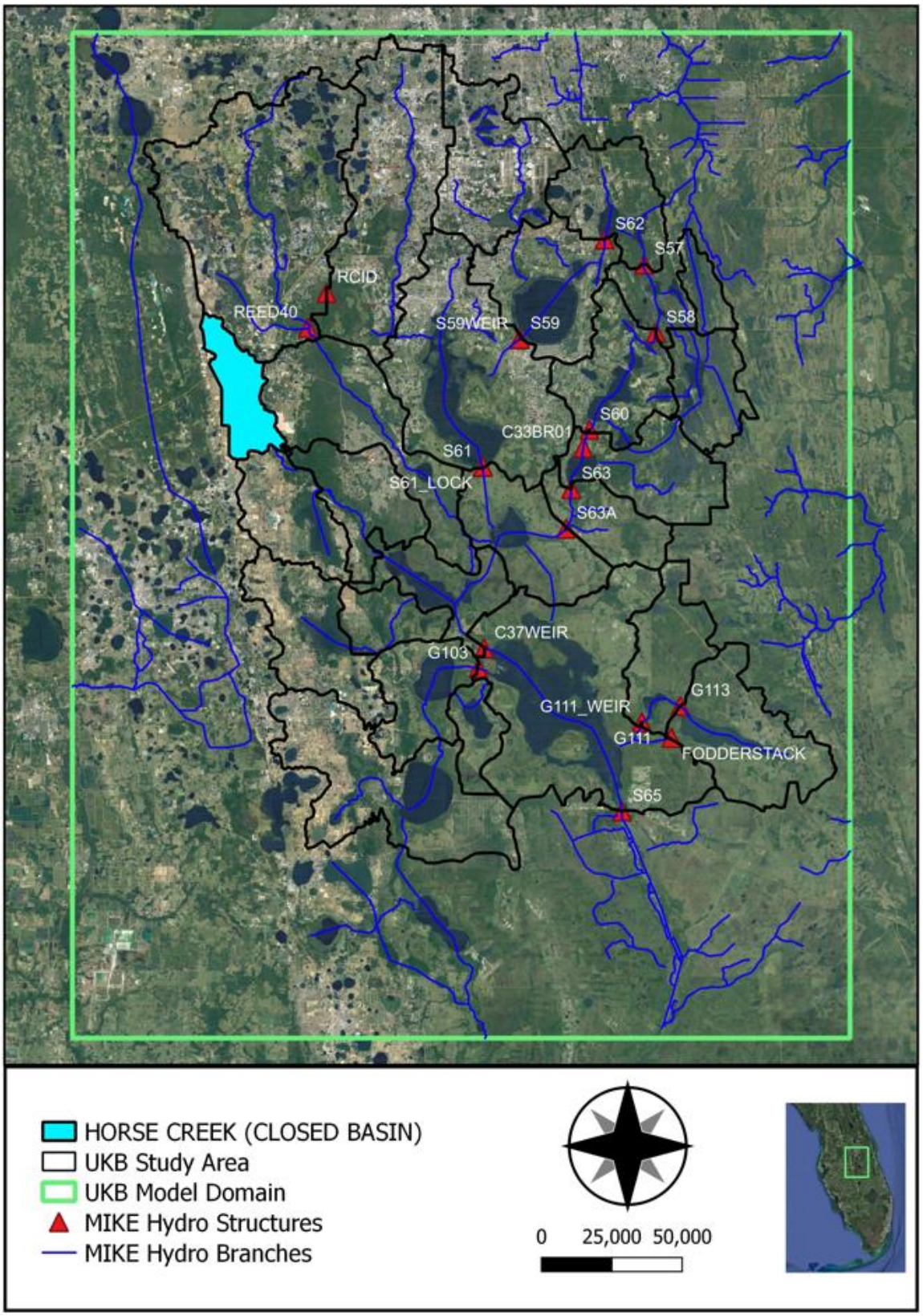

Figure 1-20: Horse Creek Watershed in UKB

1.2.4 Southern Chain Of Lakes

1.2.4.1 S63A watershed

The S63A watershed, shown in **Figure 1-21**, is upstream of Canoe Creek and connects lakes Gentry and Cypress. The C-34 canal extends from the S63 structure to S63A. Part of C-34 canal, also called Canoe Creek Canal is within this watershed.

S63A is operated to maintain an optimum headwater elevation of 55.41 ft NAVD88 (56.5 ft NGVD29) at C-34 canal, with MAGO and MPHD constraints (SFWMD, 2022c). When C-34 canal upstream of S-63A structure rises to 56.17 ft NAVD88 (57.25 ft NGVD29), S63A gates open at 6 inches per minute; when the water level rises or falls to 55.41 ft NAVD88 (56.5 ft NGVD29), the gates will become stationary; and when the water level reaches 55.1 ft NAVD88 (56.19 ft NGVD29), the gates close at the same opening rate of 6 inches per minute.

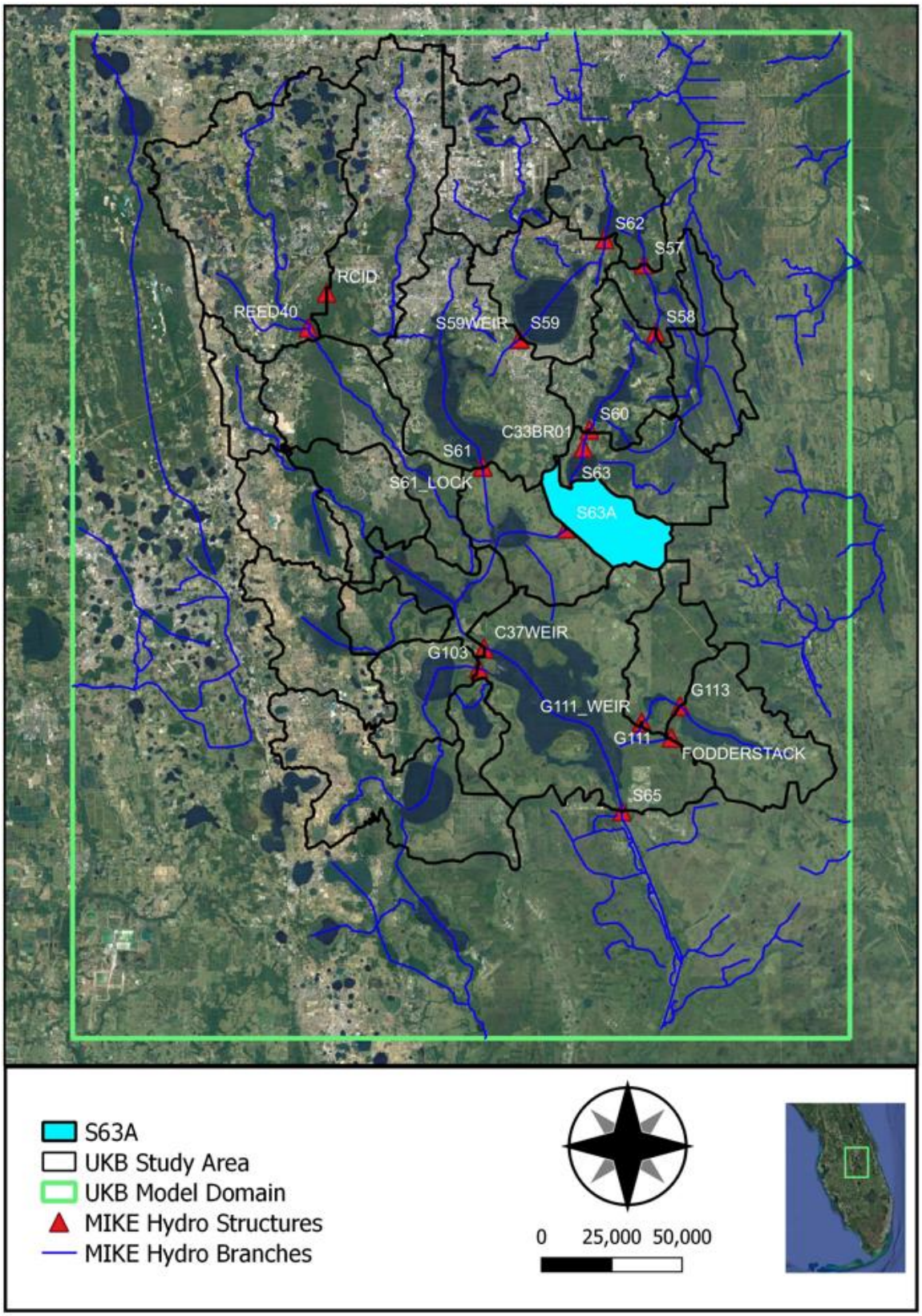

Figure 1-21: S63A Watershed in UKB

1.2.4.2 Canoe Creek Sub-watershed

This is a sub-watershed within the S63A watershed; located downstream of the S63A structure and upstream of where the C-34 canal enters Lake Cypress. There are no control structures in this sub-watershed, and it drains to C-34 canal. The topography in this sub-watershed is relatively flat and therefore has been identified as a separate drainage area in some of the past studies (Guardo, 1992). UKB FPLOS does not have a separate area to produce performance metrics for Canoe Creek sub-watershed.

1.2.4.3 Lake Cypress Watershed

The location of Lake Cypress Watershed is shown in **Figure 1-22**. Flows from Lake Gentry on the eastern chain and from Lake Tohopekaliga on the western chain converge at Lake Cypress through Canoe Creek Canal and South Port Canal (C-34 and C-35 canals, respectively). Lake Cypress receives approximately 30 percent of the flows from Reedy Creek; 70% of the Reedy Creek flows enter Lake Hatchineha (Fan A., 1986). The watersheds flowing into the creeks are also included.

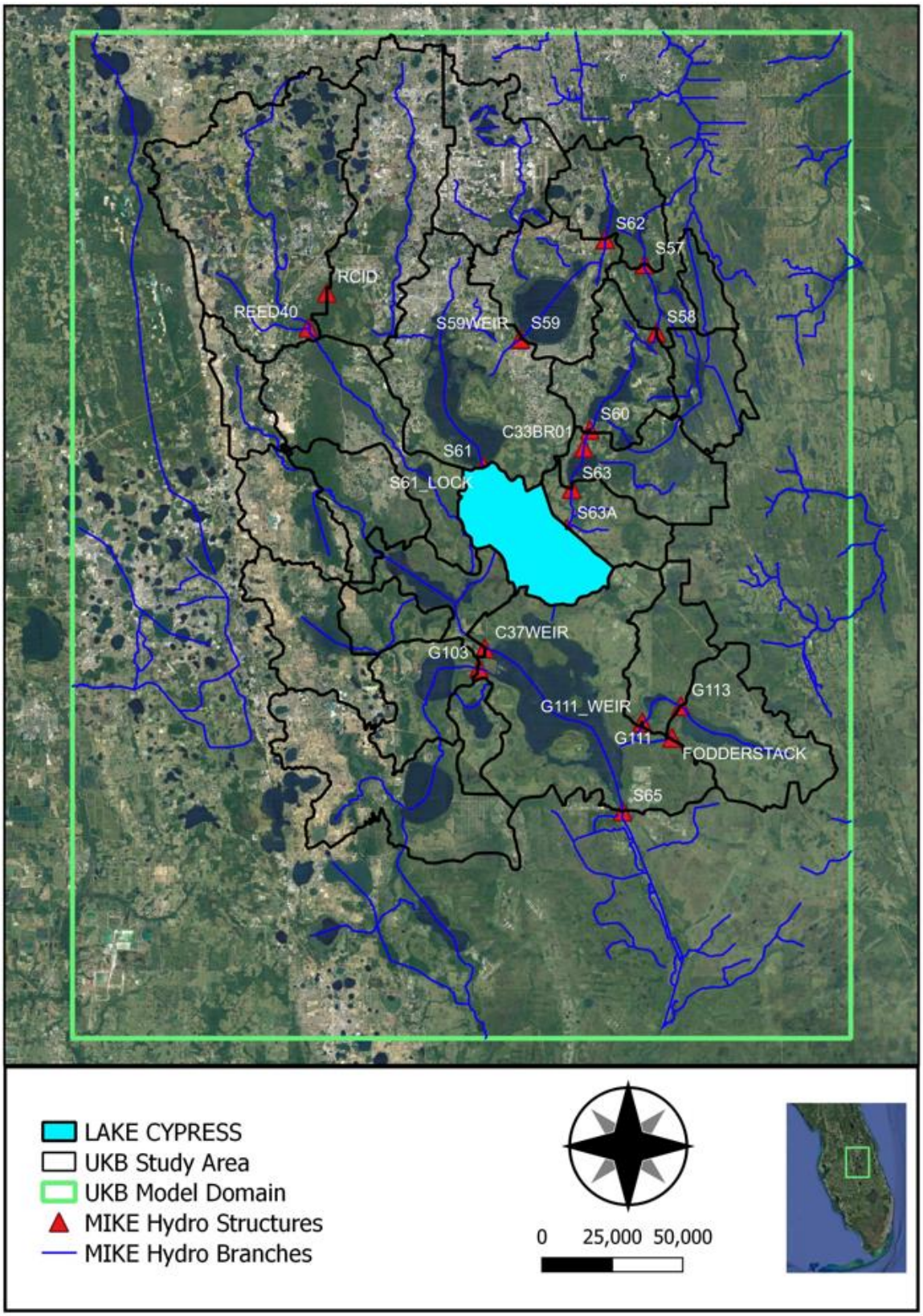

Figure 1-22: Lake Cypress Watershed in UKB

1.2.4.4 Lake Hatchineha Basin

Lake Hatchineha Basin includes 3 SFWMD watersheds: Lake Hatchineha, Lake Marion and Marion Creek; **Figure 1-23** shows the location of Lake Hatchineha Basin within the UKB. The western portion of this basin drains to Lake Marion which flows into Lake Hatchineha through Marion Creek. Snell Creek drains to Marion Creek downstream of Lake Marion. C-36 canal, also known as Cypress-Hatchineha Canal connects Lake Cypress to Lake Hatchineha. C-37 canal connects lakes Hatchineha and Kissimmee. The S65 structure, the outlet of Lake Kissimmee, regulates the lakes Kissimmee, Cypress and Hatchineha (KCH) system.

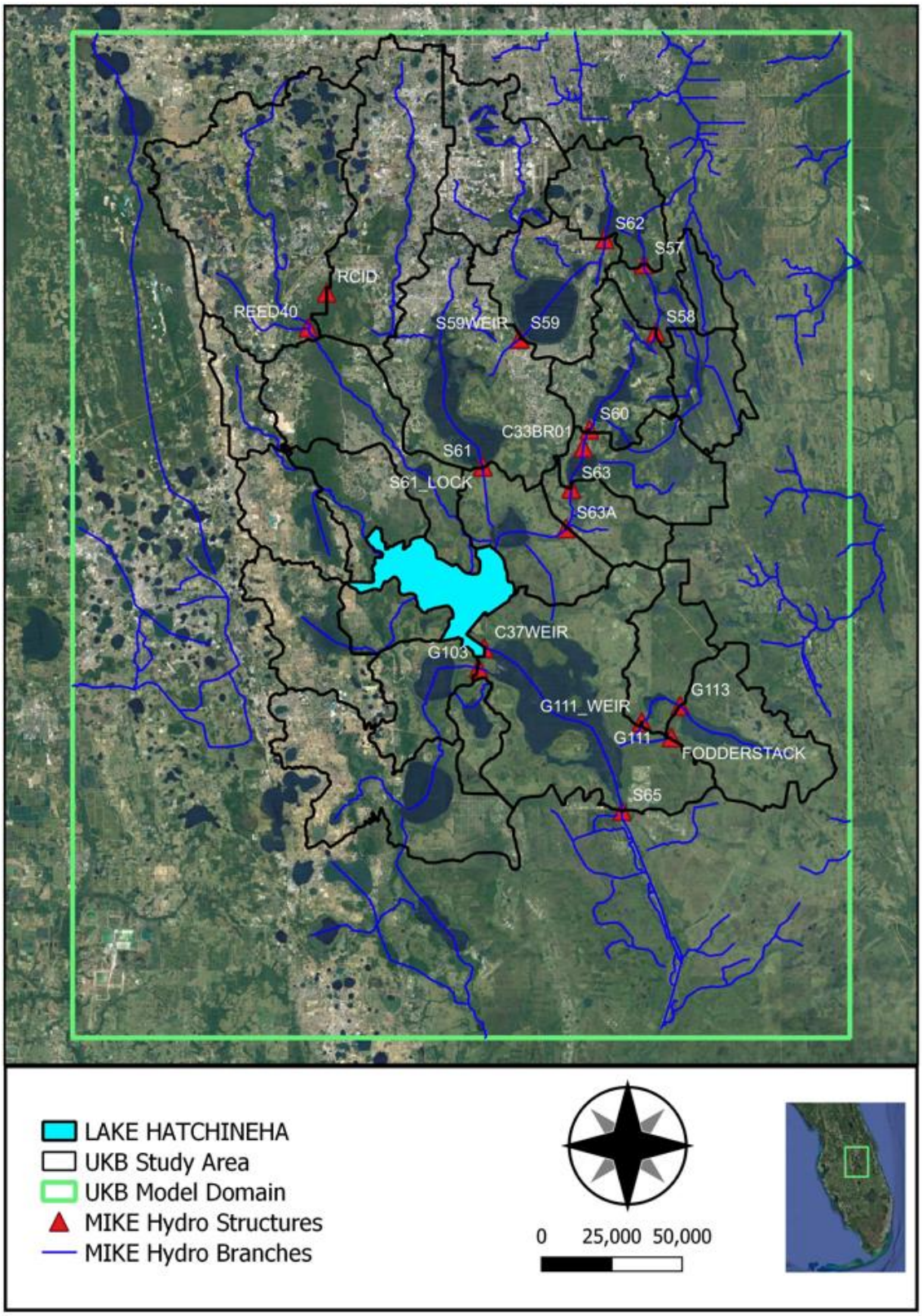

Figure 1-23: Lake Hatchineha Watershed in UKB

1.2.4.5 Lake Pierce Watershed

Lake Pierce Watershed, shown in **Figure 1-24**, flows into Lake Hatchineha through Catfish Creek. This lake does not contain any water control structures.

1.2.4.6 Reedy Creek Watershed

Reedy Creek Watershed, with its many small lakes, forms the northwest headwaters of the UKB. Walt Disney World, located in the RCID is the major driver of the hydrology in this watershed. The location of this watershed is shown in **Figure 1-25**. Reedy Creek runs for approximately 30 miles and then splits into two branches near Cypress Lake - one enters Cypress Lake and the other (Dead River) enters Lake Hatchineha. Historical data analysis showed 30% of the flow enters Lake Cypress and 70% percent flows to Lake Hatchineha (Fan, A. 1986). During extreme rainfall events, water from Reedy Creek sheet flows into Shingle Creek through the Reedy Creek swamp, north of Orange Blossom Trail. This watershed does not contain any SFWMD structures; however, there are several structures operated by RCID and Walt Disney World.

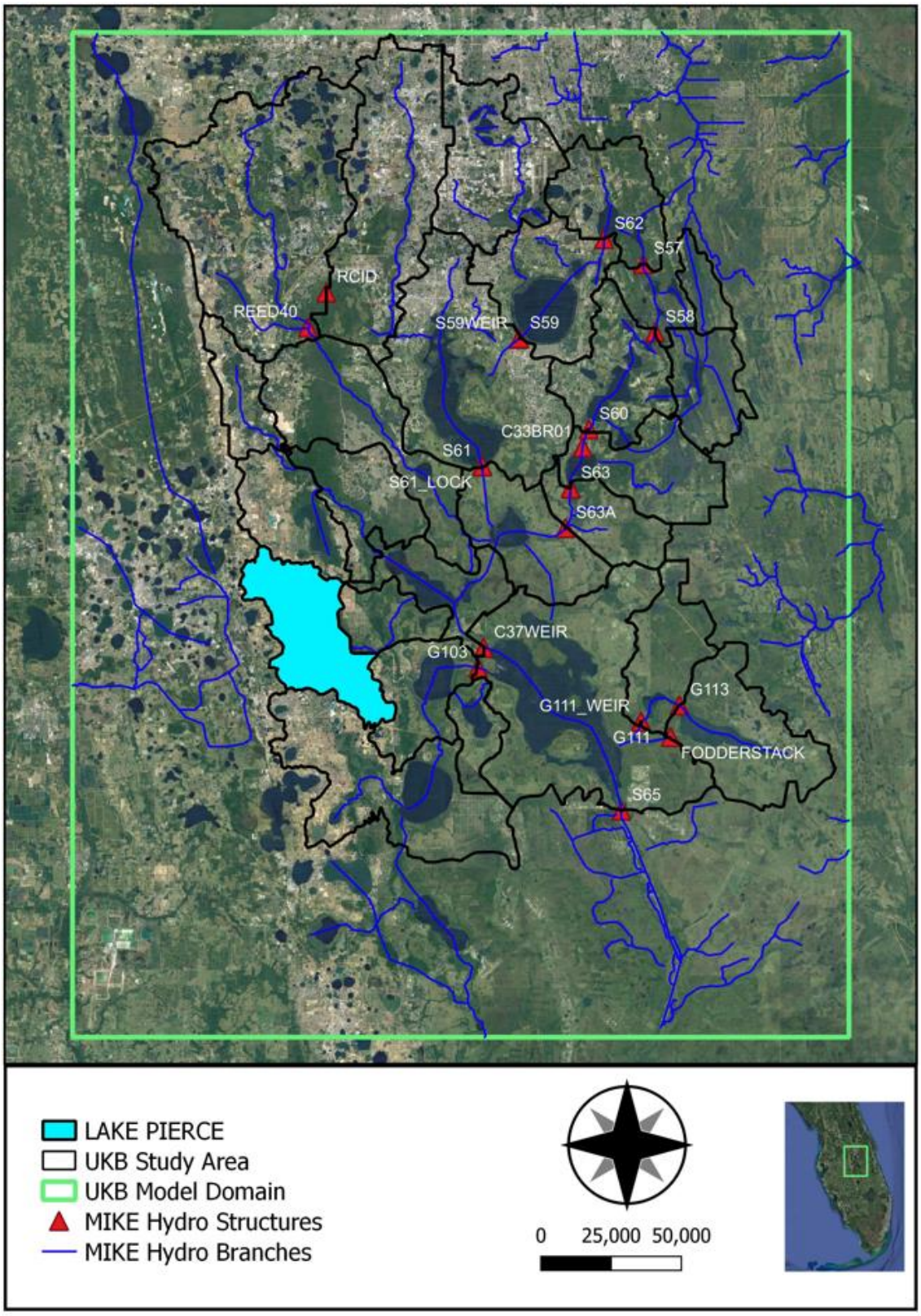

Figure 1-24: Lake Pierce Watershed in UKB

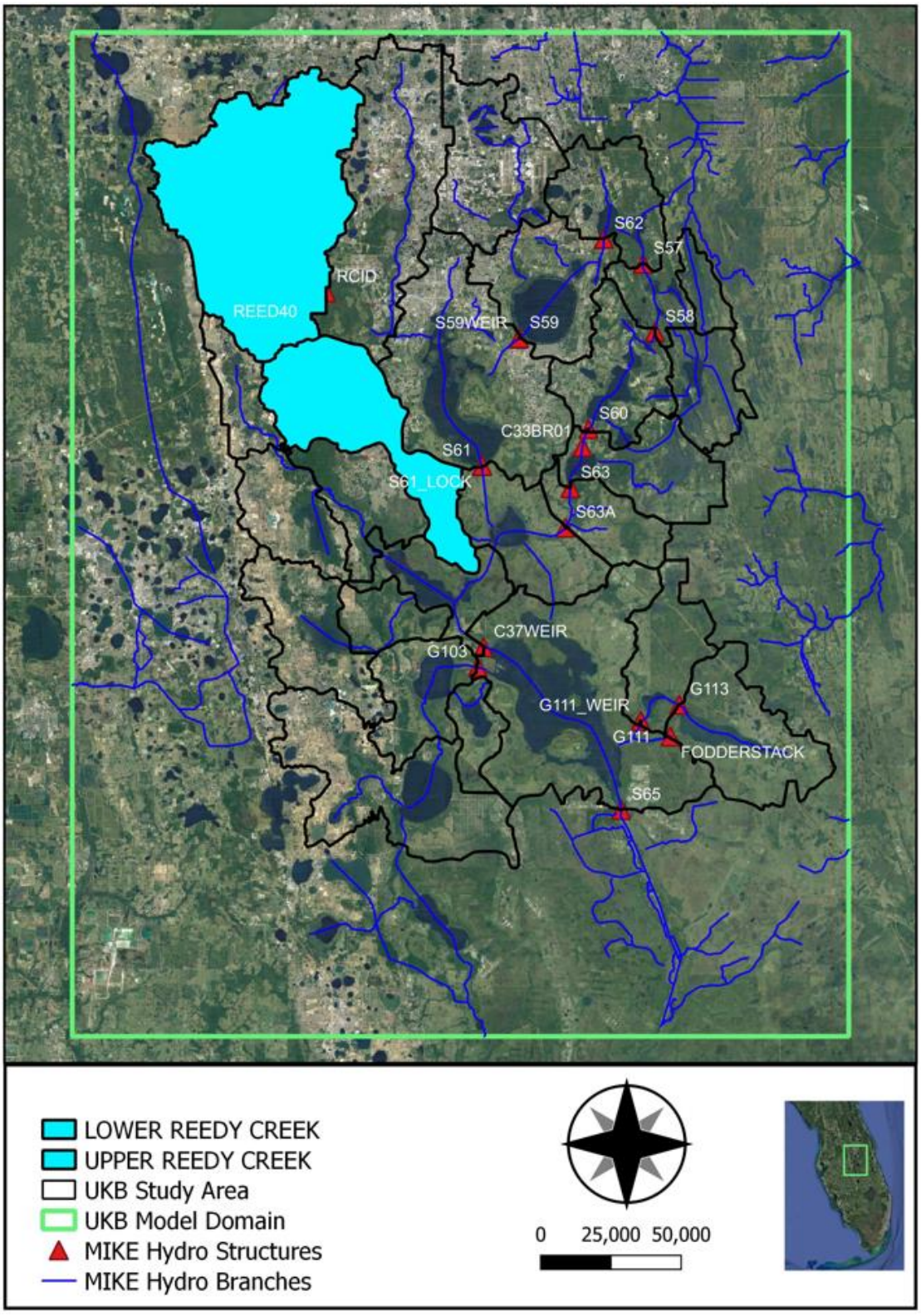

Figure 1-25: Upper and Lower Reedy Creek Watersheds in UKB

1.2.4.7 Lake Marian Watershed

Lake Marian is connected to Lake Jackson through the G113 structure, as shown in **Figure 1-26.** Lake Jackson discharges into Lake Kissimmee through Jackson Canal. Lake Marian is also connected to Lake Kissimmee through Fodderstack Slough. The G-113 structure prevents over draining of Lake Marian and maintains the water levels above 56.78 ft NAVD88 (58.0 ft NGVD29); the optimal range for Lake Marian water levels is 56.78 to 59.28 ft NAVD88 (58.0 – 60.5 ft NGVD29). There is another structure in this watershed, located west of Lake Marian on Fodderstack Slough: Fodderstack Slough Weir structure with elevation of the crest at 34.0 ft NAVD88 (35.22 ft NGVD29).

1.2.4.8 Lake Weohyakapka Basin

Lake Weohyakapka Basin includes part of Lake Rosalie Watershed and Lake Weohyakapka Watershed, shown in **Figure 1-27**. Weohyakapka Creek connects lakes Rosalie and Weohyakapka. Several small lakes - Wales, Moody, Easy, Leonore are on the western boundary of Lake Weohyakapka Watershed. Tiger Creek drains the areas on the west of Lake Weohyakapka watershed, including these lakes, and flows into Lake Weohyakapka on the west. When the stage in Lake Weohyakapka gets higher than 64.7 NAVD88 (63.5 NGVD), water flows south to Lake Arbuckle through Blue Jordan Swamp.

Lake Weohyakapka is not regulated by any water control structures, but part of Lake Rosalie watershed is regulated.

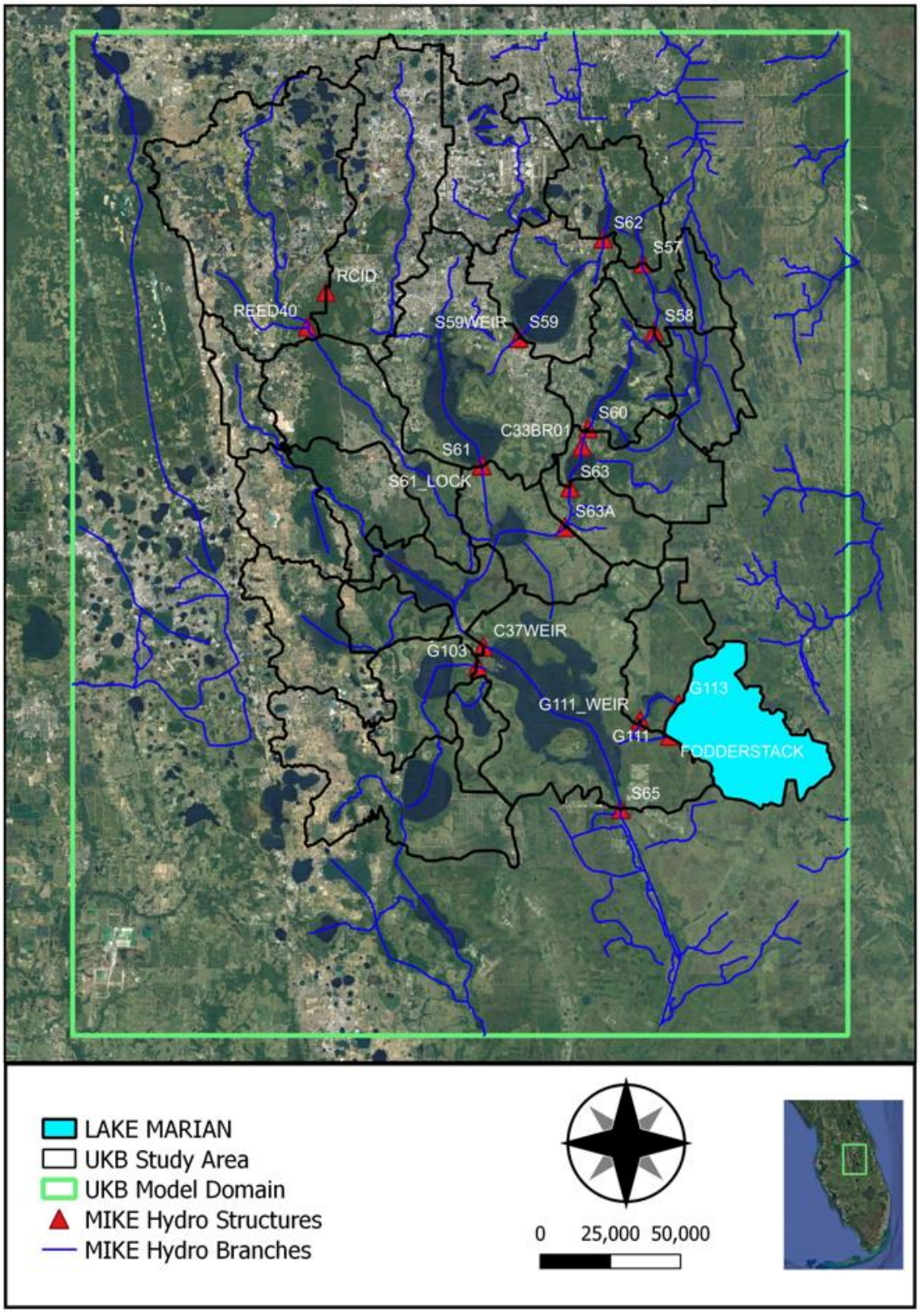

Figure 1-26: Lake Marian Watershed in UKB

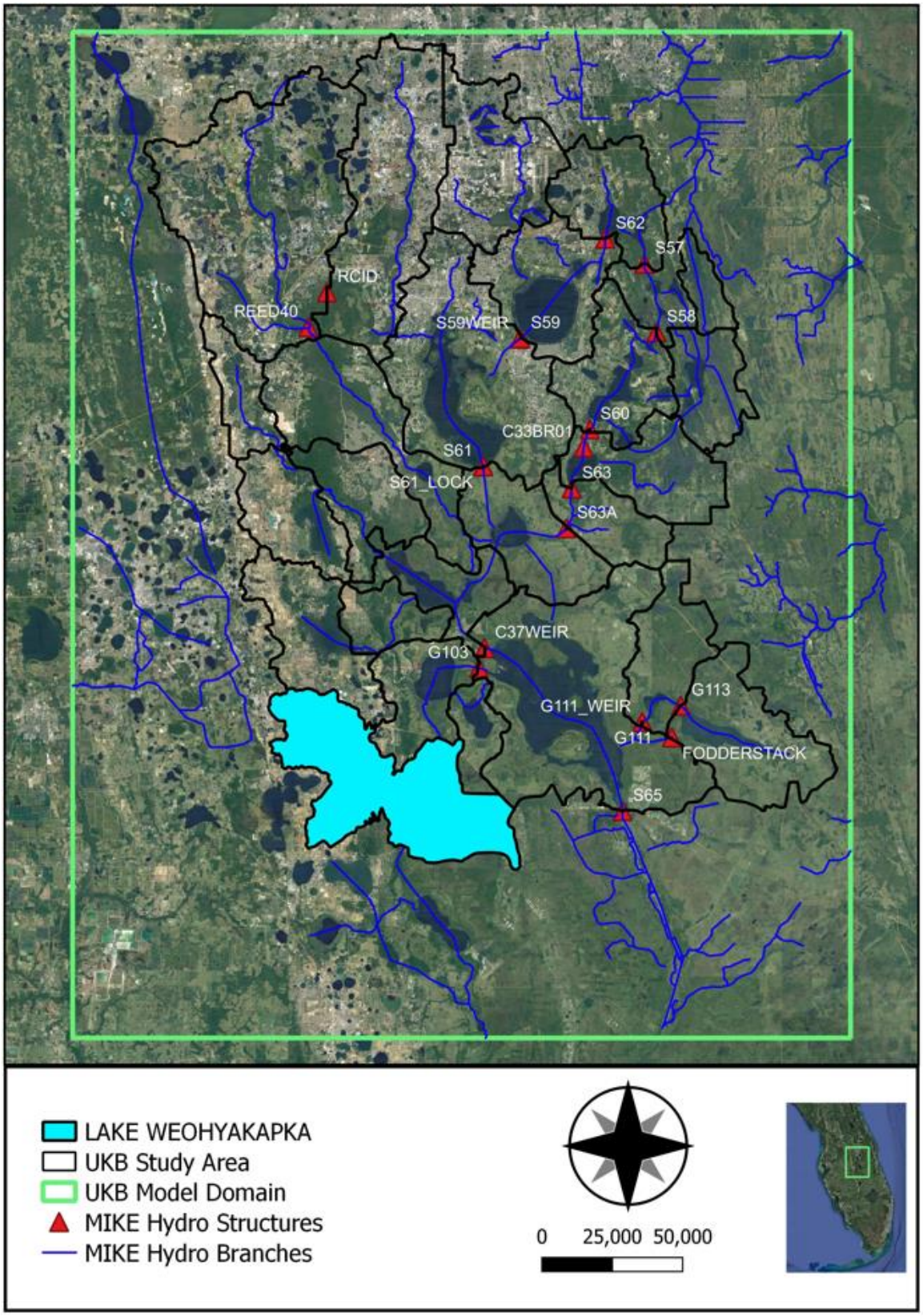

Figure 1-27: Lake Weohyakapka Watershed in UKB

1.2.4.9 Lake Kissimmee Watershed

Several watersheds surround Lake Kissimmee Watershed, shown in **Figure 1-28**. The two managed watersheds that discharge to this watershed are lakes Jackson and Rosalie. Other connected watersheds are lakes Cypress, Hatchineha, Tiger Lake, Marian and S63A. C-37 canal connects Lake Hatchineha to Lake Kissimmee and Zipperer Canal connects Lake Rosalie to Lake Kissimmee through Structure G103.

The S65 structure at the outlet of Lake Kissimmee discharges all the UKB flows to the LKB. S65 regulates the 3 lakes in the UKB – lakes Cypress, Hatchineha and Kissimmee; there are no structures separating these 3 lakes. The regulation schedule, called the Interim Regulation Schedule at the S65 structure, is shown in **Figure 1-29**.

For flood control operations, S65 is operated to return the lake to regulation schedule when the water level in Lake Kissimmee exceeds regulation schedule and is within 0.5 feet above the regulation schedule, as shown in Figure 1-29. The releases are made with maximum releases subject to MAGO and MPHD constraints (SFWMD, 2022c).

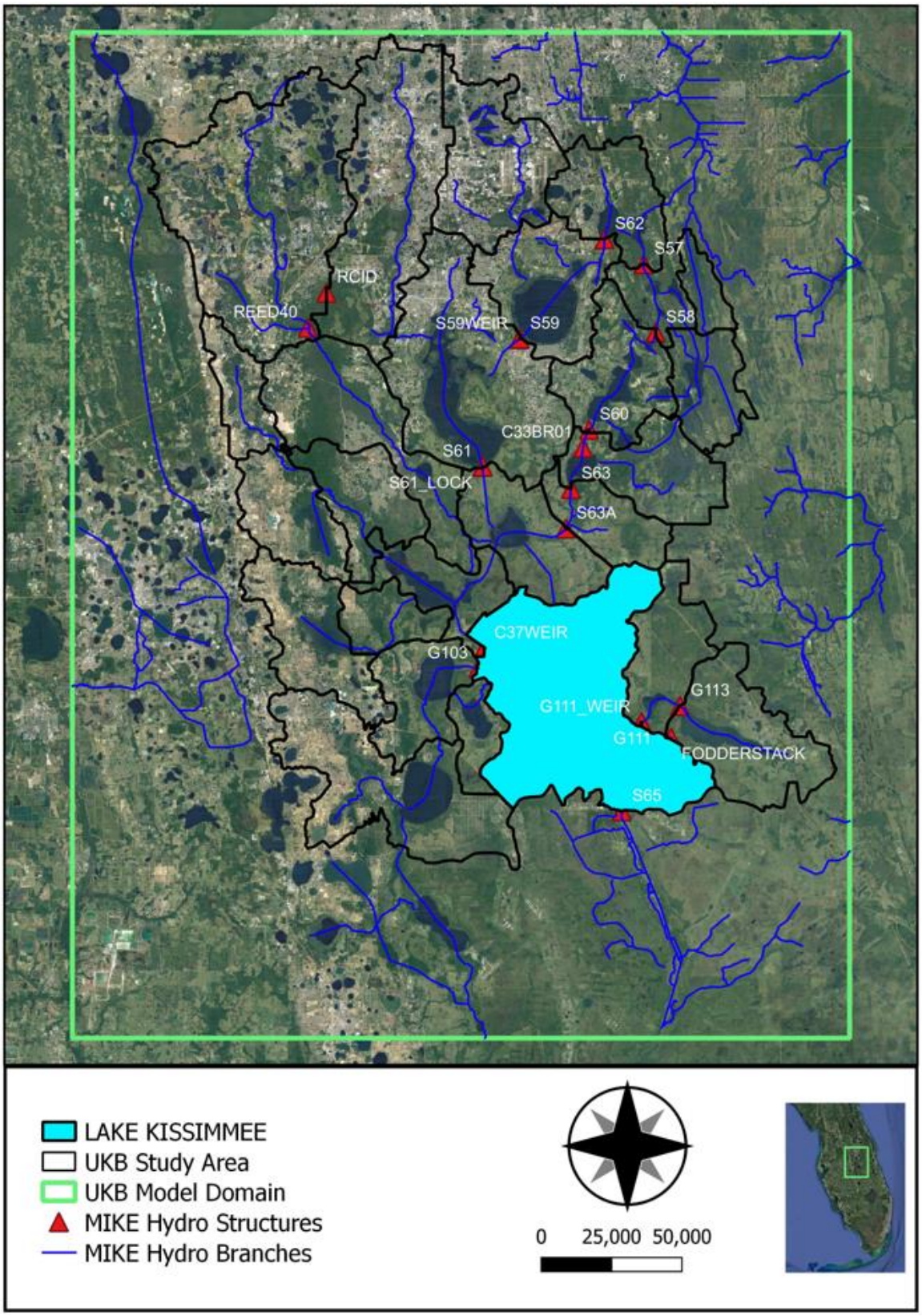

Figure 1-28: Lake Kissimmee Watershed in UKB

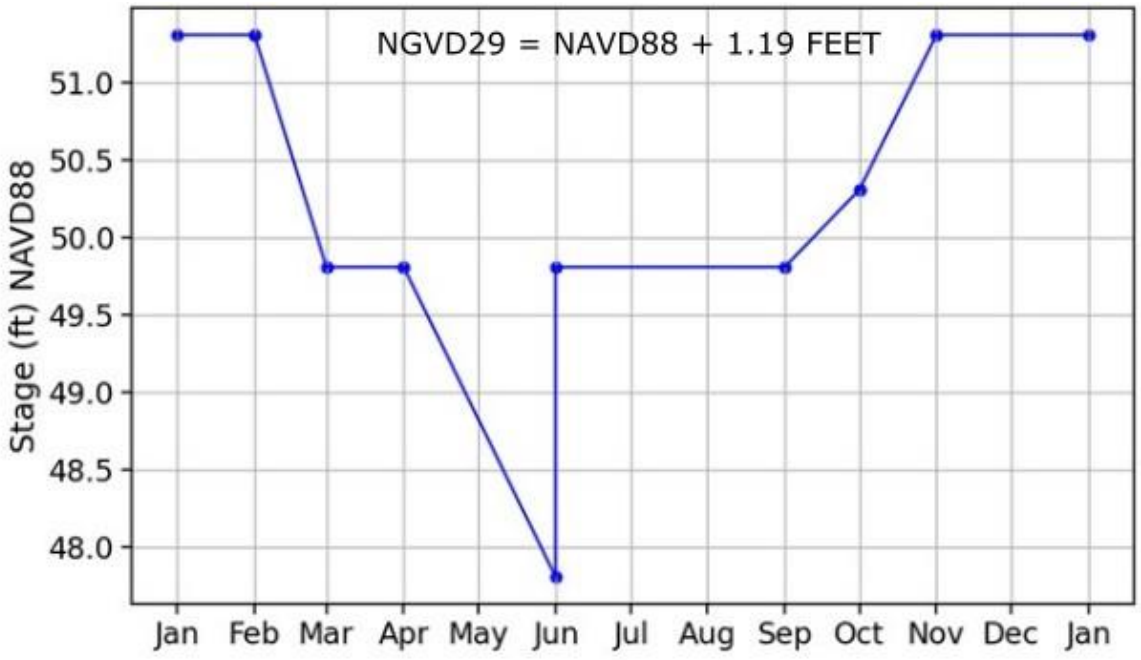

Figure 1-29: Lake Kissimmee, Hatchineha, and Cypress Regulation Schedule at S65

When Lake Kissimmee water level exceeds regulation schedule and is within 0.5 feet above the regulation schedule, a release schedule, with MAGO and MPHD constraints (SFWMD, 2022c), is followed to return the lake to regulation schedule within 15 days. Releases at S65 range from 3,000 cfs to 11,000 cfs, depending on the watershed runoff between S65 and S65A so flow at S65A does not exceed 11,000 cfs. The 3000 cfs is called the firm capacity, which is the discharge that can be made at S65 at any time, as needed. Discharges above the firm capacity can be made only if there is no flooding in the LKB.

Observed stages and flows will be analyzed to determine the relationship between S65, S65A and flooding depth determination so the relationship can be coded in the existing conditions model.

When Lake Kissimmee water levels are below the schedule line, releases are made following the quantities in the schedule to meet needs of the river restoration, navigation and water supply. The target minimum flow to the Kissimmee River is 300 cfs at S65A.

1.3 Hydrogeology of the Study Area

Florida is underlain by a thick sequence of carbonate and clastic sedimentary rocks ranging in geologic age from Paleocene to recent. The regional hydrogeologic framework of central Florida consists of four main hydrostratigraphic units (Reese and Richardson, 2008); from top to bottom, they are:

- Surficial Aquifer System (SAS), composed of mostly sand
- Intermediate confining unit (ICU), made up of mainly fine-grained clastic sediments

- Floridan Aquifer System (FAS), consisting of mainly limestone and dolostone
- Sub-Floridan confining unit (SFCU), composed of evaporites

Miller (1986) subdivided the FAS into: an Upper Floridan aquifer (UFA), a series of middle confining units (MCUs) and a Lower Floridan aquifer (LFA).

The Central Florida Water Initiative (CFWI) is a collaborative water supply planning effort among Florida's three largest water management districts (South Florida, St. Johns River and Southwest Florida water management districts), the Florida Department of Agriculture and Consumer Services (DACS), the Florida Department of Environmental Protection (DEP), and water utilities, environmental groups, business organizations, agricultural communities and other stakeholders. The CFWI Planning Area covers five counties: Orange, Osceola, Polk, Seminole and southern Lake. One of the activities in CFWI was the development of a groundwater flow model that encompasses the entire CFWI area. This model referred to as ECFTX v1.0, extends from the Gulf of Mexico on the west to the Atlantic Ocean on the east and from southern Marion County in the north to the Highlands/Glades County line in the south (CFWI, 2020). Vertically, this model simulates three-dimensional groundwater flow in the surficial aquifer, intermediate aquifer/confining unit, Upper and Lower Floridan aquifers, and associated middle confining units (CFWI, 2022).

The CFWI ECFTX model was used for hydrostratigraphy definition and initial hydrogeologic properties; the extent of this model domain is shown in **Figure 1-30**. The naming convention of the hydrogeologic units in the UKB FPLOS Model follows the ECFTX model. **Figure 1-31** shows the hydrostratigraphic conceptualization in the central Florida area.

The surficial aquifer (SA) is unconfined and consists of unconsolidated sediments. The thickness of the SA is generally less than 50 feet over much of the UKB FPLOS study area, except along ridges on the west. Lakes Wales Ridge is on the western part of the UKB FPLOS Model domain, where the SAS thickness ranges from 200-300 ft and is highly permeable. The depth-to-the-water table is shallow, ranging from topographic land surface to approximately 10 ft in most of the UKB FPLOS Model domain, except along the ridge areas where the water table ranges from about 10-50 ft deep.

The Intermediate Confining Unit (ICU) underlies the SAS; its thickness varies in Florida, and it is productive in the western regions of Florida (Reese and Richardson, 2008). In the UKB FPLOS Model domain, the ICU is leaky in the west side including areas under Lake Tohopekaliga and is fully confining in the eastern part of the domain. Thickness of ICU in the UKB FPLOS model domain ranges from less than 10 ft to 300+ ft (Sepulveda et al., 2012).

The UFA-upper is the uppermost permeable zone of the FAS and is predominantly limestone (CFWI HAT, 2020). The UFA-upper is approximately 100 ft thick in the central Florida region, and slightly thicker towards the southeast part of the UKB FPLOS Model domain.

The UFA is the lowest geologic layer in the UKB FPLOS Model. The LFA and its confining unit are not included in the UKB FPLOS Model domain.

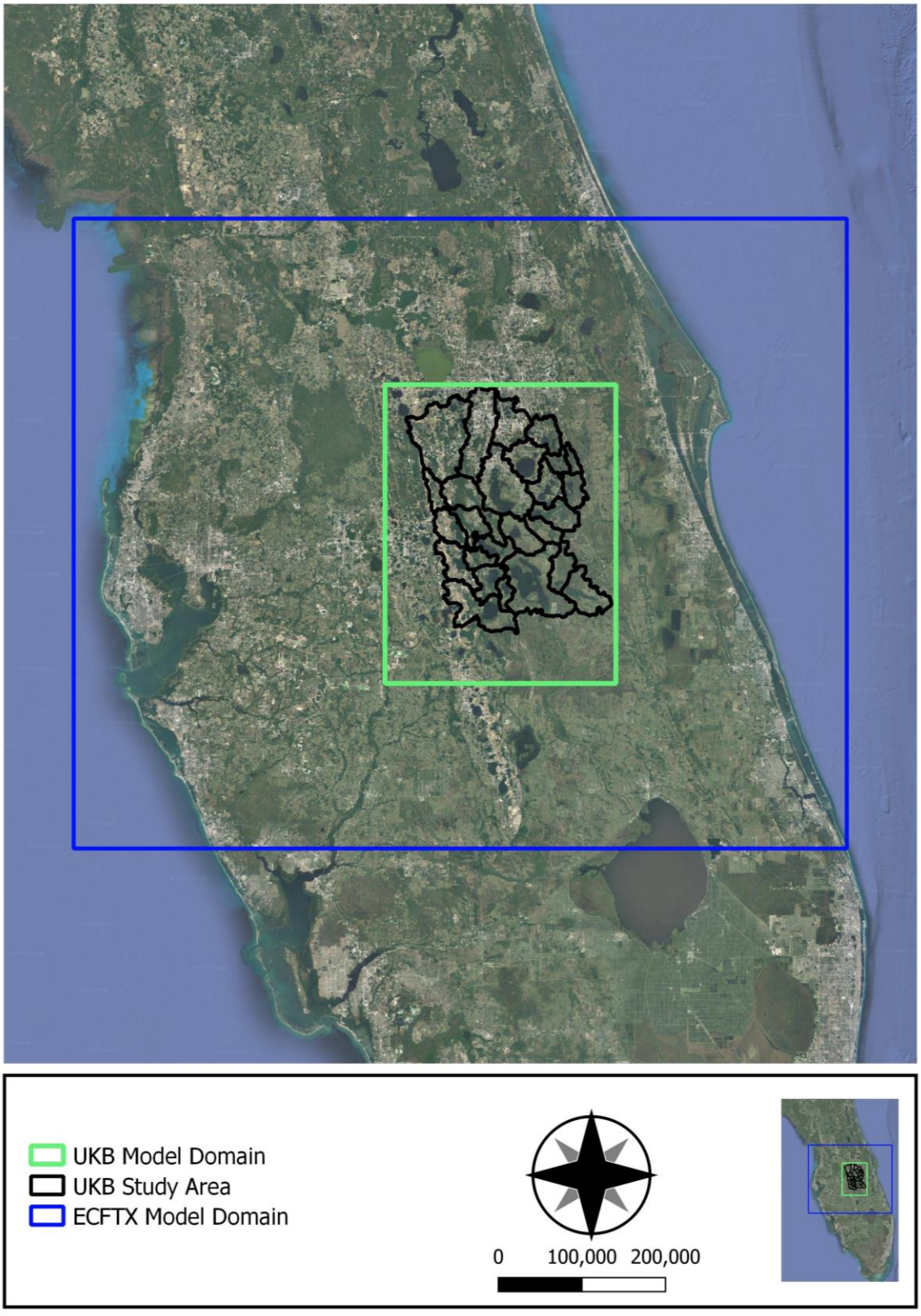

Figure 1-30: UKB FPLOS Model Domain and Study Area within ECFTX Model Domain

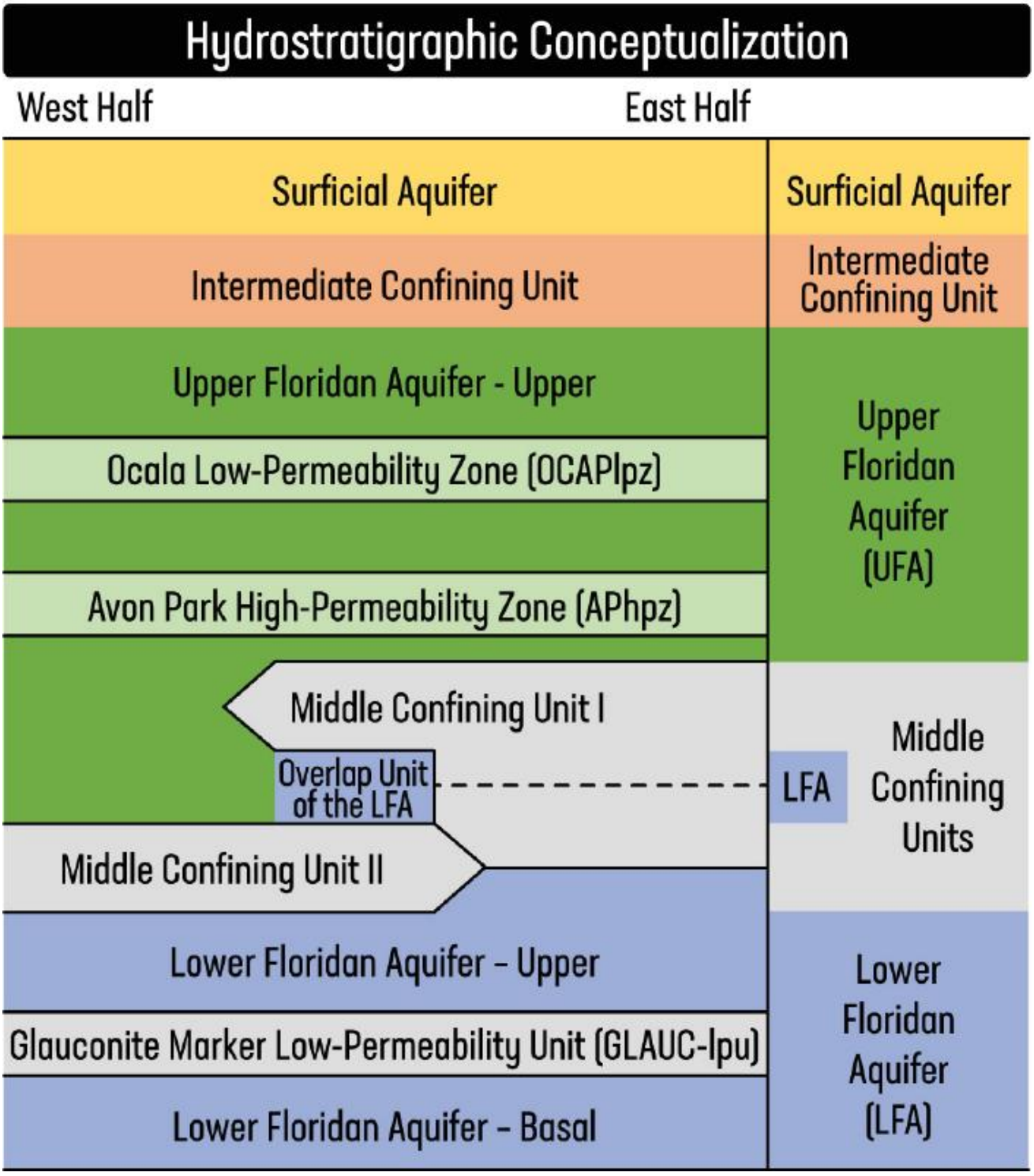

Figure 1-31. Hydrogeology in Central Florida Area (taken from ECFTX report)

1.4 Lake Management Areas

Hydrology of UKB is also described in some literature on Kissimmee by the term 'Lake Management Area' (LMA), which describes a major lake and its surrounding drainage areas. The LMAs are separated by SFWMD operated water control structures, except the KCH system. **Table 1-2** shows the LMAs and their associated structures. The LMAs are based on lakes and structures; the flows are structure flows except C-36 and C-37 canals. **Table 1-3** shows the LMAs and their associated sub-watersheds.

Much of the description and presentation of modeling and results in this report at the level of LMAs.

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Table 1-2: Lake Management Areas and their structures

Management Area	Upstream Flows		Downstream Flows		Stage Observations	
Alligator Lake LMA			S60	S58	S58	S60
Myrtle LMA	S58		S57		S57	S58
Hart LMA	S57		S62		S62	S57
Gentry LMA	S60		S63A		S63A	S60
East Toho LMA	S62		S59		S59	S62
Toho LMA	S59		S61		S61	S59
Cypress LMA	S63A	S61	C-36 Canal		S61	S63A
Hatchineha LMA	C-36 Canal		C-37 Canal			
Kissimmee LMA	C-37 Canal		S65			S65

Table 1-3: LMAs and Sub-Watersheds

LMA	Sub-watersheds IDs (Refer to Figure 1-3 and Table 1-1)					
Lake Alligator	21					
Myrtle	19	26	13			
Hart	23					
Gentry	7					
East Toho	20	25				
Toho	18	22				
Cypress	9	1	16	17		
Hatchineha	15	3	14	6	11	12
Kissimmee	4	5	10	2	24	

1.5 Previous and Related Studies and Modeling Efforts

This section contains two tables that detail the data and models utilized in this project. These encompass various aspects such as knowledge, data sets, files, or a combination of these elements section has two tables with data and models that were used in this project, in terms of knowledge, data, files or all of them.

Table 1-4: Model Sources for UKB FPLOS

Model Application/ Project	Model Software	Dates	Source
Econlockhatchee River Watershed	HSPF	Calibration: 2007 -2016 Validation: 1997 - 2006	SJRWMD
Upper St. Johns River	HSPF	Calibration: 2007 -2016 Validation: 1997 - 2006	SJRWMD
Lake Marion Creek SFWMD	HEC	Design event model (Model/report dated 7/31/13)	SWFWMD
Peace Creek Floodway Analysis	HEC-RAS	100-year design event analysis (Model/Report dated 2014)	SWFWMD
Kissimmee Basin Modeling and Operations Study (KBMOS)	MIKESHE - MIKE 11	Calibration: 1994-2004	SFWMD
Kissimmee Basin H&H Bureau Model (KB HH FB16)	MIKESHE - MIKE 11	Calibration: 1994 - 2016 Base Conditions: 1965 - 2016	SFWMD
Fanny Bass, Ham Brown, Old Hickory, Sherwood Forest, Shingle Creek	ICPR	Design event models for 24-hr rainfall events used in master plan update; report dated February 2014	Osceola County
Masterplan studies models: Boggy Creek, Lake Hart, Reedy Creek, Shingle Creek	CHAN	Design event models for rainfall events used in master plan update; models and reports dated 1996 through 2018	Orange County
East-Central Florida Transient (ECFTX) Model	SEAWAT/MODFLOW	Calibration: 2004 – 2014 Report dated February 2020	Central Florida Water Initiative Website

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Table 1-5: Data Sources for UKB FPLOS

Dataset	Description	Year	Source
Controlled water level	USJ controlled water levels - daily - yearly	Received May 2022	SJRWMD
SJRWMD GIS	SJR sub watersheds + conveyance	Received May 2022	SJRWMD
Rainfall data	2011 and 2017 NEXRAD data (hourly)	Received May 2022	SJRWMD/ SFWMD
SJRB Hydro Report	Watershed hydrology modeling of the St. Johns River Basin	Received May 2022	SJRWMD
Basin Atlas	Atlas of upper Kissimmee surface water management basins	Atlas dated 1996	SFWMD
Field Reconnaissance of Flows in Kissimmee basin	Field Reconnaissance of Flows in Kissimmee basin	Report dated September 2017 (Hurricane Irma)	SFWMD
Atlas of Flow Computations at hydraulic structures in the South Florida Water Management District	Description of current flow computation procedures at the SFWMD for hydraulic structures (Spillways, pumps, culverts, and weirs).	2015	SFWMD
Land Use data	Land Use data	Received May 2022; land use based on 2019 imagery/photos	SFWMD
SFWMD GIS data	Canals, roads, structures, watersheds etc	Received May 2022	SFWMD
Kiss_Rain_Event_Oct11	Back calculated runoff, stage estimates, hydrologic analysis	Received May 2022; analysis and report dated November 2011	SFWMD
Shingle Creek GIS	Wetlands, ditch, structures, bridges within Shingle Creek	Received June 2022; Data downloaded by Osceola County in May 2022, updated for Shingle Creek modeling in 2021	Osceola County through SFWMD
Structure Information	Structure book for selected structures in Upper Kissimmee Basin	Received September 2022; data compiled by SFWMD in July 2022	SFWMD
Master Plan Update	Osceola County Master Surface Water Management Plan	Received June 2022; Report dated February 2014	Osceola County
Osceola County GIS	Geodatabase that contains drainage ditches, sub watersheds, channels, ponds, culverts, structures within Osceola County	Received June 2022; Data downloaded by Osceola County in May 2022	Osceola County

Dataset	Description	Year	Source
Masterplan studies	Model reports for Boggy creek, Lake Hart, Reedy creek, Shingle creek	Master plan update report: Reports dated 1996 through 2018	Orange County
Technical memorandum of model migration	Technical memorandums for Cypress creek, Reedy creek, Lake Hart – reports of ICPR3 to ICPR4 model migration	August 2014	Orange County
Rating curve	Rating curve for S-40 (a.k.a. REED40)	Received May 2022	Reedy Creek Improvement District (RCID)
SWFMD NEXRAD rainfall	NEXRAD 15 min rainfall data for SWFWMD	Data downloaded from SWFWMD website in July 2022	SWFWMD
SJRWMD Groundwater monitoring network	Shapefile for the location of groundwater monitoring network	Data downloaded from SWFWMD website in July 2022	SJRWMD
Drainage Wells	Drainage Wells location, average inflow & outflow rates used in ECFTX model	ECFTX Data received from SJRWMD (Wei Jin) in September 2022	SJRWMD
Drainage Wells	Drainage Wells studies by SFWMD	(SFWMD) Data received in September 2022	Liquid Solutions Group
Observed groundwater level data	Observed groundwater level time series	Received from SJRWMD in August 2022	SJRWMD
Observed groundwater level data	Observed groundwater level time series	Received from SWFWMD in August 2022	SWFWMD
Groundwater data collection sites	GIS dataset for location of groundwater measurement locations	Received from SWFWMD in August 2022	SWFWMD
Bridges Data	GIS and design data of the bridges	Received dimensions and spatial location of the bridges from Florida Department of Transportation (FDOT) in June-September 2022	FDOT
Spillway coefficients	SFWMD updated rating equations and coefficients	2023, email exchange with Hazen	SFWMD
Bathymetry data	SFWMD Streamgauging Unit bathymetry data measurement after Hurricane Ian and Tropical Storm Nicole – to check scouring	2023	SFWMD

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2. Modeling Objectives

The SFWMD is conducting an evaluation of their regional water management infrastructure to determine the flood protection level of service (FPLOS) that is being provided by their existing SFWMD operated infrastructure under current and future conditions. Per SFWMD, "The flood protection level of service (FPLOS) describes the level of protection provided by the water management facilities within a watershed considering sea level rise (SLR), future development, and known water management issues in each watershed.". The results from this project can be used by SFWMD, local, state, and federal agencies to identify flood mitigation measures; assess where improvements to the design, retrofit, construction and/or operation of water management structures are required or would be beneficial. An integrated MIKE SHE and MIKE 1D numerical model software was selected to provide analysis of the FPLOS in UKB.

The UKB FPLOS Model was developed and calibrated to effectively simulate the main components of UKCOL hydrology. The model simulates all major water budget components critical to central Florida hydrology which include evaporation, transpiration, overland, unsaturated, and saturated zone flows, wetlands, and canal flows.

One of the modeling goals to achieve the project objective is the development of a calibrated model capable of:

- Simulating historical extreme events
- Demonstrating that the model can successfully be used to predict stages and discharges within the study area
- Predicting SFWMD LOS Program Performance Metrics (PMs) described below

SFWMD's FPLOS Program developed six (6) performance metrics to quantify the LOS provided under the current and future conditions. The FPLOS performance metrics collectively assess the flood protection level of service under current and future conditions.

- PM1: This metric evaluates the system at peak stages for current, design and future conditions. The evaluation is done for SFWMD's primary conveyance system. Determination of this requires the stages simulation to be accurate and therefore, for the best available data to be used for the cross-sections; this would impact the peak stages used for flood mapping.
- PM2: This metric evaluates the flow capacity of SFWMD primary canals, under current, design and future conditions. Determination of this requires accurate analysis of flow dynamics within the canals when operating at conditions of peak stages and peak discharges.
- PM3: This performance metric evaluates the maximum conveyance capacities of SFWMD structures, for specified conditions at the downstream structure and require thorough analysis of the outflow boundary conditions. The evaluation of this is made based on the structure design features and current structure operations.

- PM4: PM4 determines the maximum conveyance capacity within the watershed at the tidal structures which are outlets to the upstream watershed. This is not applicable in UKBFPLOS project, since the UKB is an inland watershed/basin.
- PM5: This is a stage-based metric, it evaluates if the structures are able to maintain the water levels in communities for FPLOS. This PM will use the upper limit of the stages and depths needed for protection of the infrastructure. This requires the model to accurately describe the topography and the interaction of the overbank spill weirs and inflows in the canals.
- PM6: This metric assesses the duration for flood levels to recede to non-flood levels under future conditions. This also requires the model to accurately describe the topography and the interaction of the overbank spill weirs and inflows in the canals; and for the uncertainty in peaks and time to peak to be low.

UKB FPLOS PMs will only address 5 of them – PM1, PM2, PM3, PM5 and PM6.

Considering the requirements above, an integrated surface and subsurface model was needed to analyze the dynamic exchange between the main model subdomains: Overland Flow, Open Channel Flow, Unsaturated and Saturated zones.

3. Model Software Selection

The process of model software selection in UKB FPLOS was comprehensive. Initial consideration of the available model software was relative to the hydrology of the Kissimmee Basin and the project objectives. A list of model software that met these basic requirements was then evaluated and compared based on several factors including proven track record, functionality, cost-effectiveness, SFWMD in-house skills, credibility and availability of technical support and training.

MIKE SHE and MIKE HYDRO together represent all components of Kissimmee Basin hydrology (channel flow, overland flow, groundwater, unsaturated zone flow) and meet the project objectives best. It has been used internationally with success, including in Florida, specifically in the Kissimmee Basin. In terms of proven track record and functionality relative to UKB FPLOS objectives, it rose to the top as a leader. SFWMD has owned and maintained licenses of the software for almost 20 years, has many applications based on the DHI Suite of model software and have excellent in-house skills for the model software application. This reduces overall costs and time to learn a new model and training. MIKE SHE and MIKE HYDRO have also been peer reviewed for applications in the Kissimmee Basin (Earthtech, 2005). The software technical support and training are a plus. DHI, the developer of MIKE software, not only provides comparatively fast technical support and extensive manuals, but they have also invested in the past, model software updates to meet specific needs in Florida hydrology and hydraulics. MIKE also has a user interface (both GIS and GUI) that is highly functional and user friendly.

As part of a previous modeling project for the Kissimmee Basin, a 2-day extensive workshop was conducted (Earthtech, 2005) to select the most appropriate model software. The conclusion of this workshop was that MIKE SHE – MIKE 11 (part of the DHI suite of water resources management model software) was the most appropriate to simulate the hydraulics and hydrology of the Kissimmee Basin. As part of the current UKB FPLOS project, the systematic approach to evaluate the integrated model software alternatives available to date, concluded that MIKE SHE – MIKE HYDRO is still the best simulation software for the UKB FPLOS objectives (Hazen and Sawyer, 2022). MIKE HYDRO is the successor of the MIKE 11 model software. The version of software used in this project is DHI 2022, Update 1.

4. Storm Events Evaluation and Selection

For the UKB FPLOS project, two historical storm events that occur over the study area were selected, one for calibration and another for validation of the model. It is critical for the model to be calibrated reliably over the storm events that produce peak flow and stage, total runoff volume, and initial soil moisture conditions that can reliably produce the FPLOS performance measures.

For selection of storm events, the criteria that were used include:

- Intensity and magnitude of storm events
- Uniformity of the storm event over the UKB FPLOS study area
- Peak flow produced by the rainfall at UKB structures
- Peak stage produced by the rainfall at UKB structures
- Chronological proximity of the storm event to existing conditions: The Kissimmee Basin has infrastructure and land use changes over time, so priority was placed on storm events that occurred closer to 2020
- Point precipitation frequency (PF) estimates (according to NOAA Atlas 14, Volume 9, Version 2)

4.1 Initial Screening of Storm Events

The initial assessment began with an evaluation of the 3-day rolling cumulative rainfall at stations in the study area for 2000-2020. The 72-hr daily cumulative rainfall depths were used initially for simplicity; considering the analysis was done for 20 years, 15-minute resolution over a 20-year time period was determined to be too fine. After the initial assessment to decide on the top events, remaining evaluation used 15-minute NEXRAD rainfall.

Figure 4-1 shows the location of the rainfall stations in the study area. **Figure 4-2** shows the hydrographs of all the gages and displays the magnitude and intensity of the different events from 2000 – 2020. The colors represent different rain gauges in the UKB FPLOS Study Area. All the storm events at the rain gauges were ranked and plotted in **Figure 4-3**; this shows the number of days the 99th percentile is exceeded at every rain gage within the study area. The higher the number of rain gages aligned for an event in this plot, the higher the number of gages that recorded larger than 99th percentile rainfall.

After the initial assessment of daily cumulative 72-hr rainfall events, the top four storm events promoted for further evaluation were:

- Hurricanes Charley-Frances-Ivan-Jeanne in 2004
- Tropical Storm Fay in August 2008
- No-Name Storm in October 2011
- Hurricane Irma in September 2017

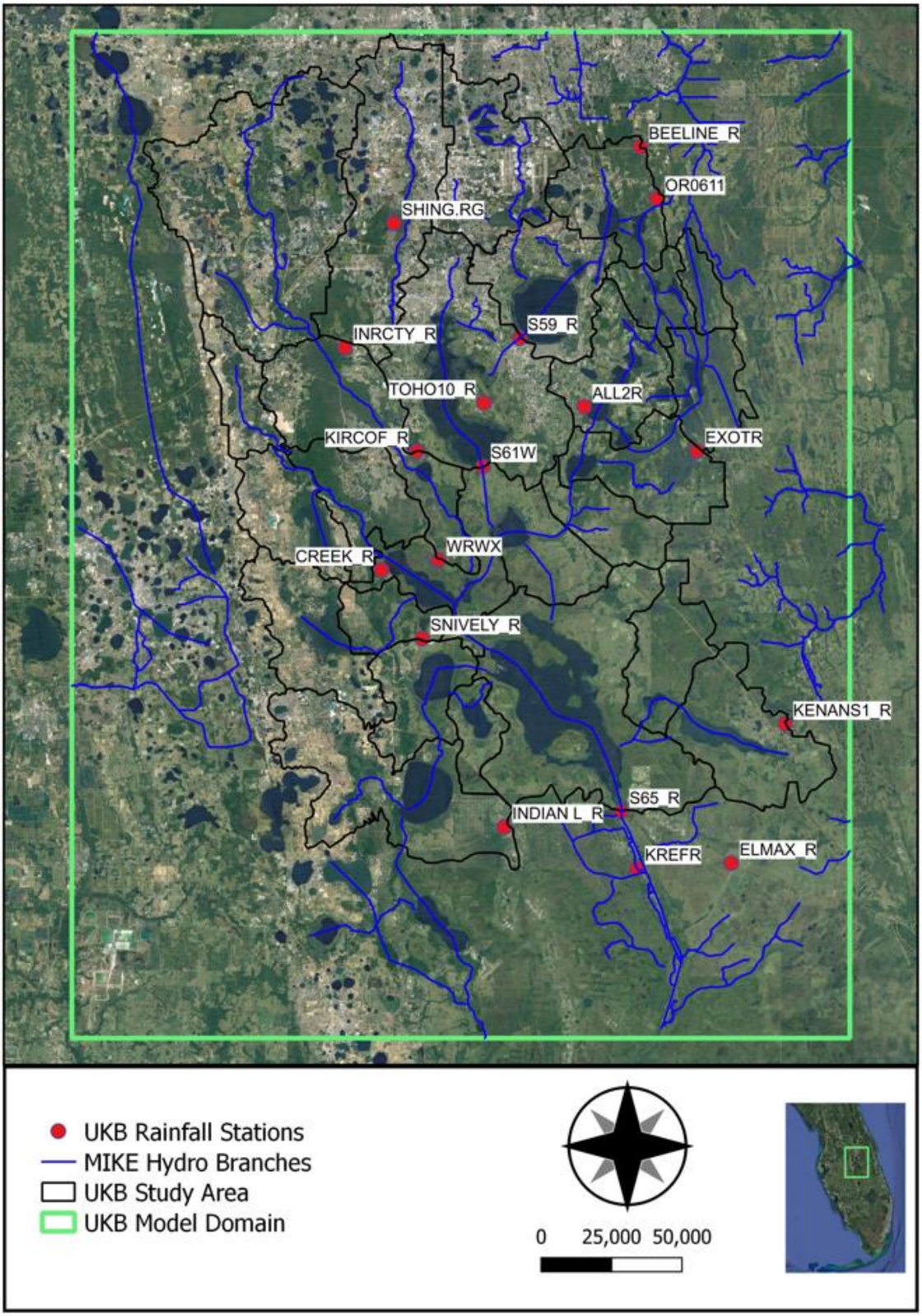

Figure 4-1: Location of Rainfall Stations in UKB FPLOS Study Area

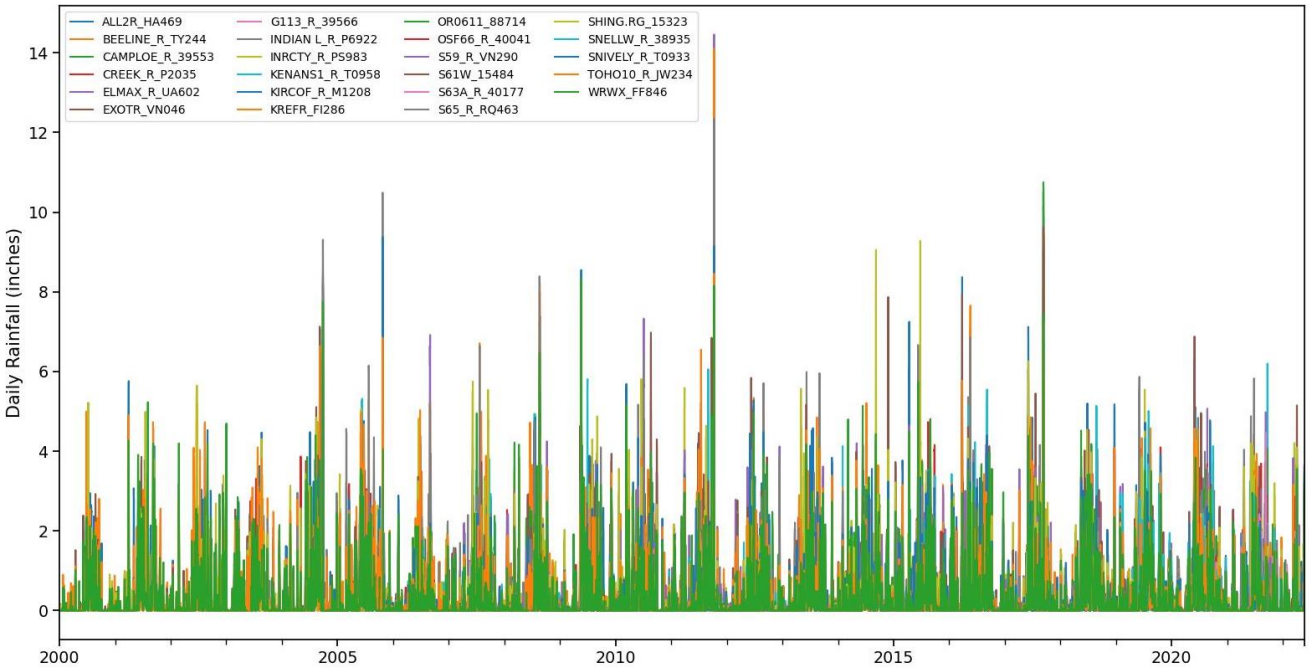

Figure 4-2: Three-day Rolling Cumulative Rainfall in Inches

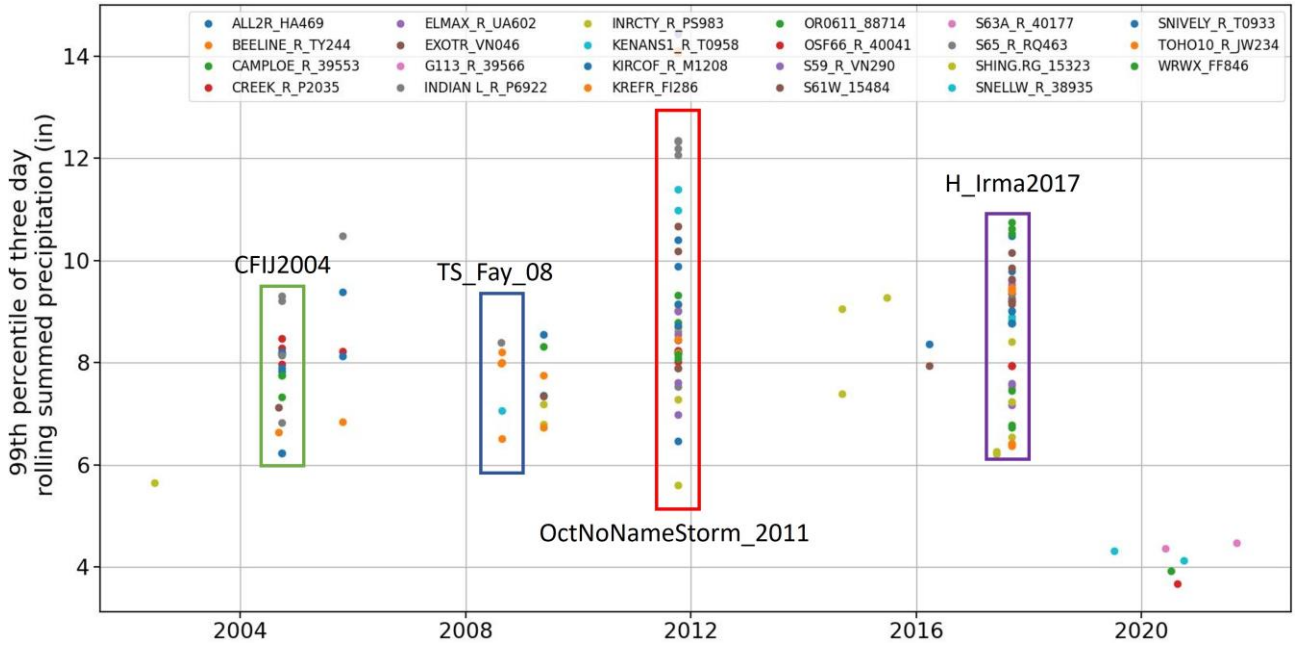

Figure 4-3: Top Ranked 3-Day Storm Events

4.2 Evaluation of Top 4 Storm Events

Storm event simulations require fine spatial and temporal resolutions. So NEXRAD is the preferred source of rainfall data, since it is available in 15-minute time intervals. But comparisons were made between NEXRAD and gage data as a QA/QC check for the 4 top events. The objective of this evaluation was to assess if there is any under or over estimation of NEXRAD data and if it needed to be gage adjusted. **Figure 4-4** shows spatial distribution of these differences. Comparisons were made at the NEXRAD pixels with rainfall stations. **Table 4-1** shows the maximum differences at these locations. Two stations showed high differences for TS Fay 2008 and CFIJ2004 events: S61W_15484 and OR0611_88714. In general, NEXRAD rainfall depths were lower than the depths recorded by the rain gages; except for CFIJ2004 when NEXRAD values were 21.88% higher than the gage data. The difference between the NEXRAD and gage data were the lowest for the October No-Name Storm of 2011 and Hurricane Irma of 2017, with NEXRAD being 1.38% and 3.36% lower respectively.

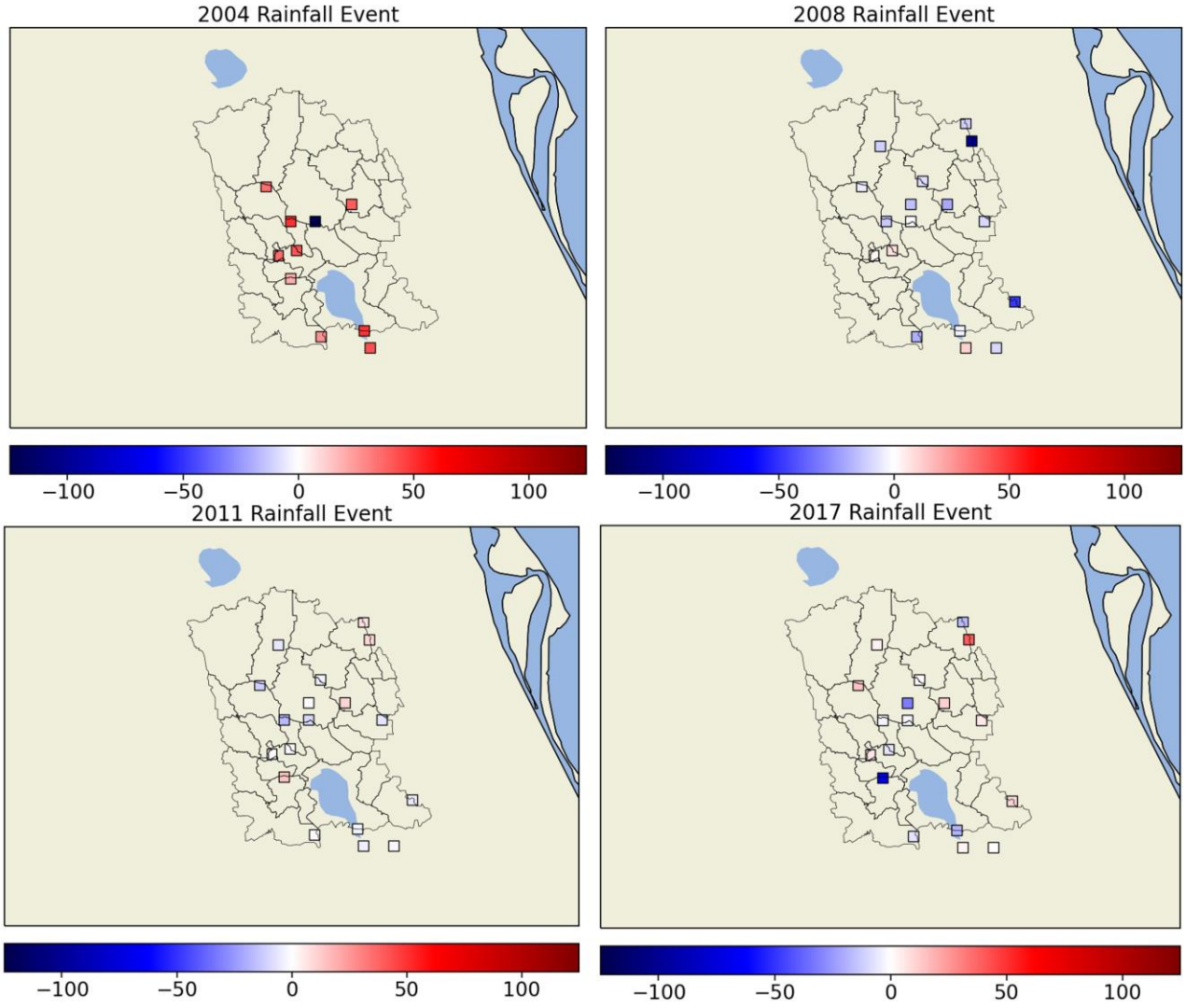

Figure 4-4: Percent differences (Gage - NEXRAD) of Daily Accumulated Rainfall Data

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Table 4-1: Maximum Difference (%) between NEXRAD and Gage Rainfall for the Top Storm Events (3-day)

Gage	Closest NEXRAD Pixel	Rainfall Event	Gage Value (in) (3-day Cumulative)	NEXRAD value (in) (3-day Cumulative)	Percent Difference
ALL2R_HA469	10104658	2004	7.64	5.119	39.52
ALL2R_HA469	10104658	2017	9.05	8.12	10.83
ALL2R_HA469	10104658	2011	7.7	6.97	9.95
ALL2R_HA469	10104658	2008	3.45	4.27	-21.24
BEELINE_R_TY244	10111297	2017	5.74	6.73	-15.88
BEELINE_R_TY244	10111297	2011	7.77	7.21	7.48
BEELINE_R_TY244	10111297	2008	2.16	2.42	-11.35
CREEK_R_P2035	10100380	2004	7.79	5.438	35.56
CREEK_R_P2035	10100380	2017	7.92	7.43	6.38
CREEK_R_P2035	10100380	2011	5.6	5.59	0.18
CREEK_R_P2035	10100380	2008	3.91	3.85	1.55
ELMAX_R_UA602	10092816	2017	9.28	9.27	0.11
ELMAX_R_UA602	10092816	2011	8.95	9.07	-1.33
ELMAX_R_UA602	10092816	2008	6.08	6.66	-9.11
EXOTR_VN046	10103242	2017	9	8.53	5.36
EXOTR_VN046	10103242	2011	7.89	8.42	-6.50
EXOTR_VN046	10103242	2008	5.85	6.39	-8.82
INDIAN L_R_P6922	10093751	2004	6.11	4.679	26.53
INDIAN L_R_P6922	10093751	2017	9.17	9.8	-6.64
INDIAN L_R_P6922	10093751	2011	7.4	7.47	-0.94
INDIAN L_R_P6922	10093751	2008	5.42	6.55	-18.88
INRCTY_R_PS983	10106066	2004	7.08	4.985	34.73
INRCTY_R_PS983	10106066	2017	8.42	7.29	14.39
INRCTY_R_PS983	10106066	2011	4.64	5.17	-10.81
INRCTY_R_PS983	10106066	2008	2.18	2.28	-4.48
KENANS1_R_T0958	10096611	2017	8.69	7.89	9.65
KENANS1_R_T0958	10096611	2011	8.11	8.48	-4.46

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Gage	Closest NEXRAD Pixel	Rainfall Event	Gage Value (in) (3-day Cumulative)	NEXRAD value (in) (3-day Cumulative)	Percent Difference
KENANS1_R_T0958	10096611	2008	4.57	7.49	-48.42
KIRCOF_R_M1208	10103226	2004	7.88	4.719	50.18
KIRCOF_R_M1208	10103226	2017	8.69	8.92	-2.61
KIRCOF_R_M1208	10103226	2011	4.78	5.67	-17.03
KIRCOF_R_M1208	10103226	2008	2.2	2.51	-13.16
KREFR_FI286	10092811	2004	6.45	4.173	42.87
KREFR_FI286	10092811	2017	9.4	9.2	2.15
KREFR_FI286	10092811	2011	8.63	8.9	-3.08
KREFR_FI286	10092811	2008	7.1	6.33	11.47
OR0611_88714	10109876	2017	10.08	6.78	39.15
OR0611_88714	10109876	2011	7.65	6.97	9.30
OR0611_88714	10109876	2008	0.76	2.32	-101.30
S59_R_VN290	10106550	2017	7	6.99	0.14
S59_R_VN290	10106550	2011	6.05	6.26	-3.41
S59_R_VN290	10106550	2008	2.47	2.68	-8.16
S61W_15484	10103230	2004	1.3	5.294	-121.14
S61W_15484	10103230	2017	9.02	9.26	-2.63
S61W_15484	10103230	2011	5.58	6.13	-9.39
S61W_15484	10103230	2008	3.57	3.6	-0.84
S65_R_RQ463	10094232	2004	8.04	4.88	48.92
S65_R_RQ463	10094232	2017	6.24	7.5	-18.34
S65_R_RQ463	10094232	2011	7.48	7.63	-1.99
S65_R_RQ463	10094232	2008	6.39	6.61	-3.38
SHING.RG_15323	10109387	2017	6.41	6.17	3.82
SHING.RG_15323	10109387	2011	4.42	4.67	-5.50
SHING.RG_15323	10109387	2008	1.57	1.76	-11.41
SNIVELY_R_T0933	10098486	2004	5.9	4.812	20.31
SNIVELY_R_T0933	10098486	2017	3.65	7.67	-71.02

Gage	Closest NEXRAD Pixel	Rainfall Event	Gage Value (in) (3-day Cumulative)	NEXRAD value (in) (3-day Cumulative)	Percent Difference
SNIVELY_R_T0933	10098486	2011	6.04	5.22	14.56
TOHO10_R_JW234	10104652	2017	6.21	8.57	-31.94
TOHO10_R_JW234	10104652	2011	5.71	5.72	-0.17
TOHO10_R_JW234	10104652	2008	2.43	2.82	-14.86
WRWX_FF846	10100857	2004	7.33	4.819	41.34
WRWX_FF846	10100857	2017	6.7	7.14	-6.36
WRWX_FF846	10100857	2011	5.93	6.03	-1.67
WRWX_FF846	10100857	2008	5.39	5	7.51

After comparison with NEXRAD, further analysis and assessments were performed for the following criteria:

1. Storm Event: The following criteria were used to rank the 4 top events based on severity.
    a. Rainfall depth of 72-hr event
    b. Peak Intensity of the 72-hr storm
    c. Uniformity of the 72-hr storm across the UKB FPLOS study area
    d. Comparison of rainfall frequency estimates at Kissimmee, according to NOAA Atlas 14, Volume 9, Version 2
2. System Response: The following criteria were used to rank the 4 top events; ranked based on causing the worst-case flooding conditions and structures operating to make the highest possible discharges.
    a. Maximum flow at structures
    b. Maximum stages at structures
3. Watershed Conditions: Ranking for this criterion was based on how chronologically close the event is to UKB FPLOS existing conditions (2020)
    a. Chronologic proximity to the existing conditions for UKB FPLOS

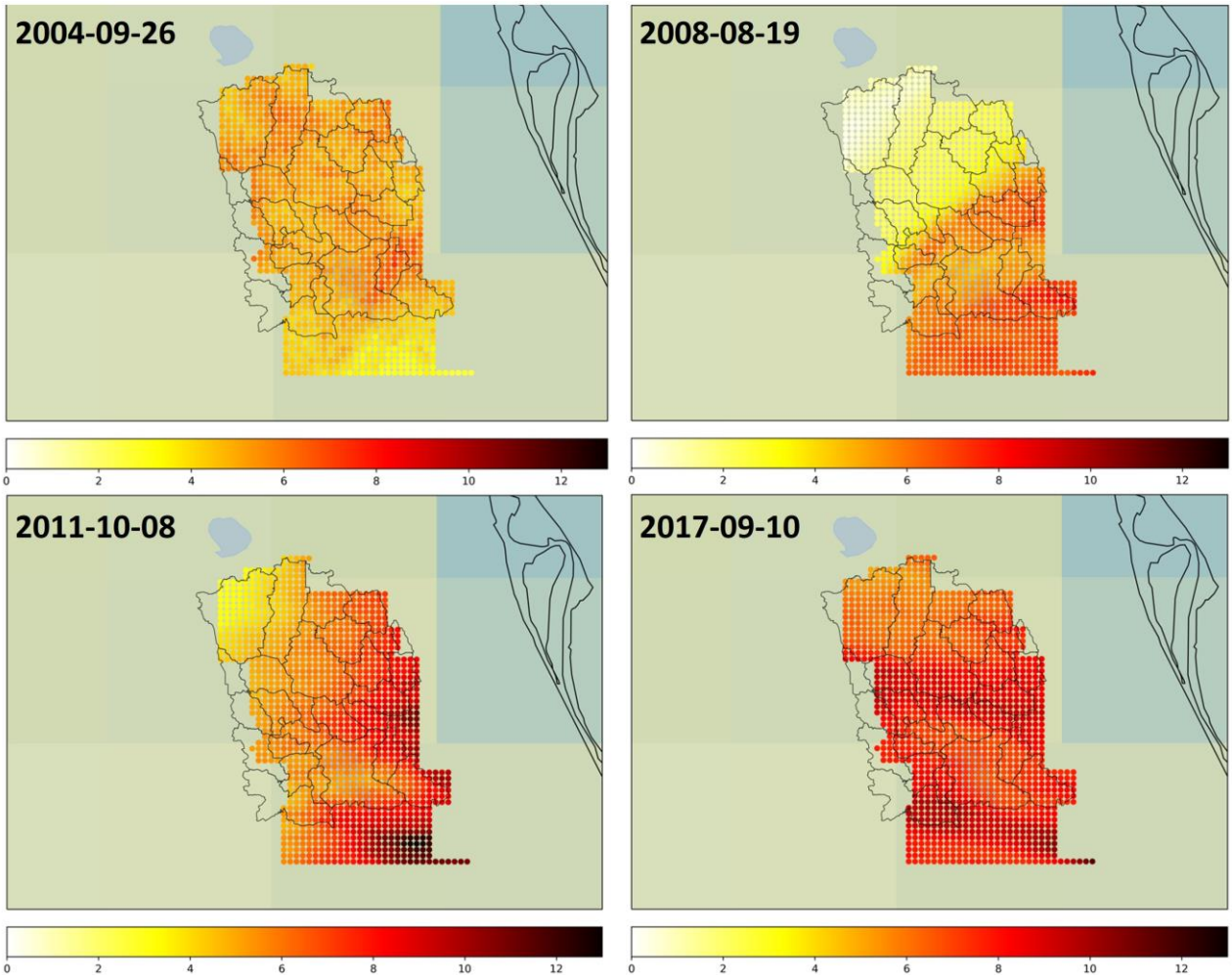

Figure 4-5: Spatial Distribution of Cumulative 24-hr Rainfall Daily Rainfall for the Top 4 Storm Events

4.2.1 Intensity, Magnitude and Uniformity of Storm Event across UKB FPLOS Study Area

The spatial distribution of the 4 events were analyzed for uniformity of rainfall. For the four different events, daily accumulated rainfall was calculated for 72 hours, shown in **Figure 4-5**. To further determine the uniformity of the rainfall distribution quantitatively, the standard deviation across the UKB FPLOS Study Area for the selected day was selected as the statistic. High standard deviation indicates there is higher variation of rainfall distribution across the study area. The ranking based on this criterion is shown in**Table 4-2**.

Table 4-3 shows the rolling cumulative rainfall for the NEXRAD cells in the study area used to classify the event; and the ranking of the events based on the cumulative rainfall. The maximum observed values of the rain gages were used in the classification.

Table 4-2: Ranking of Storm Events based on Uniformity across UKB FPLOS Study Area

Storm Event	Standard Deviation across UKB FPLOS Study Area (inches)	Rank
CFIJ2004	0.69	1
TS Fay 2008	2.17	4
OctNo-Name_2011	1.85	3
Irma_2017	1.14	2

Table 4-3: Cumulative 72-hr Rainfall in Inches for 4 Top Selected Events

Storm Event	Three Day Accumulated Rainfall Depth (inches)			Rank
	Average	Minimum	Maximum	
CFIJ [2004]	5.78	3.96	8.742	4
TSFAY [2008]	5.22	1.15	9.39	3
October NO-NAME [2011]	9.66	4.8	18.50	1
IRMA [2017]	8.45	6.61	12.04	2

Rainfall intensity was computed by aggregating 15-minute NEXRAD data into hourly values and the hourly intensity was used to rank the rainfall events, shown in **Table 4-4**. Another criterion used to evaluate the storm events was Point precipitation frequency (PF) estimates, according to NOAA Atlas 14, Volume 9, Version 2. Shown in **Table 4-5** is the NOAA classification at the Kissimmee Rainfall Station. Based on the ranges in the table, the CFIJ2004 and TS Fay (2008), events can be classified as between 10-year and 25-year rainfall events, Hurricane Irma (2017) falls between the ranges of 25-year and 200-year events and the No_Name (October 2011) event fits within the ranges of 200-year and 1000-year events.

Table 4-4: Ranking of Top Events based on Rainfall Intensity

Storm Event	Average Intensity (in/hr)	Maximum Intensity (in/hr)	Rank
CFIJ2004	0.28	1.13	4
TS Fay 2008	0.25	2.53	2
OctNo-Name_2011	0.35	2.95	1
HIrma_2017	0.31	2.15	3

Table 4-5: Point Precipitation Frequency (PF) Estimates for Station Kissimmee (NOAA Site ID 08-4625), in inches with 90% Confidence Intervals from NOAA Atlas 14, Volume 9, Version 2

Frequency (Average Recurrence Interval in Years)	72-hr Rainfall (inches)
1	**4.82** (4.19 – 5.56)
2	**5.48** (4.76 – 6.33)
5	**6.75** (5.84 – 7.83)
10	**7.99** (6.87 – 9.32)
25	**9.96** (8.38 – 12.5)
50	**11.7** (9.53 – 14.9)
100	**13.6** (10.7 – 17.9)
200	**15.8** (11.8 – 21.4)
500	**18.9** (13.5 – 26.6)
1000	**21.5** (14.8 – 30.5)

4.2.2 Hydrological Response to Storm Events

The watershed responses to the events were assessed in terms of stages and flows. Breakpoint discharge and stage data for the structures (G113, S57, S58, S59, S60, S61, S62, S63, S63A, and S65) were plotted for the four selected storm events and ranked 1-4, assuming higher flows and stages indicate worse conditions in terms of flooding and management of the structures. The events were ranked at each structure, as seen in **Table 4-6**. The October 2011 No-Name Storm and Hurricane Irma from 2017 ranked 1 and 2 for most of the structures.

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Table 4-6: Ranking of the Selected Storm Events based on Stages and Flows at Structures

Structure	Rainfall Event	Maximum Flow (cfs)	Ranking
G113	2004	392.42	2
	2008	-	-
	2011	-	-
	2017	605.52	1
S57	2004	205.08	3
	2008	151.99	4
	2011	302.77	1
	2017	240.98	2
S58	2004	9.03	2
	2008	0.0	4
	2011	40.52	1
	2017	8.55	3
S59	2004	1964.36	1
	2008	1232.48	4
	2011	1367.76	3
	2017	1522.29	2
S60	2004	537.5	3
	2008	458.33	4
	2011	666.82	1
	2017	638.65	2
S61	2004	3712.05	1
	2008	2960.4	3
	2011	2801.77	4
	2017	3145.46	2
S62	2004	517.0	4
	2008	532.63	3
	2011	704.12	2

Structure	Rainfall Event	Maximum Flow (cfs)	Ranking
	2017	706.62	1
S63	2004	981.78	3
	2008	764.96	4
	2011	1110.37	1
	2017	1023.59	2
S63A	2004	2136.43	4
	2008	2169.28	3
	2011	2739.11	1
	2017	2227.8	2
S65	2004	15044.3	1
	2008	9597.53	4
	2011	14031.53	2
	2017	13775.07	3

4.3 Storm Events Selection

The storm events were ranked for each criterion described in this section and **Table 4-7** shows the summary of the analysis. The No-Name_October 2011 Storm and Hurricane Irma (2017) were the top ranked events. Since Hurricane Irma is chronologically closer to the UKB FPLOS existing conditions (2020), it was recommended as the event for calibration and No-Name_October 2011 was recommended as the event to validate the calibrated model.

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Table 4-7: Ranking of Top 4 Storm Events for Selection of Calibration and Validation

	Factors Considered	2004	2008	2011	2017
Storm Event	Depth [3-Day]	4	3	1	2
	Intensity	4	2	1	3
	Uniformity	1	4	3	2
System Response	Maximum Flow	2	4	1	3
	Maximum Stage	2	4	1	2
Watershed Conditions	Chronologically Close to existing conditions	4	3	2	1
	Sum	17	20	9	13
	Rank	**3**	**4**	**1**	**2**

5. Model Development, Calibration and Validation

5.1 MIKE SHE Model Development

5.1.1 Simulation Specification

The UKB FPLOS model was developed using DHI's MIKE modeling framework MIKE ZERO. The model was created using the MIKE HYDRO interface and MIKE 1D simulation engine (hydrodynamic software for channel flow and infrastructure) and MIKE SHE (simulating all components of the hydrologic cycle, including precipitation and evapotranspiration, overland runoff, infiltration and vertical flow in the unsaturated zone, and 3-dimensional saturated groundwater flow). A principal advantage of the MIKE modeling framework is the ability to include all hydrologic and hydrodynamic processes and the interactions between them.

UKB FPLOS model implementation includes the four main hydrologic and hydraulic subdomains: Flow in Open Channels (OC), Overland Flow (OL), and flow in the unsaturated zone (UZ) and saturated zone (SZ).

For some processes, MIKE SHE/MIKE 1D has several options and numerical methods to effectively simulate the hydrologic processes. Below are highlights of the possible hydrologic processes and numerical approaches; with the ones used in the UKB FPLOS model italicized.

- Climate (Precipitation and Evapotranspiration)
- Rivers and Lakes (OC) (MIKE HYDRO/ MIKE 1D)
- Overland Flow (OL) simulation
    - Solutions:
        - *Finite Difference Method*
        - Simplified Overland Flow Routing Method
    - Solvers:
        - Successive Overrelaxation (SOR)
        - *Explicit*
- Unsaturated Flow (UZ) simulation
    - Solutions:
        - 1D Simplified Gravity Flow Equation
        - *Two-Layer Water Balance Method*
        - 1D Richards Equation Solution

- Saturated Flow (SZ) simulation
  - Solutions:
    - Linear Reservoir Method
    - *3D Finite Difference Method*
  - Solvers:
    - *Preconditioned Conjugate Gradient, Transient*
    - Preconditioned Conjugate Gradient, Steady State
    - Successive Overrelaxation Package (SOR)

5.1.2 Model Domain and Grid

The UKB FPLOS model is built using DHI version 2022, Update 1. The model domain covers an area of approximately 3560 sq. mi; **Figure 5-1** shows the UKB FPLOS study area and the model domain. UKB FPLOS Study Area refers to the 26 watersheds listed in Table 1-1 and UKB FPLOS Model Domain refers to the rectangular area in Figure 5-1.

The UKB FPLOS Model is discretized with a grid size of 328 ft x 328 ft (100 m x 100 m), 1090 rows, 842 columns, 3 groundwater layers (SA, ICU and UFA-upper).

5.1.3 Topography

Topographic data is important for the flooding assessment using MIKE model. It is one of the determining factors on overland flow routing, mapping of flood inundation extents, and overland and river exchanges; additionally, drain levels and saturated zone hydrostratigraphy are defined relative to the topography. The topographic data in MIKE has a uniform, averaged elevation value for each model grid cell of the model domain. The UKB FPLOS model uses topography data at a 32 ft (10 m) grid cell size, as shown in **Figure 5-2**.

This is the best available QA/QC'ed, elevation mosaic dataset that covers the UKB FPLOS Model Domain. The MIKE topography dataset was compiled from many different datasets, based on LiDAR collection in 2018. The resolution of this dataset is 2.5 ft. The extents of the sources of the LiDAR data are shown in **Figure 5-3**; the names of the datasets indicate the date and the resolution.

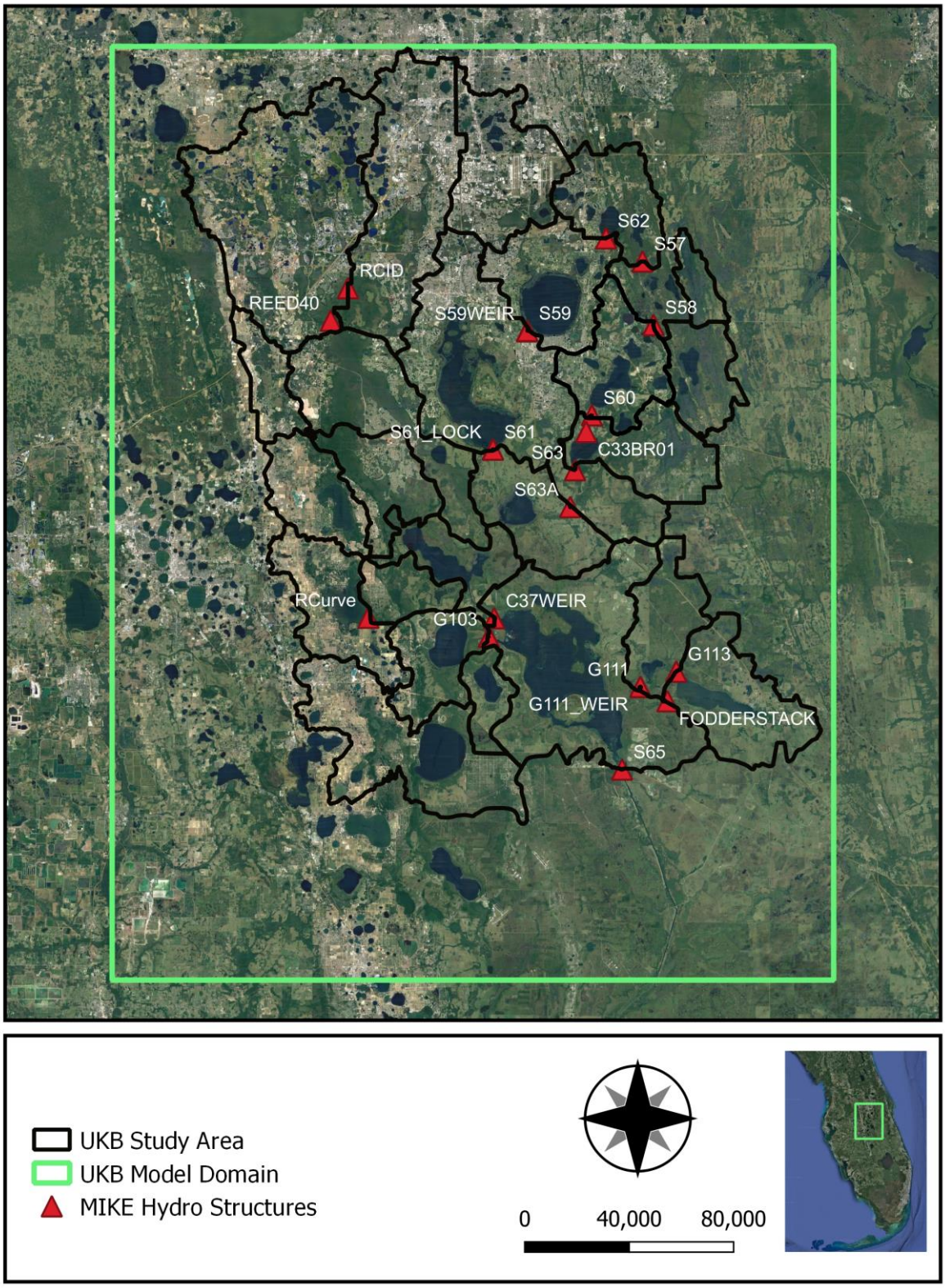

Figure 5-1: UKB FPLOS Model Domain

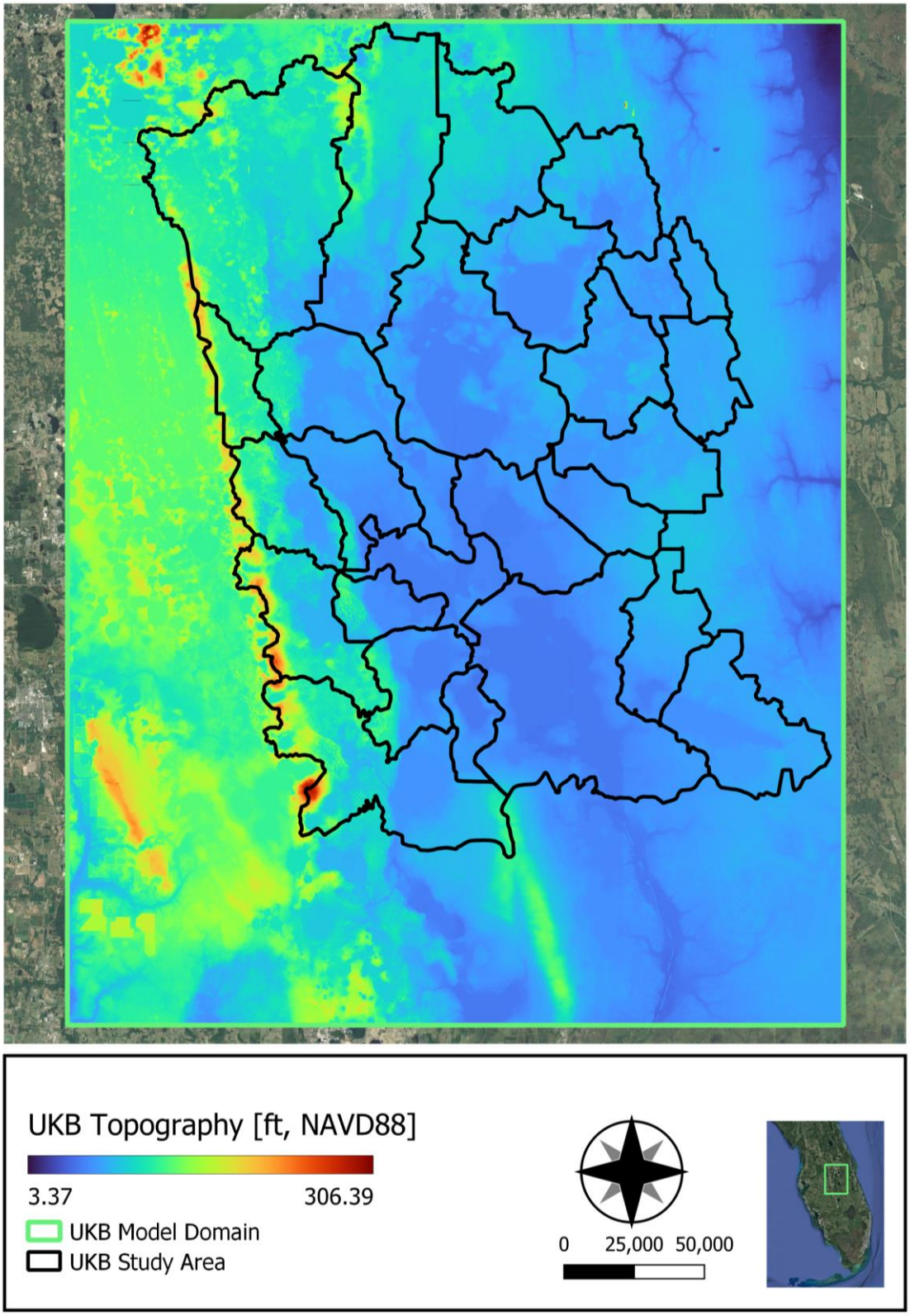

Figure 5-2: UKB FPLOS Merged Topography Dataset

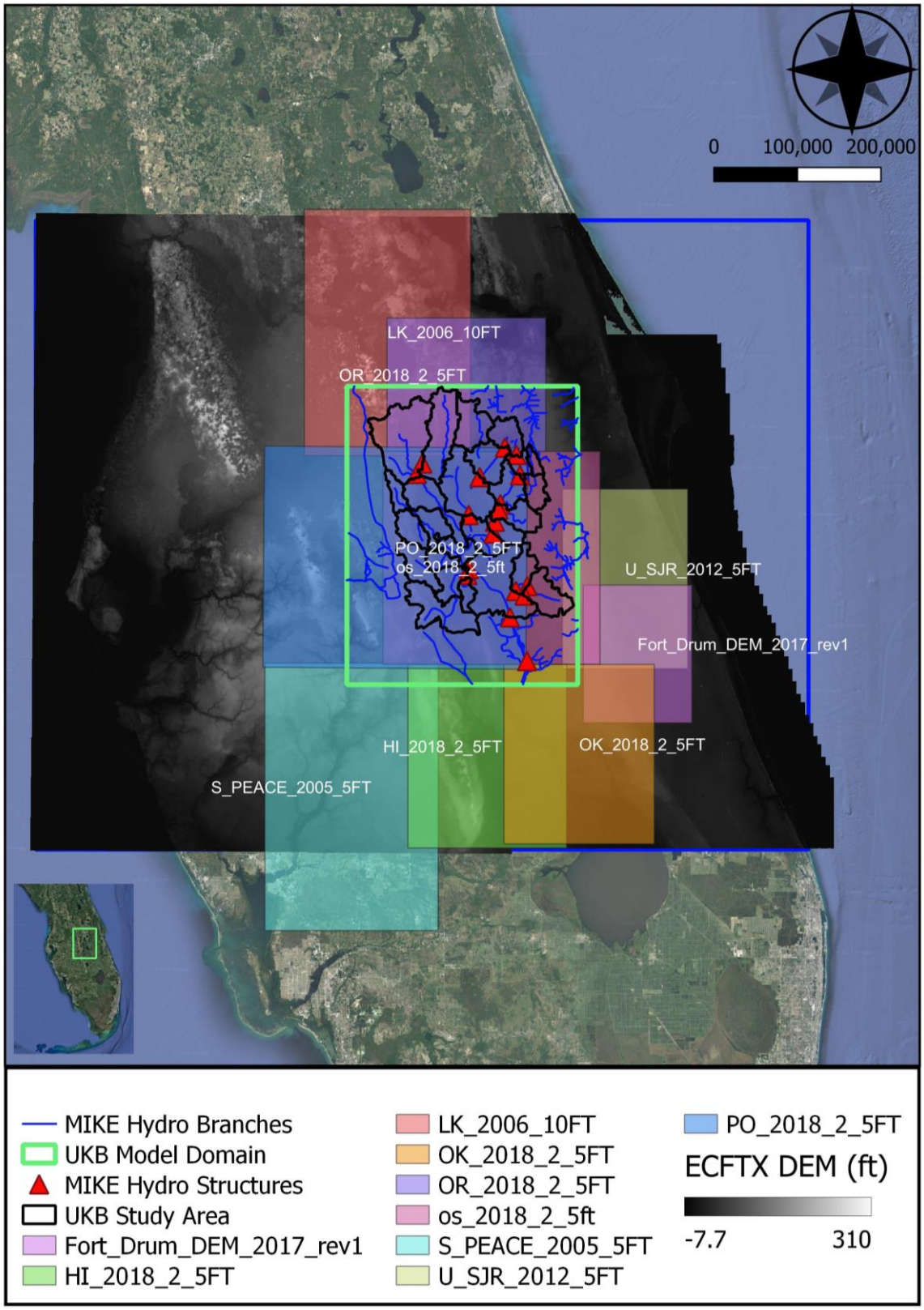

Figure 5-3: Sources of UKB FPLOS Merged Topography Dataset

5.1.4 Climate

5.1.4.1 Precipitation Rate

Rainfall is the primary hydrological driver of the model. A user can provide the input to the MIKE model by specifying precipitation spatially and temporally in various ways, depending on model objectives and data availability. **Figure 5-4** displays the monthly average rainfall in inches in the UK FPLOS area (for the stations shown in Figure 4-1) for 23 years (2000 - 2022).

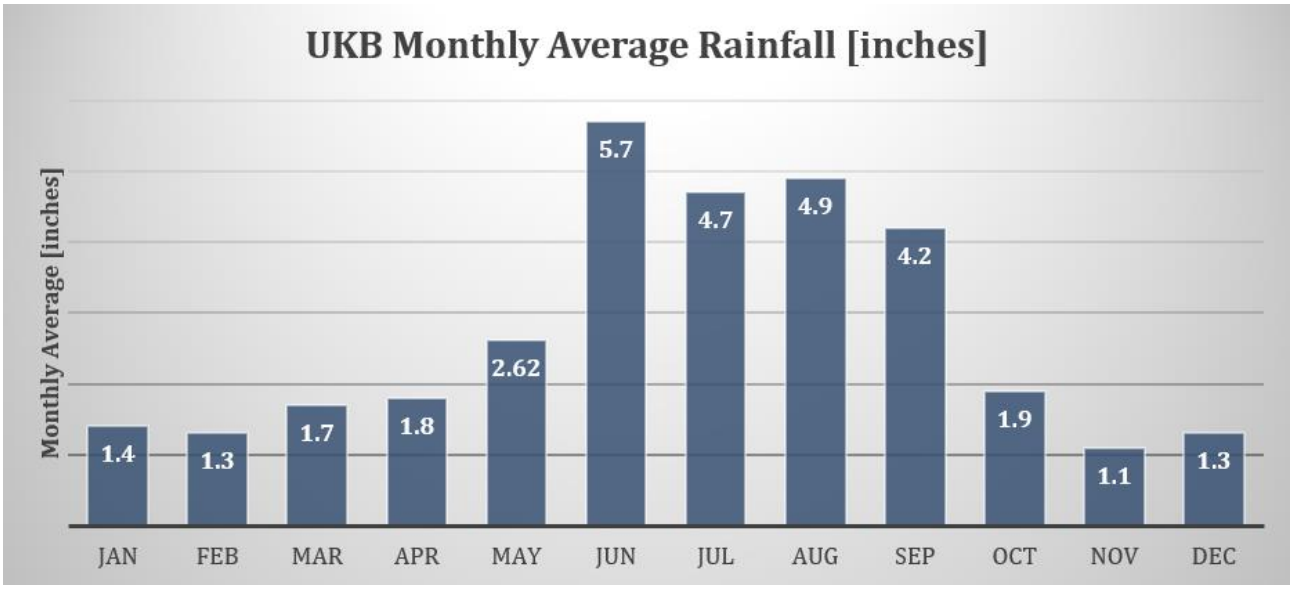

Figure 5-4: Monthly Average Rainfall UK FPLOS [2000 - 2023]

The two available sources of data from SFWMD for this project were station data (rain gage) and NEXRAD (radar rainfall):

- Gage data has been QA/QCed by SFWMD. This was generally available as breakpoint data.
- NEXRAD data was also QA/QCed; it was converted to dfs2 format at the 100 m x 100 m model resolution and is based on an underlying source resolution of 2 km x 2 km (2000 m or 6562 ft) for the UKB FPLOS model domain.

Initially, fully distributed, time varying 15-minute NEXRAD datasets were used in the UKB FPLOS model for the calibration and validation simulations since it has better spatial and temporal coverage and resolution than the gage dataset.

Comparison with the gage dataset during calibration showed differences between NEXRAD and gage datasets, as shown in **Table 5-1**. Due to the significant difference in rainfall depth between the datasets, the model was updated with gage rainfall, using the 26 watersheds as the subarea for rainfall application extent; for each watershed, the closest gage dataset was used.

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Table 5-1: Differences in NEXRAD and Gage Rainfall during Hurricane Irma

Gage Name (with DB Key)	Closest NEXRAD Pixel	Rainfall Event	Gage Data (inches) (3-day Cumulative)	NEXRAD Data (inches) (3-day Cumulative)	Percent Difference
SNIVELY_R_T0933	10098486	2017	3.65	7.67	-71.02
TOHO10_R_JW234	10104652	2017	6.21	8.57	-31.94
S65_R_RQ463	10094232	2017	6.24	7.5	-18.34
BEELINE_R_TY244	10111297	2017	5.74	6.73	-15.88
INDIAN L_R_P6922	10093751	2017	9.17	9.8	-6.64
WRWX_FF846	10100857	2017	6.7	7.14	-6.36
S61W_15484	10103230	2017	9.02	9.26	-2.63
KIRCOF_R_M1208	10103226	2017	8.69	8.92	-2.61
ELMAX_R_UA602	10092816	2017	9.28	9.27	0.11
S59_R_VN290	10106550	2017	7	6.99	0.14
KREFR_FI286	10092811	2017	9.4	9.2	2.15
SHING.RG_15323	10109387	2017	6.41	6.17	3.82
EXOTR_VN046	10103242	2017	9	8.53	5.36
CREEK_R_P2035	10100380	2017	7.92	7.43	6.38
KENANS1_R_T0958	10096611	2017	8.69	7.89	9.65
ALL2R_HA469	10104658	2017	9.05	8.12	10.83
INRCTY_R_PS983	10106066	2017	8.42	7.29	14.39
OR0611_88714	10109876	2017	10.08	6.78	39.15

For the validation event, the differences between gage and NEXRAD datasets at the gage locations were less than an inch. So, two (2) validation models were executed – one with gage data and another with NEXRAD data. The differences in the residuals of resulting stages and flows were not significant; nor did it show any bias in the results for the validation model. Therefore, the validation results discussed in this report are based on the model with gage data, so the calibration and validation models have consistency in the source of rainfall dataset.

5.1.4.2 Evapotranspiration

Potential Evapotranspiration (PET) is defined as the maximum possible evapotranspiration (ET) rate from a continuously saturated surface under ambient meteorological conditions. The reference evapotranspiration (RET) is the rate of ET from a reference vegetation type (i.e., grass or wet marsh) calculated without any shortage of water.

Time-varying PET datasets were used, and they were spatially applied based on the watersheds. The values used were filled in for missing days. The DB keys of the station used were UT749 and TV995. PET/RET is minimal during the calibration and validation storm events.

5.1.5 Land Use

The land use component of MIKE SHE defines parameters based on land use classifications and influences hydraulic and hydrologic processes. Land use and land cover classification from SFWMD, SJRWMD and SWFWMD were used to derive overland and evapotranspiration parameters; Level 4 FLUCCS code descriptions were used to assign MIKE SHE codes, to derive overland runoff and vegetation parameters.

5.1.5.1 Vegetation

The dataset used for the UKB FPLOS Model was SFWMD's Land Cover and Land Use for 2014-2016 (South Florida Water Management District, 2018). This was the latest dataset that captures land cover and land use (LCLU) within the SFWMD as it existed in 2014-16. This LCLU dataset was updated by SFWMD from earlier 2008-2009 (South Florida Water Management District, 2017) data by photointerpretation from 2014-16 aerial photography. The classification system used by SFWMD was the Florida Land Use, Cover and Forms Classification System (FLUCCS), originally compiled by the State Topographic Bureau of the Florida Department of Transportation. The classification system was amended by SFWMD considering only the land use. This dataset follows the SFWMD modified FLUCCS classification system (South Florida Water Management District, 2017). According to the metadata, LCLU features were interpreted from the county-based aerial photography (4 in - 2 ft pixel); the minimum mapping unit used to classify was 0.5 acres for wetlands and 5 acres for uplands.

The level 4 FLUCCS codes and descriptions are shown in **Table 5-2**, and **Table 5-3** shows the FLUCCS codes associated with the MIKE SHE vegetation codes. **Figure 5-5** shows the MIKE SHE land use codes in the UKB FPLOS Model.

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Table 5-2: FLUCCS Codes and Descriptions

Code	Description	Code	Description	Code	Description
1110	Fixed Single-Family Units	1900	Open Land	5420	Embayments Not Opening Directly to Gulf or Ocean
1120	Mobile Home Units	2110	Improved Pastures	5430	Saltwater Ponds
1130	Mixed Units, Fixed and Mobile Home Units	2120	Unimproved Pastures	5720	Gulf of Mexico
1180	Rural Residential	2130	Woodland Pastures	6120	Mangrove Swamp
1190	Low Density Under Construction	2140	Row Crops	6170	Mixed Wetland Hardwoods
1210	Fixed Single-Family Units	2150	Field Crops	6172	Mixed Shrubs
1220	Mobile Home Units	2160	Mixed Crops	6180	Cabbage Palm Wetland
1230	Mixed Units, Fixed and Mobile Home Units	2210	Citrus Groves	6191	Wet Melaleuca
1290	Medium Density Under Construction	2230	Other Groves	6200	Wetland Coniferous Forests
1310	Fixed Single-Family Units	2240	Abandoned Groves	6210	Cypress
1320	Mobile Home Units	2320	Poultry Feeding Operations	6215	Cypress- Domes/Heads
1330	Multiple Dwelling Units, Low Rise	2410	Tree Nurseries	6216	Cypress, Mixed Hardwoods
1340	Multiple Dwelling Units, High Rise	2430	Ornamentals	6240	Cypress, Pine, Cabbage Palm
1350	Mixed Units, Fixed and Mobile Home Units	2500	Specialty Farms	6250	Wet Pinelands Hydric Pine
1390	High Density Under Construction	2510	Horse Farms	6300	Wetland Forested Mixed
1400	Commercial and Services	2540	Aquaculture	6400	Vegetated Non-Forested Wetlands
1411	Shopping Centers	2610	Fallow Cropland	6410	Freshwater Marshes / Graminoid Prairie – Marsh
1423	Wholesale Sales	3100	Herbaceous (Dry Prairie)	6411	Freshwater Marshes-Sawgrass
1460	Oil and Gas Storage - not Ind. or Manufacturing.	3200	Upland Shrub and Brushland	6420	Saltwater Marshes / Halophytic Herbaceous Prairie
1480	Cemeteries	3210	Palmetto Prairies	6430	Wet Prairie

Code	Description	Code	Description	Code	Description
1490	Commercial, Services Under Construction.	3220	Coastal Shrub	6440	Emergent Aquatic Vegetation
1550	Other Light Industry	3300	Mixed Rangeland	6510	Tidal Flats
1560	Other Heavy Industrial	4110	Pine Flatwoods	7200	Sand Other Than Beaches
1610	Strip mines	4130	Sand Pine	7400	Disturbed Land
1620	Sand and Gravel Pits	4200	Upland Hardwood Forests	7430	Spoil Areas
1630	Rock Quarries	4220	Brazilian Pepper	7470	Dikes and Levees
1660	Holding Ponds	4240	Melaleuca	8110	Airports
1670	Abandoned Mining	4271	Oak - Cabbage Palm Forest	8113	Private Airports
1700	Institutional	4280	Cabbage Palm	8115	Grass Airports
1710	Educational Facilities	4300	Upland Mixed Forests	8140	Roads and Highways
1760	Correctional	4340	Upland Mixed Coniferous / Hardwood	8200	Communications
1810	Swimming Beach	4370	Australian Pine	8310	Electrical Power Facilities
1820	Golf Course	4410	Coniferous Plantations	8320	Electrical Power Transmission Lines
1830	Racetracks	5110	Natural River, Stream, Waterway	8330	Water Supply Plants - Including Pumping Stations
1840	Marinas and Fish Camps	5120	Channelized Waterways, Canals	8340	Sewage Treatment
1850	Parks and Zoos	5200	Lakes	8350	Solid Waste Disposal
1860	Community Recreation Facilities	5300	Reservoirs	8360	Other Treatment Ponds
1890	Other Recreational Facilities	5410	Embayments Opening Directly to Gulf or Ocean		

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Table 5-3: Codes Assigned from FLUCCS to MIKE SHE

MIKE SHE Codes	Description	FLUCCS Codes
1	Citrus	2210, 2230
2	Pasture	1920, 2110, 2120, 2130
5	Truck Crops	2140, 2150, 2160
6	Golf Course	1820
7	Bare Ground	1610, 1620, 1630, 1670, 1810, 7200, 7400, 8350, 1900, 2610, 7430, 7470, 8320
8	Hydric-Mesic-Xeric Flatwood	3100, 3200, 3210, 3300, 4110, 4410, 4130, 2240, 6240, 6250, 6180
9	Mesic-Xeric Hammock	4200, 4220, 4270, 4271, 4300, 4340, 4370, 3220
13	Hydric Hammock	4240, 4280, 6191
14	Wet Prairie	6430, 2540
16	Marsh	6400, 6410, 6440,
17	Cypress	6200, 6210, 6215, 6216
18	Swamp Forest	6170, 6172, 6190, 6300
19, 20	Water, Lakes	1660, 5110, 5120, 5200, 5300, 5410, 5420, 5430, 5720, 6510, 8360
41	Urban Low Density	1110, 1120, 1130, 1180, 1190, 1480, 2510, 2500, 2430, 2410, 8113, 8115,
42	Urban Medium Density	1210, 1220, 1230, 1290, 8330, 8340, 1840, 1850, 1860, 1890, 1460, 2320,
43	Urban High Density	1310, 1320, 1330, 1340, 1350, 1390, 1400, 1410, 1411, 1420, 1423, 1490, 1550, 1560, 1700, 1710, 1760, 1830, 8110 8140, 8200, 8310

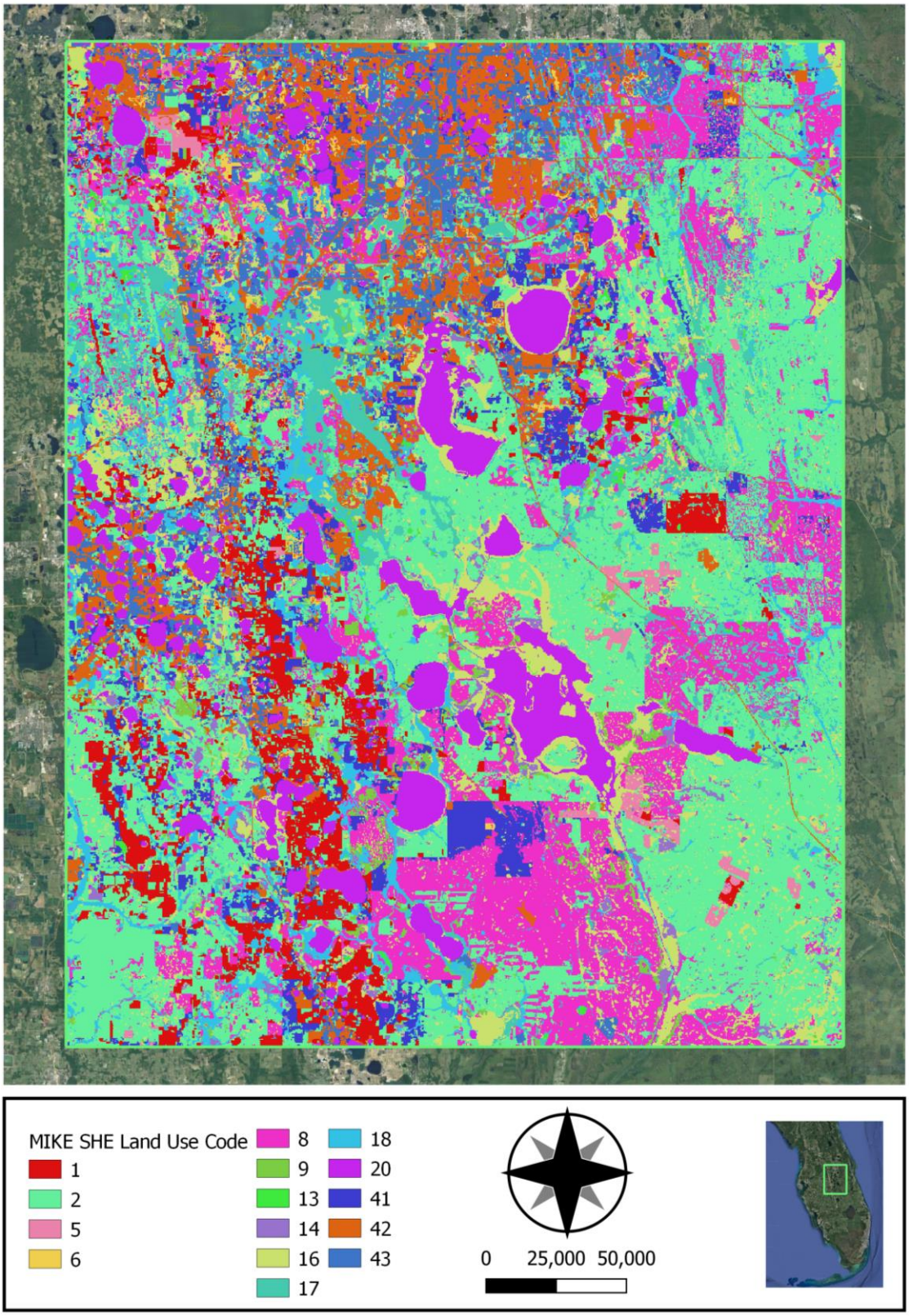

Figure 5-5: MIKE SHE Land Use Codes in UKB FPLOS Model

5.1.5.2 Irrigation

The Irrigation Command Areas (ICA) in MIKE SHE are used to describe the source of water for irrigation and method of application to the modeled area. The ICA data item is divided into two parts: The distribution dialogue for the ICAs; and source and application method for each ICA. ICAs are the grid cells in the model where irrigation will be applied based on either a calculated or specified water demand. Since the UKB FPLOS is focused on large storm events, it was assumed that irrigation will not be triggered.

5.1.6 Overland Flow Module

The finite difference equation governing the overland flow in MIKE is a two-dimensional diffusive wave approximation of the St. Venant's equations. The parameters associated with the OL module are spatially distributed and described in detail in the following sections.

5.1.6.1 Manning's Number (M)

Manning's M is the inverse of Manning's n and is equivalent to the Strickler roughness coefficient. The values capture the friction effects and impact the velocity of overland flow. The Manning's M (1/n) values in the UKB FPLOS Model are associated with the land use/vegetation types for the datasets from all 3 districts: SFWMD, SJRWMD and SWFWMD.

The 2D Manning's value map with land use categories are shown in **Table 5-4**, along with Manning's n values. The spatial distribution of Manning's M is shown in **Figure 5-6**. Initial values were derived from literature estimates and previous models in Florida, and values were adjusted as part of the calibration process, based on GIS and other data of finer resolution that could not be captured at the scale of the model cell size.

Table 5-4: Overland Flow Manning's M and n Values for each Land Use Category

MIKE SHE Codes	Description	Manning's M	Manning's n
1, 2, 5, 6, 8	Citrus, Pasture, Truck Crops, Golf Course, Flatwood	3 - 4	0.25 - 0.33
9, 13, 18	Hammock, Cypress, Swamp Forest	1.5 - 2	0.5 - 0.7
14, 16	Wet Prairie, Marsh	2.5	0.4
19, 20	Lakes, Water	25	0.04
41, 42, 43	Urban Low, Medium and High Density	10 - 21	0.05 – 0.1

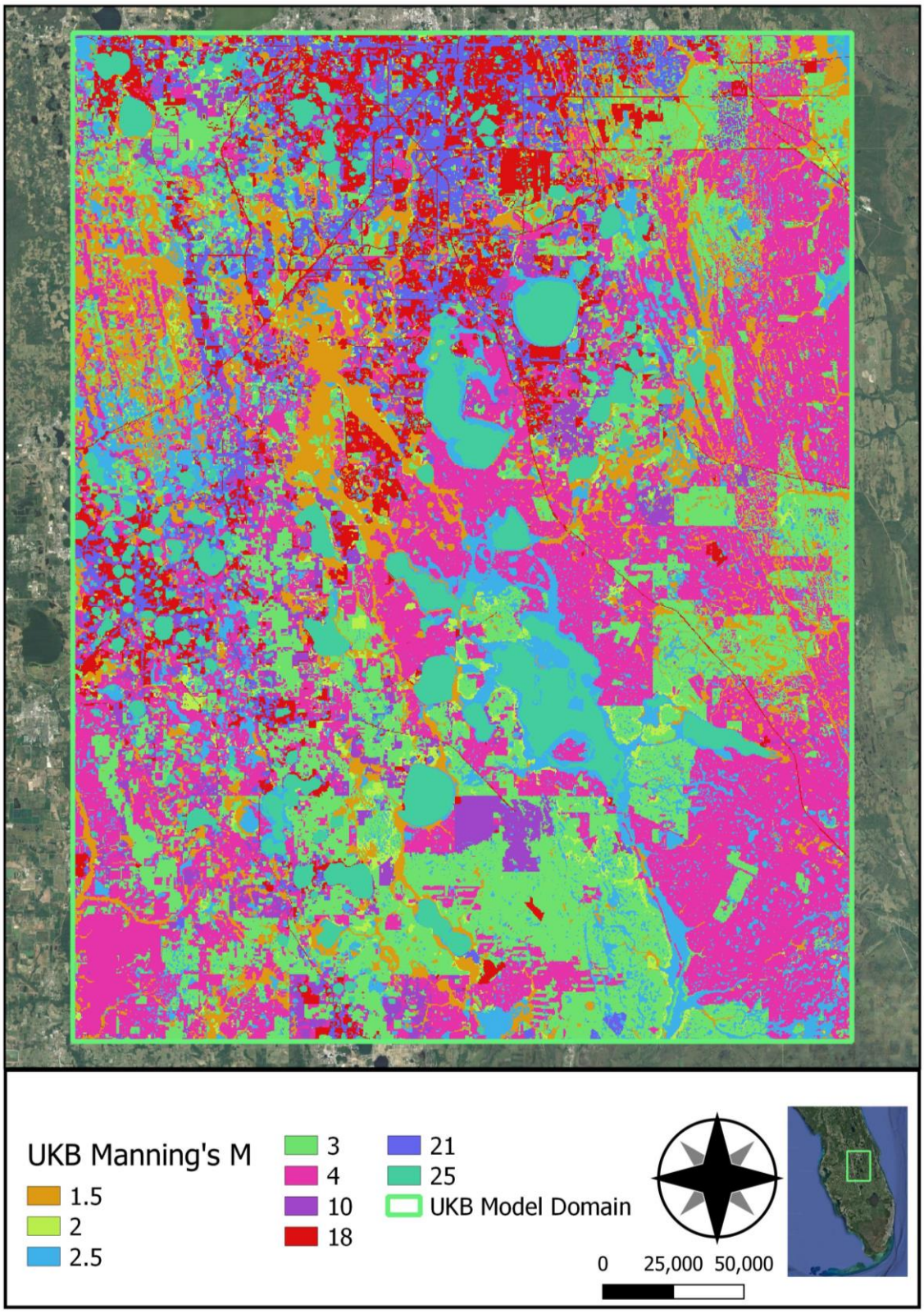

Figure 5-6: Manning's M values in the UKB FPLOS Model

5.1.6.2 Detention Storage

Detention Storage is the water depth that must be exceeded in a MIKE SHE model grid cell before overland flow can begin. Water in detention storage is not available for overland flow but is available for evapotranspiration and infiltration. Conceptually, it is a parameter that accounts for storage in microtopographic features that is lost when interpolating the high-resolution topography to the coarser grid resolution of the model topography. It is an important part of a flood model water budget. The association of the land use/vegetation types with the detention storage values for overland flow is shown in **Table 5-5** and the spatial distribution is shown in **Figure 5-7**. Initial values were derived from the previous Kissimmee MIKE model; some areas not captured in the model resolution were refined based on the imagery and date of the imagery that is closest to the calibration period.

Table 5-5: Overland Flow Detention Storage Values for each Land Use Category

MIKE SHE Codes	Description	Detention Storage (inches)
1, 5	Citrus, Flatwood, Hammock, Truck Crops, March, Cypress, Swamp Forest	0.25
17, 18	Wetland Coniferous Forests, Stream and Lake Swamps	0.4
8, 9, 13	Dry prairie, Upland Hardwood Forests, Cabbage Palm, Wet Prairie	0.3
2, 6	Pasture, Golf Course	0.25
20	Lakes, Water	0
16	Mixed Shrubs	0.15
41, 42, 43	Urban Low, Medium and High Density	0.1

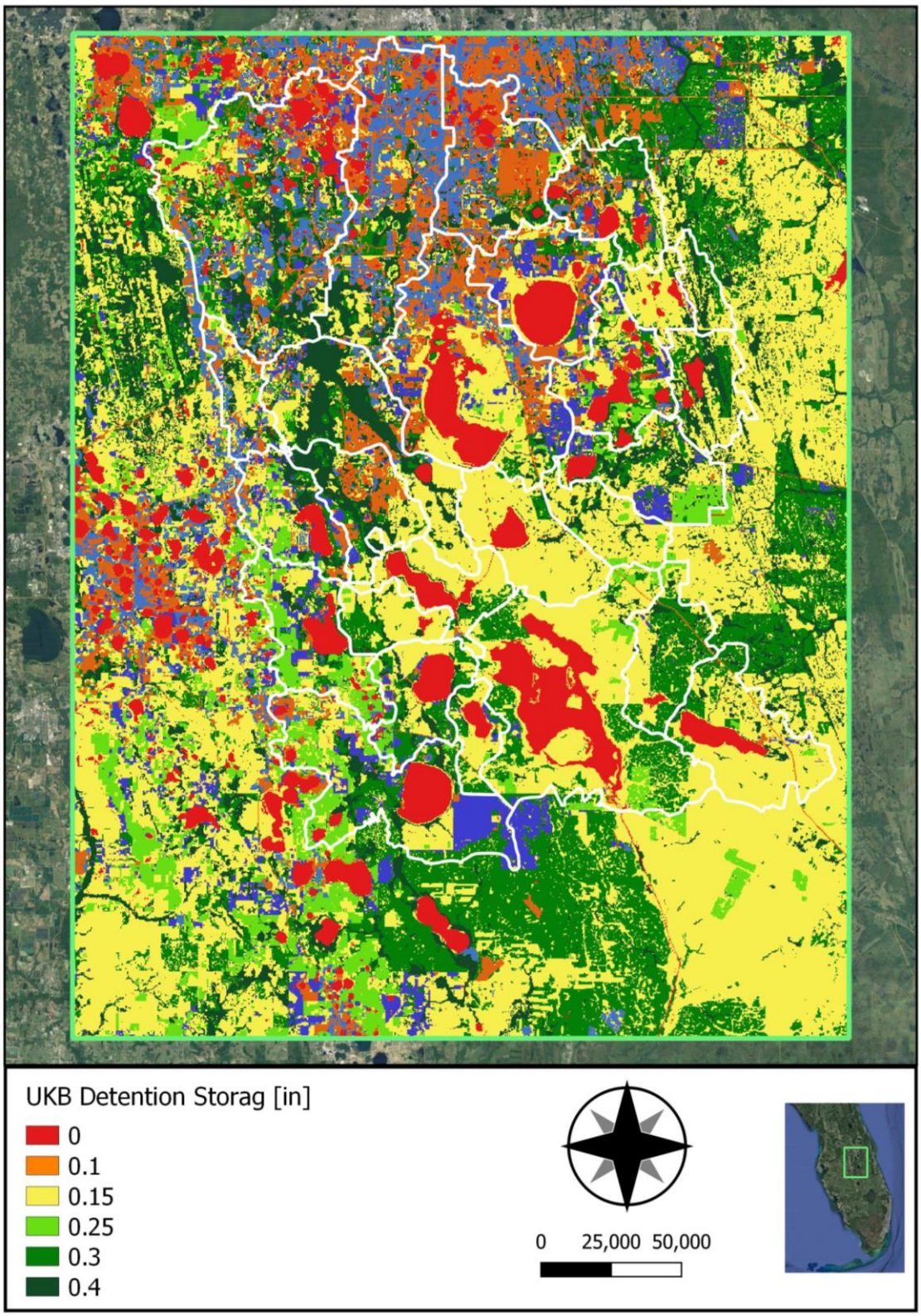

Figure 5-7: Detention Storage Depths in the UKB FPLOS Model

5.1.6.3 Initial Water Depth

The UKB FPLOS Model, for calibration, was set up to start at 0 feet (0 meters) depth (dry conditions) except for the lakes, canals, and wetlands. For the surface water bodies, the initial water levels are set at water levels under normal, average conditions.

5.1.6.4 Surface – Subsurface Leakage Coefficient

The surface – subsurface leakage coefficient represents resistance to vertical flow across the topographic surface in both directions. This parameter accounts for soil compaction or finer sediment deposits than what is in the overall soil profiles.

For the UKB FPLOS Model, surface-subsurface leakage coefficient of E-8 to E-7 per second was used for wet prairie, marsh, swamp forest to represent higher resistance to vertical flow due to organic matter and water, and lakes to represent organic matter and siltation.

5.1.6.5 Separated Flow Areas

Separated Flow Areas (SFAs) are polygons in the model that contain or limit overland flow. Conceptually, they represent major roads and levees that are finer than the resolution of the model and therefore cannot be properly represented. Within the boundaries of these SFAs, water drains to the canals within the SFAs or to specified locations.

In the UKB FPLOS Model, the SFAs presently are the watershed boundaries from the 3 WMDs. During model calibration, this approach was reevaluated to check if the SFAs boundaries are too restrictive; there was no indication that they were. There are also other datasets and models, particularly, in Econlockahatchee and Deseret Ranch on the eastern portion of the UKB FPLOS Model that have representations of SFAs – as hand drawn figures from observations, ICPR discretization, HSPF discretization and other sources. All these datasets were used in the evaluation. **Figure 5-8** shows the SFAs used in the UKB FPLOS Model.

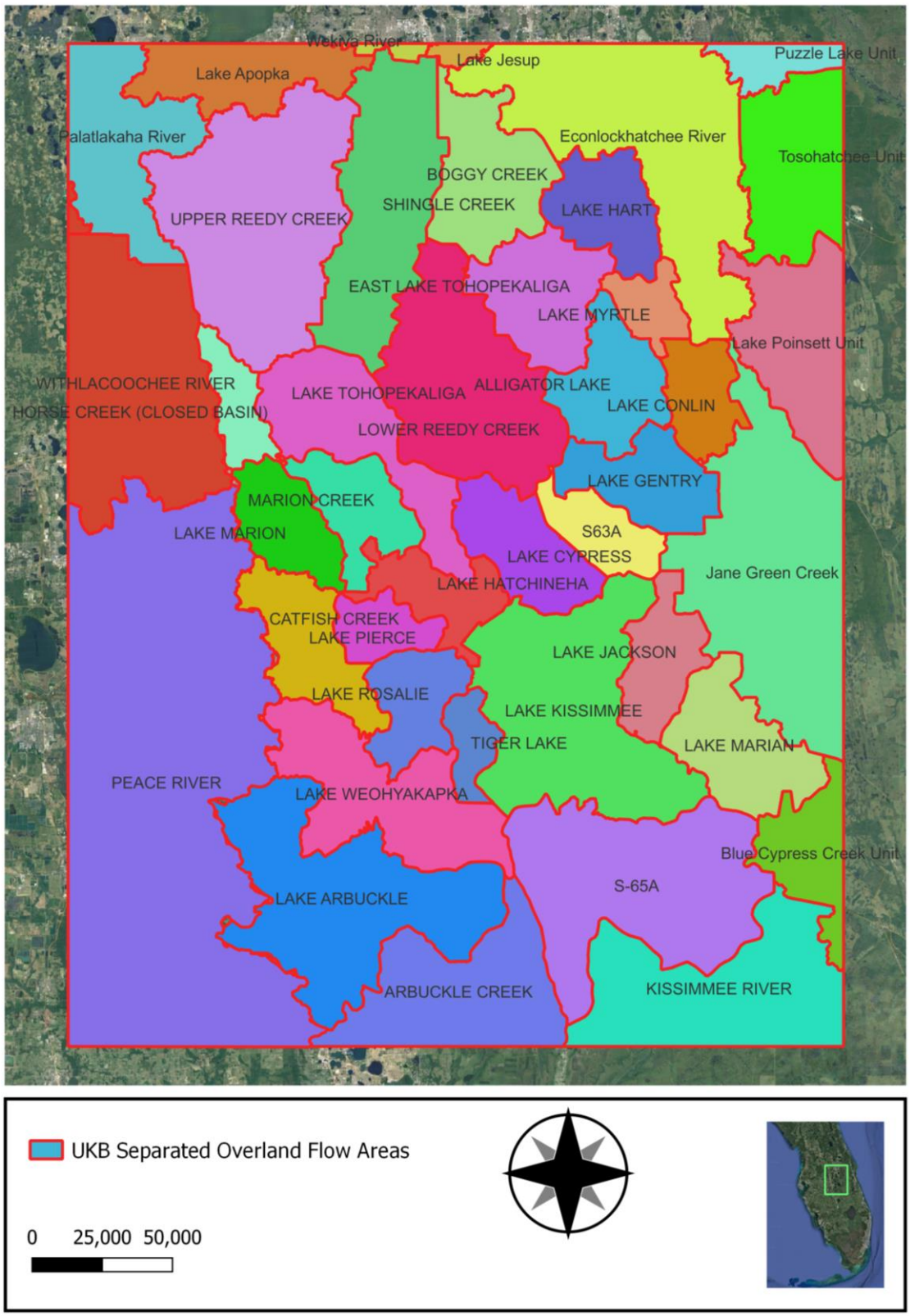

Figure 5-8: Separated Flow Areas in UKB FPLOS Model

5.1.6.6 Ponded Drainage

Ponded drainage in MIKE SHE is designed to represent urban stormwater runoff and channelized flows from smaller streams and ditches not explicitly represented in the hydraulic model. For urban runoff, there are two parameters in the model that represent impervious areas:

- Paved Area Fraction (PAF) represents the total impervious area, and infiltration is reduced by the fraction specified
- Runoff Coefficient (RC) represents directly connected impervious areas (DCIA). The fraction of the ponded water above the specified drain level is routed directly to the nearest down-gradient surface water feature by drainage runoff.

Drainage flowrates are limited by the specified inflow and outflow time constants, which represent the drainage capacity and storage of the storm drains. If the inflow and outflow time constants are equal, it implies there is no storage in the drains. In MIKE SHE, drain flowrate is calculated by multiplying the time constants by the area of the MIKE SHE OL cell size and the depth of the ponded water above it.

The UKB FPLOS Model parameters are based on land use designated as urban in the LCLU datasets. The stormwater dataset (the channels datasets) obtained from Osceola County has been used to represent ponded drainage in the model domain. The RC and time constants parameters specified in the UKB FPLOS model are shown in **Table 5-6**; the inflow and outflow time constants in the table are the average of the values used for each land use. The time constants are shown in the units as they are input into the model (per second). The values were used as calibration parameters but modified based on imagery and any land uses not captured in the cell size but influence the drainage time.

Table 5-6: Runoff Coefficients and Time Constants

Land Use	Runoff Coefficient (%)	Inflow Time Constants (1/s)	Outflow Time Constants (1/s)
Low Density Residential	5	1e-4	1e-6
Medium Density Residential	15	1e-3	7e-6
High Density Residential	45	9e-3	9e-6

5.1.7 Unsaturated Flow Module

Unsaturated Flow Zone (UZ) processes include soil moisture replenishment by rainfall and removal by evapotranspiration as well as interaction with the underlying saturated zone. In MIKE SHE, the flow in UZ is conceptualized as one-dimensional (downward) vertical flow (infiltration). MIKE SHE/ MIKE HYDRO has three (3) ways to represent the UZ:

- Full Richard's equation which is the most accurate but computationally intensive
- Gravity flow method assumes a uniform vertical gradient and considers the capillary forces negligible.

- Simple two-layer approach, which divides the unsaturated zone into a layer above the root zone and a zone below.

The UKB FPLOS Model uses the simple two-layer method. The 2-layer UZ method assumes a uniform soil profile with depth. This method is generally considered applicable in settings like the UKB where the water table is relatively shallow and the primary purpose of the UZ simulation is to represent groundwater recharge (DHI, 2021).

5.1.7.1 Soils Map and Properties

The soils data from the NRCS SSURGO database was obtained for the entire domain. The grouped units are linked to a soils database with defined parameters for each of the soil horizons. The relationship with the NRCS map units to the UKB FPLOS Model generalized categories was determined by spatial associations in the 2-dimensional grid (dfs2).

There are a total of 314 map units of soil in the UKB FPLOS Model domain. Soils representing less than 0.1% of the model area and water areas were removed and interpolated from neighboring values reducing the number of soil types from 314 to 136. The 136 soils were grouped into 36 categories based on similarity of saturated hydraulic conductivity, available water capacity (AWC), and bulk density values. In general, the process to define water content/ soil moisture content in the soil properties dialog was:

- Water Content at Wilting point (WP): Literature estimates were used; 0.05 for sand, 0.10 for fine sand, 0.15 for silt/clay/muck
- Water Content at Field capacity (FC): Wilting point plus AWC
- Water Content at Saturation (Sat) – Bulk density / 2.65 which is the Specific Gravity of soil

Assumptions:

- Saturated hydraulic conductivity (Ksat) values were taken as the lowest (limiting) value of all layers from the soils database
- AWC and bulk density values taken as the weighted average of all layers from the soils database
- Soil moisture content values were checked to ensure Sat > FC > WP; the MIKE SHE Soil with Grid Code 12 was adjusted to ensure Sat > FC > WP

Figure 5-9 shows the spatial distribution of the soil layer in the UKB FPLOS Model and **Table 5-7** shows the properties used in the model, for the 314 map units that were condensed to 36. During calibration, this was further consolidated to 6 types, simplified based on sensitivity analysis.

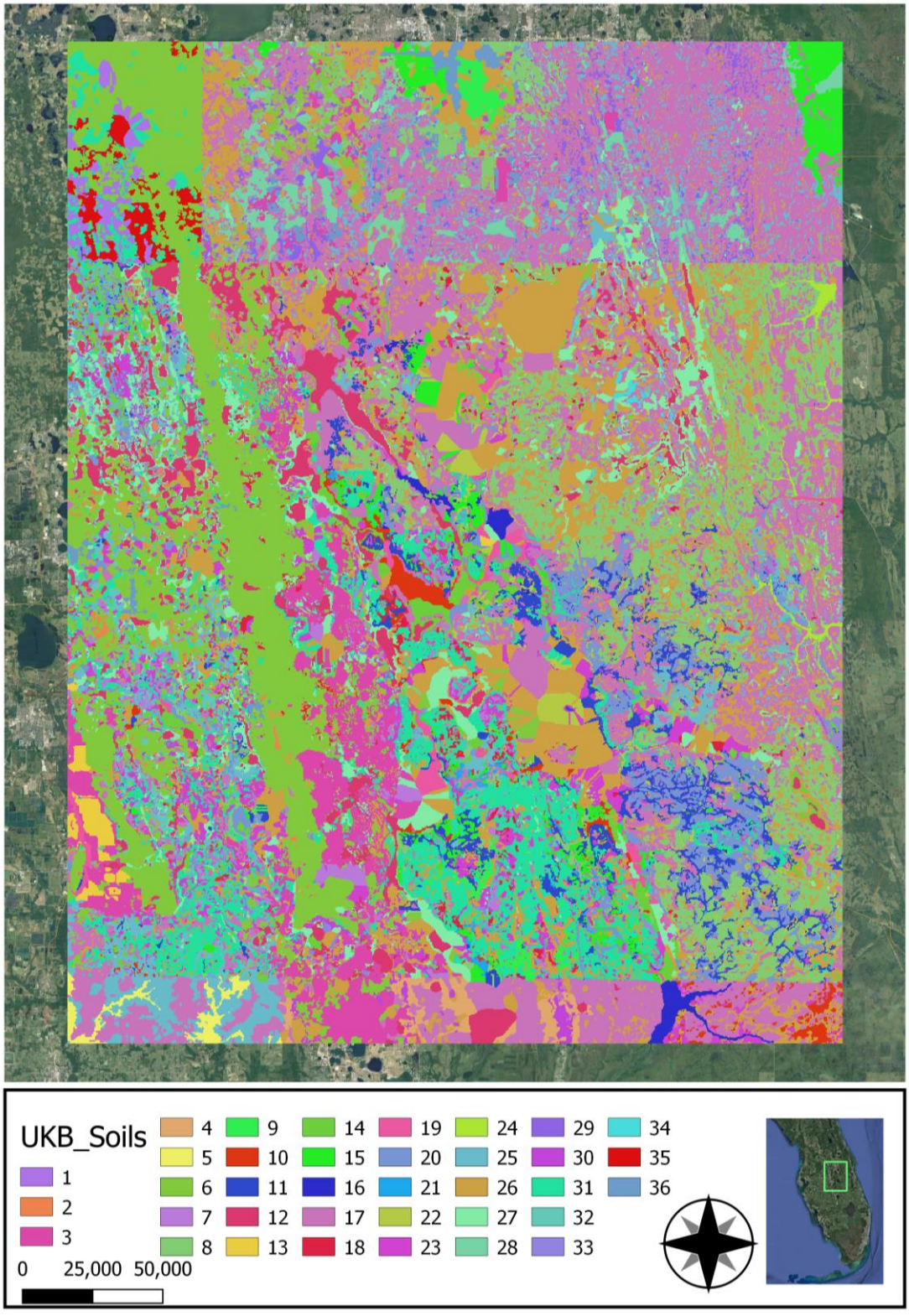

Figure 5-9: Soil Classification Based on NRCS (United States Department of Agriculture, 2021) for the UKB FPLOS Model Domain

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Table 5-7: Soil Properties in UKB FPLOS Unsaturated Zone

Soils Grid Code	AWC	Bulk Density (g/cc)	Texture	K sat (m/s)	Sat	FC	WP
1	0.10	1.46	Sand	4.20E-07	0.55	0.15	0.05
2	0.09	1.55	Fine Sand	1.40E-06	0.58	0.19	0.1
3	0.04	1.54	Sand	1.40E-04	0.58	0.09	0.05
4	0.10	1.49	Sand	1.40E-06	0.56	0.15	0.05
5	0.10	1.60	Sand	4.20E-07	0.6	0.15	0.05
6	0.05	1.61	Fine Sand	4.20E-05	0.61	0.15	0.1
7	0.07	1.52	Fine Sand	1.40E-05	0.57	0.17	0.1
8	0.09	1.57	Fine Sand	4.20E-06	0.59	0.19	0.1
9	0.07	1.51	Sand	4.20E-05	0.57	0.12	0.05
10	0.12	1.53	Fine Sand	4.20E-07	0.58	0.22	0.1
11	0.09	1.59	Fine Sand	1.40E-05	0.6	0.19	0.1
12	0.36	0.39	Silt/Clay	4.20E-05	0.3	0.25	0.1
13	0.16	1.15	Silt/Clay	1.00E-08	0.43	0.31	0.15
14	0.22	1.21	Silt/Clay	4.20E-07	0.46	0.37	0.15
15	0.08	1.56	Fine Sand	4.20E-07	0.59	0.18	0.1
16	0.13	1.52	Fine Sand	4.20E-07	0.57	0.23	0.1
17	0.08	1.56	Fine Sand	4.10E-06	0.59	0.18	0.1
18	0.05	1.57	Fine Sand	1.40E-05	0.59	0.15	0.1
19	0.15	1.42	Loam	4.20E-07	0.54	0.3	0.15
20	0.12	1.59	Fine Sand	4.20E-07	0.6	0.22	0.1
21	0.13	1.58	Fine Sand	4.00E-06	0.6	0.23	0.1
22	0.09	1.60	Fine Sand	1.40E-05	0.6	0.19	0.1
23	0.10	1.49	Fine Sand	4.00E-06	0.56	0.2	0.1
24	0.10	1.37	Sand	4.20E-07	0.52	0.15	0.05
25	0.10	1.52	Fine Sand	2.20E-06	0.57	0.2	0.1
26	0.08	1.55	Fine Sand	4.10E-05	0.59	0.18	0.1

Soils Grid Code	AWC	Bulk Density (g/cc)	Texture	K sat (m/s)	Sat	FC	WP
27	0.17	1.14	Silt/Clay	4.20E-05	0.43	0.32	0.15
28	0.20	0.88	Silt/Clay	4.20E-05	0.33	0.25	0.05
29	0.13	1.34	Silt/Clay	4.20E-05	0.51	0.28	0.15
30	0.04	1.61	Sand	1.40E-04	0.61	0.09	0.05
31	0.08	1.50	Fine Sand	4.00E-06	0.57	0.18	0.1
32	0.08	1.65	Sand	4.00E-06	0.62	0.13	0.05
33	0.13	1.47	Sand	1.40E-06	0.55	0.18	0.05
34	0.08	1.55	Fine Sand	1.40E-04	0.58	0.18	0.1
35	0.16	1.06	Silt/Clay	1.40E-05	0.4	0.31	0.15
36	0.08	1.58	Fine Sand	4.20E-07	0.6	0.18	0.1

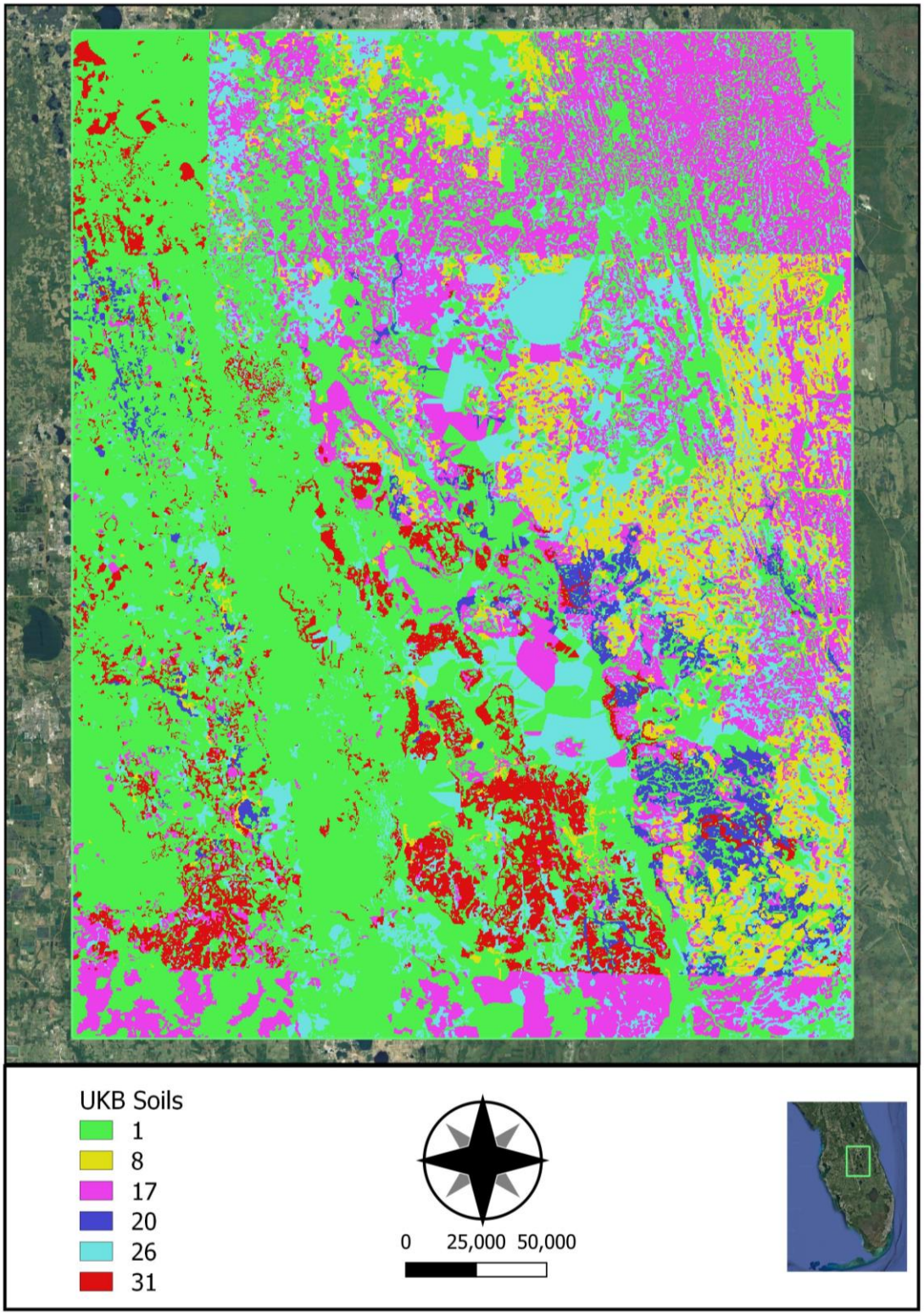

Figure 5-10: Consolidated UZ Soils based on NRCS (United States Department of Agriculture, 2021) for the UKB FPLOS Model Domain

5.1.8 Saturated Flow Module

5.1.8.1 Geological Layers

Geological layers in the Saturated Zone component were defined in the UKB FPLOS Model based on the ECFTX model (CFWI HAT, 2020). **Table 5-8** shows the hydrogeologic units and the abbreviations in ECFTX model; layers 1-3 are represented in UKB FPLOS Model. Layers 1, 2 and 3 (SA. ICU and UFA-upper) are all represented explicitly. The UKB FPLOS calibration and validation models use the surfaces made from observed data.

Table 5-8: ECFTX Model Layers (from CFWI, 2020)

Layer	Hydrogeologic Unit	Abbreviation
1	**Surficial Aquifer**	**SA**
2	**Intermediate Confining Unit**	**ICU**
3	**Upper Floridan Aquifer – upper permeable zone**	**UFA-upper**
4	Ocala-Avon Park low permeability zone	OCAPlpz
5	Avon Park high-permeability zone	APhpz
6	first component of the Middle Confining Unit	MCU_I
7	Overlap unit of the Lower Floridan aquifer	MCU_I and MCU_II
8	Third component of the MCU	MCU_II
9	first subdivision of the LFA	LFA-upper
10	second subdivision of the LFA	GLAUClpu
11	Lower Floridan aquifer – basal permeable zone	LF-basal

Figure 5-11, Figure 5-12 and **Figure 5-13** show the bottom elevations of UKB FPLOS Model layers. The horizontal and vertical conductivities and storage parameters were also extracted for each of the three UKB FPLOS Model layers. Storage coefficients of saturated zone layers is one of the required inputs in MIKE SHE; ECFTX storage coefficients were divided by the thickness of each layer to get these values.

The initial values were modified during calibration; **Figure 5-14 through Figure 5-19** show the horizontal and vertical hydraulic conductivity values and **Figure 5-20** shows the specific yield in Layer 1 of the UKB FPLOS Model currently. The extremely high values of conductivity and specific yield represent the high conductivity lakes and/or sink holes.

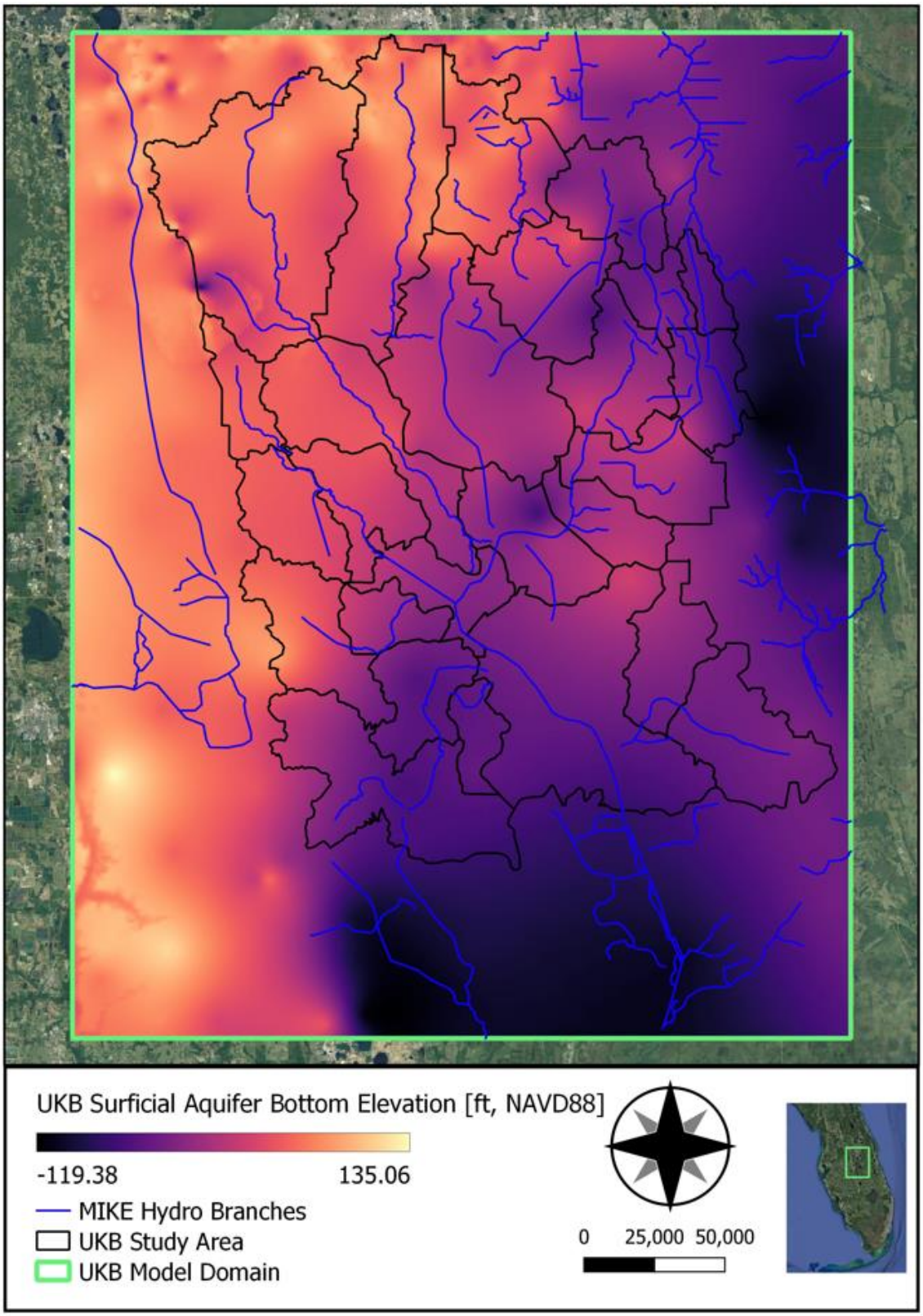

Figure 5-11: Bottom of SA (UKB FPLOS Model Layer 1)

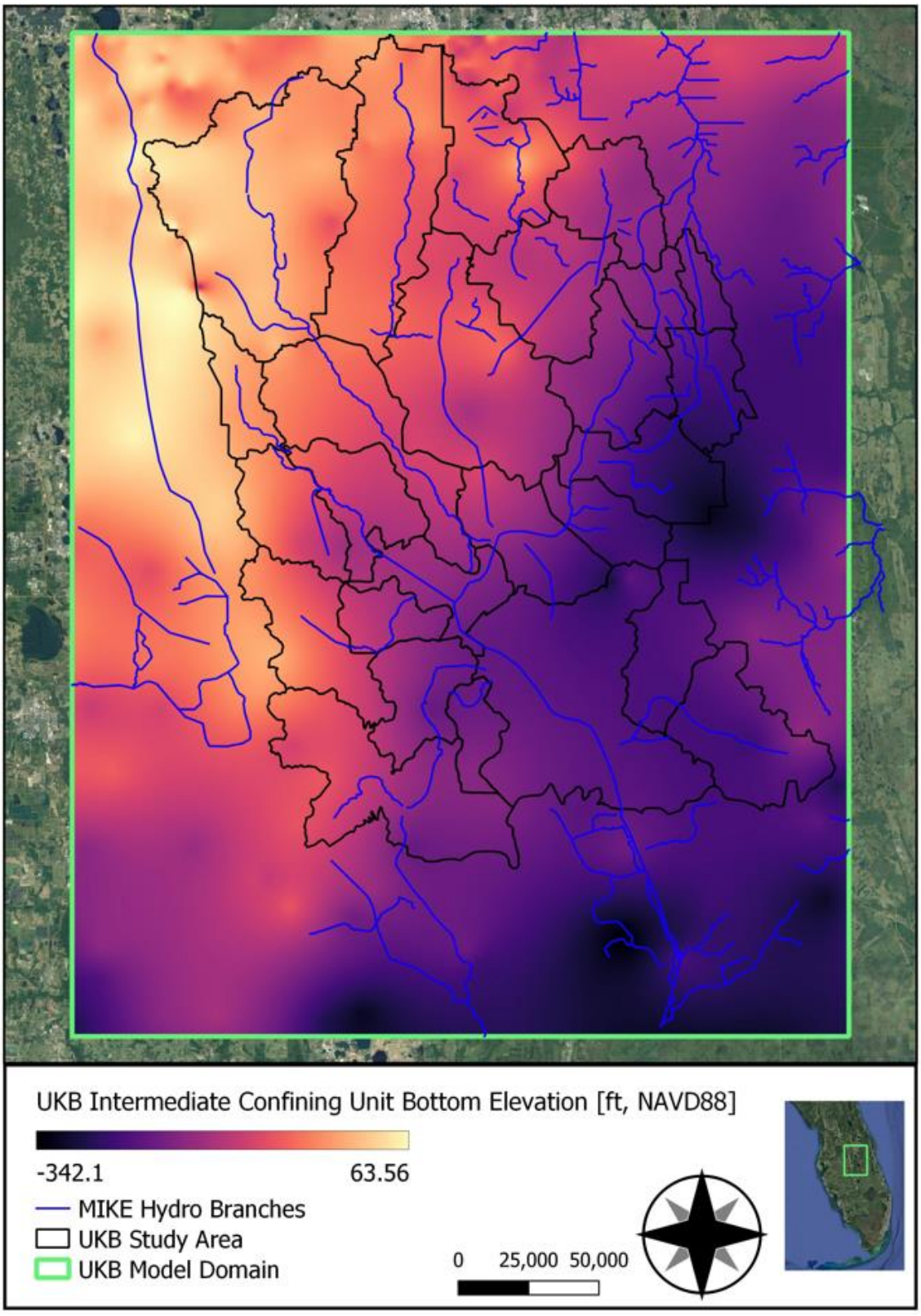

Figure 5-12: Bottom of ICU (UKB FPLOS Model Layer 2)

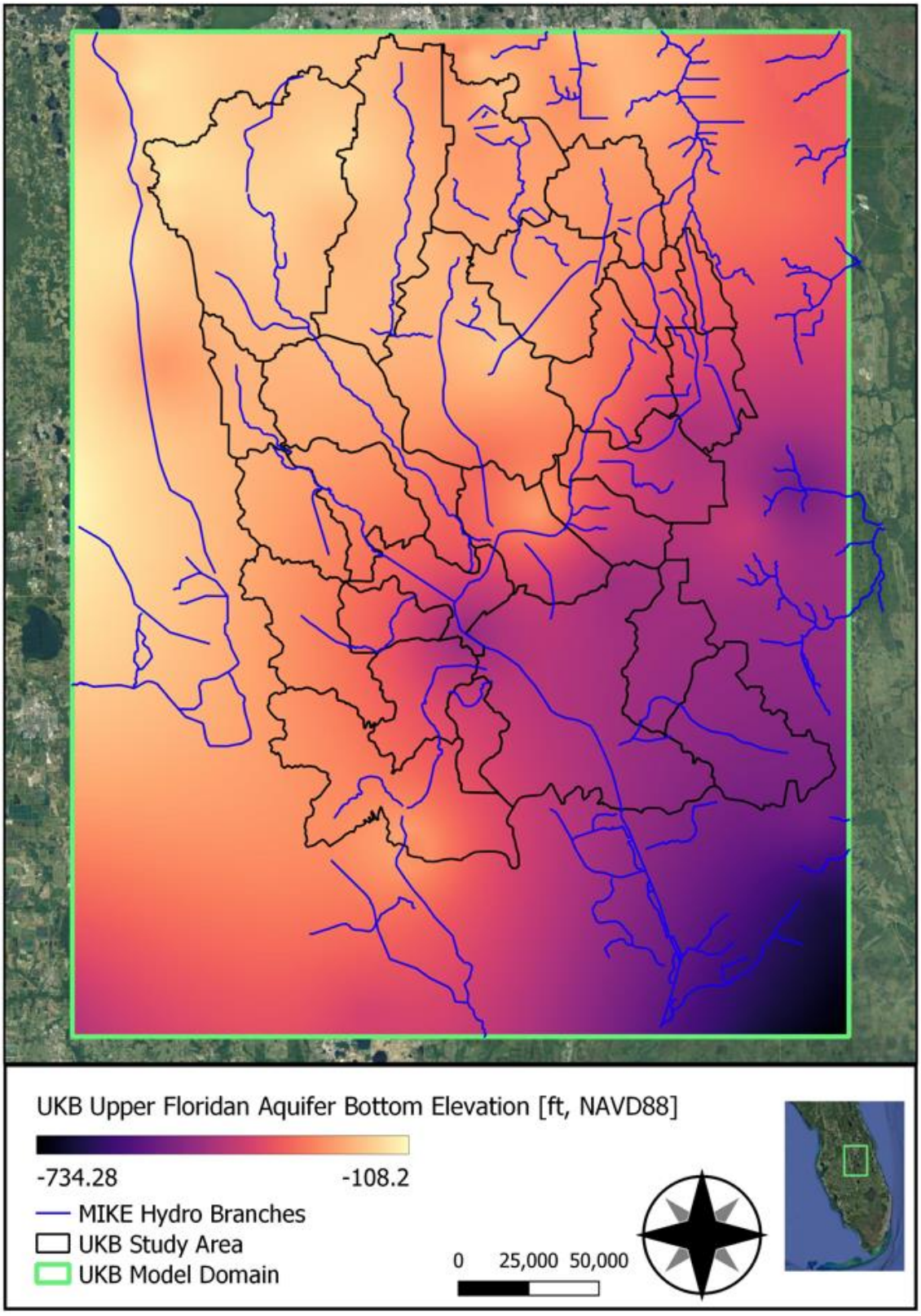

Figure 5-13: Bottom of UFA-Upper (UKB FPLOS Model Layer 3)

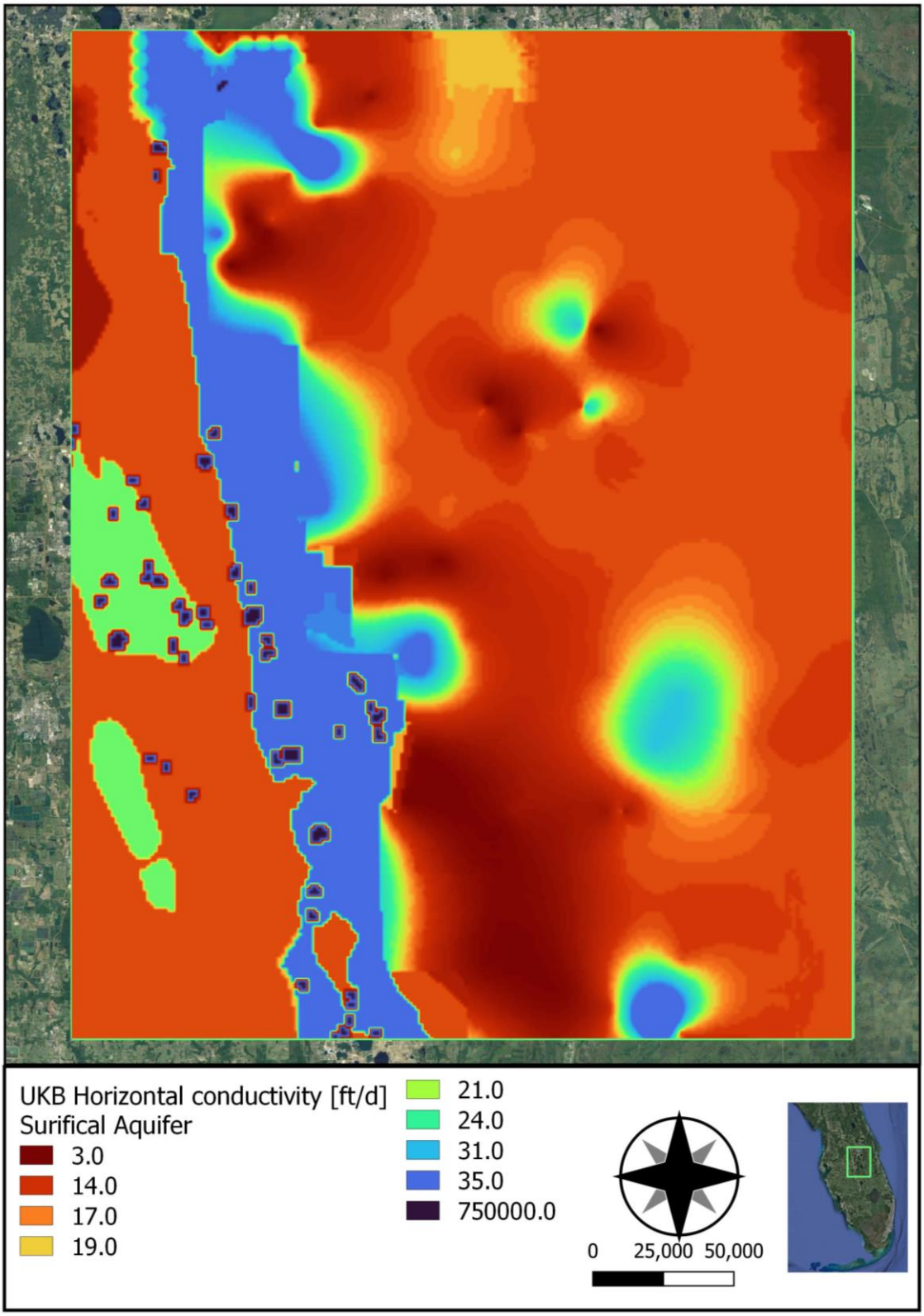

Figure 5-14: Horizontal Hydraulic Conductivity of SA (UKB FPLOS Model Layer 1)

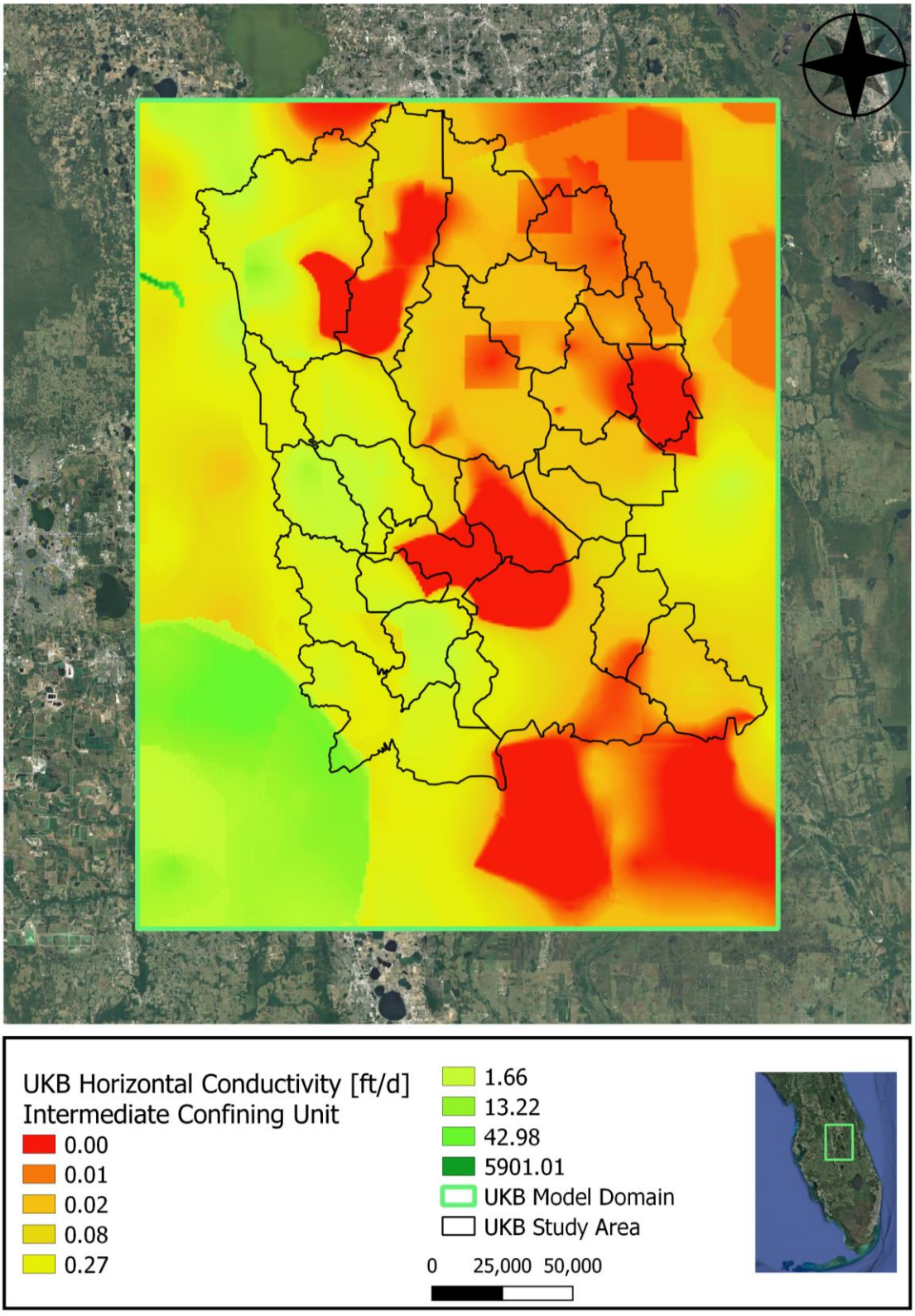

Figure 5-15: Horizontal Hydraulic Conductivity of ICU (UKB FPLOS Model Layer 2)

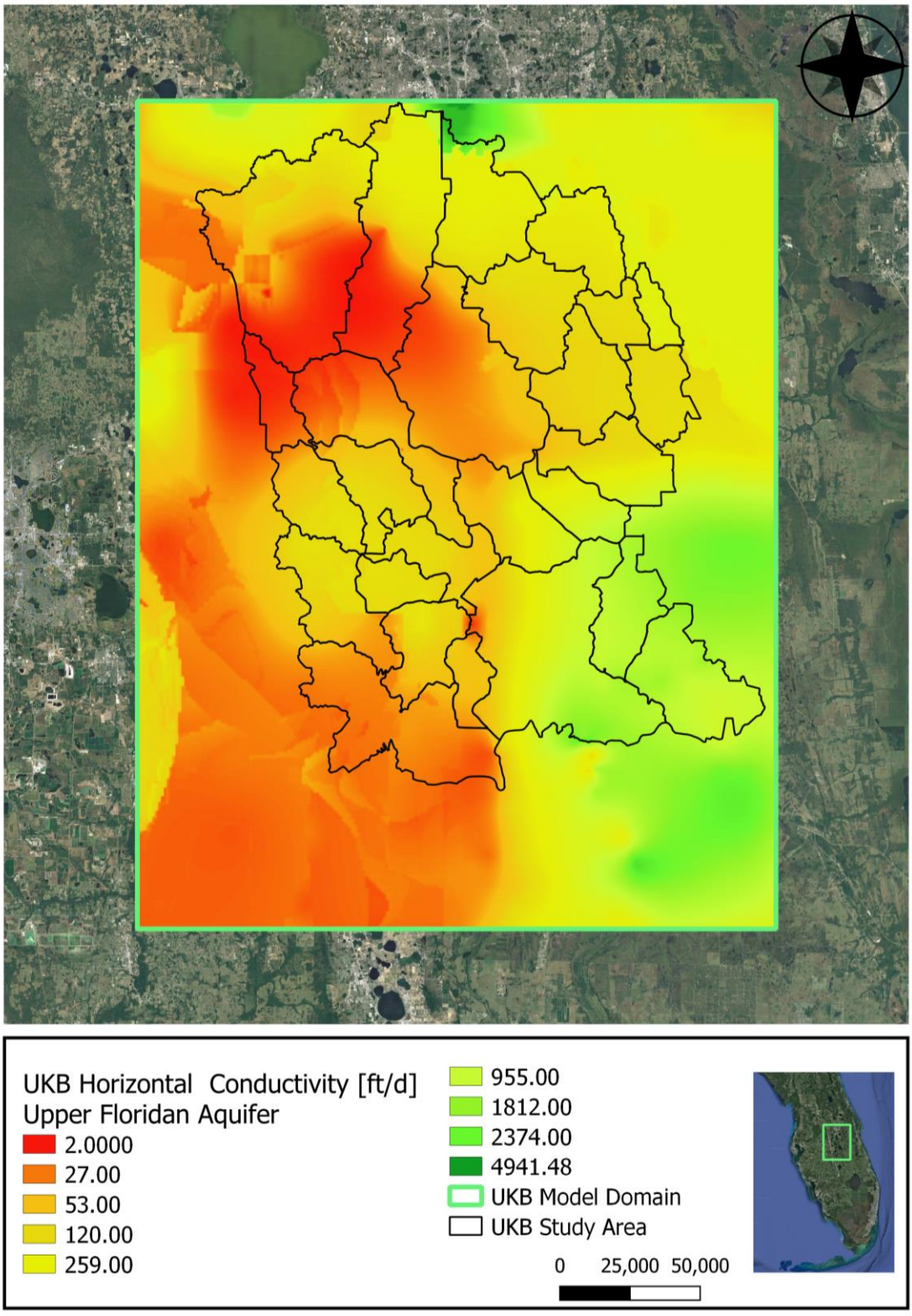

Figure 5-16: Horizontal Hydraulic Conductivity of UFA-Upper (UKB FPLOS Model Layer 3)

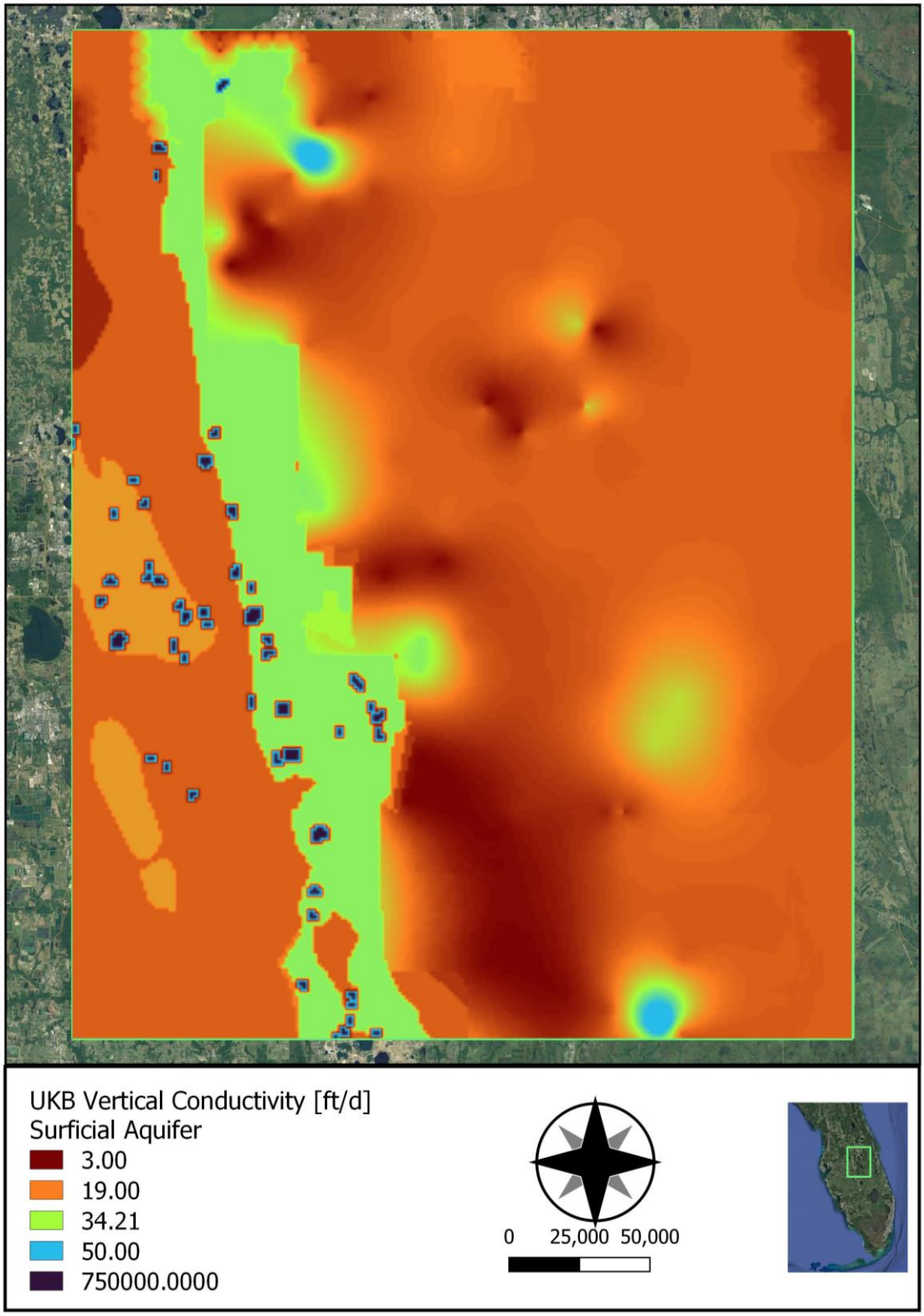

Figure 5-17: Vertical Hydraulic Conductivity of SA (UKB FPLOS Model Layer 1)

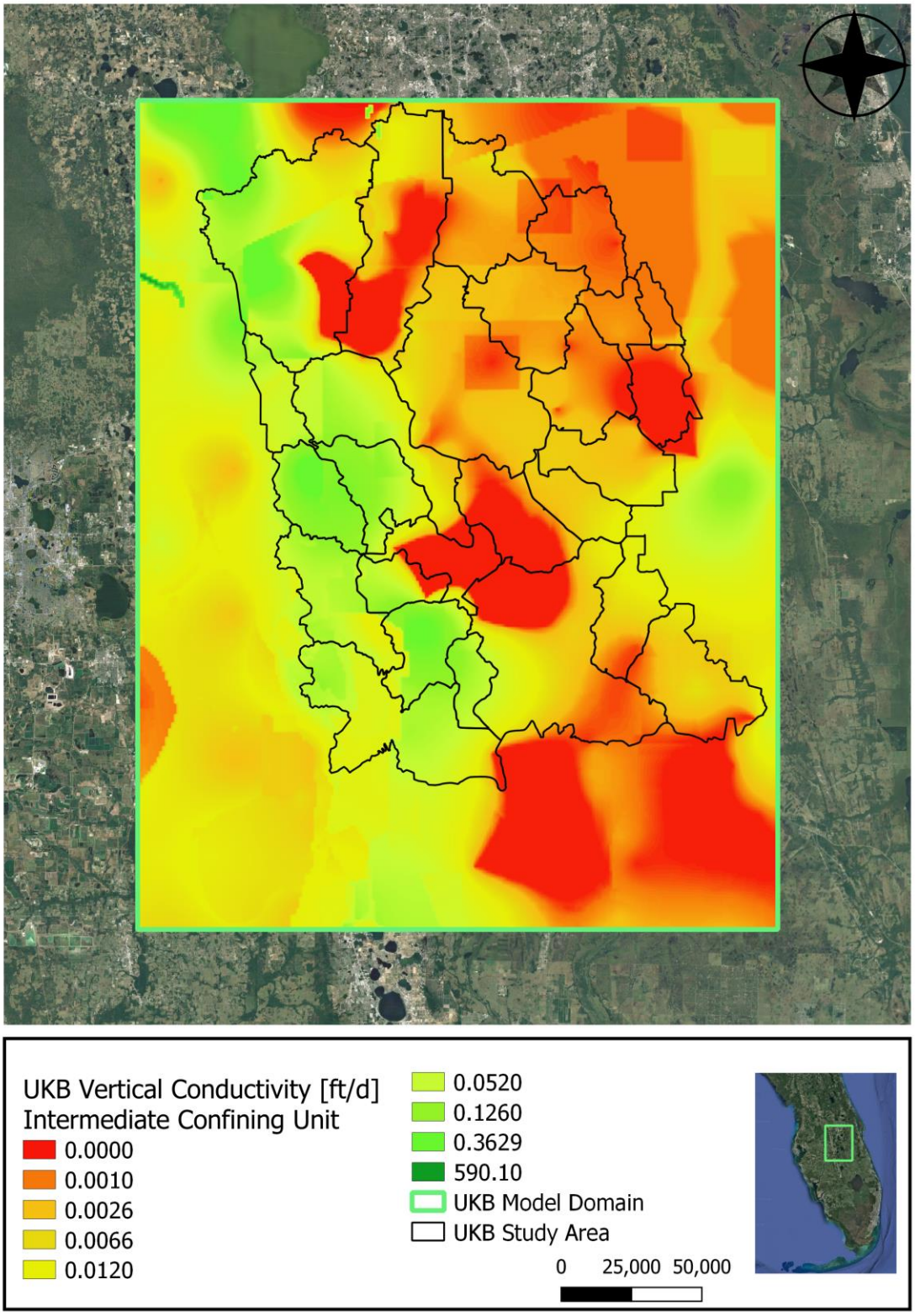

Figure 5-18: Vertical Hydraulic Conductivity of ICU (UKB FPLOS Model Layer 2)

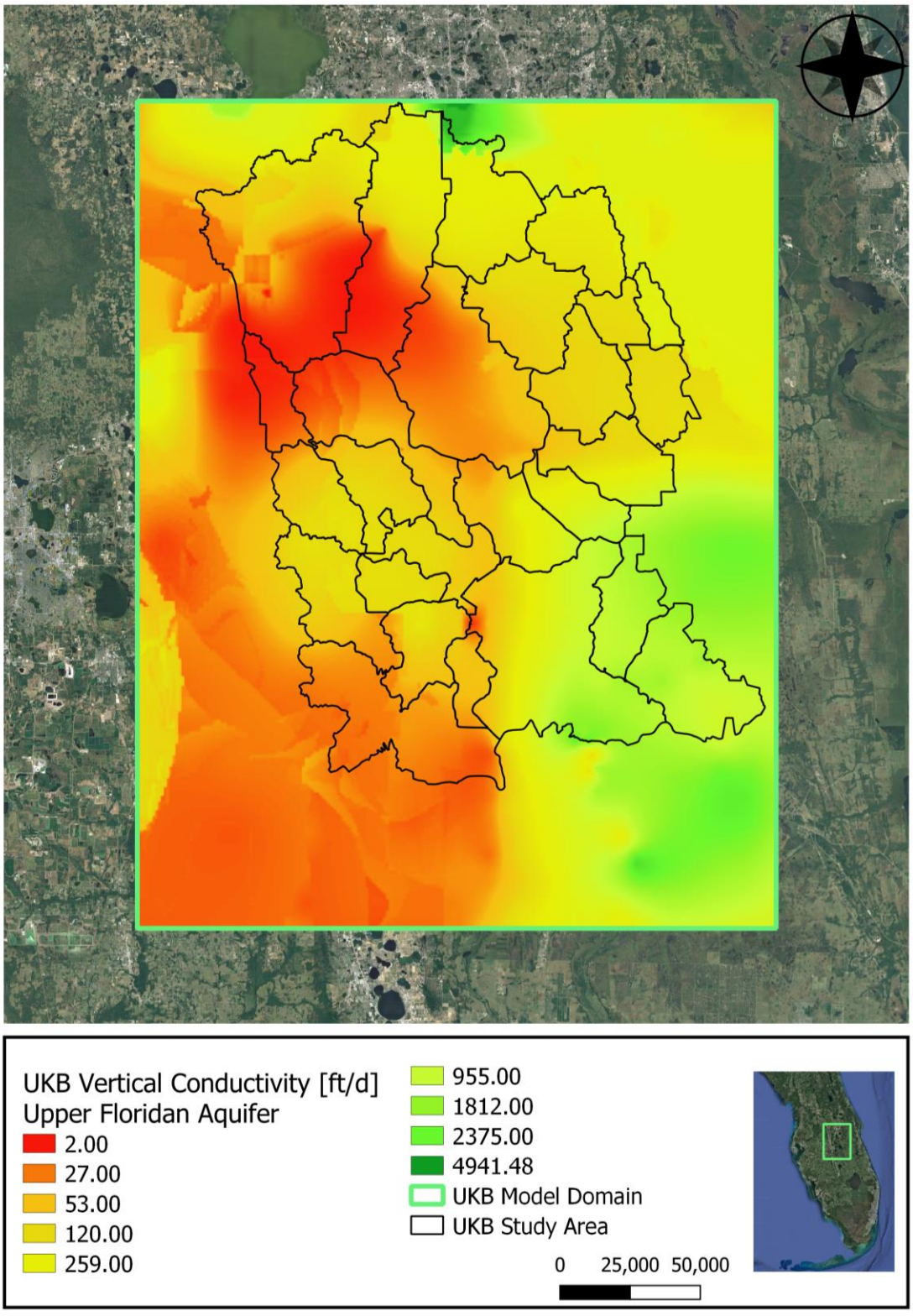

Figure 5-19: Vertical Hydraulic Conductivity of UFA-Upper (UKB FPLOS Model Layer 3)

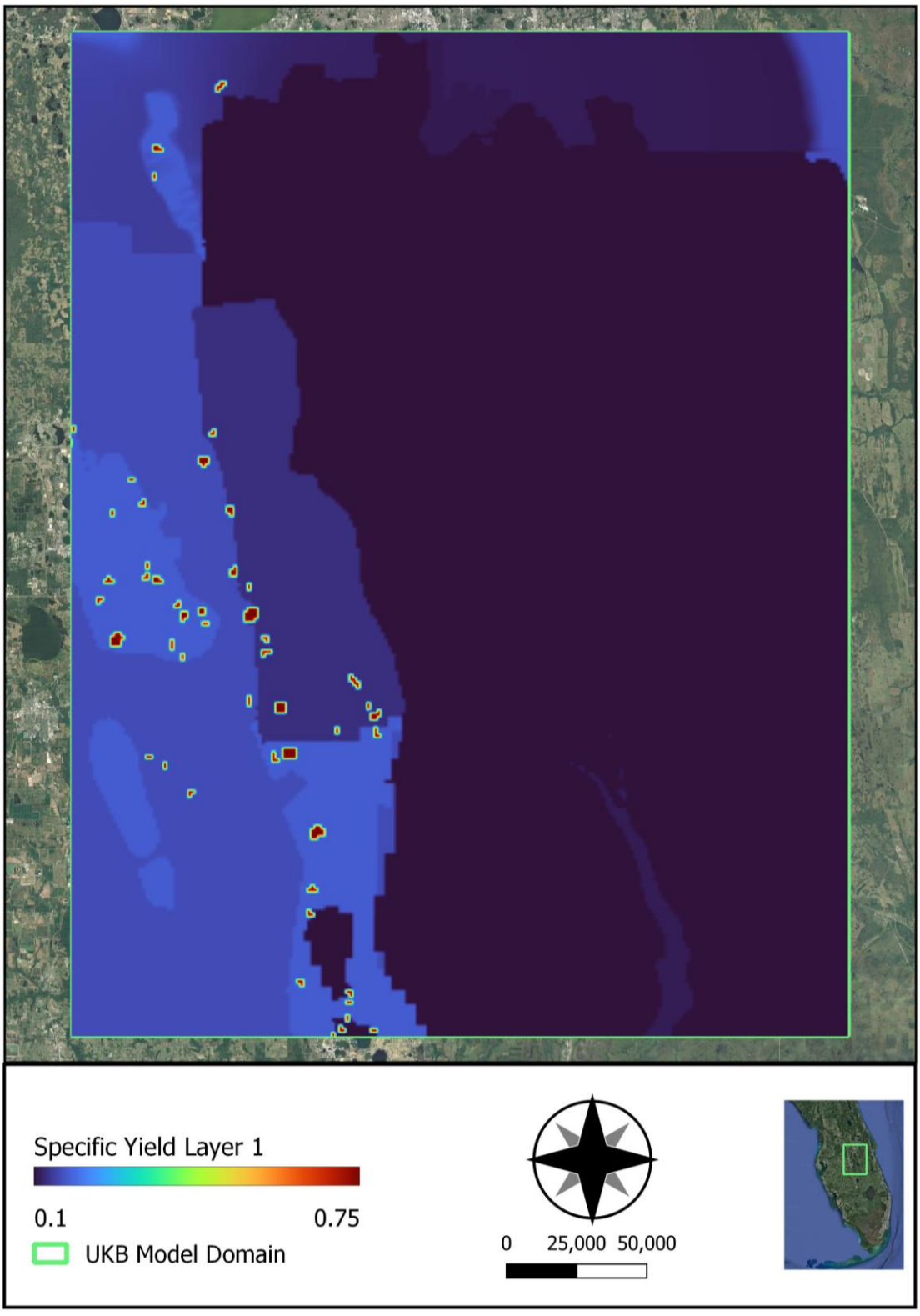

Figure 5-20: Specific Yield of Surficial Aquifer (UKB FPLOS Model Layer 1)

5.1.8.2 Saturated Zone Drainage

The drainage option in MIKE SHE can simulate flow through stormwater pipe network, but also conceptually, the drainage from saturated zone to ditches and other surface drainage features. Drainage flow occurs in the groundwater layer that the drains are located in. In MIKE SHE, the drainage system is simulated in each of the model grid cells that contains the drains. The volume and rate of outflow depends on the gradient between the water table and the drain level, and a specified time constant. The time constant represents as a lumped parameter, the density of the drainage system and the permeability of the material surrounding the drains.

In the UKB FPLOS Model, the drainage flow simulates the relatively fast surface runoff where the spatial resolution of the model cell is too large to represent small scale microtopographic variations in the UKB. The saturated zone drainage representation is based on the National Hydrography Dataset (NHD). Secondary drainage network from NHD was used to simulate the wetlands and forest that drain to the primary SFWMD network.

5.1.8.3 Stormwater Drainage Wells

Central Florida utilities have been using drainage wells for augmentation of reclaimed water systems for irrigation or other non-potable uses (Adamski and German, 2004). Approximately 400 drainage wells in the central Florida area inject stormwater by gravity into the freshwater zones of the Floridan Aquifer (Schiner and German, 1983). Approximately 50% of the drainage wells are used to dispose of runoff from roads and other impervious areas, about 35% to regulate lake levels, and about 15% for disposal of other wastewater. Water injected by drainage wells moves downgradient towards supply wells. Many factors affect the reliability of drainage wells like available storage, conditions downstream and climate conditions. These wells also typically provide recharge for the Floridan Aquifer System.

Representation of stormwater drainage wells in the UKFPLOS Model is important to reasonable simulations and predictions of water budgets and flooding extents. **Figure 5-21** shows the location of all the drainage wells and the active drainage wells in the UKB FPLOS model domain.

The documented average volume of injection is 0.5 MGD (Schiner and German, 1983). Sensitivity tests done during model calibration showed that the impacts of the stormwater drainage wells were masked by boundary effects of the model since these wells are predominantly in the northern portion of the model near the boundary. Therefore, they were not represented in the model. However, the information is included in this report, should this become important in model applications post-project.

5.1.8.4 Rapid Infiltration Basins (RIBs)

Large volumes of reclaimed water, that underwent advanced secondary treatment, are reused through land-based applications in central Florida. They provide artificial recharge to the SA through rapid infiltration basins (RIBs) and are also important to represent in the model. They are included in the UKB FPLOS Model and are shown in **Figure 5-22**.

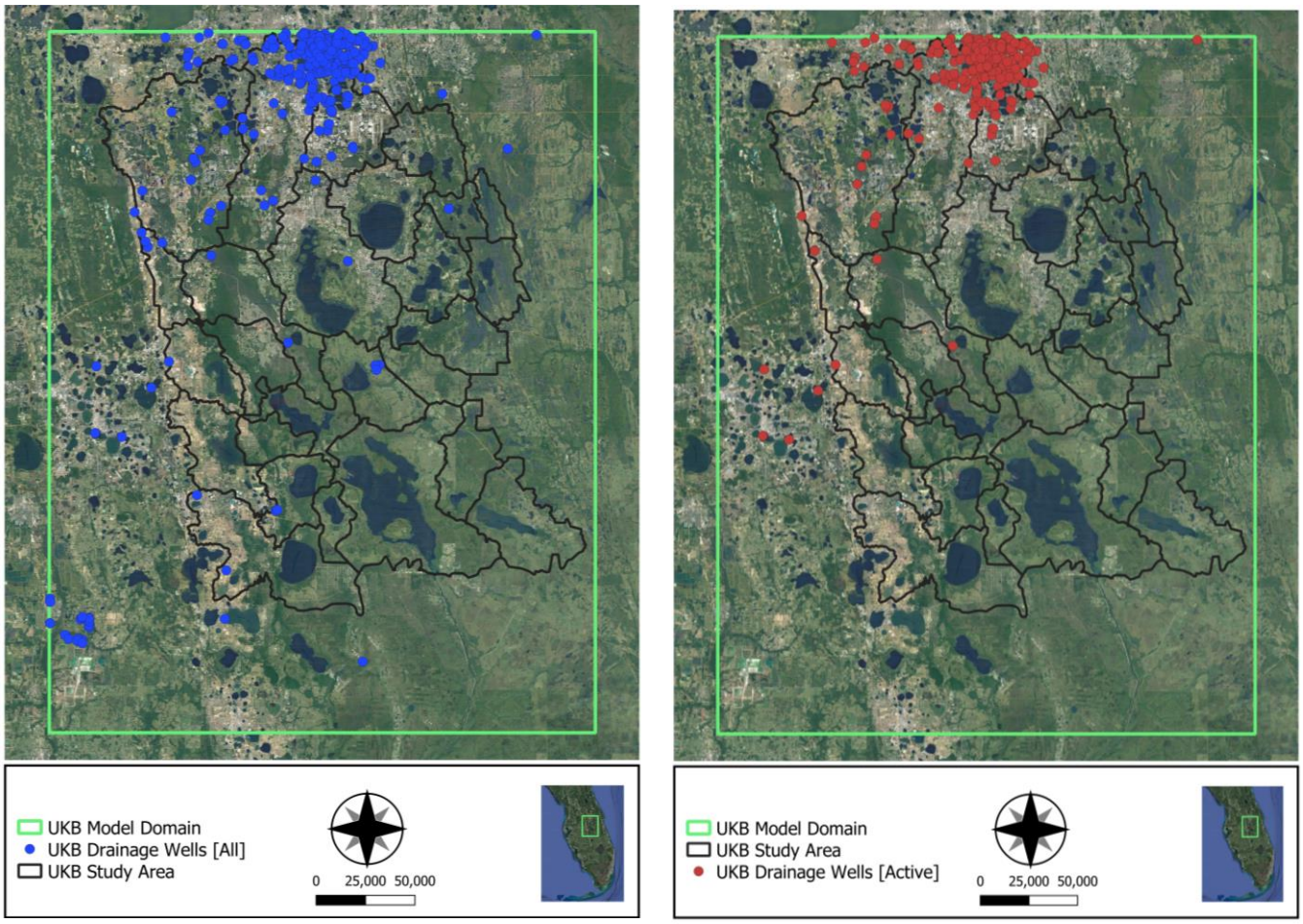

Figure 5-21: Location of Stormwater Drainage Wells in the Study Area

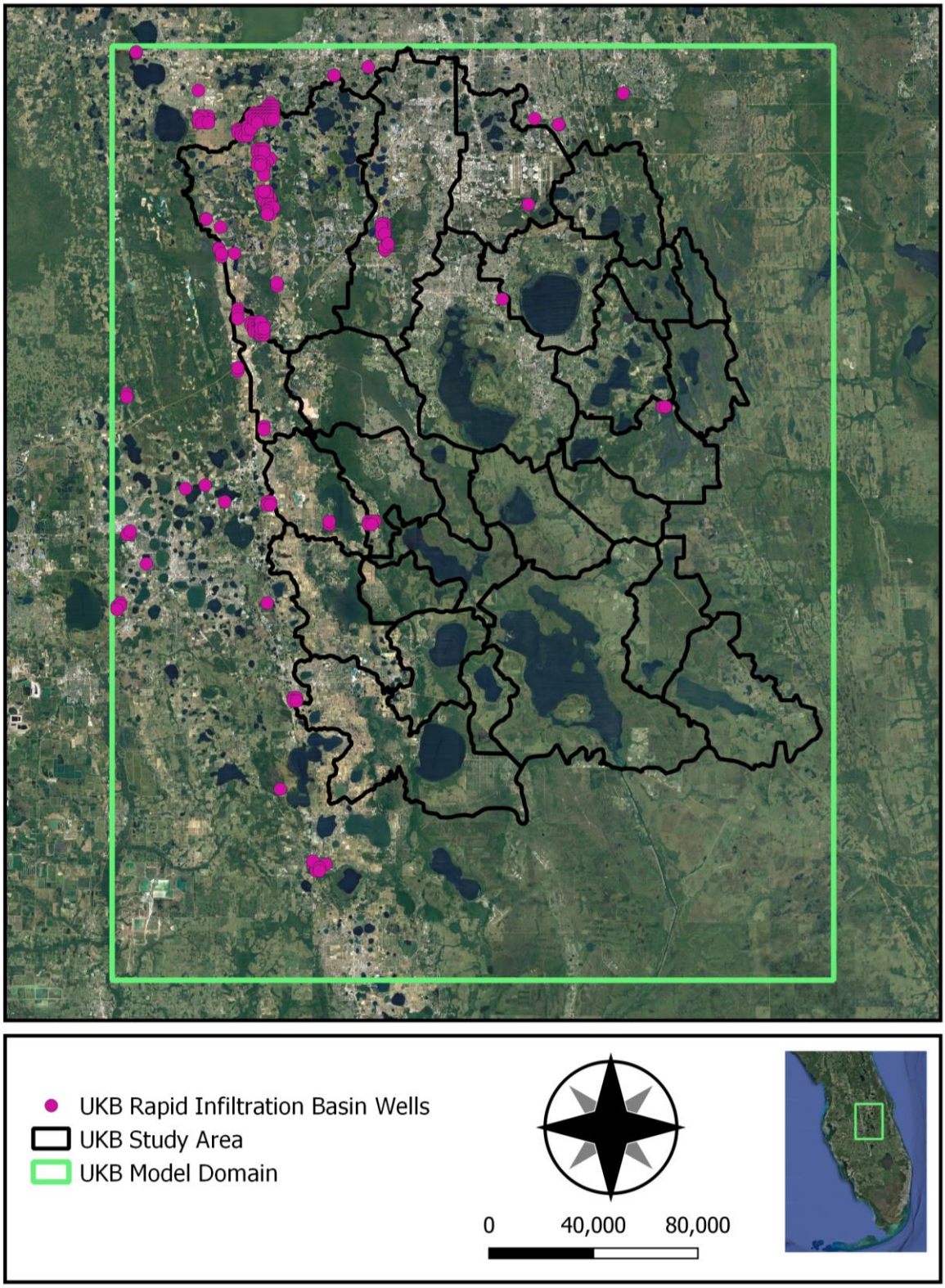

Figure 5-22. Rapid Infiltration Basin (RIBs) in the UKB FPLOS Model

5.1.8.5 Pumping Wells

The withdrawal database assembled for UKB FPLOS Model was extracted from the CFWI database of water use permits. There are generally six classes used in water supply planning in South Florida: Public Supply (PS), Agriculture (AG), Landscape/Recreational (L/R), Power Generation (PG), Commercial/ Industrial/Institutional (CII) and Domestic Self-Supply (DSS). The UKB FPLOS Model includes the PS wells, the largest use category. **Figure 5-23 and Figure 5-24** show the distribution of the water use facilities in model layers 1 and 2. UFA-upper withdrawals were not simulated in the calibrated model.

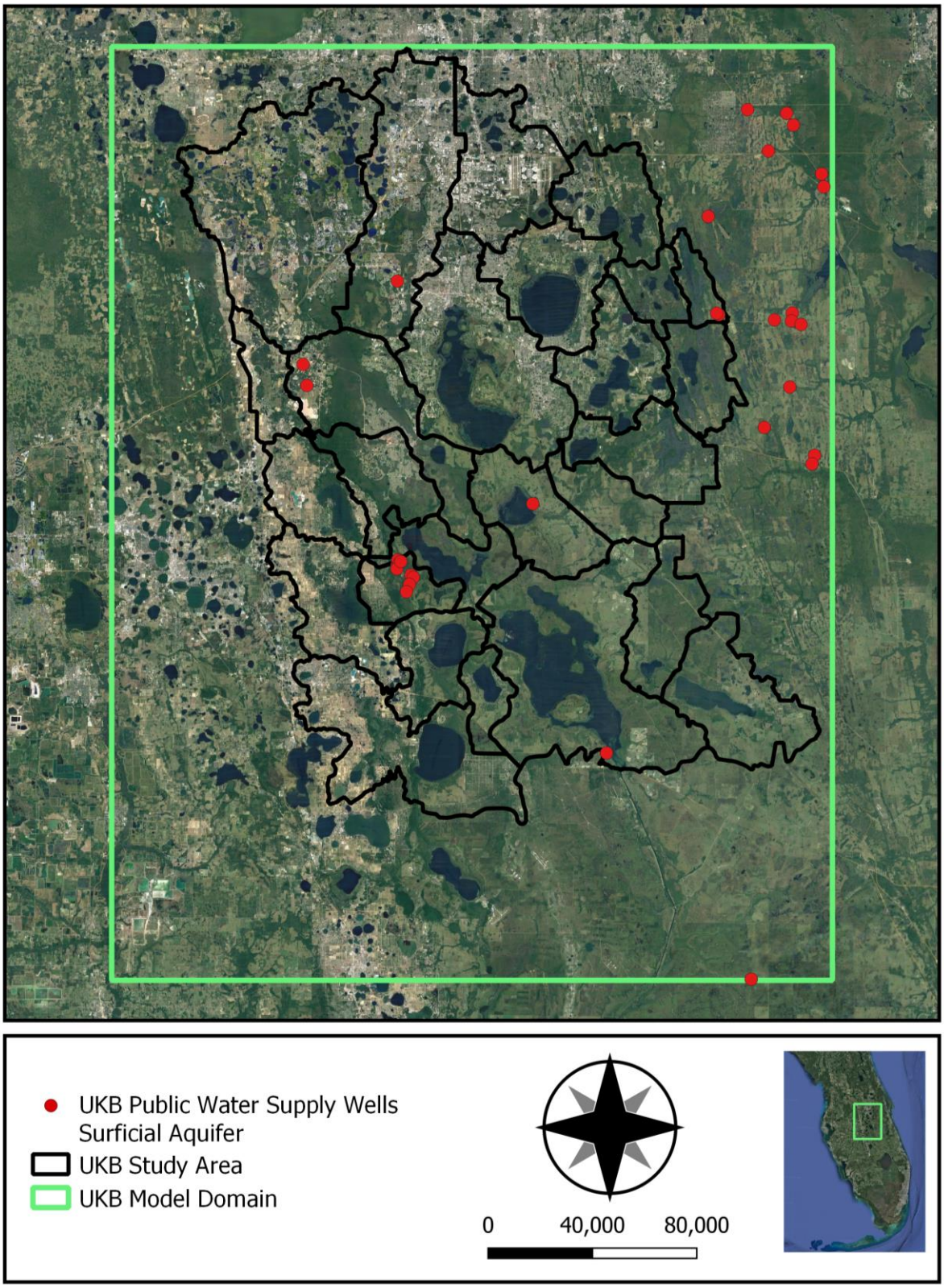

Figure 5-23: Location of PWS wells in SA (UKB FPLOS Model Layer 1)

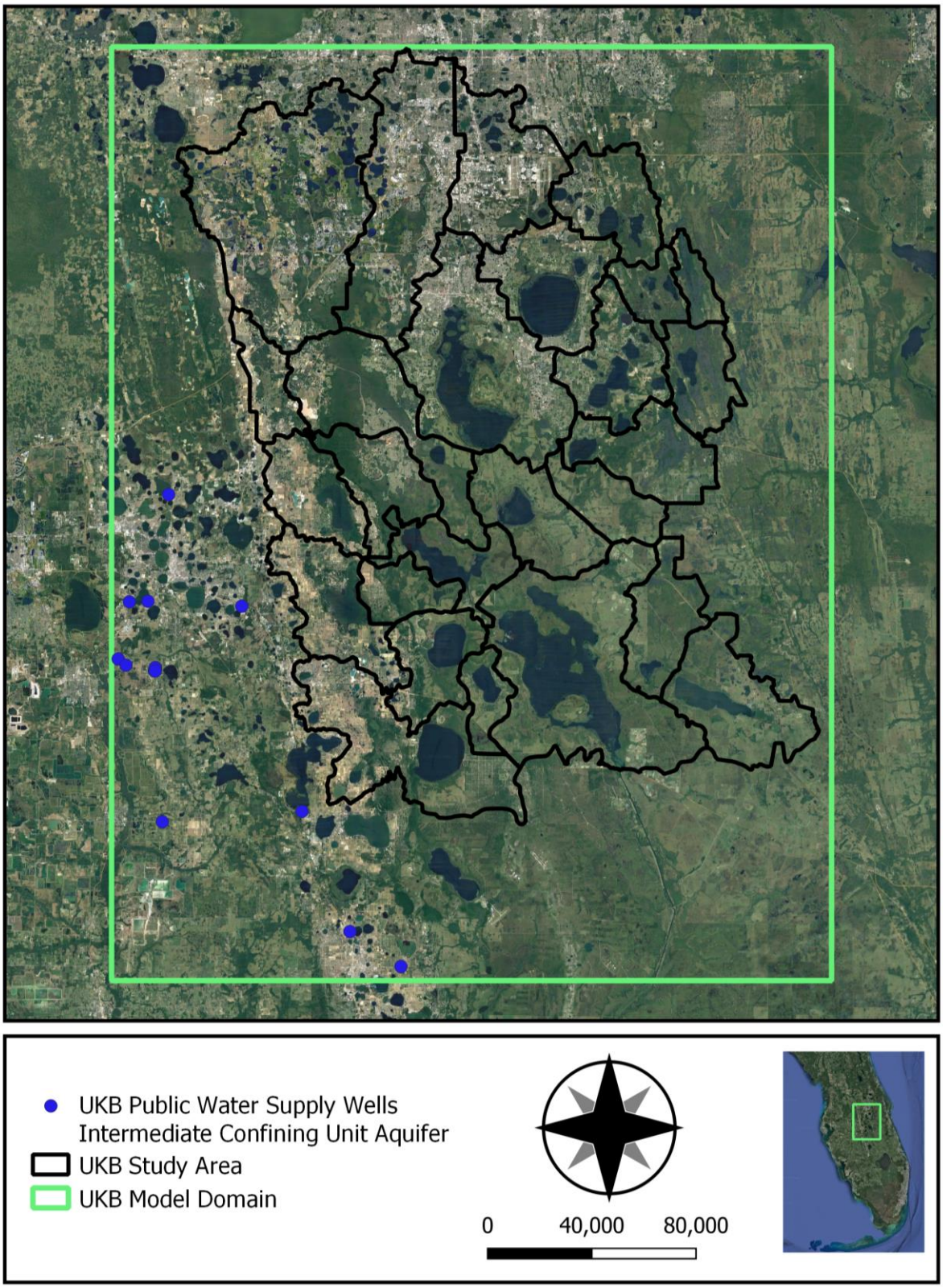

Figure 5-24: Location of PWS wells in ICU/IAS (UKB FPLOS Model Layer 2)

5.1.8.6 Initial and Outer Boundary Conditions

Daily groundwater stage data was obtained from the three districts (SFWMD, SWFWMD, and SJRWMD) for both calibration and validation events, for both the SA and UFA. Time varying dfs2 files were generated from the observed daily groundwater stages. **Figure 5-25** shows the SA and UFA stations used to generate GW initial and boundary conditions datasets. **Figure 5-26 through Figure 5-29** show the GW initial conditions for calibration and validation events.

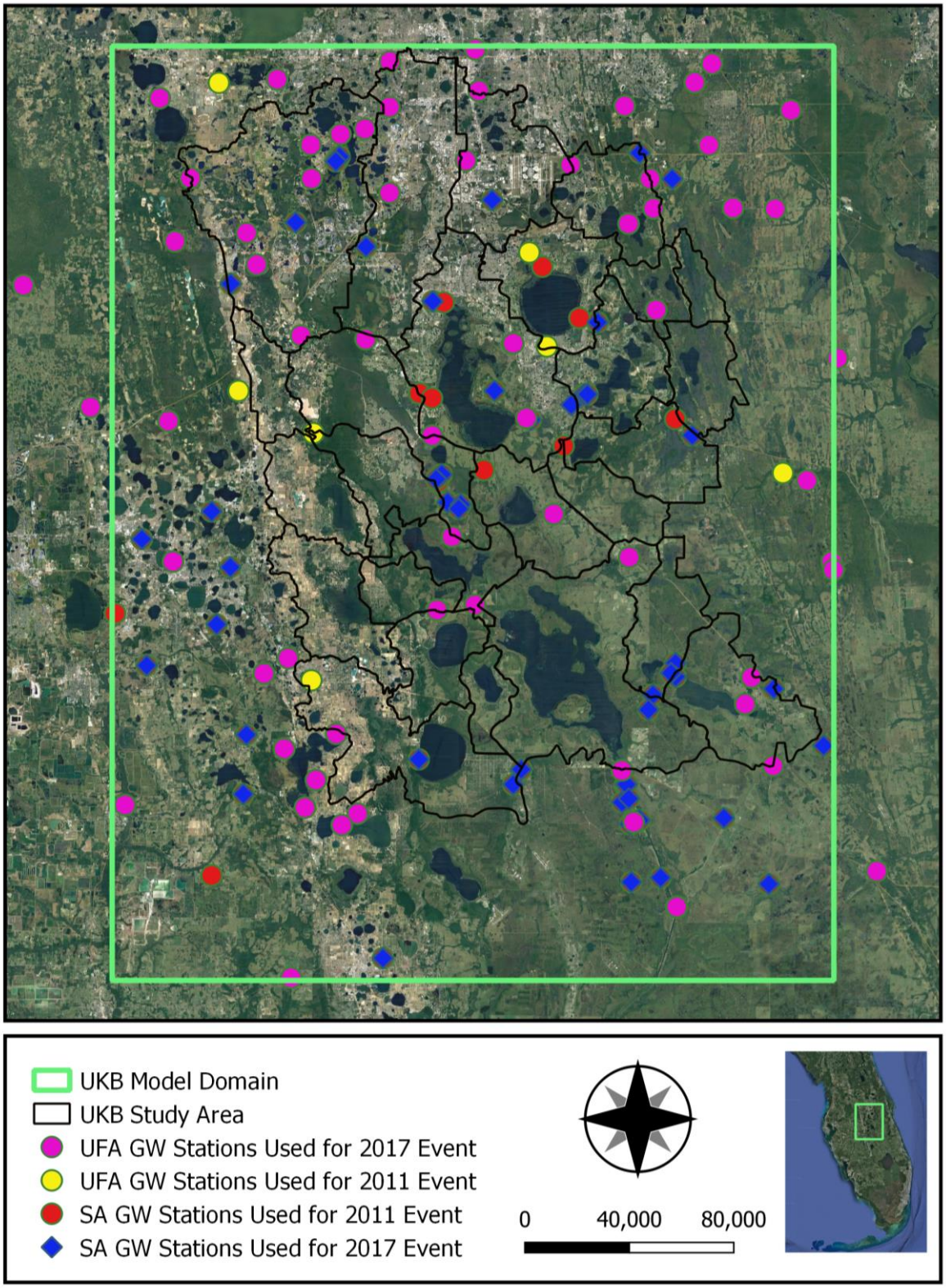

Figure 5-25: Groundwater Stations used for Initial and Boundary Conditions in UKB FPLOS Model

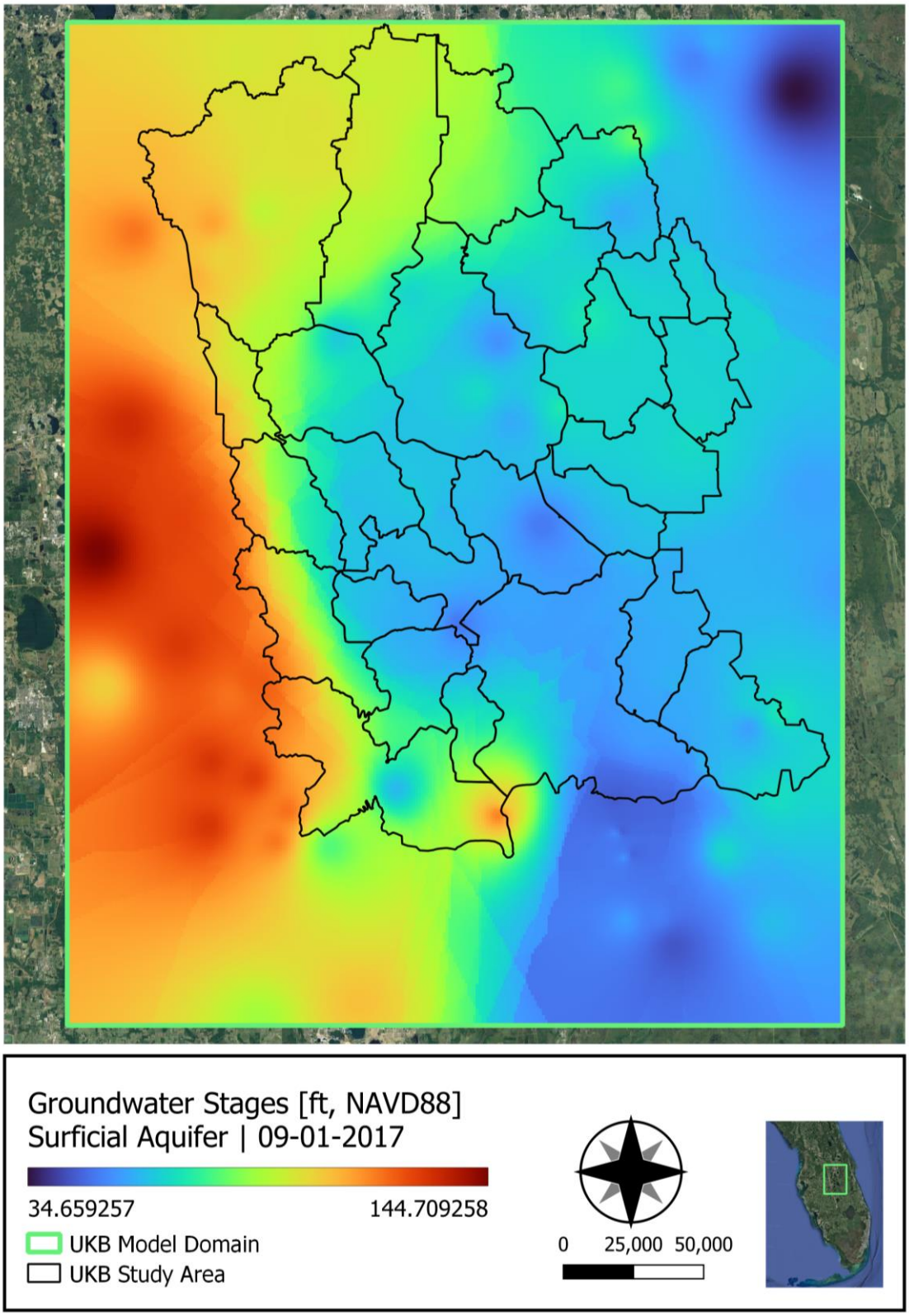

Figure 5-26: Initial Conditions in SA (UKB FPLOS Model Layer 1) for Calibration Event

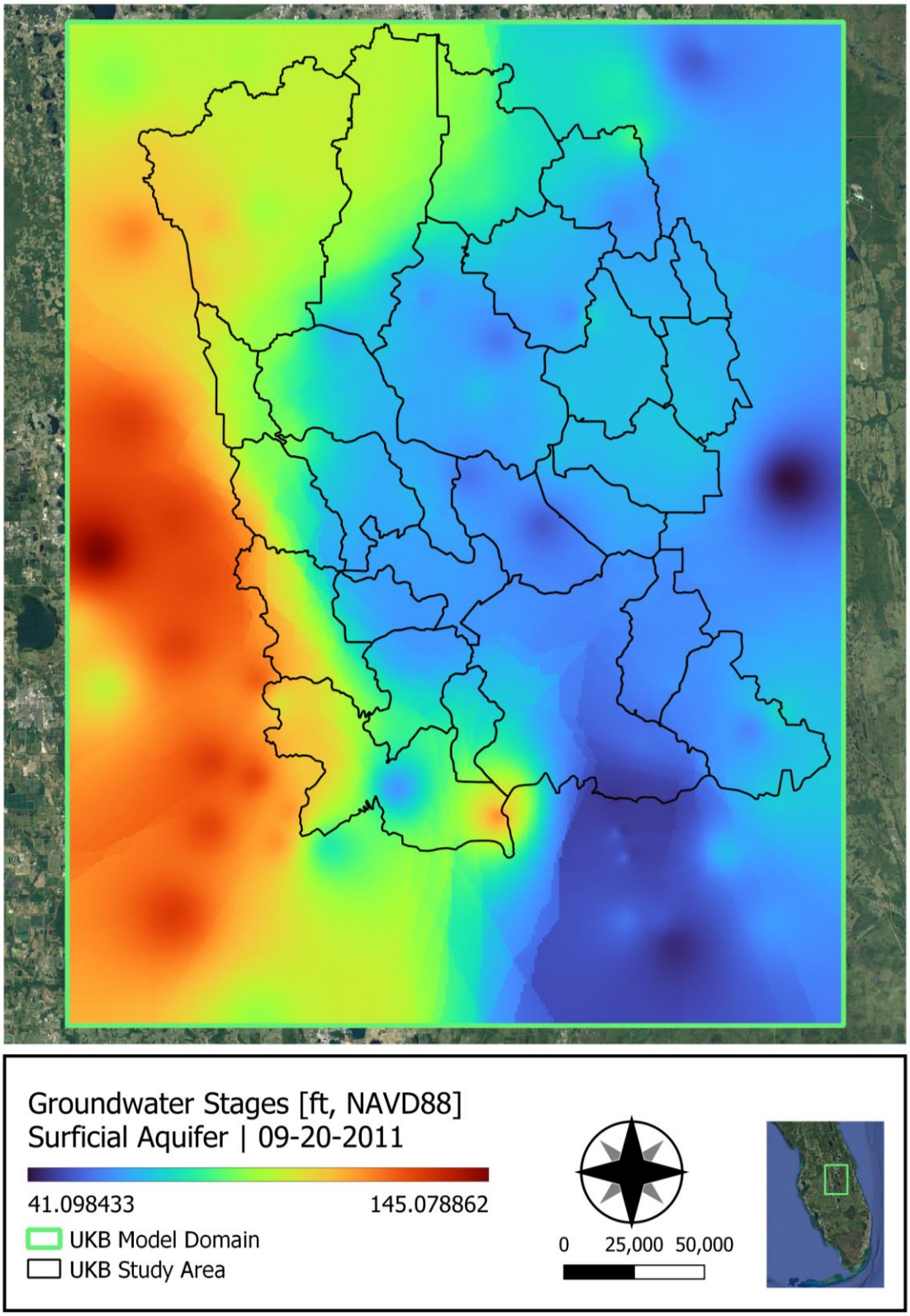

Figure 5-27: Initial Conditions in SA (UKB FPLOS Model Layer 1) for Validation Event

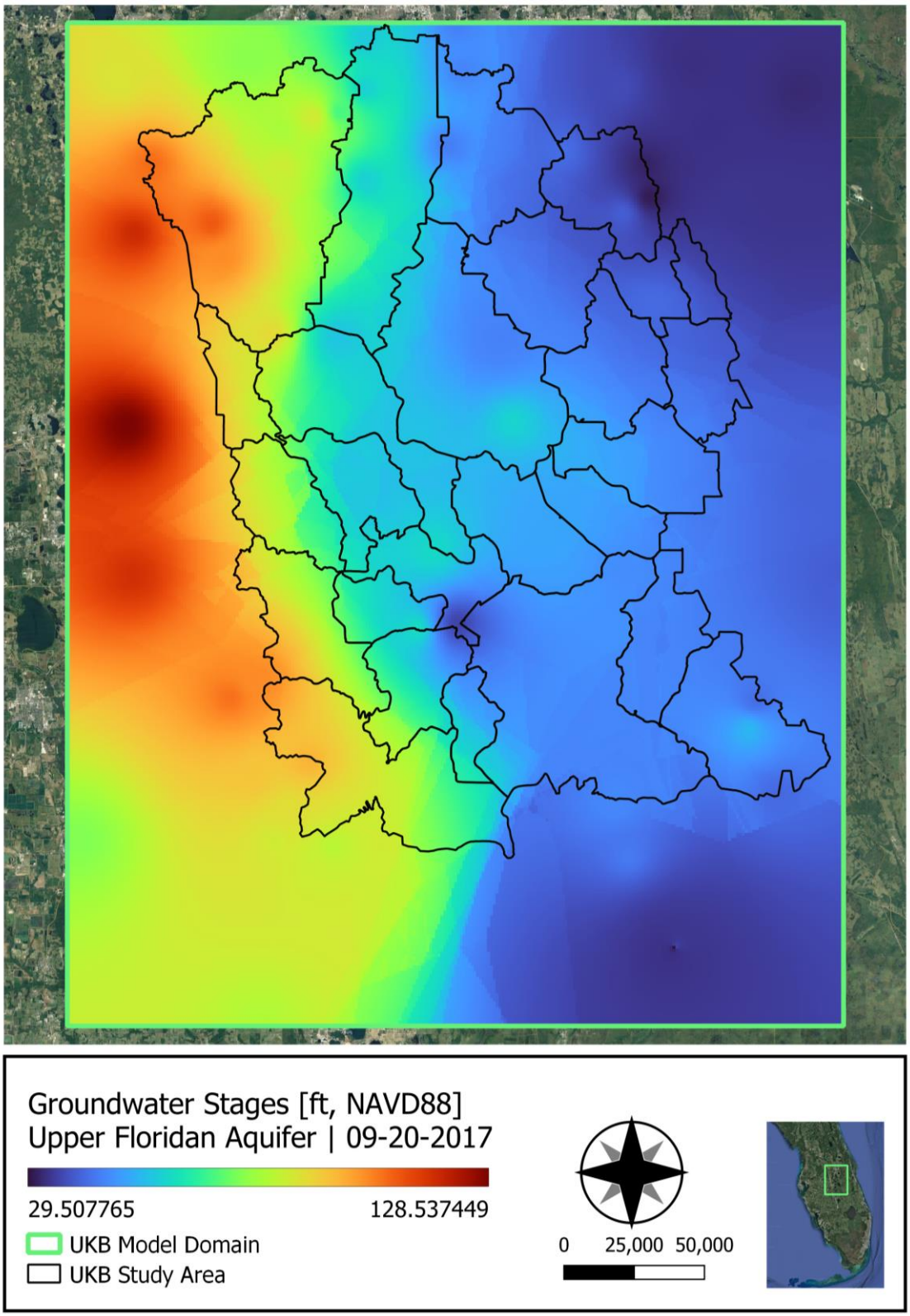

Figure 5-28: Initial Conditions in UFA-Upper (UKB FPLOS Model Layer 3) for Calibration Event

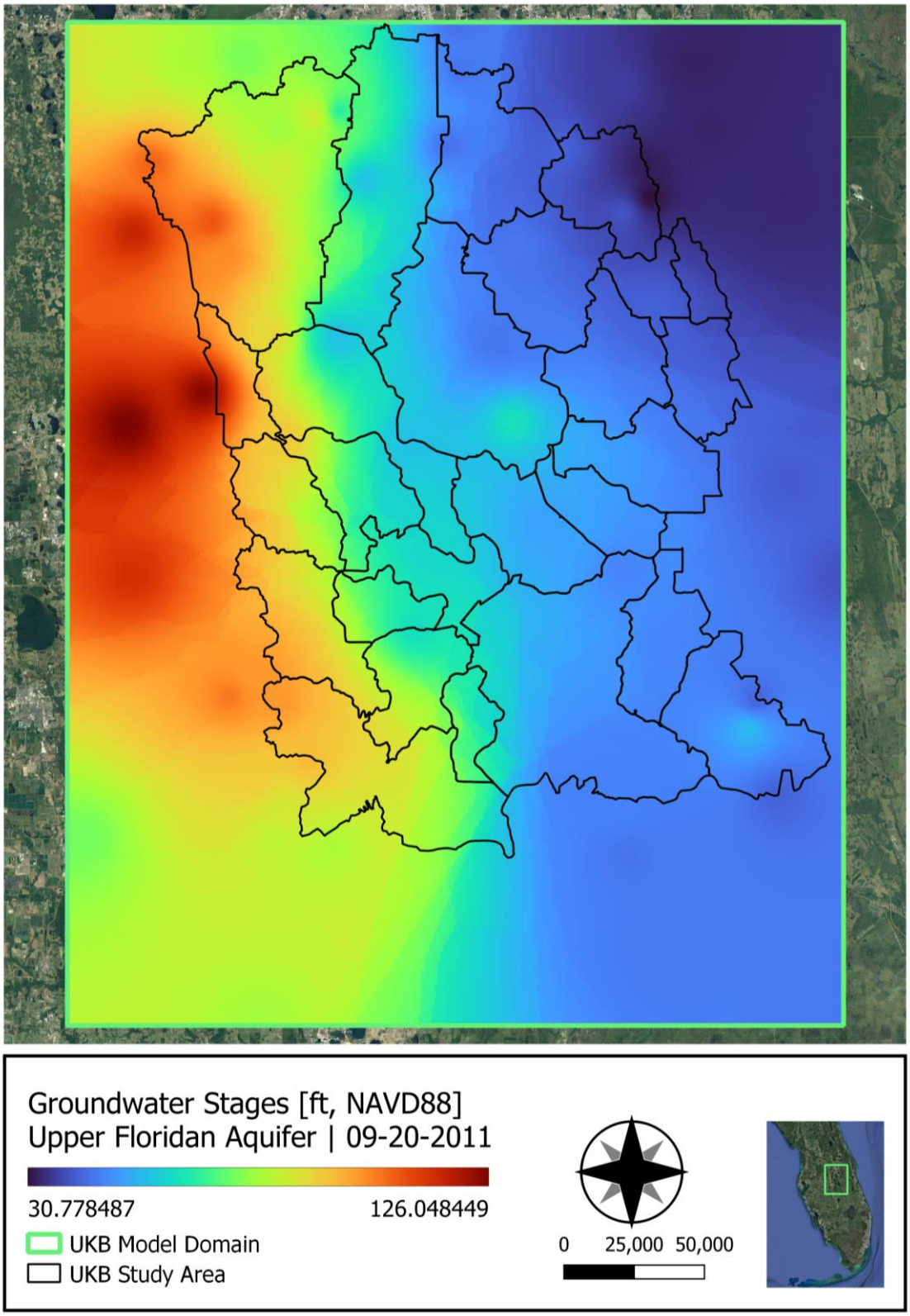

Figure 5-29: Initial Conditions in UFA-Upper (UKB FPLOS Model Layer 3) for Validation Event

5.2 MIKE HYDRO Development (Rivers and Lakes)

MIKE HYDRO is the interface to MIKE 1D, a dynamic, 1-Dimensional modelling package for modeling river channel networks, lakes and reservoirs. The Open Channel (OC) module in the UKB FPLOS Model represents channels (canals, rivers, streams, ditches, etc.), water control structures (bridges, culverts, pipes, weirs, orifices, pumps, gated structures) and overland and groundwater coupling. The SFWMD canals and the secondary and some tertiary surface water network in the UKB FPLOS Model are typically represented as 1D hydrodynamic features, which are also coupled with the 2D features (overland flow, subsurface flow) to allow exchange between all model components: SZ, OL, OC and UZ.

The MIKE HYDRO component of UKB FPLOS Model is summarized in **Table 5-9**.

Table 5-9: Summary of UKB FPLOS Model MIKE HYDRO Components

MIKE HYDRO Component	Count in UKB FPLOS Model
Branches	177
Weirs	30
Culverts	33
Bridges	40
Pumps	0
Gates	17
Direct Discharges	13
Sensors	68

5.2.1 River Networks

The UKB FPLOS MIKE HYDRO model's surface water network extends across 3 WMDs: SFWMD, SWFWMD and SJRWMD. Lakes and water bodies are implemented as MIKE HYDRO main widened channels. Canal dimensions, such as the lengths and cross-sectional widths and depths are used in MIKE HYDRO, to define the network geometry. This information was initially based on the existing models, Stormwater Atlas, the WMD databases, SWFWMD HEC-RAS models, the DEM and other sources. However, during model calibration, the need for more accurate cross-sections was evident in the simulation of the system hydraulics. New network branches were added and 3,200+ cross-sections were revised based on DEM with 2.5 ft resolution, GIS and imagery. The new cross-sections in the model have a topo ID of LIDAR_FPLOS, LIDAR or MODEL-REV to indicate the source of the data. The UKB FPLOS model was completely revised relative to the developed model or previous SFWMD Kissimmee MIKE models. The limits of each canal reach were generally strategically located at water control structures, canal intersections,

or where the geometry of the canal changes significantly. Structure representations in UKB FPLOS Model are shown in **Figure 5-30**. It should be noted that S-65A is outside the study area, hence not shown in Figure 6-1.

The area to the east, across the jurisdictional boundaries of SFWMD and SJRWMD has not been modeled in previous SFWMD models. The MIKE HYDRO network was added in this area using imagery, other models, site visit after Hurricane Ian or observed documentation. **Figure 5-31** shows the observation locations after Hurricane Ian some of the MIKE network was based on.

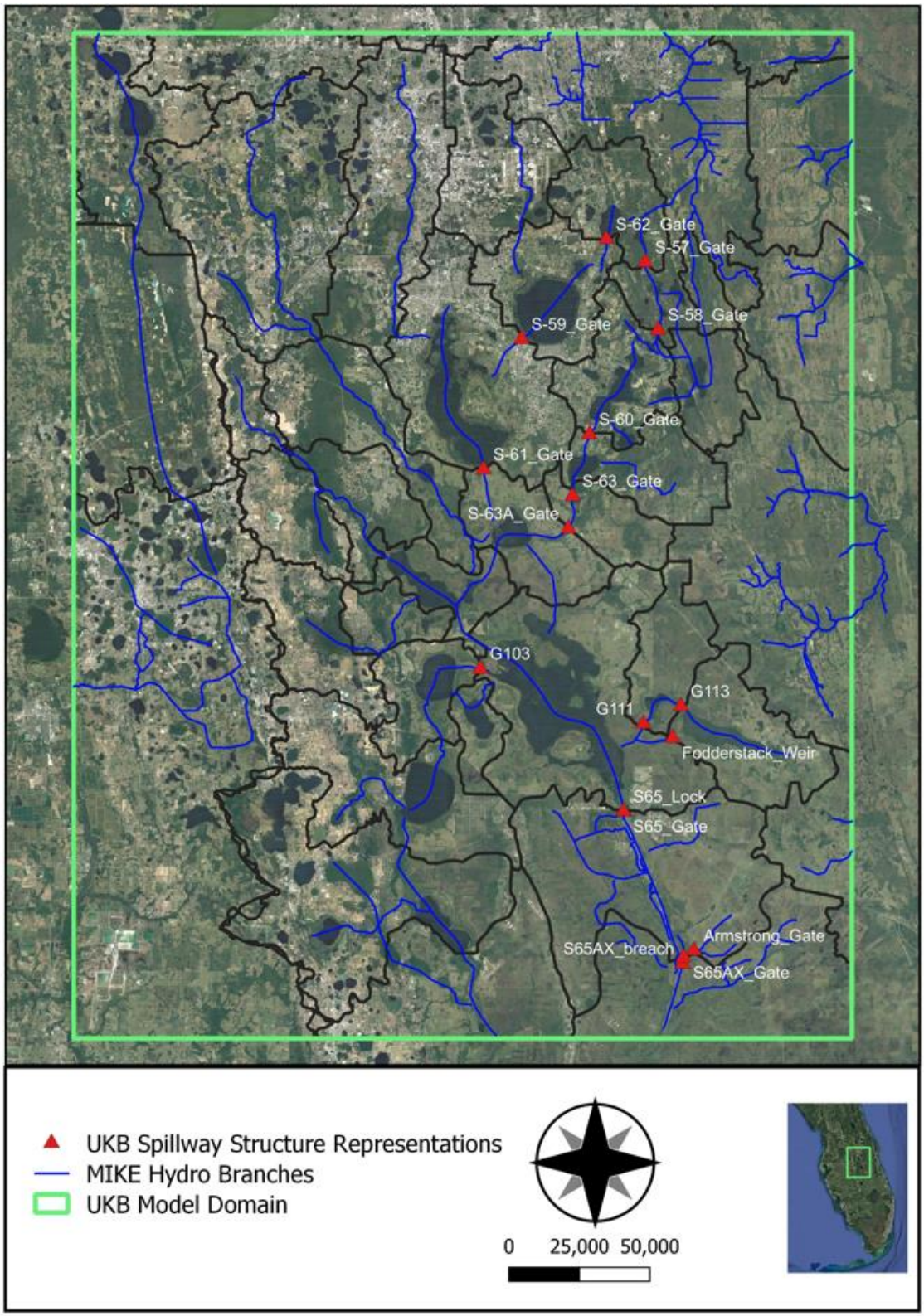

Figure 5-30: UKB FPLOS Model MIKE HYDRO Branches and Spillways

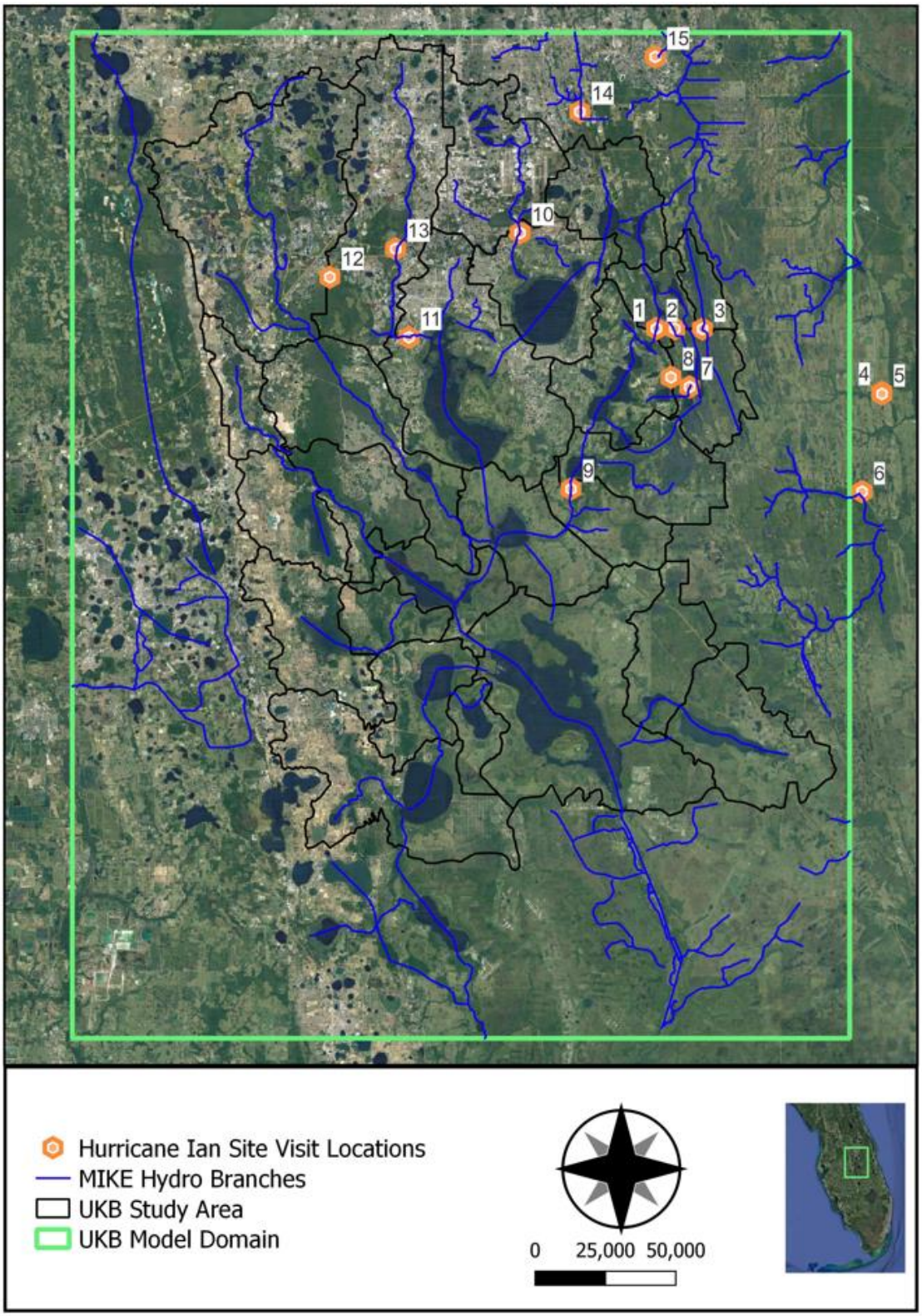

Figure 5-31: Hurricane Ian Site Visit Locations

Figure 5-32 shows the hydrology and locations of culverts in the eastern chain of lakes that were added to the model, based on observations during a helicopter flyover over the KB by SFWMD Emergency Modeling Team (EMT) and UKBFPLOS modeling team.

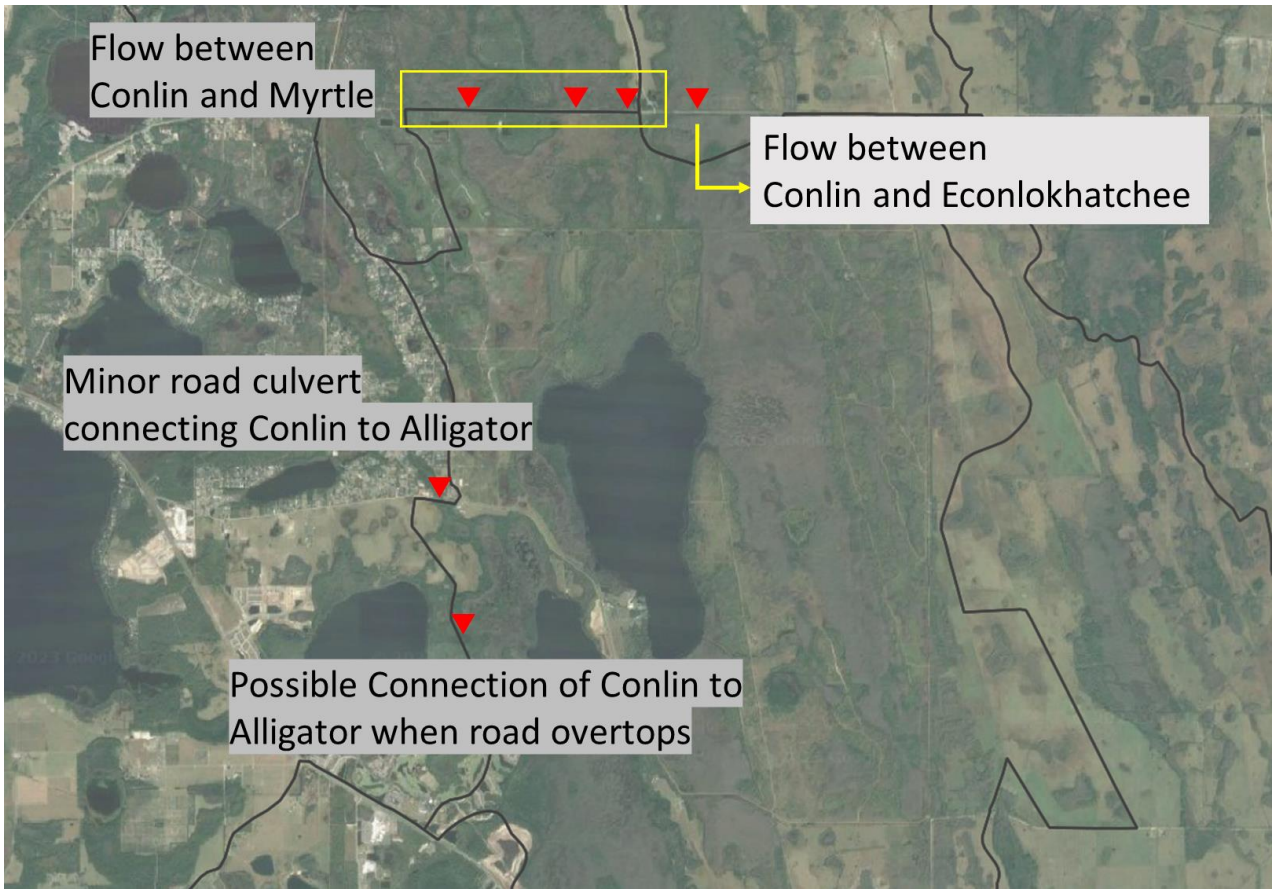

Figure 5-32: Hydrology and Culvert Connections in the Eastern Chain of Lakes

5.2.1.1 Cross Sections

Cross sections are defined by station/elevation pairs measured normal to the direction of flow; and stationing is from left to right, looking downstream. The cross-sections in MIKE require user-specified markers to establish the edges of the 1-D channels to be used in computations and the low flow and high flow regions within the channel. Left and right low flow bank markers establish the x-coordinate positions. Cross sections in the UKB FPLOS model were from different sources: surveys, existing models, cut from the DEM or marine nautical charts showing bathymetry, SFWMD published lake depths (SFWMD, 2021). They were manually checked for correctness and agreement with the topography. Maps of the cross-section locations are shown in **Figure 5-33**.

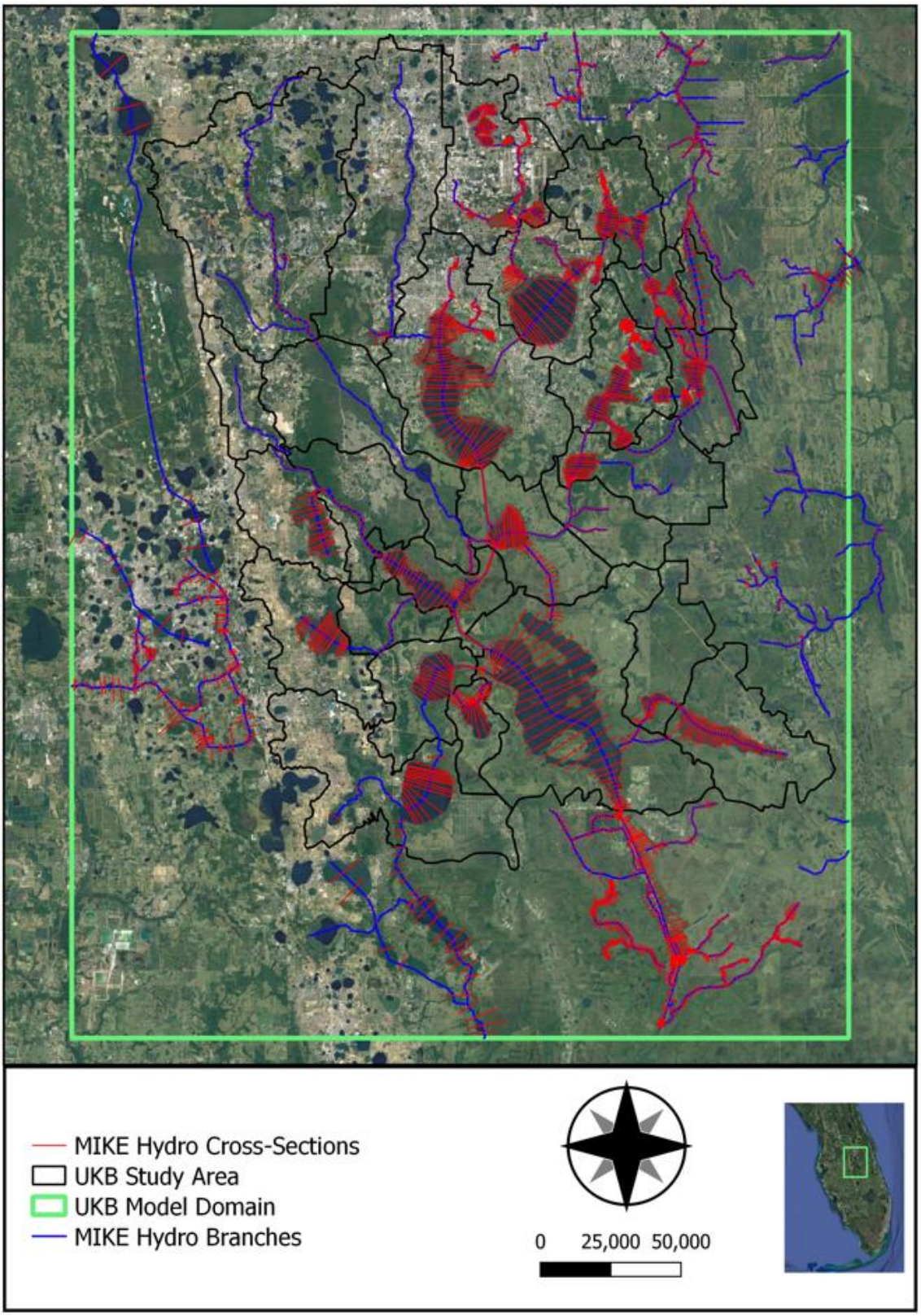

Figure 5-33: UKB FPLOS Model with Locations of Cross-Sections

Two examples of the marine charts used to generate cross-sections are shown in **Figure 5-34 and Figure 5-35**; they are for Lake Conlin and Lake Louisa respectively. All these datasets were the best available information during model construction and calibration; and were included in MIKE HYDRO as cross sections.

Lakes in UKCOL do not have uniform depths and the stages vary throughout the lake. The structures at the upstream and downstream also induce more of a 1-dimensional flow than 2-dimensional flows. Therefore, the lakes are modeled as wide cross-sections.

Bathymetry Data 2023 Collection

SFWMD collected bathymetry data using acoustic doppler profiler in several lakes upstream and downstream of the structures to assess the scouring at the lakes. This work effort compared the data with the previous bathymetry data collected. No noticeable scouring was found at S65, S62, and S57, but at S65DX1, S63A, S59, S354, S60, and S65D there are bathymetric changes indicating scouring (SFWMD, 2023).

Figure 5-34: Lake Conlin Nautical Chart

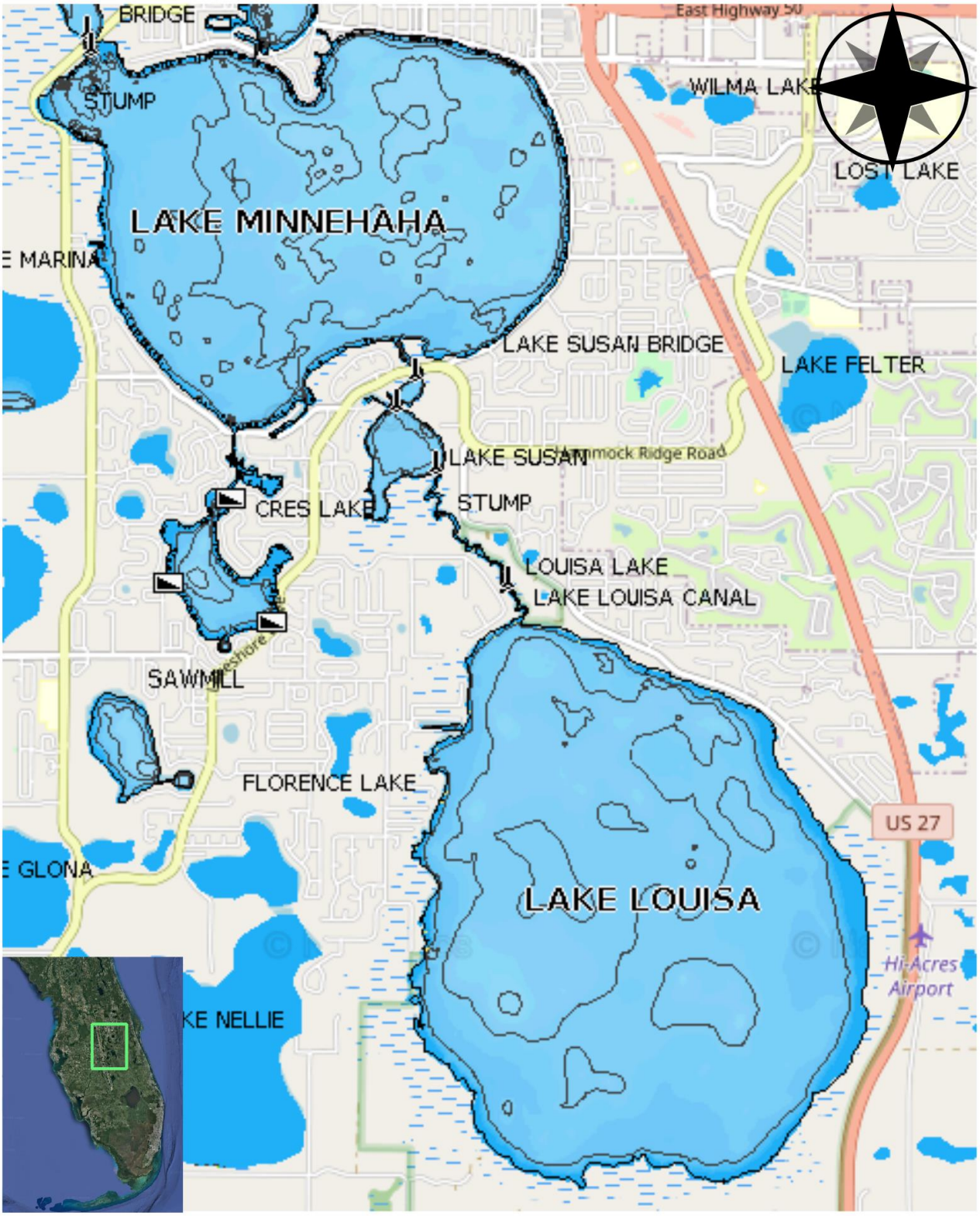

Figure 5-35: Lake Louisa Nautical Chart

5.2.2 Structures

The MIKE suite of software uses MIKE 1D, a hydrodynamic engine for modeling of surface water flow which provides a broad selection of hydraulic structures capable of being operated by defining complex control strategies.

SFWMD observed flow data at spillways are not measured but computed. In MIKE HYDRO, the spillways are simulated using the sluice formula coded by DHI in the early 2000s based on SFWMD developed flow equations. The sluice formula uses parameters for the four flow regimes: controlled submerged, uncontrolled submerged, controlled free, and uncontrolled free flow (SFWMD, 2015; Ansar and Chen 2009). The parameters for the flow have been established by the SFWMD for each individual SFWMD structure using flow rate studies. The UKB FPLOS Model uses parameters from appropriate flow rate studies provided by the SFWMD. DHI coded the SFWMD spillway equations in MIKE (Circa 2005); but SFWMD has improved the equations and DHI has not updated the equations. Differences in flows produced by MIKE and observed data may result from this.

The weirs, culverts and bridges use the energy equations; the bridges use the Federal Highway Administration WSPRO method for submergence and overtopping.

For all the structures, time series of gate openings, variable weirs and boards were downloaded from DBHYDRO and converted to their corresponding levels (gate levels in NAVD88) to make the MIKE HYDRO input datasets.

5.2.3 Control Rules

Control rules in MIKE HYDRO define the structure operations. Actions are defined in the control rules for each operable structure and at each time step, MIKE evaluates the rules for possible action. In MIKE HYDRO, the actions performed are based on the instantaneous values from the sensors. There are 53 sensors defined in the UKB FPLOS Model. The addition of sensors allows the portability of files and adds flexibility for the sensors to be used for different actions. Control rules that refer to time series as a parameter were added as an external sensor, in the UKB FPLOS Model.

In general, the UKB FPLOS Model gate operations have been coded to consider the following:

- Observed data to drive calibration and validation
    - Drivers are gate openings
- Levels vs Rules and the height of the water level in the MIKE HYDRO branch controlled by the structure
    - Levels vs rules can also include if the structure is in operation or offline due to construction, breaches etc.
- Structure design conditions
    - Maximum Allowable Gate Openings (MAGO)
    - Maximum Permissible Head Differential (MPHD)

- Is the SFWMD operating the structure during emergency(ies) (storm events)
- Wet and Dry season gate level operations

The operating rules in UKB FPLOS calibration and validation models, and design events are detailed in **Appendix K**.

5.2.4 Hydrodynamic Parameters

The fully dynamic St. Venant flow equation includes variables that are calculated during the simulation by MIKE and user defined. One of the user defined parameters in MIKE HYDRO is bed resistance which can be entered as a single global value used for the entire network or as spatially varying values. The UKB FPLOS Model has locally specified, spatially varying values.

5.2.4.1 Bed Resistance

The Manning's M values are used to define the resistance to flow in the channel. Lower n values (Higher M values) indicate smoother, typically lined canals, and higher n values (lower M values) represent natural channels or canals with vegetation growth, debris, and/or siltation.

In MIKE HYDRO, the flow area between the banks uses the main channel Manning's roughness value specified; the left bank and right bank Manning's roughness value for the overbank floodplain areas define areas outside of the main channel. The UKB FPLOS Model uses different values for the left and right banks, and for the main channel.

In general, the embankment roughness values are higher than in the main channel, as embankments typically have vegetation. In some cases, such as when the floodplain includes an access road or paved area, the Manning's roughness value on the embankment is lower than in the main channel. The Manning's n values for main channels/canals are set at 0.03, natural creeks (Shingle, Reedy, Boggy, Rosalie and others) are set at 0.3 and most embankments are set at 0.09 – 0.2.

5.2.5 Initial and Boundary Conditions

Time series files (dfs0) were used for generation of MIKE HYDRO initial and boundary conditions. These were created from QA/QC ed historical, measured data downloaded from different sources: SFWMD DBHYDRO, SJRWMD database, SWFWMD and United States Geological Survey (USGS) and county databases of hydrology data. The stages and flows were downloaded in the shortest time frequency available: as breakpoint, hourly or daily data.

Values for initial and boundary conditions were from observed data – for both surface and groundwater (MIKE HYDRO and MIKE SHE). The only open boundary condition used in MIKE HYDRO is a water level boundary, downstream of S65 structure. The rest of the boundary conditions are closed. For saturated zone, the western and northern boundaries have fixed head boundaries based on observed data; this is to capture the flows into the model domain. The eastern and southern boundaries of UKB FPLOS model use zero-flux boundary conditions.

5.2.6 MIKE SHE Coupling with MIKE HYDRO

The coupling between MIKE HYDRO and MIKE SHE is made via MIKE SHE links, which are located on the edges that separate adjacent grid cells. The entire 1D network is included in MIKE HYDRO and conveys water, but MIKE SHE will only exchange water at specified coupling reaches. The location of each MIKE SHE river link is determined from the coordinates of the MIKE Hydro River points.

In the UKB FPLOS Model, the coupling is done using the 'flood code' option. Flood codes are used to simulate flooding from canals to the adjacent flood plains and wetlands and to simulate lakes and reservoirs. Flood coded cells allow MIKE HYDRO to control the water levels in the flooded cells in MIKE SHE when the water in the 1D channel exceeds the ground topography of the MIKE SHE cells. When this happens, the water levels in the MIKE SHE flooded cells are set to the water levels in the adjacent/corresponding flood coded MIKE HYDRO branch to determine if ponded water exists on the MIKE SHE cells. If ponded water exists, then MIKE calculates unsaturated or saturated exchange flows based on the ponded water level above the cell. The flooded cells can also interact with other processes in MIKE SHE, such as infiltration, ET, and overland flow to non-flooded cells.

The spatial distribution of the flood coded cells is shown in **Figure 5-36**.

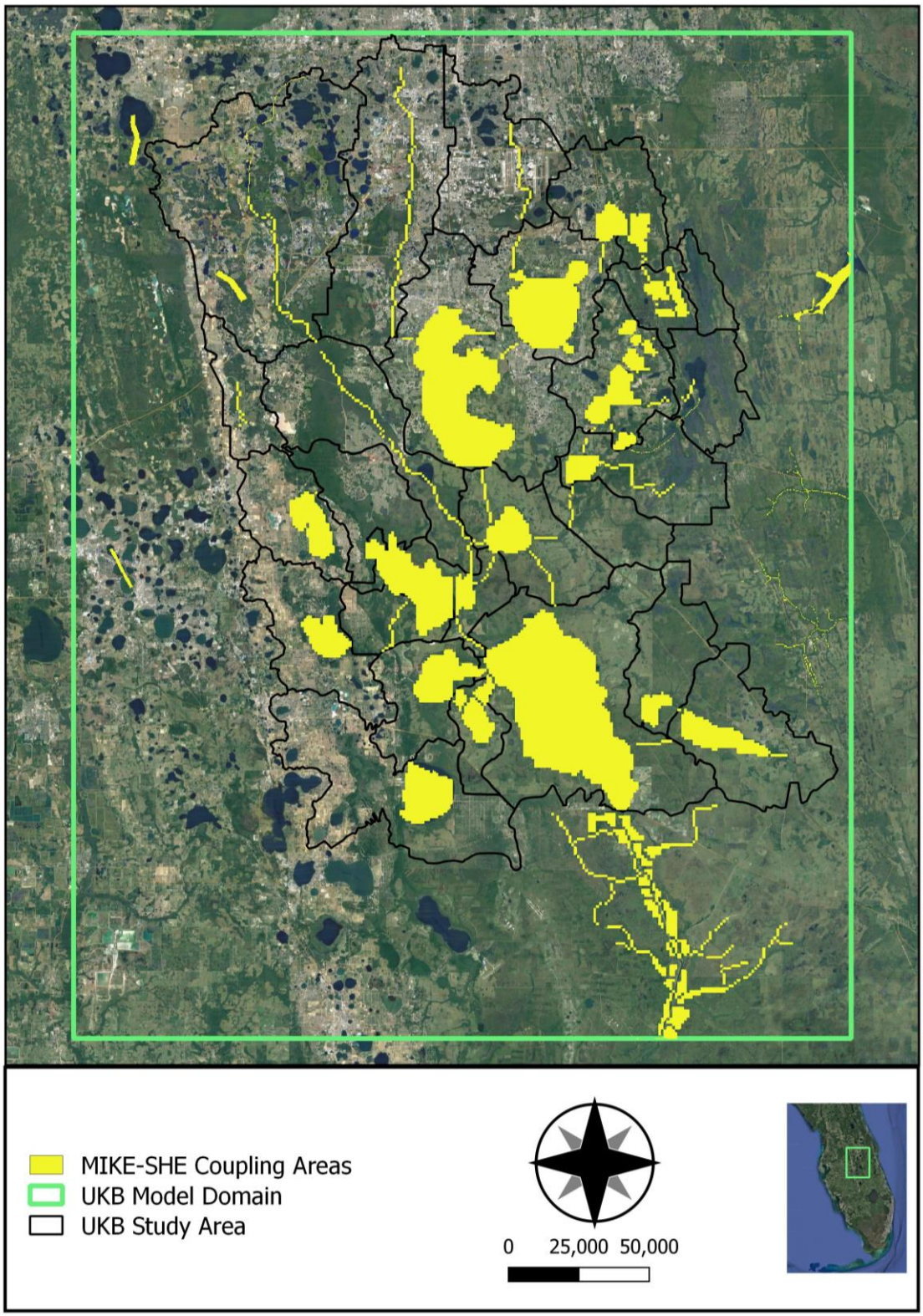

Figure 5-36: Flood Coded Cells in UKB FPLOS Model

5.3 Calibration and Validation

Model calibration, in general terms, is the adjustment of model parameters to minimize the difference between simulated and observed values, as it pertains to the model objectives. Performance of the model is assessed in terms of goodness-of-fit metrics. A model is considered calibrated when the goodness-of-fit metrics are within acceptable ranges in literature; and model calibrated parameter values are within reasonable ranges for the hydrology and hydraulics of UKB.

The objectives of the calibration of UKB FPLOS, is a model that is capable of:

- Simulating selected historical extreme events on a regional scale.
- Analyzing SFWMD's Flood Protection Level of Service (FPLOS) metrics on the primary conveyance system.
- Providing analysis of computed stages and discharges for a set of design events (10, 25, 50, 100 year) and future rainfall.
- Adequately simulating the flooding depth and extent at selected locations, relative to the performance metrics.

Calibration of the UKB FPLOS model consisted of a systematic sensitivity analysis of the preliminary model parameters; the basis of the sensitivity analysis was used in adjusting model-selected parameters relevant to the performance metrics. Observed gate operations were used during both calibration and validation events.

5.3.1 Model Calibration and Validation Criteria

For calibration and validation models, and for each monitoring station with reliable observed data, a set of statistical measures of goodness-of-fit were calculated; **Table 5-10** lists them with an explanation and targets for each statistical measure. The targets were provided by SFWMD and have been used in the FPLOS program.

Table 5-10: Statistical Parameters for Calibration of UKB FPLOS Model

Statistic	Explanation	Target
Mean Error (ME)	Mean error indicates whether simulated values are over or underestimated when history matched	+/- 0.5 ft for SW and GW stages and 20% for SW flows
MAE (Mean Absolute Error)	Sum of the absolute difference of observed minus calculated data divided by the total number of data points	+/- 0.5 ft for SW and GW stages and 20% for SW flows
RMSE (Root Mean Square Error)	Root mean square of the observed minus computed, normalized	0.5 ft for SW and GW stages and 20% for SW flows
STD (Standard Deviation)	Standard deviation of the residual	0.5 ft for SW and GW stages and 20% for SW flows
R (Correlation)	Measure of the strength of a linear relationship between two variables.	Greater than 0.5, unitless
NS Nash-Sutcliffe	The Nash-Sutcliffe efficiency (NS) is a normalized statistic; it determines the relative magnitude of the residual variance in comparison with the measured data variance. NS indicated how well the plot of observed versus simulated data fits the 1:1 line.	Greater than 0.5, unitless

The calibration statistics calculated by MIKE are based on the differences between the observed/measured data and the simulated/calculated values at the same location and time. The error, or residual is calculated as Simulated data – Observed data pair is:

(13.1) $$E_{i,t} = Obs_{i,t} - Calc_{i,t}$$

where $E_{i,t}$ is the difference between the observed and simulated values at location i and time t.

1. Mean (ME)

   The mean error at location i where n observations are available is

$$ME_i = \overline{E}_i = \frac{\sum_t (E_{i,t})}{n}$$

2. Mean Absolute Error (MAE)

   The mean of the absolute errors at location i where n observations are available is

$$MAE_i = \overline{|E_i|} = \frac{\sum_t |E_{i,t}|}{n}$$

3. Root Mean Square Error (RMSE)

   The root mean square error at location i where n observations are available is

$$RMSE_i = \sqrt{\frac{\sum_t (E_{i,t})^2}{n}}$$

4. Standard Deviation of the Residuals (STDres)

   The standard deviation of the residuals at location i where n observations are available is

$$STDres_i = \sqrt{\frac{\sum_t ((E_{i,t}) - \overline{E}_i)^2}{n}}$$

   Standard deviation is a measure to assess how well the dynamics of observed data are simulated.

5. Correlation Coefficient (R)

   The correlation coefficient is a measure of the linear dependency between observed and simulated datasets. The closer the value is to 1.0, the higher the dependency. The correlation coefficient at location i is

$$r_i = \frac{\sum_t (Calc_{i,t} - \overline{Calc}_{i,t}) \cdot (Obs_{i,t} - \overline{Obs}_{i,t})}{\sqrt{\sum_t (Calc_{i,t} - \overline{Calc}_{i,t})^2 \cdot \sum_t (Obs_{i,t} - \overline{Obs}_{i,t})^2}}$$

   where $\overline{Obs_i}$ and $\overline{Calc_i}$ are the averages of the observations and simulations at location i respectively.

6. Nash Sutcliffe Coefficient (NS or R2)

   The Nash-Sutcliffe coefficient at location i where n observations are available is

$$R2 = 1 - \frac{\sum_t (E_{i,t})^2}{\sum_t (Obs_{i,t} - \overline{Obs_i})^2}$$

   where $\overline{Obs_i}$ is the average of the observations at location i. NS or R2 will be 1.0 if there is a perfect match.

7. Differences at peak
    - Peak Headwater Difference = Simulated Stage of structure headwater – Observed Stage of structure headwater
    - Peak Tailwater Difference = Simulated Stage of structure tailwater – Observed Stage of structure tailwater

5.3.2 Calibration and Validation Data Selection

The selected storm events for UKB FPLOS are:

- Storm Event for calibration: September 9-11, 2017 (Hurricane Irma)
- Storm Event for validation: October 7-9, 2011 (October No Name Storm)

The data needed for the UKB FPLOS model calibration and validation included the following:

- Surface water:
    - Breakpoint stages and flows
    - Gate openings (converted to gate levels for MIKE HYDRO)
- Groundwater:
    - Groundwater levels for all SAS and UFA wells within the study area
- Other:
    - Structure operations during the events
    - Flooding information available like high water marks and anecdotal information.

One-dimensional time series datasets were used for calibration and validation of surface water stages and flows. These datasets were created from historical, measured data from SFWMD database DBHYDRO, SJRWMD and USGS databases. All data were downloaded as breakpoint data when available, and daily or sub-daily data if breakpoint data were unavailable. The calculation of descriptive statistics requires matching pairs of observed and computed data; and MIKE generated statistics were used.

5.3.3 Classification of the calibration/validation stations

Calibration and validation data were divided into primary and secondary calibration/validation stations.

Primary stations are locations where calibration metrics are generated, model parameters were adjusted to meet calibration targets at these locations. Sensitivity Analysis and the collective understanding of the system shows these stations will directly impact on the five (5) UKB FPLOS performance metrics. These are locations within the 26 study area watersheds.

Stations are classified as secondary calibration data for one of a few reasons:

- Locations outside of the UKB FPLOS study area.
- Incomplete dataset.
- Data are not an accurate representation of SFWMD primary system; for example, a station that is location at the edge of a lake and represents the overland water depth rather than the lake levels.
- Stations which are not operated by SFWMD and therefore not QA/QC ed by SFWMD, as it relates to SFWMD primary system.
- Wells that are in the same aquifer but different depths, so have different recorded water levels. But in the UKB FPLOS regional scale model, they are in the same MIKE SHE cell and therefore cannot have different water levels. In this case, one of them is excluded from calculation of statistics.
- Data in areas with high topographic relief, but the model cell is averaged elevation and therefore the difference between the ground levels of observed data and modeled cell are higher than 5 ft.
- Surface water and groundwater stations, which are too close to the boundary and are impacted by the boundary data.

Figure 5-37 through **Figure 5-39** show the surface and ground water primary and secondary targets that were used for calibration and validation; **Table 5-11** shows the stations with the LMAs they are located in.

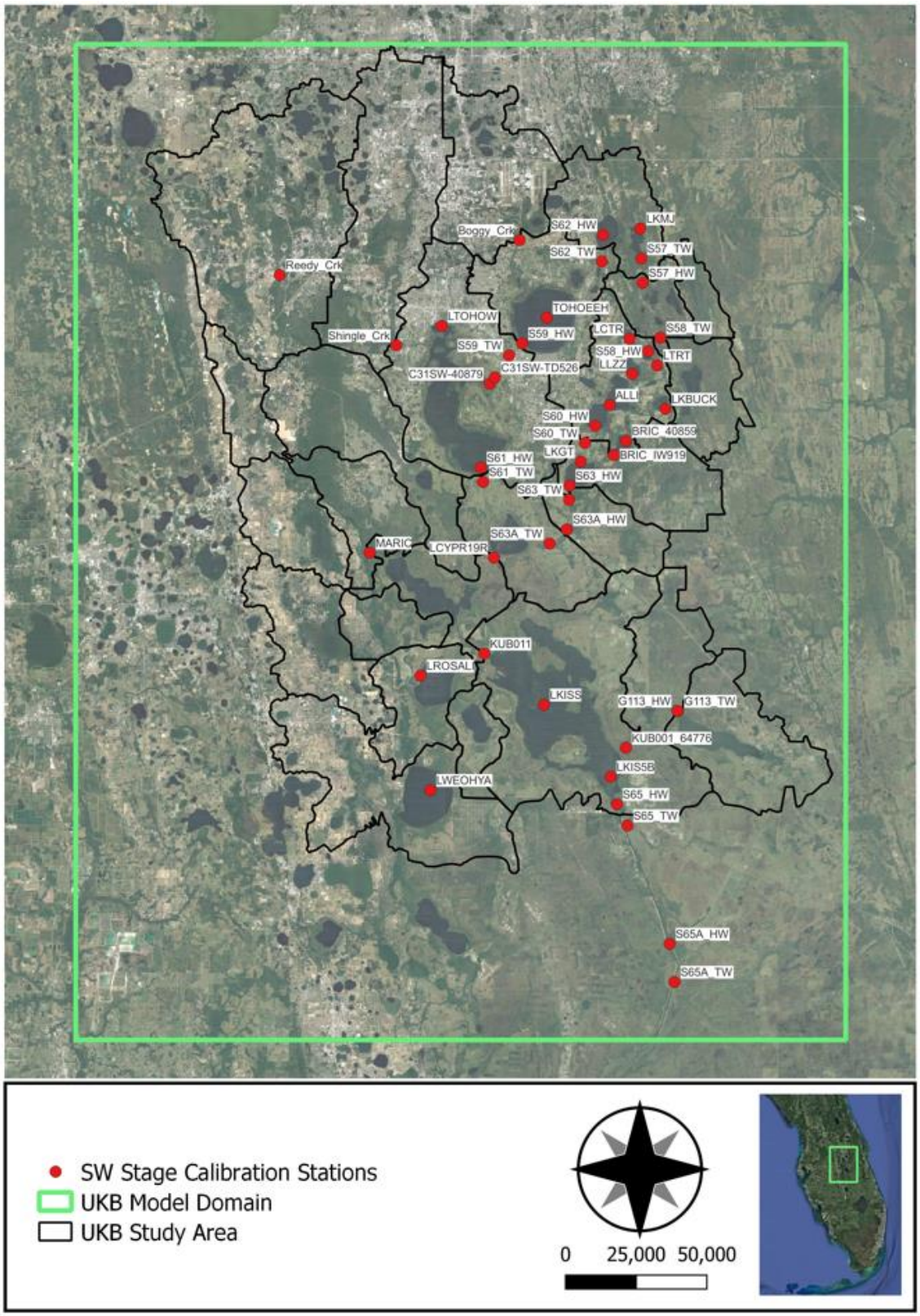

Figure 5-37: Surface Water Calibration and Validation Targets in UKB FPLOS Model

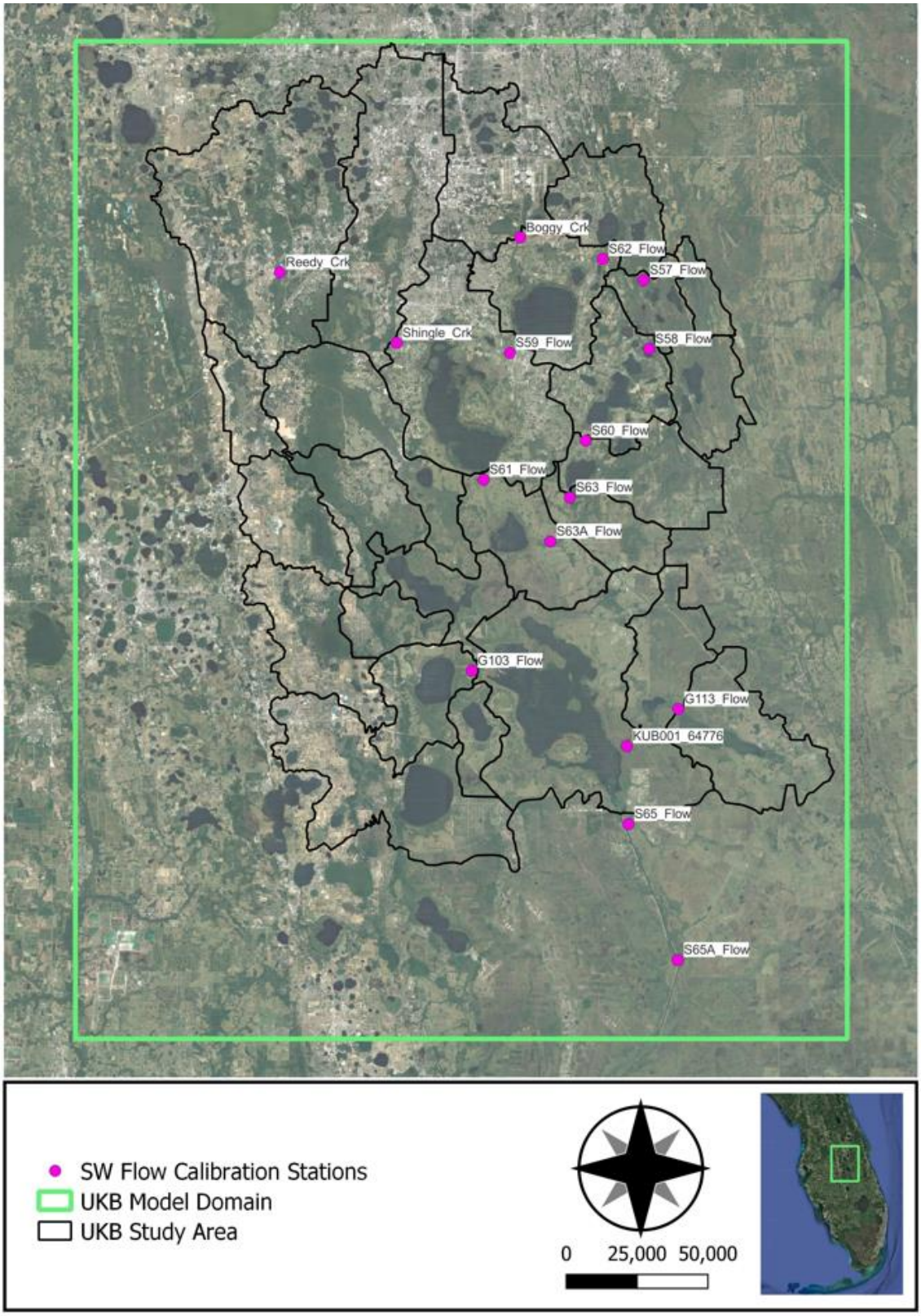

Figure 5-38: Surface Water Flow Calibration and Validation Targets in UKB FPLOS Model

Hazen

Table 5-11: Surface Water Primary Calibration Stations and Relationship to LMA

Management Area	Regulating Water Control Structure Flows				Regulating Water Control Structure Stages			
	Primary Flow Calibration Station				Primary Stage Calibration Station			
LMA	Upstream Flows		Downstream Flows		Upstream Stage		Downstream Stage	
Alligator Lake LMA			S60	S58			S60HW	S60TW
Myrtle LMA	S58		S57		S58HW	S58TW	S57HW	S57TW
Hart LMA	S57		S62		S57HW	S57TW	S62HW	S62TW
Gentry LMA	S60		S63A		S60HW	S60TW	S63A_HW	S63A_TW
East Toho LMA	S62		S59		S62HW	S62TW	S59HW	S59TW
Toho LMA	S59		S61		S59HW	S59TW	S61HW	S61TW
Cypress LMA	S63A	S61	-		S61HW, S63A_HW	S61TW, S63A_TW	LCYPR19R	LCYPR19R
Hatchineha LMA	-		C-37 Canal		LCYPR19R	LCYPR19R	MARIC	MARIC
Kissimmee LMA	KUB011		S65		MARIC	MARIC	S65HW	S65TW

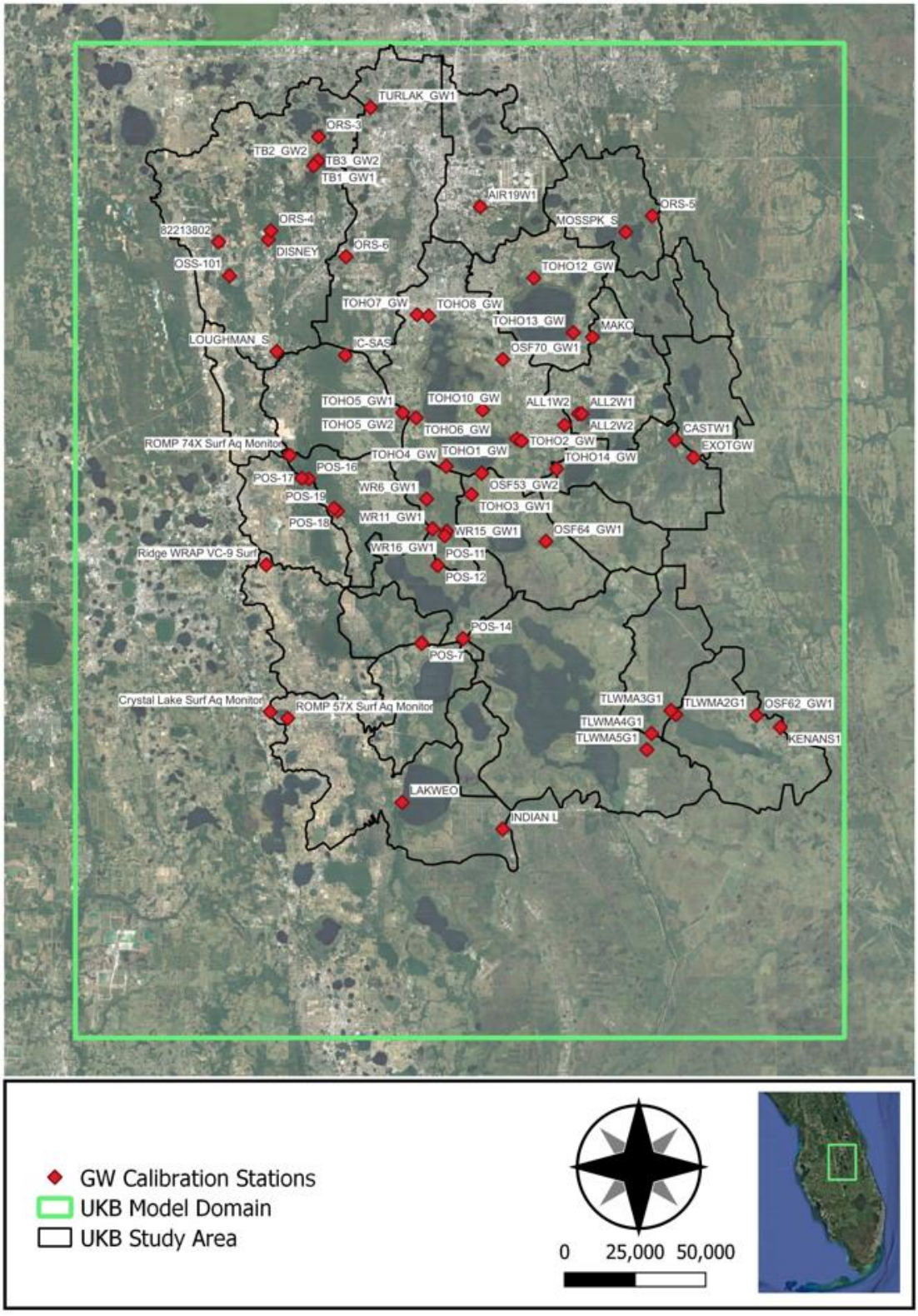

Figure 5-39: Location of Groundwater Calibration and Validation in UKB FPLOS Model

5.3.4 Sensitivity Analysis

Traditionally, the objective of model sensitivity analysis is quantification of the uncertainty of a model resulting from uncertainty in estimation of model parameters, stresses, and boundary conditions. UKB FPLOS sensitivity analysis was performed with pre-calibration model and focused on the following goals:

- Determination if a model reproduces known and understood UKCOL hydrology and hydraulics in the UKB
- Identification of the parameters that have the most impact on FPLOS performance metrics, so calibration can focus on them
- Examination of drainage wells and their importance to UKB FPLOS model efforts
- Recommendation of parameters that need further examination to improve confidence in model output.

A sensitivity analysis was done - running simulations for shorter time periods, with the pre-calibration model while systematically varying selected parameters and examining the effect of the changes to the FPLOS performance metrics. The criteria used to determine model sensitivity were:

- Change in the water budget of more than 20%
- Surface water stages and groundwater levels of higher than +/- 10% of the starting value in at least 50% of calibration stations
- Change in the surface water flows higher than +/- 20% of the initial value in at least 50% of SFWMD primary conveyance structures

The sensitivity analysis was conducted for the following input parameters:

- Initial and boundary conditions
- OL subdomain: Manning's M, detention storage depth, and root zone depth
- UZ subdomain: saturated moisture content and saturated conductivity
- SZ subdomain: Horizontal and vertical hydraulic conductivity, specific yield and storativity
- Drainage wells: Presence/need for drainage wells representation in the model
- OC subdomain: Manning's roughness and location of downstream boundary condition in the model

The sensitivity analysis is summarized in **Table 5-12**. The sensitivity analysis focused on the storm event itself, so the sensitivity model runs were hot started 1-2 days before the storm event, from a pre-calibration version of the model. In general, the model was found to be more sensitive to at least one parameter in each MIKE model subdomain that play a major role in antecedent moisture conditions

(initial conditions of overland and open channel water depths); specific yield in saturated zone and saturated moisture content in unsaturated zone. This is because the sensitivity analysis focused on the response of the system to the storm event.

Table 5-12: Summary of Sensitivity Analysis with Preliminary UKB FPLOS Model

MIKE Model Subdomain	Model Parameters	Source of Starting Values	Range of Sensitivity Testing	Results
Boundary Conditions	Boundary Conditions OL	DBHYDRO & USGS data	± 0.5 ft	Sensitive
	Boundary Conditions SZ	DBHYDRO & USGS data	± 0.5 ft	Sensitive to western BC because of the volume of flow and limited sensitivity to eastern BC
Initial Conditions	Overland water depth	MIKE modeling in the region	± 0.5 ft	Highly sensitive; effects extended about 1 week in lakes and 2 days in the creeks
	Groundwater Layer 1	MIKE modeling in the region	± 0.5 ft	Sensitive
	MIKE HYDRO	MIKE modeling in the region	± 1 ft	Highly sensitive
	OC Manning's Roughness	Engineering Practice	± 10%	Sensitive
Overland	OL Manning's Roughness	Engineering Practice	±10%	Sensitive
	Detention Storage	Based on Land Use	± 25%	Sensitive
	Depth of Root Zone	Based on Land Use	± 15%	Sensitive
Unsaturated Zone	Saturated Moisture Content	NRCS database values	± 25%	Highly sensitive
	Saturated Conductivity	NRCS database values	± 25%	Sensitive
Saturated Zone (Surficial Aquifer)	Horizontal Conductivity	SFWMD MODFLOW and aquifer test data	± 25%	Sensitive
	Vertical Conductivity	SFWMD Data	± 25%	Limited sensitivity
	Specific Yield	SFWMD Data	± 10%	Highly sensitive during initial simulation
	Storativity	SFWMD MODFLOW and aquifer test data	± 10%	Limited sensitivity
Drainage WELLS	Injection rate	Published reports	± 10%	Unclear; masked by boundary effects since most drainage wells are near model boundary

MIKE Model Subdomain	Model Parameters	Source of Starting Values	Range of Sensitivity Testing	Results
Open Channel	Manning's Roughness	SFWMD Kissimmee MIKE model	± 10%	Sensitive
	Downstream Boundary condition location	DBHYDRO data	N/A	Sensitive if BC is within 1 mile (approximately) of S65 structure
Climate Drivers	Rainfall Sources in Calibration Model	Gage and NEXRAD datasets	SFWMD datasets used for both sources	Very sensitive, because of the differences in gage and NEXRAD data; largest differences of up to 4 inches for the calibration event
	Rainfall Sources in Validation Model	Gage and NEXRAD datasets	SFWMD datasets used for both sources	Not sensitive, because of the differences in gage and NEXRAD data being less than an inch for the validation event

5.3.5 Calibration Strategy

The overall strategy for calibration was to match the observed data at S65 first, to ensure the volume of water in the study area was simulated correctly, since S65 is the outlet of the upper basin. Then the calibration focused on the uppermost part of KCOL – Lake Alligator that discharges south to Lake Gentry and north to Lakes PMJ. Calibration then focused on the entire system.

For model calibration and application, UKB FPLOS had the following models constructed and tested:

- A 328 ft x 328 ft (100 m x 100 m) model with more than 1000s of updated cross sections to provide refined representation of the floodplain and other details. The execution time of this model is 13-15 hours, for a 2-month storm event simulation.
- A 656 ft x 656 ft (200 m x 200 m) grid model, in addition to the 100m model. This helps to provide faster evaluation of various parameters, to check results and make calibration adjustments. The execution time of model is 5-6 hours.
- The 328 ft (100-m) and 656 ft (200-m) models are driven by observed gate openings for calibration and validation. This is important since the structures in Kissimmee Basin are driven by calendar-based regulation schedules, and the schedules will be used in model applications. The models have a capability of changing the operational model to be driven by observed flows at structures. This is changed through an external logical switch for each structure and allows selective change of operation by editing an external timeseries (as a '.dfs0' file), without modifying the network file. The 328 ft (100-m) and 656 ft (200-m) models (100-m and 200-m) driven by observed flows help isolate any differences that comes from the structure flow equations being different from DHI equations.
- A sub-regional/local model that focuses just on the eastern lakes (and Toho, E. Toho) – with

simplified groundwater layers. This helps understand the behavior of the eastern part of the system. The execution time of these runs is in the 1-2 hours range.

All the above models were used in the calibration phase of UKB FPLOS, for various reasons.

5.3.6 Calibration Parameters

This section describes the model parameters that were considered for modification to calibrate the model.

5.3.6.1 Open Channel Subdomain

Canal Manning's n:

Manning's n, canal roughness coefficient represents the streambed/canal roughness and generally, have the most direct impact on the surface water stages, flows and velocities. Initial values were set from the SFWMD calibrated long-term Kissimmee model in the SFWMD jurisdictional area, SJRWMD and SWFWMD models data in SJRWMD and SWFWMD jurisdictional areas; when none of these were available, literature values were used. In general, the high flow areas in the main channel have values of 0.03; the low flow areas which are the embankments or floodplains and areas around the lakes and canals have values of 0.09-0.2. Any adjustments during calibration were within these established values.

Canal Leakage Coefficient

Canal leakage coefficient is a critical model calibration parameter in the UKB FPLOS model. The larger the value of the canal leakage coefficient, the stronger the direct connection between the river/MIKE 1D is to the saturated zone. The initial values in the UKB FPLOS model were based on the SFWMD's previous Kissimmee MIKE model efforts; aquifer hydraulic conductivity and riverbed values were used to determine the lake and canal exchanges with the aquifer. The calibration methodology included analysis of upstream and downstream observed structure flows and adjustments of canal leakance coefficients to best match the surface water discharges and stages. Additionally, the closest shallow groundwater gages were also examined to ensure consistency. The final canal leakage coefficients varied between 1.0 E-6 and 1.0 E-3.

MIKE HYDRO Flow Network and Cross-Sections

The MIKE HYDRO network, which is comprised of branches and corresponding cross sections to represent the lake storage and the flow paths was an important factor for ensuring proper distribution of flows between the lakes, floodplains and canals and reasonable calibration to represent the physical parameters of the system.

To fully capture the potential flows, the floodplains required adequate representation; the previous Kissimmee models accounted for the floodplain storage mostly by triangular shaped cross sections that represent the average ground surface. For this model development, the LIDAR data with resolution of 2.5 ft was used to define accurate floodplain cross sections, which are represented by a high zone (with Manning's M of 5 units) and a low flow zone which was defined with Manning's M of 30 units to

represent the stream conditions during low flow. The high zone was defined by simultaneous analysis of the aerial (to determine changes in vegetation indicating flooded areas) and the topography to determine the best configuration of the floodplain cross sections.

Particularly important is the eastern portion of the model domain which allows discharges from the KCOL system to the eastern boundary and requires implementation of multiple flow paths and associated floodplains to be then included in the model. This flow network in the eastern portion of the model domain is complicated and historically has been the least understood. This was studied in the imagery, observed in the field after Hurricane Ian, cross-checked with data analysis performed after October No-Name storm of 2011; and then replicated in MIKE. The cross-sections were also improved using the latest 2.5 ft-resolution LiDAR data provided by SFWMD.

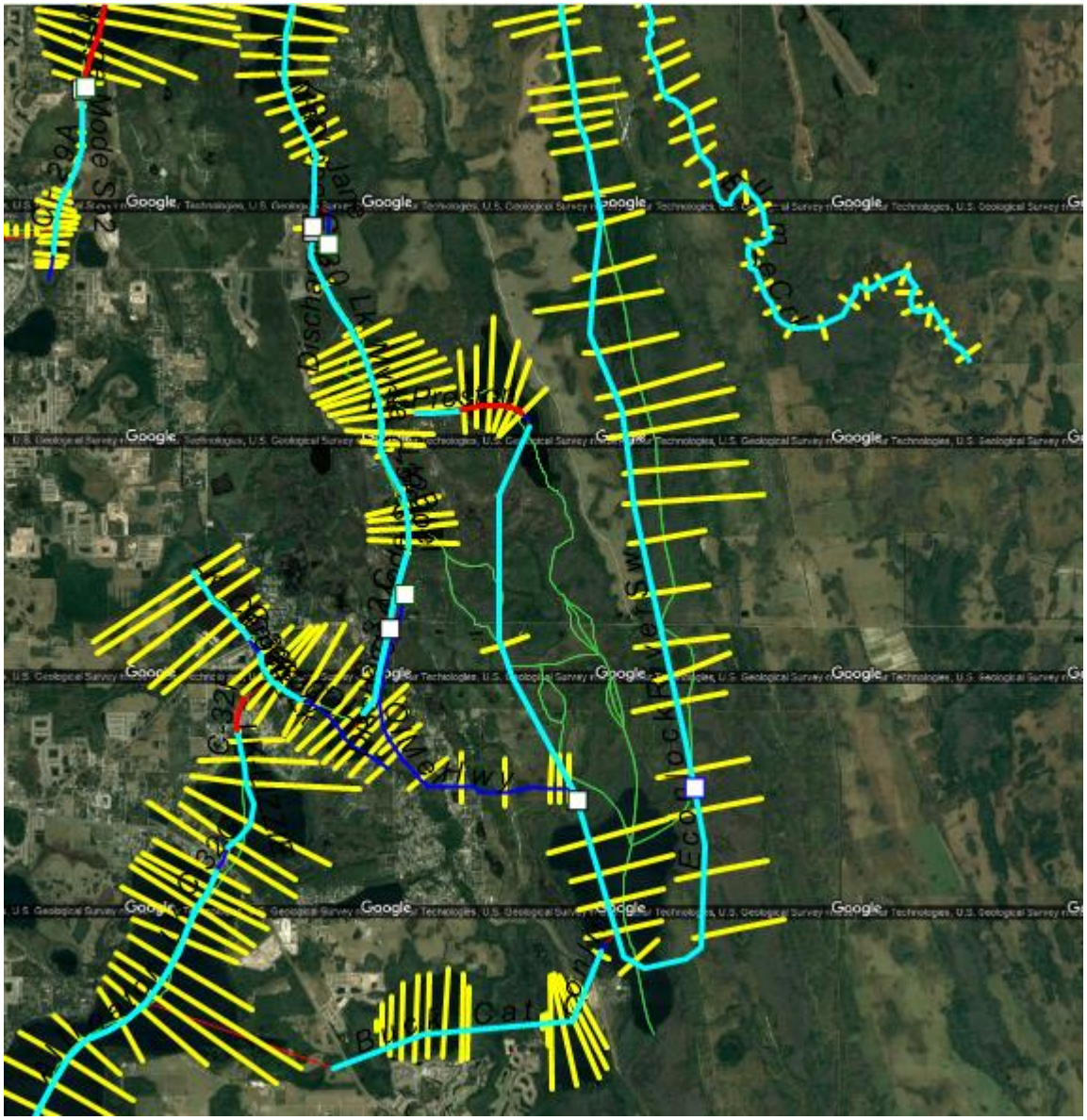

Figure 5-40: Flow Network Refinement at Econlockhatchee River

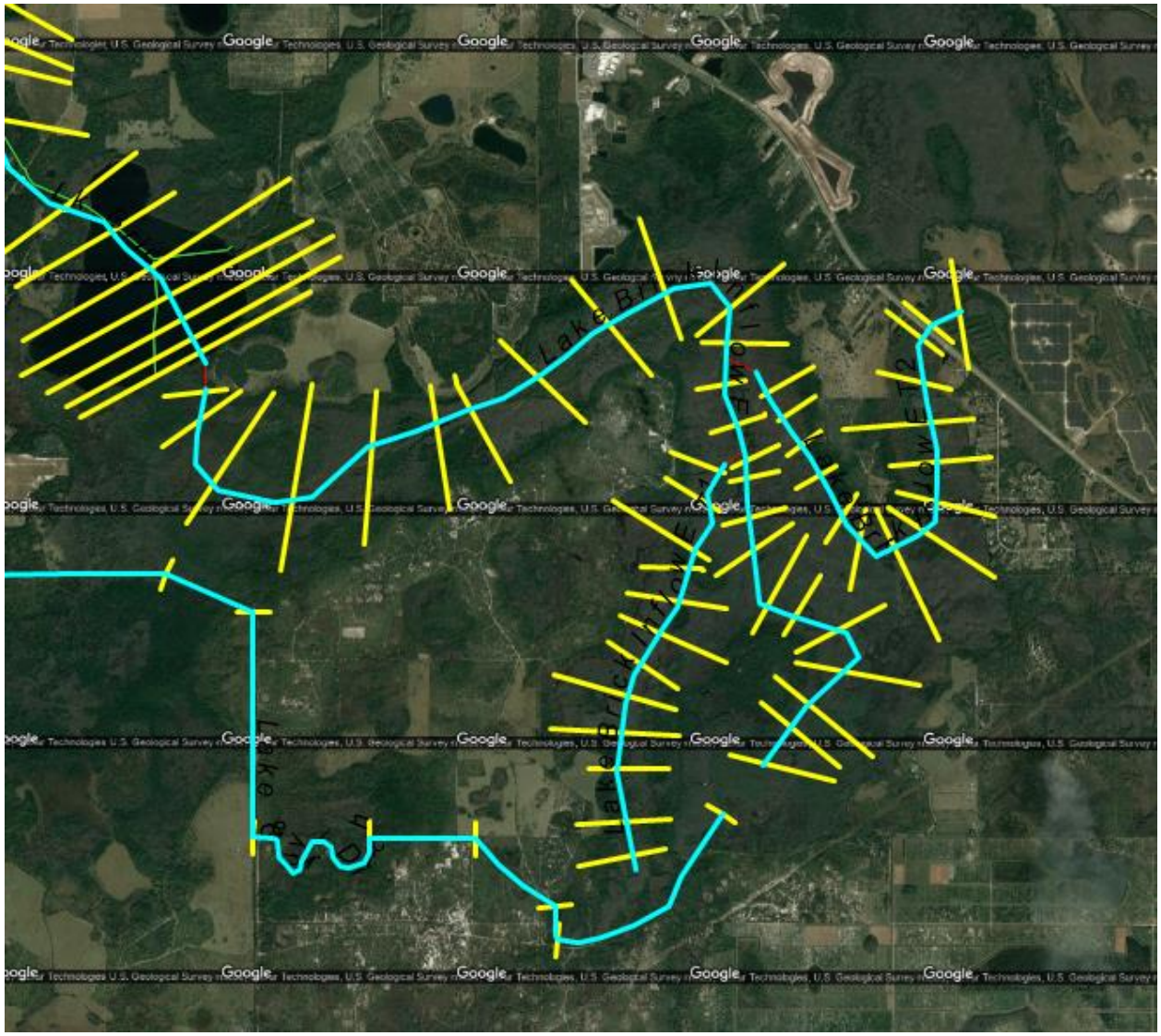

Figure 5-41: Flow Network Refinement at Lakes Brick and Gentry

In addtion, other flow network was detailed based on how critical the areas are, to UKB FPLOS project performance measures. An example is the USGS creeks, as shown in **Figure 5-42**.

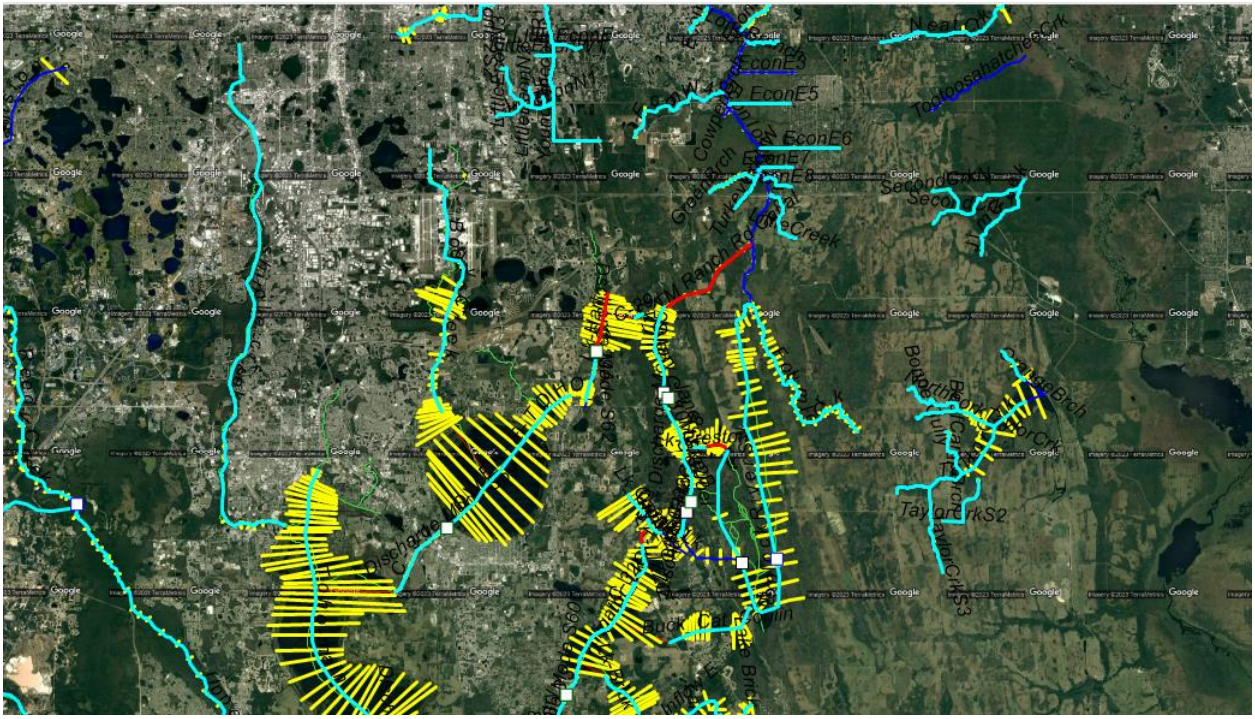

Figure 5-42: Flow Network Refinement at USGS Creeks

5.3.6.2 Overland Subdomain

All the OL parameters were based on land use/land cover dataset provided by SFWMD, as described in the earlier sections of this report.

Crop Coefficients:

ET Crop coefficient is a vegetation specific physical parameter that impacts on the evapotranspiration, and therefore the water budget. As in most models, the UKB FPLOS model was very sensitive to this parameter during tests with pre-calibration model runs. The initial values were calibrated values from SFWMD's SFWMM model and were also used in SFWMD Kissimmee model (KB HH Mod) so was not modified for this model.

Manning's M:

Manning's M values in MIKE models lump the effects of friction due to the vegetation and bedding on surface flows. This parameter directly influences the velocity of overland flow and indirectly influences the volume and the rate of infiltration to the saturated zone. The values in UKB FPLOS model are constant over the entire flow regime, but spatially distributed based on land use. The initial values of this were based on the SFWMD long-term MIKE model calibrated for the Kissimmee Basin. Calibrated values range between 6 and 28 $m^{1/3}/sec$.

Detention Storage Depth:

Detention Storage is water trapped in surface depressions. Detention storage depth values were based on land use, and the existing values taken from SFWMD calibrated Kissimmee MIKE model values for areas in the study area. For areas outside the study area, values were based on land use and land cover values within the study area. Calibrated values range between 0.1 and 0.4 inches.

Ponded Drainage Parameters:

Three of the ponded drainage parameters – runoff coefficient, inflow and outflow time constants were all considered calibration parameters. The inflow and outflow time constants represent the storage of the urban drainage network. These were used as calibration parameters; the final values ranged between $1e^{-3}$ / sec to $7e^{-6}$ / sec.

5.3.6.3 Unsaturated Zone Subdomain

Unsaturated zone parameters were modified based on the sensitivity analysis. The 36 soil zones were simplified and consolidated to 6 soil zones/types, as shown in **Figure 5-10**.

5.3.6.4 Saturated Zone Subdomain

Horizontal and Vertical Hydraulic Conductivity Values

The surfaces representing the saturated zone were from ECFTX model developed by SFWMD, SWFWMD and SJRWMD. A series of pre-calibration sensitivity simulations were conducted by varying the K_H and K_V values for SAS, and the datasets examined for agreement with APT values. The MODFLOW values were as high as 1,000,000 ft/d to represent the high conductivity lakes in the western part of the model domain. Those values, and the remaining datasets were modified to preserve the ratio in the dataset and keep the values within the field observed values.

5.3.7 Assessment of Calibration

UKCOL hydrology, and SFWMD management of UKCOL is uncommon. They are best described by the concept of Lake Management Area (LMA) which has water control structures upstream and downstream of this drainage area. The model calibration was assessed at 3 levels:

1. Regional Level assessment: Hydraulic gradient through the system

   This criterion assessed the performance of the model at the regional level. The KCH system is the dominant hydrology of the UKB. This criterion evaluates if the gradient through this system and the changes in the gradient as the storm progressed, matches observed data gradient.

2. Sub-regional level assessment: Calibration of the LMA

   This criterion evaluated the model at sub-regional level. The following are the sub-criteria/guidelines that were used to assess if a LMA is reasonably calibrated:

- Upstream structure flows are within the criterion (+/- 20% of observed flows) during at least 50% of the evaluation period
- Downstream structure flows are within the criterion (+/- 20% of observed flows) during at least 50% of the evaluation period
- Upstream structure headwater stage within 0.5 ft mean error (ME) during the evaluation period
- Upstream structure tailwater stage within 0.5 ft ME during the evaluation period
- Downstream structure headwater stage within 0.5 ft mean error (ME) during the evaluation period
- Downstream structure tailwater stage within 0.5 ft ME during the evaluation period

3. Watershed level assessment: Individual stations

   The third criterion assessed the model performance at each watershed level: analyzing the statistics of stages, flows and cumulative flows at individual stations. The statistics were computed for surface water and groundwater stations. Water budgets were also computed for reasonableness checks.

5.3.8 Calibration Results

Calibration was conducted by parameter adjustments as described in the previous section and analyzing the performance of the model at the primary and secondary calibration targets and water budgets. After each calibration run, the metrics were used to evaluate model performance using the calibration criteria. The results of the UKB FPLOS model calibration are presented and discussed in this section. Out of all the observation sites, all stations in the SFWMD conveyance system were the primary focus and are described in detail in this section, while the remaining stations were used to ensure best model performance within the study area.

The assessment was done for calibration within the UKB FPLOS study area consisting of 26 watersheds shown in **Figure 1-3**. The model domain was extended beyond the study area to establish hydrology in the study area where the performance metrics will be computed and to minimize the boundary effects. Information and lessons of sensitivity analysis from the model domain were used to reduce the non-uniqueness of the solutions.

The results of calibration and validation are also presented for the entire simulation period and for the storm event. SFWMD experience with past FPLOS projects, analyses of watershed behavior during storm events, preliminary model sensitivity analysis was used to decide on length of simulations for calibration for parameter adjustment and validation.

A total of 61 surface water observations at SFWMD structures were used to guide the history-matching in the UKB FPLOS calibration process; 45 stage monitoring and 16 flow sites were used for surface water calibration within the UKB FPLOS study area. Groundwater contribution is significant in the

UKB FPLOS model domain; 79 Surficial Aquifer observations were used for quantification of the saturated zone impacts on surface water and interaction.

Model performance of calibration at structure headwater and tailwater stages were evaluated with stage and flow mean error (ME), mean absolute error (MAE), RMSE, STD, R and NS metrics; all of which are summarized for surface water stages and flows. **Appendix A** lists the stations that were available, and which were excluded from calculations of statistics, with comments on the reason. The list of all surface water and groundwater stations with their DBKEYs are listed in **Appendix B**. **Appendix C** has all the plots from the calibration model during the period of simulation (POS), including the stations that were excluded from calculation of statistics. The plots are for the entire POS, which includes the periods statistics are computed for. Calibration plots focused on the storm event (Hurricane Irma) are shown in **Appendix D**.

5.3.8.1 Analysis of stage gradient during Hurricane Irma

One way to assess the calibration of the UKB FPLOS model to Irma is a snapshot of KCOL as the storm progresses. The observed and simulated head gradient across the KCOL system is captured in **Figure 5-43** and **Figure 5-44**.

The plots show that there are differences in the initial conditions on the 10th of September between the observed and simulated stages. The data clearly shows the pre-storm drawdown performed by SFWMD in preparation for the hurricane. After the initial conditions, the response to the storm is captured well in the UKBFPLOS calibration model. On 9/18/17, at S61, the observed stage is 54.1 ft, falling to 52.37 ft at S65HW. In the model, the stage at S61TW is approximately 54 ft NAVD88, gradually lowering to 52.5 ft NAVD88, keeping the gradient across the system at approximately 1.5 ft as in observed data. The stages used for this analysis were the end-of-day values.

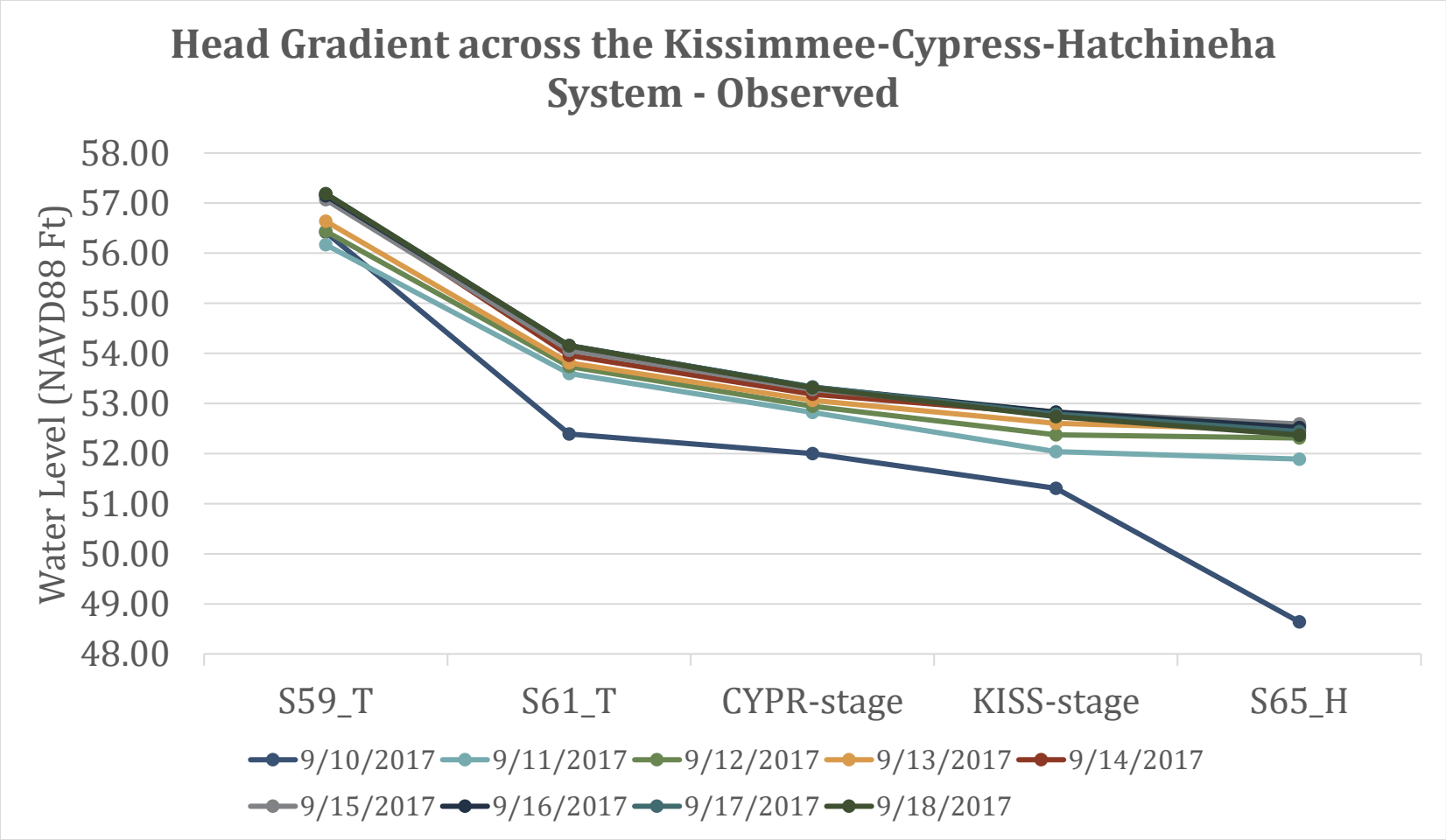

Figure 5-43: Observed Stage Gradient (ft NAVD88) across KCH after Hurricane Irma

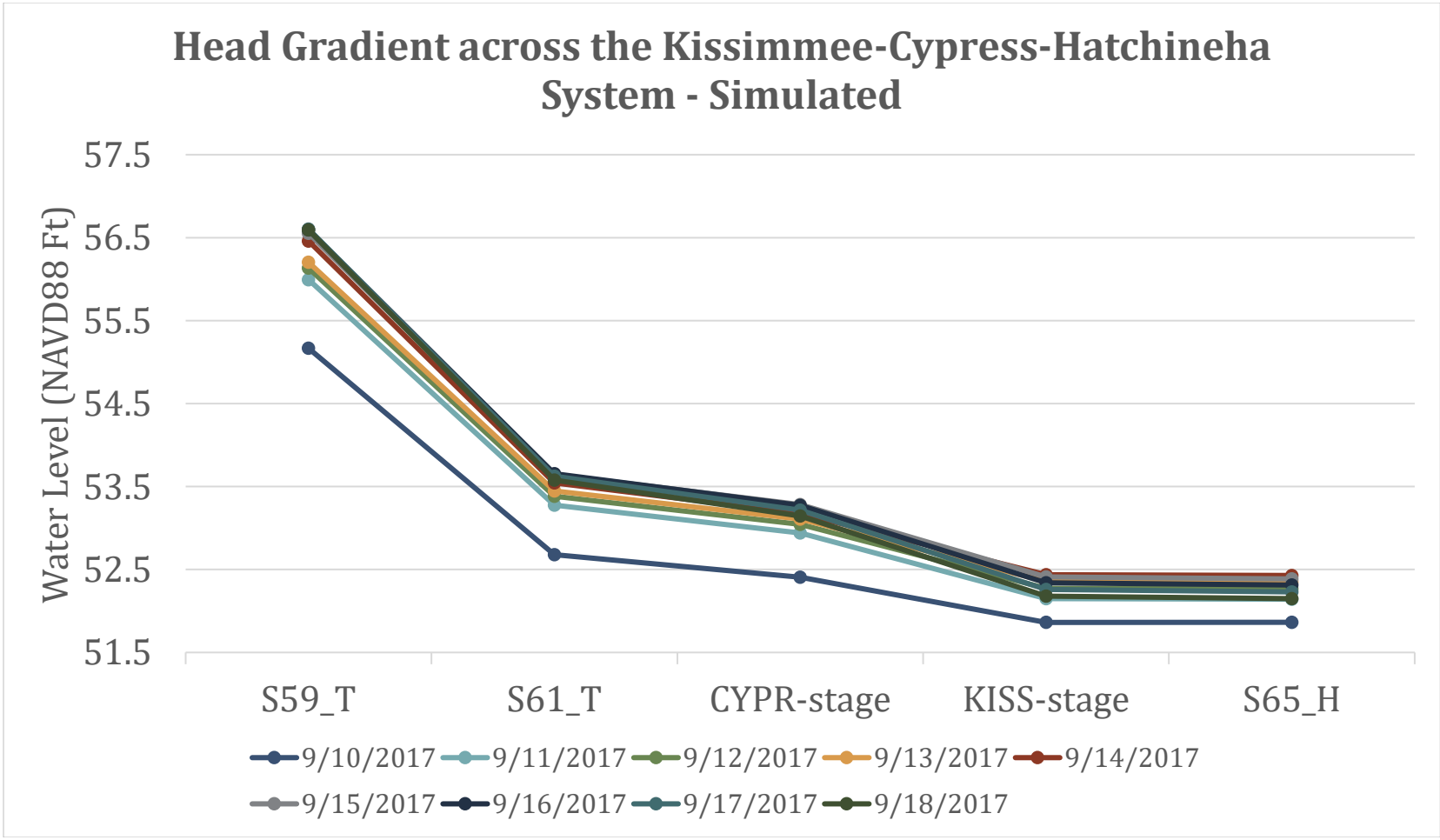

Figure 5-44: Simulated Stage Gradient (ft NAVD88) across KCH after Hurricane Irma

5.3.8.2 Water Balance

The water budgets were checked for reasonableness; the water balance computed for each watershed for the calibration model is shown in **Appendix I**. Detailed water budgets were developed for the UKB FPLOS watersheds. **Figure 1-3** shows the model watersheds areas used to extract the water budget data. The water budget analysis includes summary of the water balances for all four model subdomains: OL – Two-dimensional Overland Flow, UZ – One dimensional flow in the Unsaturated Zone, SZ – Three-dimensional flow in Saturated Zone and OC – Open Channel and hydraulics (canals, water control structures).

The general water balance which is used in DHI's Water balance Tool is based on the equation:

Error = Inflows + Outflows - Change in Storage

In MIKE SHE-MIKE HYDRO, the inflows are defined as positive if the water flux enters the control volume (Inflows), and negative when the water flux leaves the control volume (Outflows).

The simulations cover the following period: 9/10/2017 - 10/30/2017.

The water balance over the MIKE SHE domain in inches is summarized for the calibration and validation events; MIKE HYDRO items are treated as boundary flows and are included in 'Base flow from/to River', OL to River', and 'SZ Drain to/from River'. A reasonable check of the water balance indicates ET, overland to river and saturated zone drains to river are the major drivers during the storm event, as expected.

5.3.8.3 Calibration Statistics

The hydrology of KCOL is driven by the lakes, which are heavily managed. **Table 5-11** is a tabular description of each LMA in UKBFPLOS Model domain, and their hydrological components. The calibration metrics were computed for a subset of the POS: 09/10/2017 to 10/10/2017. Statistics were also computed for another subset focused on Hurricane Irma: 09/10/2017 – 09/18/2017. Calibration results are presented here for each LMA and further statistics.

The results of calibration for surface water stages are presented in the following formats:

1. Spatial plots of the ME statistic in this section (surface and groundwater stages and flows plots for the calibration period are attached in **Appendix D**)
2. Tabular format of all the MIKE SHE generated calibration statistics for 09/10/2017 to 10/10/2017 and 09/10/2017 – 09/18/2017 (**Appendix E**[1])
3. For each LMA, stage and flow calibration statistics are summarized in this section

[1] In Appendix E and F, a statistics value of -1 indicates that observed data were not available.

For surface water flows, the calibration statistics were computed as follows: hourly or break point (in the case of observed flows) flow data are considered for the analysis. Both the simulated and observed flow time series are aggregated to an hourly time scale and for each hour in the POS, the observed and simulated flow values are compared, and the percentage of times where simulated flows are within ±20% observed flows are computed. Calibration statistics for surface water flows discussed below references **Table 5-13**.

A python script was used to run through the datasets to extract peak time and stages and flows at peak time. However, the script picked up values from a non-Irma related response or instabilities. So, the results of the python script were QA/QC ed, and peaks were selected manually when necessary.

Calibration statistics for groundwater stage station are presented in the following formats:

1. Spatial plots of the ME statistic (surface and groundwater stages and flows plots for the calibration period are attached in **Appendix D**)

2. Tabular format of GW stations with peak stages and time to peak criteria.

5.3.8.3.1 Surface Water

Plots of the SW ME statistic are shown in **Figure 5-54** and **Figure 5-55**, which depict where a residual of 0.5 ft and 0.75 ft are met spatially for 09/10/2017 – 10/10-2017. **Figure 5-56** and **Figure 5-57** show spatially the same for 09/10/2017 – 09/18/2017. **Table 5-13** shows the percentage of time during the simulation when the modeled flows are within 20% of the observed flows. Calibration statistics are summarized below for each LMA.

Alligator Lake LMA

Surface water management in this LMA include S58 and S60 structures, both are outflow structures. In the calibration simulations, S58 HW and S58 TW had a ME of [0.4ft, 0.1ft] and R of [0.9, 0.6] respectively. The UKBFPLOS model S60 HW and S60 TW had a ME of [0.4 ft, 0.7 ft] and a R of [0.9, 0.8] for S60 HW and S60 TW. (Refer to **Figure 5-45** for the calibration plots). The calibration modeled flows at S60 were within 20% of the observed values, 26% of the time. S58 was kept closed during the simulation.

During Hurricane Irma, S58 HW and S58 TW had a ME of [0.5ft, -0.7ft] and R of [0.9, 0.9] respectively. ME of [0.4ft, 0.8ft] and R of [0.9, 0.9] describes S60 HW and S60 TW.

Myrtle LMA

This LMA has S58 and S57 as surface water inflow and outflow structures. S57 HW and S57 TW had a ME of [0.2ft, 0.03ft] and a R of [0.9, 0.9] respectively (refer to **Figure 5-45** and **Figure 5-46** for stage calibration plots). Simulated flows at S57 during the POS were within 20% of observed flows 72% of the time. S58 was kept closed 100% of the time during the POS.

For the storm event, S57 HW and S57 TW had a ME of [0.1ft, -0.1ft] and R of [0.9, 0.9] respectively. As for surface water flows, simulated flows at S57 during the storm event are within 20% of observed

flows 63% of the time. S58 was simulated as closed 100% of the time during the storm event (refer to **Figure 5-50** for calibration plots).

Hart LMA

S57 and S62 are inflow and outflow structures in this LMA. S62 HW and S62 TW had a ME of [-0.03ft, 0.6ft] and R of [0.9, 0.9] respectively (refer to Figure 5-46 for stage calibration plots, **Figure 5-52** for cumulative flow plots). S62 simulated flows were within the 20% margin of the observed flows 77% of the POS (refer to **Figure 5-50** for calibration plots).

For Irma, S62 HW and S62 TW had ME of [-0.1ft, 0.7ft] and R of [0.9, 0.9] respectively. S62 simulated flows were within the 20% margin of the observed flow for 16% of the simulation time and for 9% of the storm event period.

Gentry LMA

In this LMA S60, S63, and S63A were used as calibration stations; with inflows into Lake Gentry from S60 and outflows through S63. S63A is downstream of S63. S63 HW and S63 TW simulated stages had ME of [0.9ft, 0.6ft] and R of [0.6, -0.1] respectively (refer to **Figure 5-47** and **Figure 5-48** for calibration plots). ME of [0.7ft, -0.3ft] and R of [0.3, 0.9] are what simulated stages have for S63A HW and S63A TW respectively. During the POS, S60, S63, S63A structures met the 20% flow criterion 26%, 30%, and 64% of the time (refer to **Figure 5-51** for calibration plots and **Figure 5-53** for cumulative flow plots).

S63 HW and S63 TW simulated stages had ME of [1.6ft, 1.6ft] and R of [0.9, 0.1] respectively during the storm event. At S63A HW and S63A TW, the calibrated model had a ME of [1.9ft, -0.5ft] and R of [0.2, 0.9]. During the storm event, S60, S63, S63A surface water flows were 1%, 14%, and 39% of the time within 20% of their respective observed flows.

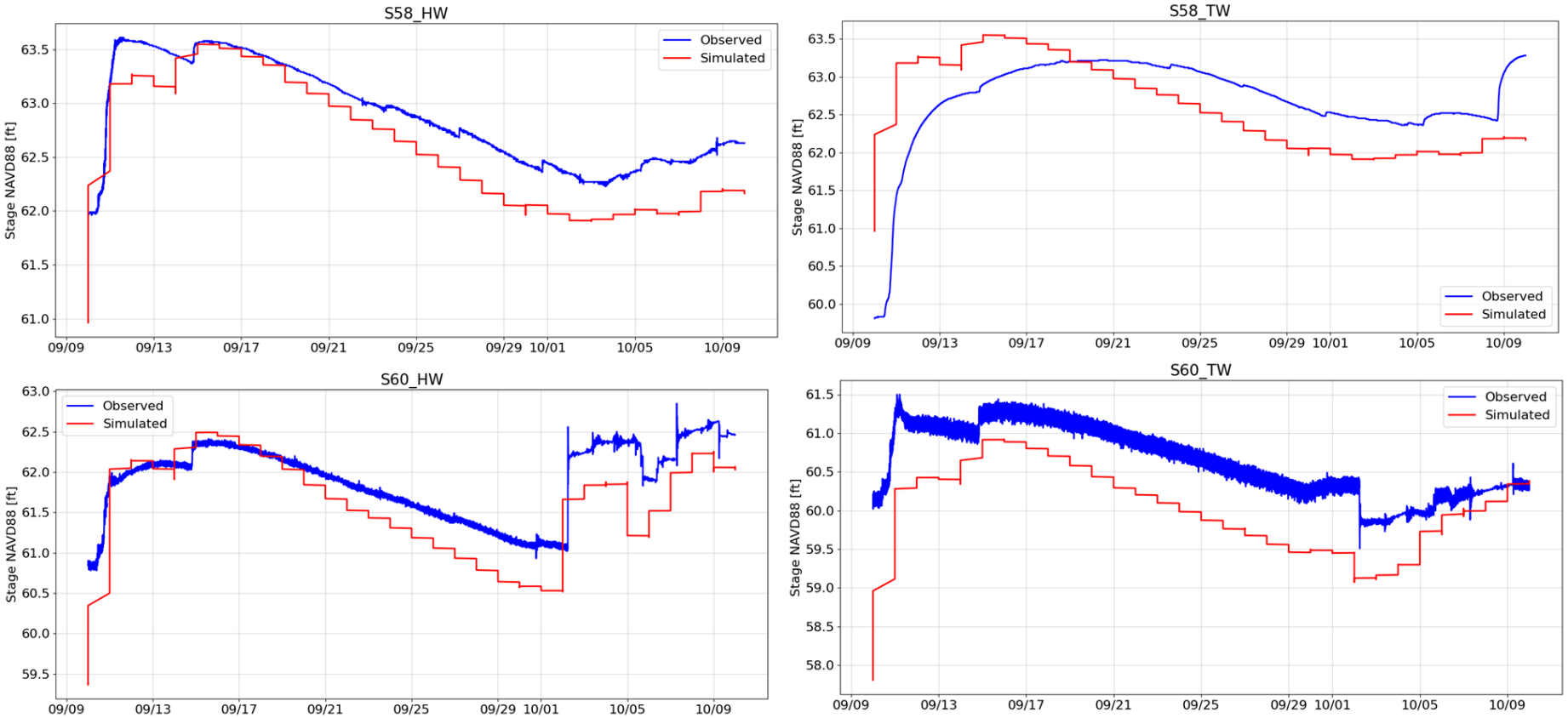

Figure 5-45: S58 and S60 Observed and Simulated Stages for Hurricane Irma

East Toho LMA

S62 and S59 structures are at the inlet and outlet of this LMA. S59 HW and S59 TW had a ME of [0.7ft, 0.6ft] and R of [0.9, 0.9] respectively (refer to **Figure 5-46** and **Figure 5-48**). For 97% of the time, S59 simulated flows were within 20% of the observed S59 flows.

For Irma, S59 HW and S59 TW had ME of [0.6ft, 0.5ft] and R of [0.9, 0.9] respectively. S59 simulated flows were within 20% of the observed flow (refer to **Figure 5-50** for calibration plots and **Figure 5-52** for cumulative flow plots) for 89% of the time.

Toho LMA

The surface water hydrology in this LMA is regulated by S59 and S61 structures. S61 HW and S61 TW had ME of [-0.1ft, 0.1ft] and R of [0.7, 0.9] respectively (refer to **Figure 5-48** and **Figure 5-49**). For 96% of the time, S61 simulated flows were within 20% of the observed flows.

For Hurricane Irma, S61 HW and S61 TW have a ME of [-0.7ft, -0.2ft] and R of [0.8, 0.9] respectively. For 89% of the time, S61 simulated flows were within the 20% margin of the observed flow (refer to **Figure 5-50** for calibration plots and **Figure 5-52** for cumulative flow plots).

Cypress LMA

S61 and S63A structures inflow into this LMA. There are no outflow structures in this LMA but there is a stage recorder LCYPR19R is at the outlet in C36 Canal. Calibration statistics of S61 and S63A were included in Gentry LMA and Toho LMA. LCYPR19R had a ME of -0.5ft and R of 0.9 (**Figure 5-47** and **Figure 5-49** show the calibration). LCYPR19R for Hurricane Irma had a ME of -0.5ft and R of 0.9.

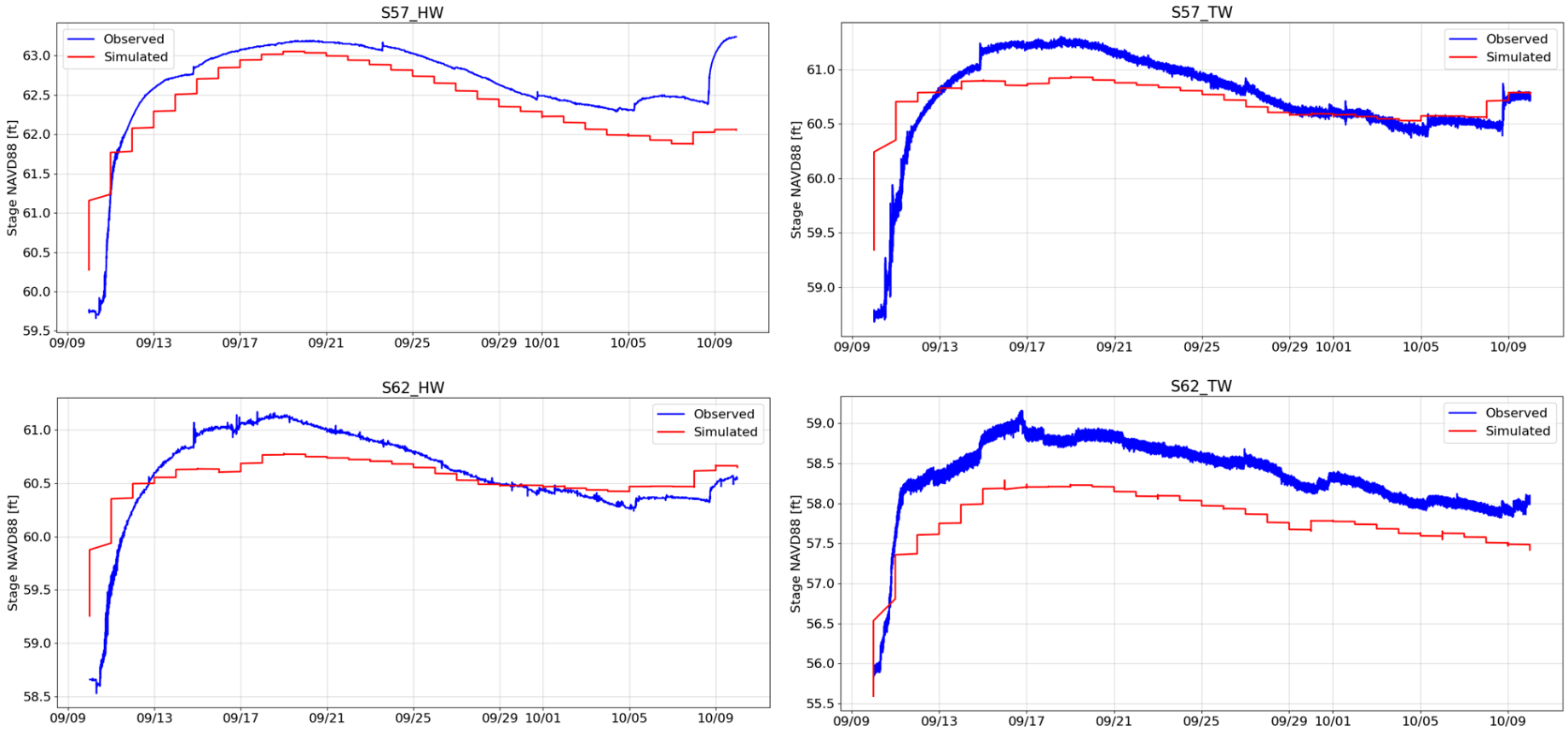

Figure 5-46: S57 and S62 Observed and Simulated Stages for Hurricane Irma

Hatchineha LMA

Hatchineha has no inflow structures; however, at the inlet of this LMA are stage calibration stations MARIC and LCYPR19R, and KUB011 is at the downstream end of the LMA. MARIC had ME of -0.6ft and R of 0.9. KUB011 which is also at the inlet of the Kissimmee LMA, had a ME of -0.5ft and R of 0.9. KUB001 met the 20% range criteria 18% of the time during the POS.

During the storm event, MARIC had ME of 0.2 ft and R of 0.9. KUB011 which is also at the inlet of the Kissimmee LMA, had a ME of -0.1ft and R of 0.8. KUB001 simulated flows were within the 20% range of the observed flow 2% of the time during the storm event.

Kissimmee LMA

In this LMA, KUB011 and S65 were the calibration stations; KUB011 is a stage recorder with observed stages and S65 is the outlet of Lake Kissimmee and the entire UKB. S65 HW and S65 TW had ME of [-0.6ft, -0.3ft] and R of [0.4, 0.9] respectively (refer to **Figure 5-49** for calibration plots). During the POS, S65 simulated flows were within the 20% margin of the S65 observed flows for 61% of the time (refer to **Figure 5-51** for calibration plots and **Figure 5-53** for cumulative flow plots).

For the storm event, S65 HW and S65 TW have ME of [-1.3ft, -0.03ft] and R of [0.5, 0.9] respectively. S65 simulated flows were within the 20% margin of the observed flow for 59% of the time.

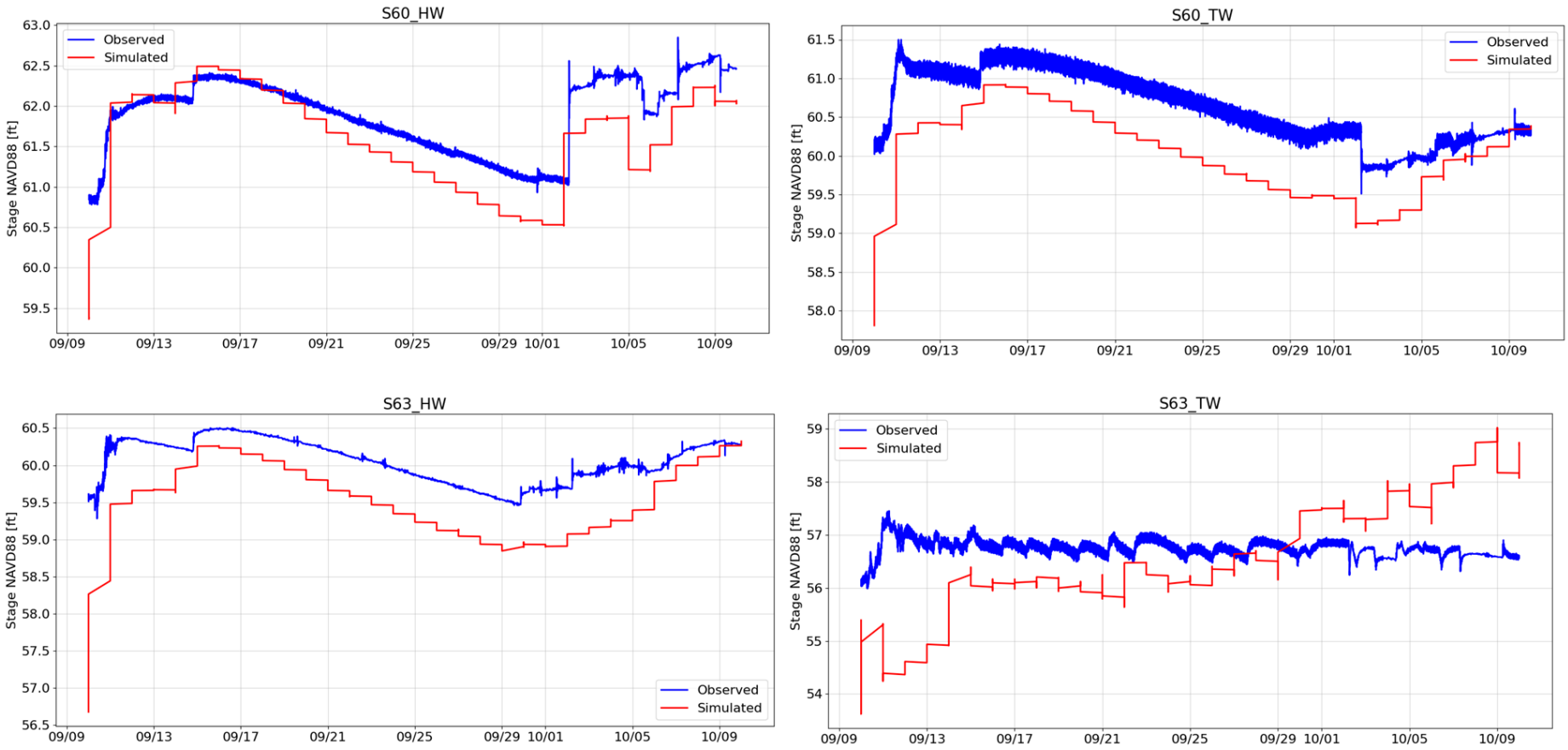

Figure 5-47: S60 and S63 Observed and Simulated Stages during Hurricane Irma

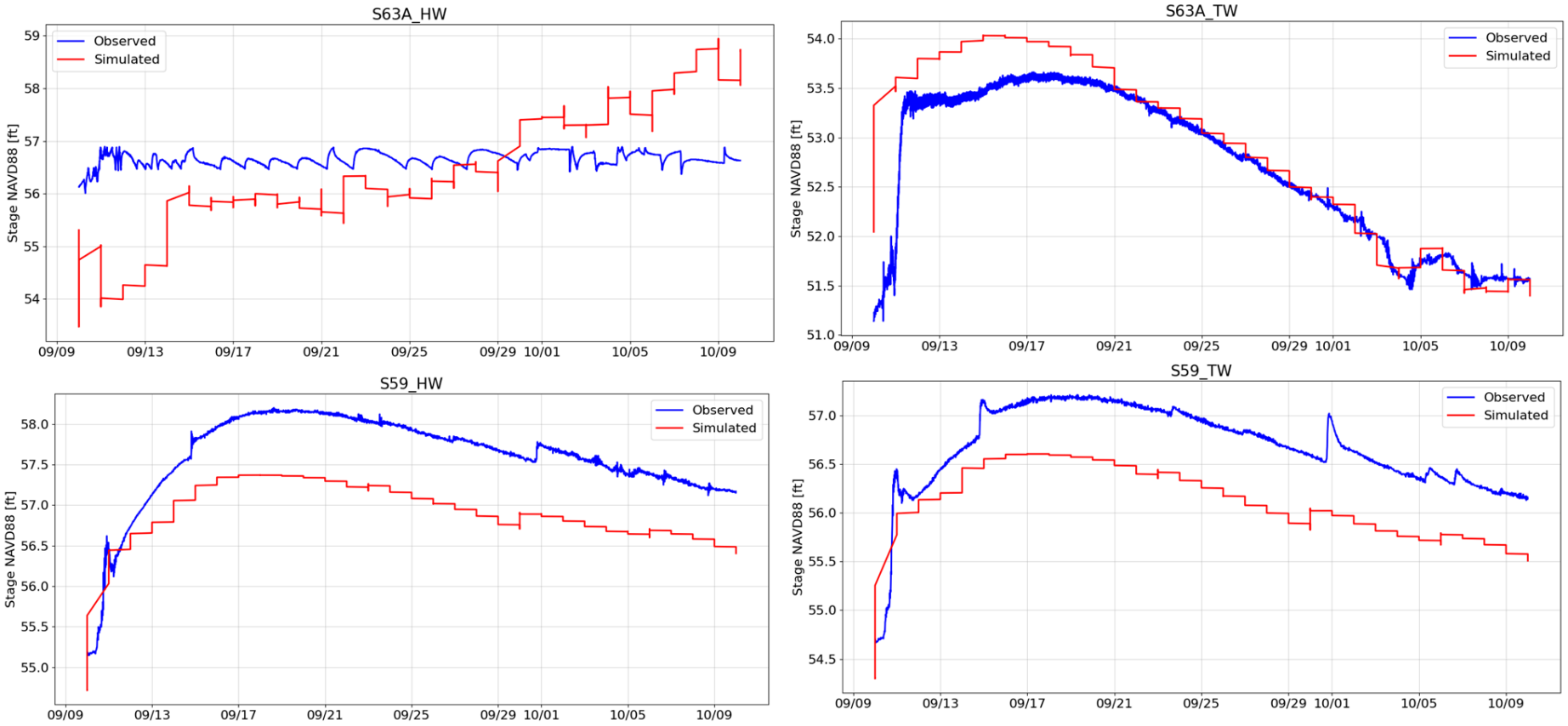

Figure 5-48: S63A and S59 Observed and Simulated Stages during Hurricane Irma

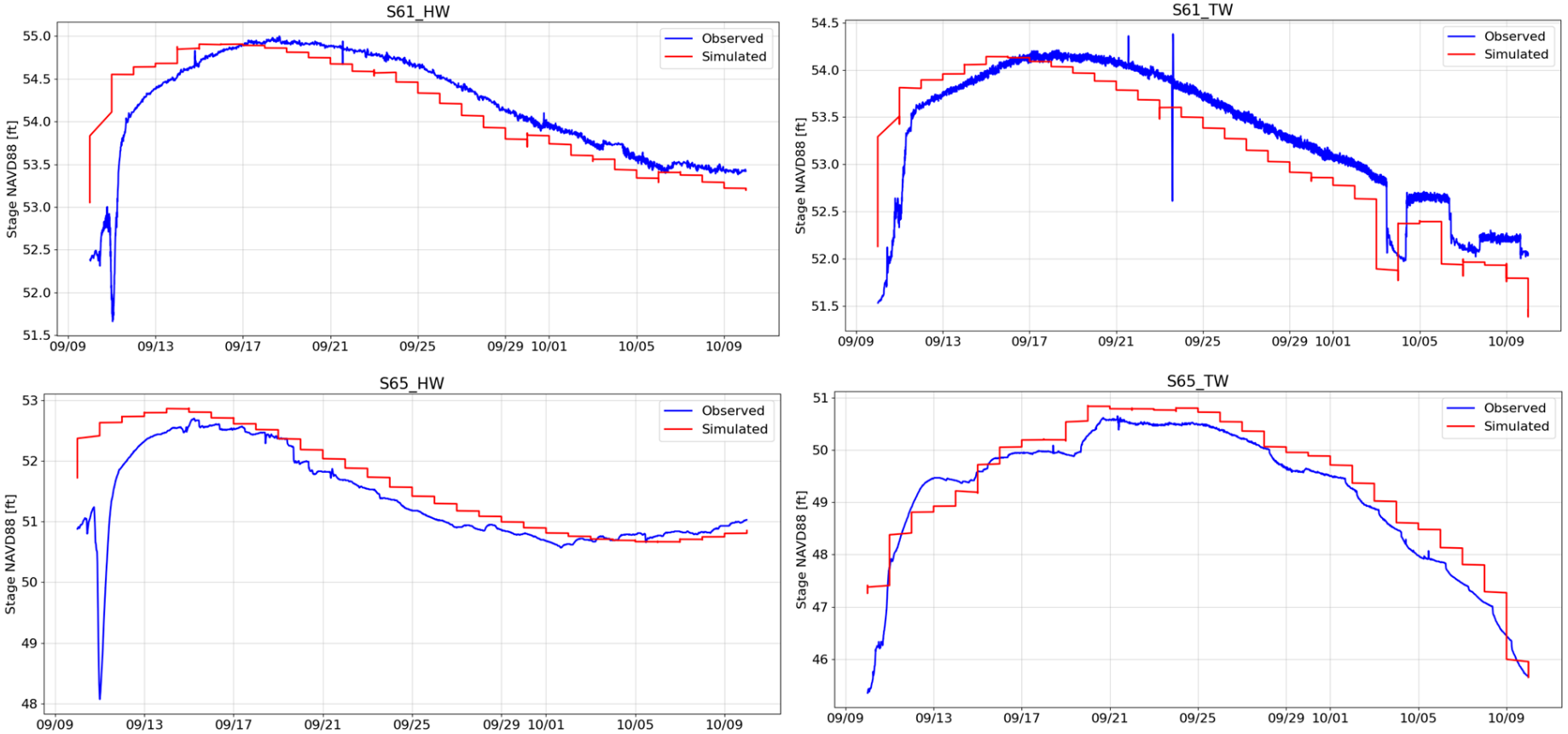

Figure 5-49: S61 and S65 Observed and Simulated Stages during Hurricane Irma

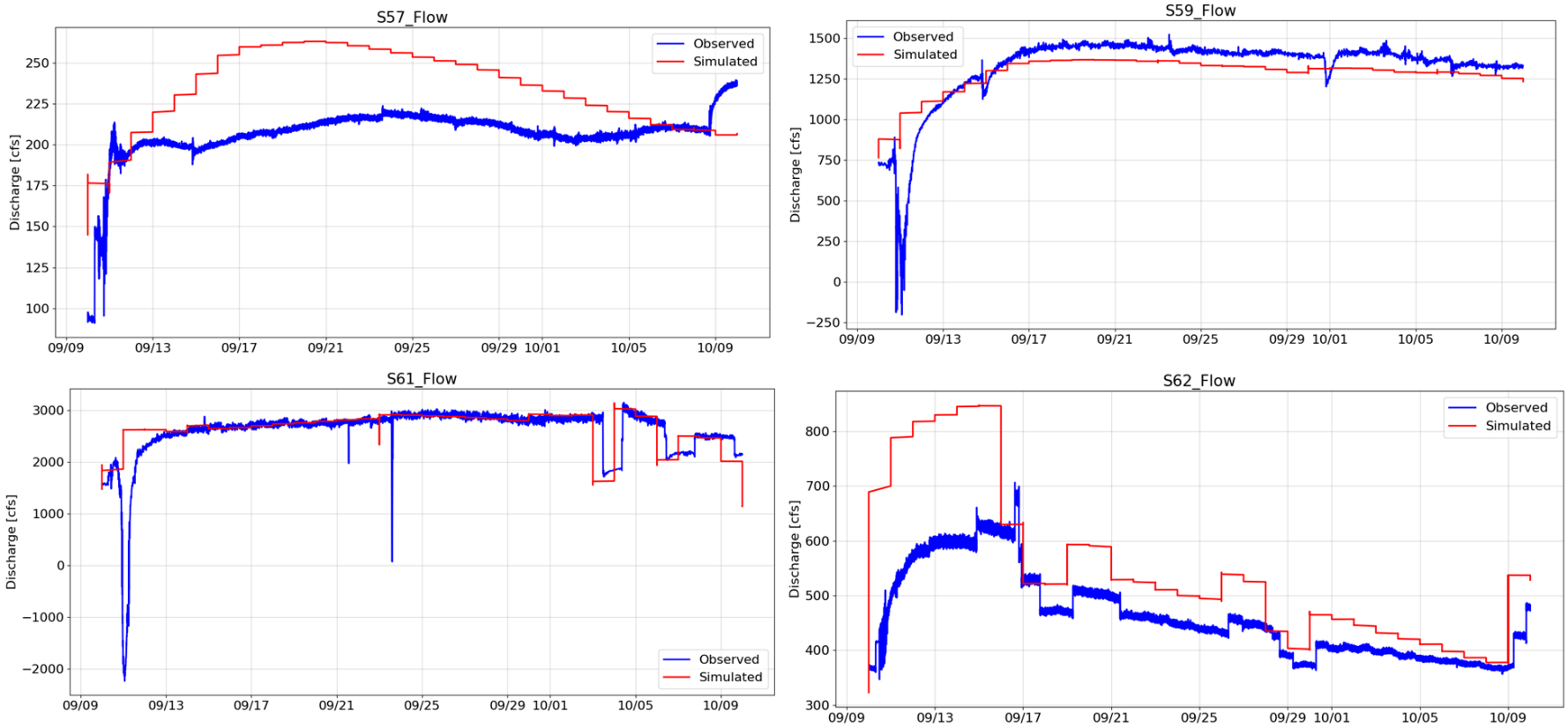

Figure 5-50: S57, S59, S61, and S62 Observed and Simulated Discharges during Hurricane Irma

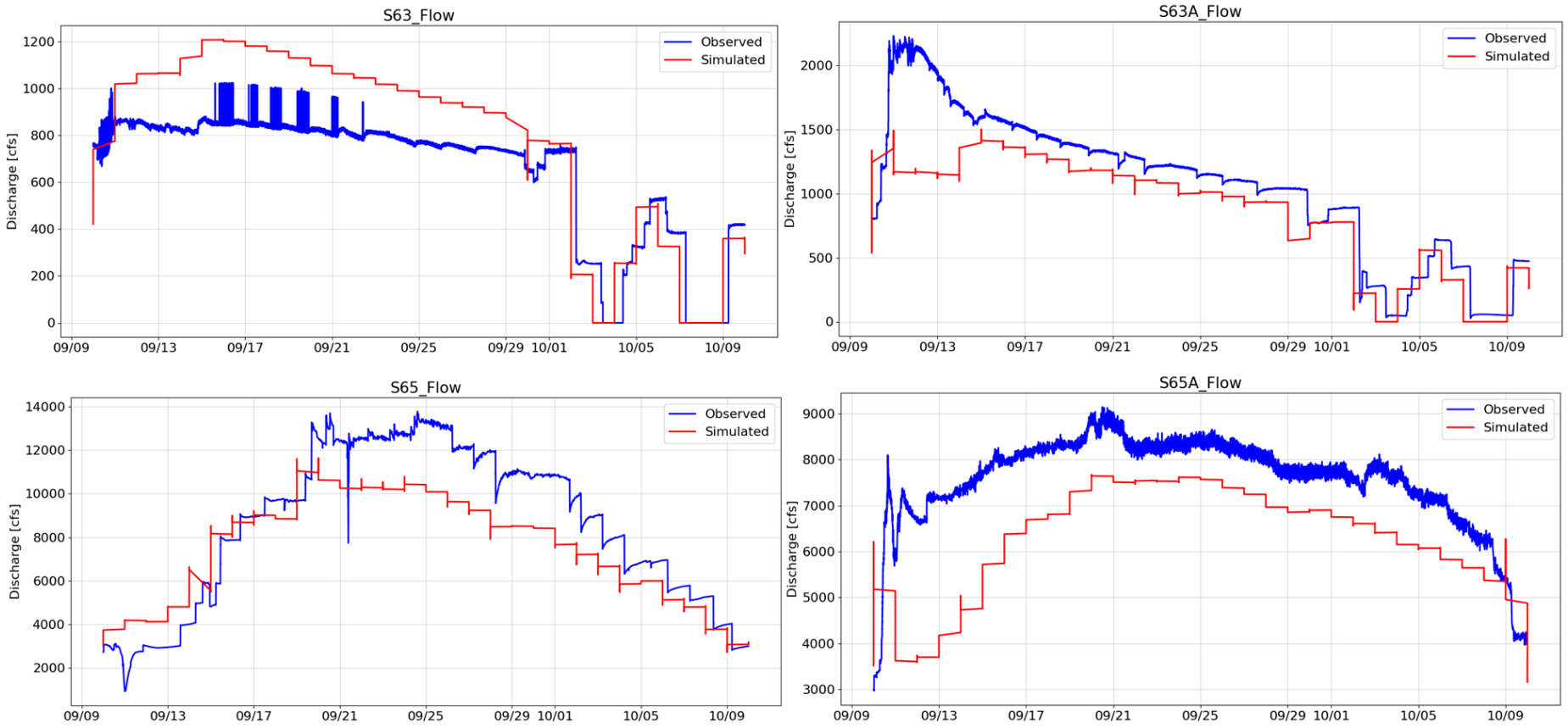

Figure 5-51: S63, S63A, S65, and S65A Observed and Simulated Discharges during Hurricane Irma

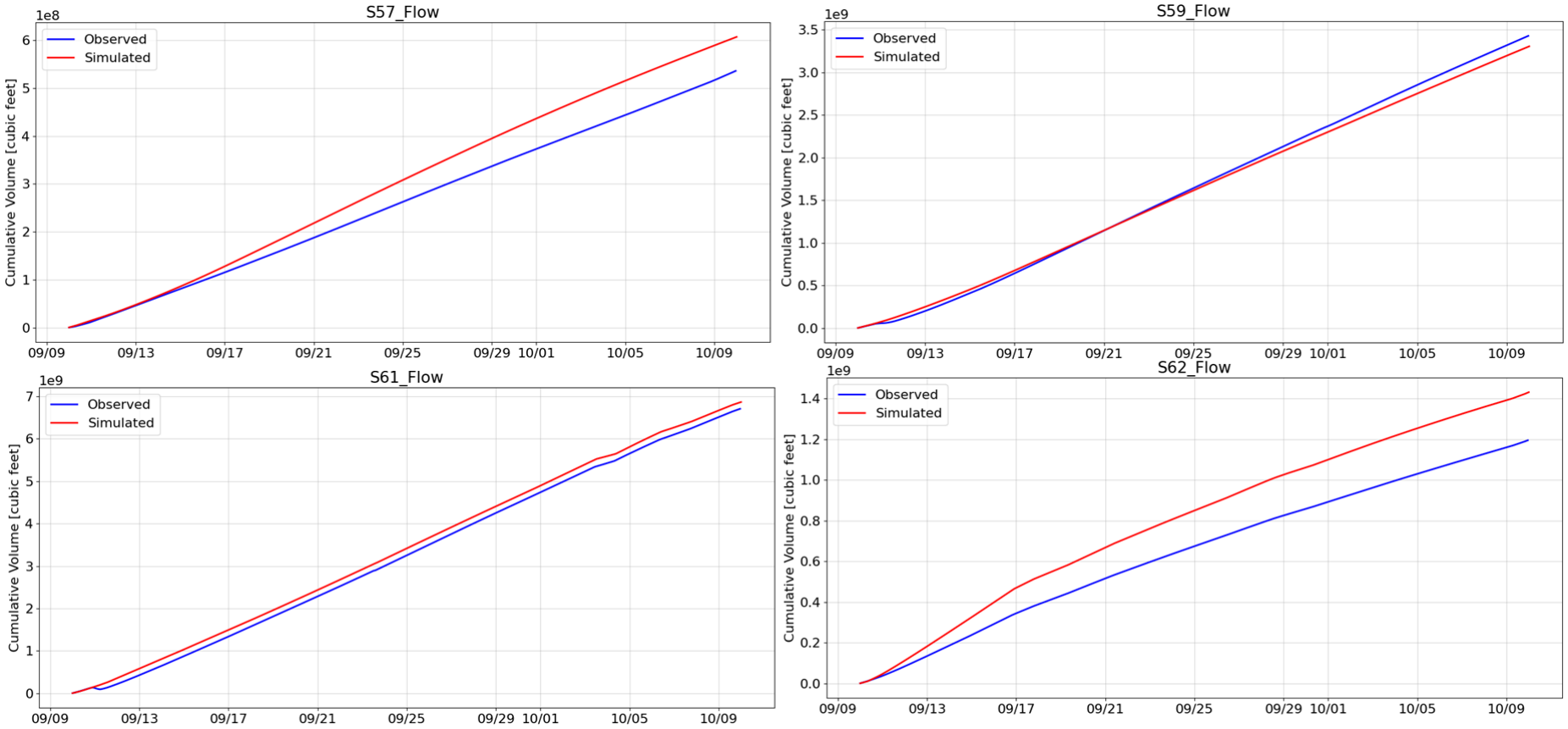

Figure 5-52: S57, S59, S61, and S62 Observed and Simulated Cumulative Flows during Hurricane Irma

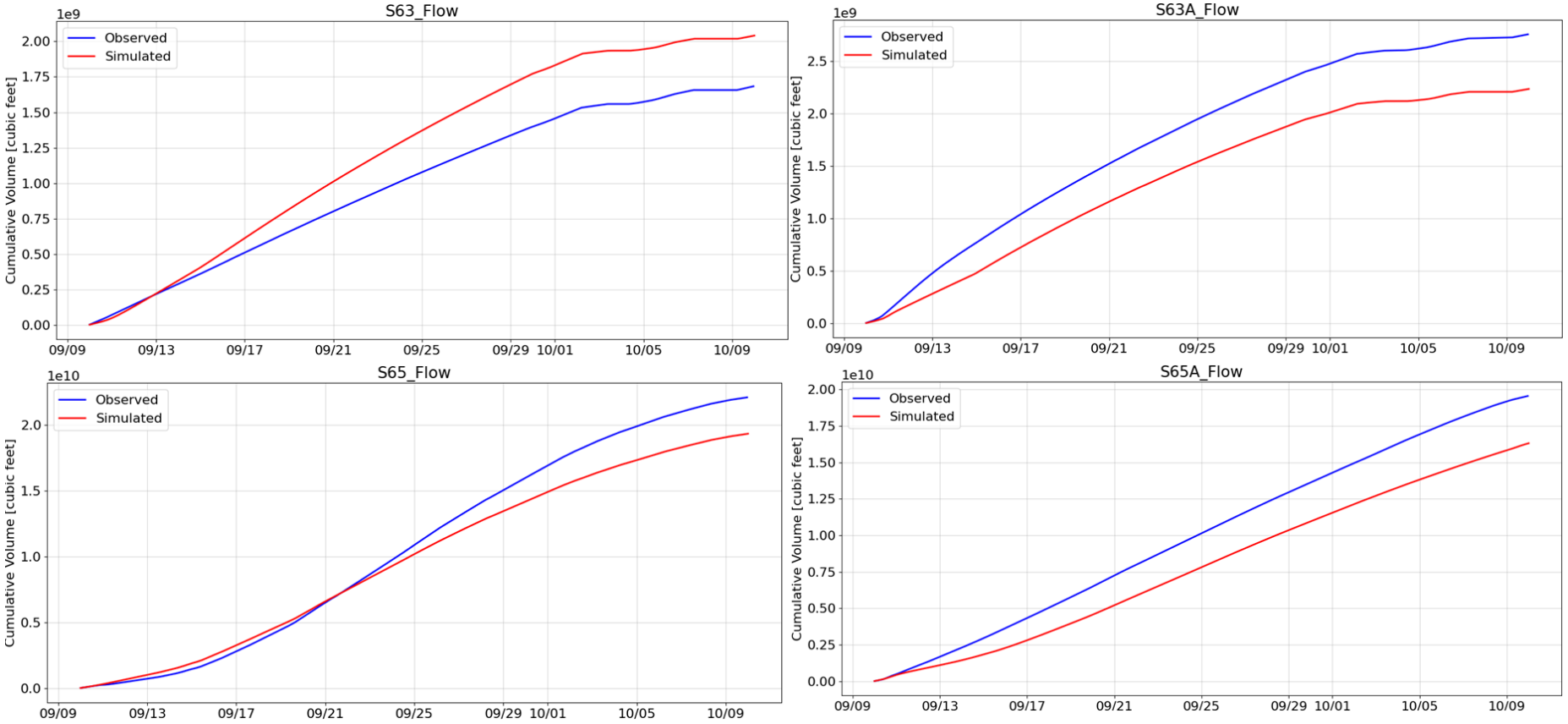

Figure 5-53: S63, S63A, S65, and S65A Observed and Simulated Cumulative Flows during Hurricane Irma

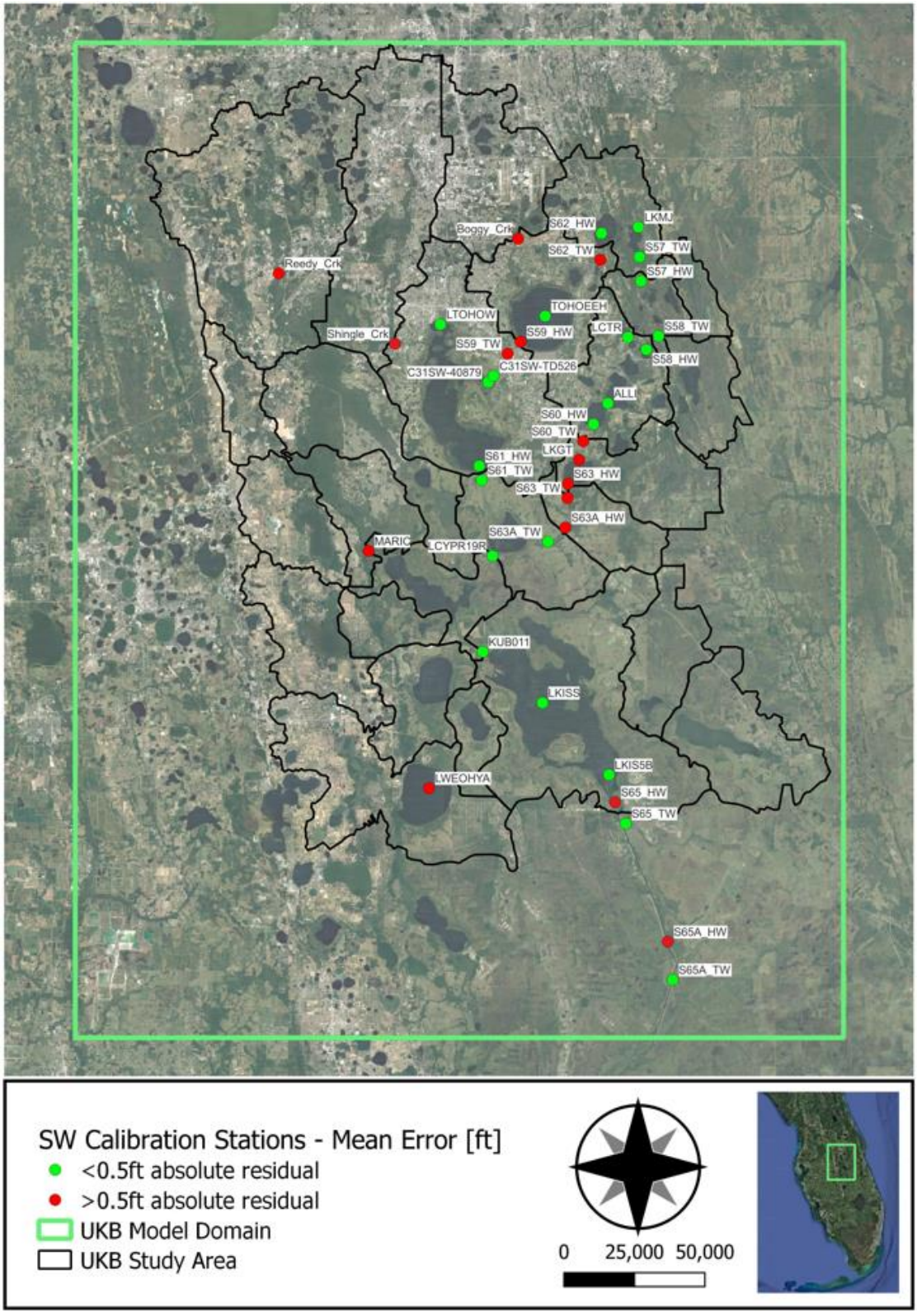

Figure 5-54: Mean Error [ft] Distribution in SW Calibration (9/10-10/10) with threshold of 0.5 ft

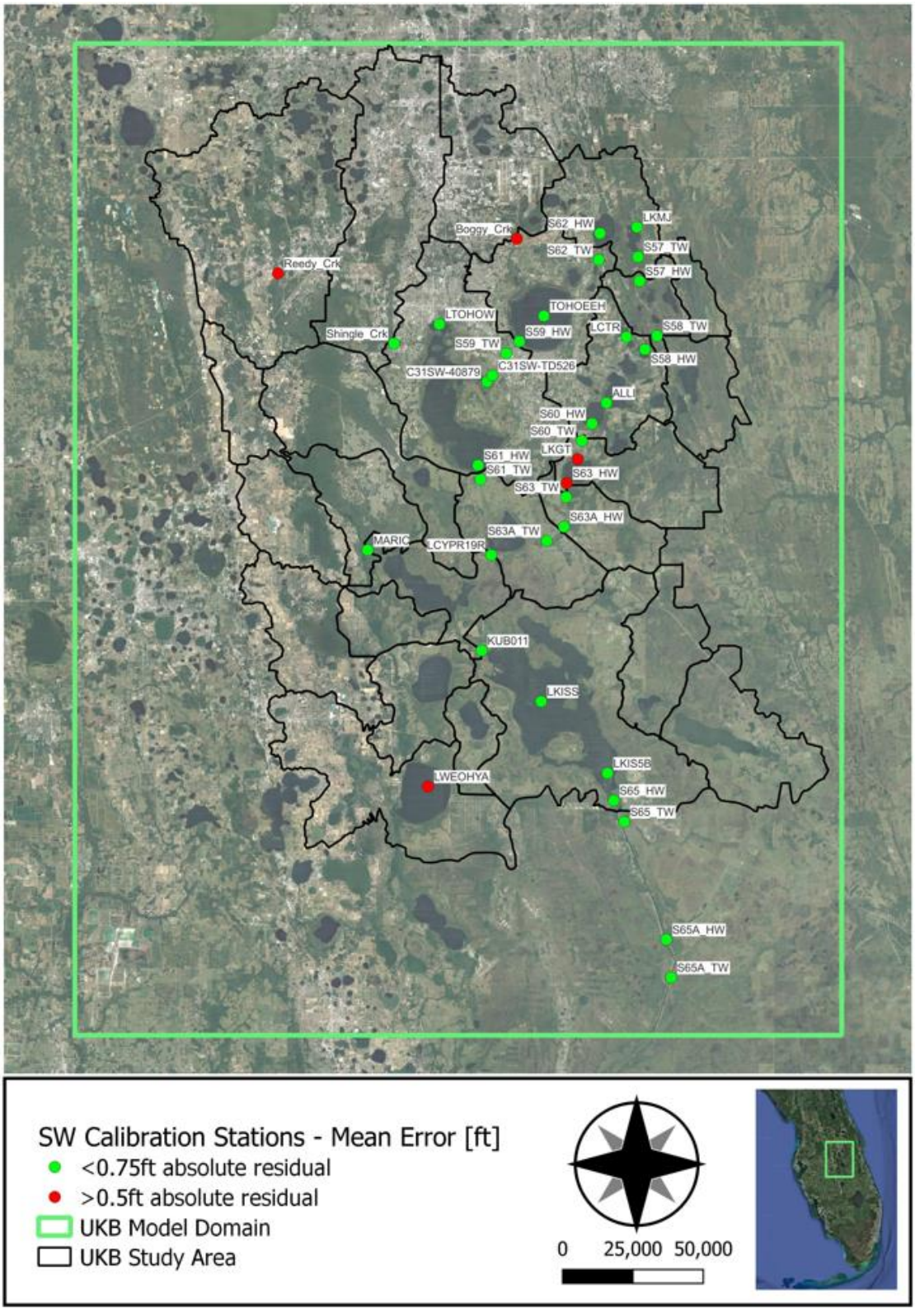

Figure 5-55: Mean Error [ft] Distribution in SW Calibration (9/10-10/10) with threshold of 0.75 ft

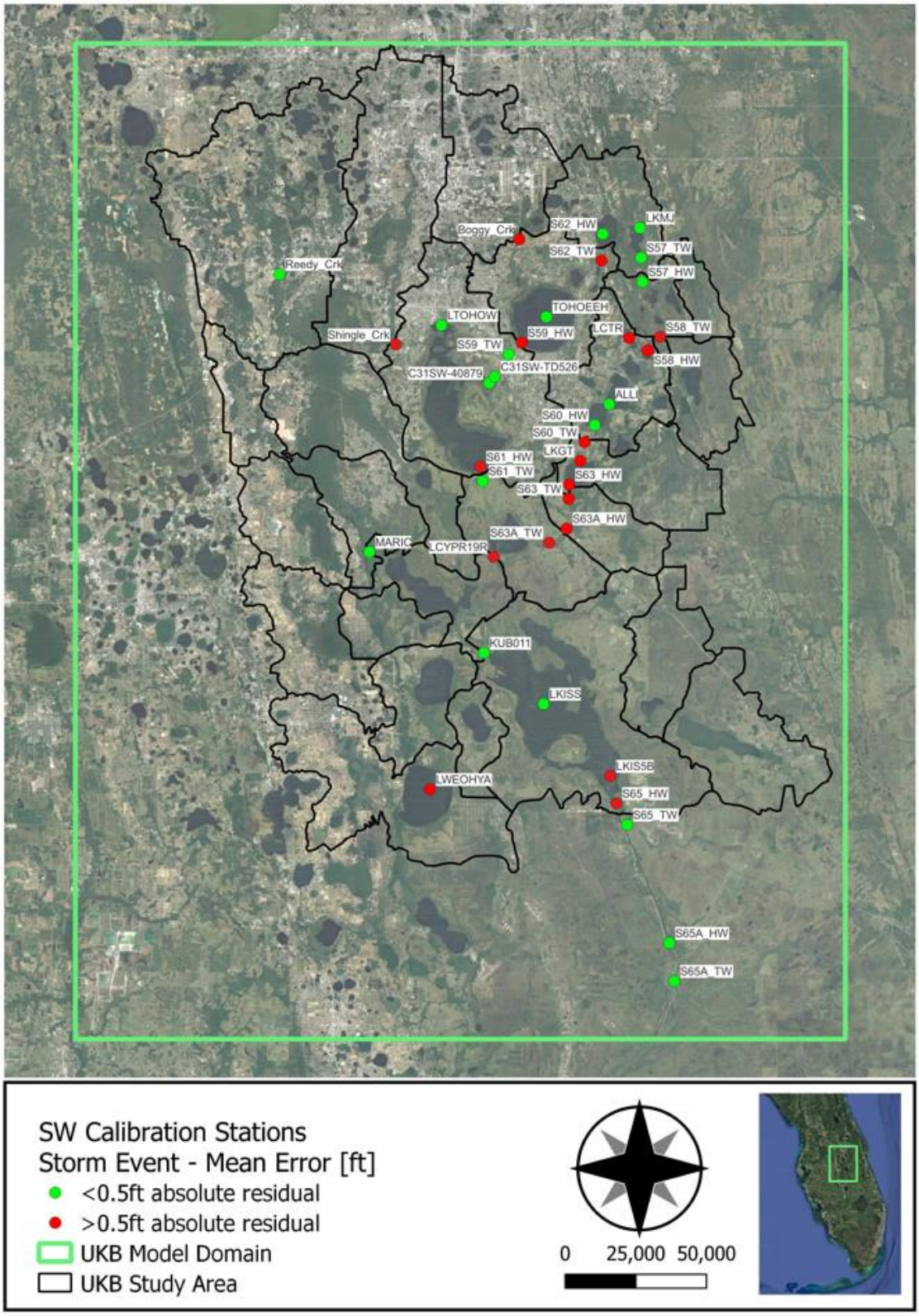

Figure 5-56: Mean Error [ft] Distribution in SW Calibration (9/10-9/18) with threshold of 0.5 ft

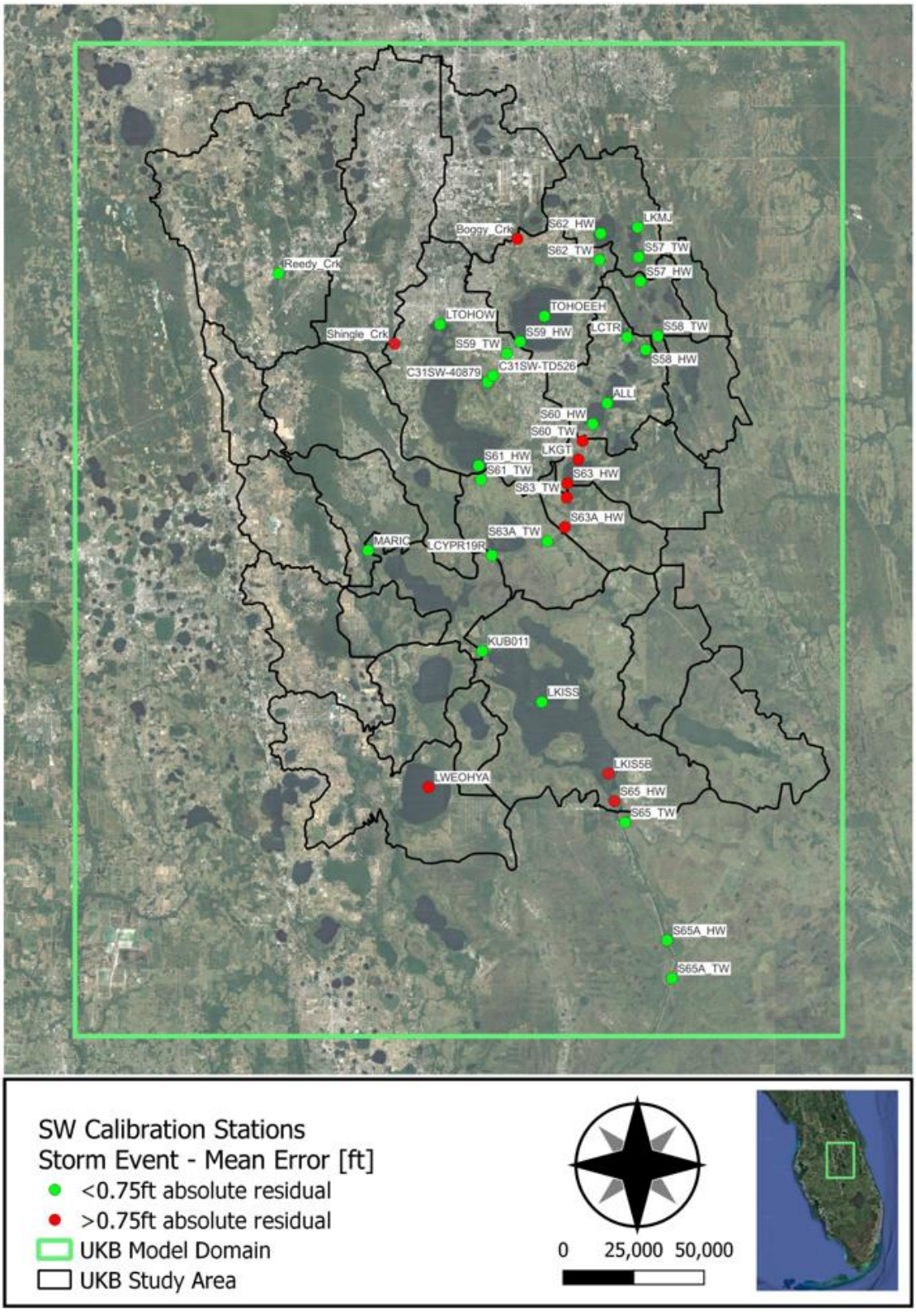

Figure 5-57: Mean Error [ft] Distribution in SW Calibration (9/10-9/18) with threshold of 0.75 ft

Table 5-13: Percentage of Time where Simulated Flows are within 20% of Observed Flow at Hourly Time Steps

Station	POS	Storm Event
G113	0%	2%
S65A	75%	15%
G103	No data	No data
S57	72%	63%
S58	100% Structure closed	100% Structure closed
S59	97%	89%
S60	26%	1%
S61	96%	89%
S62	77%	16%
S63	30%	14%
S63A	64%	39%
S65	61%	59%
Shingle Creek	1%	4%
Reedy Creek	19%	32%
Boggy Creek	28%	34%
KUB001	18%	2%

Table 5-14 shows the difference between the observed and simulated stages at peak. In terms of magnitude, the difference at peak stages between observed and simulated data is highest at S63A HW at 2.0 ft. Differences in peak flows and the time of occurrence is summarized in **Table 5-15** for SFWMD primary conveyance structures and the USGS creeks. The largest difference in magnitude as well as the longest difference in time for flows is at station Boggy Creek, at 2000 cfs and 0.6 days. Of the primary SFWMD structures, the largest magnitude difference between the simulated and observed flows is 1800 cfs at S65.

Hazen

Table 5-14: Peak Stage Comparison for SW Stations during POS

Calibration Station	Peak Stage (FT NAVD88)		Time of Peak Stage	
	Simulated	Observed	Simulated	Observed
G113_HW	56.3	57.3	9/12/2017 0:00	9/14/2017 0:00
G113_TW	56.3	56.8	9/11/2017 12:00	9/11/2017 12:00
S57_HW	63.1	63.2	9/20/2017 0:00	9/20/2017 0:00
S57_TW	60.8	61.3	9/17/2017 0:00	9/17/2017 0:00
S58_HW	63.3	63.5	9/15/2017 0:00	9/14/2017 20:00
S58_TW	63.5	63.2	9/15/2017 0:00	9/19/2017 12:00
S59_HW	57.4	58.2	9/19/2017 0:00	9/19/2017 0:00
S59_TW	56.6	57.3	9/20/2017 0:00	9/20/2017 0:00
S60_HW	62.2	62.6	9/16/2017 6:00	9/16/2017 6:00
S60_TW	60.9	61.5	9/16/2017 12:00	9/11/2017 8:00
S61_HW	54.8	55	9/17/2017 0:00	9/17/2017 0:00
S61_TW	54.1	54.1	9/17/2017 0:00	9/17/2017 0:00
S62_HW	60.8	61.2	9/18/2017 0:00	9/18/2017 0:00
S62_TW	58.3	59.1	9/16/2017 18:00	9/16/2017 18:00
S63_HW	60.3	60.5	9/17/2017 0:00	9/17/2017 0:00
S63_TW	Peak Unclear	57.5	Peak Time Unclear	9/11/2017 6:00
S63A_HW	Peak Unclear	57	Peak Time Unclear	9/11/2017 22:00
S63A_TW	54	53.7	9/16/2017 0:00	9/16/2017 0:00
S65_HW	52.7	52.6	9/14/2017 18:00	9/15/2017 8:00
S65_TW	50.8	50.6	9/20/2017 14:00	9/20/2017 14:00

Hazen

Table 5-15: Peak Discharge Comparison for SW Stations during POS

Calibration Station	Peak Discharge (CFS)		Time of Peak Discharge	
	Simulated	Observed	Simulated	Observed
G113_Flow	78	610	9/10/2017 18:00	9/10/2017 18:00
S57_Flow	265	Peak unclear	9/21/2017 0:00	-
S58_Flow	-100	-65	9/29/2017 0:00	9/26/2017 12:00
S59_Flow	1375	1450	9/20/2017 0:00	9/20/2017 0:00
S60_Flow	990	600	9/16/2017 6:00	9/16/2017 6:00
S61_Flow	3150	3150	10/4/2017 11:00	10/4/2017 11:00
S62_Flow	840	700	9/17/2017 22:00	9/17/2017 20:00
S63_Flow	1200	1050	9/16/2017 0:00	9/16/2017 0:00
S63A_Flow	1500	2220	9/11/2017 0:00	9/11/2017 0:00
S65_Flow	11600	13400	9/19/2017 17:00	9/19/2017 17:00
02264495-Q - Shingle	1600	2820	9/12/2017 0:00	9/13/2017 0:00
02266300-Q - Reedy	1480	2250	9/11/2017 22:00	9/11/2017 15:00
02262900-Q - Boggy	2180	4180	9/11/2017 12:00	9/12/2017 2:00

5.3.8.3.2 Groundwater

Figure 5-58 and **Figure 5-59** show spatially the ME statistics for groundwater. Analysis of the calibration statistics of groundwater shows 20% of groundwater stations had a ME of ±0.5 ft; if a threshold of 1 ft is used for ground water stages, 57% of the stations met the ME criterion and 4% of the stations met the R with an average value of 0.9.

Statistics were also computed for groundwater stations in the UKB FPLOS Study area for the calibration model, focused on the storm event. 13% of groundwater stations had a ME of ±0.5 ft and 9% of the stations had an R of 0.9. Using a threshold of 1 ft for the groundwater stations, 54% and 9% of the stations met the ME criterion and had a R of 0.9. These are shown spatially in **Figure 5-60** and **Figure 5-61**.

Table 5-16 shows the comparison of observed and simulated peak groundwater levels in response to Hurricane Irma at SFWMD primary structures. The average difference between the peaks of observed and simulated groundwater levels was -0.6 ft.

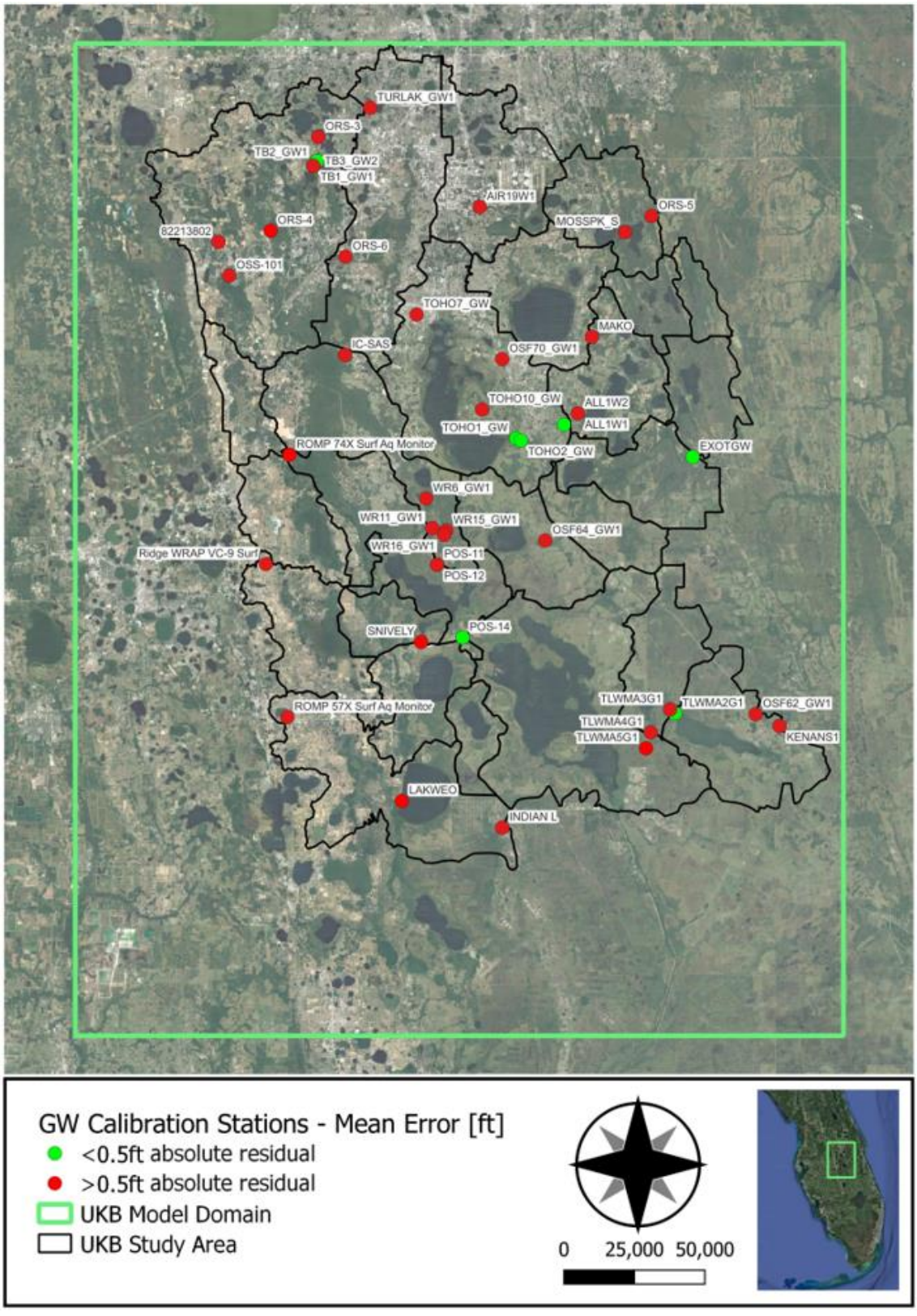

Figure 5-58: Mean Error [ft] Distribution in GW Calibration (9/10-10/10) with threshold of 0.5 ft

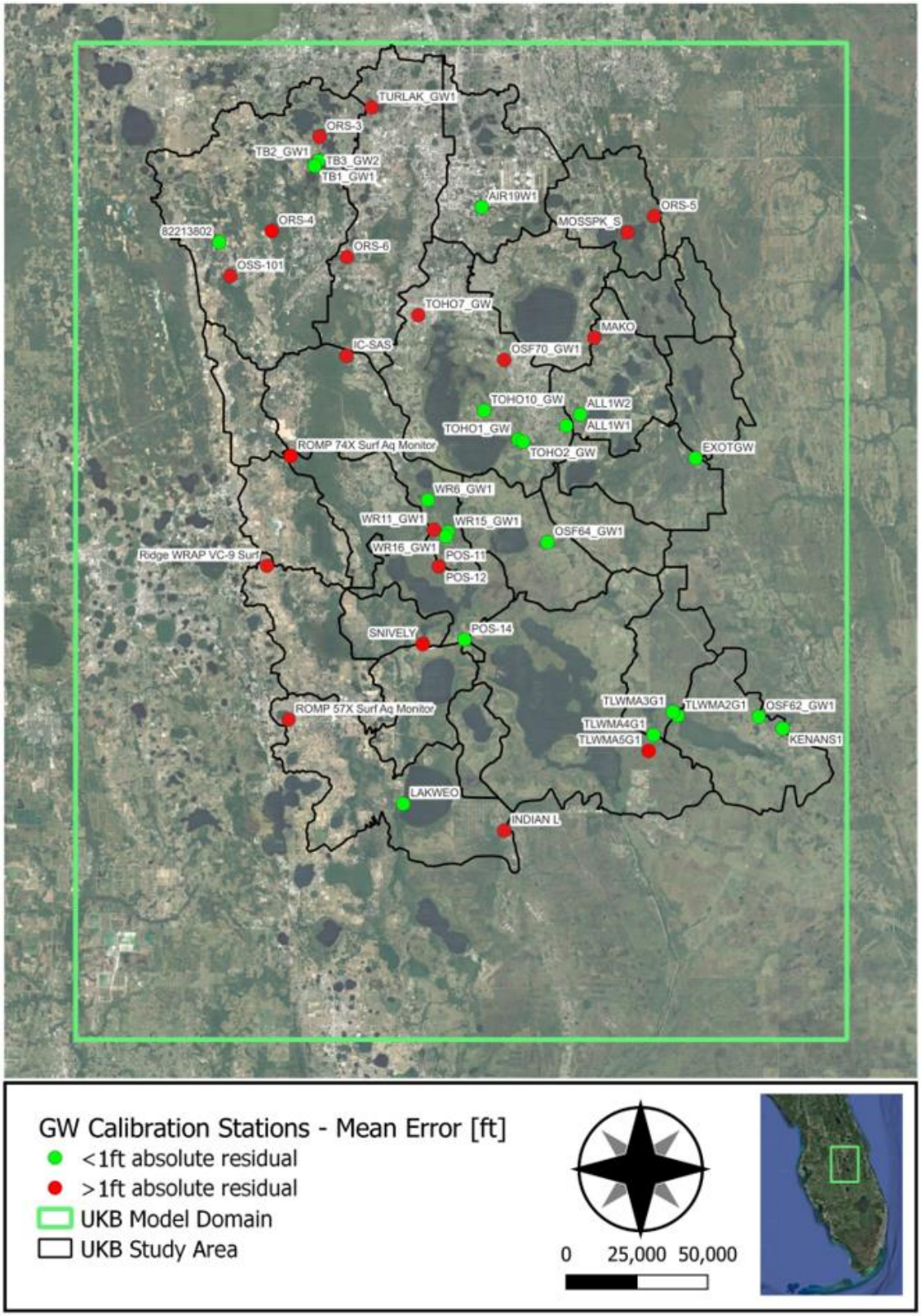

Figure 5-59: Mean Error [ft] Distribution in GW Calibration (9/10-10/10) with threshold of 1 ft

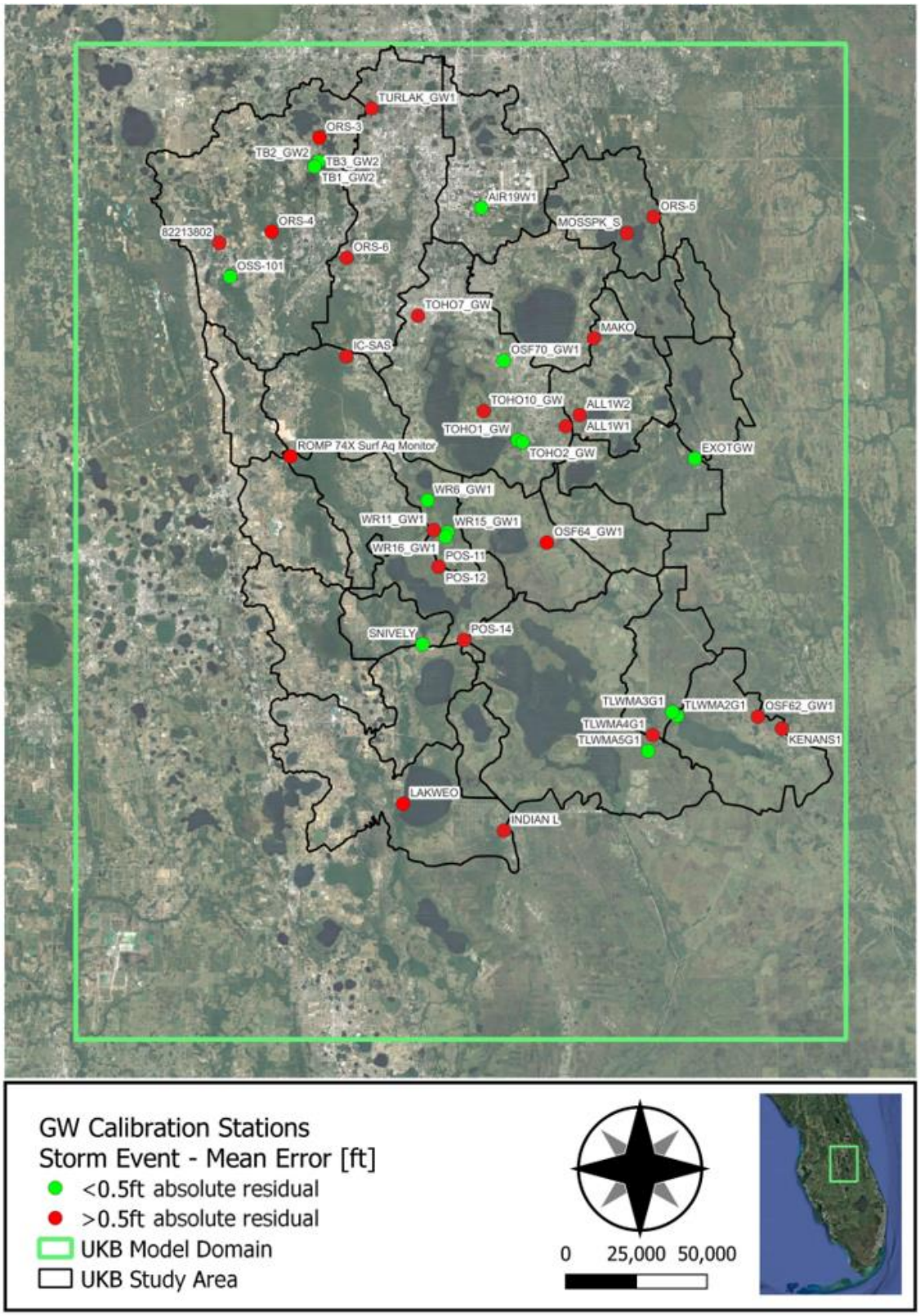

Figure 5-60: Mean Error [ft] Distribution in GW Calibration (9/10-9/18) with threshold of 0.5 ft

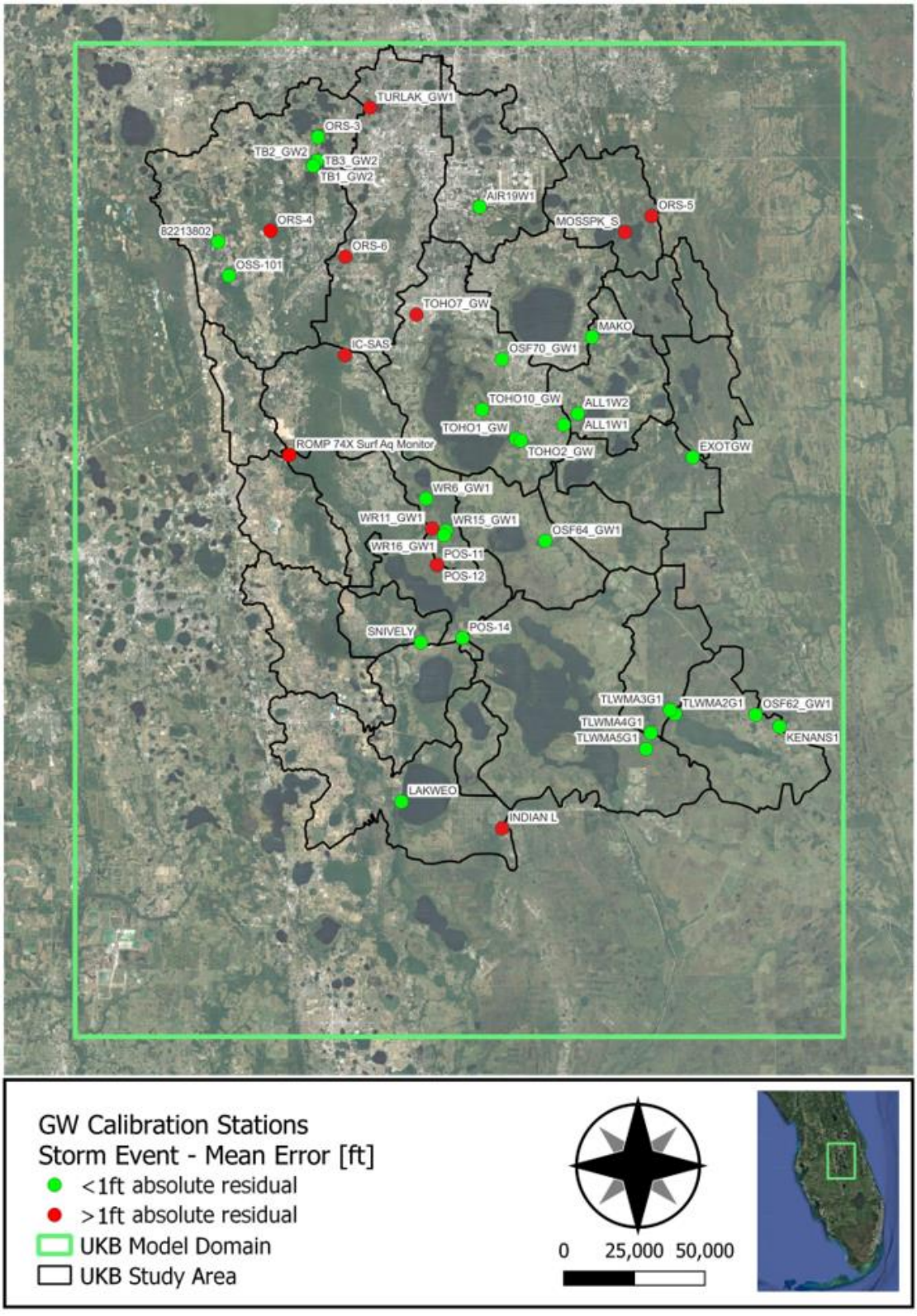

Figure 5-61: Mean Error [ft] Distribution in GW Calibration (9/10-9/18) with threshold of 1 ft

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Table 5-16: Peak Water Levels Comparison for GW Stations during POS

Calibration Station	Peak Stage (FT NAVD88)	
	Simulated	Observed
82213802	115.8	114
MOSSPK_S	67.5	62
AIR19W1	Peak Unclear	Peak Unclear
ORS-3	Peak Unclear	102.8
ORS-6	94.1	93
ORS-7	94.1	93
ORS-5	66.6	64
ORS-4	88.7	95.5
ALL1W1	72	72
ALL1W2	72	72
EXOTGW	70.5	70.3
MAKO	76.8	75.4
OSF62_GW1	69.2	68.8
OSF64_GW1	55.4	54
OSF70_GW1	59.5	59
IC-SAS	Peak Unclear	Peak Unclear
KENANS1	75.1	74.2
OSS-101	Peak Unclear	104.4
WR6_GW1	64.5	64
WR9_GW1	68.6	66.5
WR11_GW1	68	66.2
WR15_GW1	Peak Unclear	Peak Unclear
TOHO1_GW	63.5	63
TOHO2_GW	64.8	64.6
TOHO10_GW	70.1	69.8
TOHO15_GW	74.9	74.2

Calibration Station	Peak Stage (FT NAVD88)	
	Simulated	Observed
TOHO7_GW	69.8	68.8
TB2_GW1	101.7	100.7
TB2_GW2	101.8	100.1
TB3_GW1	Peak Unclear	Peak Unclear
TB3_GW2	Peak Unclear	Peak Unclear
TB1_GW1	100.8	100.5
TB1_GW2	100.9	100.5
TURLAK_GW1	Peak Unclear	Peak Unclear
TLWMA2G1	61.2	60.7
TLWMA3G1	Peak Unclear	Peak Unclear
TLWMA4G1	Peak Unclear	Peak Unclear
TLWMA5G1	58.6	58.7
POS-14	Peak Unclear	Peak Unclear
WR16_GW1	65.3	65
INDIAN L	119.3	117.5
POS-11	64.5	65
POS-12	64.5	62.5
LAKWEO	62.8	63.5
SNIVELY	63.3	62.8
Ridge WRAP VC-9 Surf	Peak Unclear	Peak Unclear
ROMP 57X Surf Aq Monitor	Peak Unclear	Peak Unclear
ROMP 74X Surf Aq Monitor	90.2	91.5

5.3.8.4 Summary of Calibration Status

Several statistical measures were used to describe the simulated data in comparison to the observed data, to evaluate the calibration. For the surface water stages, 62% met the criteria of +/- 0.5 ft ME during POS. 87% met the +/- 0.75 ft ME criterion during POS. For the primary stations, R was higher than 0.5 for all. For the surface water flows during the POS, all spillways except S60 and S63 met the

flow criterion of +/- 20% more than 50% of the time. All the USGS creeks met the flow criterion for less than 50% of the time.

Analysis of the calibration statistics of groundwater showed 20% of groundwater stations with a ME of ±0.5 ft and 57% of the stations met the ME criterion of ±1 ft.

5.3.9 Validation Results

The calibrated model was simulated using a second independent storm event – October No-Name Storm of 2011. Reasonable model performance using an independent storm event validated the adequacy of the UKB FPLOS model setup and parameterization. SFWMD structures used observed operations in the form of observed gate levels/openings, where available. Initial and boundary conditions for surface and groundwater were from observed DBHYDRO data.

The No-Name storm of October 2011 was used to validate the UKB FPLOS model. This storm exceeded the 100-year rainfall with a duration of 5-days for the Kissimmee Basin. This has resulted in the overwhelming of the drainage system right after the event. For validation, several input datasets were modified such as rainfall, initial and boundary conditions, as well as observed stages and flows. Validation results were presented for the POS of 09/29/2011 – 10/27/2011 and then for the storm event for a period of 10/03/2011 – 10/27/2011. The analysis period of this event is longer than that of the calibration event because the recession part of the storm event hydrograph was longer for this event.

5.3.9.1 Analysis of Stage Gradient during October No-Name Storm of 2011

A method to evaluate the calibration of the UKB FPLOS model is take a snapshot in time of KCOL during the storm's progression, and compare it with the model's performance during a different storm. The observed and simulated head gradient across the entire KCOL system during the 2011 No-Name storm event is captured in **Figure 5-62** and **Figure 5-63**.

The plots reveal that while the gradient remains fairly flat before the storm, it dramatically rises to around 2 feet as the storm approaches. The modeled results aligned fairly well with this striking change in observed data.

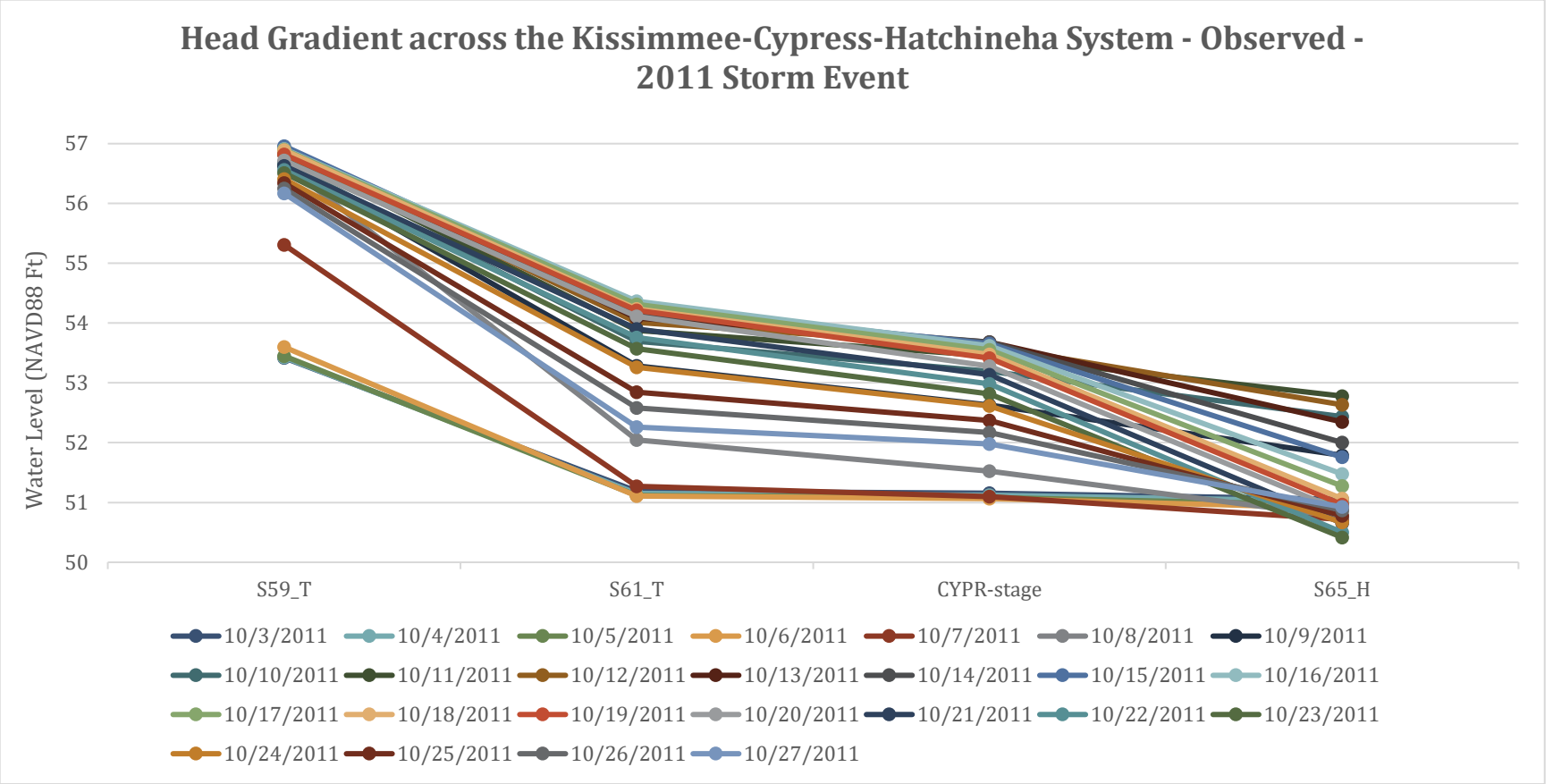

Figure 5-62: Observed Stage Gradient (ft NAVD88) across KCH after October 2011 No-Name Storm

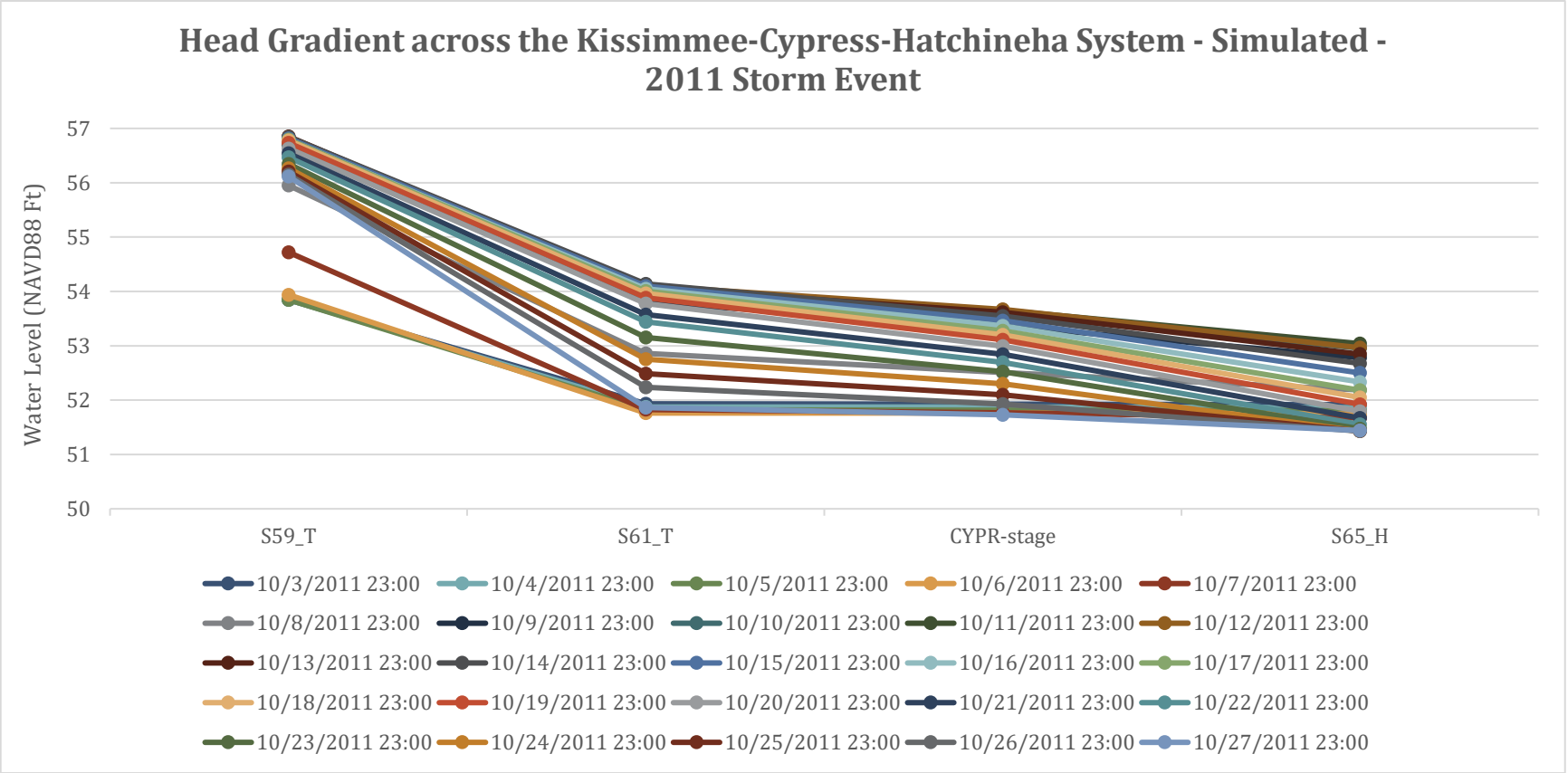

Figure 5-63: Simulated Stage Gradient (ft NAVD88) across KCH after October 2011 No-Name Storm

5.3.9.2 Water Balance

The water budgets were checked for reasonableness in comparison with back-calculated watershed numbers from prior events. The water balance computed for each watershed for the validation model is summarized in **Appendix I**. Detailed water budgets were developed for the UKB FPLOS watersheds. **Figure 1-3** shows the model watersheds areas used to extract the water budget data. The water budget analysis includes summary of the water balances for all four model subdomains: OL – Two-dimensional Overland Flow, UZ – One dimensional flow in the Unsaturated Zone, SZ – Three-dimensional flow in Saturated Zone and OC – Open Channel and hydraulics (canals, water control structures).

The water balance over the MIKE SHE domain in inches is summarized for the validation events; MIKE HYDRO items are treated as boundary flows and are included in 'Base flow from/to River', OL to River', and 'SZ Drain to/from River'. A reasonable check of the water balance indicates ET, overland to river and saturated zone drains to river are the major drivers during the storm event, as was the case with the calibration model.

5.3.9.3 Validation Statistics for POS

The UKB hydrology, being a managed system, is best understood in the context of the lakes. **Table 5-11** is a tabular description of each LMA in UKBFPLOS Model domain, and their inflows and outflows. The status of calibration was assessed around the ability of the model to simulate the hydrology of the LMAs. The validation of the calibration model was also evaluated in the same context.

The validation metrics were computed for a subset of the POS: 09/29/2011 to 10/27/2011. Statistics were also computed for another subset focused on the storm event October No-Name Storm of 2011, for 10/03/2011 to 10/27/2011. Validation results are presented here for each LMA and further statistics can be found in the appendices.

The results of calibration for surface water stages are presented in the following formats:

1. Spatial plots of the ME statistics in this section (surface water and groundwater stages and flows plots for the validation period are attached in Appendix F)
2. Tabular format of all the MIKE SHE generated validation statistics for 09/29/2011 to 10/27/2011 and 10/03/2011 – 10/27/2011 (Appendix G)
3. For each LMA, stage and flow calibration statistics are summarized in this section

Methodologies used in computation of surface water flows, and the validation statistics is the same as in calibration statistics as described in the calibration section. Validation statistics for groundwater stations are presented in the following formats: Spatial plots of the ME statistic.

5.3.9.3.1 Surface Water

Plots of the SW ME statistic are shown in **Figure 5-72** and **Figure 5-73** which depict where a residual of 0.5 ft and 0.75 ft are met spatially for 09/29/2011 – 10/27/2011. **Figure 5-74** and **Figure 5-75** show spatially the same for 10/03/2011 – 10/27/2011. **Table 5-20** shows the percentage of time during the simulation the modeled flows are within 20% of the observed flows. Validation statistics are summarized below for each LMA.

Alligator Lake LMA

Surface water management in this LMA include S58 and S60 structures, both are outflow structures. In the validation simulations, S58 HW and S58 TW had a ME of [0.6ft, 0.5ft] and R of [0.9, 0.8] respectively. The UKBFPLOS model had a ME of [0.6, 0.8] for S60 HW and S60 TW and R of [0.9, 0.9]. The validation modeled flows at S60 were within 20% of the observed values for 12% of the time. S58 was kept closed during the simulation (refer to **Figure 5-64** for validation plots).

For October No-Name Storm, S58 HW and S58 TW had a ME of [0.5ft, 0.6ft] and R of [0.9, 0.8] respectively. ME of [0.6ft, 0.8ft] and R of [0.9, 0.9] describes S60 HW and S60 TW.

Myrtle LMA

This LMA has S58 and S57 as surface water inflow and outflow structures. S57 HW and S57 TW had a ME of [-0.0ft, -1.2ft] and a R of [0.9, 0.9] respectively. Simulated flows at S57 during the POS were within 20% of the observed flows for 43% of the time. S58 was closed during the simulation (refer to **Figure 5-64** and **Figure 5-65** for validation plots).

For the storm event, S57 HW and S57 TW had a ME of [0.0ft, -1.2ft] and a R of [0.9, 0.9] respectively. As for surface water flows, simulated flows at S57 during the storm event were within 20% of observed flows for 50% of the time.

Hart LMA

Inflow and outflow structures in this LMA are S57 and S62. S62 HW and S62 TW had an ME of [-0.8ft, 0.5ft] and R of [0.9, 0.9] respectively. S62 simulated flows were within the 20% margin of the observed flows for 48% of the POS (refer to **Figure 5-65** for validation plots).

For October No Name Storm of 2011, S62 HW and S62 TW had an ME of [-0.9ft, 0.5ft] and R of [0.9, 0.9] respectively. S62 simulated flows were within the 20% margin of the observed flow during the storm event for 48% of the time.

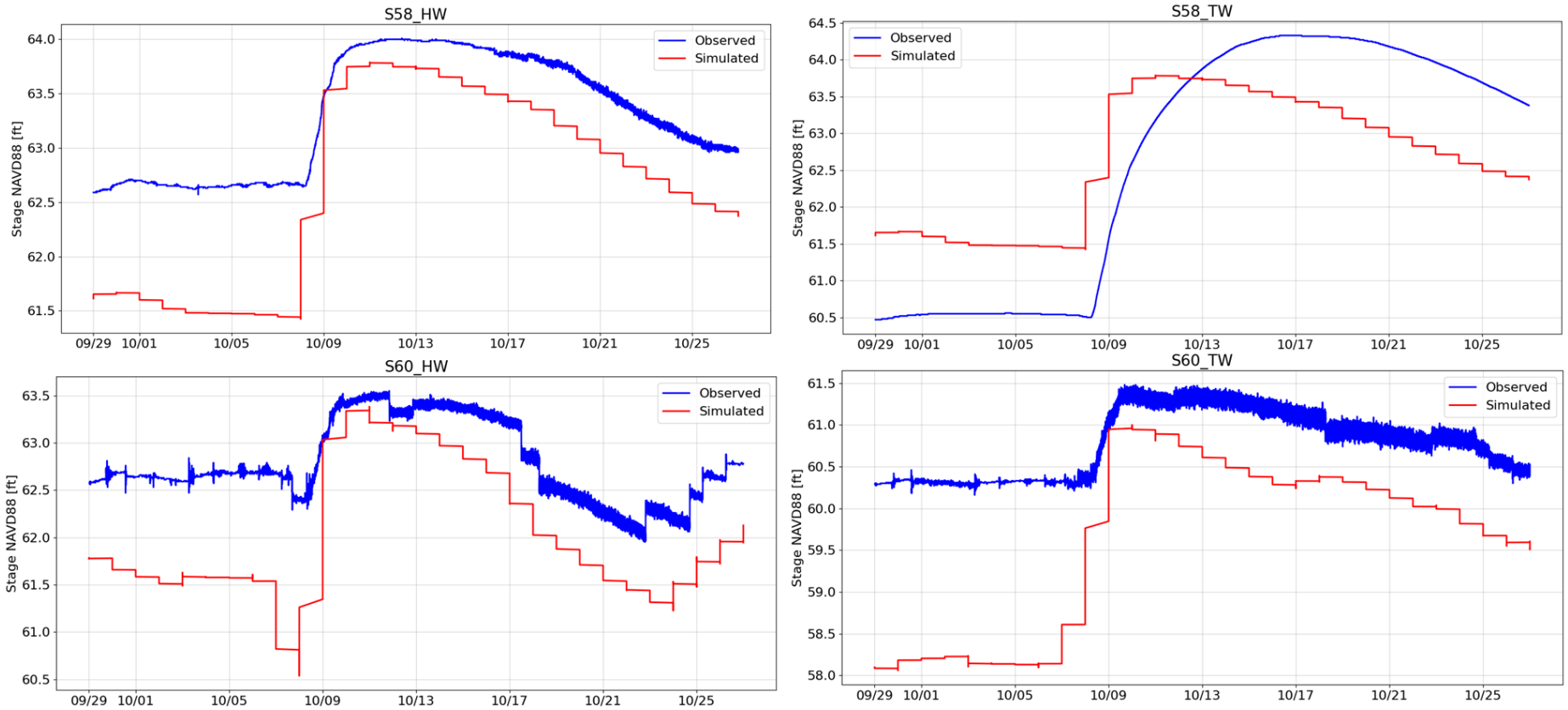

Figure 5-64: S58 and S60 Observed and Simulated Stages during 2011 October No-Name Storm

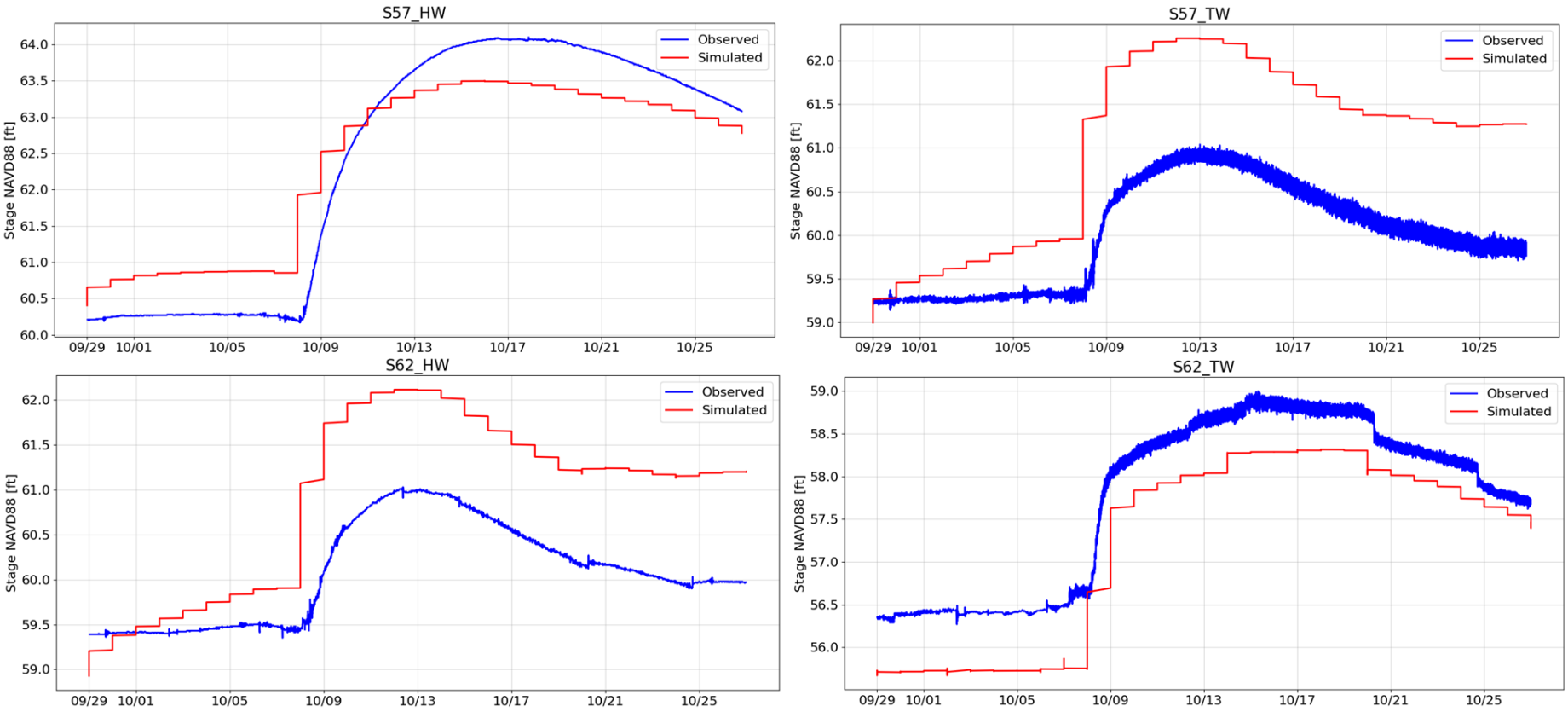

Figure 5-65: S57 and S62 Stages Observed and Simulated Stages during 2011 October No-Name Storm

Gentry LMA

In this LMA S60, S63, and S63A were used as validation stations; with inflows into Lake Gentry from S60 and outflows through S63; and then through S63A, which is downstream of S63. S63 HW and S63 TW simulated stages had ME of [1.4ft, 0.7ft] and R of [0.9, 0.4] respectively. ME of [0.9ft, -0.4ft] and R of [0.6, 0.9] are what simulated stages had for S63A HW and S63A TW respectively. During the POS, S60, S63, S63A surface water flows met the 20% flow criterion for 12%, 64%, and 63% of the time (refer to **Figure 5-66** and **Figure 5-67** for validation plots).

S63 HW and S63 TW simulated stages had ME of [1.3ft, 0.7ft] and R of [0.9, 0.3] respectively during the storm event. At S63A HW and S63A TW, the calibrated model had a ME of [0.8ft, -0.4ft] and R of [0.6, 0.9]. During the storm event, S60, S63, S63A surface water flows were 7%, 64%, and 74% of the time within 20% of their respective observed flows.

East Toho LMA

S62 and S59 structures are at the inlet and outlet of E. Toho LMA. S59 HW and S59 TW had ME of [0.6ft, 0.1ft] and R of [0.9, 0.9] respectively. S59 simulated flows were, for 74% of the time, within 20% of the observed S59 flows. Hart LMA's description above has details on S62 validation statistics (refer to **Figure 5-65** and **Figure 5-67** for validation plots).

For October No Name Storm of 2011, S59 HW and S59 TW had ME of [0.6ft, 0.1ft] and R of [0.9, 0.9] respectively. S59 simulated flows were within 20% of the observed flow 40% of the simulation time.

Toho LMA

The surface water hydrology in Toho LMA is regulated by S59 and S61 structures. S61 HW and S61 TW had ME of [-0.6ft, -0.1ft] and R of [0.9, 0.9] respectively. During 57% of the time, S61 simulated flows were within the 20% of the observed flows (refer to **Figure 5-67** and **Figure 5-68** for validation plots).

For October No Name Storm, S61 HW and S61 TW have ME of [-0.6ft, -0.1ft] and R of [0.9, 0.9] respectively. During 54% of the time, simulated S61 flows were within the 20% margin of the observed flows.

Cypress LMA

S61 and S63A structures inflow into Cypress LMA. There are no outflow structures in this LMA but there is a stage recorder LCYPR19R at the outlet in C36 Canal. Validation statistics of S61 and S63A are included in Gentry LMA and Toho LMA. LCYPR19R had a ME of -0.3ft and R of 0.9 (refer to **Figure 5-67** and **Figure 5-68** for validation plots).

LCYPR19R for the October 2011 No Name event had a ME of -0.2ft and R of 0.9.

Hatchineha LMA

Hatchineha has no inflow structures; however, at the inlet of this LMA are stage validation stations MARIC and LCYPR19R, and KUB011 is at the downstream end of the LMA. MARIC has ME of -

0.6ft and R of 0.9. KUB011 which is also at the inlet of the Kissimmee LMA, had a ME of -0.3ft and R of 0.9.

During the storm event, MARIC had ME of -0.5 ft and R of 0.9. KUB011 which is at the inlet of the Kissimmee LMA, had a validation ME of -0.3ft and R of 0.9.

Kissimmee LMA

In this LMA KUB011 and S65 are the monitoring stations used for validation; KUB011 is a stage recorder with observed stages and S65 is the outlet of Lake Kissimmee and is the outflow structure of the entire UKB. S65 HW and S65 TW had ME of [-1.0ft, -1.1ft] and R of [0.9, 0.9] respectively. During the validation time period, S65 simulated flows were within the 20% margin of the S65 observed flows for 63% of the time (refer to **Figure 5-68** for validation plots).

For the storm event, S65 HW and S65 TW had ME of [-1.0ft, -0.9ft] and R of [0.9, 0.9] respectively. S65 simulated flows are within the 20% margin of the observed flows for 72% of the time.

Table 5-17: Peak Stage Comparison for SW Stations during Validation Period

Calibration Station	Peak Stage (FT NAVD88)		Time of Peak Stage	
	Simulated	Observed	Simulated	Observed
G113_HW	57	59.8	10/11/2011 0:00	10/11/2011 0:00
G113_TW	57	59.5	10/11/2011 12:00	10/11/2011 12:00
S57_HW	63.5	64	10/18/2011 0:00	10/18/2011 0:00
S57_TW	62.3	61	10/13/2011 0:00	10/13/2011 0:00
S58_HW	63.8	64	10/12/2011 0:00	10/12/2011 0:00
S58_TW	63.8	64.3	10/12/2011 12:00	10/18/2011 0:00
S59_HW	57.5	57.7	10/16/2011 0:00	10/16/2011 0:00
S59_TW	56.8	57	10/14/2011 0:00	10/14/2011 0:00
S60_HW	63.4	63.5	10/11/2011 0:00	10/11/2011 0:00
S60_TW	61	61.3	10/10/2011 0:00	10/9/2011 20:00
S61_HW	55.7	55.3	10/14/2011 0:00	10/14/2011 10:00
S61_TW	54.3	54.4	10/16/2011 0:00	10/16/2011 0:00
S62_HW	62.2	61	10/13/2011 0:00	10/13/2011 0:00
S62_TW	58.3	58.9	10/14/2011 19:00	10/16/2011 0:00
S63_HW	60.5	61.2	10/10/2011 0:00	10/9/2011 18:00

Calibration Station	Peak Stage (FT NAVD88)		Time of Peak Stage	
	Simulated	Observed	Simulated	Observed
S63_TW	57.1	57.2	Peak Time Unclear	Peak Time Unclear
S63A_HW	57	57	Peak Time Unclear	Peak Time Unclear
S63A_TW	54.2	54	10/9/2011 12:00	10/15/2011 4:00
S65_HW	53.1	52.8	10/12/2011 0:00	10/12/2011 0:00
S65_TW	51.4	50.8	10/16/2011 0:00	10/16/2011 0:00

Table 5-17 shows the difference between the observed and simulated stages at peak. In terms of magnitude, the difference at peak stages between observed and simulated data was highest at G113 HW at 2.8ft. Differences in peak flows and the time of occurrence is summarized in **Table 5-18** for SFWMD primary conveyance structures and the USGS creeks. The largest difference in magnitude as well as the longest difference in time for flows is at Boggy Creek stations (2700 cfs) and S61(5.5 days) respectively. Of the primary SFWMD structure, the largest magnitude difference between the simulated and observed flows was 1100 cfs at S65.

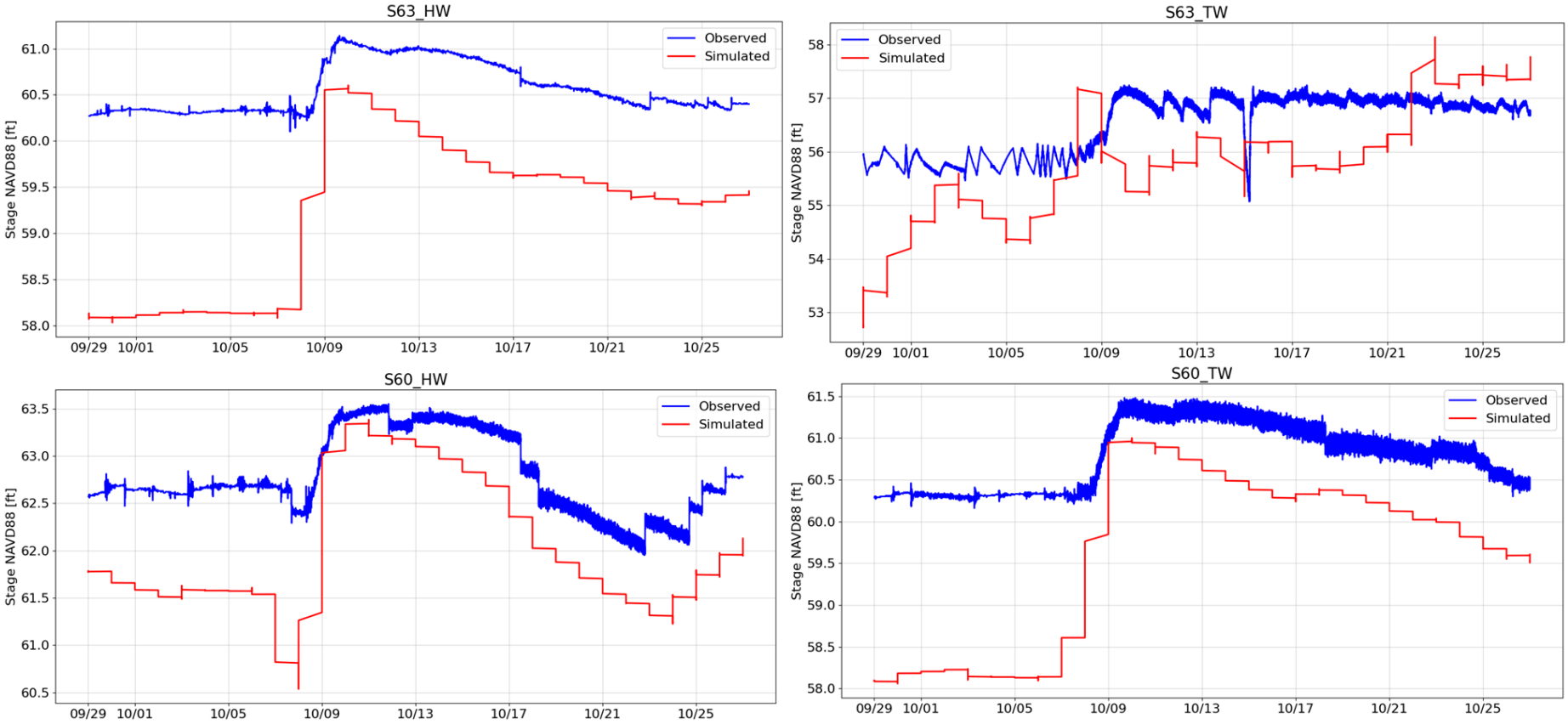

Figure 5-66: S63 and S60 Observed and Simulated Stages during 2011 October No-Name Storm

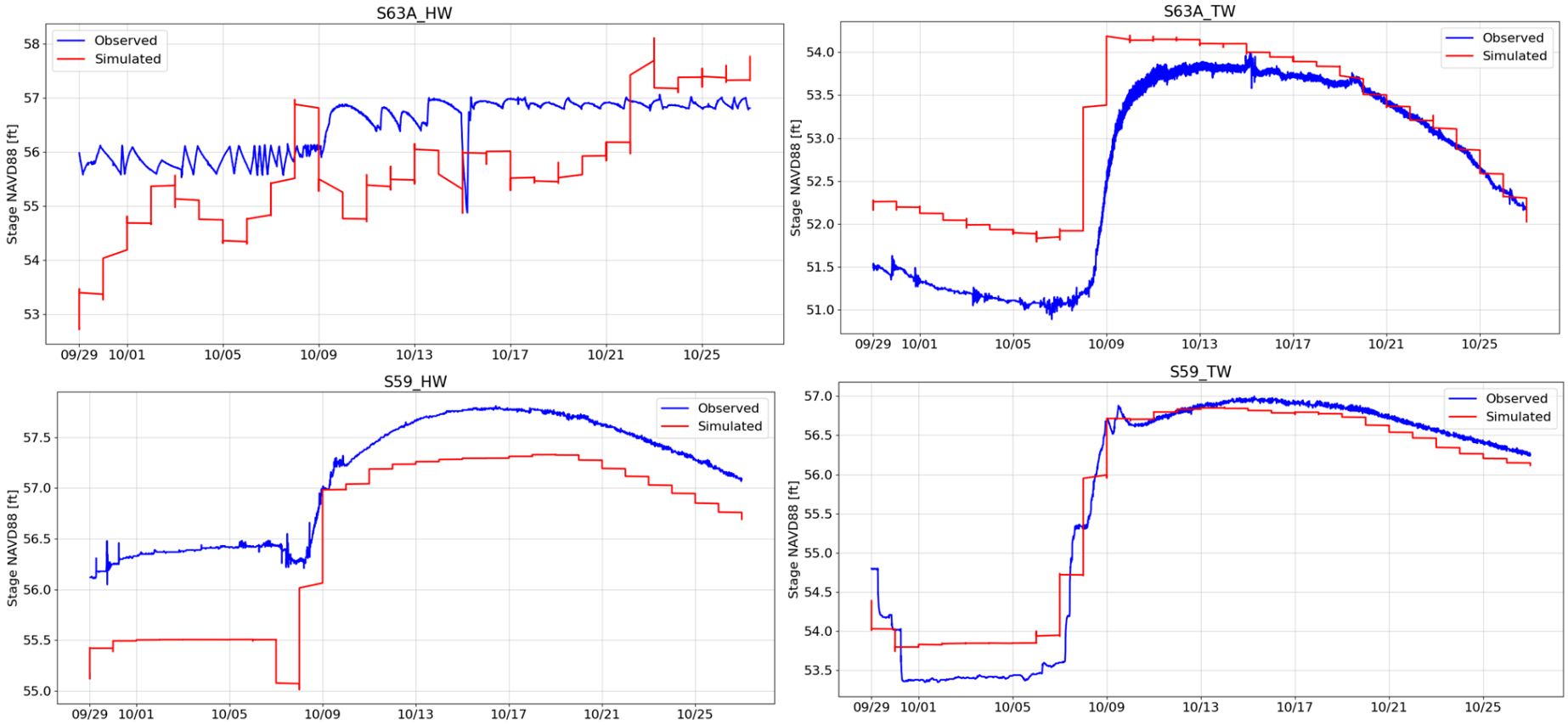

Figure 5-67: S63A and S59 Observed and Simulated Stages during 2011 October No-Name Storm

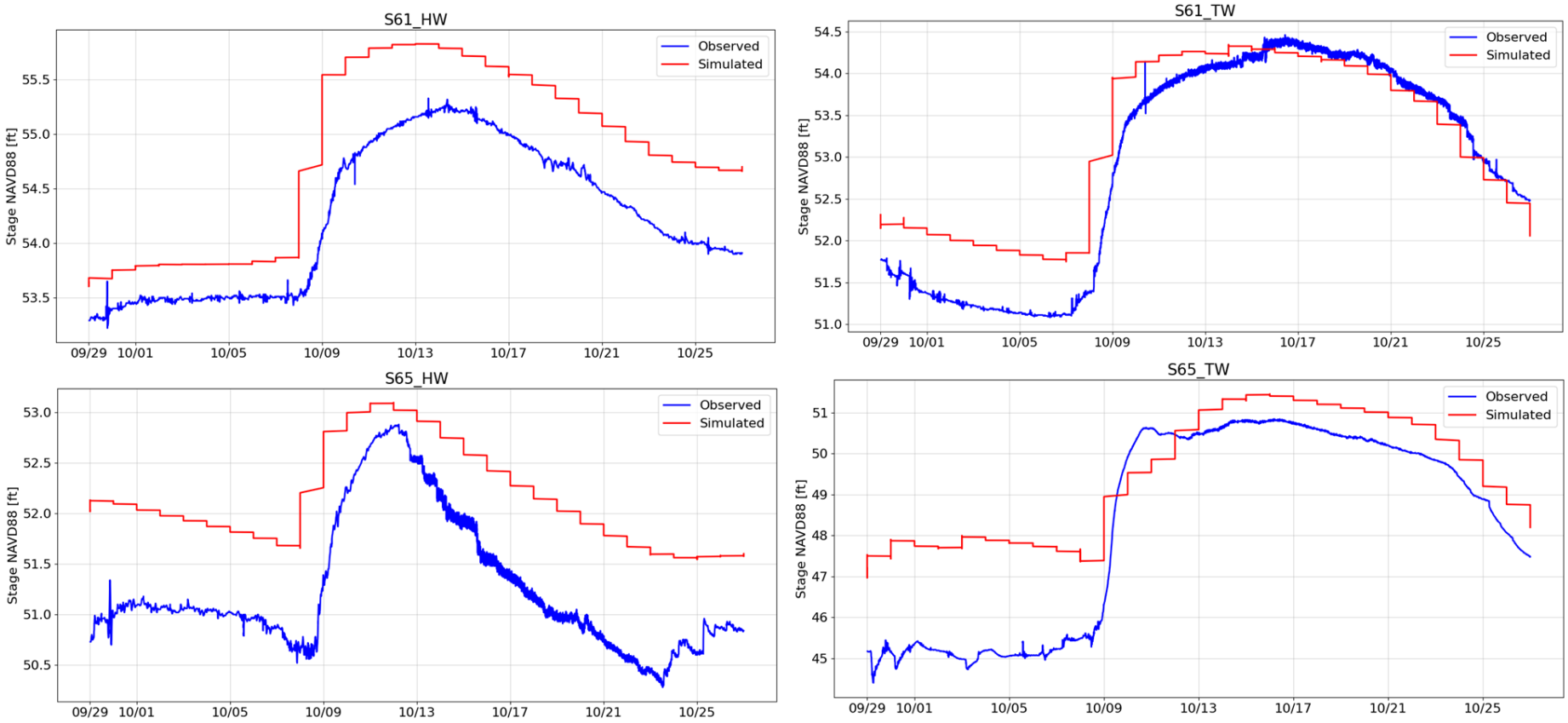

Figure 5-68: S61 and S65 Observed and Simulated Stages during 2011 October No-Name Storm

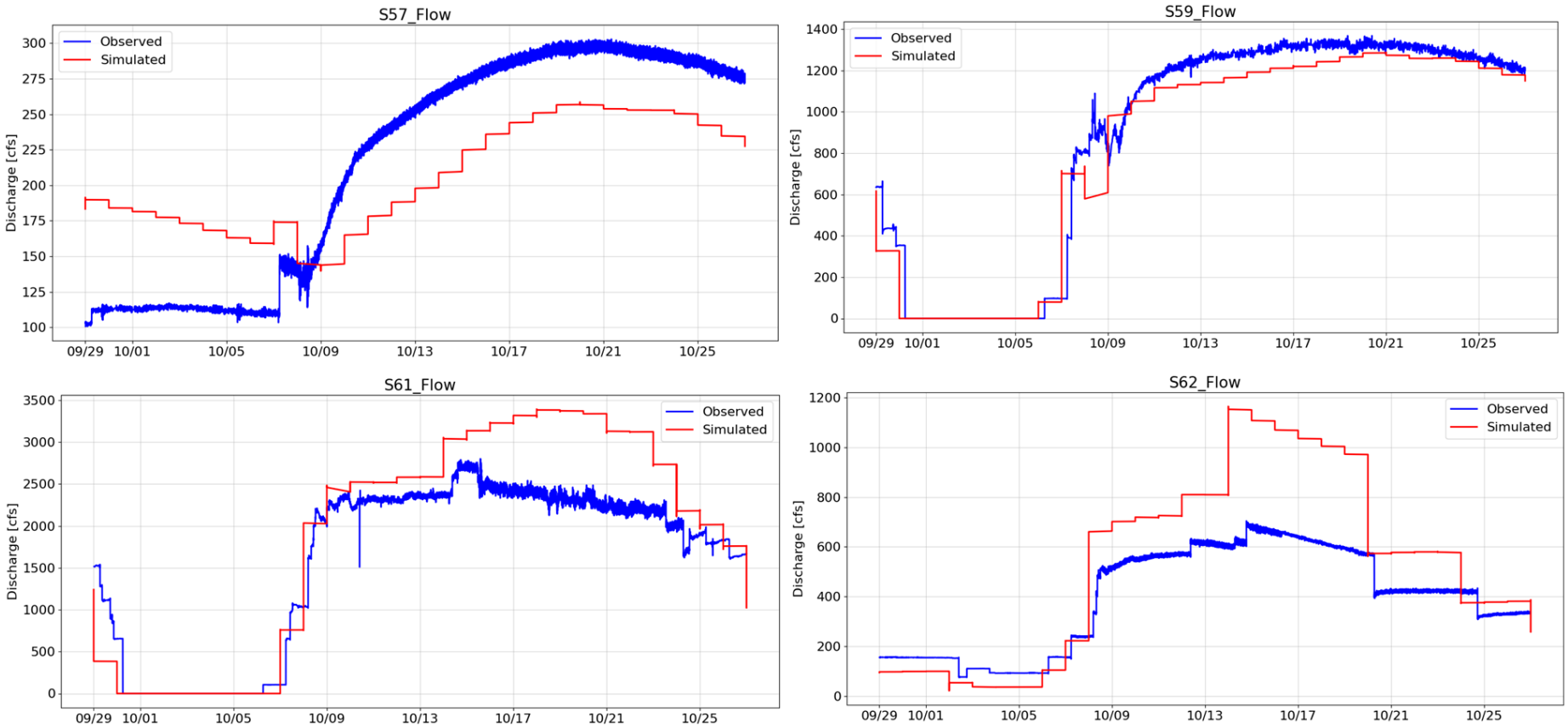

Figure 5-69: S57, S59, S61, and S62 Observed and Simulated Discharges during 2011 October No-Name Storm

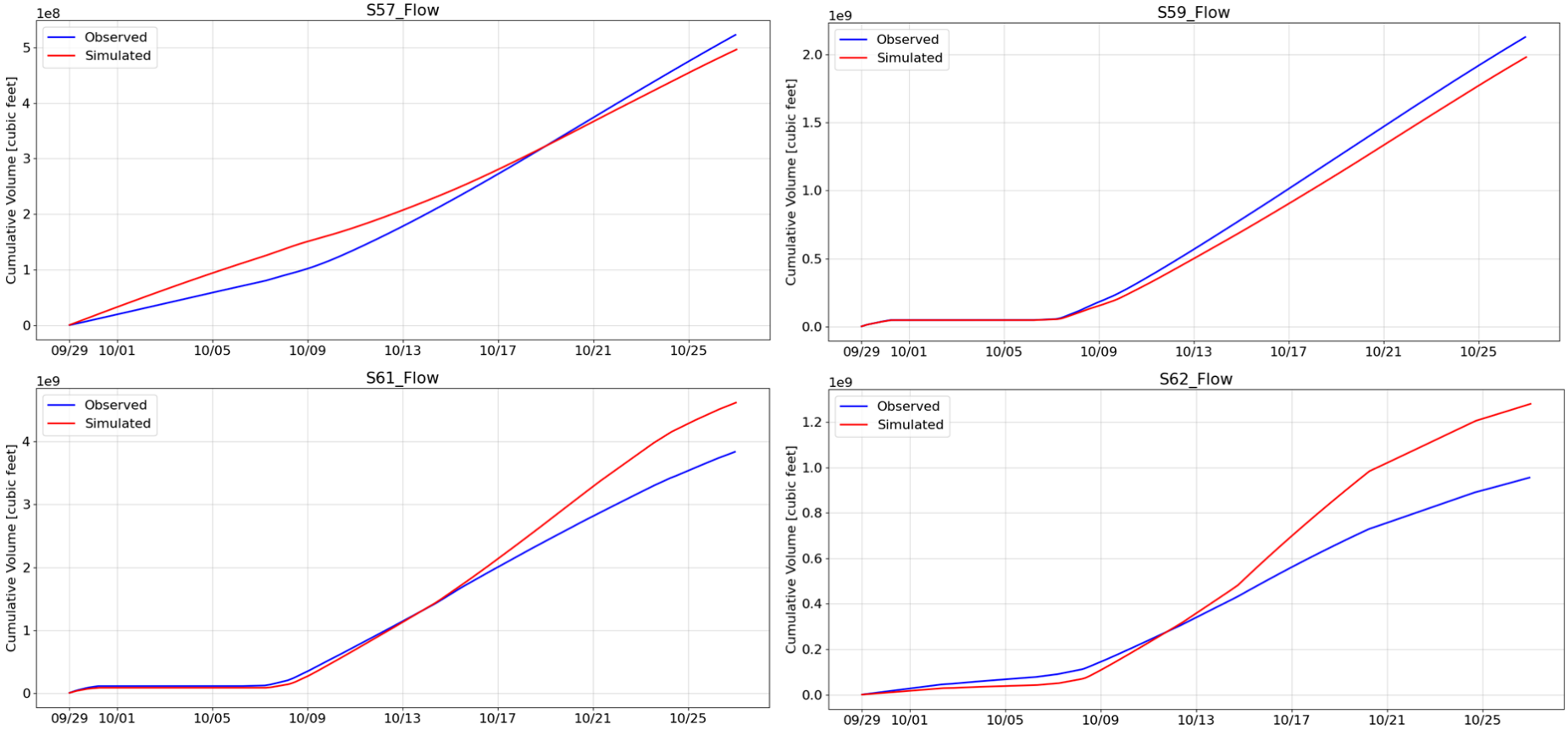

Figure 5-70: Cumulative Volume Comparisons for S57, S59, S61, S62

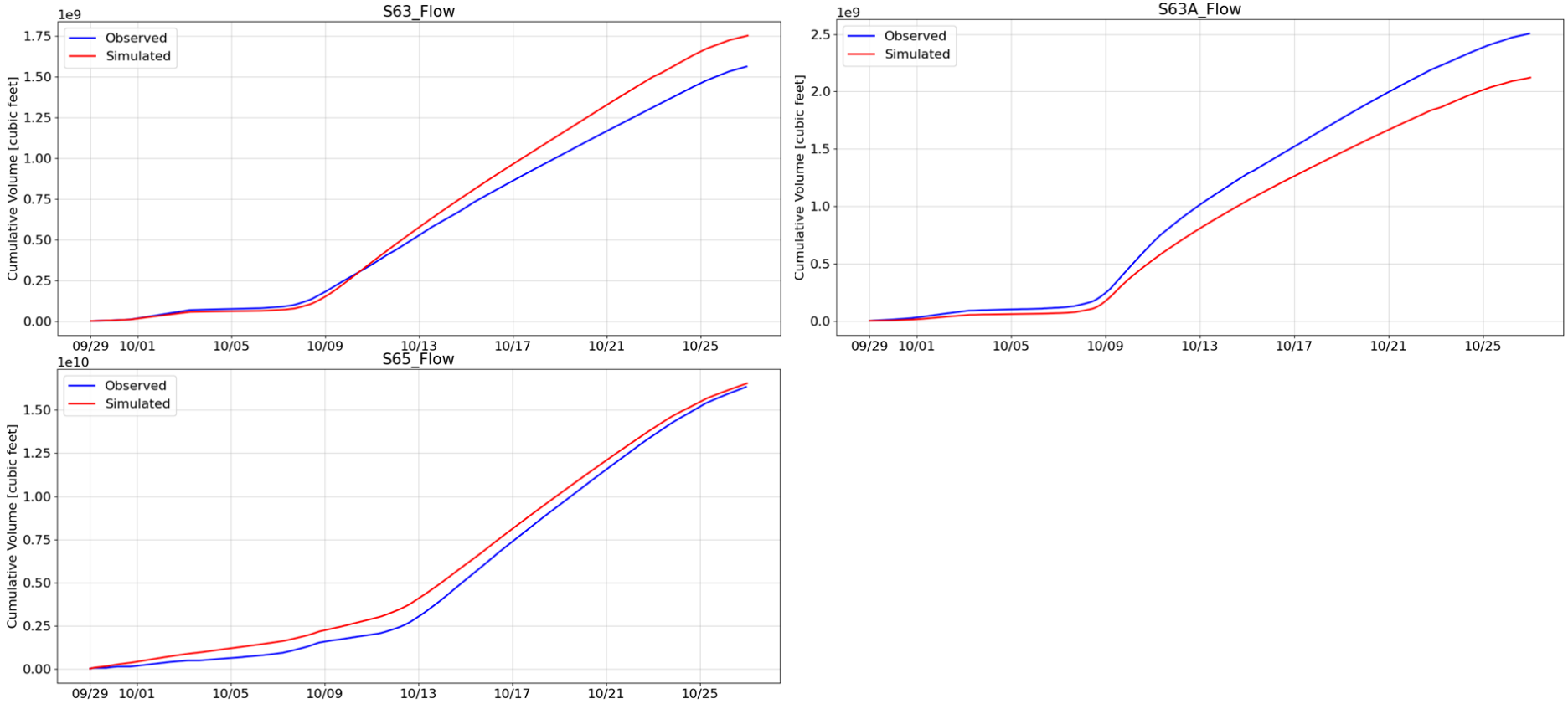

Figure 5-71: Cumulative Volume Comparisons for S63, S63A, and S65

Hazen

Table 5-18: Peak Discharge Comparison for SW Stations during Validation Period

Calibration Station	Peak Discharge (CFS)		Time of Peak Discharge	
	Simulated	Observed	Simulated	Observed
G113_Flow	280	No Data	10/20/2011 0:00	
S57_Flow	253	300	10/21/2011 12:00	10/21/2011 12:00
S58_Flow	-125	-110	10/25/2011 12:00	10/22/2011 0:00
S59_Flow	1300	1400	10/21/2011 12:00	10/20/2011 20:00
S60_Flow	850	650	10/18/2011 12:00	10/18/2011 14:00
S61_Flow	3300	2800	10/20/2011 0:00	10/14/2011 12:00
S62_Flow	1100	700	10/14/2011 20:00	10/14/2011 20:00
S63_Flow	1250	1400	10/10/2011 0:00	10/11/2011 0:00
S63A_Flow	2450	2750	10/9/2011 6:00	10/9/2011 12:00
S65_Flow	12800	13900	10/15/2011 22:00	10/15/2011 23:00
02264495-Q - Shingle	2480	1480	10/9/2011 12:00	10/12/2011 0:00
02266300-Q - Reedy	1200	570	10/9/2011 14:00	10/10/2011 13:00
02262900-Q - Boggy	3800	1100	10/9/2011 12:00	10/11/2011 0:00

Table 5-19: Peak Stages Comparison for GW Stations during Validation Period

Calibration Station	Peak Stage (FT NAVD88)	
	Simulated	Observed
82213802	112	110.5
MOSSPK_S	67.5	61.8
AIR19W1	85.5	85
ORS-3	Peak Unclear	101
ORS-6	Peak Unclear	93
ORS-7	Peak Unclear	93
ORS-5	66.5	63.5
ORS-4	88.2	94.1
ALL1W1	71.9	72.3
ALL1W2	72	72.3
EXOTGW	70.5	70.6
MAKO	76.3	75
OSF62_GW1	69.2	69.2
OSF64_GW1	55.7	55
OSF70_GW1	59.2	60.1
IC-SAS	Peak Unclear	66.3
KENANS1	75.1	74.6
OSS-101	Peak Unclear	103
WR6_GW1	64.3	64.2
WR9_GW1	68.6	68.2
WR11_GW1	67.7	67.1
WR15_GW1	62.9	62.8
TOHO1_GW	64.5	63.8
TOHO2_GW	64.8	64.8
TOHO10_GW	70	70

Calibration Station	Peak Stage (FT NAVD88)	
	Simulated	Observed
TOHO15_GW	74.8	74.8
TOHO7_GW	69.7	69.2
TB2_GW1	101.5	99.4
TB2_GW2	101.5	99.2
TB3_GW1	100	98.1
TB3_GW2	100	98
TB1_GW1	100.5	99.9
TB1_GW2	100.5	99.7
TURLAK_GW1	Peak Unclear	Peak Unclear
TLWMA2G1	61.3	60.9
TLWMA3G1	60.5	60.5
TLWMA4G1	60.5	59.8
TLWMA5G1	58.6	58.7
POS-14	Peak Unclear	No Obs Data
WR16_GW1	65.4	65.3
INDIAN L	119.4	117.7
POS-11	64.5	65.1
POS-12	64.5	62.6
LAKWEO	62.8	63.5
SNIVELY	64.6	65
Ridge WRAP VC-9 Surf	114	Peak Unclear
ROMP 57X Surf Aq Monitor	Peak Unclear	Peak Unclear
ROMP 74X Surf Aq Monitor	90	90

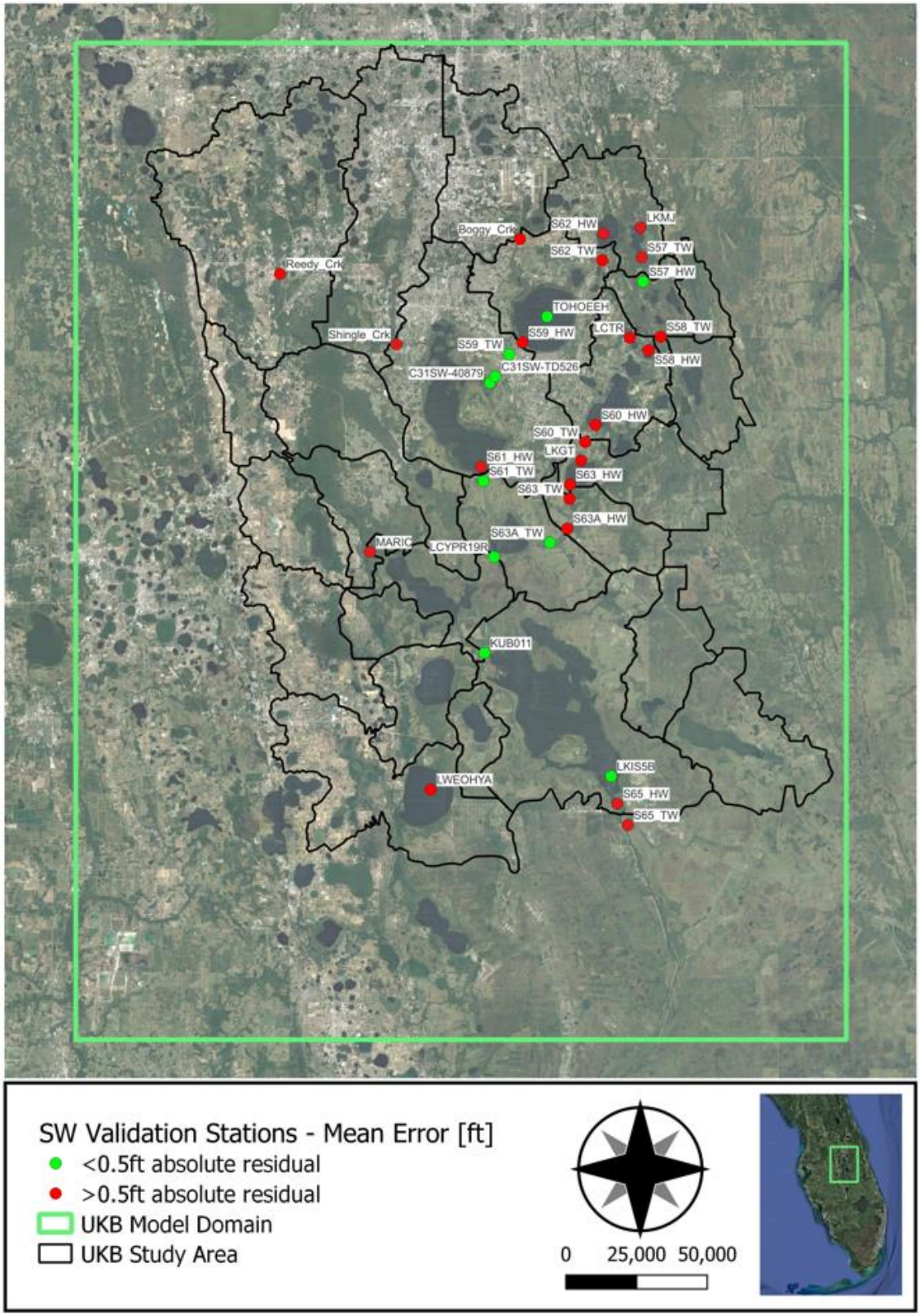

Figure 5-72: Mean Error [ft] Distribution in SW Validation (9/29-10/27) with threshold of 0.5 ft

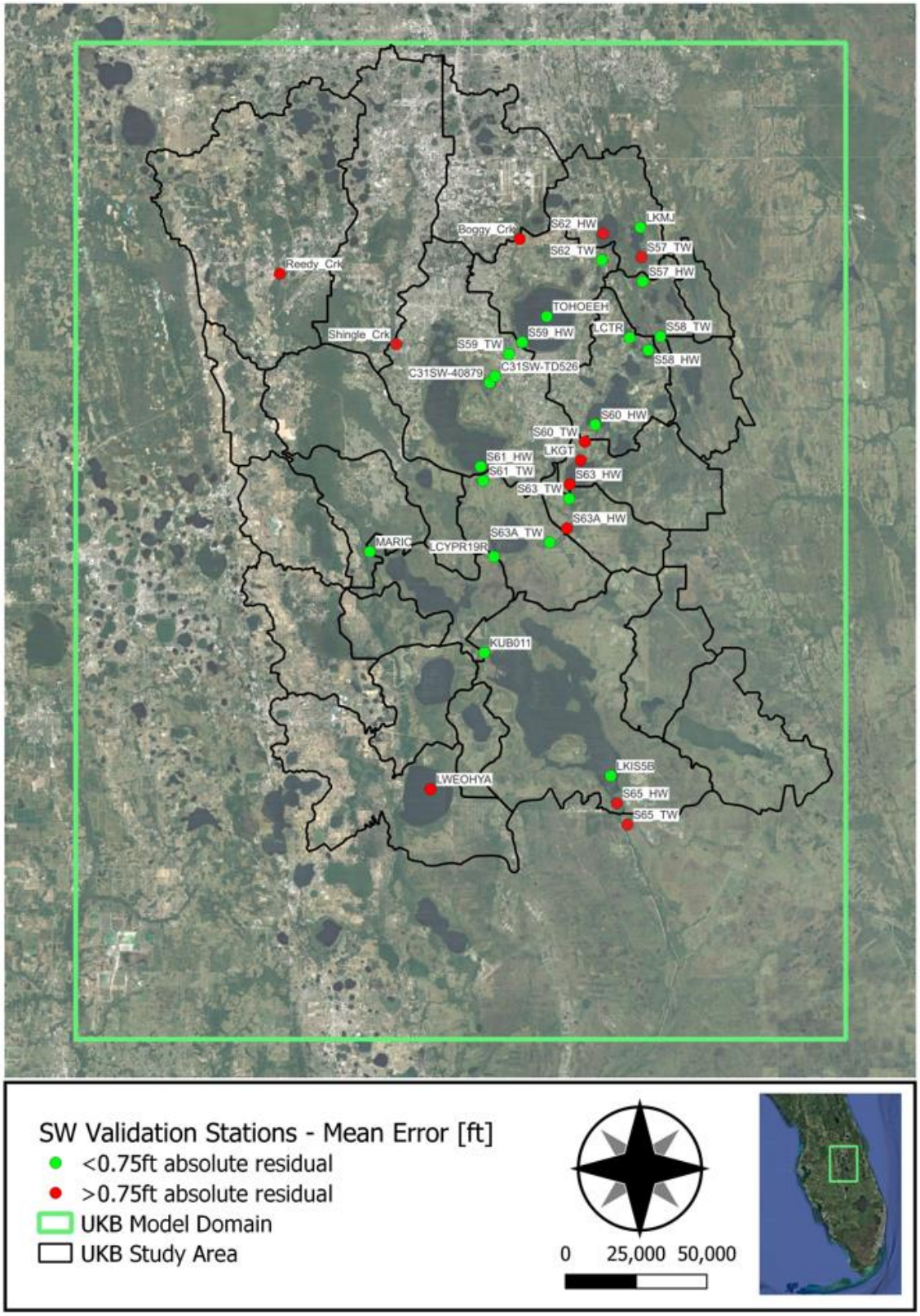

Figure 5-73: Mean Error [ft] Distribution in SW Validation (9/29-10/27) with threshold of 0.75 ft

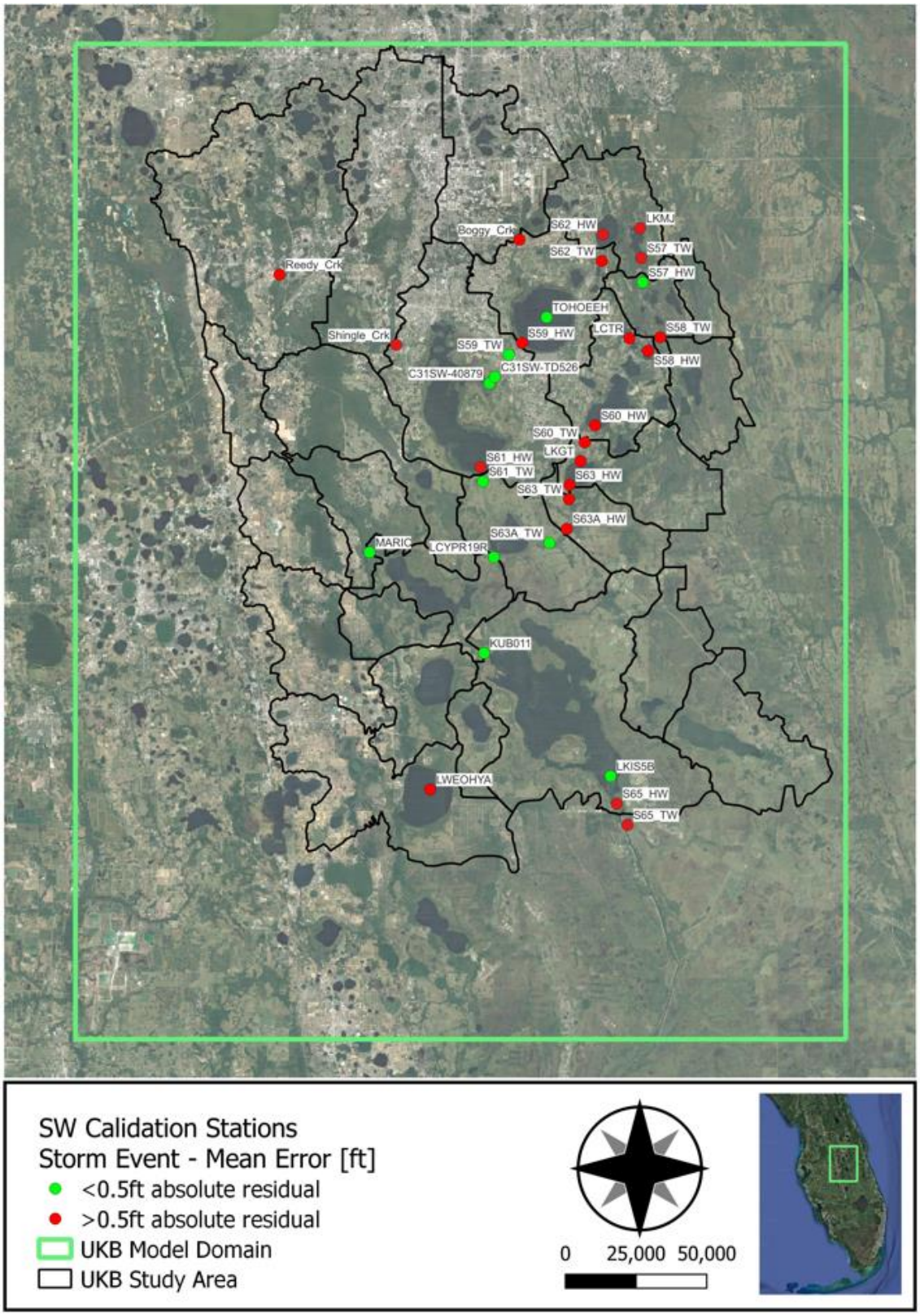

Figure 5-74: Mean Error [ft] Distribution in SW Validation (10/03-10/27) with threshold of 0.5 ft

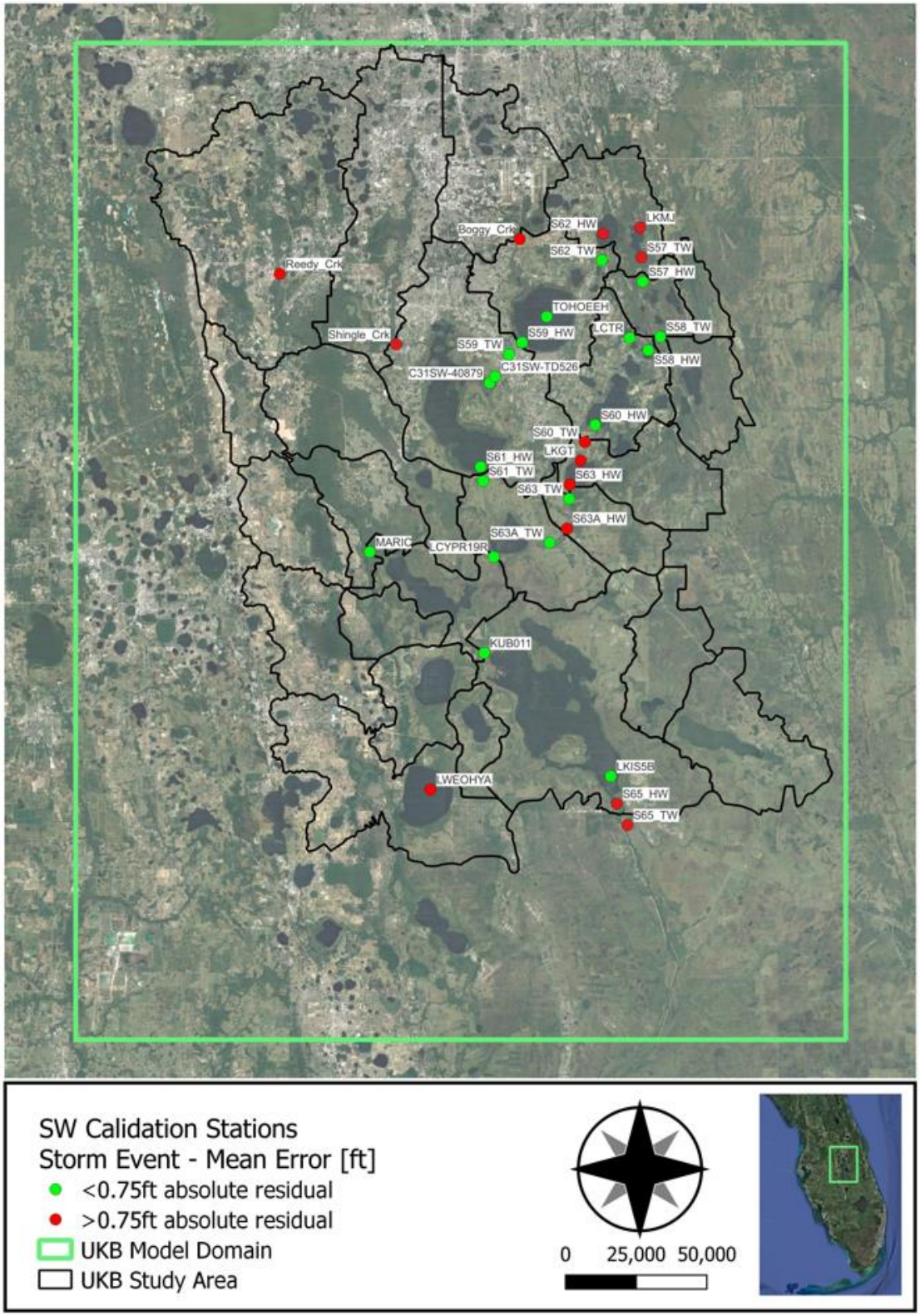

Figure 5-75: Mean Error [ft] Distribution in SW Validation (10/03-10/27) with threshold of 0.75 ft

Table 5-20: Percentage of Times where Simulated Flows are within 20% of Observed Flows at Hourly Time Steps

Station	POS	Storm Event
G113	-	-
S65A	37%	42%
G103	-	-
S57	43%	50%
S58	-	-
S59	74%	81%
S60	12%	7%
S61	57%	54%
S62	48%	56%
S63	64%	64%
S63A	63%	74%
S65	63%	72%
Shingle Creek	29%	32%
Reedy Creek	2%	0%
Boggy Creek	2%	0%
KUB001	-	-

5.3.9.3.2 Groundwater

Figure 5-74 and **Figure 5-75** show spatially the ME statistics for groundwater. Analysis of the validation statistics of groundwater shows 27% of groundwater stations had a ME of ±0.5 ft; if a threshold of 1 ft is used for ground water stages, 49% of the stations met the ME criterion and 34% of the stations meet the R with an average value of 0.9.

Statistics were also computed for groundwater stations in the UKB FPLOS Study area for the validation model, focused on the storm event. 24% of groundwater stations have a ME of ±0.5 ft and 36% of the stations have a R of 0.8 or higher. Using a threshold of 1 ft for the groundwater stations, 47% and 36% of the stations met the ME criteria and have a R value of 0.8 or higher. These are shown spatially in **Figure 5-78** and **Figure 5-79**.

Table 5-19 shows the comparison of observed and simulated peak groundwater levels in response to NoName event at SFWMD primary structures. The average difference between the peaks of observed and simulated groundwater levels was -0.53ft.

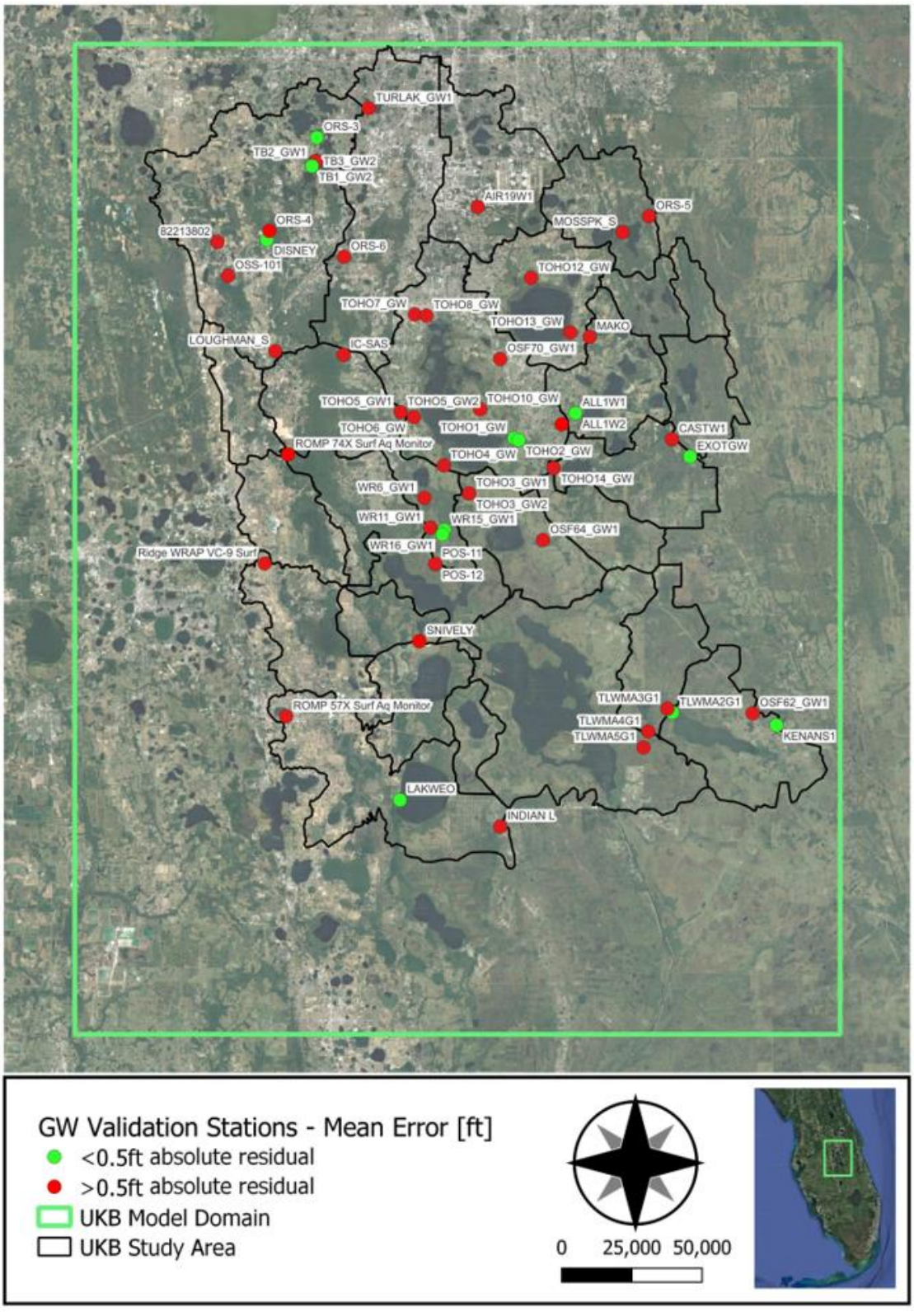

Figure 5-76: Mean Error [ft] Distribution in GW Validation (9/29-10/27) with threshold of 0.5 ft

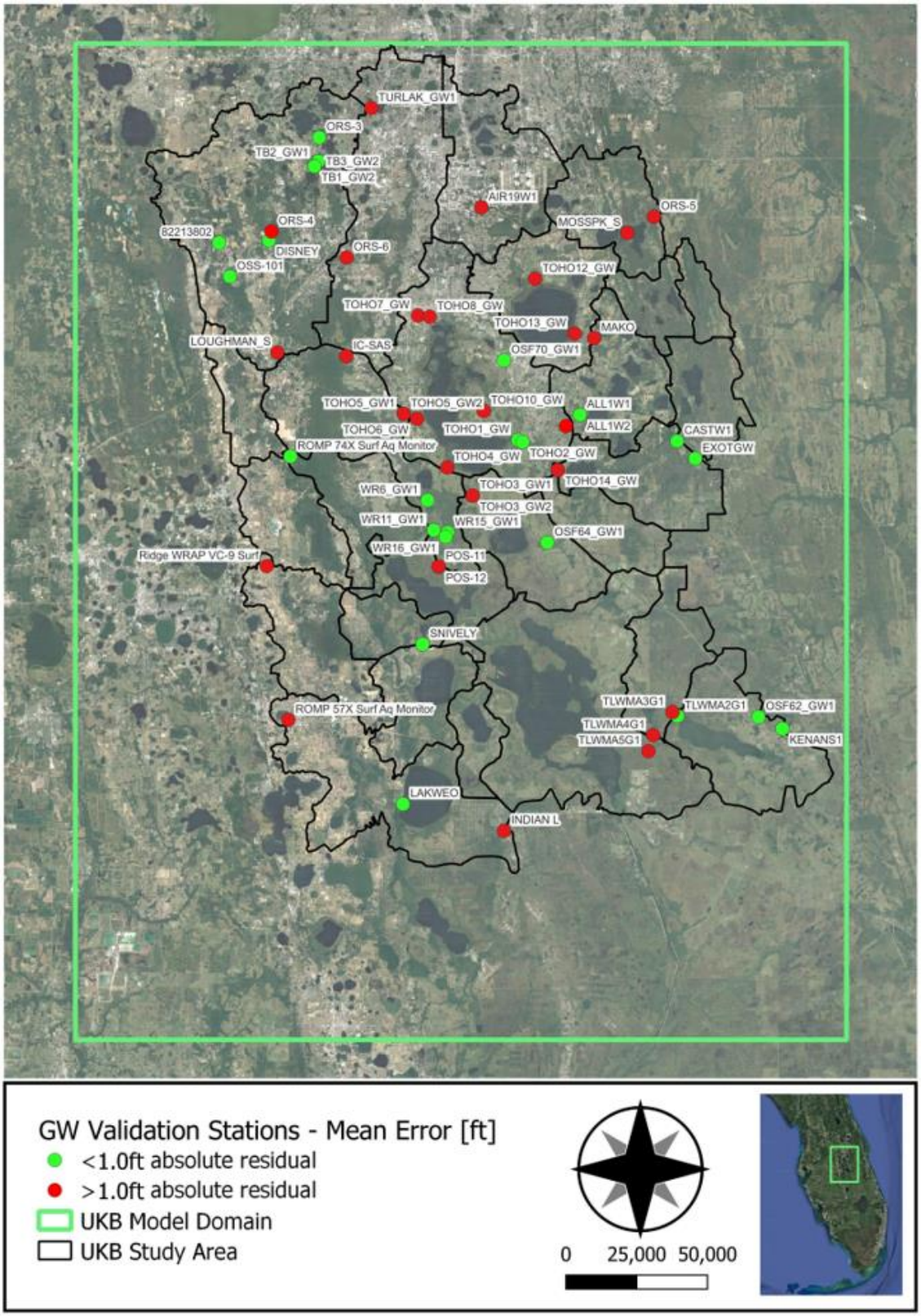

Figure 5-77: Mean Error [ft] Distribution in GW Validation (9/29-10/27) with threshold of 1 ft

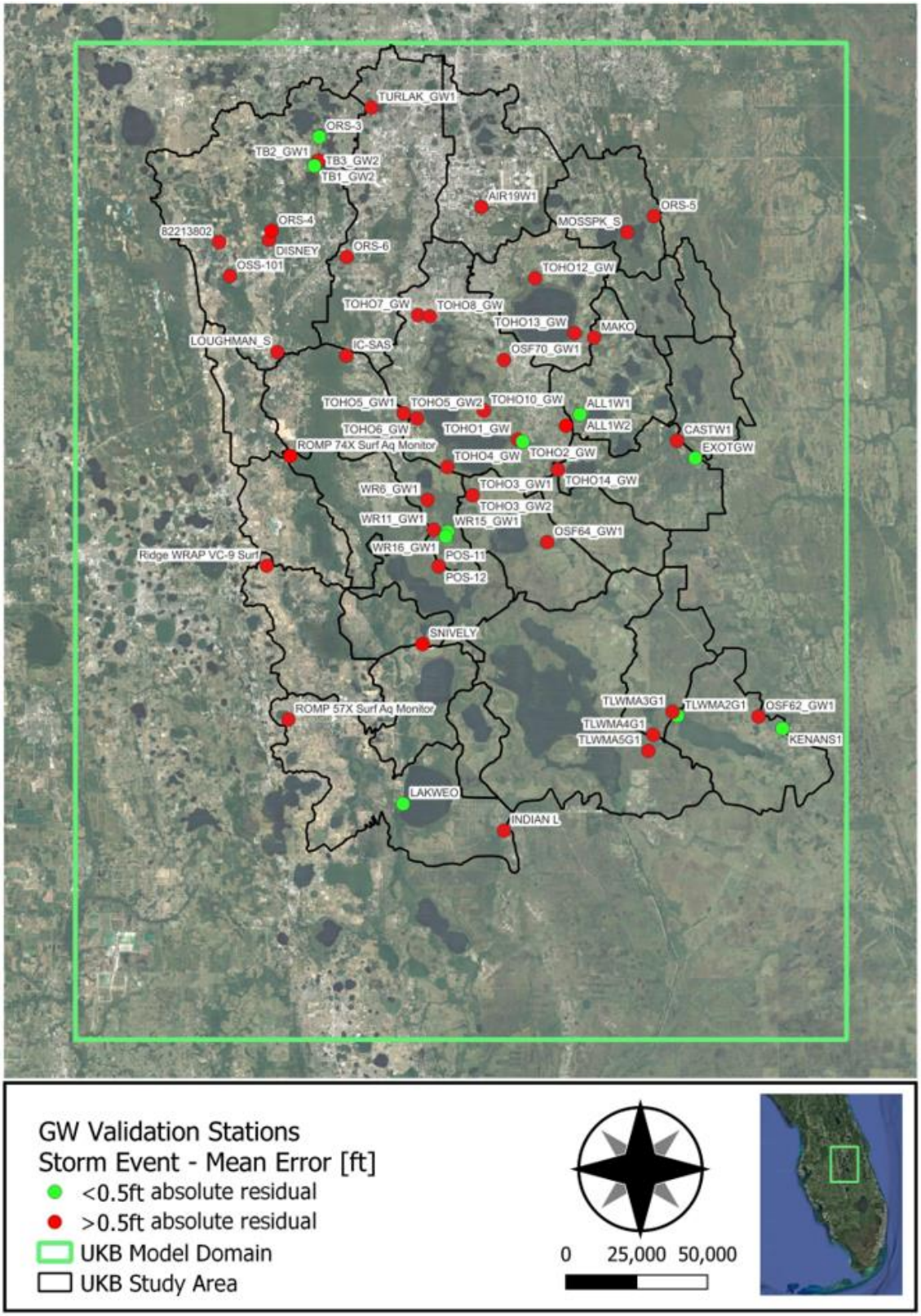

Figure 5-78: Mean Error [ft] Distribution in GW Validation (10/03-10/27) with threshold of 0.5 ft

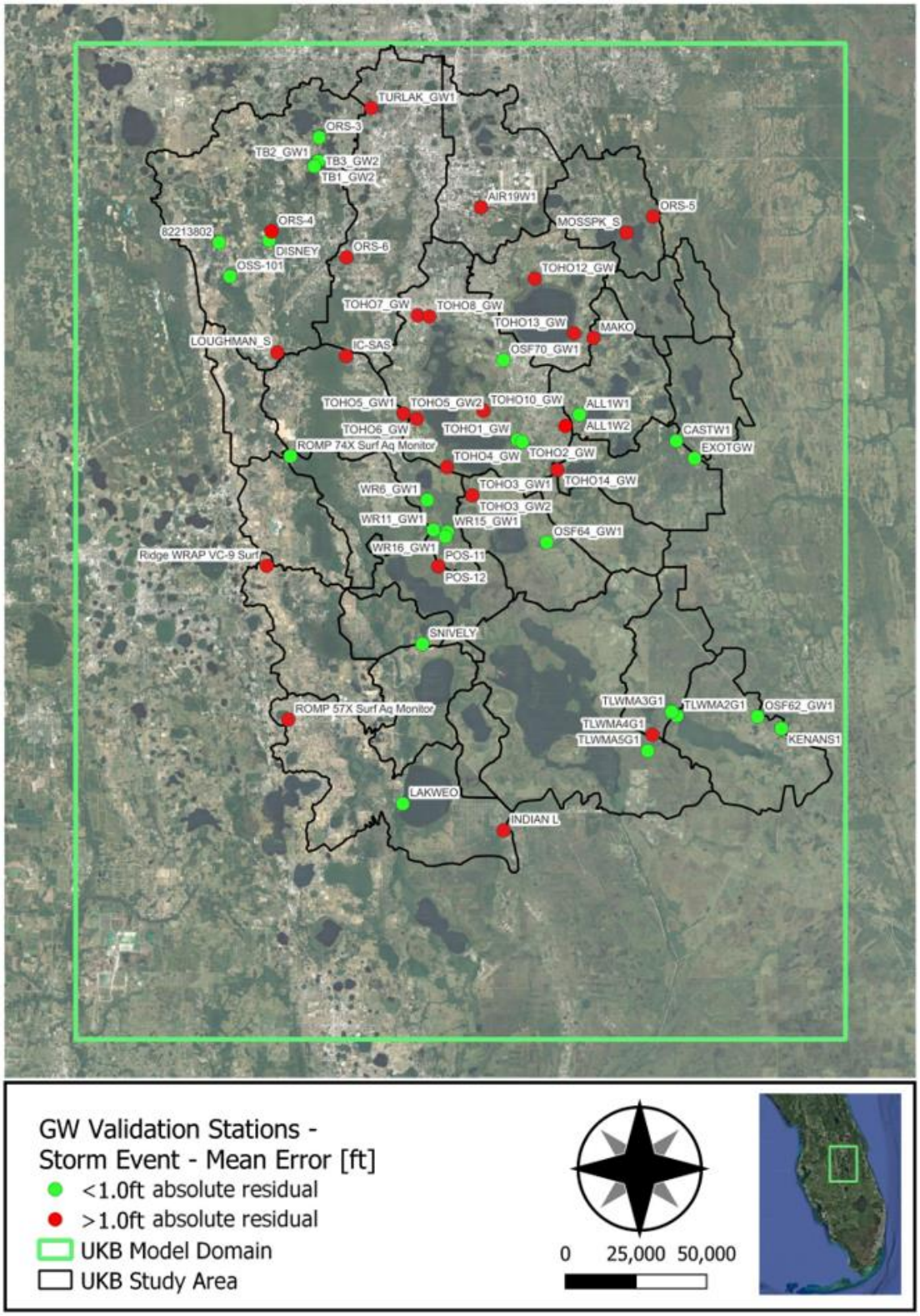

Figure 5-79: Mean Error [ft] Distribution in GW Validation (10/03-10/27) with threshold of 1 ft

5.3.9.4 Summary of Validation Status

Several statistical measures were used to describe the simulated data in comparison to the observed data, to assess the validation of the calibrated model. For the surface water stages, 29% met the criteria of +/- 0.5 ft ME, and 88% of them have R value of 0.8 or above. 59% met the +/- 0.75 ft ME criterion was used, with 88% of them with R value of 0.8 or above. For the surface water flows, all spillways except S60 and S65A met the flow criterion of +/- 20% more than 40% of the time. Among the USGS creeks, Reedy Creek had missing data for the validation event. Shingle and Boggy creeks have less than 30% of the time with simulated flows within 20% of the observed flows, but analysis shows all the creeks are time-shifted in the simulation.

Analysis of the validation statistics of groundwater showed 27% of groundwater stations with a ME of ±0.5 ft and 49% of the stations met the ME criterion of ±1 ft.

6. Current Conditions Model Setup

This section of the report outlines the modifications of datasets and processes based on the calibration model, including explanation of the simulation period, design storm rainfall, the initial and boundary conditions, the withdrawals, and the water control structure operations; as well as changes to the MIKE HYDRO model network that were implemented.

The current conditions model was set up for sixty days and includes a 9-to-10-day warm-up period prior to the simulation period, the three-day design storm event, and the following weeks to capture the recession hydrograph until it returns to baseflow.

6.1 Design Rainfall

The calibration model was the basis of the design events model, which used Hurricane Irma (Sep 10-12, 2017). For the design storm simulations, the Hurricane Irma rainfall was replaced by rainfall that represents the spatially distributed 3-day design storm rainfall distribution by SFWMD. Rainfall depths were obtained from the National Oceanic and Atmospheric Administration (NOAA) Rainfall Atlas ASCII Grid data. NOAA documentation indicated that the raster datasets of rainfall precipitation frequency were spatially interpolated by point precipitation frequency estimates. The precipitation frequency estimates were calculated using regional statistics for the annual maximum data series.

Flood events of a magnitude which are expected to be equaled or exceeded at least once on the average during any recurrence interval (5-, 10-, 25-, or the 100-yr period) were selected as design events. These storm events, referred to as the 5-, 10-, 25-, or the 100-yr floods, have a 20-, 10-, 4-, and 1-percent annual chance, respectively, of being equaled or exceeded during a given year.

The original NOAA grid provided total precipitation inches, in 2 km x 2 km grids. For the UKB FPLOS model, the NOAA grids were used to develop a spatial grid timeseries based on a five-minute timestep using the same resolution as the original grid from NOAA (2023). **Table 6-1** through **Table 6-4** provide the summary statistics for the 26 watersheds for the selected 5-, 10-, 25-, and the 100-yr return periods and **Figure 6-1** through **Figure 6-4** provide the spatial distribution of the storm events for the same.

For the design events, the SFWMD rainfall distribution for a 72-hr rainfall provided in SFWMD's Environmental Resource Permit, (ERP SFWMD, 2016) were used to create rainfall hyetographs. The cumulative percentage for the 72-hr rainfall distribution shown in **Table 6-5** and the plot is shown in **Figure 6-5**. The methodology for development of the rainfall curves was the following: Rainfall hyetograph was multiplied with the value of rainfall in each raster, which is the total cumulative rainfall for the 72-hr rain event.

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Table 6-1: Design Rainfall - Statistics for 5-yr 72-hr Event UKB FPLOS Model Domain (Current Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	6.54	6.66	0.12	6.59	0.03
Tiger Lake	6.46	6.52	0.06	6.48	0.01
Lake Rosalie	6.42	6.47	0.05	6.44	0.01
Lake Kissimmee	6.46	6.77	0.31	6.62	0.08
Lake Jackson	6.67	6.83	0.16	6.75	0.04
Lake Pierce	6.43	6.45	0.02	6.44	0.01
Lake Gentry	6.55	6.60	0.05	6.56	0.01
Horse Creek (Closed Basin)	6.35	6.44	0.09	6.41	0.02
Lake Cypress	6.48	6.63	0.15	6.53	0.03
Lake Marian	6.75	6.91	0.16	6.83	0.04
Marion Creek	6.42	6.53	0.11	6.46	0.02
Catfish Creek	6.42	6.44	0.03	6.42	0.01
Lake Conlin	6.56	6.67	0.11	6.60	0.03
Lake Marion	6.42	6.46	0.03	6.43	0.01
Lake Hatchineha	6.42	6.50	0.08	6.45	0.02
Lower Reedy Creek	6.43	6.63	0.20	6.51	0.05
Upper Reedy Creek	6.30	6.54	0.24	6.40	0.06
Lake Tohopekaliga	6.54	6.71	0.17	6.63	0.05
Lake Myrtle	6.59	6.68	0.09	6.63	0.02
East Lake Tohopekaliga	6.57	6.71	0.14	6.65	0.03
Lake Alligator	6.55	6.63	0.08	6.57	0.02
Shingle Creek	6.47	6.68	0.21	6.56	0.05
Lake Hart	6.64	6.70	0.06	6.66	0.01
Lake Weohyakapka	6.41	6.52	0.11	6.44	0.02
Boggy Creek	6.53	6.68	0.15	6.61	0.04
Econlockhatchee River Swamp	6.62	6.70	0.08	6.67	0.02

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Table 6-2: Design Rainfall- Statistics for 10-yr 72-hr Event for UKB FPLOS Model Domain (Current Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	7.80	7.96	0.15	7.86	0.04
Tiger Lake	7.64	7.72	0.07	7.67	0.02
Lake Rosalie	7.63	7.68	0.06	7.64	0.01
Lake Kissimmee	7.67	8.04	0.37	7.86	0.10
Lake Jackson	7.96	8.15	0.19	8.05	0.05
Lake Pierce	7.64	7.69	0.05	7.67	0.01
Lake Gentry	7.82	7.89	0.07	7.85	0.02
Horse Creek (Closed Basin)	7.62	7.71	0.09	7.68	0.02
Lake Cypress	7.71	7.90	0.19	7.78	0.04
Lake Marian	7.94	8.21	0.27	8.11	0.07
Marion Creek	7.65	7.80	0.16	7.71	0.03
Catfish Creek	7.63	7.66	0.03	7.64	0.01
Lake Conlin	7.85	8.01	0.15	7.91	0.04
Lake Marion	7.65	7.72	0.07	7.68	0.01
Lake Hatchineha	7.64	7.73	0.10	7.67	0.02
Lower Reedy Creek	7.68	7.95	0.28	7.79	0.07
Upper Reedy Creek	7.57	7.85	0.28	7.68	0.07
Lake Tohopekaliga	7.81	8.07	0.26	7.94	0.07
Lake Myrtle	7.90	8.03	0.13	7.96	0.03
East Lake Tohopekaliga	7.86	8.06	0.20	7.98	0.05
Lake Alligator	7.84	7.96	0.12	7.86	0.03
Shingle Creek	7.76	8.03	0.28	7.88	0.07
Lake Hart	7.98	8.08	0.10	8.01	0.02
Lake Weohyakapka	7.56	7.70	0.14	7.63	0.03
Boggy Creek	7.84	8.04	0.20	7.95	0.05
Econlockhatchee River Swamp	7.94	8.06	0.11	8.01	0.03

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Table 6-3: Design Rainfall - Statistics for 25-yr 72-hr Event for UKB FPLOS Model Domain (Current Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	9.76	9.95	0.19	9.84	0.04
Tiger Lake	9.41	9.54	0.13	9.48	0.03
Lake Rosalie	9.42	9.54	0.12	9.48	0.03
Lake Kissimmee	9.45	9.99	0.54	9.74	0.13
Lake Jackson	9.92	10.20	0.29	10.05	0.07
Lake Pierce	9.47	9.64	0.17	9.56	0.04
Lake Gentry	9.80	9.92	0.12	9.86	0.03
Horse Creek (Closed Basin)	9.63	9.72	0.09	9.69	0.02
Lake Cypress	9.63	9.84	0.21	9.72	0.05
Lake Marian	9.77	10.28	0.52	10.07	0.12
Marion Creek	9.55	9.79	0.24	9.66	0.05
Catfish Creek	9.51	9.56	0.06	9.53	0.01
Lake Conlin	9.88	10.09	0.21	9.95	0.05
Lake Marion	9.56	9.69	0.13	9.64	0.03
Lake Hatchineha	9.53	9.64	0.11	9.57	0.03
Lower Reedy Creek	9.59	10.00	0.41	9.78	0.10
Upper Reedy Creek	9.58	9.88	0.30	9.68	0.07
Lake Tohopekaliga	9.79	10.15	0.36	9.98	0.10
Lake Myrtle	9.94	10.12	0.18	10.01	0.04
East Lake Tohopekaliga	9.88	10.14	0.25	10.03	0.06
Lake Alligator	9.84	10.01	0.17	9.89	0.04
Shingle Creek	9.73	10.10	0.37	9.90	0.10
Lake Hart	10.03	10.20	0.17	10.09	0.04
Lake Weohyakapka	9.27	9.57	0.30	9.42	0.07
Boggy Creek	9.82	10.10	0.28	9.98	0.07
Econlockhatchee River Swamp	10.00	10.16	0.16	10.09	0.04

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Table 6-4: Design Rainfall - Statistics for 100-yr 72-hr Event UKB FPLOS Model Domain (Current Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	13.33	13.60	0.27	13.44	0.06
Tiger Lake	12.56	12.86	0.30	12.72	0.08
Lake Rosalie	12.63	12.93	0.30	12.81	0.08
Lake Kissimmee	12.60	13.61	1.01	13.10	0.21
Lake Jackson	13.35	13.96	0.62	13.65	0.15
Lake Pierce	12.77	13.24	0.47	13.03	0.11
Lake Gentry	13.42	13.70	0.27	13.55	0.06
Horse Creek (Closed Basin)	13.36	13.44	0.08	13.42	0.02
Lake Cypress	13.13	13.47	0.34	13.27	0.08
Lake Marian	13.07	14.14	1.07	13.65	0.25
Marion Creek	13.05	13.44	0.39	13.25	0.09
Catfish Creek	12.92	13.06	0.14	12.98	0.03
Lake Conlin	13.62	13.92	0.29	13.74	0.07
Lake Marion	13.07	13.38	0.31	13.24	0.08
Lake Hatchineha	12.92	13.17	0.25	13.06	0.05
Lower Reedy Creek	13.10	13.72	0.63	13.45	0.14
Upper Reedy Creek	13.28	13.59	0.32	13.37	0.07
Lake Tohopekaliga	13.42	13.90	0.48	13.71	0.13
Lake Myrtle	13.70	13.94	0.24	13.79	0.06
East Lake Tohopekaliga	13.62	13.88	0.26	13.76	0.05
Lake Alligator	13.51	13.75	0.24	13.62	0.05
Shingle Creek	13.30	13.83	0.54	13.55	0.16
Lake Hart	13.72	14.08	0.36	13.86	0.09
Lake Weohyakapka	12.28	12.99	0.71	12.62	0.18
Boggy Creek	13.38	13.82	0.44	13.63	0.11
Econlockhatchee River Swamp	13.79	14.01	0.22	13.91	0.06

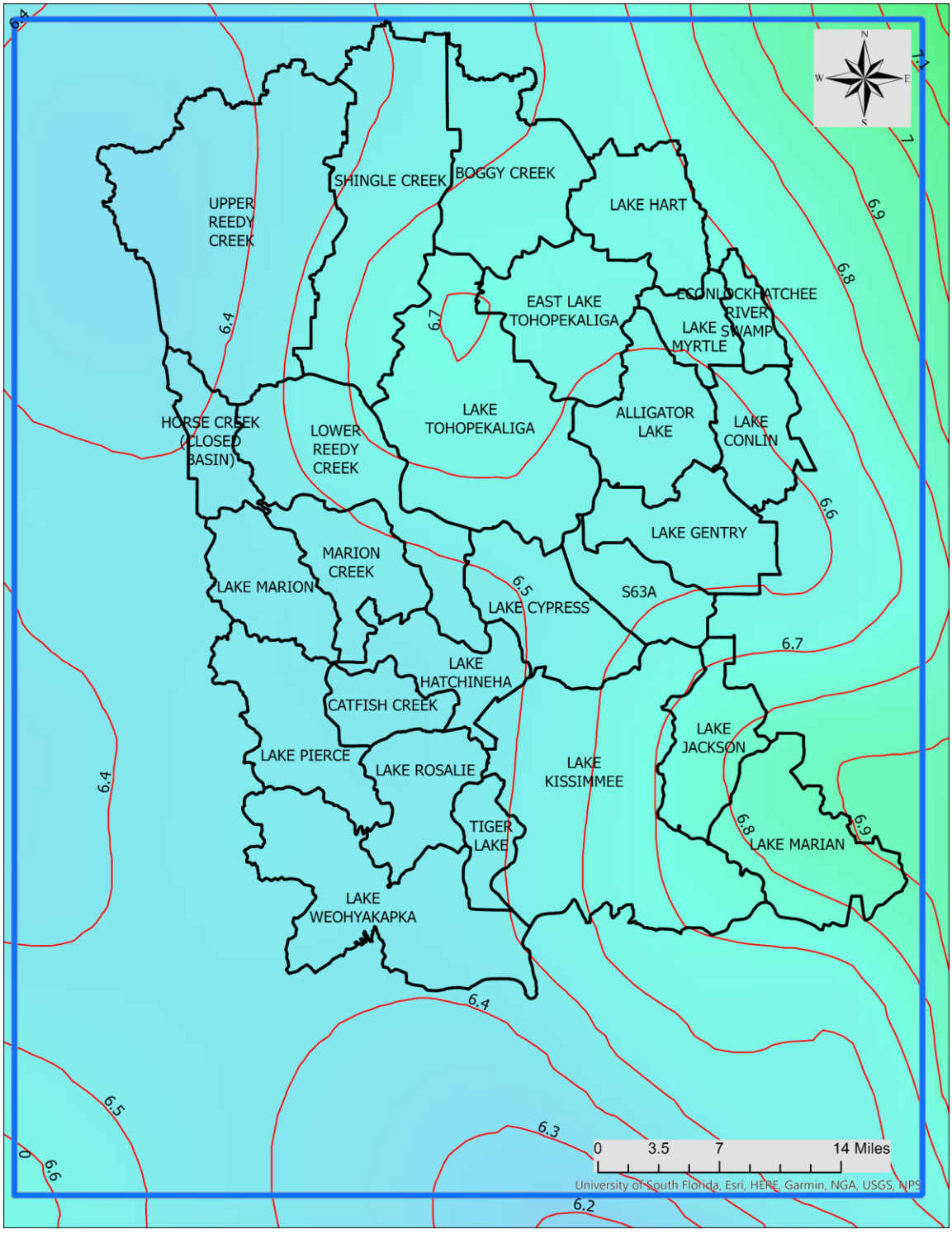


Figure 6-1: Rainfall Distribution for 5-yr 3-day Design Event (contours represent total rainfall in inches)

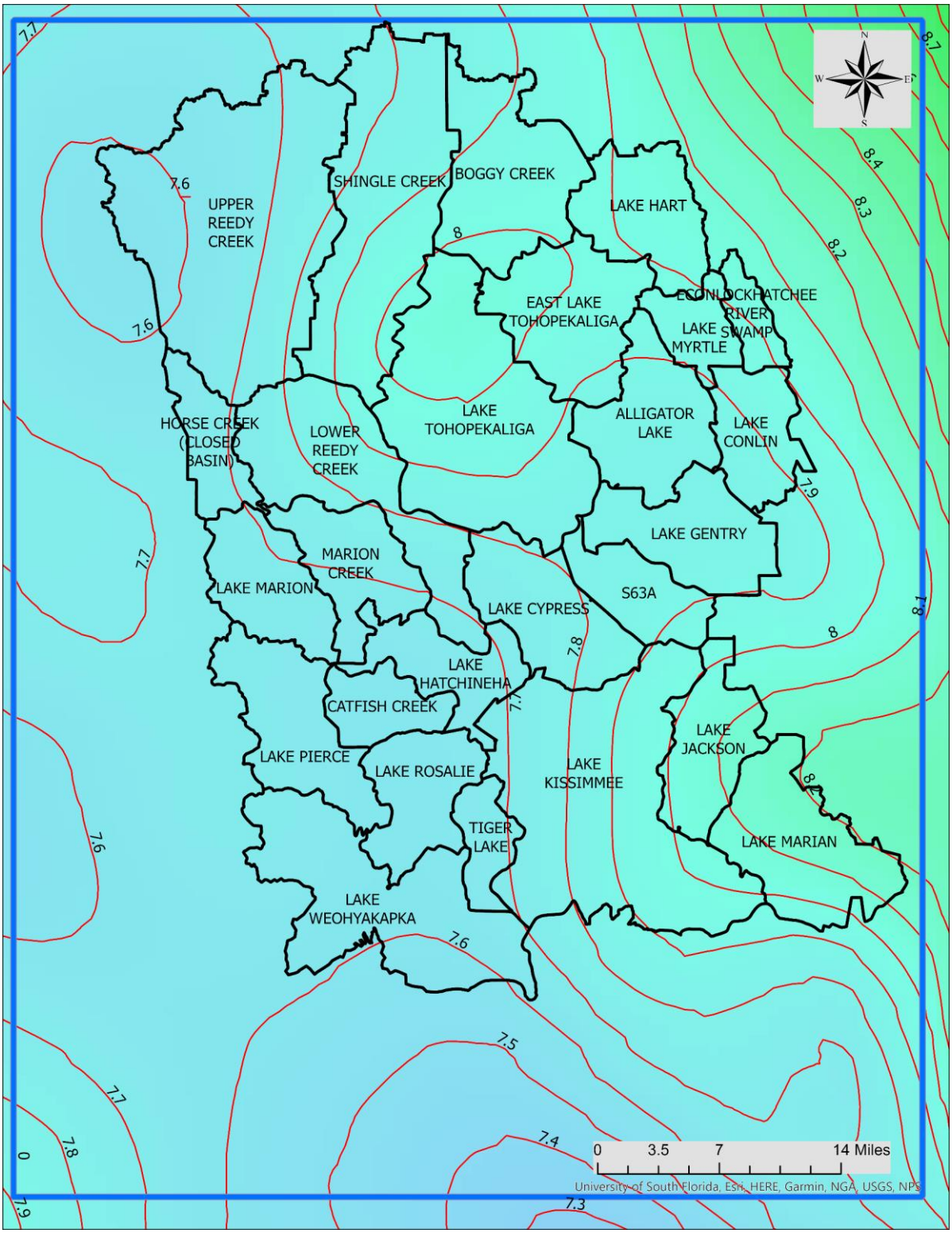


Figure 6-2: Rainfall Distribution for 10-yr 3-day Design Event (contours represent total rainfall in inches)

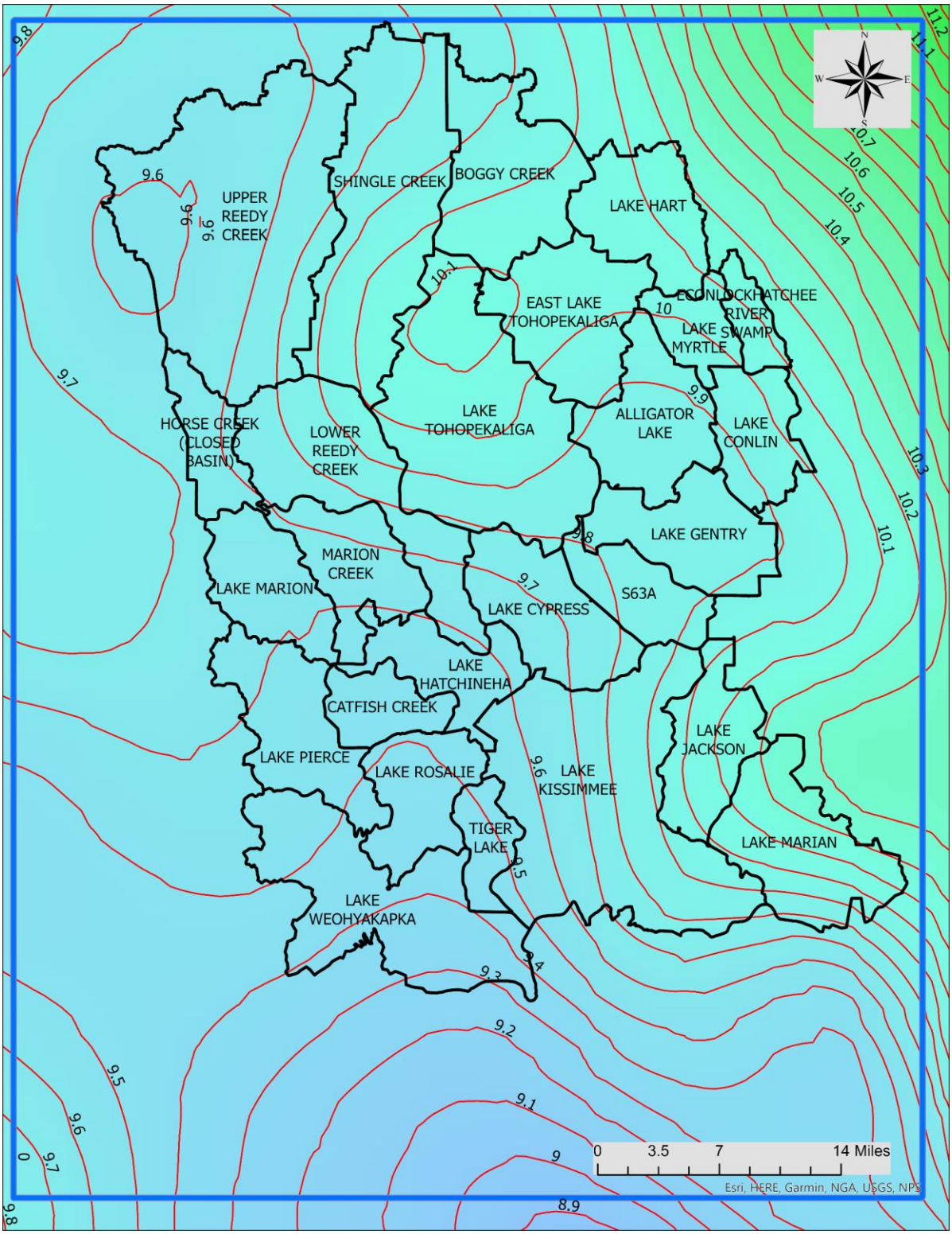


Figure 6-3: Rainfall Distribution for 25-yr 3-day Design Event (contours represent total rainfall in inches)

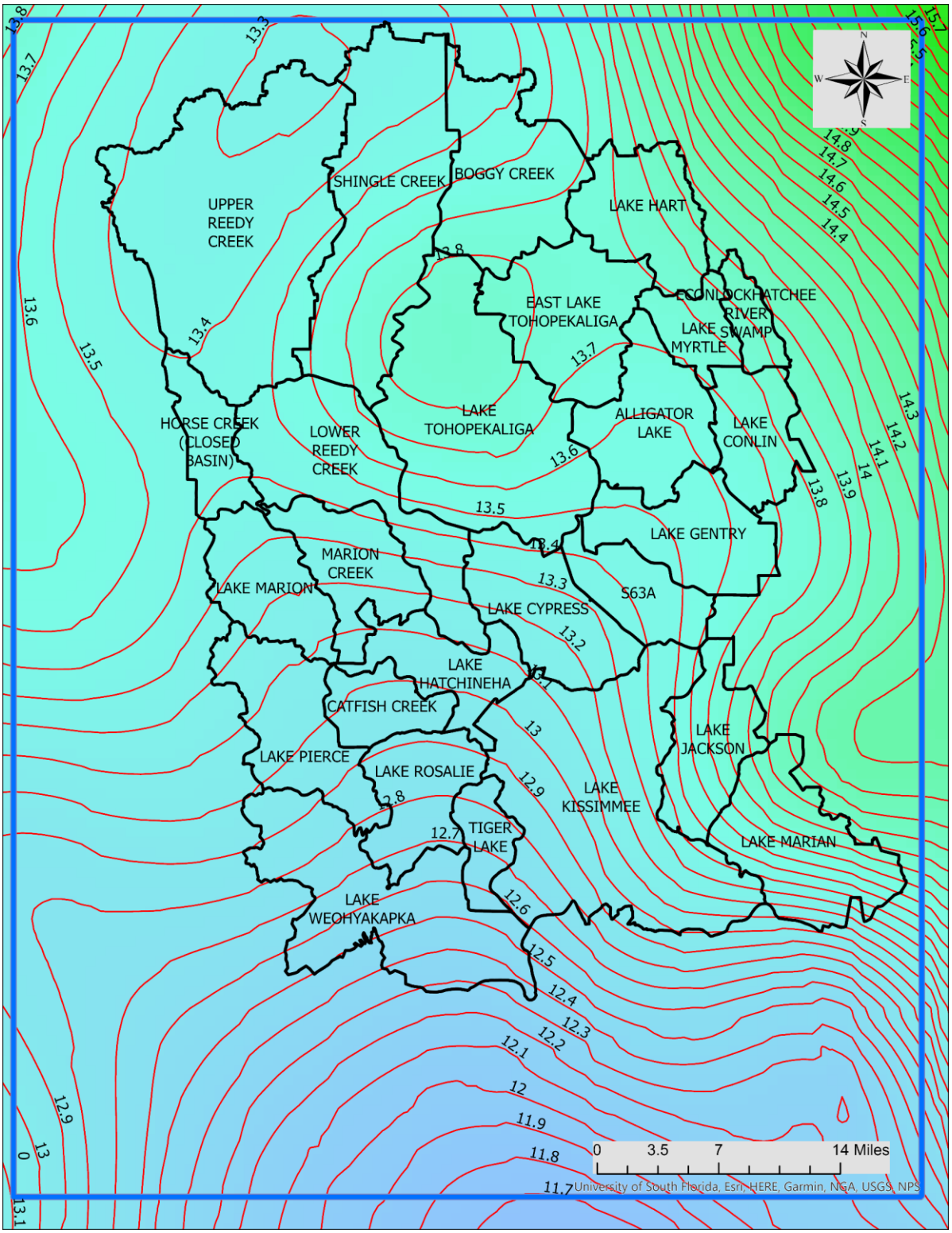


Figure 6-4. Rainfall Distribution for 100-yr 3-day Design Event (contours represent total rainfall in inches)

Table 6-5: SFWMD 72-hr Rainfall Distribution

Time (hours)	Cumulative Percentage of Peak 24-hr Rainfall	Cumulative 100% 72-hr Rainfall
0	0.0%	0.0%
24	14.6%	10.7%
48	35.9%	26.4%
58	57.2%	42.1%
59	62.8%	46.2%
59.5	67.8%	49.9%
59.75	82.8%	60.9%
60	101.5%	74.7%
60.5	108.8%	80.1%
61	112.6%	82.9%
62	117.7%	86.6%
72	135.9%	100.0%

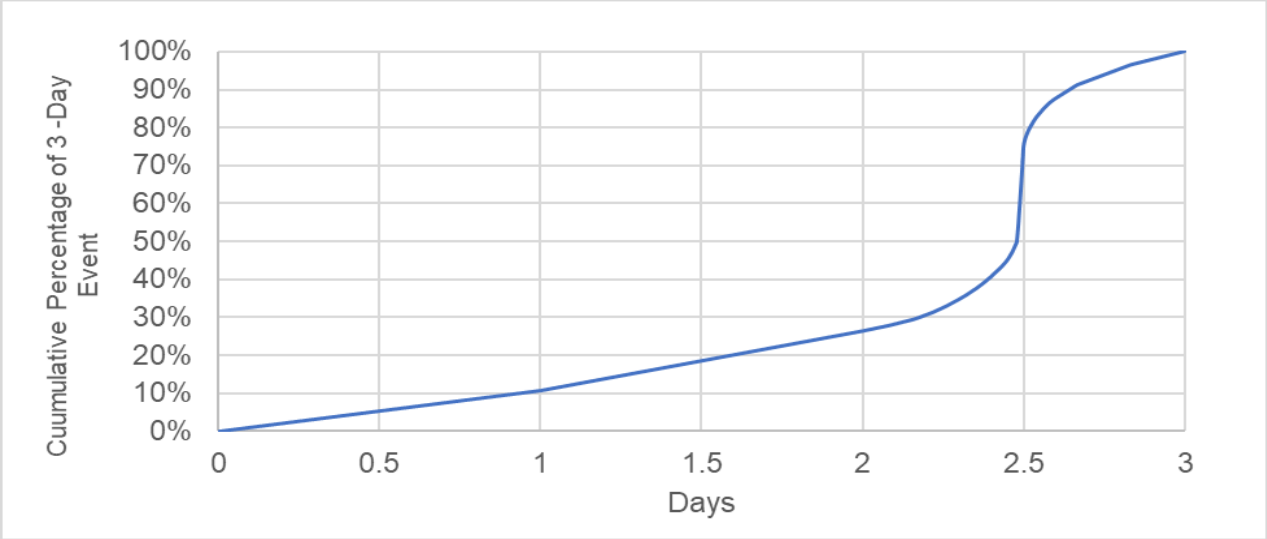


Figure 6-5: Unit Rainfall Distribution for 72-hr Rainfall Event

6.2 Structure Operations

In general, the structures in UKB FPLOS Model under current conditions are based on the year 2020.

6.2.1 Management of the UKB System

Simulations of the UKB system is by combining the 3 management considerations:

- Lake regulation schedules
- Gate operation criteria
- Flow rating equations

6.2.1.1 Regulation Schedules

The lakes are regulated by calendar based, rigid operating schedules. The structure regulation schedules represent the management aspect of the KCOL system aimed at optimizing flood control and other SFWMD core missions. The general trend of the lake regulation schedules reaches the minimum and maximum at the beginning and end of the wet season in preparation for flood control and water supply demands, respectively. Regulation schedules are, nevertheless, subjective rules which are managed by SFWMD using the best professional judgment to manage the water resources for central and south Florida. The schedules will be input in the model as breakpoint data. The schedules for each LMA are discussed further in this section.

6.2.1.2 Gate Operating Criteria

The objective of the MAGO for each spillway is to prevent damage to the riprap around the structures due to excessive velocity of water. The criteria were established by the U. S. Army Corps of Engineers (USACE) and presented in charts later in the section. The family of curves were represented using an equation for each spillway and were coded using functions and variables in MIKE. MPHD determines the maximum difference between the headwater and tailwater of structures to preserve the structure integrity. The MAGO and MPHD were coded as constraints in the UKB FPLOS model. Therefore, they have higher priority relative to the regulation schedule when the model determines the discharges at structures.

Table 6-6 summarizes the UKB FPLOS model structure parameters, including the MPHD for the structures. The MAGO curves for the structures in **Figure 6-6 through Figure 6-12** show the headwater and tailwater elevations in NGVD29.

Table 6-6: Summary of UKB FPLOS Structure Parameters

Structure	Type	Crest/invert Elevation [ft NAVD88]	Number of Gates	Design Capacity in cfs	MPHD in feet; Elevations in ft NAVD88	Corresponding Levels in NGVD29
S57	Culvert	51.44	2	170	4	5.07
S58	Culvert	53.43	2	160	4	5.06
S59	Gated spillway	48.13	1	590 - 820	8	8.97
S60	Gated spillway	53.95	1	450	8.5	9.55
S61	Gated spillway	35.85	1	1570 - 2300	10 if HW < 53.95 6 if 53.95 <HW<57.95 reduced (6ft) if HW > 57.95	10 if HW < 55 6 if 55<HW<59 reduced (6ft) if HW > 59
S62	Gated spillway	54.27	1	450 - 640	7.2	8.23
S63	Gated spillway	52.91	1	715	11 if HW ≤ 62.91 10 if HW > 62.91	11 if HW ≤ 64 10 if HW > 64
S63A	Gated spillway	48.27	2	870 - 2000	11 if HW ≤ 56.87 10 if HW > 56.87	11 if HW ≤ 58 10 if HW > 58
S65	Gated spillway	38.11	5	3000 [firm capacity] 11000 [full capacity]	10	11.19
G113	Culvert	50.61	2	440	N/A	N/A
G111	Culvert	43.79	10 gates; 5 barrels	Unknown	N/A	N/A
G103	Gate-Controlled Weir	47.84	5	600	N/A	N/A

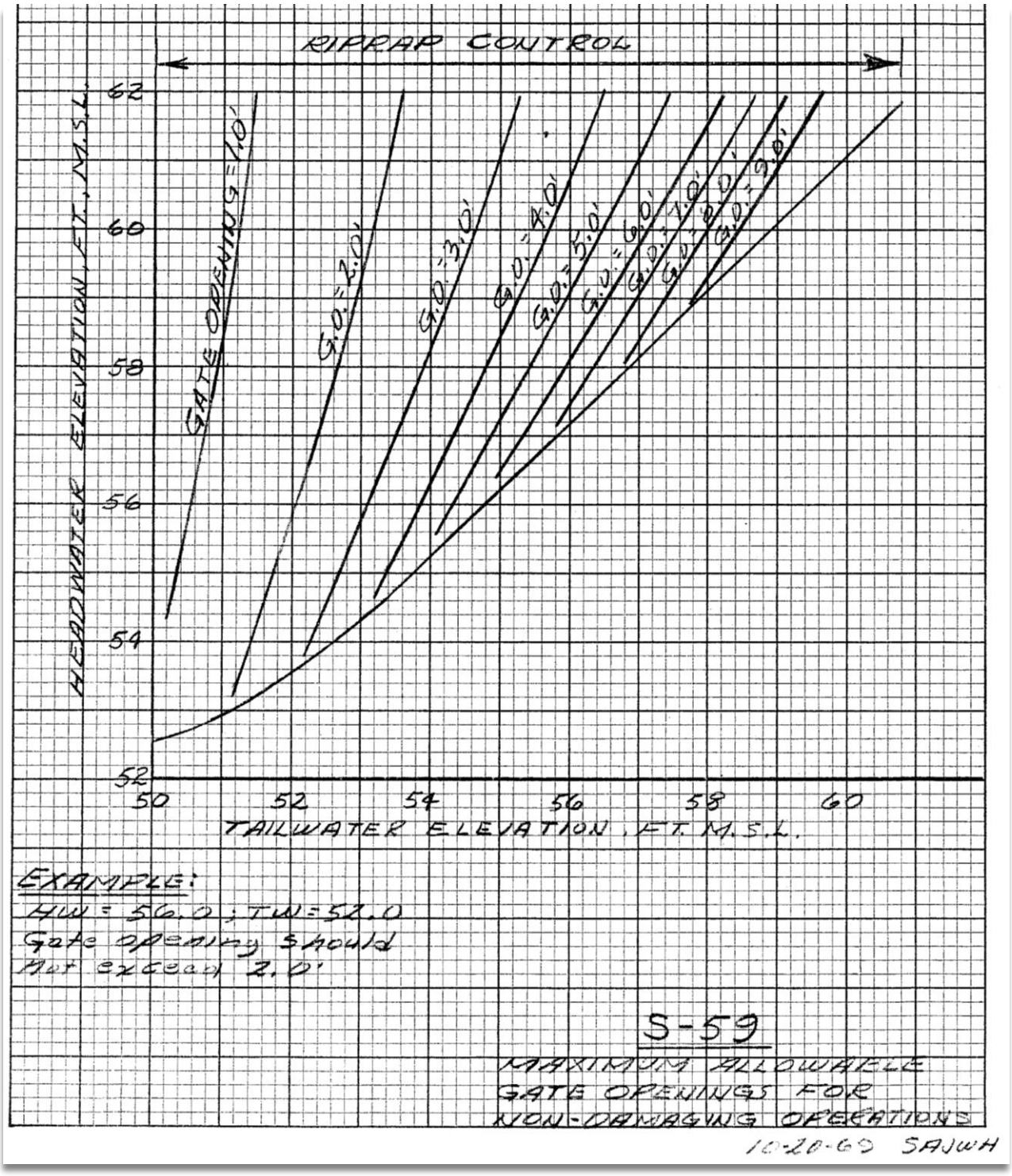


Figure 6-6: Structure S59 MAGO Curves

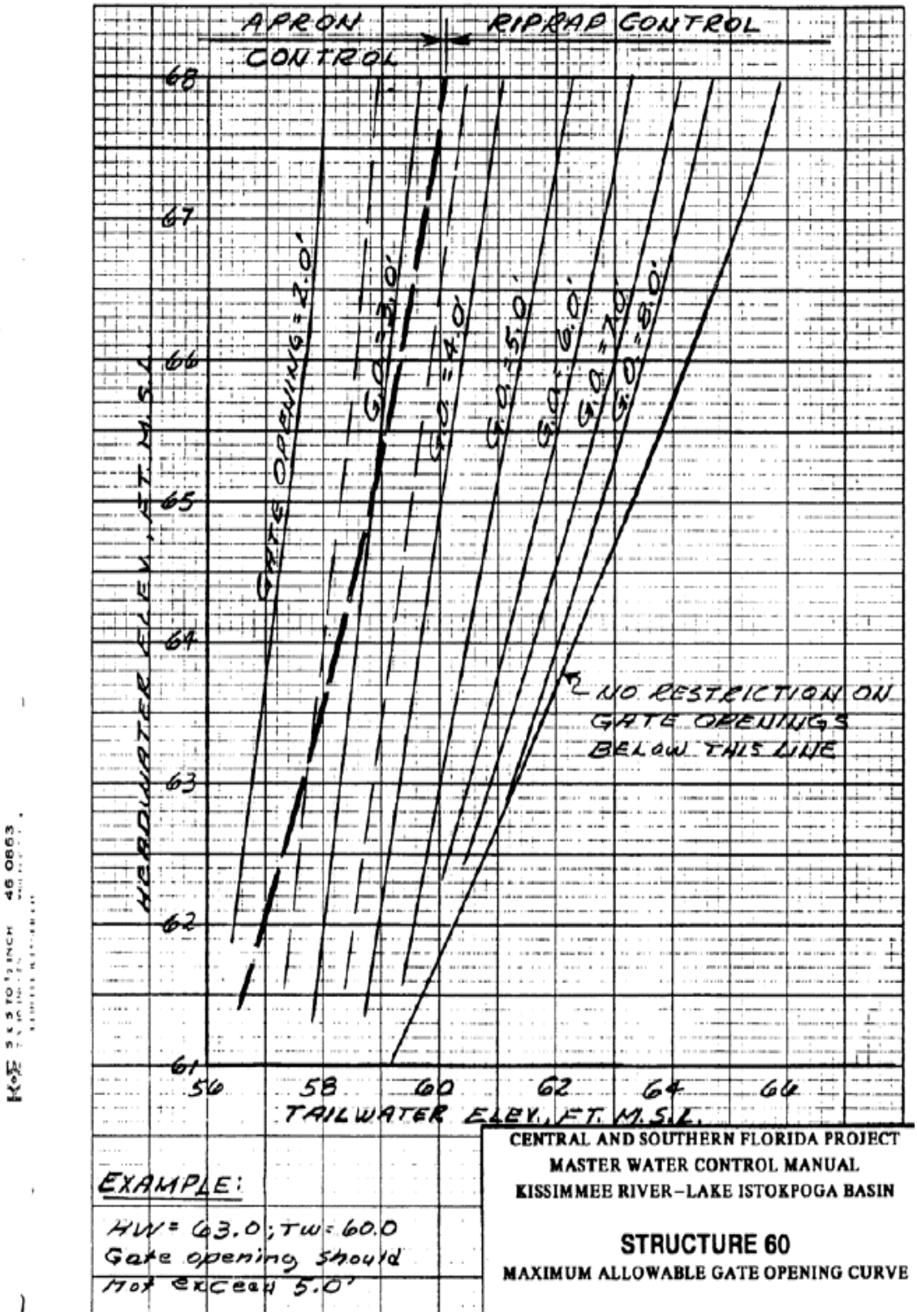


Figure 6-7: Structure S60 MAGO Curves

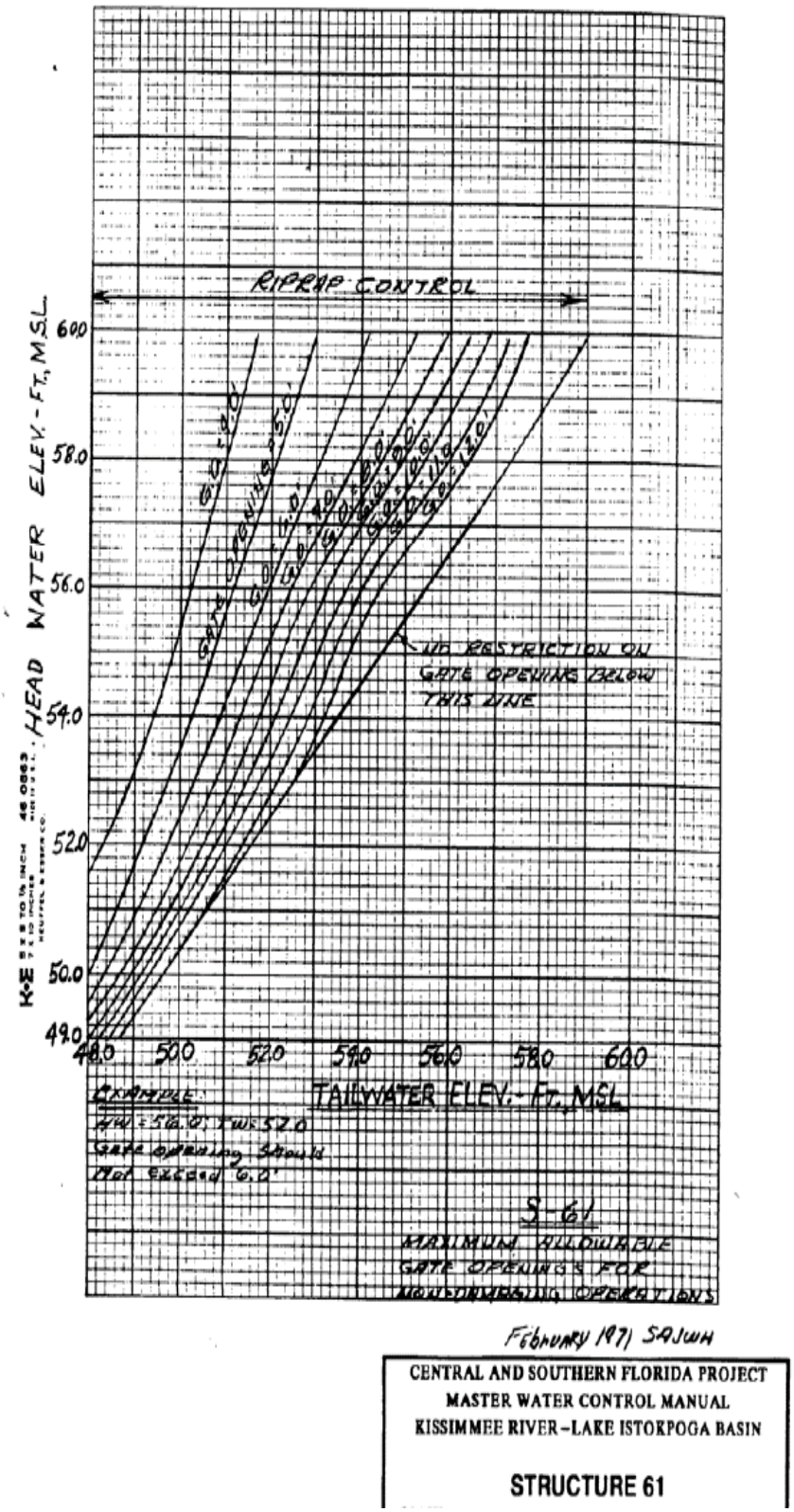


Figure 6-8: Structure S61 MAGO Curves

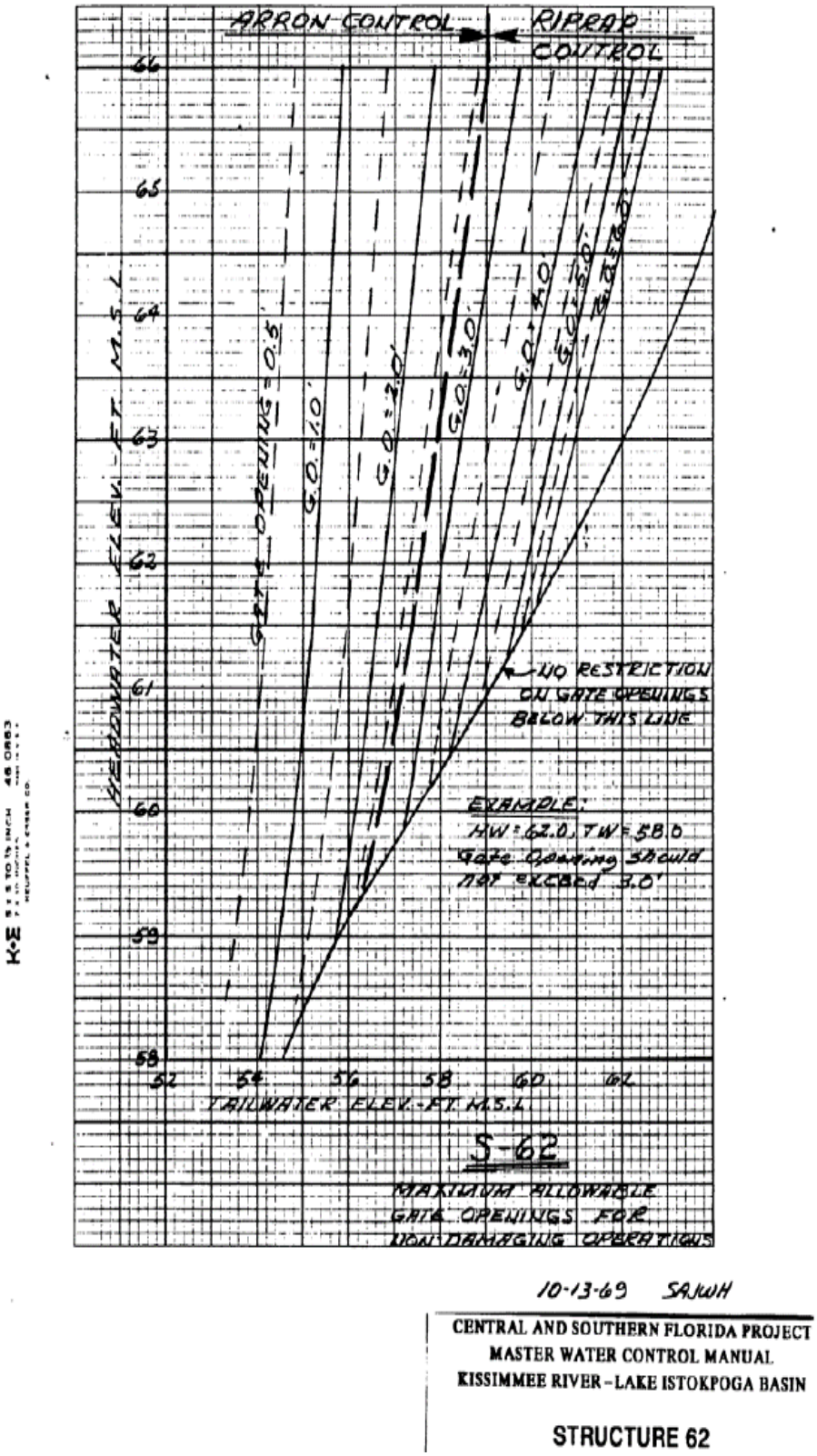


Figure 6-9: Structure S62 MAGO Curves

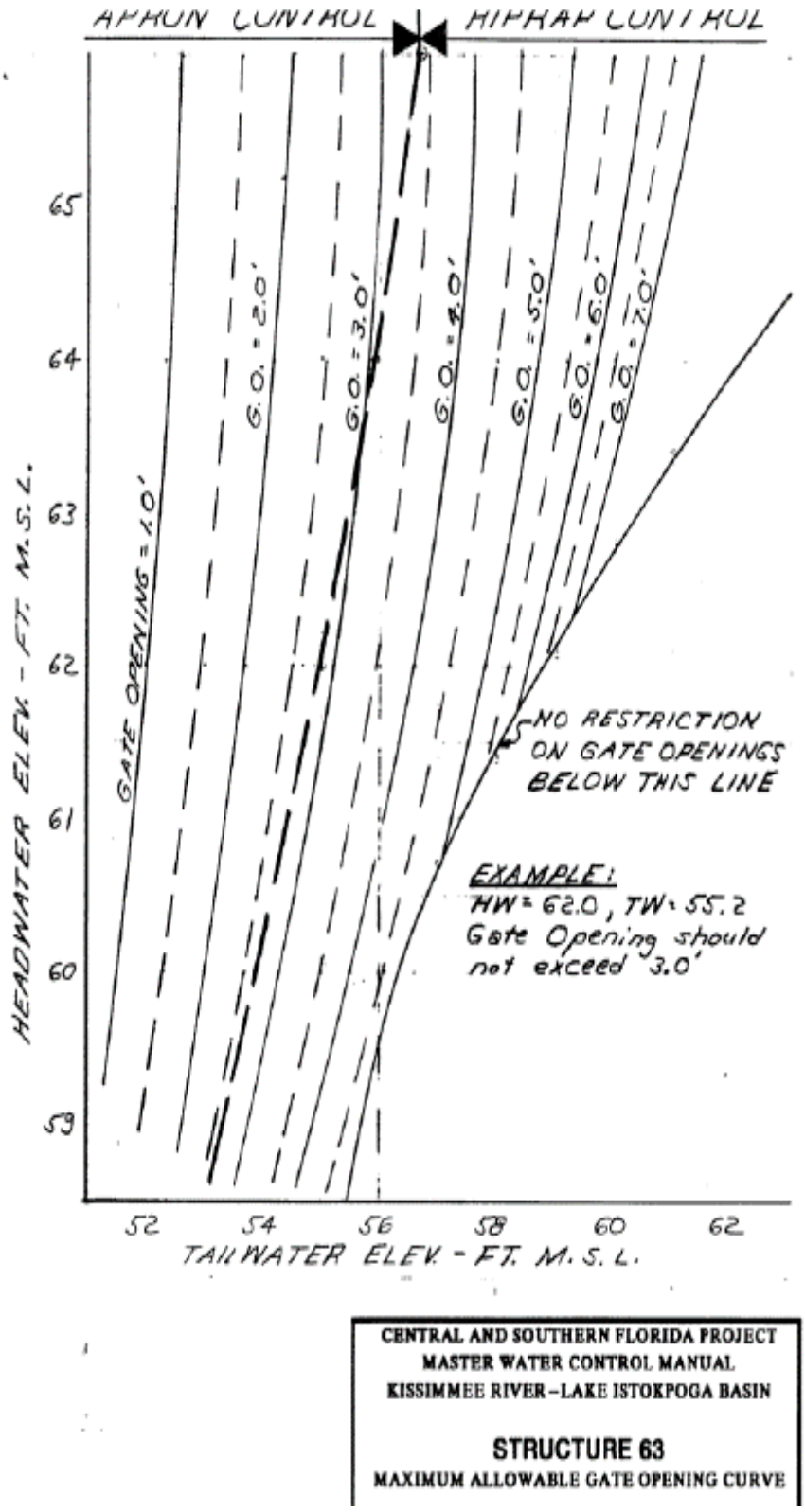


Figure 6-10: Structure S63 MAGO Curves

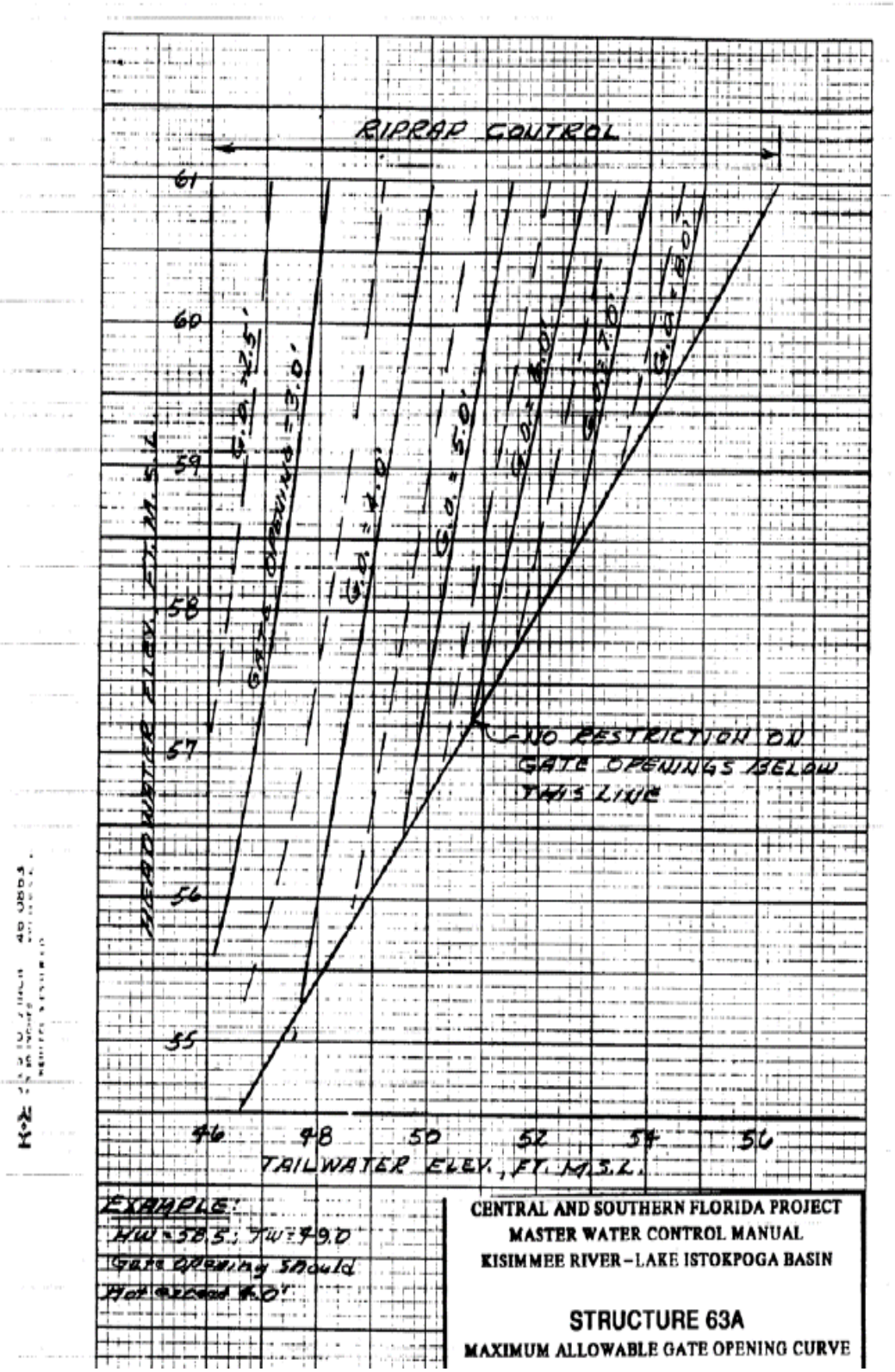


Figure 6-11: Structure S63A MAGO Curves

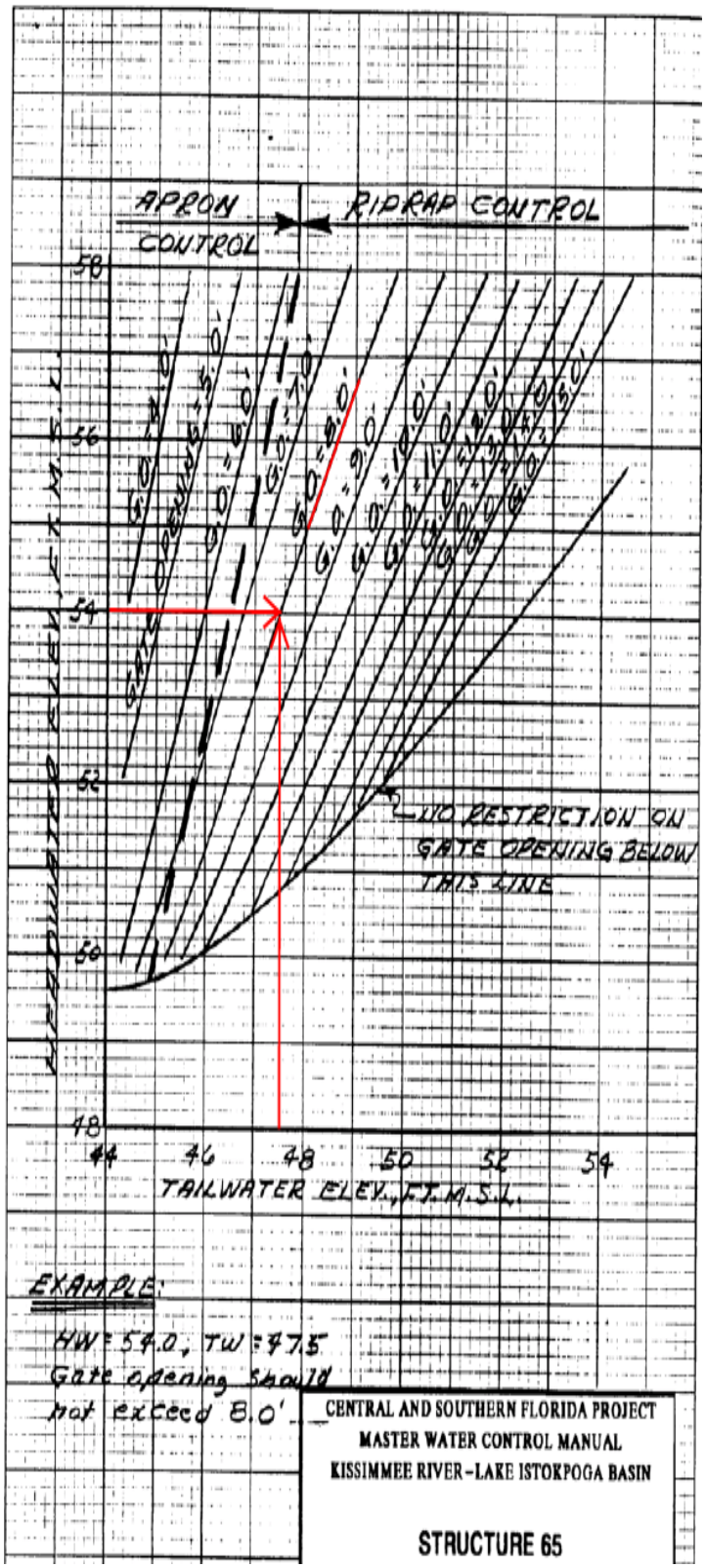


Figure 6-12: Structure S65 MAGO Curves

6.2.1.3 Flow Equations at Structures

The flows at UKB structures (spillways, culverts and weirs) are calculated by SFWMD using their well-documented flow equations (used in NEXFLOW program), based on relevant physical and hydraulic parameters for each site (SFWMD, 2015).

Spillways

Historically, UKB spillway flows (S59, S60, S61, S62, S63, S63A and S65) were computed using equations developed by USACE in the 1960s from small scale model testing of central Florida spillways. Flows through gated spillways are classified into 5 categories, based on the head water and tail water stages and the gate openings. The categories are listed below and depicted in **Figure 8-13 through Figure 8-17** (SFWMD, 2015) on the following pages.

- Controlled-free,
- Controlled-submerged,
- Uncontrolled-free,
- Uncontrolled-submerged, and
- Over-the-top flows.

Equations to compute flows were implemented at UKB spillways based on Ansar and Chen (2009), for the first four flow categories and the transitions between them. The SFWMD equations were coded in DHI software; but SFWMD revised the rating at structures after DHI's implementation.

Figures 8-13 through 8-17 show the spillway coefficients used in the UKB FPLOS calibration model, which were not modified for the current or the future conditions, unless there are structural changes. The a and b in the table refer to the coefficient and exponent respectively in the flow equations. The MIKE models developed by SFWMD and USACE use consistent coefficient values. These coefficients are based on established calibration procedures and ensure compatibility with other SFWMD MIKE model applications. While SFWMD has updated the rating curves at structures, DHI's model structure and coefficient calculations have remained unchanged.

It is important to note that the DHI implementation of spillway equations has only the Case-5 parameters. For Case-2 and Case-6, the default Case-5 parameters were used. The b term in DHI MIKE Zero interface is only used when needed.

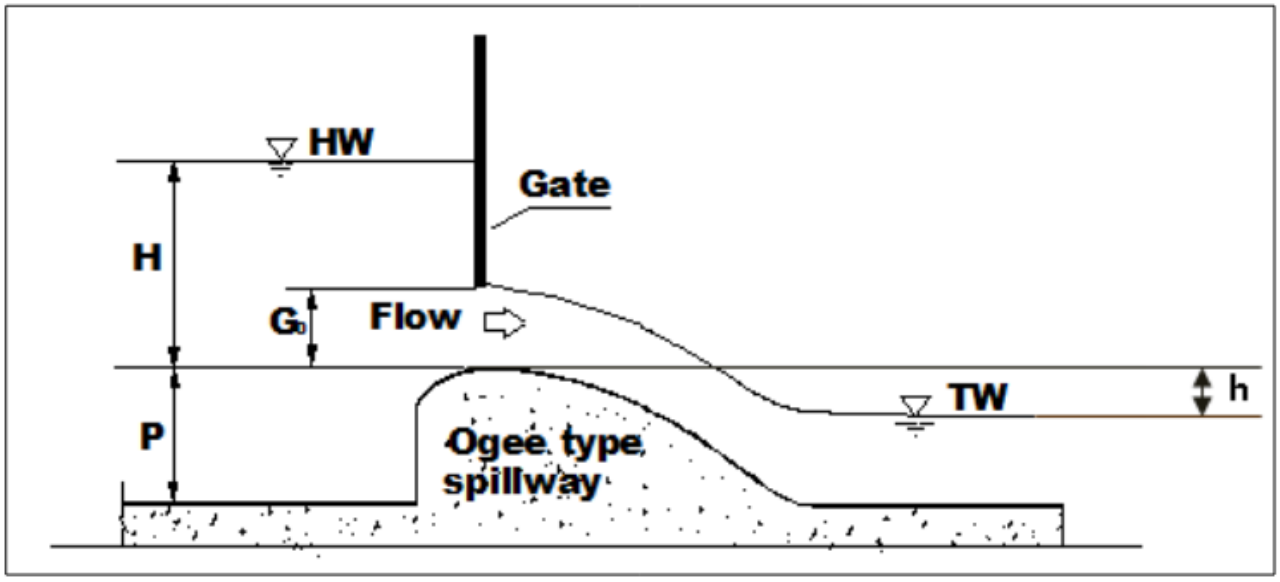


Figure 6-13: Controlled-Free Flow Regime

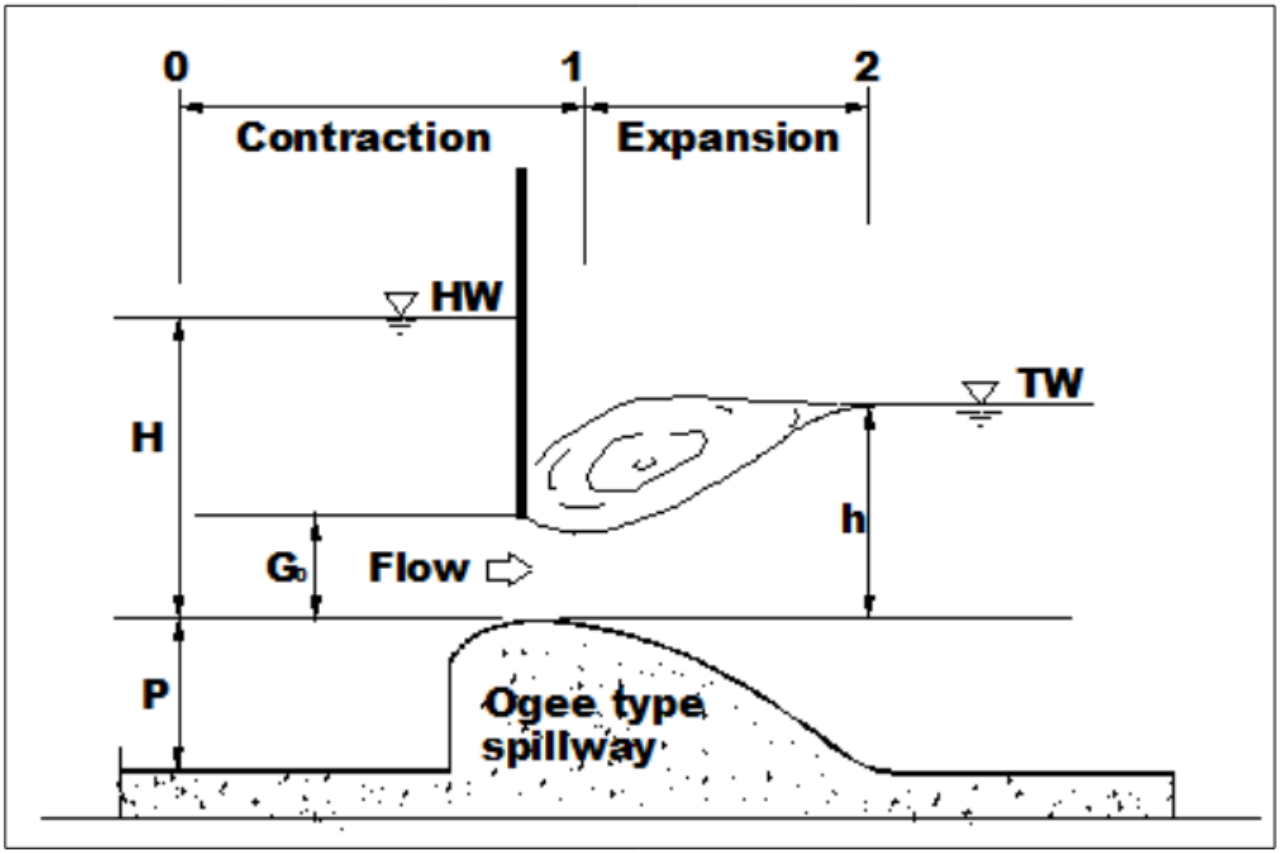


Figure 8-14: Controlled-Submerged Flow Regime

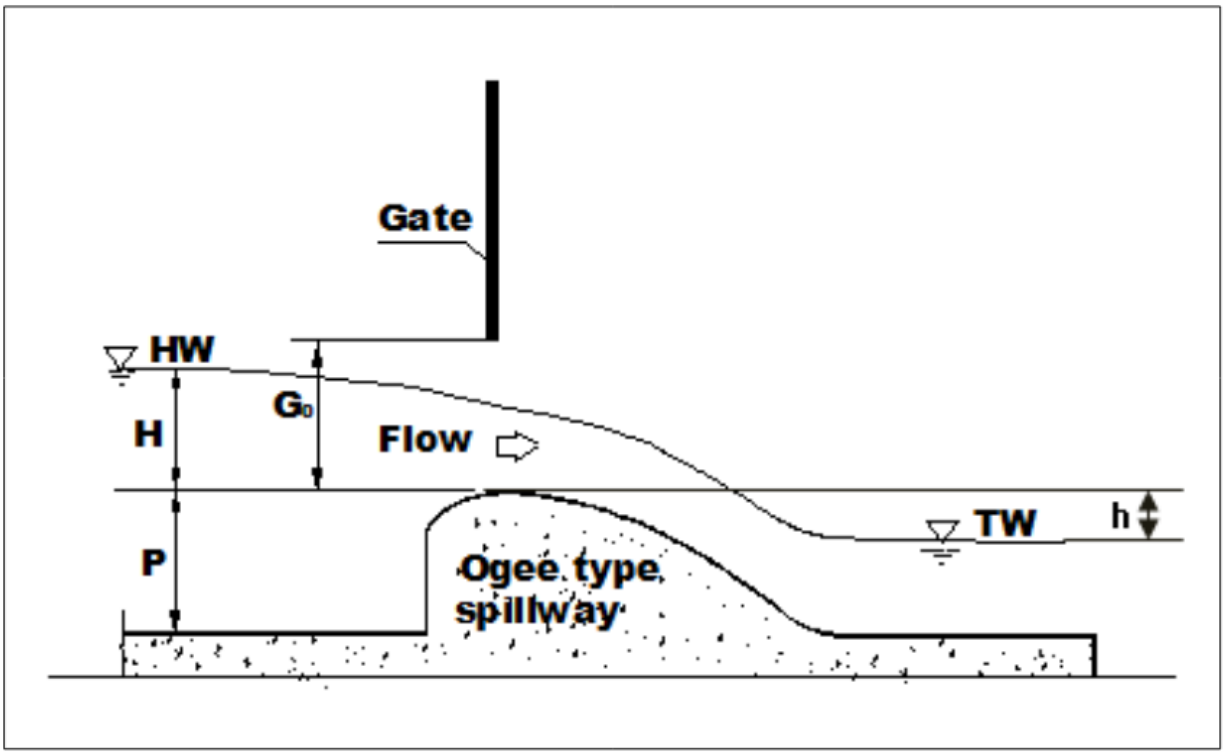


Figure 8-15: Uncontrolled-Free Flow Regime

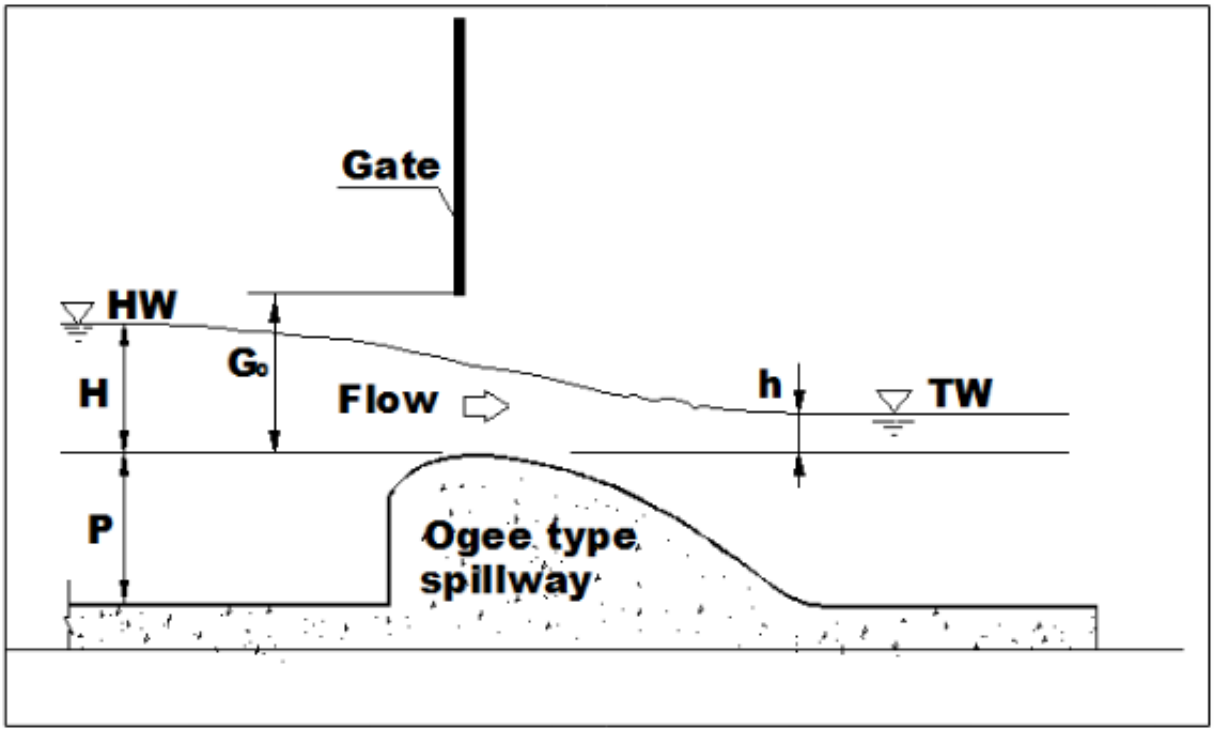


Figure 8-16: Uncontrolled-Submerged Flow Regime

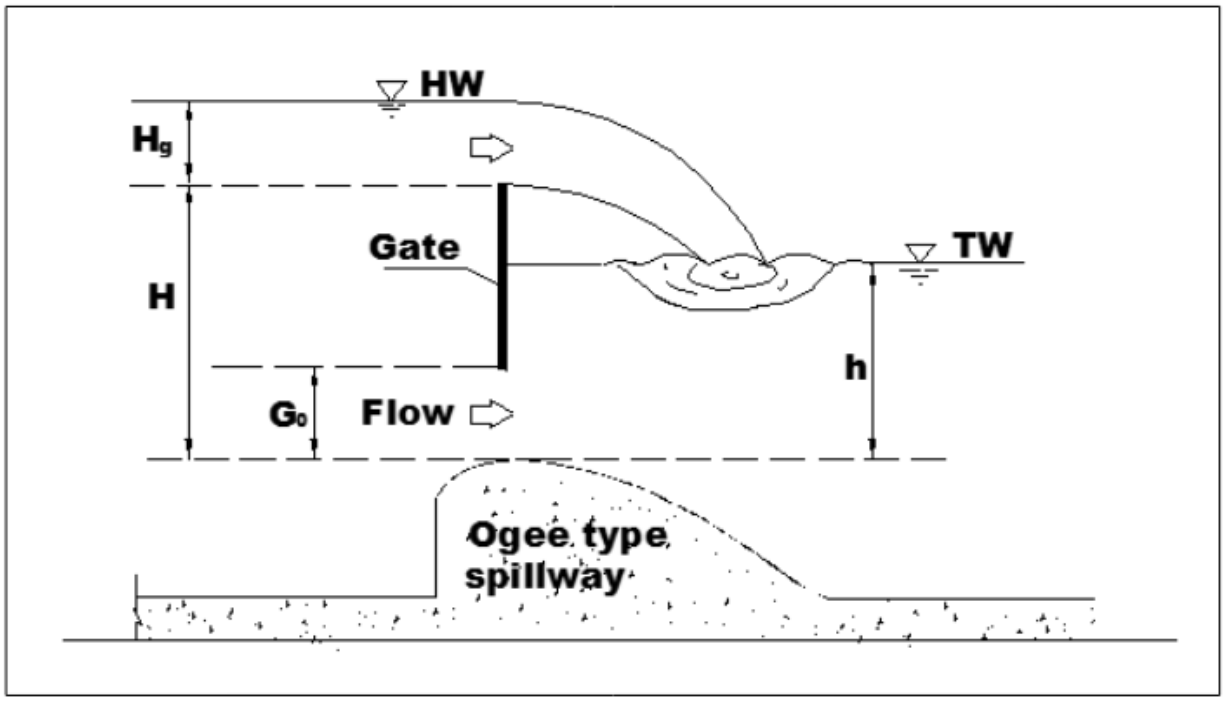


Figure 8-17: Over-the-Top Flow Regime

Culverts

There are 4 culverts in UKB: S57, S58, G113 and G111. SFWMD has developed rating equations for the five flow types in culverts and they are summarized in Atlas of Flow computations (SFWMD, 2015). They are: Open Channel Flow with critical depth at inlet, Open Channel Flow with critical depth at outlet, Open Channel Flow with tranquil flow, Full Pipe Flow and Orifice Flow. So SFWMD's culvert flow equations are based on head water, tail water, and gate opening conditions at each culvert. However, the energy equation in MIKE HYDRO was used to calculate the flows over the culverts.

Weirs

G103 is a five-bay sheet pile gated weir on Zipper Canal in UKB. SFWMD's free-flow weir equations predict unsubmerged flows over the weirs, while Villemonte's (1947) equation is used to describe the submerged flows (SFWMD, 2015) at the weirs. The energy equation in MIKE HYDRO was used to calculate the flows over G103. Differences between G103 flows computed by MIKE and SFWMD equations are and should be expected.

Hazen

Table 6-7: Case-5 Spillway Coefficients used in UKB FPLOS Modeled Structures

MIKE Structure Name	Controlled Submerged Flow				Controlled Free Flow				Uncontrolled Submerged Flow				Uncontrolled Free Flow
	CS-coef a	CS-exp b	CS-hi limit	CS-low limit	CF-coef a	CF-exp b	CF-hi limit	CF-lo limit	US-coef a	US exp b	US-hi limit	US-lo limit	UF-coef a
Armstrong Gate	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
G103	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
G111	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
G113	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
S-57_Gate	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
S-58_Gate	1.12	0.21	1.05	0.95	0.89	0.17	1.55	1.45	0.86	0.38	0.7	0.6	0.77
S-59_Gate	1.12	0.21	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S-60_Gate	1.12	0.21	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S-61_Gate	1	0.27	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S-62_Gate	1.12	0.21	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S-63_Gate	1.12	0.21	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S-63A_Gate	1.12	0.21	1.01	1	0.86	0.38	1.51	1.49	0.89	0.17	0.67	0.66	0.77
S65_Gate	1.04	0.3	1.01	1	0.86	0.35	1.51	1.49	0.838	0.167	0.67	0.66	0.7
S65A_Gate	1.04	0.3	1.01	1	0.86	0.35	1.51	1.49	0.838	0.167	0.67	0.66	0.7

6.2.2 Coding of Structure Operations

A sensor is a time-varying variable which is used as a parameter for a control rule. The sensors in UKB FPLOS track ongoing simulation conditions, and/or an external item read in a time series (observed data). The sensors keep track of water levels, discharges and gate levels for decision making; specifically, the head and tailwater at structures, discharges and gate levels at the spillways and culverts in UKB FPLOS model domain.

UKB FPLOS structures in the model were regulated with a number of rules. In general, for each rule, MIKE requires 2 items to be specified:

- A condition: Criteria to be fulfilled for the rule to be executed, and
- An action: The action to be performed by the structure when the specified condition is true

The gate level changes in the model are made every 15 minutes, even if the solver time steps are smaller; this is to prevent chatter in the hydrographs due to frequent changes in gate levels. **Table 6-8** shows the priority of structure operations that are coded in the current conditions model, for each structure.

Appendix K has the details of the structure operations in UKB FPLOS model.

Table 6-8: Priority of Structure Operations

Priority	Condition	Action	Details
1	If (HW-TW) ≤ MPHD	Gate Level Unchanged	
2	If (HW-TW) > MAGO	Set Gate Level to MAGO	
3	If HW > Max regulatory schedule and Time since last Gate Change ≥ 15 minutes	Open gate	Calculation of discharge to ramp down to regulation schedule
4	Default	Gate Level Unchanged	

6.3 Initial Conditions

Initial conditions are critical in modeling because they define the starting point of the system being modeled. They specify the state of the system at the beginning of the simulation and determine the behavior of the system over time. The integrated model requires initial conditions for surface water stages, groundwater levels and overland water depths, summarized in this section.

It is acceptable to use initial conditions from observed data in a storm event hydrologic model because these conditions provide valuable information about the specific characteristics of the storm event being modeled, and the response of the watershed to it. By using observed data, such as precipitation, lake levels and volumes, evapotranspiration, withdrawals, and other input data from Hurricane Irma, the model was

calibrated to stages and flows. This means the model can reasonably simulate the behavior of the hydrologic system during the storm event of similar characteristics in terms of spatial variation. In an integrated modeling tool like MIKE SHE – MIKE HYDRO, and a hydrologic system dominated by lakes like in the UKB, it is critical to have a complete system initialization to be in sync with each other. That is the value added by taking initial conditions from a model that was calibrated to the storm event, similar in spatial pattern like the precipitation that will be used in base conditions simulations. If the response of the watershed to the storm event is representative of the system, it is reasonable to use the data observed during the event as starting conditions. That is why Hurricane Irma initial conditions were used in the model for current and future conditions modeling. The rainfall events substituted in the place of the observed (and calibrated) rainfall, do vary in the simulations.

Observed data from 9/1/2017 were used for initial conditions in the model; the entire system is consistent with each other and the observed data. Hurricane Irma conditions were calibrated to, so there is confirmation that the system captures the responses to Irma well. September in south Florida is also typically wet/hurricane period. The initial conditions were fine-tuned during calibration using observed data where available and using simulated data when not available. All of these make 9/1/2017 a reliable date for initial conditions for design events.

6.3.1 Surface Water Stages

The initial values for surface water used the observed stages of 9/1/2017 used for the calibration model at structure headwater and tailwater locations and mid-lake levels. The model has a warm-up period of approximately 10 days for equilibration of water levels within canals, and exchanges with overland and saturated zones. At locations with no nearby observed data, the initial levels were set at 1 ft above the basin or canal average to provide numerical stability in simulation.

6.3.2 Groundwater

Groundwater levels were obtained from SFWMD, SJRWMD, and SWFWMD. Water levels from September 1, 2017, were spatially interpolated to raster, and 2D time-series dfs2 datasets were generated. These datasets provided continuous observed water levels consistent with those of the lakes and wetlands. Figure 5-26, Figure 5-27, Figure 5-28 and Figure 5-29 illustrate the initial water levels in saturated zone layers 1, 2, and 3; these levels were also used in the UKBFPLOS calibration model.

6.3.3 Overland

The UKB FPLOS MIKE model necessitates an initial overland water depth for optimal performance. Accurate initial water depths in KCOL are crucial for effective model operation and prediction. The initial water levels of observed lakes and canals were utilized as starting points, and simulations were conducted iteratively. The overland water depth from the stabilized model was then used in model calibration and in the development of the current and future conditions models.

6.4 Boundary Conditions

Boundary conditions define the limits or constraints of the UKB FPLOS system at its lateral and vertical boundaries. They specify how the system interacts with the areas outside of the UKBFPLOS model domain. Accurate boundary conditions are crucial for capturing the real-world behavior of the system and ensuring that the model accurately represents the system's response to external influences.

The design events model for current conditions simulations requires time-varying datasets for the boundary conditions of surface water, groundwater, and overland flow. The time series boundary conditions were applied at the specified surface water reaches, and boundary cells of the overland and groundwater model domain.

When selecting boundary conditions for a hydrologic model, it is important to ensure that they are necessary for accurately capturing the flow dynamics within the model domain. At the same time, boundary conditions should be placed far enough from the study area or main area of interest so that their effects on the model results are minimized. By striking a balance between necessity and distance, UKB FPLOS Model ensures that the hydrologic model accurately represents real-world conditions while minimizing the influence of boundary effects.

All of the above considerations were accounted for, when the model domain was selected, and the downstream boundary condition was placed downstream of S65A. Boundary effects do not extend upstream into the UKB study area. The gradient across S65 is generally in the order of many feet, so the determining factor for gate opening is the headwater of S65, and tailwater when MAGO and MPHD apply. This further confirms that the boundary effects do not extend that far upstream.

It is important to note that the boundary conditions are the same for all current conditions design events.

6.4.1 Surface Water Stages

The surface water boundaries were closed except for the stage boundary set downstream of S65A, where the calibration model used the observed stage data from the station FTKISS. FTKISS was chosen as the downstream boundary condition for the design events simulations due to its proximity to C-38 Canal, its location in a natural channel, and the availability of historical stage data. The UKB FPLOS Model domain includes the S65A watershed, and FTKISS is located downstream of S65A.

6.4.2 Groundwater

Groundwater levels were downloaded from SFWMD, SJRWMD and SWFWMD; 9/1/2017 levels were spatially interpolated to raster and 2D timeseries dfs2 datasets were generated for boundary conditions. These were used for calibration and current conditions simulations as well.

6.4.3 Overland

The UKB FPLOS MIKE model requires initial overland water depth. Initial water depths in KCOL are critical for model performance and prediction. Observed lakes and canal water levels were used, simulations

were done iteratively and the overland water depth from the stabilized model was used in the model calibration and will be used in the current conditions model.

6.5 Evapotranspiration

The calibration model used observed ET data time series from DBHydro, for the current conditions model (Hazen and Sawyer, 2023). ET values used in the model reflect those measured within Kissimmee Basin.

6.6 Groundwater Withdrawals

Groundwater withdrawals used in the current conditions calibrated model were not modified (Hazen and Sawyer, 2023).

6.7 Drainage Wells

Data on drainage wells, the pumpage/injection rates and their efficiency are not readily available. Drainage wells lose efficiency with time. No reliable data were found on the reduced rate of the drainage well, so they were not implemented in current conditions simulations.

6.8 Planned Projects

Planned projects (CERP, DWM, other) were considered, but none of them are operational before 2020, the date represented by the current conditions. So they were not implemented in the model.

7. Future Conditions Model Setup

The future conditions model was set up for 60 days and included a 30-day antecedent conditions prior to the simulation period, 7-day warm-up period, the three-day design storm event, and the following weeks to capture the recession hydrograph until it returns to baseflow. The future conditions model used the same antecedent conditions as the calibration model, which used observed conditions throughout the system as the hydrologic representation of the basin-wide antecedent conditions. One of the challenges unique to H&H simulations in Kissimmee Basin is formulation of antecedent conditions that are reasonable representations of the varying hydrology across the domain yet are relatively smooth enough for a software to execute without instabilities or incorrect solutions to the equations. Therefore, starting from the calibrated model for the entire domain provided the best basin-wide initial conditions. This section provides a summary of the model revisions and updates to set up the future conditions model, starting with the current conditions model (described in the previous section).

7.1 Design Rainfall

The calibration model, which was the basis of the design events model, used gage data for Hurricane Irma (Sep 10-12, 2017). The calibration model was set up to simulate August 1st to October 30th, 2017; the future conditions model used the same antecedent conditions as of September 1st, thereby providing the beginning of September as warm-up period. For the future conditions design storm simulations, the Hurricane Irma rainfall was replaced by rainfall that represents the spatially distributed 3-day design storm rainfall distribution by SFWMD.

7.1.1 Construction of Design Rainfall for Future Conditions Model from Current Conditions Model

Rainfall time series grids generated for current conditions model was the starting point of the UKB FPLOS future conditions rainfall climate drivers. The change factors in **Table 7-1** recommended in SFWMD reports on future climate scenarios (Irizarry-Ortiz, M.M. and J.F. Stamm. 2022; SFWMD 2022d) were used to generate the future conditions rainfall time series grids.

Table 7-1: Change Factors used to Produce Future Conditions Rainfall Datasets

Storm Event Frequency	Change Factor *
5-yr	1.13
10-yr	1.16
25-yr	1.2
100-yr	1.26

***** *Change factors based on Table 2 of Technical Memorandum: 'ADOPTION OF FUTURE EXTREME RAINFALL CHANGE FACTORS FOR FLOOD RESILIENCY PLANNING IN SOUTH FLORIDA April 27, 2022*

Table 7-2 through Table 7-5 provide the summary statistics for the 26 watersheds for the selected 5-, 10-, 25-, and the 100-yr return periods used in future conditions model. **Figure 7-1 through Figure 7-4** provide the spatial distribution of the storm events for the same. The data shows the total three-day rainfall distribution over all KB watersheds.

Table 7-2: Design Rainfall - Statistics for 5-yr 72-hr Event UKB FPLOS Model Domain (Future Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	7.39	7.53	0.14	7.45	0.03
Tiger Lake	7.30	7.37	0.07	7.32	0.01
Lake Rosalie	7.25	7.31	0.06	7.28	0.01
Lake Kissimmee	7.30	7.65	0.35	7.48	0.09
Lake Jackson	7.54	7.72	0.18	7.63	0.05
Lake Pierce	7.27	7.29	0.02	7.28	0.01
Lake Gentry	7.40	7.46	0.06	7.41	0.01
Horse Creek (Closed Basin)	7.18	7.28	0.10	7.24	0.02
Lake Cypress	7.32	7.49	0.17	7.38	0.03
Lake Marian	7.63	7.81	0.18	7.72	0.05
Marion Creek	7.25	7.38	0.12	7.30	0.02
Catfish Creek	7.25	7.28	0.03	7.25	0.01
Lake Conlin	7.41	7.54	0.12	7.46	0.03
Lake Marion	7.25	7.30	0.03	7.27	0.01
Lake Hatchineha	7.25	7.35	0.09	7.29	0.02
Lower Reedy Creek	7.27	7.49	0.23	7.36	0.06
Upper Reedy Creek	7.12	7.39	0.27	7.23	0.07
Lake Tohopekaliga	7.39	7.58	0.19	7.49	0.06
Lake Myrtle	7.45	7.55	0.10	7.49	0.02
East Lake Tohopekaliga	7.42	7.58	0.16	7.51	0.03
Alligator Lake	7.40	7.49	0.09	7.42	0.02
Shingle Creek	7.31	7.55	0.24	7.41	0.06
Lake Hart	7.50	7.57	0.07	7.53	0.01
Lake Weohyakapka	7.24	7.37	0.12	7.28	0.02
Boggy Creek	7.38	7.55	0.17	7.47	0.05
Econlockhatchee River Swamp	7.48	7.57	0.09	7.54	0.02

Table 7-3: Design Rainfall - Statistics for 10-yr 72-hr Event for UKB FPLOS Model Domain (Future Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	9.05	9.23	0.17	9.12	0.05
Tiger Lake	8.86	8.96	0.08	8.90	0.02
Lake Rosalie	8.85	8.91	0.07	8.86	0.01
Lake Kissimmee	8.90	9.33	0.43	9.12	0.12
Lake Jackson	9.23	9.45	0.22	9.34	0.06
Lake Pierce	8.86	8.92	0.06	8.90	0.01
Lake Gentry	9.07	9.15	0.08	9.11	0.02
Horse Creek (Closed Basin)	8.84	8.94	0.10	8.91	0.02
Lake Cypress	8.94	9.16	0.22	9.02	0.05
Lake Marian	9.21	9.52	0.31	9.41	0.08
Marion Creek	8.87	9.05	0.19	8.94	0.03
Catfish Creek	8.85	8.89	0.03	8.86	0.01
Lake Conlin	9.11	9.29	0.17	9.18	0.05
Lake Marion	8.87	8.96	0.08	8.91	0.01
Lake Hatchineha	8.86	8.97	0.12	8.90	0.02
Lower Reedy Creek	8.91	9.22	0.32	9.04	0.08
Upper Reedy Creek	8.78	9.11	0.32	8.91	0.08
Lake Tohopekaliga	9.06	9.36	0.30	9.21	0.08
Lake Myrtle	9.16	9.31	0.15	9.23	0.03
East Lake Tohopekaliga	9.12	9.35	0.23	9.26	0.06
Alligator Lake	9.09	9.23	0.14	9.12	0.03
Shingle Creek	9.00	9.31	0.32	9.14	0.08
Lake Hart	9.26	9.37	0.12	9.29	0.02
Lake Weohyakapka	8.77	8.93	0.16	8.85	0.03
Boggy Creek	9.09	9.33	0.23	9.22	0.06
Econlockhatchee River Swamp	9.21	9.35	0.13	9.29	0.03

Table 7-4: Design Rainfall - Statistics for 25-yr 72-hr Event for UKB FPLOS Model Domain (Future Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	11.71	11.94	0.23	11.81	0.05
Tiger Lake	11.29	11.45	0.16	11.38	0.04
Lake Rosalie	11.30	11.45	0.14	11.38	0.04
Lake Kissimmee	11.34	11.99	0.65	11.69	0.16
Lake Jackson	11.90	12.24	0.35	12.06	0.08
Lake Pierce	11.36	11.57	0.20	11.47	0.05
Lake Gentry	11.76	11.90	0.14	11.83	0.04
Horse Creek (Closed Basin)	11.56	11.66	0.11	11.63	0.02
Lake Cypress	11.56	11.81	0.25	11.66	0.06
Lake Marian	11.72	12.34	0.62	12.08	0.14
Marion Creek	11.46	11.75	0.29	11.59	0.06
Catfish Creek	11.41	11.47	0.07	11.44	0.01
Lake Conlin	11.86	12.11	0.25	11.94	0.06
Lake Marion	11.47	11.63	0.16	11.57	0.04
Lake Hatchineha	11.44	11.57	0.13	11.48	0.04
Lower Reedy Creek	11.51	12.00	0.49	11.74	0.12
Upper Reedy Creek	11.50	11.86	0.36	11.62	0.08
Lake Tohopekaliga	11.75	12.18	0.43	11.98	0.12
Lake Myrtle	11.93	12.14	0.22	12.01	0.05
East Lake Tohopekaliga	11.86	12.17	0.30	12.04	0.07
Alligator Lake	11.81	12.01	0.20	11.87	0.05
Shingle Creek	11.68	12.12	0.44	11.88	0.12
Lake Hart	12.04	12.24	0.20	12.11	0.05
Lake Weohyakapka	11.12	11.48	0.36	11.30	0.08
Boggy Creek	11.78	12.12	0.34	11.98	0.08
Econlockhatchee River Swamp	12.00	12.19	0.19	12.11	0.05

Table 7-5: Design Rainfall - Statistics for 100-yr 72-hr Event UKB FPLOS Model Domain (Future Conditions)

Name	Minimum (inches)	Maximum (inches)	Range (inches)	Mean (inches)	St. Dev. (inches)
S63A	16.80	17.14	0.34	16.93	0.08
Tiger Lake	15.83	16.20	0.38	16.03	0.10
Lake Rosalie	15.91	16.29	0.38	16.14	0.10
Lake Kissimmee	15.88	17.15	1.27	16.51	0.26
Lake Jackson	16.82	17.59	0.78	17.20	0.19
Lake Pierce	16.09	16.68	0.59	16.42	0.14
Lake Gentry	16.91	17.26	0.34	17.07	0.08
Horse Creek (Closed Basin)	16.83	16.93	0.10	16.91	0.03
Lake Cypress	16.54	16.97	0.43	16.72	0.10
Lake Marian	16.47	17.82	1.35	17.20	0.32
Marion Creek	16.44	16.93	0.49	16.70	0.11
Catfish Creek	16.28	16.46	0.18	16.35	0.04
Lake Conlin	17.16	17.54	0.37	17.31	0.09
Lake Marion	16.47	16.86	0.39	16.68	0.10
Lake Hatchineha	16.28	16.59	0.32	16.46	0.06
Lower Reedy Creek	16.51	17.29	0.79	16.95	0.18
Upper Reedy Creek	16.73	17.12	0.40	16.85	0.09
Lake Tohopekaliga	16.91	17.51	0.60	17.27	0.16
Lake Myrtle	17.26	17.56	0.30	17.38	0.08
East Lake Tohopekaliga	17.16	17.49	0.33	17.34	0.06
Alligator Lake	17.02	17.33	0.30	17.16	0.06
Shingle Creek	16.76	17.43	0.68	17.07	0.20
Lake Hart	17.29	17.74	0.45	17.46	0.11
Lake Weohyakapka	15.47	16.37	0.89	15.90	0.23
Boggy Creek	16.86	17.41	0.55	17.17	0.14
Econlockhatchee River Swamp	17.38	17.65	0.28	17.53	0.08

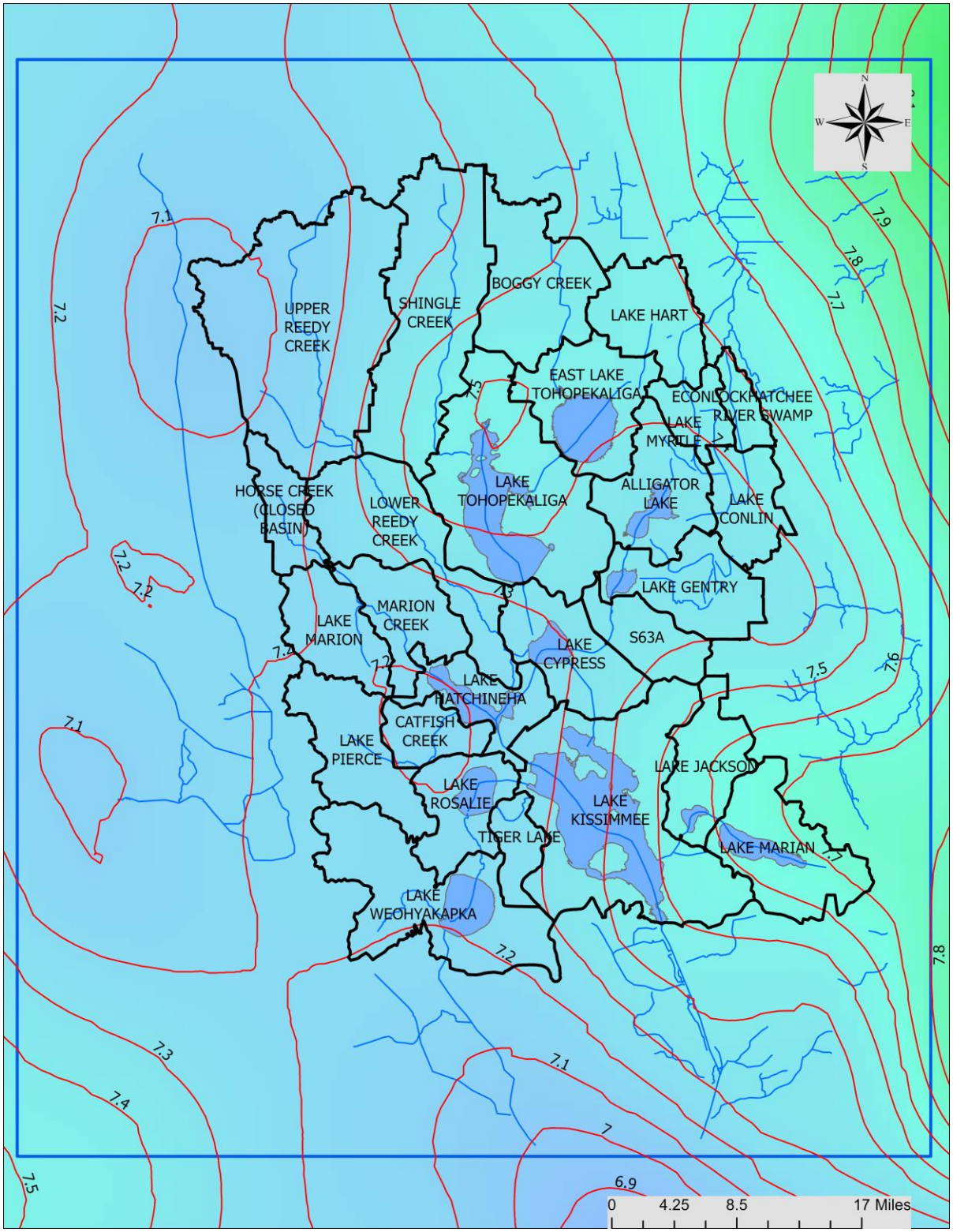


Figure 7-1: Rainfall Distribution for 5-yr 3-day Design Event (contours represent total rainfall in inches)

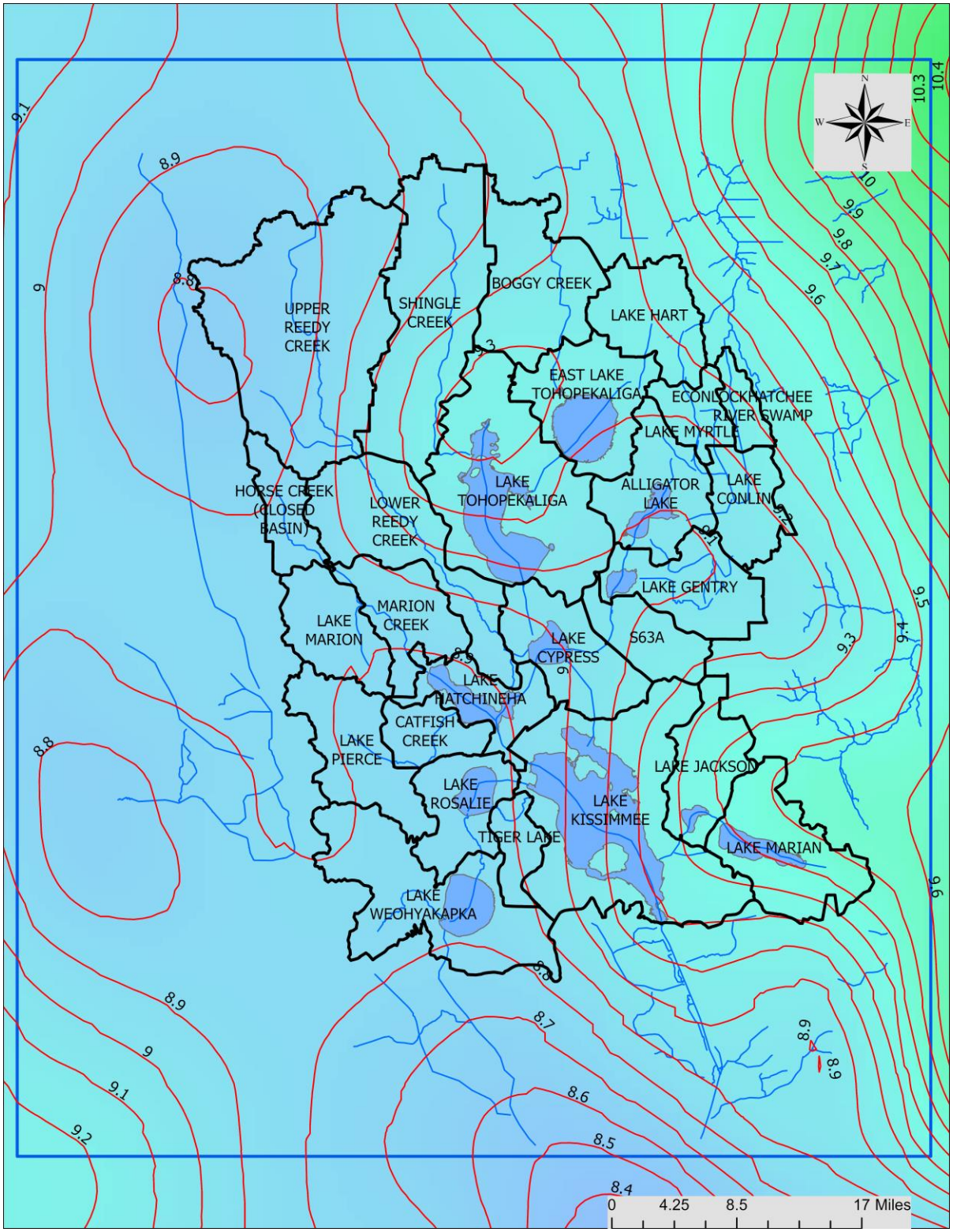


Figure 7-2: Rainfall Distribution for 10-yr 3-day Design Event
(contours represent total rainfall in inches)

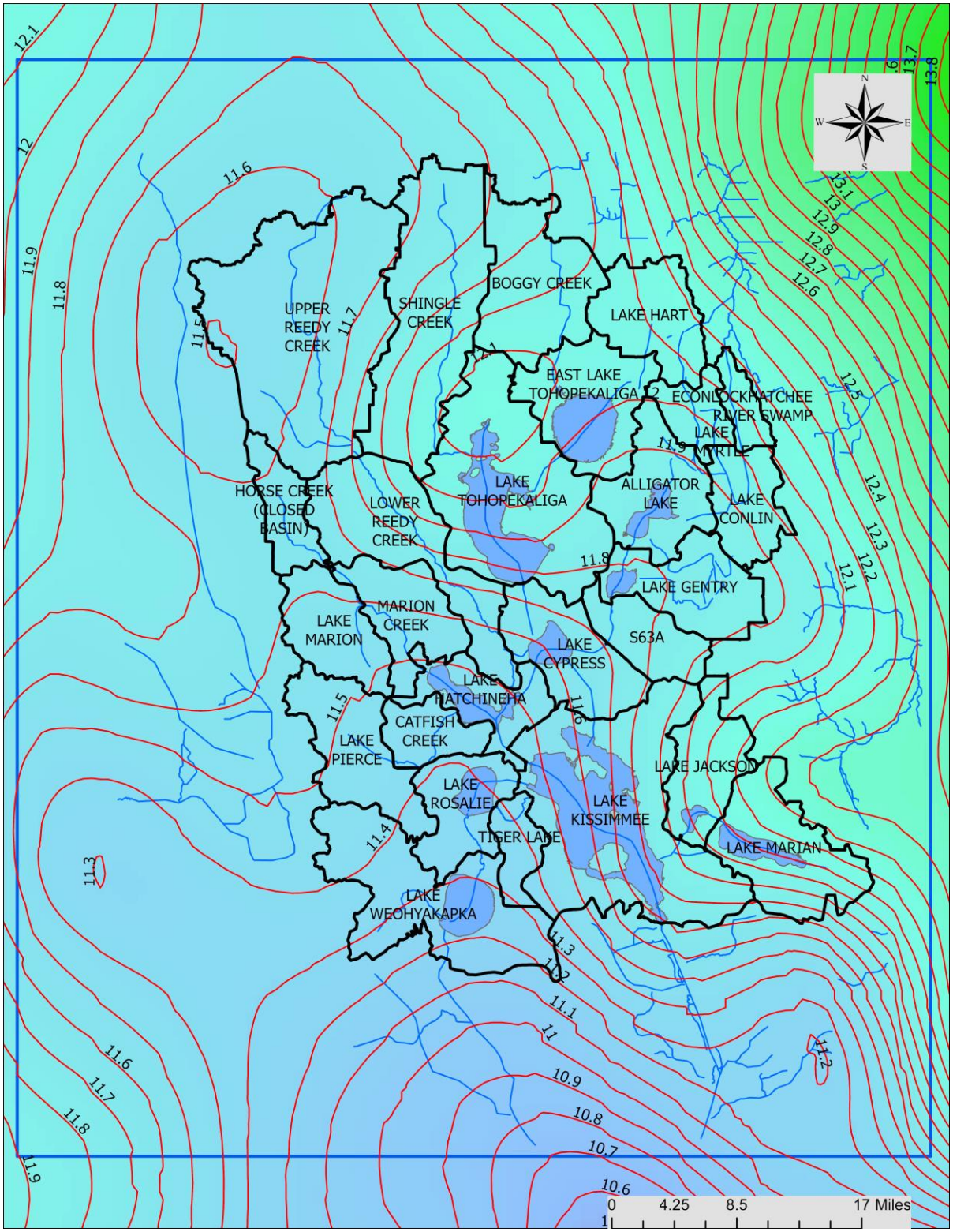


Figure 7-3: Rainfall Distribution for 25-yr 3-day Design Event
(contours represent total rainfall in inches)

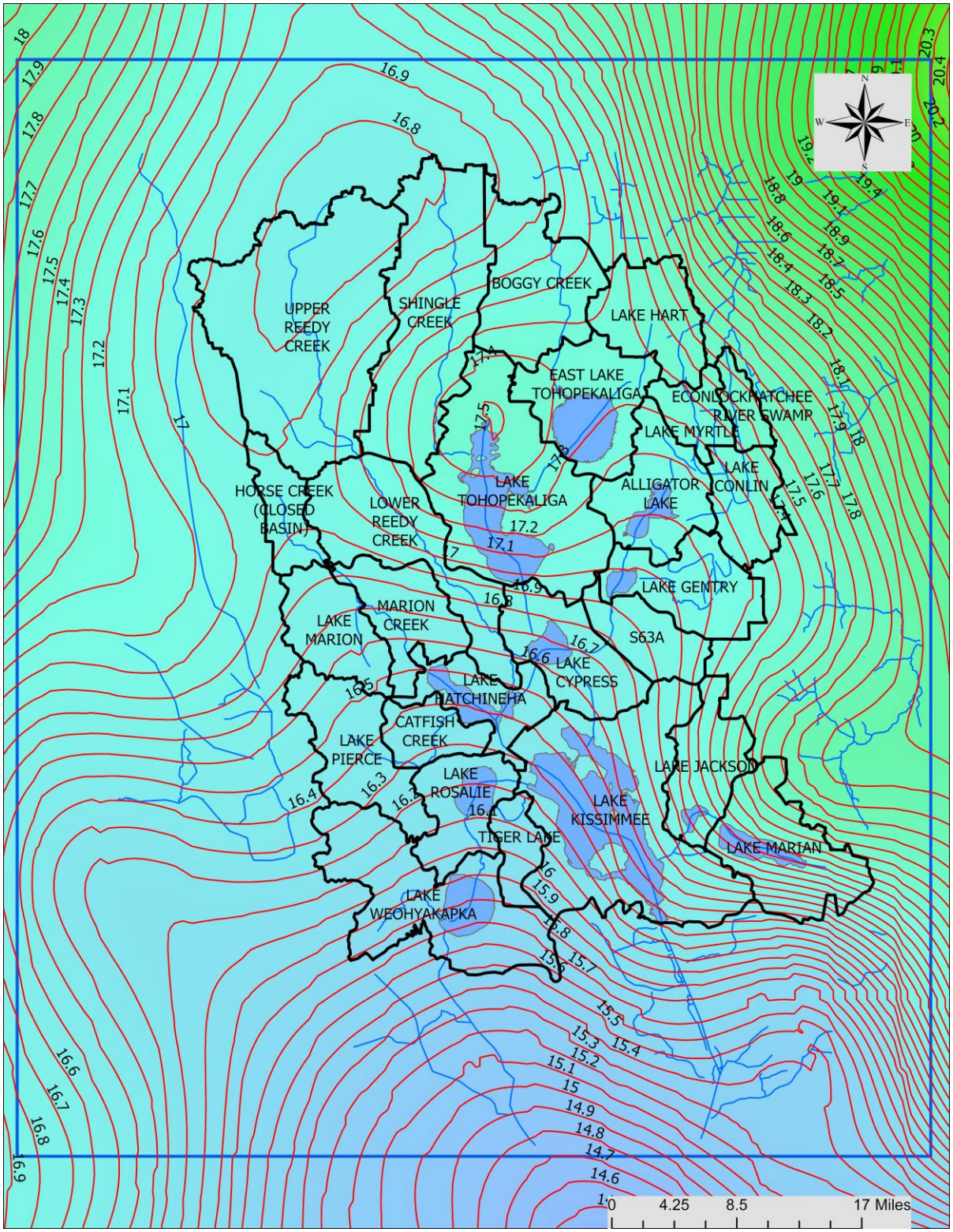


Figure 7-4: Rainfall Distribution for 100-yr 3-day Design Event
(contours represent total rainfall in inches)

7.2 Structure Operations

The structure operations for the UKB FPLOS Future Conditions model were modified only for the K-C-H system, with S65 being the outlet for this system and the entire UKB. The S65 structure at the outlet of Lake Kissimmee discharges all the UKB flows to the LKB. S65 regulates the 3 lakes in the LKB – lakes Cypress, Hatchineha and Kissimmee; there are no structures separating these 3 lakes. The regulation schedule, called the Headwaters Revitalization Schedule (HRS) at the S65 structure is shown in **Figure 7-5**.

Currently, the KCH system is regulated by the Interim Operating Schedule, also shown in Figure 7-5. The USACE current plan is for the first increment of HRS to be in place by wet season of 2024 and implemented fully by 2025 (SFWMD, 2023; Koebel et al., 2023). The intent of the HRS is to raise the flood control line and provide additional storage of 100,000 acre-feet. The higher water levels of the HRS will benefit the littoral zone (USACE, 1996).

For flood control operations, S65 is operated to return the lake to regulation schedule when the water level in Lake Kissimmee exceeds regulation schedule and is within 0.5 feet above the regulation schedule, as shown in Figure 7-5. The releases are made with maximum releases subject to MAGO and MPHD constraints (SFWMD, 2022c).

When Lake Kissimmee water level exceeds the regulation schedule and is within 0.5 feet above the regulation schedule, a release schedule, with MAGO and MPHD constraints (SFWMD, 2022c), is followed to return the lake to regulation schedule within 15 days. Releases at S65 range from 3,000 cfs to 11,000 cfs, depending on watershed runoff between S65 and S65A so flow at S65A does not exceed 11,000 cfs. The 3000 cfs is called the firm capacity, which is the discharge that can be made at S65 at any time, as needed. Discharges above the firm capacity can be made only if there is no flooding in the LKB.

When Lake Kissimmee water levels are below the HRS schedule line, releases are made following the quantities in the schedule to meet needs of the river restoration, navigation and water supply. The target minimum flow to the Kissimmee River is 300 cfs at S65A (SFWMD, 2022d).

Although Kissimmee LMA has a complex operating schedule and zones of releases, the focus of this project is above the regulatory schedule line, which is how the structure is operated under flood control mode.

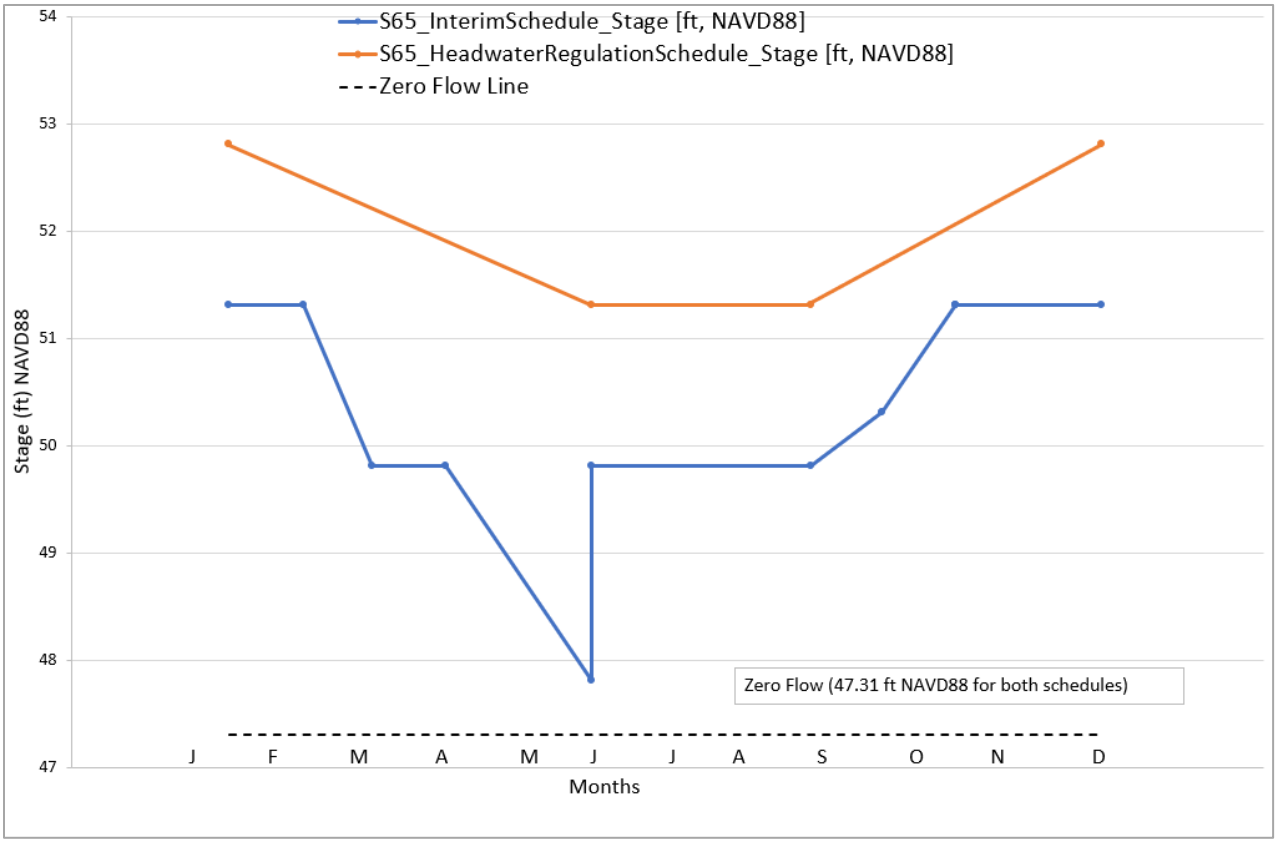


Figure 7-5: Lake Kissimmee, Hatchineha, and Cypress Regulation Schedule at S65 [NAVD88 = NGVD29 – 1.19 Feet]

Details of the structure operations can be found in **Appendix K**.

7.3 Initial Conditions

The integrated model requires initial conditions for surface water and groundwater conditions, and overland water depths (or levels). The initial values for surface water, groundwater and overland depths used in the calibrated model were used in the current conditions and the future conditions model, except for Lake Kissimmee. Lake Kissimmee used HRS in future conditions model, the maximum regulation schedule in HRS is 1.5 ft higher than the Interim Regulation Schedule used in current conditions model. So, Lake Kissimmee initial stages are 1 ft higher than that of current conditions model.

7.4 Boundary Conditions

The design events model for future conditions simulations requires time-varying datasets for the boundary conditions of surface water, groundwater, and overland flow. The timeseries boundary conditions applied at the specified surface water reaches, and boundary cells of the overland and groundwater model domain were not modified from current conditions. The same boundary conditions applied to all design events.

7.5 Evapotranspiration

Evapotranspiration was not modified from current conditions.

7.6 Groundwater Withdrawals

Groundwater withdrawals were not modified from current conditions.

7.7 Drainage Wells

Drainage wells were not implemented in future conditions, the same as current conditions.

7.8 Planned Projects in UKB

Planned projects (Central Everglades Restoration Planning (CERP), Dispersed Water Management (DWM), Kissimmee Reservations, other) will be included if they will be operational Circa 2070 (Date for future base conditions).

7.8.1 Water Reservations for Kissimmee River and Chain of Lakes

SFWMD developed the Kissimmee River and Chain of Lakes Water Reservations to protect fish and wildlife. According to the technical document on the reservations, "Protection of fish and wildlife means ensuring the health and sustainability of fish and wildlife communities through natural cycles of drought, flood, and population variation" (SFWMD, 2020). The proposed Water Reservation includes:

- Upper Chain of Lakes (Lakes Hart and Mary Jane; Lakes PMJ; Lake E. Toho; Lake Toho; the Alligator Chain of Lakes; and Lake Gentry)
- Headwaters Revitalization Lakes (KCH System and Tiger Lake)
- Kissimmee River and floodplain as well as interconnected canals.

The Headwaters Revitalization Lakes are directly associated with the performance of the Kissimmee River Restoration Project (KRRP) (USACE, 1996) and have a federal regulation schedule intended to meet the flow requirements of the KRRP. The KRRP was developed to address concerns about the adverse effects of the C & SF Flood Control Project on the Kissimmee River - specifically the altered hydrology, loss of floodplain wetlands and their habitat and reduced populations of several species of fish and wildlife. Direct and indirect withdrawals of water from the UKCOL and contributing water bodies will be limited to existing legal use allocations and quantities of surface water up to the proposed Water Reservation (WR) stages given in the draft Water Reservation rules (SFWMD, 2020).

The Water Reservation Line (WRL) seasonal high stage is the upper stage limit or threshold which preserves the maximum littoral extent in each UKCOL waterbody and ensures no reduction in wetland extent will occur below that defined stage elevation. For all UKCOL reservation waterbodies, the WR document specifies the seasonal high stage as the upper stage limit of the current stage regulation schedule and to occur starting on the first day (November 1st) the regulation schedule allows that stage to be reached.

The KRR process established the WRL seasonal high stages at the same stage and timing as the authorized regulation schedule, in order to capture the water levels needed to maintain the current shoreward extent of littoral/wetland vegetation in the KCOL waterbodies. Water levels do occasionally exceed the regulatory schedule lines in UKCOL, which trigger flood control releases (SFWMD, 2020).

The WRL does not take priority over the flood control release line. Therefore, the WRL line was not coded into a model which focuses on assessment of flooding impacts.

7.8.2 DWM projects

Dispersed Water Management (DWM) projects are designed to store and treat excess surface water from the regional system on private lands, which would otherwise discharge to Lake Okeechobee. There are 3 that were considered for inclusion in this model: El Maximo, Eagle Haven Ranch and Partin Family Ranch. The dates of the operations of these projects are 2020 and after; therefore, they were not included in the current conditions model (2020). They were included in the UKB FPLOS future conditions model.

Figure 7-6 on the following page shows the locations of all DWM projects within the SFWMD jurisdictional area; these are the 3 projects of interest:

- El Maximo: This is outside of the UKB FPLOS study area and therefore was not implemented.
- Eagle Haven Ranch (Lost Oak Ranch): This is located inside the UKB FPLOS study area. This DWM is operational and operates passively, with an estimated storage capacity of 374 acre-feet per year. There isn't a formal operations plan for this DWM, but there are control elevations (service elevations) at control structures and discharge culverts within each water management area (WMA) on the property. **Table 7-6** shows the structures and control elevations, as outlined in the contract with SFWMD. These service elevations were used to estimate retention within the project; and any water level above these elevations will result in project discharge (runoff). This information was used to modify the MIKE HYDRO cross sections and topography datasets in the model.
- Partin Family Ranch: This is located inside the UKB FPLOS study area. This was included in the model, with modifications in MIKE HYDRO cross sections and DEM.

Table 7-6: Water Control Structures and Control Elevations at Eagle Haven Ranch

Structure	Location	Latitude	Longitude	Service Elevation ft NAVD88	Service Elevation ft NGVD 29
S-3	WMA5	27.8264	-81.232	53.74	54.93
S-4	WMA4	27.836	-81.2334	54.33	55.52
S-5	WMA3	27.8359	-81.2532	58.4	59.59
S-7	WMA2	27.8409	-81.2589	55.94	57.13
S-9	WMA1	27.8431	-81.2654	55.9	57.09

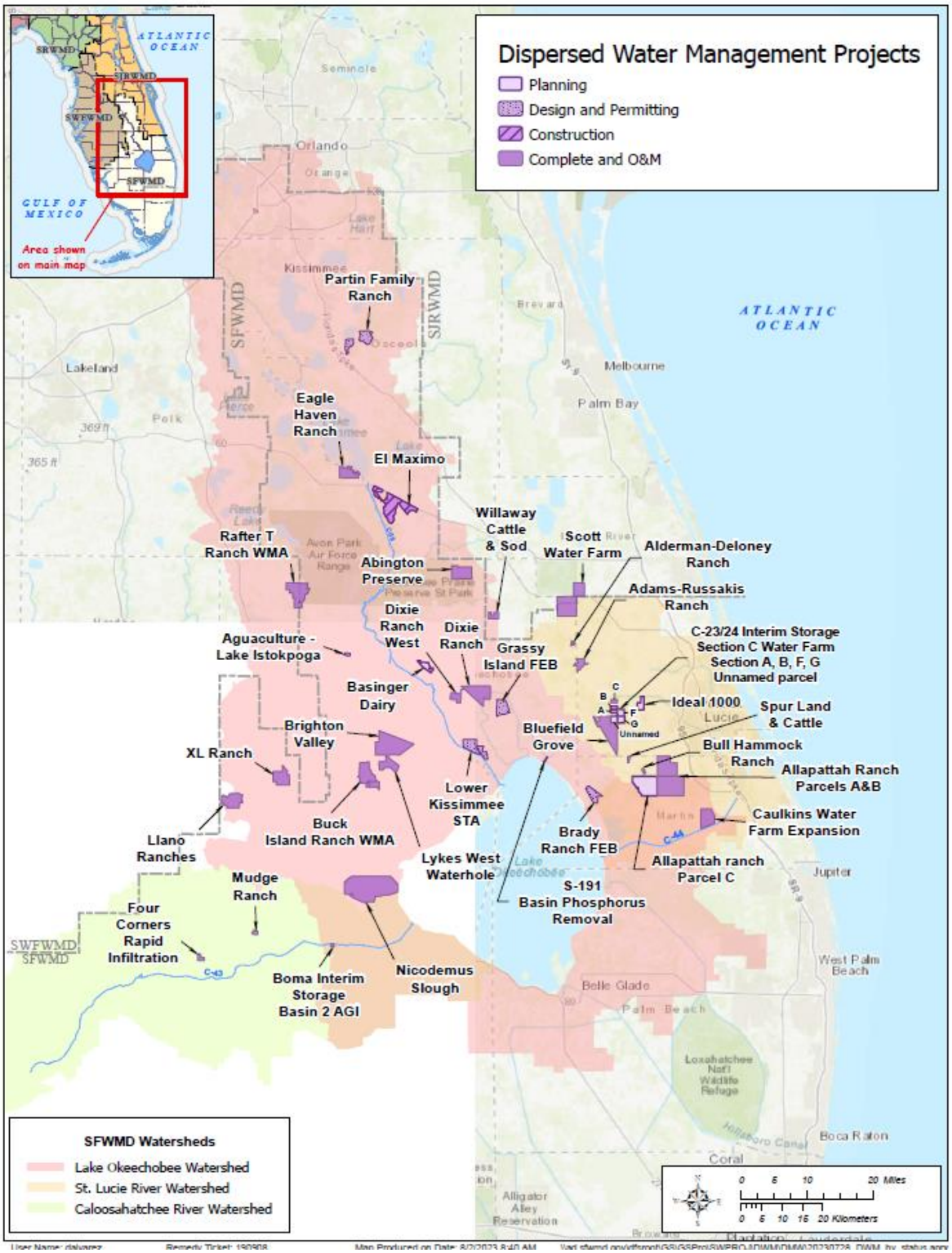


Figure 7-6: DWM Projects within SFWMD Jurisdictional

The projects were implemented based on the following assumptions:

- All projects are above-ground impoundments, which will require additional grading for the creation of levees and flattening the reservoir bottoms.
- Each above-ground impoundment features control inlet and outlet structures, facilitating the management of residence time storage and release. Inflow and outflow canal branches are provided for this purpose, integrating the projects into the drainage system.
- Presently, gravity inflows and outflows of the structures are implemented with no control. Essentially, water naturally flows into the project area and is released naturally. As a result, each project area uses uncontrolled storage.
- The storage areas are represented with MIKE HYDRO cross sections and additionally the future topography was modified for these areas to reflect the bottom grading to specified elevations.
- There is no data available that could be used as initial condition for the DWM projects, so the model assumes the conditions as of 9/1/17.

7.8.3 MIKE HYDRO Network

As described in the previous section, DWM projects were implemented by modifications to MIKE HYDRO as shown in **Figure 7-7**. The modifications were made to the network and cross-sections, based on available information.

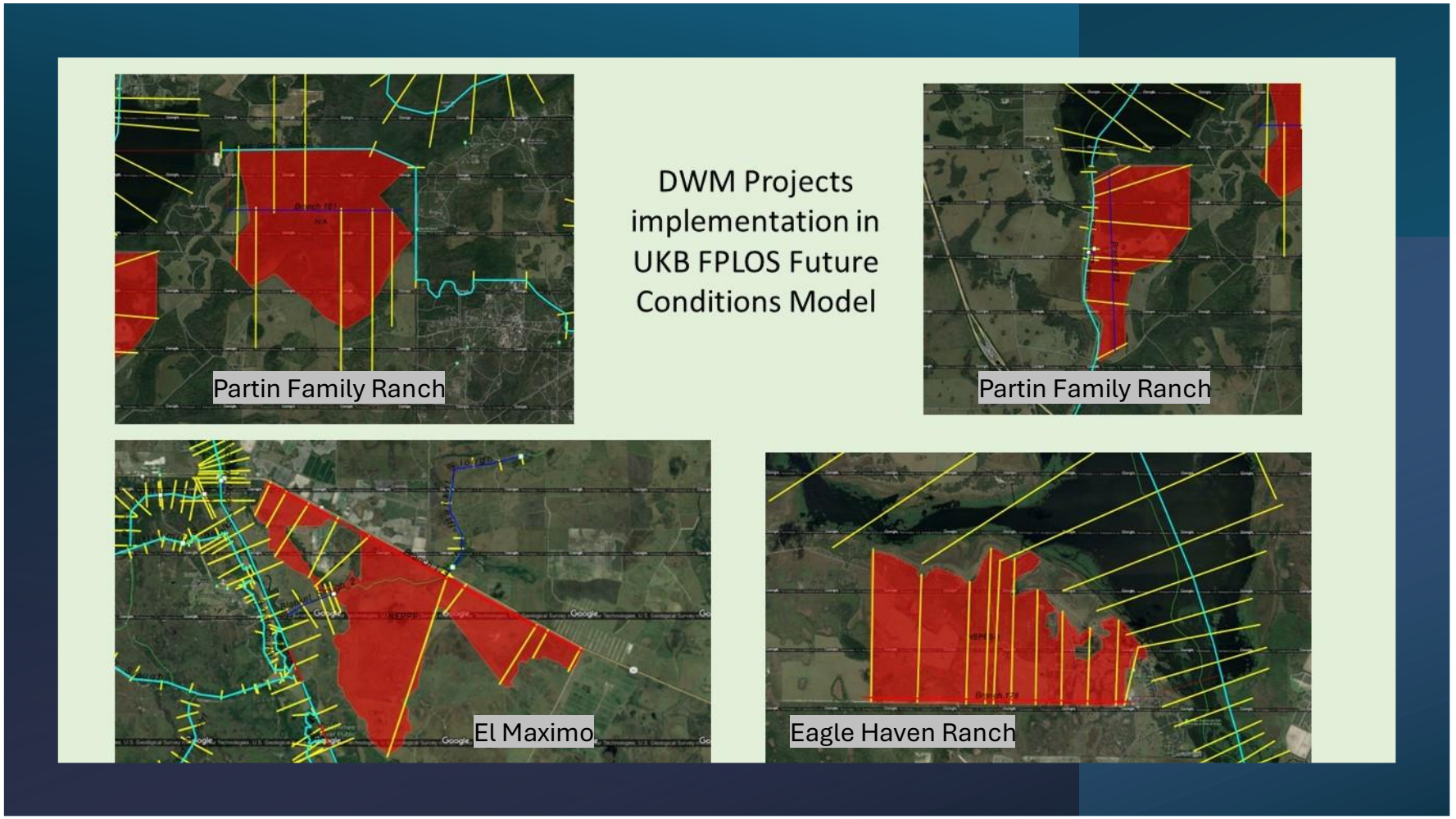


Figure 7-7: DWM Projects Implementation in UKB FPLOS Future Conditions Mode

7.9 Other

7.9.1 Land Use

Future land use GIS datasets from Orange and Osceola counties (2050 and 2040 respectively) were merged and compared to the land use dataset used in the current conditions UKB FPLOS model. The differences were only in the type of urban land use (low, medium and high density); none that would result in a change of parameters. So, the parameters were not modified.

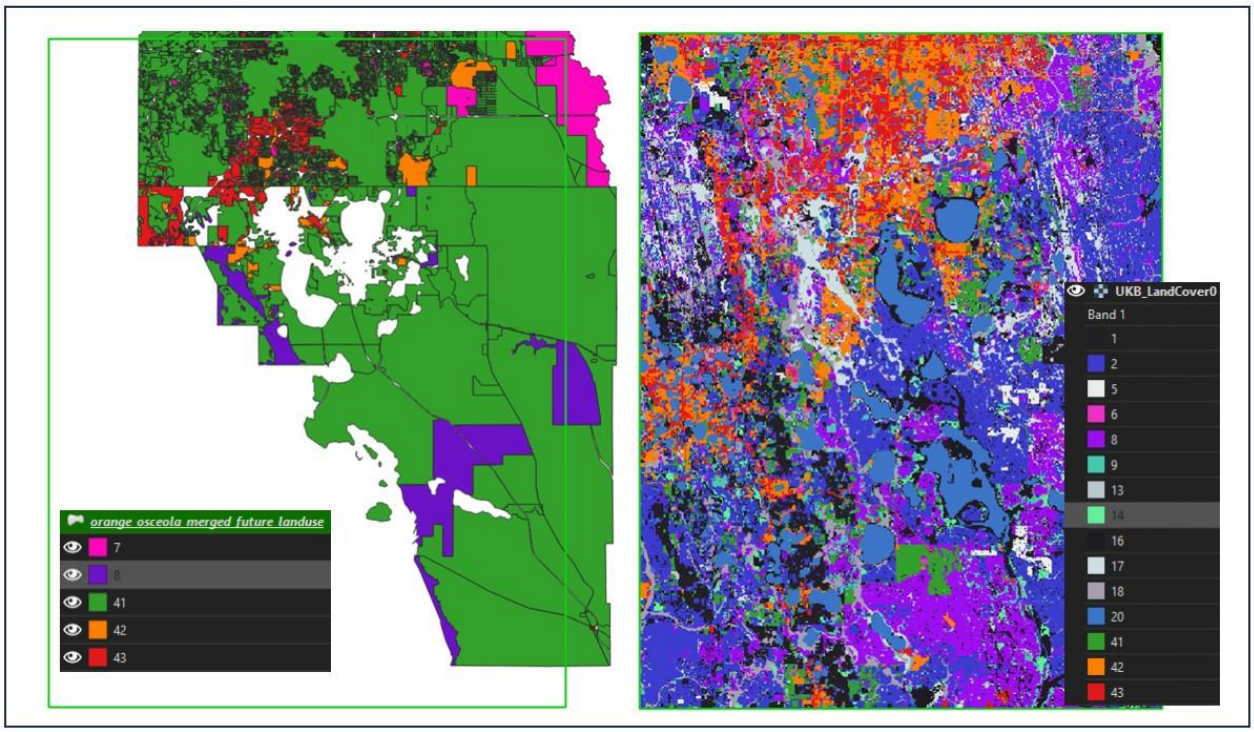


Figure 7-8: Comparison of Future Land Use Datasets: Orange and Osceola Counties LU with SFWMD LU

7.9.2 Topography

The topography was modified in the location of future SFWMD projects, described in Section 7.2.

8. Alignment of Downstream Boundary Condition to Rainfall Peak under current and future conditions

The calibrated UKBFPLOS model was used to perform design event simulations. Generally, model sensitivity analysis tests how the model responds to various parameters. The goal of the sensitivity analysis for this task was slightly different: it was done to understand how the watershed behaves for different alignments of the peak storm event intensity and the model boundary conditions. This was done to ensure that the selected alignment of the boundary conditions with the peak of the design rainfall does not result in underestimation of the flooding. This section explains the methodology, observations, results and conclusion of the sensitivity analysis.

To investigate the model behavior in response to the timing of peak rainfall intensity relative to the peak boundary condition stages, a set of simulations were created by shifting the rainfall peak one day at a time - 3 days before and 2 days after the downstream boundary condition peak stage. The downstream boundary condition used in UKB FPLOS Model is the observed time series at FTKISS station, downstream of S65A structure. The flooding in UKB is at its worst when discharges through S65 are limited. So, for the sensitivity analysis, the discharges through S65 were analyzed for various alignment of the design rainfall.

Figure 8-1 shows the S65 flows and the design rainfall at selected model cell. It shows that the S65 discharges do not change for different design rainfall events. The S65 headwater and tailwater differential is approximately 3ft, and so the discharges are determined by:

- Head differentials when subject to structure restrictions (MPHD or MAGO) or
- Headwater when determined by regulatory schedule; the SFWMD regulatory schedule at the UKB structures are calendar based.

Although the boundary condition at FTKISS is needed for simulations, it is not a dominant factor in determining the flows out of S65, except when the local basin is flooding (as determined by specific gages).

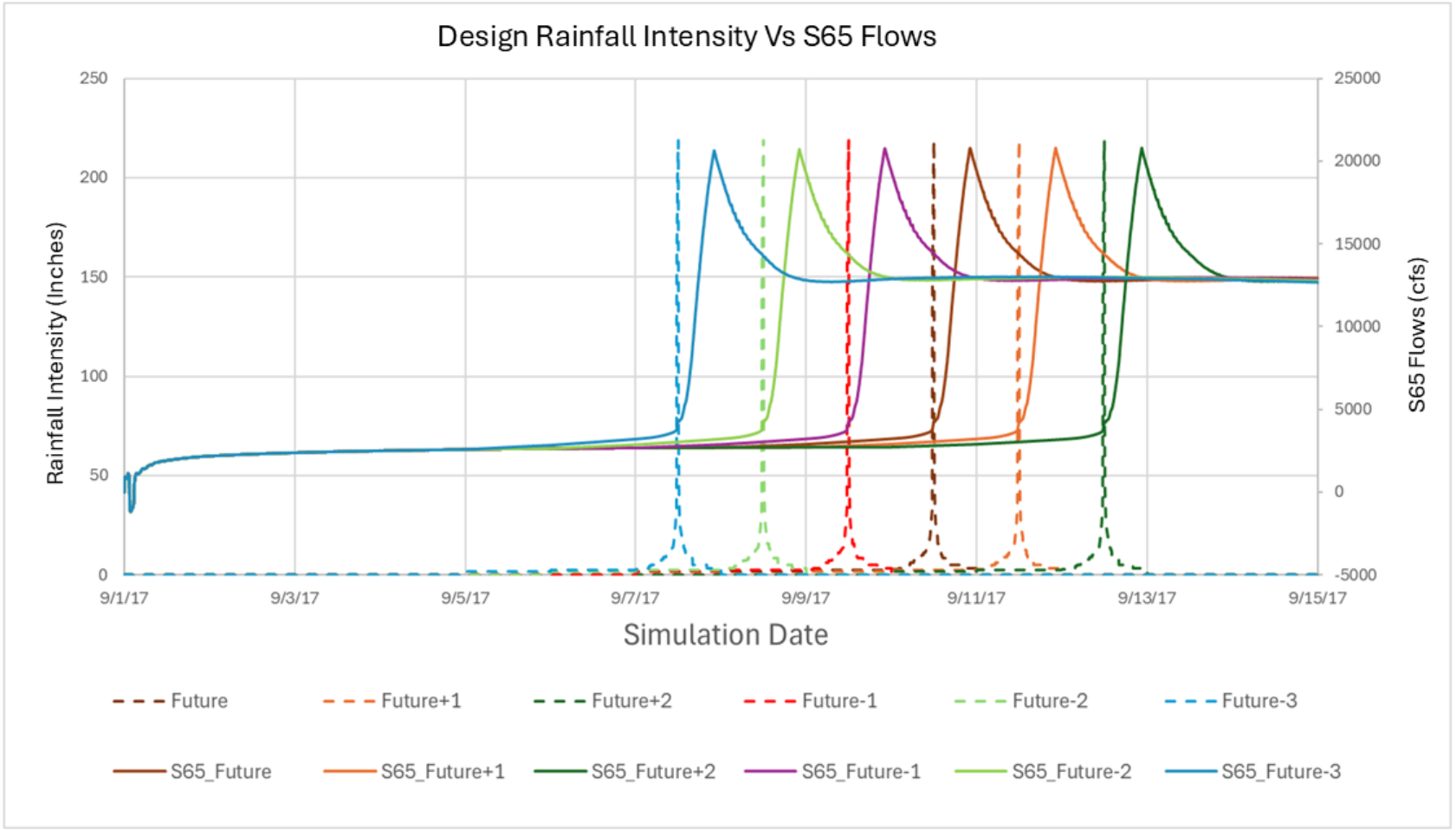


Figure 8-1: Rainfall intensity and Flow at Structure S65 for One-Day Shift Increment for 100-Yr Design Storm

9. UKB FPLOS Performance Metrics

The SFWMD FPLOS program uses six (6) formal PMs to evaluate the FPLOS for SFWMD's primary water management infrastructure. However, UKB FPLOS uses 5 of the 6 PMs: PM #1, PM #2, PM #3, PM #5 and PM #6. PM #4 is to assess the performance of coastal structures and is not applicable to UKB FPLOS. However, it was modified for UKB to assess performance of the system relative in flooding in the LKB. This PM will now be referred to as PM #4-UKB. The UKB FPLOS PMs are defined in this section, and how the model configuration will be used in evaluation of these PMs.

- Maximum Stage in Primary Canals (PM #1): This is the peak stage profile in the primary SFWMD canal conveyance system for current and future conditions. When the stages exceed the bank elevations of the UKB lakes or canals that connect them, the conveyance of the primary system is inadequate to drain the adjacent communities. Additionally, when simulated stages overflow the banks, it could cause localized flooding in areas drained by the secondary or tertiary drainage system. To evaluate PM #1, the water level profiles were developed for the three-day design storms (5-, 10-, 25-, and 100-yr simulations). The largest of the simulated design storms that stayed within the MIKE HYDRO canal banks defined the PM #1 LOS of the SFWMD primary system evaluated.

  Bank elevations shown on the profile figures were based on the MIKE HYDRO cross-section data. For the purposes of this performance metric, as part of the current and future conditions analyses, several cross-section banks were modified/extended based on the LiDAR or other sources of reliable data. This was done to ensure model simulations capture levees and areas at which the SFWMD primary canals would be considered out-of-bank. QA/QC checks were done to accurately represent the level at which bank exceedance would start to occur. Also shown in the PM #1 figures are control structures, and/or primary canal junctions.

- Maximum Daily Discharge Capacity in Primary Canals (PM #2): PM #2 is defined as the maximum discharge capacity of each of the SFWMD primary canals or the UKB lakes. It determines the maximum flow capacity of the primary systems in UKB FPLOS watersheds for current conditions represented. The conveyance capacity of the primary system was the net discharge corresponding to the largest of the 4 simulated design events, without any adverse impacts. This PM was analyzed for each LMA.

  Lake/River/Canal segments are generally defined as surface water network between water control structures; however, if there are no intermediate water control structures along the primary canals, then the associated segment is the entire lake or canal segment.

- Performance of Structure (PM #3): PM #3 determines the effective capacity of SFWMD structures. The structure flow capacities will be evaluated over the range of simulated design storms with the original static design conditions. Evaluation of this PM was based on structure design features and considered the SFWMD operational protocols under current and future conditions. This PM was evaluated with current conditions and then under future conditions. The objective of this evaluation is to determine if there are any degradations in structure performance due to future rainfall, operations, new projects, land use and/or other modifications in the future conditions model.

- Performance of Structure (PM #4): PM #4 determines the maximum conveyance capacity within the watershed at the tidal structures which are outlets to the upstream watershed; and will assess the peak discharge of the coastal structures under a range of design storms. Typically, the modeled scenario considers the combined effect of design storm events and storm surge – all averaged over the tidal cycle for coastal structures. PM #4 is used to illustrate the maximum discharge for each specific tidal boundary condition and design storm event. This metric examines the behavior of the system under worst case conditions and is used to check if conditions exceed design limits. UKB is an inland project, so PM #4 cannot be used as is usually done. But it is still important to assess the flooding in response to the flooding in LKB. To differentiate the use of this PM for an inland system, the PM called PM #4-UKB was assessed using a range of stages downstream of S65.

- Frequency of Flooding (PM #5): PM #5 determines the overall ability of the SFWMD water management infrastructure to maintain water levels within communities; it is a stage-based metric. PM #5 evaluated the overland (OL) flooding depths for 72-hr rainfall of 5-, 10-, 25- and 100-yr recurrence interval of design storms. It evaluated the peak flood stages and water depths at which critical areas and local infrastructure (roads, wetlands, urban areas, agricultural areas) get impacted. Flood inundation maps developed from model output of peak stages for each simulated storm event were used in evaluation of PM #5.

  For the purposes of this UKB FPLOS evaluation, flood inundation maps were prepared using MIKE SHE gridded model output for each design storm event, as depth of overland water greater than 0.25 ft. The threshold of 0.25 ft has been the standard in SFWM FPLOS Program. Flooding depths were representative of the overland water depths on the UKB FPLOS model grid cell.

- Duration of Flooding (PM #6): PM #6 is for assessment of the time required for surface water stages to recede to non-flood levels for storm events under current and future conditions. PM #6 quantifies the flooding duration at specific locations in UKB FPLOS Study Area. A threshold of 0.25-ft flooding depth was used to determine the length of time that this is exceeded.

- Volume in lake due to design storm (PM #UKB-V_{Lake}): This assessment was made for every LMA, to quantify the volume of water in the lake after the storm for each design event. The analysis was done for 10/9/2017 00:00 to 12/9/2017 24:00, to capture the recession completely. At each cross-section within each lake, the difference in volume between the last and first time steps was assumed to be the cumulative volume at that cross-section. The sum of the computed cumulative volumes at all cross-sections within the lake was quantified as the total excess volume after the event, due to that event. This metric is to identify vulnerabilities in terms of lake capacity and flow conveyance for each LMA under several storm scenarios.

- Duration of lake recovery to regulation schedule (PM #UKB-T_{Reg}): UKB is unique, in the hydrology of the basin is driven by the lakes. It is important to assess the time for the lake to return to the regulation schedule, if and after the lake stage exceeds the schedule line. This was done for the lakes regulated by SFWMD's primary conveyance structures.

The evaluation of the above PMs under both current and future conditions, were the basis for recommendations of flood mitigation strategies to reduce risk of flooding.

9.1 Analyses of Model Results for FPLOS PMs

9.1.1 Definition of thresholds for PM Analyses

UKB has many lakes, big and small, and wetlands that shape its water system. No dataset shows the size or the edges of these water bodies accurately. This means there will be areas that show inundation but are not areas of concern. **Table 9-1** provides the areas of the major lakes in each LMA, which can be used as reference to assess land areas in each LMA that are inundated.

Setting thresholds for assessments was a challenge. They were chosen based on practicality for FPLOS rating and descriptions. The flooding depth threshold was 0.25 ft was used, which has been the SFWMD FPLOS Program standard. The flooding duration threshold used was about 6 hours, which indicates flash flooding; and the focus of FPLOS Program is long-term flooding. So, longer than 6 hours indicates persistent flooding. For LMA level assessments, land use from the UKB FPLOS model was used. 'Marsh', 'Swamps' and 'Water' make up 25% of the model area. So, 25% LMA area inundation was used as the threshold in PM5 and PM6 assessment.

Table 9-1: LMAs and Areas of Major Lakes

Lake Management Area	Lake Name	Area (acres)	Area (sq. miles)
Alligator	Alligator	3533.9	5.5
Alligator	Lizzie	815.7	1.3
Myrtle	Myrtle	558.6	0.9
Myrtle	Preston	689.7	1.1
Hart	Hart	1977.8	3.1
Gentry	Gentry	1737.9	2.7
E Toho	E Toho	10328.6	16.1
Toho	Toho	19139.2	29.9
Cypress	Cypress	3624.6	5.7
Hatchineha	Hatchineha	6094.0	9.5
Hatchineha	Rosalie	4540.1	7.1
Kissimmee	Kissimmee	32084.1	50.1
Kissimmee	Jackson	1084.4	1.7
Marian	Marian	4808.7	7.5
Marion	Marion	2951.6	4.6

9.1.2 Presentation of Results

SFMWD FPLOS metrics need thorough analyses of the model results. For the PM metrics presentation, this report provides modeling analyses of 1-dimensional channel hydraulics, 2-dimensional overland inundation, and to a lesser extent 3-dimensional groundwater flow. All of them are integrated in the model and are linked numerically during the simulation, but produce results outputs at different time steps, limited by the user due to practicality of file sizes. The MIKE 1D channel hydraulics simulation time step uses a maximum of 2 minutes and output channel flow and stage results every one (1) hour; the MIKE SHE 2D overland flow equations are solved using a maximum time step of one (1) hour and overland depth maps are output one (1) hour and saturated zone flow equation is solved with a maximum of one (1) hour and results are output every six (6) hours. Therefore, there are some differences in how the results were evaluated for the various PMs.

MIKE HYDRO 1-D stages and flows results were used to evaluate PM #1 through PM #4. Sometimes, structure operations cause a temporary spike, producing stages and flows resulting from numerical oscillations and not real. To address this, 12-hour moving average numbers were used. 2-D overland depth maps were used to evaluate PM #5 and PM #6, which are the maximum depth and duration of flooding, respectively. For the purposes of this analysis, 0.25 ft was the threshold for overland flooding.

MIKE HYDRO default assumption is to define flow direction as positive with increasing chainages looking downstream of the river. Therefore, in PM #1 plots, flow direction' defines the left and right sides of the river; the sides are observed looking in the direction of the main flow. The left and right banks are observed looking in the direction of decreasing chainage numbers, generally.

In addition, it should be noted that the UKB FPLOS design event models were run over the simulation period of 9/1/2017 through 10/30/2017; however, these dates are arbitrary and do not represent actual hydrologic conditions on these dates.

10. FPLOS Assessment for Current and Future Conditions in UKB

The UKB FPLOS Model domain was established to better capture the hydrologic boundaries and establish hydrology into the study area. The PMs are used to identify areas at risk of flooding, given the uncertainties inherent to all models. This section summarizes the assessment in UKB for the flood protection provided by SFWMD primary system using the implementation of the model for four design events (5-, 10-, 25-, and 100-yr).

The maximum stages in the lakes and primary canals were extracted from the results of each design storm simulation, under both current and future conditions. Generally, the gates were fully open at times of peak flow.

10.1 Alligator Lake LMA

Figure 10-1 on the following page shows Alligator Lake LMA with the lakes, main roadways and waterways.

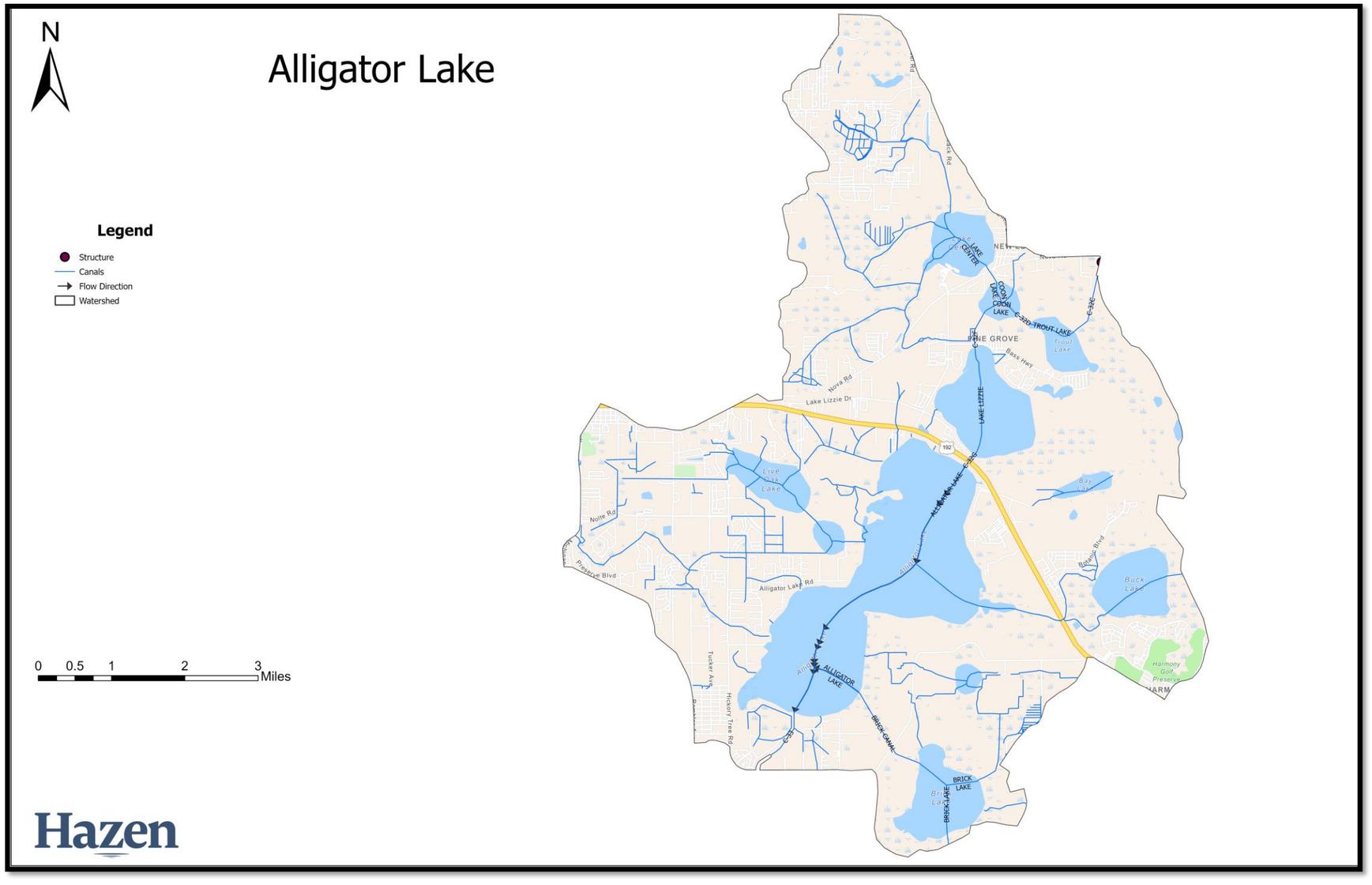


Figure 10-1: Alligator Lake with Main Structures, Waterways and Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.1.1 PM #1

Alligator Lake LMA stages are controlled by structures S60 on the south and S58 on the north; S60 is on C-33 canal and S58 is on C-32C canal. Assessment of PM #1 for this LMA focused on Alligator Lake and the canals.

For Lake Alligator LMA under current conditions, the left bank is exceeded in several places for the 100-year event, while the right bank sees exceedances at various points, though more consistently than the left bank. For 5-, 10-, and 25-year storms, the water level stays within the banks, except in two locations where they exceed the right and left banks separately. In Canal C-32C, both the left and right banks are exceeded for the 100-year event, while the smaller design events stay within the bank elevations. For the C-33 Canal, there are no exceedances for either the left or right bank for all storms.

Under future conditions, Lake Alligator LMA continues to see left bank exceedances for the 100-year event, with some additional right bank exceedances in more areas compared to the current conditions. The C-33 Canal remains stable with no exceedances for either bank, just as under the current conditions. In Canal C-32C, both banks are exceeded for the 100-year event, and there is a slight increase in water levels of the 25-year storm event after S58 Gate compared to current conditions, making it exceed both banks.

Based on the available profiles along Alligator Lake and C-32C and C-33 canals shown in **Figure 10-2 through Figure 10-7** under current and future conditions, it is evident that Alligator Lake water levels mostly stay above banks for all simulation events at some locations along the banks. Along Canal C-33, the water levels stay within the banks for all storms. For 25- and 100-yr storms, the water overtops the Canal C-32C banks. When comparing current to future conditions, there is a slight increase in water levels, but the general trends of bank exceedance remain consistent, especially in Canals C-32C and C-33.

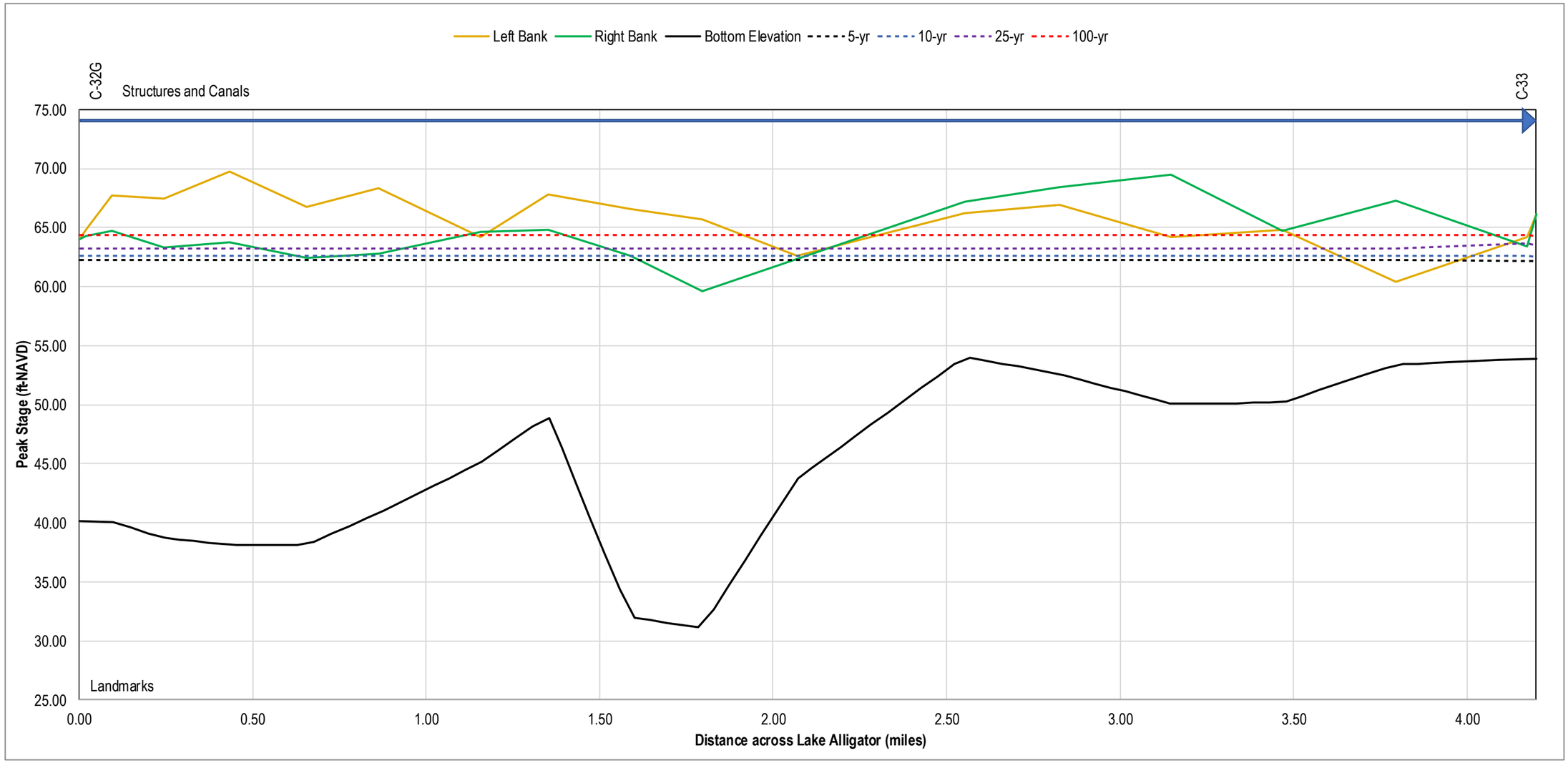


Figure 10-2: Maximum Stage Profile for Alligator Lake LMA under Current Conditions

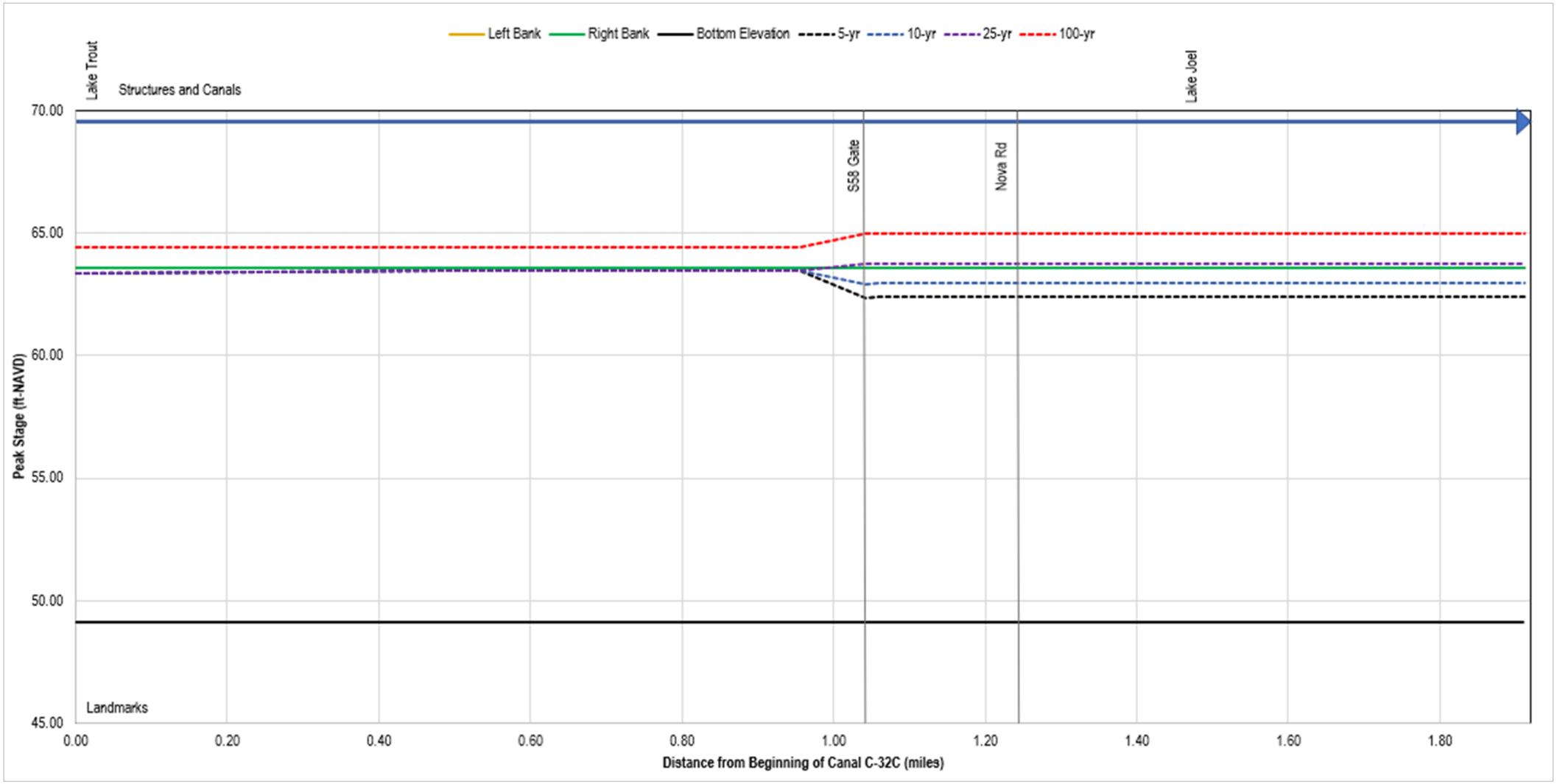


Figure 10-3: Maximum Stage Profile for C-32C Canal under Current Conditions

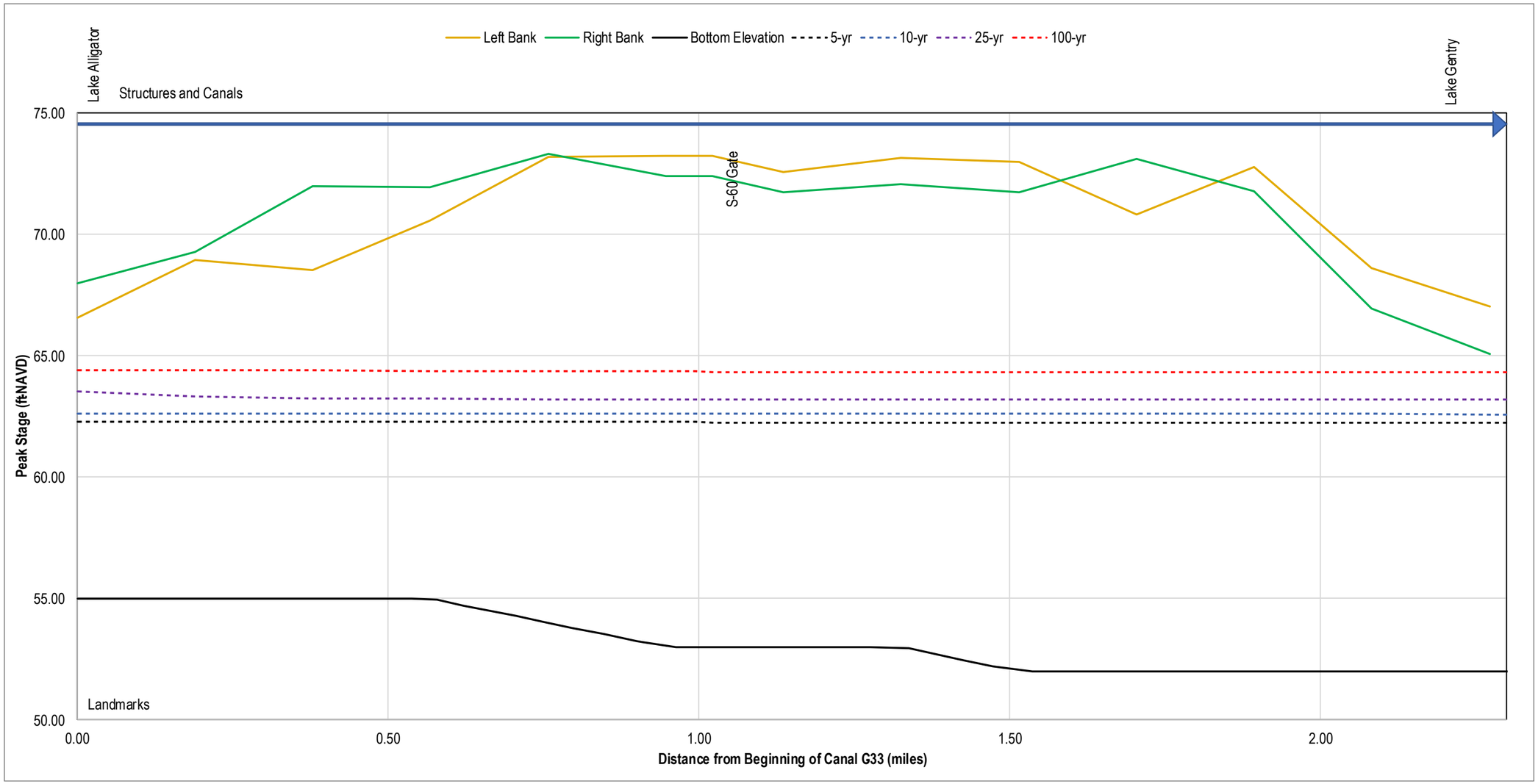


Figure 10-4: Maximum Stage Profile for C-33 Canal under Current Conditions

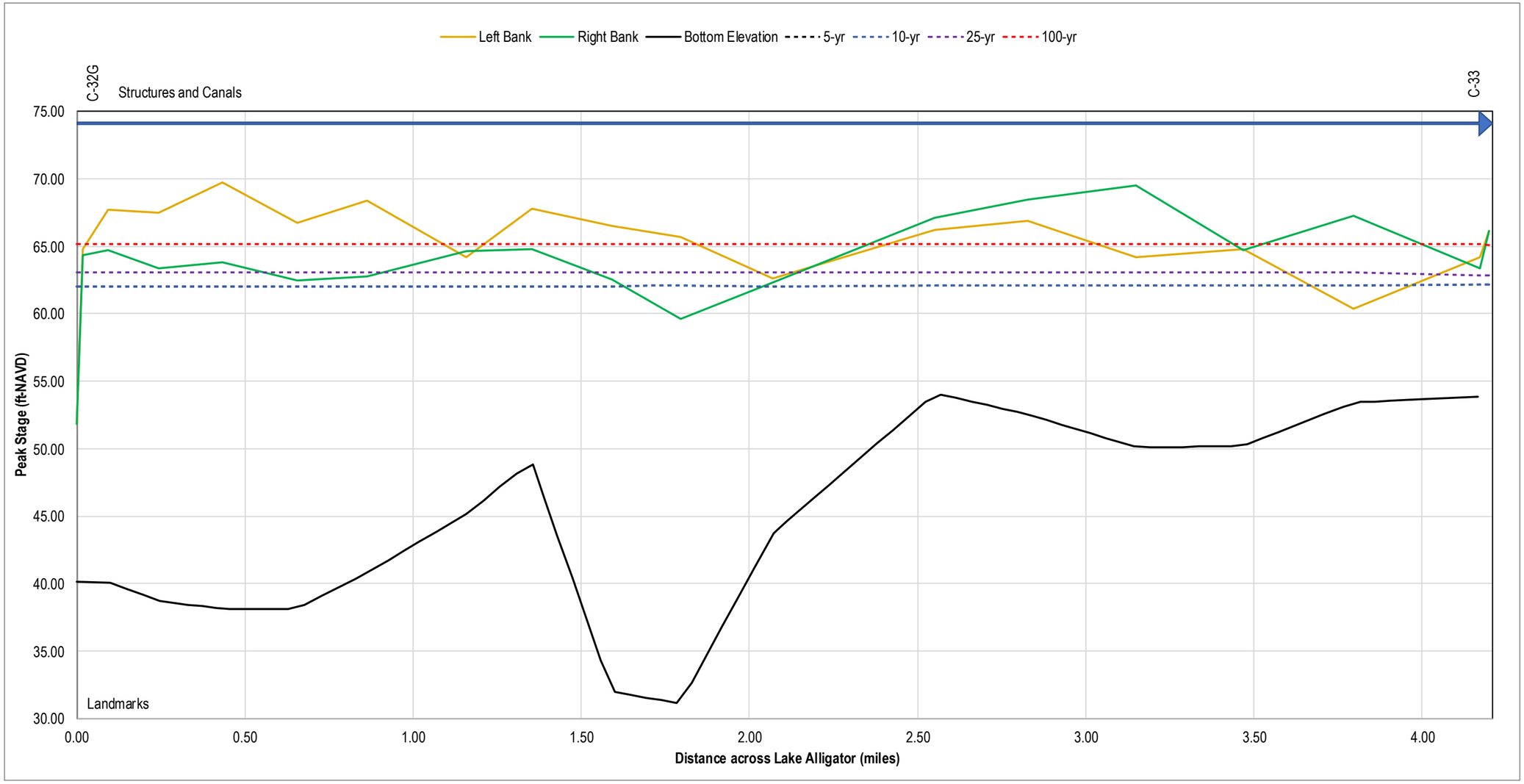


Figure 10-5: Maximum Stage Profile for Alligator Lake under Future Conditions

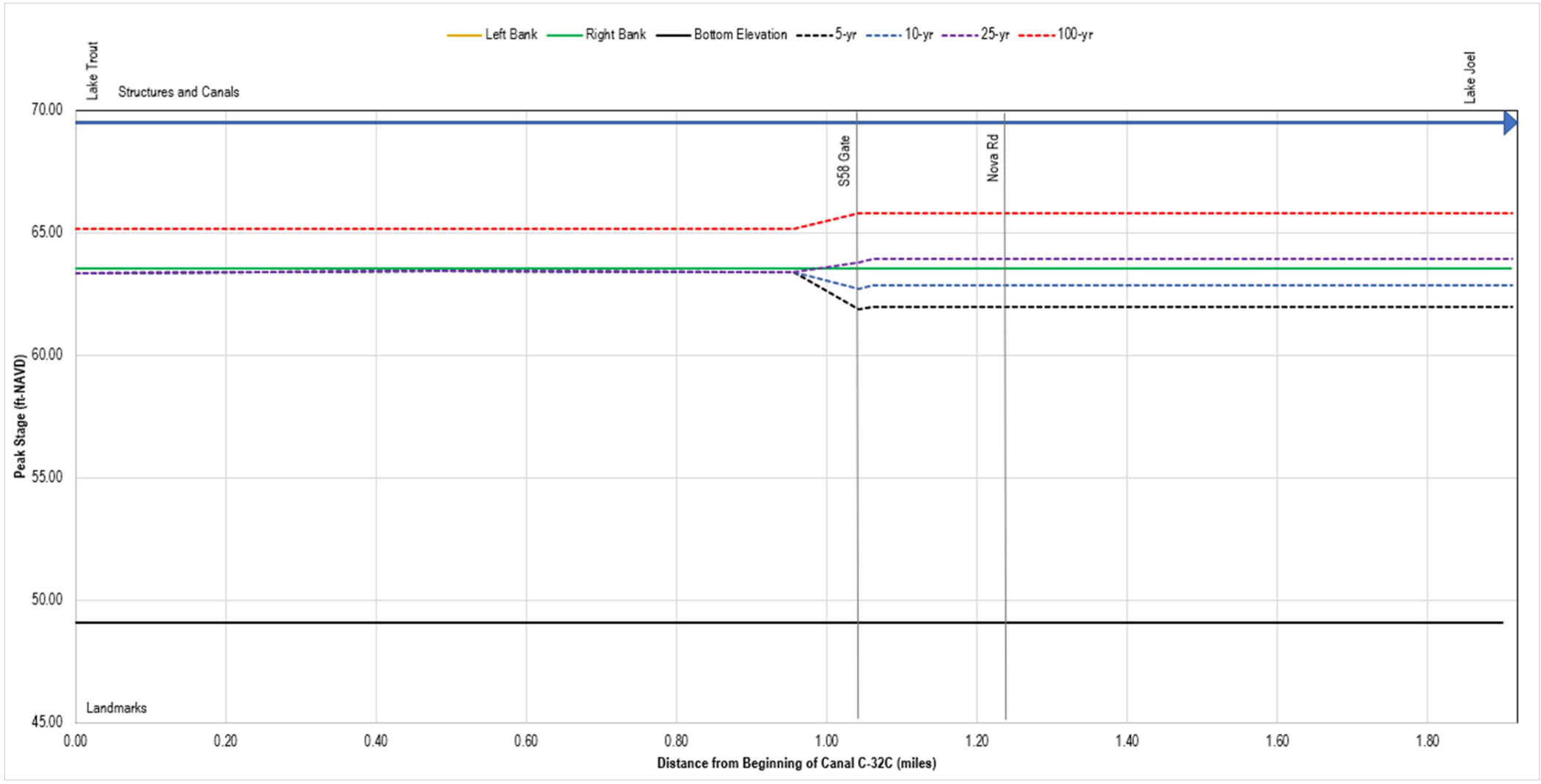


Figure 10-6: Maximum Stage Profile for C-32C Canal under Future Conditions

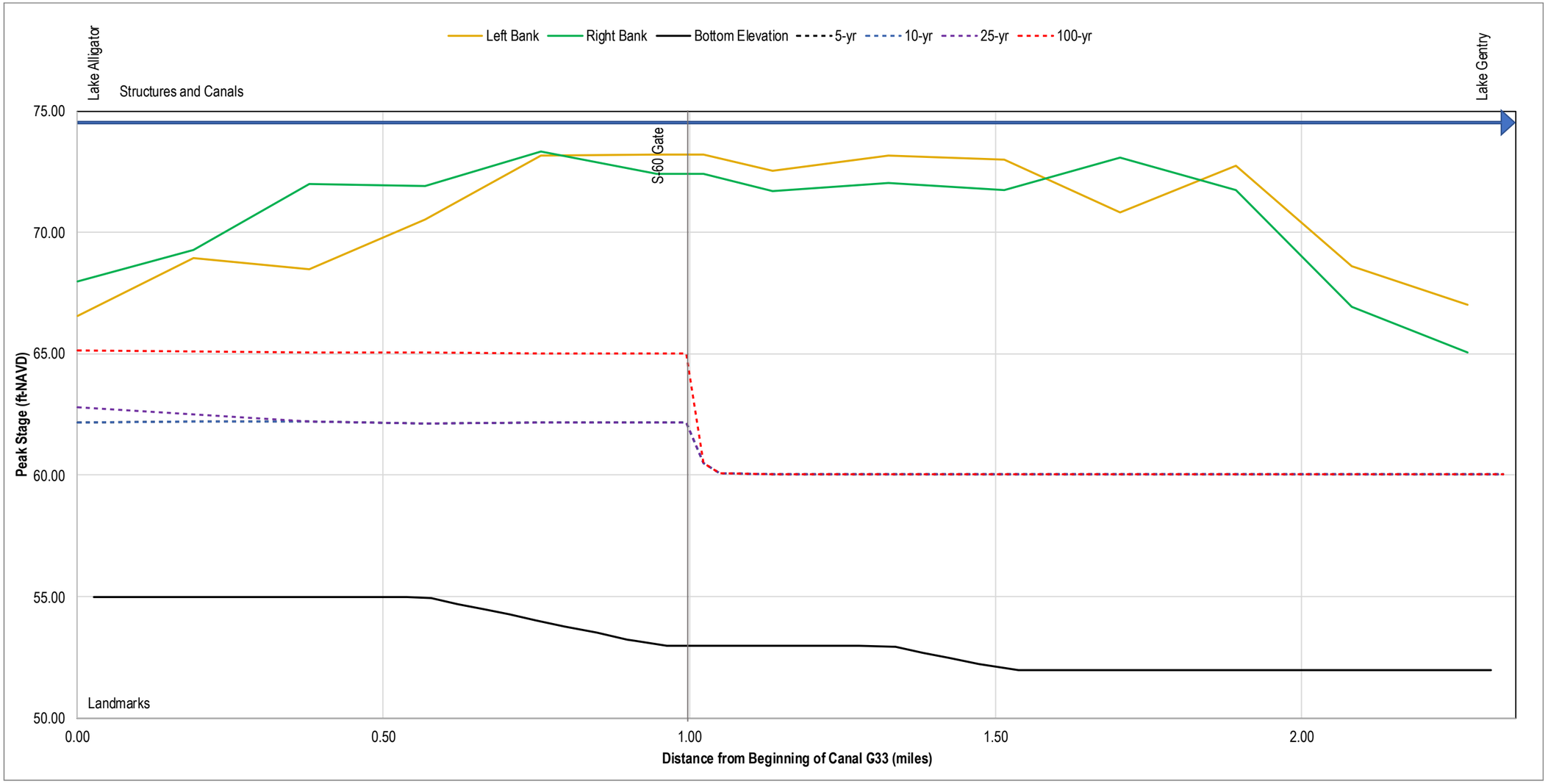


Figure 10-7: Maximum Stage Profile for C-33 Canal under Future Conditions

10.1.2 PM #2

Alligator Lake LMA has no upstream water control structure but has two downstream structures: S58 and S60. S58 was closed during Hurricane Irma. For the design storms, the operations were coded to open S58 if S60 is flowing at capacity. Therefore, the maximum daily discharge capacity of this LMA is addition of both S58 and S60. Per the Districts ERP Handbook, Alligator Lake has an allowable discharge rate of 13.0 CSM, based on a design frequency of 10-yrs. **Table 10-1** summarizes Alligator Lake LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area; and **Table 10-2** provides the same information under future conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 13.9 CSM under current conditions and 14.1 CSM under future conditions, which is higher than SFWMD's allowable discharge of 13 CSM based on a 10-yr design frequency.

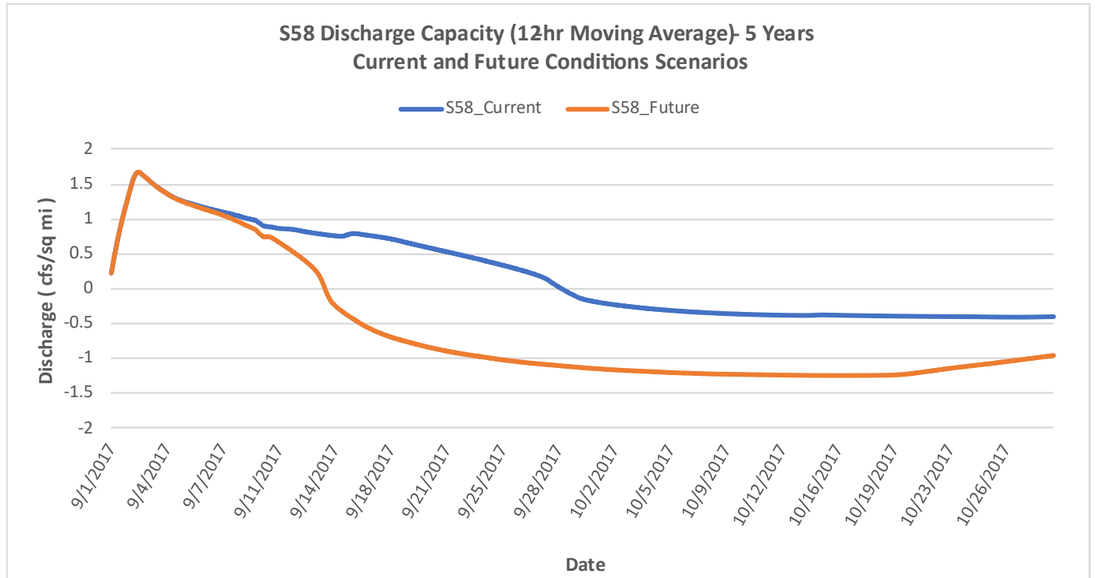
Table 10-1: Drainage Discharge Capacity (cfs/sq. mi) of Alligator Lake LMA under Current Conditions

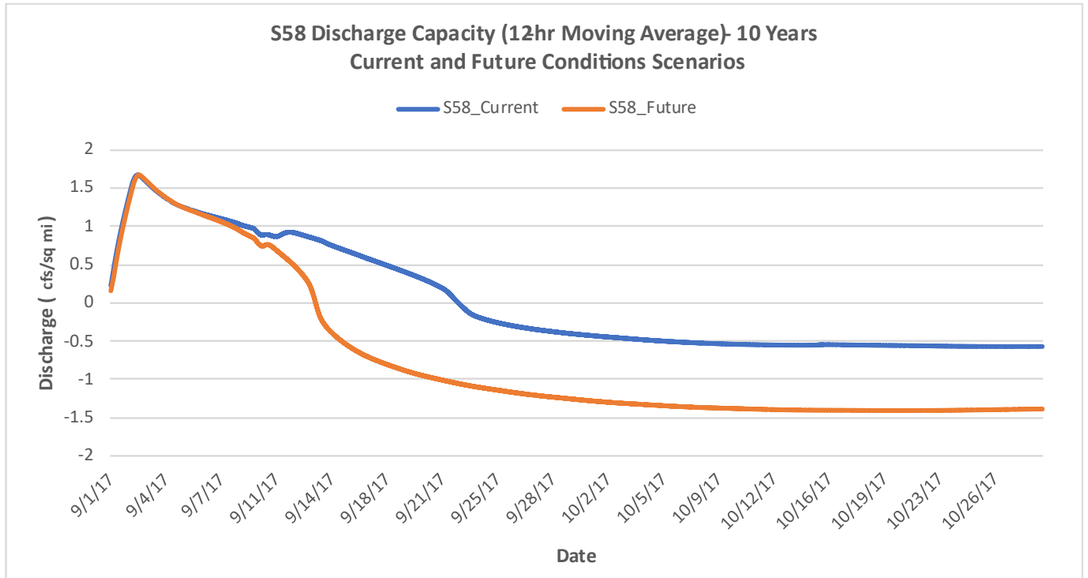
LMA	Total Area (mi^2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Alligator Lake	52.6	11.6	13.9	13.8	13.9	13.0

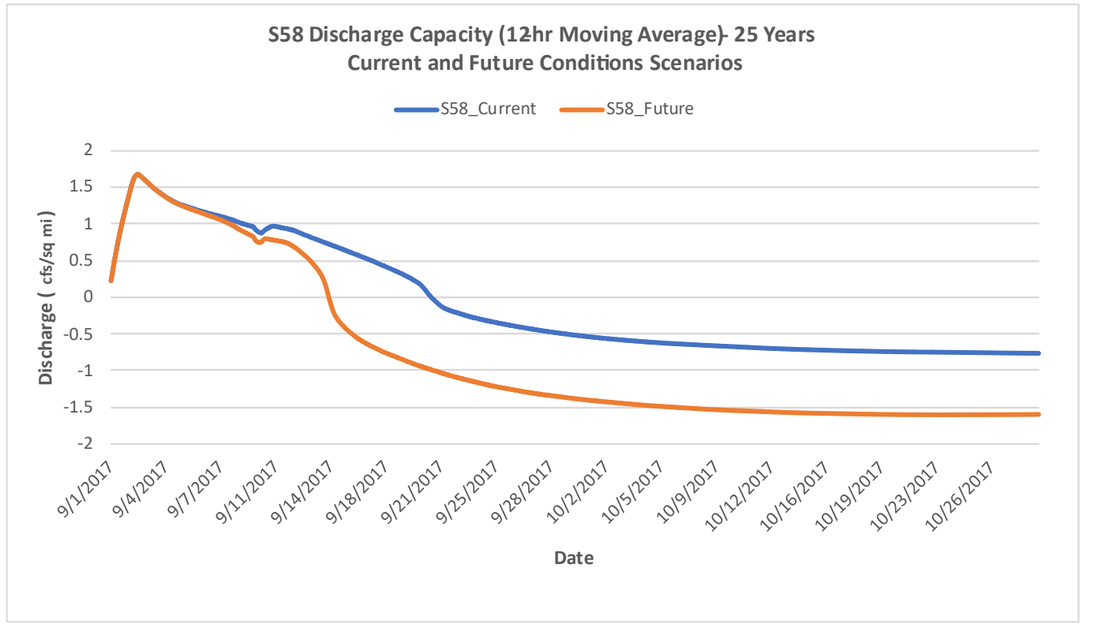
Table 10-2: Drainage Discharge Capacity (cfs/sq. mi) of Alligator Lake LMA under Future Conditions

LMA	Total Area (mi^2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Alligator Lake	52.6	14.1	14.1	18.5	21.5	13.0

Figure 10-8 shows the peak 12-hour moving average discharge per square mile of drainage area, calculated for each design storm at structure S60. **Figure 10-9** shows the maximum daily discharge through structure S60 for both current and future conditions. They show the allowable discharge being exceeded under future conditions for the 10-yr design event; it is exceeded for 25-yr and 100-yr design events under current and future conditions.







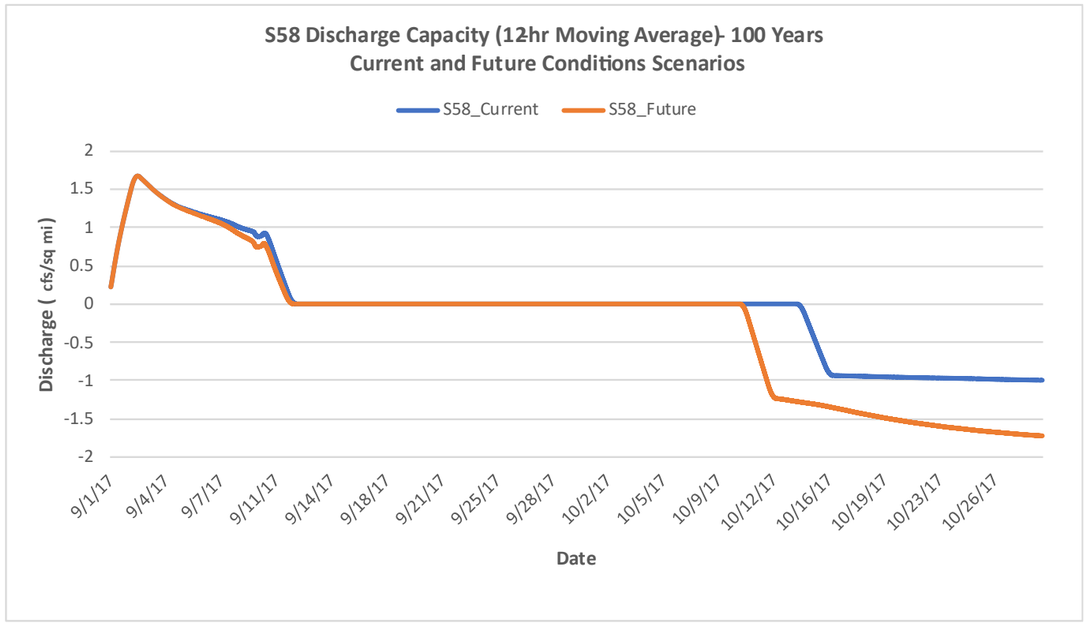
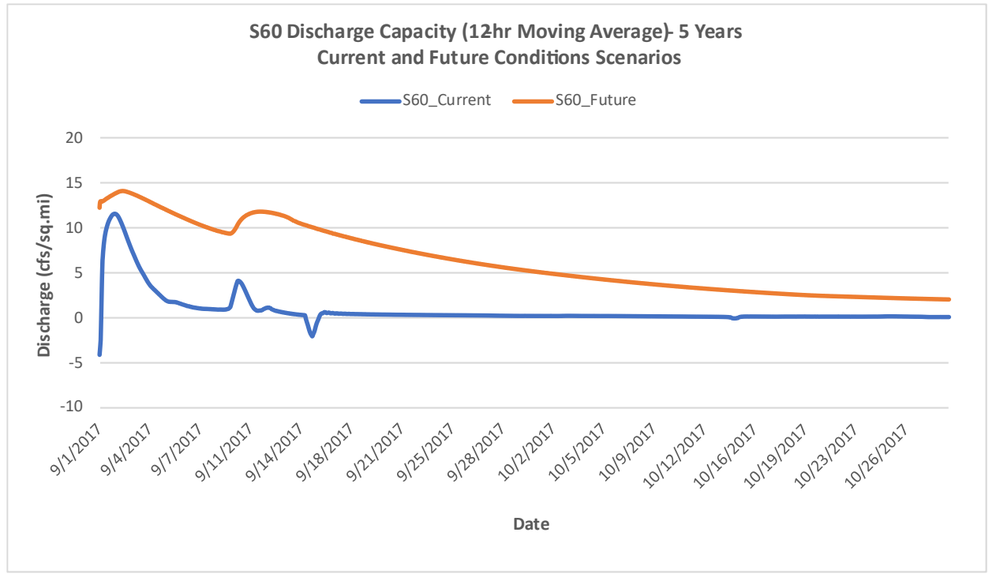
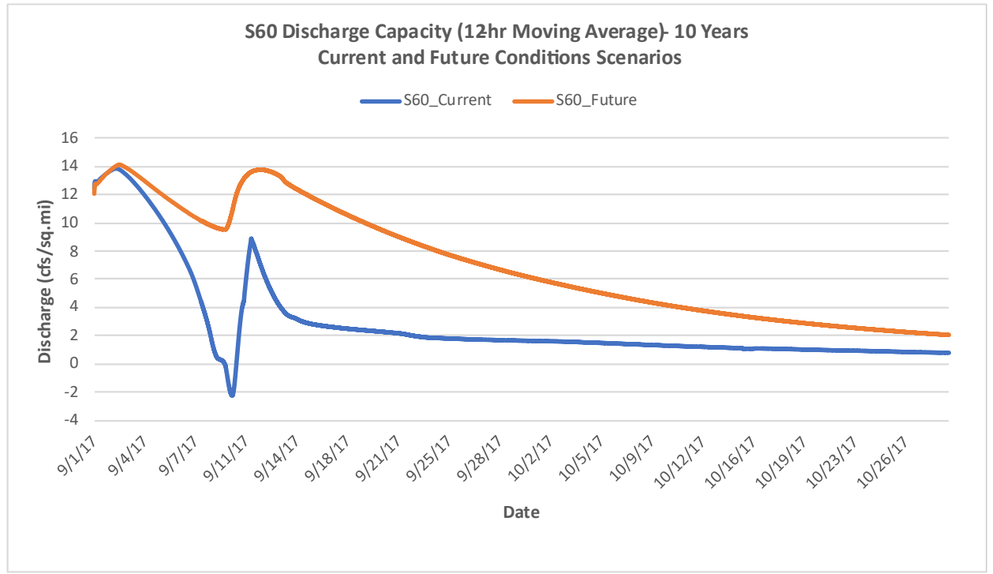
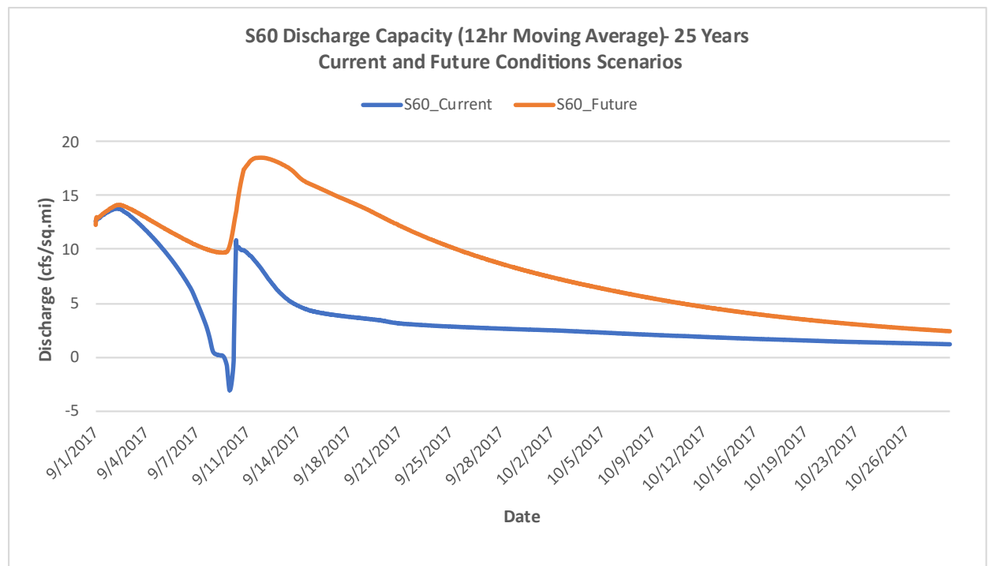


Figure 10-8: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S58 for 5-, 10-, 25-, 100- Year Design Storms







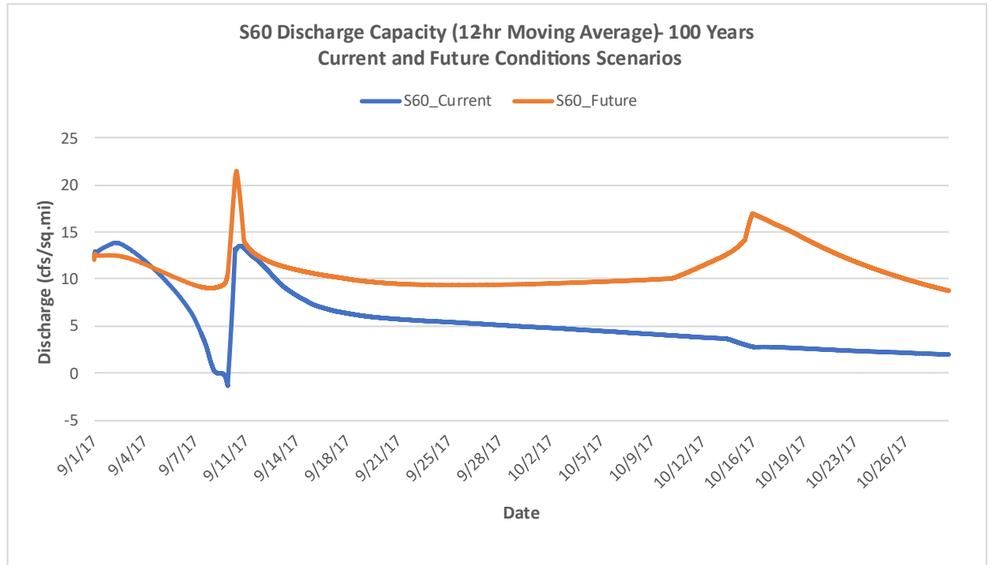


Figure 10-9: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S60 for 5-, 10-, 25-, 100- Year Design Storms

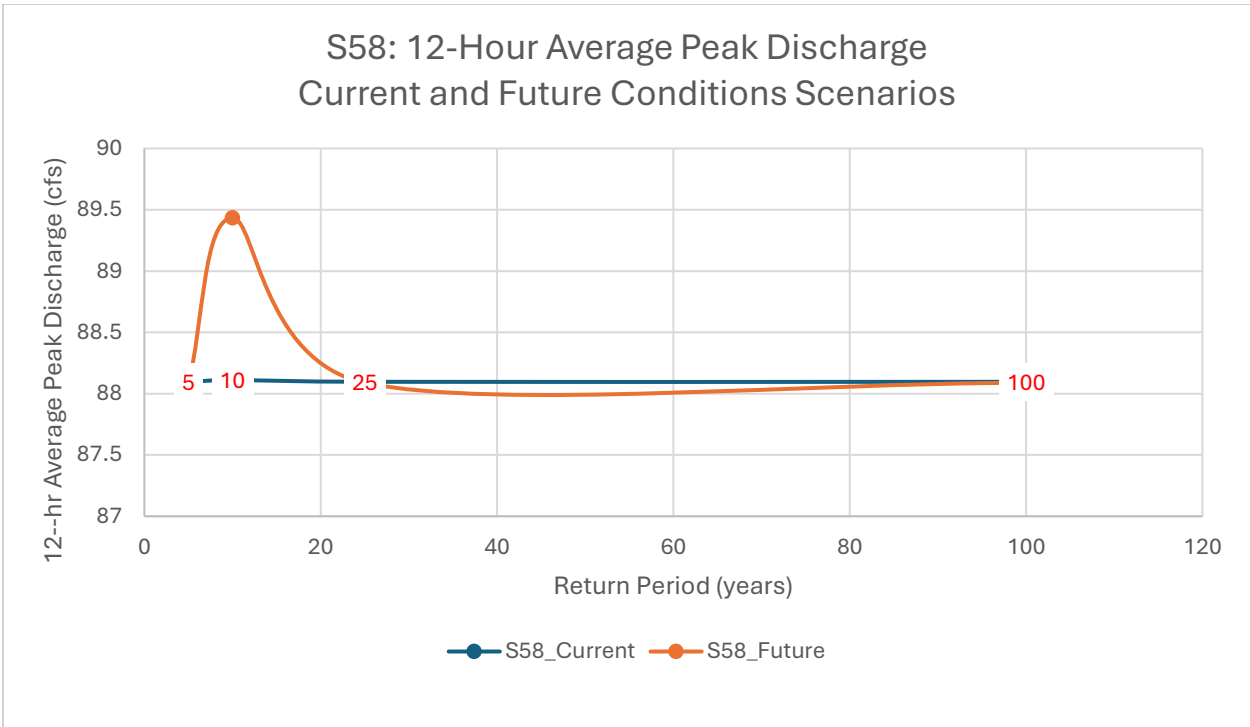


Figure 10-10: Structure S58 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

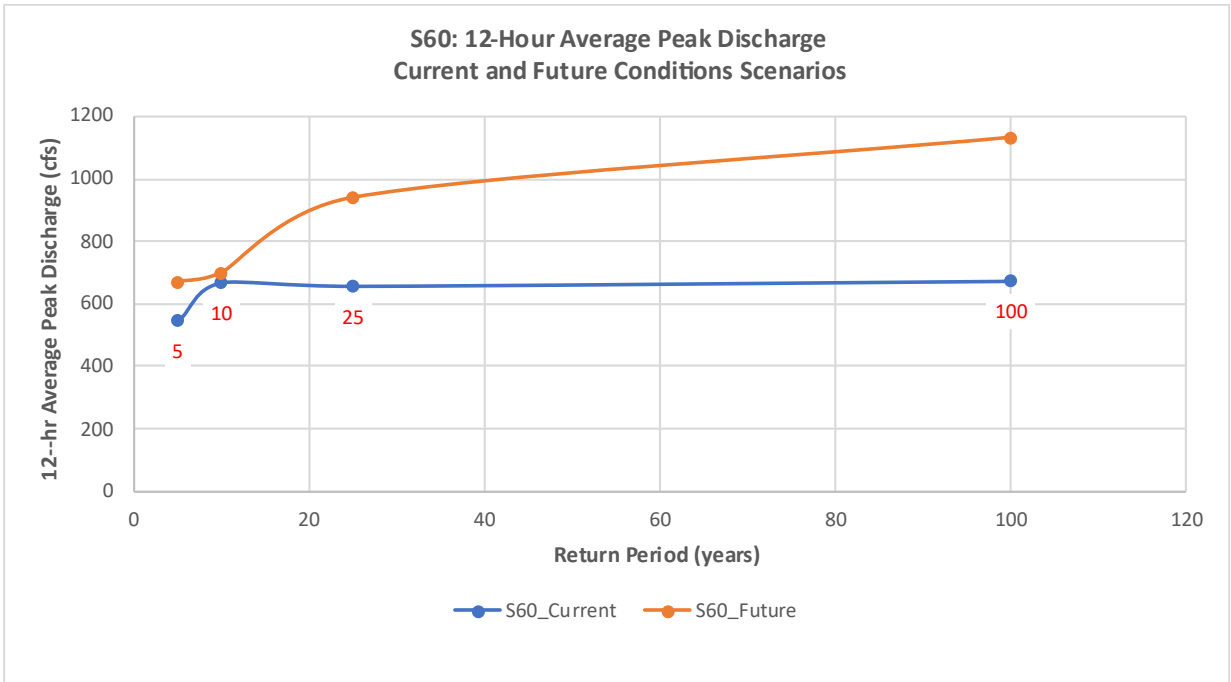


Figure 10-11: Structure S60 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

10.1.3 PM #3

The SFWMD structure design parameters in **Table 10-3** and **Table 10-4** on the following page were used to evaluate the S58 and S60 performance against the various design storms.

Table 10-5 through **Table 10-10** provide a summary of current and future conditions simulated instantaneous stages and flows at structure S58 at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S58, the simulated headwater elevation at S58 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is less than the SPF headwater elevation for both current and future conditions, except for 100-yr design storm simulation under current conditions. This is right at the start of the 100-yr design storm, and the system is still adjusting, and the model has some instabilities. The simulated flow at structure S58 is lower than the design discharge (160 cfs) for all design storm events under current and future conditions. During the peak headwater at the structure S58, the simulated tailwater elevation at S58 is lower than design tailwater elevation for smaller storms but higher for the larger storms; but the tailwater stages are lower than the SPF tailwater elevation for all storms under current and future conditions.

Table 10-3: Design Parameters for Structure S58

Structure	S58
Design Discharge	160 cfs (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	63.74 feet NAVD88 (64.8 feet NGVD29)
Design Tailwater Elevation	62.14 feet NAVD88 (63.2 feet NGVD29)
SPF Headwater Elevation	66.34 feet NAVD88 (67.4 feet NGVD29)
SPF Tailwater Elevation	65.74 feet NAVD88 (66.8 feet NGVD29)
Optimum Headwater	60.94 to 62.94 feet NAVD88 (62.0 to 64.0 feet NGVD29)
Optimum Tailwater	58.94 to 60.94 feet NAVD88 (60.0 to 62.0 feet NGVD29)

Table 10-4: Design Parameters for Structure S60

Structure	S60
Design Discharge	450 cfs (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	61.25 to 63.15 feet NAVD88 (62.3 to 64.2 feet NGVD29)
Design Tailwater Elevation	60.65 to 62.25 feet NAVD88 (61.7 to 63.3 feet NGVD29)
SPF Headwater Elevation	66.35 feet NAVD88 (67.4 feet NGVD29)
SPF Tailwater Elevation	65.35 feet NAVD88 (66.4 feet NGVD29)
Optimum Headwater	60.95 to 62.95 feet NAVD88 (62.0 to 64.0 feet NGVD29)
Optimum Tailwater	58.94 to 60.94 feet NAVD88 (60.0 to 62.0 feet NGVD29)

Table 10-5: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	62.21	62.31	0.00	-0.10	10/2/17 10:00 PM
10-year	62.57	62.89	0.00	-0.32	10/3/17 5:00 PM
25-year	63.24	63.28	0.00	-0.05	9/26/17 3:00 AM
100-year	64.43	63.71	0.00	0.72	9/17/17 4:00 PM

Table 10-6: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	61.44	60.57	39.30	0.87	10/28/17 3:00 PM
10-year	61.88	61.02	39.21	0.86	9/11/17 6:00 PM
25-year	63.13	62.34	37.48	0.79	9/12/17 3:00 PM
100-year	65.16	64.49	0.00	0.67	9/15/17 12:00 AM

Table 10-7: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	62.37	62.16	0.00	0.21	10/10/2017 13:00 PM
10-year	62.95	62.51	0.00	0.44	10/8/2017 20:00 PM
25-year	63.75	63.07	0.00	0.68	10/8/2017 20:00 PM
100-year	64.99	63.65	0.00	1.34	10/20/17 3:43 AM

Table 10-8: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	61.99	59.80	-50.27	2.19	10/2/17 5:00 AM
10-year	62.86	60.19	-68.00	2.67	10/2/2017 21:00 PM
25-year	63.97	60.74	-75.70	3.23	10/2/2017 23:00 PM
100-year	65.80	61.18	-6.39	5.62	10/02/2017 21:00 PM

Table 10-9: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW at Peak Discharge (ft NAVD88)	Head Differential at Peak TW (ft)	Date & Time
5-year	90.13	60.79	59.50	1.29	9/02/17 19:00 PM
10-year	90.13	60.82	59.31	1.52	9/02/17 19:00 PM
25-year	90.13	60.87	59.39	1.48	9/02/17 19:00 PM
100-year	90.13	63.42	61.89	1.53	9/02/17 19:00 PM

Table 10-10: Summary of the Instantaneous Conditions at Structure S58 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW at Peak Discharge (ft NAVD88)	Head Differential at Peak TW (ft)	Date & Time
5-year	90.13	60.59	59.27	1.32	9/02/17 19:00 PM
10-year	90.13	60.49	59.34	1.15	9/02/17 19:00 PM
25-year	90.13	60.59	59.47	1.12	9/02/17 19:00 PM
100-year	90.13	62.48	57.92	4.56	9/02/17 19:00 PM

Table 10-11 through **Table 10-16** provide a summary of current and future conditions simulated instantaneous stages and flows at structure S60 at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S60, the simulated headwater elevation at S60 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is less than the SPF headwater elevation for both current and future conditions. The simulated flow at structure S60 is lower than the design discharge (450 cfs) for all design storm events under current conditions, however the flows are higher for all the design storms under future conditions. During the peak headwater at the structure S60, the simulated tailwater elevation at S60 is lower than design tailwater elevation during only 5-yr and 10-yr design storm events under current conditions. Under future conditions, the simulated tailwater stages are lower for all design storm events. The simulated flow at structure S60 is lower than the design discharge (450 cfs) for 3 of the 4 design storm events (except 100-yr) under current conditions; however, under future conditions the flows are higher for all the design storms. During the peak flow at S60 structure, the headwater and tailwater elevations are lower than the design elevations for 5-yr and 10-yr design storm events under current and future conditions.

Table 10-11: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	62.25	62.24	75.46	0.01	10/2/17 12:00 AM
10-year	62.61	62.60	108.06	0.01	10/2/17 6:00 AM
25-year	63.20	63.19	171.01	0.02	9/20/17 6:00 AM
100-year	64.35	64.32	341.39	0.03	9/18/17 2:43 AM

Table 10-12: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	59.75	58.31	595.90	1.43	9/12/17 3:00 PM
10-year	60.34	58.67	699.73	1.67	9/13/17 12:55 AM
25-year	61.60	59.40	939.71	2.20	9/12/17 9:00 PM
100-year	65.02	58.06	555.49	6.95	9/16/17 11:00 AM

Table 10-13: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	62.24	62.25	75.46	-0.01	10/2/17 12:00 AM
10-year	62.60	62.61	108.05	-0.01	10/2/17 2:00 AM
25-year	63.19	63.20	170.91	-0.01	9/20/17 6:25 AM
100-year	64.33	64.35	339.55	-0.03	9/18/17 5:43 AM

Table 10-14: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	58.31	59.75	595.8155	1.43	9/12/17 2:00 PM
10-year	58.67	60.34	699.6866	1.67	9/13/17 2:00 AM
25-year	59.40	61.59	935.5676	2.19	9/13/17 5:55 AM
100-year	60.06	62.88	1212.71	2.82	9/11/17 5:25 AM

Table 10-15: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (cfs)**	**Head Differential at Peak Q (ft)**	**Date & Time**
5-year	316.65	62.06	62.03	0.03	9/15/17 5:00 PM
10-year	476.50	62.09	61.98	0.11	9/11/17 7:00 PM
25-year	785.79	62.65	62.11	0.54	9/10/17 11:10 PM
100-year	920.25	63.21	62.39	0.83	9/10/17 3:40 PM

Table 10-16: Summary of the Instantaneous Conditions at Structure S60 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at Peak Q (ft)**	**Date & Time**
5-year	595.90	59.75	58.31	1.43	9/12/17 4:00 PM
10-year	699.79	60.34	58.67	1.67	9/12/17 11:00 PM
25-year	941.78	61.59	59.36	2.24	9/11/17 11:25 PM
100-year	1212.71	62.88	60.06	2.82	9/11/17 5:25 AM

Figure 10-12 through **Figure 10-15** show the plots of the 12-hr moving average flows at S58 and S60 under current and future conditions

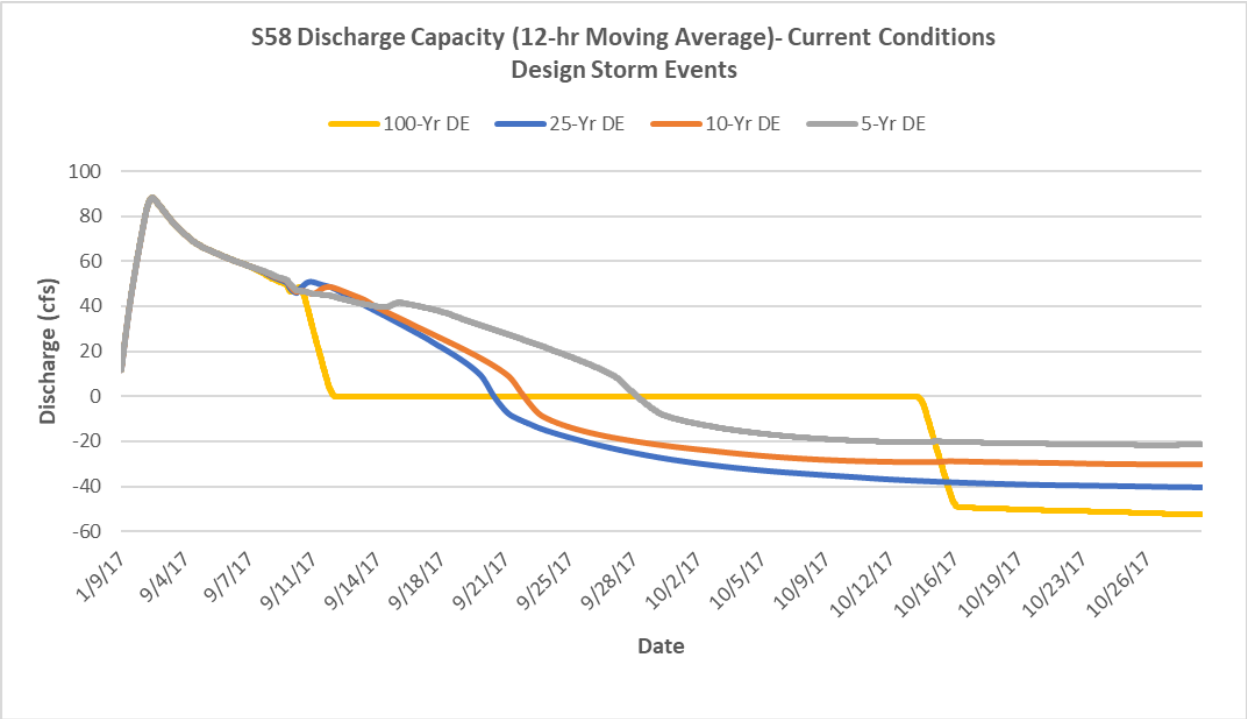


Figure 10-12: 12-hr Moving Average Flows in cfs at S58 under Current Conditions

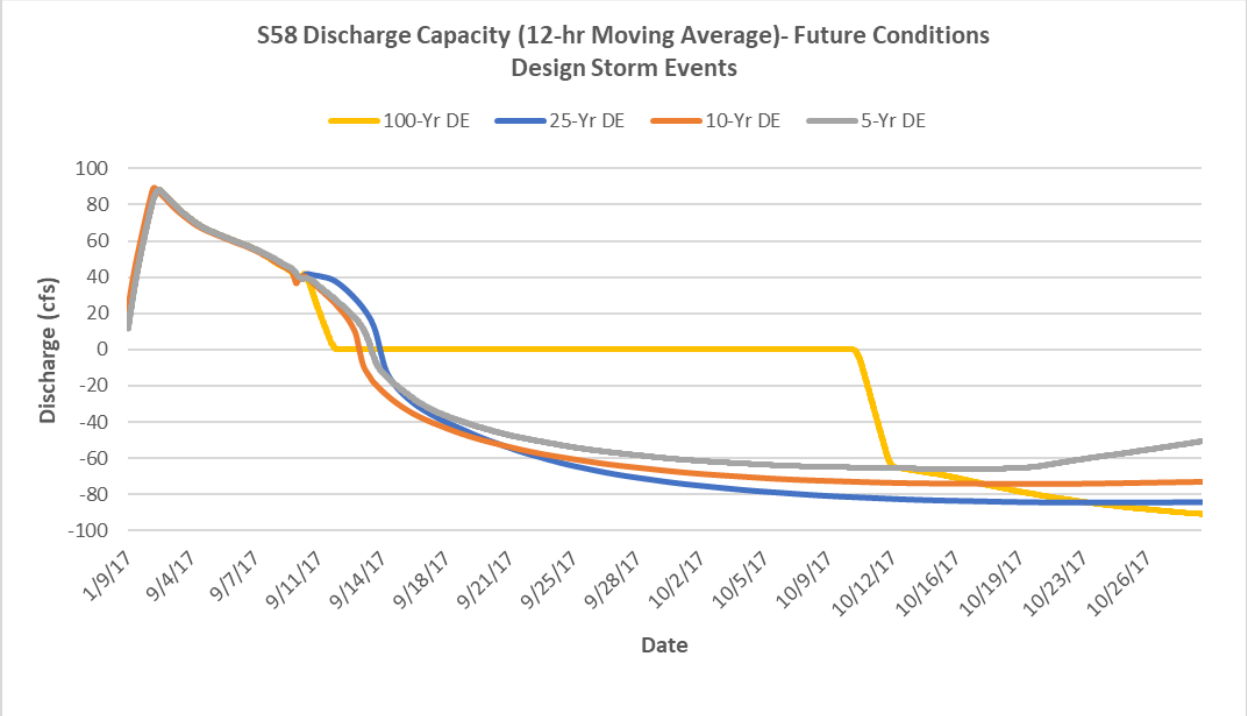


Figure 10-13: 12-hr Moving Average Flows in cfs at S58 under Future Conditions

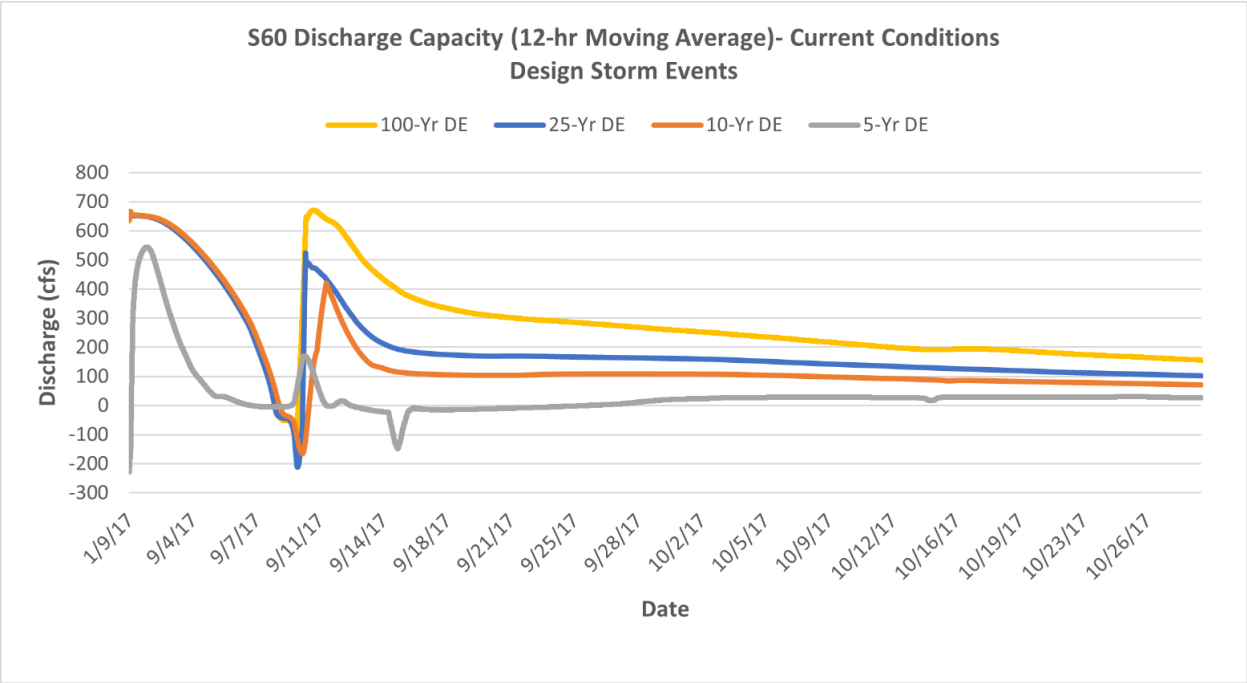


Figure 10-14: 12-hr Moving Average Flows in cfs at S60 under Current Conditions

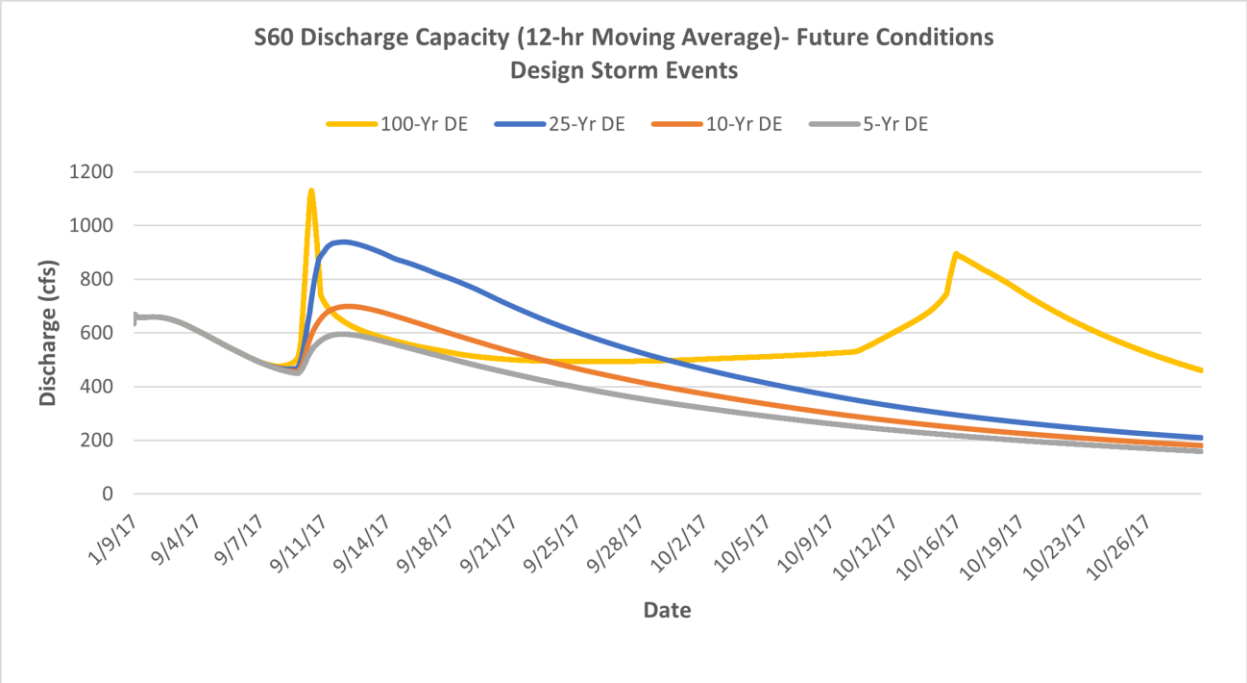


Figure 10-15: 12-hr Moving Average Flows in cfs at S60 under Future Conditions

10.1.4 PM #4

PM#4-UKB was not assessed in this LMA.

10.1.5 PM #5

The maximum overland depth was extracted for each design storm event and evaluated for Alligator Lake LMA under current and future conditions. **Table 10-17** is a tabulation of the flood inundation area in square miles for the entire LMA, shown in incremental intervals of feet. **Figure 10-16** presents the data in a plot format. **Figure 10-17** through **Figure 10-20** present the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions.

The table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding and higher than 2.5 feet of flooding. The flooding at the lower depths can be considered nuisance flooding, or the time periods during sheet flow. The higher depths of flooding are the large lakes, which is expected. The plot clearly shows how stages are increasing under future conditions, relative to current conditions. In addition, future conditions simulations show the middle range of flooding depths in the table with larger areas of inundation, compared to the current conditions. It should be noted that areas with less flooding depths in future conditions relative to current conditions correspond to areas where the DEM/topography was increased in the future conditions model due to DWM projects.

Table 10-17: Incremental Flood Inundation Area (sq. mi.) in Alligator Lake LMA

Flooding Depth (ft)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	26.85	23.72	18.73	11.14	27.41	24.20	18.88	11.19
0.25<=Depth<0.5	6.39	5.86	7.93	10.14	5.85	7.08	8.07	10.19
0.5<=Depth<0.75	3.99	5.49	2.81	3.37	3.98	3.85	2.85	3.46
0.75<=Depth<1.0	3.34	3.40	2.09	2.30	3.33	3.36	5.17	2.31
1.0<=Depth<1.25	1.95	2.72	1.88	1.81	1.96	2.70	3.47	1.86
1.25<=Depth<1.5	1.61	1.53	1.84	1.71	1.61	1.54	2.37	1.81
1.5<=Depth<1.75	1.27	1.22	1.63	1.69	1.27	1.22	1.60	1.86
1.75<=Depth<2.0	0.80	1.53	1.36	2.05	0.79	1.53	1.37	1.93
2.0<=Depth<2.25	0.59	0.69	1.39	1.75	0.60	0.69	1.37	1.66
2.25<=Depth<2.5	0.50	0.68	0.65	1.60	0.50	0.68	0.66	1.49
>2.5	5.36	5.81	6.88	15.09	5.36	5.80	6.89	14.92
Lake Alligator LMA	**33702.65**	**acres**						
	52.66	**sq. mi.**						

***Note**: The lake area of 12.24 sq. mi. w included in this table computations.*

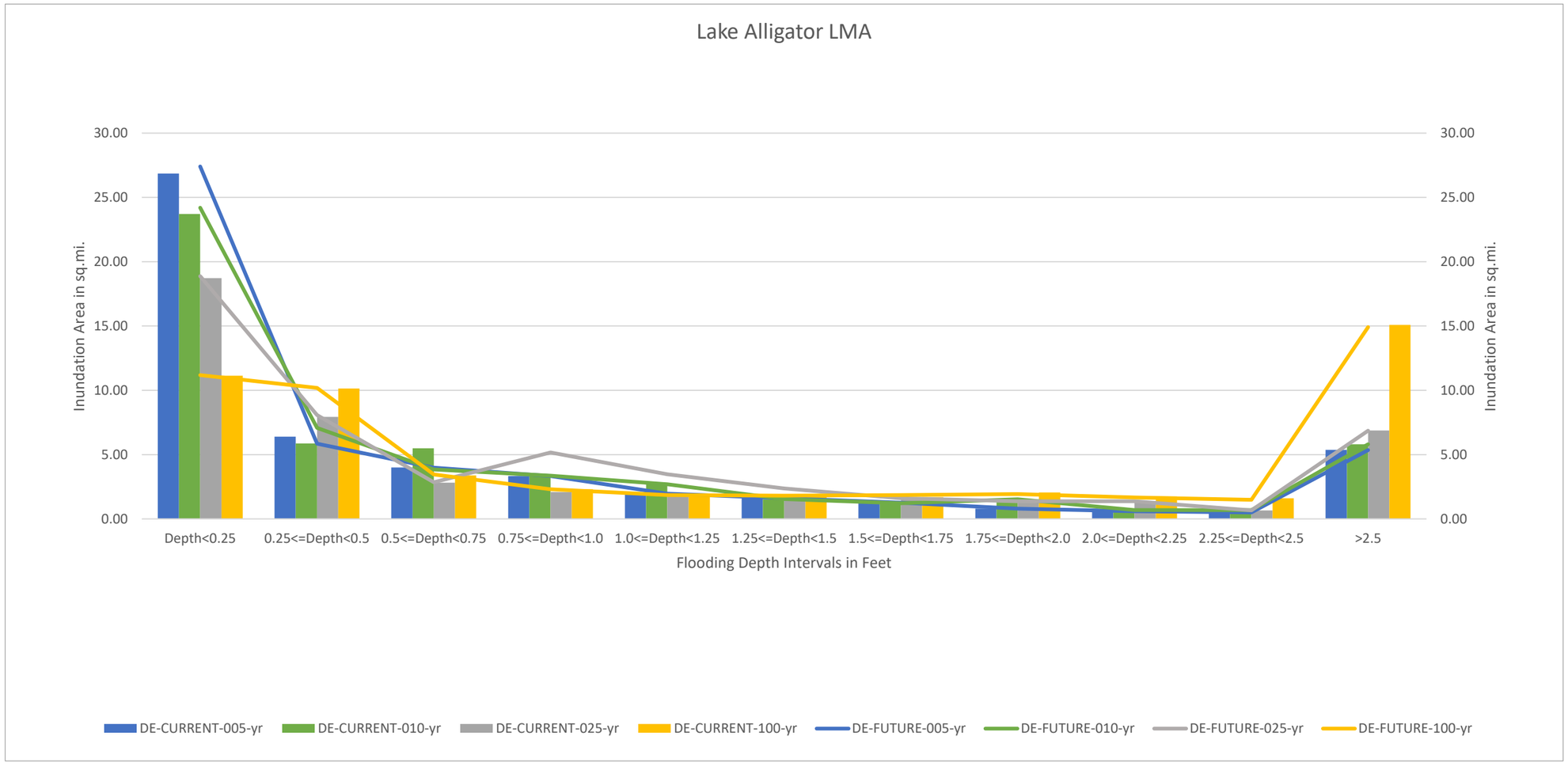


Figure 10-16: Incremental Flood Inundation Depth in Feet in Alligator Lake LMA under Current and Future Conditions

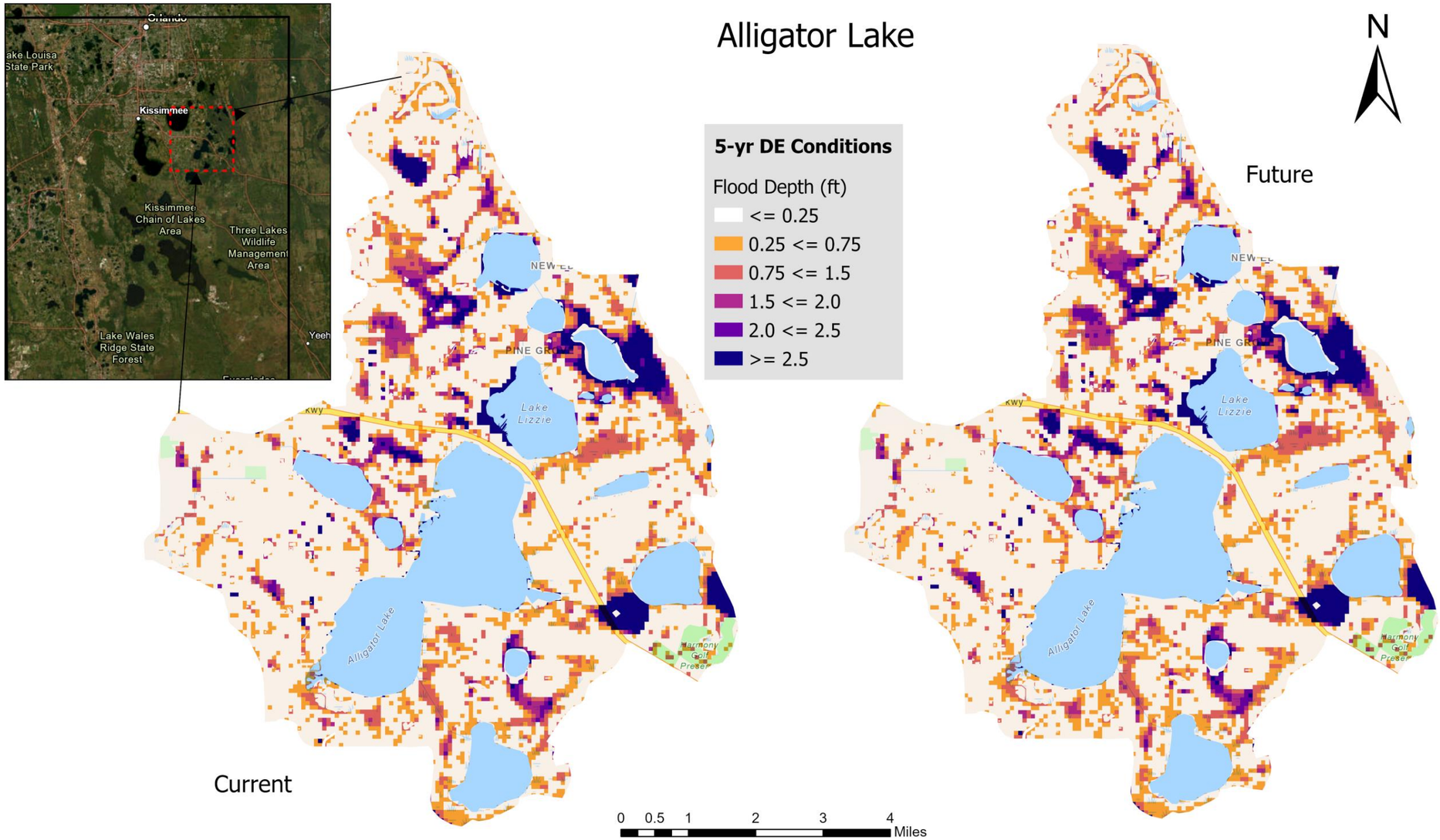


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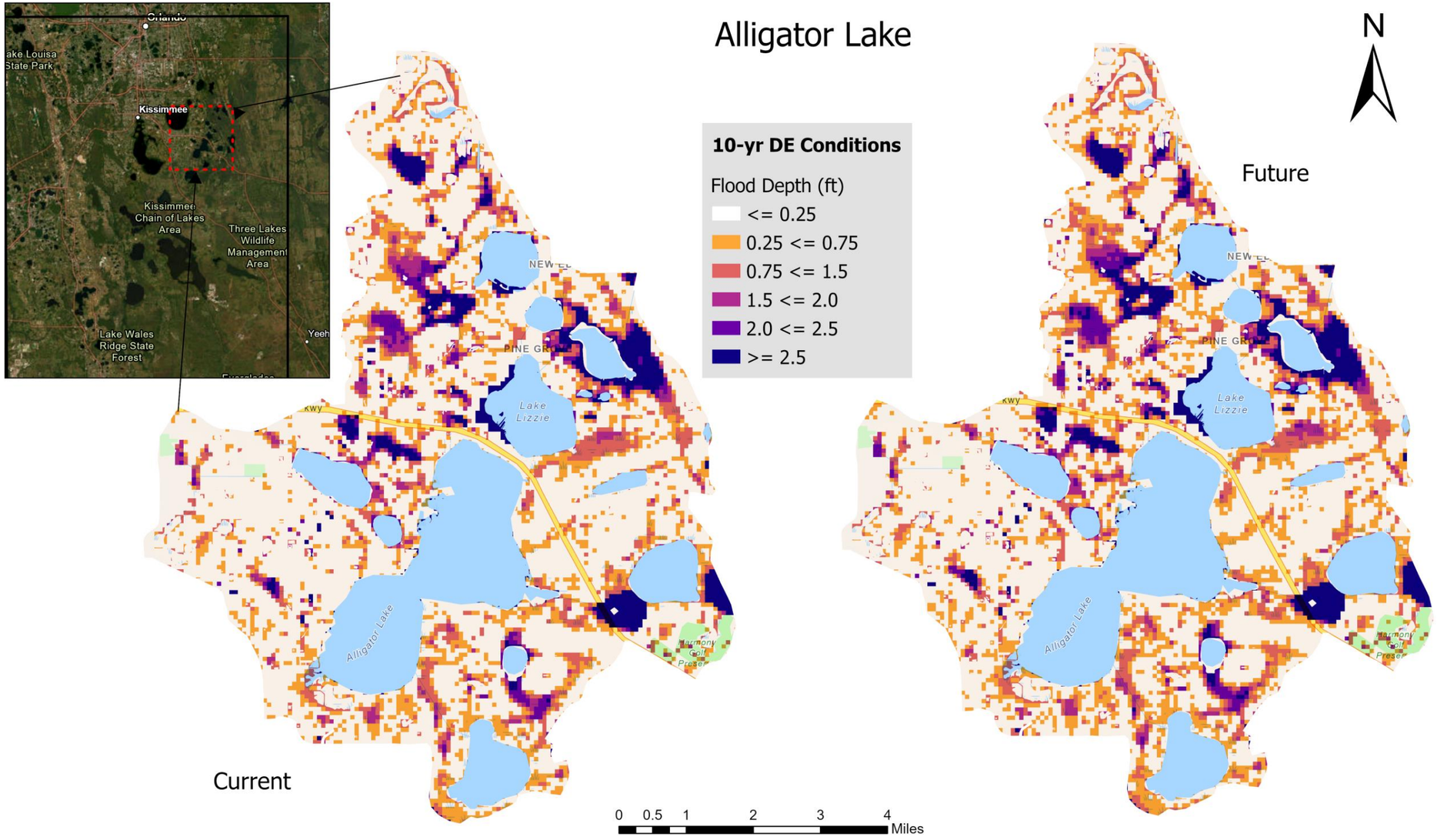
Figure 10-17: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA



Hazen

Figure 10-18: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

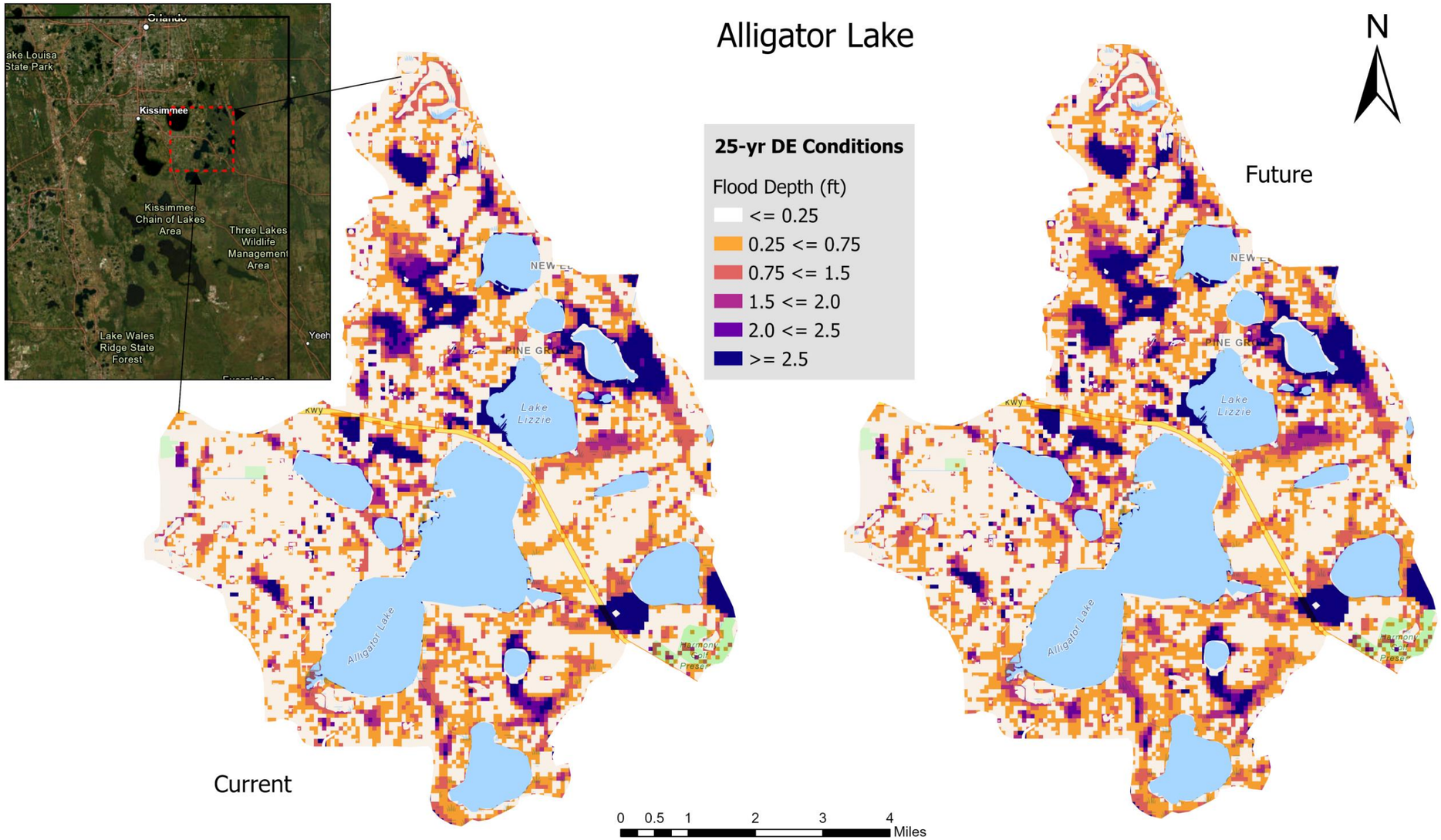


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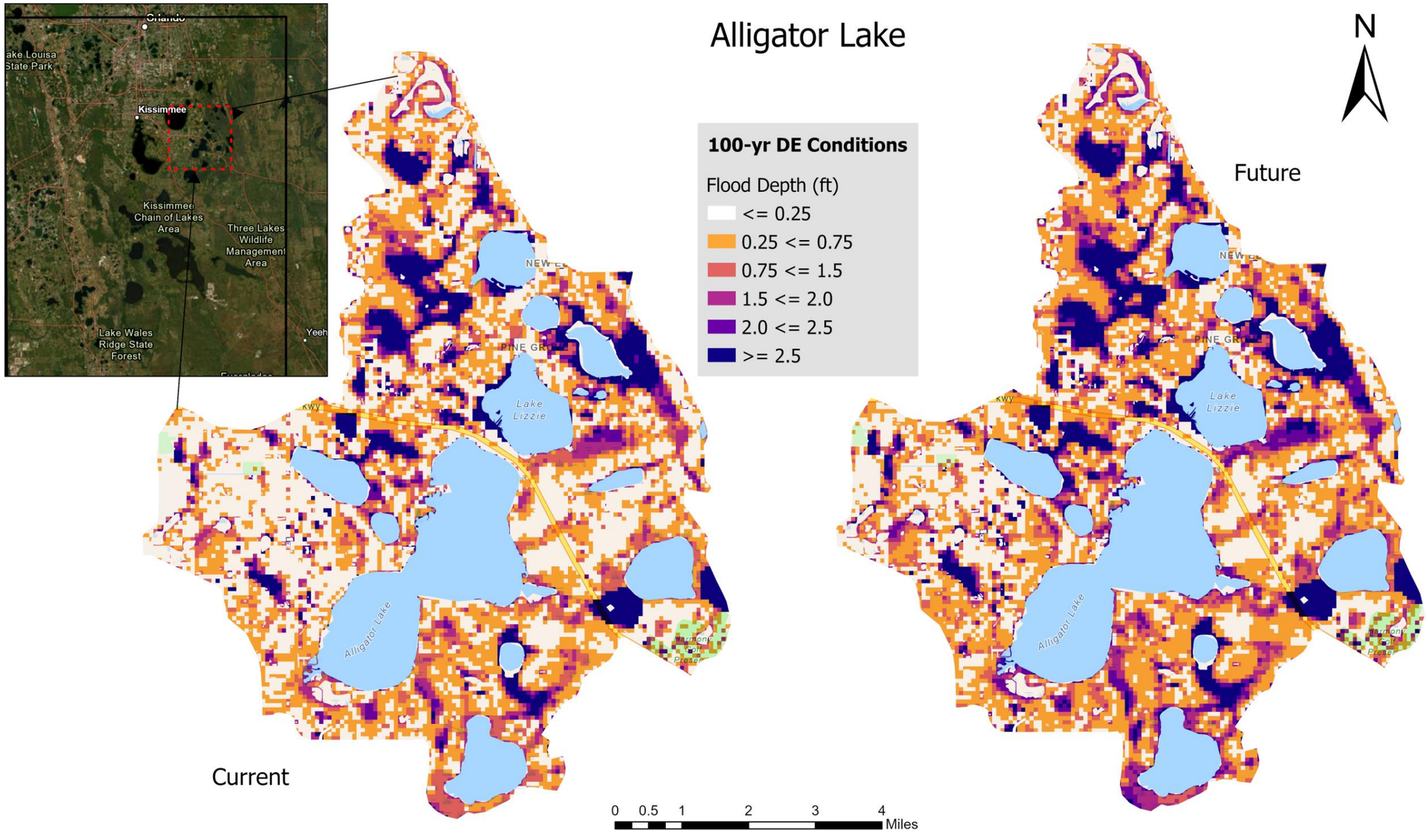
Figure 10-19: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA



Hazen

Figure 10-20: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

10.1.6 PM #6

Table 10-18 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Alligator Lake LMA. **Figure 10-21** provides a graphical representation of this data, with the flood duration plotted against the percentage of area inundated for the areas within the Alligator Lake LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-22 through Figure 10-25** are maps of flood duration for the design events under current and future conditions.

The table, the plot and the figures show that the greatest distribution of flooding duration in the LMA is between zero and one hour of flooding, which can be considered insignificant, except on the roads. Flooding duration at the higher end is on the large lakes, as expected. Due to this reason, Alligator Lake was masked out in the figures. Some of the mid-range flooding depths observed are at the edges of the medium and small sized lakes.

This LMA stays flooded for extended periods of time surrounding the lakes and canals. Based on this analysis, the areas of concern are:

- Nova Road on C-32C canal
- Nova Road between lakes Coon and Center
- Irlo Bronson Highway between lakes Lizzie and Alligator
- Areas surrounding lakes Conlin, Buck, Bay, Brick and Cat

Table 10-18: Areas of Incremental Flood Duration (sq. mi.) in Alligator Lake LMA

Flooding Duration (Hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	29.81	26.65	22.80	15.75	29.97	27.42	23.05	15.83
1<=Hours<4	1.43	1.88	3.11	6.10	1.36	2.10	3.22	6.12
4<=Hours<8	0.87	0.91	1.16	1.37	0.89	1.09	1.19	1.44
8<=Hours<12	0.58	0.49	0.78	0.71	0.59	0.58	0.69	0.74
12<=Hours<24	1.28	1.32	1.40	1.46	1.39	1.67	1.64	1.54
24<=Hours<48	1.58	1.26	1.02	1.10	1.64	1.80	1.27	1.12
48<=Hours<96	1.61	0.82	1.04	1.13	1.59	1.79	1.17	1.17
96<=Hours<168	3.34	0.71	1.00	0.90	1.45	1.56	2.31	0.94
168<=Hours<240	0.50	0.48	0.58	0.61	0.86	0.99	4.97	0.74
240<=Hours<360	0.60	0.37	0.63	0.75	1.01	1.00	0.69	0.93
360<=Hours<1417	11.06	17.77	19.14	22.78	11.91	12.65	12.46	22.08
Lake Alligator LMA	**33702.65**	**acres**						
	52.66	**sq.mi.**						

Note: *The lake area of 12.24 sq. mi. was included in this table computations*

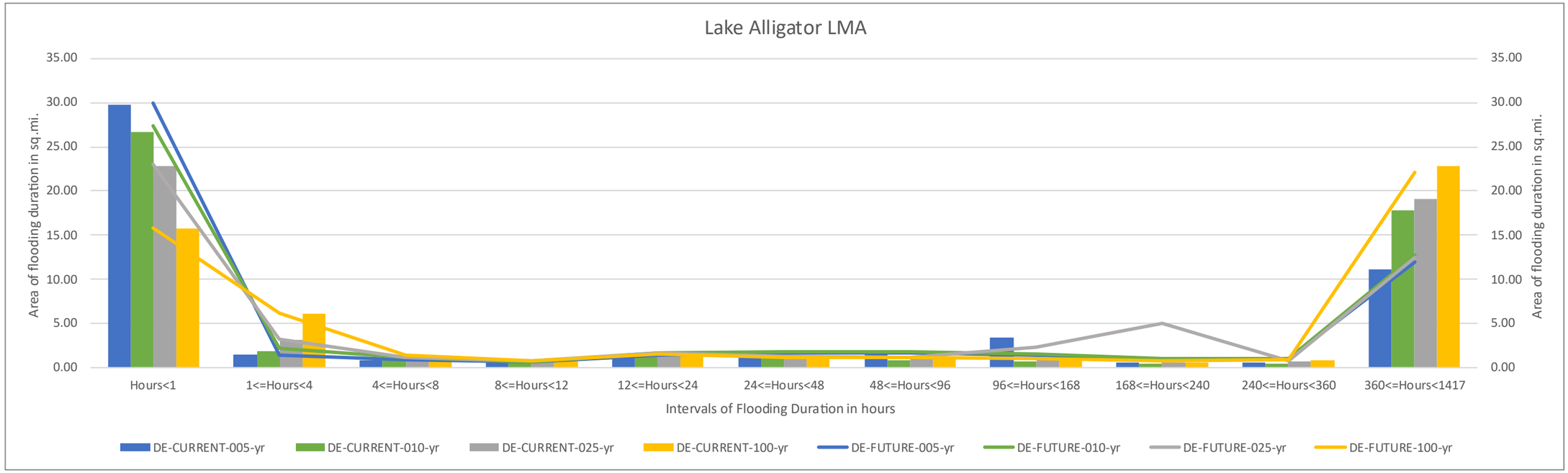


Figure 10-21: Areas of Incremental Flood Duration in hours in Alligator Lake LMA under Current and Future Conditions

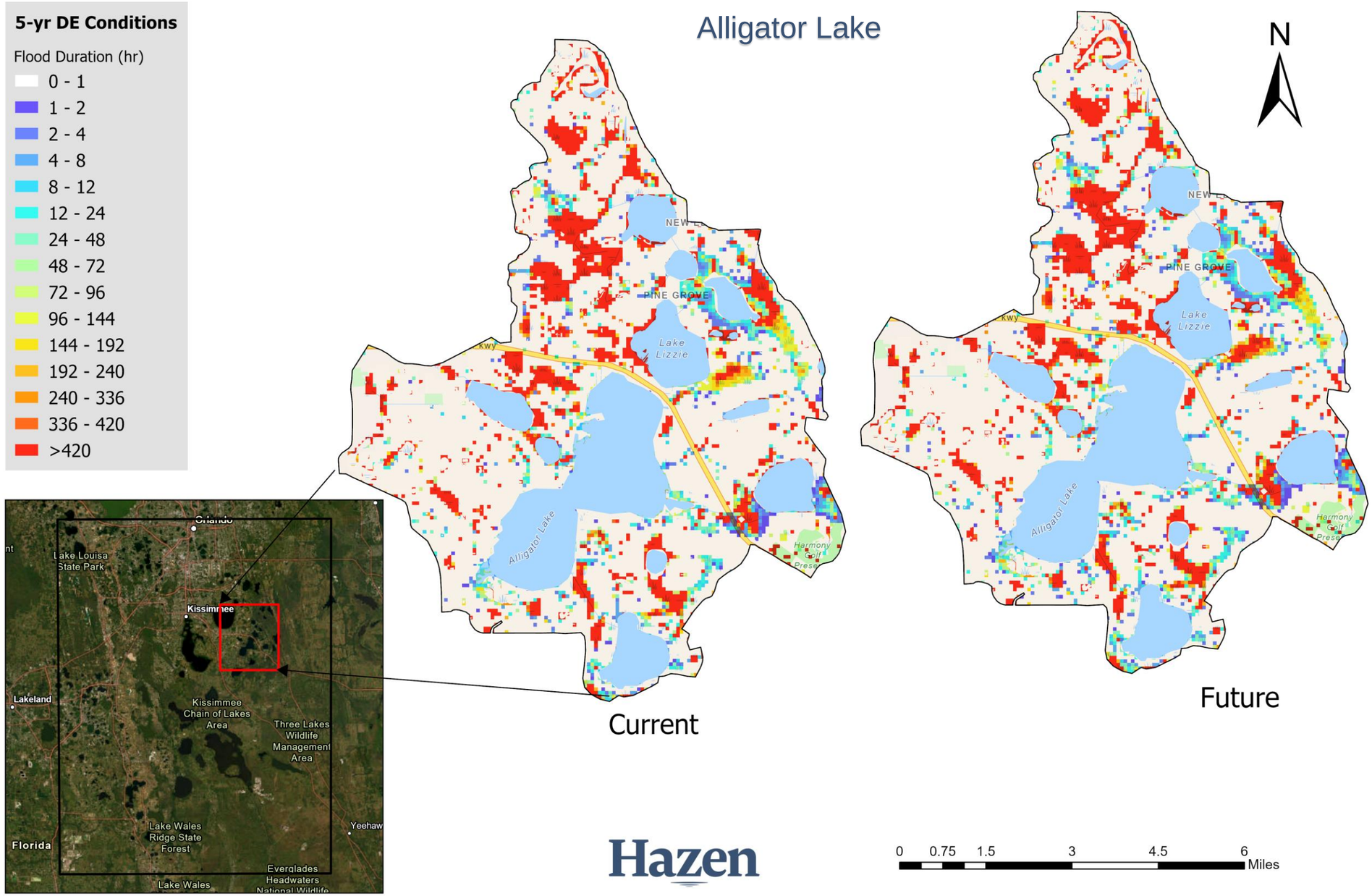


Figure 10-22: Flood Duration Maps for 5-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

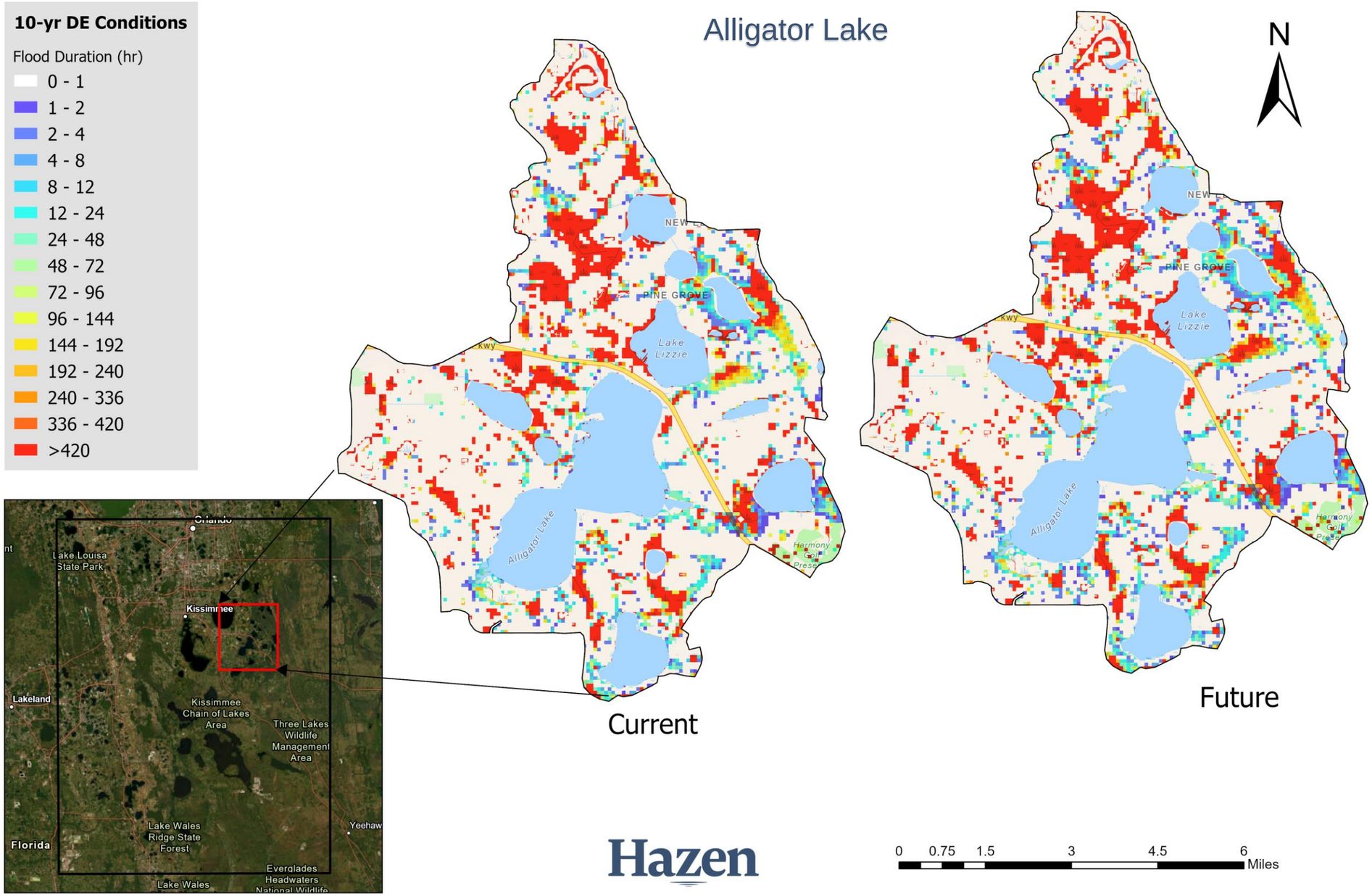


Figure 10-23: Flood Duration Maps for 10-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

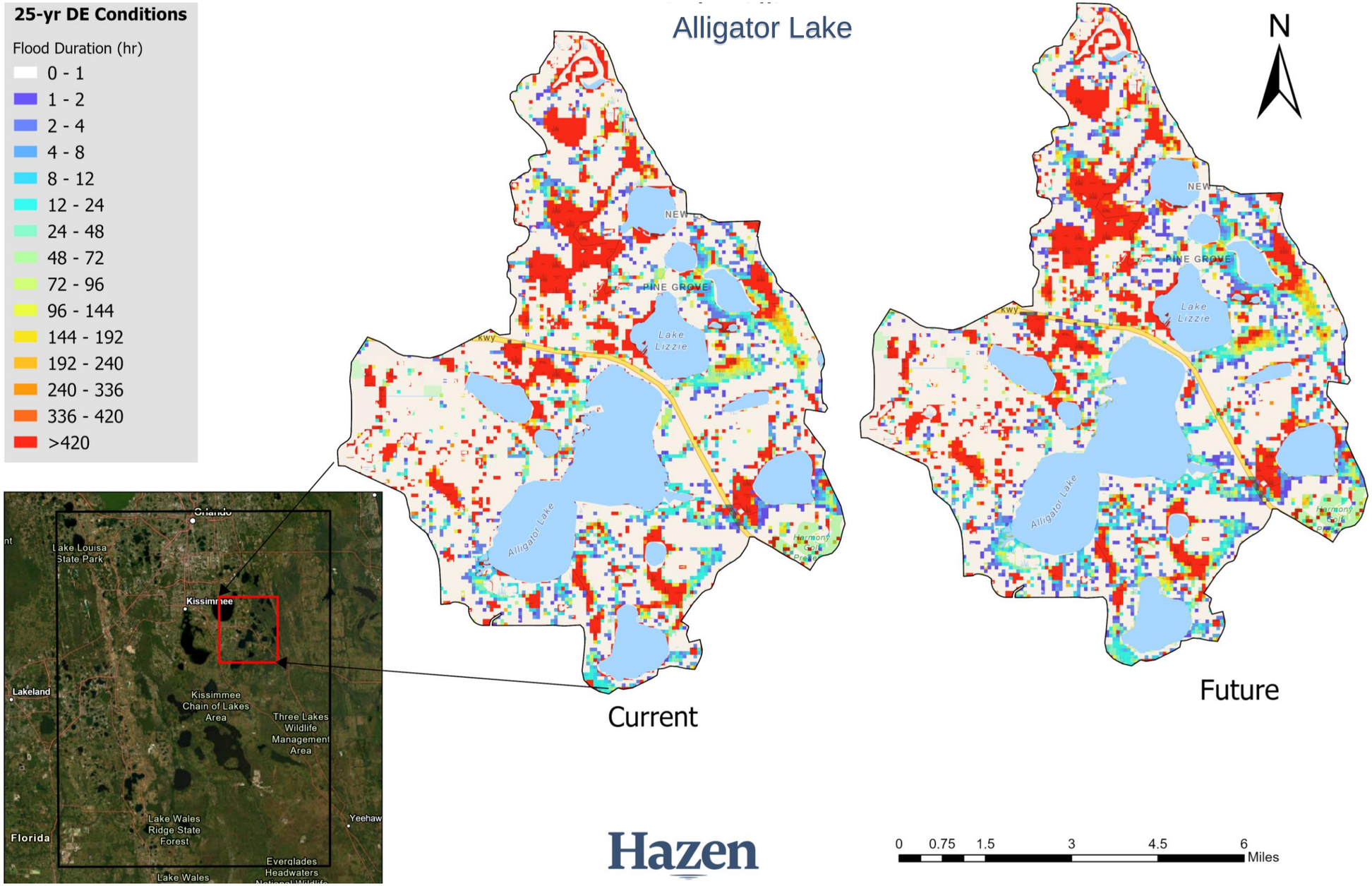


Figure 10-24: Flood Duration Maps for 25-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

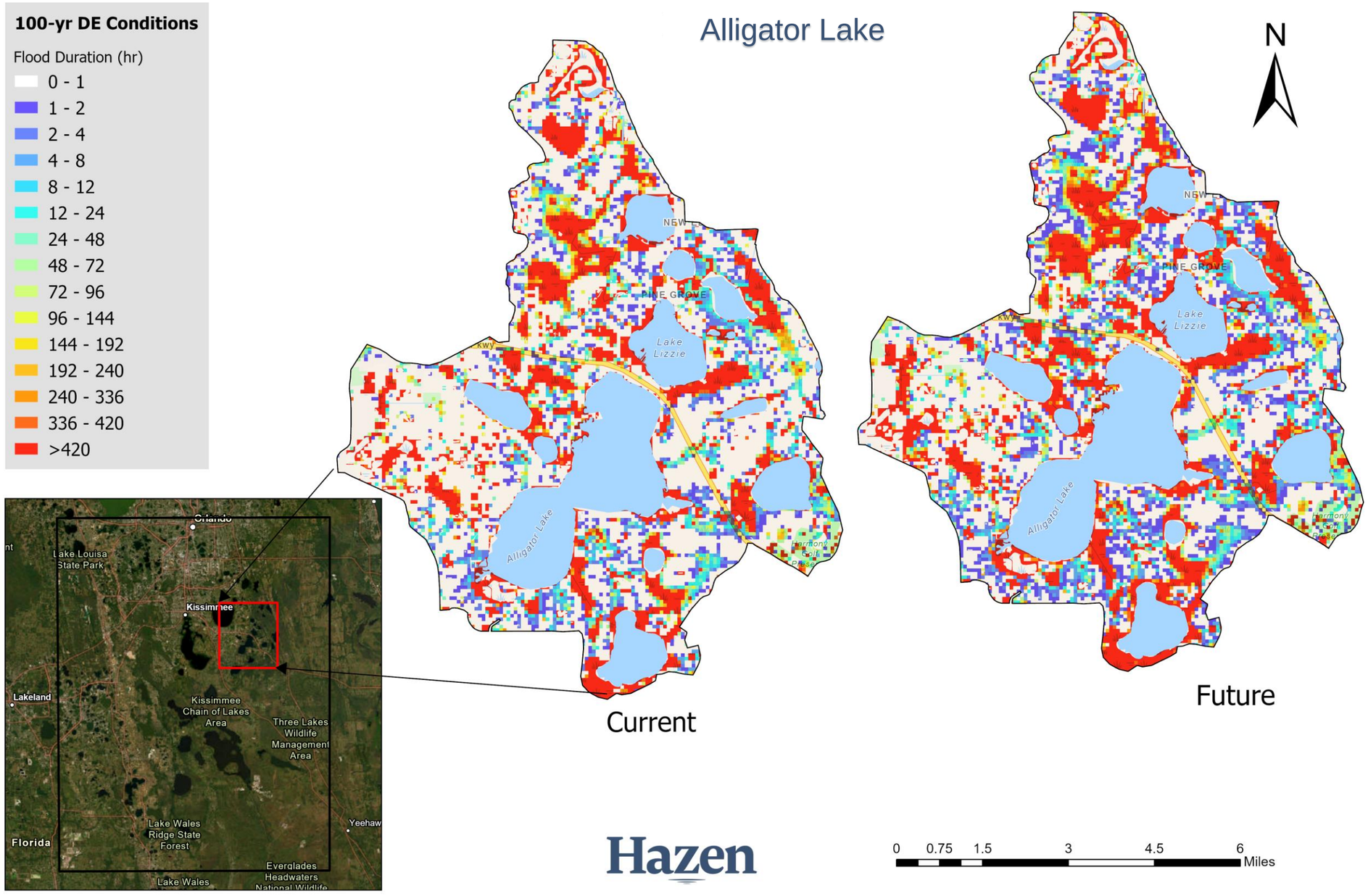


Figure 10-25: Flood Duration Maps for 100-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

10.1.7 FPLOS Rating of Alligator Lake LMA

Table 10-19 provides a summary of evaluation of Alligator Lake LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The FPLOS rating for the LMA ranges from <5-yr to 100-yr under current conditions, and from < 5-yr to 100-yr under future conditions. The overall FPLOS rating for Lake Alligator LMA, considering all PMs is less than 5-yr.

Table 10-19: Summary of PM Analysis in Alligator Lake LMA

Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Alligator, C-32C and C-33 canals	Lake Alligator water levels mostly stay above banks for all simulation events; The water levels exceed the banks almost all along the banks of Alligator Lake for 100-yr storms. Along Canal C-33, the water levels stay within the banks for all storms. For 25- and 100- yr storms, the water overflows the Canal C-32C banks.	<5-yr in Alligator Lake 100-yr in Canal C-33 25-yr in Canal C-32C	<5-yr in Alligator Lake 100-yr in Canal C-33 10-yr in Canal C-32C
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 13.9 CSM under current conditions and 14.1 CSM under future conditions, which is higher than SFWMD's permissible discharge allowance of 13 CSM based on a 10-yr design frequency.	10-yr discharge higher than permissible discharge	10-yr discharge higher than permissible discharge
PM#3	Entire LMA	The simulated flow at structure S60 is lower than the design discharge (450 cfs) for all design storm events in the current condition, however for future conditions the flows are higher for all the design storms.	< 25-yr	< 5-yr
PM#4-UKB	-	-	-	-
PM#5	Entire LMA	Approximately 50% of the LMA has flooding depth lower than 0.25 ft for the 5-yr storm and approximately only 21% of the LMA has flooding depth lower than 0.25 ft for the 100-yr storm. The area of inundation increases for all storms in future conditions.	5-yr	5-yr
PM#6	Entire LMA	Approximately 61% of the LMA stays flooded for shorter than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately only 44% of the LMA stays flooded for shorter than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm. The duration changes by less than 1% for all storms under future conditions	5-yr	5-yr

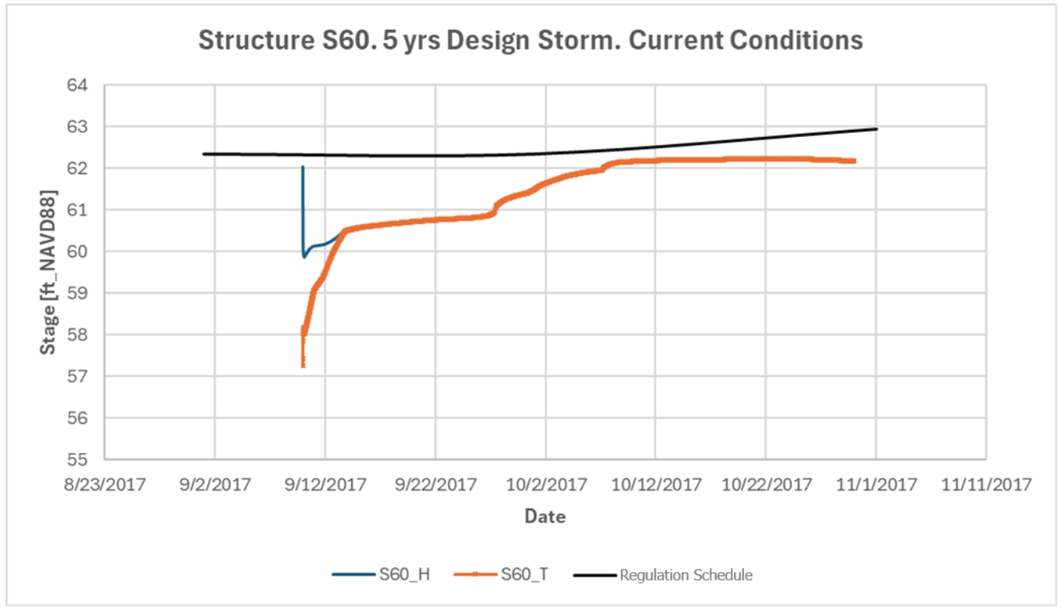
Lake Alligator LMA Overall FPLOS	**Current Conditions**	**< 5-yr**	
	Future Conditions	**< 5-yr**	

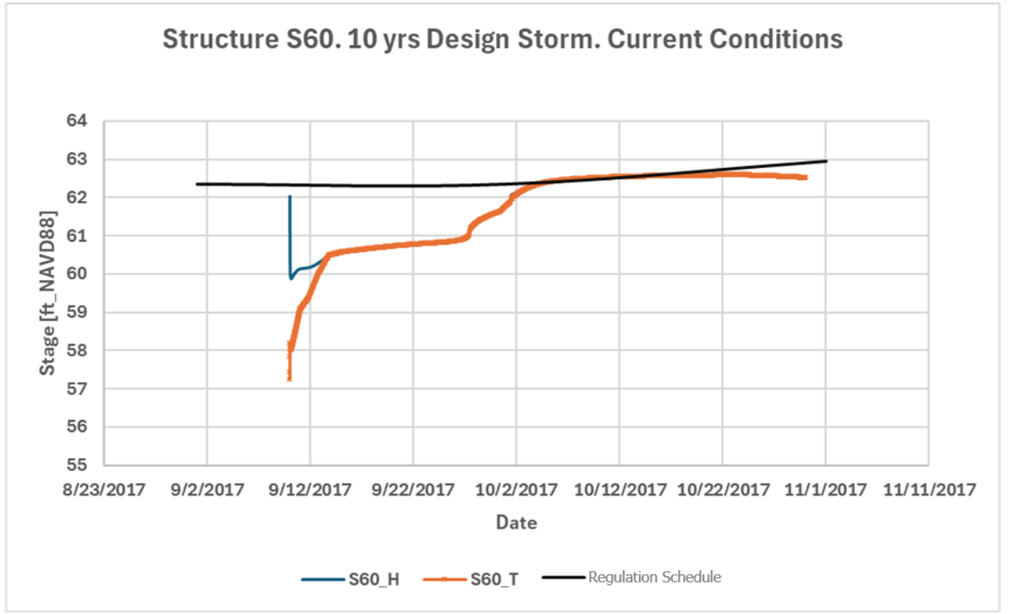
10.1.8 Volume in Lake due to Design Storm UKB-$V_{Alligator}$: Alligator Lake

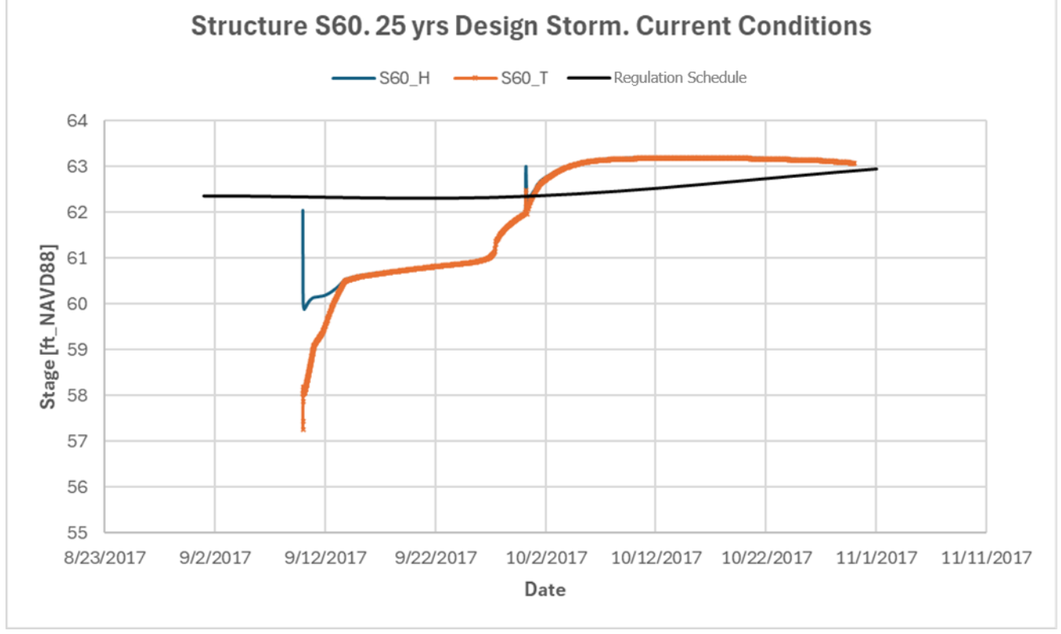
Table 13-3 summarizes the water volume change in Lake Alligator, measured as the difference between final and initial volume during the simulation. The results show that both current and future conditions have excess water due to the design storms; however, the results would vary if the initial lake levels were different. **Table 13-4** shows the volume of water at peak for Lake Alligator.

10.1.9 Duration of Lake Recovery to Regulation Schedule UKB-T_{Reg}: Alligator Lake

This section examines at S60, the duration of water levels above the regulation schedule in Alligator Lake. **Figure 10-26** through **Figure 10-28** shows headwater and tailwater stages at S60, as well as the regulation schedule for the entire simulation period, for current and future conditions respectively. Structure S60, as shown, stays around the regulation schedule for about 10 days and then falls below it under 10-yr current conditions. Under current conditions for 25- and 100-yr design storms, lake water levels do not get to the regulation schedule line after exceeding it. In all scenarios, except for the 100-year design case, the water level in Alligator Lake never surpasses the regulation schedule for future conditions.







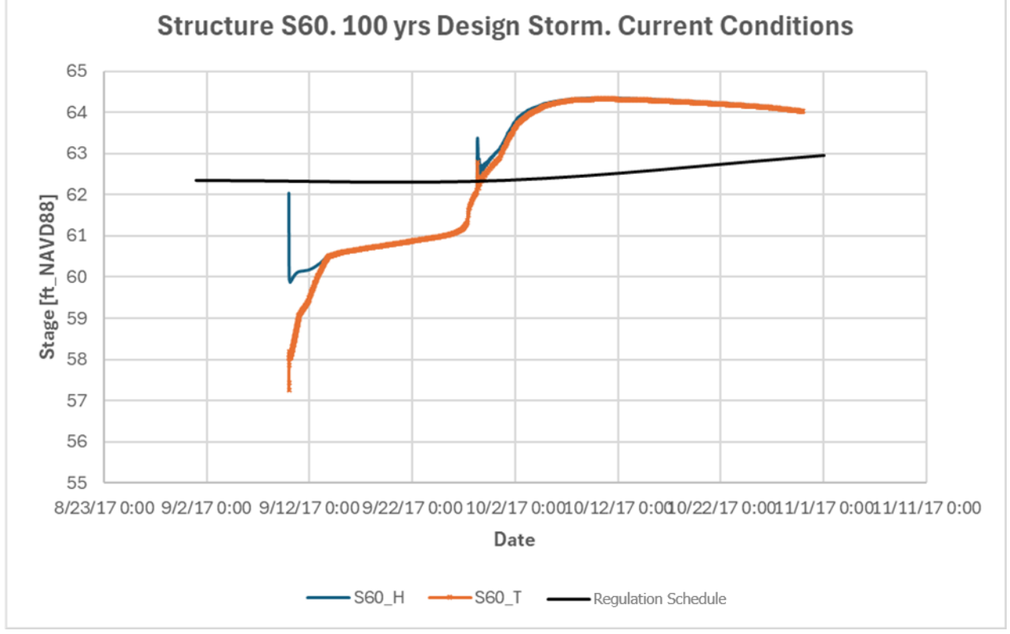
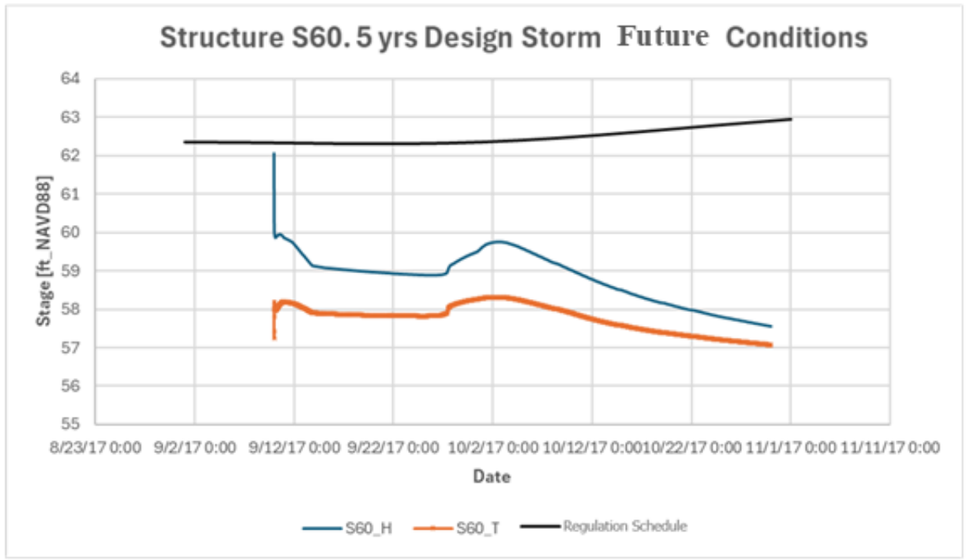
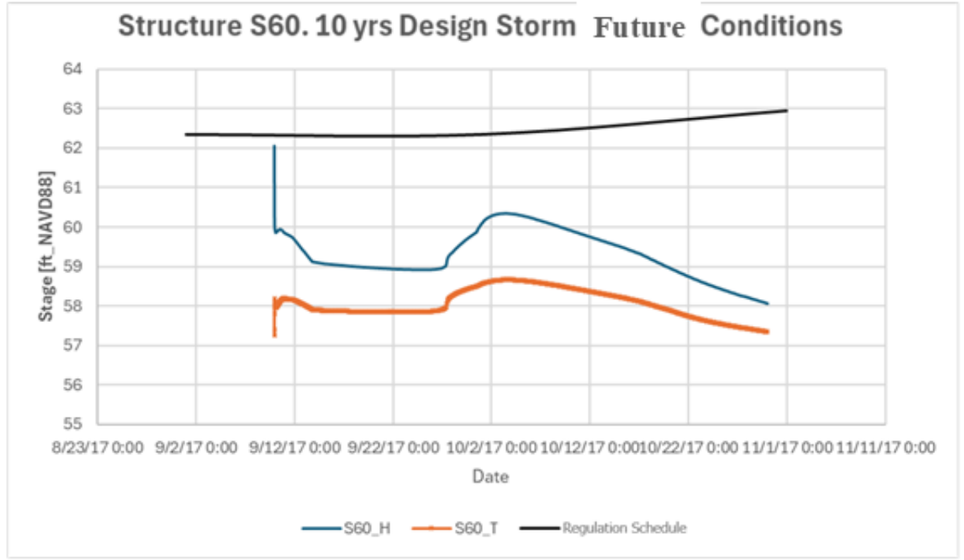
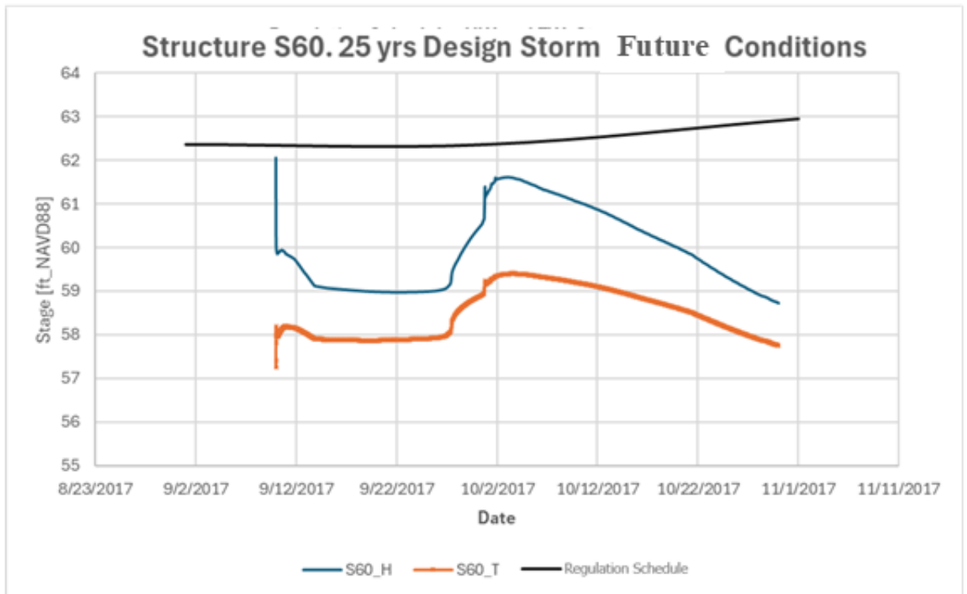


Figure 10-26: Headwater and Tailwater at Structure S60 and Regulation Schedule for 5-, 10-, 25- and 100-yr Design Storms under Current Conditions







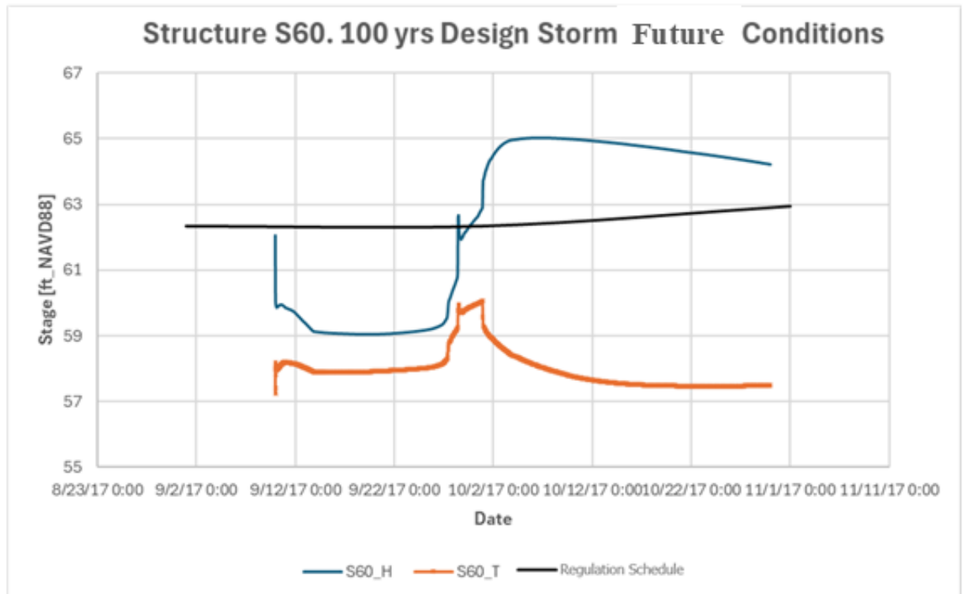


Figure 10-27: Headwater and Tailwater at Structure S60 and Regulation Schedule for 5-, 10-, 25- and 100-yr Design Storm under Future Conditions

10.2 Lake Myrtle LMA

Figure 10-28 shows Lake Myrtle LMA with primary structures, roadways and waterways

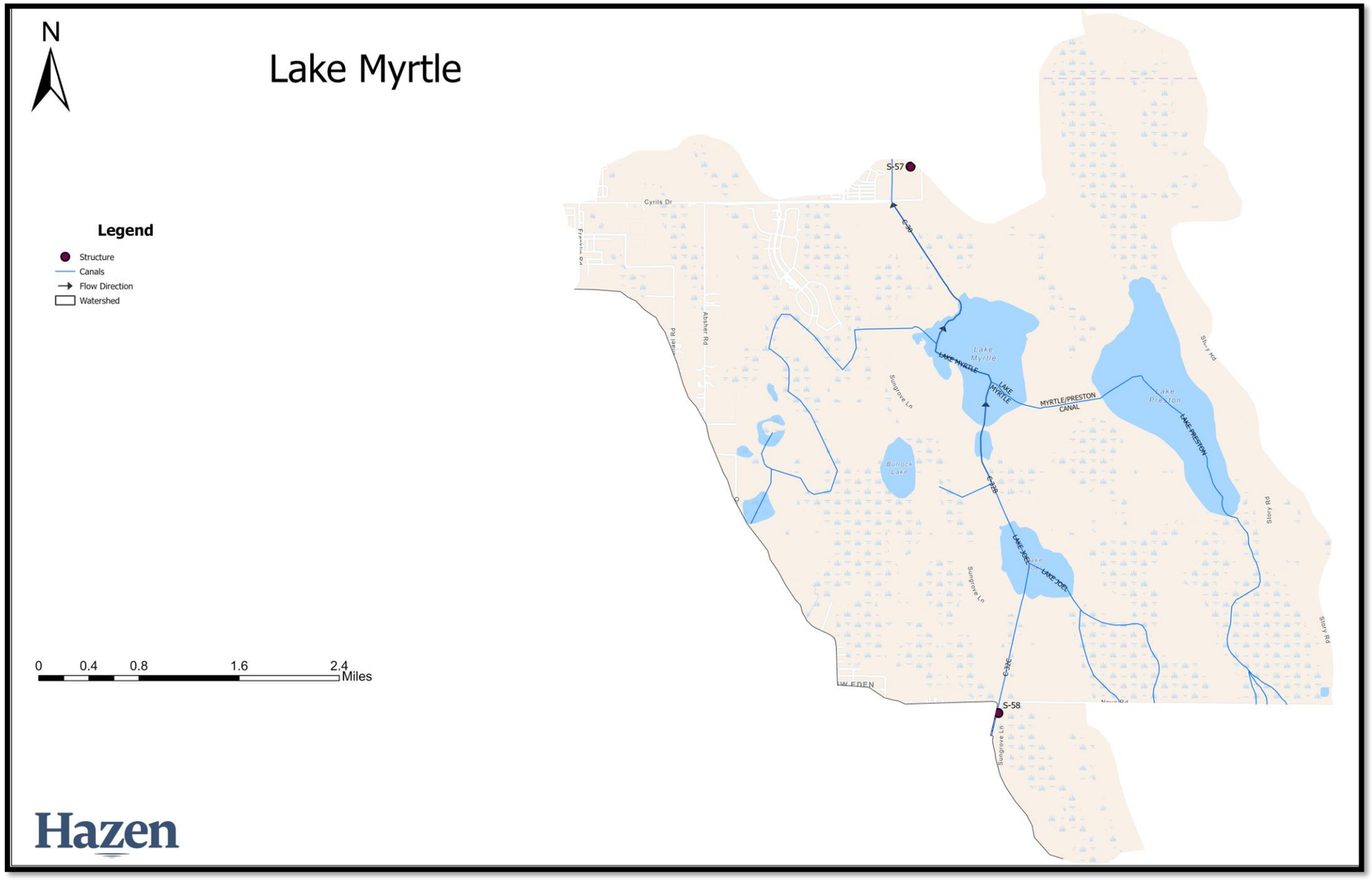


Figure 10-28: Lake Myrtle LMA with Main Structures, Waterways and Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.2.1 PM #1

PM #1 for Lake Myrtle LMA was analyzed at lakes Myrtle and Preston, and C-32B and C-30 canals; Canal C-32B connects lakes Myrtle and Joel, and Canal C-30 connects Lake Myrtle to Lake Mary Jane, the downstream LMA. **Figure 10-29 through Figure 10-33** show the water profiles under current conditions and **Figure 10-34 through Figure 10-35** show the same under future conditions. **Figure 10-36** shows Lake Myrtle LMA and the canals connecting the lakes in this LMA.

For Lake Myrtle LMA, under current conditions, both the left and right banks are exceeded for the 100-year event. The water levels at the left bank exceed at a few locations while the right bank sees more exceedances for the smaller design storms, such as the 5-, 10-, and 25-year events. In Canal C-32B, under current conditions, both banks are exceeded for 100-, 25-, and 10-year design storms, with all water levels staying above the bank elevations, and a 5-year storm event doesn't exceed any bank. For Lake Preston, both the left and right banks are exceeded only during the 100-year event in most areas. The smaller events, 5- and 10-year, stay within the banks in all places, except 25-year, exceeding both banks in a few places, with the right bank being exceeded more. In Canal C-30, the banks are exceeded for the 100-year event for the first half of the area, while smaller events remain within the banks.

Under future conditions, Lake Myrtle sees similar trends, with the 100- and 25-year event exceeding both the left and right banks at all locations. The water levels for the smaller events exceed bank levels in various locations, with the right bank experiencing more significant exceedances. For Canal C-32B, the future conditions mirror the current scenario for all design storms. For Lake Preston, the banks are exceeded only during the 100-year event, with no exceedances for smaller events of 5- and 10-year, except 25-year storm exceeding one or both banks in two places. In Canal C-30, the pattern of exceedance is similar to current conditions, with the 100-year event exceeding both banks before the S57 gate. Future conditions generally show higher water levels but follow the same overall pattern of exceedances as current conditions, particularly for the 100-year event across these LMAs.

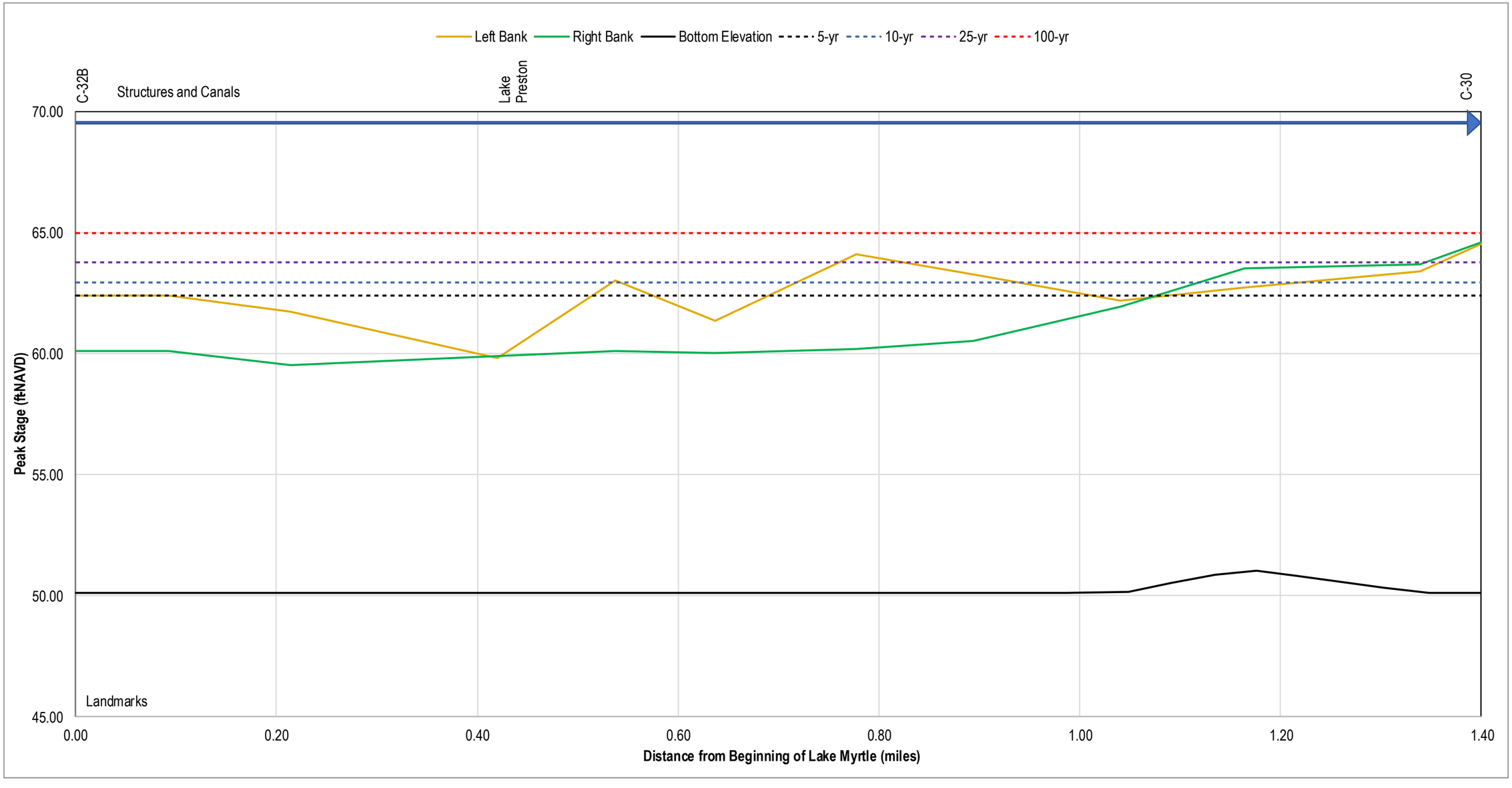


Figure 10-29: Maximum Stage Profile for Lake Myrtle under Current Conditions

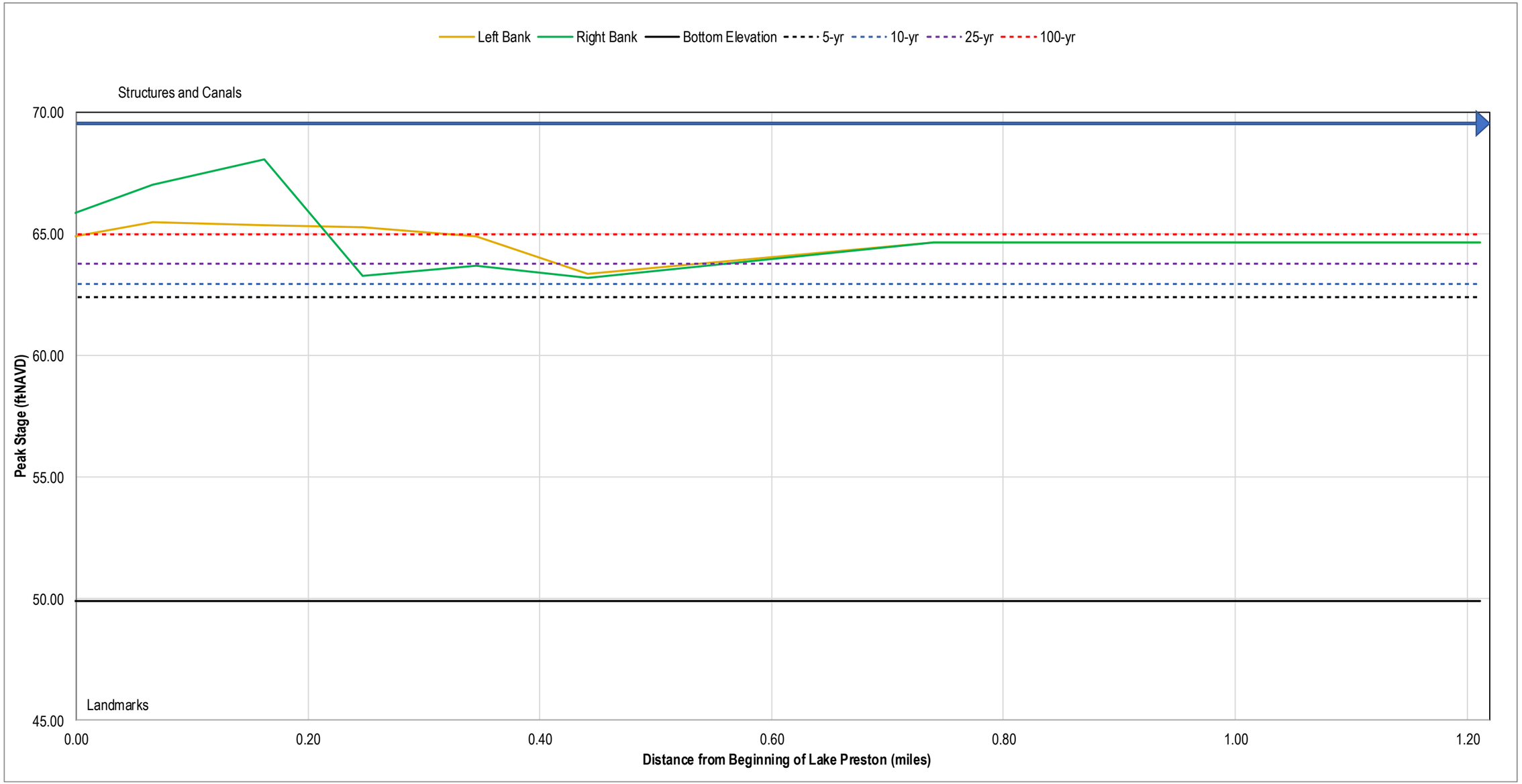


Figure 10-30: Maximum Stage Profile for Lake Preston under Current Conditions

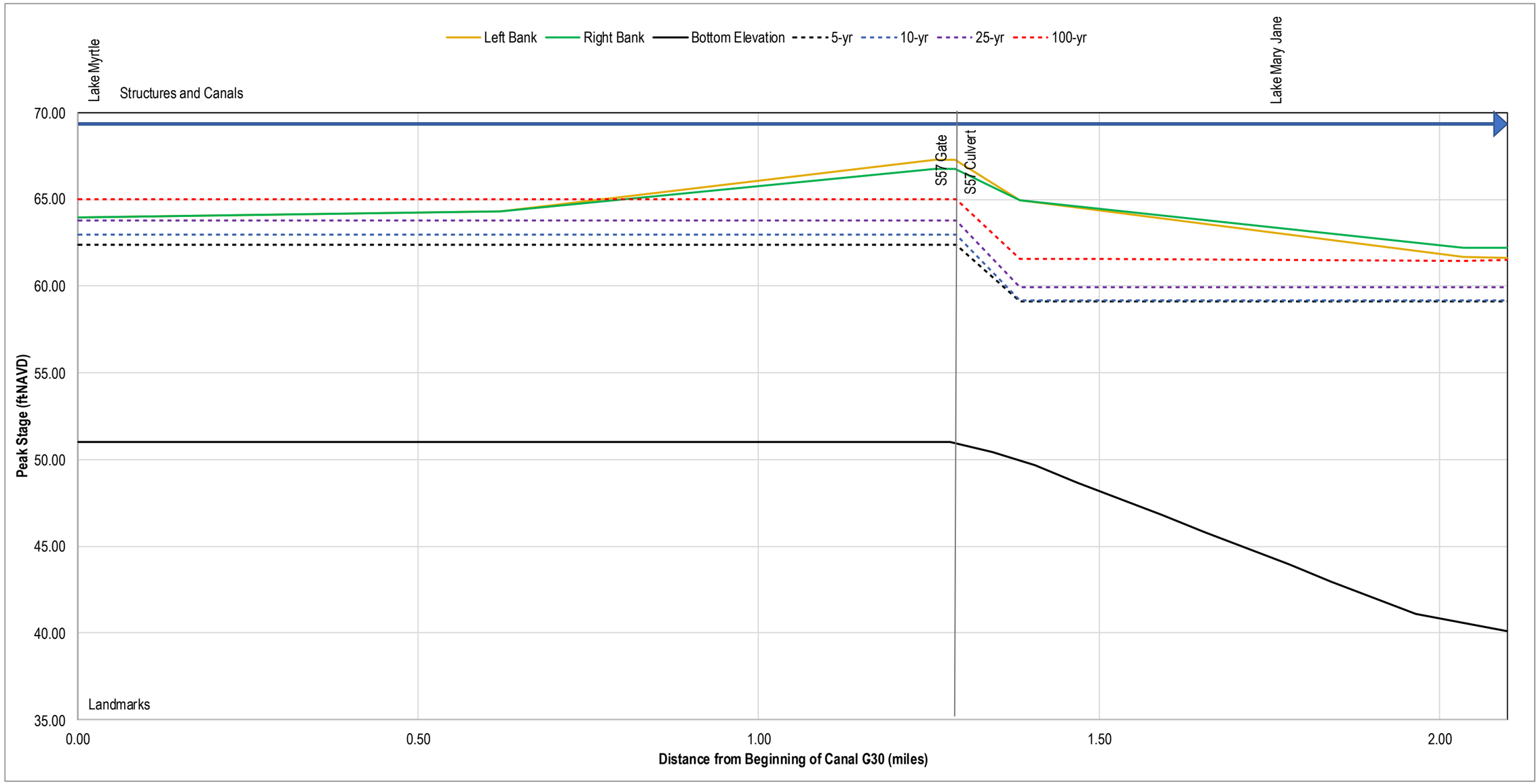


Figure 10-31: Maximum Stage Profile for C-30 Canal under Current Conditions

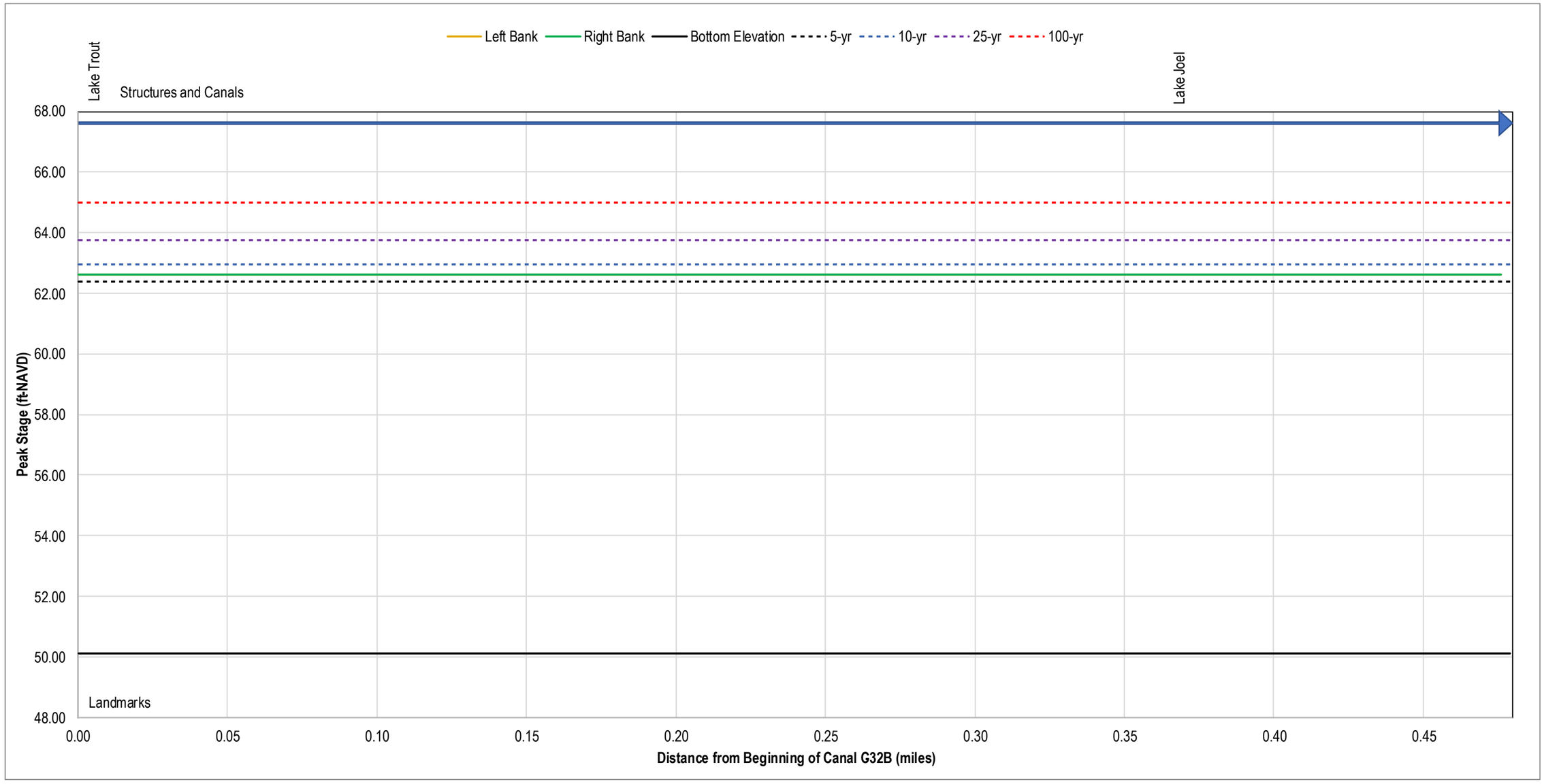


Figure 10-32: Maximum Stage Profile for C-32B Canal under Current Conditions

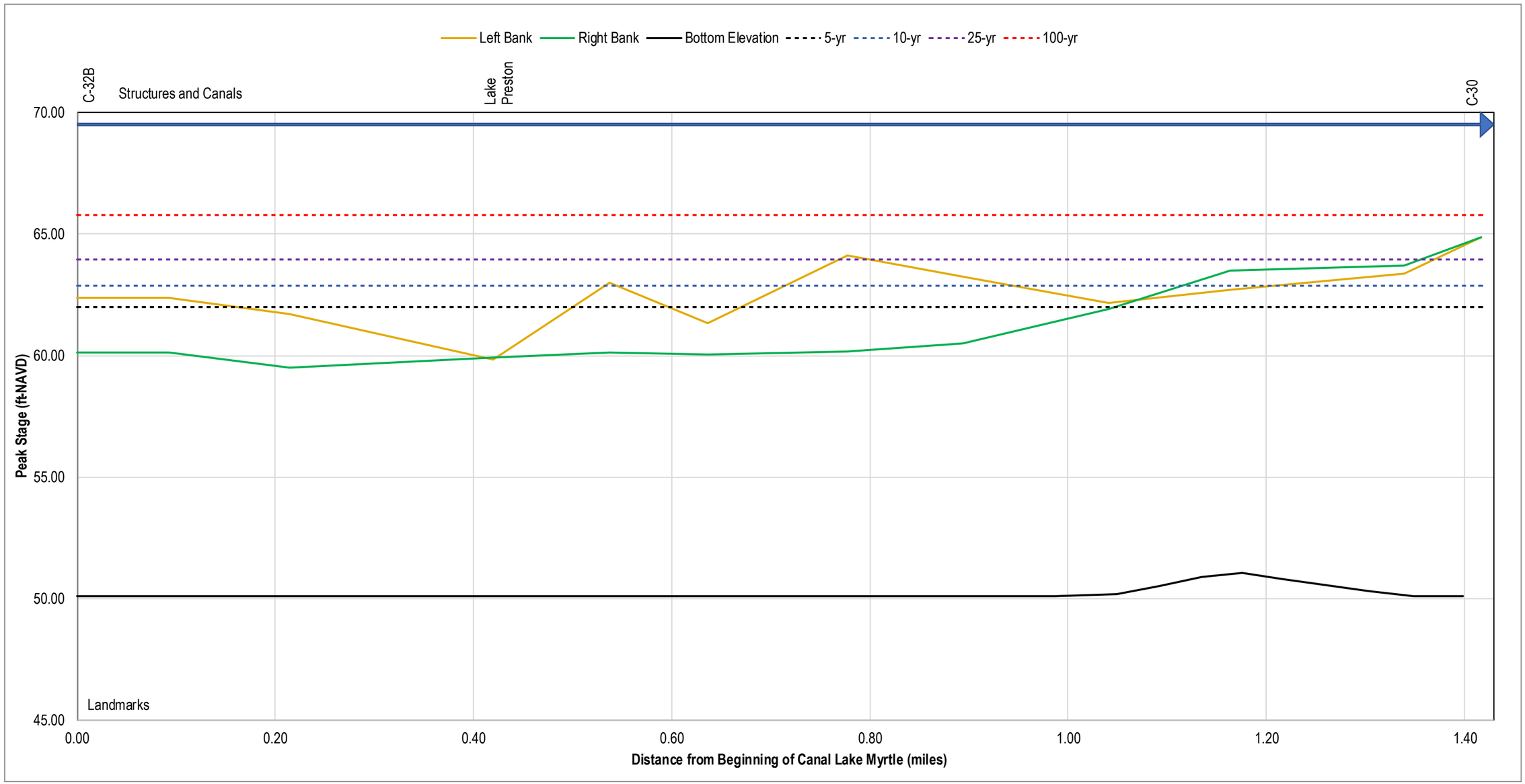


Figure 10-33: Maximum Stage Profile for Lake Myrtle under Future Conditions

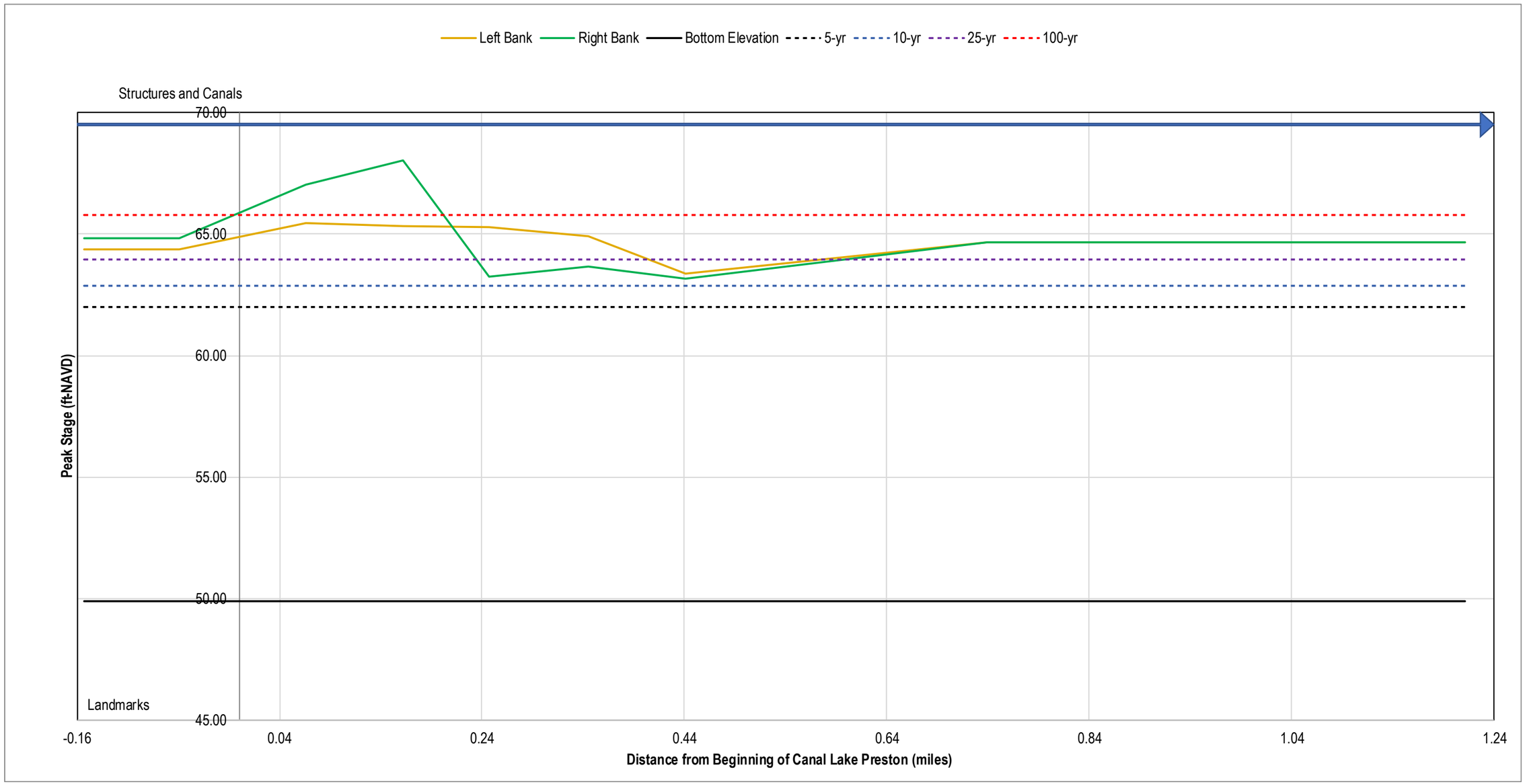


Figure 10-34: Maximum Stage Profile for Lake Preston under Future Conditions

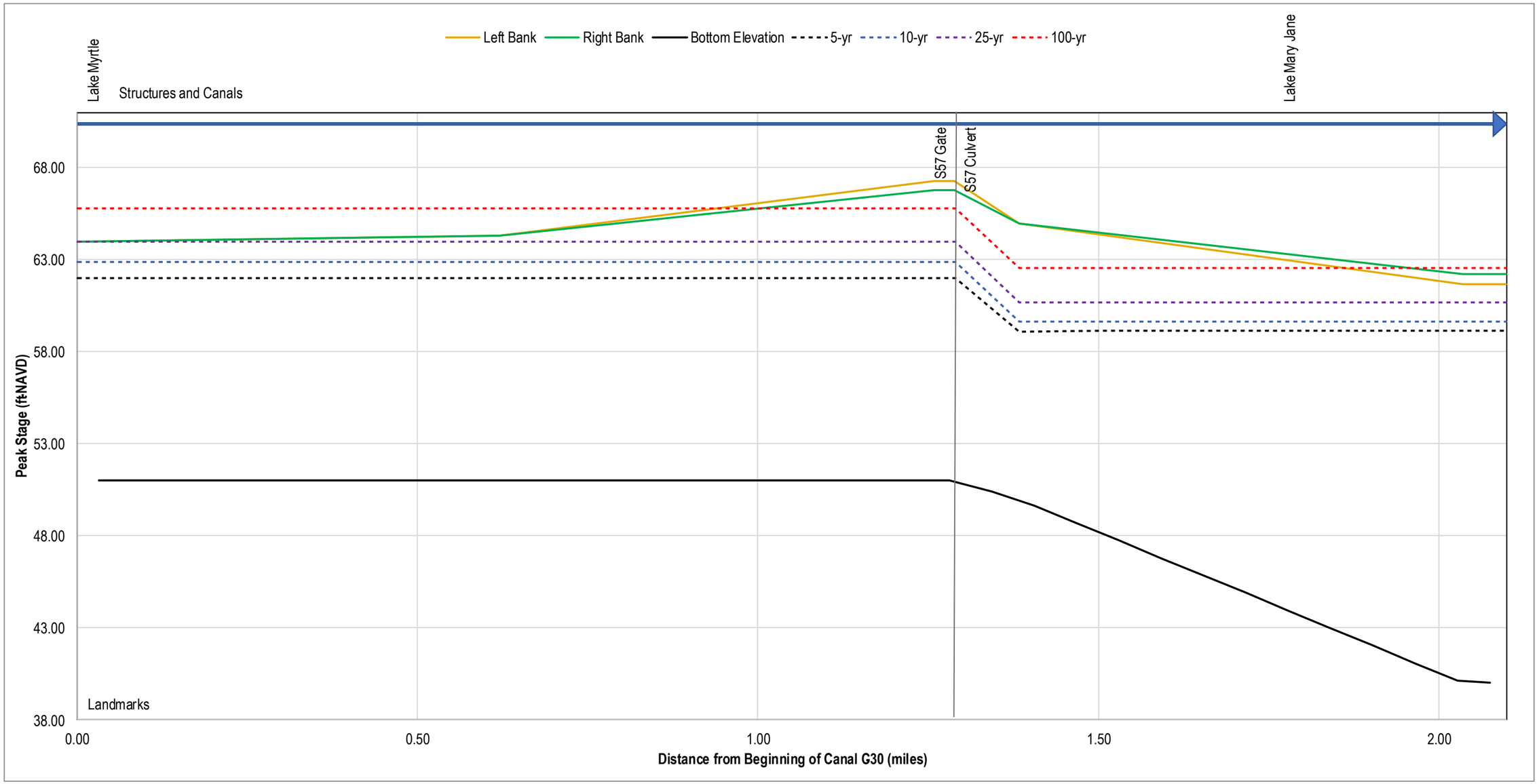


Figure 10-35: Maximum Stage Profile for Canal C-30 under Future Conditions

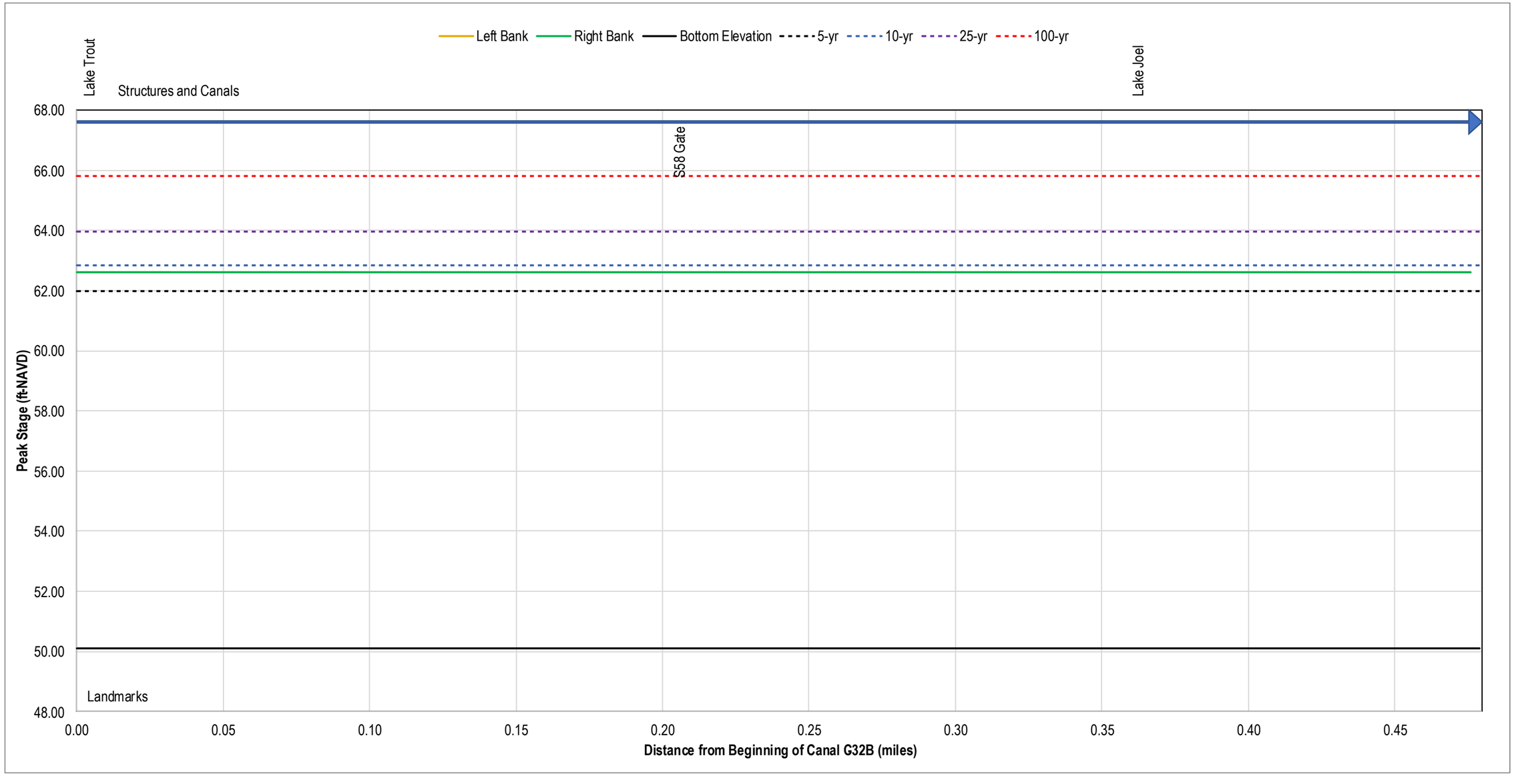


Figure 10-36: Maximum Stage Profile for Canal C-32B under Future Conditions

10.2.2 PM #2

Lake Myrtle LMA's upstream water control structure is S58, and its downstream structure is S57. The maximum daily discharge capacity of this LMA is determined by the discharge capacity of S57. Per the District's ERP Handbook, Lake Myrtle LMA has an allowable discharge rate of 3.6 CSM, based on a design frequency of 10-yrs. **Table 10-20** and **Table 10-21** summarize Lake Myrtle LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area under current and future conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 2 and 1.8 CSM under current and future conditions respectively, both of which are lower than SFWMD's permissible discharge allowance of 3.6 CSM based on a 10-yr design frequency.

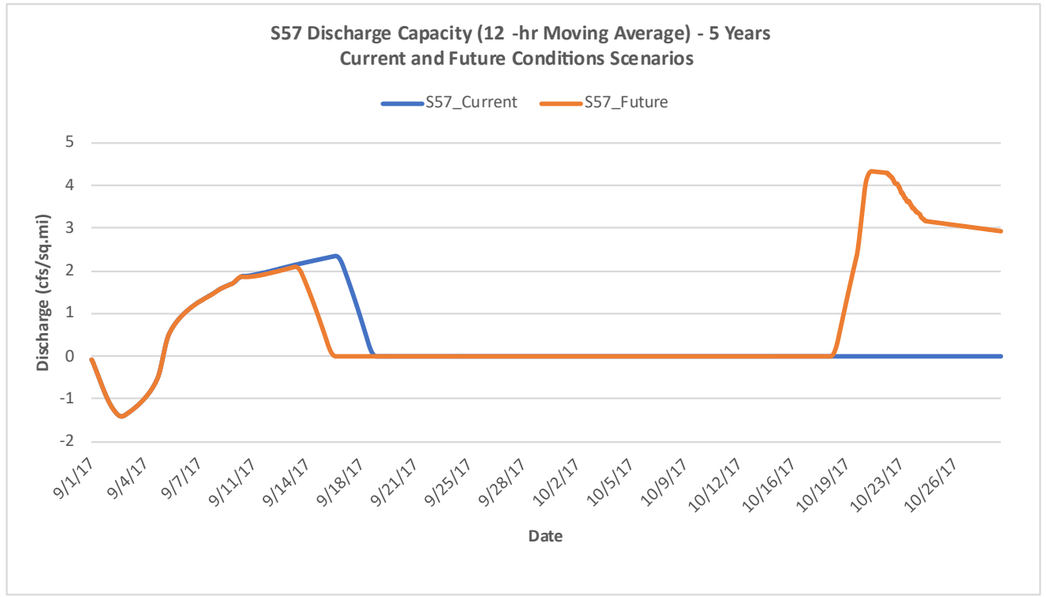
Table 10-20: Drainage Discharge Capacity (cfs/sq. mi) of Lake Myrtle LMA under Current Conditions

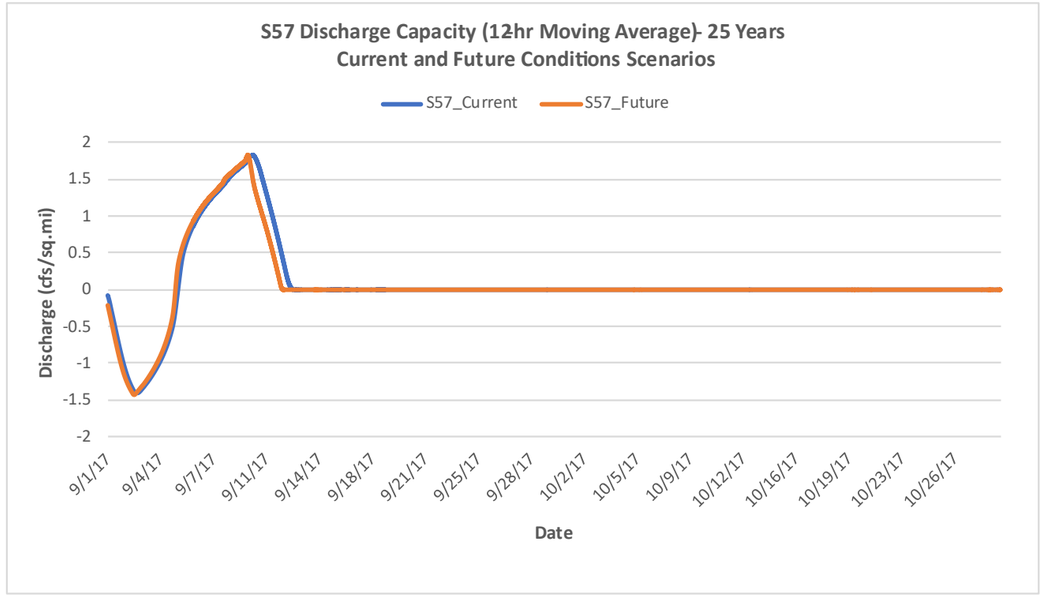
LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Myrtle	69.9	2.3	2.0	1.8	1.8	3.6

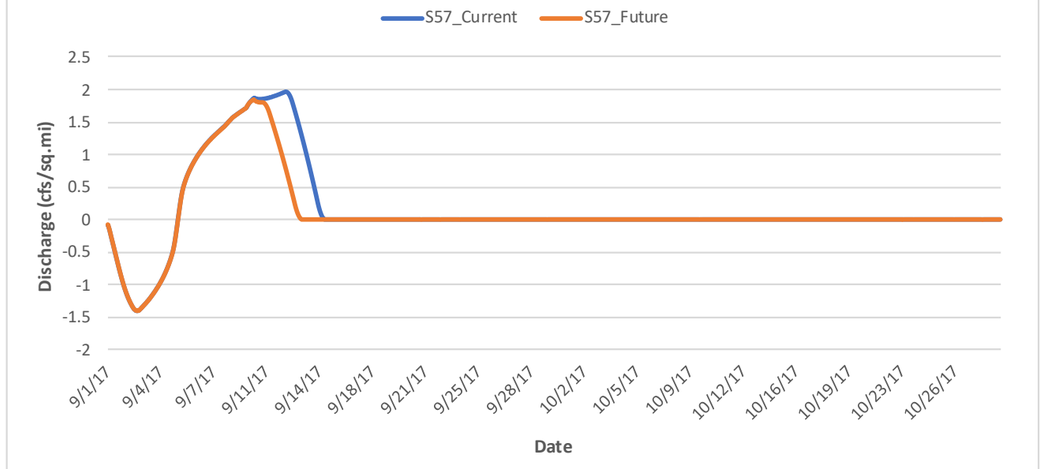
Table 10-21: Drainage Discharge Capacity (cfs/sq. mi) of Lake Myrtle LMA under Future Conditions

LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Myrtle	69.9	4.3	1.8	1.8	1.8	3.6

Figure 10-37 shows the peak 12-hour average discharge per square mile of drainage area, calculated for each design storm at structure S57. **Figure 10-38** shows the maximum daily discharge through structure S57 under both current and future conditions. The simulated peak 12-hour moving average discharge per square mile during the event 25-year and future for current conditions is 1.75 CSM which is much lower than the district allowable discharge of 3.57 CSM for a 10-yr single event for this structure.







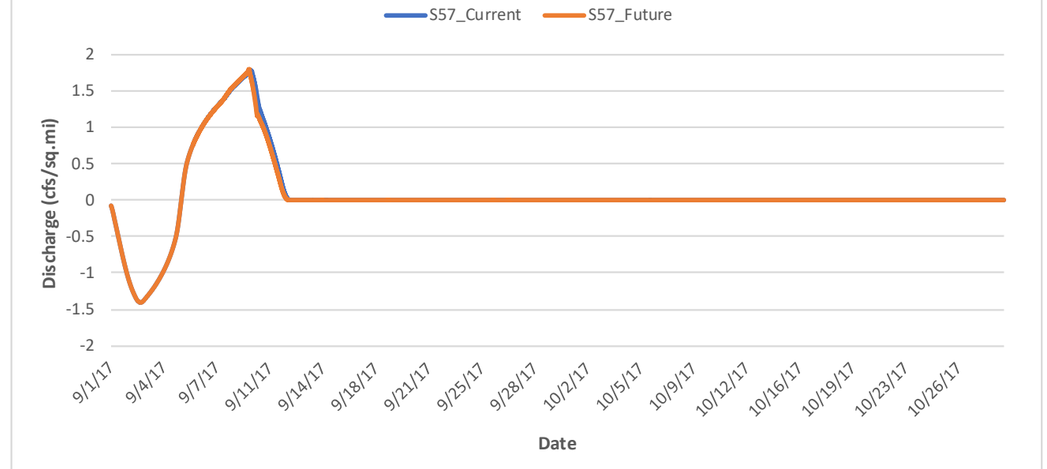


Figure 10-37: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S57 for 5-, 10-, 25-, 100- Year Design Storm

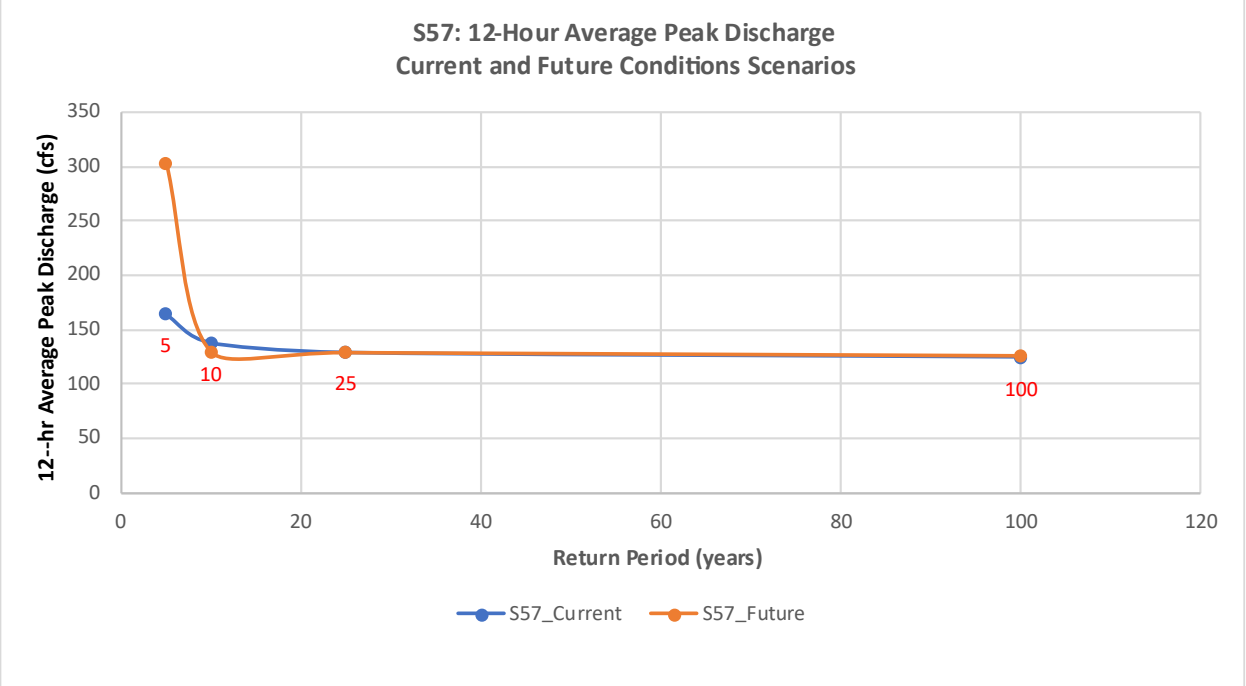


Figure 10-38: Structure S57 12-Hour Average Peak Discharge vs Return Period under Current and Future Scenarios for 5-, 10-, 25-, 100- yr Design Storms

10.2.3 PM #3

The SFWMD structure design parameters, shown in **Table 10-22** were used to evaluate the S57 performance against the various design storms: summarize simulated instantaneous stages and flows at structure S57 under current conditions and future conditions at the time of peak tailwater, peak headwater and peak discharge. At the time of peak tailwater at structure S57, the simulated headwater elevation at S57 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is less than the SPF headwater elevation under both current and future conditions. The simulated flow at structure S57 is higher than the design discharge (170 cfs) for all design storm events in the current conditions and future conditions.

During the peak headwater at the structure S57, the simulated tailwater elevation at S57 is lower than design tailwater elevation during only 5-yr and 10-yr design storm events for current conditions and future conditions. The simulated flows at structure S57 are higher than the design discharge (170 cfs) for all the design storms.

During the peak flow at structure S57, the headwater elevations are lower than the design headwater for only 5-yr design storm event for current and future conditions. However, the tailwater elevation is consistently lower for all design storms for both conditions. **Figure 10-39 and Figure 10-40** show the same as time series.

Table 10-22: Design Parameters for Structure S57

Structure	S57
Design Discharge	170 cfs (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	61.73 feet NAVD88 (62.8 feet NGVD29)
Design Tailwater Elevation	60.53 feet NAVD88 (61.6 feet NGVD29)
SPF Headwater Elevation	65.73 feet NAVD88 (66.8 feet NGVD29)
SPF Tailwater Elevation	65.63 feet NAVD88 (66.7 feet NGVD29)
Optimum Headwater	58.93 to 60.93 feet NAVD88 (60.0 to 62.0 feet NGVD29)
Optimum Tailwater	57.93 to 59.93 feet NAVD88 (59.0 to 61.0 feet NGVD29)

Table 10-23: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-yr	58.70	60.60	68.86	1.9	9/12/17 10:55 AM
10-yr	59.19	61.13	68.47	1.94	9/13/17 3:43 AM
25-yr	59.91	61.97	0.00	2.06	9/13/17 11:43 AM
100-yr	61.39	61.64	31.04	0.25	9/10/17 5:50 PM

Table 10-24: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	62.00	56.77	0.00	5.23	10/2/17 3:00 AM
10-year	62.86	57.19	0.00	5.67	10/2/17 8:00 PM
25-year	63.97	58.03	0.00	5.94	10/2/17 11:00 PM
100-year	65.81	58.58	0.00	7.23	10/11/17 5:00 AM

Table 10-25: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at Peak Q (ft)	Date & Time
5-year	82.83	61.18	58.43	2.75	9/17/17 4:00 AM
10-year	69.44	61.12	59.19	1.93	9/13/17 2:25 AM
25-year	68.77	60.48	58.63	1.85	9/10/17 12:50 PM
100-year	71.44	60.98	58.98	2.00	9/10/17 12:10 PM

Table 10-26: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head at Peak TW Differential (ft)	Date & Time
5-year	59.01	60.90	68.70	1.89	9/12/17 10:00 PM
10-year	59.63	61.57	18.55	1.94	9/13/17 4:43 AM
25-year	60.66	62.64	0.00	1.99	9/13/17 6:43 PM
100-year	62.54	64.27	0.00	1.74	9/14/17 6:43 PM

Table 10-27: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW Differential (ft)	Date & Time
5-year	62.38	55.89	0	6.49	10/10/17 1:00 PM
10-year	62.95	56.21	0	6.74	10/9/17 1:00 AM
25-year	63.75	56.78	0	6.97	10/8/17 5:00 PM
100-year	64.99	57.10	0	7.89	10/14/17 6:00 PM

Table 10-28: Summary of the Instantaneous Conditions at Structure S57 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at Peak Q Differential (ft)**	**Date & Time**
5-year	151.84	61.30	55.75	5.55	10/21/17 5:00 AM
10-year	67.87	60.47	58.61	1.86	9/10/17 1:30 PM
25-year	68.84	60.59	58.80	1.79	9/10/17 12:20 PM
100-year	81.38	62.09	59.49	2.60	9/10/17 12:10 PM

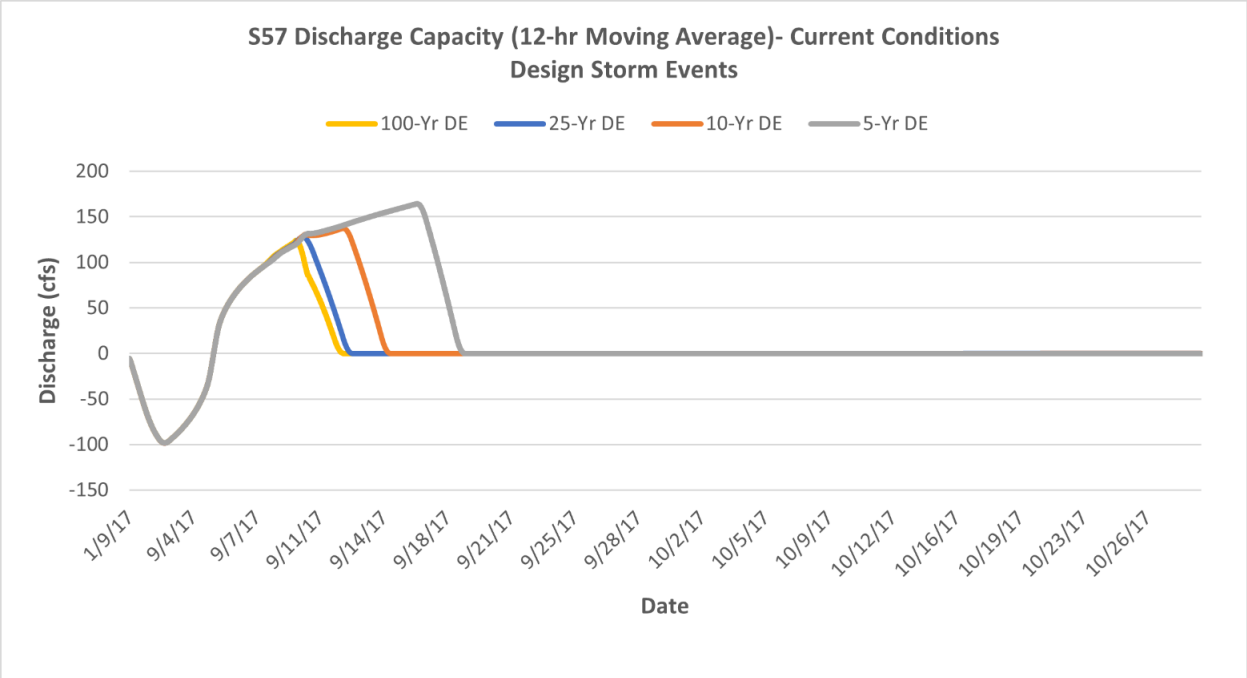


Figure 10-39: 12-hr Moving Average Flows in cfs at S57 under Current Conditions

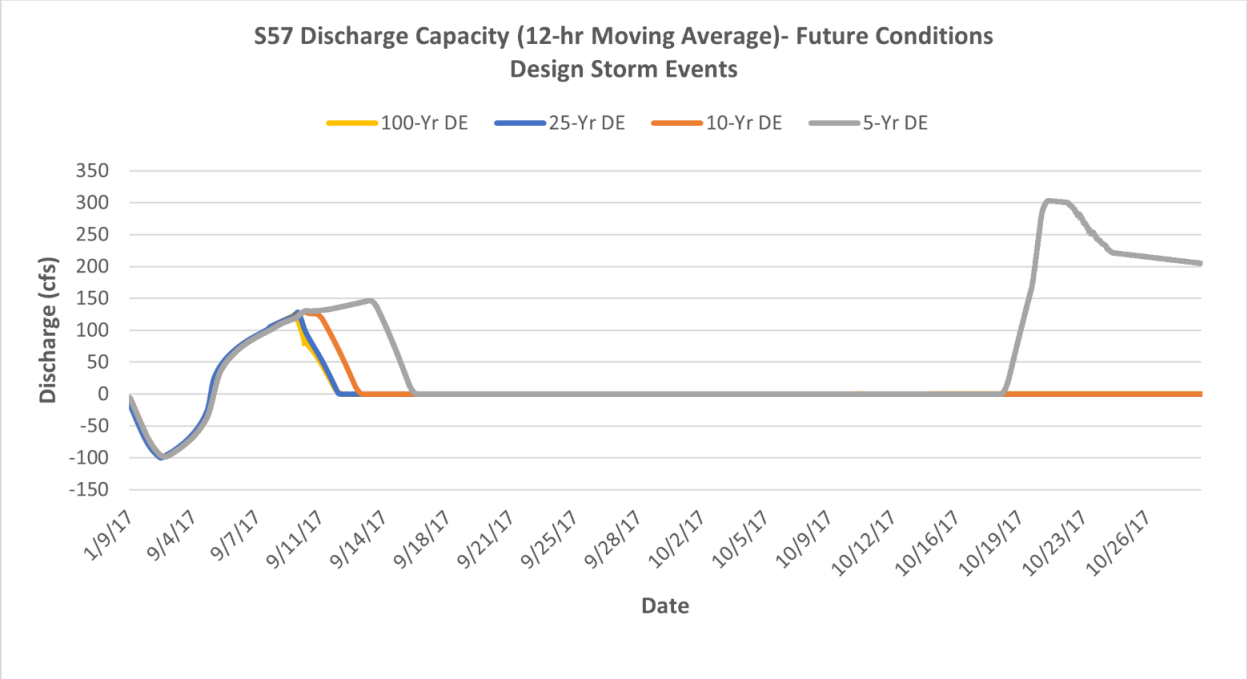


Figure 10-40: 12-hr Moving Average Flows in cfs at S57 under Future Conditions

10.2.4 PM #4

PM#4-UKB was not assessed in this LMA.

10.2.5 PM #5

The maximum overland depth was extracted for each design storm event and evaluated for Lake Myrtle LMA under current and future conditions. **Table 10-29** is a tabulation of the flood inundation area in square miles for this LMA, shown in incremental intervals of feet. **Figure 10-41** demonstrates this data in a plot format. **Figure 10-42 through Figure 10-45** present the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions for Lake Myrtle LMA.

The table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding and 0.25 to 0.75 feet of flooding. The higher depths of flooding are the large lakes, which is expected. The areas of flooding at depths less than 0.5 feet are still significant. The plot also shows how the areas of inundation are large even for the mid-ranges. Examination of this LMA shows that between 40 to 70% of the LMA are inundated with depths greater than 0.5 ft for all storms under current and future conditions.

The groundwater levels were also examined in this LMA. **Figure 10-46** shows the Floridan wells in this LMA: OSF-110 is in the Upper Floridan Aquifer, OSF-22 in the Middle Floridan Aquifer and OSF-111 is in the Lower Floridan Aquifer. All 3 have water levels (potentiometric surfaces) in the 25- and 100-yr

storms at or higher than the bottom of the canals and lakes, throughout the simulation after the occurrence of the storm. For the smaller storms, the groundwater levels are higher than the DEM and canal thalwegs at some locations. The inundation is primarily caused by rainfall and retention of water on the wetlands, then exasperated by higher groundwater.

Table 10-29: Incremental Flood Inundation Area (sq. mi.) in Lake Myrtle LMA

Flooding Depth (ft)	De-Curren 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	31.88	27.40	19.57	7.44	32.03	27.54	19.66	7.47
0.25<=Depth<0.5	8.82	8.14	10.41	14.83	8.92	8.25	10.50	14.86
0.5<=Depth<0.75	9.15	8.31	5.79	4.74	9.12	8.37	5.82	4.76
0.75<=Depth<1.0	6.40	7.75	7.16	4.33	6.46	7.78	7.23	4.31
1.0<=Depth<1.25	3.38	4.97	7.06	4.32	3.74	4.97	7.12	4.43
1.25<=Depth<1.5	1.91	3.05	5.00	6.31	1.85	3.03	5.03	6.27
1.5<=Depth<1.75	1.57	2.21	3.15	6.29	1.27	2.58	3.14	6.24
1.75<=Depth<2.0	0.92	1.25	2.72	4.97	0.93	1.26	2.72	5.03
2.0<=Depth<2.25	0.67	0.66	1.78	3.24	0.90	0.62	1.74	3.20
2.25<=Depth<2.5	0.40	0.87	0.75	2.48	0.32	0.54	0.67	2.50
>2.5	4.81	5.32	6.54	10.97	4.37	4.98	6.29	10.86
Lake Myrtle LMA	**44750.72**	**acres**						
	69.92	**sq.mi.**						

***Note**: The lake area of 22.90 sq. mi. was included in this table computations.*

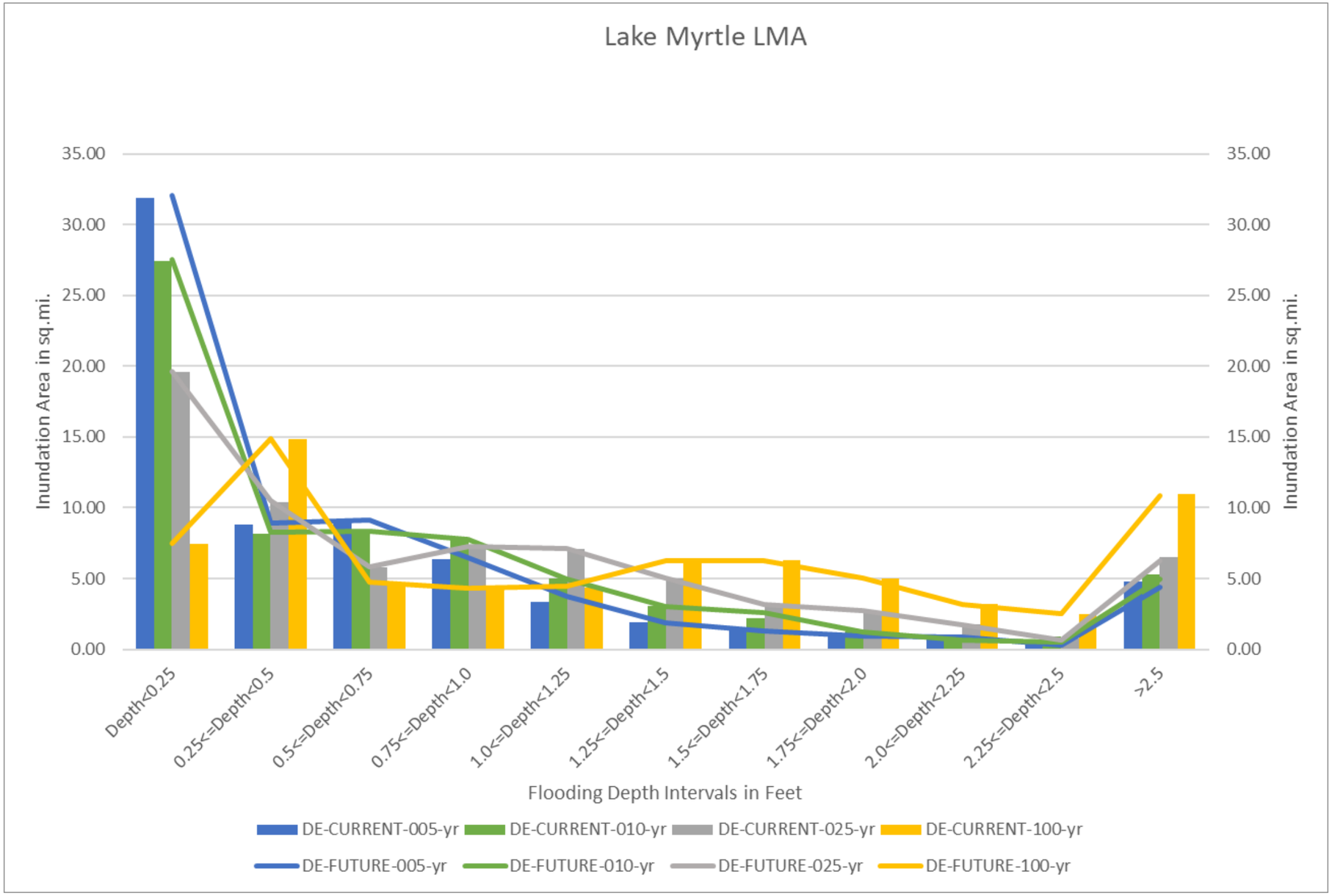


Figure 10-41: Incremental Flood Inundation Depth in feet in Lake Myrtle LMA under Current and Future Conditions

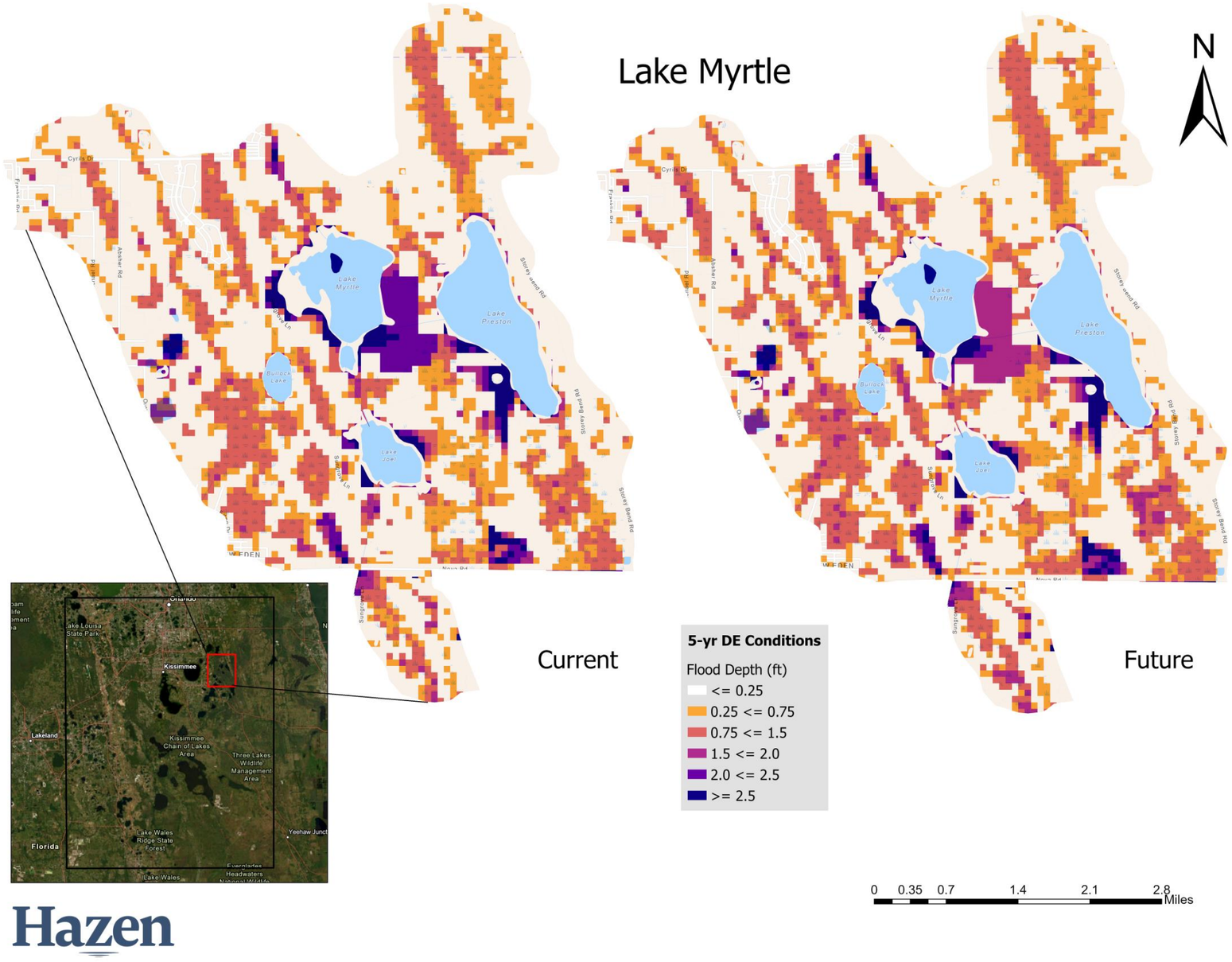


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Figure 10-42: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

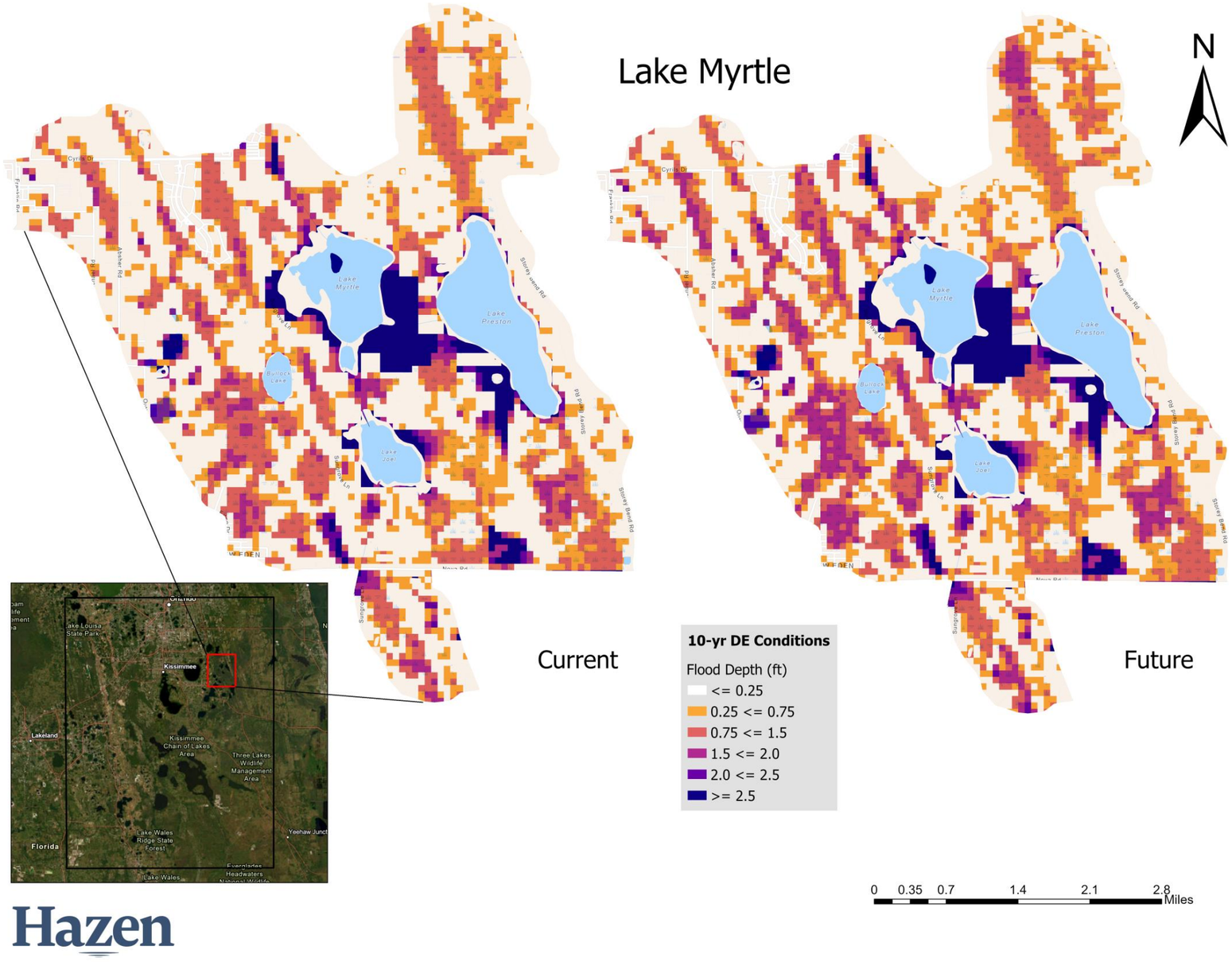

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Figure 10-43: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

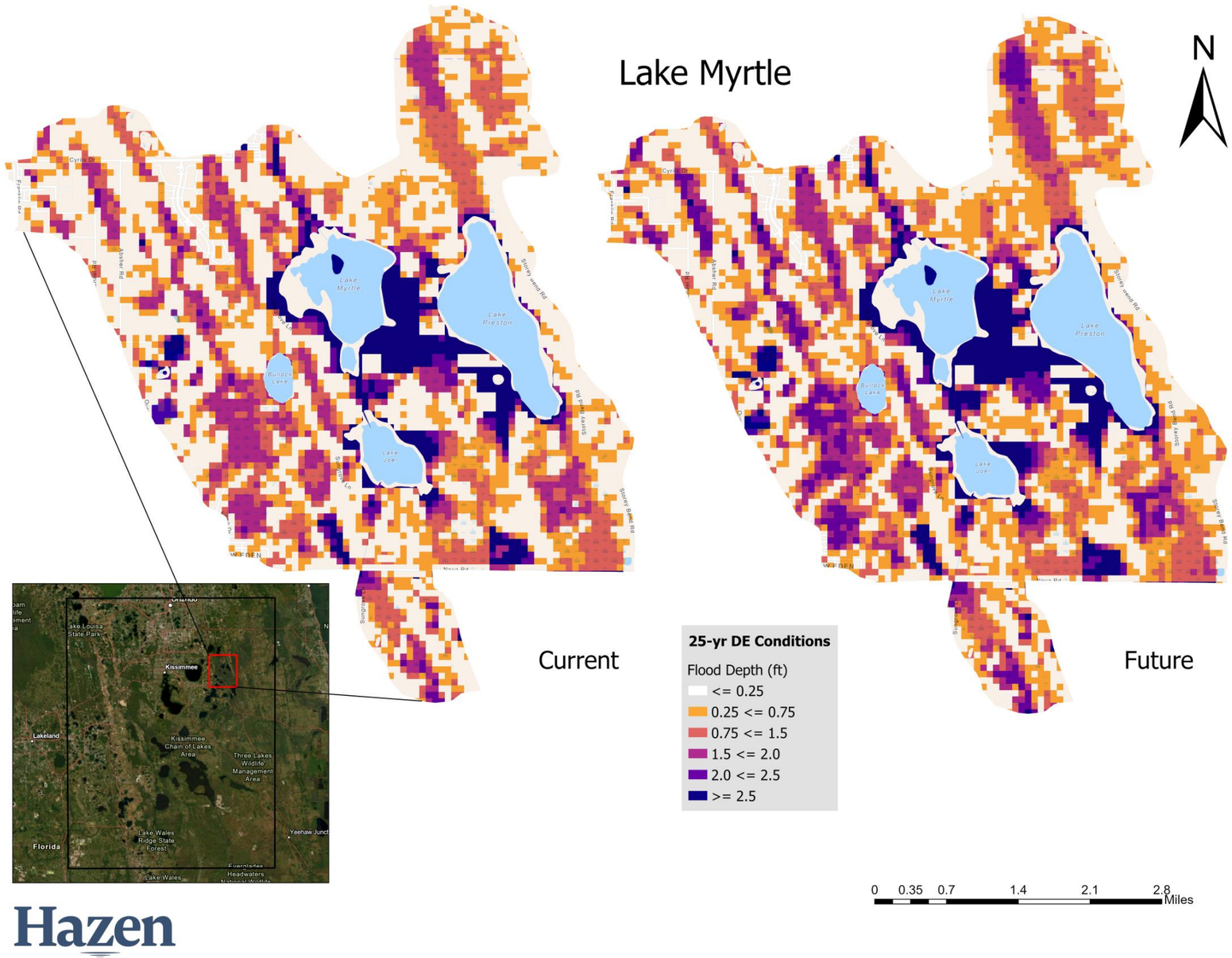

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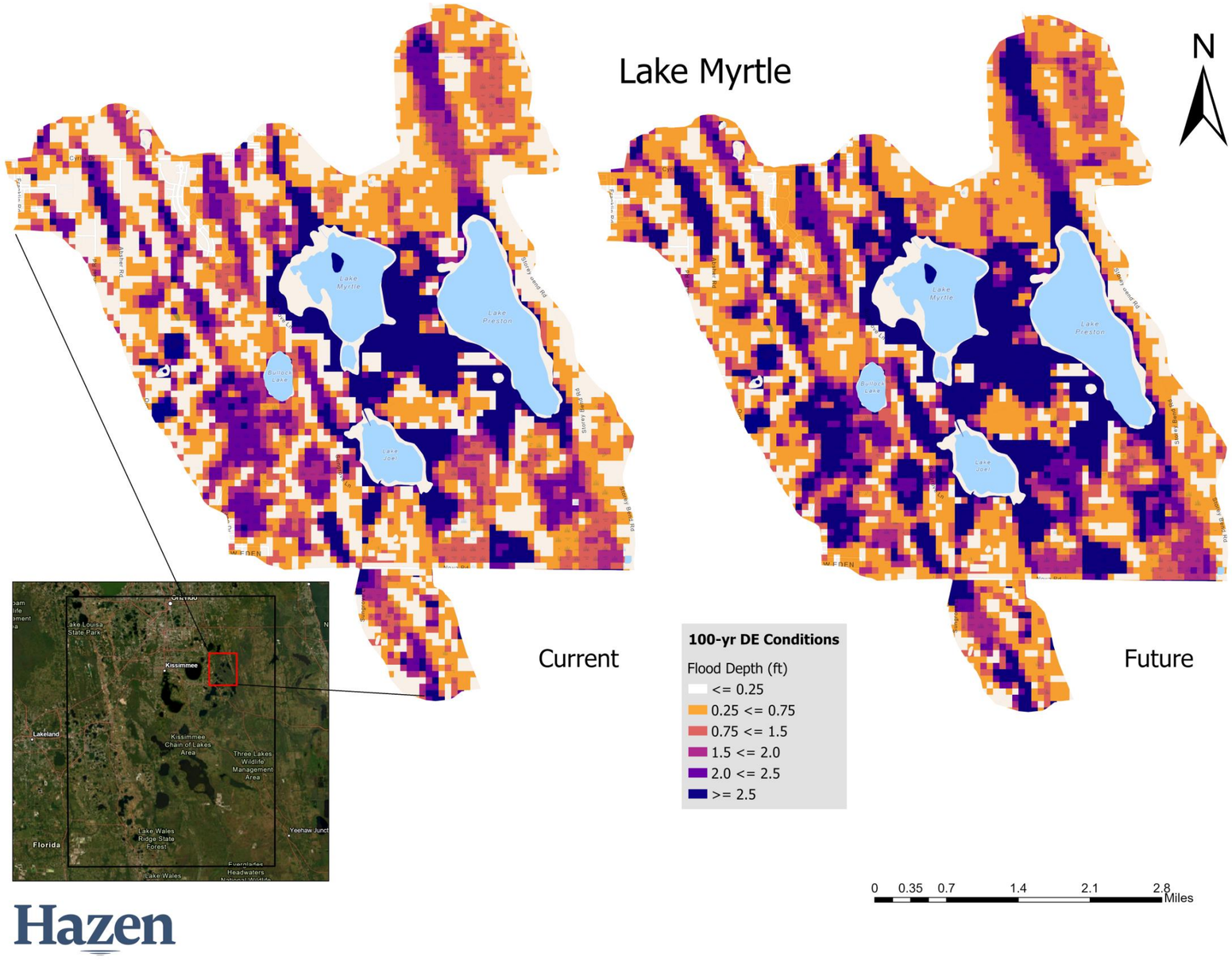
Figure 10-44: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA



Hazen

Figure 10-45: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

Figure 10-46 Floridan Wells in Lake Myrtle LMA

10.2.6 PM #6

Table 10-30 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Myrtle LMA.

Figure 10-47 provides a graphical representation of this data, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Myrtle LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-48 through Figure 10-51** show the flood duration in Lake Myrtle LMA under current and future conditions. None of the lakes were masked in this LMA.

The table and the plot show that the greatest distribution of flooding duration in the LMA is between zero and one hour of flooding, which can be considered insignificant. Flooding duration at the higher end is on the large lakes, which is also expected.

Table 10-30: Areas of Incremental Flood Duration (sq. mi.) in Lake Myrtle LMA

Flooding Duration (Hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	28.65	26.24	22.54	15.97	28.65	26.25	22.58	15.97
1<=Hours<4	0.80	1.19	2.14	4.76	0.79	1.19	2.14	4.75
4<=Hours<8	0.74	0.81	0.70	0.90	0.74	0.82	0.71	0.91
8<=Hours<12	0.47	0.52	0.51	0.53	0.49	0.52	0.51	0.51
12<=Hours<24	0.96	1.02	1.40	1.29	0.97	1.02	1.41	1.30
24<=Hours<48	0.97	1.12	1.27	1.39	1.02	1.13	1.31	1.34
48<=Hours<96	1.16	1.55	1.38	1.63	1.13	1.54	1.35	1.57
96<=Hours<168	1.02	1.14	1.16	1.43	1.02	1.14	1.18	1.52
168<=Hours<240	0.73	0.76	0.92	0.89	0.72	0.78	0.97	0.95
240<=Hours<360	0.88	1.08	1.21	1.32	0.90	1.08	1.40	1.51
360<=Hours<1417	11.00	11.94	14.15	17.27	10.95	11.91	13.83	17.04
Lake Myrtle LMA	**44750.72**	**acres**						
	69.92	**sq.mi.**						

Note: *The lake area of 22.90 sq. mi. was included in this table computations.*

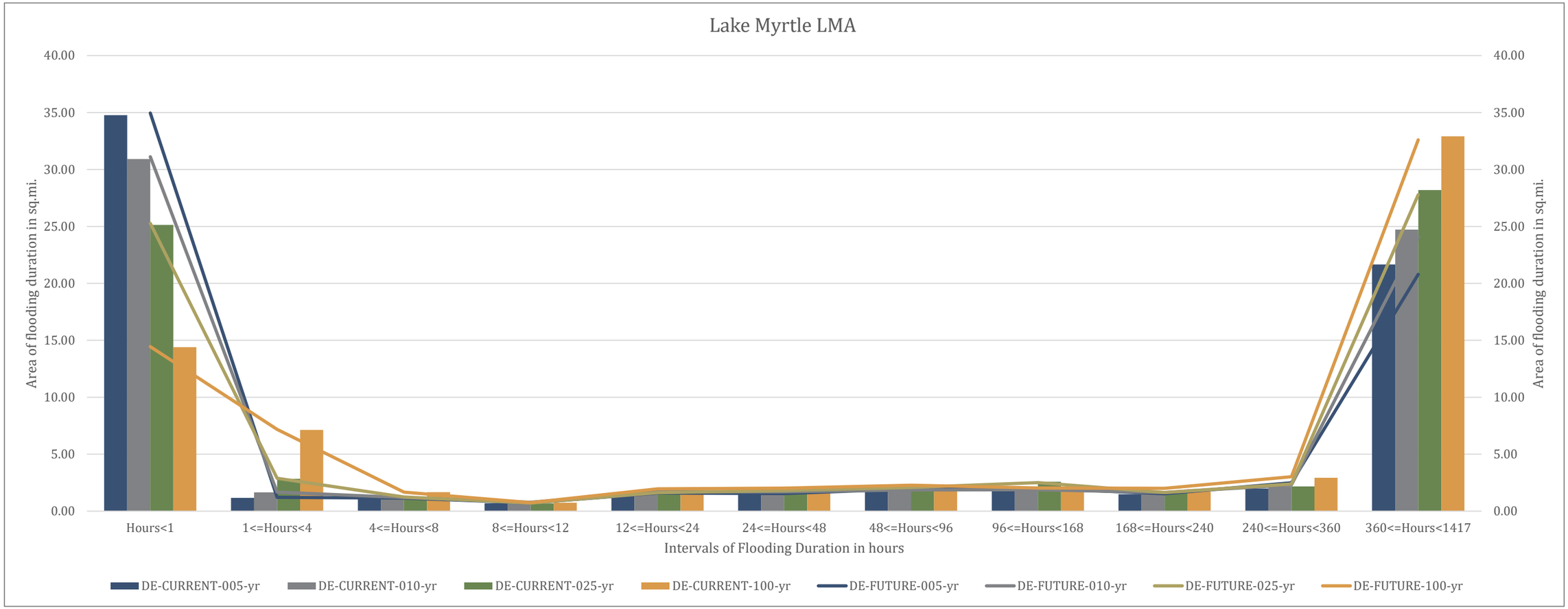


Figure 10-47: Areas of Incremental Flood Duration in hours in Lake Myrtle LMA under Current and Future Conditions

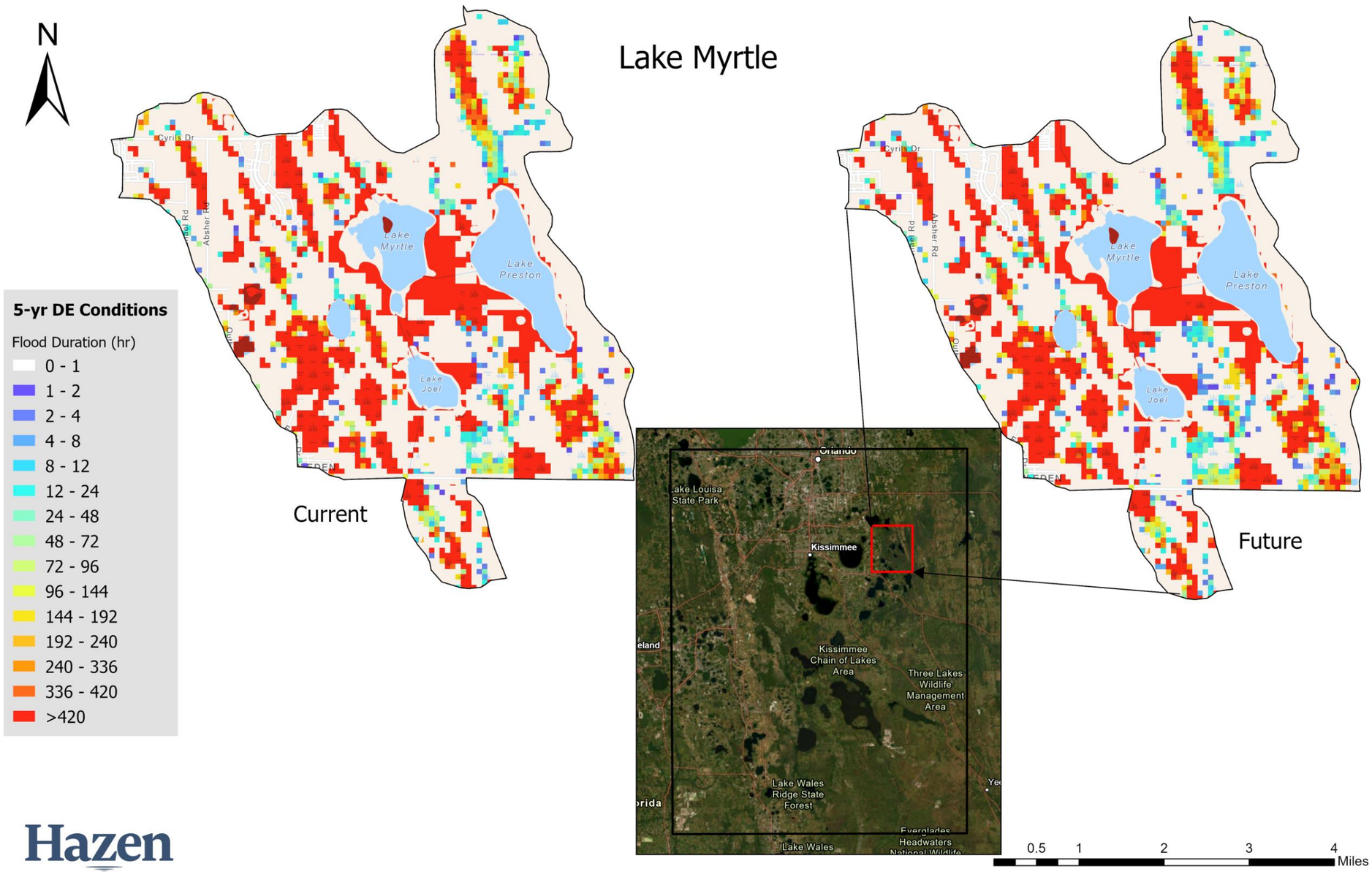


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Figure 10-48: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

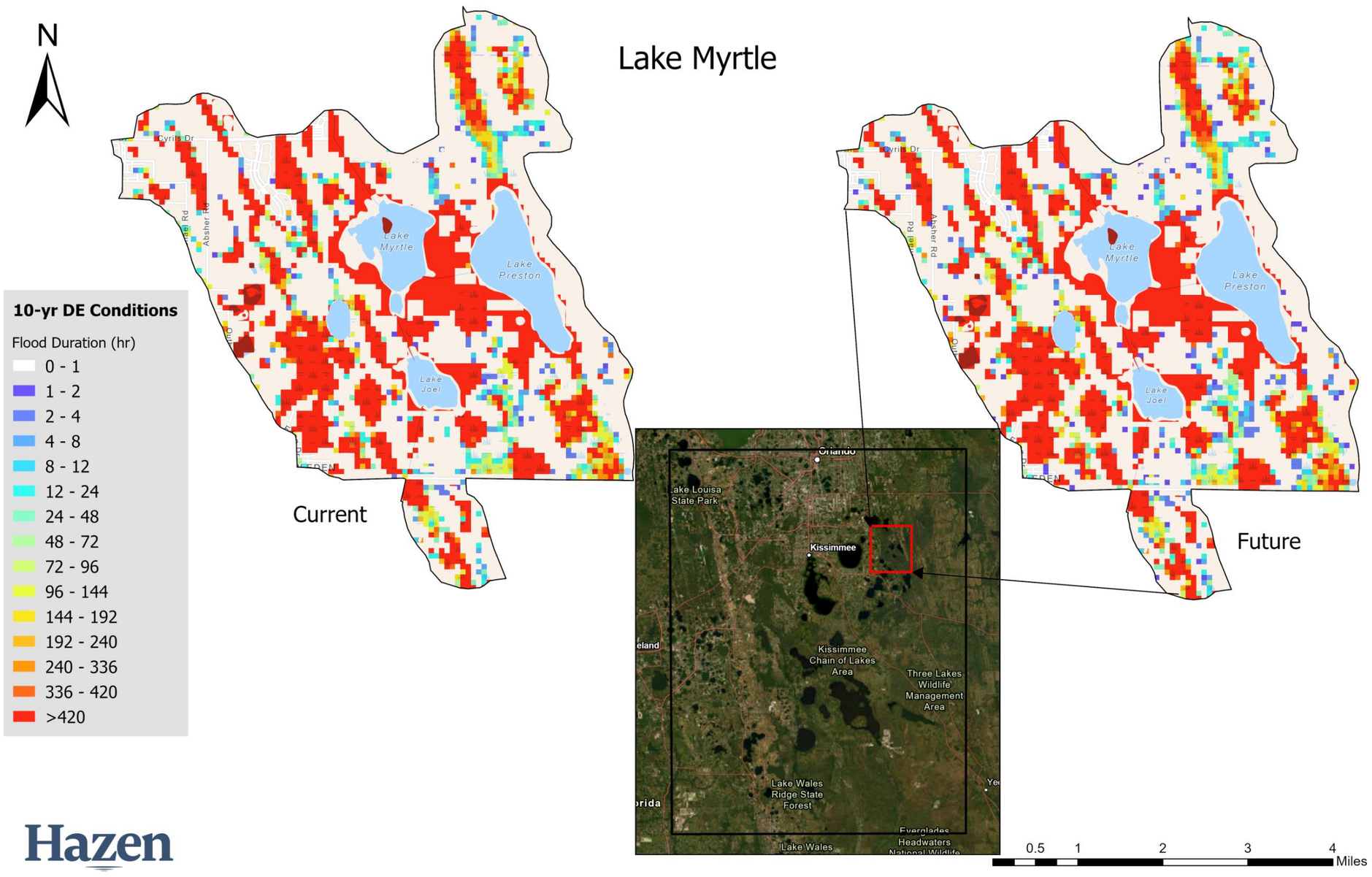

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Figure 10-49: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

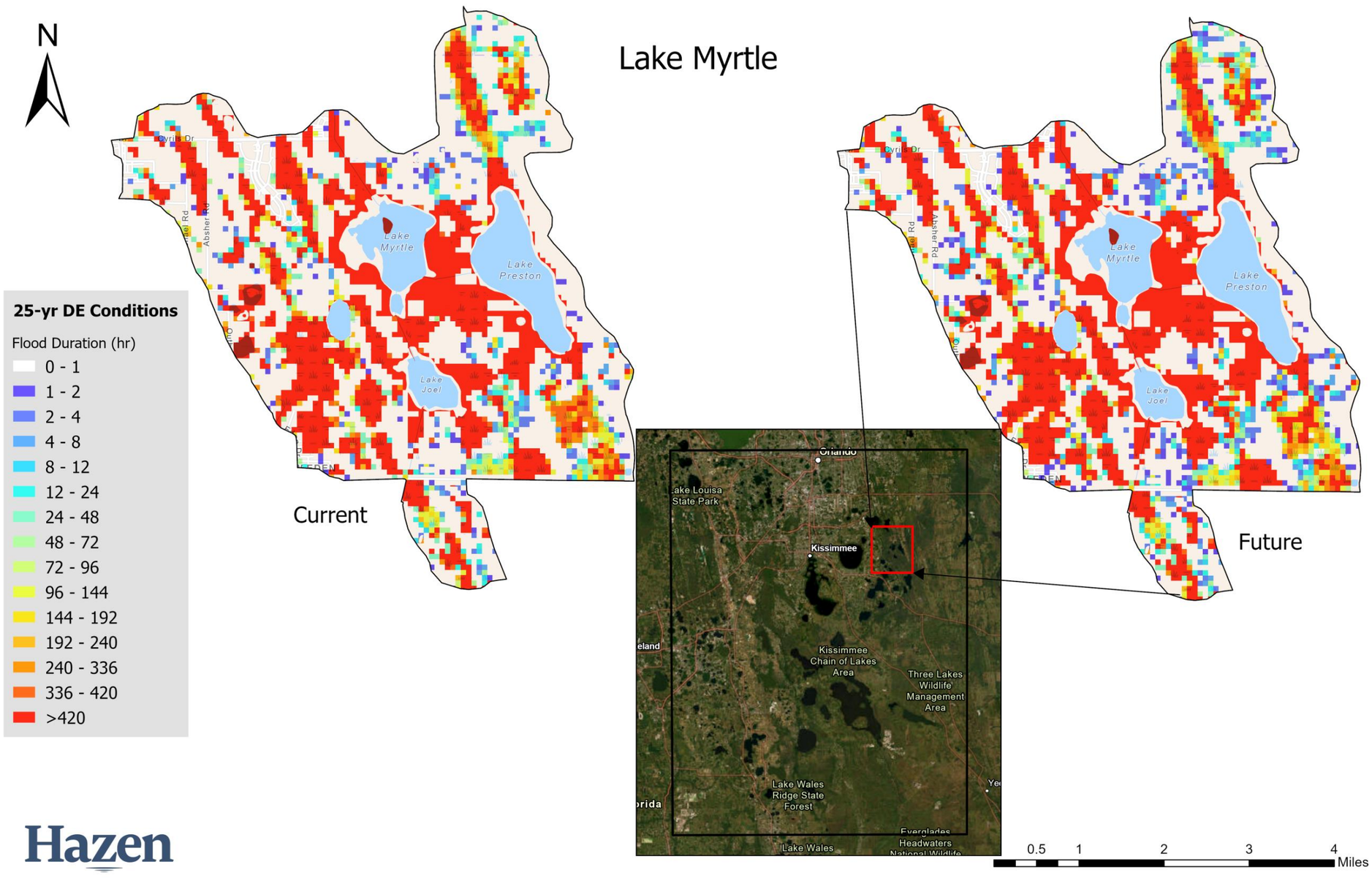

Figure 10-50: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

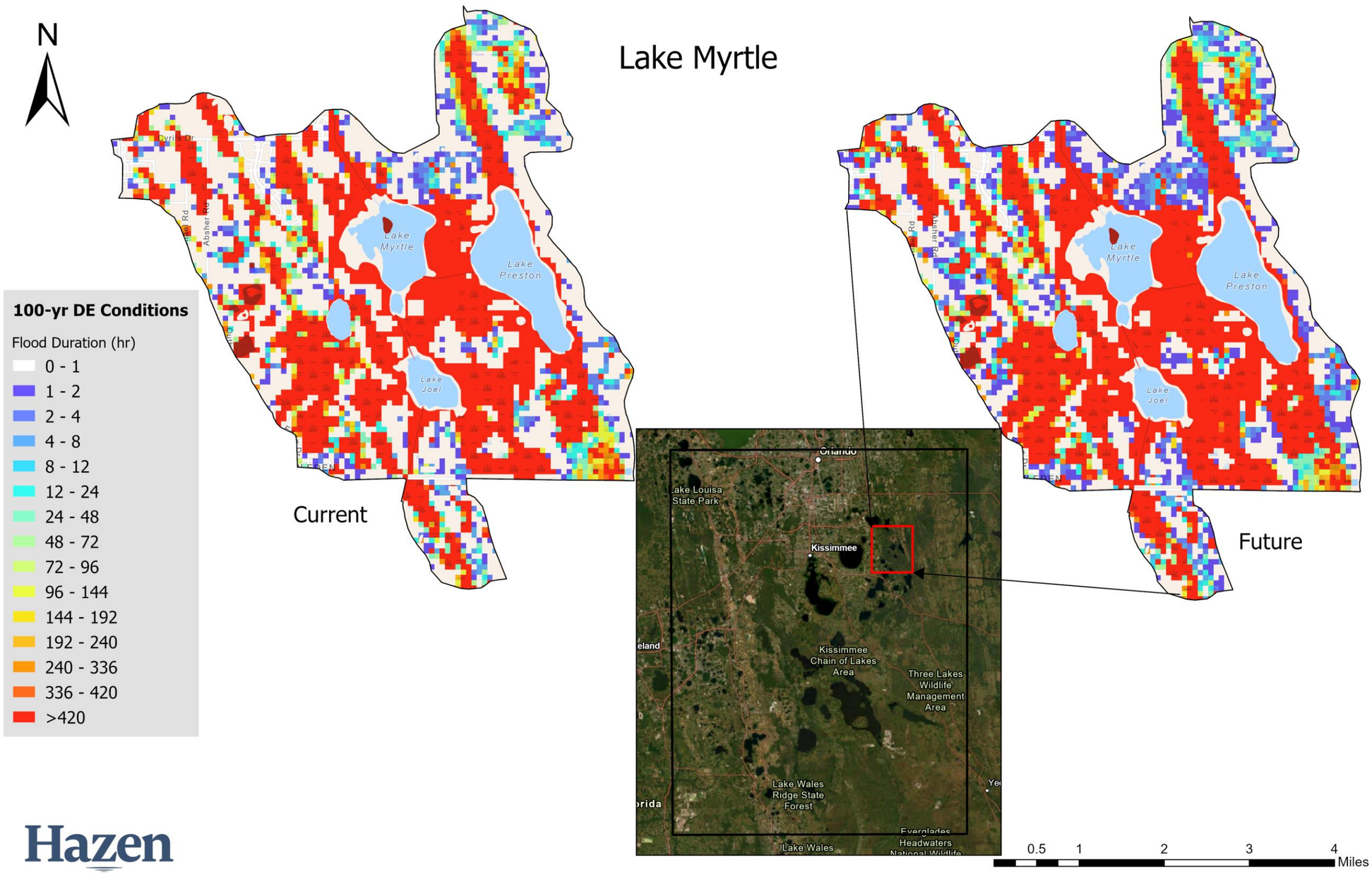

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Figure 10-51: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

10.2.7 FPLOS Rating of Lake Myrtle LMA

Table 10-31 provides a summary of evaluation of Lake Myrtle LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The FPLOS rating for the LMA is on average <5-yr to 5-yr for most PMs; however, the rating for PM#1 ranges from 5-yr to 25-yr under both current and future conditions.

Table 10-31: Summary of PM Analysis in Lake Myrtle LMA

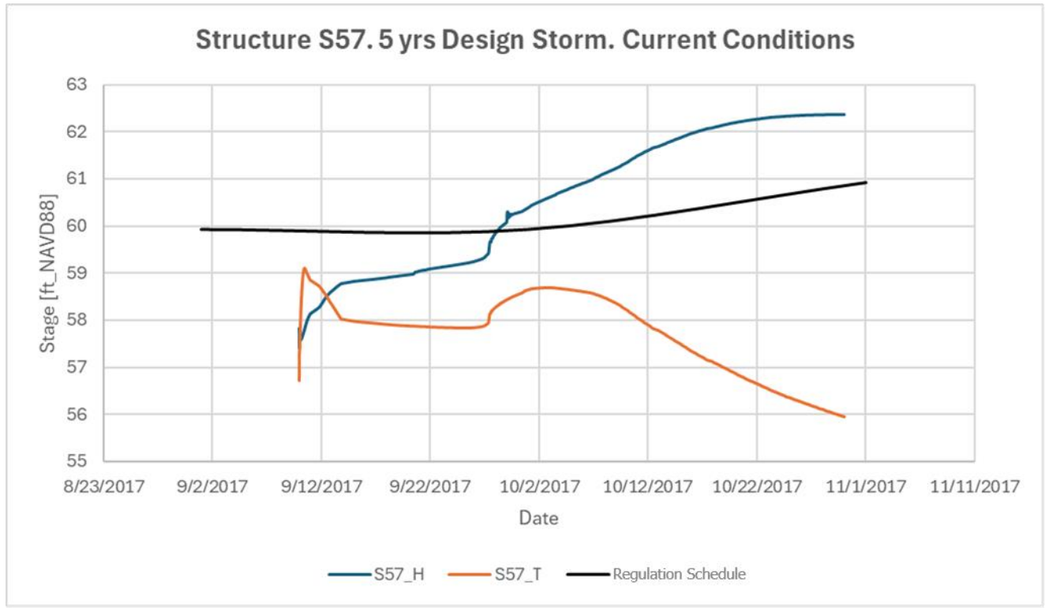
Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Current Conditions
PM#1	Lake Myrtle, Lake Preston, C-32B and C-30 canals	The banks in Lake Myrtle are exceeded for all design storms for both current and future conditions, and canal C-32B are exceeded for 10-, 25- and 100-yr storms and the banks are exceeded in the 100-yr design storm in Lake Preston. Portions of the banks in canal C-30 are exceeded for 100-yr design storm.	<5-yr in Lake Myrtle 25-yr in Lake Preston 5-yr in Canal C-32B 25-yr in Canal C-30	<5-yr in Lake Myrtle 10-yr in Lake Preston 5-yr in Canal C-32B 25-yr in Canal C-30
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 2.0 CSM under current conditions and 1.8 CSM under future conditions. Both are within SFWMD's permissible discharge allowance of 3.6 CSM based on a 10-yr design frequency.	10-yr discharge lower than permissible discharge	10-yr discharge lower than permissible discharge
PM#3	Entire LMA	During both peak tailwater and peak headwater, the simulated flow at structure S57 is higher than the design discharge of 170 cfs for all design storm events in the current and future conditions.	< 5-yr	< 5-yr
PM#4-UKB	-	-	-	-
PM#5	Entire LMA	The greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding and higher than 2.5 feet of flooding. Between 40 to 70% of the LMA are inundated with depths greater than 0.5 ft for all storms under current and future conditions.	< 5-yr	< 5-yr
PM#6	Entire LMA	Approximately 53% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 33% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm under both current and future conditions.	5-yr	5-yr
Lake Myrtle LMA Overall FPLOS		**Current Conditions**		**< 5-yr**
		Future Conditions		**< 5-yr**

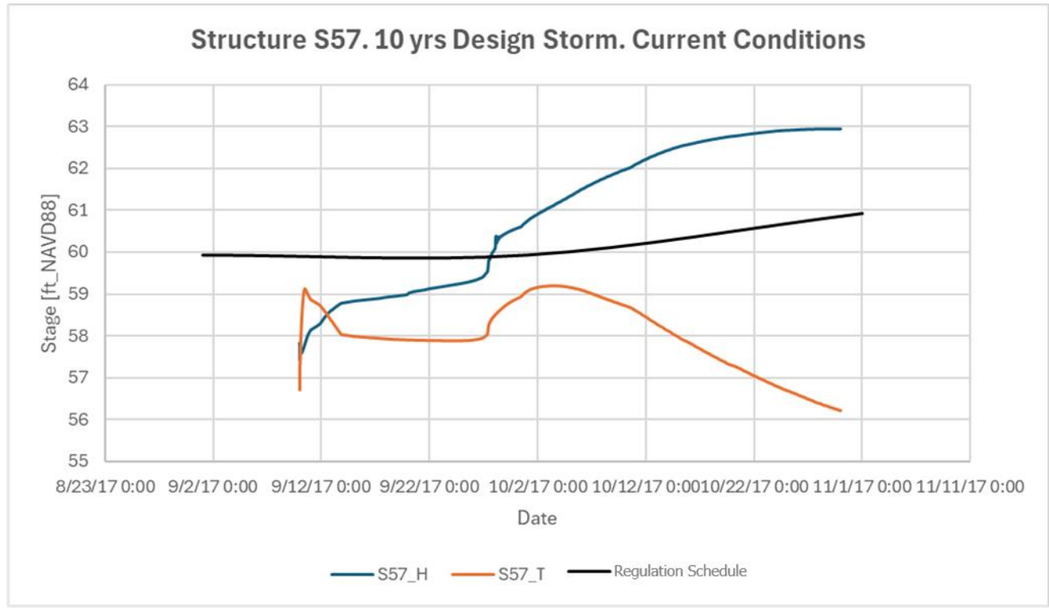
10.2.8 Volume in Lake due to Design Storm UKB-V_{Myrtle}: Lake Myrtle

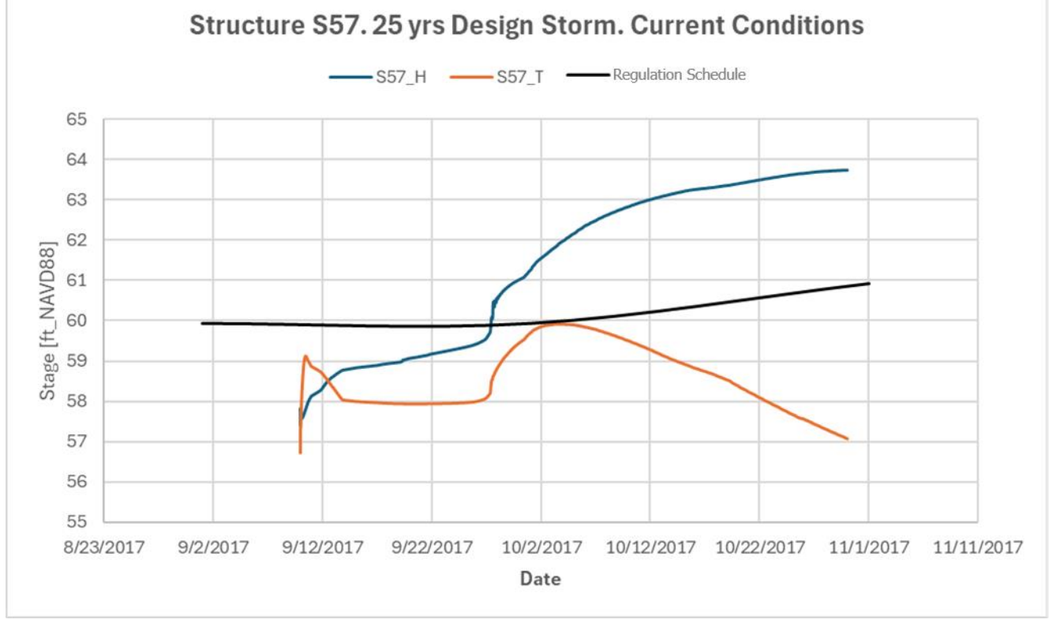
Table 13-3 summarizes the water volume change in Lake Myrtle, measured as the final minus the initial volume during the simulation. The volume is higher for future conditions than current conditions for each design storm, as expected from the higher rainfall intensity of future events. **Table 13-4** shows the volume of water at peak.

10.2.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}: Lake Myrtle

The examination of water levels relative to the regulation schedule in Lake Myrtle focused on its main regulation structure, S57. **Figure 10-52 and Figure 10-53** display the upstream and downstream stages at S57, along with the regulation schedule for the whole simulation period, for current and future conditions respectively. The water levels in Lake Myrtle remain above the regulation schedule line for the entire simulation time and do not return to the schedule line for any of the design storms.







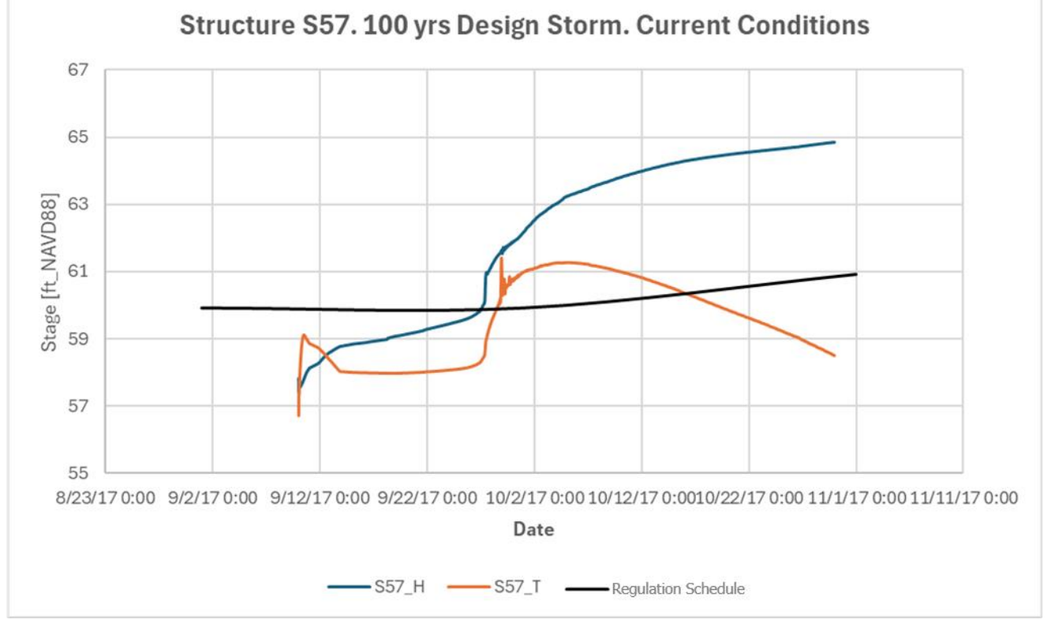
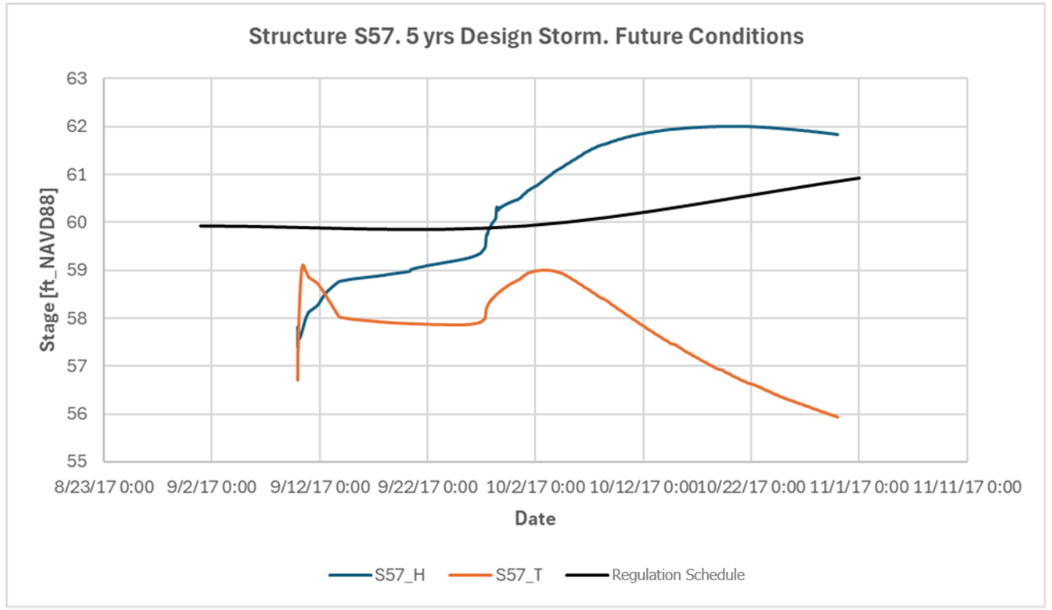
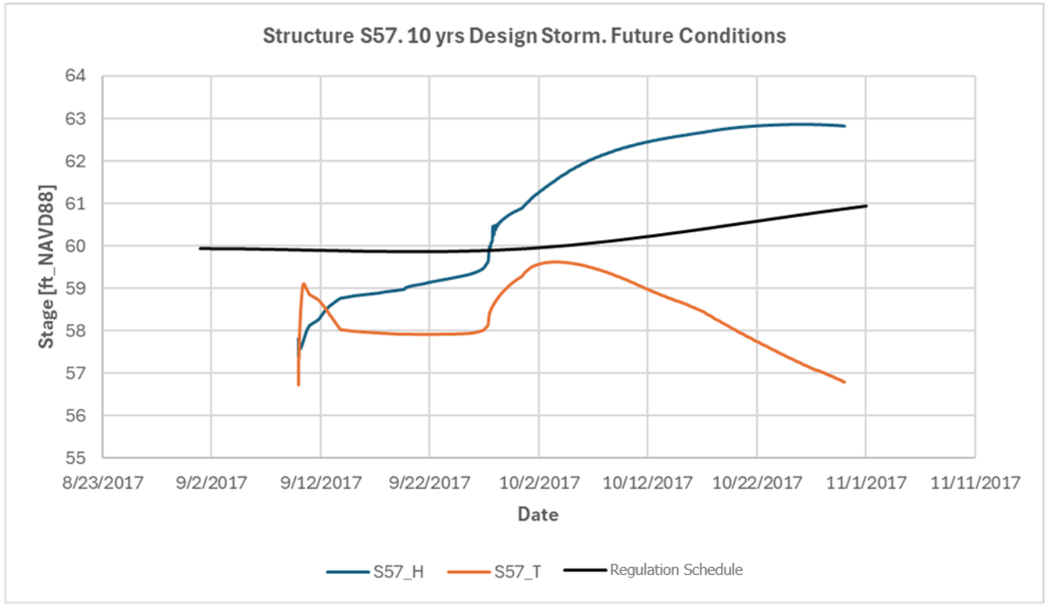
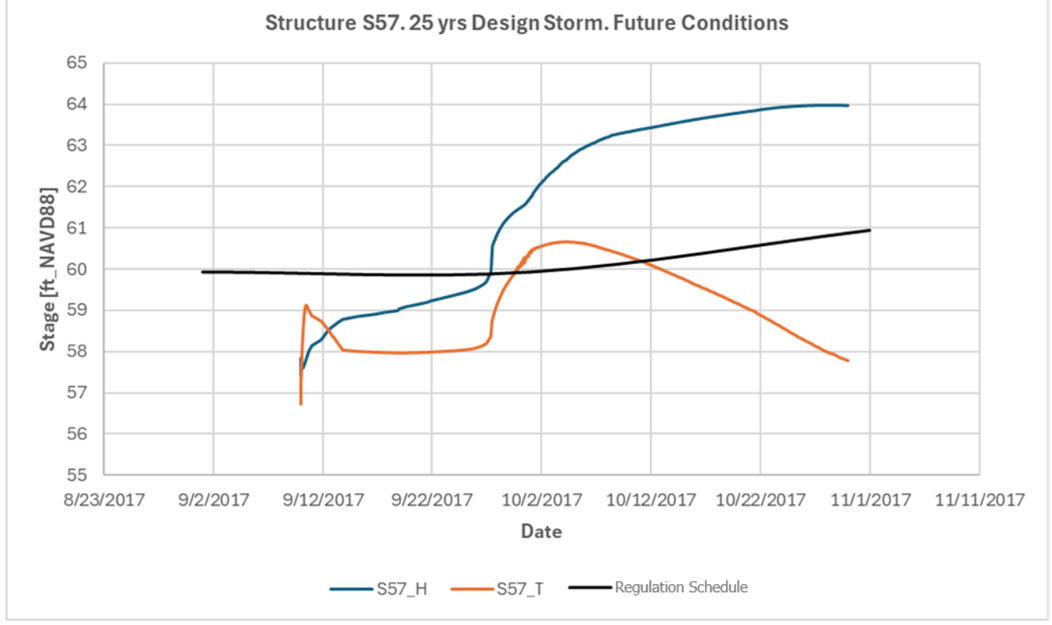


Figure 10-52: Headwater and Tailwater at Structure S57 vs. Lake Myrtle Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







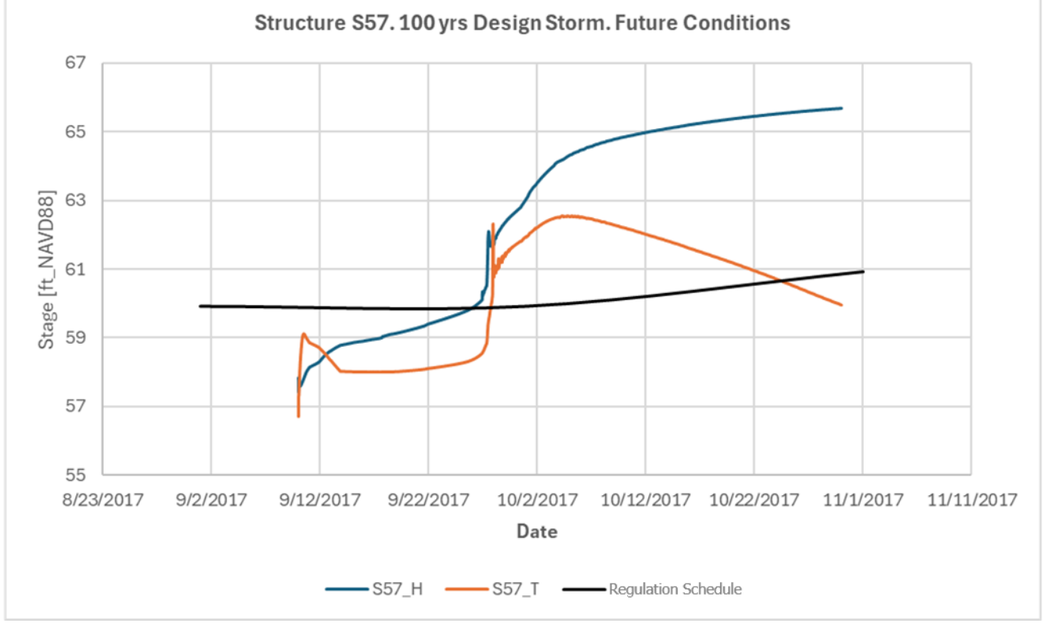


Figure 10-53: Headwater and Tailwater at Structure S57 vs. Lake Myrtle Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Future Conditions

10.3 Lake Hart LMA

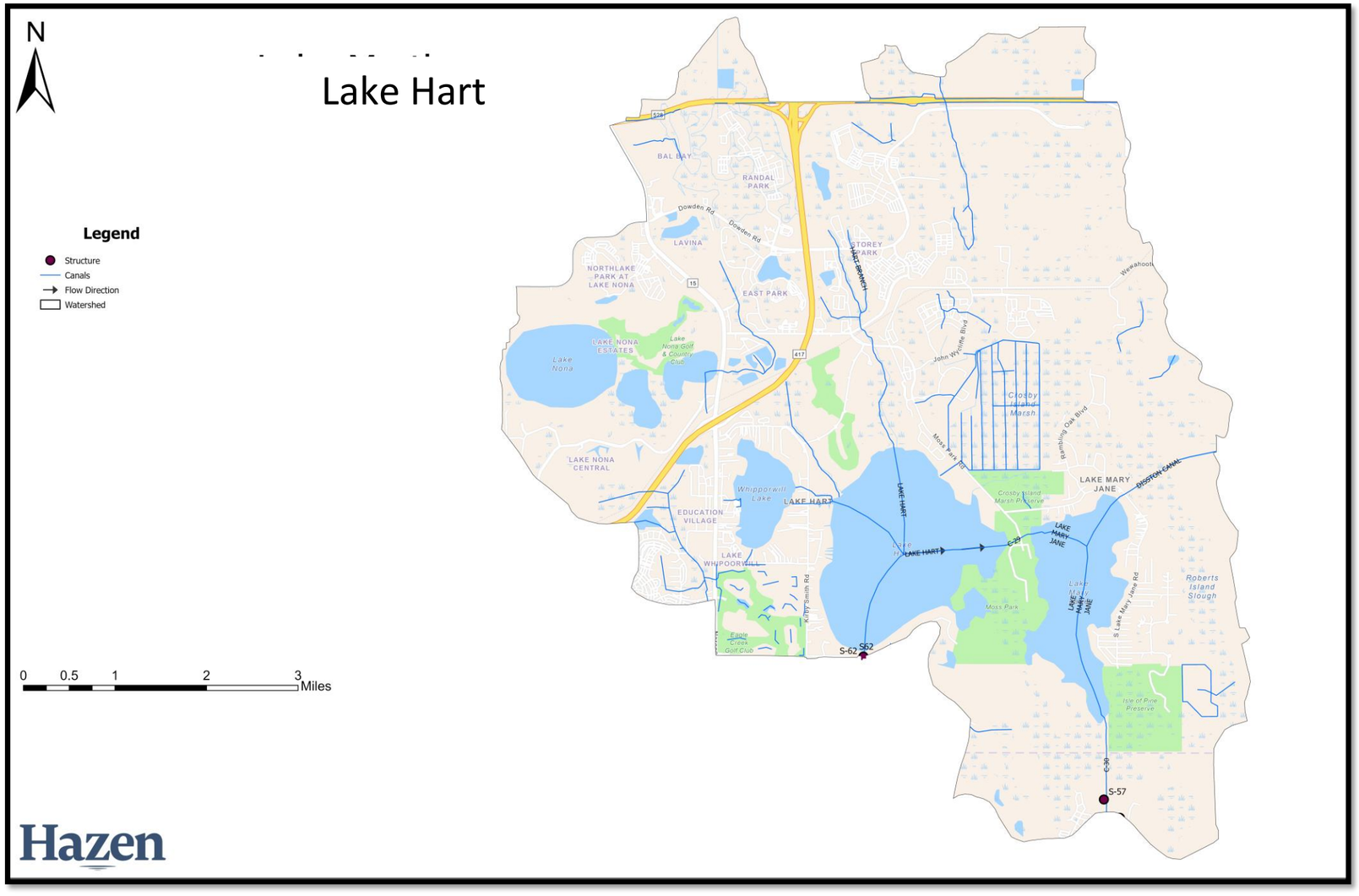


Figure 10-54: Lake Hart LMA with Main Structures, Waterways and Roadways

***Note**: Flow direction used in this instance was to help determine left and right banks.*

10.3.1 PM #1

Performance of the system at Lake Hart LMA was evaluated at Lake Hart and C-29A canal, shown in **Figure 10-55 and Figure 10-56** under current conditions; and **Figure 10-57 and Figure 10-58** under future conditions.

For Lake Hart LMA, under current conditions, the left bank is exceeded by all design events in most areas, while the right bank also sees some exceedances for the 100-year event, although less consistently. For Canal C-29A, the left and right banks are not exceeded at any point for all design events, keeping the water levels contained, except in one location where the right bank is exceeded during the 100-year event.

Under future conditions, the trends remain similar, with both the left and right banks of Lake Hart being exceeded in most areas during the 100-year event. The water levels for the smaller events also continue to exceed the left bank in most places and the right bank in some locations. For Canal C-29A, right bank exceedances during the 100-year event happen twice, with slight increases in water levels compared to current conditions. This comparison shows that future conditions bring higher water levels but maintain the same general pattern of exceedances, particularly during the 100-year event.

At Lake Hart, lake levels exceed the banks almost everywhere for all design events under current and future conditions. Plots show that for Canal C-29A, the water levels stay within banks for current and future conditions upstream of Lake Ajay. Across Lake Ajay, the profiles show that the right bank is overtopped under current and future conditions; however, this is because Lake Ajay has a linear shaped island that the water levels exceed.

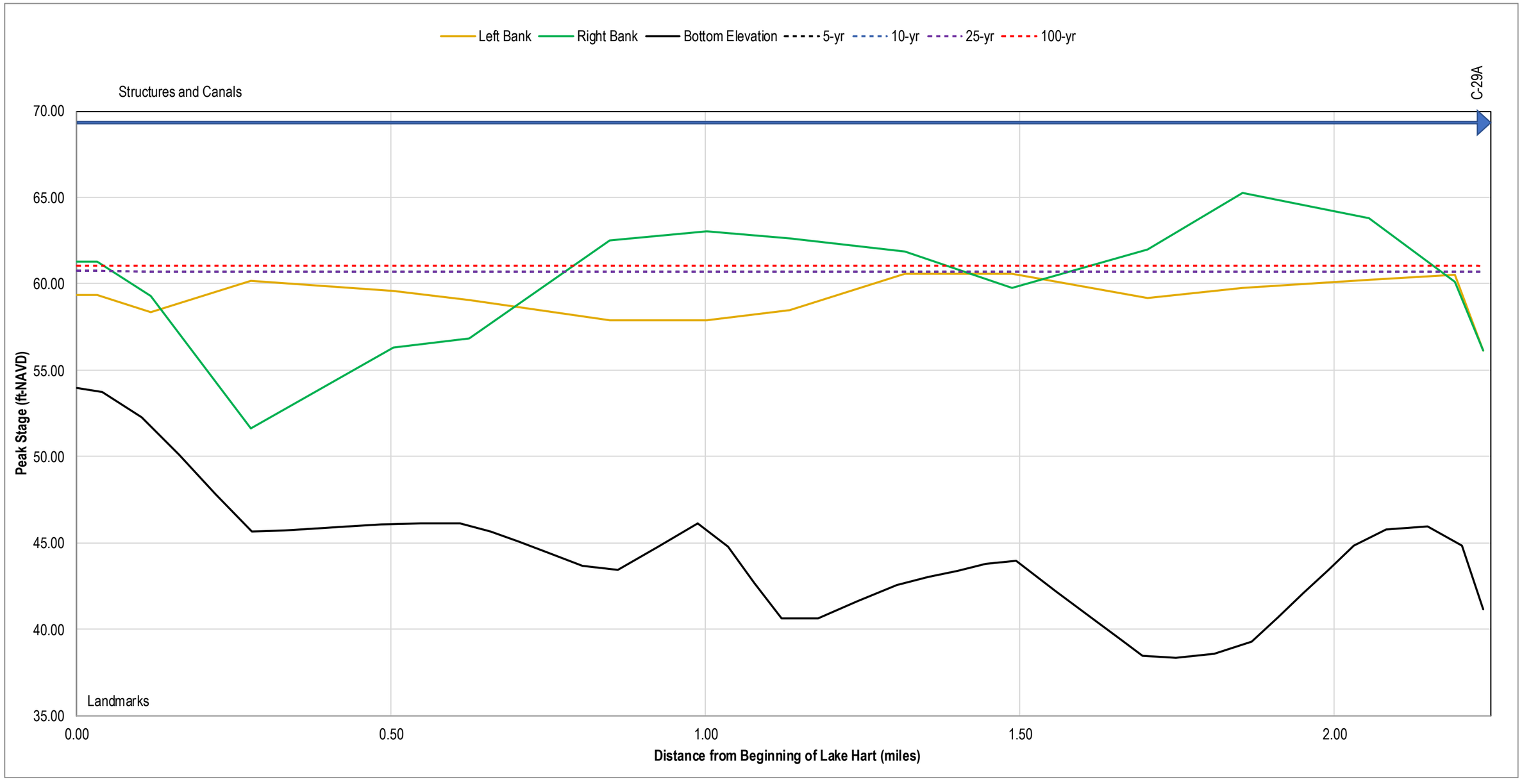


Figure 10-55: Maximum Stage Profile for Lake Hart under Current Conditions

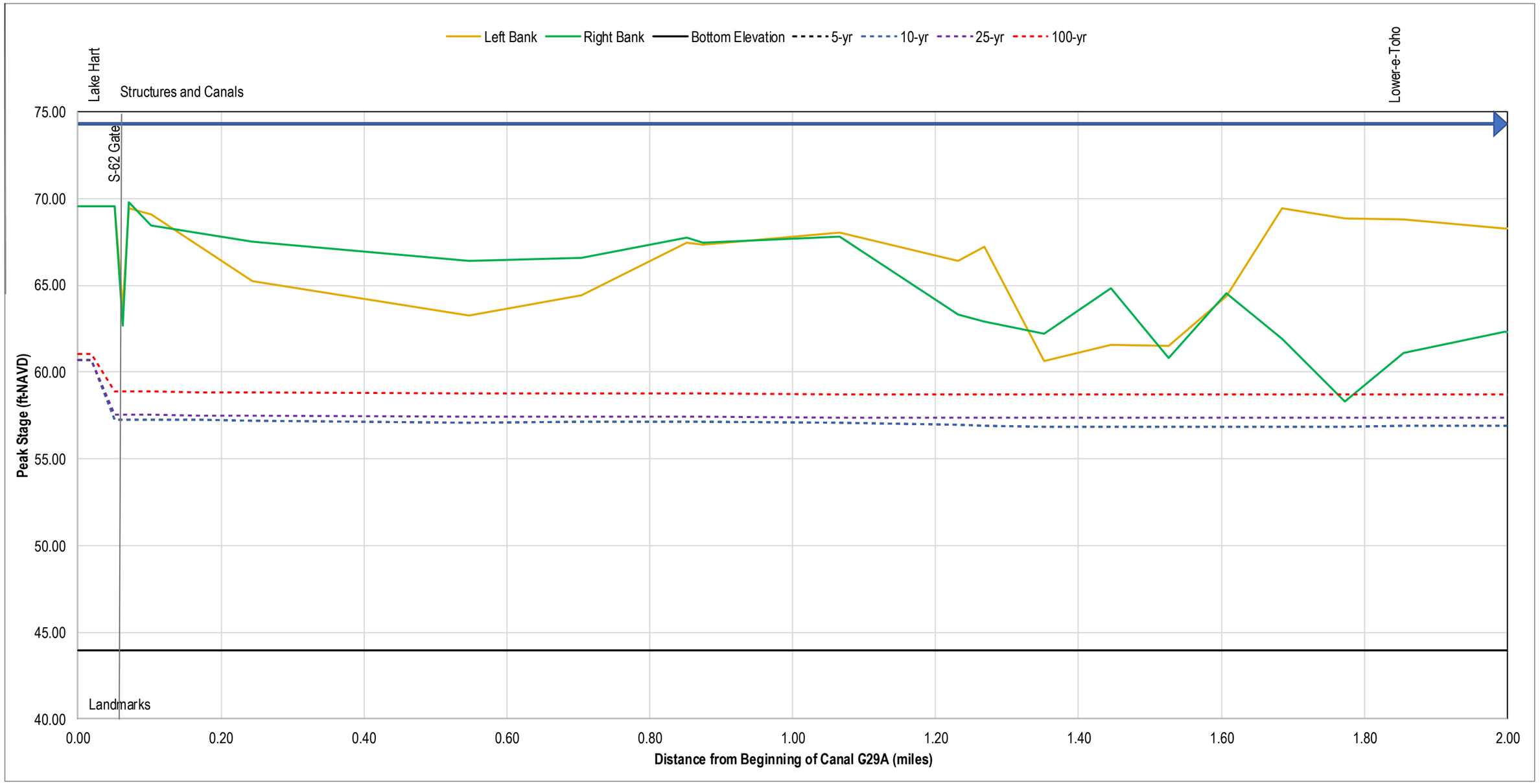


Figure 10-56: Maximum Stage Profile for Canal C-29A under Current Conditions

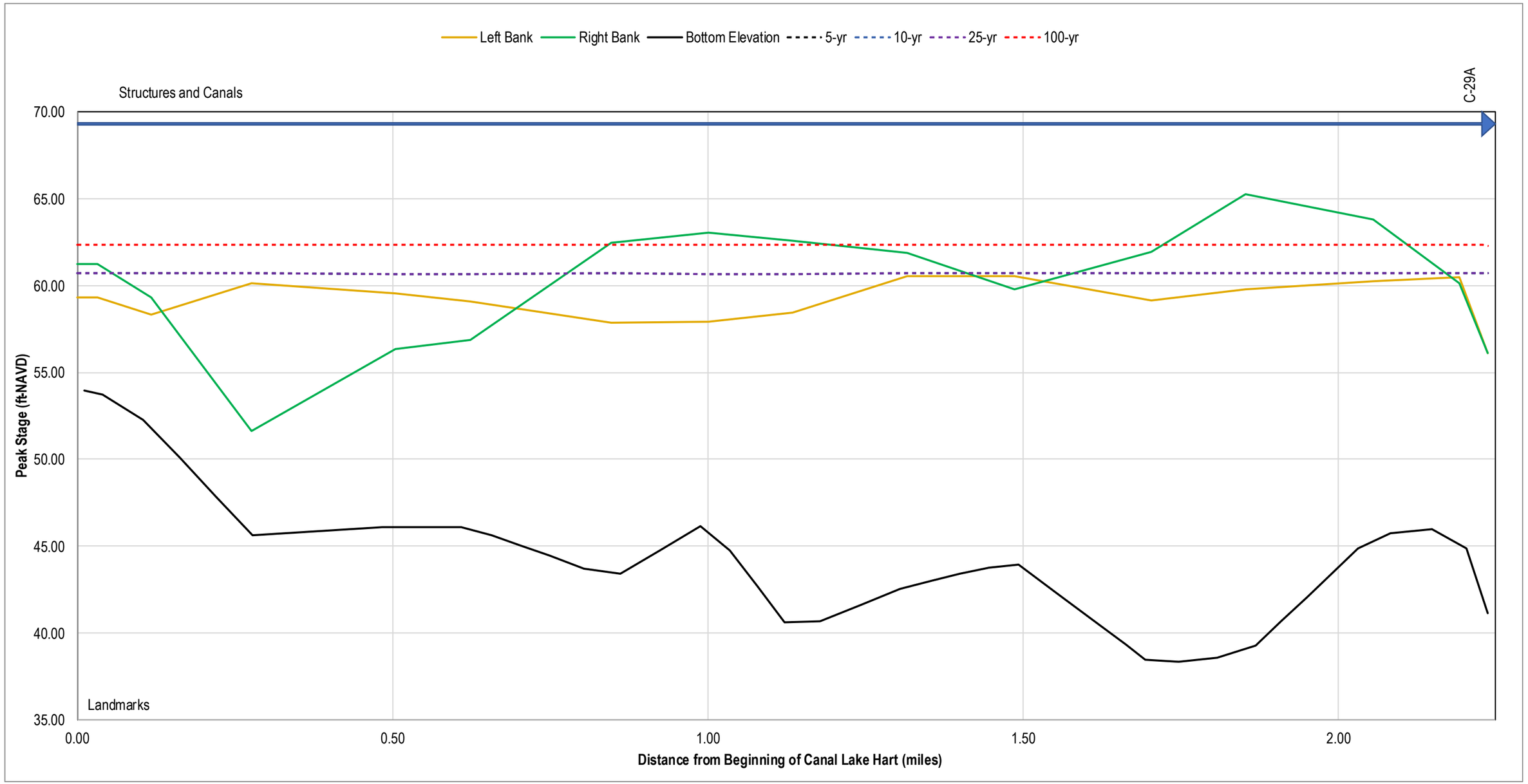


Figure 10-57: Maximum Stage Profile for Lake Hart under Future Conditions

Figure 10-58: Maximum Stage Profile for Canal C-29A under Future Conditions

10.3.2 PM #2

Lake Hart LMA's upstream water control structure is S57, and downstream structure is S62. The maximum daily discharge capacity of this LMA is determined by S62 discharge capacity. Per the Districts ERP Handbook, Lake Hart LMA has an allowable discharge rate of 10.6 CSM, based on a design frequency of 10-yrs. **Table 10-32 and Table 10-33** summarize Lake Hart LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area under current and future hydrologic conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 12.0 CSM under current conditions and 13.8 CSM under future conditions, which are higher than SFWMD's permissible discharge allowance of 10.6 CSM based on a 10-yr design frequency.

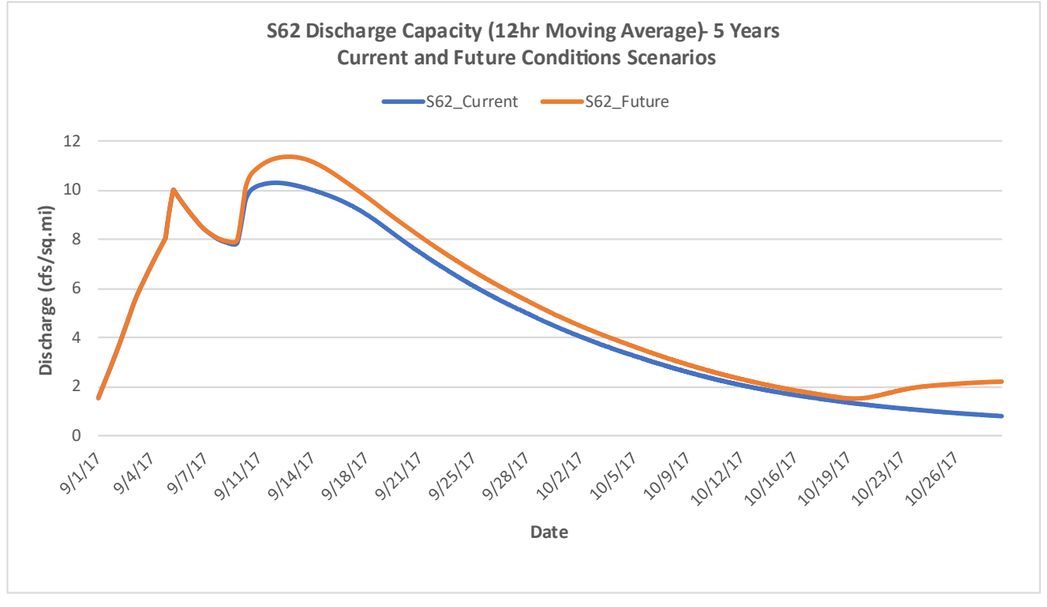
Table 10-32: Drainage Discharge Capacity (cfs/sq. mi) of Lake Hart LMA under Current Conditions

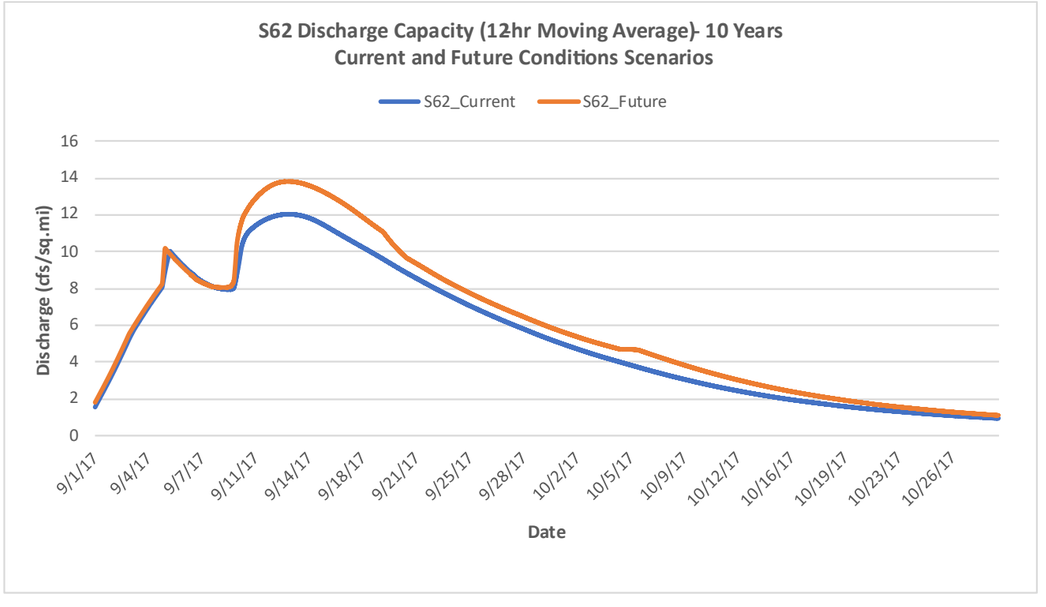
LMA	Total Area (mi^2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Hart	47.39	10.3	12.0	15.0	21.0	10.6

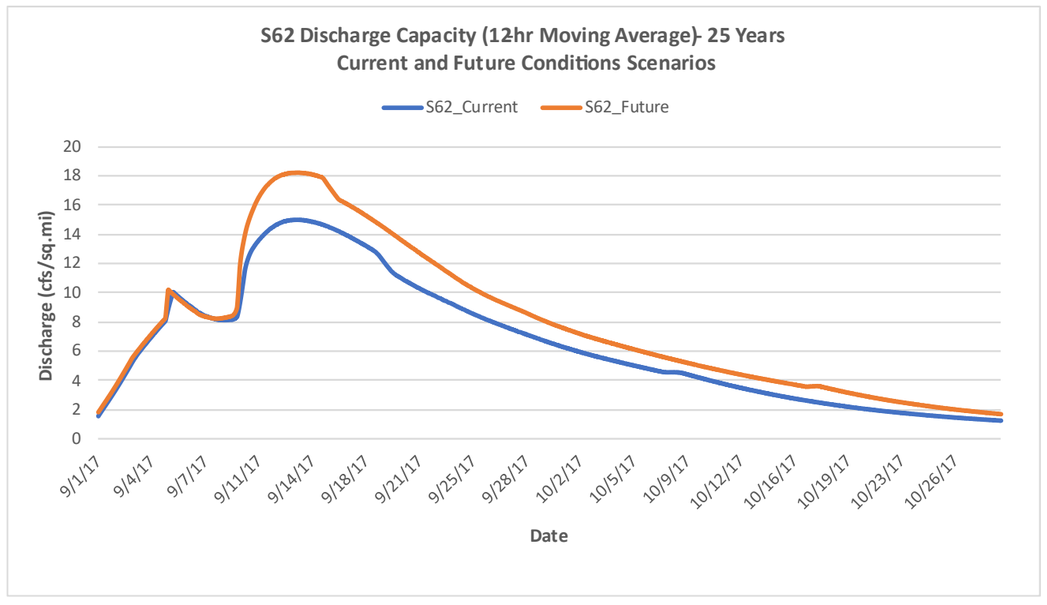
Table 10-33: Drainage Discharge Capacity (cfs/sq. mi) of Lake Hart LMA under Future Conditions

LMA	Total Area (mi^2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Hart	47.39	11.4	13.8	18.2	24.7	10.6

Figure 10-59 shows the peak 12-hour moving average discharge per square mile of drainage area, calculated for each design storm at structure S62 under current and future conditions. **Figure 10-60** shows the maximum daily discharge through structure S62 for both current and future conditions. The moving average numbers also show they exceed the allowable discharge for 10-yr events under current and future conditions.







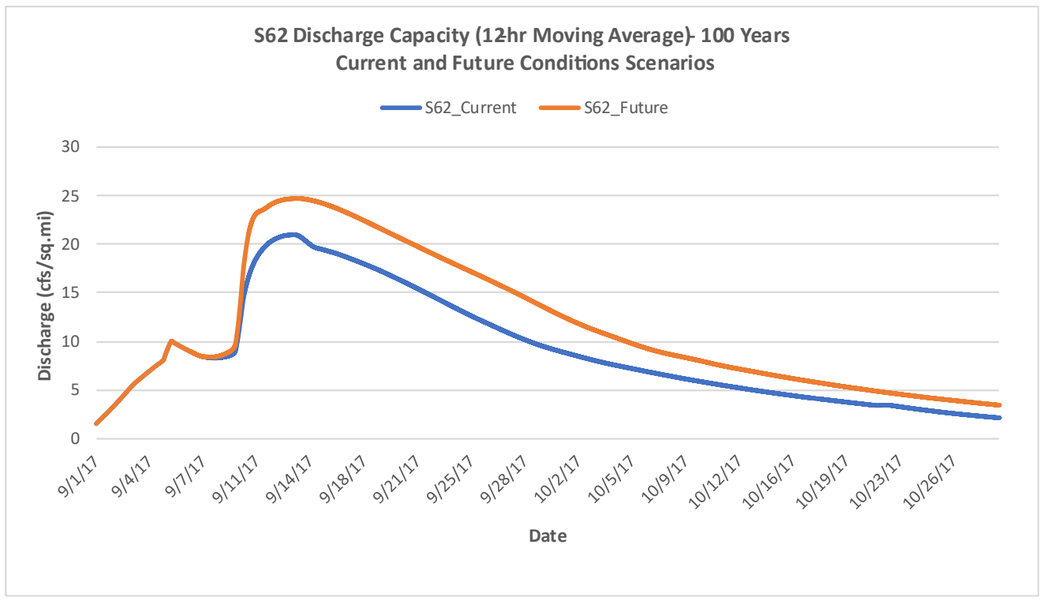


Figure 10-59: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S62 for 5-, 10-, 25-, 100- Yr Design Storms

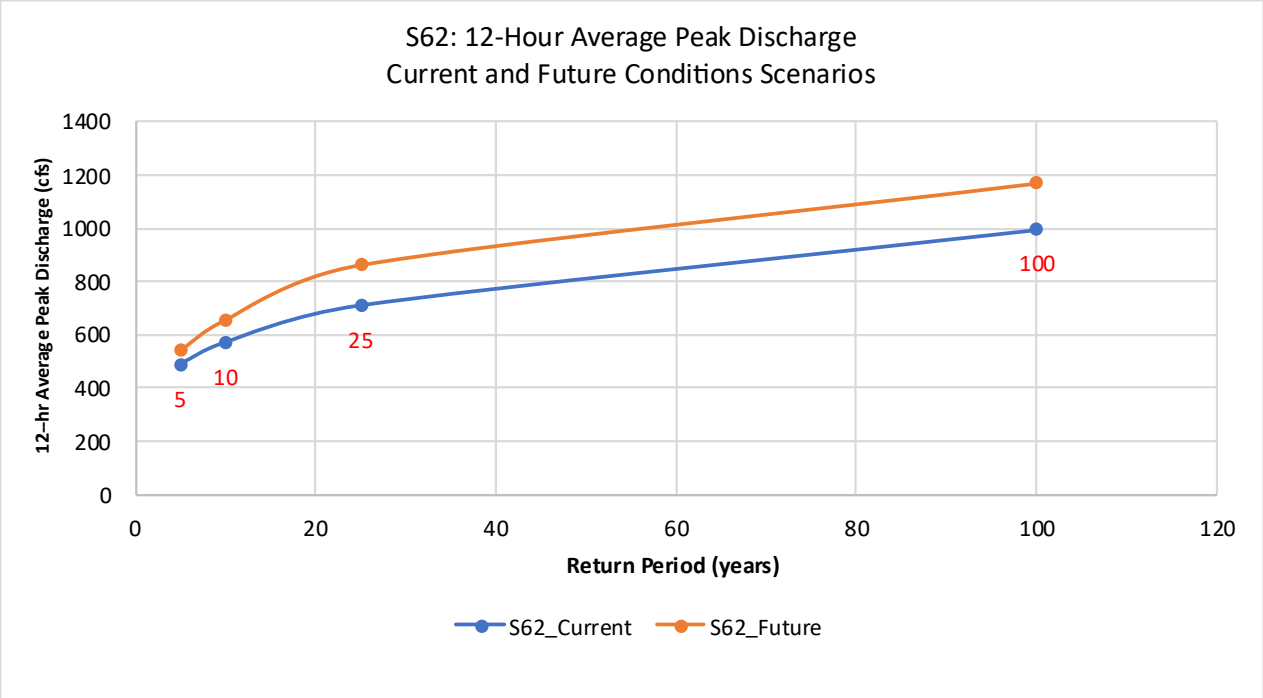


Figure 10-60: Structure S62 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

10.3.3 PM #3

The SFWMD structure design parameters, shown in **Table 10-34**, were used to evaluate the S62 performance against the various design storms. **Table 10-35 through Table 10-40** summarize simulated instantaneous stages and flows at structure S62 under current conditions and future conditions at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S62, the simulated headwater elevation at S62 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is lower than the SPF headwater elevation for both current and future conditions. The simulated flow at structure S62 is higher than the design discharge (640 cfs) except for only 5-yr design storm event in the current condition and future conditions.

During the peak headwater at the structure S62, the simulated tailwater elevation at S62 is lower than design tailwater elevation during 5-yr, 10-yr and 25-yr design storm events for current conditions and future conditions. The simulated flows at structure S62 are higher than the design discharge (640 cfs) for all design storm events except for 5-yr DE.

During the peak flow at Structure S62, the headwater elevations are lower than the design headwater for 5-yr and 10-yr design storm events under current and future conditions. However, the simulated tailwater elevations is lower than design elevation for 3 of the 4 design storm events (except 100-yr) for both conditions. Time series plots of S62 in **Figure 10-61 and Figure 10-62** demonstrate this.

Table 10-34: Design Parameters for Structure S62

Structure	S62
Design Discharge	450 to 640 cfs (Lower Profile and Peak Stage) (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	58.57 to 60.27 feet NAVD88 (59.6 to 61.3 feet NGVD29)
Design Tailwater Elevation	57.77 to 59.07 feet NAVD88 (58.8 to 60.1 feet NGVD29)
SPF Headwater Elevation	65.67 feet NAVD88 (66.7 feet NGVD29)
SPF Tailwater Elevation	64.87 feet NAVD88 (65.9 feet NGVD29)
Optimum Headwater	57.97 to 59.97 feet NAVD88 (59.0 to 61.0 feet NGVD29)
Optimum Tailwater	54.97 to 56.97 feet NAVD88 (56.0 to 58.0 feet NGVD29)

Table 10-35: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak TW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-year	58.47	56.51	488.57	1.96	9/12/17 4:00 PM
10-year	58.93	56.84	570.55	2.09	9/13/17 5:43 PM
25-year	59.66	57.45	709.97	2.21	9/14/17 3:43 AM
100-year	61.03	58.74	985.55	2.28	9/14/17 11:00 AM

Table 10-36: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak HW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-year	58.76	56.73	538.94	2.02	9/13/17 12:00 PM
10-year	59.38	57.22	654.71	2.16	9/13/17 6:43 PM
25-year	60.41	58.16	862.88	2.25	9/14/17 9:00 AM
100-year	62.33	60.08	1164.69	2.25	9/15/17 2:00 AM

Table 10-37: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at peak HW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-year	56.51	58.47	488.00	-1.96	9/13/17 3:25 AM
10-year	56.85	58.90	566.09	-2.05	9/14/17 4:25 PM
25-year	57.53	59.52	681.86	-1.99	9/16/17 9:55 AM
100-year	58.86	60.86	879.99	-2.00	9/17/17 3:00 PM

Table 10-38: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at peak TW (cfs)	Head Differential at peak TW (ft)	Date & Time
5-year	56.74	58.74	536.88	-2.00	9/14/17 7:00 AM
10-year	57.29	59.26	632.04	-1.97	9/15/17 12:00 AM
25-year	58.27	60.29	812.69	-2.02	9/16/17 10:25 AM
100-year	60.27	62.08	1060.69	-1.81	9/18/17 4:00 PM

Table 10-39: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at peak Q (ft)	Date & Time
5-year	488.60	58.47	56.51	1.96	9/12/17 5:43 PM
10-year	570.57	58.93	56.84	2.09	9/13/17 6:00 PM
25-year	709.97	59.66	57.45	2.21	9/14/17 3:43 AM
100-year	995.18	61.02	58.72	2.30	9/14/17 5:00 AM

Table 10-40: Summary of the Instantaneous Conditions at Structure S62 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at peak Q (ft)**	**Date & Time**
5-year	538.97	58.76	56.73	2.02	9/13/17 1:00 PM
10-year	654.76	59.38	57.22	2.15	9/13/17 8:00 PM
25-year	862.88	60.41	58.16	2.25	9/14/17 9:00 AM
100-year	1172.42	62.31	59.97	2.34	9/14/17 7:00 AM

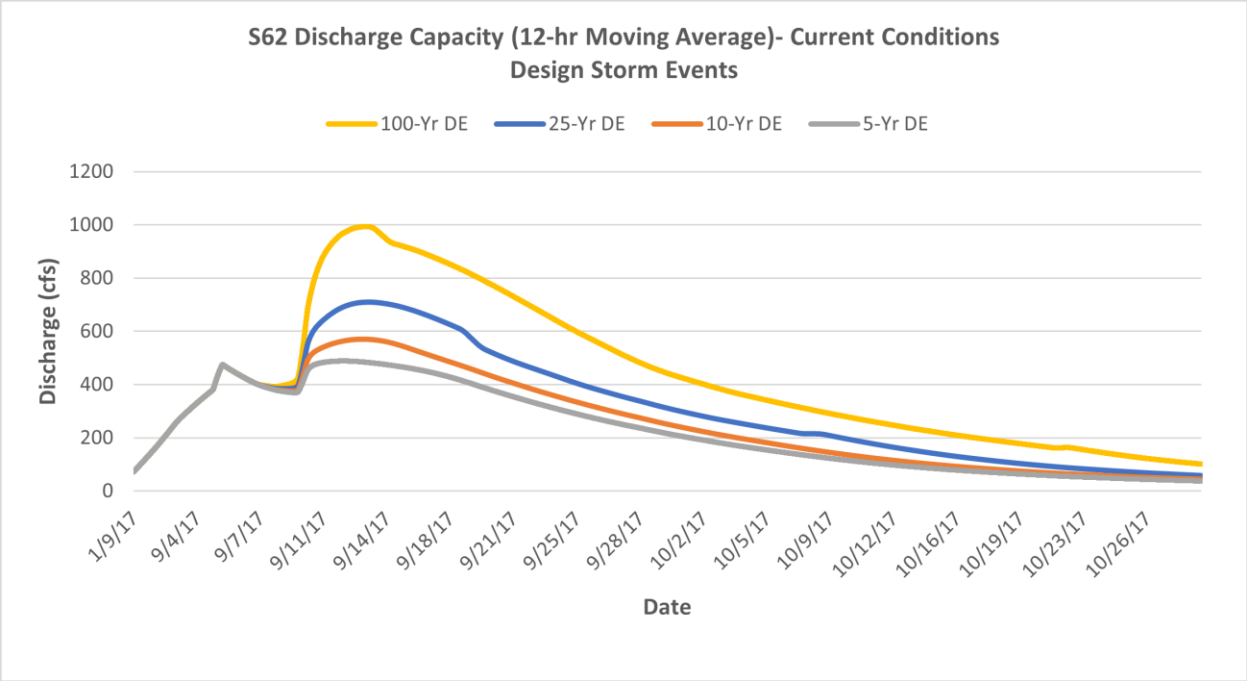


Figure 10-61: 12-hr Moving Average Flows in cfs at S62 under Current Conditions

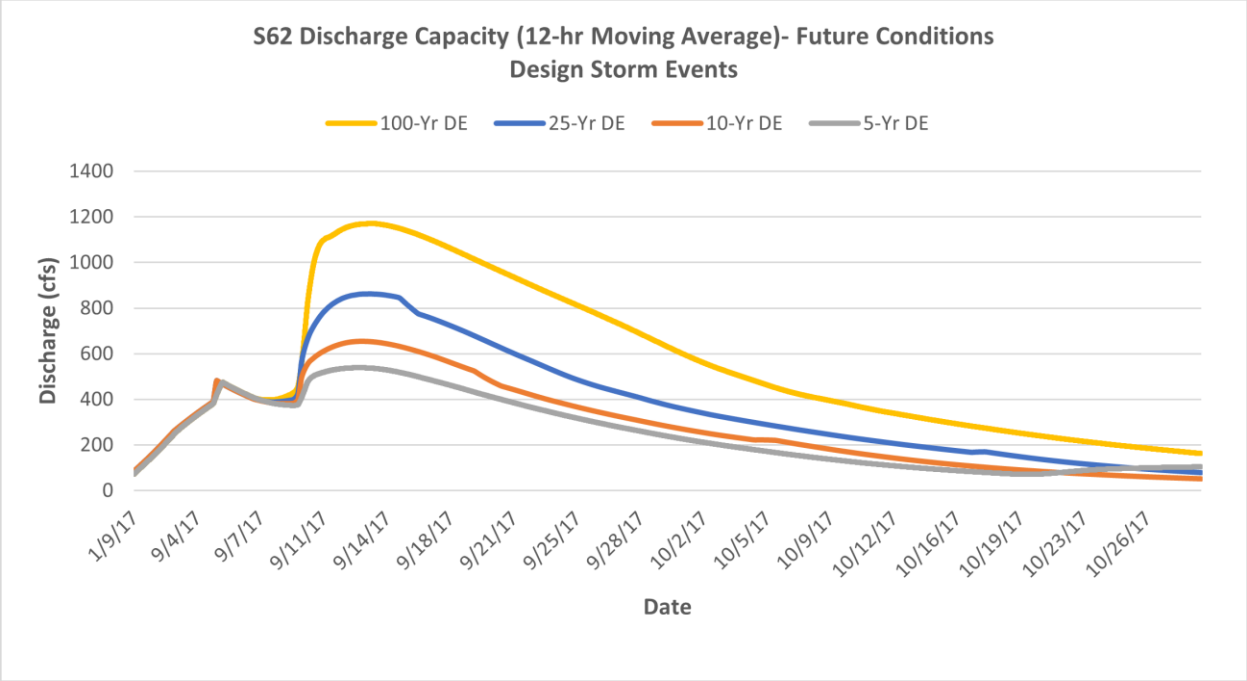


Figure 10-62: 12-hr Moving Average Flows in cfs at S62 under Future Conditions

10.3.4 PM #4

PM#4-UKB was not assessed for this LMA.

10.3.5 PM #5

The maximum overland depth was extracted for each of the design storms under current and future conditions and evaluated for Lake Hart LMA under current and future conditions. **Table 10-41** shows the flood inundation area in square miles for this LMA, tabulated in incremental intervals of feet. **Figure 10-62** provides this data in a plot format. **Figure 10-63, Figure 10-64, Figure 10-65, and**. **Figure 10-66** present the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions for Lake Hart LMA.

The table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not significant. There is also high distribution of flooding depths higher than 2.5 feet of flooding, which are in lakes Mary Jane, Hart, Whippoorwill, Nona, Mud, Buck and Red. There is almost an equal acreage of areas that are inundated between 0.25 to 0.5 ft range; these are Crosby Island Swamp to the north of Lake Mary Jane, Econlockhatchee River Swamp to the east of Lake Mary Jane and Split Oak Park Forest between lakes Hart and Mary Jane.

Table 10-41: Incremental Flood Inundation Area (sq. mi.) in Lake Hart LMA

Flooding Depth (ft)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	26.89	23.77	18.97	11.43	26.90	23.78	18.98	11.43
0.25<=Depth<0.5	4.63	5.31	6.76	9.34	4.61	5.31	6.78	9.35
0.5<=Depth<0.75	3.53	3.42	3.08	3.02	3.55	3.42	3.09	3.03
0.75<=Depth<1.0	3.10	3.22	2.78	2.26	3.07	3.20	2.78	2.24
1.0<=Depth<1.25	2.70	2.50	2.45	2.21	2.70	2.51	2.44	2.20
1.25<=Depth<1.5	1.13	2.05	2.95	2.11	1.17	2.05	3.00	2.12
1.5<=Depth<1.75	0.58	1.72	2.07	2.15	0.58	1.71	2.07	2.14
1.75<=Depth<2.0	0.20	0.54	1.50	2.47	0.21	0.54	1.46	2.49
2.0<=Depth<2.25	0.17	0.27	1.33	1.94	0.16	0.27	1.35	1.95
2.25<=Depth<2.5	0.10	0.10	0.60	1.44	0.10	0.11	0.58	1.44
>2.5	4.35	4.49	4.89	9.03	4.35	4.48	4.88	9.00
Lake Hart LMA	**30324.73**	**acres**						
	47.38	**sq. mi.**						

Note: *The lake area of 9.30 sq. mi. was included in this table computations.*

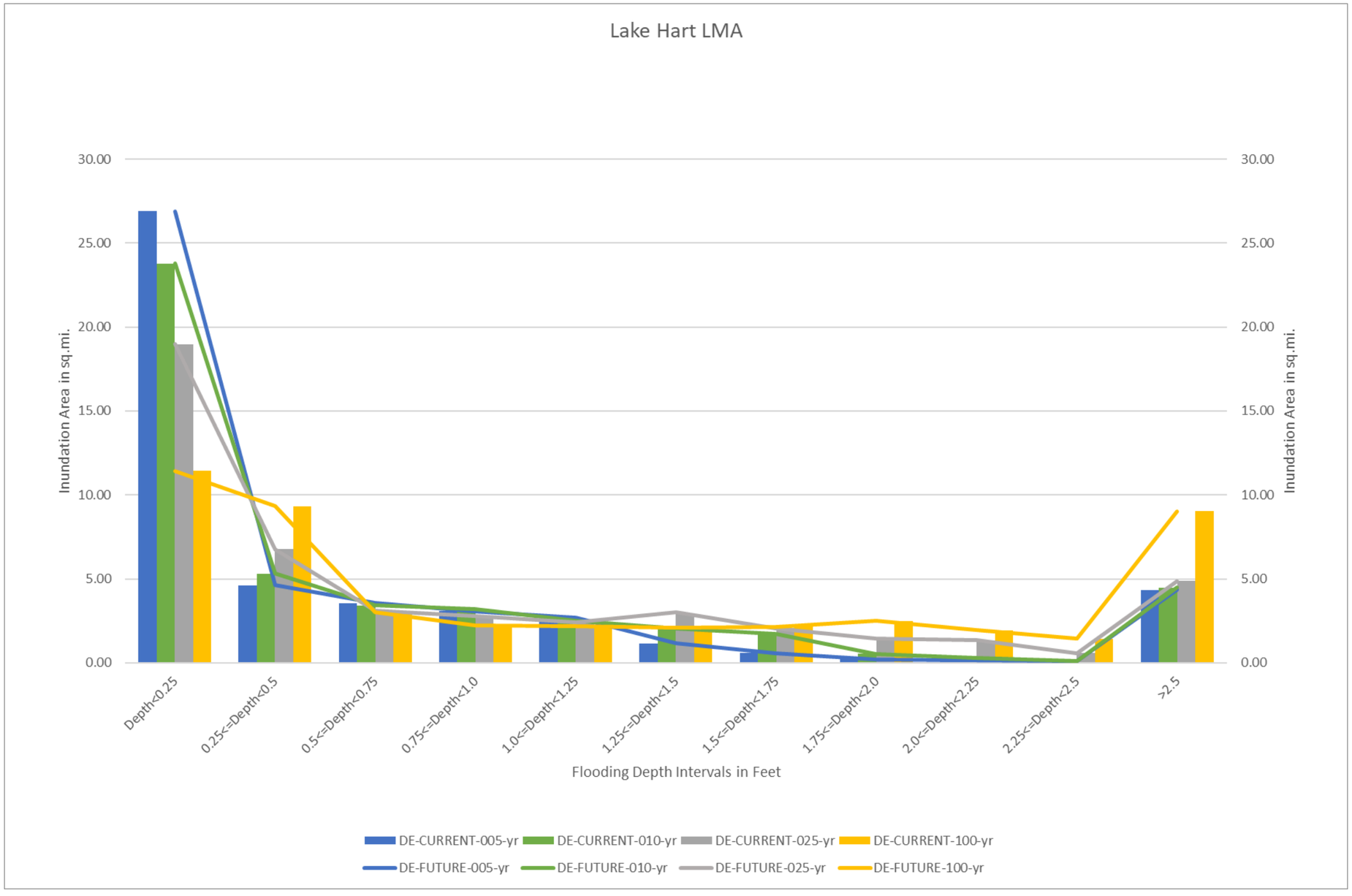


Figure 10-63: Incremental Flood Inundation Depth in feet in Lake Hart LMA under Current and Future Conditions

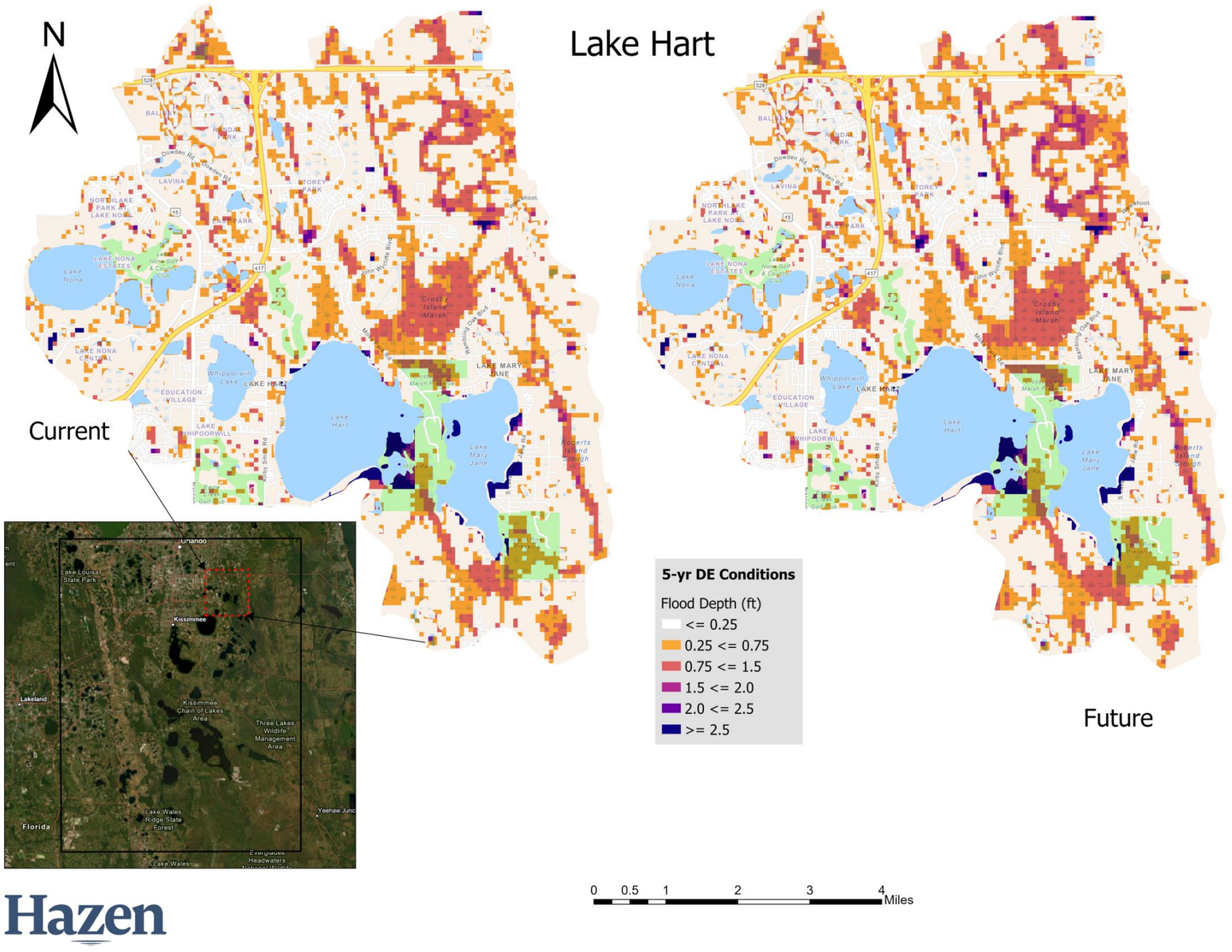
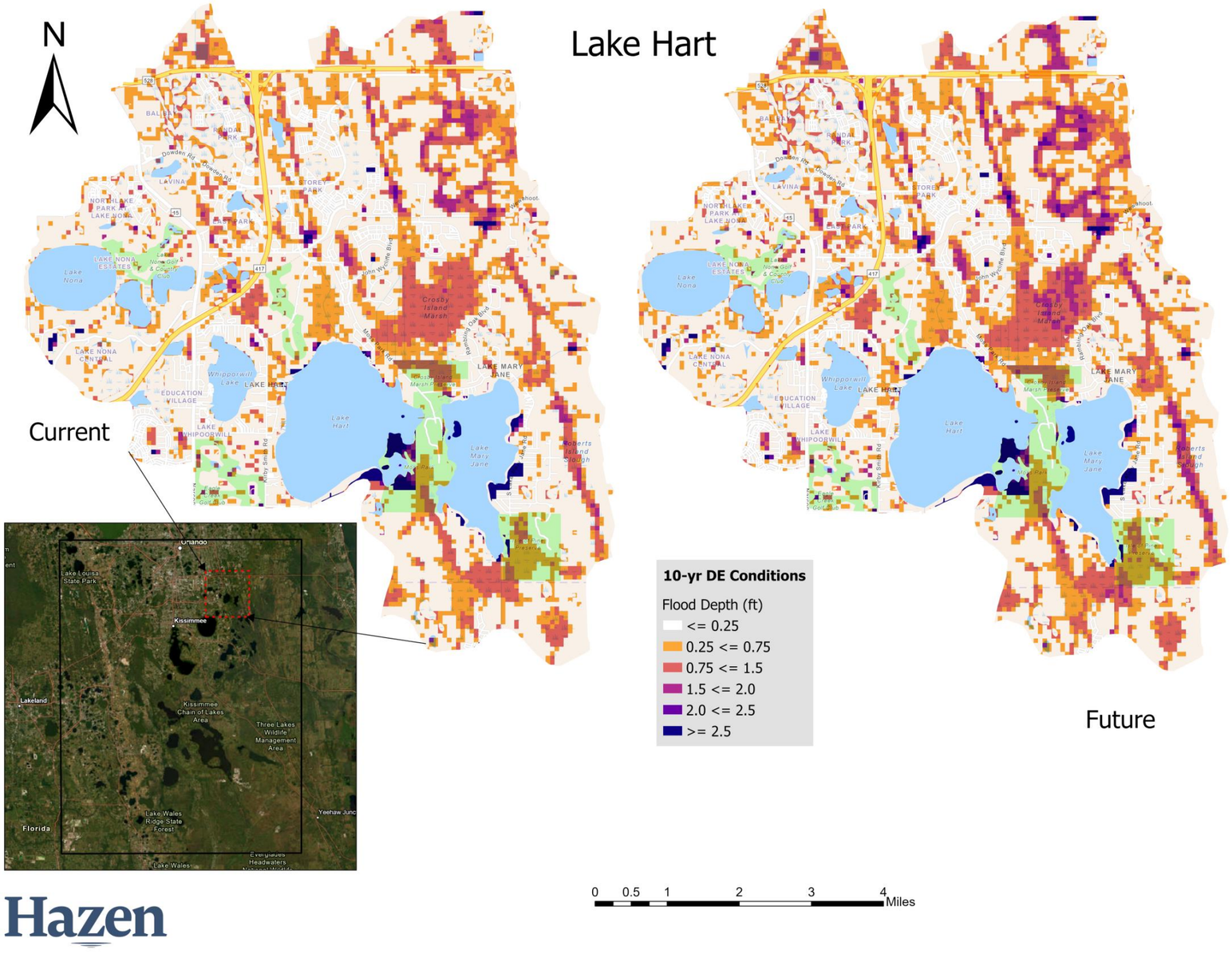


Figure 10-64: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA



Hazen

Figure 10-65: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA

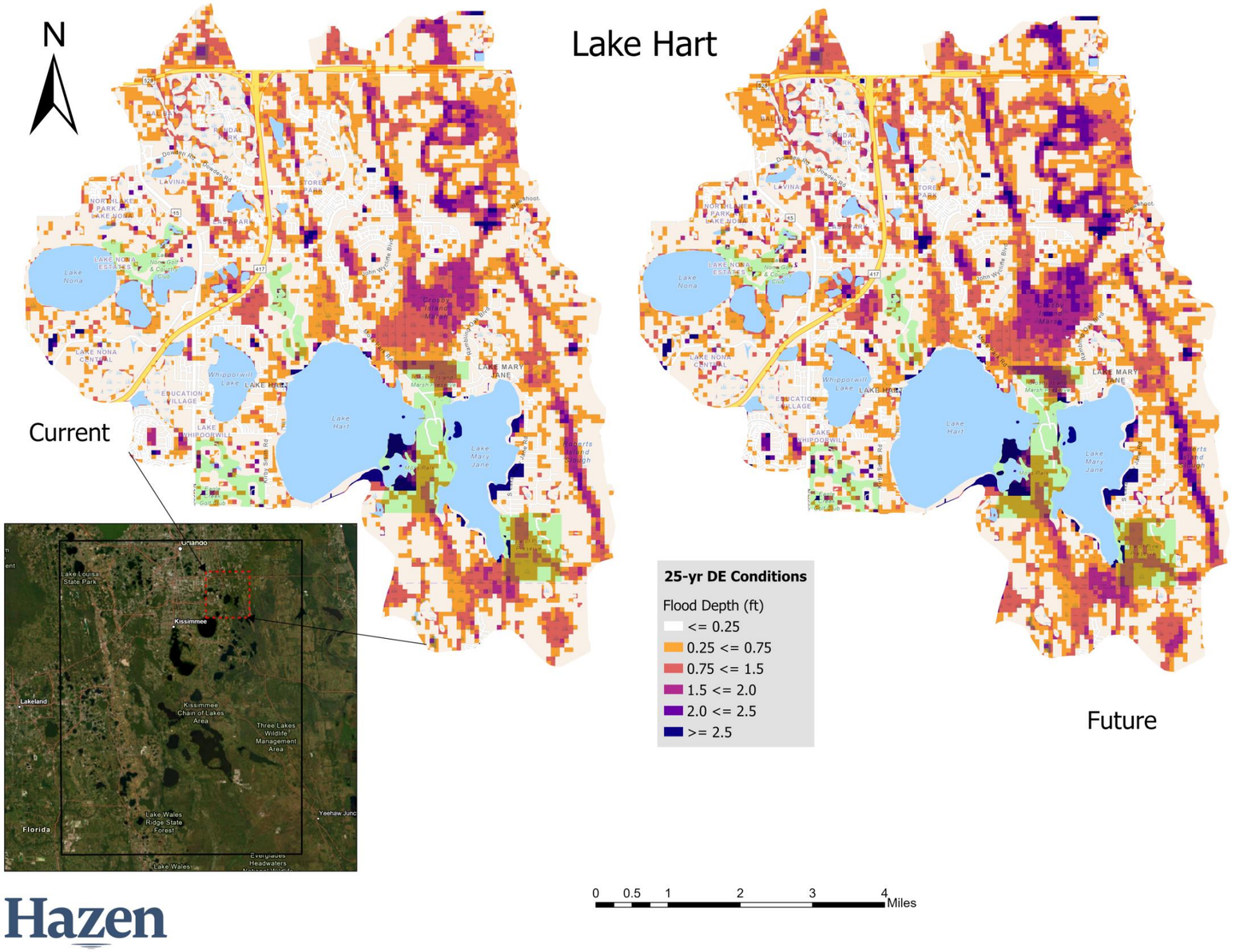


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Figure 10-66: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA

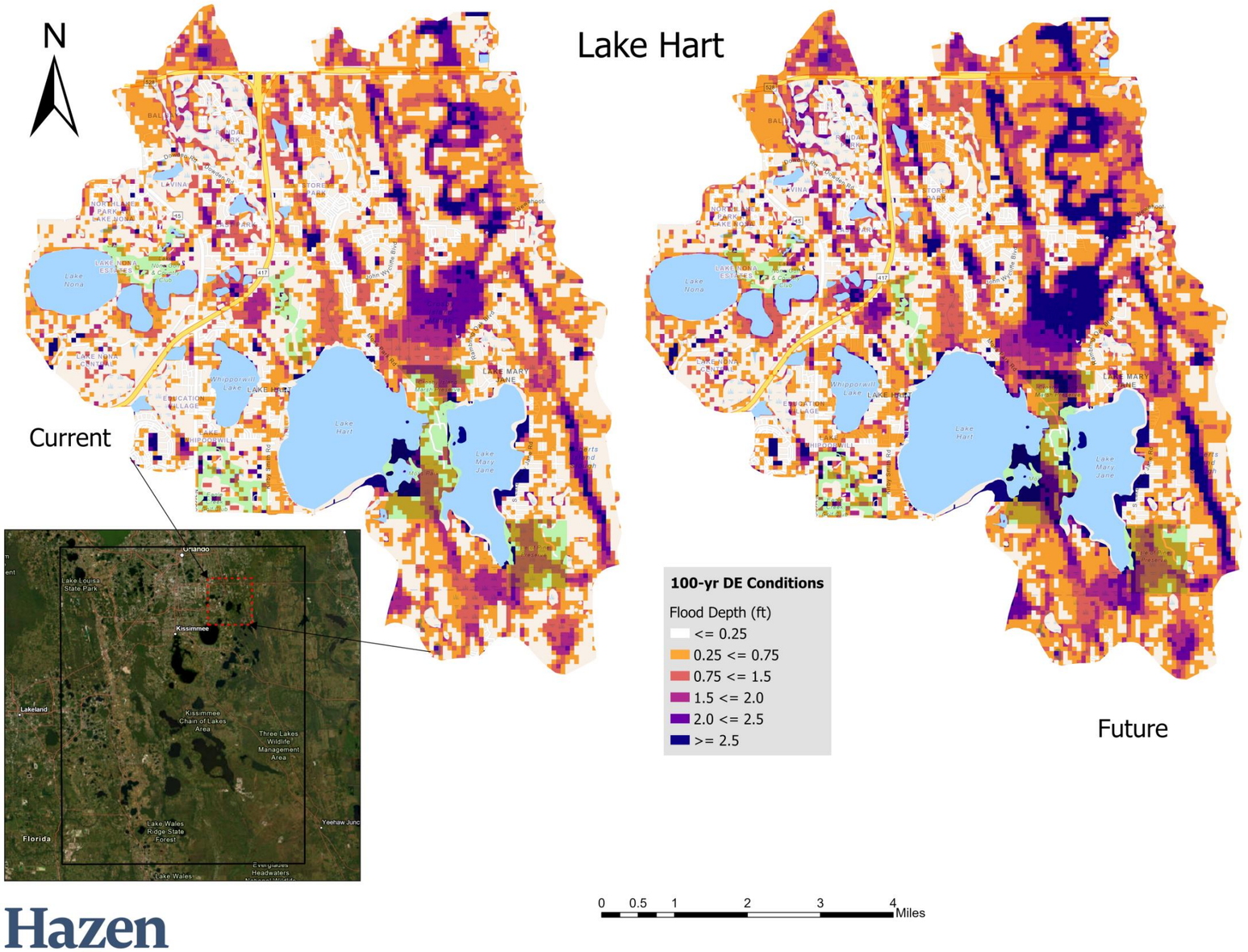

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Figure 10-67: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA

10.3.6 PM #6

Table 10-42 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Hart LMA. **Figure 10-68** show these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Hart LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-69 through Figure 10-72** show duration of the flood waters in Lake Hart LMA.

The table and the plot show that the greatest distribution of flooding duration in Lake Hart LMA is between zero and one hour of flooding, which can be considered insignificant. The second greatest distribution of flooding duration in Lake Hart LMA are areas with longer than 360 hours of flooding, which are the lakes.

Table 10-42: Areas of Incremental Flood Duration (sq. mi.) in Lake Hart LMA

Flooding Duration (Hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	28.65	26.24	22.54	15.97	28.65	26.25	22.58	15.97
1<=Hours<4	0.80	1.19	2.14	4.76	0.79	1.19	2.14	4.75
4<=Hours<8	0.74	0.81	0.70	0.90	0.74	0.82	0.71	0.91
8<=Hours<12	0.47	0.52	0.51	0.53	0.49	0.52	0.51	0.51
12<=Hours<24	0.96	1.02	1.40	1.29	0.97	1.02	1.41	1.30
24<=Hours<48	0.97	1.12	1.27	1.39	1.02	1.13	1.31	1.34
48<=Hours<96	1.16	1.55	1.38	1.63	1.13	1.54	1.35	1.57
96<=Hours<168	1.02	1.14	1.16	1.43	1.02	1.14	1.18	1.52
168<=Hours<240	0.73	0.76	0.92	0.89	0.72	0.78	0.97	0.95
240<=Hours<360	0.88	1.08	1.21	1.32	0.90	1.08	1.40	1.51
360<=Hours<1417	11.00	11.94	14.15	17.27	10.95	11.91	13.83	17.04
Lake Hart LMA	**30324.73**	**acres**						
	47.38	**sq. mi.**						

Note: *The lake area of 9.30 sq. mi. was included in this table computations.*

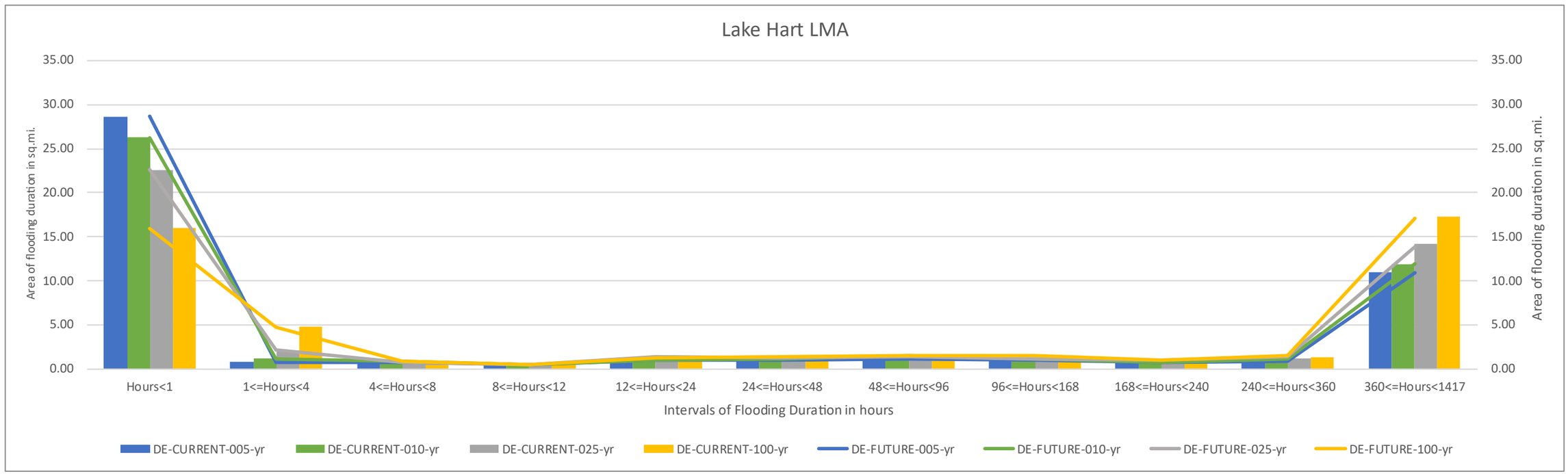

Figure 10-68 Areas of Incremental Flood Duration in Hours in Lake Hart LMA under Current And Future Conditions

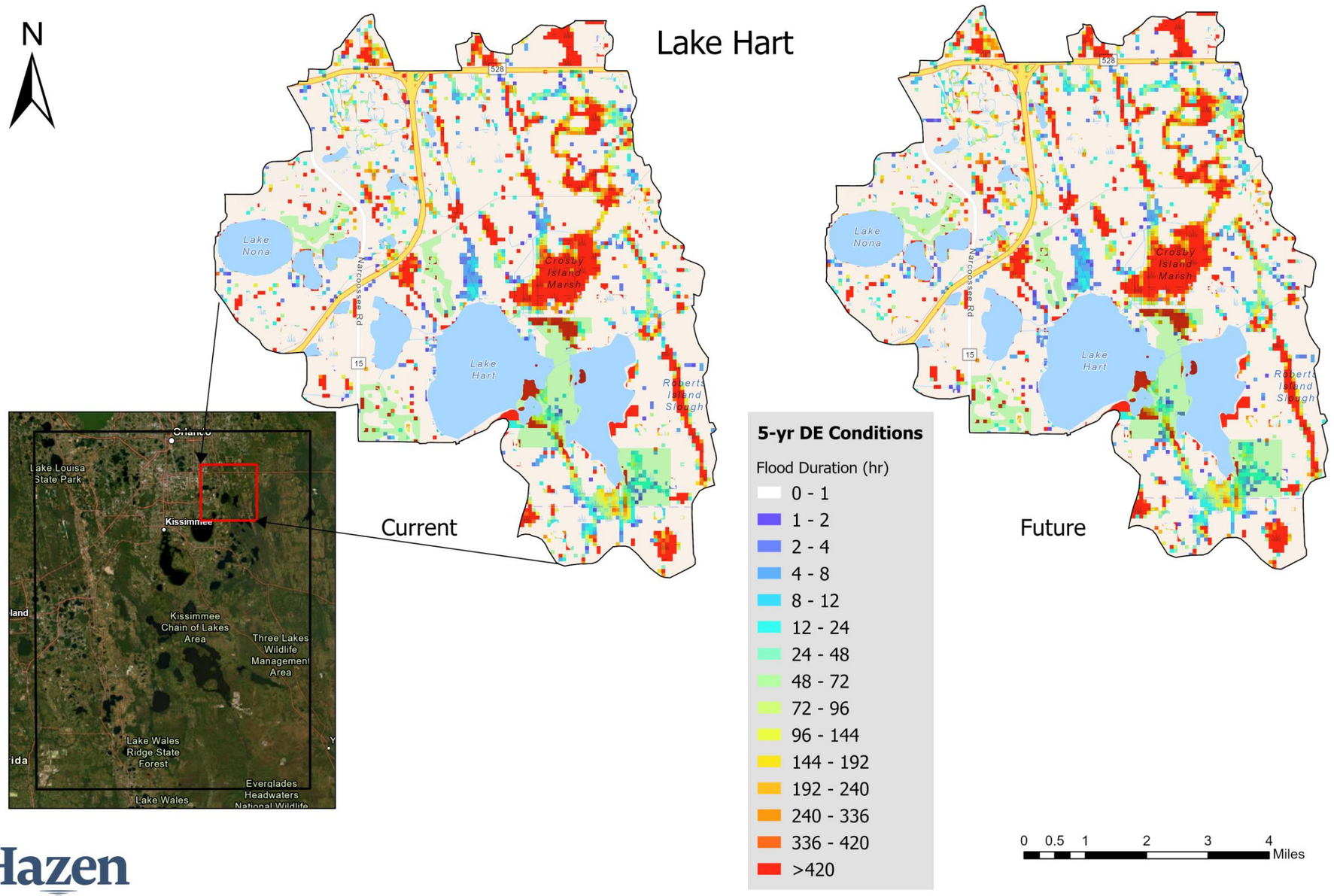

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Figure 10-69: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

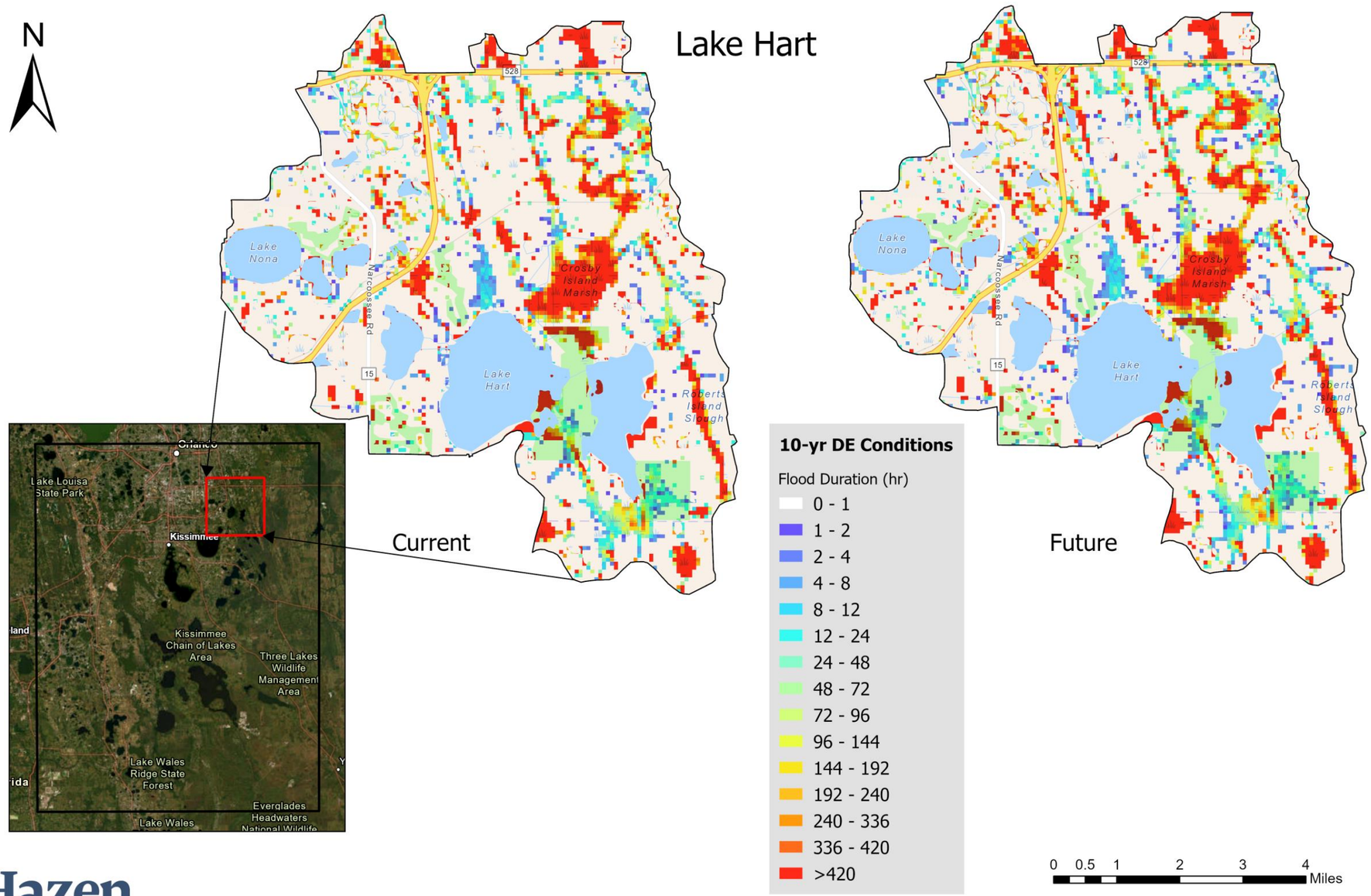

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Figure 10-70: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

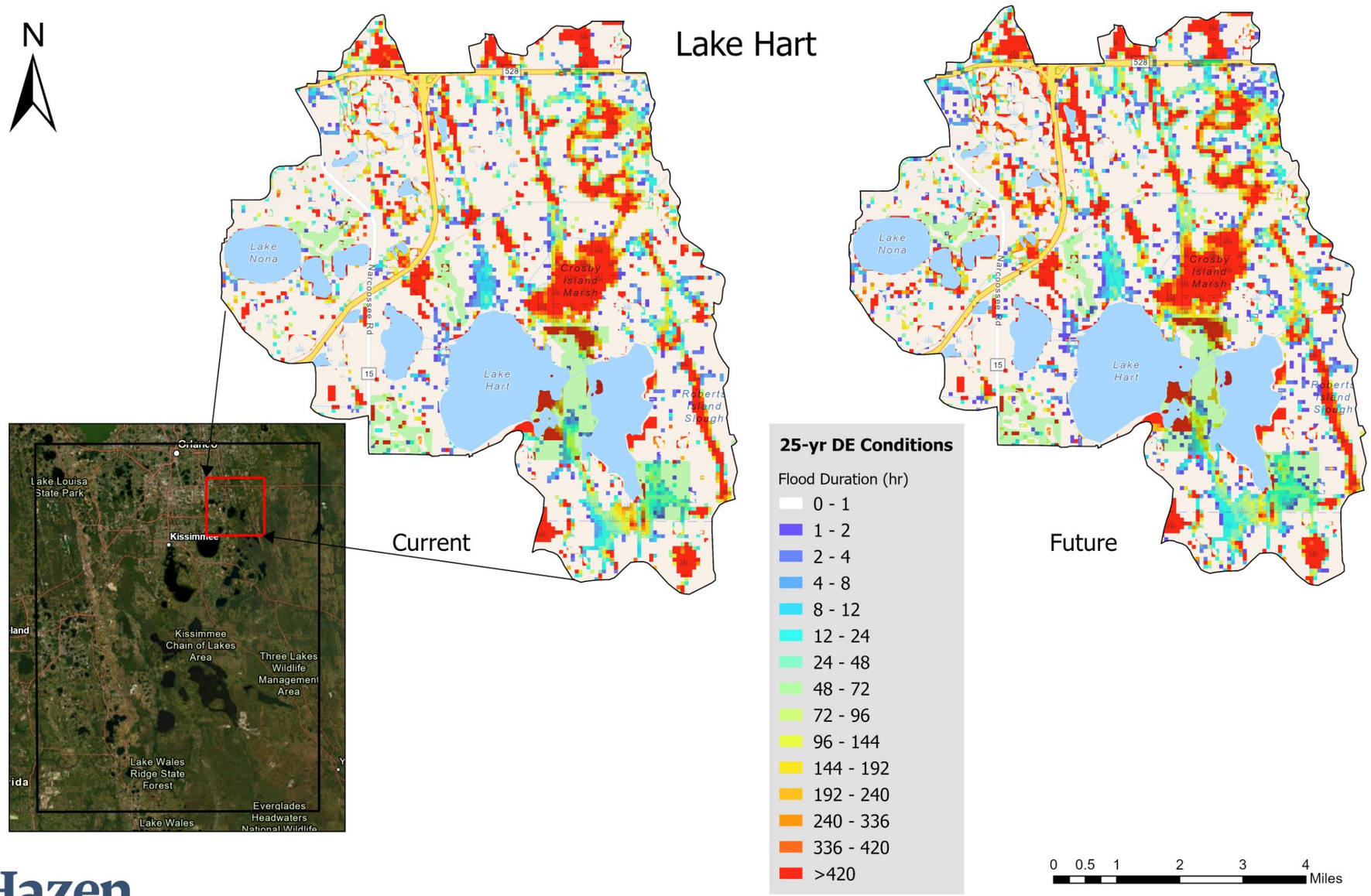

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Figure 10-71: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

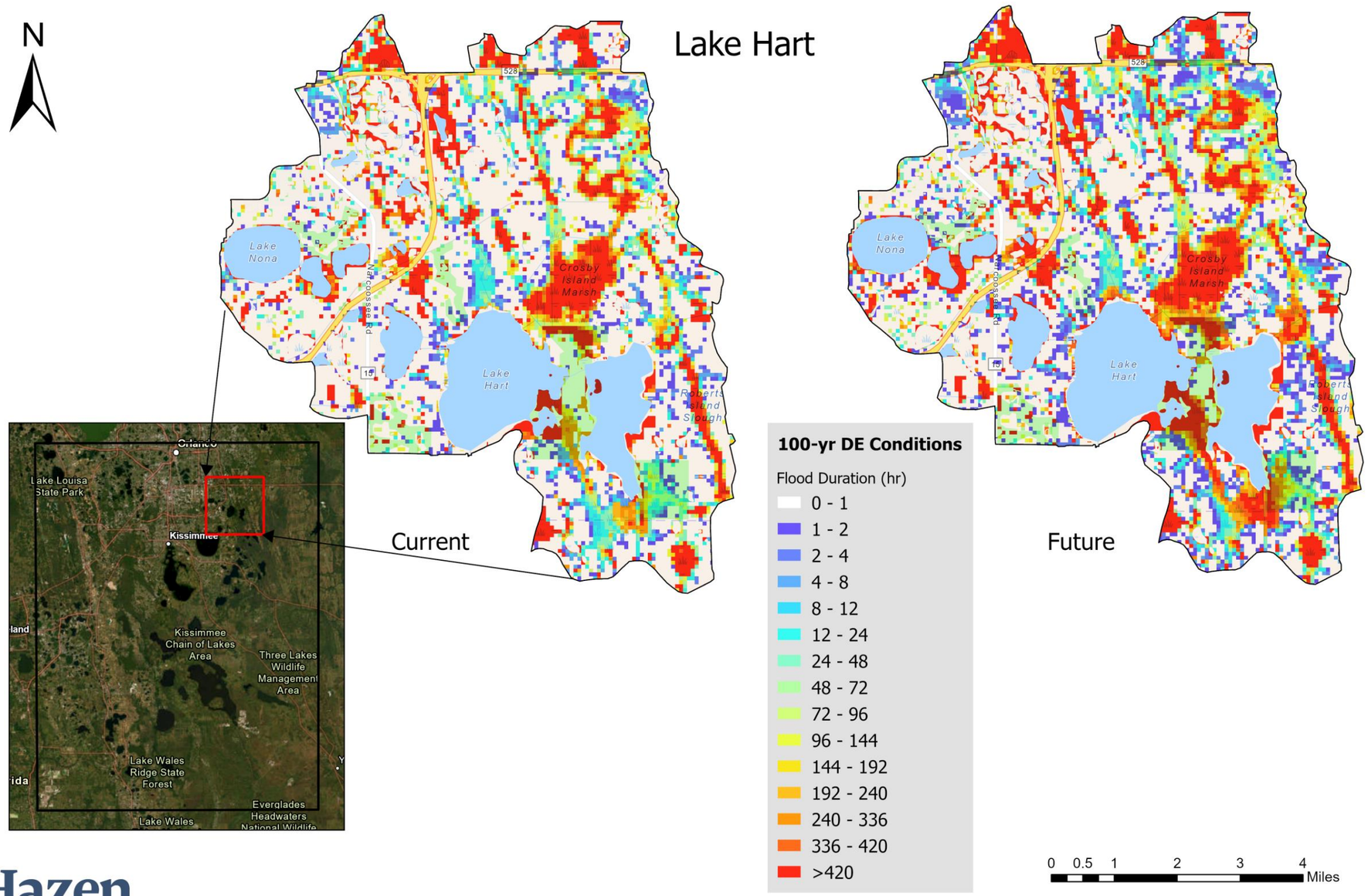

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Figure 10-72: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

10.3.7 FPLOS Rating of Lake Hart LMA

Table 10-43 provides a summary of evaluation of Lake Hart LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The average or overall FPLOS rating for the LMA is < 5-yr.

Table 10-43: Summary of PM Analysis in Lake Hart LMA

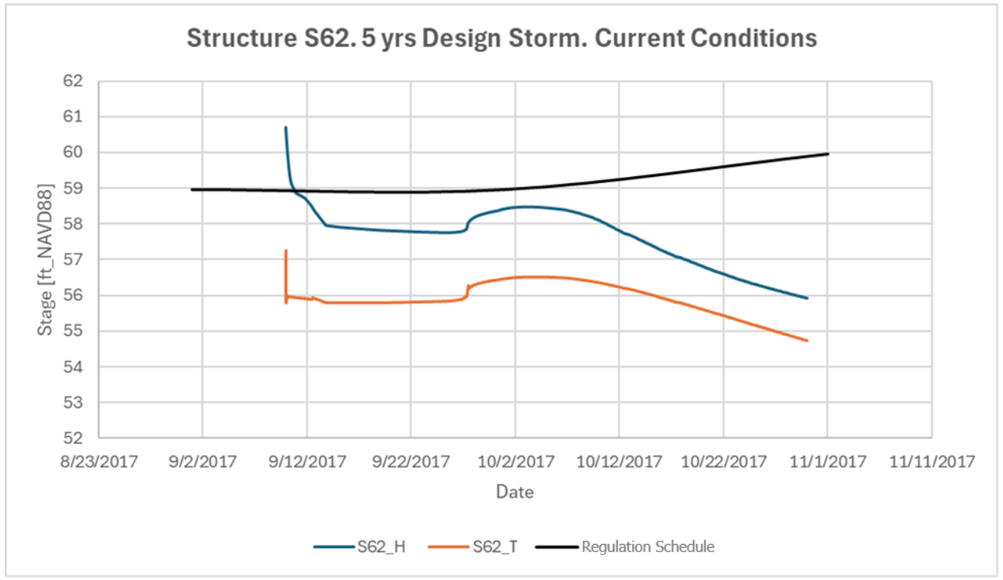
Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Hart, C-29A canal	At Lake Hart, water levels exceed the banks for all design events under both current and future conditions. Along Canal C-29A, water levels remain within banks for current and future conditions upstream of Lake Ajay, only surpassing right bank for 100- yr events for both future and current conditions.	<5-yr in Lake Hart 25-yr in Canal C-29A	<5-yr in Lake Hart 25-yr in Canal C-29A
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 12.0 CSM under current conditions and 13.8 CSM under future conditions. Both exceed SFWMD's permissible discharge allowance of 10.6 CSM based on a 10-yr design frequency.	10-yr discharge higher than permissible discharge	10-yr discharge higher than permissible discharge
PM#3	Entire LMA	During peak tailwater, the simulated flow at structure S62 is higher than the design discharge (640 cfs) for only 5-yr design storm event in the current condition and future conditions. During peak headwater, the simulated flows at structure S62 are higher than the design discharge (640 cfs) for all design storm events except for 5-yr.	5-yr	5-yr
PM#4-UKB	-	-	-	-
PM#5	Entire LMA	For 5- and 10-yr design events under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 60% of the LMA is inundated more than 0.25-ft for a 25-yr design event and approximately 75% of the LMA is inundated more than 0.25-ft for a 100-yr event under both current and future conditions.	10-yr	10-yr
PM#6	Entire LMA	Approximately 65% of the LMA stays flooded for shorter than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 45% of the LMA stays flooded for shorter than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm.	5-yr	5-yr
Lake Hart LMA Overall FPLOS		**Current Conditions**	**< 5-yr**	
		Future Conditions	**< 5-yr**	

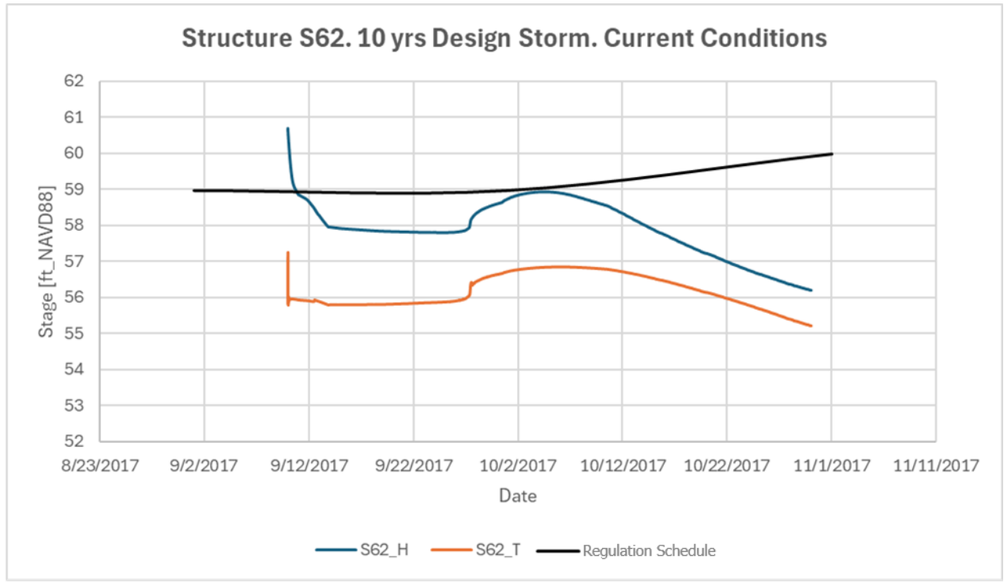
10.3.8 Volume in Lake due to Design Storm UKB-V_{Hart}: Lake Hart

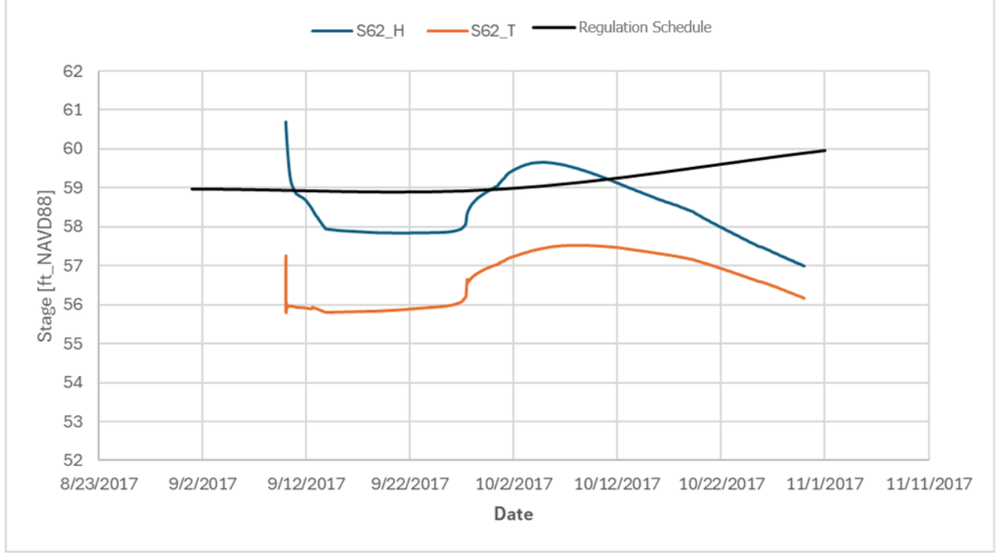
Table 13-3 shows how much water volume accumulated in Lake Hart, measured by subtracting the starting volume from the final volume after each storm event. The table also shows that the volume is higher under future conditions. This is important if any developments are planned around this lake, which may increase the runoff to the lake. **Table 13-4** shows the volume of water at peak.

10.3.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}: Lake Hart

This section examines the water level and regulation schedule of Lake Hart by analyzing its main regulation structure, S62. **Figure 10-73 and Figure 10-74** display the headwater and tailwater stages at S62, along with the regulation schedule for the whole simulation period, for current and future conditions respectively. Water levels in Lake Hart don't exceed or slightly exceed the regulation schedule for either 5- or 10-yr events under current and future conditions. For 25-yr design event under current and future conditions, lake stage returns to the regulation schedule in less than a month after the storm occurrence. Water levels for 100-yr design event scenarios under current and future 100 years stay above regulation schedule for about a month







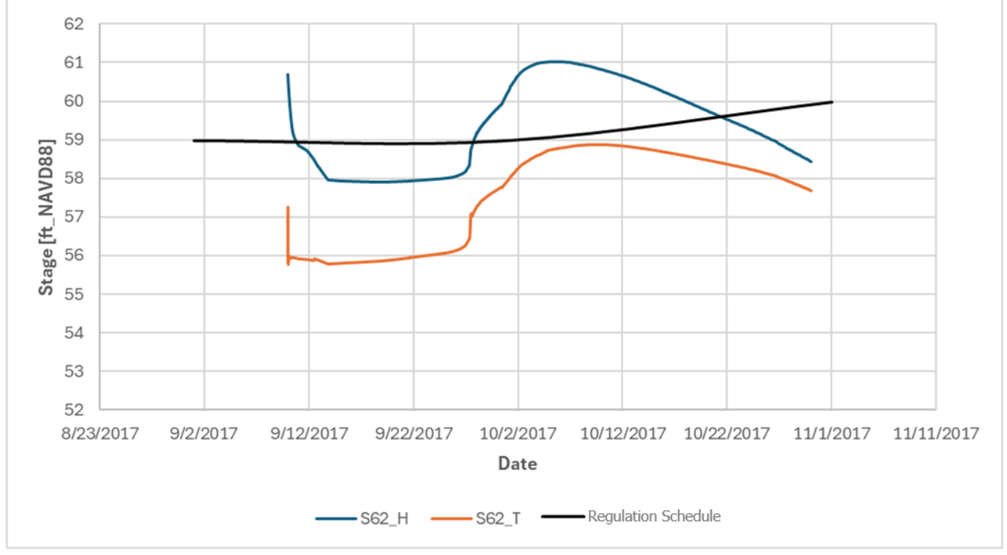
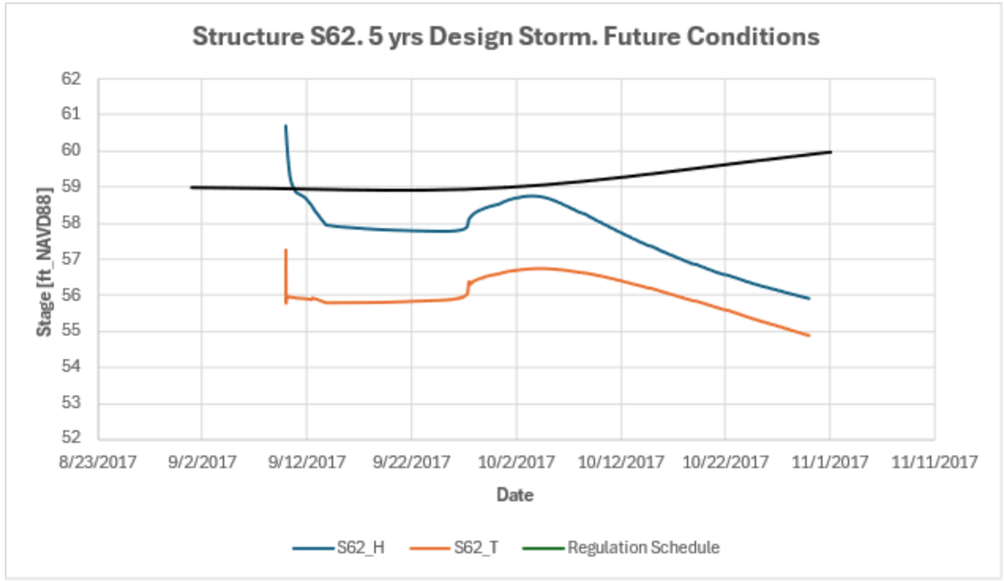
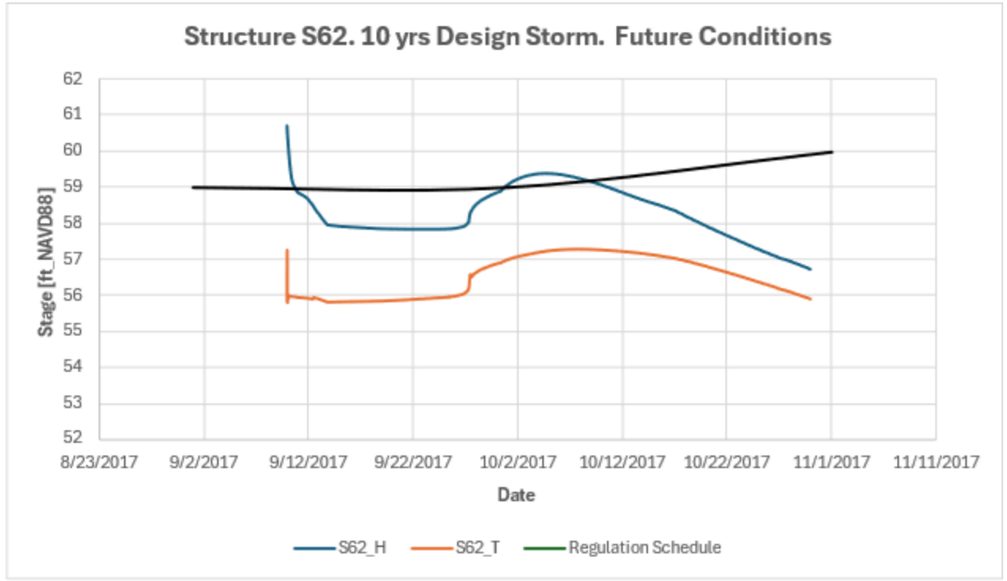
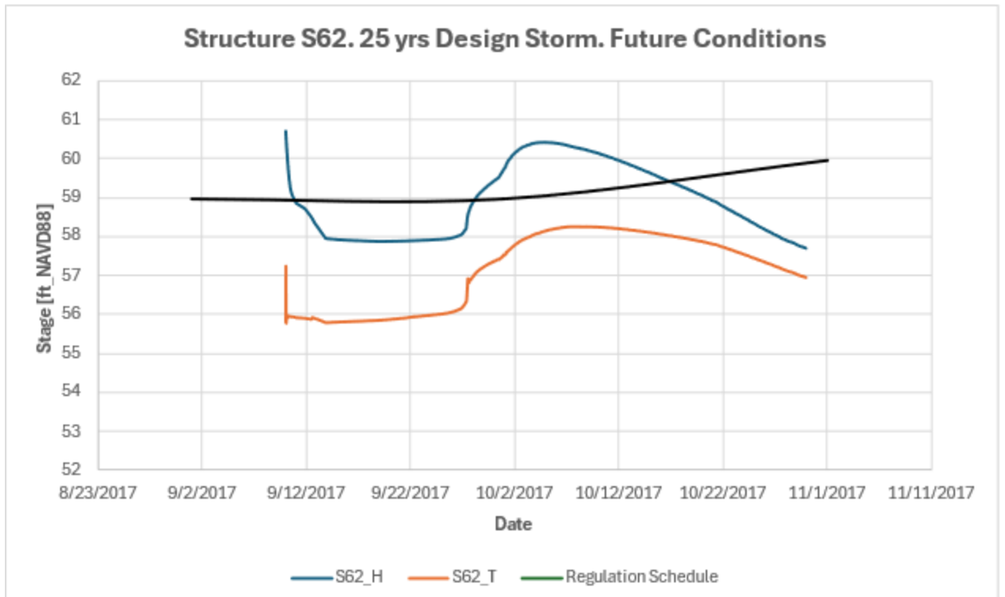


Figure 10-73: Headwater and Tailwater at Structure S62 vs. Lake Hart Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







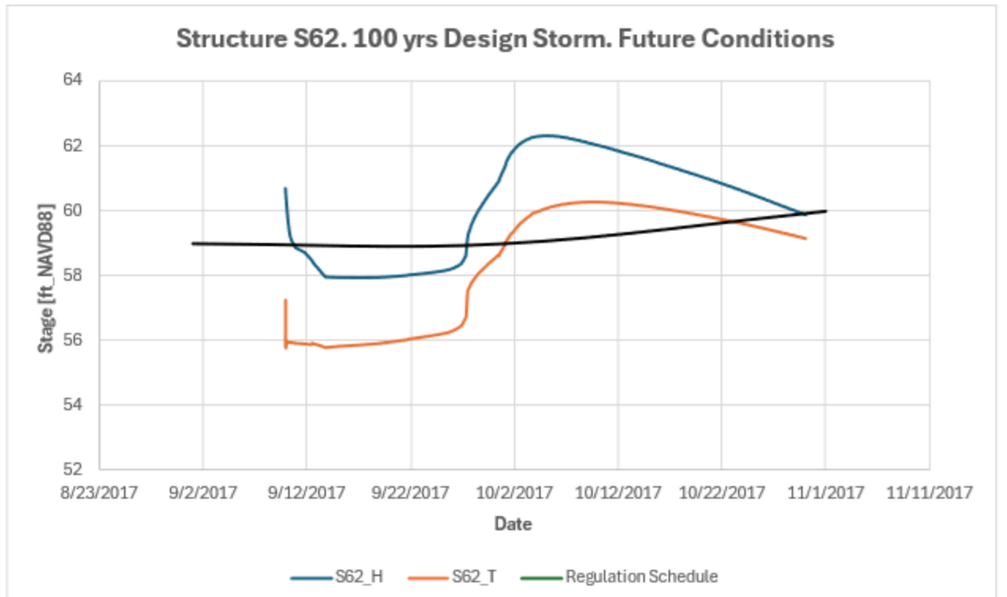


Figure 10-74: Headwater and Tailwater at Structure S62 vs. Lake Hart Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Future Condition

10.4 Lake Gentry LMA

Figure 10-75 shows Lake Gentry LMA with the primary conveyance system features and roadways.

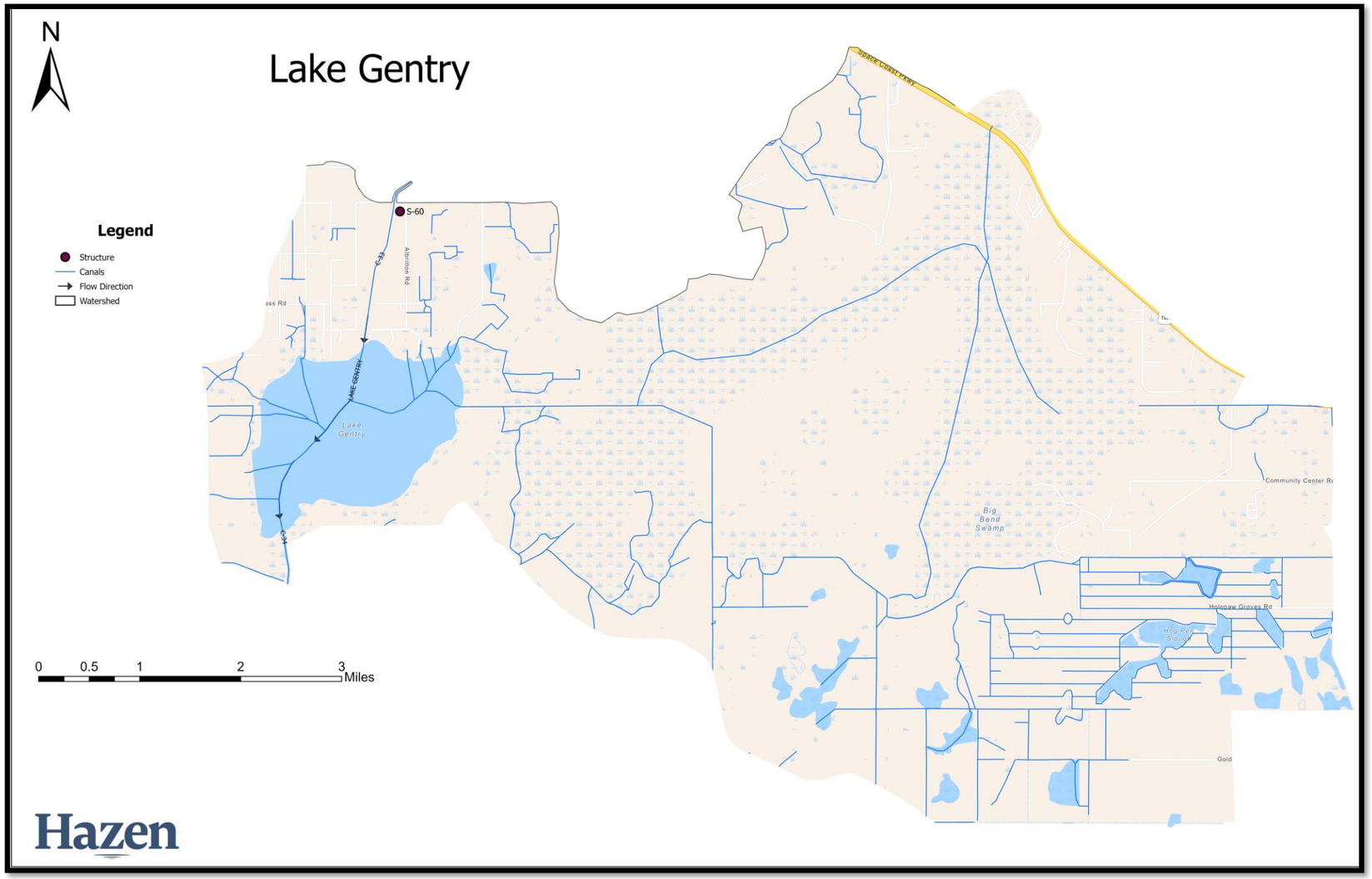


Figure 10-75: Lake Gentry LMA with Main Structures, Waterways And Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.4.1 PM #1

Performance of the system at Lake Gentry LMA was assessed at Lake Gentry and C-34 canal. The profiles are shown in **Figure 10-76 through Figure 10-79** for Lake Gentry and Canal C-34 respectively under current and future conditions.

For Lake Gentry, under current conditions, the left and right banks generally remain within the bank limits across most storm events. However, during the 100-year event, there are some exceedances, particularly on the right bank in the middle sections of the lake. The smaller storm events, such as the 5-, 10-, and 25-year events, remain well below the bank elevations, except in a few locations where they exceed the right bank mostly. In Canal C-34, all design storms exceed both banks, particularly up to the S-63A Gate and bypass, and 100-year storms in all areas.

Under future conditions, for Lake Gentry, both the left and right banks stay within limits for all events. In Canal C-34, the 100-year event causes exceedances for both banks and smaller storms exceeding both banks after the S-63A structure bypass. Overall, current conditions show slightly higher water levels than future conditions.

To summarize, under current and future conditions, portions of Lake Gentry banks are overtopped for 5-, 10- and 25-yr storms, and almost the entire length of the banks are overtopped for the 100-yr storm. For all the design storms, water levels exceed Canal C-34 banks.

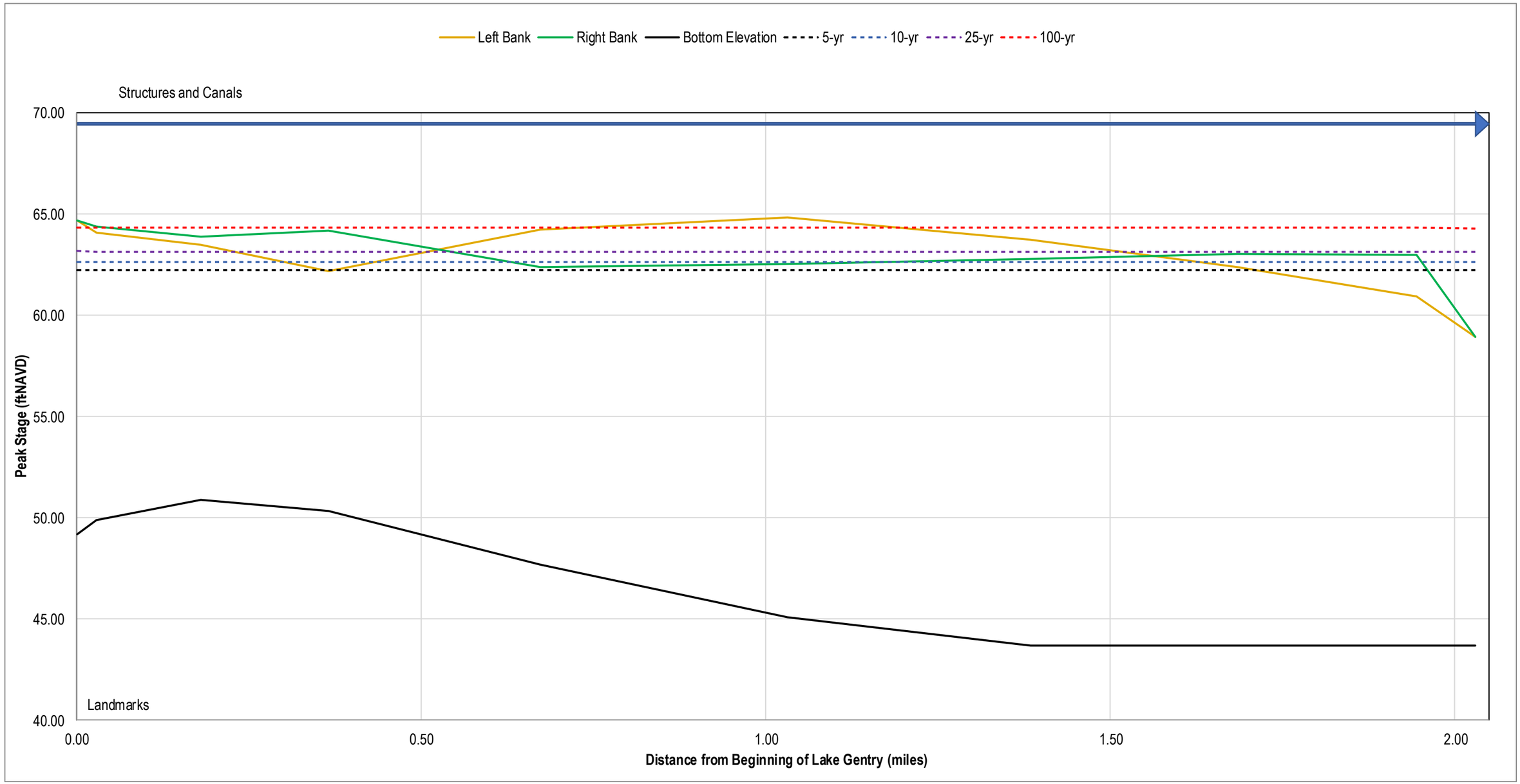


Figure 10-76: Maximum Stage Profile for Lake Gentry under Current Conditions

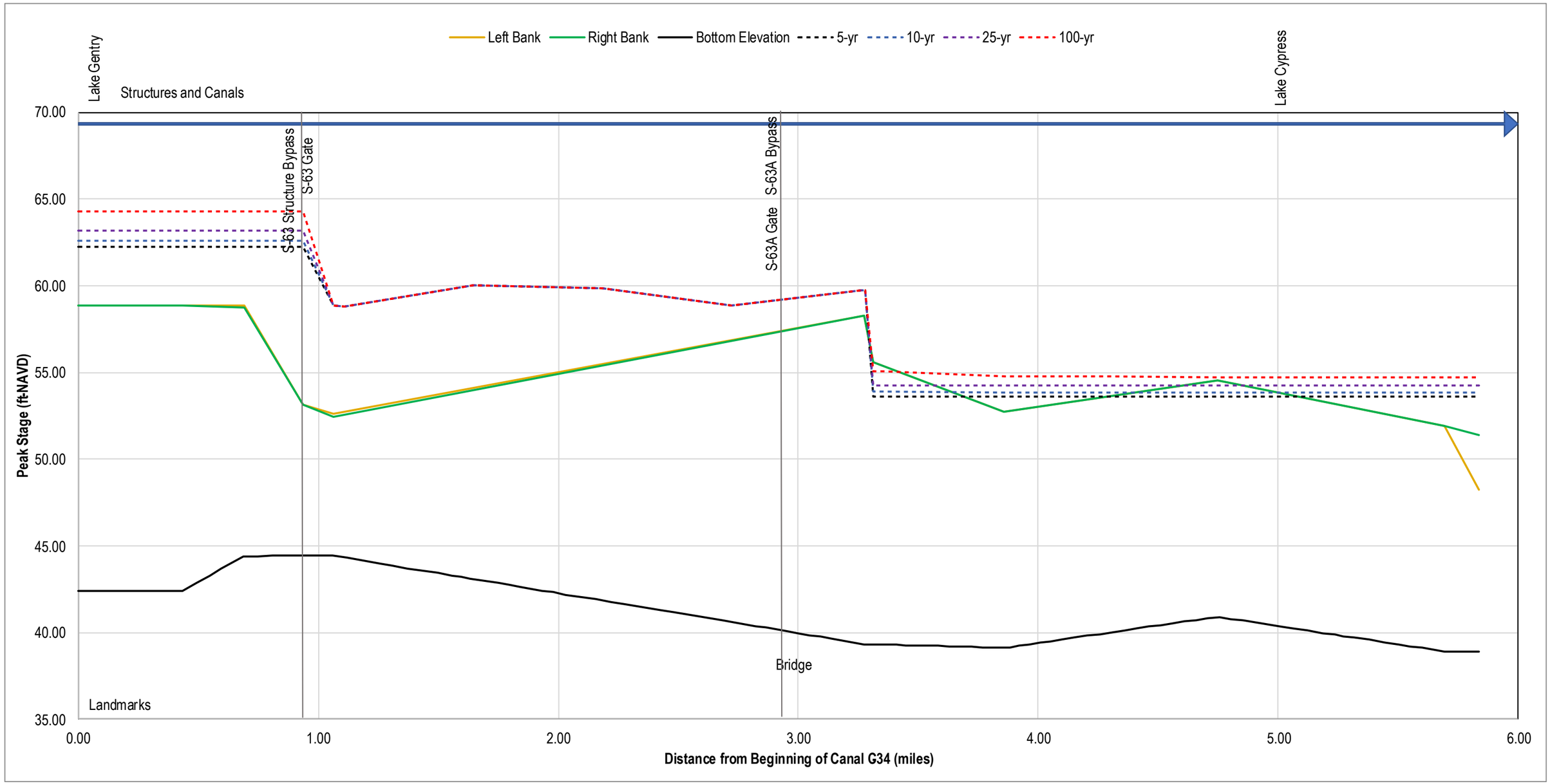


Figure 10-77: Maximum Stage Profile for Canal C-34 under Current Conditions

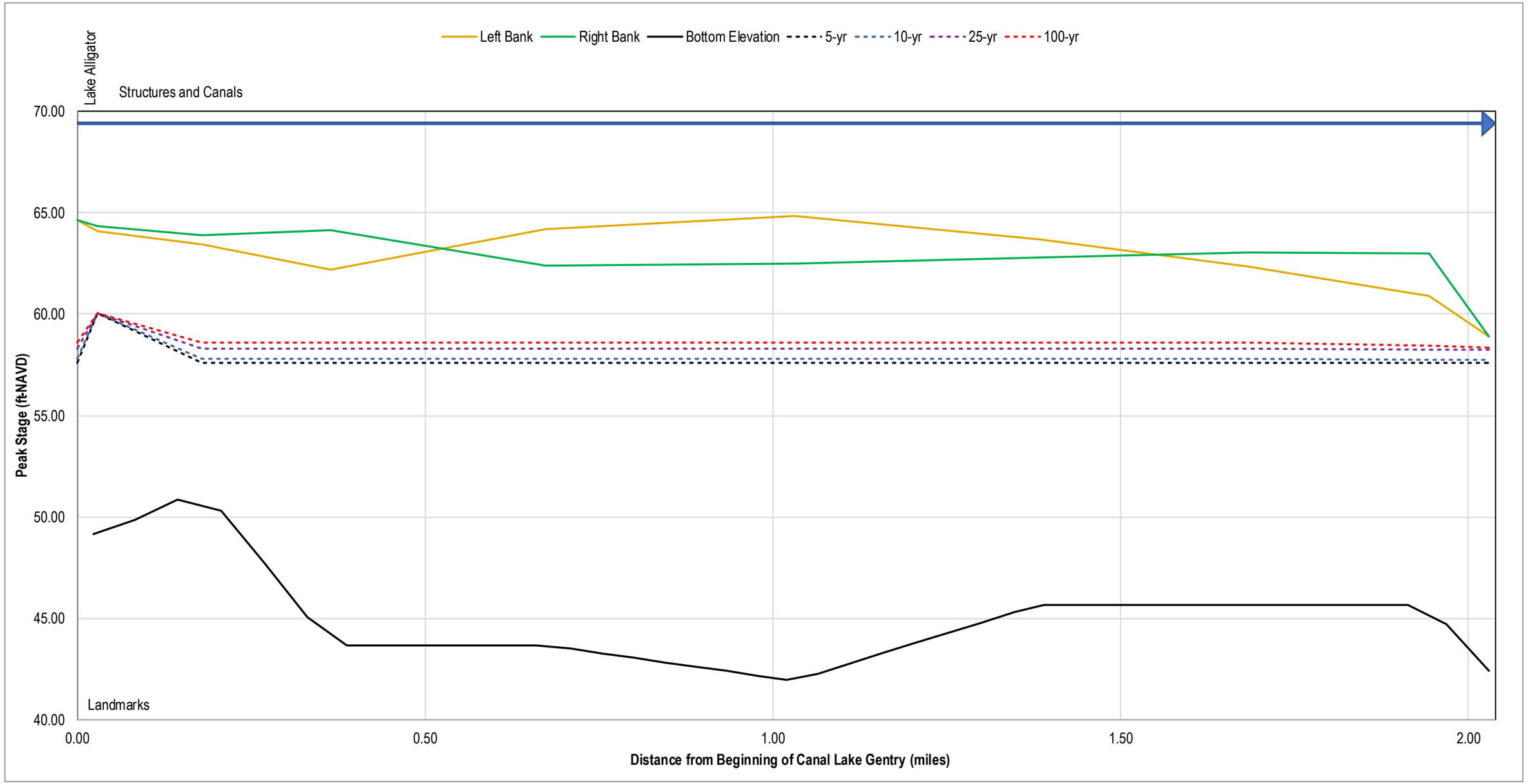


Figure 10-78: Maximum Stage Profile for Lake Gentry under Future Conditions

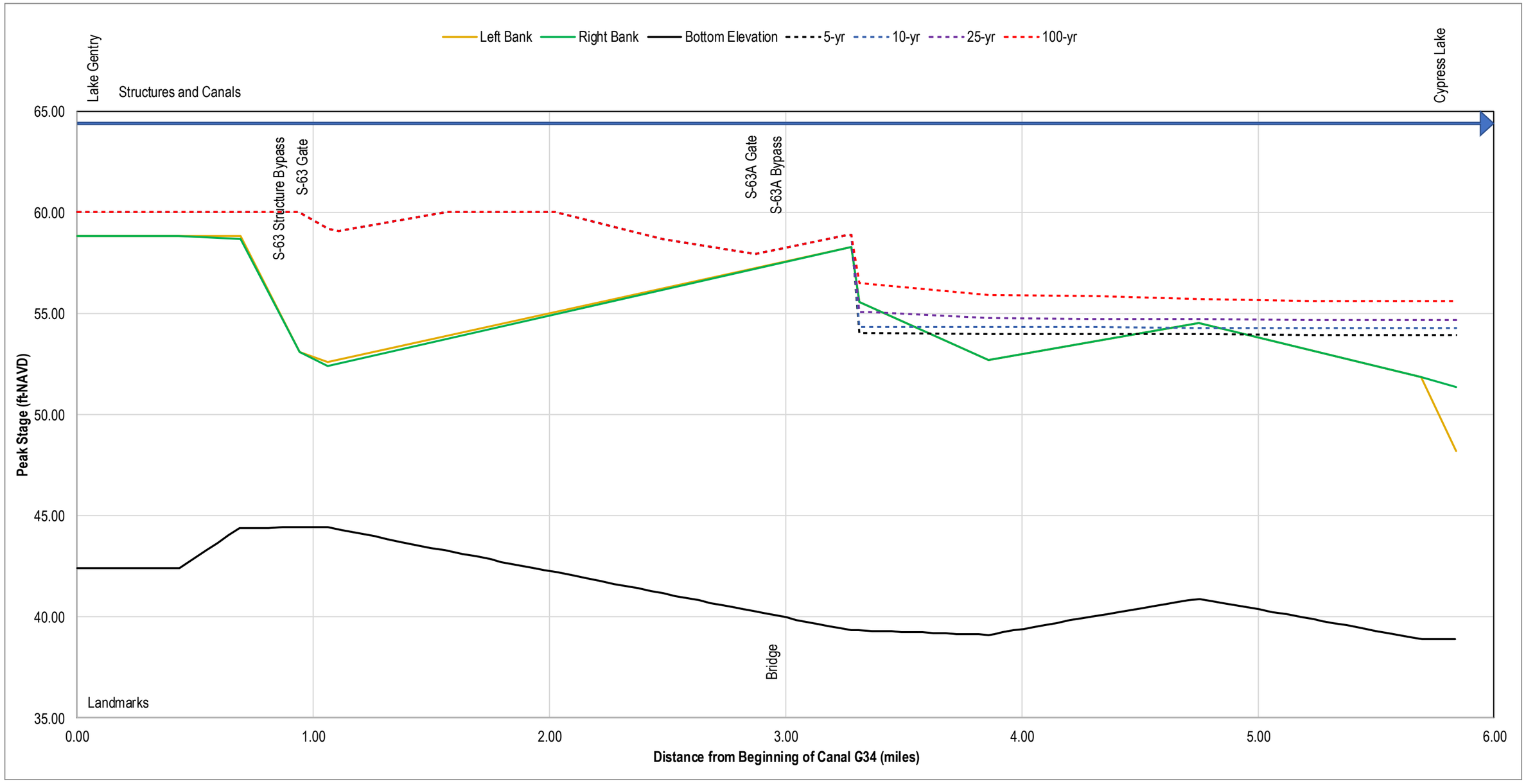


Figure 10-79: Maximum Stage Profile for Canal C-34 under Future Conditions

10.4.2 PM #2

Lake Gentry LMA's upstream water control structure is S60, and downstream structure is S63A. The maximum daily discharge capacity of this LMA is determined by S63A discharge capacity. Per the Districts ERP Handbook, Lake Gentry LMA has an allowable discharge rate of 13.8 CSM, based on a design frequency of 10-yrs. **Table 10-44** summarizes Lake Gentry LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area; and **Table 10-45** provides that information for Lake Gentry LMA under future conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 6.9 CSM and 17.3 CSM under current and future conditions respectively, which means current condition is lower than SFWMD's permissible discharge allowance of 13.8 CSM based on a 10-yr design frequency, but future conditions are higher.

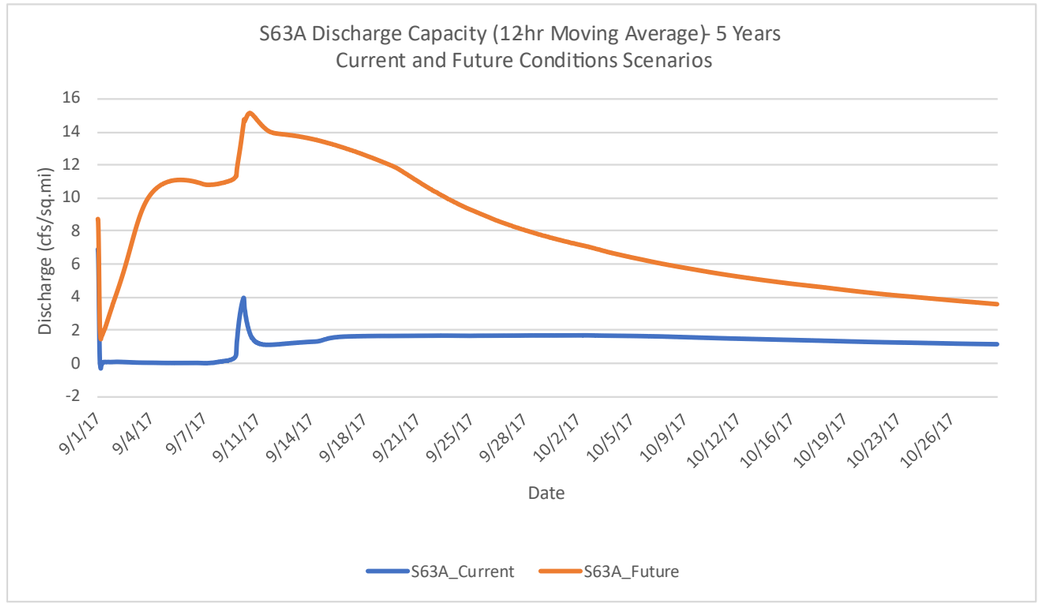
Table 10-44: Drainage Discharge Capacity (cfs/sq. mi) of Lake Gentry LMA under Current Conditions

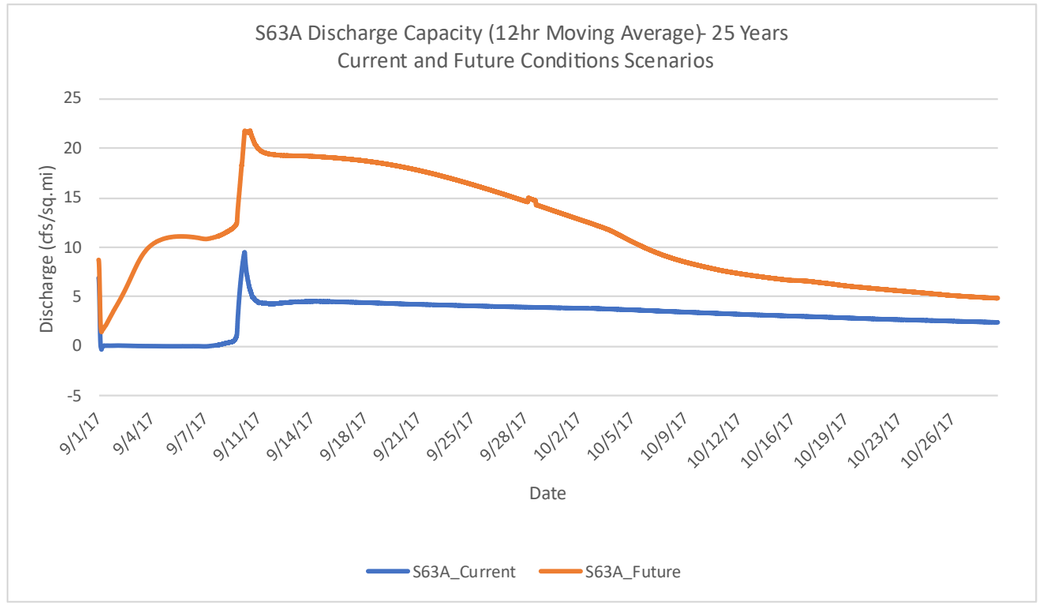
LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Gentry	50.69	6.9	6.9	9.5	20.0	13.8

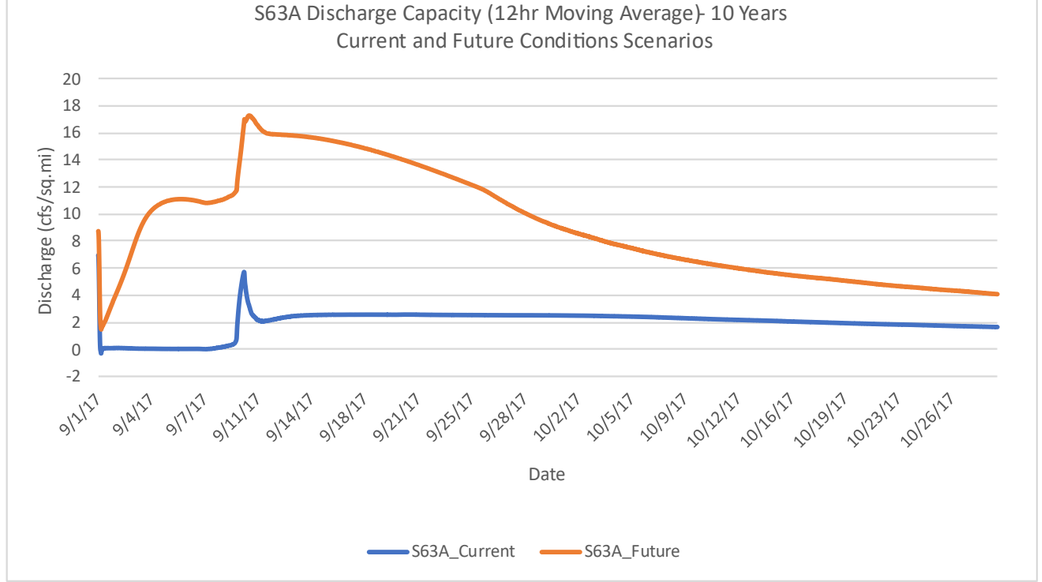
Table 10-45: Drainage Discharge Capacity (cfs/sq. mi) of Lake Gentry LMA under Future Conditions

LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Gentry	50.69	15.2	17.3	21.8	54.0	13.8

Figure 10-80 shows the peak 12-hour average discharge per square mile of drainage area, calculated for each design storm at structure S63A; and **Figure 10-81** shows the maximum daily discharge through structure S63A for both current and future conditions. The moving averages are consistent with the numbers in the table.







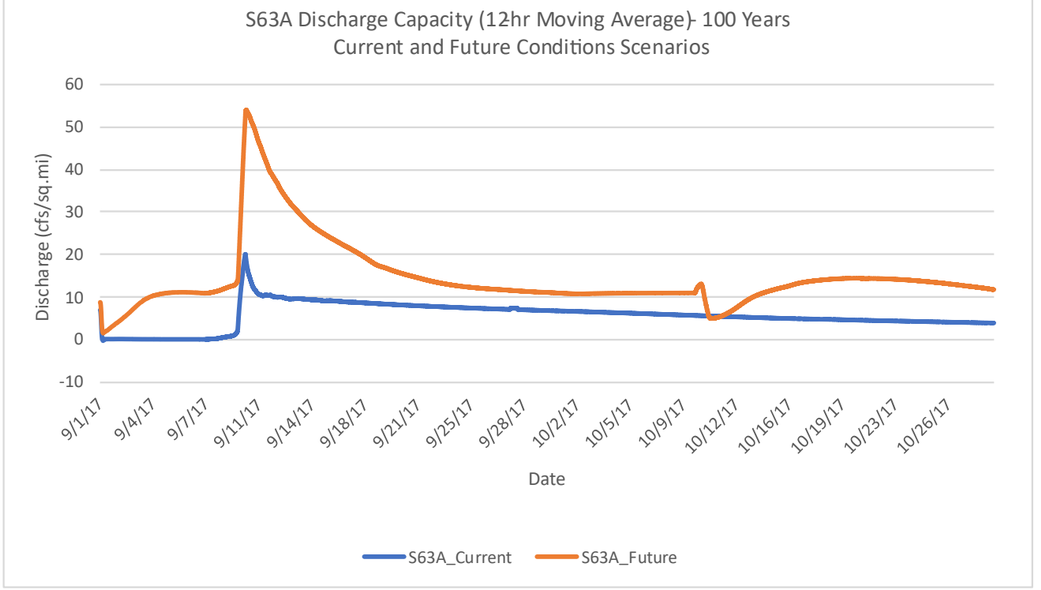


Figure 10-79: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S63A for 5-, 10-, 25-, 100- Year Design Storms

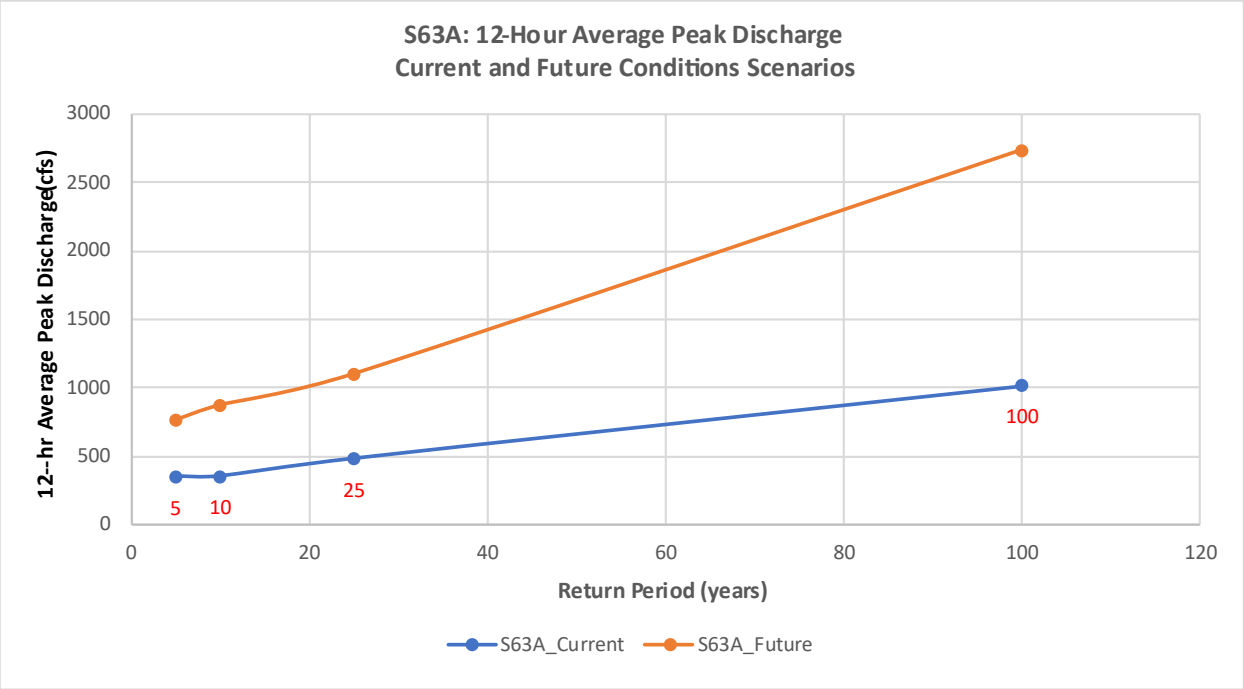


Figure 10-80: Structure S63A 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

10.4.3 PM #3

The SFWMD structure design parameters (**Table 10-46**) were used to evaluate the S63A performance against the various design storms; **Table 10-47** shows the design parameters for Structure S63. **Table 10-48 through Table 10-53** provides a summary of current conditions and future conditions simulated instantaneous stages and flows at structure S63A at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S63A, the simulated headwater elevation at S63A for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is lower than the design headwater elevation for current conditions. For future conditions the headwater levels are consistently higher for all design storms. The simulated flow at structure S63A is less than the design discharge (2000 cfs) for all design storm events in the current and future conditions.

During the peak headwater at the structure S63A, the simulated tailwater elevation at S63A is higher than design tailwater elevation for all design storm events for both current conditions and future conditions. The simulated flows at structure S63A are higher than the design discharge (2000 cfs) for all design storm events except for 5-yr design event simulation.

During the peak flow at Structure S63A, the headwater elevations are lower than the design headwater for all design storm events for current and future conditions. However, the simulated tailwater elevations are always higher than design elevation for all design storms in future conditions and 3 of 4 design storms in current conditions. **Figure 10-82 and Figure 10-83** show the same as time series.

Table 10-46: Design Parameters for Structure S63A

Structure	S63A
Design Discharge	870 to 2,000 cfs (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	55.87 feet NAVD88 (57.0 feet NGVD29)
Design Tailwater Elevation	53.3 (52.17) to 53.8 (52.67 feet NAVD88) feet NGVD29
SPF Headwater Elevation	59.87 feet NAVD88 (61.0 feet NGVD29)
SPF Tailwater Elevation	57.17 feet NAVD88 (58.3 feet NGVD29)
Optimum Headwater	55.37 feet NAVD88 (56.5 feet NGVD29)
Optimum Tailwater	47.37 to 51.37 feet NAVD88 (48.5 to 52.5 feet NGVD29)

Table 10-47: Design Parameters for Structure S63

Structure	S63
Design Discharge	715 cfs (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	59.41 to 61.71 feet NAVD88 (60.5 to 62.8 feet NGVD29)
Design Tailwater Elevation	56.41 to 56.61 feet NAVD88 (57.5 to 57.7 feet NGVD29)
SPF Headwater Elevation	65.11 feet NAVD88 (66.2 feet NGVD29)
SPF Tailwater Elevation	60.11 feet NAVD88 (61.2 feet NGVD29)
Optimum Headwater	58.91 to 60.91 feet NAVD88 (60 to 62 feet NGVD29)
Optimum Tailwater	55.91 feet NAVD88 (57 feet NGVD29)

Table 10-48: Summary of the Instantaneous Conditions at Structure S63A at Time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak HW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-yr	53.58	53.58	61.99	0.00	9/13/17 6:25 PM
10-yr	53.88	53.87	128.90	0.00	9/14/17 10:25 AM
25-yr	54.25	54.24	230.82	0.01	9/15/17 1:25 PM
100-yr	55.05	55.04	497.50	0.01	9/14/17 2:25 PM

Table 10-49: Summary of the Instantaneous Conditions at Structure S63A at Time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-year	54.04	54.02	694.77	0.02	9/14/17 12:00 PM
10-year	54.38	54.36	794.18	0.02	9/15/17 6:25 AM
25-year	55.11	55.09	976.21	0.02	9/14/17 11:25 PM
100-year	56.70	56.50	2496.11	0.20	9/11/17 7:00 AM

Table 10-50: Summary of the Instantaneous Conditions at Structure S63A at Time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at peak TW (ft)	Date & Time
5-year	53.58	53.58	61.99	0.00	9/13/17 6:25 PM
10-year	53.87	53.88	128.90	0.00	9/14/17 10:25 AM
25-year	54.24	54.25	230.73	0.01	9/15/17 4:25 PM
100-year	55.04	55.05	483.38	0.01	9/14/17 5:25 PM

Table 10-51: Summary of the Instantaneous Conditions at Structure S63A at time of Peak Tailwater under Future Conditions

Instantaneous values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at peak TW (cfs)	Head Differential at peak TW (ft)	Date & Time
5-year	54.02	54.04	694.77	-0.02	9/14/17 12:00 PM
10-year	54.36	54.38	793.11	-0.02	9/15/17 9:25 AM
25-year	55.09	55.11	976.21	-0.02	9/14/17 11:25 PM
100-year	56.50	56.70	2496.11	-0.20	9/11/17 7:00 AM

Table 10-52: Summary of the Instantaneous Conditions at Structure S63A at time of Peak Discharge under Current Conditions

Instantaneous values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at peak Q (ft)**	**Date & Time**
5-year	559.9	52.60	52.59	0.01	9/10/17 1:00 PM
10-year	685.41	52.88	52.87	0.02	9/10/17 1:00 PM
25-year	905.03	53.37	53.34	0.03	9/10/17 1:00 PM
100-year	2084.48	55.12	55.00	0.12	9/10/17 3:00 PM

Table 10-53: Summary of the Instantaneous Conditions at Structure S63A at time of Peak Discharge under Future Conditions

Instantaneous values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at peak Q (ft)**	**Date & Time**
5-year	878.78	53.88	53.86	0.02	9/10/17 12:40 PM
10-year	1070.64	54.02	54.00	0.02	9/10/17 12:40 PM
25-year	1577.66	54.34	54.31	0.03	9/10/17 12:30 PM
100-year	2827.027	55.87	55.59	0.28	9/10/17 2:20 PM

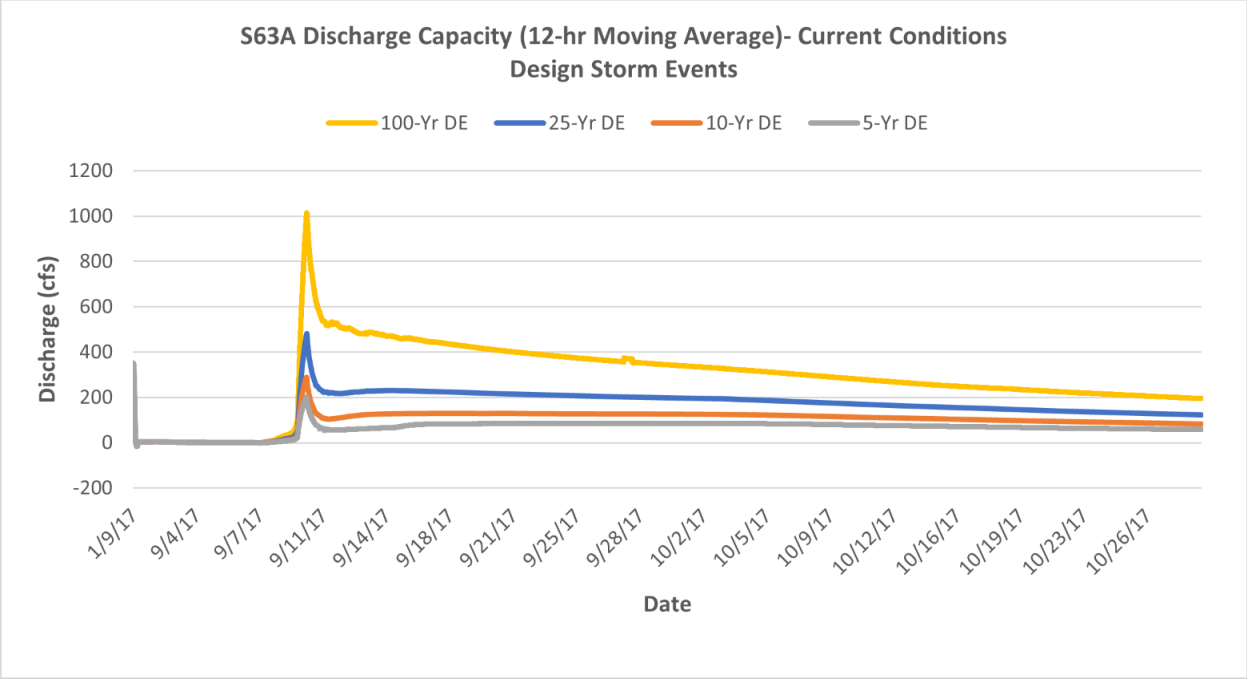


Figure 10-82: 12-hr Moving Average Flows in cfs at S63A under Current Conditions

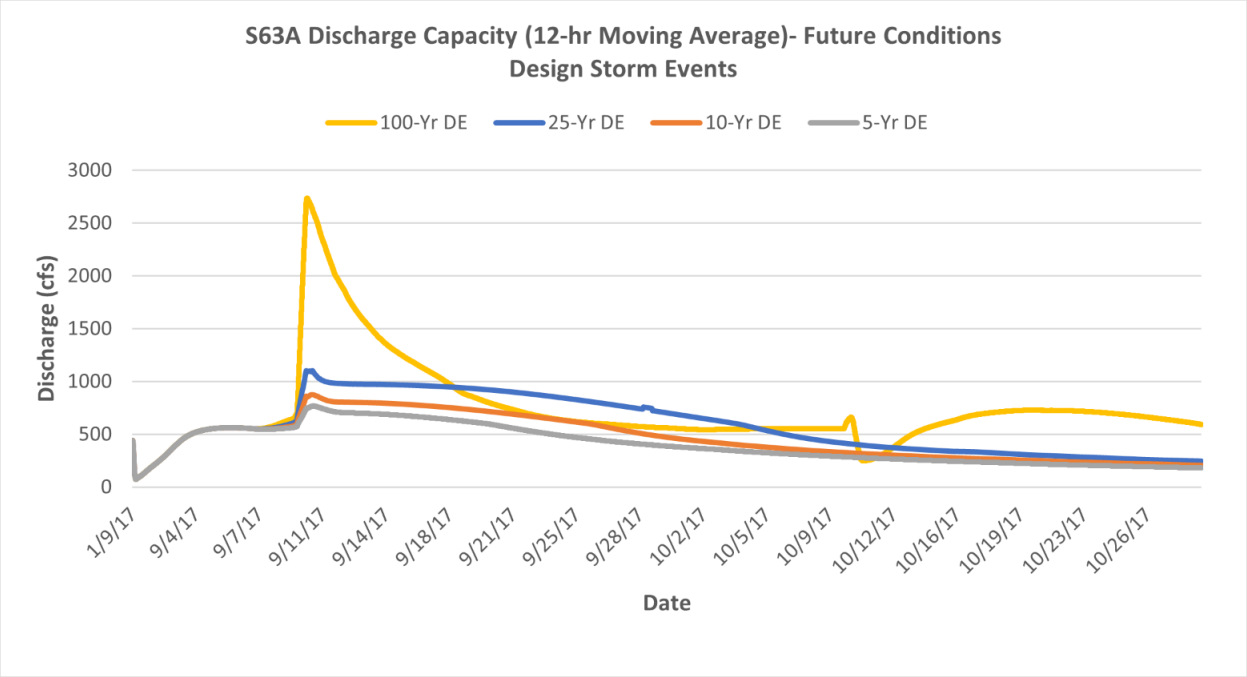


Figure 10-83: 12-hr Moving Average Flows in cfs at S63A under Future Conditions

10.4.4 PM #4

PM#4-UKB was not assessed for this LMA.

10.4.5 PM #5

The gridded data of maximum overland depth was extracted for each of the design storms under current and future conditions and evaluated for Lake Gentry LMA under current and future conditions. **Table 10-54** shows the flood inundation area in square miles for this LMA, tabulated in incremental intervals of feet and **Figure 10-84** shows the distribution graphically. **Figure 10-85 through Figure 10-86** show the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions for Lake Gentry LMA.

Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not significant. There is also a high distribution of flooding depths between 0.5 and 1.5 feet for 5- and 10-yr storms, and between 0.5 and 2 feet for 25- and 100-yr storms. Assessment of this shows these are in Gentry Ditch and Big Bend Swamp, both to the east of Lake Gentry. The flow paths from these swamps following storms have been well documented, observed; this is evident in the simulated overland flow maps in the flow patterns and the direction of the sheet flow.

Table 10-54: Incremental Flood Inundation Area (sq. mi.) in Lake Gentry LMA

Flooding Depth (ft)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	27.39	24.05	17.88	8.83	28.09	24.80	18.66	9.42
0.25<=Depth<0.5	6.38	6.78	9.03	11.57	6.19	6.89	9.75	12.54
0.5<=Depth<0.75	5.29	5.15	4.65	4.42	4.88	4.92	4.74	5.09
0.75<=Depth<1.0	4.15	4.33	4.25	3.93	4.45	3.85	4.12	4.33
1.0<=Depth<1.25	2.39	3.39	3.54	3.71	2.94	3.94	3.15	3.78
1.25<=Depth<1.5	1.05	1.85	3.38	3.54	0.90	2.32	3.20	3.34
1.5<=Depth<1.75	0.56	1.02	2.07	2.99	0.41	0.97	2.74	2.54
1.75<=Depth<2.0	0.41	0.56	1.19	2.78	0.30	0.35	1.30	2.00
2.0<=Depth<2.25	0.16	0.37	0.57	1.98	0.17	0.17	0.34	2.39
2.25<=Depth<2.5	0.08	0.20	0.43	1.32	0.05	0.11	0.15	1.80
>2.5	2.77	2.93	3.63	5.56	2.26	2.33	2.50	3.41
Lake Gentry LMA	**32407.82**	**acres**						
	50.64	**sq.mi.**						

Note: *The lake area of 5.40 sq. mi. was included in this table computations*.

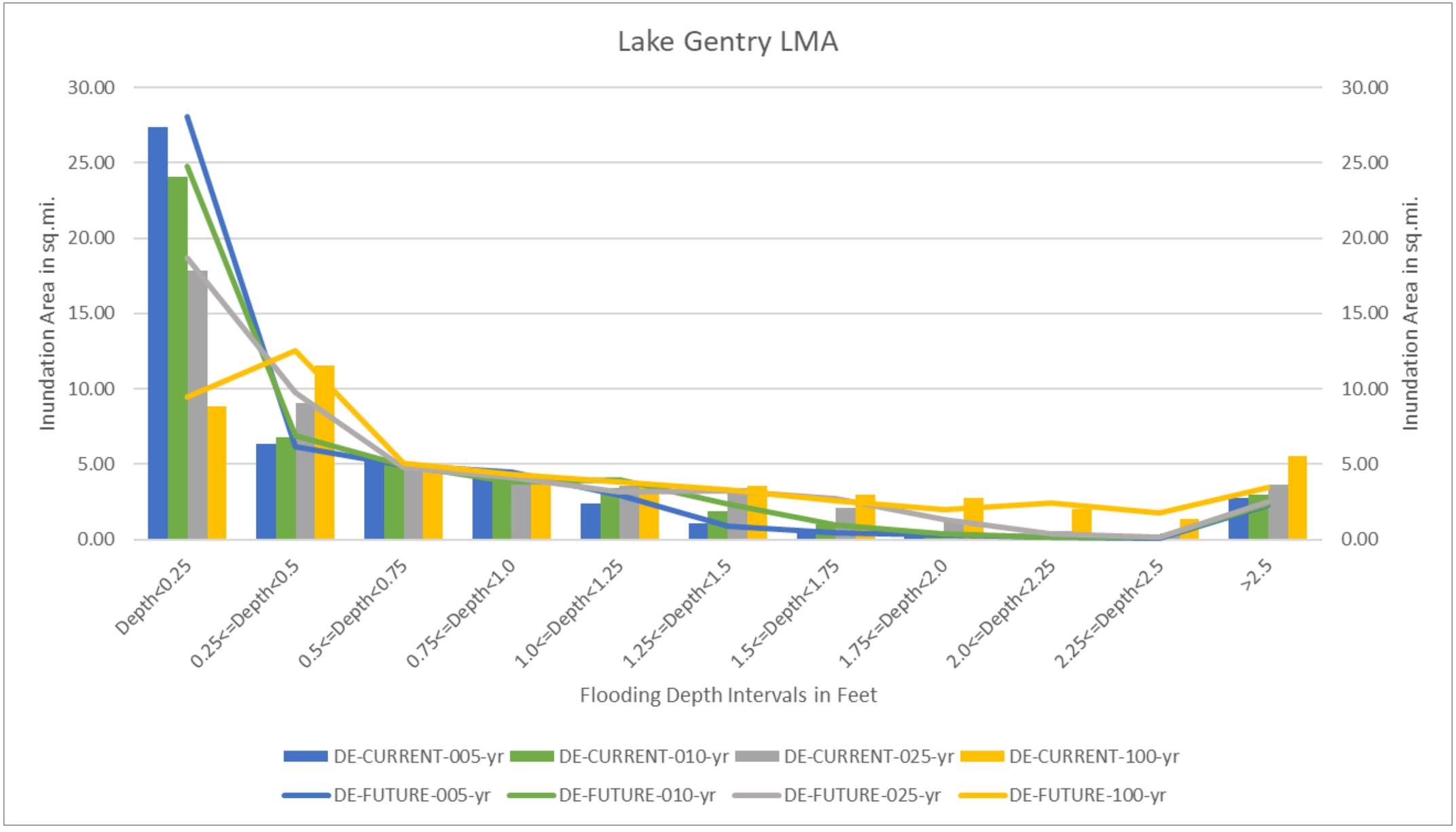


Figure 10-84: Incremental Flood Inundation Depth in feet in Lake Gentry LMA under Current and Future Conditions

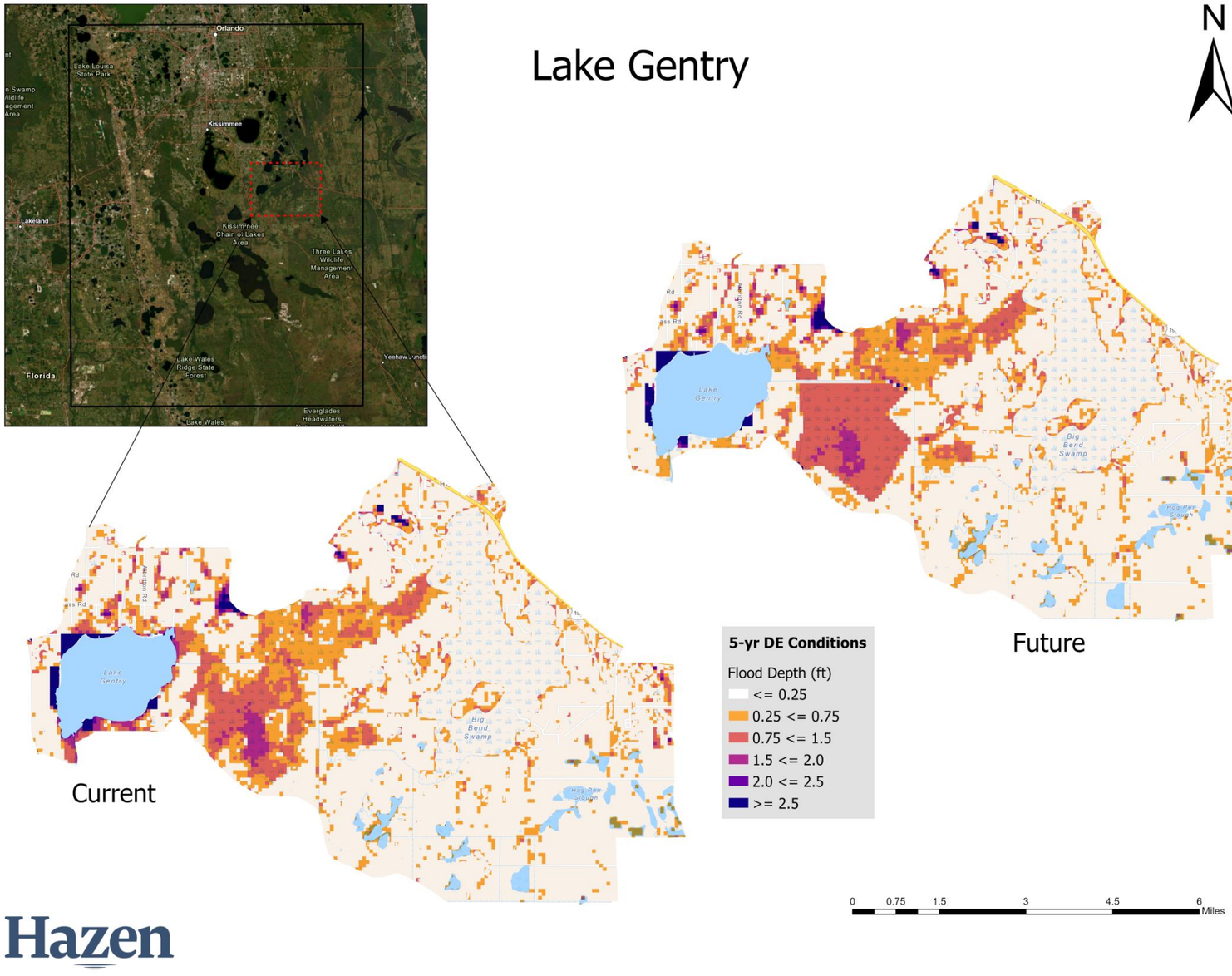


Figure 10-85: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

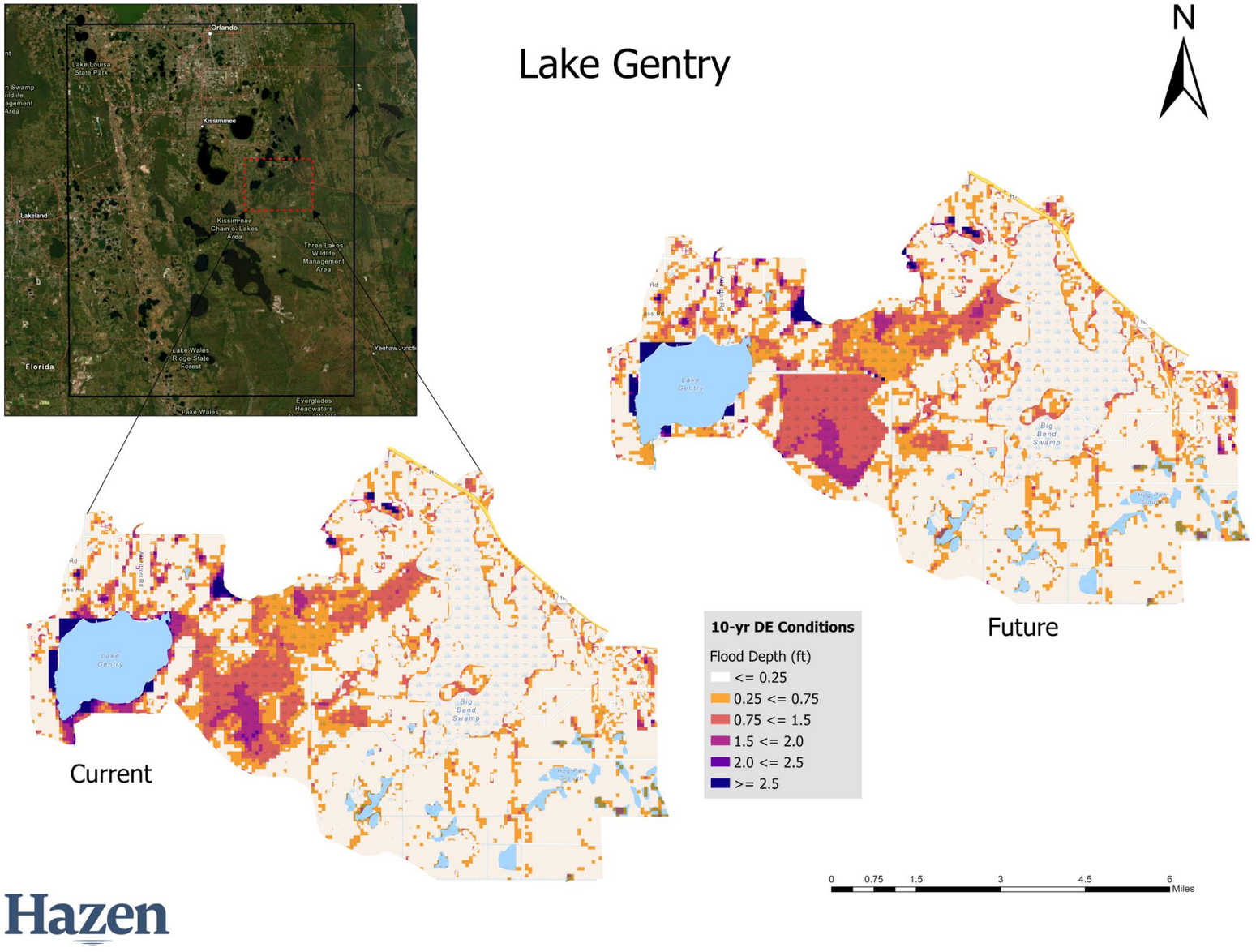


Figure 10-86: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

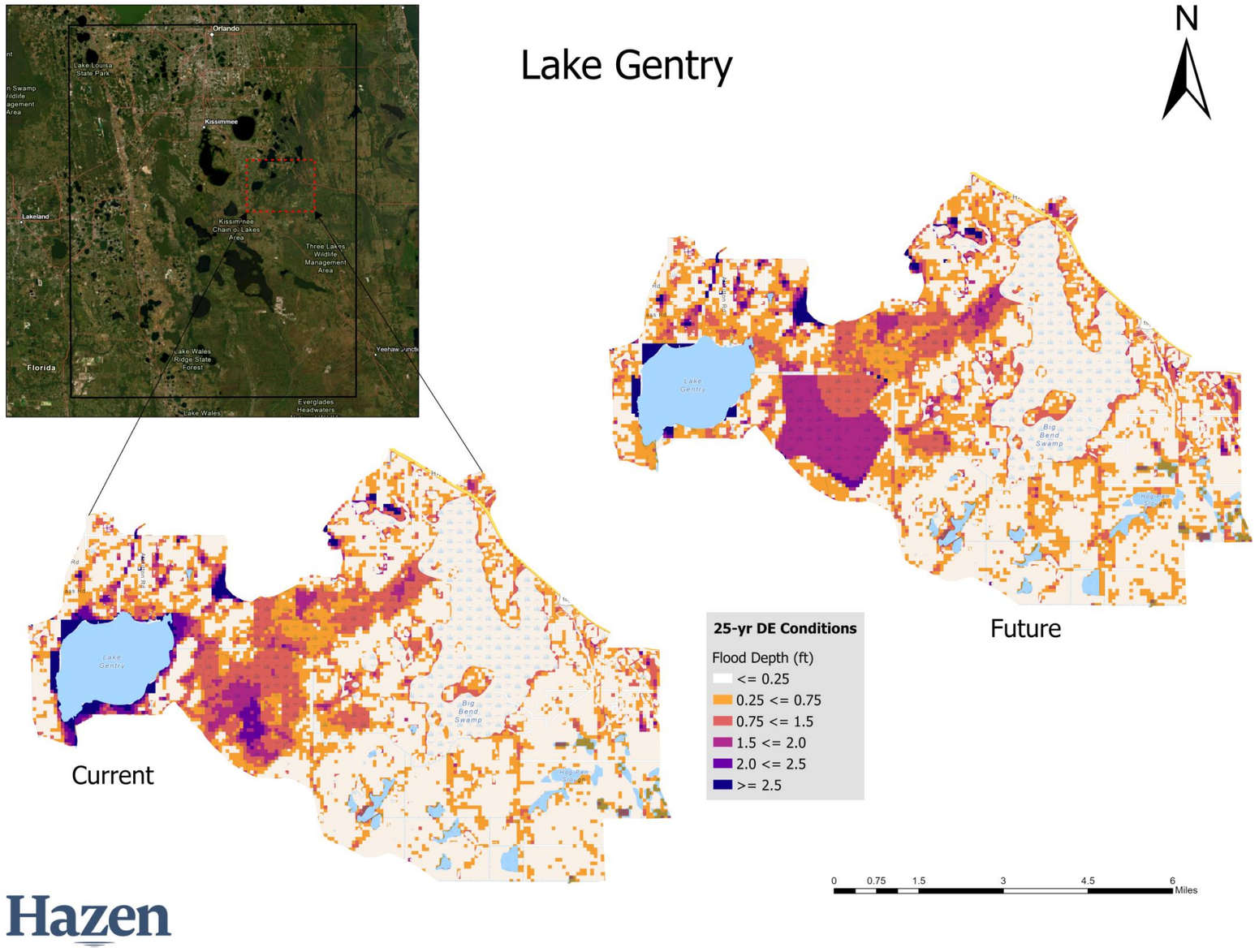


Figure 10-87: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

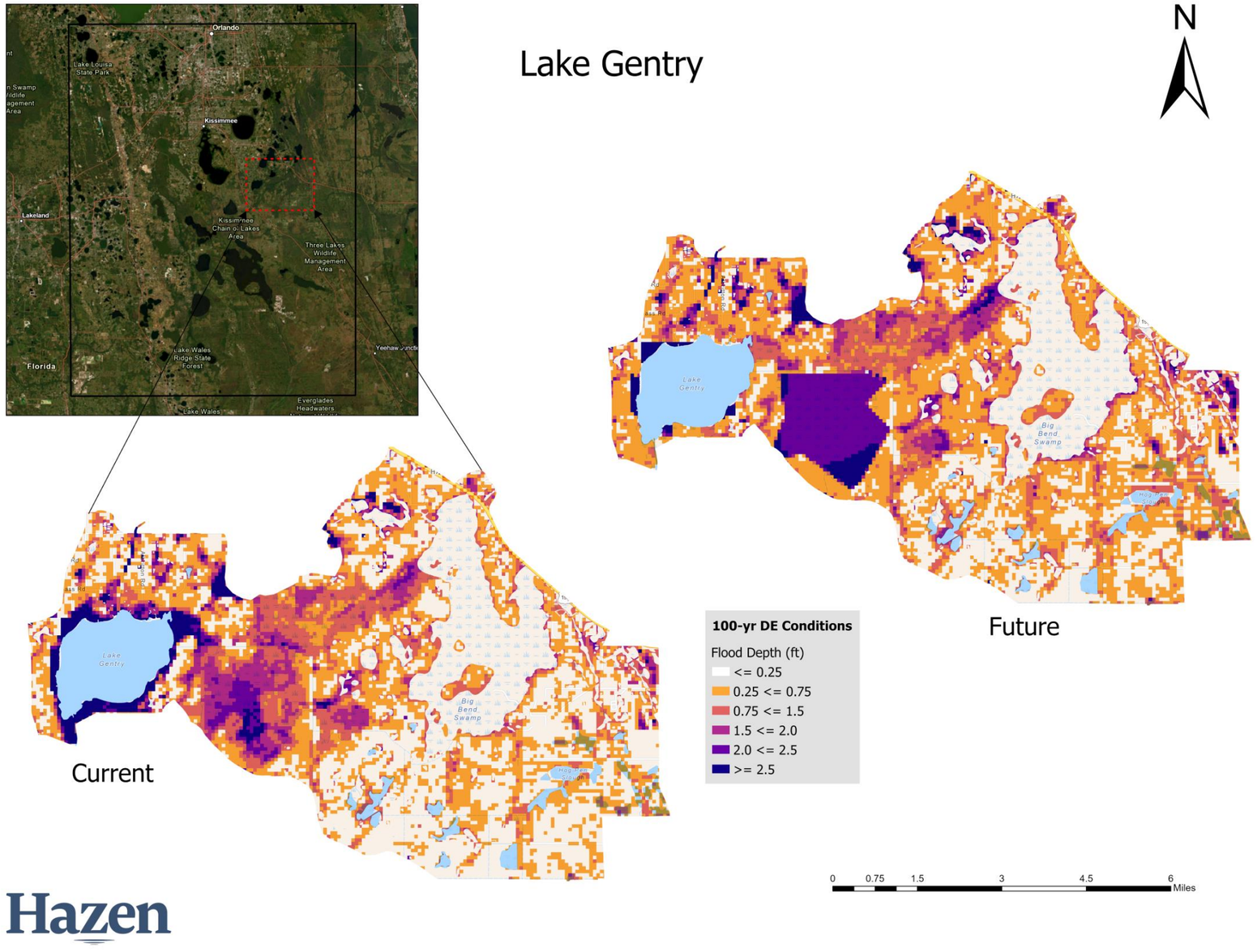


Figure 10-88: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

10.4.6 PM #6

Table 10-55 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Gentry LMA. **Figure 10-89** show these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Gentry LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-90 through Figure 10-93** show the flood duration in Lake Gentry LMA, except in Lake Gentry which is expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration In Lake Gentry LMA is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration in Lake Hart LMA are areas with longer than 360 hours of flooding, which are the lakes.

Figure 12-85 demonstrates that less than 2.5% of the LMA area will remain dry under current and future conditions. The probability that 25% or more of the LMA area will be inundated is about 16% for a 5-yr storm event and about 18% for a 100-yr storm event.

Table 10-55: Areas of Incremental Flood Duration (sq. mi.) in Lake Gentry LMA

Flooding Duration (Hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	31.03	28.89	25.72	19.59	30.48	27.94	23.64	14.94
1<=Hours<4	0.58	0.81	1.40	3.31	0.94	1.50	2.83	6.59
4<=Hours<8	0.51	0.63	0.87	1.15	0.68	0.97	1.31	1.82
8<=Hours<12	0.34	0.46	0.54	0.74	0.51	0.61	0.78	1.03
12<=Hours<24	1.11	1.24	1.72	2.17	1.27	1.58	2.12	2.62
24<=Hours<48	1.02	1.20	1.55	2.00	1.26	1.47	1.97	2.55
48<=Hours<96	1.14	1.24	1.42	1.54	1.24	1.42	1.68	2.01
96<=Hours<168	0.96	1.01	1.11	1.40	1.03	1.08	1.46	1.76
168<=Hours<240	0.78	0.76	0.85	0.93	0.82	0.81	0.98	1.13
240<=Hours<360	0.92	1.07	1.08	1.10	1.08	1.19	1.14	1.58
360<=Hours<1417	12.25	13.33	14.39	16.71	11.32	12.07	12.72	14.63
Lake Gentry LMA	**32407.82**	**acres**						
	50.64	**sq. mi.**						

Note: *The lake area of 5.40 sq. mi. was included in this table computations.*

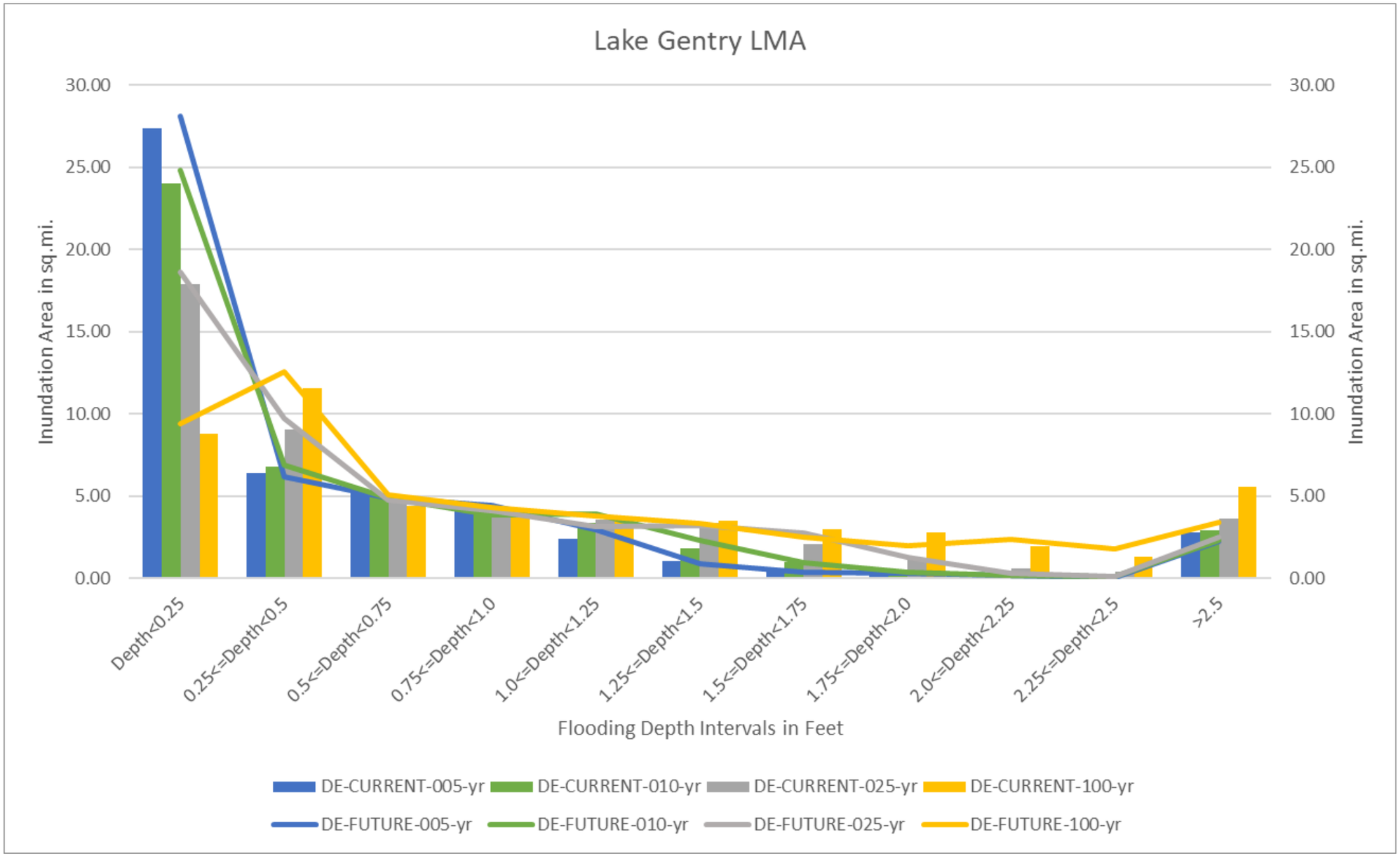
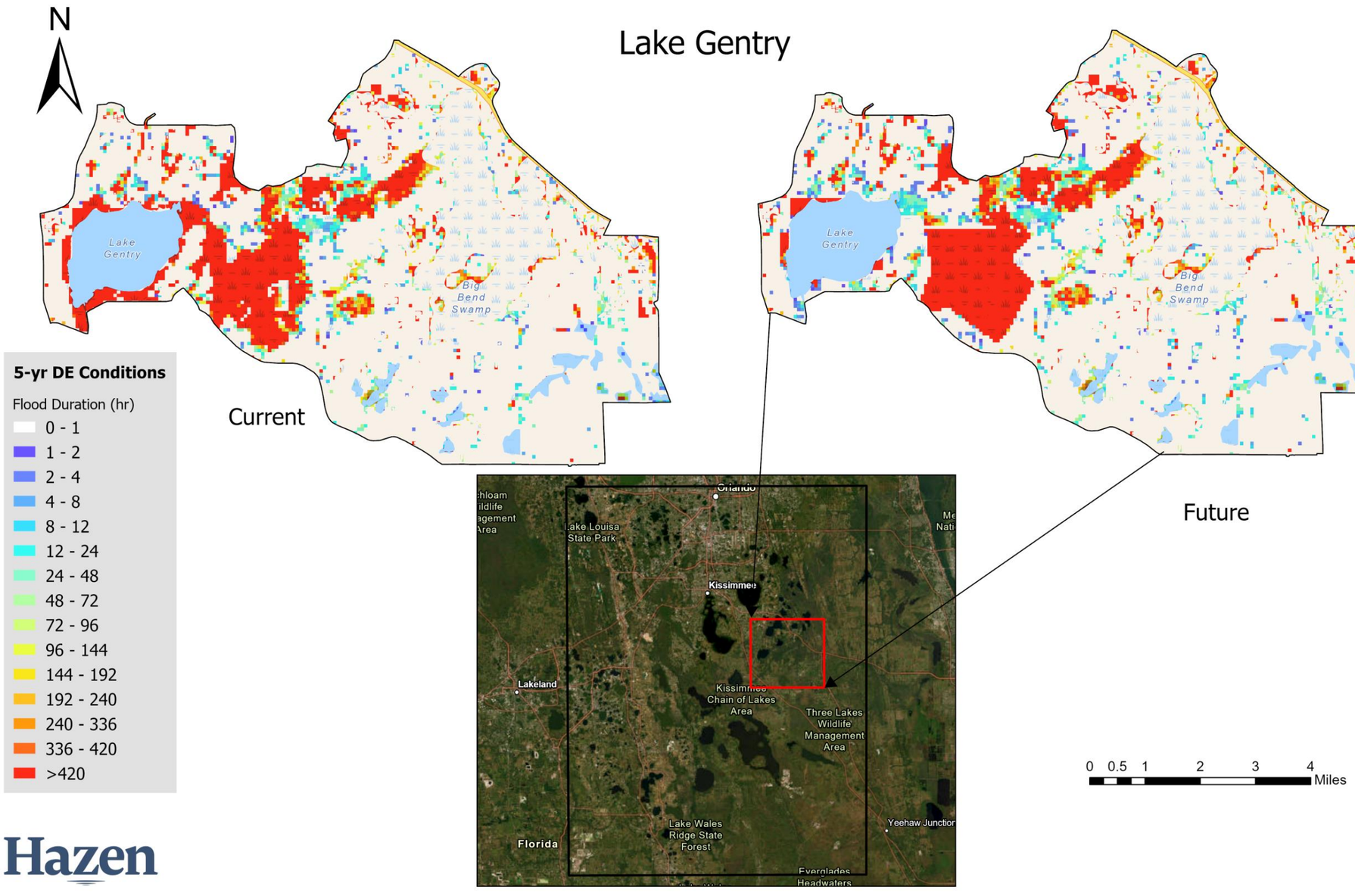
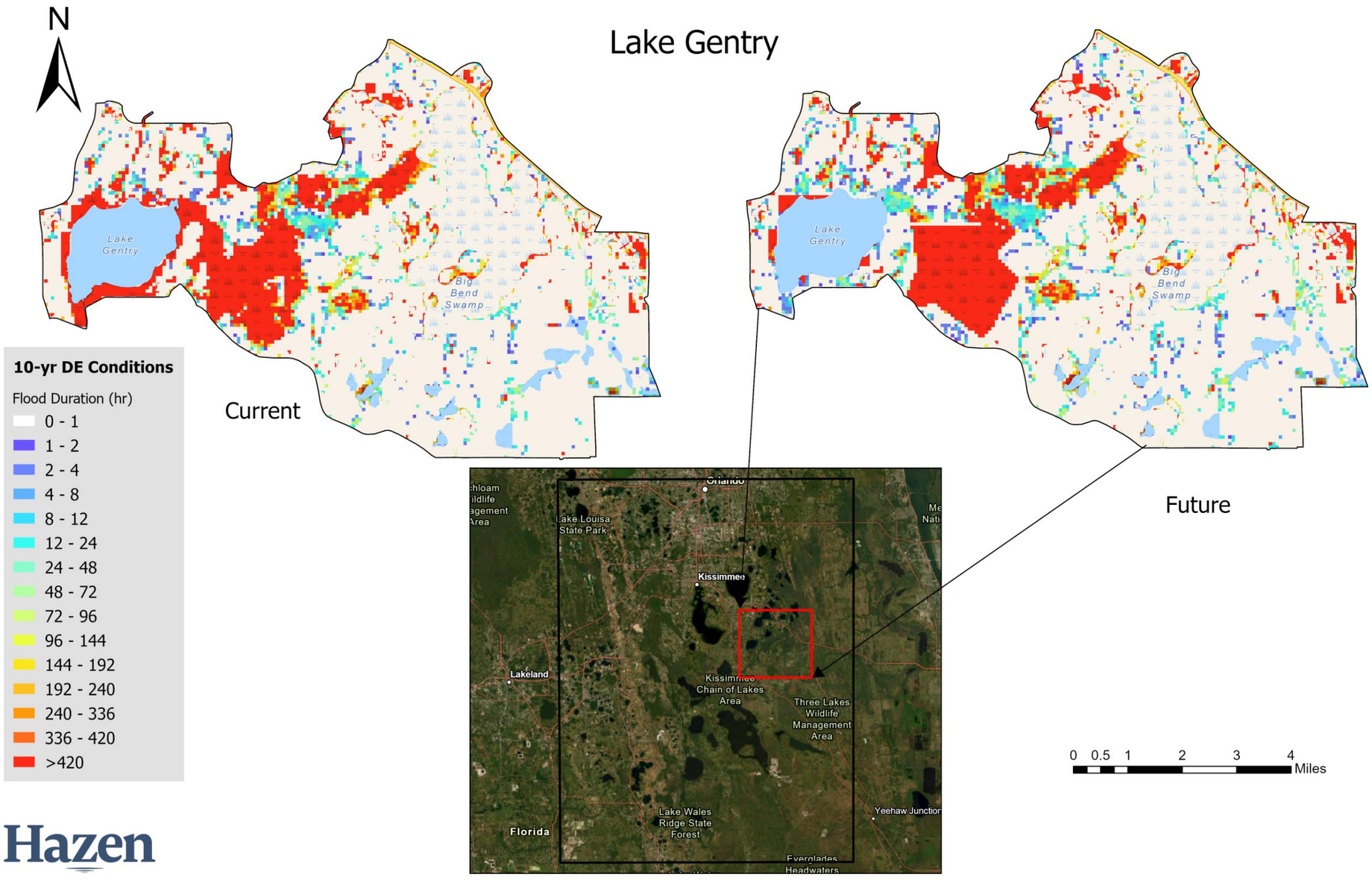


Figure 10-89: Areas of Incremental Flood Duration in Hours in Lake Gentry LMA under Current and Future Conditions



Hazen

Figure 10-90: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions



Hazen

Figure 10-91: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions

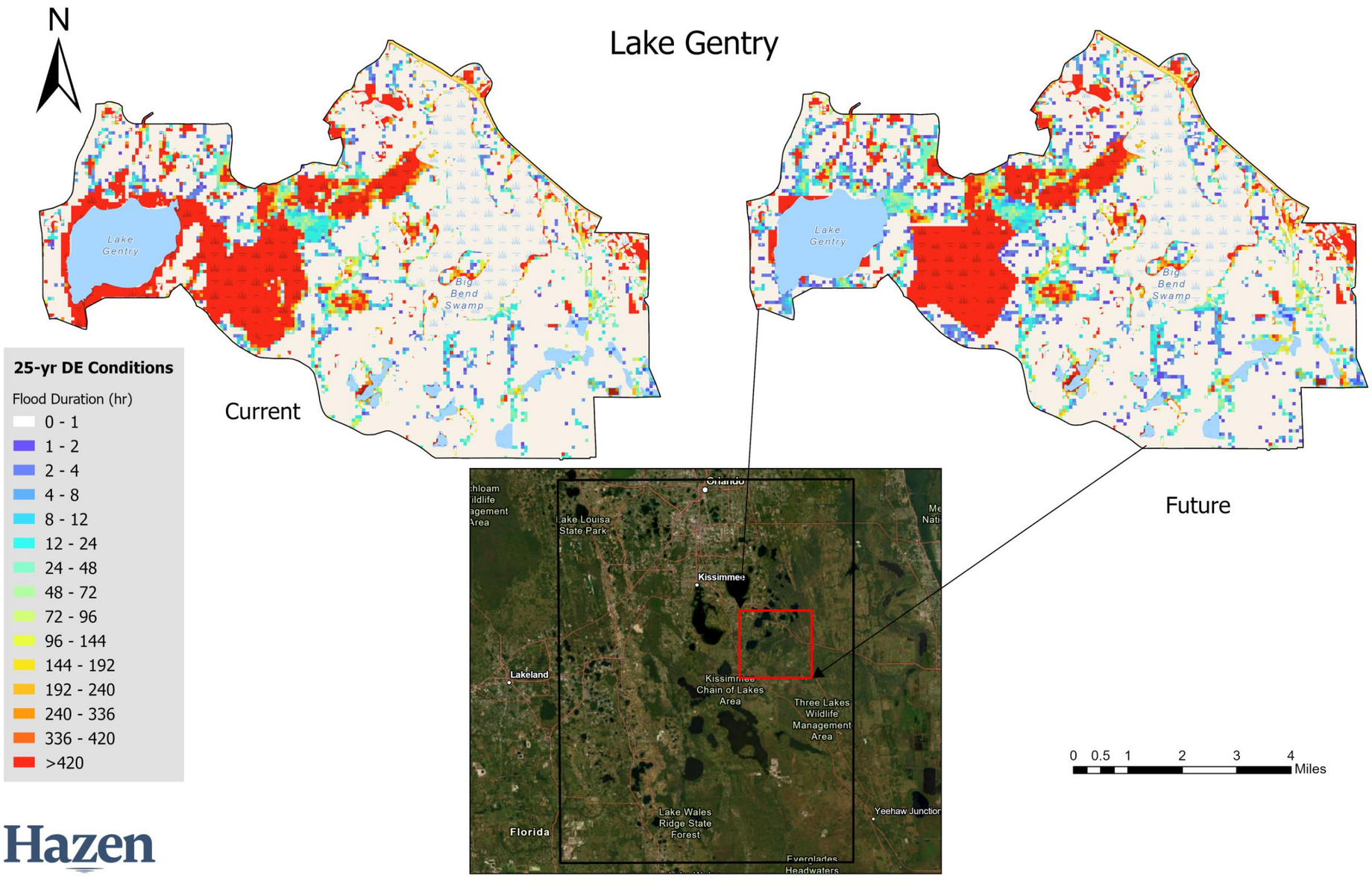


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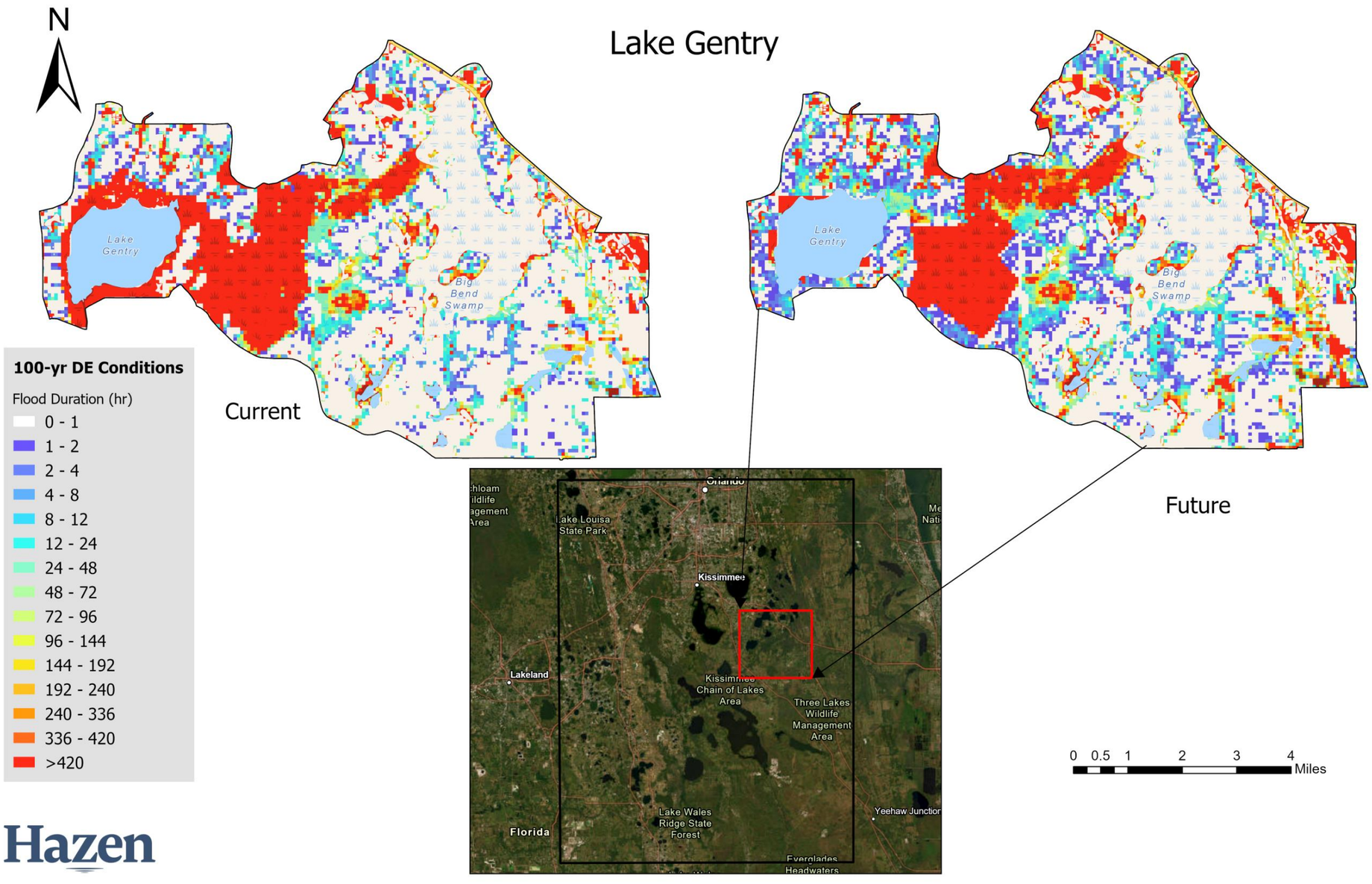
Figure 10-92: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions



Hazen

Figure 10-93: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions

10.4.7 FPLOS Rating of Lake Gentry LMA

Table 10-56 provides a summary of evaluation of Lake Gentry LMA performance using FPLOS metrics to provide flood protection under current and future conditions. Under current and future conditions, the FPLOS rating for the LMA ranges from <5-yr to 10-yr. The overall rating for this LMA is <5-yr under current and future conditions.

Table 10-56: Summary of PM Analysis in Lake Gentry LMA

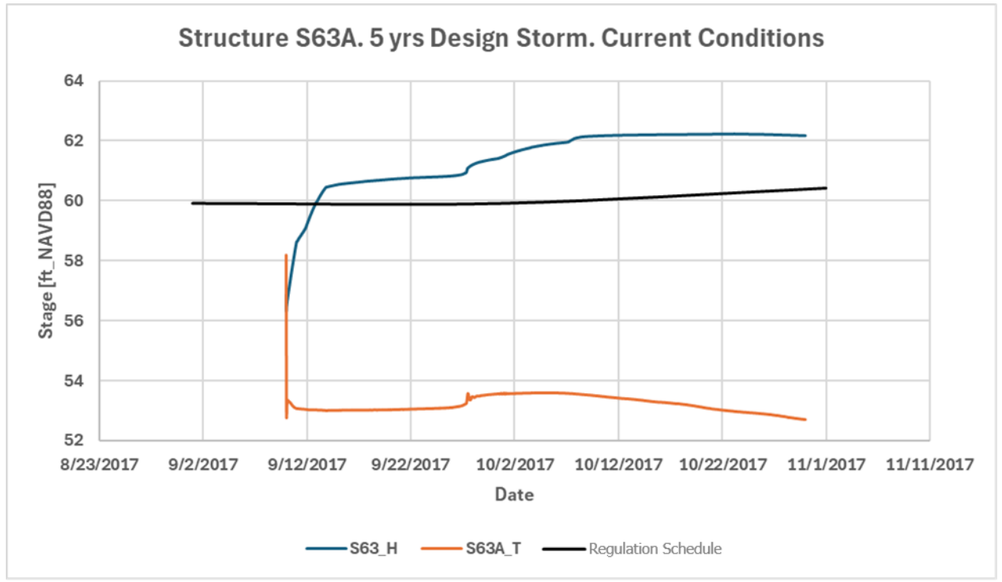
Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Gentry, C-34 canal	Portions of Lake Gentry banks are overtopped for 5-, 10- and 25-yr storms, and almost all banks are overtopped for the 100-yr storm. Water levels exceed the Canal C-34 banks for all design storms.	<5-yr in Lake Gentry <5-yr in Canal C-34	<5-yr in Lake Gentry <5-yr in Canal C-34
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 6.9 CSM under current conditions and 17.3 CSM under future conditions, which means the current condition is less than SFWMD's permissible discharge allowance of 13.8 CSM based on a 10-yr design frequency but future conditions are higher.	10-yr discharge less than permissible discharge	10-yr discharge higher than permissible discharge
PM#3	Entire LMA	During peak tailwater, the simulated flow at structure S63A is less than the design discharge (2000 cfs) for all storm events in the current and future conditions. During peak headwater, the simulated flow at structure S63A is higher than the design discharge (2000 cfs) for all design storm events except for 5-yr.	5-yr	5-yr
PM#4-UKB	-	-	-	-
PM#5	Entire LMA	For 5-yr design events under both current and future conditions, only 46% of the LMA is inundated more than 0.25-ft. The percentage of inundated areas with depths more than 0.25-ft increase for 10- and 25-, and 100-yr events, up to a maximum of around 82% for a 100-yr event under both current and future conditions.	5-yr	5-yr
PM#6	Entire LMA	Under current conditions, approximately 61% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm, while under future conditions approximately 63% of the LMA stays flooded. Approximately 41% and 46% of the LMA under current and future conditions, respectively, remains flooded for longer than 6 hours at flooding depths higher than 0.25 ft for the 100-yr storm.	5-yr	5-yr
Lake Gentry LMA Overall FPLOS		**Current Conditions**		**< 5-yr**
		Future Conditions		**< 5-yr**

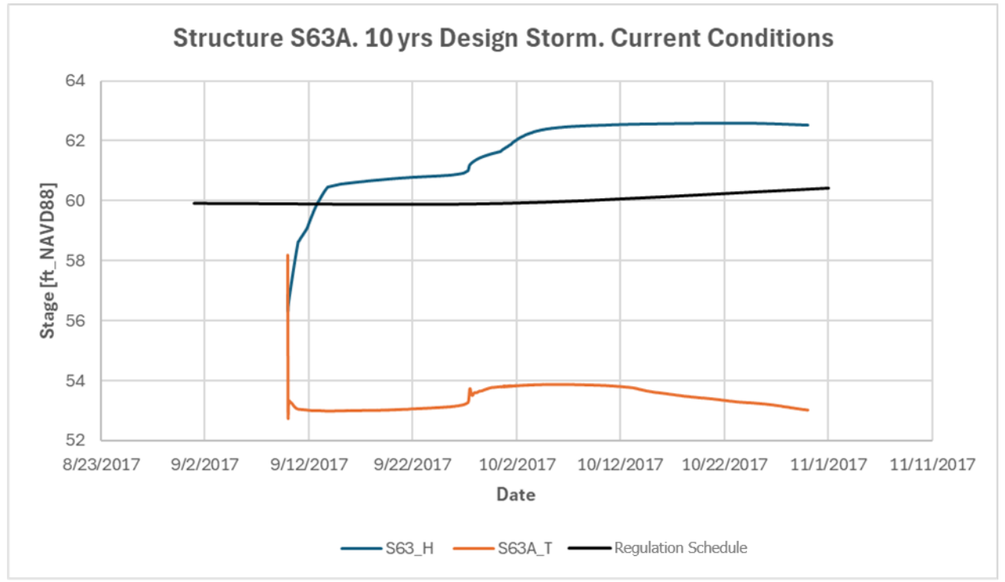
10.4.8 Volume in Lake due to Design Storm: Lake Gentry

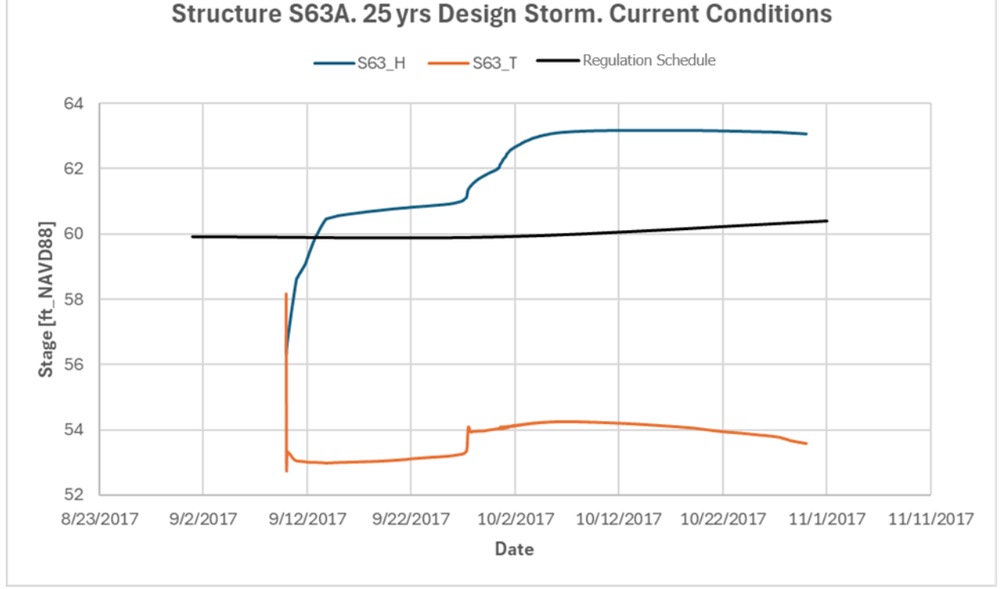
Table 13-3 summarizes how Lake Gentry's water volume is impacted by the design events under current and future conditions. Under current conditions, Lake Gentry had a higher volume under current conditions than under future conditions. For a 25-yr design event under current conditions, the volume is 4555 ac. ft, much higher than under future conditions which is 1588.5 ac. ft. This is because there is a larger volume available in Lake Kissimmee for Lake Gentry to discharge to, under future conditions due to the headwaters revitalization schedule. **Table 13-4** shows the volume of water at peak.

10.4.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}: Lake Gentry

The impact of the design events relative to the regulation schedule in Lake Gentry is assessed by examining its main regulation structure, S63A. **Figure 10-94 and Figure 10-95** show the water levels upstream and downstream of S63A, and the regulation schedule for the whole simulation period, under current and future conditions. Under current conditions, water level in Lake Gentry is higher than regulation schedule for 5-, 10-, and 25- yr design storms, but lower than regulation schedule under future conditions for these events. Water levels for a 100-yr design event under current and future is always higher than regulation schedule.







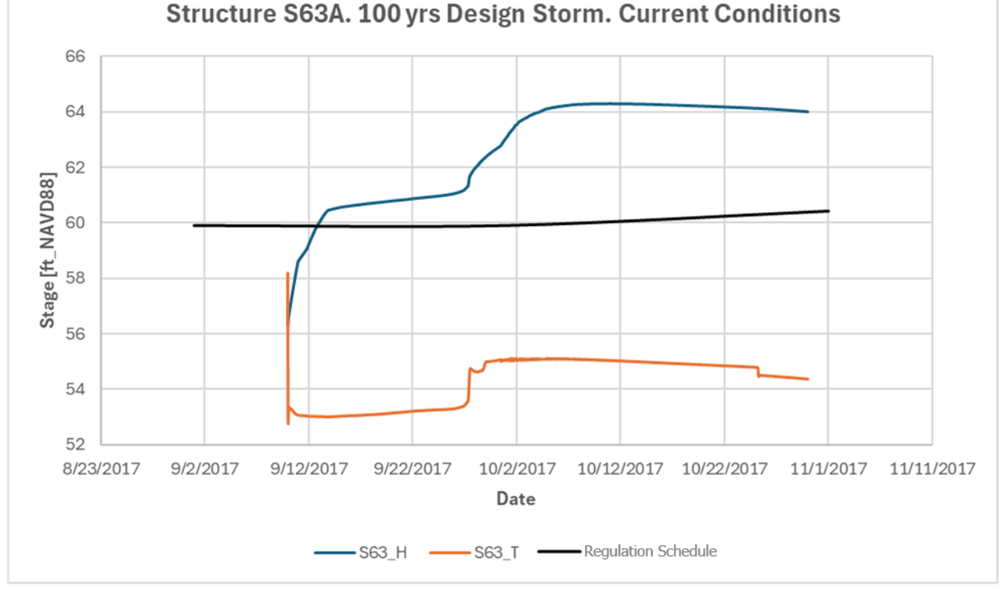
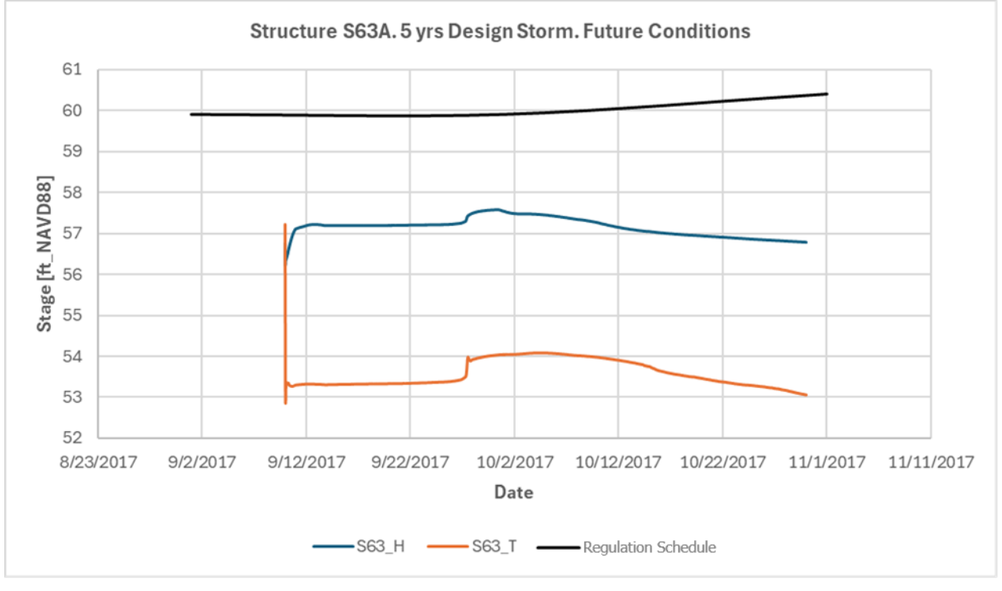
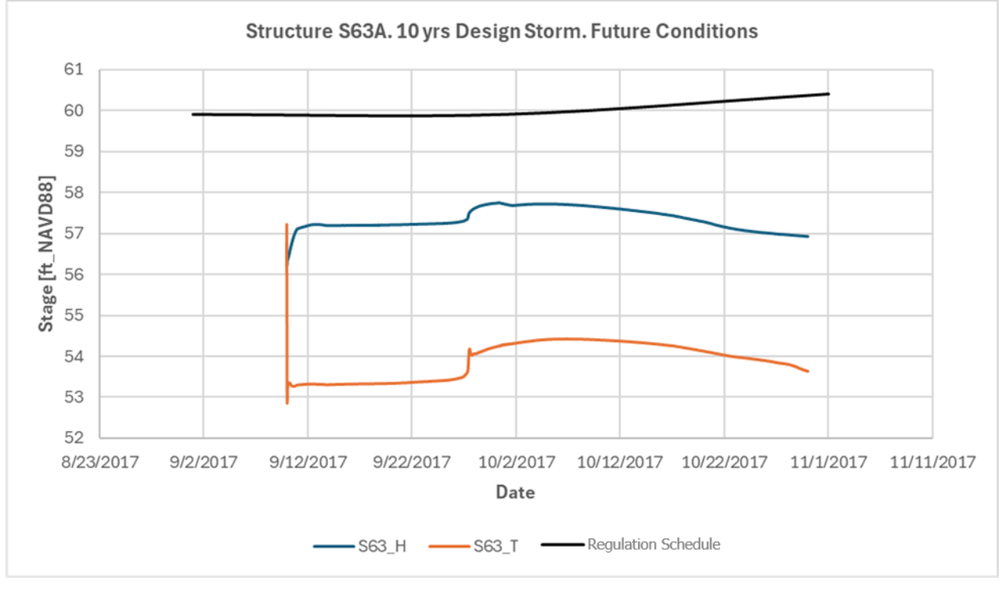
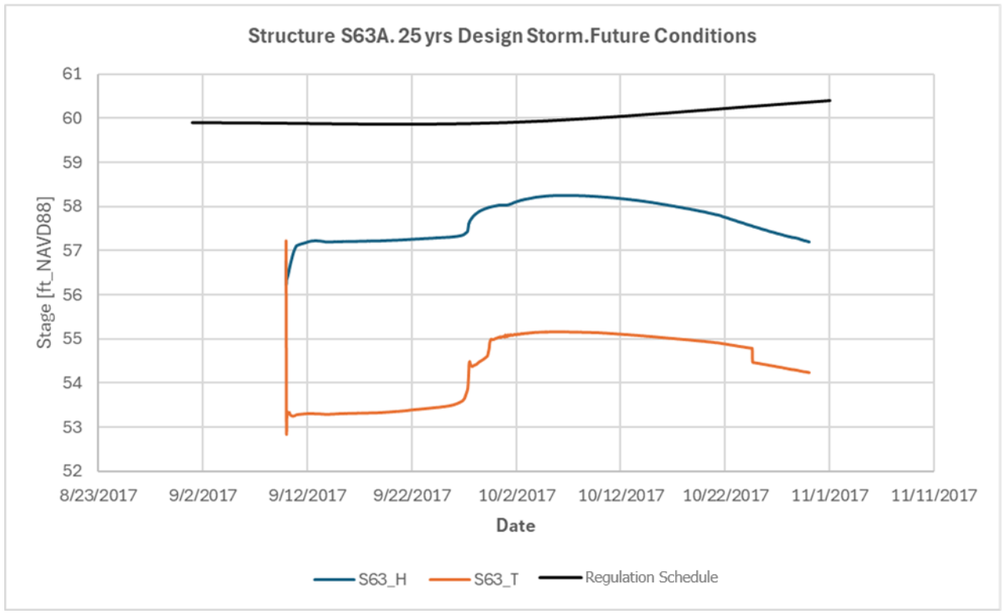


Figure 10-94: Headwater and Tailwater at Structure S63A vs. Lake Gentry Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







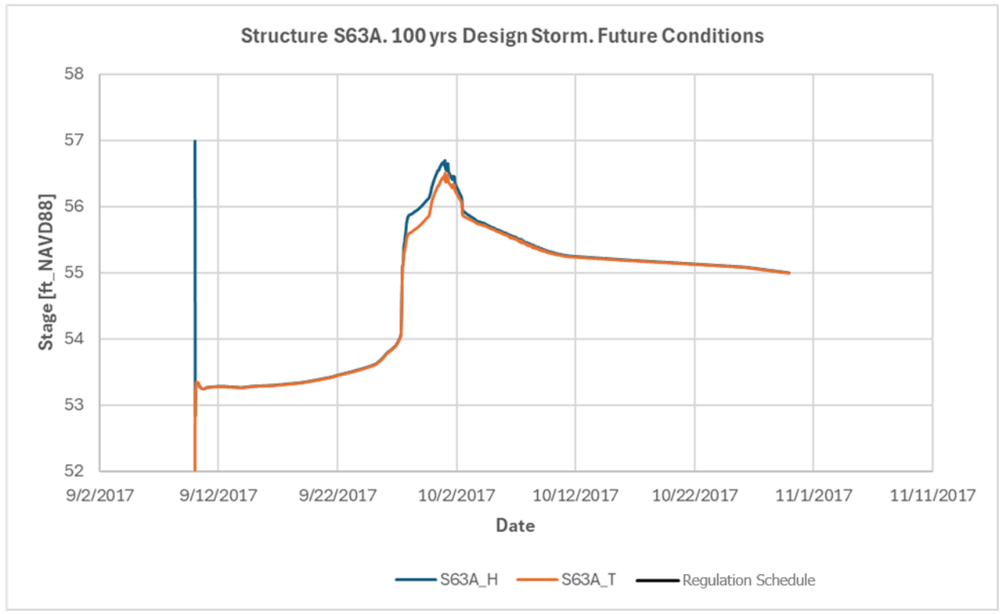


Figure 10-95: Headwater and Tailwater at Structure S63A vs. Lake Gentry Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Future Conditions

10.5 East Lake Toho LMA

Figure 10-96 shows the East Toho LMA with its main waterways and water control structures, and the roadways.

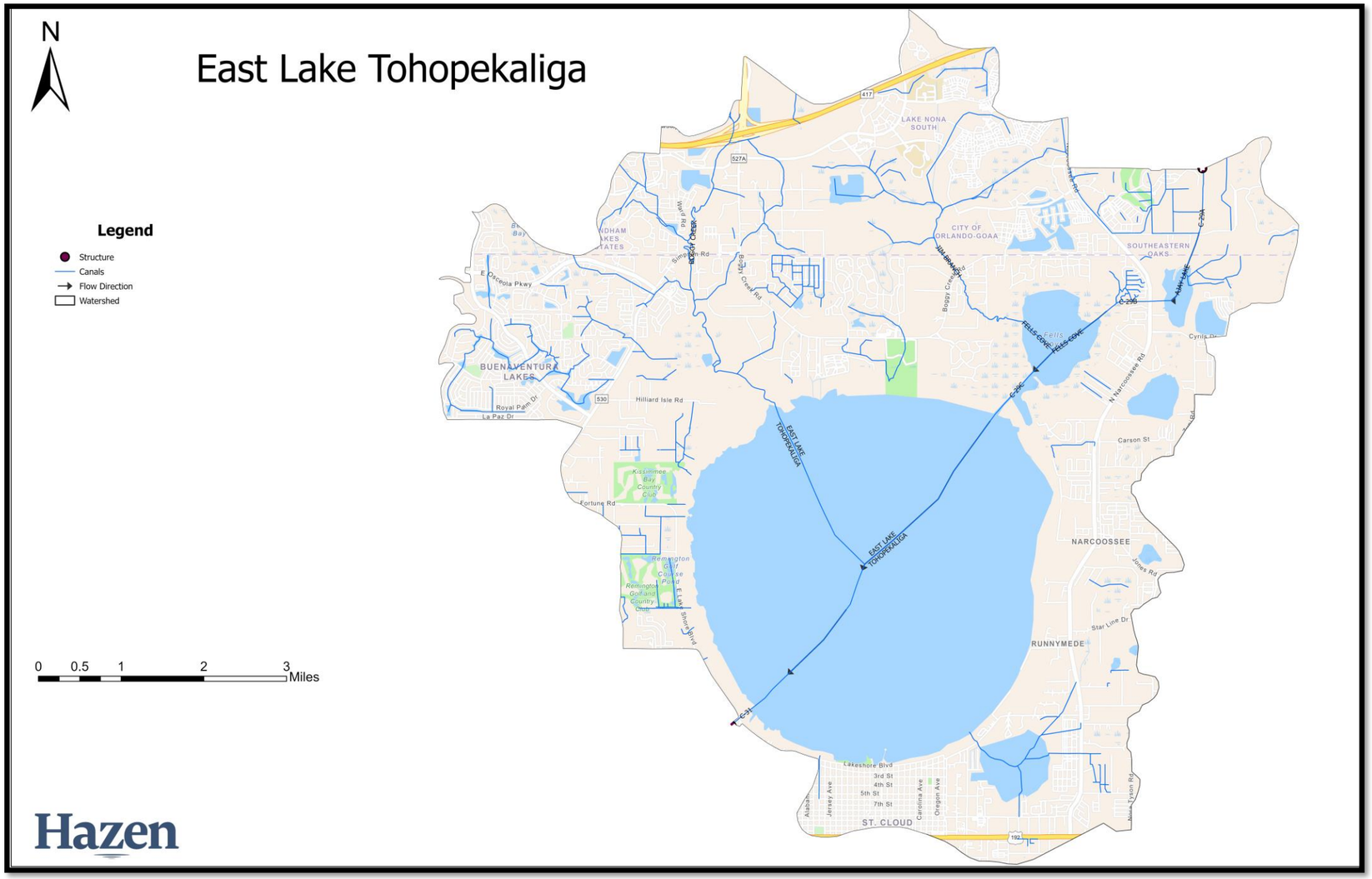


Figure 10-96: East Lake Tohopekaliga LMA with Main Structures, Waterways and Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.5.1 PM #1

Performance of the system at East Lake Toho LMA was assessed at East Lake Toho, as shown in **Figure 10-97 and Figure 10-98** under current and future conditions. For East Lake Toho, under current conditions, the left and right banks do not have exceedances during any of the design storms, for the most part. Under future conditions, the exceedances during the 100-year event become more pronounced, especially exceeding the left bank near the S-59 structure and the right bank in the middle of the LMA. For the other storms, water levels remain similar to current conditions, with all staying within the bank limits. The comparison between current and future conditions indicates a slight increase in water levels under future conditions, with more frequent and higher exceedances during the 100-year event.

In summary, at East Lake Toho, lake levels do not exceed the banks for 5-, 10-, and 25- yr events, except at 1 or 2 locations along the right bank. Under current and future conditions, water levels overtop the right bank in one portion of the canal for 100- yr design storm.

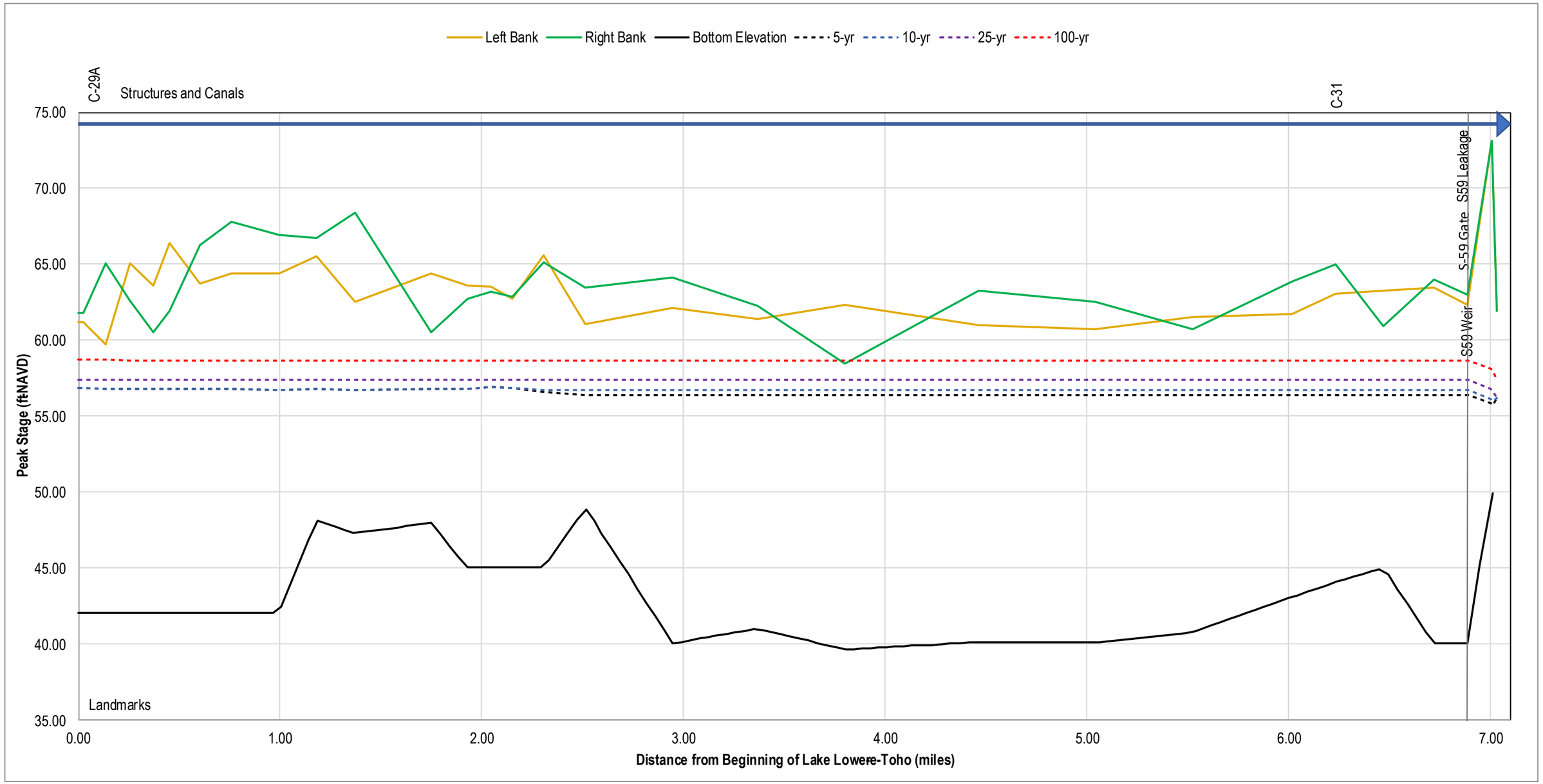


Figure 10-97: Maximum Stage Profile for East Lake Toho under Current Conditions

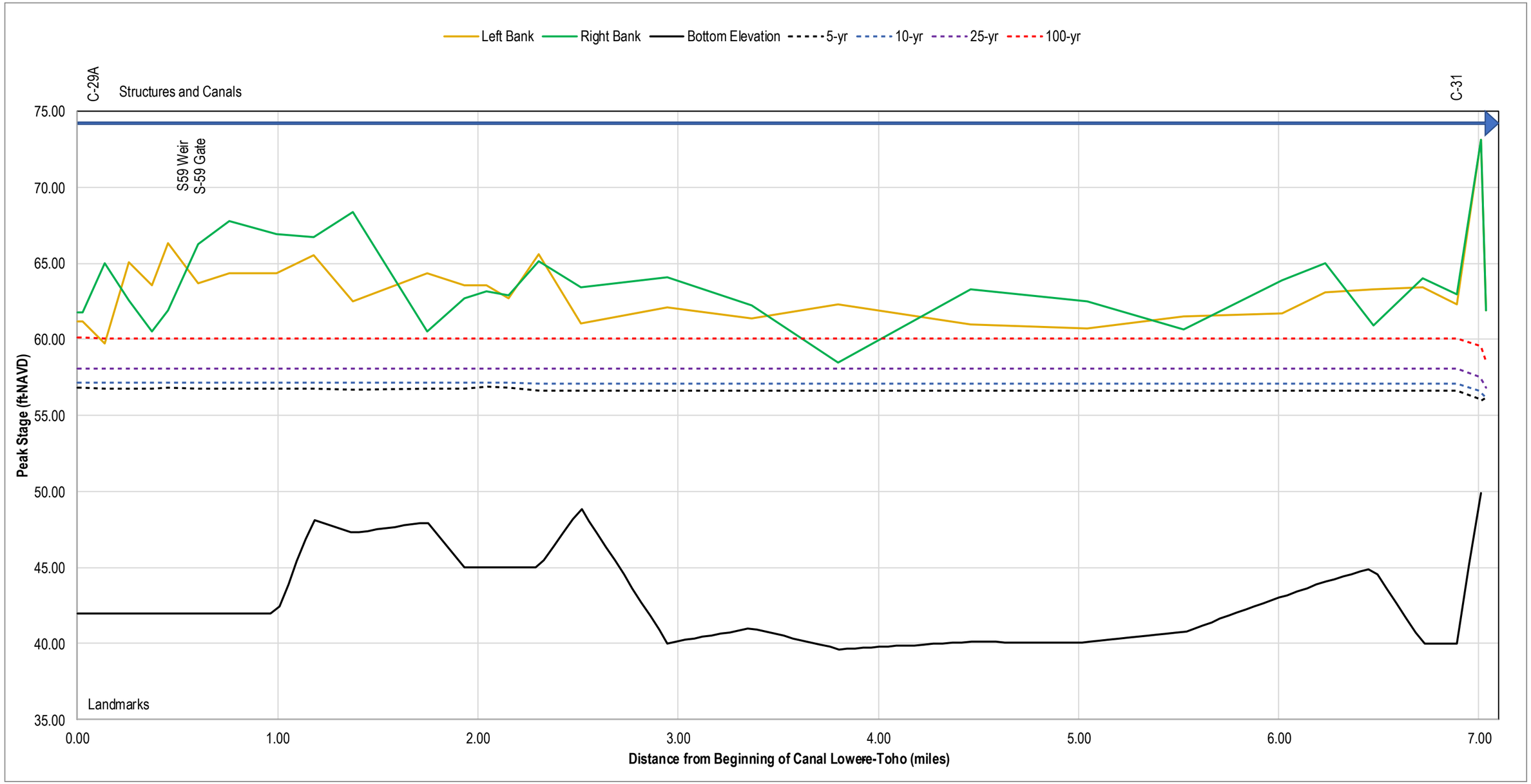


Figure 10-98: Maximum Stage Profile for East Lake Toho under Future Conditions

10.5.2 PM #2

East Lake Toho LMA's upstream water control structure is S62, and downstream structure is S59. The maximum daily discharge capacity of this LMA is determined by S59 discharge capacity. Per the Districts ERP Handbook, East Lake Toho LMA has an allowable discharge rate of 16.1 CSM, based on a design frequency of 10-yrs. **Table 10-57 and Table 10-58** tabulate East Lake LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area under current and future hydrologic conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 7.3 CSM and 7.9 CSM under current and future conditions respectively, both of which are lower than SFWMD's permissible discharge allowance of 16.1 CSM based on a 10-yr design frequency.

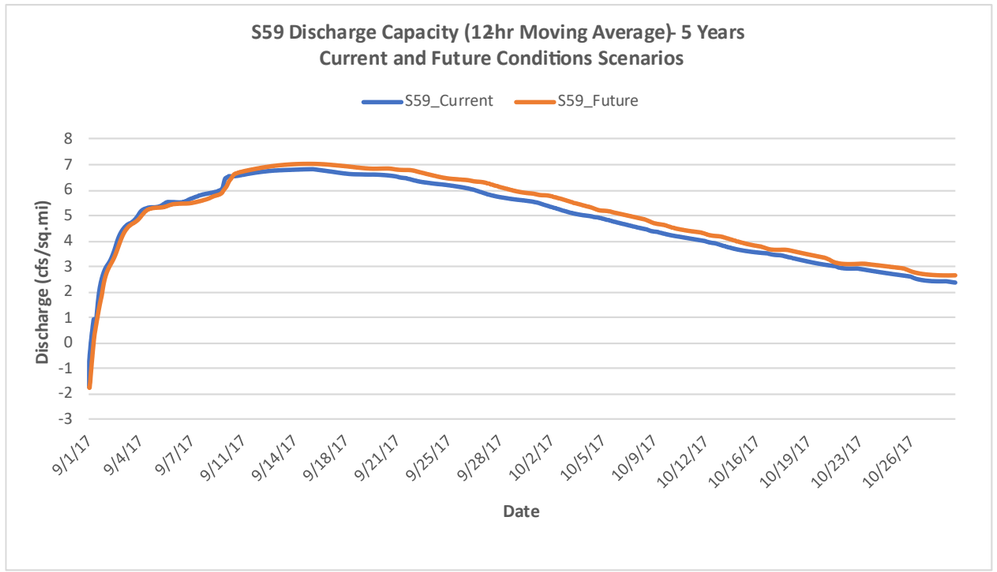
Table 10-57: Drainage Discharge Capacity (cfs/sq. mi) of East Lake Toho LMA under Current Conditions

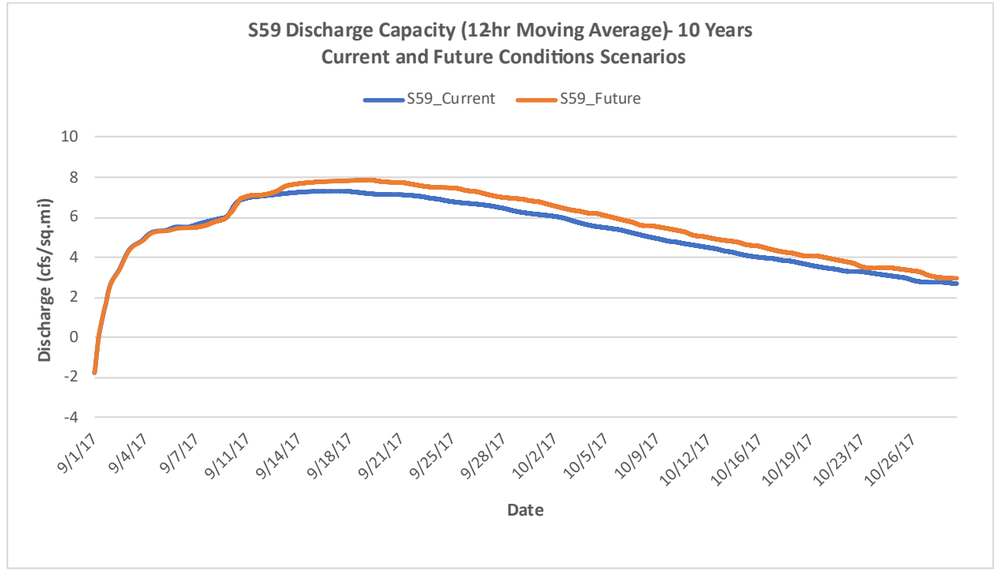
LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
East Lake Toho	132.8	6.8	7.3	8.3	11.1	16.1

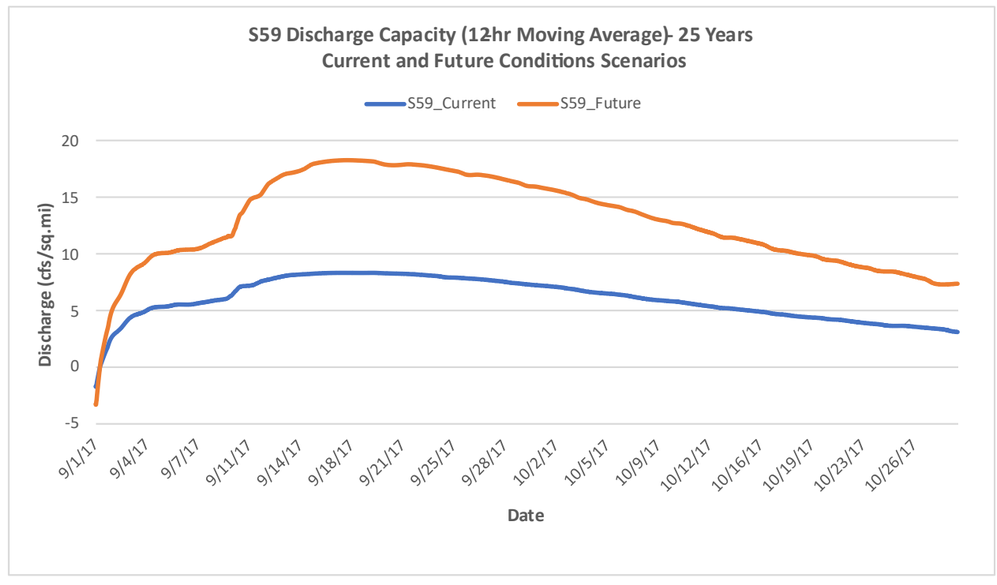
Table 10-58: Drainage Discharge Capacity (cfs/sq. mi) of East Lake Toho LMA under Future Conditions

LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
East Lake Toho	132.8	7.0	7.9	9.6	13.1	16.1

Figure 10-99 shows the peak 12-hour average discharge per square mile of drainage area, calculated for each design storm at structure S59. **Figure 10-100** shows the maximum daily discharge through structure S59 for both current and future conditions. They show the discharges are lower than the SFWMD allowable discharge for all design storms under current and future conditions.







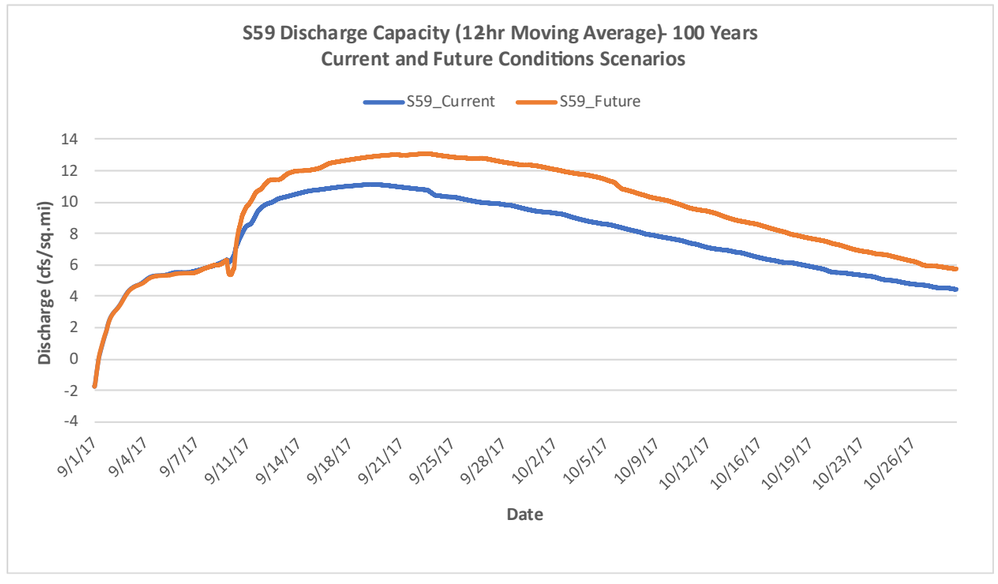


Figure 10-99: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S59 for 5-, 10-, 25-, 100- yr Design Storms

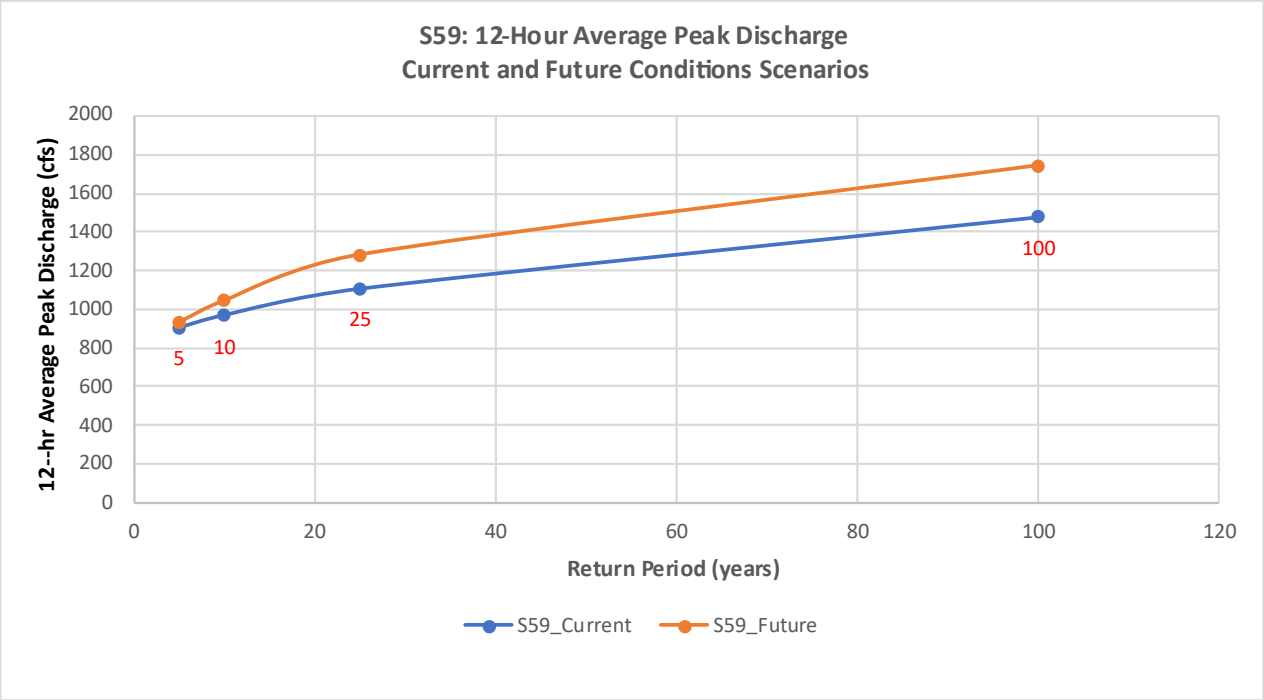


Figure 10-100: Structure S59 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- yr Design Storms

10.5.3 PM #3

The SFWMD structure design parameters (**Table 10-59**) were used to evaluate the S59 performance against the various design storms. **Table 10-60 through Table 10-65** summarize current conditions and future conditions simulated instantaneous stages and flows at Structure S59 at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S59, the simulated headwater elevation at S59 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is lower than the SPF headwater elevation for both current and future conditions. The simulated flow at structure S59 is higher than the design discharge (820 cfs) for all design storm events in the current and future conditions.

During the peak headwater at the structure S59, the simulated tailwater elevation at S59 is lower than design tailwater elevation during 5-yr, and 10-yr design storm events for current conditions and future conditions. The simulated flows at structure S59 are higher than the design discharge (820 cfs) for all design storm events for both current and future conditions.

During the peak flow at Structure S59, the headwater and tailwater elevations are lower than the design headwater for 5-yr and 10-yr design storm event for current and future conditions. This is demonstrated in **Figures 10-101 and 10-102**.

Table 10-59: Design Parameters for Structure S59

Structure	S59
Design Discharge	590 to 820 cfs (Lower Profile to Peak Stage) (30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	54.83 to 56.53 feet NAVD88 (55.8 to 57.5 feet NGVD29)
Design Tailwater Elevation	55.3 (54.33) to 56.9 (55.93 feet NAVD88) feet NGVD29
SPF Headwater Elevation	61.83 feet NAVD88 (62.8 feet NGVD29)
SPF Tailwater Elevation	61.2 (60.23 feet NAVD88) feet NGVD29
Optimum Headwater	54.03 to 57.03 feet NAVD88 (55.0 to 58.0 feet NGVD29)
Optimum Tailwater	52.03 to 54.03 feet NAVD88 (53.0 to 55.0 feet NGVD29)

Table 10-60: Summary of the Instantaneous Conditions at Structure S59 at Time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak TW (cfs)	Head Differential at peak TW (ft)	Date & Time
5-yr	55.68	55.29	889.36	0.39	9/12/17 5:43 PM
10-yr	55.99	55.56	960.32	0.43	9/14/17 4:55 PM
25-yr	56.65	56.15	1104.53	0.50	9/16/17 9:00 PM
100-yr	57.96	57.37	1455.80	0.59	9/17/17 7:55 PM

Table 10-61: Summary of the Instantaneous Conditions at Structure S59 at Time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak HW (cfs)	Head Differential at peak HW (ft)	Date & Time
5-year	55.95	55.56	925.41	0.38	9/13/17 10:00 PM
10-year	56.48	56.06	1031.10	0.43	9/16/17 4:43 AM
25-year	57.43	56.85	1277.50	0.58	9/17/17 10:00 AM
100-year	59.51	58.55	1693.21	0.96	9/18/17 9:00 PM

Table 10-62: Summary of the Instantaneous Conditions at Structure S59 at Time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at peak TW (ft)	Date & Time
5-year	55.29	55.67	881.91	0.38	9/12/17 4:00 AM
10-year	55.57	55.97	936.79	0.40	9/12/17 12:00 PM
25-year	56.15	56.64	1093.25	0.49	9/15/17 2:43 PM
100-year	57.38	57.94	1424.45	0.55	9/15/17 9:00 PM

Table 10-63: Summary of the Instantaneous Conditions at Structure S59 at Time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at peak TW (cfs)	Head at Peak TW (ft)	Date & Time
5-year	55.57	55.94	912.70	0.37	9/12/17 7:00 PM
10-year	56.06	56.47	1021.36	0.41	9/15/17 3:55 AM
25-year	56.86	57.42	1266.19	0.56	9/16/17 10:55 AM
100-year	58.58	59.48	1658.32	0.90	9/17/17 5:55 AM

Table 10-64: Summary of the Instantaneous Conditions at Structure S59 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at peak HW (ft)	Date & Time
5-year	906.49	55.18	55.18	0.44	9/16/17 1:00 PM
10-year	970.95	55.49	55.49	0.46	9/18/17 8:00 AM
25-year	1107.95	56.09	56.09	0.52	9/20/17 3:55 AM
100-year	1479.72	57.30	57.30	0.62	9/19/17 10:25 PM

Table 10-65: Summary of the Instantaneous Conditions at Structure S59 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Discharge at Peak TW (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at peak HW (ft)**	**Date & Time**
5-year	934.38	55.92	55.52	0.41	9/16/17 1:00 AM
10-year	1045.64	56.38	55.91	0.48	9/20/17 2:25 AM
25-year	1281.22	57.43	56.84	0.59	9/18/17 7:25 AM
100-year	1741.60	59.26	58.21	1.05	9/23/17 8:25 PM

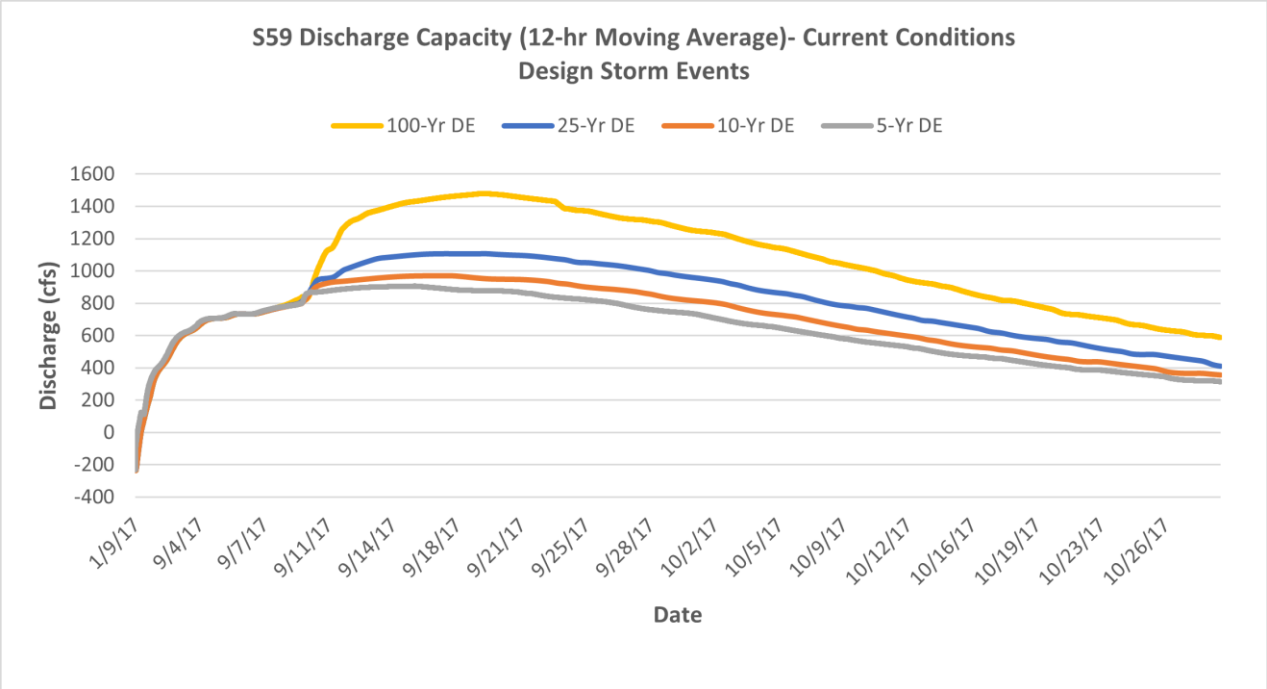


Figure 10-101: 12-hr Moving Average Flows in cfs under Current Conditions

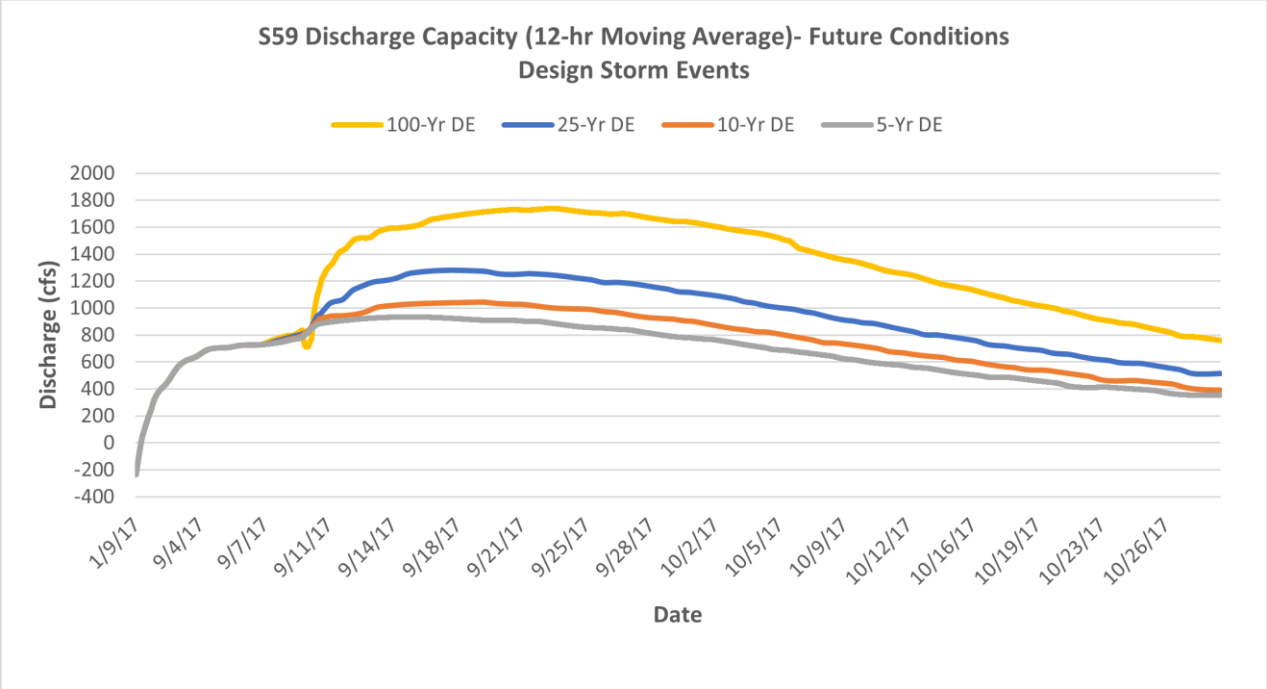


Figure 10-102: 12-hr Moving Average Flows in cfs under Future Conditions

10.5.4 PM #4

PM#4-UKB was not evaluated for this LMA.

10.5.5 PM #5

The gridded data of maximum overland depth was extracted for all simulated design storms under current and future conditions and evaluated for East Lake Toho LMA.

Table 10-66 and **Figure 10-103** represent the flood inundation area in square miles for this LMA, in incremental intervals of feet as table and plot, respectively. **Figure 10-104 through Figure 10-107** show the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions for East Lake Toho LMA. Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not significant. There is also a high distribution of flooding depths above 2.5 feet which are the large lakes in this LMA. Assessment of this shows the flooding depths that are between 0.25 feet and greater than 2.5 feet, are in the smaller lakes, and the canals and ditches that connect them.

Table 10-66: Incremental Flood Inundation Area (sq. mi.) in East Lake Toho LMA

Flooding Depth (ft)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	81.16	73.98	63.14	45.49	81.15	73.97	63.12	45.46
0.25<=Depth<0.5	9.50	12.17	16.49	21.95	9.43	12.13	16.46	21.93
0.5<=Depth<0.75	5.15	5.47	5.99	8.58	5.07	5.42	6.00	8.57
0.75<=Depth<1.0	6.44	4.45	4.46	5.16	6.51	4.40	4.46	5.14
1.0<=Depth<1.25	3.94	5.62	3.71	3.99	3.95	5.51	3.70	4.00
1.25<=Depth<1.5	3.87	3.87	4.17	3.58	3.94	3.97	4.10	3.56
1.5<=Depth<1.75	1.34	2.80	4.25	2.83	1.34	2.80	4.21	2.85
1.75<=Depth<2.0	1.11	2.83	3.34	2.98	1.09	2.95	3.30	2.97
2.0<=Depth<2.25	0.93	1.17	2.62	3.51	0.93	1.19	2.62	3.46
2.25<=Depth<2.5	1.04	0.98	2.01	2.80	1.04	0.96	2.09	2.81
>2.5	18.25	19.40	22.54	31.87	18.27	19.42	22.67	31.98
Lake East Toho LMA	**84944.81**	**acres**						
	132.73	**sq.mi.**						

***Note**: The lake area of 28.50 sq. mi. was included in this table computations.*

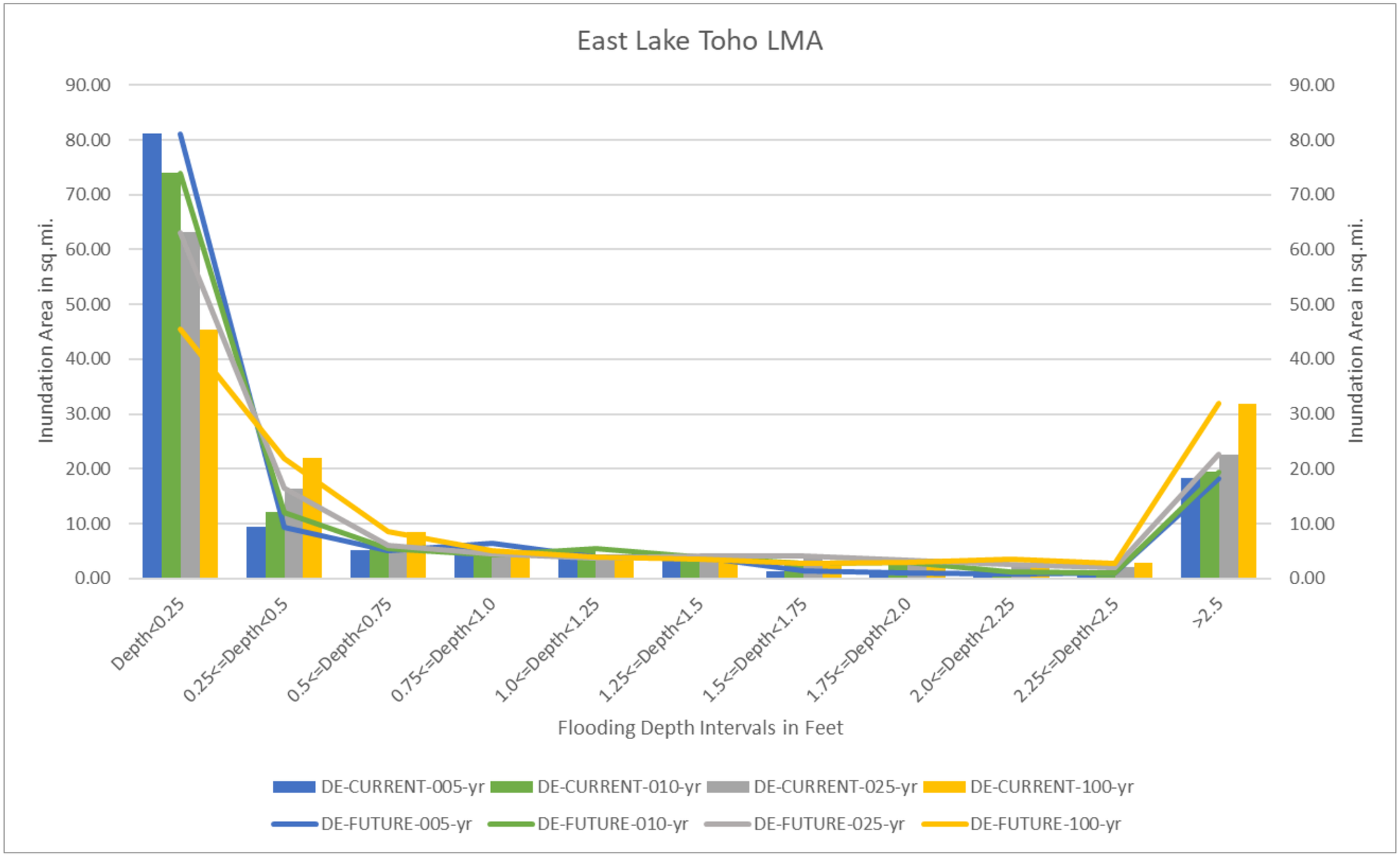


Figure 10-103: Incremental Flood Inundation Depth in feet in East Lake Toho LMA under Current and Future Conditions

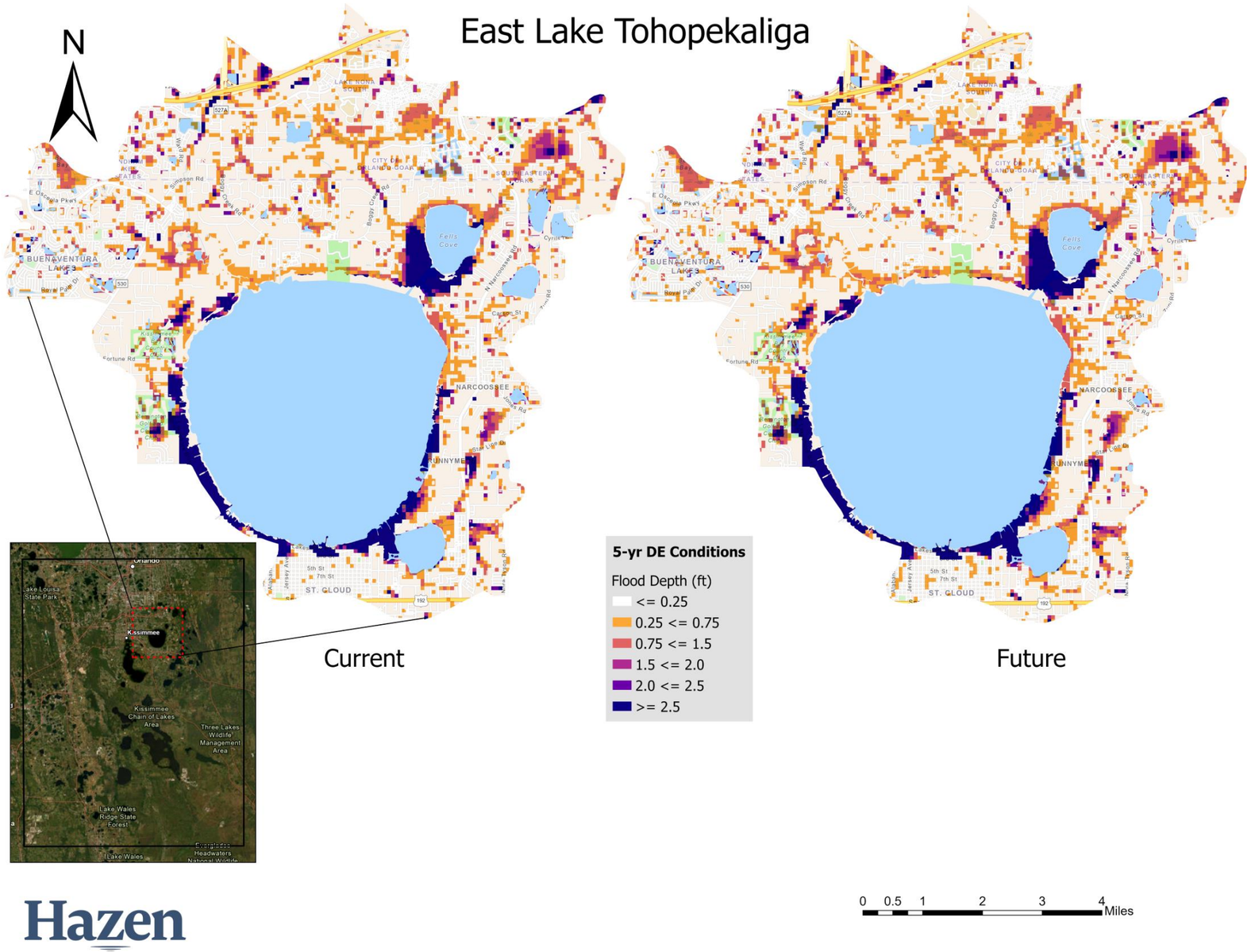


Figure 10-104: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA

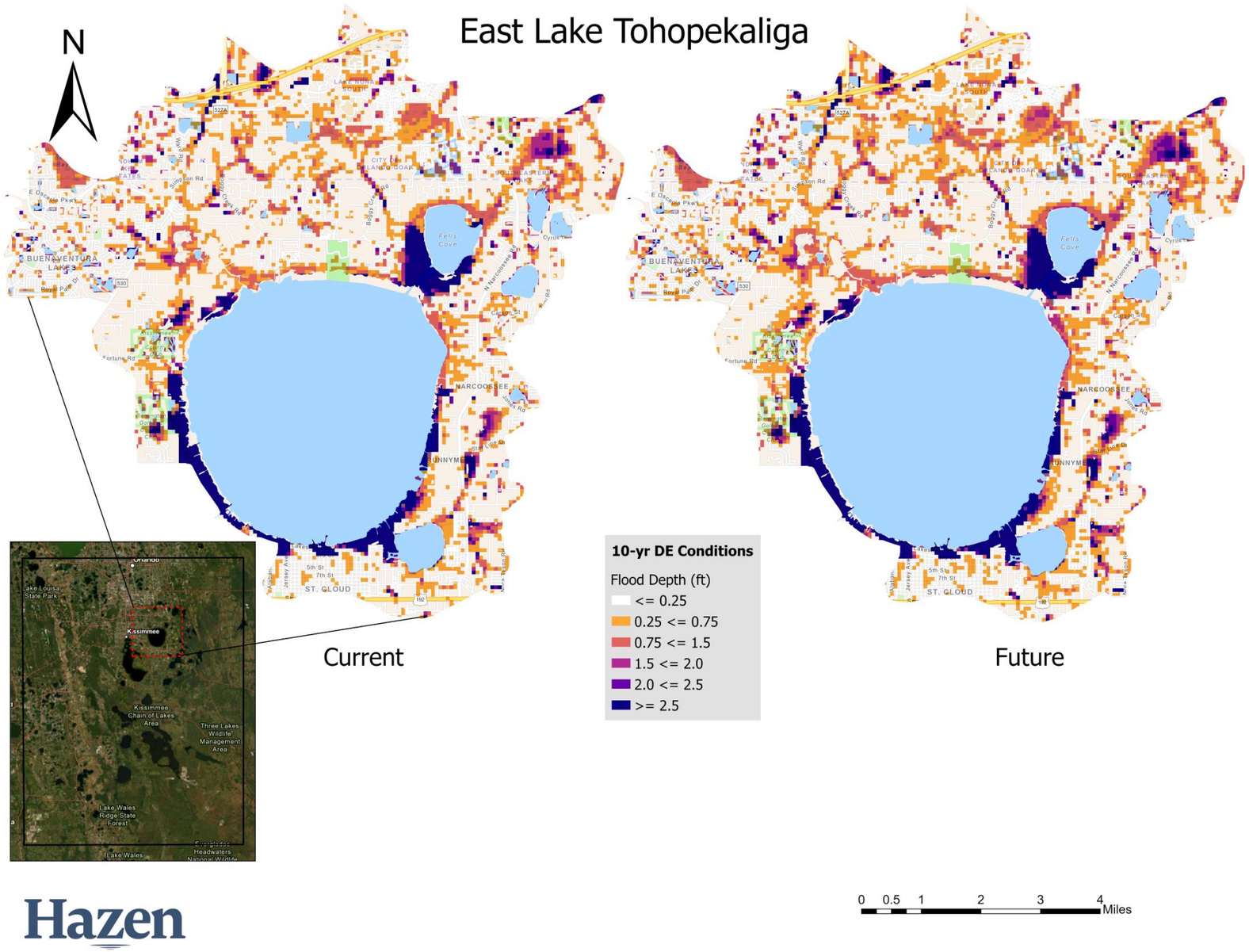


Figure 10-105: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA

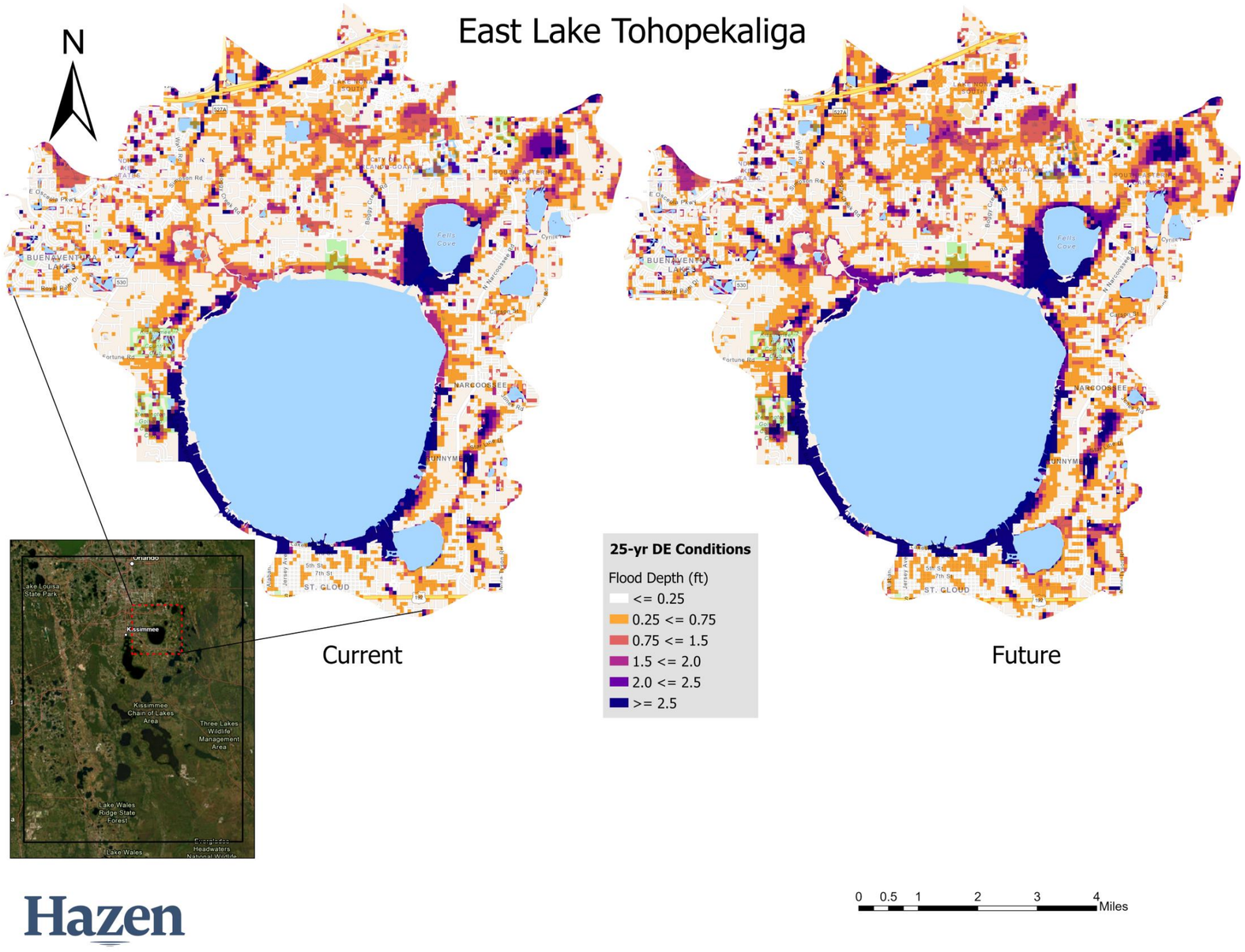
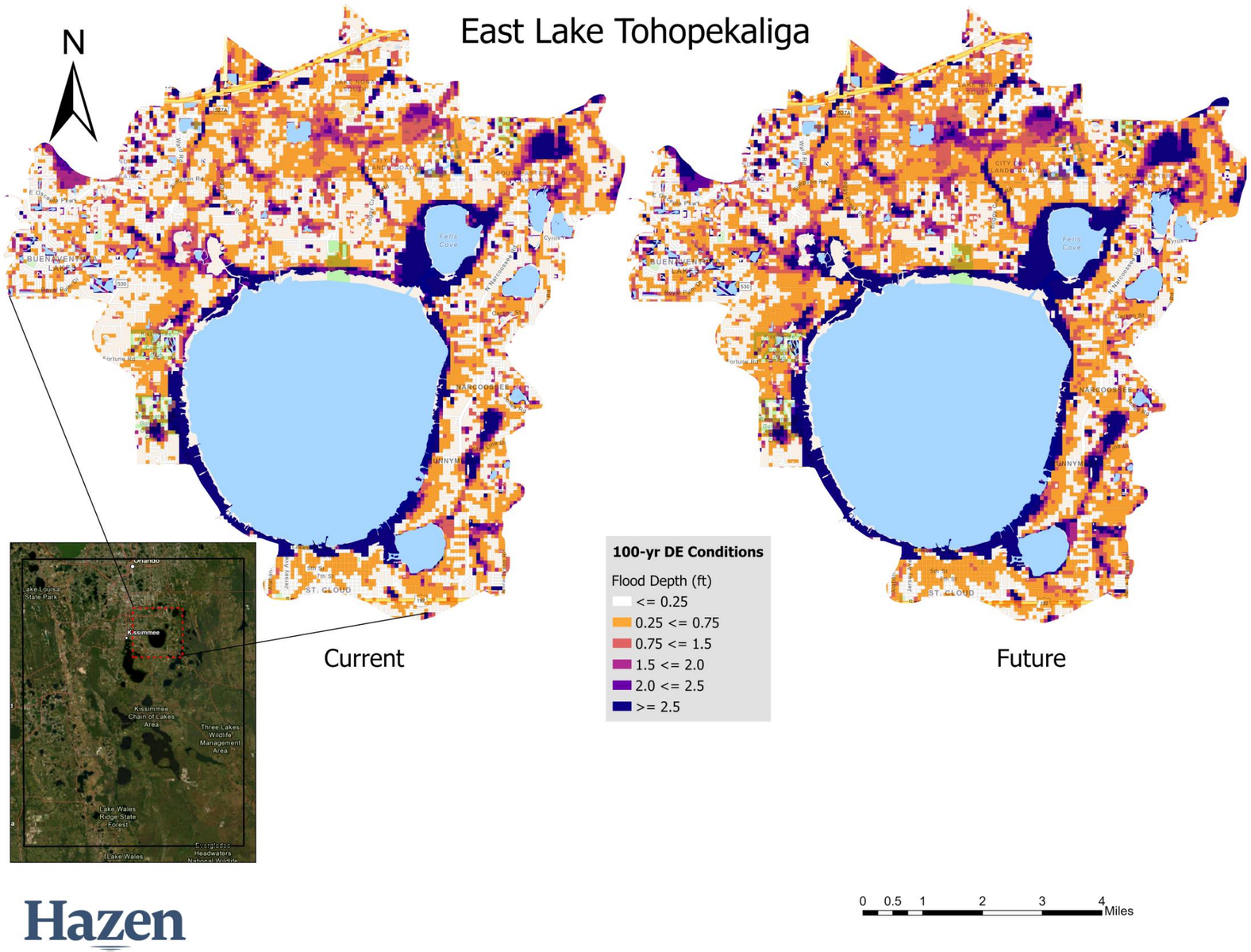


Figure 10-106: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA



Hazen

Figure 10-107: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA

10.5.6 PM #6

Table 10-67 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within East Lake Toho LMA. **Figure 10-108** show these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the East Lake Toho LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-109 through Figure 10-112** show the flood duration in East Lake Toho LMA, except in East Lake Toho which is expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration in East Lake Toho LMA is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration in East Lake Toho LMA are areas with longer than 360 hours of flooding, which are the lakes. The maps show that for all the design storms, the simulations show that the LMA drains at reasonable times; this is due to that the majority of the LMA consists of lakes and the topographic gradient is high in this LMA.

Table 10-67: Areas of Incremental Flood Duration (sq. mi.) in East Lake Toho LMA

Flooding Duration (Hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	85.44	80.10	71.19	55.58	85.42	80.09	71.16	55.54
1<=Hours<4	2.58	3.73	6.80	13.05	2.53	3.69	6.76	13.02
4<=Hours<8	1.29	1.37	1.90	2.53	1.29	1.36	1.88	2.49
8<=Hours<12	0.66	0.74	0.93	1.39	0.63	0.73	0.92	1.39
12<=Hours<24	1.39	1.85	2.41	3.42	1.42	1.80	2.43	3.38
24<=Hours<48	1.52	1.88	2.33	3.25	1.55	1.91	2.35	3.14
48<=Hours<96	1.69	2.20	2.78	3.70	1.72	2.19	2.76	3.70
96<=Hours<168	1.51	1.80	2.57	3.42	1.43	1.80	2.54	3.53
168<=Hours<240	0.98	1.07	1.53	2.27	0.80	1.04	1.51	2.24
240<=Hours<360	1.45	1.12	1.39	2.03	1.29	1.03	1.34	2.02
360<=Hours<1417	34.22	36.86	38.92	42.09	34.66	37.07	39.07	42.29
Lake East Toho LMA	**84944.81**	**acres**						
	132.73	**sq.mi.**						

***Note**: The lake area of 28.50 sq. mi. was included in this table computations.*

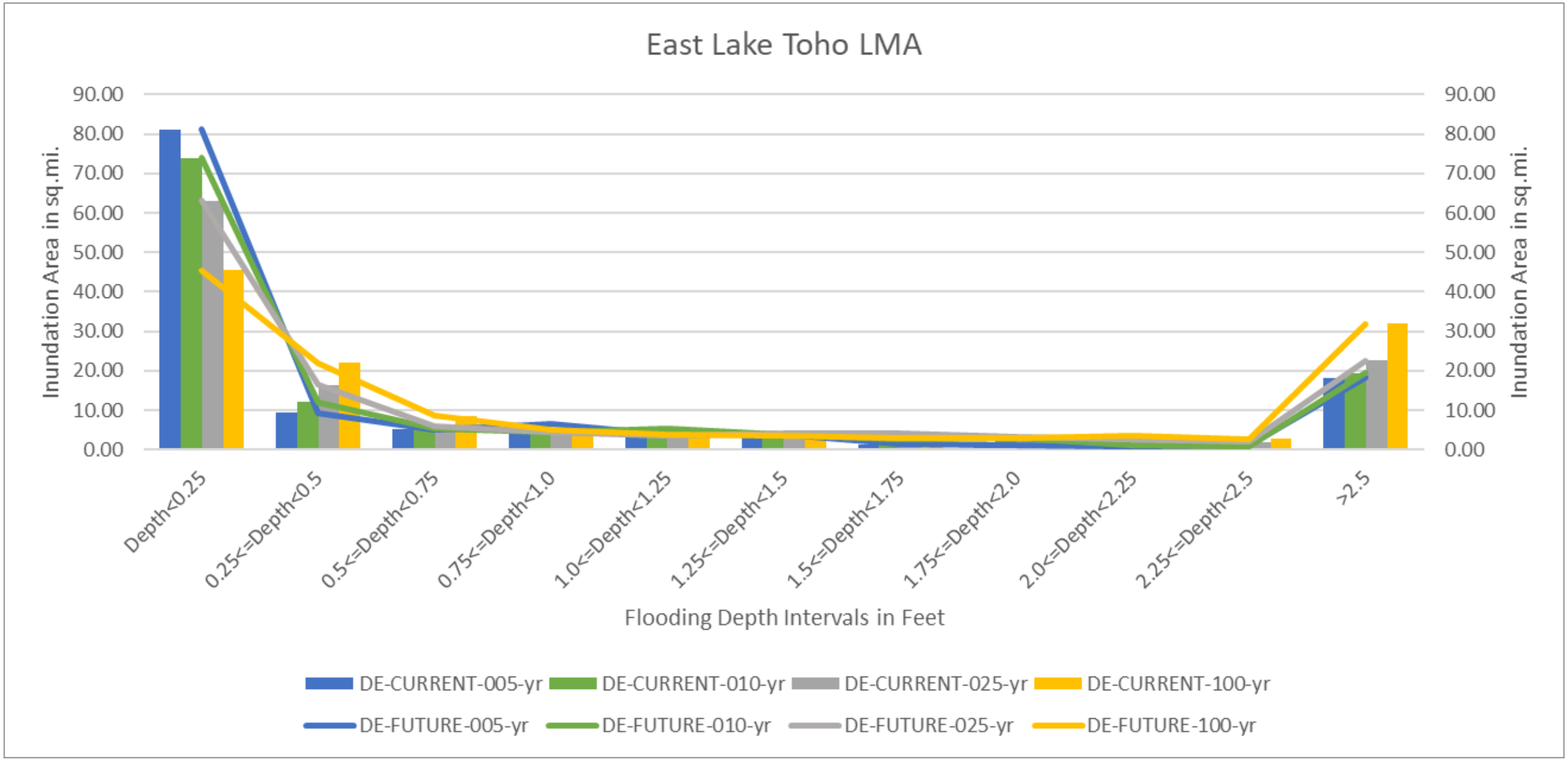


Figure 10-108: Areas of Incremental Flood Duration in hours in East Lake Toho LMA under Current and Future Conditions

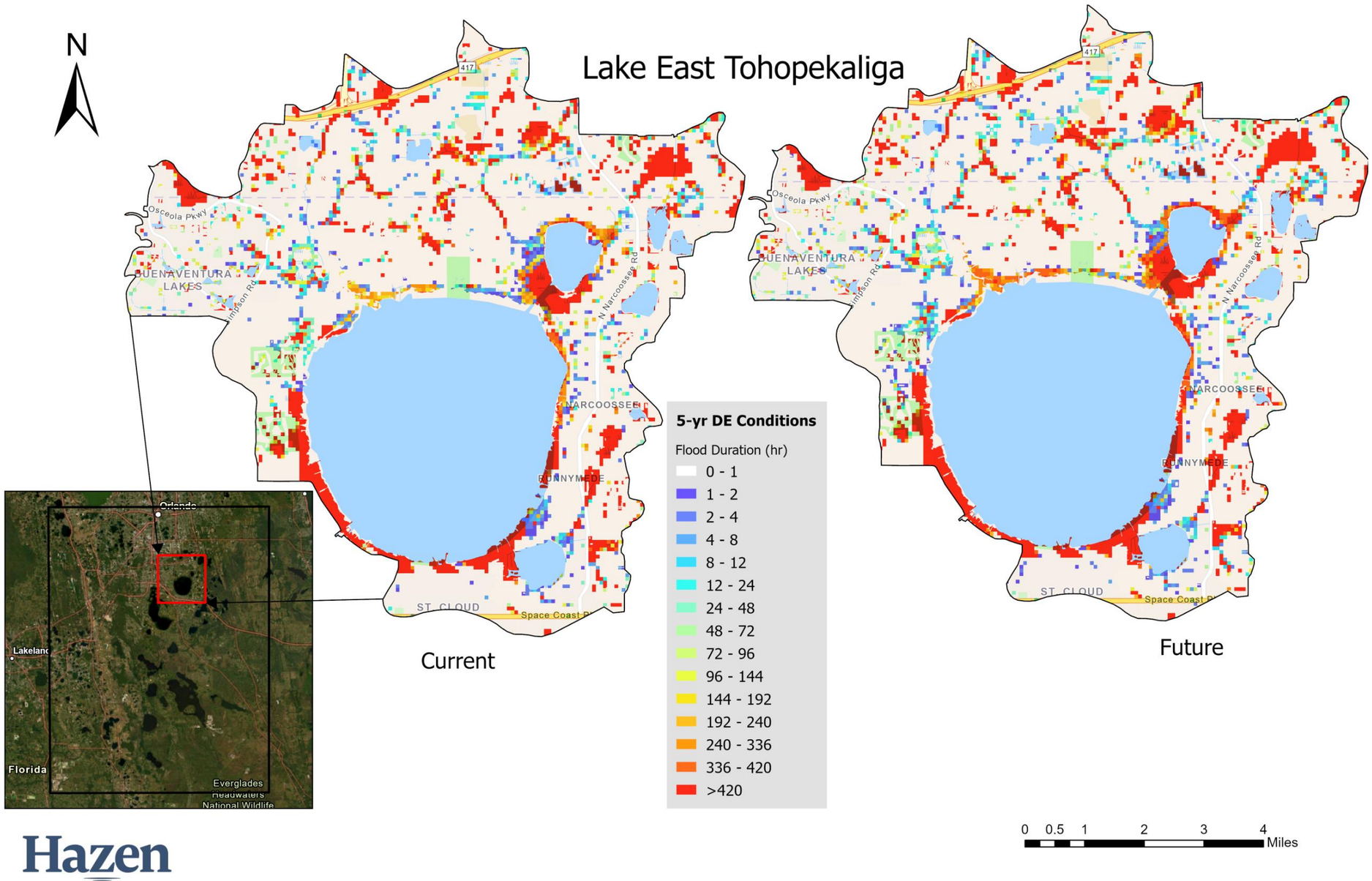


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Figure 10-109: Flood Duration Maps for 5-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

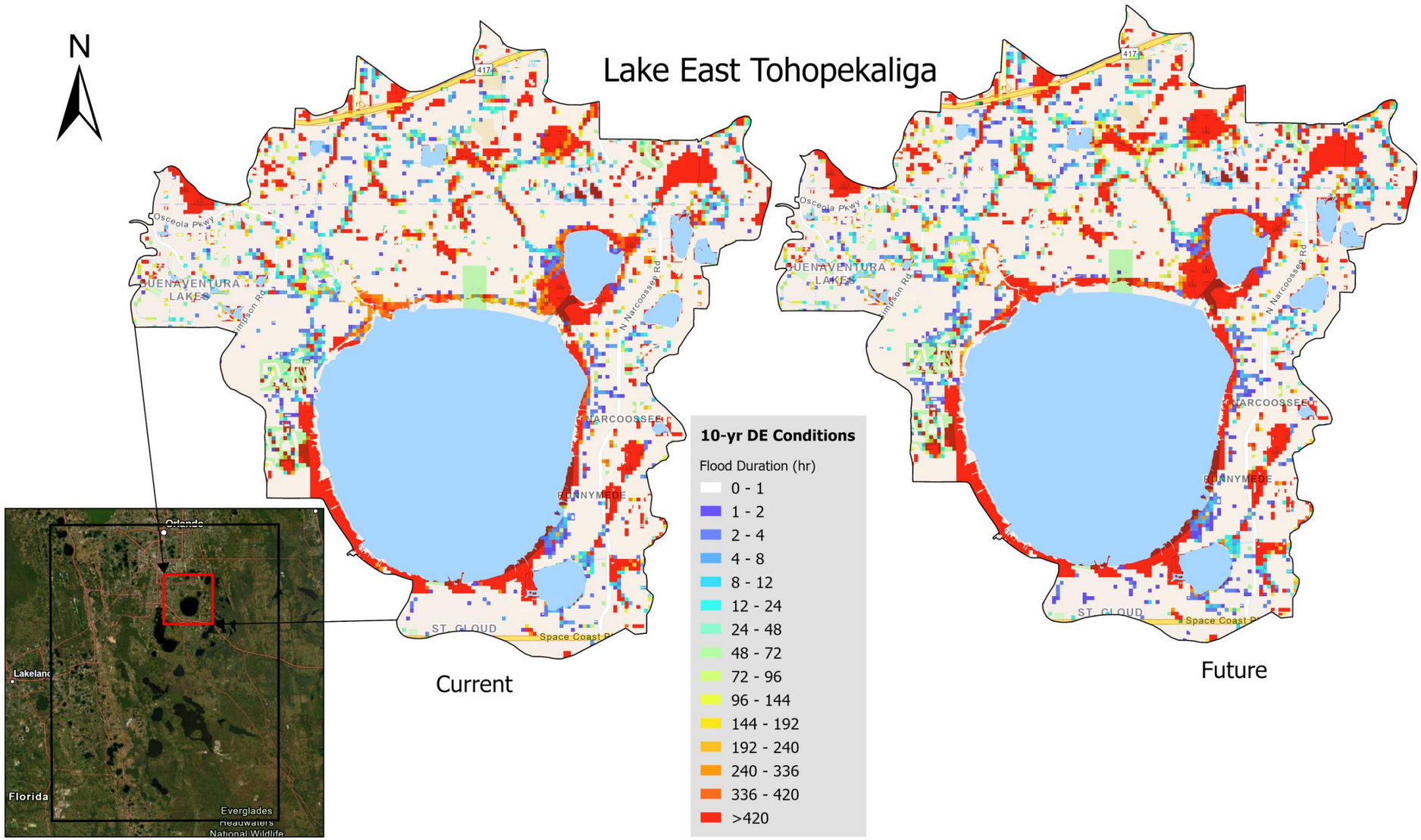

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Figure 10-110: Flood Duration Maps for 10-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

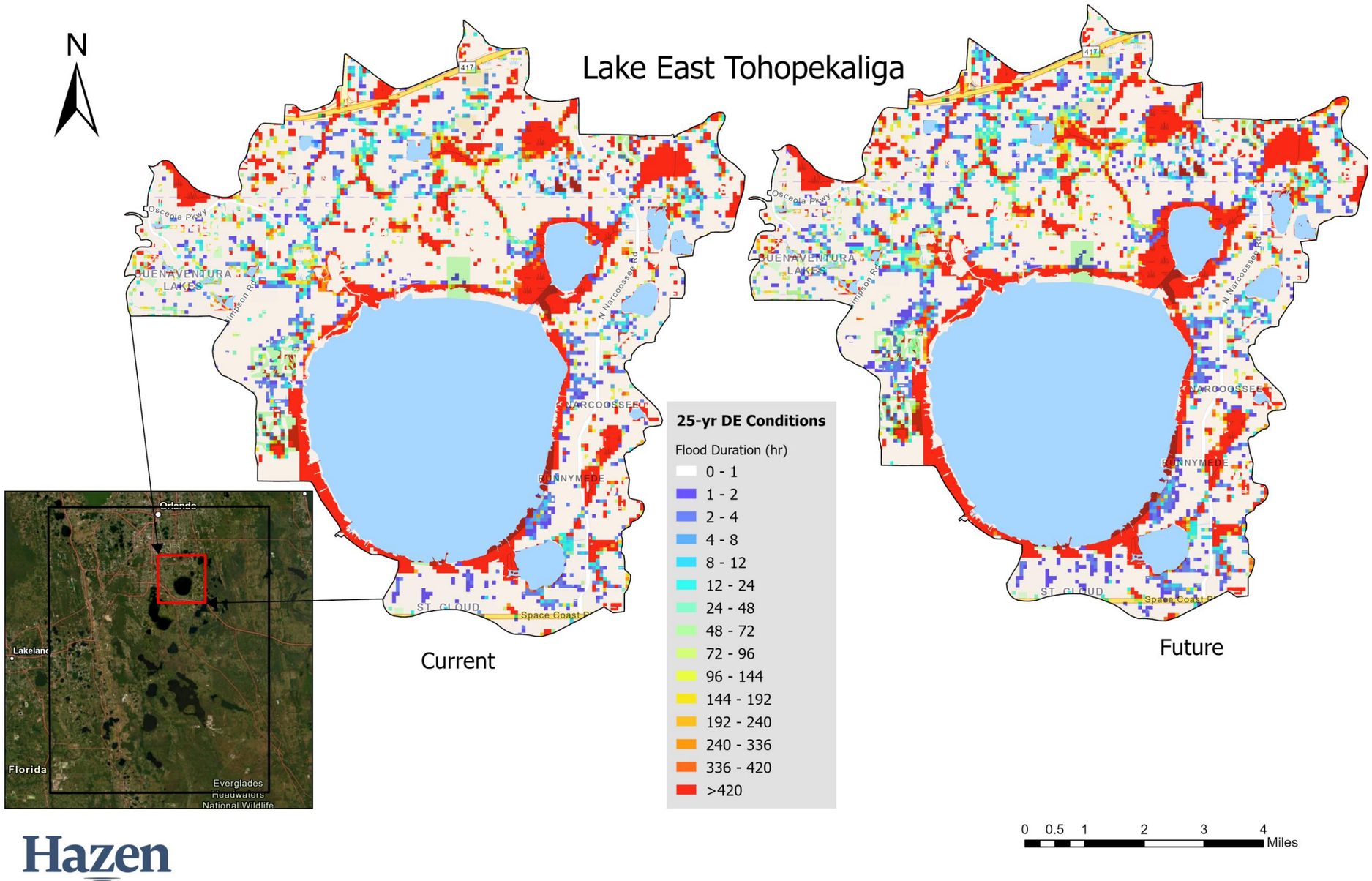

Figure 10-111: Flood Duration Maps for 25-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

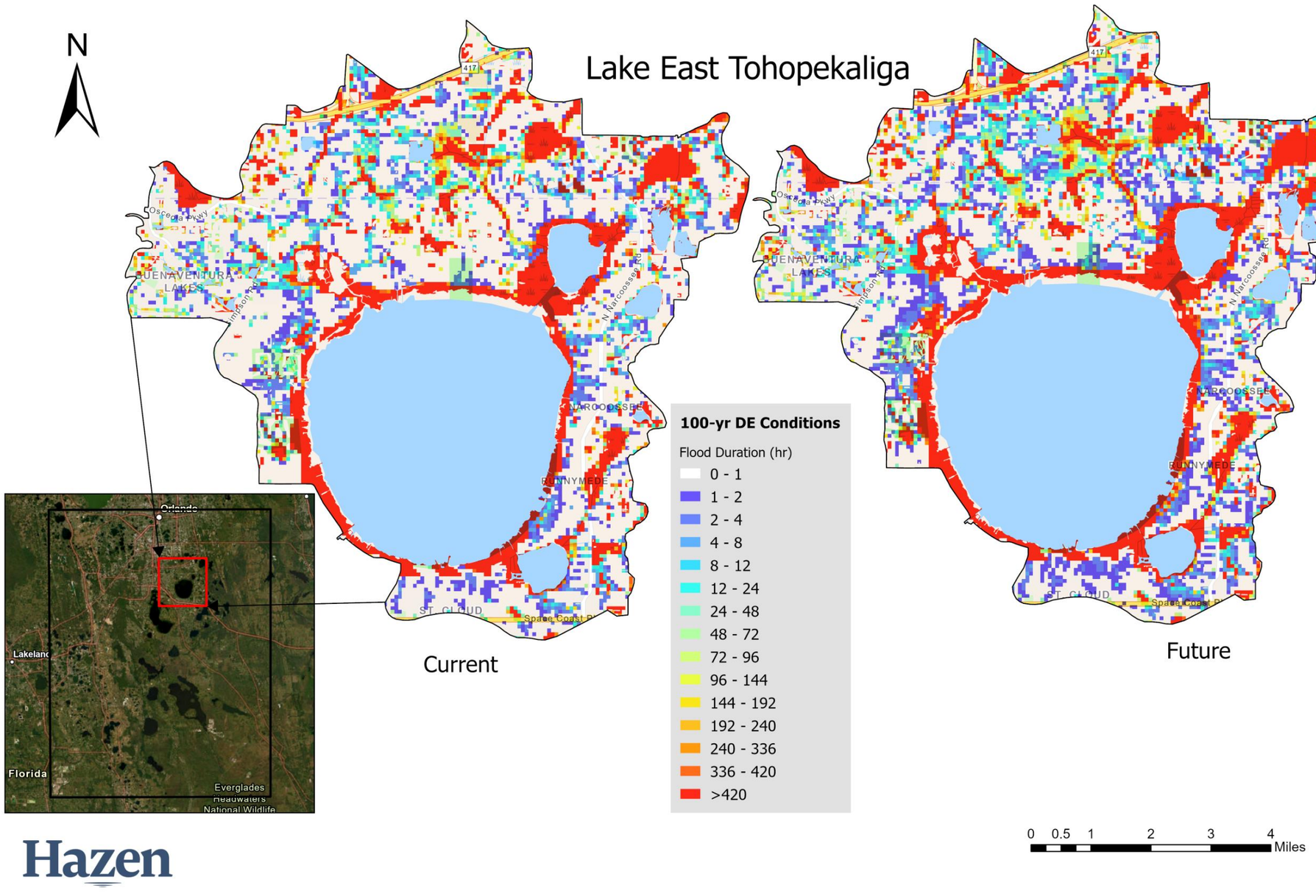

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Figure 10-112: Flood Duration Maps for 100-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

10.5.7 FPLOS Rating of East Lake Toho LMA

Table 10-68 provides a summary of evaluation of East Lake Toho LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The FPLOS rating for the LMA ranges from <5-yr (PM#3) to 25-yr (PM#1) under both current and future conditions.

Table 10-68: Summary of PM Analysis in East Lake Toho LMA

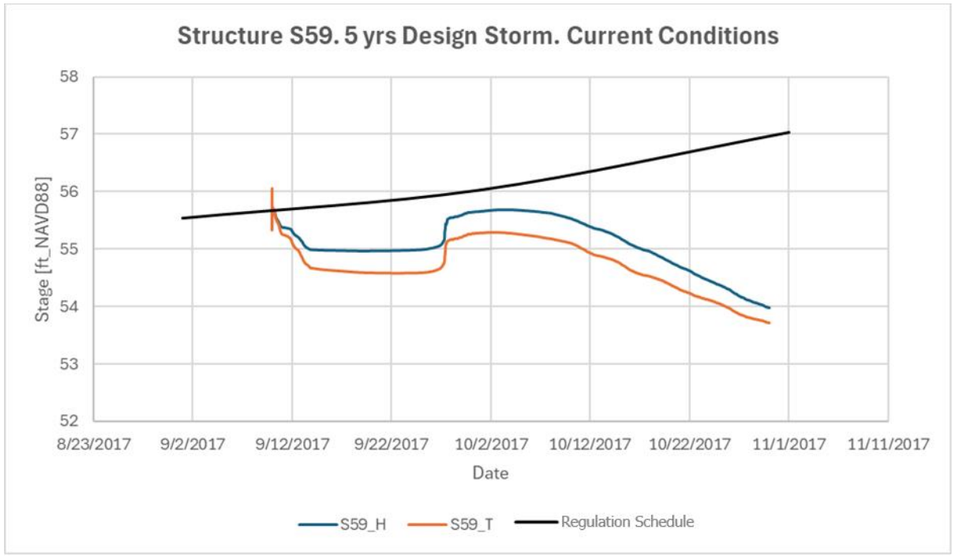
Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	East Lake Toho	At East Lake Toho, lake levels do not exceed the banks 5-, 10, and 25-yr events under current and future conditions.	25-yr	25-yr
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 7.3 CSM under current conditions and 7.9 CSM under future conditions, which are both less than SFWMD's permissible discharge allowance of 16.1 CSM based on a 10-yr design frequency.	10-yr discharge less than permissible discharge	10-yr discharge less than permissible discharge
PM#3	Entire LMA	During both peak tailwater and peak headwater, the simulated flow at structure S59 is higher than the design discharge (820 cfs) for all design storm events in the current and future conditions.	< 5-yr	< 5-yr
PM#4-UKB	-	-	-	-
PM#5	Entire LMA	For 5- and 10-yr design events under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 52% of the LMA is inundated more than 0.25-ft for a 25-yr design event and approximately 66% of the LMA is inundated more than 0.25-ft for a 100-yr event under both current and future conditions.	10-yr	10-yr
PM#6	Entire LMA	Approximately 67% of the East Lake Toho LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 54% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm.	5-yr	5-yr
East Lake Toho LMA Overall FPLOS		**Current Conditions**		**5-yr**
		Future Conditions		**5-yr**

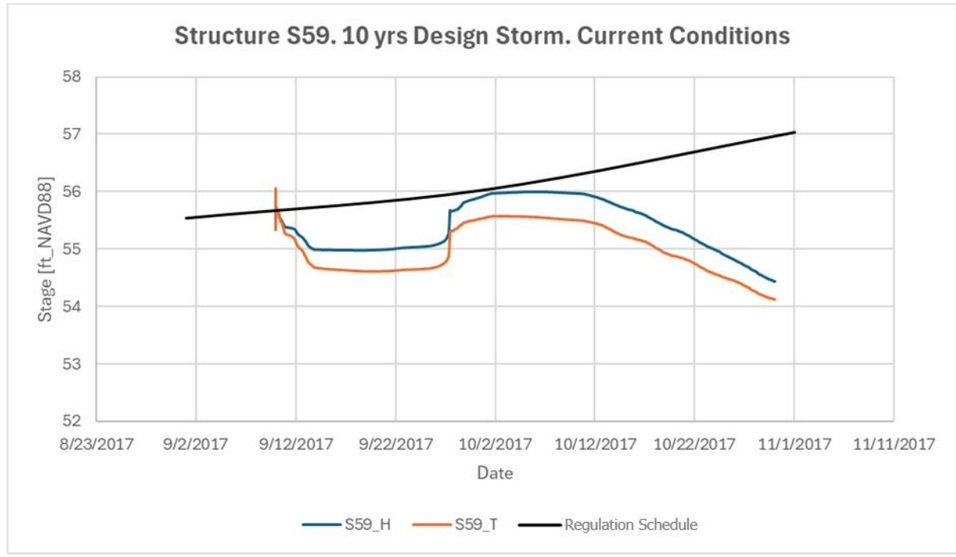
10.5.8 Volume in Lake due to Design Storm UKB-V_{ETOho}: East Lake Toho

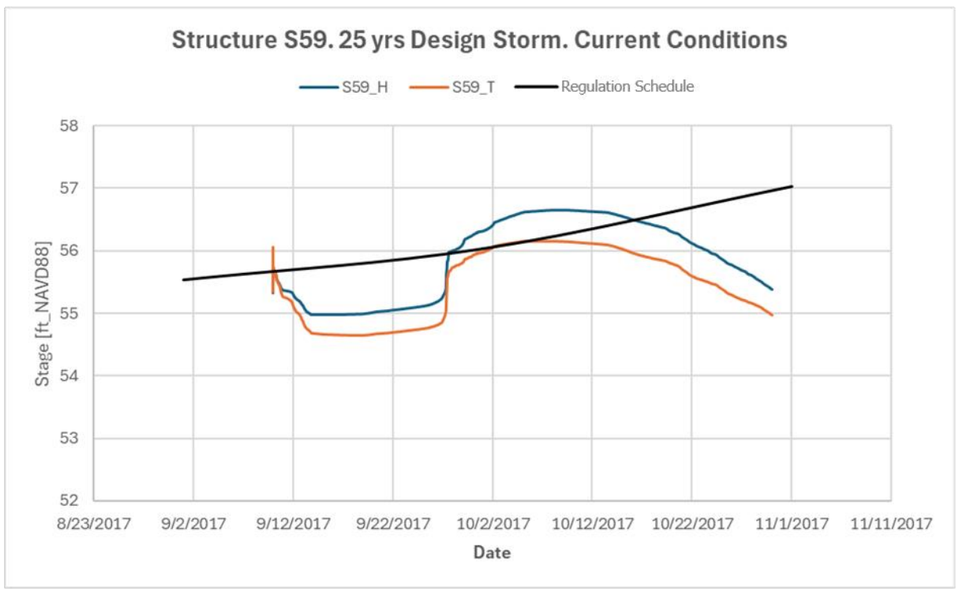
A summary of the change in water volume in Lake East Toho is shown in **Table 13-3**, calculated as the final volume at the end of storm event minus the initial volume at the start of the simulation. It can be observed that the volume is higher under future conditions than under current conditions for each set of design storms, which is reasonable because of the higher rainfall under future conditions. **Table 13-4** shows the volume of water at peak.

10.5.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}: East Lake Toho

This section examines the water level and regulation schedule in East Lake Toho by using its main regulation structure, S59. **Figure 10-113 and Figure 10-114** display headwater and tailwater stages at S59, as well as the regulation schedule for the whole simulation period, under current and future conditions respectively. The water levels in this lake do not exceed the regulation schedule for 5- and 10-yr design event simulations under current conditions. For a 100-yr design event under current conditions, the lake takes over 2 weeks to return to the regulation schedule and under future conditions, the lake remains above the schedule till the end of simulation. The system response to the 25-yr events falls in between the timelines of the smaller events and the 100-yr event.







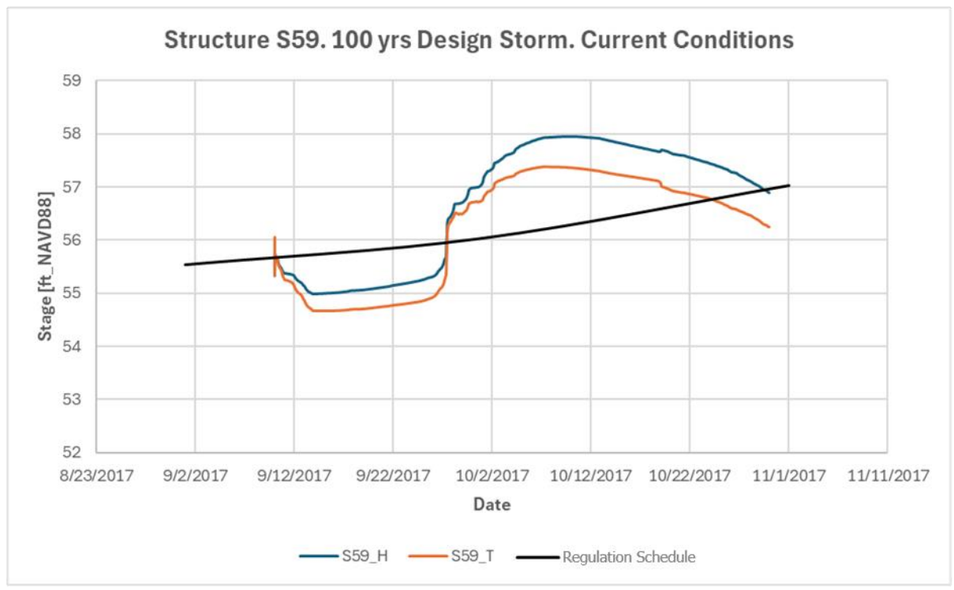
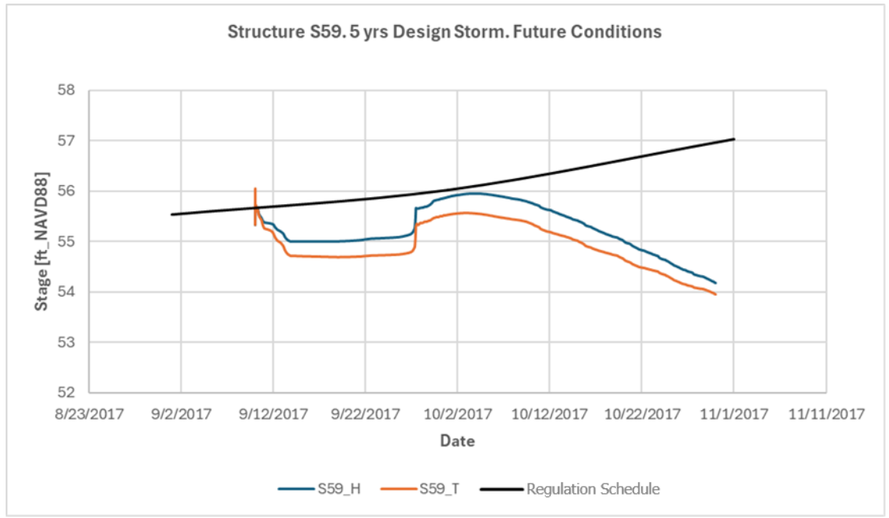
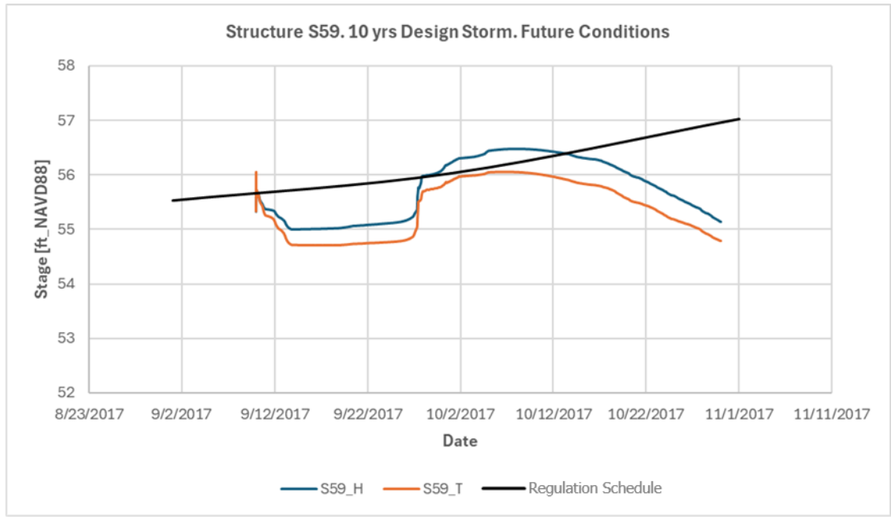
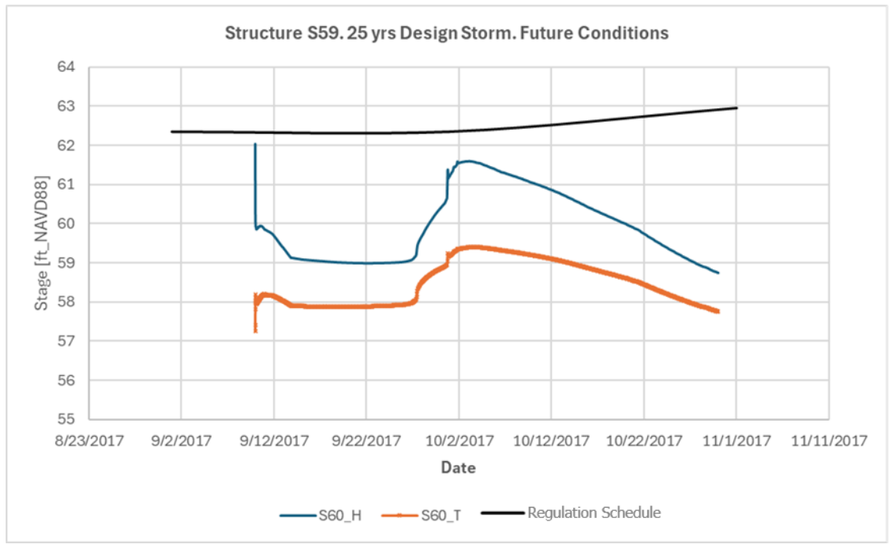


Figure 10-113: Headwater and Tailwater at Structure S59 vs. East Lake Toho Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







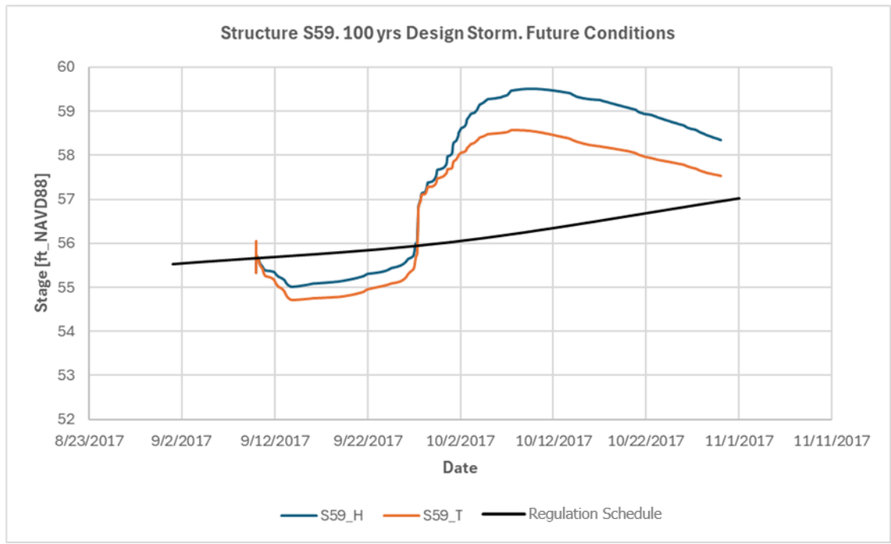


Figure 10-114: Headwater and Tailwater at Structure S59 vs. East Lake Toho Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Future Conditions

10.6 Lake Toho LMA

Figure 10-115 shows Lake Toho LMA with the primary conveyance features and their structures, and the roadways.

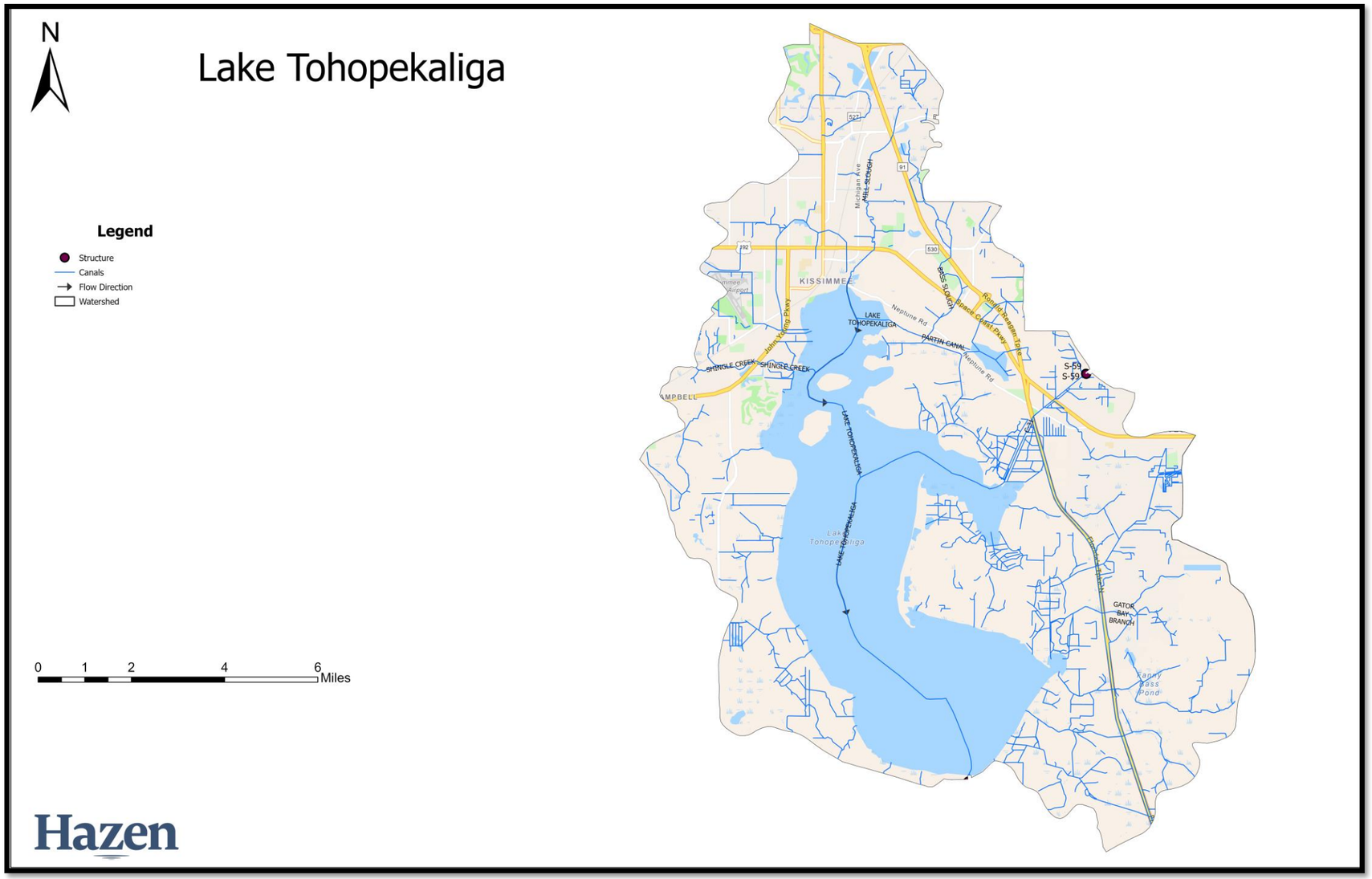


Figure 10-115: Lake Tohopekaliga LMA with Main Structures, Waterways and Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.6.1 PM #1

Performance of the system at Lake Toho LMA was assessed at Lake Toho and C-35 canal, shown in **Figure 10-116 through Figure 10-119** under current and future conditions respectively. The water levels in Lake Toho overtops the banks in most locations for all design storms under current and future conditions; along Canal C-35, water levels exceed the banks for all the simulated current and future events. This is due to the definition of the canal banks; and needs verification.

Summarizing Lake Toho FPLOS, under current and future conditions, both the left and right banks experience significant fluctuations, with the right bank frequently exceeding the bank elevations during all storms. The left bank also sees exceedances, although these are less frequent and less pronounced than on the right bank. In Canal C-35, under both current and future conditions, the banks are exceeded for all the storm events, with water levels remaining above the bank elevations across all scenarios.

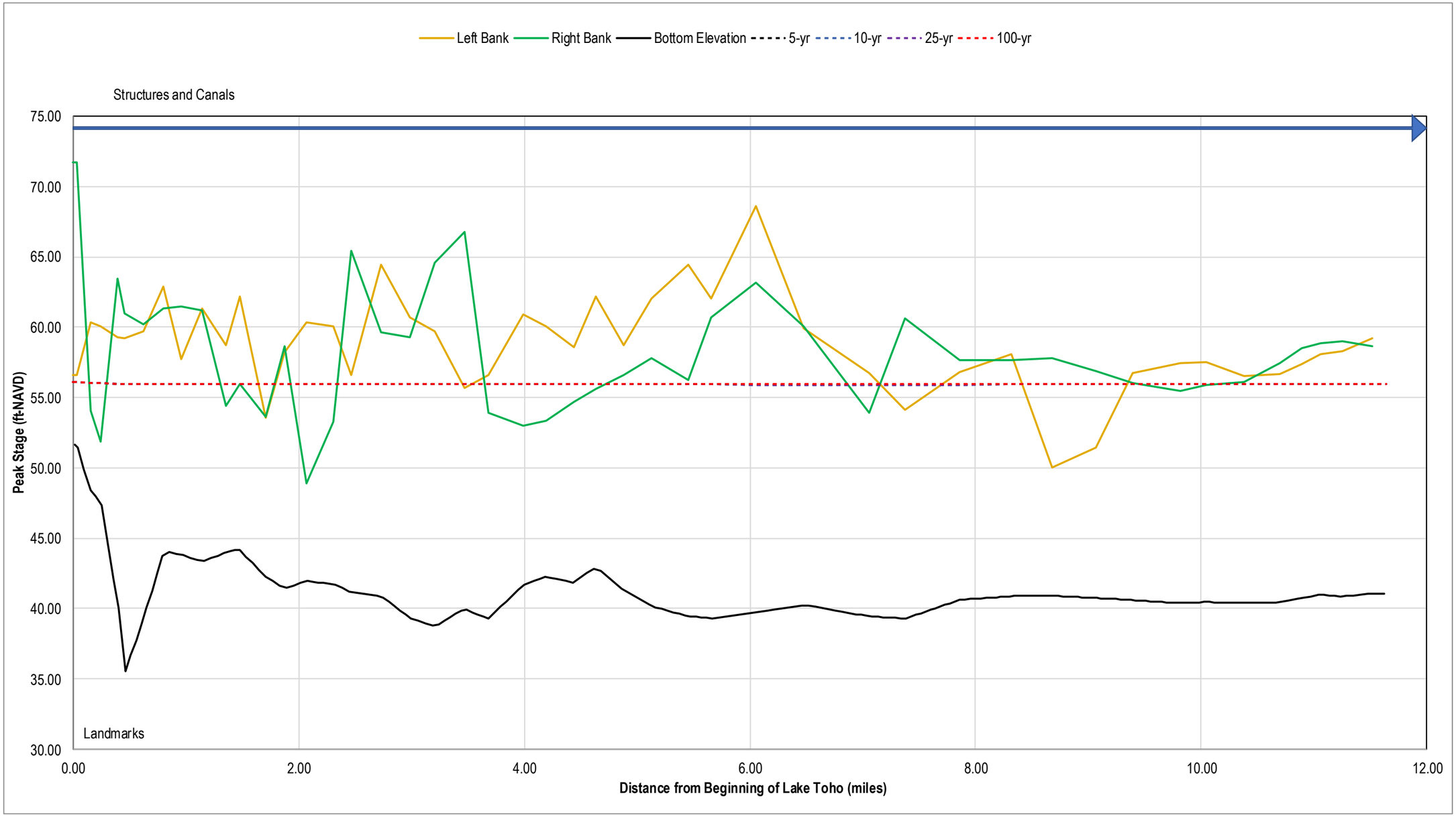


Figure 10-116: Maximum Stage Profile for Lake Toho under Current Conditions

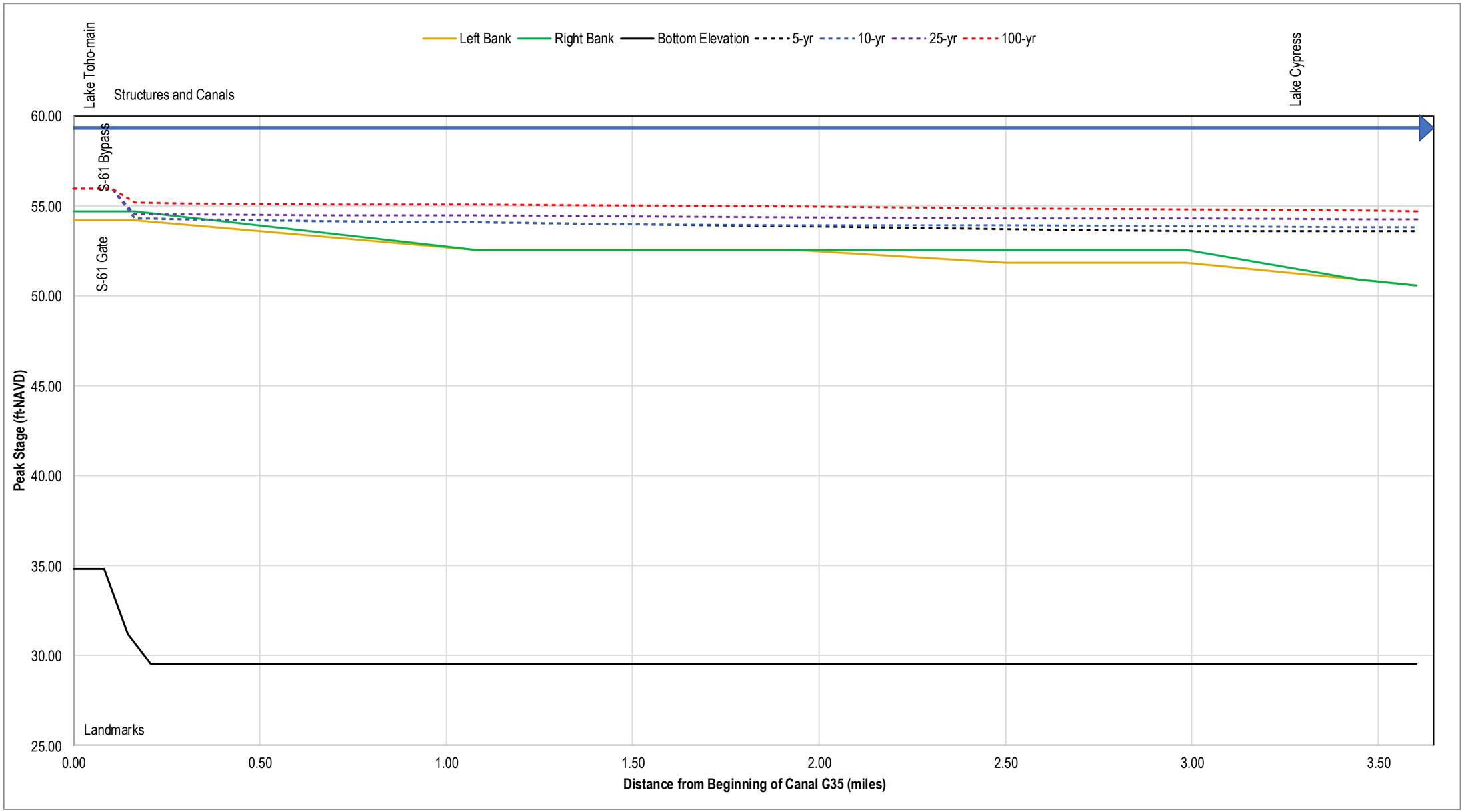


Figure 10-117: Maximum Stage Profile for Canal C-35 under Current Conditions

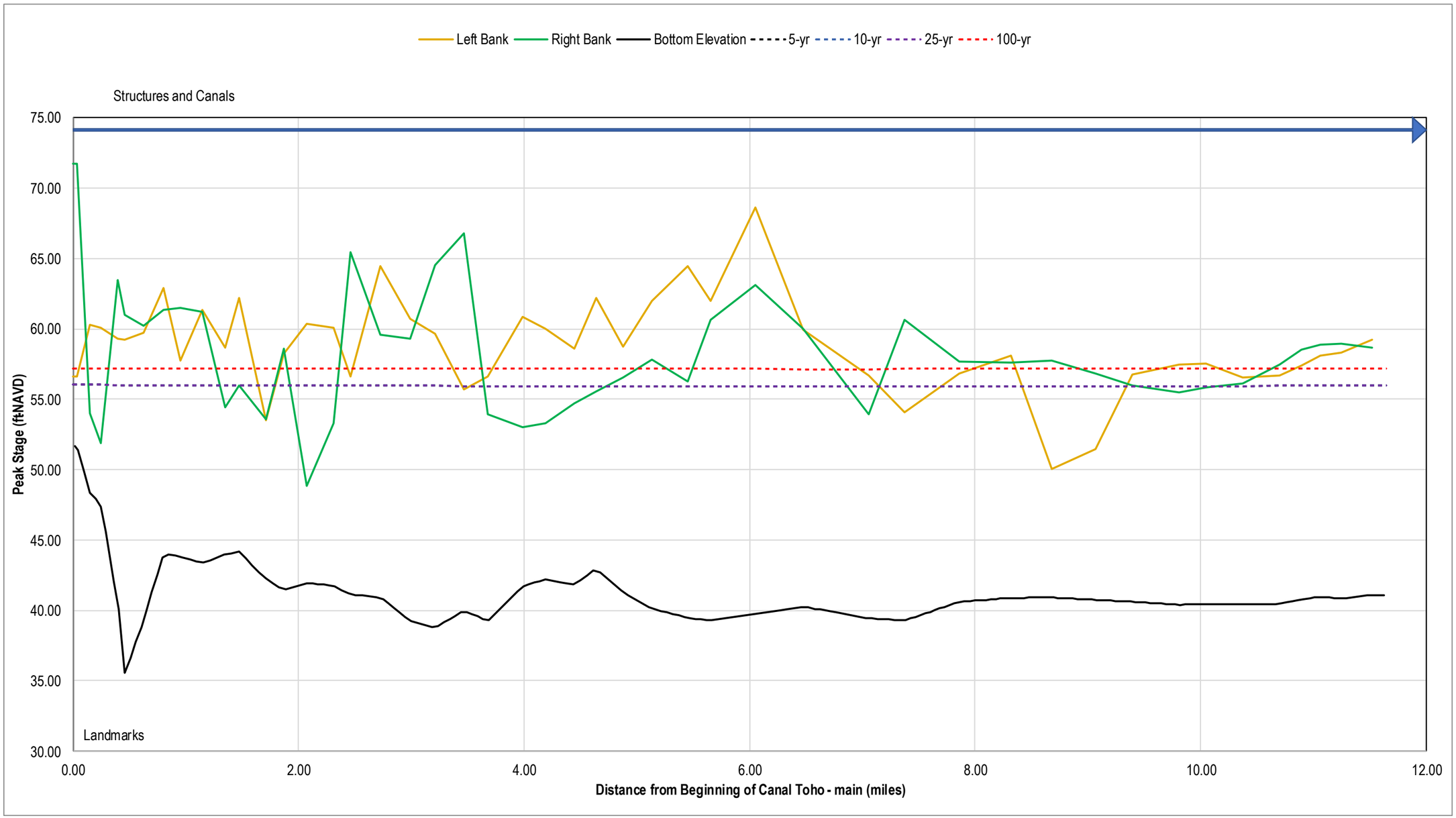


Figure 10-118: Maximum Stage Profile for Lake Toho under Future Conditions

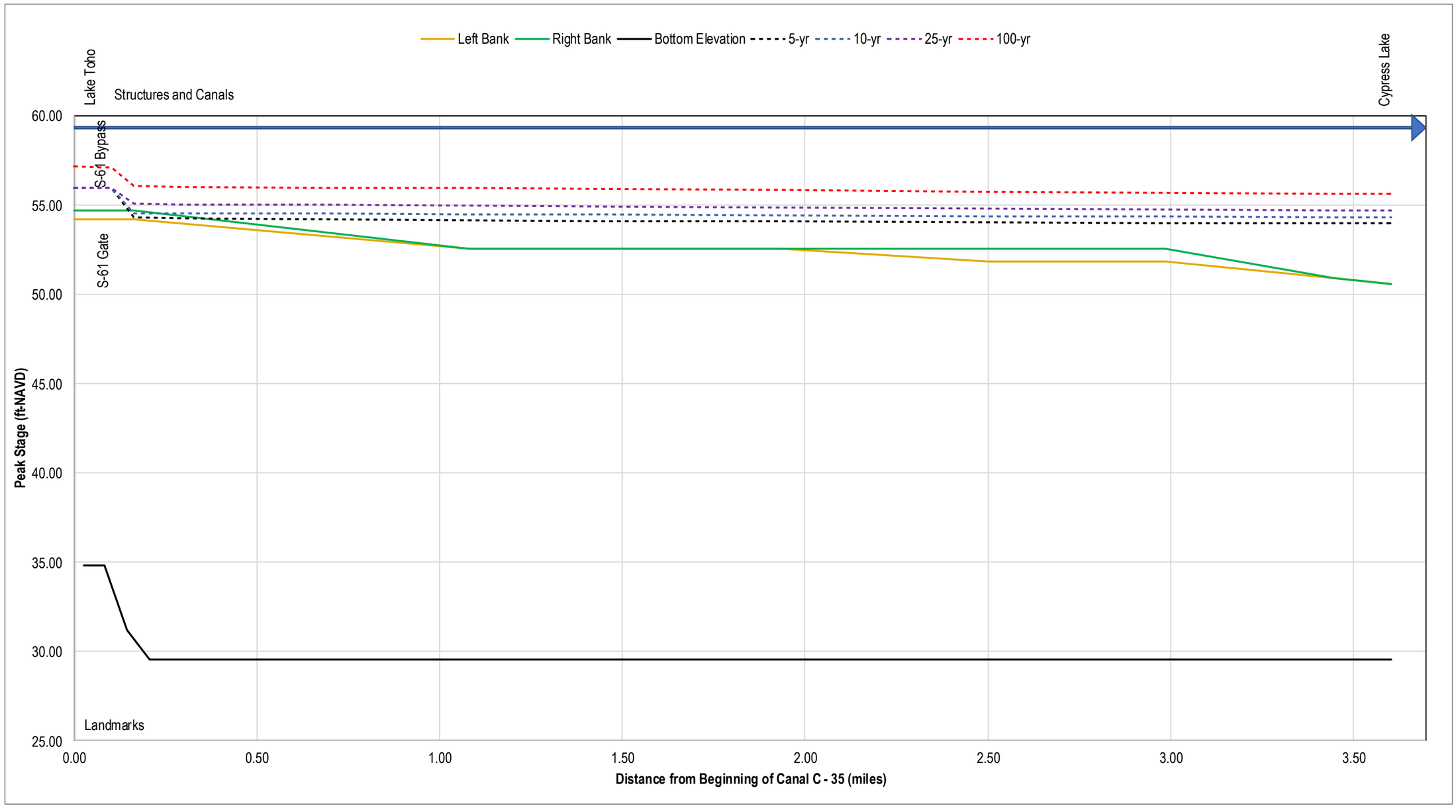


Figure 10-119: Maximum Stage Profile for Canal C-35 under Future Conditions

10.6.2 PM #2

Lake Toho LMA's upstream water control structure is S59, and downstream structure is S61. The maximum daily discharge capacity of this LMA is determined by S61 discharge capacity. Per the Districts ERP Handbook, Lake Toho LMA has an allowable discharge rate of 17.5 CSM, based on a design frequency of 10-yrs. **Table 10-69 and Table 10-70** captures Lake Toho LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area under current and future conditions modeling. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 16.8 CSM and 16.2 CSM under current and future hydrologic conditions modeling, which are lower than SFWMD's permissible discharge allowance of 17.5, based on a 10-yr design frequency.

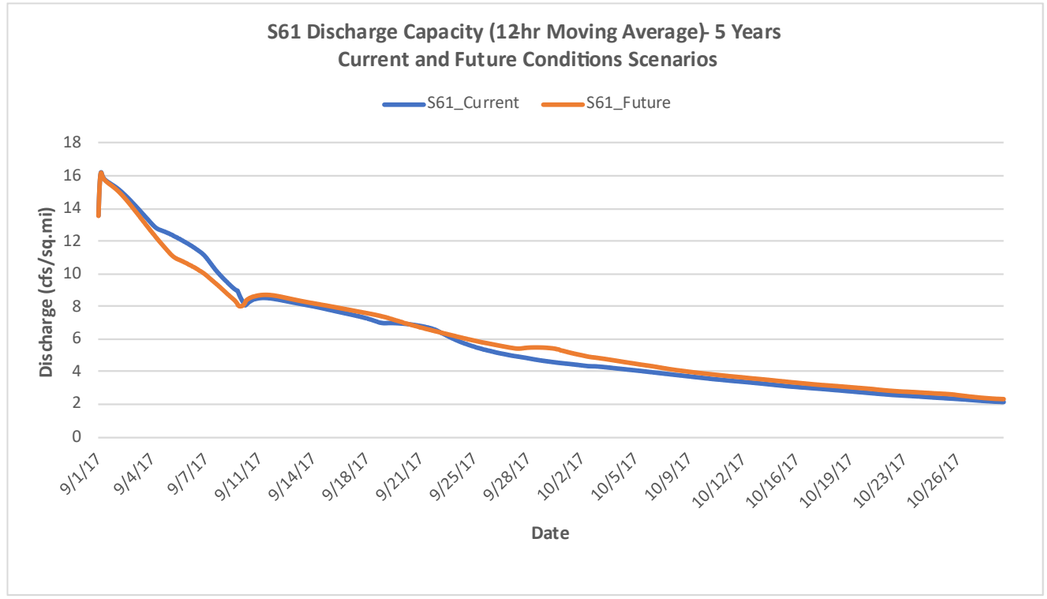
Table 10-69: Drainage Discharge Capacity (cfs/sq. mi) of Lake Toho LMA under Current Conditions

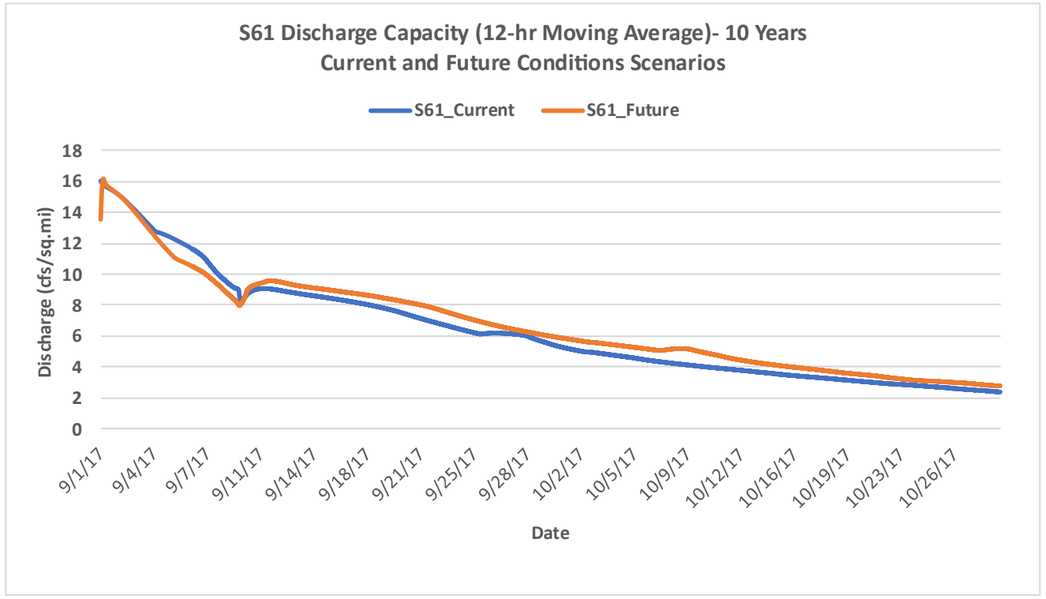
LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Toho	248.8	16.2	16.8	16.2	16.2	17.5

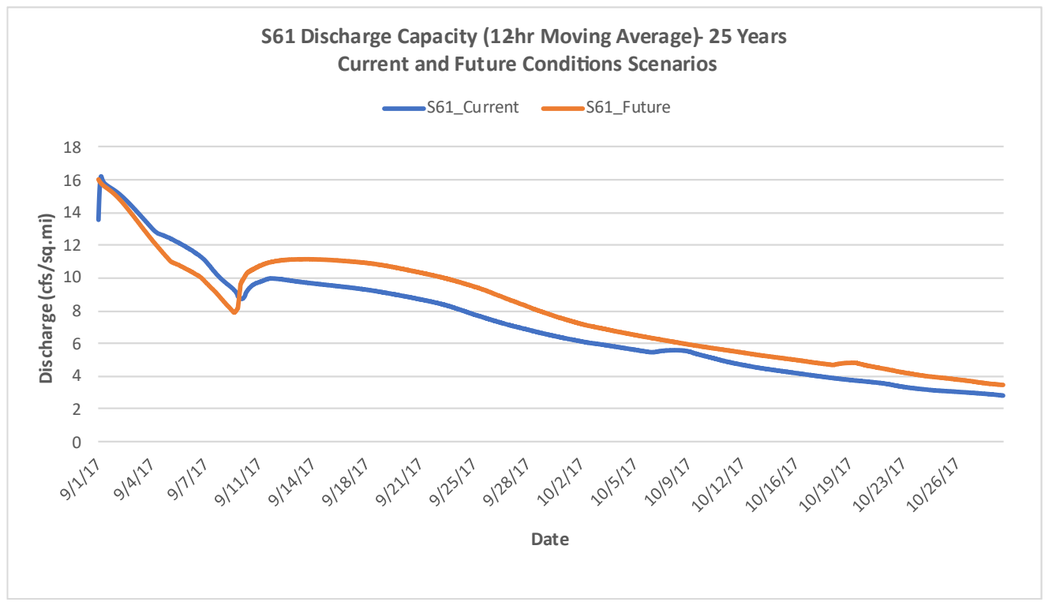
Table 10-70: Drainage Discharge Capacity (cfs/sq. mi) of Lake Toho LMA under Future Conditions

LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Toho	248.8	16.2	16.2	16.0	16.2	17.5

Figure 10-120 shows the peak 12-hour average discharge per square mile of drainage area, calculated for each design storm at structure S61. **Figure 10-121** shows the maximum daily discharge through structure S61 for both current and future conditions. The simulated peak 12-hour moving average discharge per square mile during the events 5 through 100-year and future for current conditions is 16 CSM which is lower to the district allowable discharge of 17.5 CSM for this structure, so this structure is expected to convey water properly under extreme design storms.







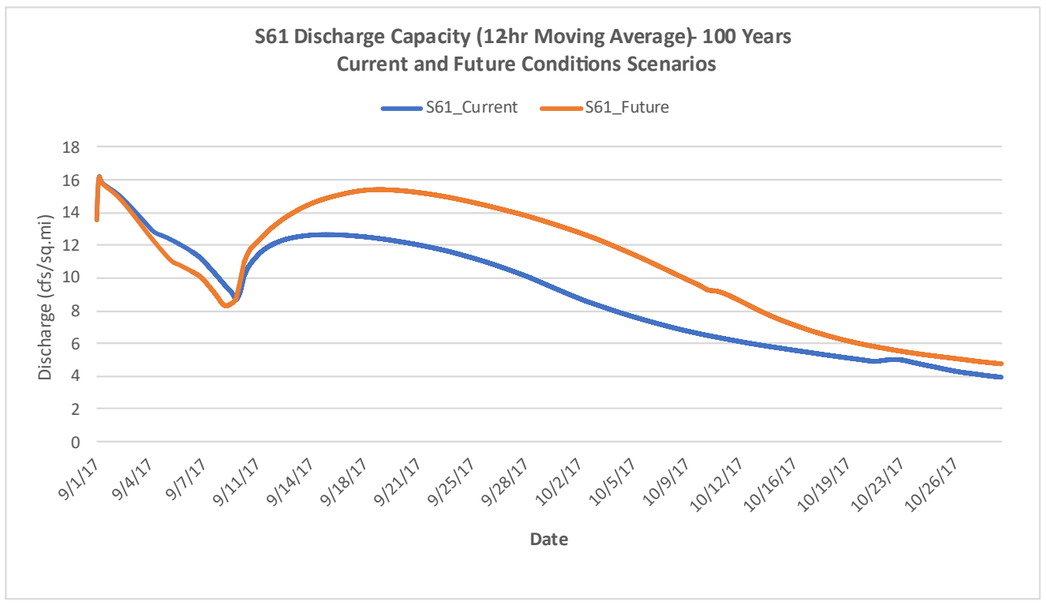


Figure 10-120: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S61 for 5-, 10-, 25-, 100- Year Design Storms

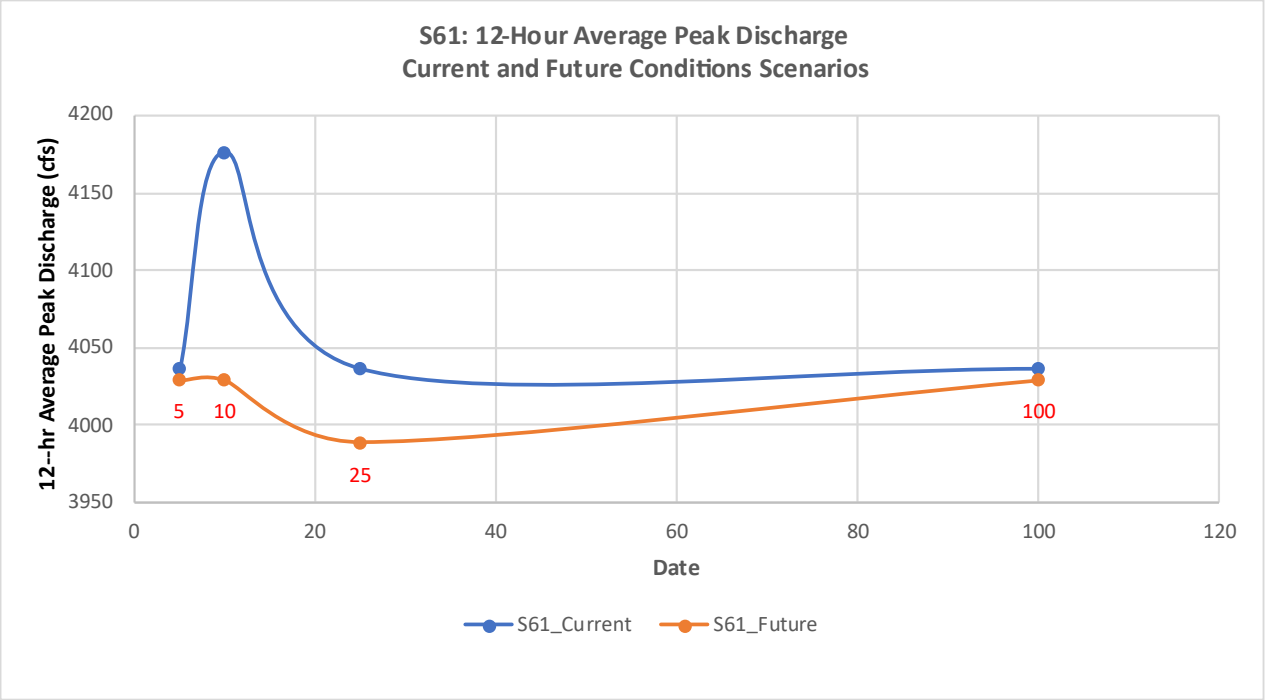


Figure 10-121: Structure S61 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

10.6.3 PM #3

The SFWMD structure design parameters (**Table 10-71**) were used to evaluate the S61 performance against the various design storms. **Table 10-72 through Table 10-78** provide a summary of current conditions and future conditions simulated instantaneous stages and flows at structure S61 at the time of peak tailwater, peak headwater and peak discharge. The results show that, at the time of peak tailwater at structure S61, the simulated headwater elevation at S61 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is higher than the design headwater elevation for both current and future conditions. The simulated flow at structure S61 is higher than the design discharge (2300 cfs) for almost all design storm events in the current and future conditions (except for 5-yr design storm).

During the peak headwater at the structure S61; the simulated tailwater elevation at S61 is higher than design tailwater elevation for all design storm events for current and future conditions. The simulated flows at structure S61 are higher than the design discharge (2300 cfs) for all the design storms.

During the peak flow at structure S61, the headwater and tailwater elevations are consistently higher than design headwater and tailwater for all design storm events for current and future conditions. **Figure 10-122 and Figure 10-123** show the same as time series.

Table 10-71: Design Parameters for Structure S61

Structure	S61
Design Discharge	1,570 to 2,300 cfs (Lower Profile to Peak Stage)
SPF Discharge	(30% SPF)
SPF Discharge	N/A
Design Headwater Elevation	51.95 to 53.65 feet NAVD88 (53.0 to 54.7 feet NGVD29)
Design Tailwater Elevation	51.75 to 53.25 feet NAVD88 (52.8 to 54.3 feet NGVD29)
SPF Headwater Elevation	58.85 feet NAVD88 (59.9 feet NGVD29)
SPF Tailwater Elevation	58.45 feet NAVD88 (59.5 feet NGVD29)
Optimum Headwater	50.95 to 53.95 feet NAVD88 (52.0 to 55.0 feet NGVD29)

Table 10-72: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	54.11	53.82	2121.06	0.29	9/12/17 1:00 AM
10-year	54.39	54.07	2225.30	0.32	9/13/17 2:25 AM
25-year	54.92	54.52	2420.83	0.40	9/14/17 7:00 AM
100-year	55.94	55.17	3152.82	0.76	9/15/17 12:00 PM

Table 10-73: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	54.47	54.17	2145.63	0.29	9/12/17 9:00 PM
10-year	54.89	54.53	2336.13	0.36	9/13/17 8:00 AM
25-year	55.61	55.05	2777.86	0.56	9/14/17 9:43 PM
100-year	57.13	55.96	3743.39	1.16	9/16/17 3:00 PM

Table 10-74: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Discharge at Peak TW (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at Peak TW (ft)	Date & Time
5-year	53.82	54.10	2102.42	-0.28	9/12/17 4:25 PM
10-year	54.08	54.39	2190.14	-0.31	9/13/17 9:00 PM
25-year	54.53	54.91	2391.30	-0.38	9/15/17 9:00 AM
100-year	55.17	55.93	3147.02	-0.76	9/15/17 4:00 AM

Table 10-75: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	54.18	54.46	2088.67	-0.28	9/13/17 12:00 AM
10-year	54.55	54.89	2266.07	-0.34	9/14/17 9:00 PM
25-year	55.05	55.61	2775.78	-0.56	9/15/17 2:25 AM
100-year	56.06	57.04	3502.25	-0.99	9/14/17 3:25 AM

Table 10-76: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at Peak Q (ft)	Date & Time
5-year	2261.44	53.43	53.36	0.07	9/10/17 12:10 AM
10-year	2286.36	53.47	53.40	0.07	9/10/17 12:10 AM
25-year	2495.57	54.87	54.45	0.43	9/12/17 7:00 AM
100-year	3153.99	55.93	55.17	0.76	9/15/17 8:00 PM

Table 10-77: Summary of the Instantaneous Conditions at Structure S61 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at Peak Q (ft)**	**Date & Time**
5-year	2170.95	54.45	54.15	0.30	9/11/17 11:00 PM
10-year	2396.95	54.87	54.48	0.39	9/12/17 2:00 AM
25-year	2777.92	55.61	55.05	0.56	9/14/17 8:00 PM
100-year	3843.86	57.04	55.79	1.24	9/19/17 10:00 AM

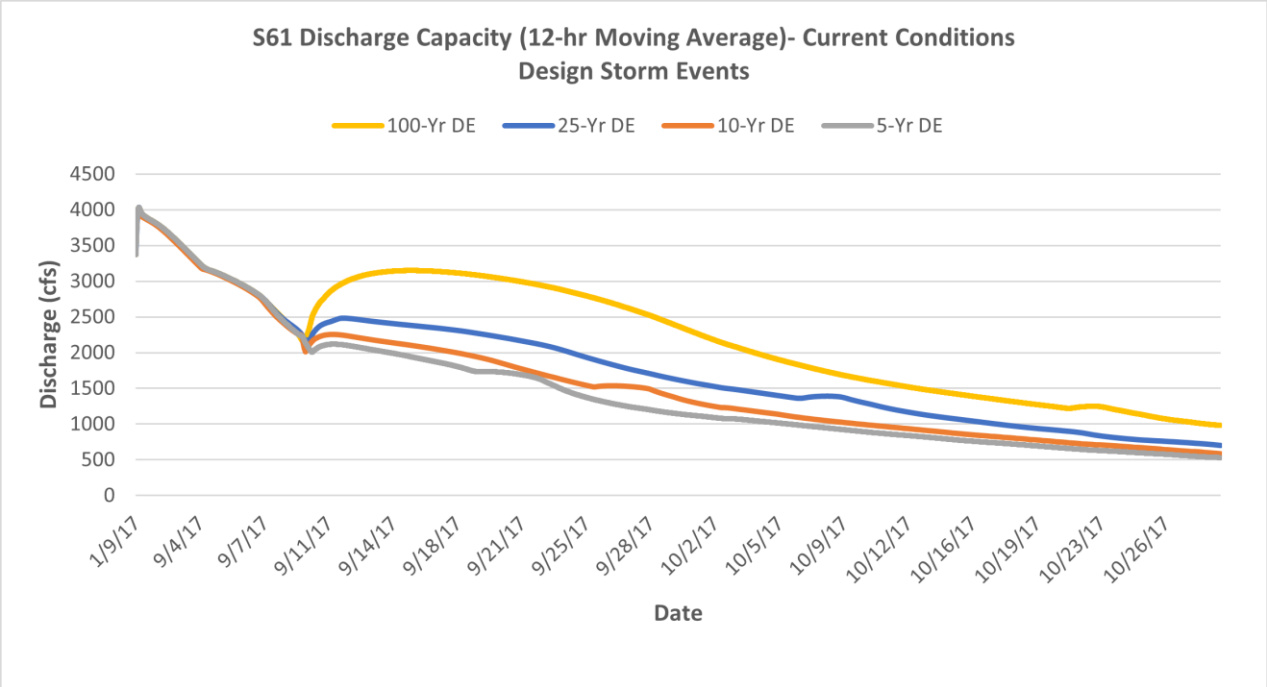


Figure 10-122: 12-hr Moving Average Flows in cfs at S61 under Current Conditions

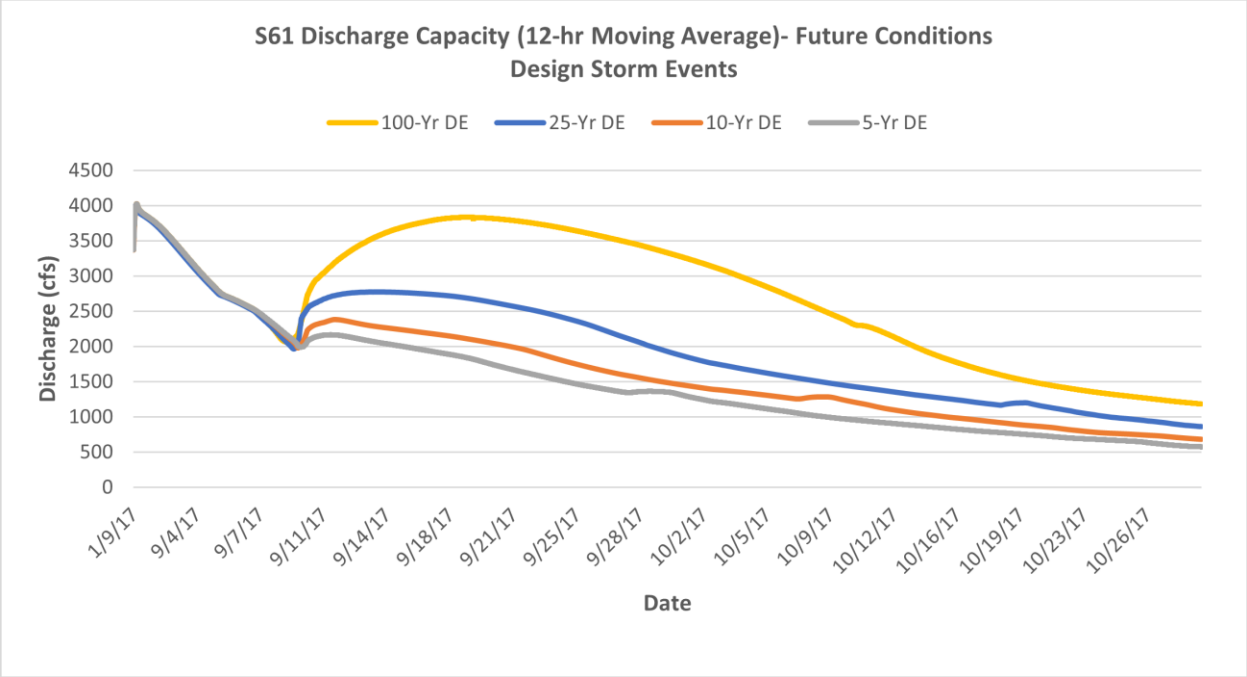


Figure 10-123: 12-hr Moving Average Flows in cfs at S61 under Future Conditions

10.6.4 PM #4

PM#4-UKB was not evaluated for this LMA.

10.6.5 PM #5

The gridded data of maximum overland depth was extracted for the simulated design storms under current and future conditions and evaluated for Lake Toho LMA under current and future conditions. **Table 10-78** shows the flood inundation area in square miles for this LMA, tabulated in incremental intervals of feet and **Figure 10-124** shows the distribution graphically. **Figure 10-125 through Figure 10-128** show the maximum overland flooding depths in the UKB FPLOS domain for the 5-, 10-, 25- and 100-yr design events, under current and future conditions for Lake Toho LMA.

Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not significant. There is also a high distribution of flooding depths greater than 2.5 feet, which indicates the lake depths. The ranges in between them are the location of small wetlands.

Table 10-78: Incremental Flood Inundation Area (sq. mi.) in Lake Toho LMA

Flooding Depth (ft)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Depth<0.25	146.77	134.56	113.34	79.20	146.75	134.55	113.30	79.09
0.25<=Depth<0.5	16.37	21.22	31.34	44.19	16.33	21.24	31.39	44.07
0.5<=Depth<0.75	9.72	9.03	10.30	14.14	9.77	8.99	10.26	14.04
0.75<=Depth<1.0	10.80	8.83	7.53	8.86	10.75	8.81	7.53	8.61
1.0<=Depth<1.25	6.49	9.04	7.29	6.82	6.55	9.07	7.29	6.60
1.25<=Depth<1.5	5.53	6.45	7.77	6.39	5.51	6.47	7.76	6.39
1.5<=Depth<1.75	4.25	5.27	6.89	5.90	4.24	5.24	7.03	5.83
1.75<=Depth<2.0	3.19	4.53	5.92	5.62	3.21	4.59	6.02	5.56
2.0<=Depth<2.25	2.69	3.81	4.93	5.26	2.68	3.78	4.91	5.20
2.25<=Depth<2.5	2.26	2.82	4.37	6.00	2.27	2.83	4.36	6.02
>2.5	40.82	43.33	49.22	66.52	40.84	43.33	49.03	67.48
Lake Toho LMA	**159291.30**	**acres**						
	248.9	**sq.mi.**						

Note: *The lake area of 44.60 sq. mi. was included in this table computations.*

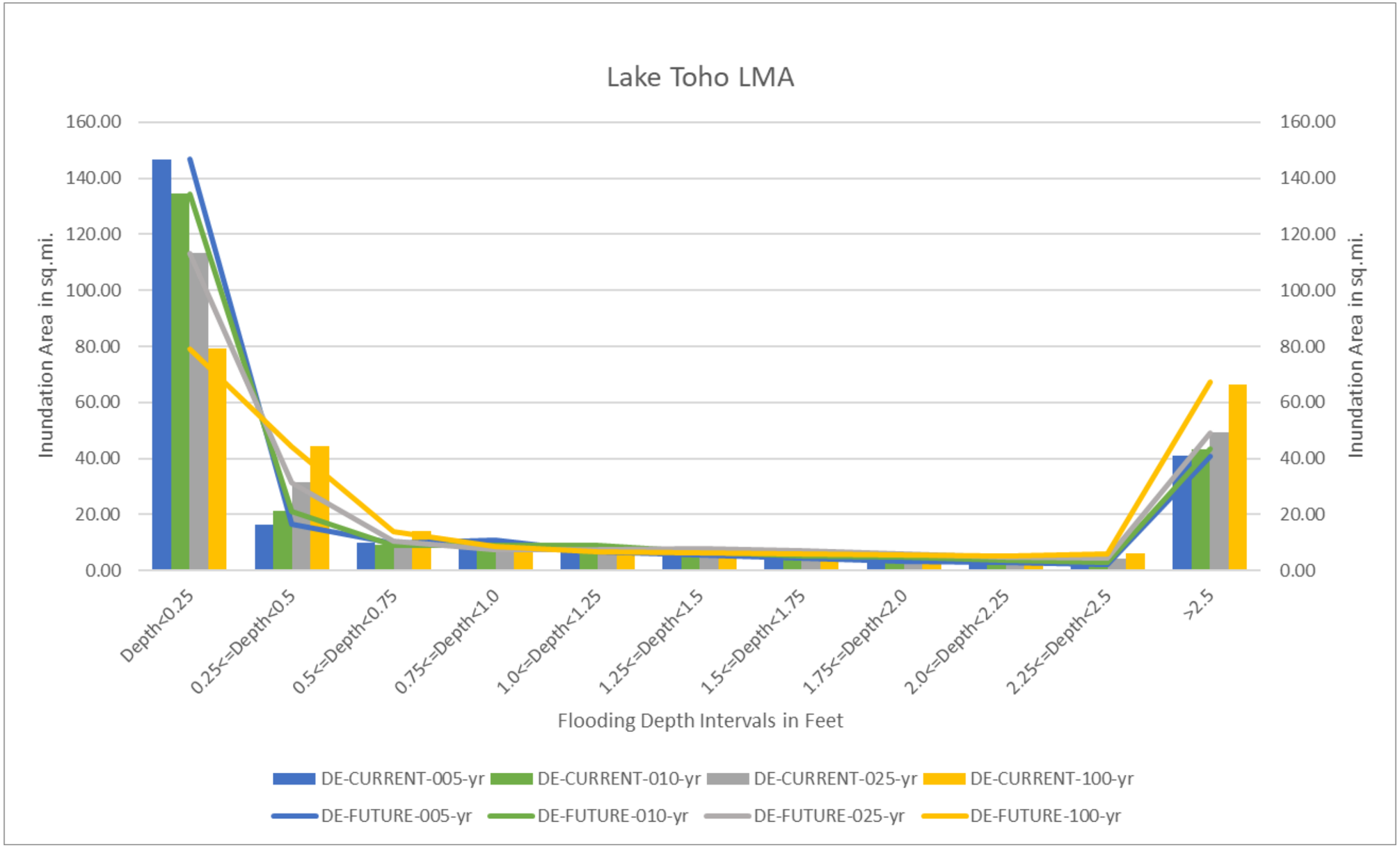


Figure 10-124: Incremental Flood Inundation Depth in feet in Lake Toho LMA under Current and Future Conditions

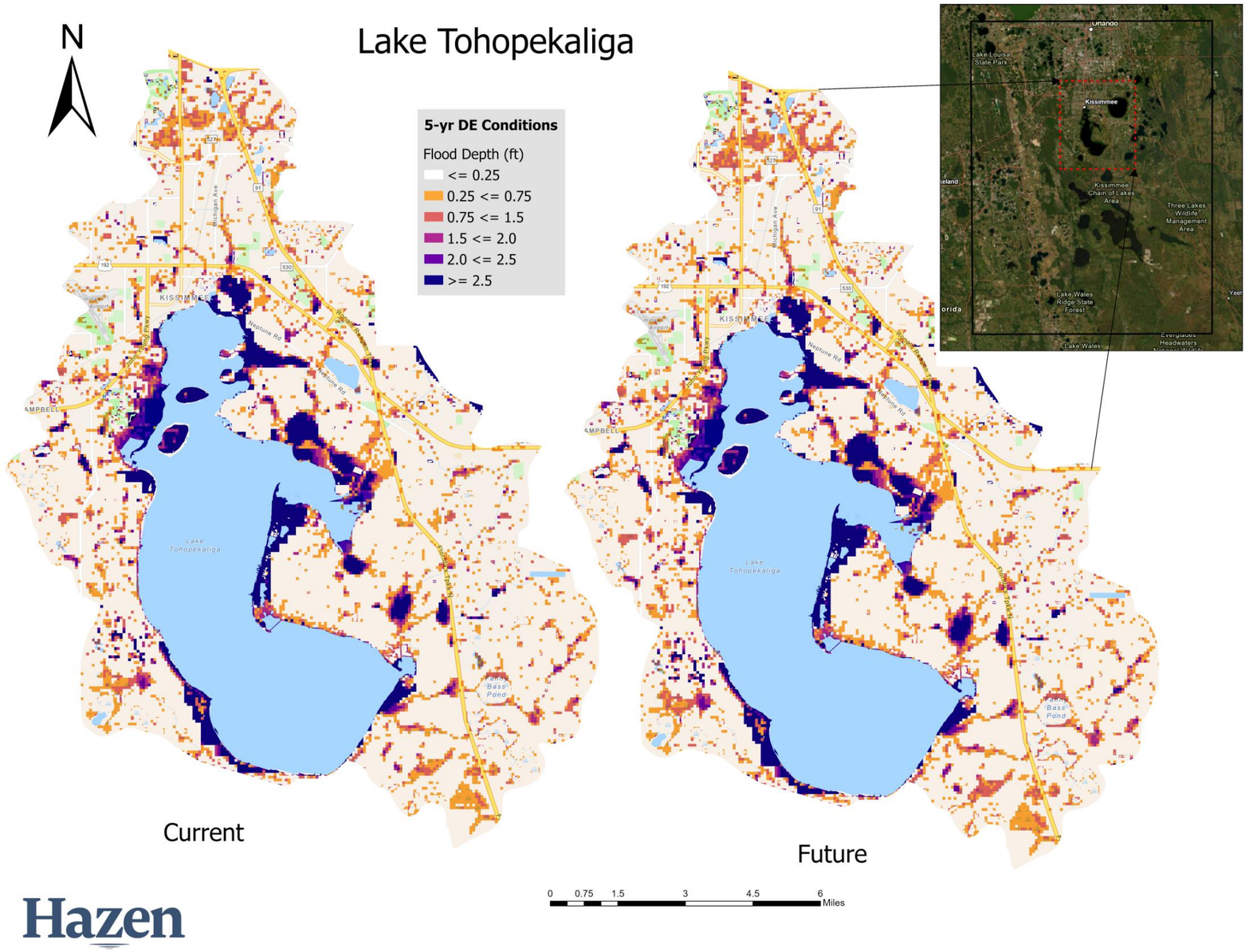


Figure 10-125: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA

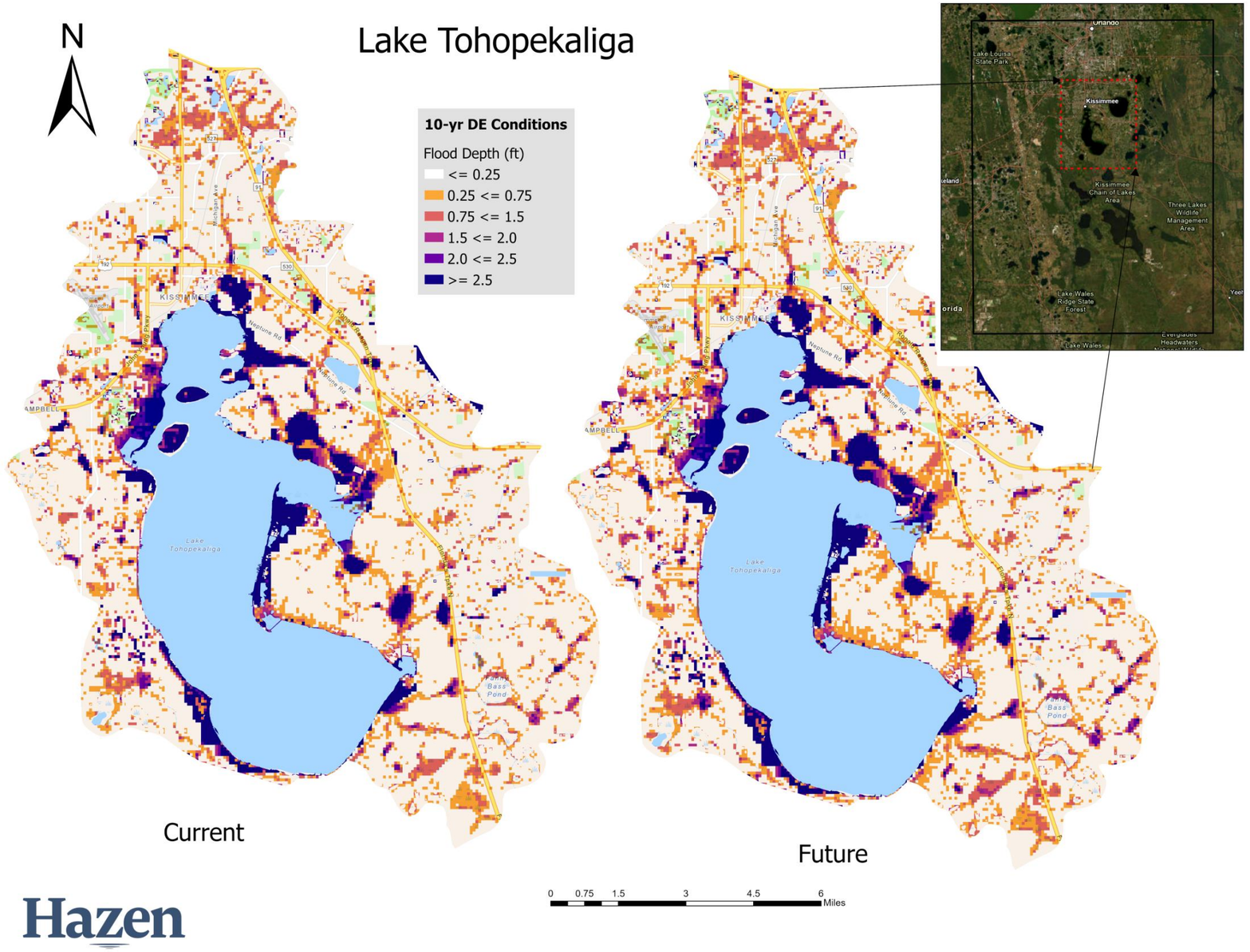


Figure 10-126: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA

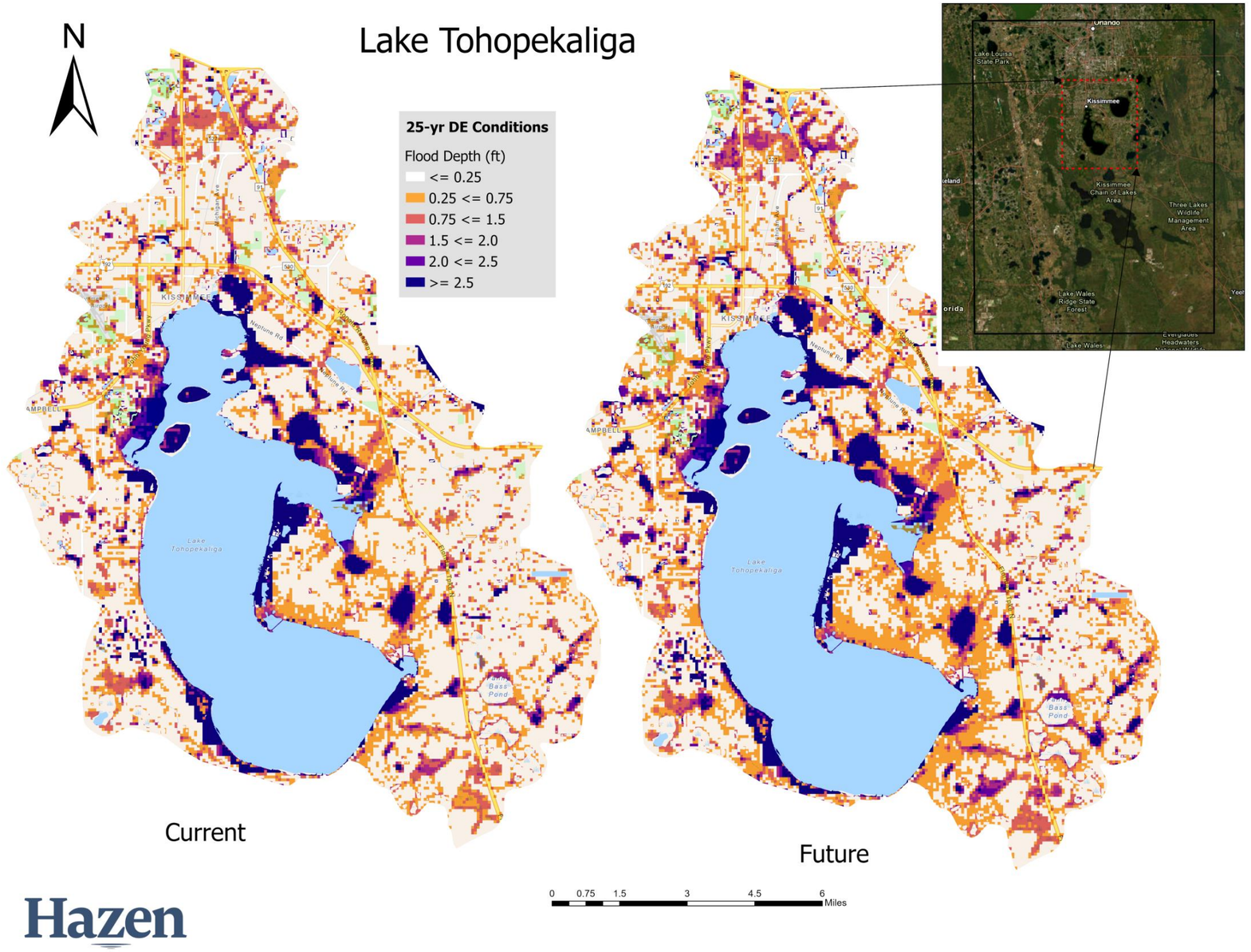


Figure 10-127: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA

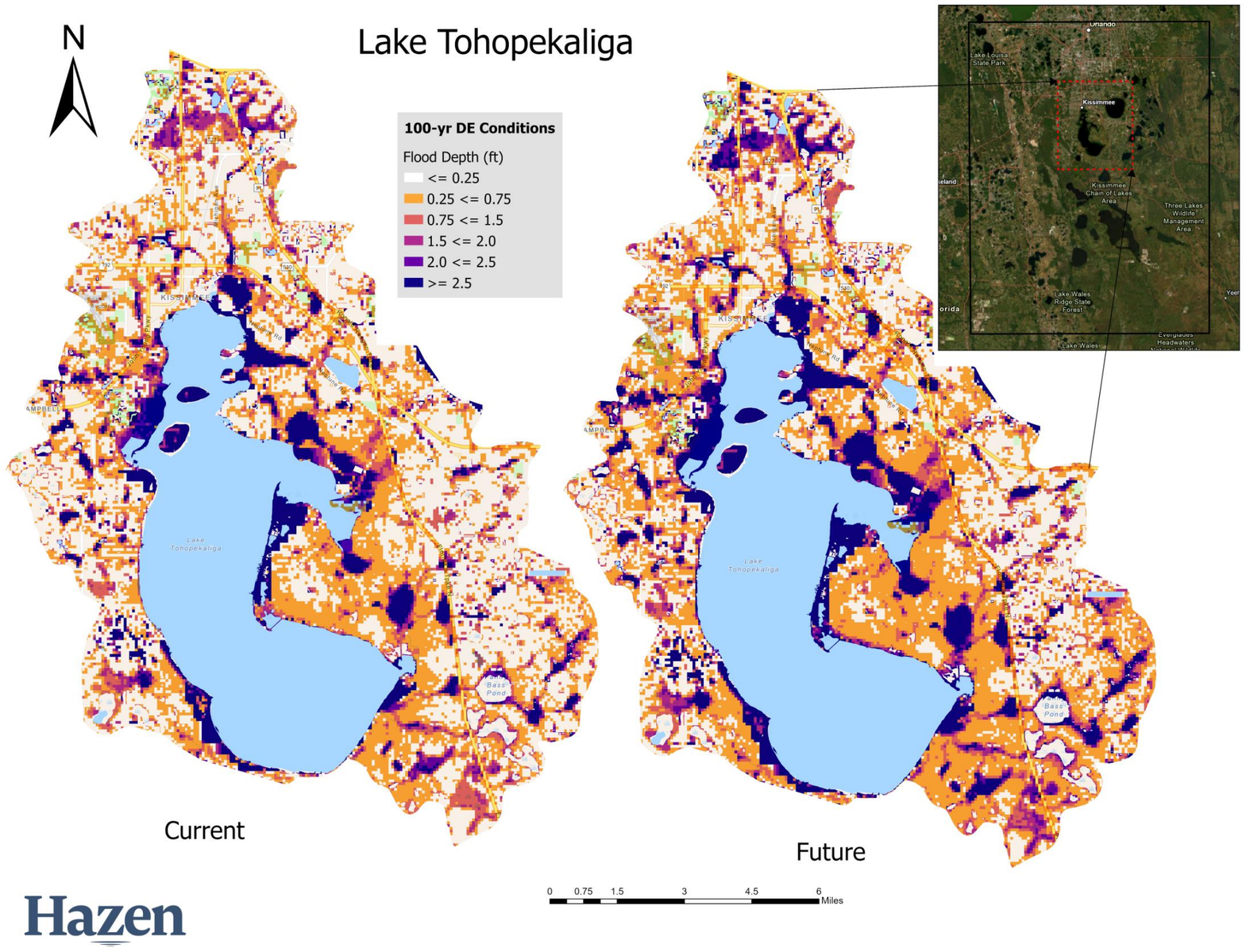


Figure 10-128: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA

10.6.6 PM #6

Table 10-79 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Toho LMA. **Figure 10-129** show these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Toho LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-130 through Figure 10-133** show the flood duration in Lake Toho LMA, except in Lake Toho which is expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration in Lake Toho LMA is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration in Lake Toho LMA are areas with longer than 360 hours of flooding, which are the lakes. The maps show that for the design storms, the simulations show that about 10% of the LMA takes between 4 and 96 hours to drain. These areas are on the fringes of the lakes and channels connecting the lakes.

Table 10-79: Areas of Incremental Flood Duration (sq. mi.) in Lake Toho LMA

Flooding Duration (hours)	De-Current 005-Yr	De-Current 010-Yr	De-Current 025-Yr	De-Current 100-Yr	De-Future 005-Yr	De-Future 010-Yr	De-Future 025-Yr	De-Future 100-Yr
Hours<1	154.16	145.25	129.95	98.79	154.16	145.23	129.96	98.59
1<=Hours<4	4.97	7.09	12.83	27.82	4.96	7.10	12.86	27.50
4<=Hours<8	2.00	2.62	3.23	4.47	2.00	2.62	3.26	4.21
8<=Hours<12	1.10	1.34	1.66	2.01	1.11	1.33	1.62	1.98
12<=Hours<24	2.75	3.29	4.29	5.57	2.82	3.26	4.14	5.50
24<=Hours<48	3.44	3.98	4.81	5.06	3.49	4.01	4.24	4.82
48<=Hours<96	4.92	5.40	5.41	6.25	4.74	4.66	4.98	6.28
96<=Hours<168	5.89	5.10	5.16	6.35	4.28	4.70	4.87	6.31
168<=Hours<240	3.55	3.77	4.03	4.43	2.94	3.47	3.87	4.47
240<=Hours<360	3.54	4.84	5.37	5.93	2.98	3.76	5.03	5.83
360<=Hours<1417	62.58	66.22	72.16	82.21	65.42	68.75	74.07	83.38
LAKE TOHO LMA	**159291.30**	**acres**						
	248.9	**sq.mi.**						

Note: *The lake area of 44.6 sq. mi. was included in this table computations.*

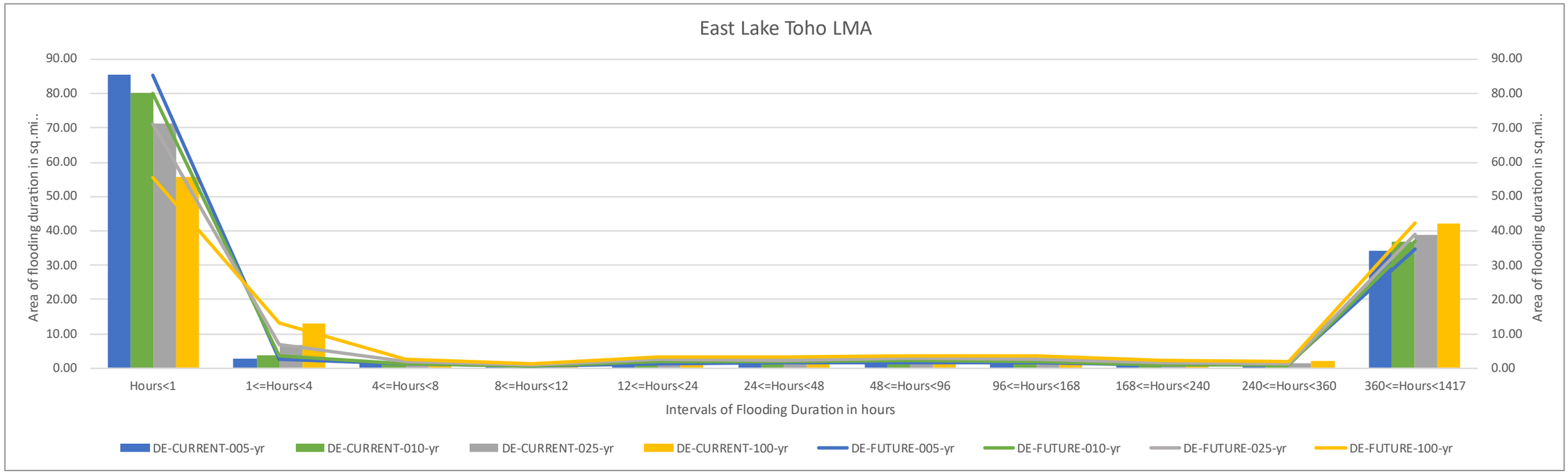


Figure 10-129: Areas of Incremental Flood Duration in hours in Lake Toho LMA under Current and Future Conditions

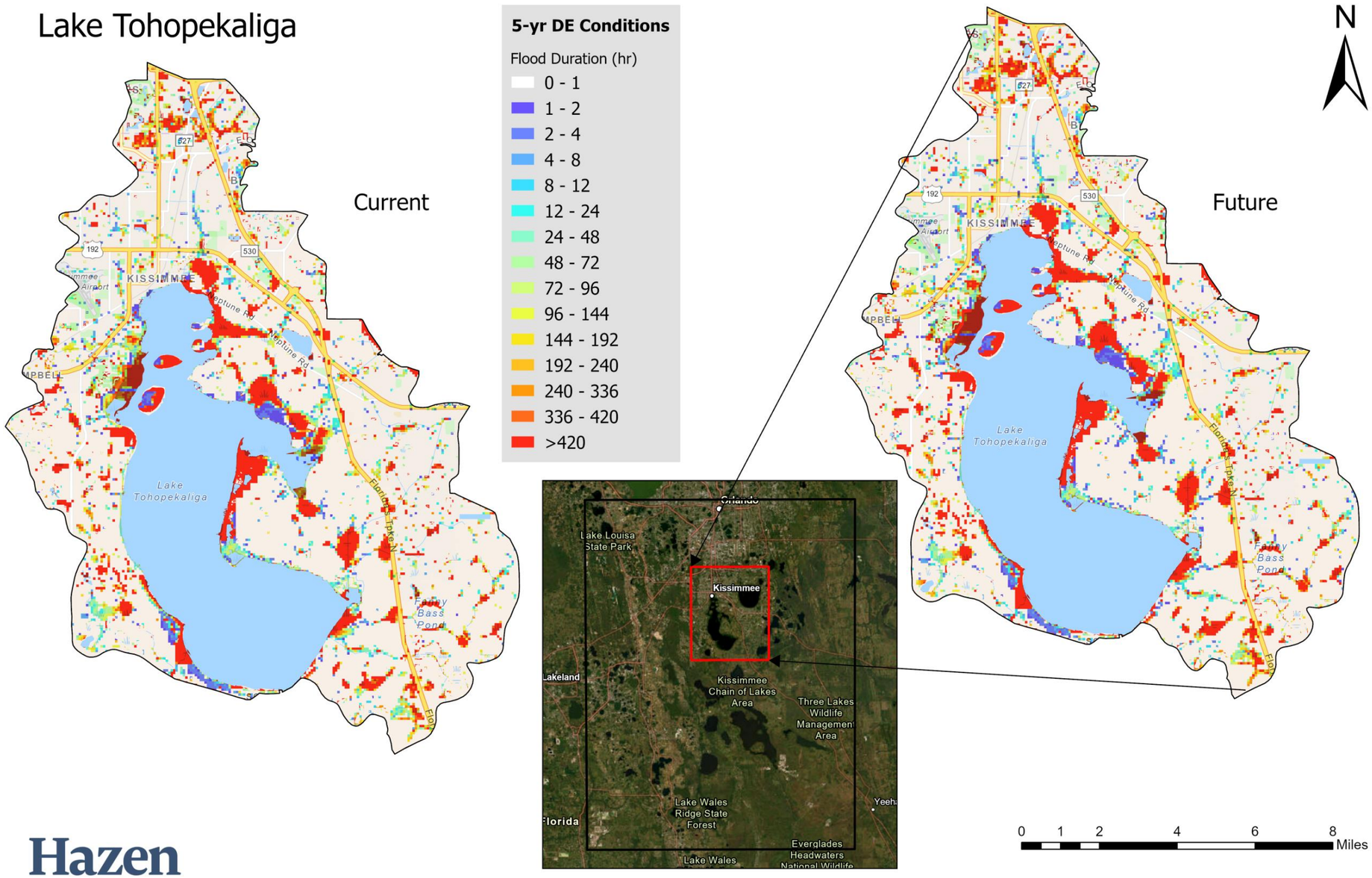
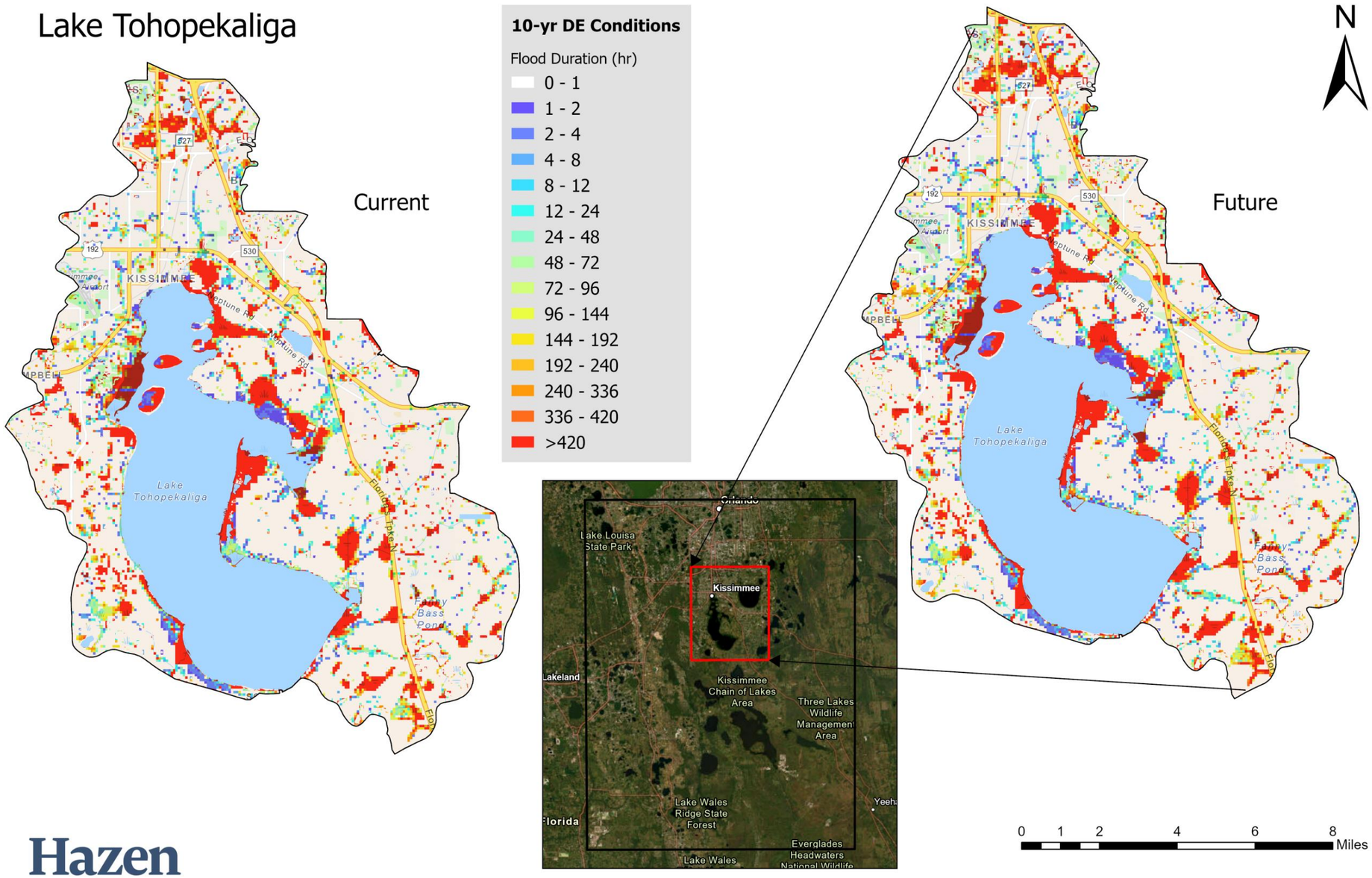


Figure 10-130: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions



Hazen

Figure 10-131: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions

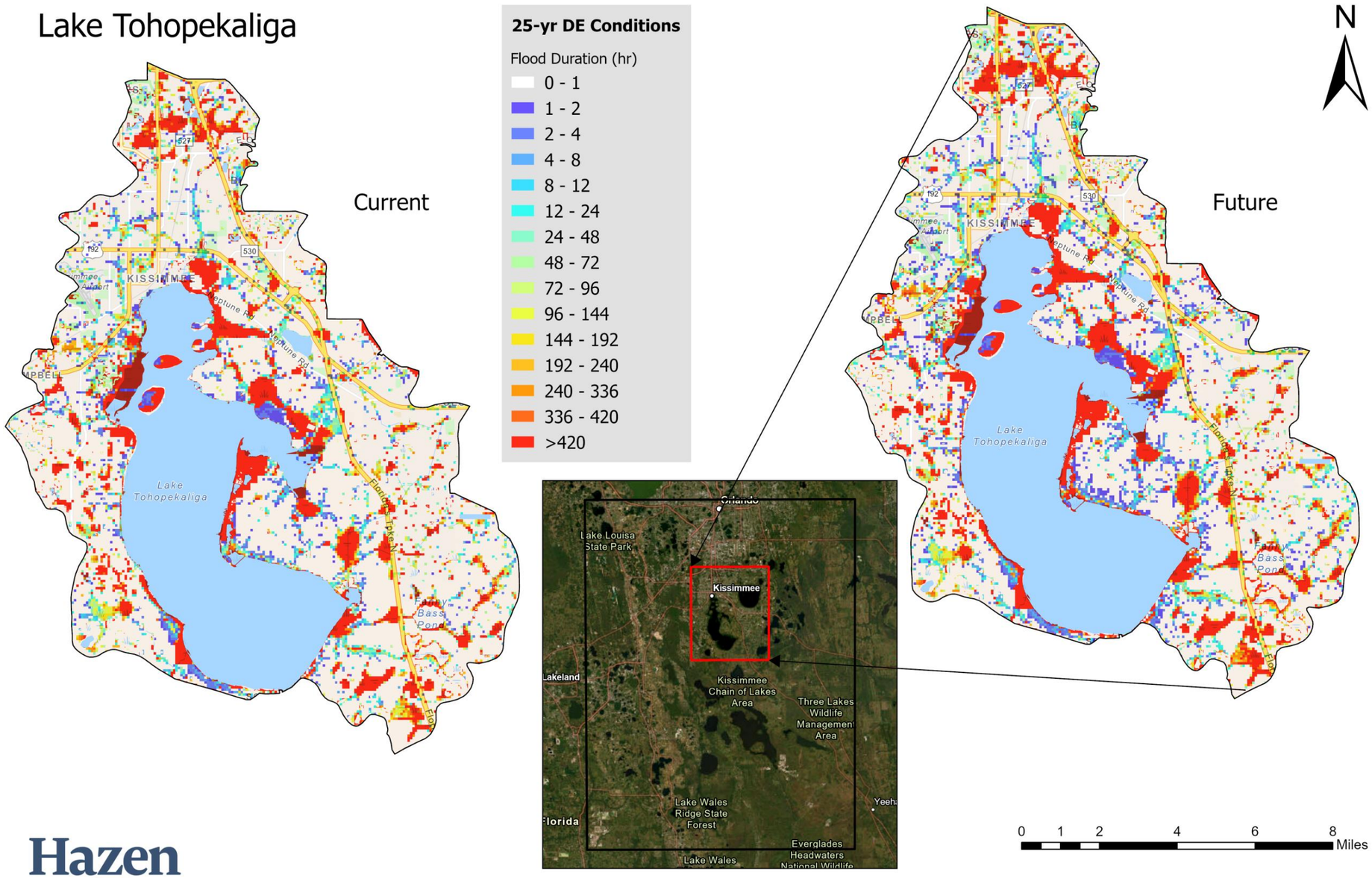


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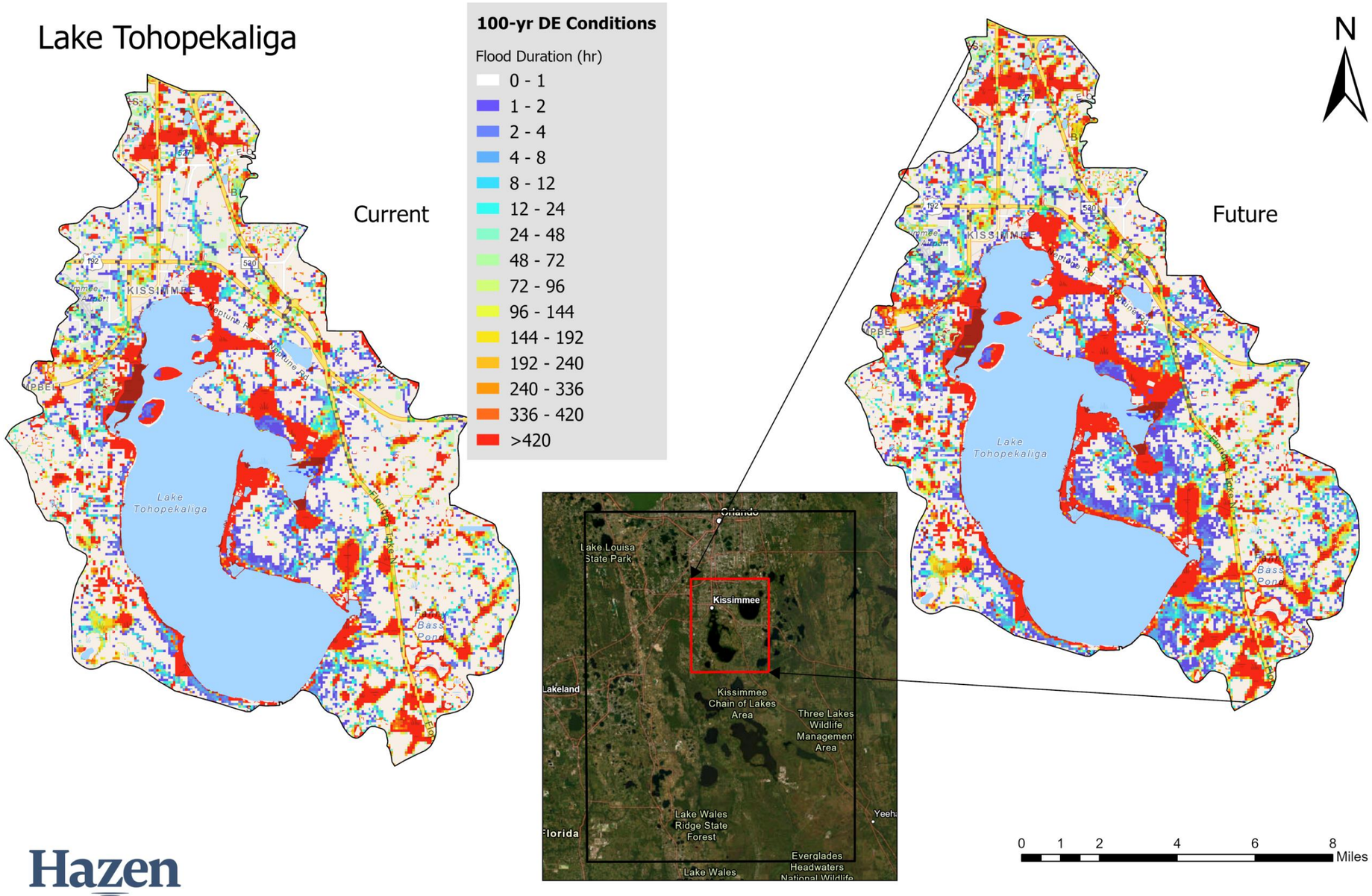
Figure 10-132: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions



Hazen

Figure 10-133: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions

10.6.7 FPLOS Rating of Lake Toho LMA

Table 10-80 provides a summary of evaluation of Lake Toho LMA performance using FPLOS metrics to provide flood protection under current and future conditions. Lake Toho is assigned a FPLOS rating of <5-yr to 10-yr, and an overall rating of <5-yr, under current and future conditions.

Table 10-80: Summary of PM Analysis in Lake Toho LMA

Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Toho, C-35 canal	The water levels are exceeded at banks within Lake Toho in most locations in current and future conditions. Water levels exceed the banks almost all along C-35 canal for all storms.	<5-yr in Lake Toho < 5-yr in Canal C-35	<5-yr in Lake Toho < 5-yr in Canal C-35
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 16.2 CSM under current conditions and 16.2 CSM under future conditions, which are both less than SFWMD's permissible discharge allowance of 17.5 CSM based on a 10-yr design frequency.	10-yr discharge less than permissible discharge	10-yr discharge less than permissible discharge
PM#3	Entire LMA	During peak tailwater, the simulated flow at structure S61 is higher than the design discharge (2300 cfs) for all design storm events in the current and future conditions, except for 5-yr design storm. During peak headwater, the simulated flow at structure S61 is higher than the design discharge (2300 cfs) for all design storm events in the current and future conditions.	< 5-yr	< 5-yr
PM#4-UKB	-	-		
PM#5	Entire LMA	For 5- and 10-yr design events under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 54% of the LMA is inundated more than 0.25-ft for a 25-yr design event and approximately 68% of the LMA is inundated more than 0.25-ft for a 100-yr event under both current and future conditions.	10-yr	10-yr
PM#6	Entire LMA	Approximately 65% of the Lake Toho LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 53% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm.	5-yr	5-yr

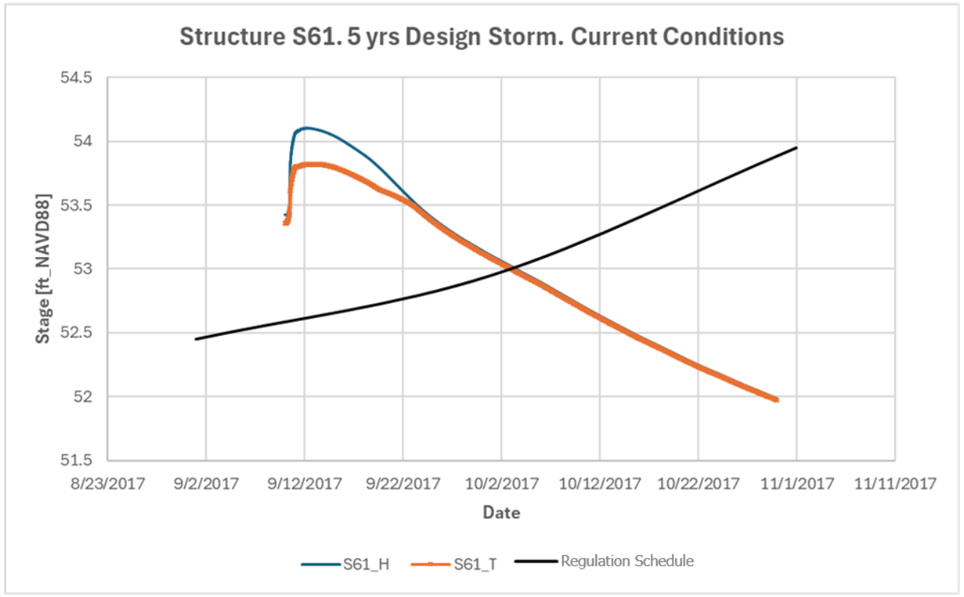
Lake Toho LMA Overall FPLOS	**Current Conditions**	**< 5-yr**
	Future Conditions	**< 5-yr**

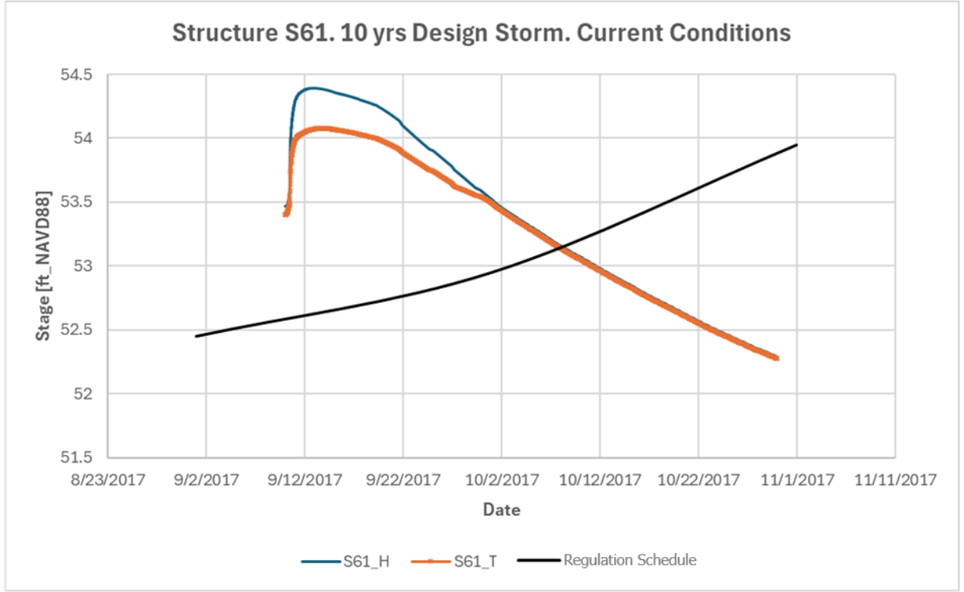
10.6.8 Volume in Lake due to Design Storm UKB-V_{Toho}: Lake Toho

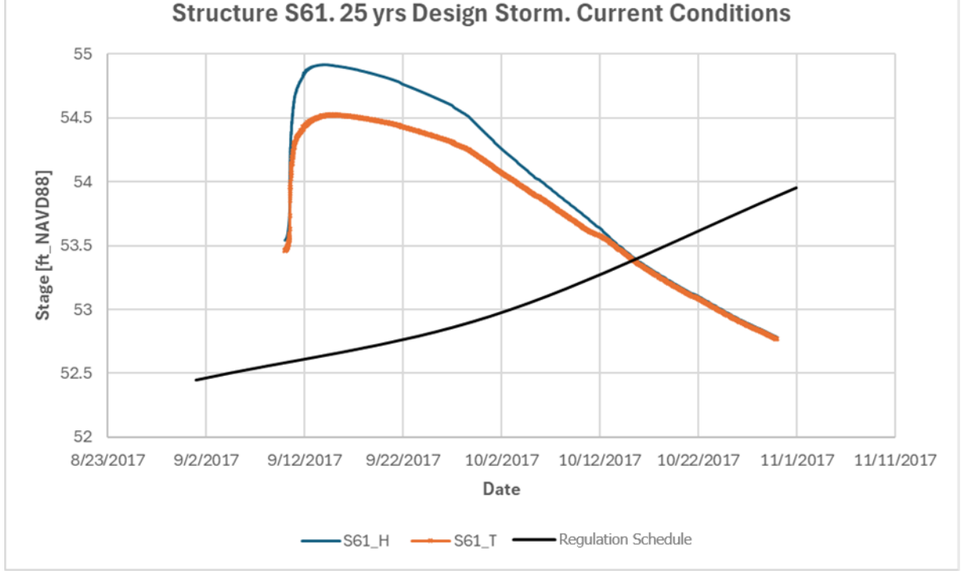
Table 13-3 presents a comparison of the water volume change in Lake Toho, computed as the final volume at the end of the storm event minus the initial volume at the start of the simulation. There is a higher volume for rain events under future conditions than for current conditions for each set of design storms, which is consistent with the rainfall intensity of future conditions events. Also, water volume grows exponentially for each design storm, from 11634 ac-ft for 5-yr design storm to 37976 ac-ft for 100-yr design storm under current conditions; and from 14743 ac-ft for 5-yr design storm to 51813 ac-ft for 100-yr design event under for future conditions. **Table 13-4** shows the volume of water at peak.

10.6.9 Duration of lake recovery relative to regulation schedule UKB-T_{Reg}: Lake Toho

This section examines the water level and regulation schedule in Lake Toho by using its main regulation structure, S61. **Figure 10-134 and Figure 10-135** display headwater and tailwater stages at S61, as well as the regulation schedule for the whole simulation period, under current and future conditions respectively. The water levels in this lake exceed the regulation schedule for all design event simulations for both current and future conditions. Under current and future conditions, the time for the lake levels to return to the regulation schedule is between 10 and 45 days for different events.







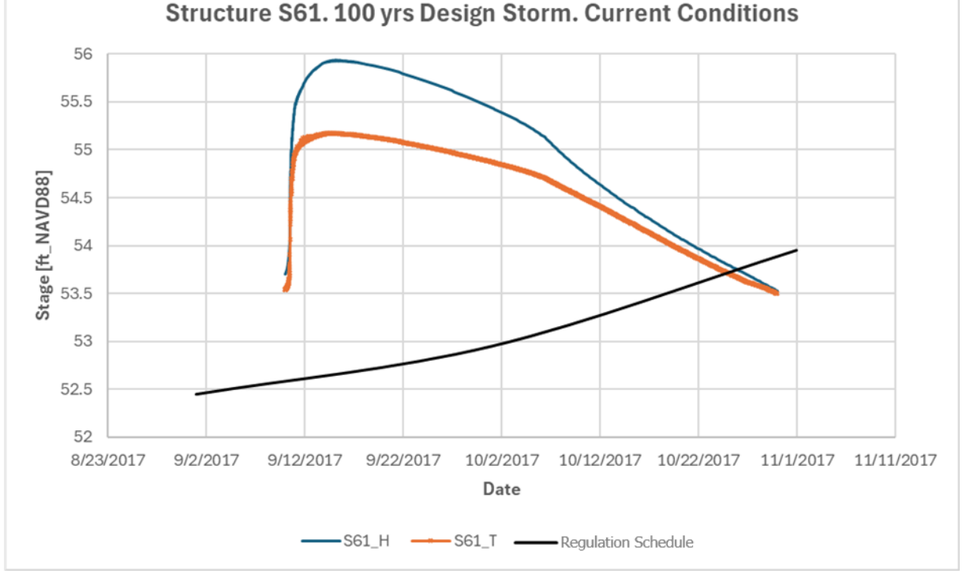
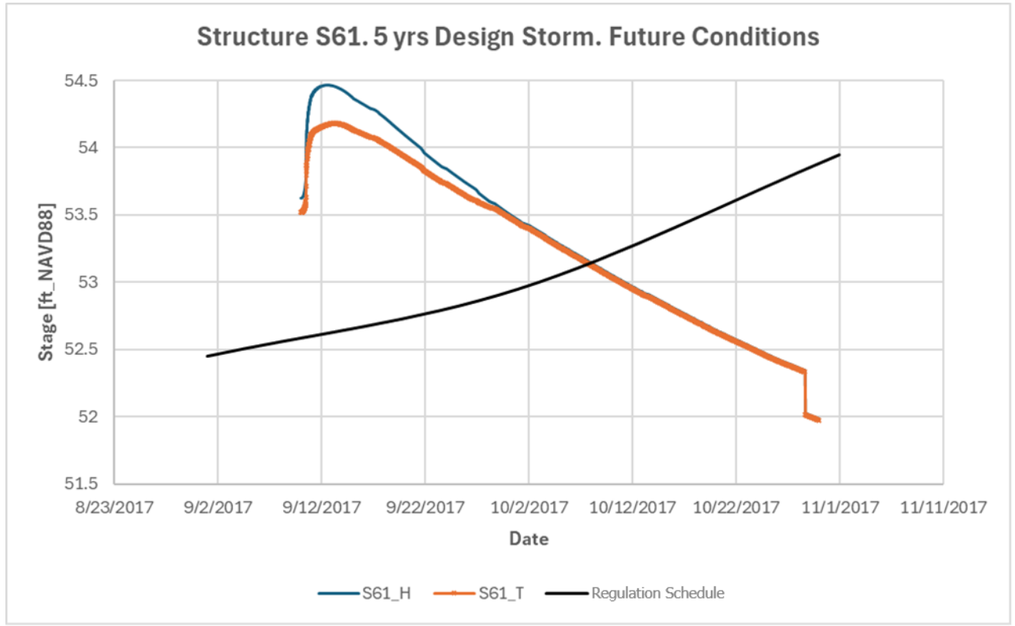
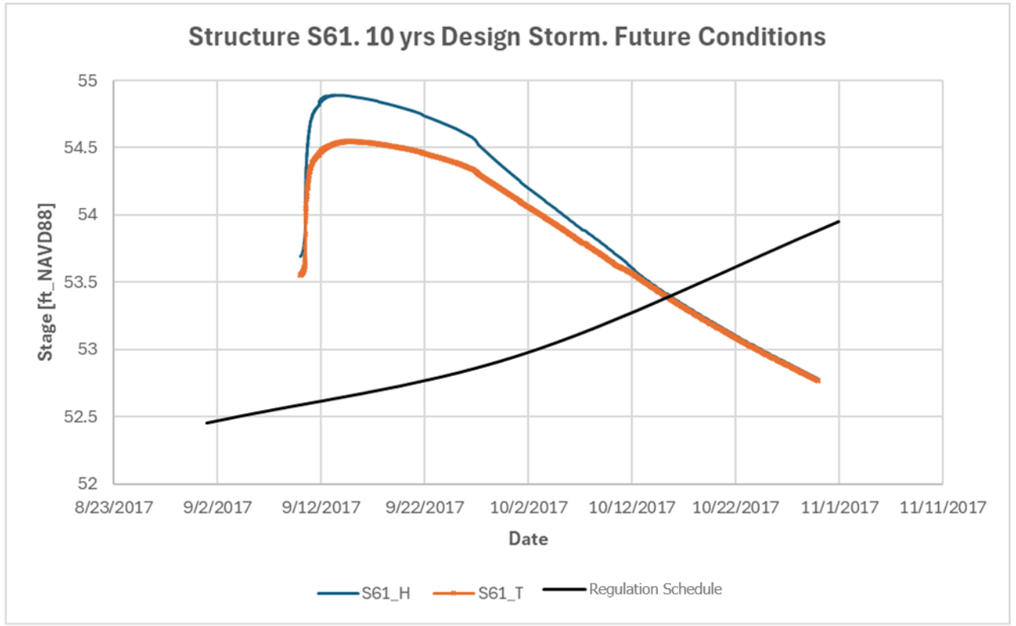
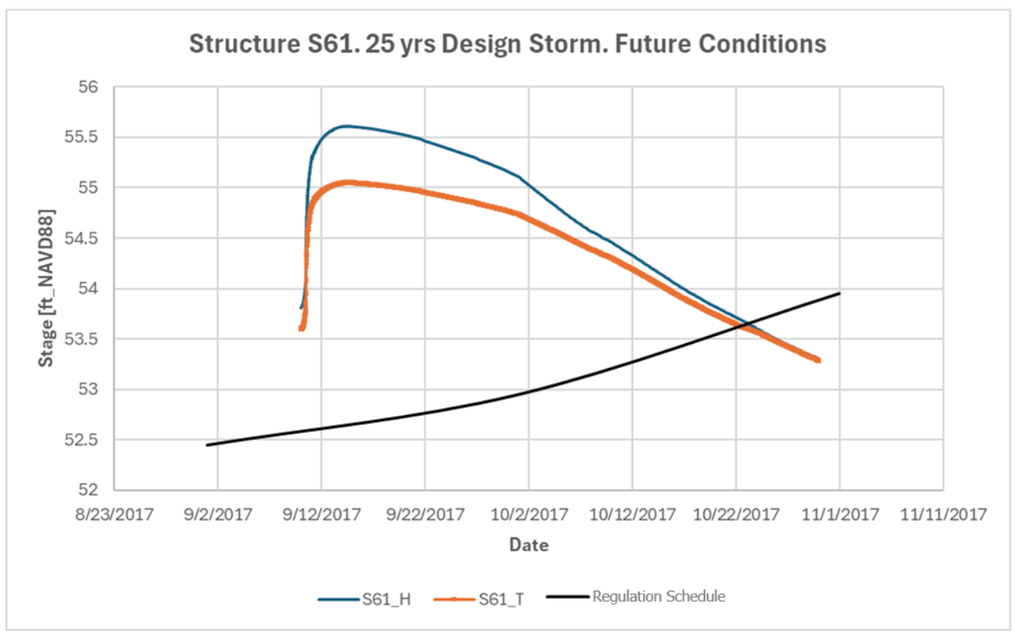


Figure 10-134: Headwater and Tailwater at Structure S61 vs. East Lake Toho Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







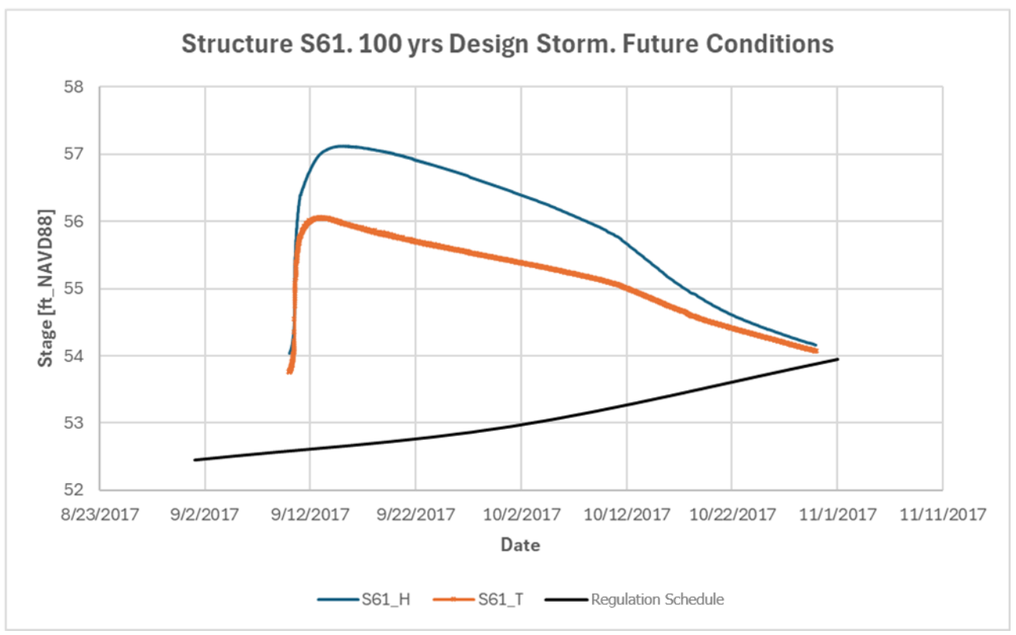


Figure 10-135: Headwater and Tailwater at Structure S61 vs. Lake Toho Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions

10.7 Cypress Lake LMA

10.7.1 PM #1

Performance of the system at Lake Cypress LMA was assessed at Lake Cypress and C-36 canal. The profiles are shown in **Figure 10-136 and Figure 10-137** for Lake Cypress and Canal C-36 respectively under current conditions; and the same are shown in **Figure 10-138 and Figure 10-139** under future conditions. Profiles show water levels exceed the bank elevations under current and future conditions, almost all along the banks.

In summary, under current and future conditions, for Lake Cypress, the left bank is exceeded almost all along the lake, and the right bank is exceeded from the middle to the downstream reaches. For the canal C-36, both the left and right banks remain well below water level for all storms.

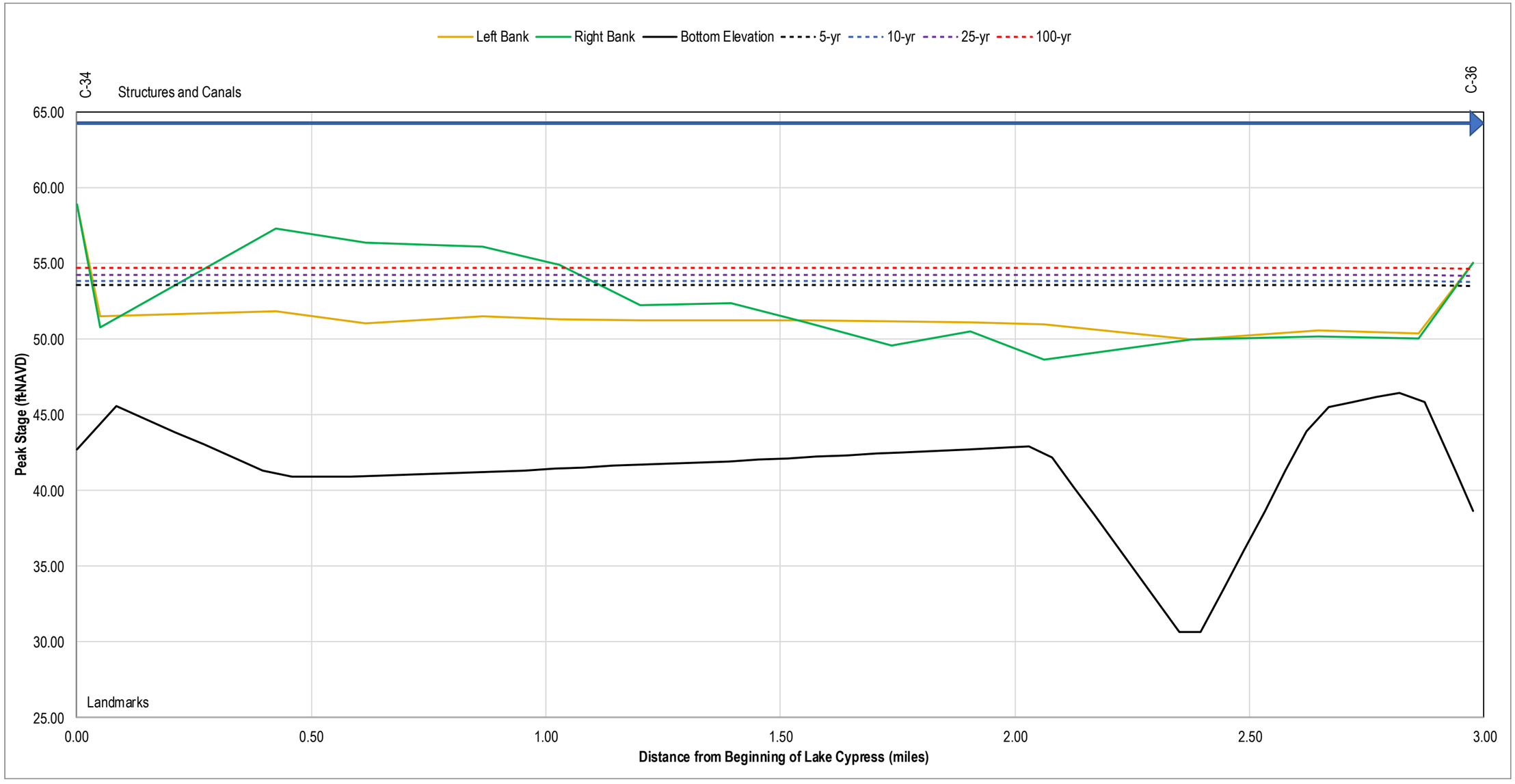


Figure 10-136: Maximum Stage Profile for Cypress Lake under Current Conditions

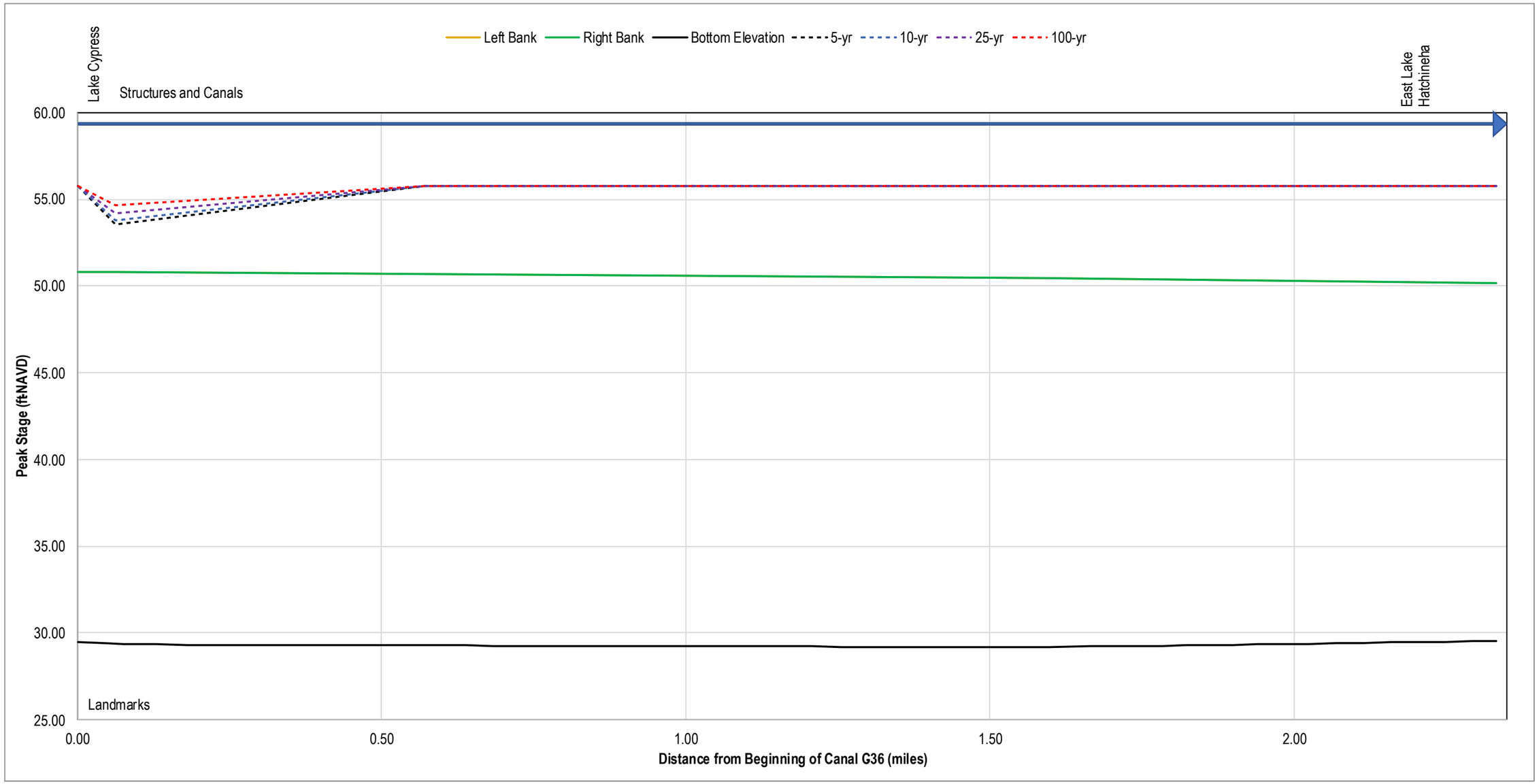


Figure 10-137: Maximum Stage Profile for Canal C-36 under Current Conditions

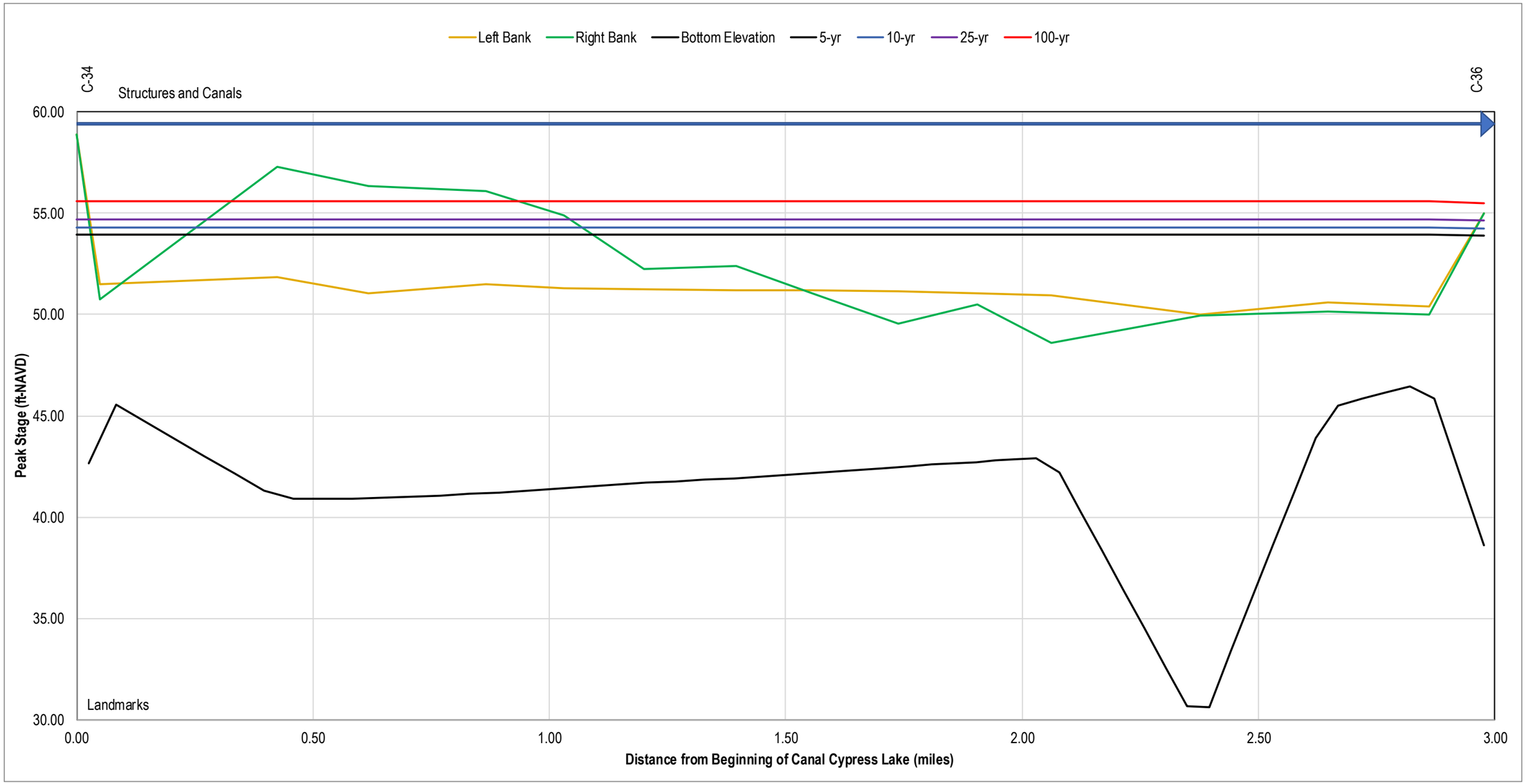


Figure 10-138: Maximum Stage Profile for Cypress Lake under Future Conditions

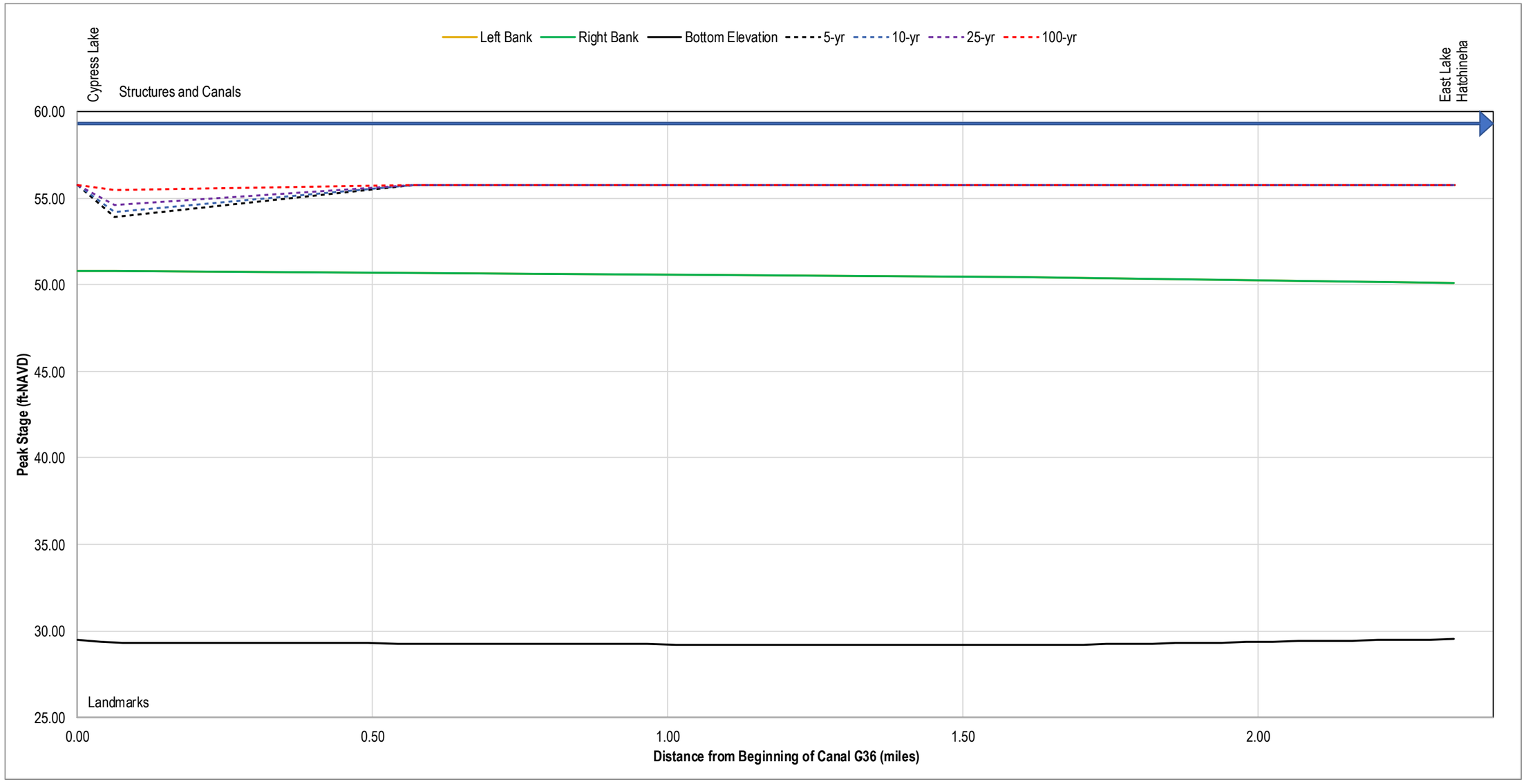


Figure 10-139: Maximum Stage Profile for Canal C-36 under Future Conditions

10.7.2 PM #2

Lake Cypress LMA does not have a downstream structure, so PM #2 for Lake Cypress LMA is included in Lake Kissimmee LMA for analysis.

10.7.3 PM #3

Lake Cypress LMA does not have a downstream structure and is controlled by S65, so PM #3 for Lake Cypress LMA is included in Lake Kissimmee LMA for analysis.

10.7.4 PM #4

PM#4-UKB was evaluated for this LMA, as part of the KCH system.

10.7.5 PM #5

The gridded data of maximum overland depth was extracted for all design storms under current and future conditions and evaluated for Lake Cypress LMA under current and future conditions. **Table 10-81** and **Figure 10-140** shows the flood inundation area in square miles for this LMA, in tabular and graphical formats respectively. **Figure 10-168 through Figure 10-171** show the maximum overland flooding depths in the KCH System for the 5-, 10-, 25- and 100-yr design events, under current and future conditions.

Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not substantial. The tables, plots and maps show that the inundation depths for the remainder of the LMA are high. Assessment of this, along with Lake Gentry LMA shows the inundation is water from Gentry Ditch and Big Bend Swamp, both to the east of Lake Gentry. The flow paths from these swamps following storms have been well documented and observed; this is apparent in the simulated overland flow maps in the flow patterns and the direction of the sheet flow. The inundation in both Lake Cypress and Lake Gentry LMA comes from these wetlands that are hydraulically and hydrologically connected to these LMAs.

Table 10-81: Incremental Flood Inundation Area (sq. mi.) in Lake Cypress LMA

Flooding Depth (ft)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Depth<0.25	201.73	184.26	155.48	109.04	200.82	183.51	155.04	109.02
0.25<=Depth<0.5	27.71	32.62	43.75	56.78	26.47	31.42	42.54	55.88
0.5<=Depth<0.75	23.58	18.36	17.36	21.51	23.12	17.68	16.83	20.75
0.75<=Depth<1.0	24.55	21.96	15.15	14.59	24.04	21.42	14.78	14.10
1.0<=Depth<1.25	18.38	20.93	17.98	13.34	18.62	20.40	17.51	13.07
1.25<=Depth<1.5	12.07	16.10	17.64	11.64	12.88	15.86	17.09	11.53
1.5<=Depth<1.75	8.53	12.20	16.31	11.10	8.90	12.83	15.79	11.02
1.75<=Depth<2.0	6.25	8.79	13.45	14.92	6.10	9.50	13.24	14.46
2.0<=Depth<2.25	4.92	7.41	10.81	13.73	4.59	7.10	10.46	13.51
2.25<=Depth<2.5	3.78	4.69	8.69	12.90	3.56	4.53	9.12	12.60
>2.5	13.06	17.23	27.94	65.01	15.46	20.31	32.17	68.64
Lake Cypress LMA	**220523.92**	**acres**						
	344.57	**sq.mi.**						

Note: *The lake area of 36.90 sq. mi. was included in this table computations.*

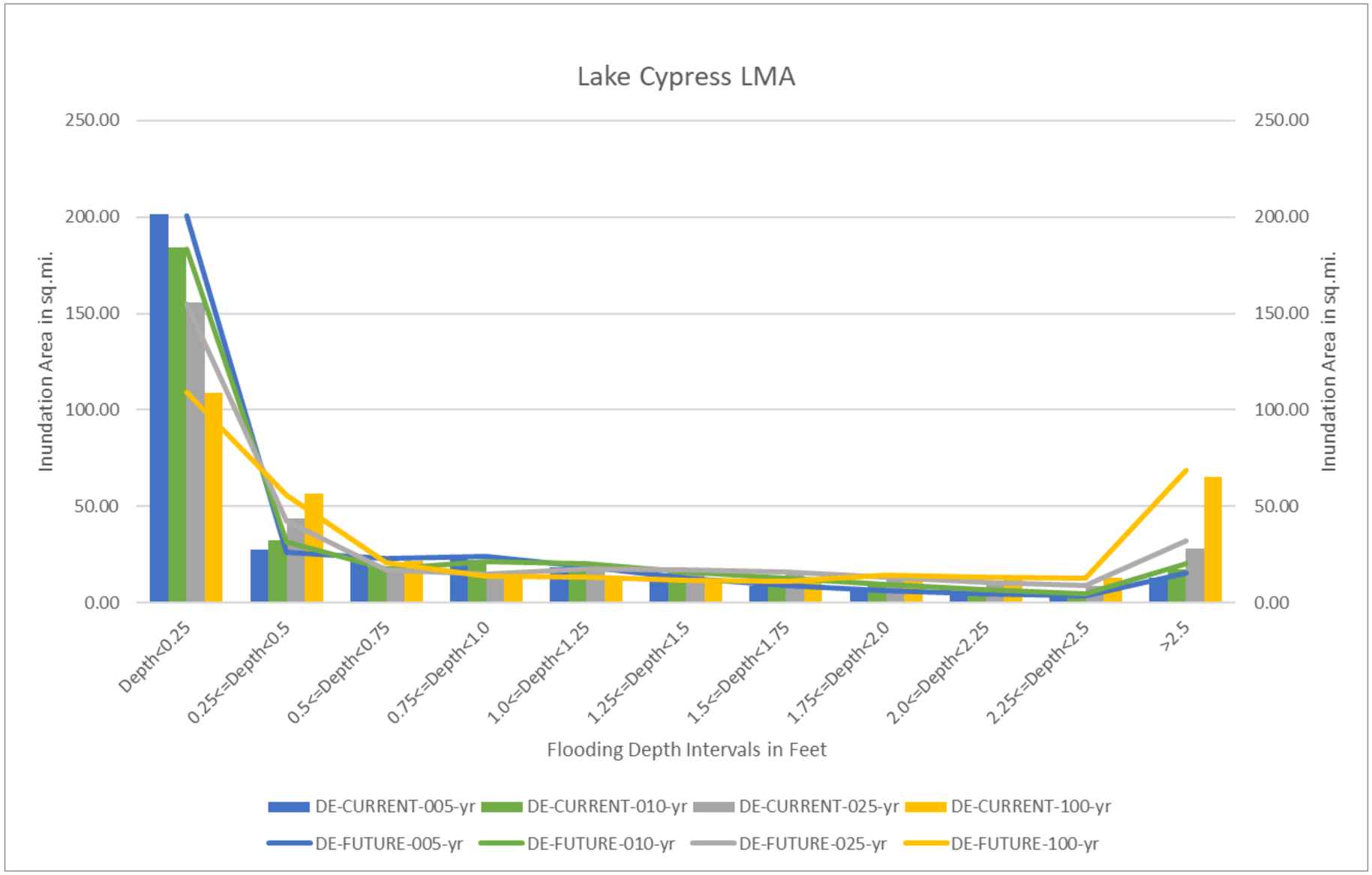


Figure 10-140: Incremental Flood Inundation Depth in feet in Lake Cypress LMA under Current and Future Conditions

10.7.6 PM #6

Table 10-82 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Cypress LMA. **Figure 10-141** presents these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Cypress LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions**. Figure 10-164 through Figure 10-168** show the flood duration in KCH System, except in lakes Cypress, Hatchineha and Kissimmee which are expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration in Lake Gentry LMA is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration in Lake Cypress LMA are areas with longer than 360 hours of flooding, which are the lakes. The maps show that for about 10% of the LMA takes 4 hours to 2 weeks to drain, depending on the size of the storm and the location. This has been observed and is of interest to FPLOS – these swamps take significant time to drain, and this volume of water will flow to Lake Kissimmee and can only be discharged if the LKB is not in flooding conditions.

Table 10-82 Areas of Incremental Flood Duration (sq. mi.) in Lake Cypress LMA

Flooding Duration (hours)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Hours<1	214.46	202.23	182.56	144.22	211.99	198.73	176.78	133.51
1<=Hours<4	4.09	6.30	11.69	26.07	4.17	6.72	13.02	29.22
4<=Hours<8	2.67	3.02	4.14	5.58	2.73	3.11	4.35	6.11
8<=Hours<12	1.57	1.89	2.15	2.85	1.57	1.87	2.19	3.19
12<=Hours<24	4.37	5.05	5.85	7.12	4.47	5.14	5.99	8.05
24<=Hours<48	4.56	4.99	5.45	6.59	4.45	5.02	5.83	7.15
48<=Hours<96	4.27	5.15	6.13	6.97	4.24	5.11	6.26	7.66
96<=Hours<168	3.55	3.98	4.91	6.53	3.51	4.06	5.15	6.97
168<=Hours<240	2.39	2.59	3.26	4.36	2.22	2.56	3.27	4.74
240<=Hours<360	3.74	3.58	3.73	5.53	3.57	3.14	3.72	5.65
360<=Hours<1417	98.90	105.79	114.70	128.77	101.66	109.10	118.00	132.33
Lake Cypress LMA	**220523.92**	**acres**						
	344.57	**sq.mi.**						

Note: *The lake area of 36.90 sq. mi. was included in this table computations.*

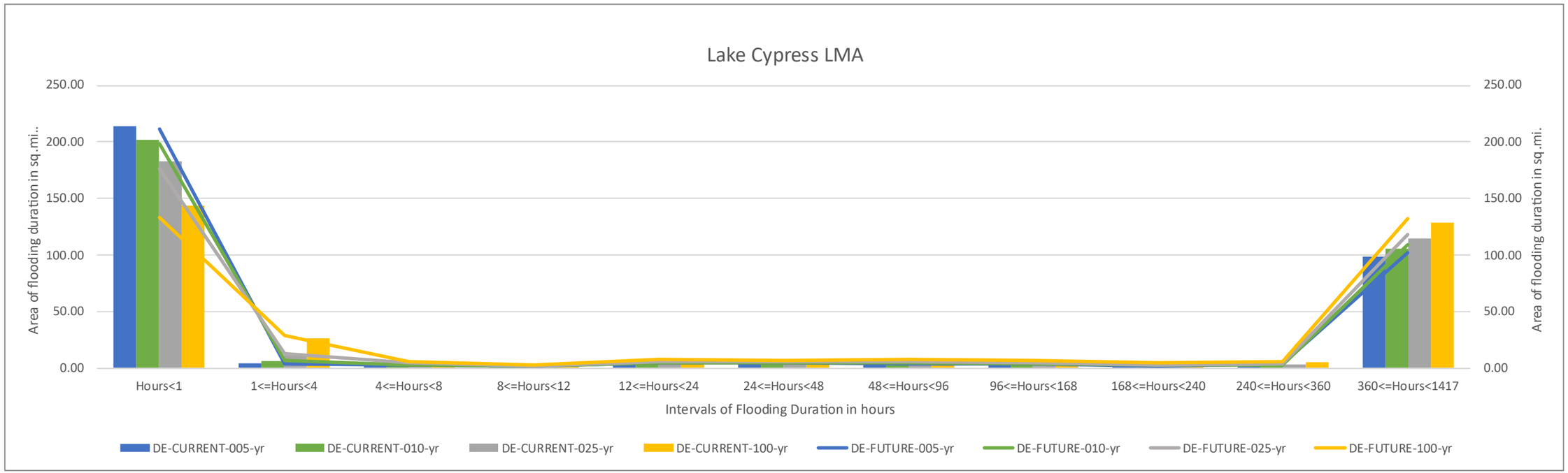


Figure 10-141: Areas of Incremental Flood Duration in hours in Lake Cypress LMA under Current and Future Conditions

10.7.7 FPLOS Rating of Cypress Lake LMA

Table 10-83 provides a summary of evaluation of Cypress Lake LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The FPLOS rating for the LMA ranges from <5-yr to 10-yr under both current and future conditions for the PMs; and therefore an overall FPLOS rating of <5-yr.

10.7.8 Volume in Lake due to Design Storm UKB-$V_{Cypress}$: Lake Cypress

Table 13-3 provides a summary of the change in water volume in Cypress Lake, calculated by subtracting the initial volume at the start of the simulation from the final volume at the end of the storm event. It also shows that the volume is higher for future conditions than for current conditions for each set of design storms, which is expected due to the rainfall magnitude under future conditions. **Table 13-4** shows the volume of water at peak.

10.7.9 Duration of Lake Recovery Relative to regulation schedule UKB-T_{Reg}: Lake Cypress

Lake Cypress LMA's level depends on S65 and does not have its own outlet structure, so it is evaluated with Lake Kissimmee LMA.

Table 10-83: Summary of PM Analysis in Lake Cypress LMA

Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Cypress, C-36 canal	The water levels exceed the banks almost all along Lake Cypress and C-36 canal for all storms.	< 5-yr	< 5-yr
PM#2	-	-		
PM#3	-	-		
PM#4-UKB	-	-		
PM#5	Entire LMA	For 5- and 10-yr design events under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 55% of the LMA is inundated more than 0.25-ft for a 25-yr design event and approximately 68% of the LMA is inundated more than 0.25-ft for a 100-yr event under both current and future conditions.	10-yr	10-yr
PM#6	Entire LMA	Approximately 64% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 49% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm under both current and future conditions.	5-yr	5-yr

Lake Cypress LMA Overall FPLOS	**Current Conditions**	**< 5-yr**
	Future Conditions	**< 5-yr**

10.8 Lake Hatchineha LMA

10.8.1 PM #1

Performance of the system at Lake Hatchineha LMA was assessed at East Lake Hatchineha and West Lake Hatchineha and C-37 canal, shown in **Figure 10-142 through Figure 10-144** under current conditions; and **Figure 10-145 through Figure 10-148** under future conditions. The water levels exceed the banks for all the simulated current and future events in most locations.

For the Lake Hatchineha LMA, the current and future conditions show that the water levels exceed the right bank in West Lake Hatchineha for all storms, particularly in the middle portions of the LMA. The water levels stay above the left bank generally for all design events. In East Lake Hatchineha, the right bank is exceeded for all storm events, though less prominently, while the left bank remains below the water level for all design events in almost half of the LMA area. The C-37 Canal water levels exceed the right and left banks in various locations along the LMA for all design storms.

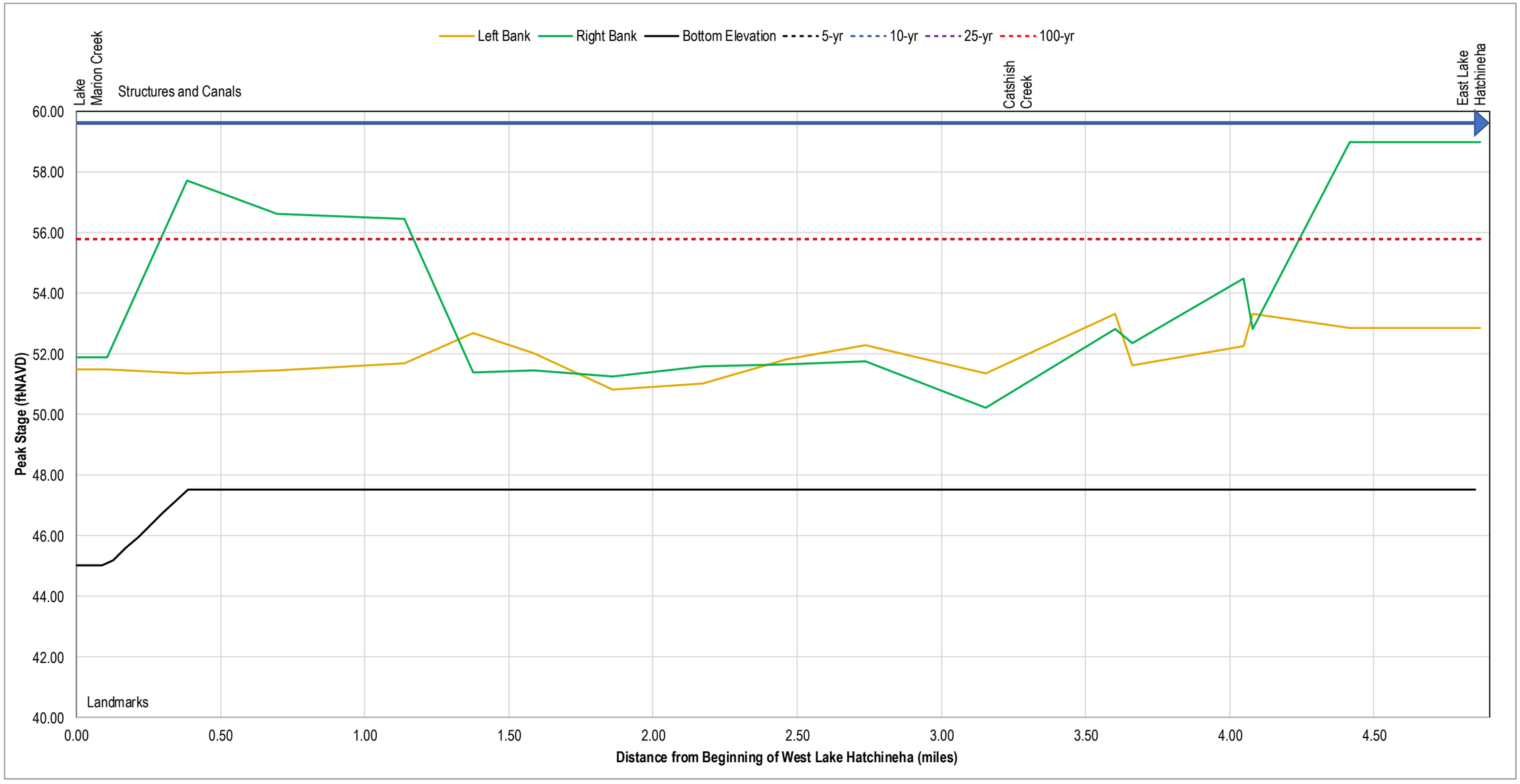


Figure 10-142: Maximum Stage Profile for West Hatchineha under Current Conditions

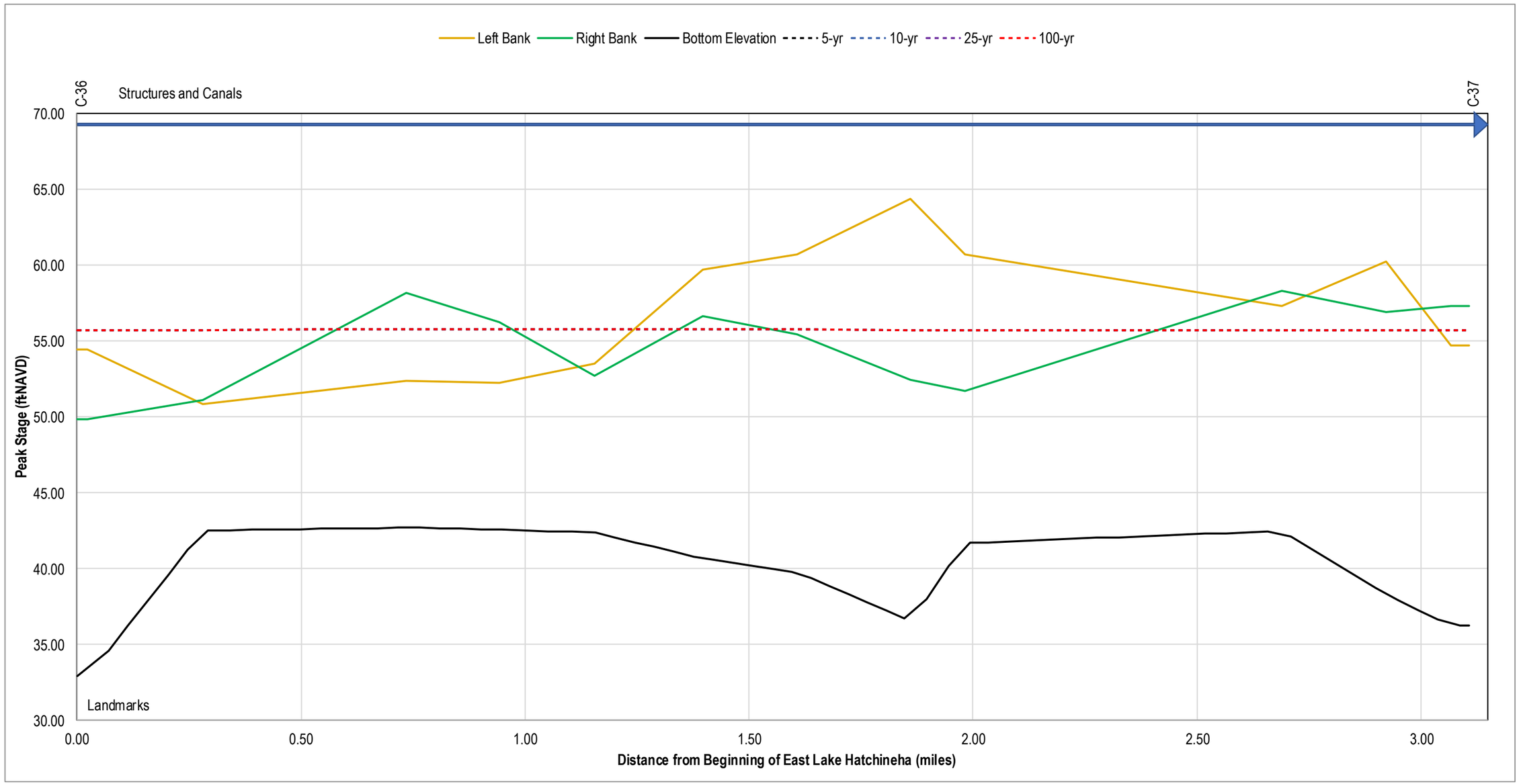


Figure 10-143: Maximum Stage Profile for East Hatchineha under Current Conditions

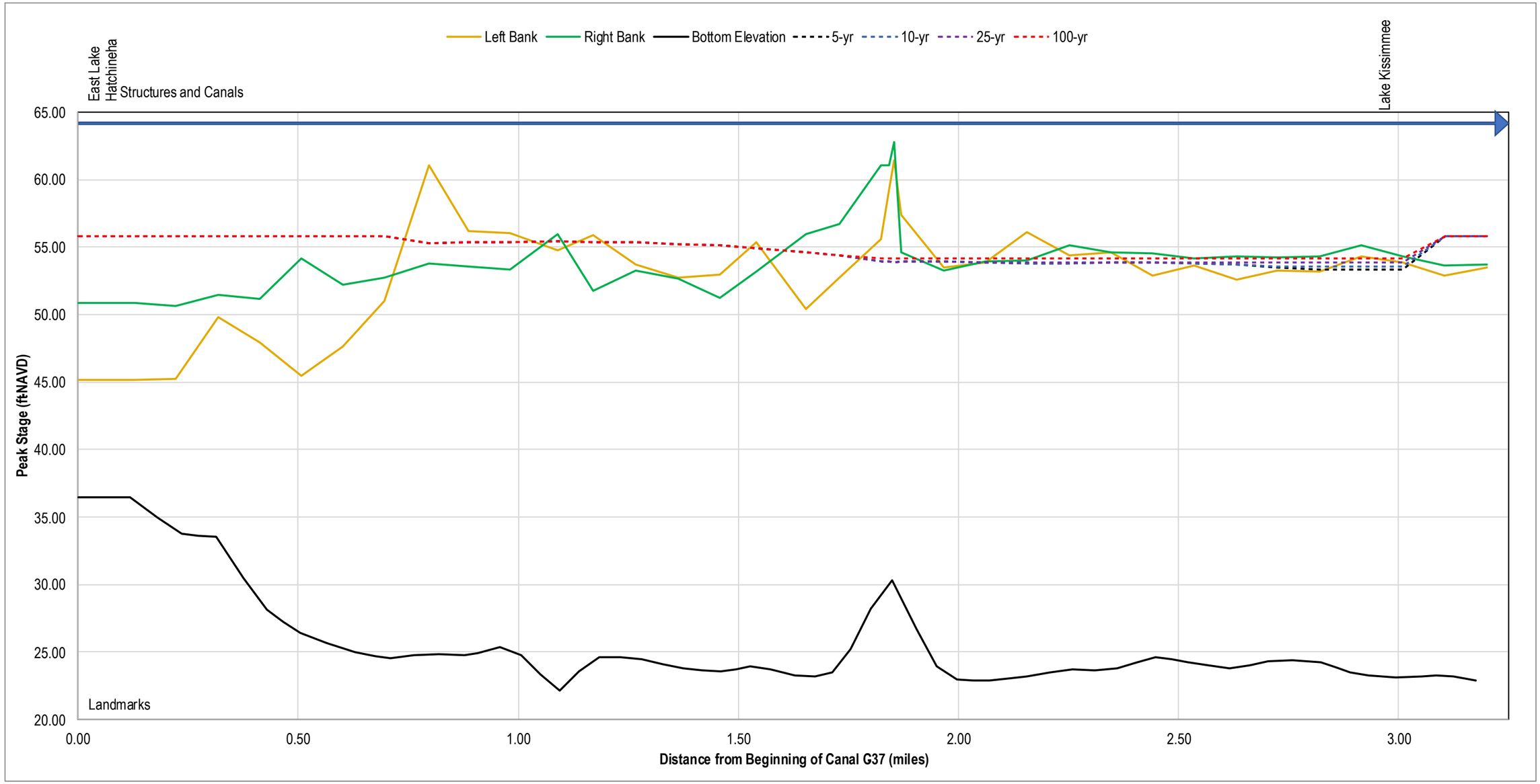


Figure 10-144: Maximum Stage Profile for Canal C-37 under Current Conditions

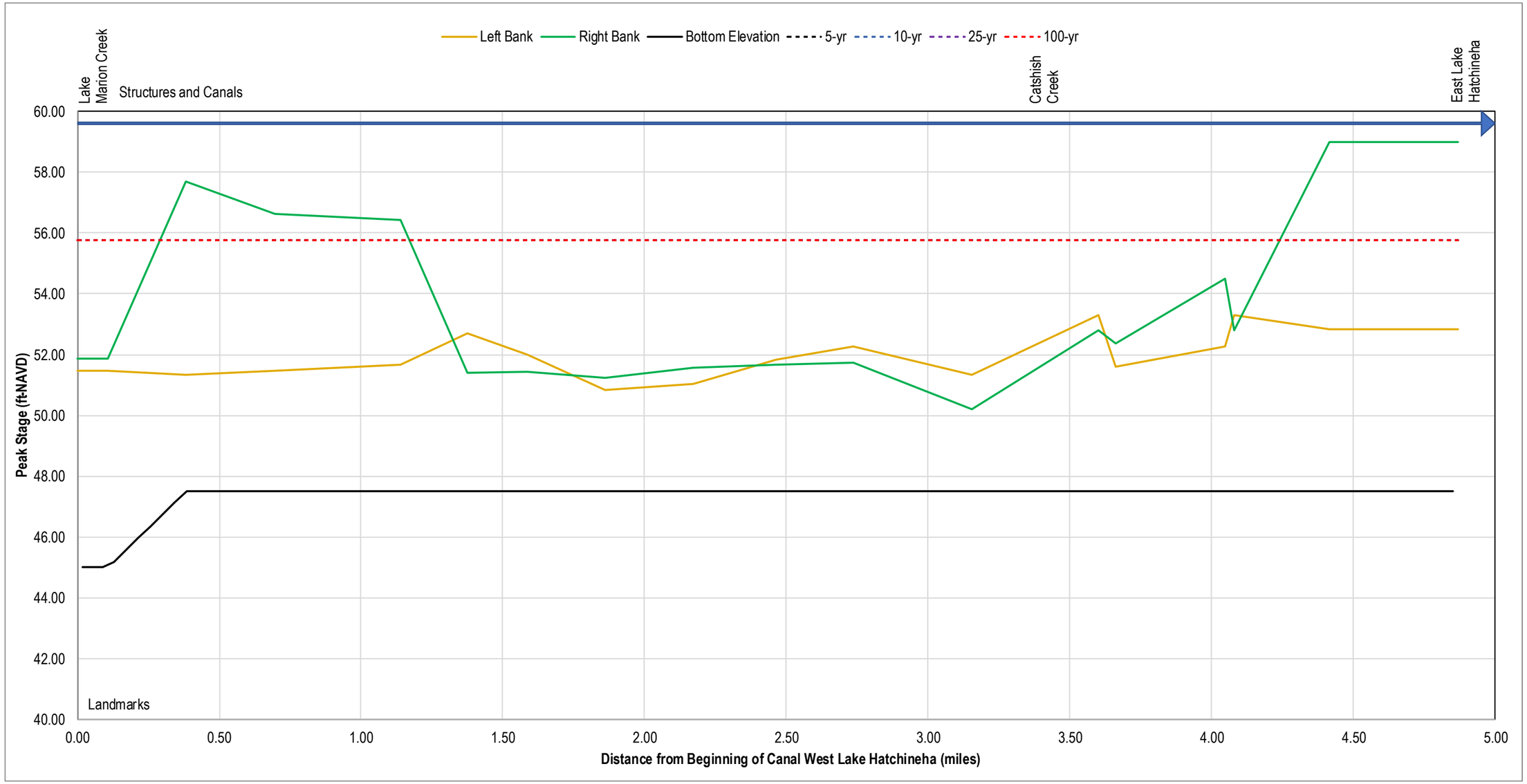


Figure 10-145: Maximum Stage Profile for West Hatchineha under Future Conditions

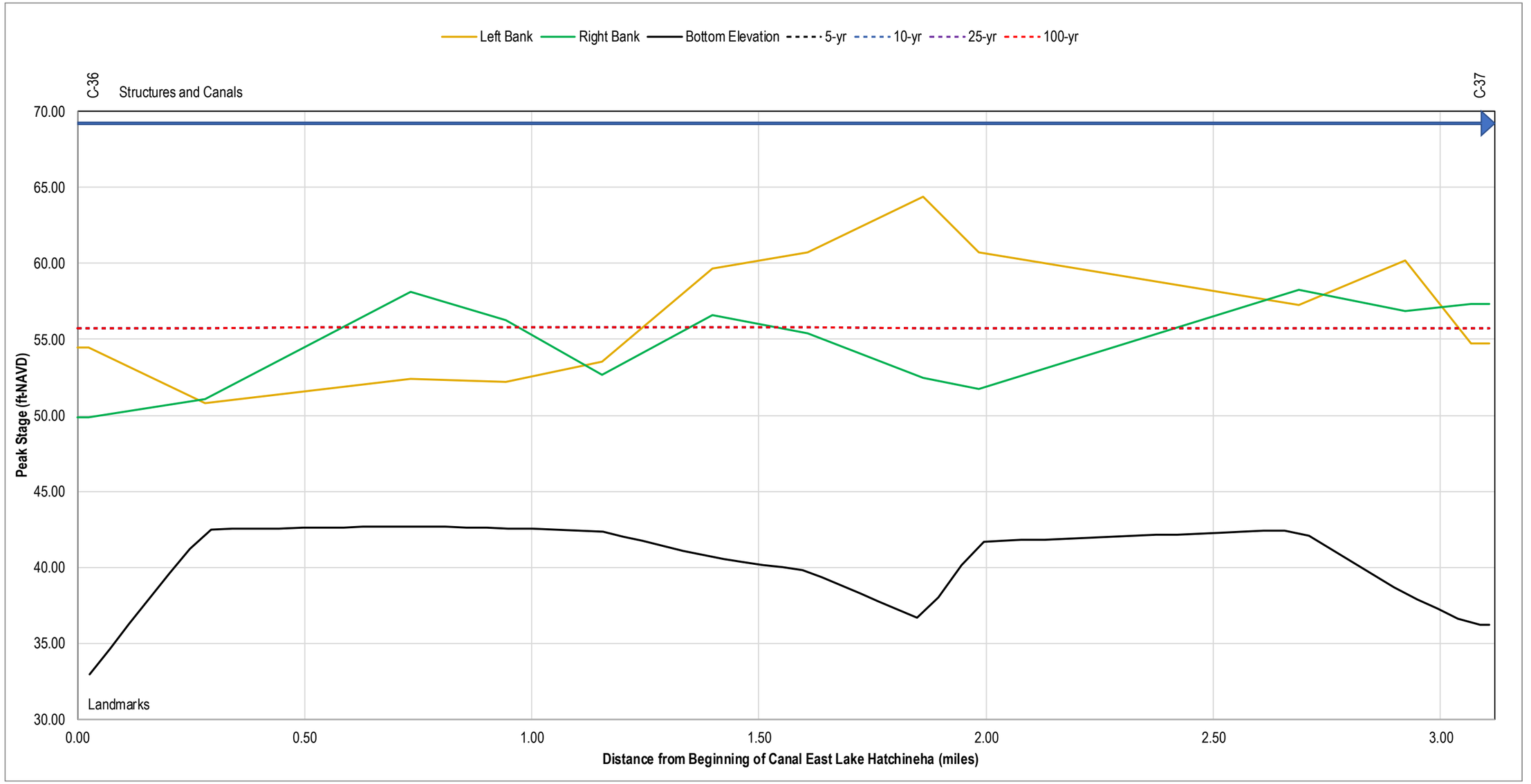


Figure 10-146: Maximum Stage Profile for East Hatchineha under Future Conditions

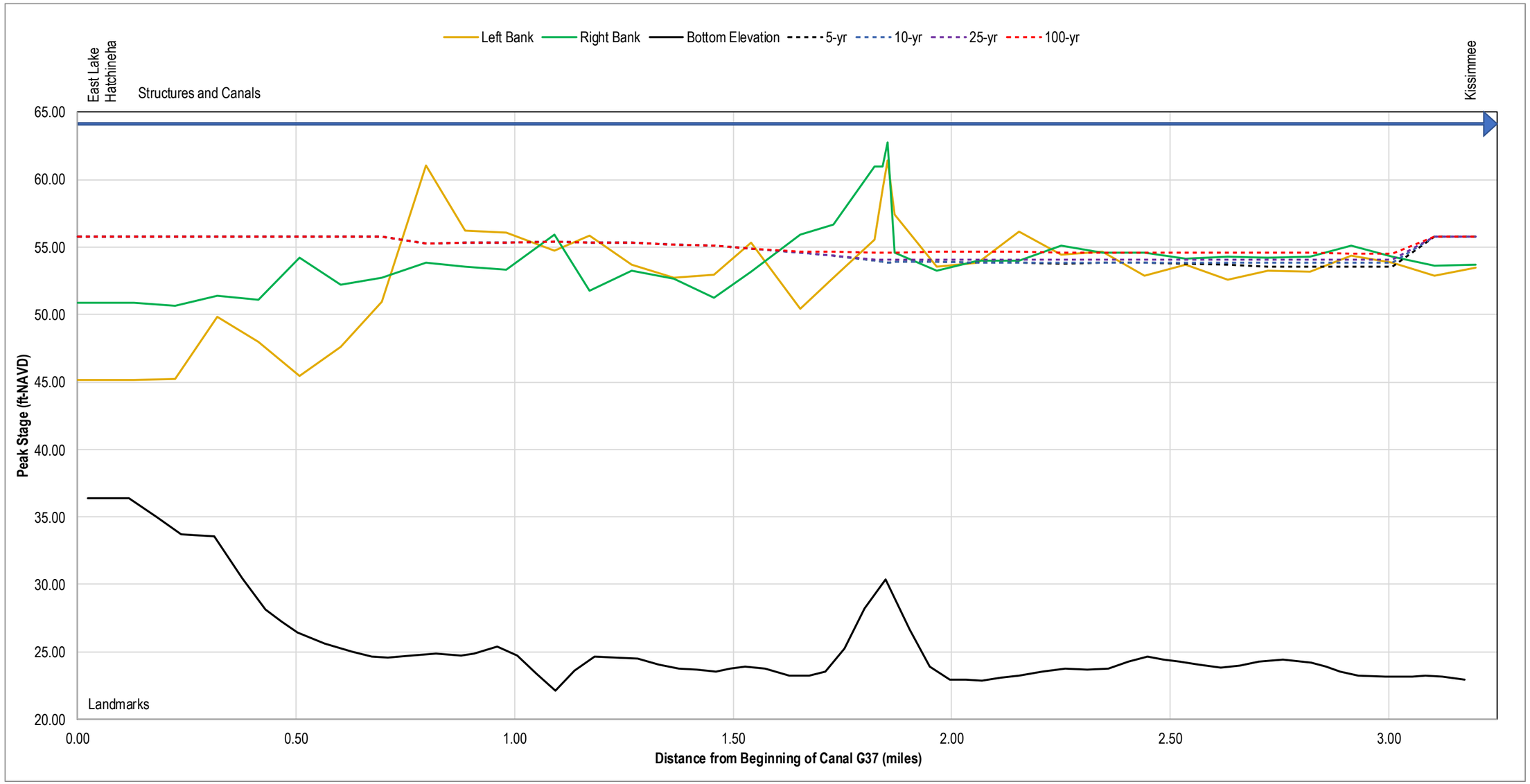


Figure 10-147: Maximum Stage Profile for Canal C-37 under Future Conditions

10.8.2 PM #2

Lake Hatchineha LMA does not have a downstream structure, so PM #2 is included in Lake Kissimmee LMA for analysis.

10.8.3 PM #3

Lake Hatchineha LMA does not have a downstream structure and is controlled by S65, so PM #3 for Lake Hatchineha LMA is included in Lake Kissimmee LMA for analysis.

10.8.4 PM #4

PM#4-UKB was assessed in this LMA, as part of the KCH system.

10.8.5 PM #5

The gridded data of maximum overland depth was extracted for each of the design storms under current and future conditions and evaluated for Lake Hatchineha LMA. **Table 10-84** and **Figure 10-148** are tabular and graphical representations of the flood inundation area in square miles for this LMA. **Figure 10-168 through Figure 10-171** show the maximum overland flooding depths in the KCH System for the 5-, 10-, 25- and 100-yr design events, under current and future conditions.

Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not substantial. The tables, plots and maps show that the inundation depths for the remainder of Lake Hatchineha LMA are high. Assessment of this, along with Lake Cypress and Lake Gentry LMAs, and the wetlands from the western portion of the model domain show the inundation is water from Gentry Ditch and Big Bend Swamp to the east and Catfish Creek State Preserve and other wetlands areas to the west that drain to this LMA. The inundation in this LMA comes from these wetlands that are hydraulically and hydrologically connected to these LMAs, and groundwater that has a high gradient towards this LMA.

Table 10-84: Incremental Flood Inundation Area (sq. mi.) in Lake Hatchineha LMA

Flooding Depth (ft)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Depth<0.25	124.89	115.92	100.69	74.65	124.59	115.53	100.31	74.58
0.25<=Depth<0.5	13.22	16.91	24.50	34.03	13.18	16.68	23.54	34.61
0.5<=Depth<0.75	8.36	8.32	8.63	11.26	8.26	8.33	8.34	11.32
0.75<=Depth<1.0	7.78	6.64	7.17	7.02	7.83	6.60	7.05	6.94
1.0<=Depth<1.25	6.85	7.28	5.91	6.19	6.88	7.32	5.80	6.10
1.25<=Depth<1.5	4.19	6.12	6.12	5.41	4.23	6.20	6.14	5.31
1.5<=Depth<1.75	3.17	4.37	6.20	5.10	3.17	4.32	6.43	5.08
1.75<=Depth<2.0	2.80	3.36	5.04	5.30	2.68	3.27	4.90	5.17
2.0<=Depth<2.25	2.41	2.61	3.66	5.51	2.31	2.60	3.50	5.97
2.25<=Depth<2.5	2.21	2.35	3.01	5.17	2.23	2.22	3.04	5.50
>2.5	43.19	45.19	48.13	59.42	43.70	46.01	50.01	58.49
Lake Hatchineha LMA	**140199.96**	**acres**						
	219.06	**sq.mi.**						

Note: *The lake area of 33.10 sq. mi. was included in this table computations.*

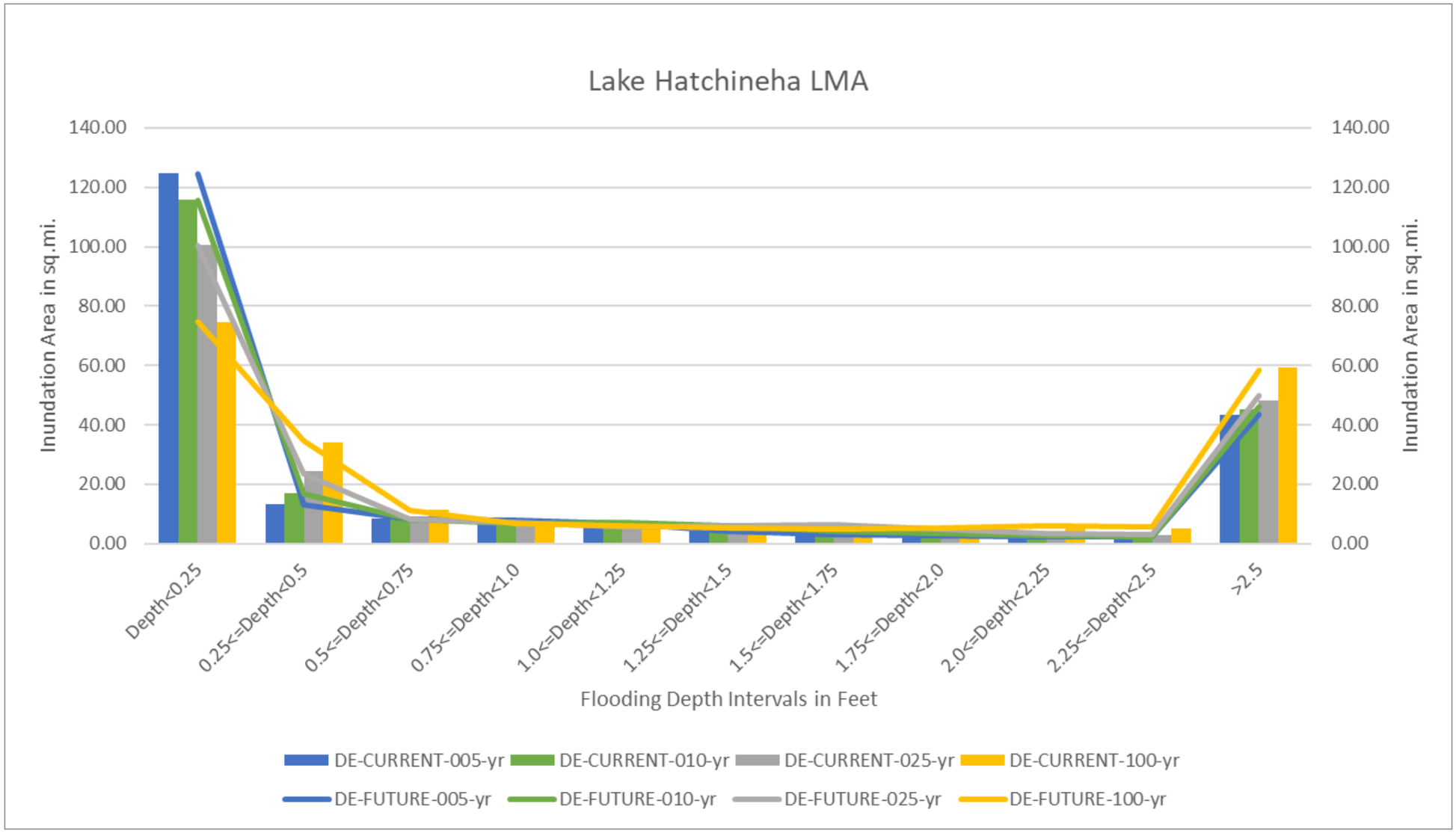


Figure 10-148: Incremental Flood Inundation Depth in feet in Lake Hatchineha LMA under Current and Future Conditions

10.8.6 PM #6

Table 10-86 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within Lake Hatchineha LMA. Figure present these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the Lake Hatchineha LMA for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-166 through Figure 10-169** show the flood duration in KCH System, except in lakes Cypress, Hatchineha and Kissimmee which are expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration in Lake Hatchineha LMA is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration in Lake Hatchineha LMA are areas with longer than 360 hours of flooding, which are the lakes. **Figure 10-149** demonstrates that less than 2% of the LMA area will remain dry under current and future conditions. The probability that 25% or more of the LMA area will be inundated is about 18% for a 5-yr storm event and just over 20% for a 100-yr storm event. The maps show that for about 25% of the LMA takes 4 hours to 2 weeks to drain, depending on the size of the storm and the location within the LMA. This is a large volume of water that will flow to Lake Kissimmee and can only be discharged if the LKB is not in flooding conditions.

Table 10-85: Areas of Incremental Flood Duration (sq. mi.) in Lake Hatchineha LMA

Flooding Duration (Hours)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Hours<1	132.91	128.01	119.34	100.61	130.39	124.20	113.26	89.87
1<=Hours<4	2.47	3.37	6.17	15.21	2.73	4.22	8.46	20.05
4<=Hours<8	2.17	2.51	3.26	4.39	2.35	2.71	3.32	4.87
8<=Hours<12	1.24	1.47	1.70	1.95	1.13	1.29	1.54	1.93
12<=Hours<24	3.35	3.43	3.39	3.54	2.80	2.96	3.26	3.88
24<=Hours<48	3.42	3.44	3.14	3.34	3.11	2.94	2.90	3.49
48<=Hours<96	3.09	3.19	3.11	3.51	2.49	2.81	3.23	3.82
96<=Hours<168	2.49	2.57	2.51	2.64	2.00	2.17	2.30	2.90
168<=Hours<240	1.68	1.94	2.46	1.81	1.42	1.41	1.59	2.22
240<=Hours<360	2.12	2.37	3.11	2.67	1.68	2.08	2.36	2.42
360<=Hours<1417	64.14	66.75	70.88	79.40	68.96	72.26	76.81	83.61
Lake Hatchineha LMA	**140199.96**	**acres**						
	219.06	**sq.mi.**						

Note: *The lake area of 33.10 sq. mi. was included in this table computations.*

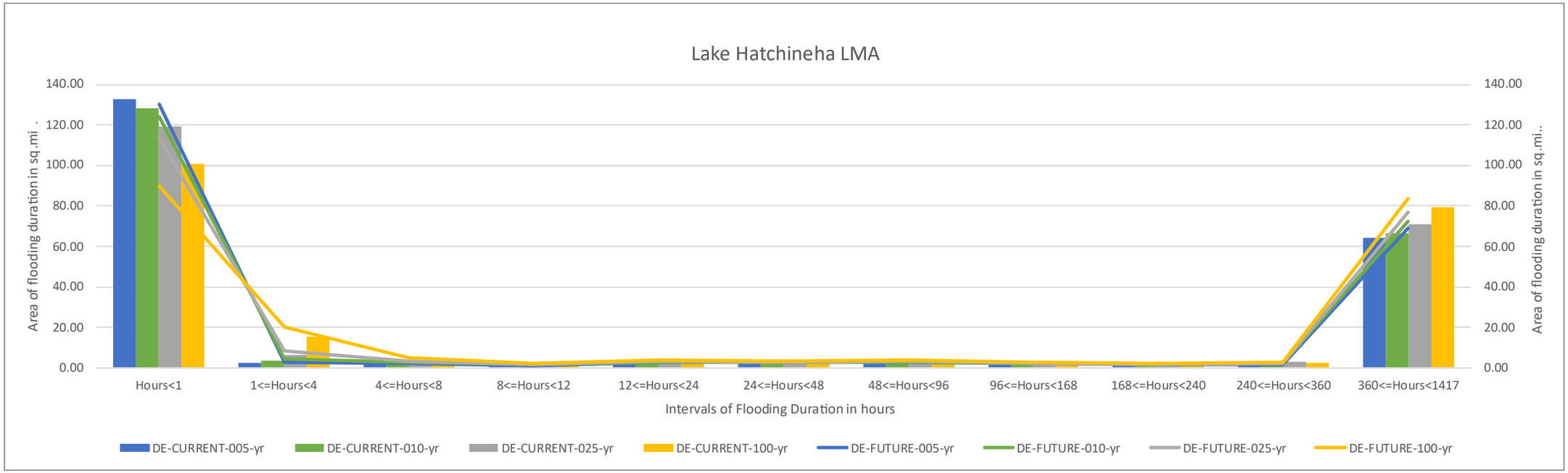


Figure 10-149 Areas of Incremental Flood Duration in hours in Lake Hatchineha LMA under Current and Future Condition

10.8.7 FPLOS Rating of Lake Hatchineha LMA

Table 10-86 provides a summary of evaluation of Lake Hatchineha LMA performance using FPLOS metrics to provide flood protection under current and future conditions. The FPLOS rating for the LMA ranges from <5-yr to 10-yr under both current and future conditions for the various PMs; the overall rating assigned to this LMA, therefore is <5-yr.

Table 10-86: Summary of PM Analysis in Lake Hatchineha LMA

Metric	Water Bodies Included in Analyses	Outcomes Based on Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions	Comments
PM#1	East Lake Hatchineha, West Lake Hatchineha, C-37 canal	The water levels exceed the banks for all the simulated current and future events in most locations.	< 5-yr	< 5-yr	Overflows for smaller storms, so stages are the same for all
PM#2	-	-			
PM#3	-	-			
PM#4-UKB	-	-			
PM#5	Entire LMA	For 5- and 10-yr design events under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 54% of the LMA is inundated more than 0.25-ft for a 25-yr design event and approximately 66% of the LMA is inundated more than 0.25-ft for a 100-yr event under both current and future conditions.	10-yr	10-yr	
PM#6	Entire LMA	Approximately 62% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 52% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm under both current and future conditions.	5-yr	5-yr	
Lake Hatchineha LMA Overall FPLOS		**Current Conditions**		**< 5-yr**	
		Future Conditions		**< 5-yr**	

10.8.8 Volume in Lake due to Design Storm UKB-V_{Lake}: Lake Hatchineha

Table 13-3 presents a summary of the water volume change in Lake Hatchineha, computed as the final volume at the end of the storm event minus the initial volume at the start of the simulation. There is higher volume in the lake under future conditions compared to current conditions for each set of design storms, due to the higher rainfall under future conditions, and the higher lake regulation schedule. Also, water volume increases exponentially for each design storm, from 1093.1 ac-ft for the 5-yr design storm to 2406.9 ac-ft for the 100-yr storm under current conditions; and from 1293.7 ac-ft for the 5-yr design storm to 3065.3 ac-ft for the -100-yr design storm under future conditions. **Table 13-4** shows the volume of water at peak.

10.8.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}: Lake Hatchineha

There is no downstream structure for Lake Hatchineha LMA and S65 controls it, so the evaluation is the same as for Lake Kissimmee LMA.

10.9 Lake Kissimmee LMA

Figure 10-150 shows the K-C-H system conveyance, their structures and roadway.

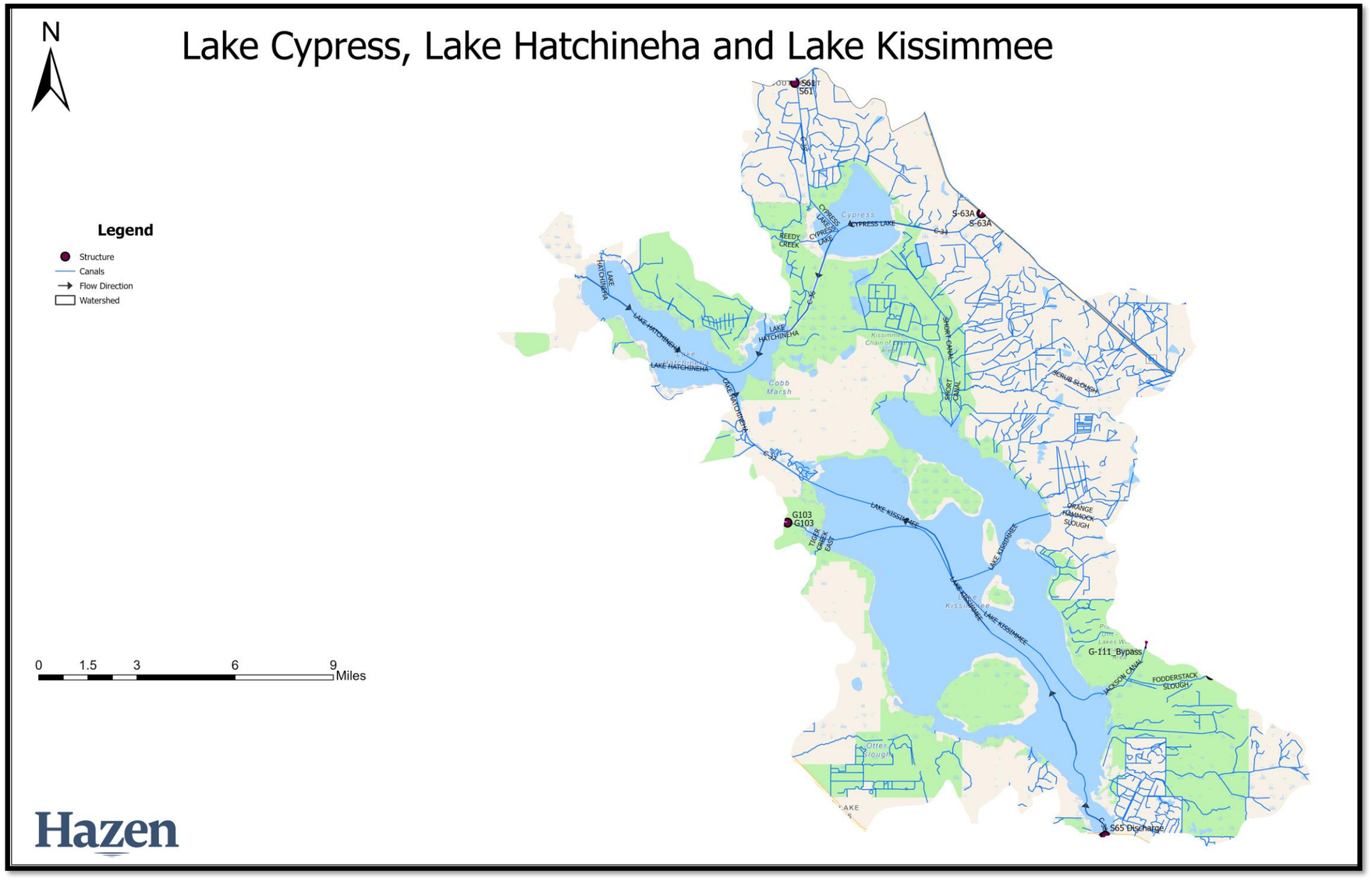


Figure 10-150: K-C-H System with Main Structures, Waterways and Roadways

Note: *Flow direction used in this instance was to help determine left and right banks.*

10.9.1 PM #1

Performance of the system at Lake Kissimmee LMA was assessed at Lake Kissimmee and C-38 canal. This was also assessed at Zipper Canal, Jackson canal, Lake Marion Creek and Lake Marian. **Figure 10-151 through Figure 10-154** show the water surface profiles for the conveyance system under current and future conditions. For all design storms under current and future conditions, water levels stay within the banks in Lake Kissimmee for all design storms except 100- yr when the water levels exceed the banks in multiple locations. In all or portions of Canal C-38, Jackson Canal, Lake Marian and Lake Marion, the banks are overtopped for all simulated storms.

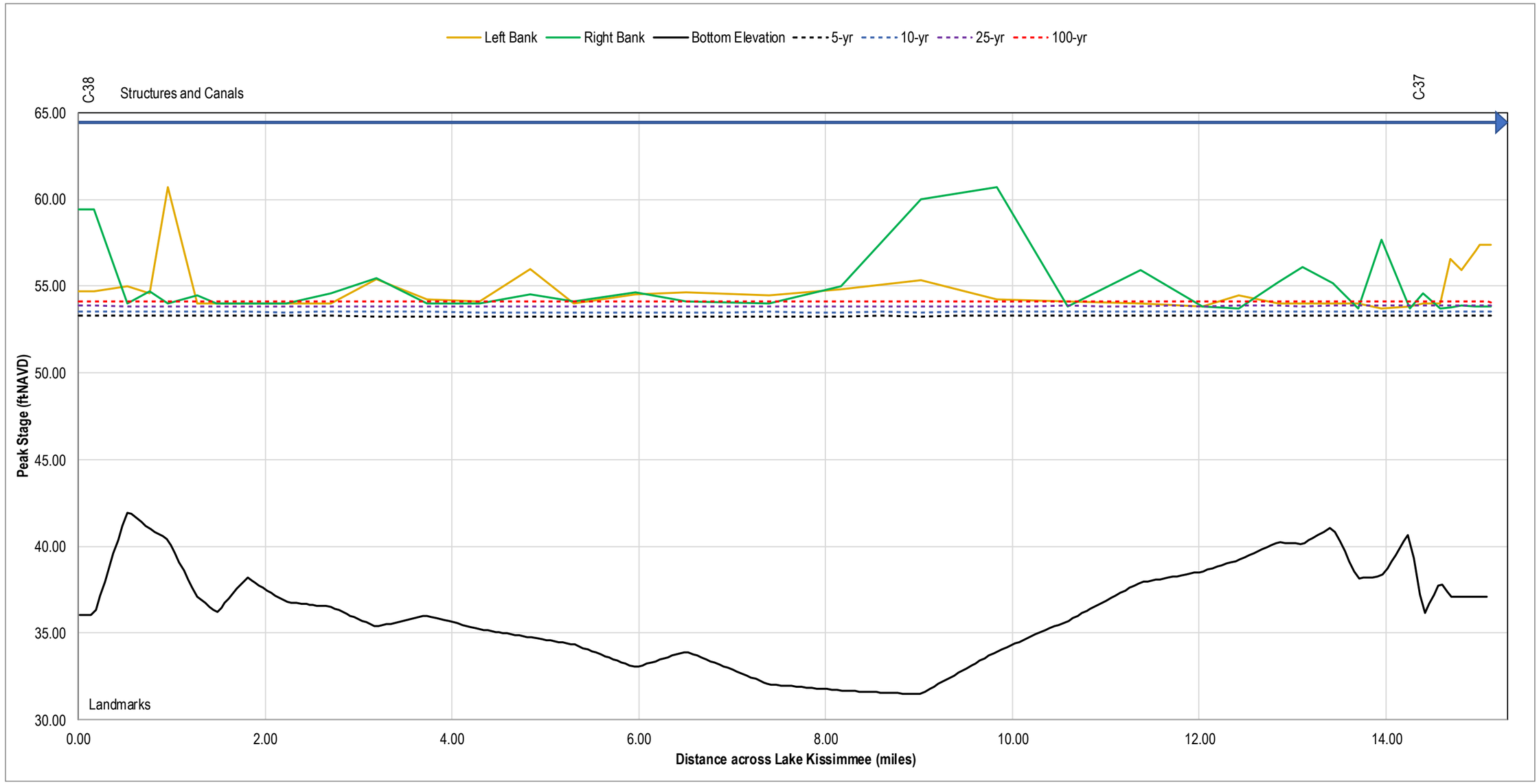


Figure 10-151: Maximum Stage Profile for Lake Kissimmee under Current Conditions

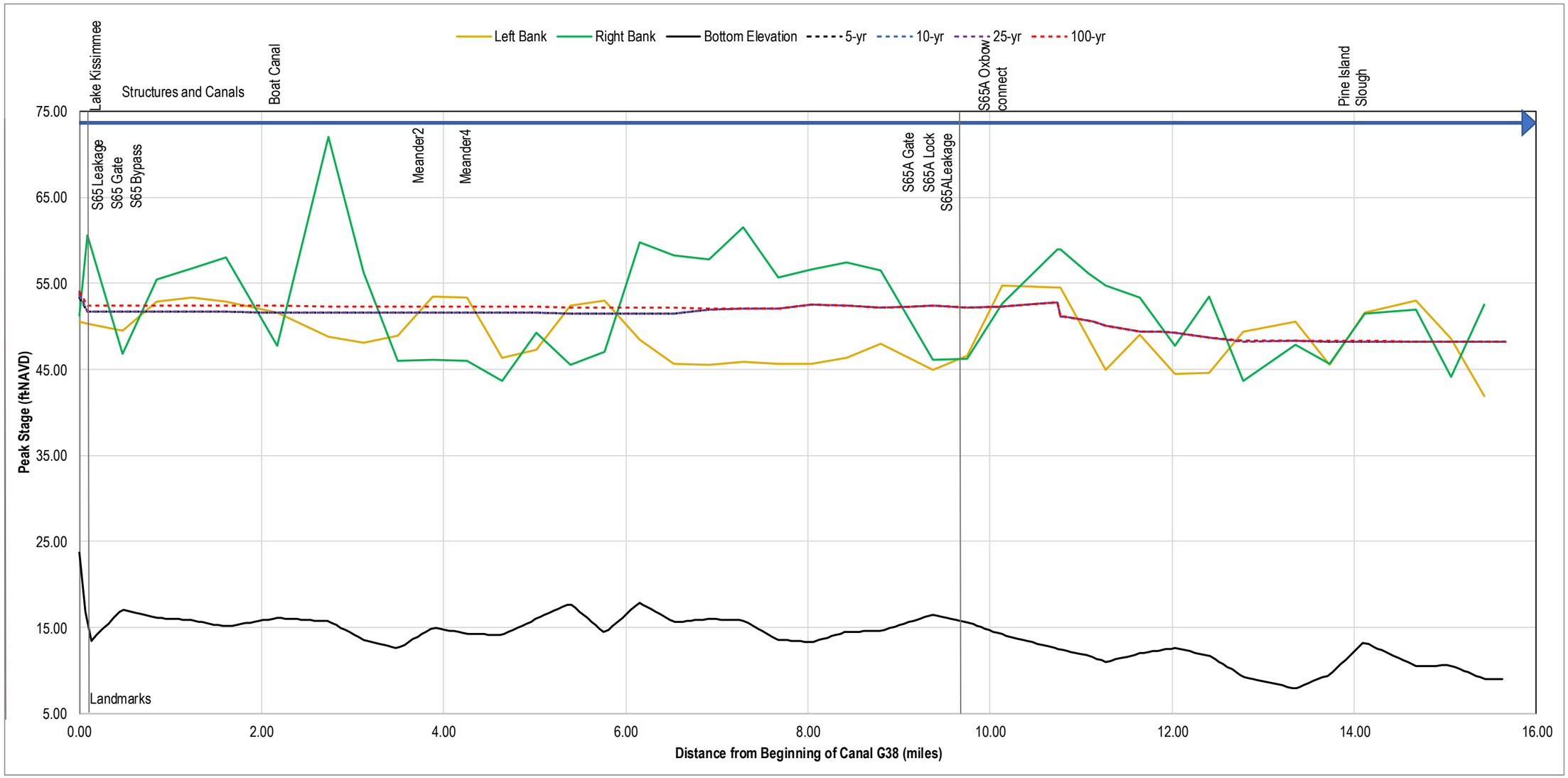


Figure 10-152: Maximum Stage Profile for Canal C-38 under Current Conditions

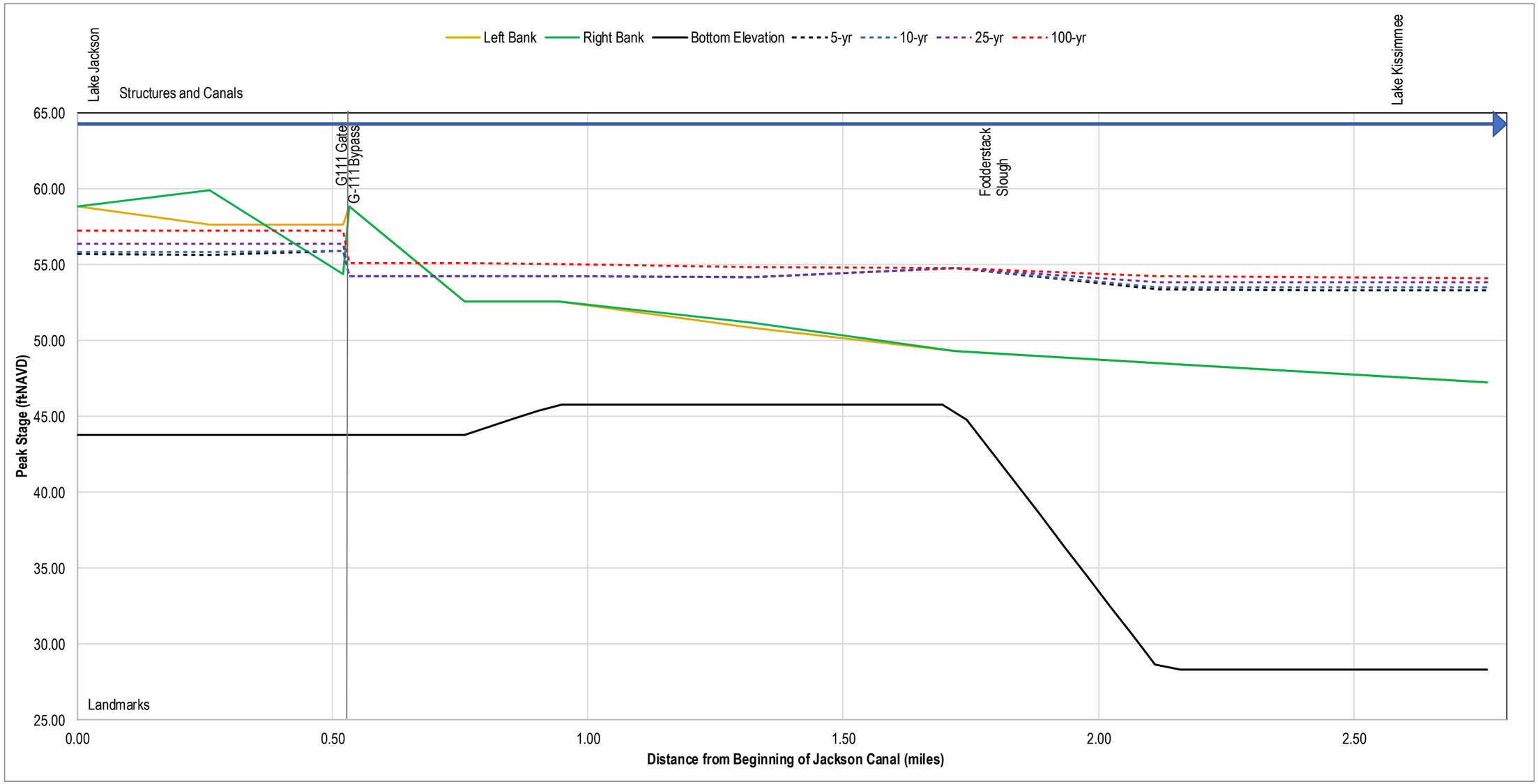


Figure 10-153: Maximum Stage Profile for Jackson Canal under Current Conditions

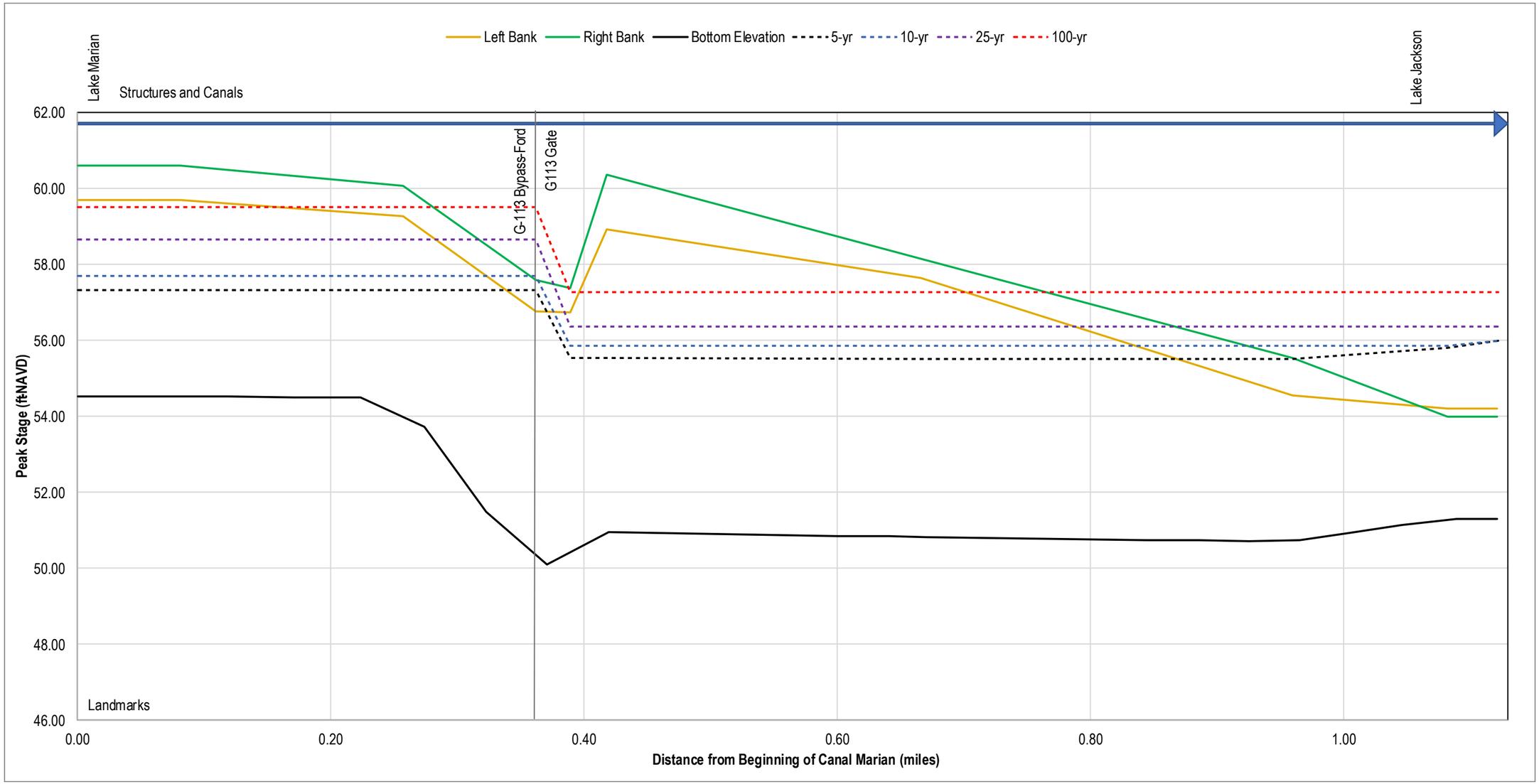


Figure 10-154: Maximum Stage Profile for Lake Marian under Current Conditions

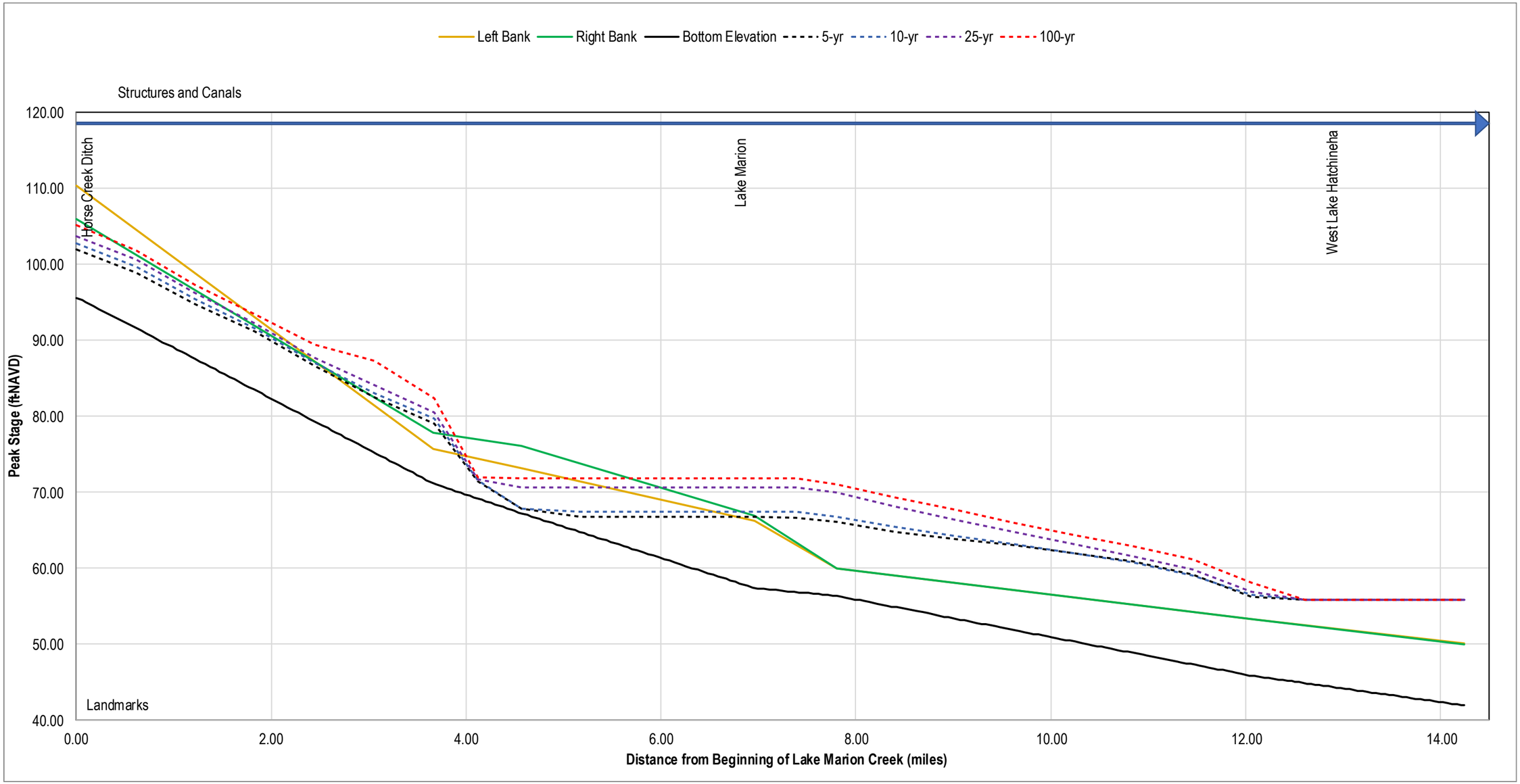


Figure 10-155: Maximum Stage Profile for Marion Creek under Current Conditions

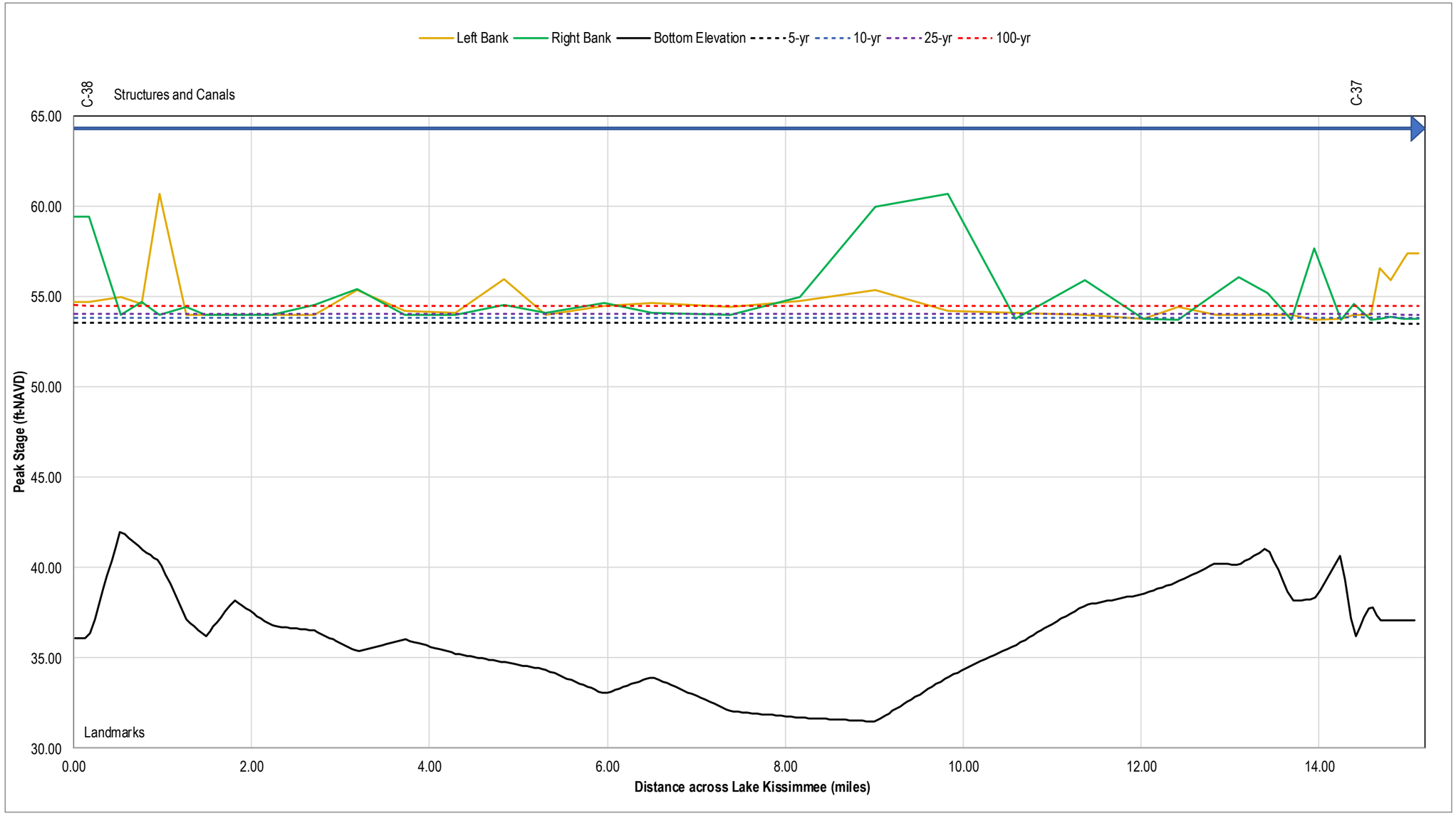


Figure 10-156: Maximum Stage Profile for Lake Kissimmee under Future Conditions

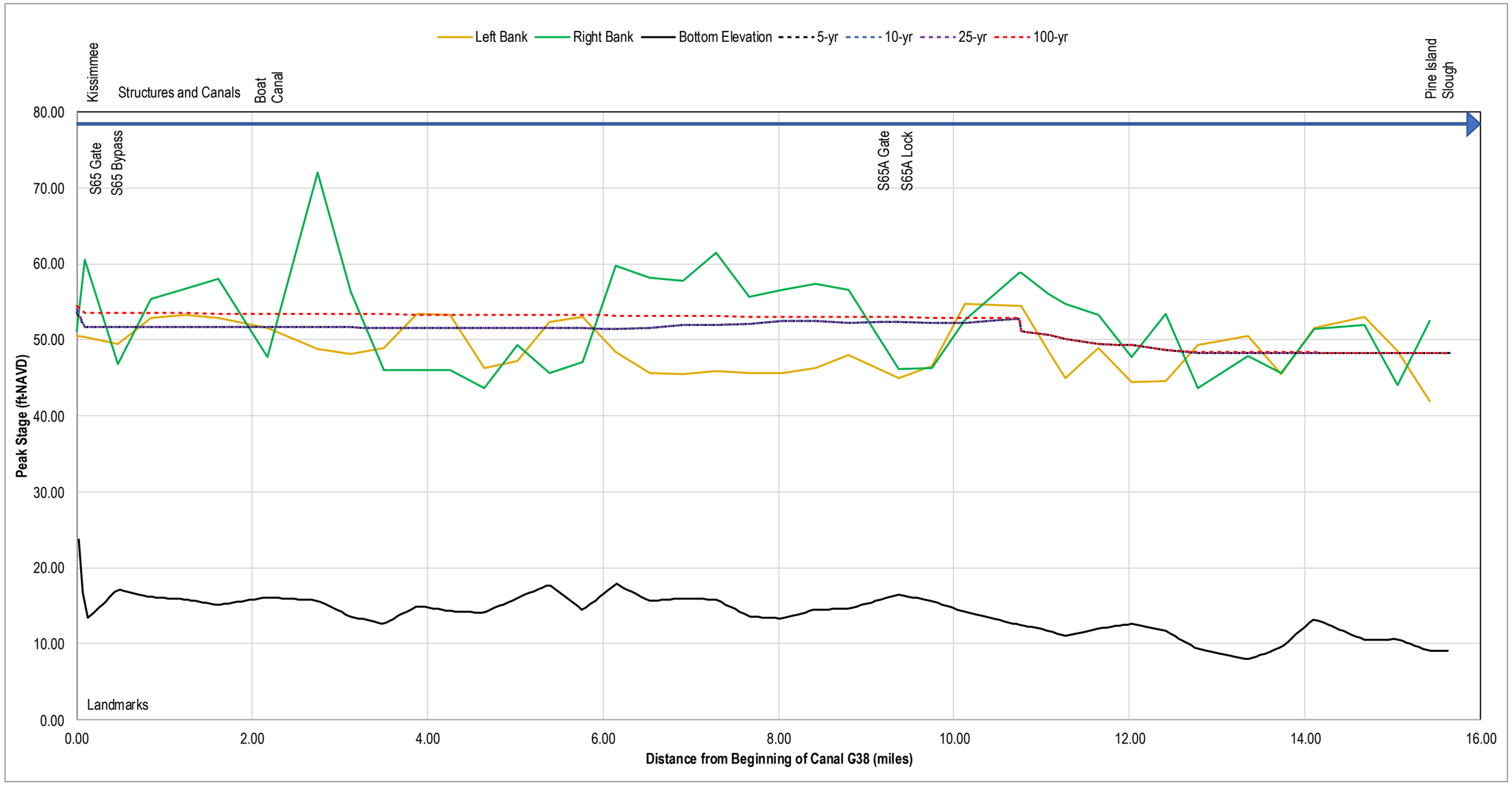


Figure 10-157: Maximum Stage Profile for Canal C-38 under Future Conditions

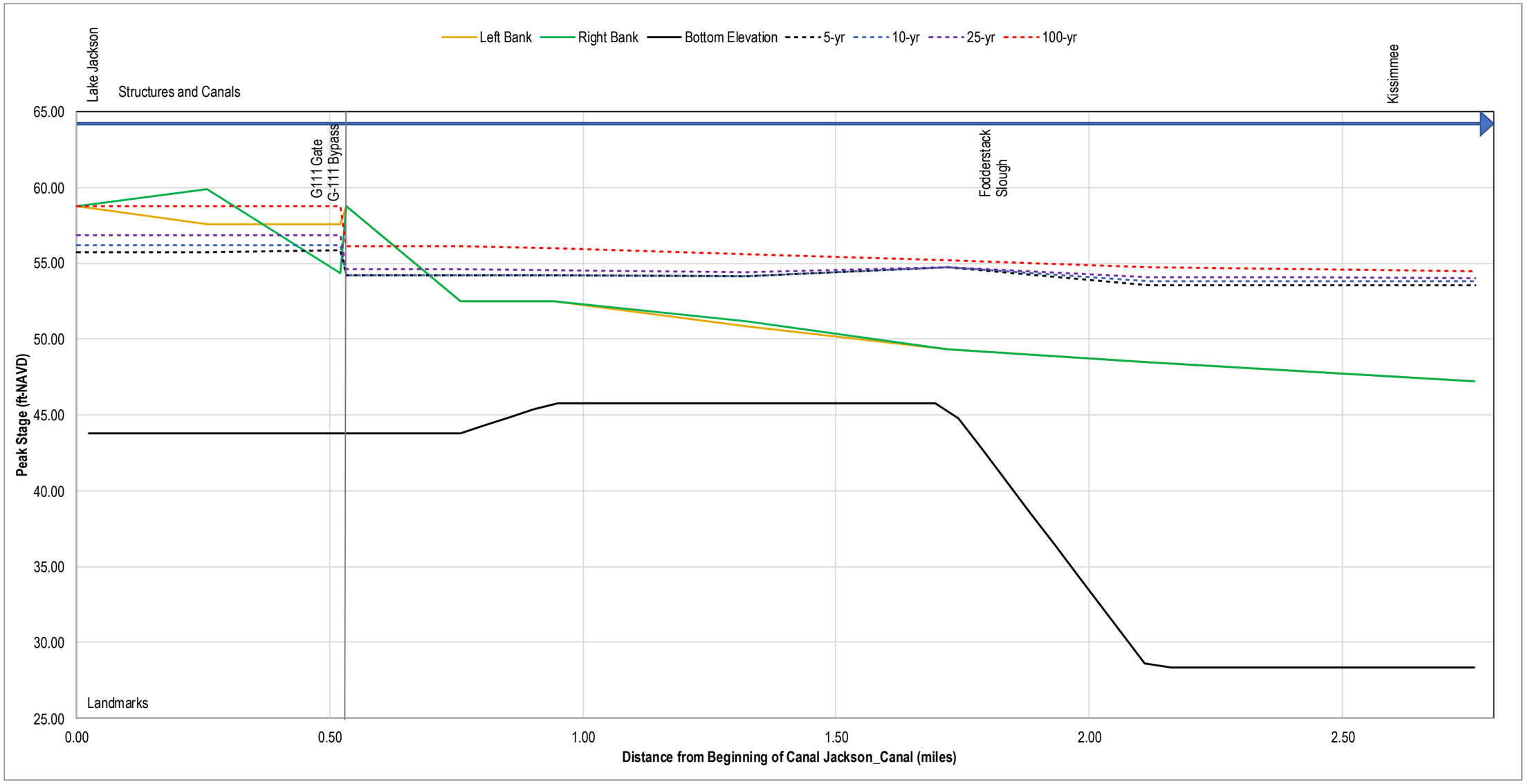


Figure 10-158: Maximum Stage Profile for Jackson Canal under Future Conditions

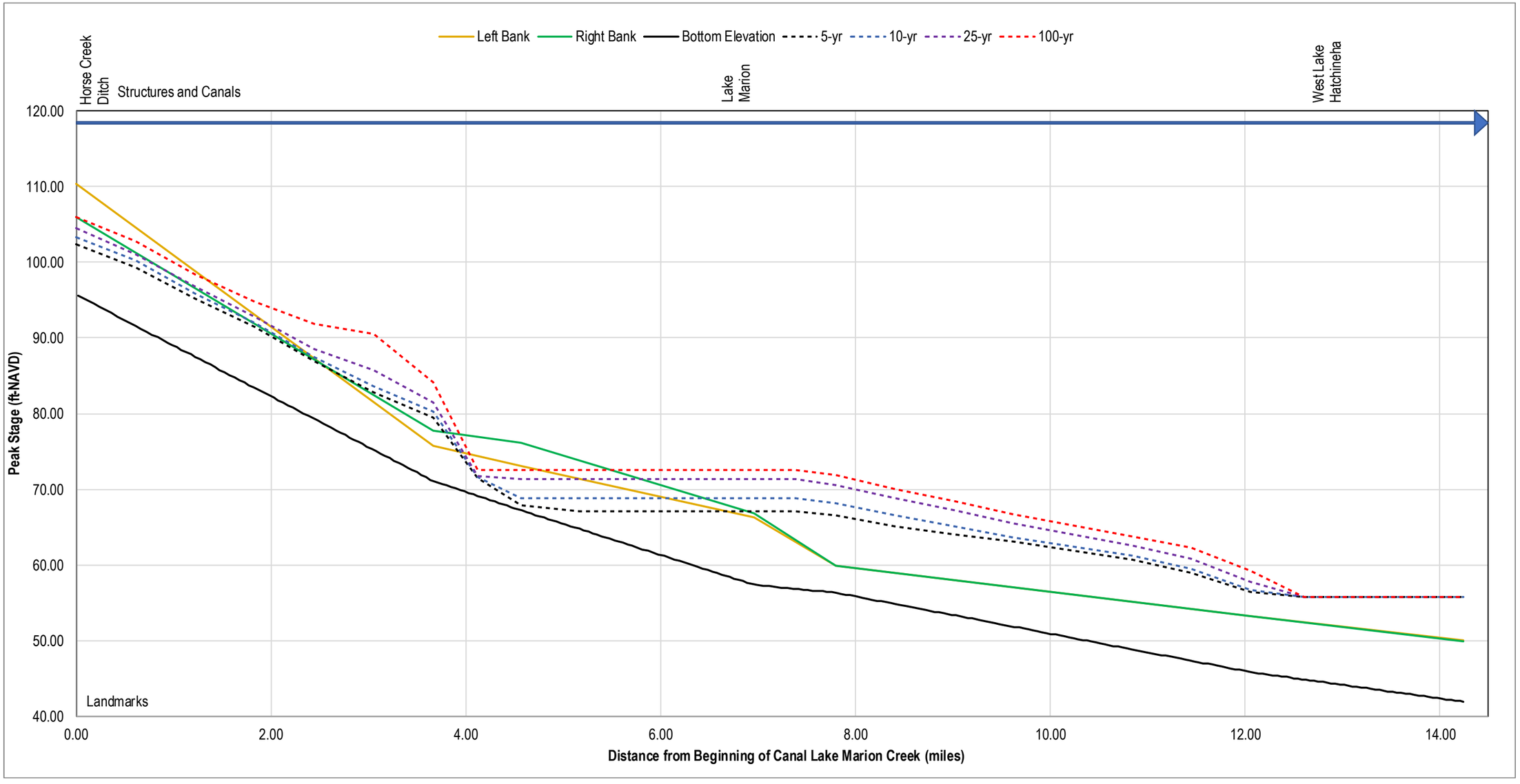


Figure 10-159: Maximum Stage Profile for Marion Creek Canal under Future Conditions

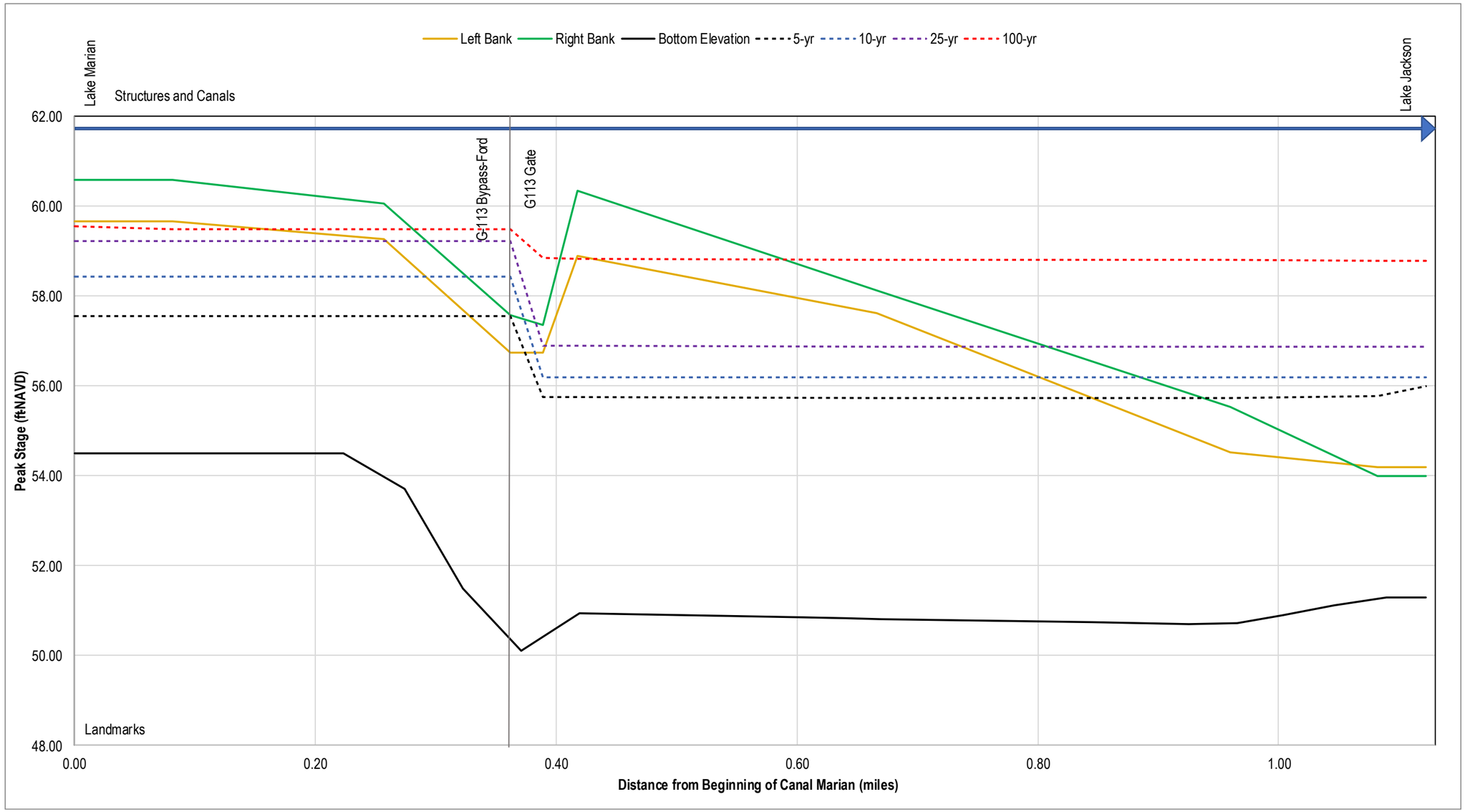


Figure 10-160: Maximum Stage Profile for Lake Marian under Future Conditions

10.9.2 PM #2

Lake Kissimmee LMA's upstream water control structure is S61, and downstream structure is S65. The maximum daily discharge capacity of this LMA is determined by S65 discharge capacity. Per the District's ERP Handbook, Lake Kissimmee LMA has an allowable discharge rate of 31.1 CSM, based on a design frequency of 10-yrs.

Lakes Cypress, Hatchineha and Kissimmee do not have control structures between them. So for PM #2 analysis, the 3 lakes are grouped into K-C-H system LMA. **Table 10-87** summarizes K-C-H system LMA's contributing area, and the peak 12-hour discharge from the contributing drainage area; **Table 10-88** provides the same information under future conditions. The peak 12-hour discharge per square mile was calculated by dividing the peak discharge by the watershed's contributing drainage area, consistent with the Arc Hydro Enhanced Database (AHED). The simulated peak discharge per square mile during the 10-yr event was 3.8 CSM and 4.2 CSM respectively under current and future hydrologic conditions, which are lower than SFWMD's permissible discharge allowance of 31.1 CSM based on a 10-yr design frequency.

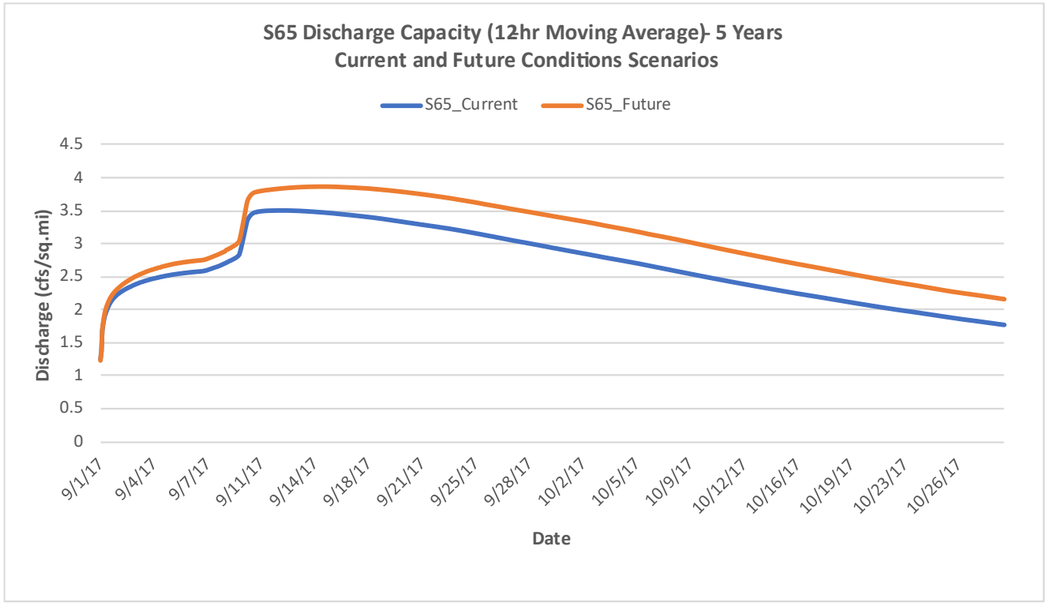
Table 10-87: Drainage Discharge Capacity (cfs/sq. mi) of K-C-H system of LMA under Current Conditions

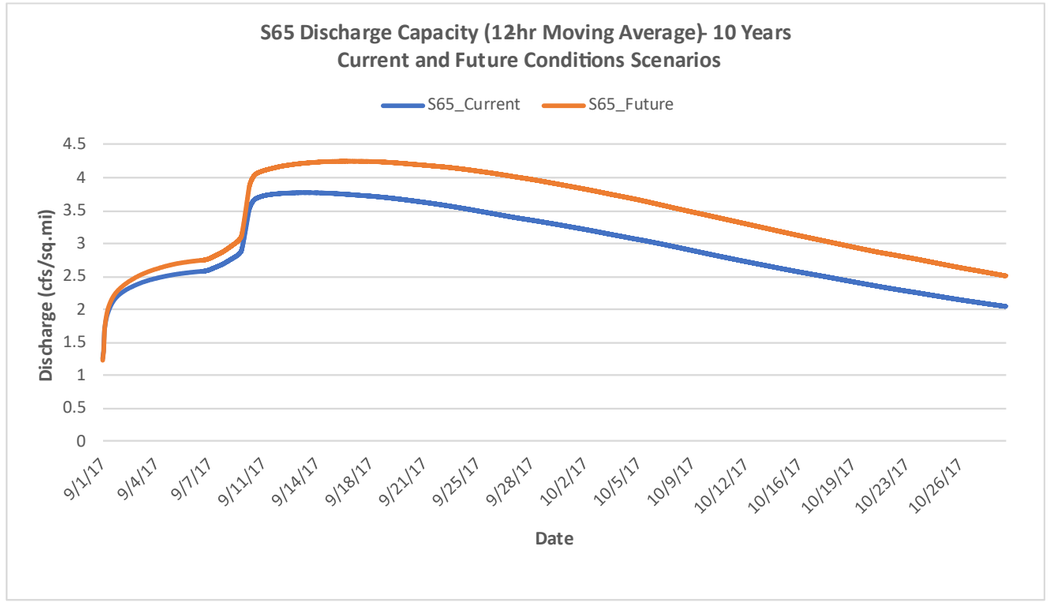
LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
K-C-H system	978.5	3.5	3.8	4.3	16.4	31.1

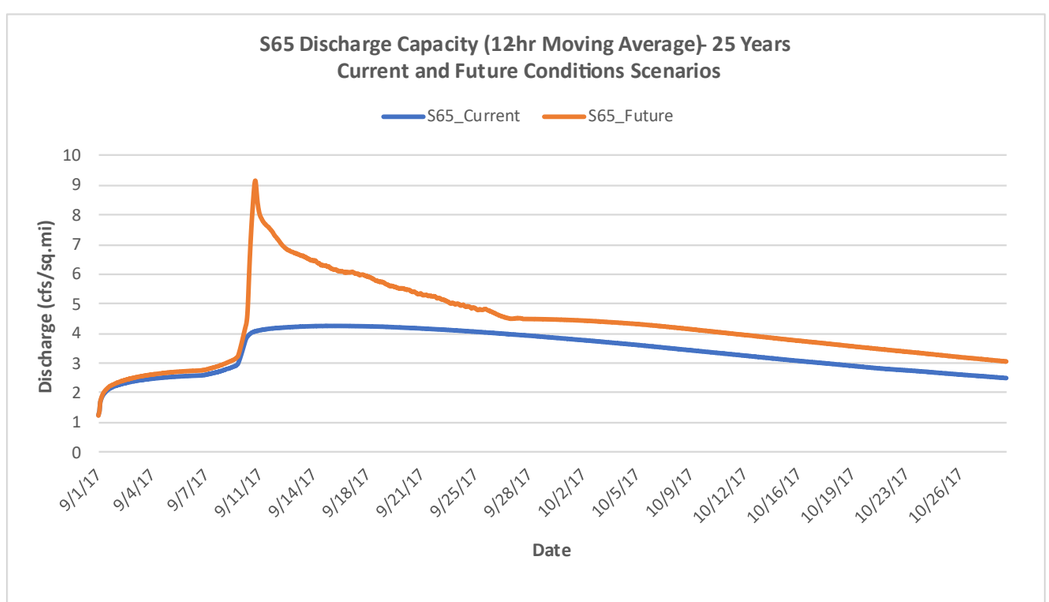
Table 10-88: Drainage Discharge Capacity (cfs/sq. mi) of K-C-H system of LMA under Future Conditions

LMA	Total Area (mi2)	5-yr (CSM)	10-yr (CSM)	25-yr (CSM)	100-yr (CSM)	Allowable 10-yr ERP (CSM)
Lake Toho	978.5	3.9	4.2	9.2	18.4	31.1

Figure 10-161 shows the peak 12-hour average discharge per square mile of drainage area, calculated for each design storm at structure S65. **Figure 10-162** shows the maximum daily discharge through structure S65 for both current and future conditions. The simulated peak 12-hour moving average discharge per square mile during the events 5- to 100-year for future for current conditions is less than 19 CSM which is considerably lower than he district permissible discharge allowance of 31 CSM for this structure. Overall, peak discharge per square mile for current and future conditions are below the District allowable discharge in all design storms, suggesting a pretty good conveyance under extreme rainfall events.







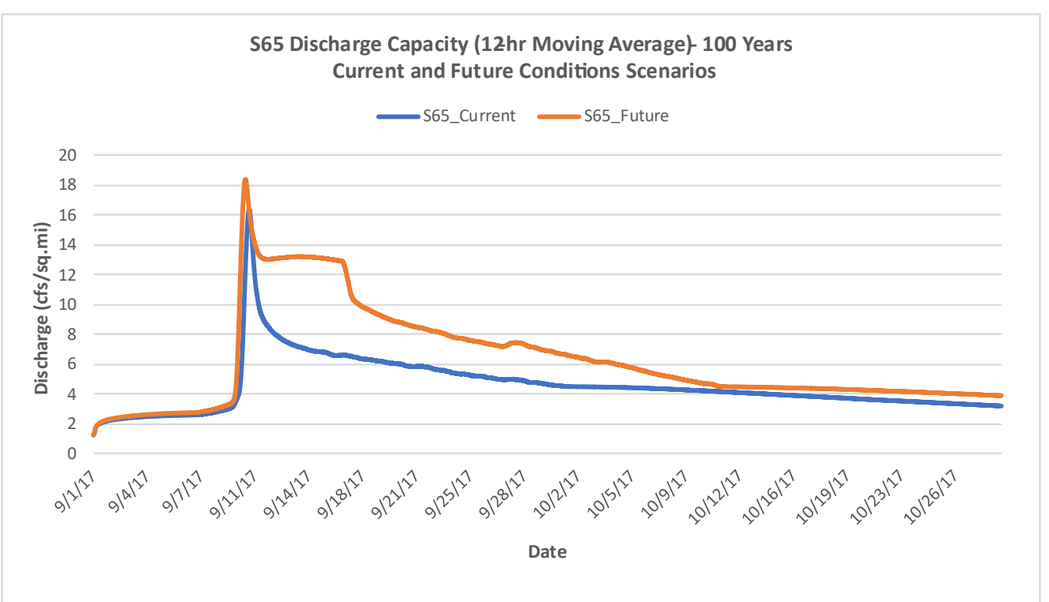


Figure 10-161: 12-hour Moving Average Discharge Hydrographs (cfs/sq-mi) for Structure S65 for 5-, 10-, 25- and 100- Year Design Storms

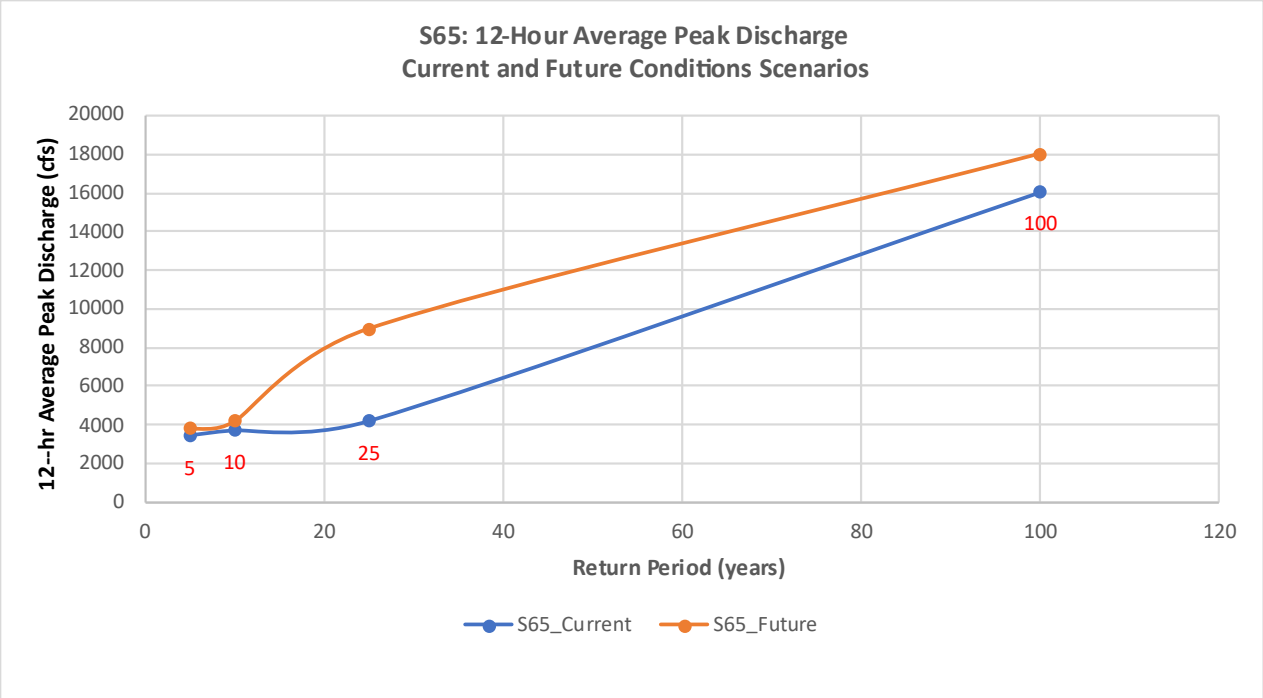


Figure 10-162: Structure S65 12-Hour Average Peak Discharge vs Return Period for Current and Future Scenarios for 5-, 10-, 25-, 100- Year Design Storms

10.9.3 PM #3

The SFWMD structure design parameters (**Table 10-89**) were used to evaluate the S65 performance against the various design storms. **Table 10-90 through Table 10-95** summarize simulated instantaneous stages and flows at structure S65 under current conditions and future conditions at the time of peak tailwater, peak headwater and peak discharge. Simulations show that, at the time of peak tailwater at structure S65, the simulated headwater elevation at S65 for all design storm events (5-yr, 10-yr, 25-yr, and 100-yr) is less than the SPF headwater elevation and higher than design headwater levels for both current and future conditions. The simulated flow at structure S65 is higher than the firm capacity (3000 cfs) for all design storm events in the current conditions and future conditions.

During the peak headwater at the structure S65; the simulated tailwater elevation at S65 is higher than design tailwater elevation during all design storm events for current and future conditions. The simulated flows at structure S65 are higher than the design discharge (3000 cfs) for all the design storms for current and future conditions.

During the peak flow at structure S65, the headwater and tailwater elevations are consistently higher than design headwater and tailwater for all design storm events for current and future conditions. **Figure 10-163 and Figure 10-164** have the same data as time series.

Table 10-89: Design Parameters for Structure S65

Structure	S65
Design Discharge	Firm capacity 3,000 cfs; Full capacity 11,000 cfs
SPF Discharge	11,000 cfs (not updated after backfilling of C-38)
Design Headwater Elevation	50.31 feet NAVD88 (51.5 feet NGVD29)
Design Tailwater Elevation	47.81 feet NAVD88 (49.0 feet NGVD29)
SPF Headwater Elevation	56.81 feet NAVD88 (58.0 feet NGVD29)
SPF Tailwater Elevation	51.91 feet NAVD88 (53.1 feet NGVD29)
Optimum Headwater	47.31 to 51.31 feet NAVD88 (48.5 to 52.5 feet NGVD29)
Optimum Tailwater	45.11 feet NAVD88 (46.3 feet NGVD29)

Table 10-90: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Headwater under Current Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	53.32	49.58	3433.03	3.74	9/12/17 4:55 PM
10-year	53.51	49.71	3699.39	3.80	9/14/17 4:55 AM
25-year	53.85	49.85	4173.35	4.00	9/16/17 1:43 PM
100-year	54.09	49.98	9524.55	4.11	9/10/17 11:40 PM

Table 10-91: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Headwater under Future Conditions

Instantaneous Values					
Design Storm	Peak HW (ft NAVD88)	TW (ft NAVD88)	Discharge at peak HW (cfs)	Head Differential at Peak HW (ft)	Date & Time
5-year	53.57	49.78	3782.42	3.79	9/15/17 11:00 AM
10-year	53.84	49.99	4167.28	3.85	9/18/17 3:43 AM
25-year	54.02	49.96	5810.47	4.05	9/11/17 1:25 AM
100-year	54.45	53.07	15668.43	1.38	9/11/17 7:00 AM

Table 10-92: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Tailwater under Current Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	TW (ft NAVD88)	Discharge at Peak HW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	49.58	49.58	3424.35	3.73	9/12/17 6:00 PM
10-year	49.74	49.74	3688.23	3.76	9/12/17 11:55 PM
25-year	50.00	50.00	4104.24	3.81	9/12/17 11:00 PM
100-year	52.44	52.44	10891.94	1.56	9/11/17 5:00 PM

Table 10-93: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Tailwater under Future Conditions

Instantaneous Values					
Design Storm	Peak TW (ft NAVD88)	HW (ft NAVD88)	Discharge at Peak TW (cfs)	Head Differential at Peak TW (ft)	Date & Time
5-year	49.89	53.55	3751.94	3.66	9/12/17 8:00 PM
10-year	50.09	53.79	4083.05	3.70	9/12/17 6:00 PM
25-year	51.39	54.00	7386.03	2.61	9/12/17 11:55 AM
100-year	53.54	54.42	12773.89	0.88	9/12/17 5:00 AM

Table 10-94: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Discharge under Current Conditions

Instantaneous Values					
Design Storm	Peak Discharge (cfs)	HW (ft NAVD88)	TW (ft NAVD88)	Head Differential at Peak Q (ft)	Date & Time
5-year	3433.03	53.32	49.58	3.74	9/12/17 4:55 PM
10-year	3700.87	53.51	49.71	3.80	9/14/17 2:43 AM
25-year	4173.35	53.85	49.85	4.00	9/16/17 1:43 PM
100-year	19617.32	54.00	51.45	2.56	9/11/17 4:55 AM

Table 10-95: Summary of the Instantaneous Conditions at Structure S65 at time of Peak Discharge under Future Conditions

Instantaneous Values					
Design Storm	**Peak Discharge (cfs)**	**HW (ft NAVD88)**	**TW (ft NAVD88)**	**Head Differential at Peak Q (ft)**	**Date & Time**
5-year	3782.49	53.57	49.78	3.79	9/15/17 1:00 PM
10-year	4167.28	53.84	49.99	3.85	9/18/17 3:43 AM
25-year	10773.28	54.00	50.35	3.65	9/11/17 3:55 AM
100-year	20757.34	54.39	51.80	2.59	9/10/17 10:10 PM

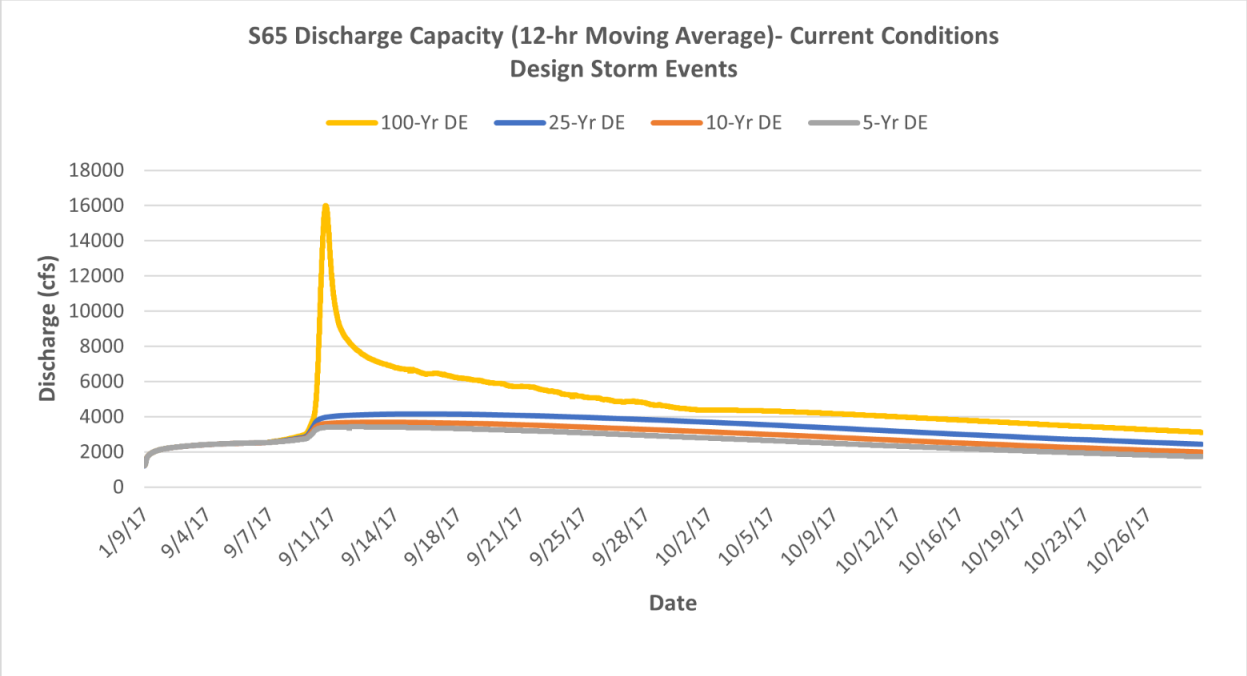


Figure 10-163: 12-hr Moving Average Flows in cfs at S65 under Current Conditions

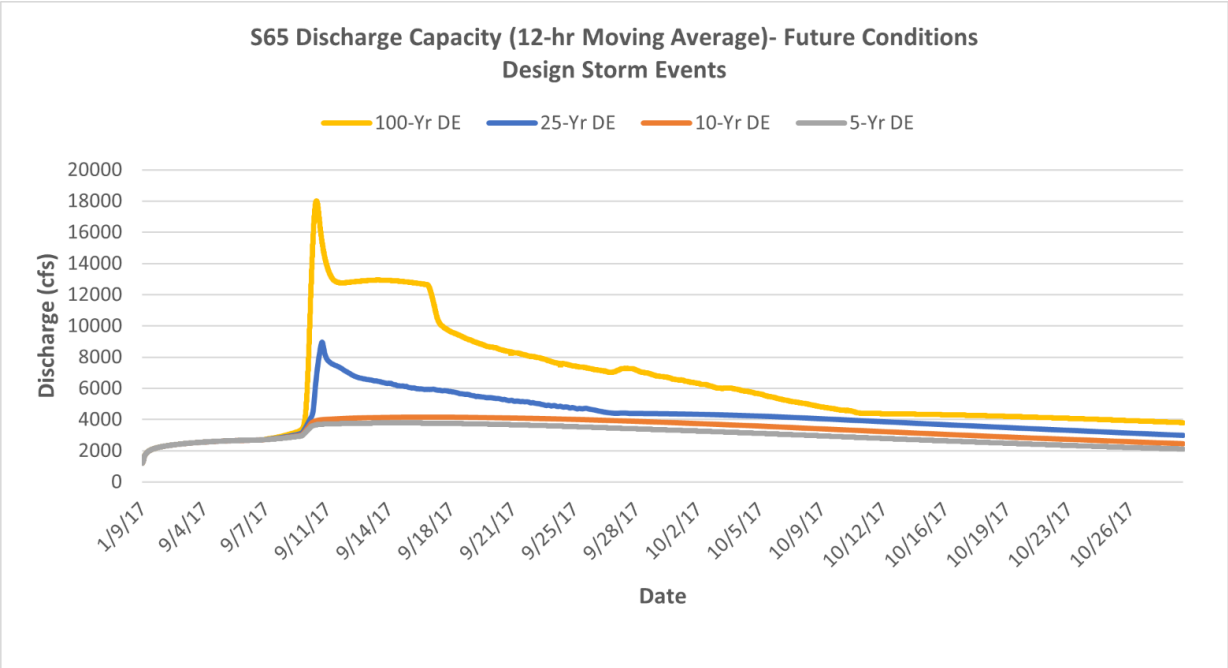


Figure 10-164: 12-hr Moving Average Flows in cfs at S65 under Future Conditions

10.9.4 PM #4

PM#4-UKB is a limited and simplified evaluation of how flooding in the LKB impacts this LMA – KCH. This assists in the formulation of a future work plan or an extension of this work plan.

Figure 10-165 (excerpt from South Florida Environmental Report) shows the conditional release rules in KB. It shows when FTKISS > 51.75ft NGVD (50.56 ft NAVD), S65 discharges needs to be restricted to 3000 cfs. FTKISS is the southern downstream boundary condition in UKB FPLOS Model. For assessment of PM#4-UKB, the downstream BC was set to 50.56 ft NAVD, the stage in FT-KISS that would indicate flooding, for 7 days. **Figure 10-166** shows the downstream boundary conditions used in the design event simulations at the south model domain (DS_BC), and the modified BC (DS_BC_PM4-UKB) to assess the impact of LKB flooding on UKB.

Table 3: Conditional Release Capacity

Observed Elevation at Site	Decision Tree (Elevations are ft NGVD29)				S-65 Release Capacity (cfs) (Use smallest value of four sites)
KREN	>	52.00 - **B**	---------->		Up to 3,000
	<=	52.00 - **B**	and >	51.32	3,000 to 18,000
	<	51.32	---------->		Up to 18,000
FTKISS	>	51.75 - **B**	---------->		Up to 3,000
	<=	51.75 - **B**	and >	50.05	3,000 to 18,000
	<	50.05	---------->		Up to 18,000
PC11	>	41.54 - **B**	---------->		Up to 3,000
	<=	41.54 - **B**	and >	39.40	3,000 to 18,000
	<	39.40	---------->		Up to 18,000
PD42	>	37.75 - **B**	---------->		Up to 3,000
	<=	37.75 - **B**	and >	35.10	3,000 to 18,000
	<	35.10	---------->		Up to 18,000

* B is a "buffer" variable. A value of 0.25 ft will be used for B unless otherwise recommended by SFWMD and agreed to by USACE Jacksonville District.

**Site KREN is intended to have a Project Lands Boundary elevation of 52.87 ft. This release capacity guidance table will be updated from 52.00 ft to 52.87 ft when the appropriate flowage easements are in place to allow for increased elevations and updated release capacity.

*** All S65 releases are subject to MAGOs constraints at this structure.

Figure 10-165: Conditional Release Capacity in Kissimmee Basin

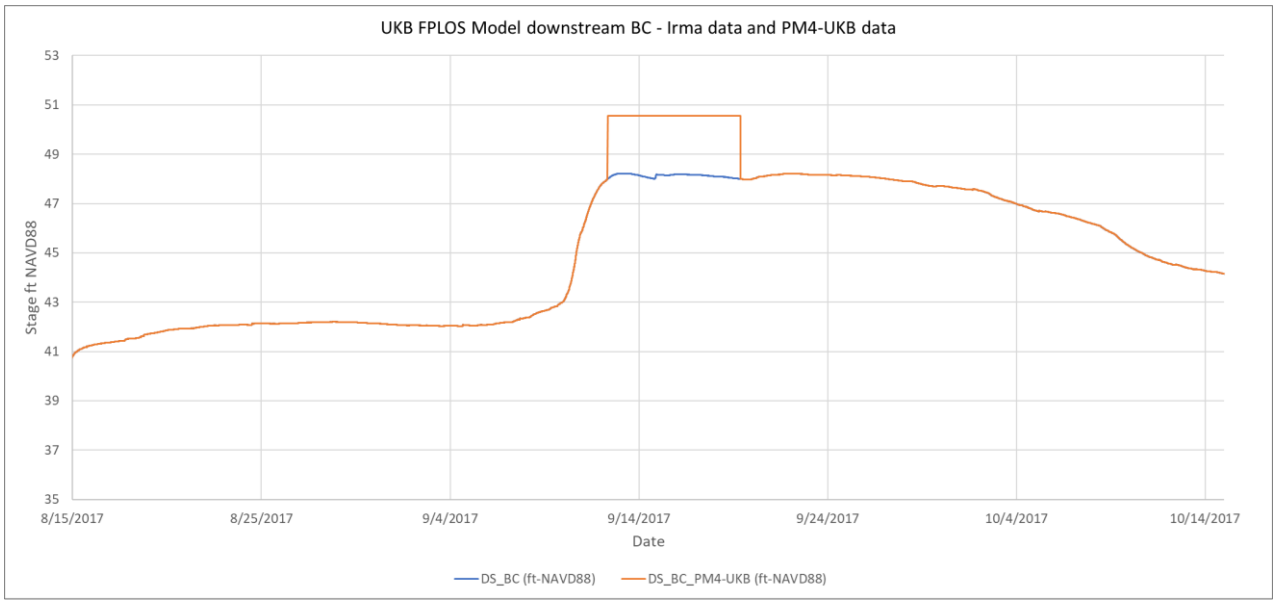


Figure 10-166: UKBFPLOS Model Downstream Boundary Conditions for PM#4-UKB

Four (4) runs were made to assess the impact of LKB flooding on S65 discharges: 5- and 100-yr under current conditions and 5- and 100-yr under future conditions. **Figure 10-167** illustrates the extreme case – it shows the comparison of the 100-yr design event simulations with DS_BC and DS_BC_PM4-UKB BC, under future conditions. Under future conditions, the S65HW increases by 0.1 ft for 100-yr design event with DS_BC_PM4-UKB, relative to the 100-yr run with DS_BC. The higher stage lasts longer than 1 week. That increases the inundation depth by 0.1 ft for 1 week over the 103653 acres of Lake Kissimmee and its surrounding areas. This would also impact other lakes and reduce the ability to discharge through the structures.

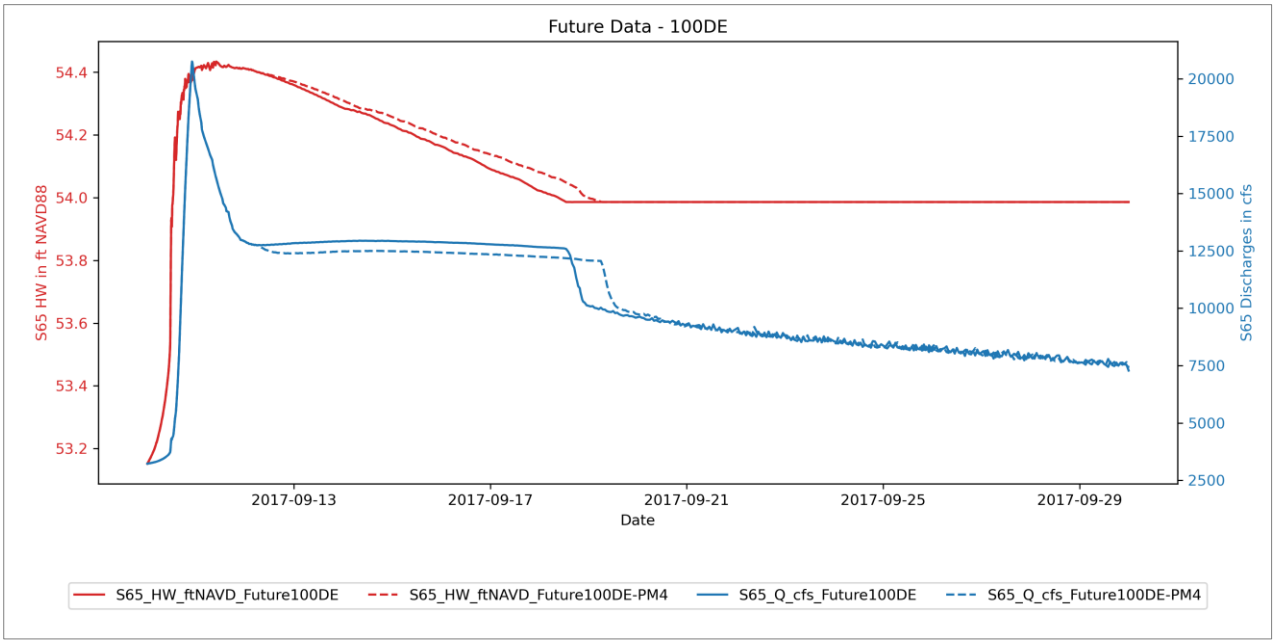


Figure 10-167: Assessment of PM#4-UKB at S65 HW for 100-yr Design Event under Future Conditions

10.9.5 PM #5

The gridded data of maximum overland depth was extracted for each of the design storms under current and future conditions and evaluated for the Lake Kissimmee LMA. **Table 10-96 and Figure 10-167** are tabular and graphical representations of the flood inundation area in square miles for this LMA. **Figure 10-168 through Figure 10-172** show the maximum overland flooding depths in the KCH System for the 5-, 10-, 25- and 100-yr design events, under current and future conditions.

Collectively, the table, plot and the figures show that the greatest distribution of flooding depths in the LMA are between zero and 0.25 feet of flooding, which is not critical. The tables, plots and maps show that the inundation depths for the remainder of Lake Kissimmee are less than 1 ft. Most of these areas area small lakes and wetlands.

Table 10-96: Incremental Flood Inundation Area (sq. mi.) in Lake Kissimmee LMA

Flooding Depth (ft)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Depth<0.25	214.05	184.31	136.66	81.57	209.40	180.64	134.67	80.99
0.25<=Depth<0.5	41.04	56.26	80.32	87.69	39.81	53.73	76.61	84.92
0.5<=Depth<0.75	20.12	22.98	27.61	39.70	19.93	22.58	26.70	38.60
0.75<=Depth<1.0	13.58	16.04	18.90	23.98	13.09	15.49	18.30	23.20
1.0<=Depth<1.25	9.06	11.90	14.25	17.71	8.86	10.95	13.64	17.46
1.25<=Depth<1.5	7.08	8.10	10.97	13.73	7.95	7.93	10.43	13.43
1.5<=Depth<1.75	5.63	6.59	8.09	11.71	5.35	7.41	8.19	11.37
1.75<=Depth<2.0	4.49	5.44	6.70	9.54	3.75	5.54	7.59	9.26
2.0<=Depth<2.25	6.18	6.61	7.63	7.38	5.41	5.93	7.79	8.23
2.25<=Depth<2.5	9.54	9.02	8.90	7.65	8.36	8.19	8.30	8.68
>2.5	84.14	87.65	94.86	114.26	93.02	96.51	102.70	118.76
Lake Kissimmee LMA	**265541.50**	**acres**						
	414.91	**sq.mi.**						

Note: *The lake area of 83.50 sq. mi. was included in this table computations.*

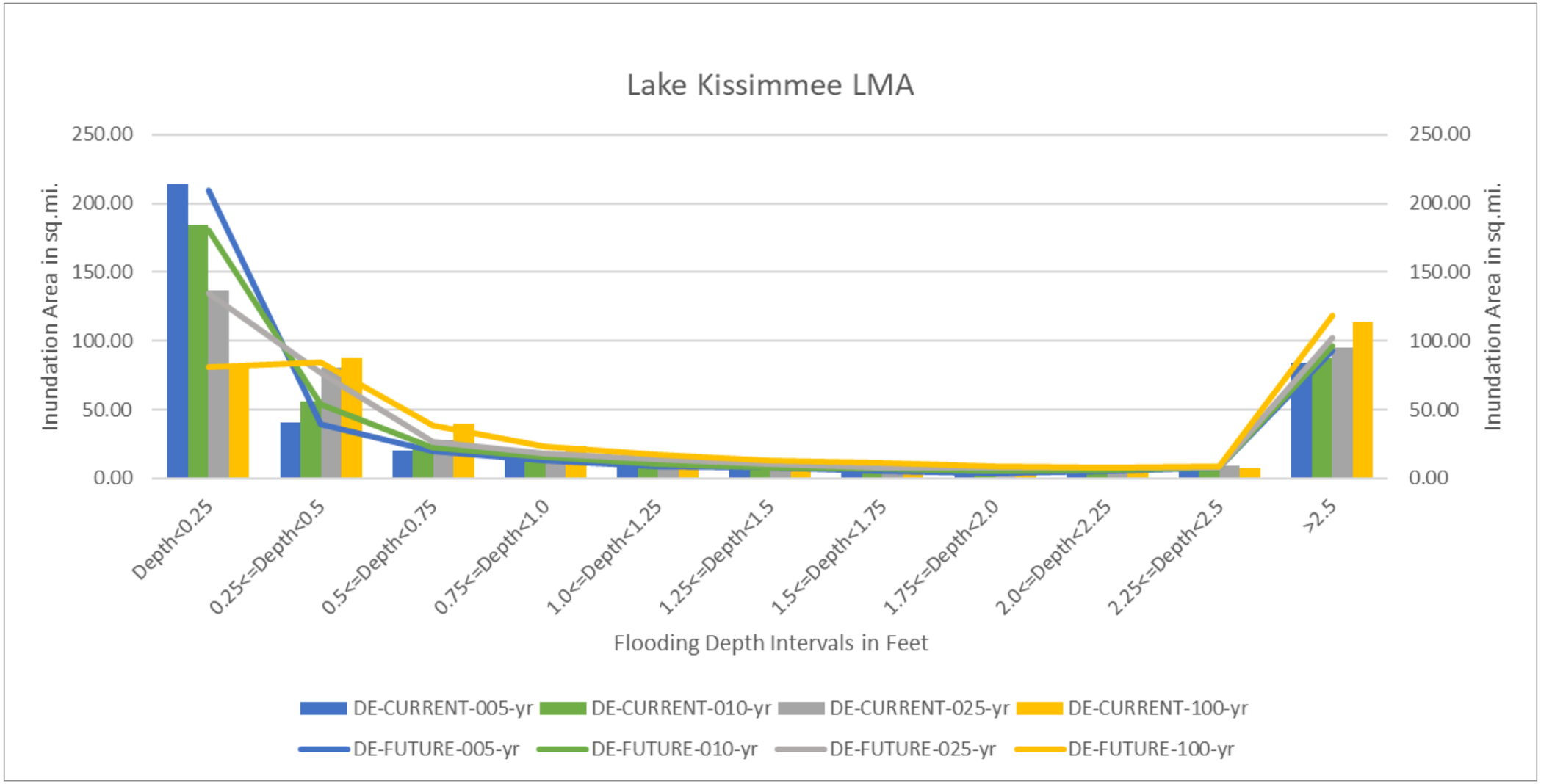


Figure 10-168: Incremental Flood Inundation Depth in feet in Lake Kissimmee LMA under Current and Future Conditions

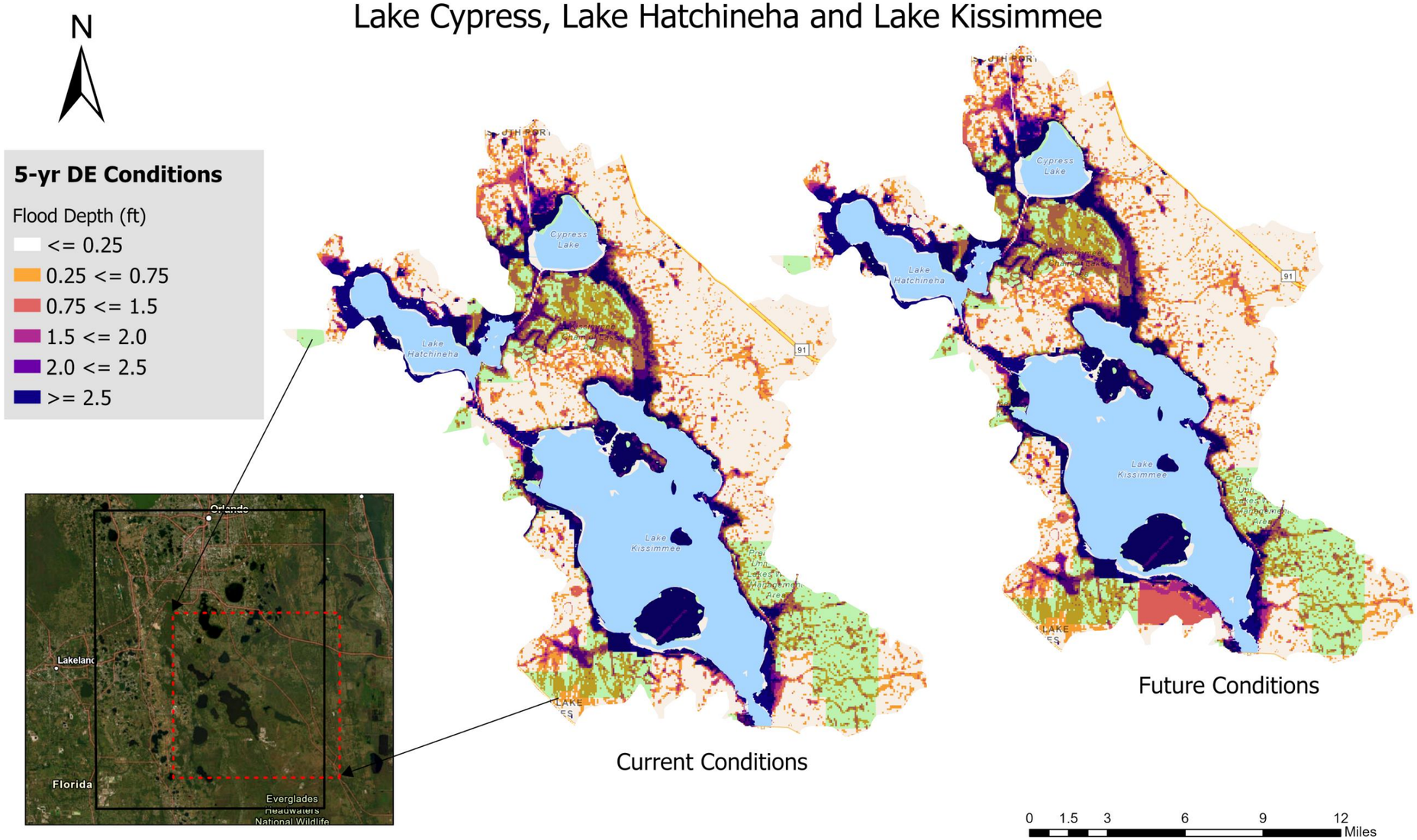
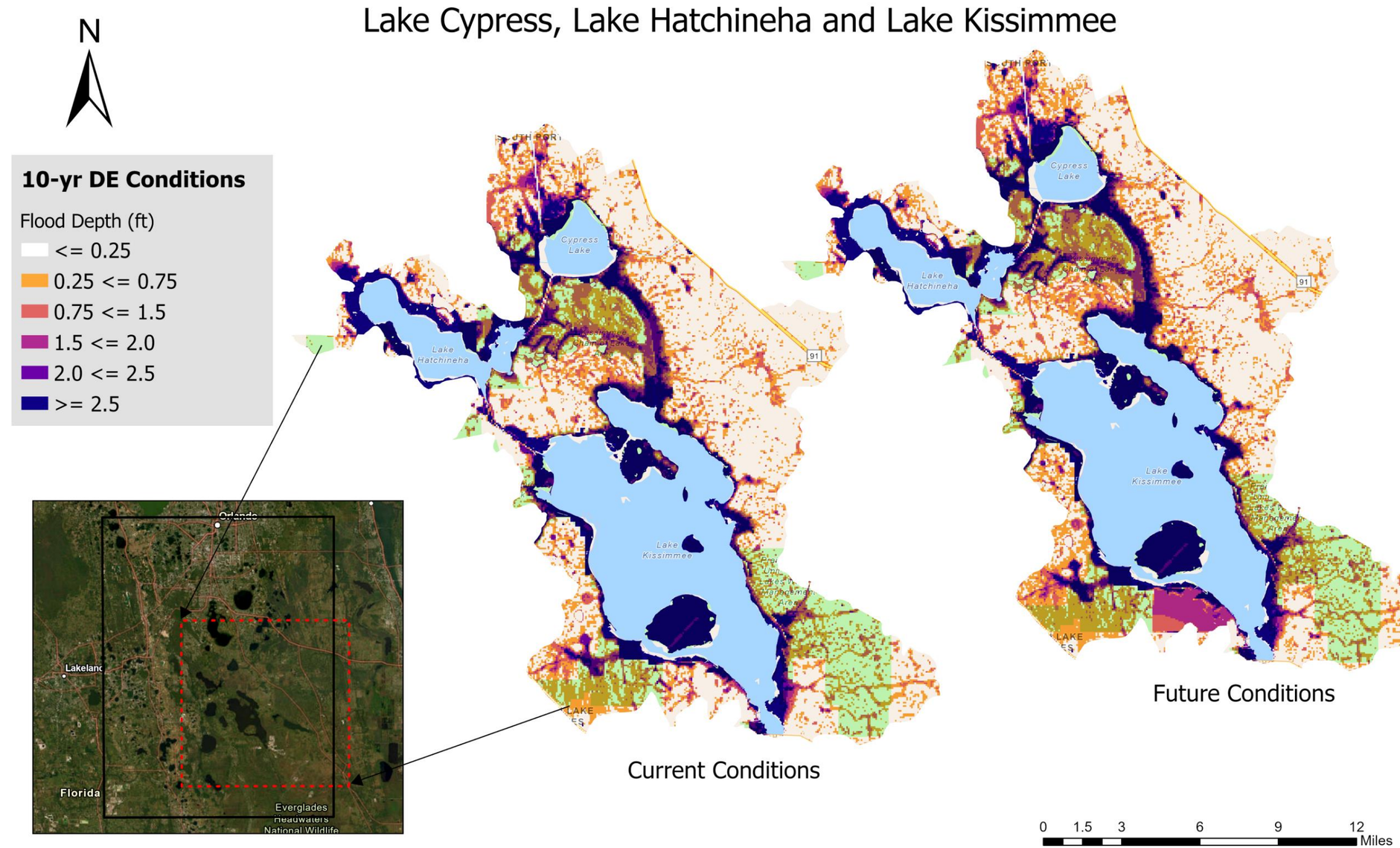
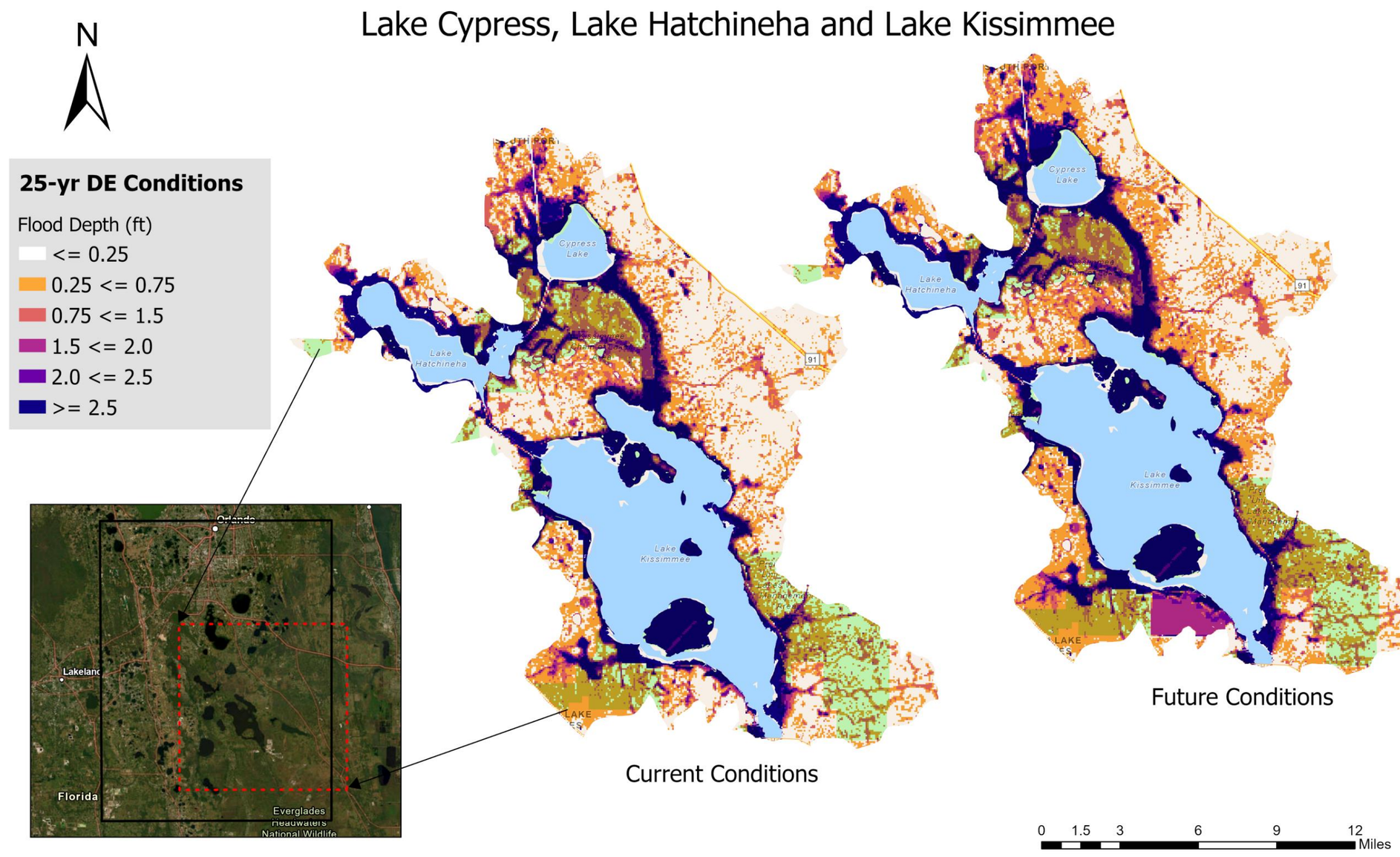


Figure 10-169: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



Hazen

Figure 10-170: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



Hazen

Figure 10-171: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA

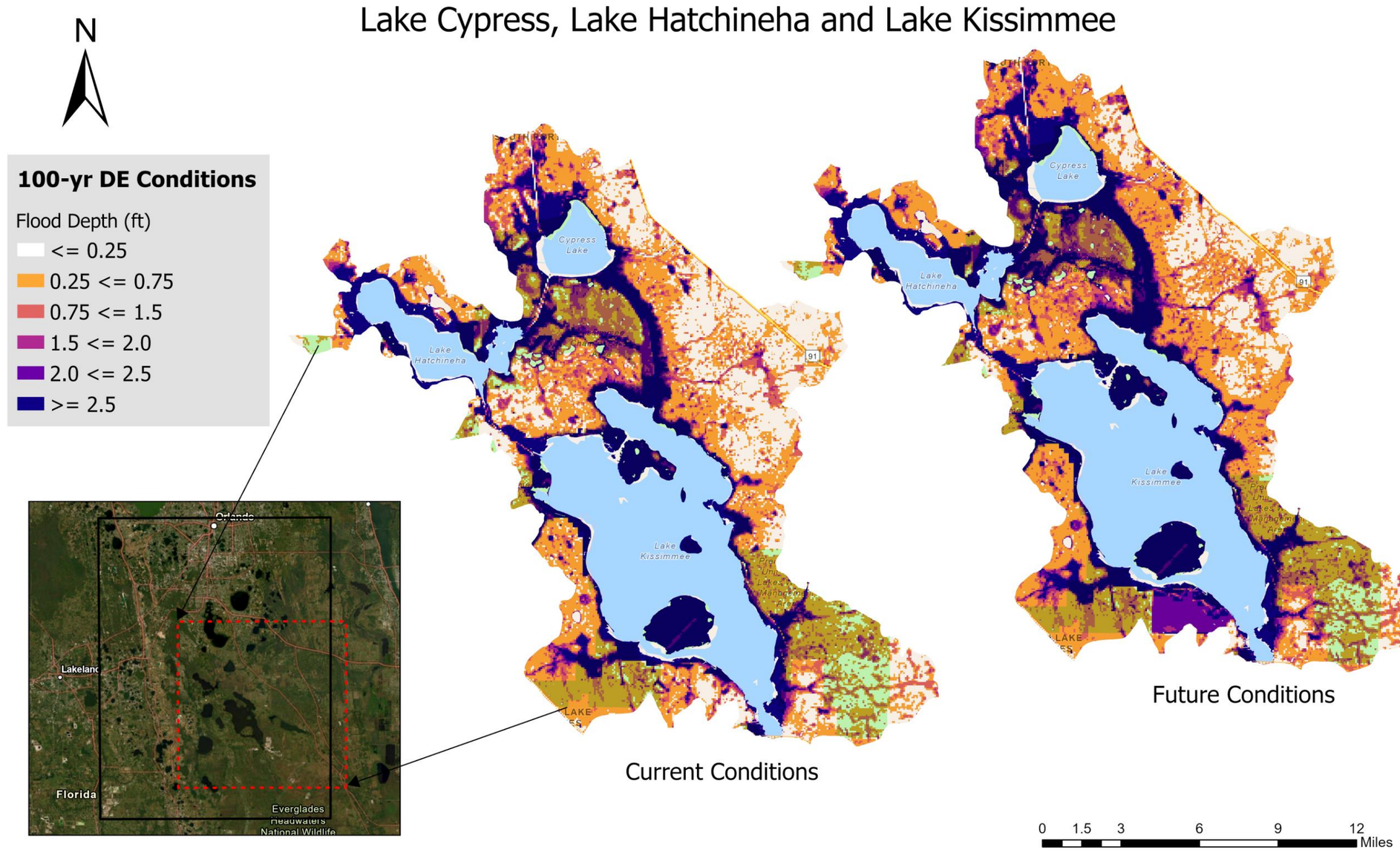


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Figure 10-172: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA

10.9.6 PM #6

Table 10-97 tabulates the total area of flood inundation (in square miles) per flood duration interval/range within KCH System. **Figure 10-173** show these data graphically, with the flood duration plotted against the percentage of area inundated for the areas within the KCH System for the 5-, 10-, 25- and 100-yr storm, under current and future conditions. **Figure 10-174 through Figure 10-177** show the flood duration in KCH System, except lakes Kissimmee, Cypress and Hatchineha which are expected to have water year-round.

The table and the plot show that the greatest distribution of flooding duration in KCH System is between zero and one hour of flooding, which is insignificant. The second greatest distribution of flooding duration are areas with longer than 360 hours of flooding, which are the lakes. The maps show 85% of the areas are within the extreme categories of flood duration (< 1 hour, longer than 360 hours). The remaining 85% of the areas have standing water that does not drain; these are areas surrounding the lakes or connections between the lakes.

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Table 10-97. Areas of Incremental Flood Duration (sq. mi.) in Lake Kissimmee LMA

Flooding Duration (hours)	De-Current-005-Yr	De-Current-010-Yr	De-Current-025-Yr	De-Current-100-Yr	De-Future-005-Yr	De-Future-010-Yr	De-Future-025-Yr	De-Future-100-Yr
Hours<1	236.80	221.78	196.70	144.89	226.94	208.12	173.96	108.62
1<=Hours<4	7.35	9.57	15.41	36.80	7.79	11.26	22.39	47.03
4<=Hours<8	4.97	6.55	9.48	13.28	5.59	7.58	10.88	16.45
8<=Hours<12	2.76	3.72	4.88	6.47	3.28	3.98	5.62	7.99
12<=Hours<24	6.74	8.49	10.31	14.64	7.03	8.81	11.60	16.89
24<=Hours<48	6.52	7.64	9.27	12.46	6.64	8.23	10.44	14.18
48<=Hours<96	5.72	6.96	9.07	11.48	6.13	7.56	9.92	14.20
96<=Hours<168	5.37	6.03	6.91	9.29	5.36	6.00	7.66	11.97
168<=Hours<240	3.74	4.24	4.62	5.67	3.52	4.03	4.85	7.07
240<=Hours<360	5.22	5.09	5.90	6.63	4.90	5.18	6.09	7.59
360<=Hours<1417	129.73	134.83	142.36	153.29	137.72	144.15	151.49	162.91
Lake Kissimmee LMA	**265541.50**	**acres**						
	414.91	**sq.mi.**						

***Note**: The lake area of 83.50 sq. mi. was included in this table computations.*

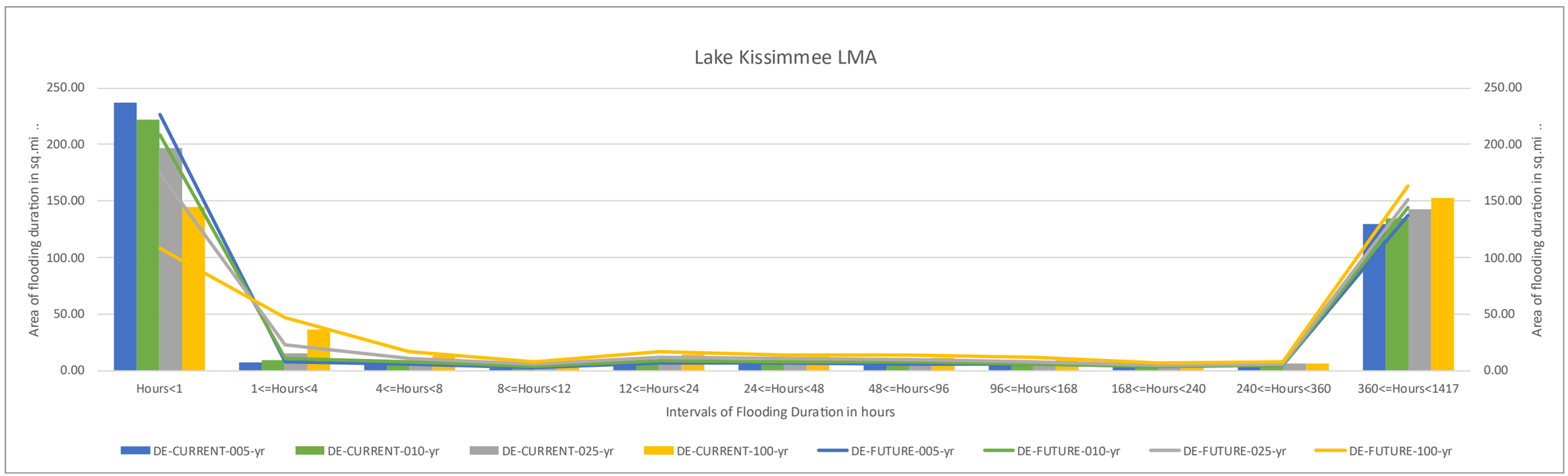

Figure 10-173: Areas of Incremental Flood Duration in hours in Lake Kissimmee LMA under Current and Future Conditions

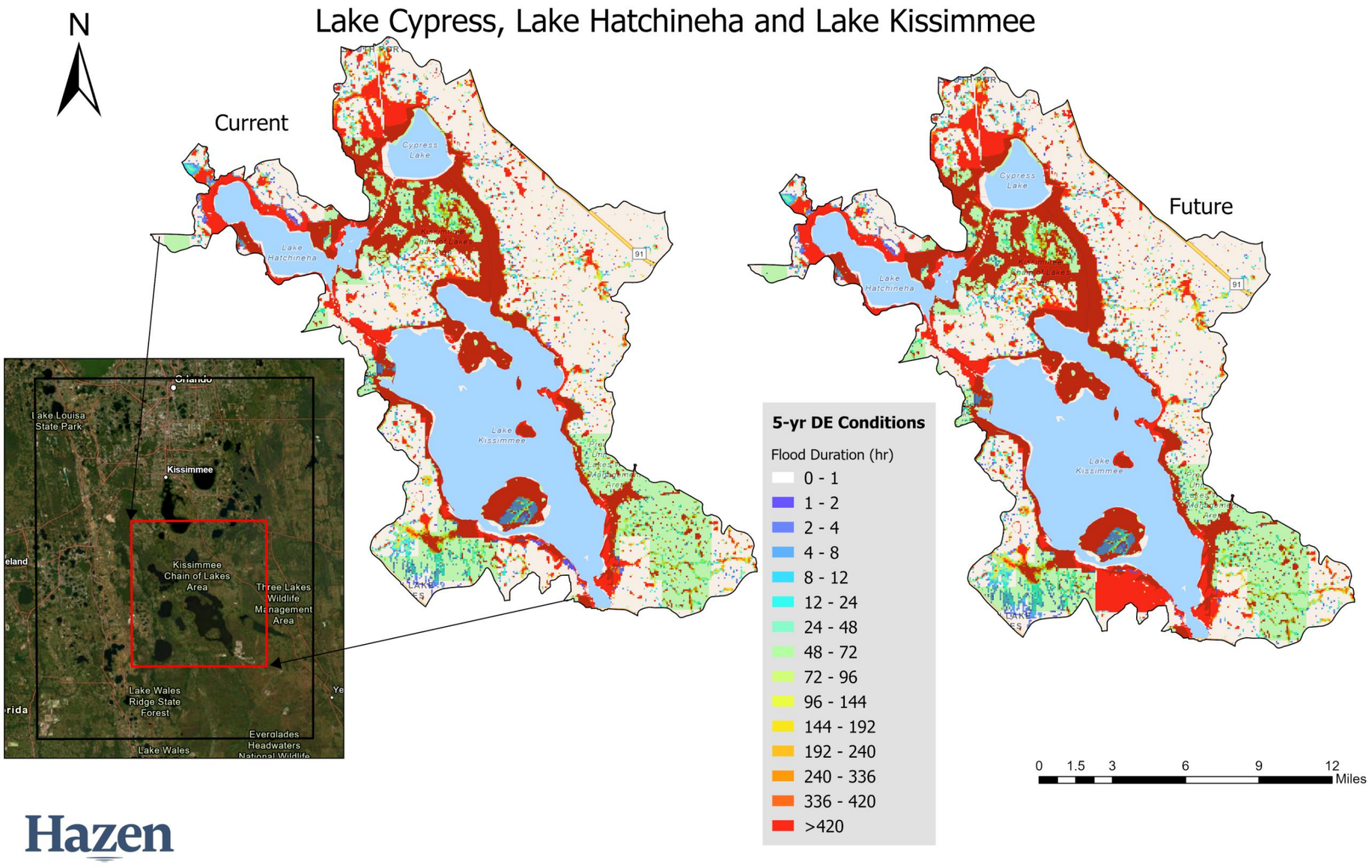

Figure 10-174: Flood Duration Maps for 5-yr 3-day Design Storm for KCH System under Current and Future Conditions

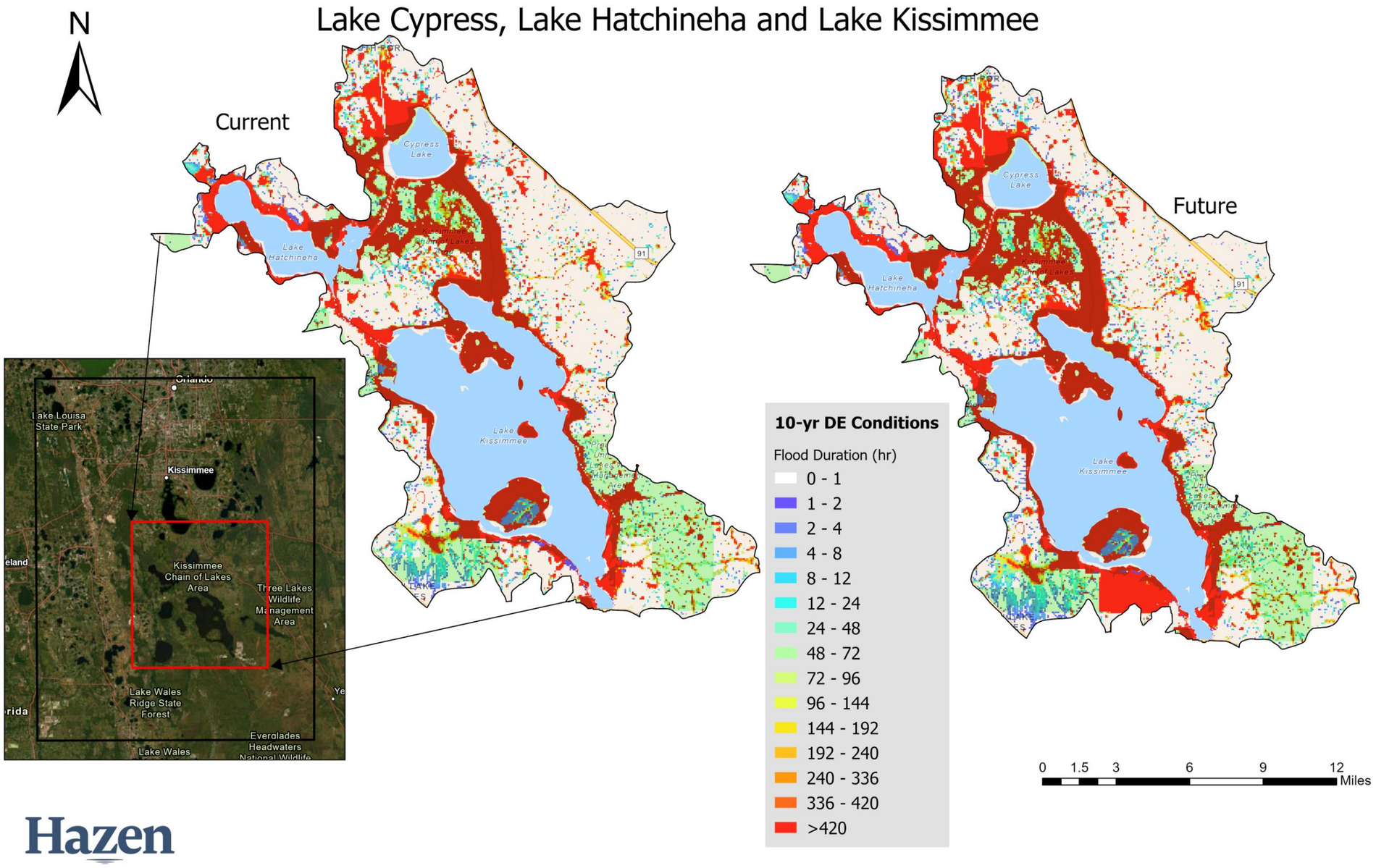

Figure 10-175: Flood Duration Maps for 10-yr 3-day Design Storm for KCH System under Current and Future Conditions

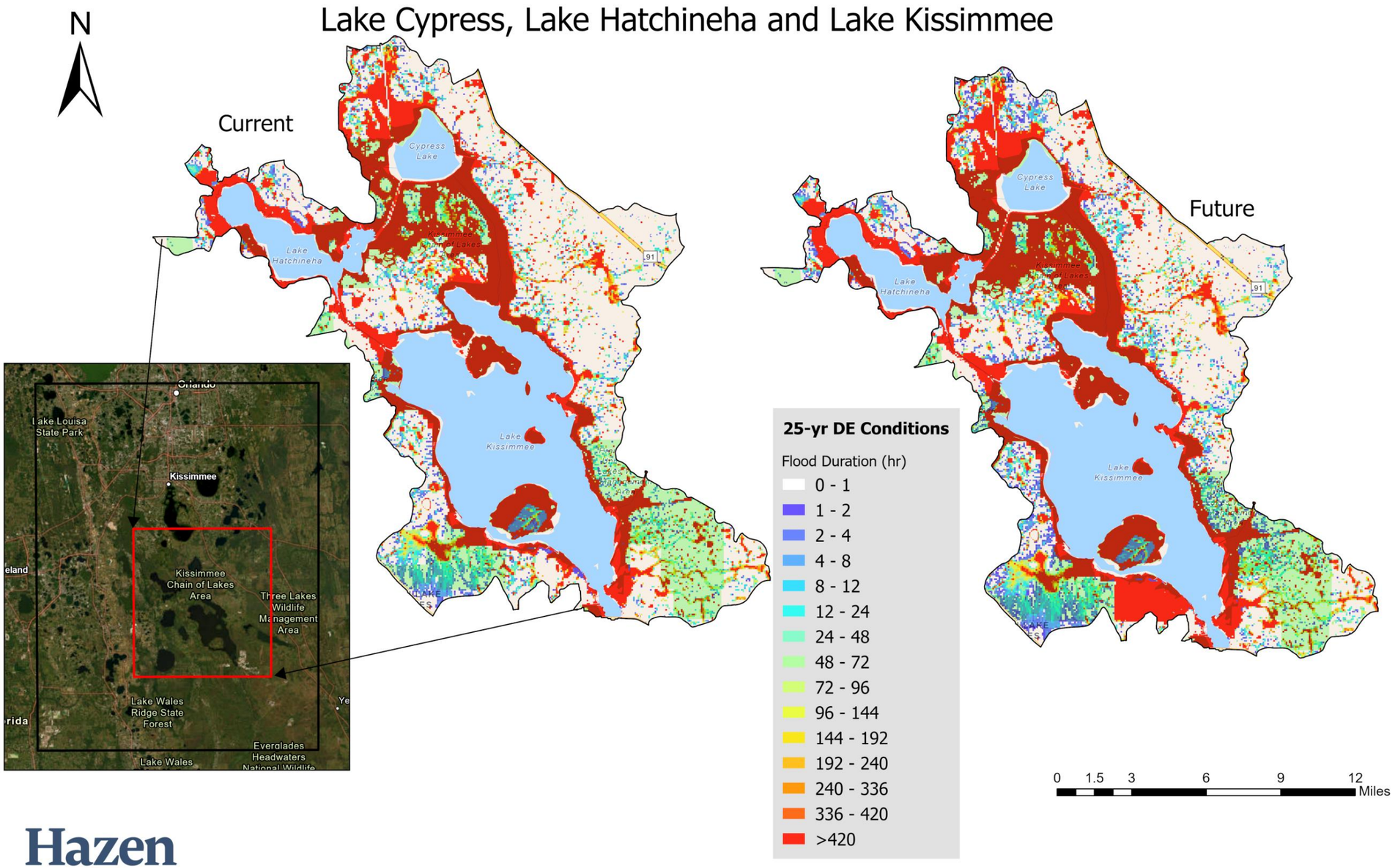

Figure 10-176: Flood Duration Maps for 25-yr 3-day Design Storm for KCH System under Current and Future Conditions

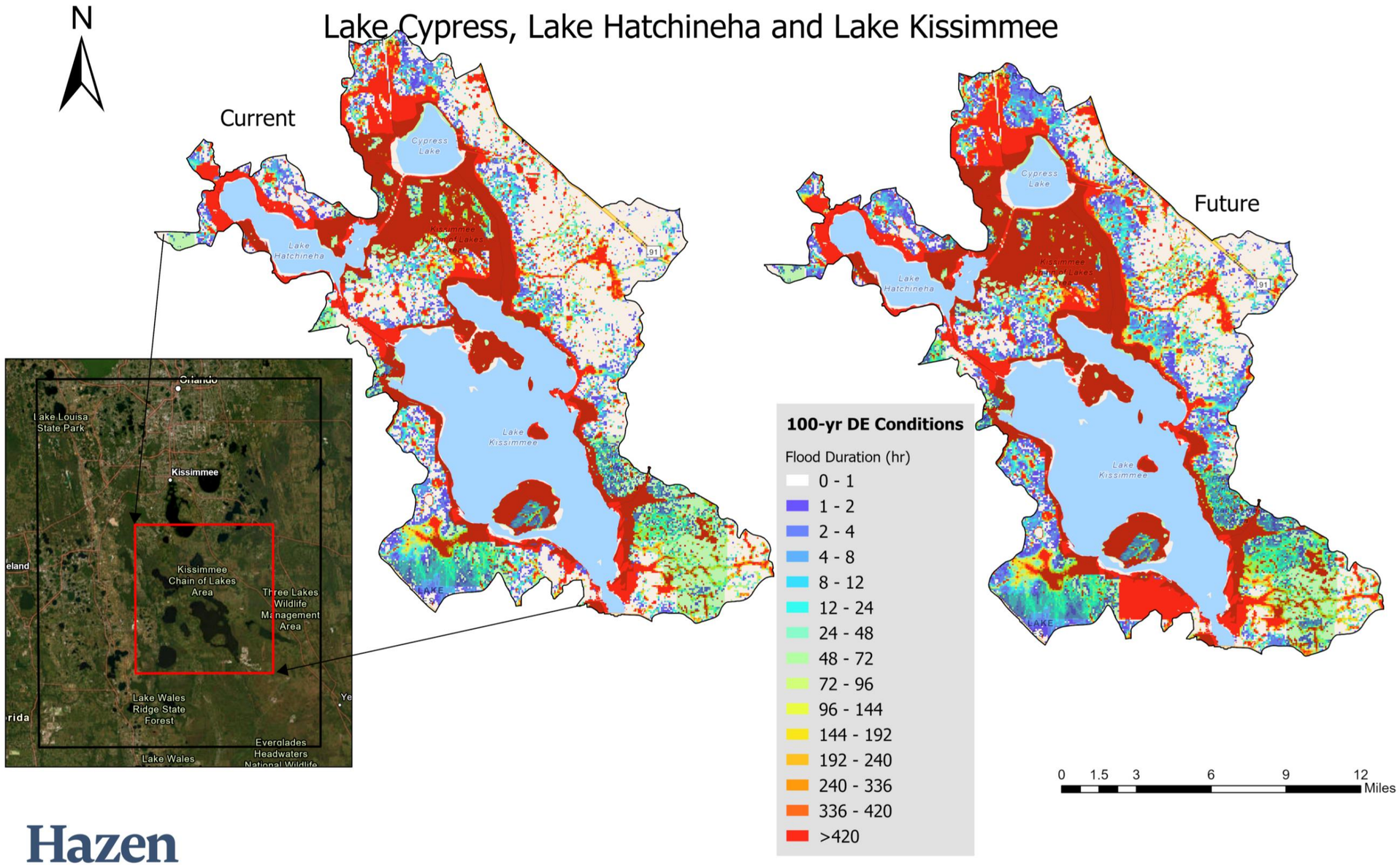

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Figure 10-177: Flood Duration Maps for 100-yr 3-day Design Storm for KCH System under Current and Future Conditions

10.9.7 FPLOS Rating of Lake Kissimmee LMA

Table 10-98 provides a summary of an evaluation of Lake Kissimmee LMA performance using FPLOS metrics to provide flood protection under current and future conditions. Lake Kissimmee is assigned an FPLOS rating of <25-yr for PM#1 under current conditions, and 25-yr under future conditions; this is due to the different tributaries of Lake Kissimmee providing LOS ranging from <5-yr to 25-yr. The future conditions model also uses the HRS which provides 1.5 ft of additional storage in the lake compared to the interim schedule used in the current conditions model. Assessment under all other PMs range in FPLOS rating from <5-yr to 10-yr. Due to all of these reasons, Lake Kissimmee LMA is assigned an overall rating of 5-yr under current and future conditions.

Table 10-98: Summary of PM Analysis in Lake Kissimmee LMA

Metric	Water Bodies Included in Analyses	Outcomes Based On Model Results	FPLOS Rating Under Current Conditions	FPLOS Rating Under Future Conditions
PM#1	Lake Kissimmee, C-38 canal, Jackson canal, Lake Marian, Lake Marion	For all design storms under current and future conditions, water levels stay within the banks in Lake Kissimmee for all design storms except 100-yr where water level is exceeding banks in multiple locations for less than 0.3 feet. Water levels are exceeded for portions of Canal C-38, Jackson Canal, Lake Marian and Lake Marion Creek in all design storms for both current and future conditions.	25-yr in Lake Kissimmee < 5-yr in Canal C-38 < 5-yr in Jackson Canal 5-yr in Lake Marian < 5-yr in Lake Marion Creek	25-yr in Lake Kissimmee < 5-yr in Canal C-38 < 5-yr in Jackson Canal 5-yr in Lake Marian < 5-yr in Lake Marion Creek
PM#2	Entire LMA	The simulated peak discharge per square mile during the 10-yr event was 3.8 CSM under current conditions and 4.2 CSM under future conditions, which are both less than SFWMD's permissible discharge allowance of 31.1 CSM based on a 10-yr design frequency.	10-yr discharge less than permissible discharge	10-yr discharge less than permissible discharge
PM#3	Entire LMA	During both peak tailwater and peak headwater, the simulated flow at structure S65 is higher than the firm capacity (3000 cfs) for all design storm events in the current conditions and future conditions.	< 5-yr	< 5-yr
PM#4-UKB	KCH System	S65 discharges needs to be restricted to 3000 cfs. Note: This is a limited analysis to set the stage for future work efforts. Under current conditions, discharges at S65 limited by downstream flooding had a bigger impact than under future conditions. This is due to larger storage capacity in Lake Kissimmee under future conditions (higher regulation schedule)	-	-
PM#5	Entire LMA	For the 5-yr design event, under both current and future conditions, less than 50% of the LMA is inundated more than 0.25-ft. Approximately 55%, 67%, and 80% of the LMA is inundated more than 0.25-ft for the 10-, 25-, and 100-yr design events, respectively, under both current and future conditions.	5-yr	5-yr
PM#6	Entire LMA	Approximately 62% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 5-yr storm and approximately 52% of the LMA stays flooded for longer than 6 hours for flooding depth higher than 0.25 ft for the 100-yr storm under both current and future conditions.	5-yr	5-yr

Lake Kissimmee LMA Overall FPLOS	**Current Conditions**	**5-yr**
	Future Conditions	**5-yr**

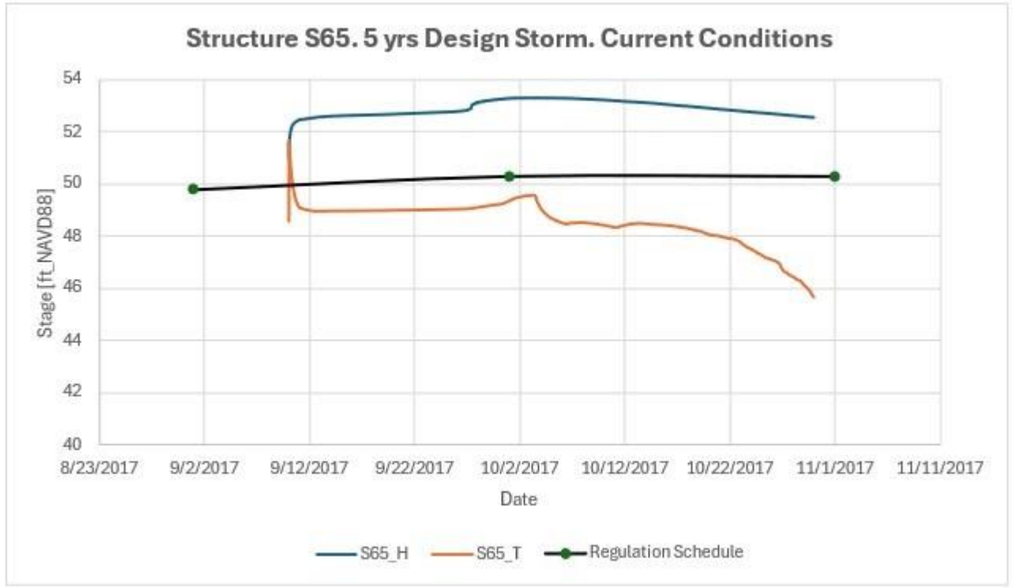
10.9.8 Volume in Lake due to Design Storm UKB-$V_{Kissimmee}$: Lake Kissimmee

Table 13-3 presents a summary of the water volume change in Lake Kissimmee, computed as the difference between volumes at the end of the storm event and the initial volume. There is higher volume in the lake under future conditions compared than under current conditions for each set of design storms, due to the higher rainfall under future conditions, and the higher lake regulation schedule. **Table 13-4** shows the volume of water at peak.

10.9.9 Duration of Lake Recovery Relative to Regulation Schedule UKB-T_{Reg}. Lake Kissimmee

This sub- section examines the water level and regulation schedule in Lake Kissimmee by reviewing its main regulation structure, S65. **Figure 10-178 and Figure 10-179** display the headwater and tailwater stages at S65, as well as the regulation schedule for the whole simulation period, for current and future conditions respectively. Generally, the water levels at this lake remain above regulation schedule, in flood control zone for almost the entire simulation period. **Figure 10-180** shows the current and future discharges through S65, the outlet of the UKB.

It should be noted that the regulation schedule shown in current and future are the interim regulation schedule, for comparison purposes. But the future conditions model uses headwater revitalization schedule.



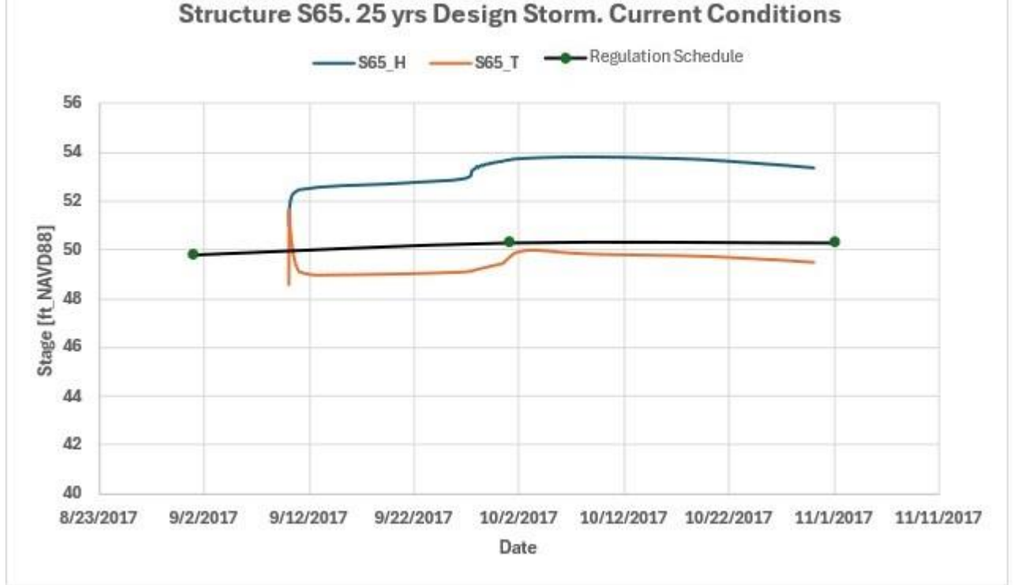
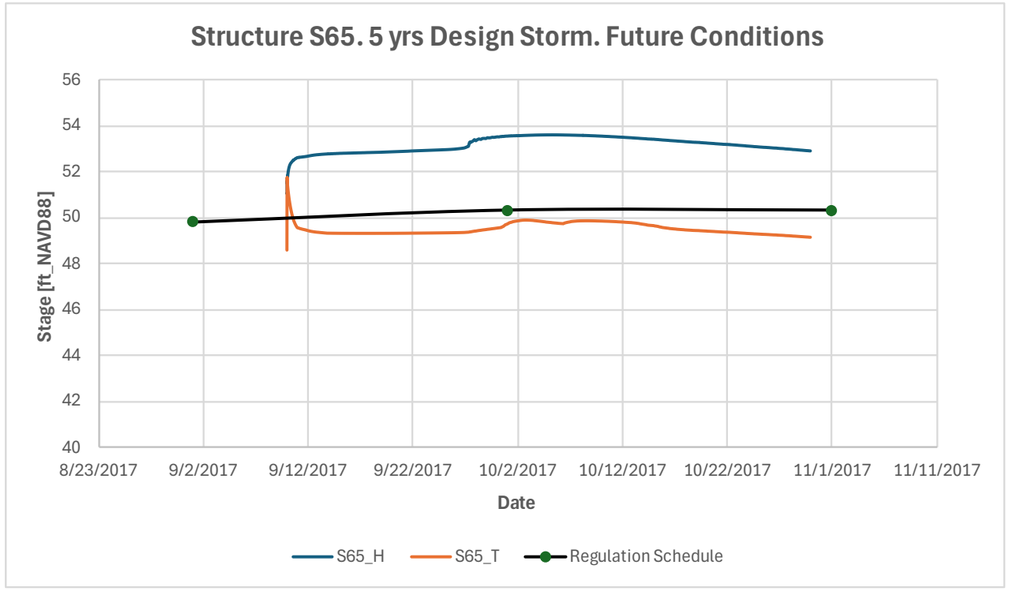
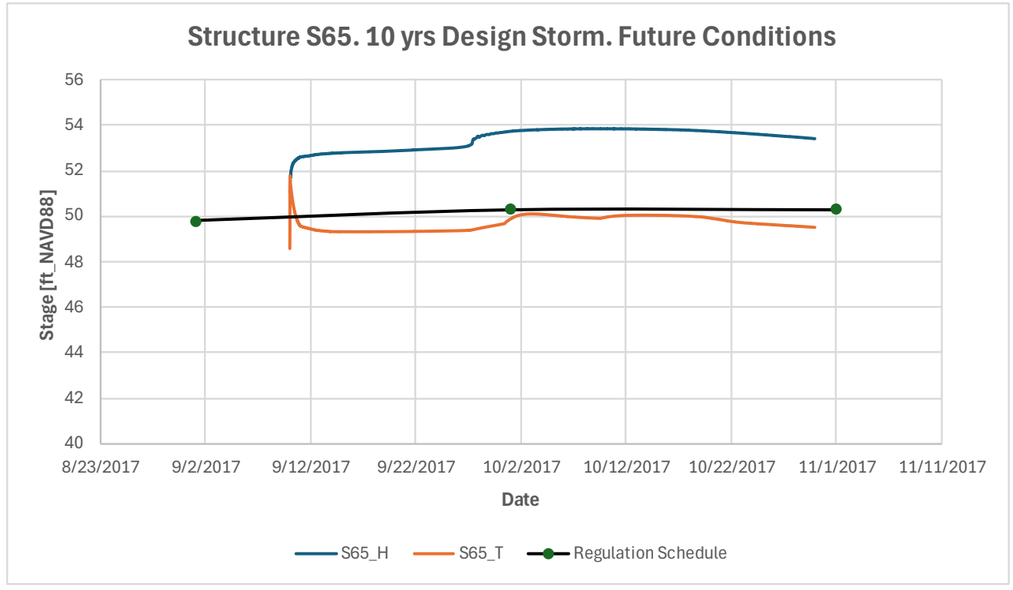
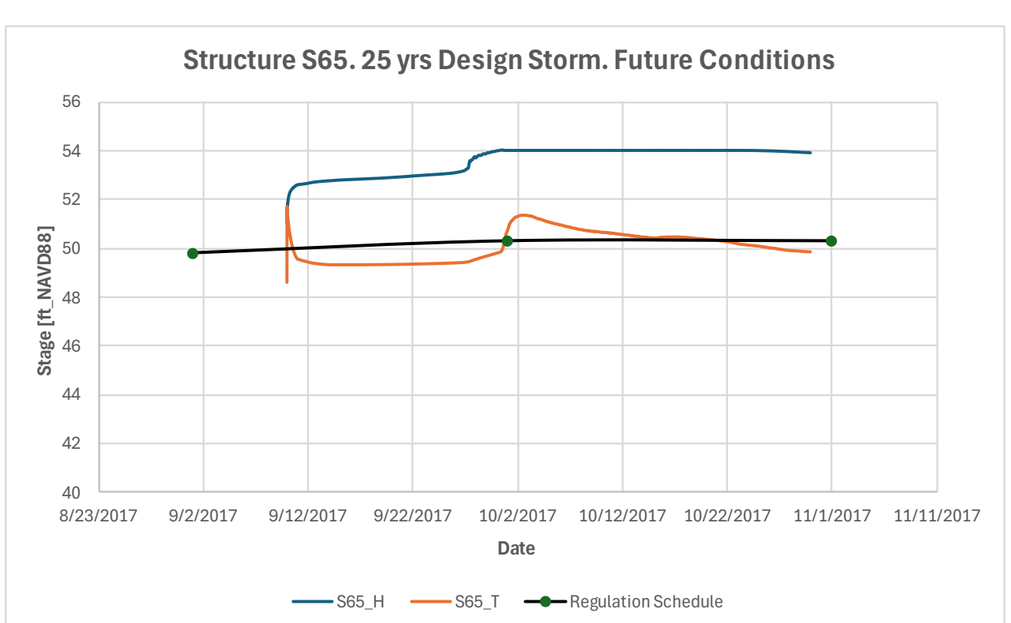


Figure 10-178: Headwater and Tailwater at Structure S65 vs. Lake Kissimmee Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Current Conditions







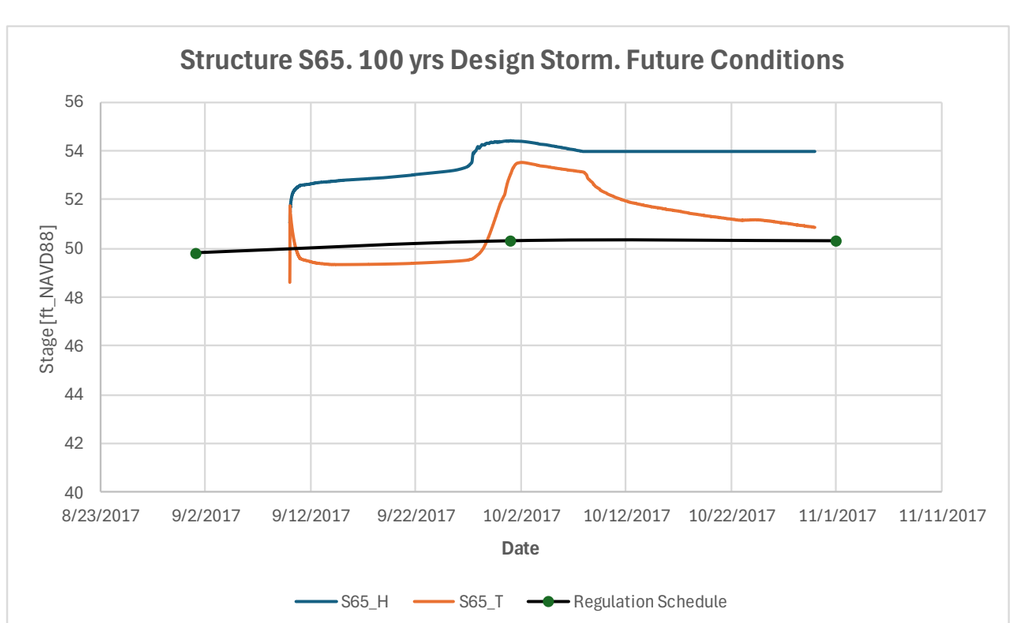


Figure 10-179: Headwater and Tailwater at Structure S65 vs. Lake Kissimmee Regulation Schedule. 5-, 10-, 25-, and 100- Years Design Storm. Future Conditions

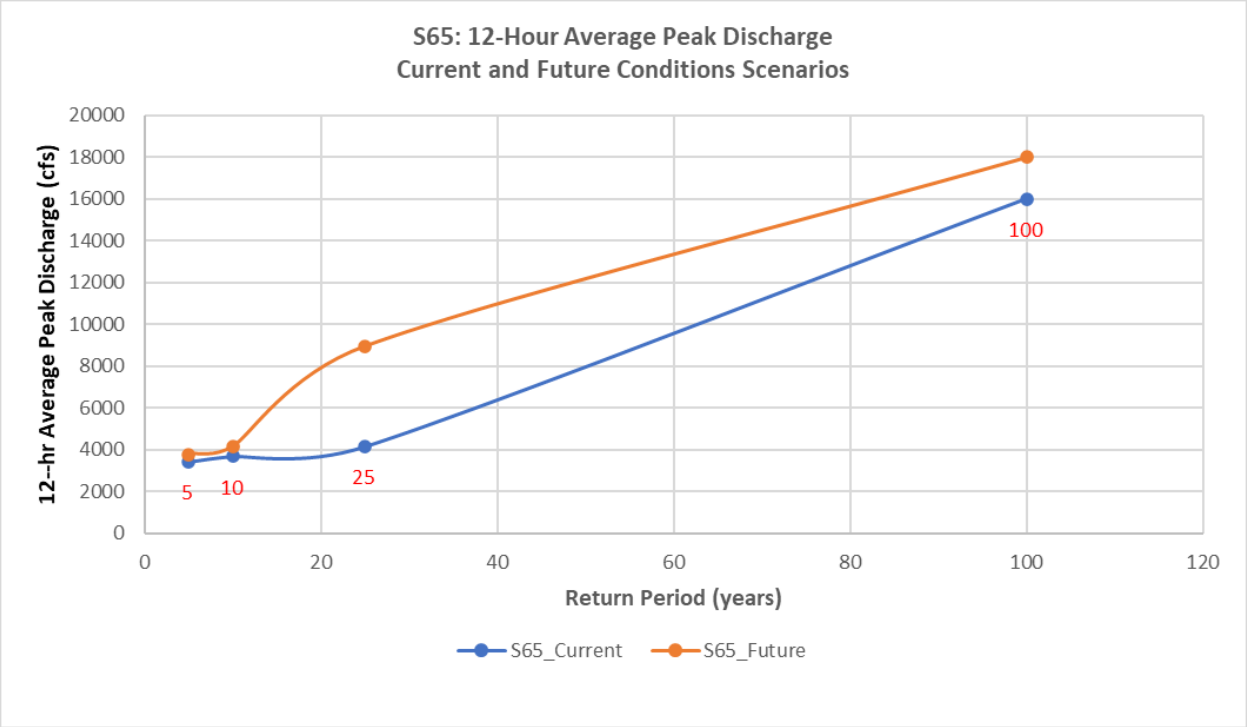


Figure 10-180: S65 12-Hour Average Peak Discharge Current and Future Conditions Scenarios

11. Water Budget for the Design Events under Current and Future Conditions

Water budgets for each watershed in the study area were analyzed to understand how different water budget components affect the watershed hydrology for various storm events. The analysis also checked the reasonableness of the relative magnitude of the water balance.

Appendix L shows the summary table of the water budget, with columns for each UKB watershed, and rows for each water budget component. Water budget calculations were done for each design storm, under for current and future scenarios.

Groundwater is a major contributor in all watersheds, as shown by calibration and validation, and design event water budgets. Overland flows to the lakes and canals is also a big hydrologic factor, accounting for 20% to 50% of rainfall, which is reasonable for the Kissimmee Basin, dominated by the hydrology of the lakes. The exchange and movement of water between watersheds is significant in the saturated zone, up to 15% of the precipitation.

The KB, like many areas in SFWMD jurisdiction, is characterized by flat relief with numerous lakes, wetlands, and rivers. In terms of precipitation, the Kissimmee Basin receives an average annual rainfall of around 50 inches, with most of this precipitation occurring during the wet season from June to September. This rainfall recharges the lakes, rivers, and wetlands within the basin, contributing to the overall water budget. Evapotranspiration is another important factor in the water budget of the UKB, with the high rates of evaporation from surface water bodies and wetlands. Runoff plays a significant role in the water budget of the Kissimmee Basin, as excess rainfall that is not absorbed by the soil or vegetation flows into the numerous rivers, streams, and lakes in the area. In terms of saturated zone processes, groundwater recharge is also an important component of the water budget in the Kissimmee Basin, with water seeping into the aquifers through the unsaturated zone.

Overall, the hydrologic water budget in the design events show the values of all these terms are within the literature values and previous modeling studies in the area.

12. Vulnerability Analysis

12.1 Alligator Lake LMA

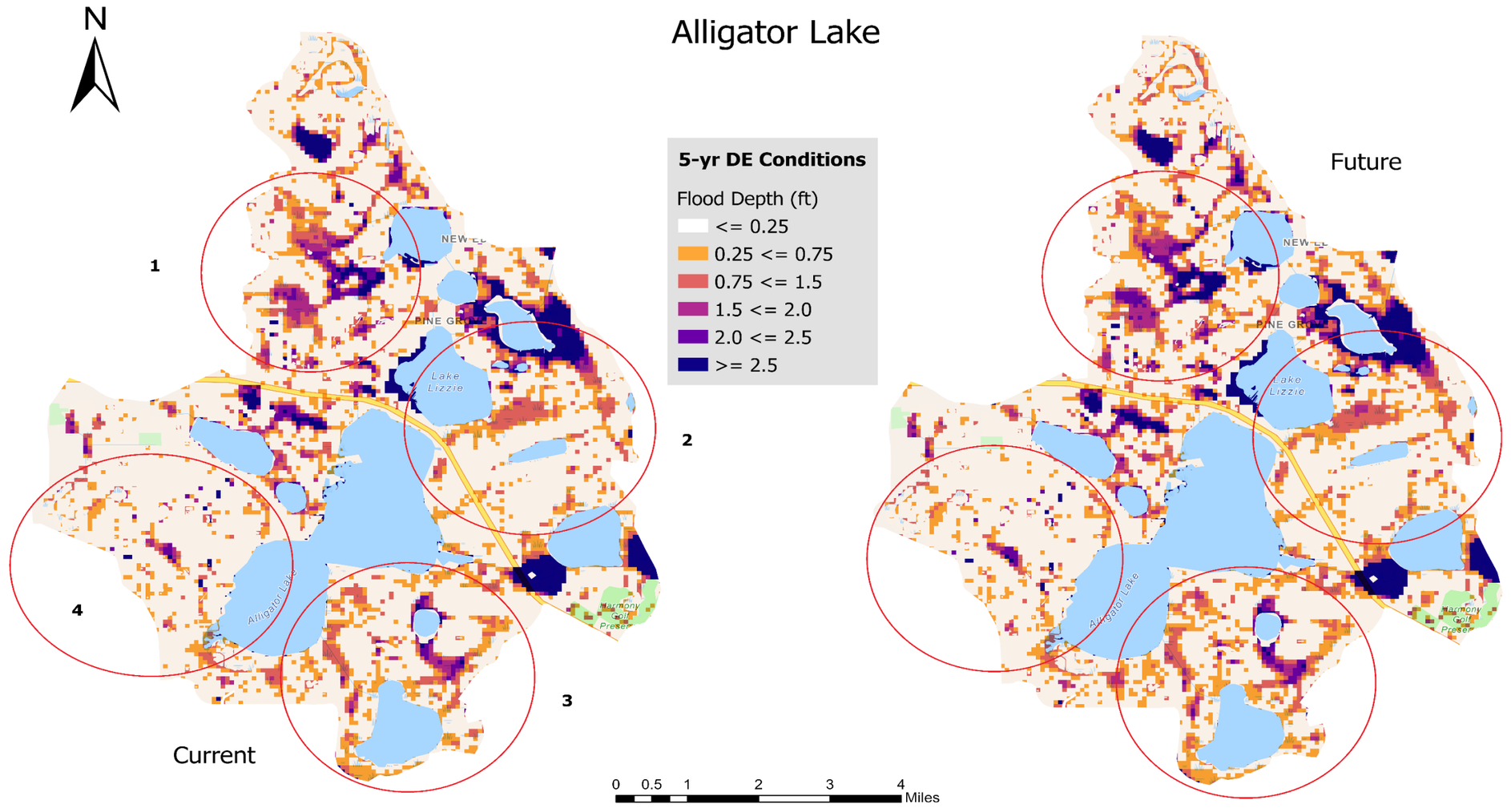


Figure 12-1: Alligator Lake LMA in Sections

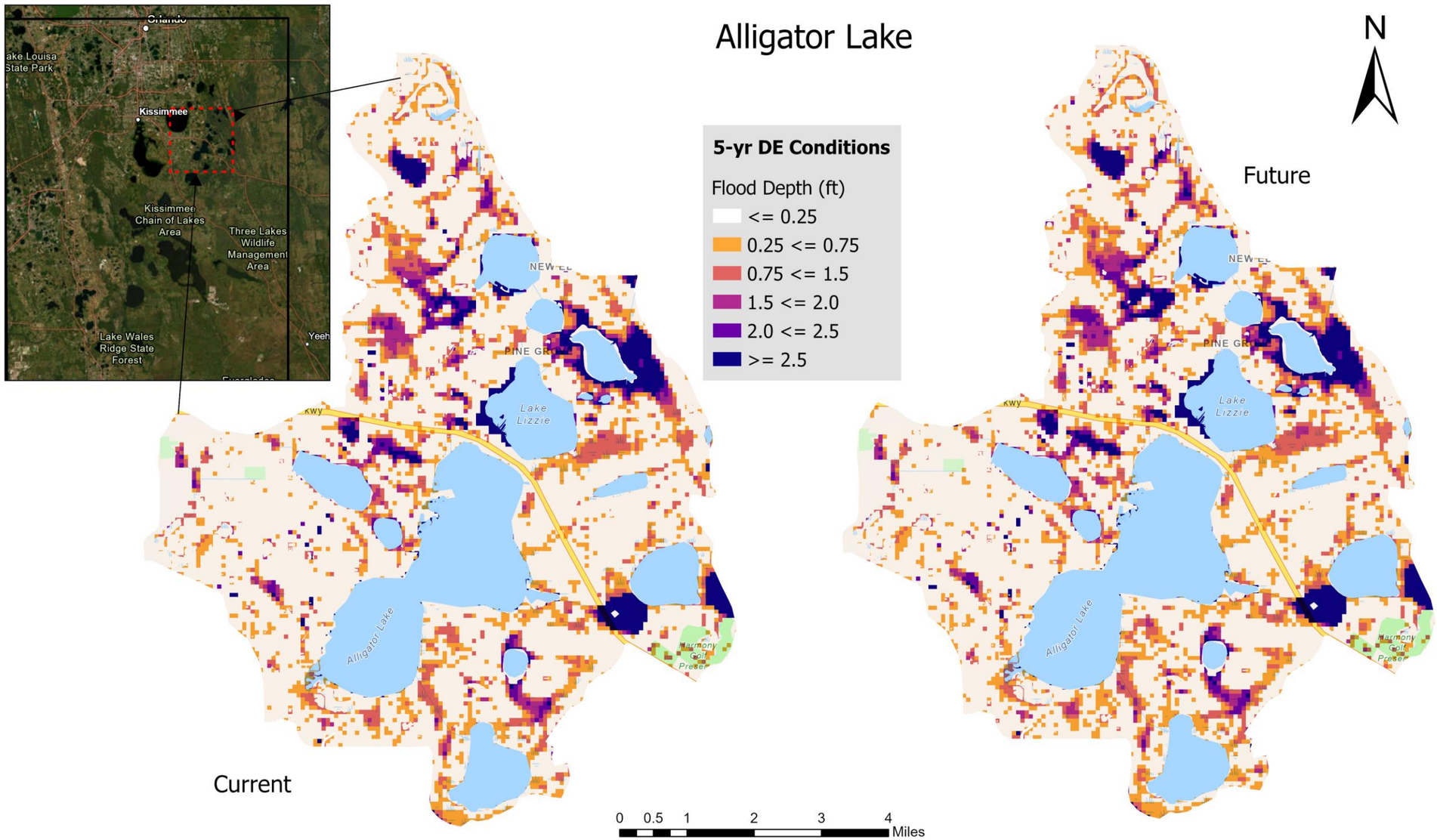


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Figure 12-2: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

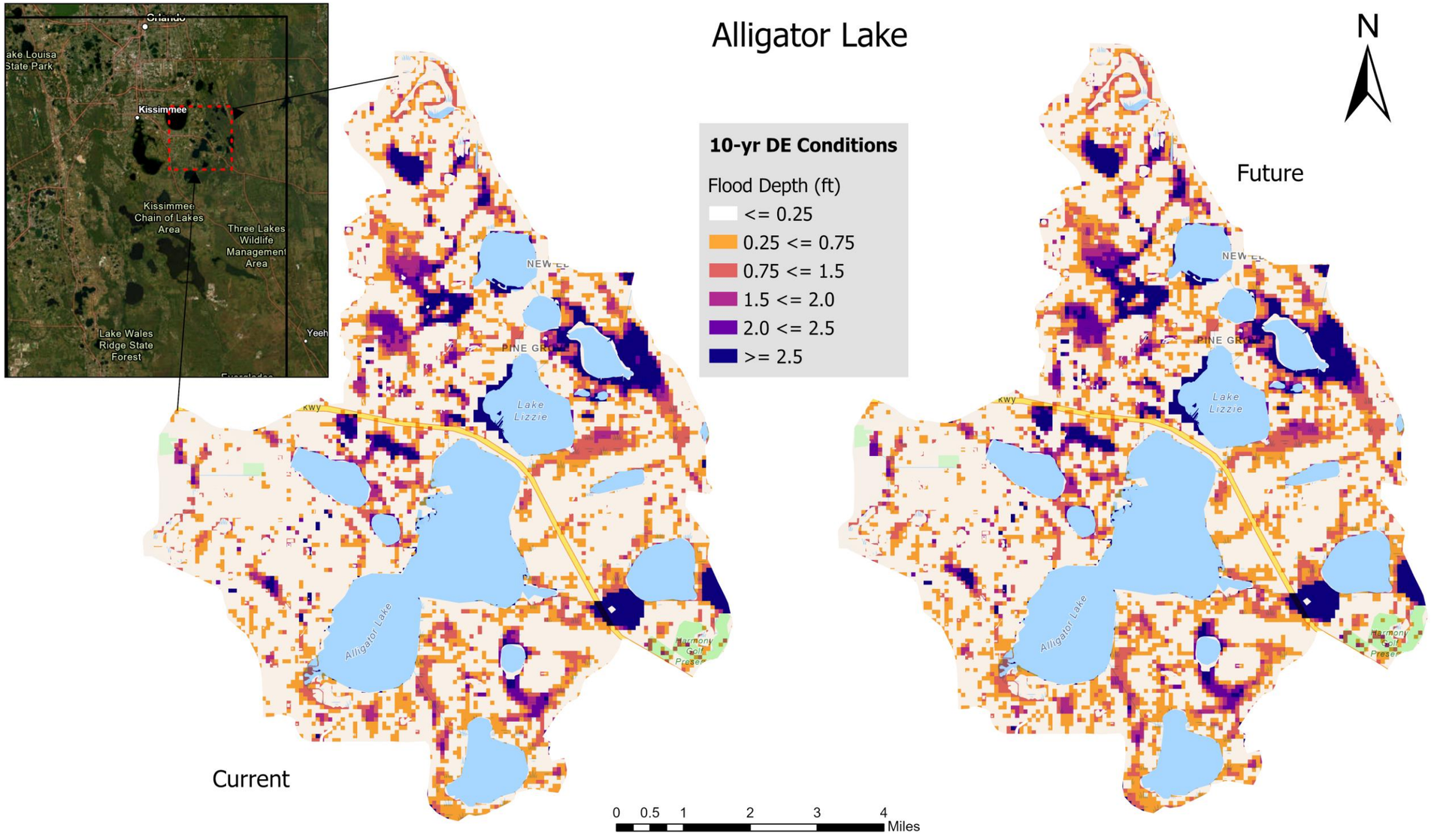

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Figure 12-3: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

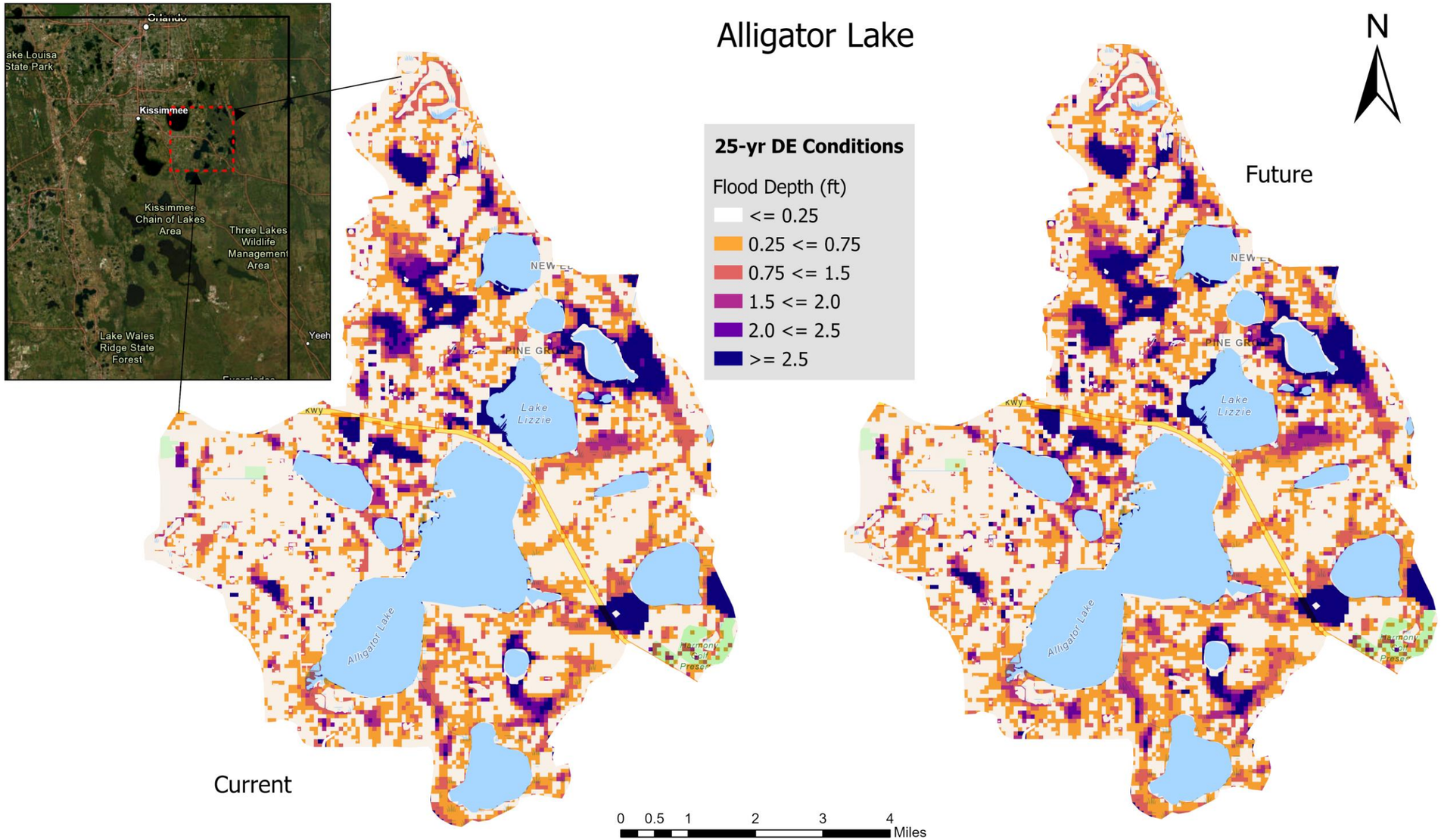

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Figure 12-4: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

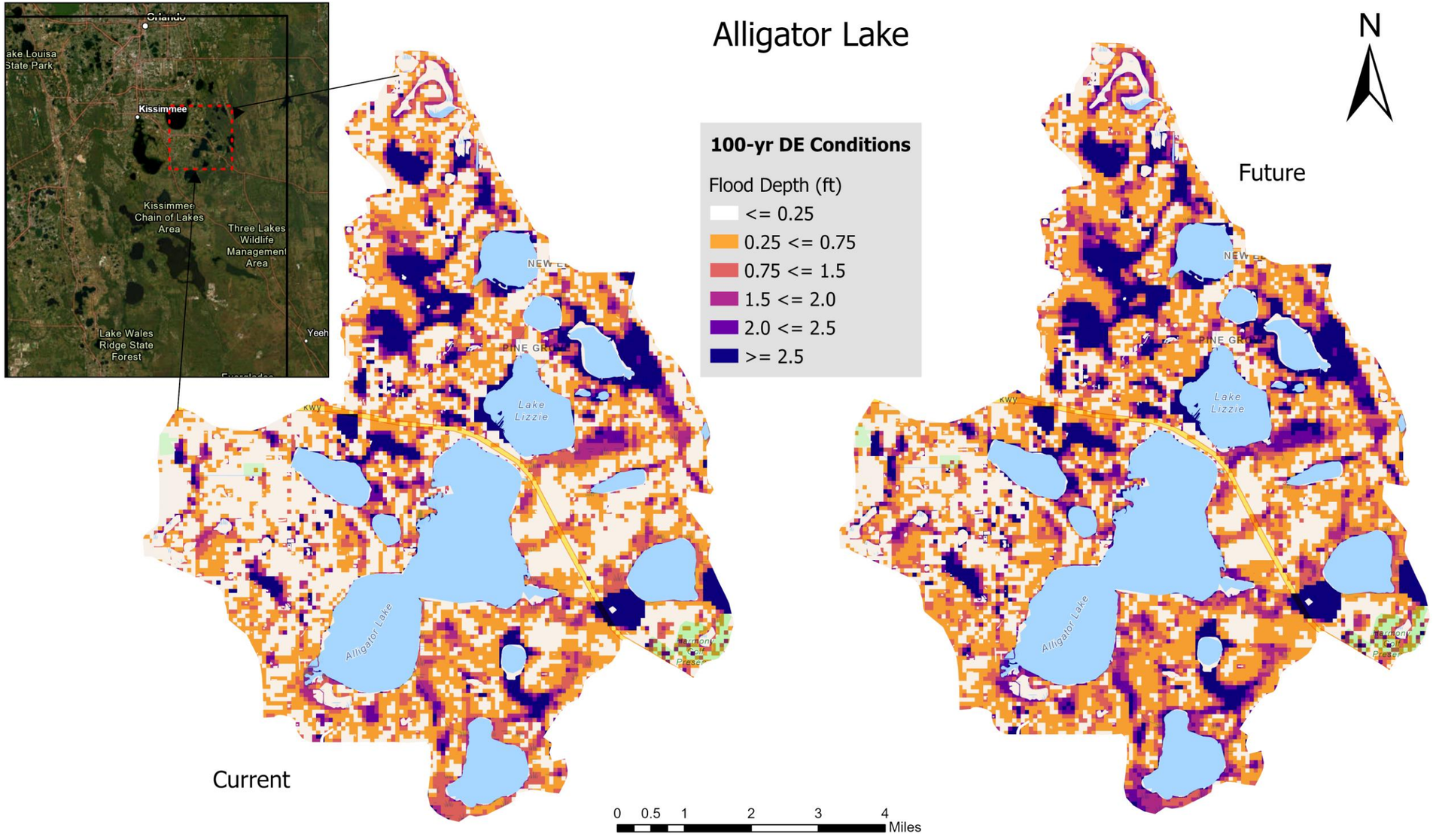

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Figure 12-5: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Alligator Lake LMA

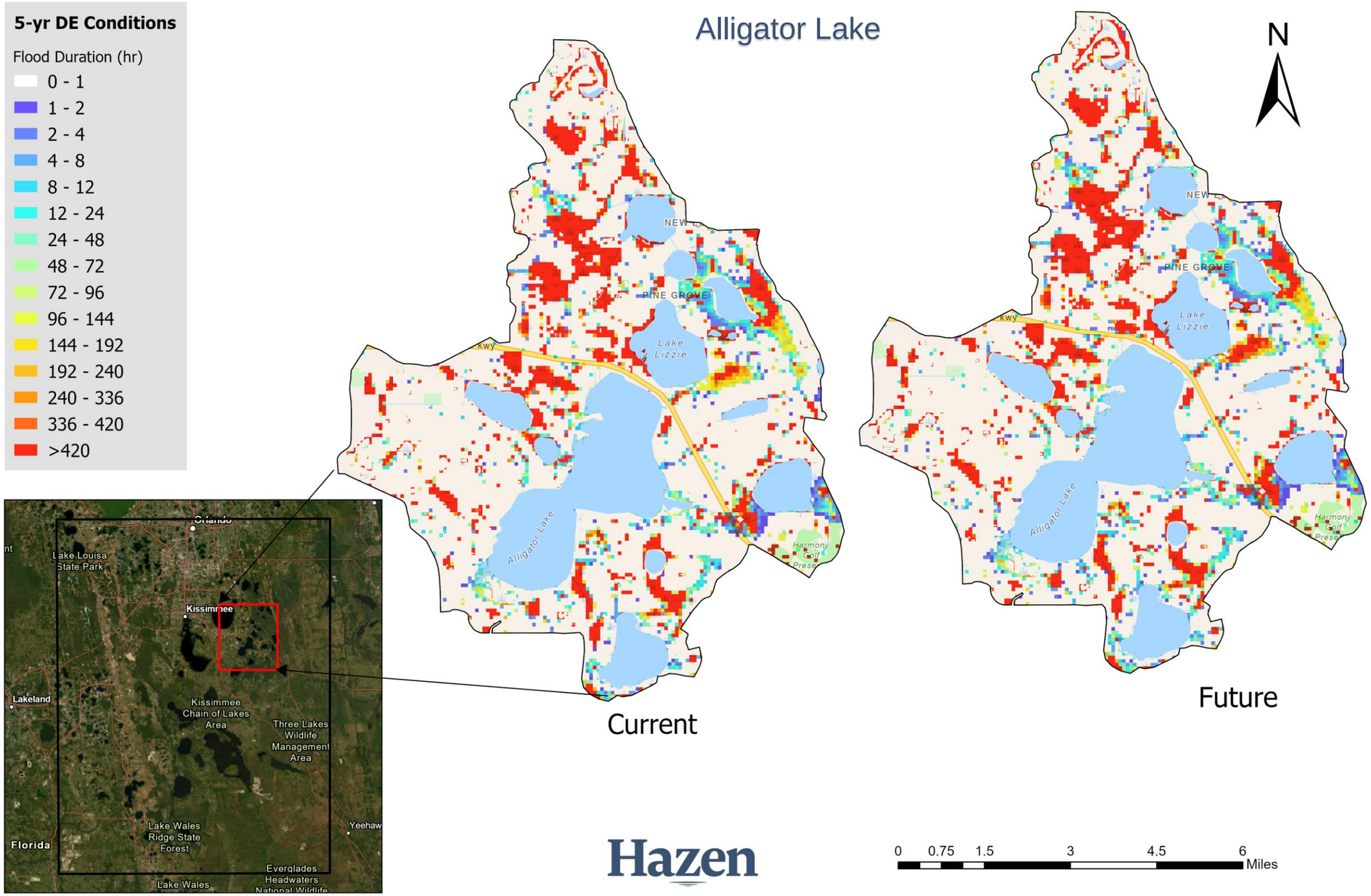

Figure 12-6: Flood Duration Maps for 5-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

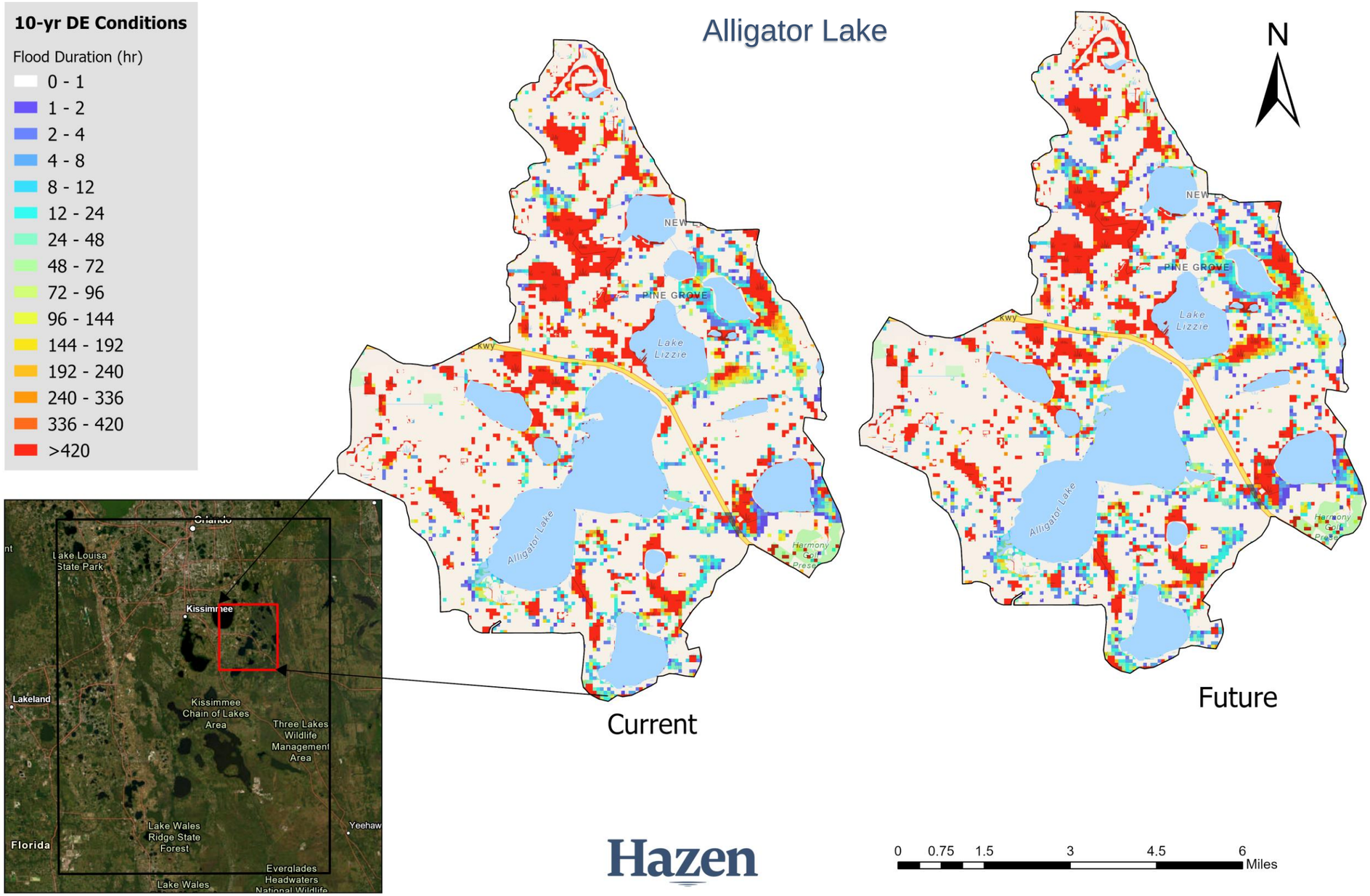

Figure 12-7: Flood Duration Maps for 10-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

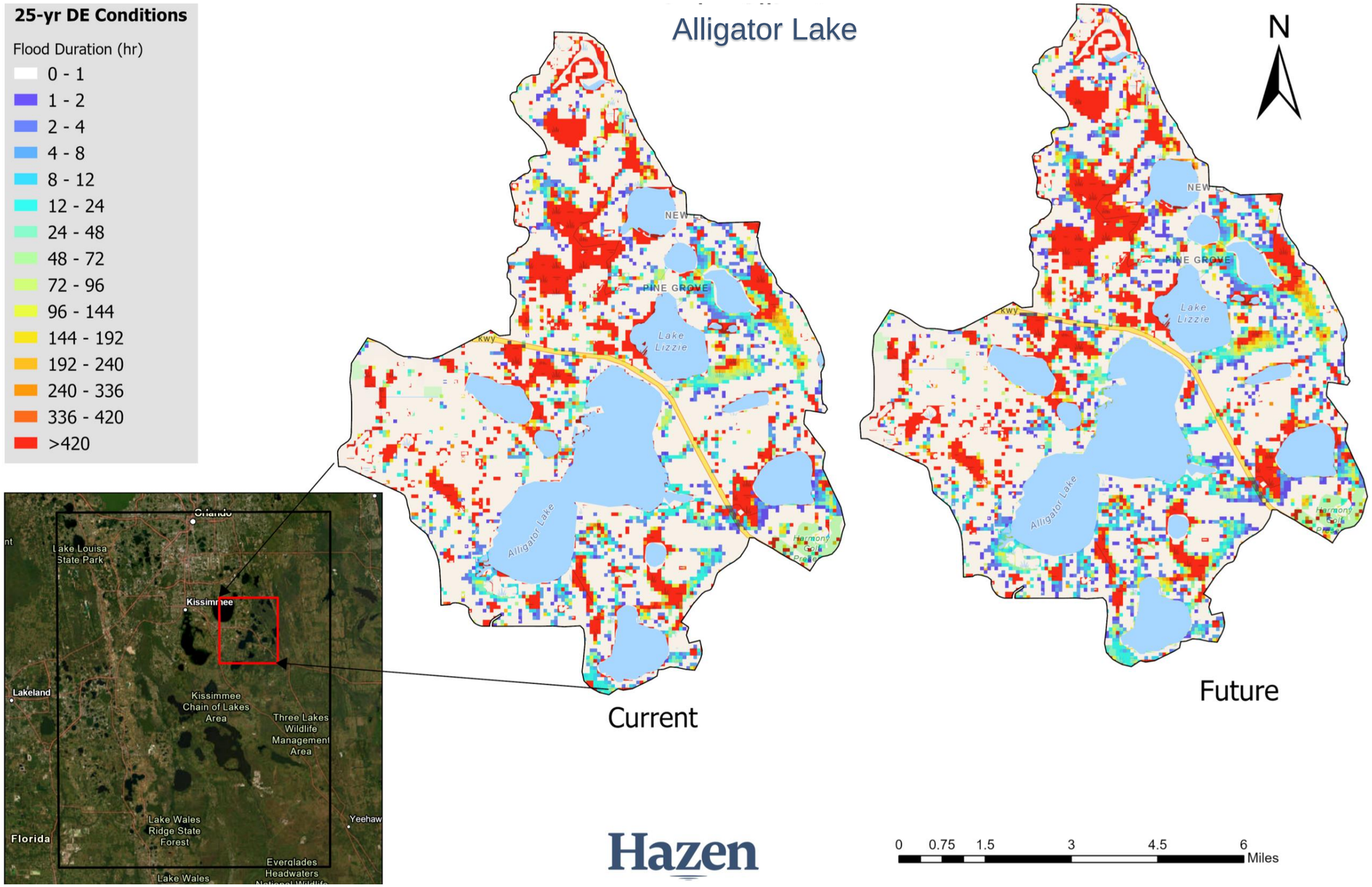

Figure 12-8: Flood Duration Maps for 25-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

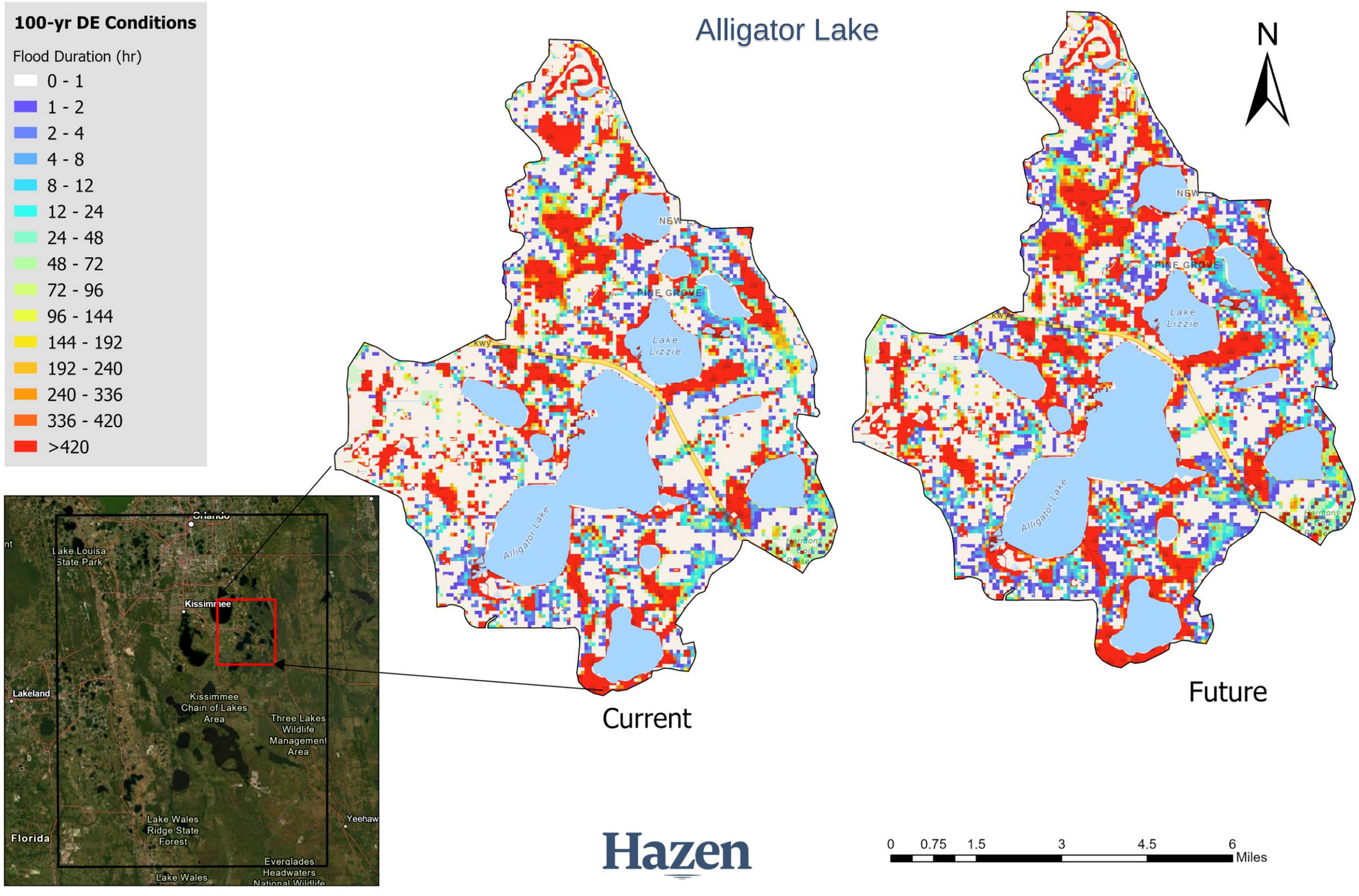

Figure 12-9: Flood Duration Maps for 100-yr 3-day Design Storm for Alligator Lake LMA under Current and Future Conditions

Figure 12-10: Aerial Image of Section 1 for Alligator Lake LMA

Section 1 from **Figure 12-1** highlights the region northwest of Alligator Lake, west of Lake Center, and north of E. Irlo Bronson Memorial Highway. During the 5-year event as seen in **Figure 12-2**, an average flooding depth of 2.3 feet was concentrated in the central areas, which are typically characterized as wetlands. The extent of flooding increased when comparing current to future conditions. As we move from the 10-year to the 100-year event, **Figure 12-3 to Figure 12-5**, flooding expanded across the entire area, with a notable rise in severity between current and future scenarios.

The PM6 analysis for the Alligator LMA as seen in **Figure 12-6 to Figure 12-9** reveals a strong correlation between flood depth and flood duration. In the central region of Section 1, prolonged flooding was observed, with water taking over 420 hours to drain. This trend was consistent across both current and future conditions for all design events. Conversely, the outer areas of Section 1 experienced significantly faster drainage, with water receding in under 2 hours under current conditions and within 4 hours under future conditions. This analysis confirms that the central portion of Section 1 is highly vulnerable across all design events.

Figure 12-11: Aerial Image of Section 2 for Alligator Lake LMA

Section 2 focuses on the area east of E. Irlo Bronson Memorial Highway, between Lakes Buck, Lizzie, Trout, and Colin. Significant flooding depths were observed around the banks of Trout Lake and along the southern shore of Lake Lizzie. However, there was a minimal difference in flooding severity between current and future conditions for these locations during smaller storm events.

As the storm intensity increased from the 5-year to the 100-year design event, flooding expanded noticeably, with the 100-year event showing the most severe flooding. For the 25-year and 100-year events, there was a definite difference between current and future conditions.

In the PM6 analysis (Figure 12-6 to Figure 12-9), an intriguing contradiction emerged around the banks of the lakes, where higher flood depths were observed. Despite these depths, most banks drained under smaller design events within 2 to 4 hours. However, as the design events intensified, areas like the banks of Lake Lizzie exhibited significantly longer drainage times. Notably, for all design events, the future conditions consistently showed extended flood durations compared to the current conditions, highlighting the increasing vulnerability of these areas under changing hydrological scenarios.

Figure 12-12: Aerial Image of Section 3 for Alligator Lake LMA

Section 3 examines the region between Brick Lake and Alligator Lake, where flooding patterns vary by storm intensity and future projections. Flooding occurred with similar intensity around the banks of Pearl Lake under current and future conditions during the 5-year and 10-year design events.

However, as the design storm escalated from the 25-year to the 100-year event, flooding became more pronounced around the banks of Alligator Lake and Brick Lake. There was a significant increase in flooding severity for these higher-intensity events between current and future conditions.

The PM6 (Figure 12-6 to Figure 12-9) analysis for Section 3 mirrors the trends observed in Section 2, where most lake banks with higher flood depths drained relatively quickly, within 2 to 4 hours. However, notable exceptions emerged. The banks of Pearl Lake consistently experienced prolonged flood durations across all design events, emphasizing their persistent vulnerability. In contrast, the banks of Brick Lake exhibited faster drainage for the 5-, 10-, and 25-year events, but as the intensity increased to the 100-year event, the flood duration exceeded 420 hours. This analysis highlights Pearl Lake's consistent susceptibility to prolonged flooding, while Brick Lake's vulnerability becomes critical under the 100-year event.

Figure 12-13: Aerial Image of Section 4 for Alligator Lake LMA

For this section, the level of flooding between current and future conditions was only noticeable in the 100-year event where there was a difference in flooding between these two conditions. The 5, 10, and 25-year events had flooding but with relatively lower depths except for the small area between Rambler Avenue and Hickory Tree Road, which saw a higher depth of flood.

The PM6 (Figure 12-6 to Figure 12-9) analysis reveals a persistent pattern of prolonged flood durations within the small area between Rambler Avenue and Hickory Tree Road, consistent across all design events. Flood durations for other areas within this section were relatively shorter during smaller design events. However, as the design events intensified, the flood durations increased significantly, spreading uniformly across the section. In all scenarios, future conditions consistently exhibited slightly higher flood durations compared to current conditions, emphasizing the growing impact of future hydrologic changes on this area's flood resilience.

12.2 Lake Myrtle LMA

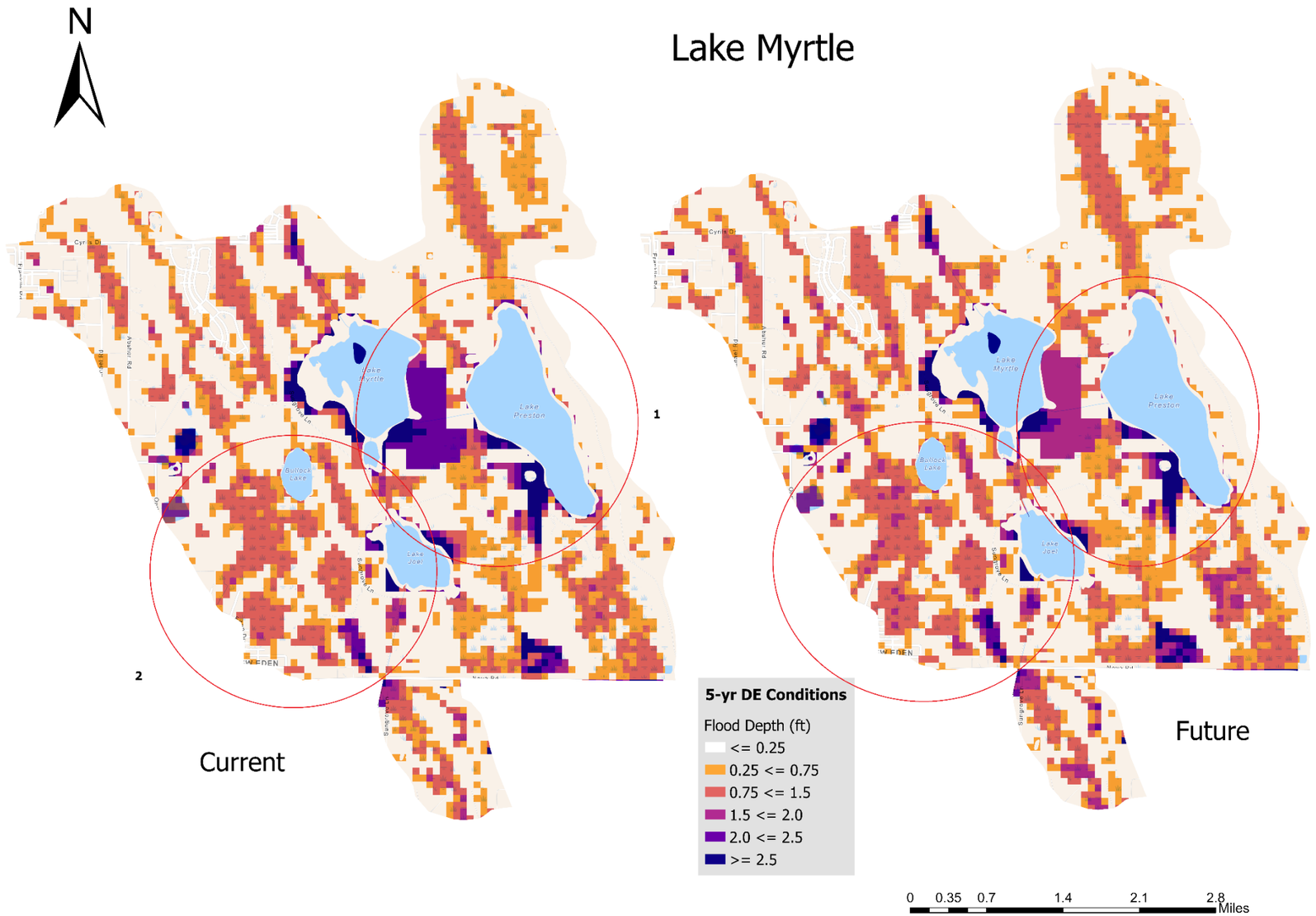

Figure 12-14: Lake Myrtle LMA in Sections

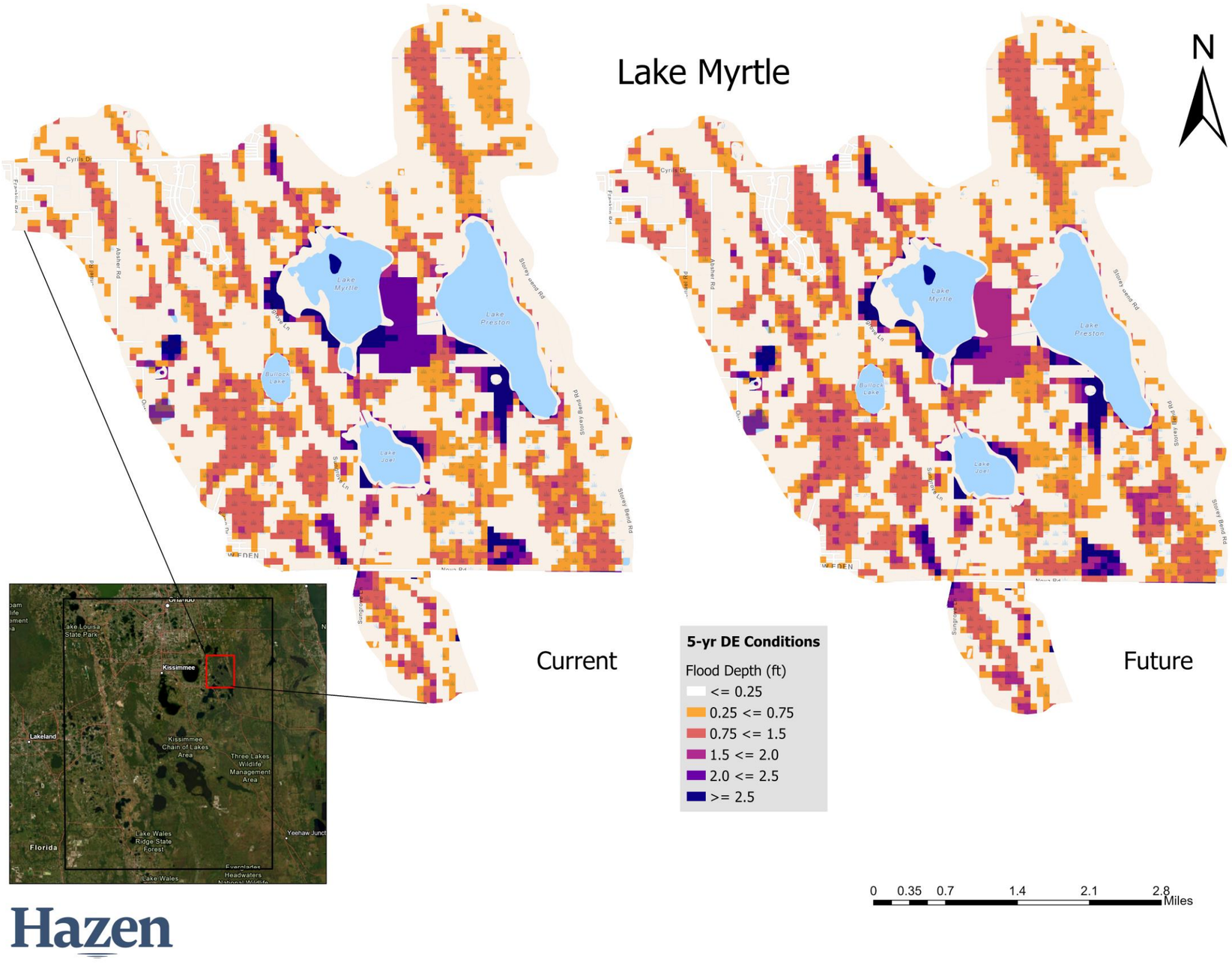

Figure 12-15: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

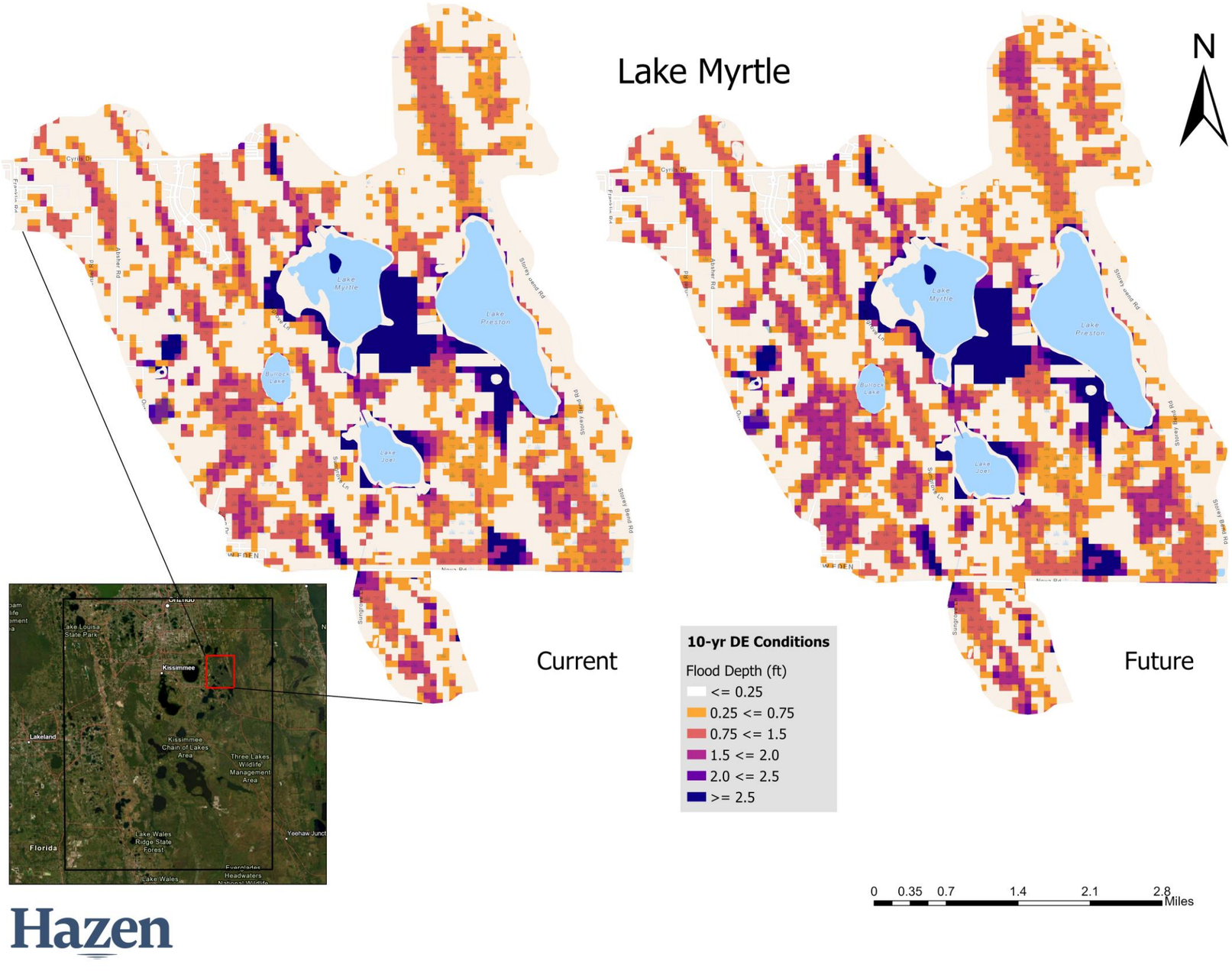

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Figure 12-16: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

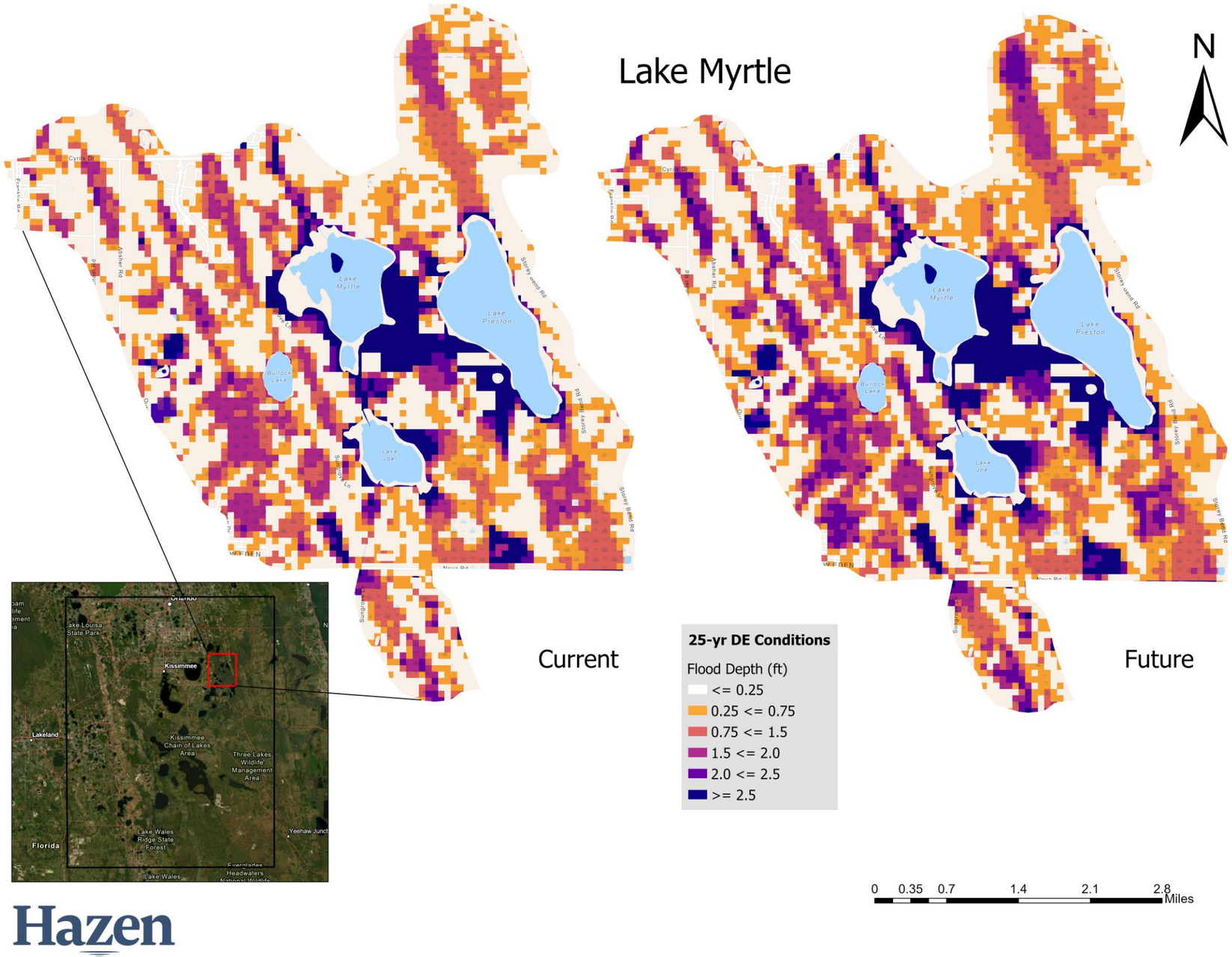

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Figure 12-17: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

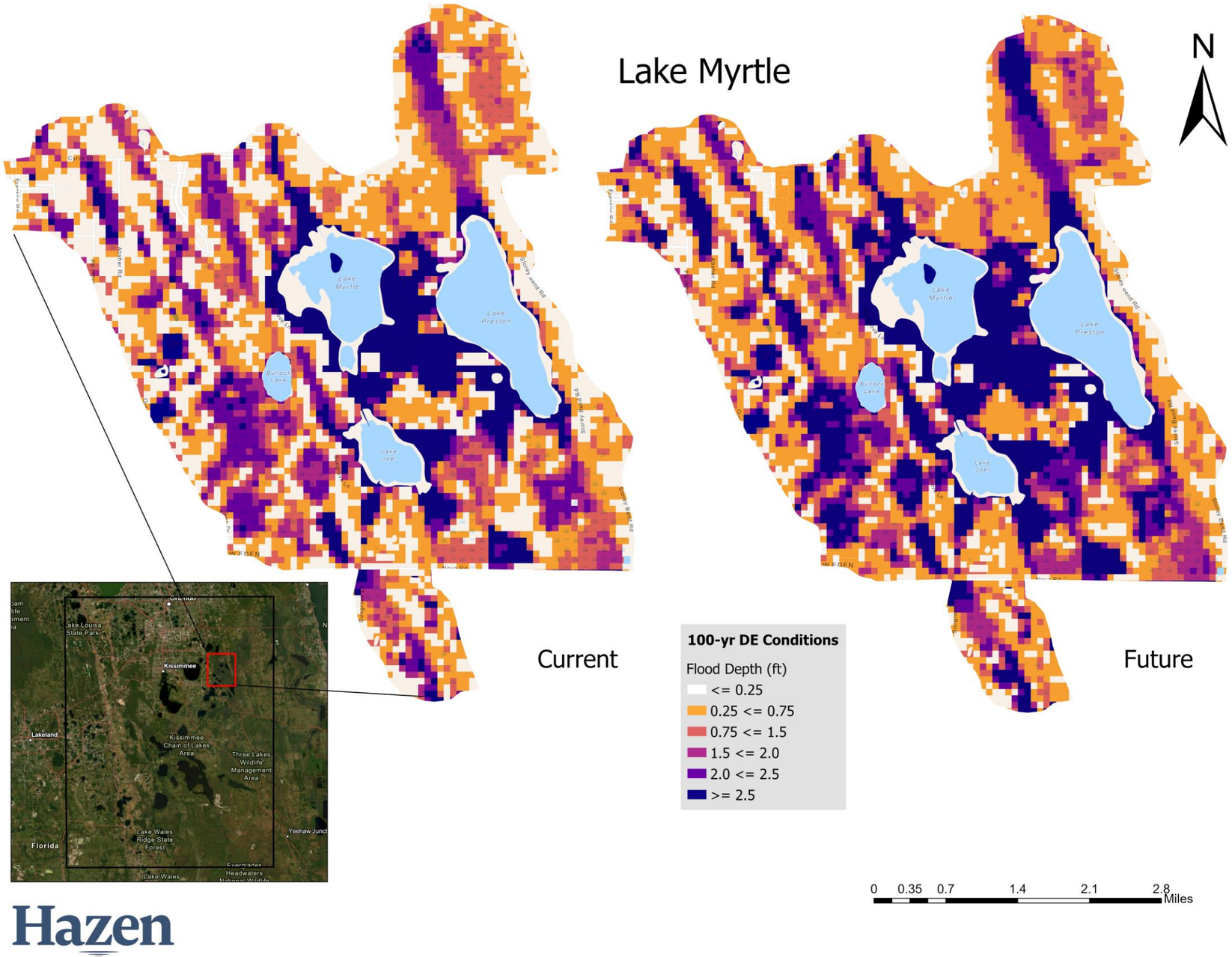

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Figure 12-18: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Myrtle LMA

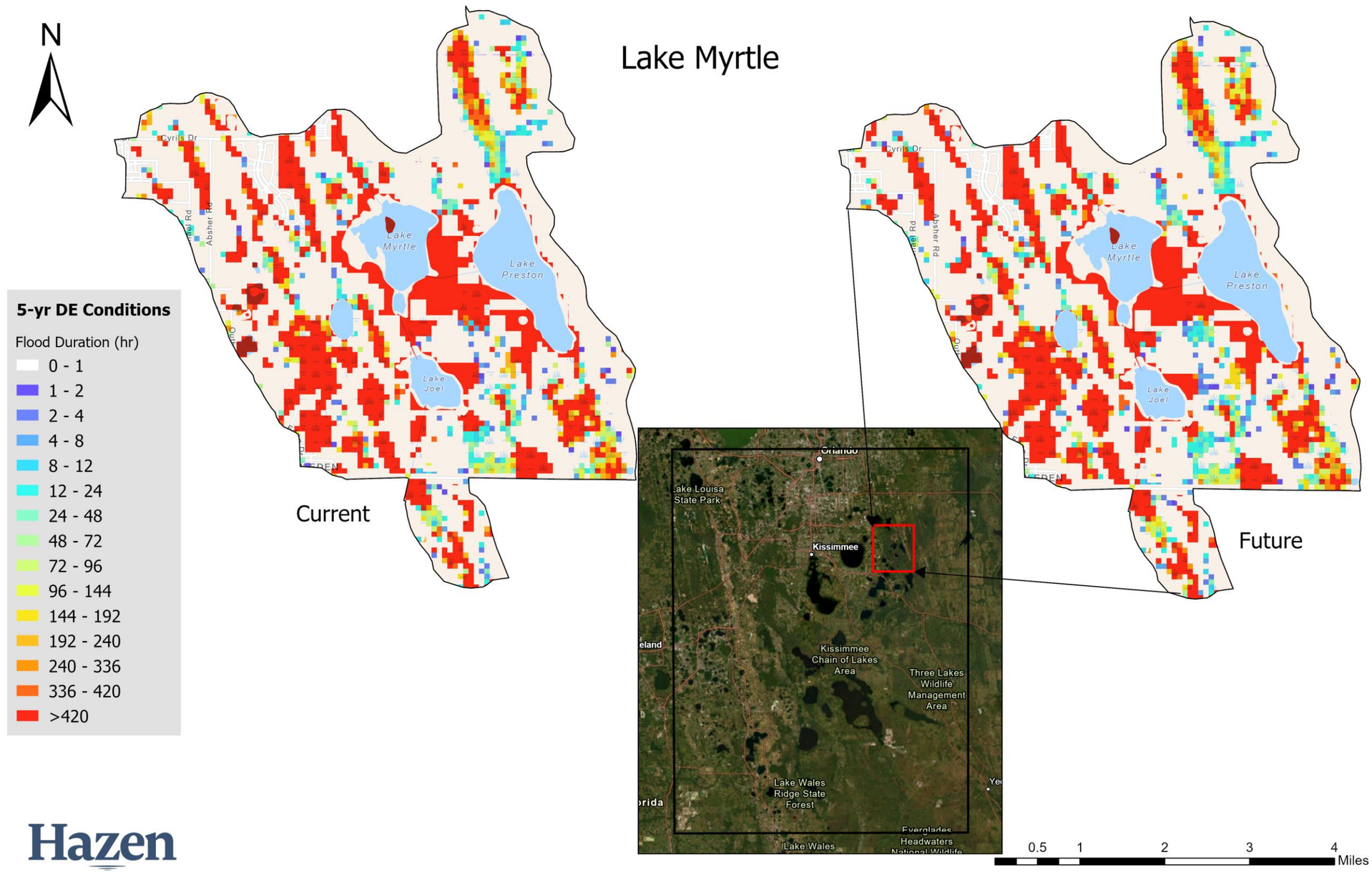

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Figure 12-19: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

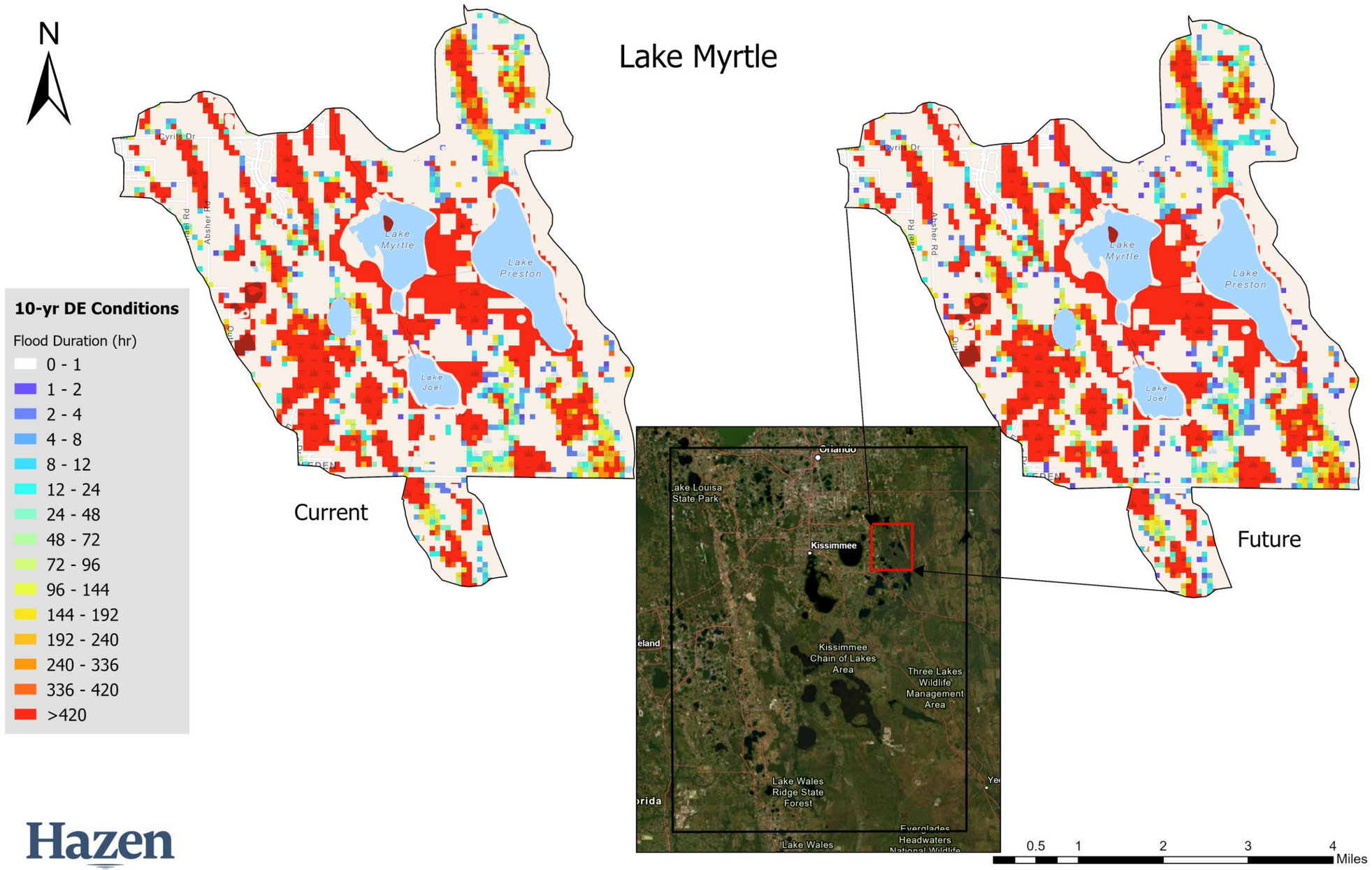

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Figure 12-20: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

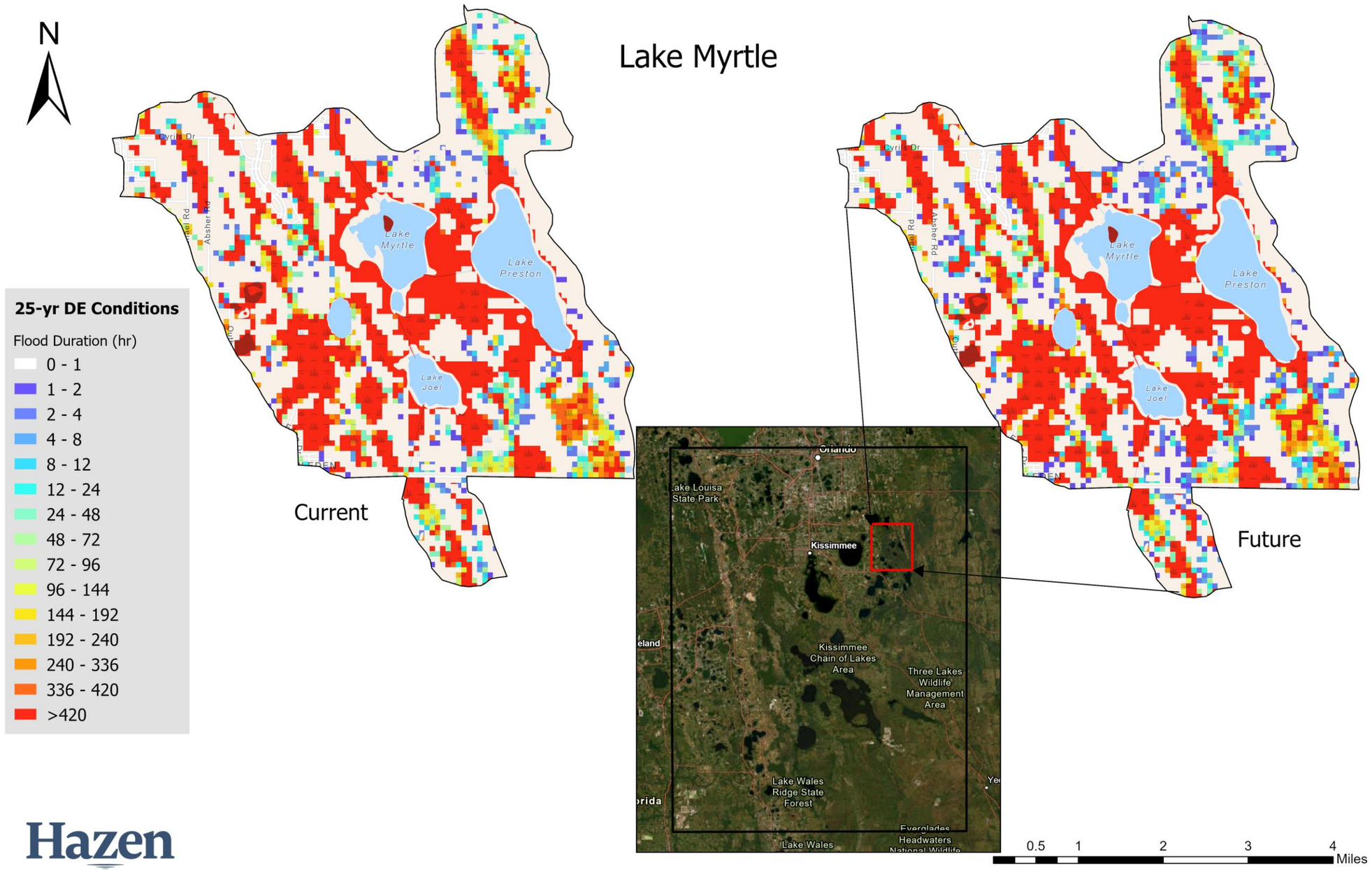

Figure 12-21: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

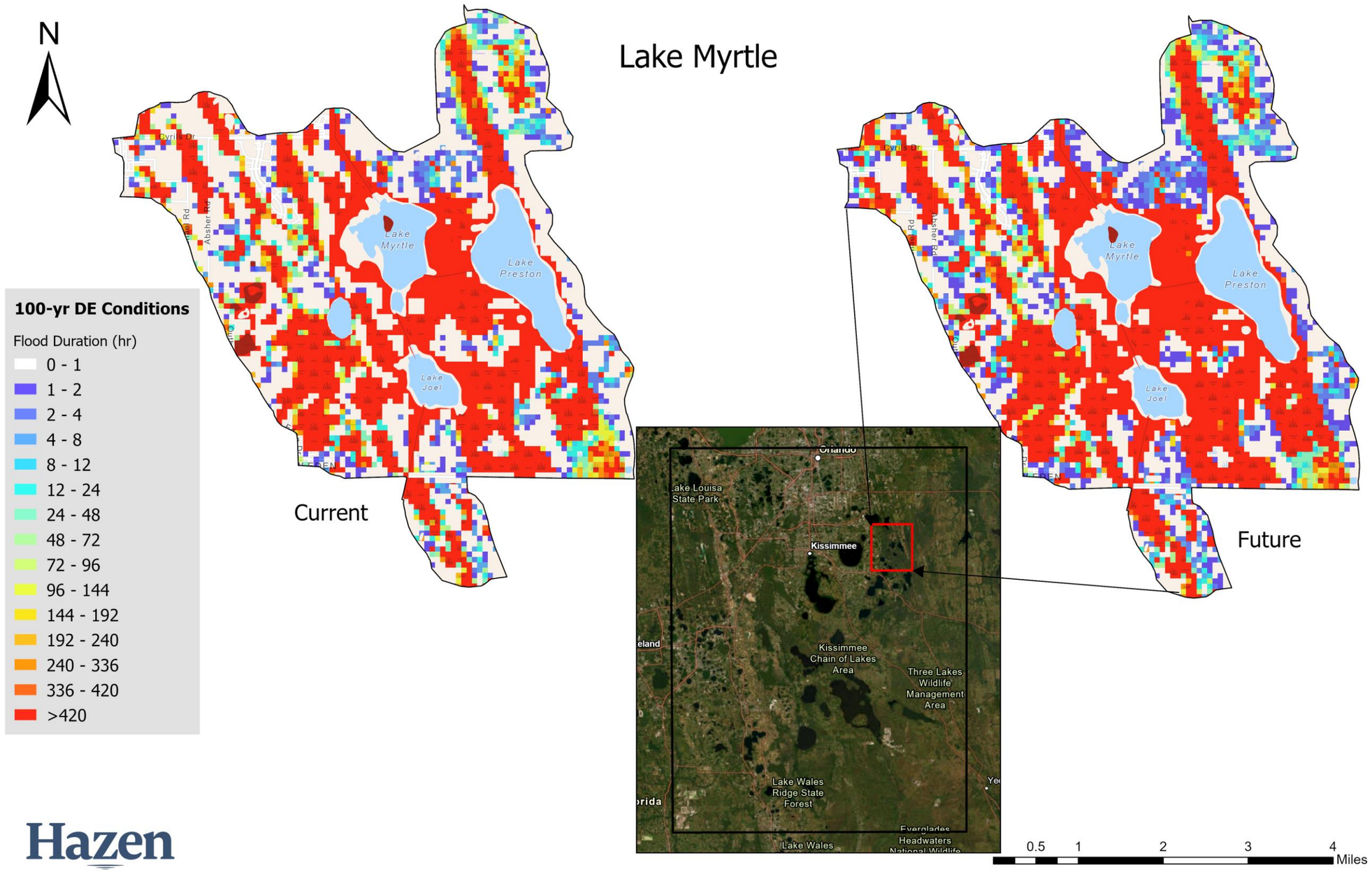

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Figure 12-22: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Myrtle LMA under Current and Future Conditions

Figure 12-23: Aerial Image of Section 1 for Lake Myrtle LMA

Section from **Figure 12-14** looks at the areas between Lakes Myrtle, Preston, and Joel, highlighting notable flooding patterns across different storm intensities. During the 5-year design event, significant flooding was observed along the eastern banks of Lake Myrtle and the southern bank of Lake Preston. Interestingly, the flood depth around Lake Myrtle was higher under current conditions than in future conditions, a deviation from the typical trend.

In other areas within this section, flood intensity remained relatively consistent between current and future scenarios. As the design event increased, overall flood depths increased throughout the region. However, future conditions consistently exhibited greater flood depths for higher-intensity events than current conditions.

The PM6 analysis saw the same trends observed in PM5, highlighting the areas mentioned above experiencing extended flood durations exceeding 420 hours of drainage time, especially around the banks

of Lake Myrtle. Notably, there were no significant differences between future and current conditions. These areas with higher duration are vulnerable to current and future conditions.

Figure 12-24: Aerial Image of Section 2 for Lake Myrtle LMA

Section 2 (Figure 12-14) of the Lake Myrtle LMA focuses on the region east of Lake Joel, identified as wetlands in the aerial imagery shown in Figure 2-1. This area experienced flooding across all design events, from the 5-year to the 100-year storm, with flood depth increasing progressively as the storm intensity grew. Additionally, flood depths consistently rose when comparing current conditions to future scenarios for each design event.

PM6 analysis for section 2 exhibited significantly higher flood durations around the wetlands, increasing as design events intensified from the 5-year to the 100-year events. Notable differences emerged between future and current conditions across all design scenarios, with future conditions consistently showing prolonged durations, highlighting the vulnerability of the wetlands to flooding, emphasizing their sensitivity to extended inundation under all design events.

12.3 Lake Hart LMA

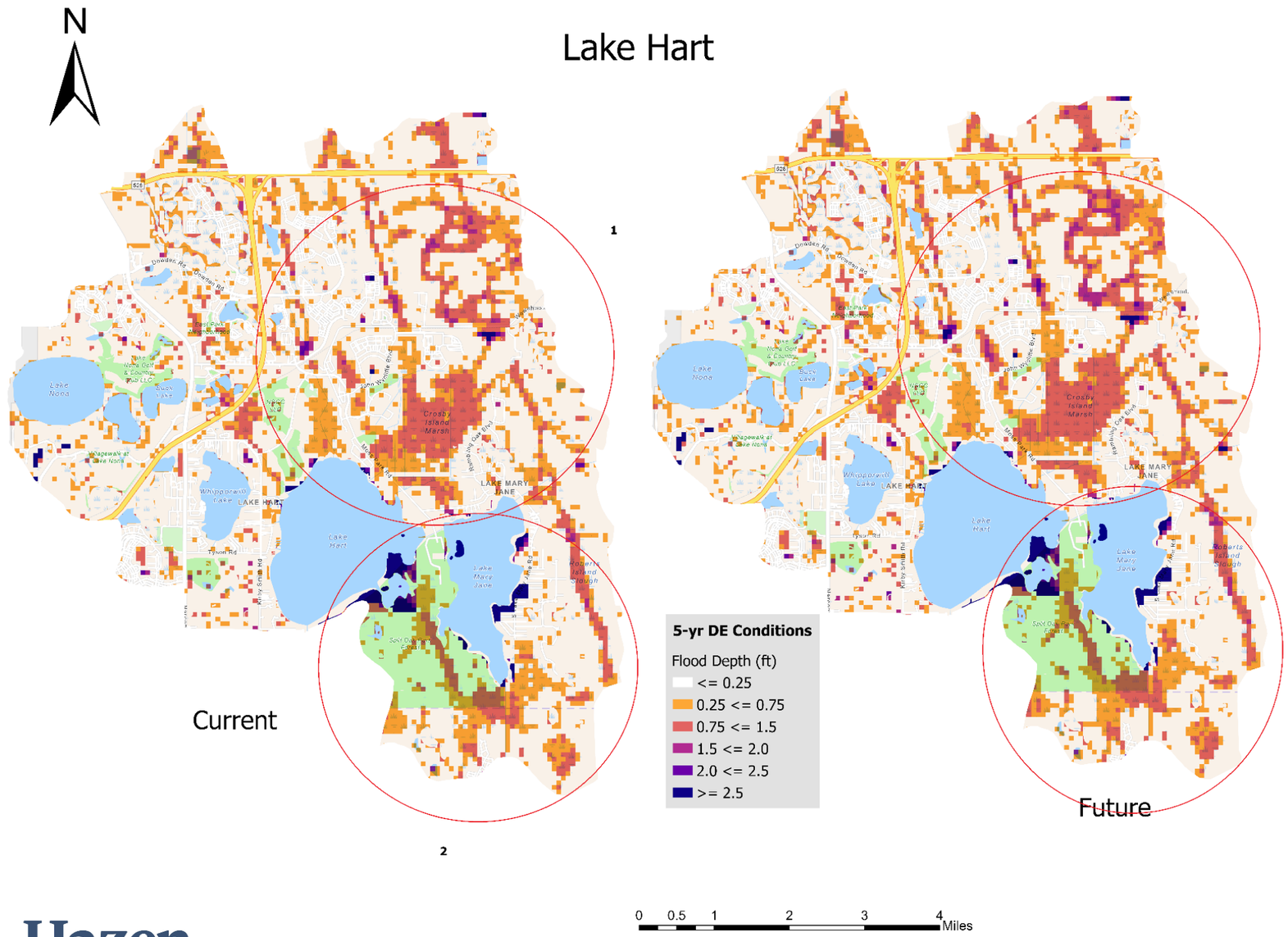
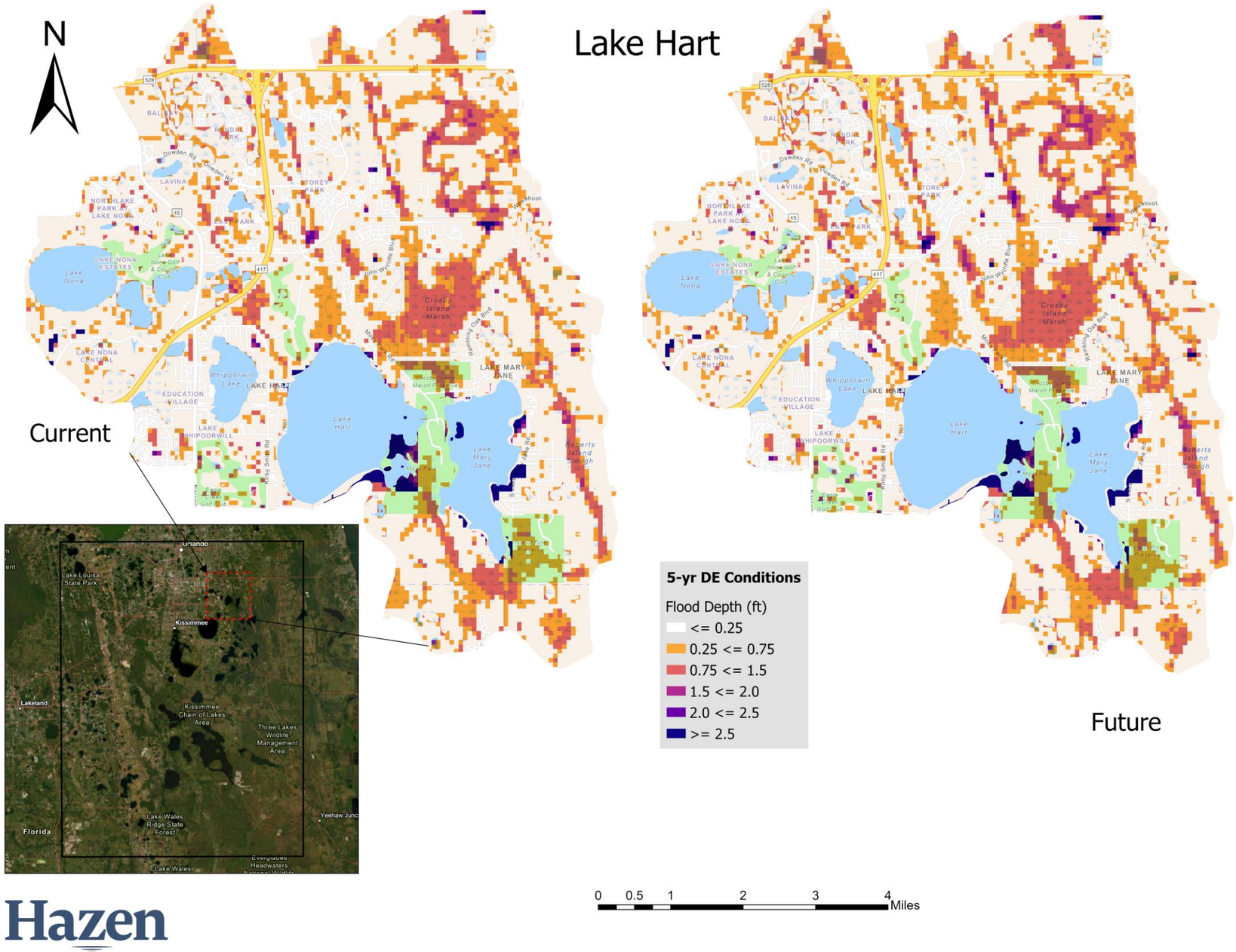
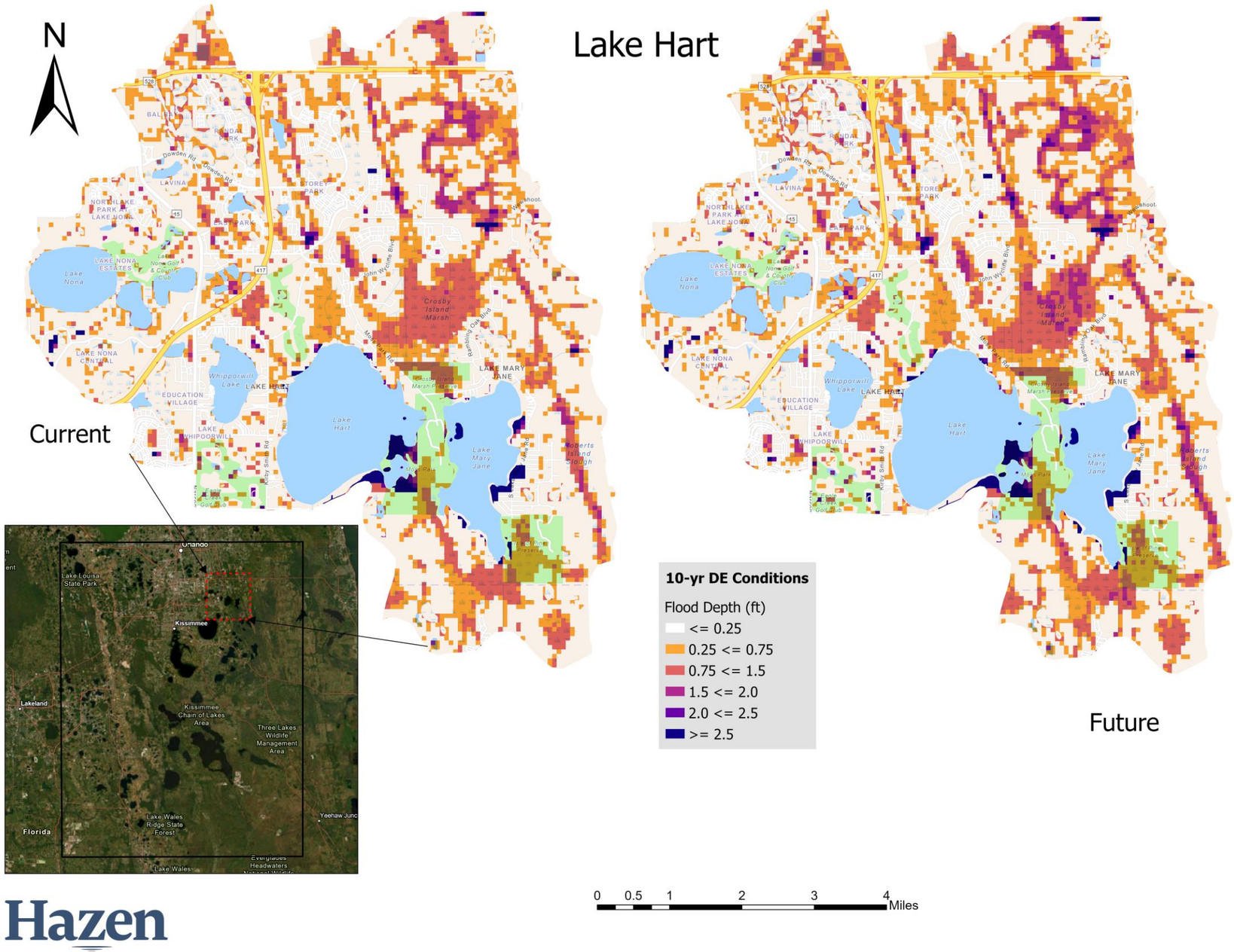


Figure 12-25: Lake Hart LMA in Sections



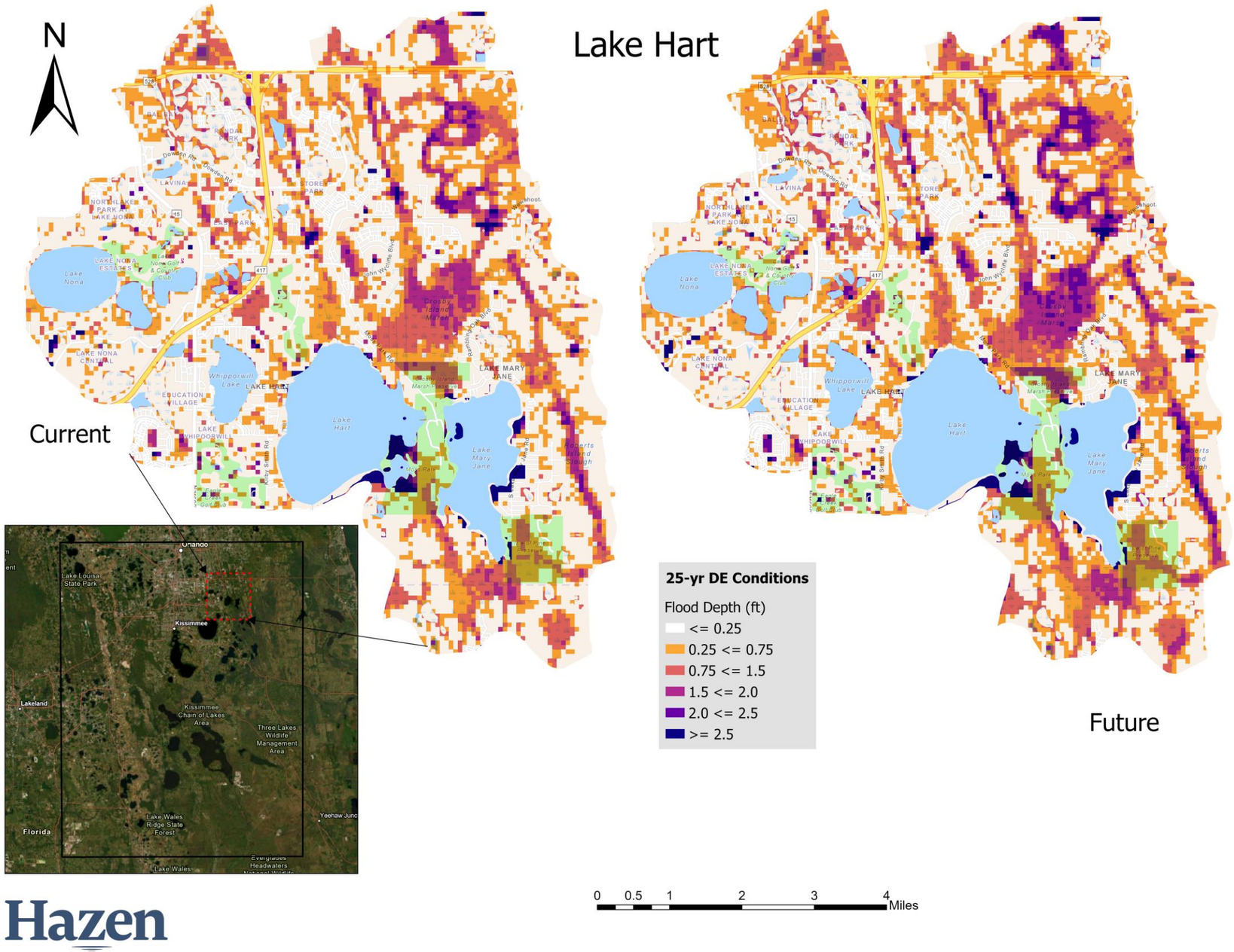
Hazen

Figure 12-26: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA



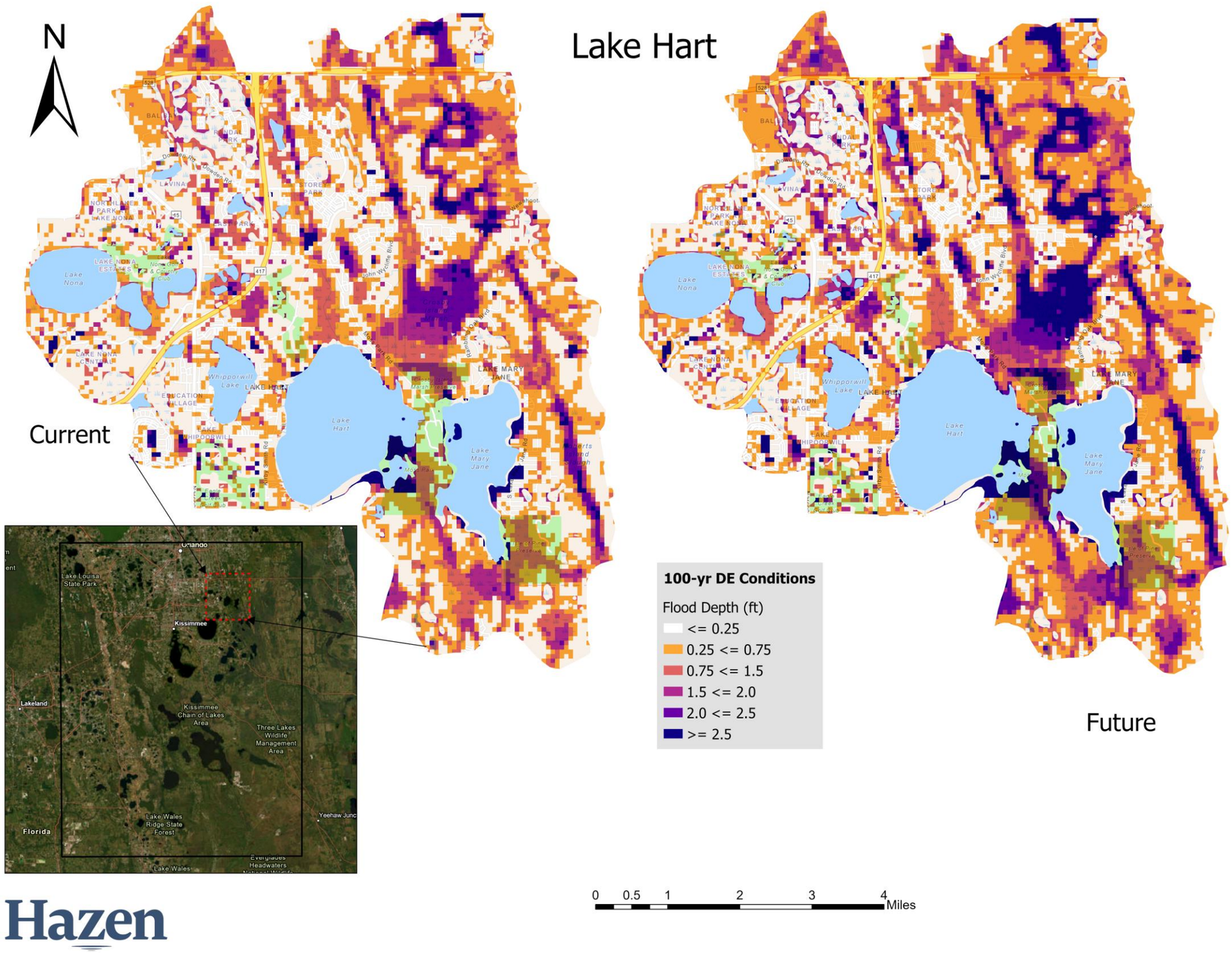
Hazen

Figure 12-27: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA



Hazen

Figure 12-28: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA



Hazen

Figure 12-29: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Hart LMA

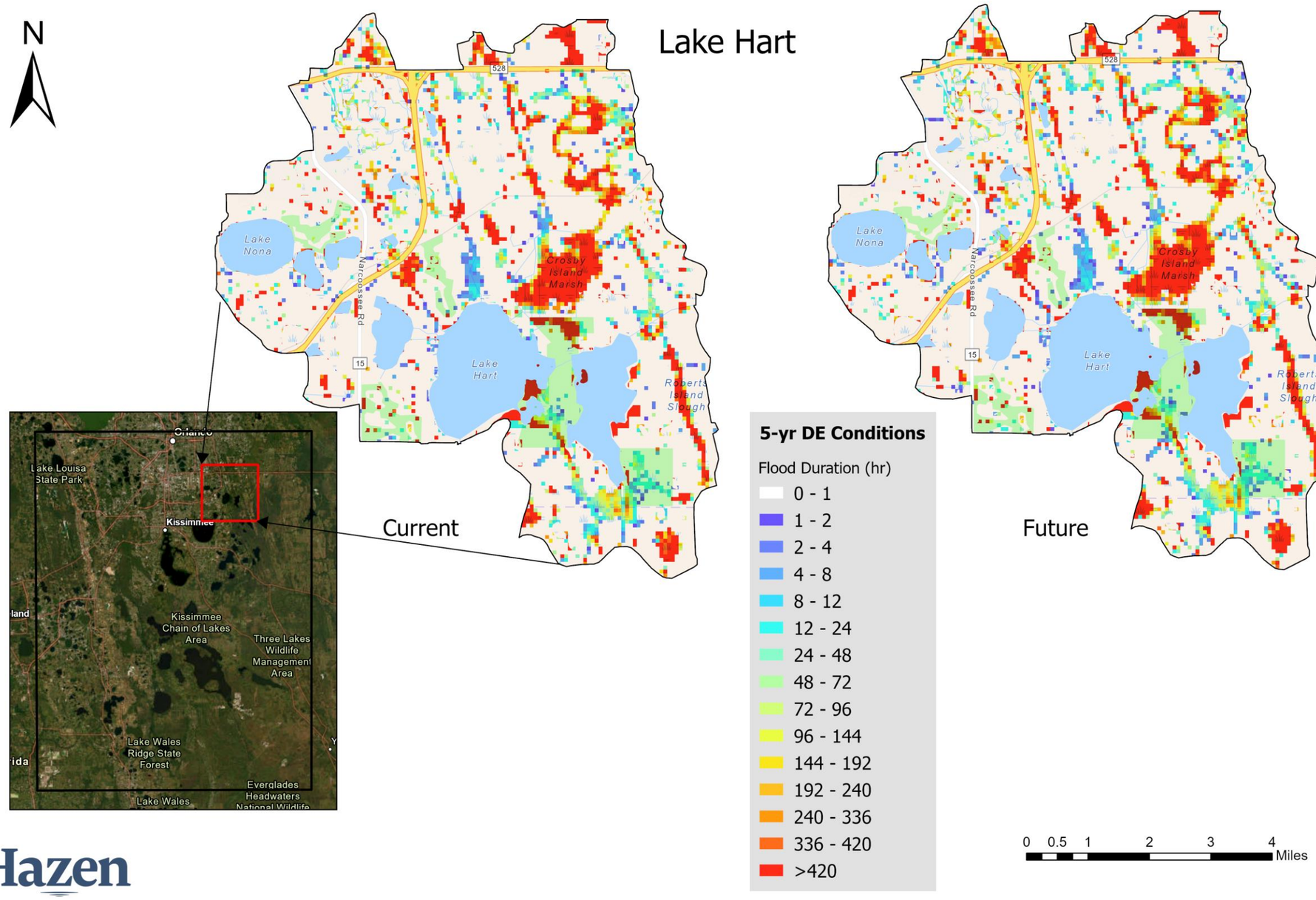


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Figure 12-30: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

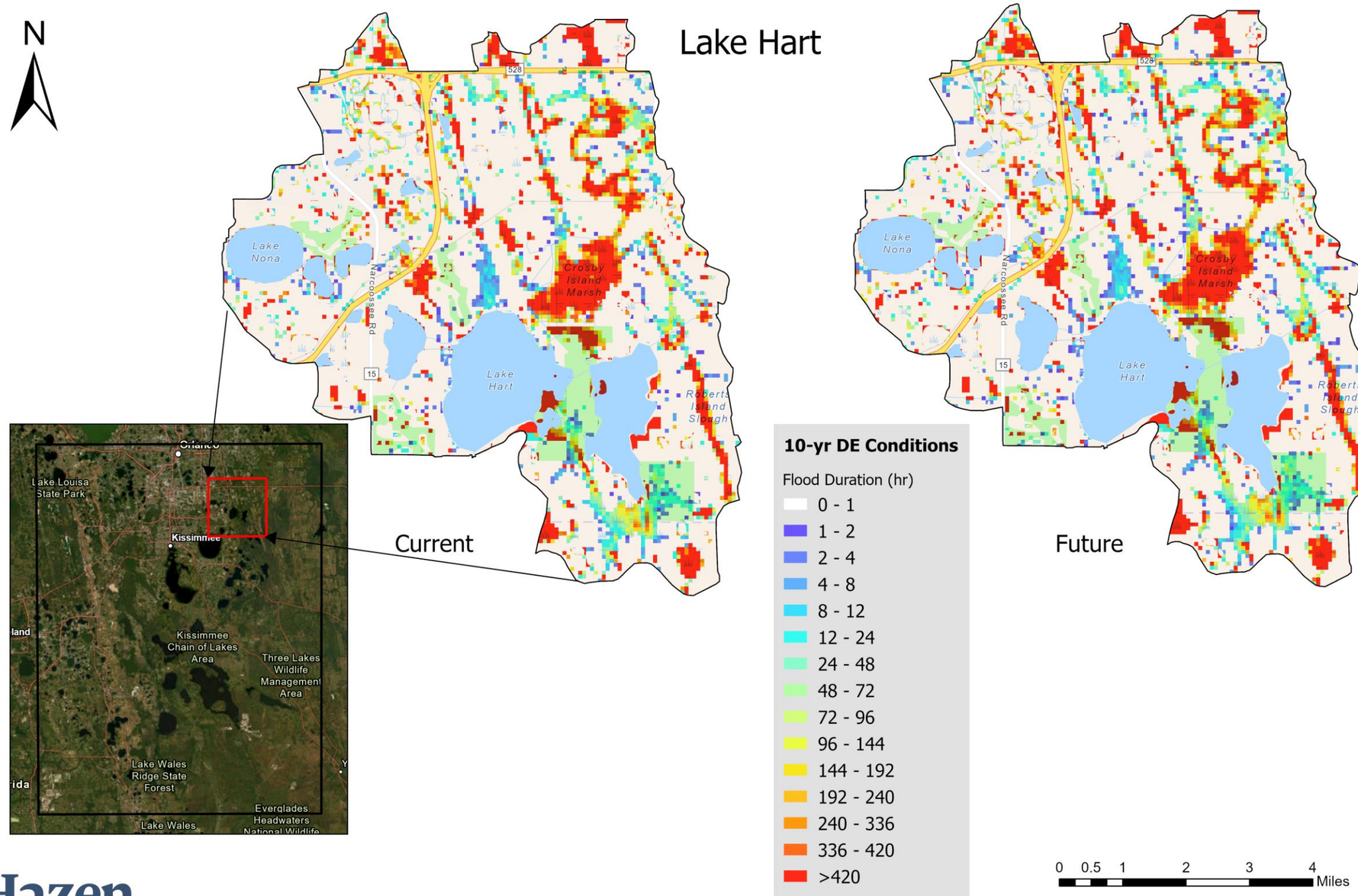

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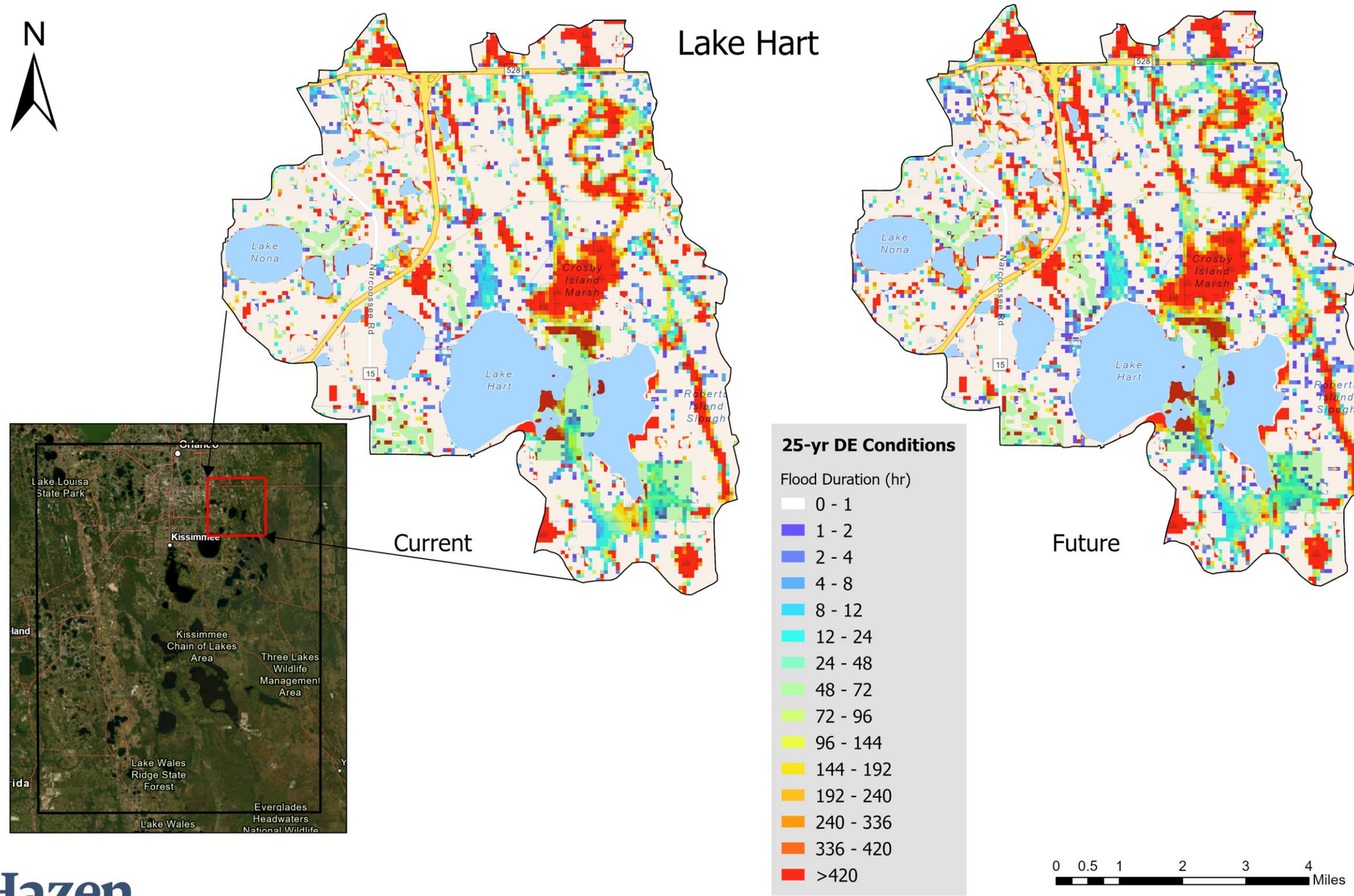
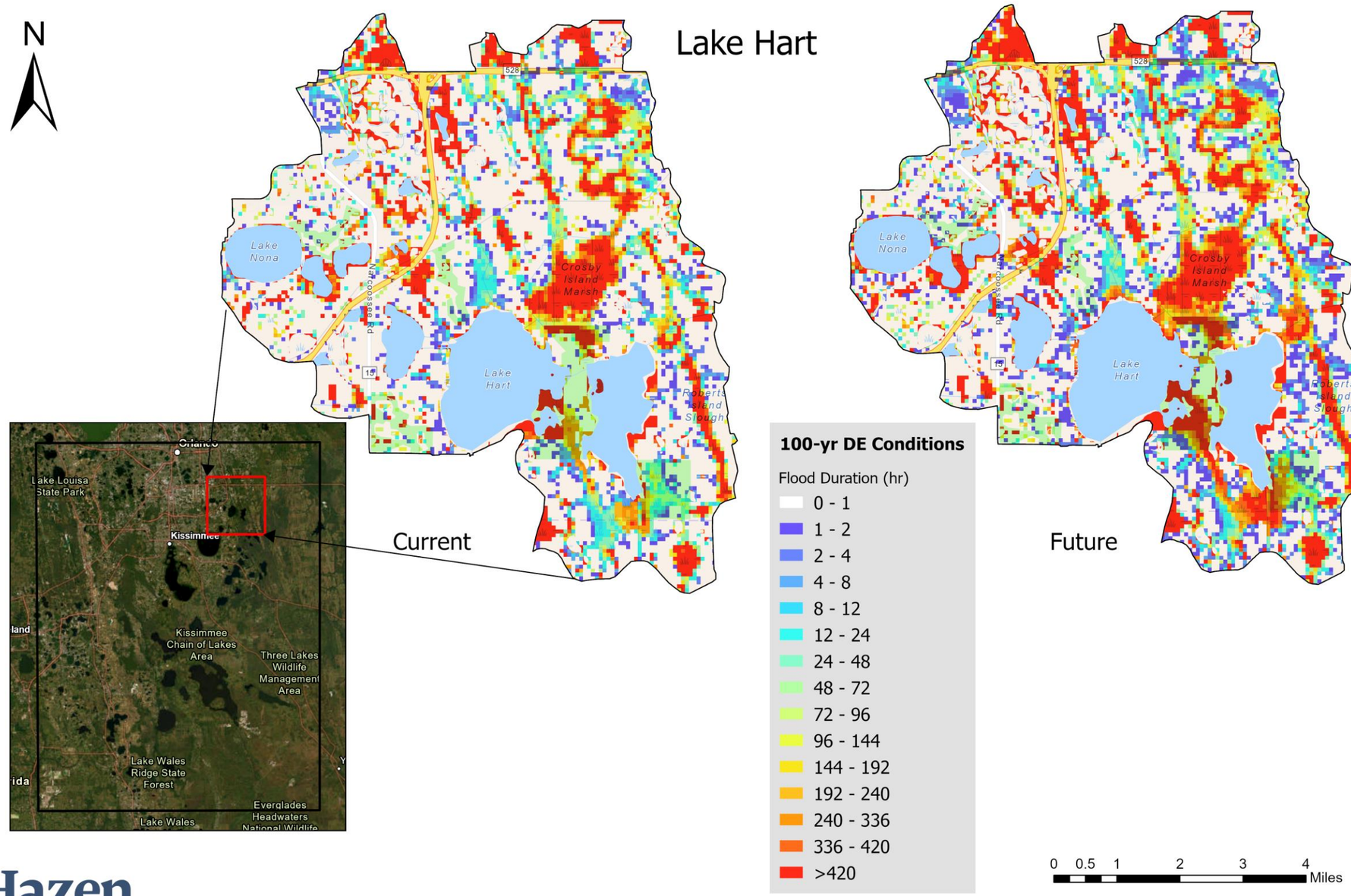
Figure 12-31: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions



Hazen

Figure 12-32: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions



Hazen

Figure 12-33: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Hart LMA under Current and Future Conditions

Figure 12-34: Aerial Image of Section 1 for Lake Hart LMA

Section 1 of the Hart LMA from **Figure 12-25** examines the area east of State Road 417 and south of State Road 528. Flooding was observed across all design events, from the 5-year to the 100-year storms, particularly around Crosby Island Marsh and the northeastern portion of the section. As storm intensity increased, flood depths rose progressively, with future conditions consistently showing greater flood depths than current conditions for each design event.

The PM6 analysis revealed prolonged flood durations exceeding 420 hours in areas such as Crosby Island Marsh. As design events intensified, the extent of regions experiencing higher flood durations also expanded. While differences between future and current conditions were minimal, the consistently elevated durations emphasize the vulnerability of these areas to flooding.

Figure 12-35: Aerial Image of Section 2 for Lake Myrtle LMA

Section 2 (Figure 12-25) focuses on the region between Lake Hart and Lake Mary Jane, including areas east of Lake Mary Jane. During the 5-year design event, significant flood depths were observed along portions of the banks of both lakes, while regions such as Roberts Island Slough experienced flood depths ranging from 0.25 to 0.75 feet.

Flood depths increased with higher design events. While the 5-year and 10-year events showed no notable differences between current and future conditions, the 25-year and 100-year events revealed a marked increase in flood depths under future conditions.

For flood duration, the same banks inundated during the 5-year event exhibited prolonged drainage times exceeding 420 hours. However, the overall section maintained a moderate average drainage time of less than 24 hours during the 100-year event. Notably, the 100-year event was the only scenario that revealed apparent differences between current and future conditions.

12.4 Lake Gentry LMA

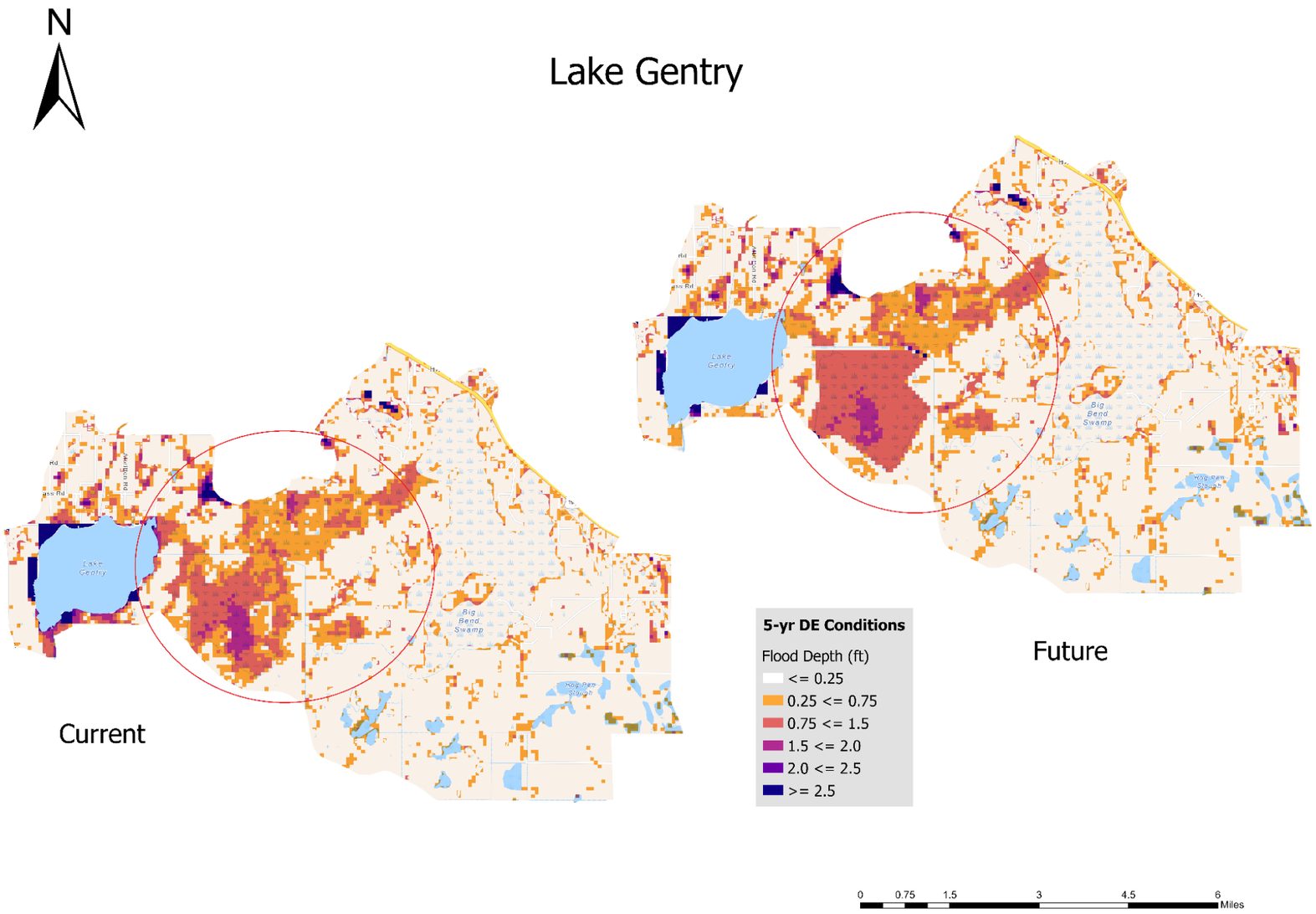


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Figure 12-36: Lake Gentry LMA Section

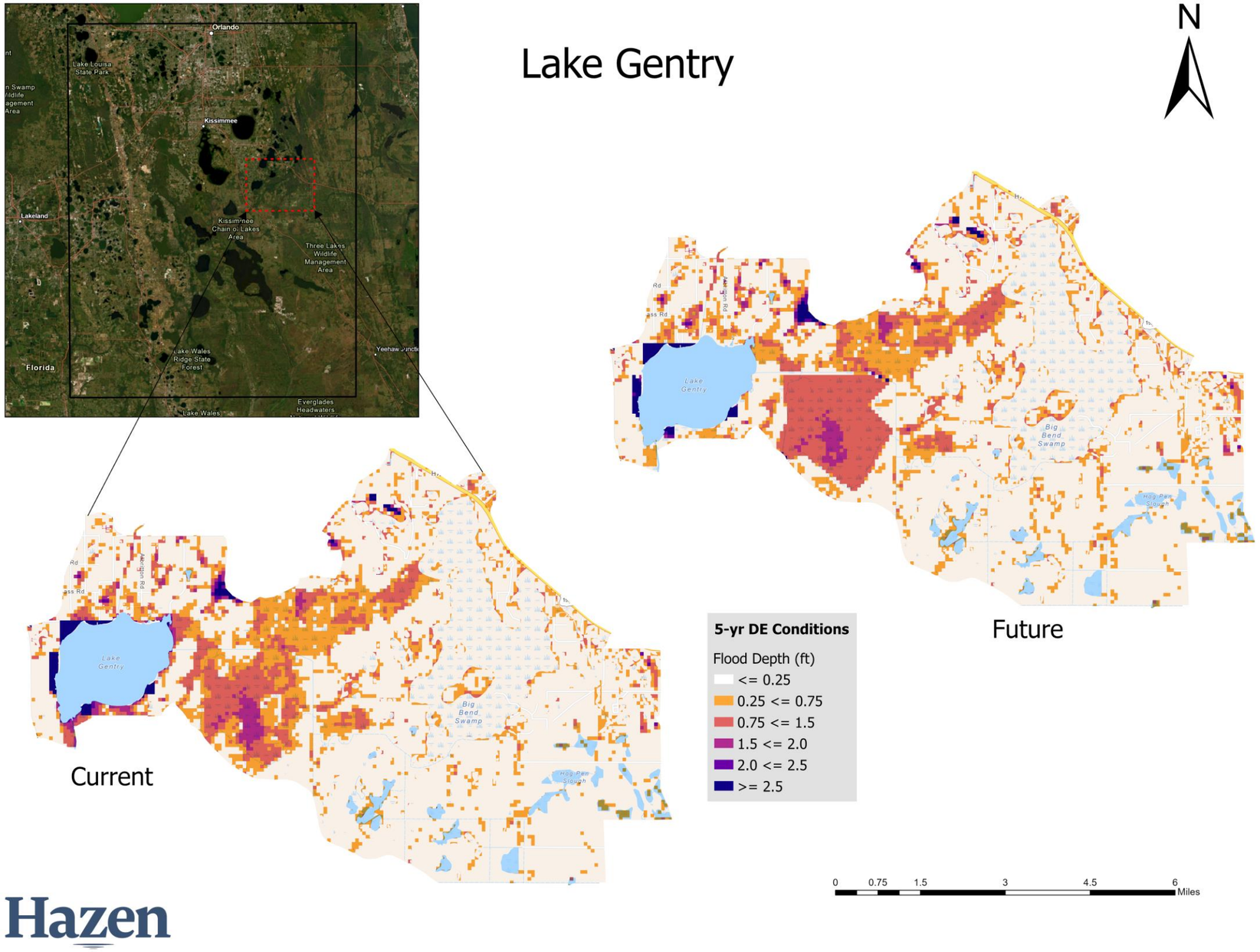

Figure 12-37: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

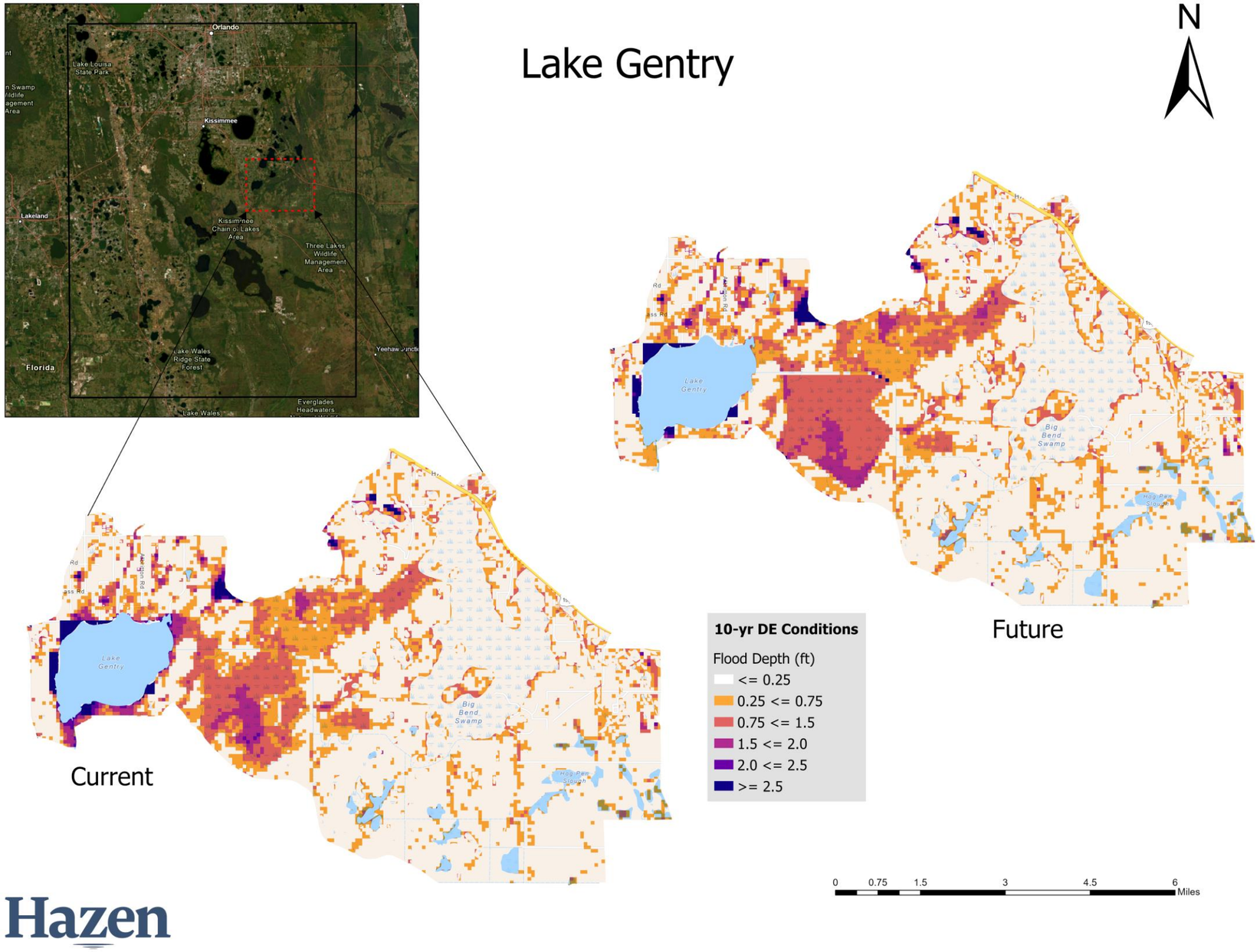

Figure 12-38: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

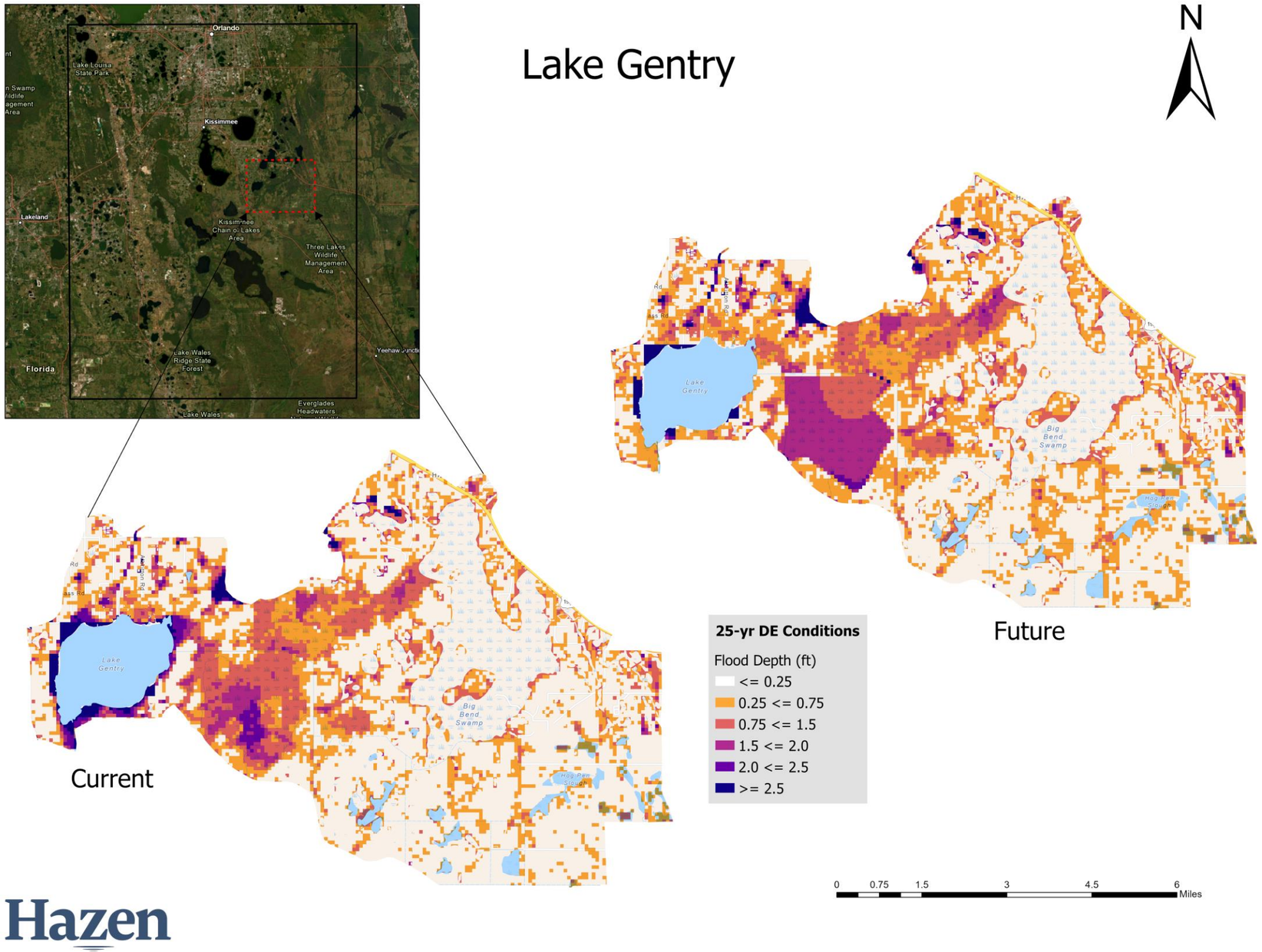

Figure 12-39: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA

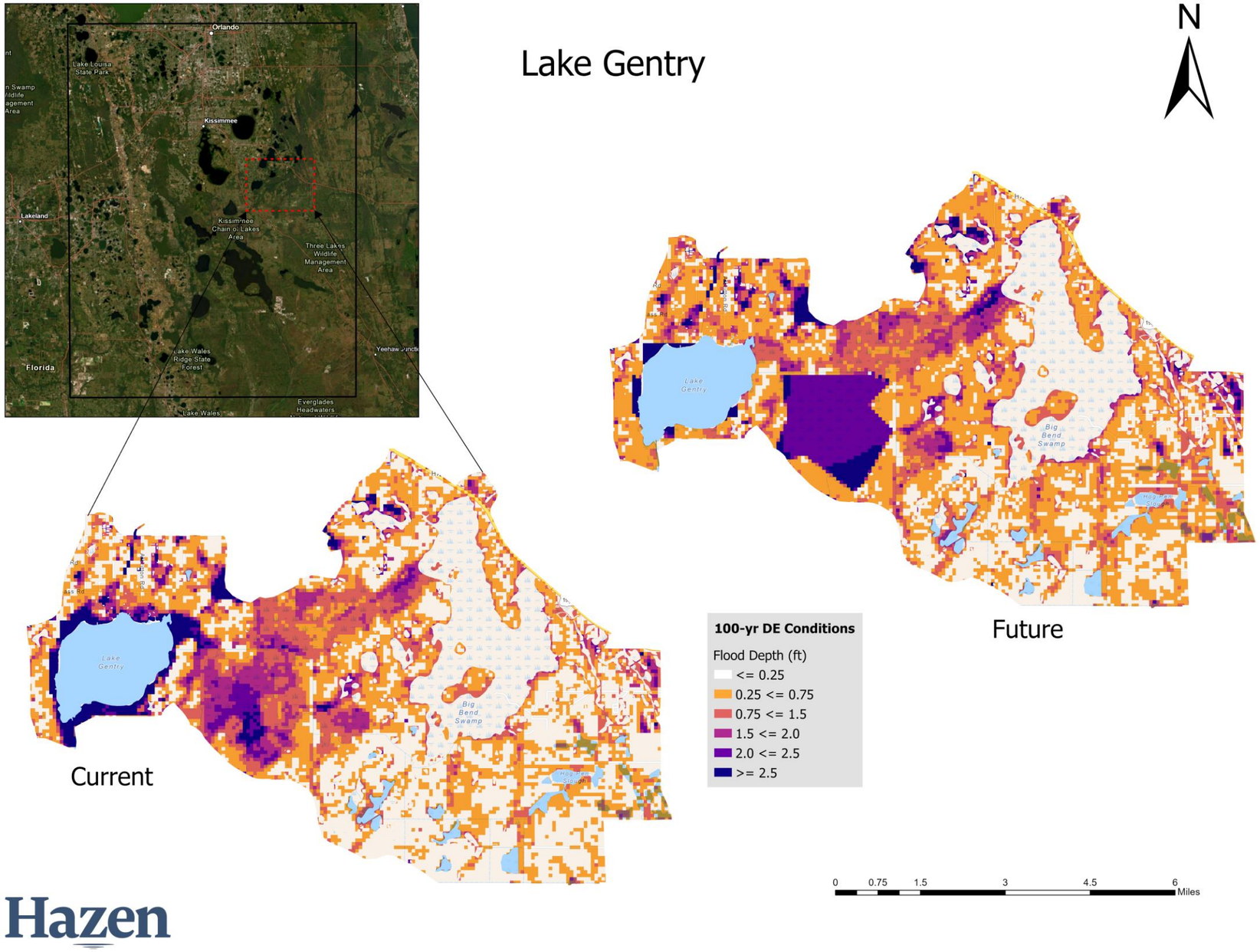

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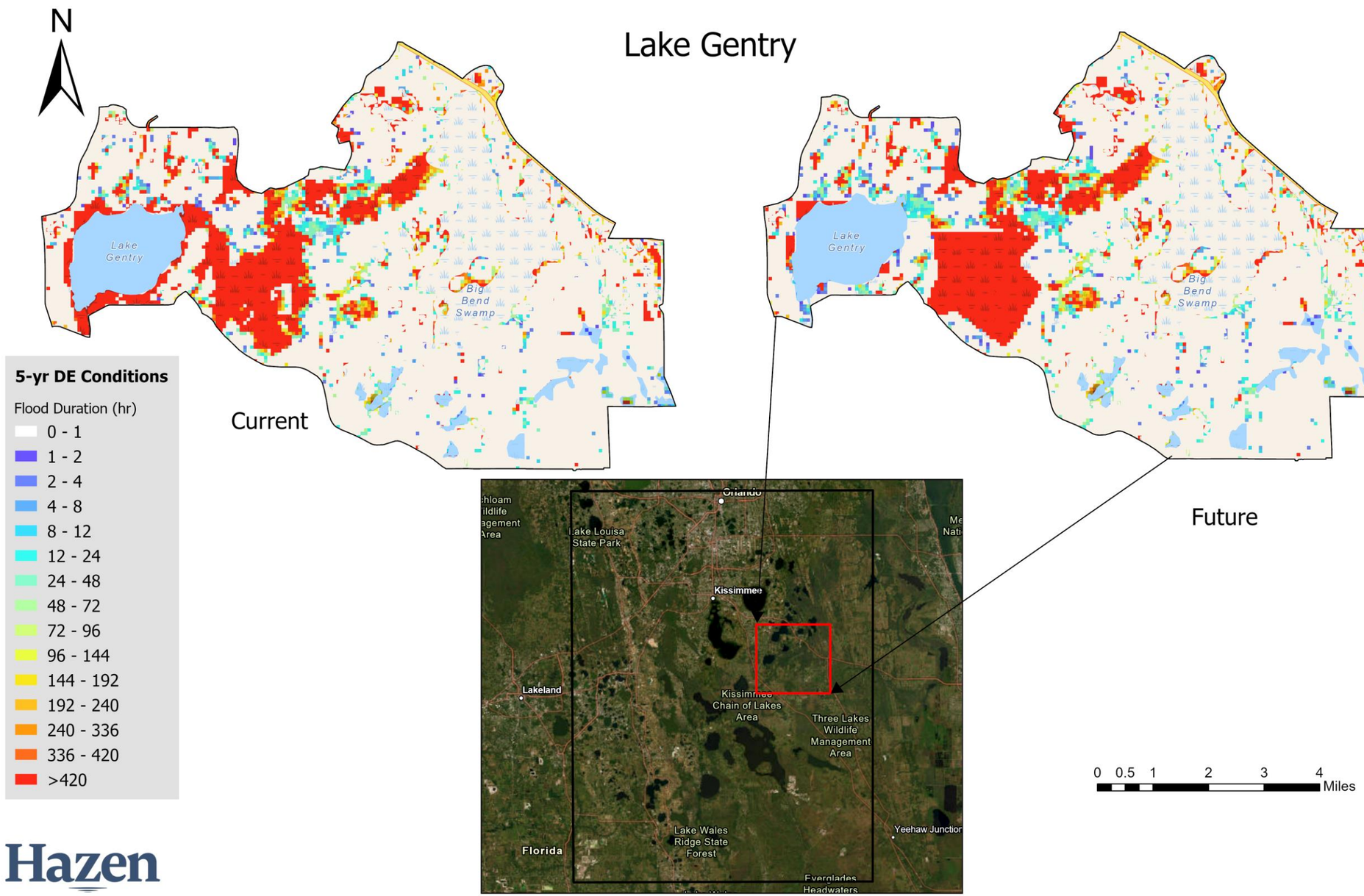
Figure 12-40: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Gentry LMA



Hazen

Figure 12-41: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions

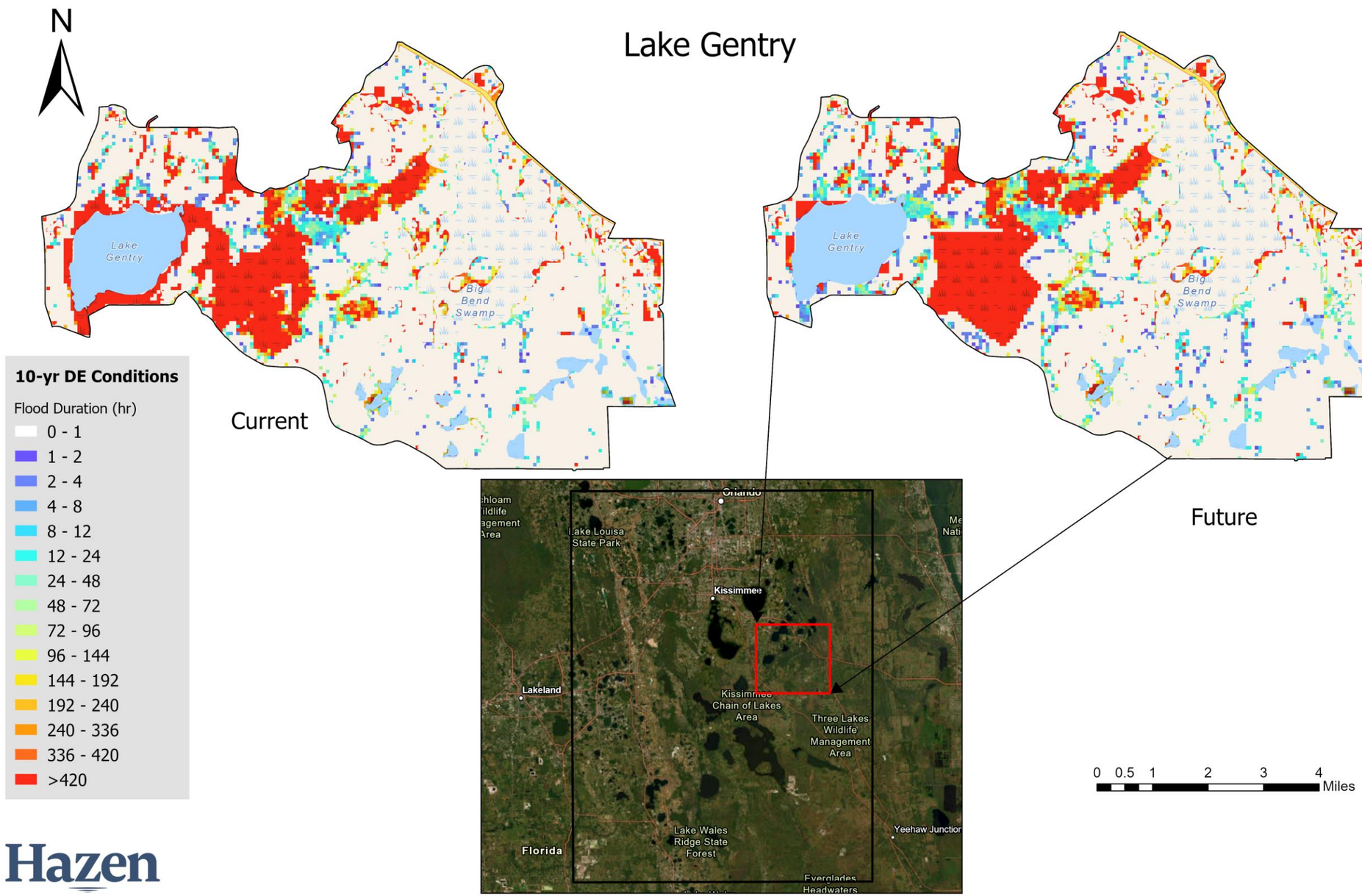


Figure 12-42: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions

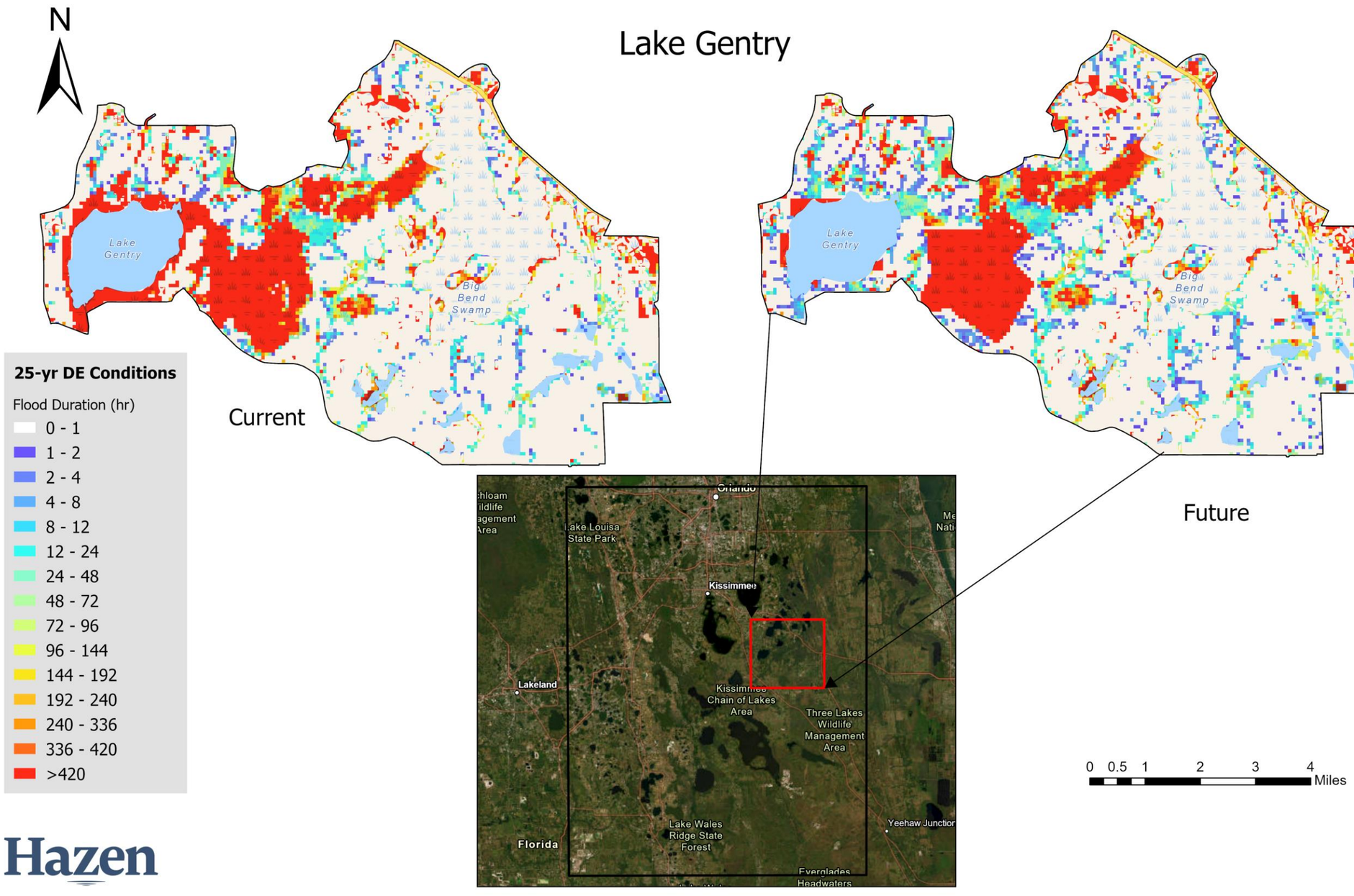


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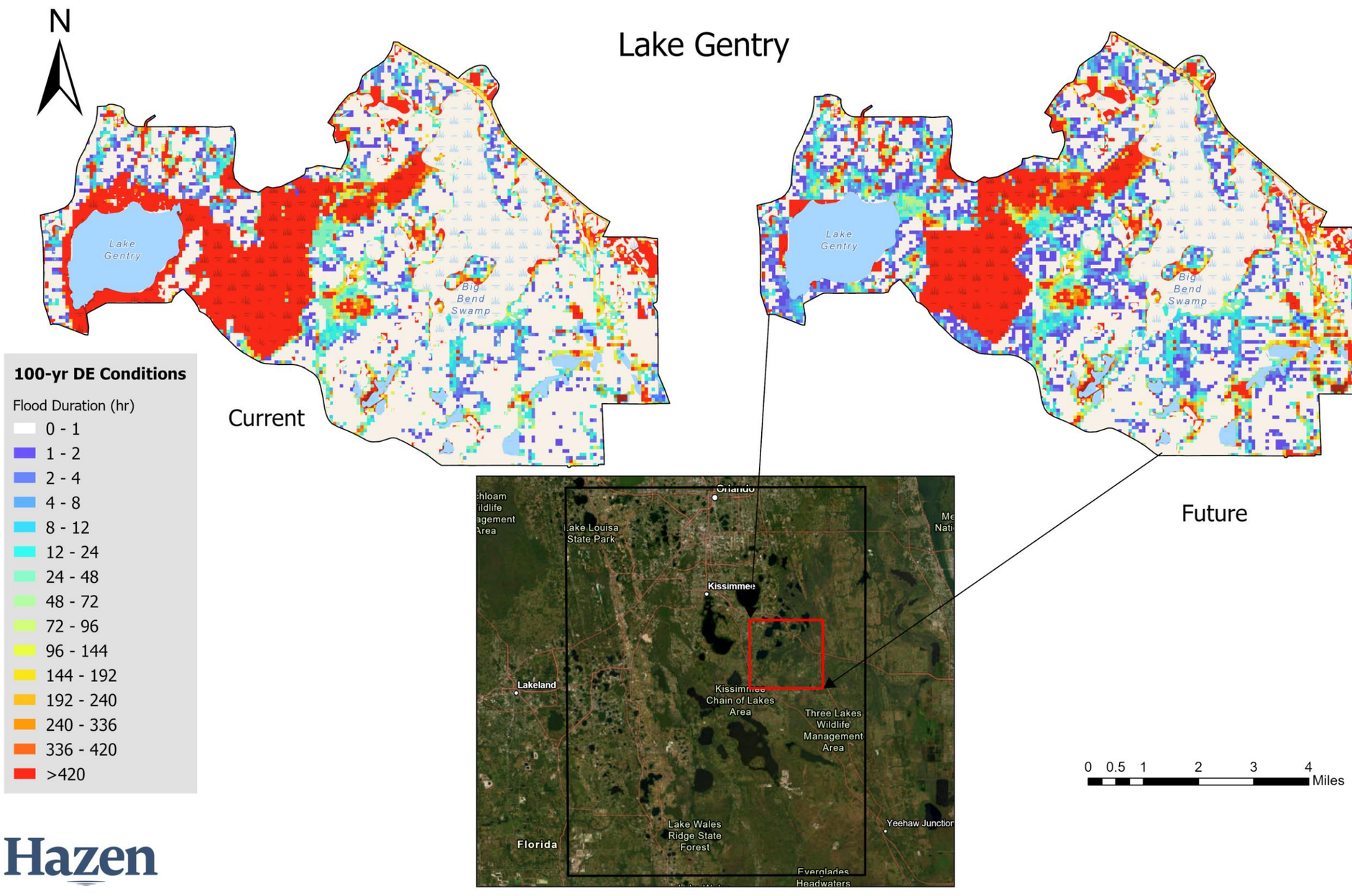
Figure 12-43: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions



Hazen

Figure 12-44: Flood Duration Maps for 100-yr 3-day Design Storm for Lake Gentry LMA under Current and Future Conditions

Figure 12-45: Aerial Image of Section 1 for Lake Gentry LMA

In addition to the banks of Lake Gentry, which experienced higher flood depths across all design events, the circled area east of Lake Gentry, as shown in Figure 12-36, also saw significant flooding throughout. Flood depths increased progressively with the severity of the design events, and a consistent rise in flood depth was observed when comparing current conditions to future scenarios for each design event.

Flood duration analysis revealed a strong correlation with extended flood depths, notably exceeding 420 hours in areas with greater depths. As design events intensified, there was a noticeable progression in the extent of regions experiencing prolonged durations. Around the banks of Lake Gentry, future conditions exhibited lower flood durations compared to the higher values recorded under current conditions. However, overall, this section demonstrated a trend of increased flood duration areas under future scenarios, highlighting its vulnerability to prolonged flooding.

12.5 East Lake Tohopekaliga LMA

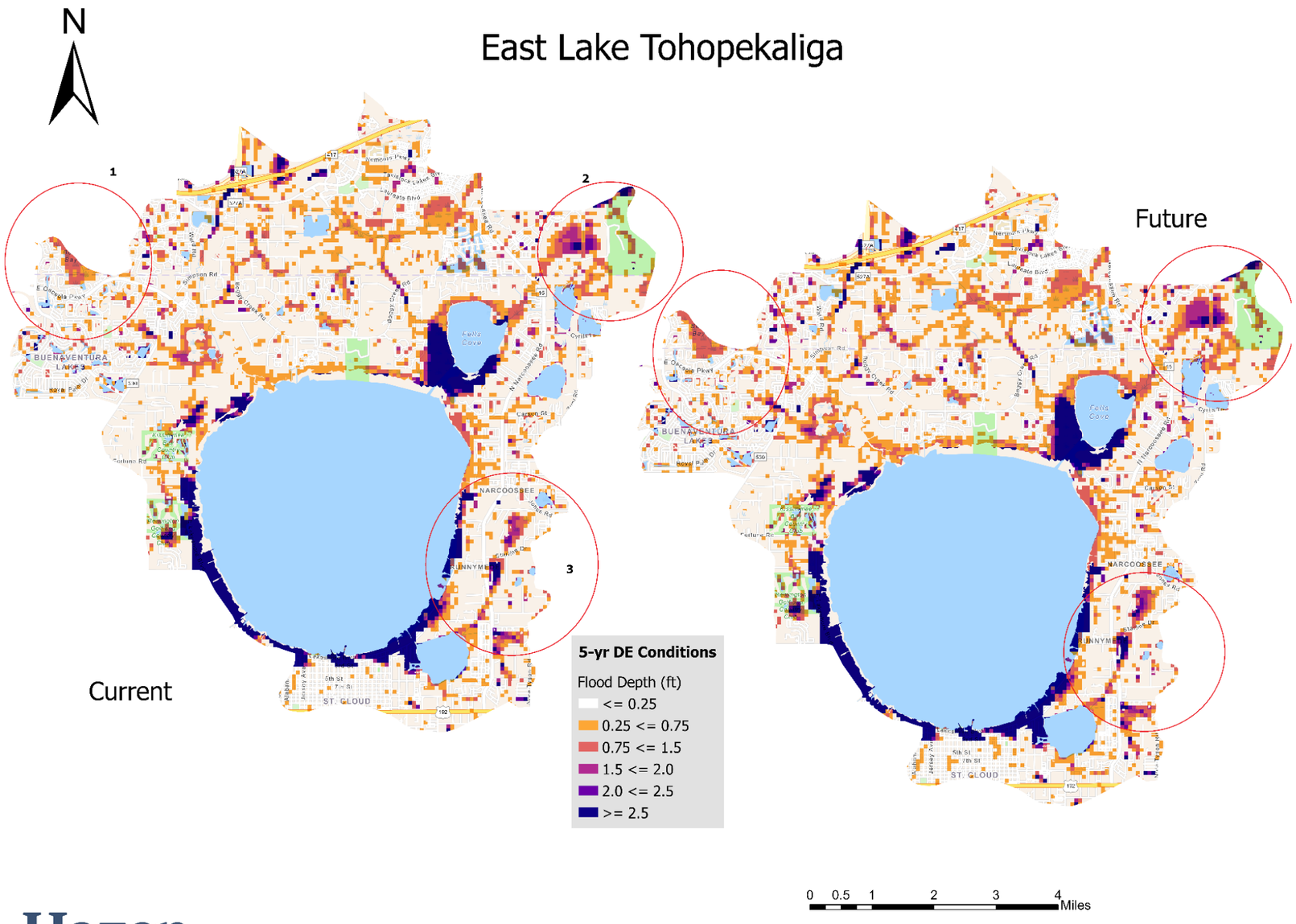


Figure 12-46: East Lake Toho in Sections

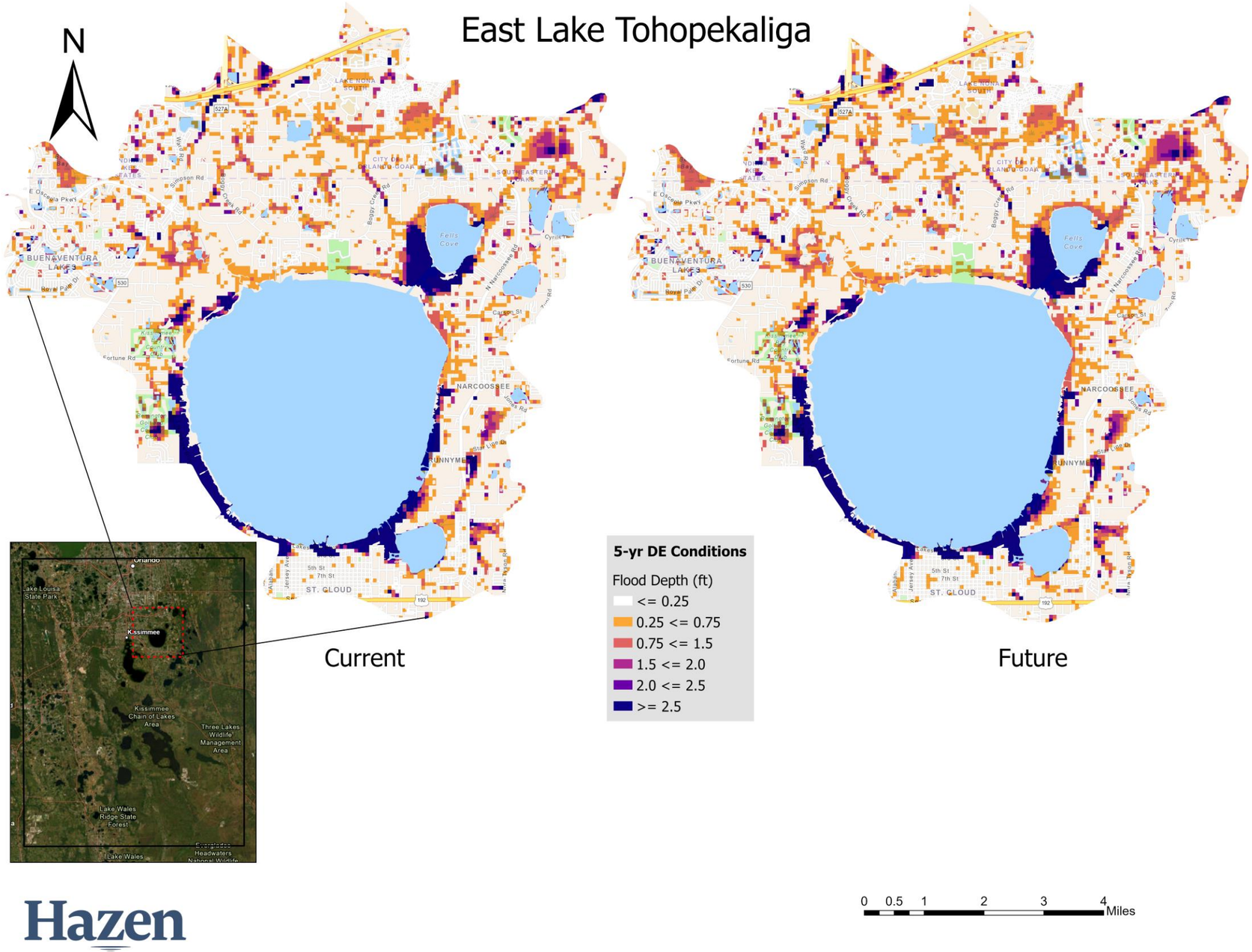


Figure 12-47: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA

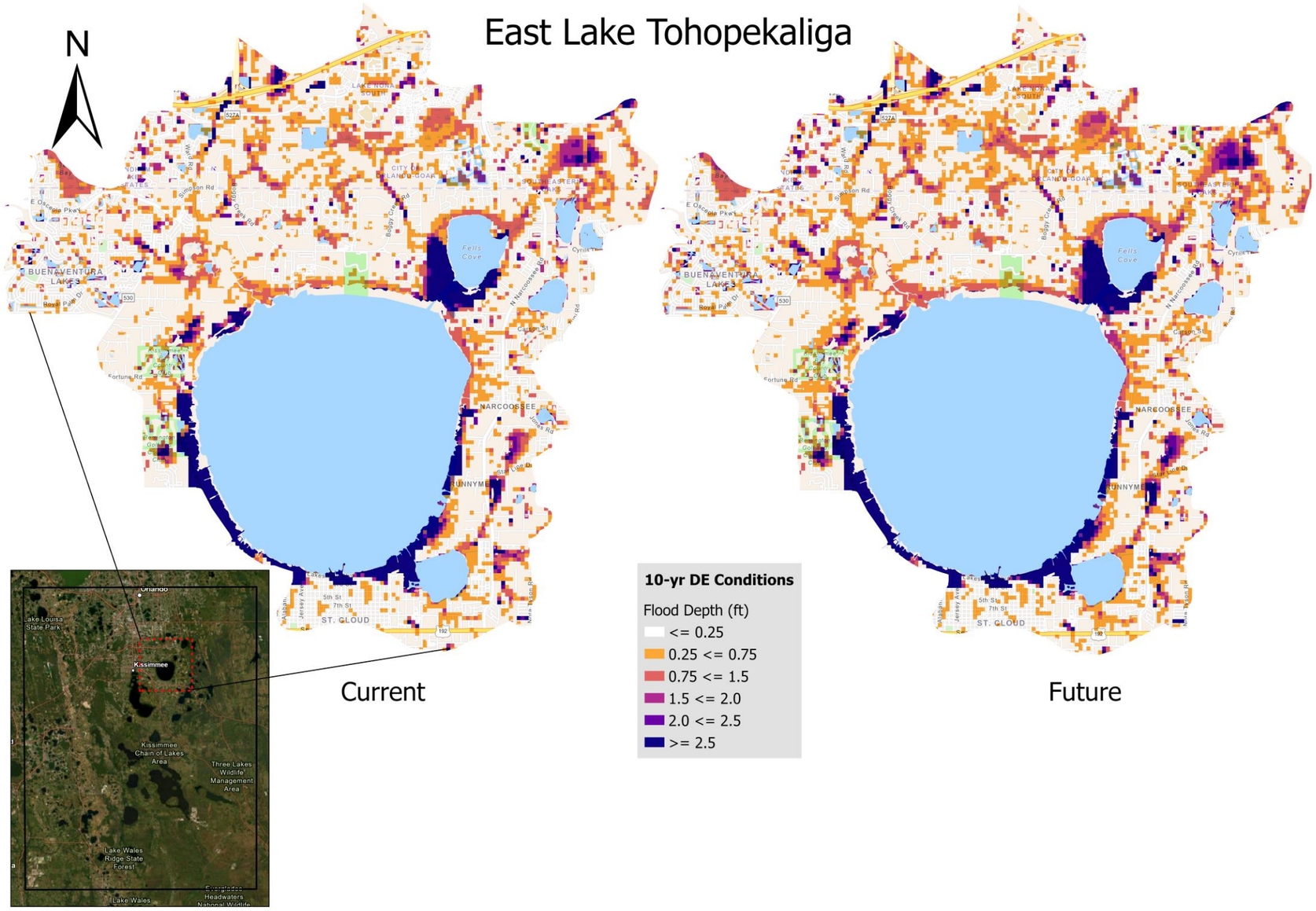
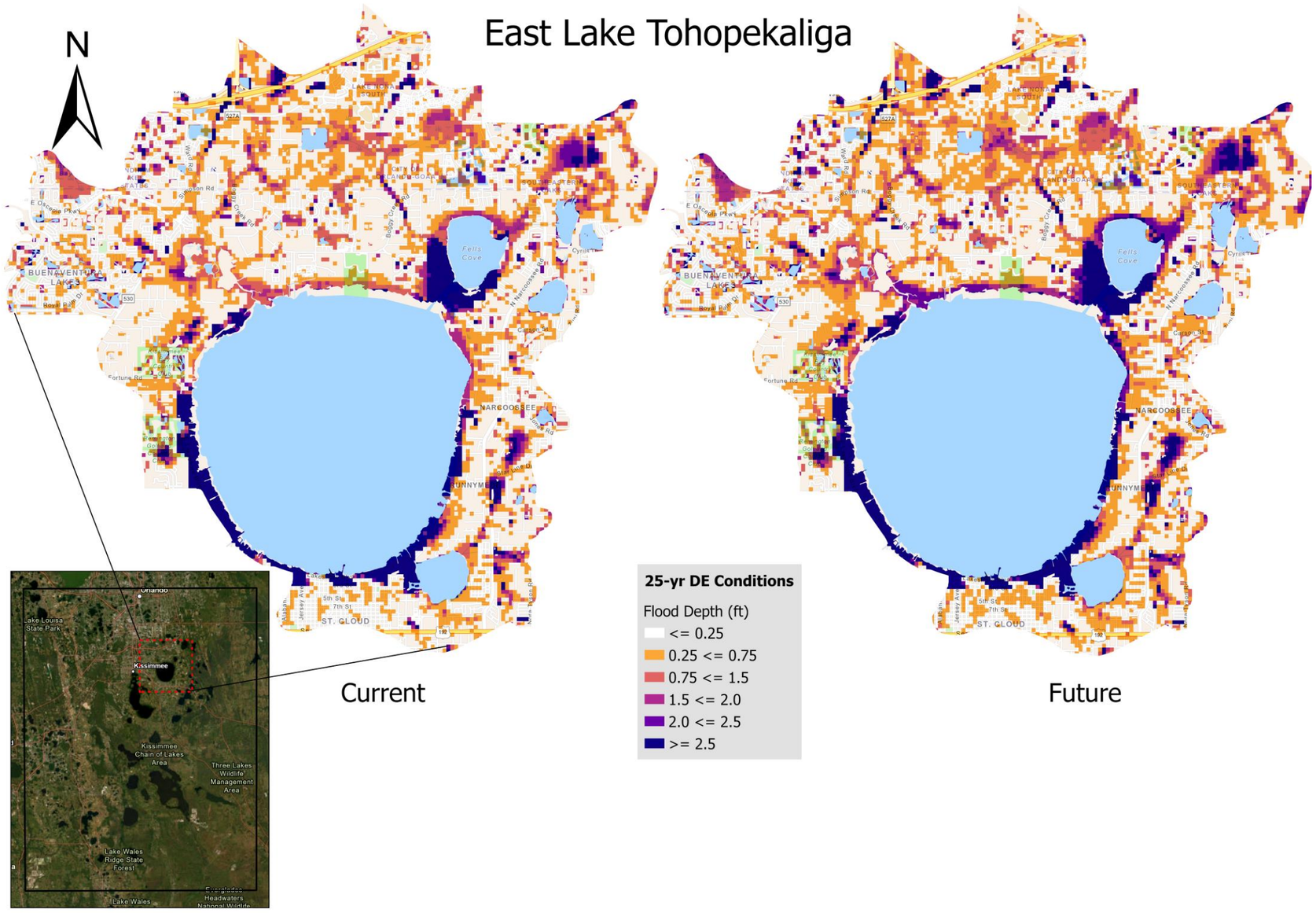
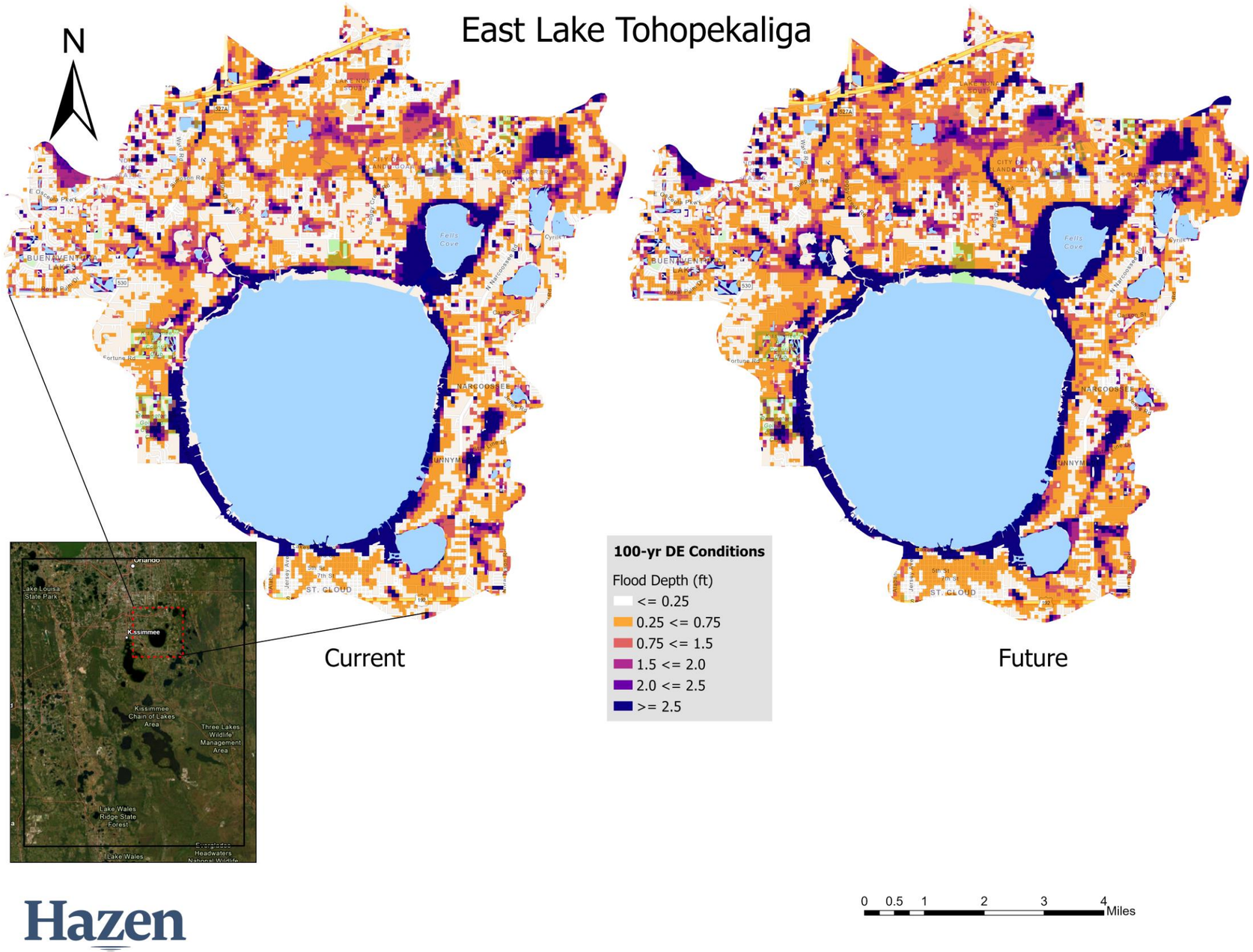


Figure 12-48: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA



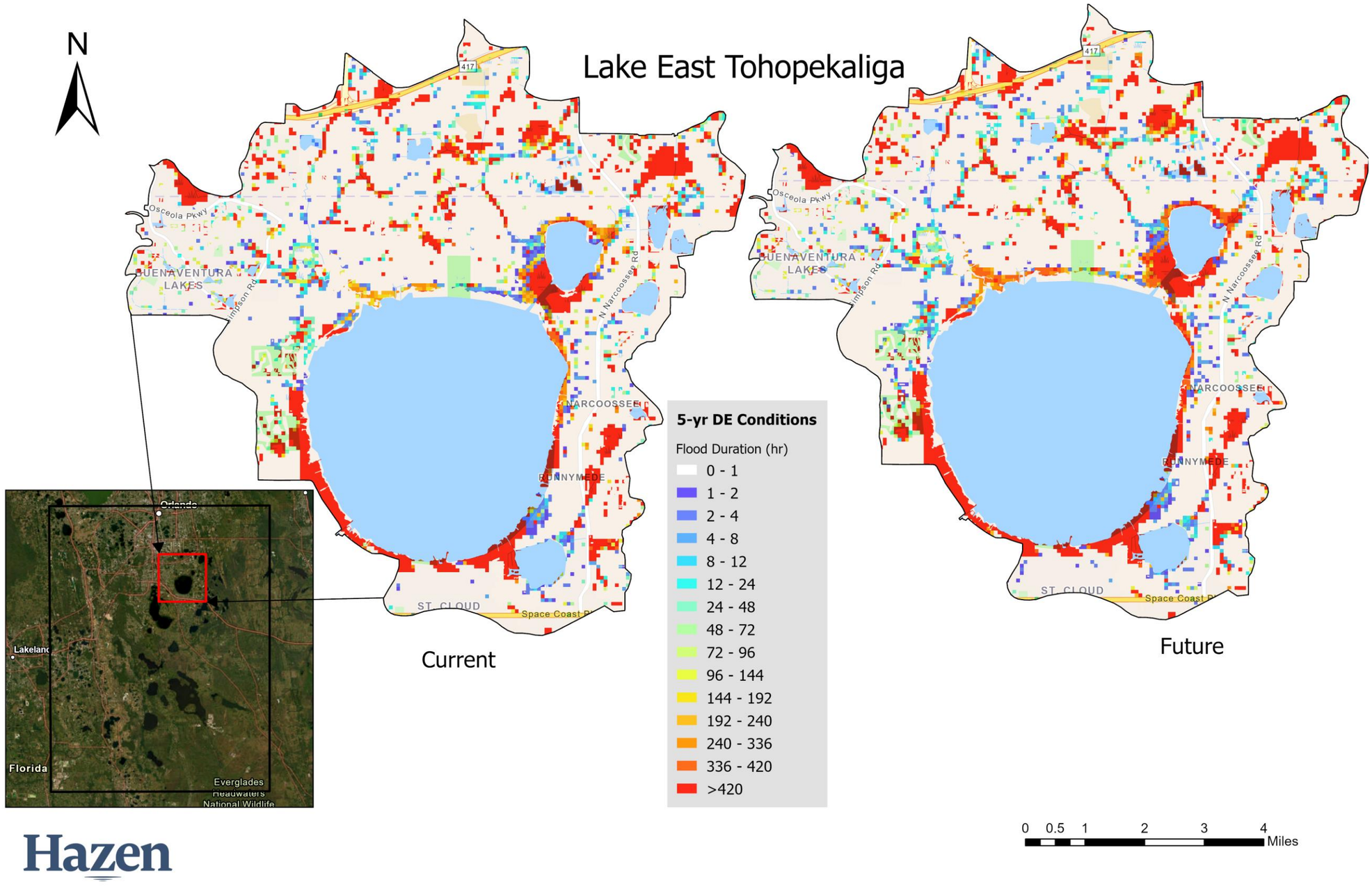
Hazen

Figure 12-49: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA



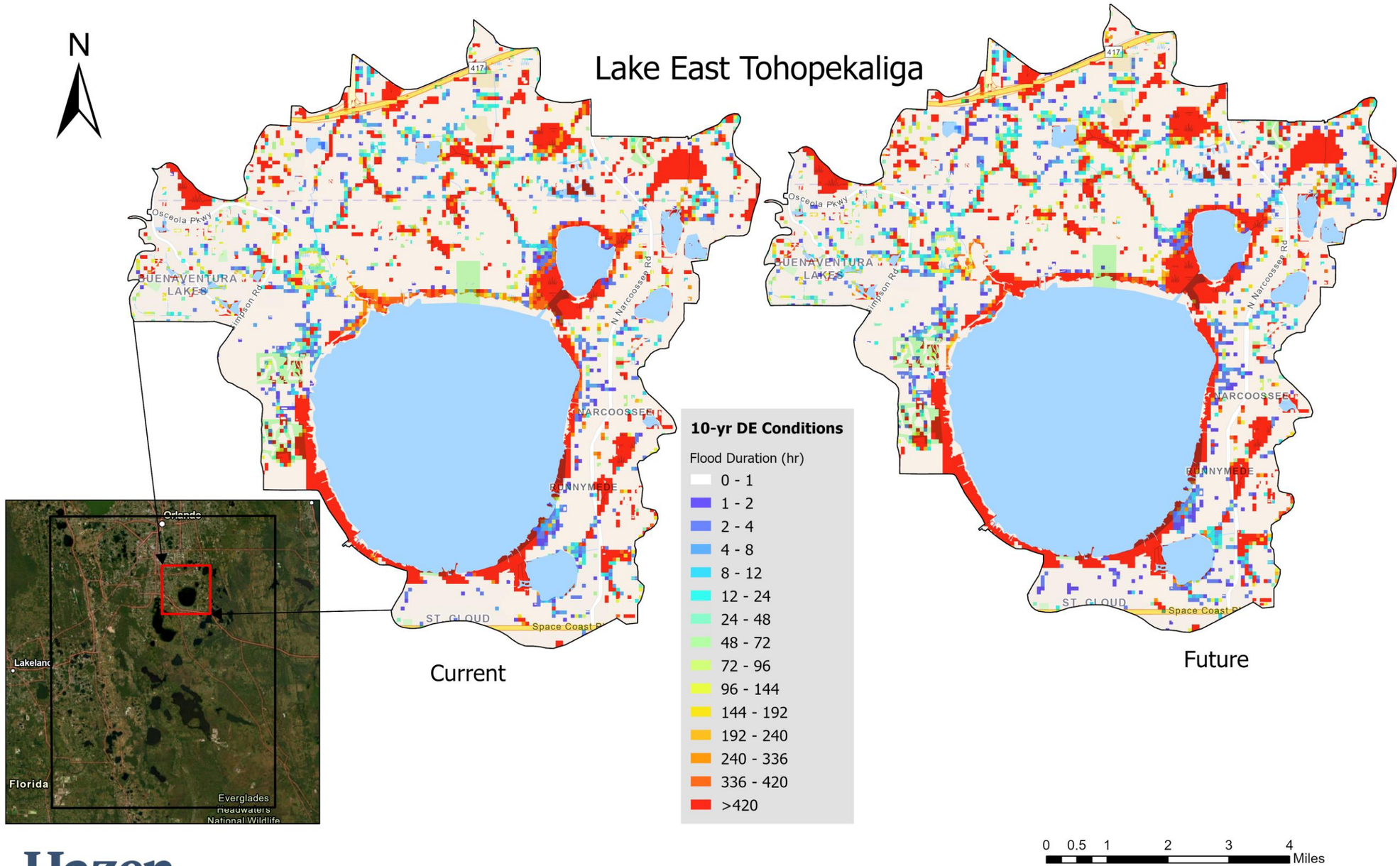
Hazen

Figure 12-50: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in East Lake Toho LMA



Hazen

Figure 12-51: Flood Duration Maps for 5-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions



Hazen

Figure 12-52: Flood Duration Maps for 10-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

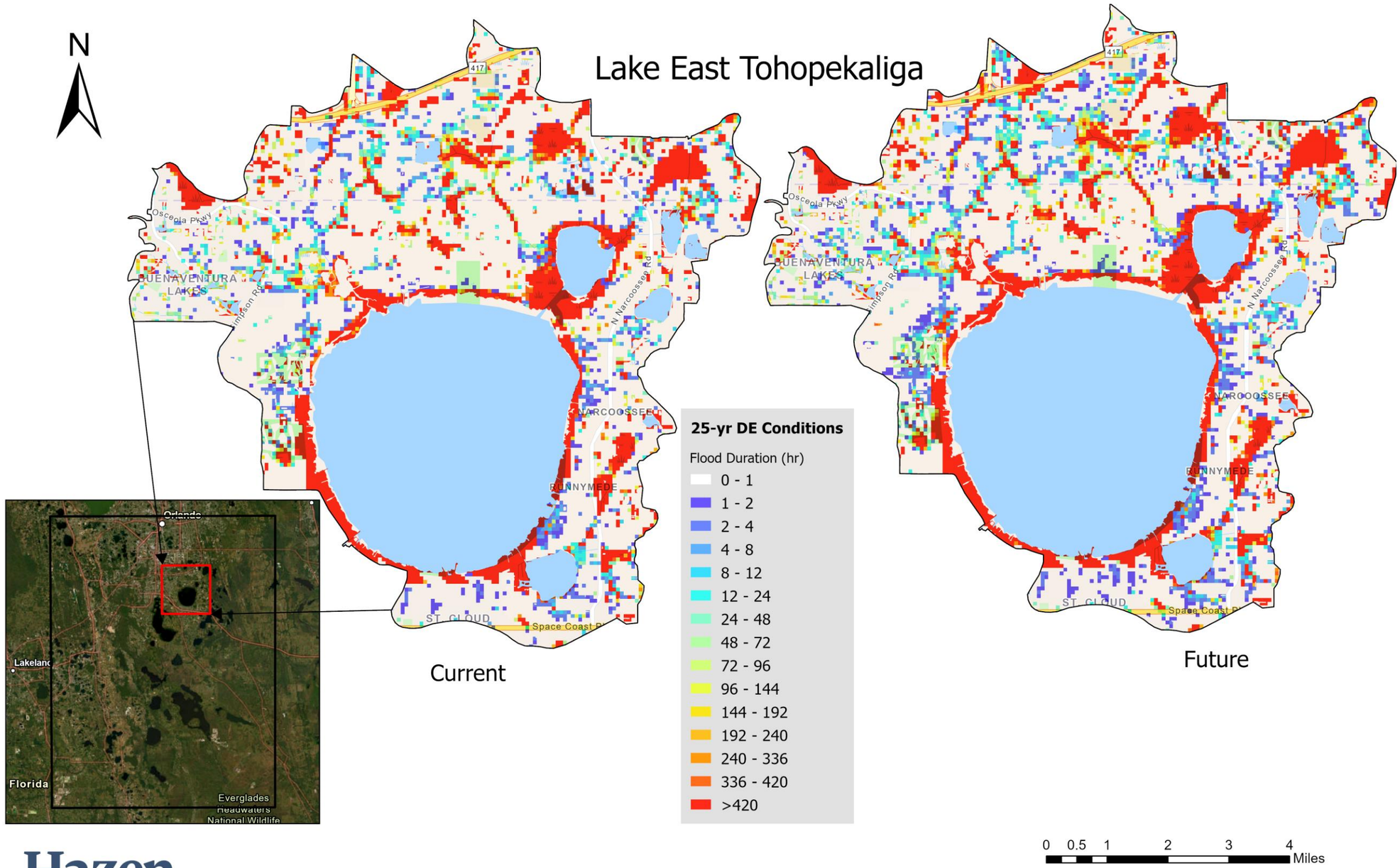


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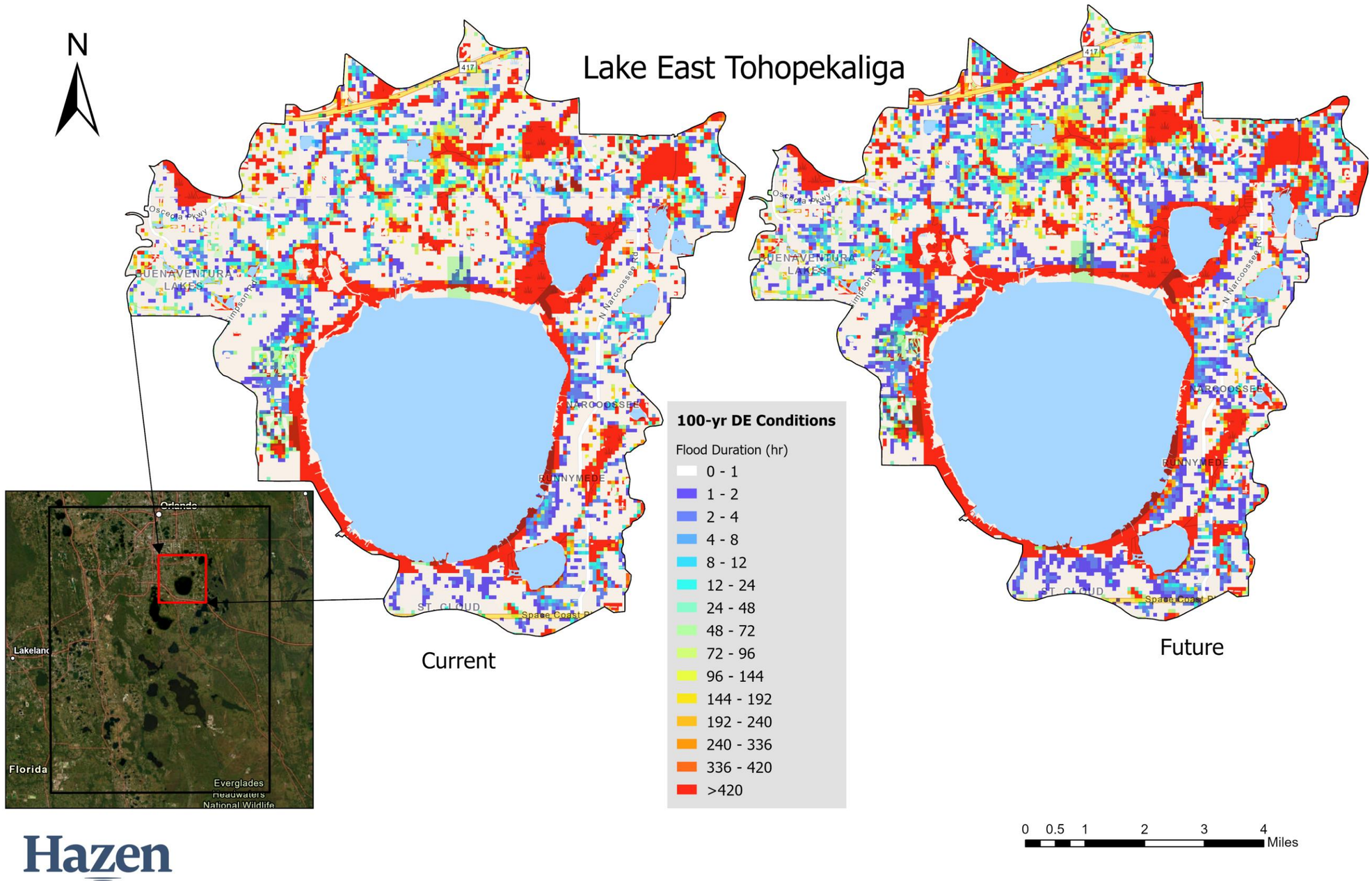
Figure 12-53: Flood Duration Maps for 25-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions



Hazen

Figure 12-54: Flood Duration Maps for 100-yr 3-day Design Storm for East Lake Toho LMA under Current and Future Conditions

Figure 12-55: Aerial Image #1 of Section 1 for East Tohopekaliga Lake LMA

Figure 12-56: Aerial Image #2 of Section 1 for East Tohopekaliga Lake LMA

Figure 12-57: Aerial Image of Section 3 for East Tohopekaliga Myrtle LMA

For the East Lake Tohopekaliga LMA, the banks of East Lake Tohopekaliga and Fells Cove experienced the most significant flood depths across all design events shown in **Figure 12-47 to Figure 12-50**. The three sections are shown in **Figure 12-36**—Bear Bay, Southeastern Oaks, and the area east of the lake—experienced flooding from the 5-year to the 100-year design events. While there was no noticeable difference in flood depths between current and future conditions for the 5-year and 10-year events, significant differences emerged for the 25-year and 100-year events, with future scenarios showing consistently higher depths showing the escalating flood risks in these areas under future conditions and more intense storms.

As observed in previous analyses, the banks of Lake East Tohopekaliga and the southern edges of Fells Cove consistently exhibited prolonged flood durations exceeding 420 hours. While the selected sections showed extended durations in only localized areas, the extent of flooding increased with the intensity of design events. Areas with less than an hour duration during the 5-year event gradually transitioned to average drainage times of under 4 hours as events intensified. However, the regions along the banks and those with durations surpassing 420 hours remain significantly more vulnerable to flooding than other areas. Future conditions consistently recorded higher flood durations, emphasizing the increasing flood risk under projected scenarios.

12.6 Lake Toho LMA

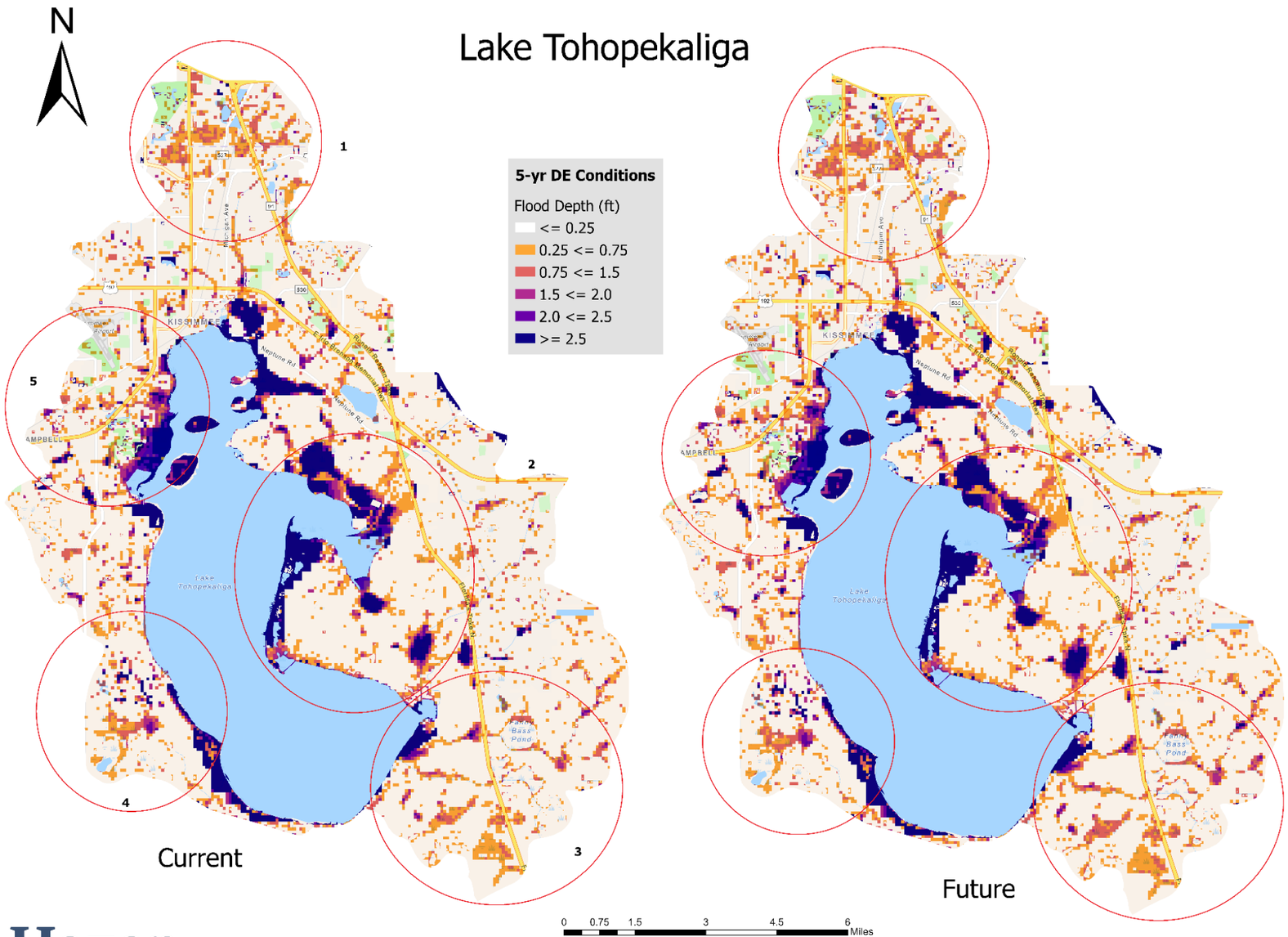
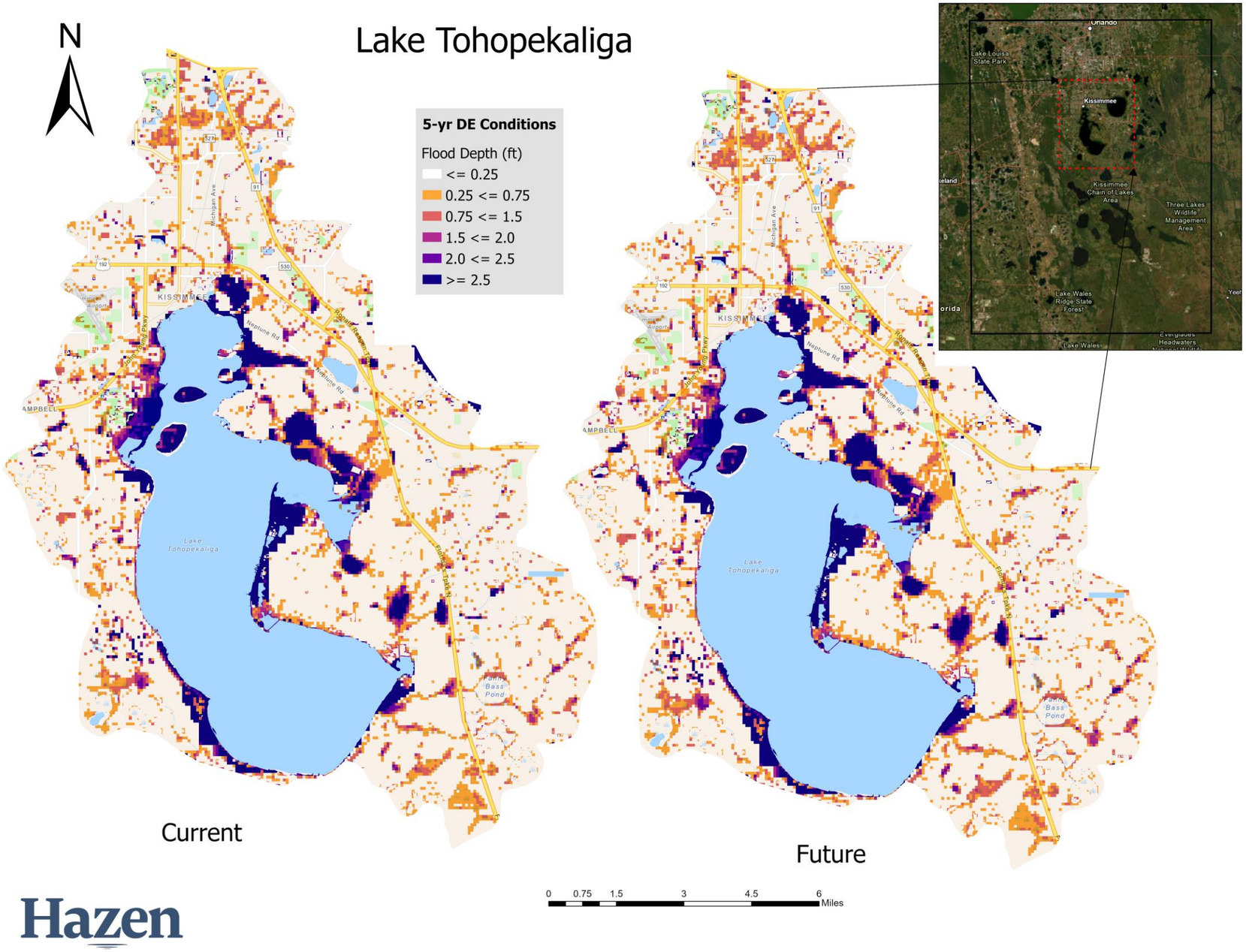
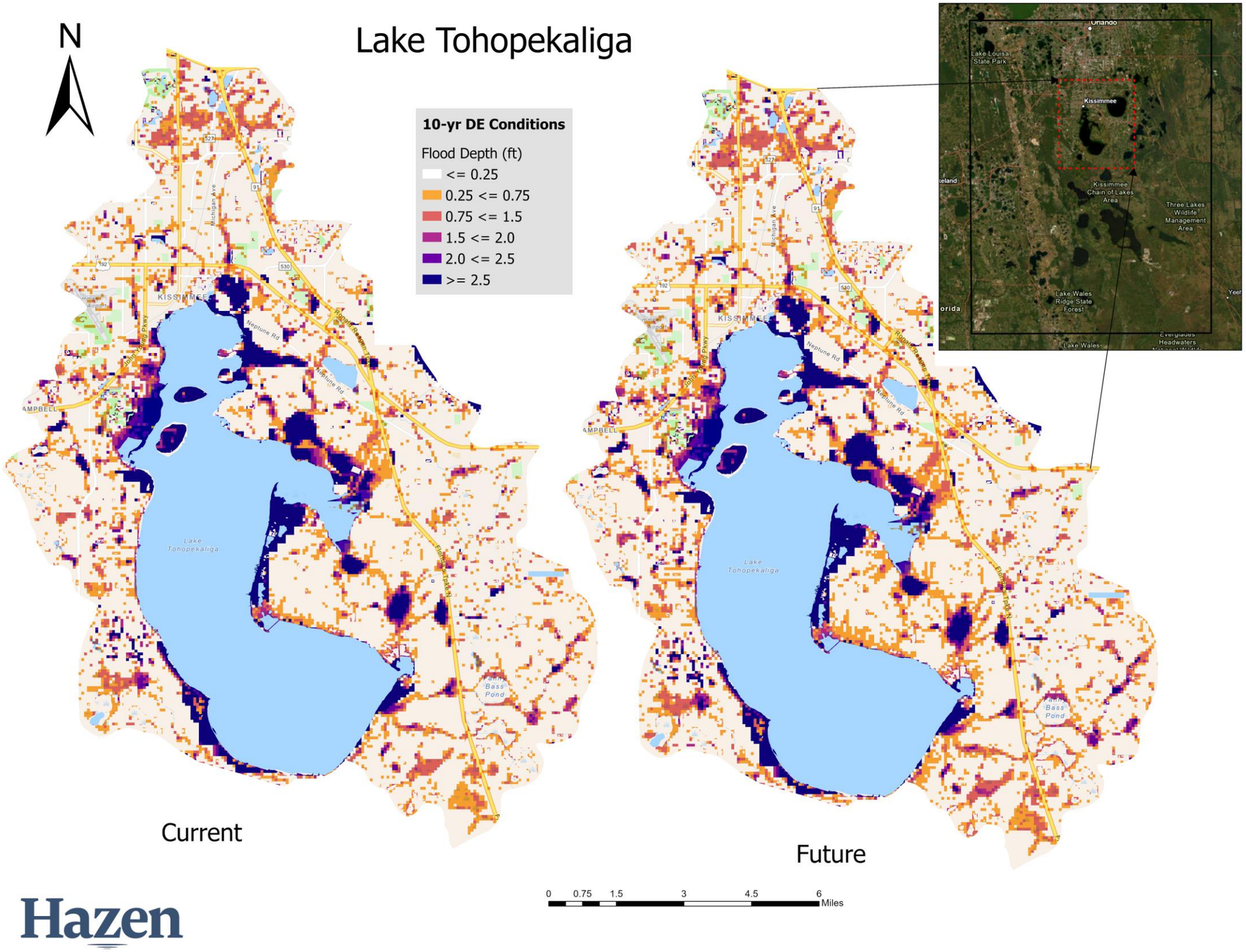


Figure 12-58: Lake Toho LMA in Sections



Hazen

Figure 12-59: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA



Hazen

Figure 12-60: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA

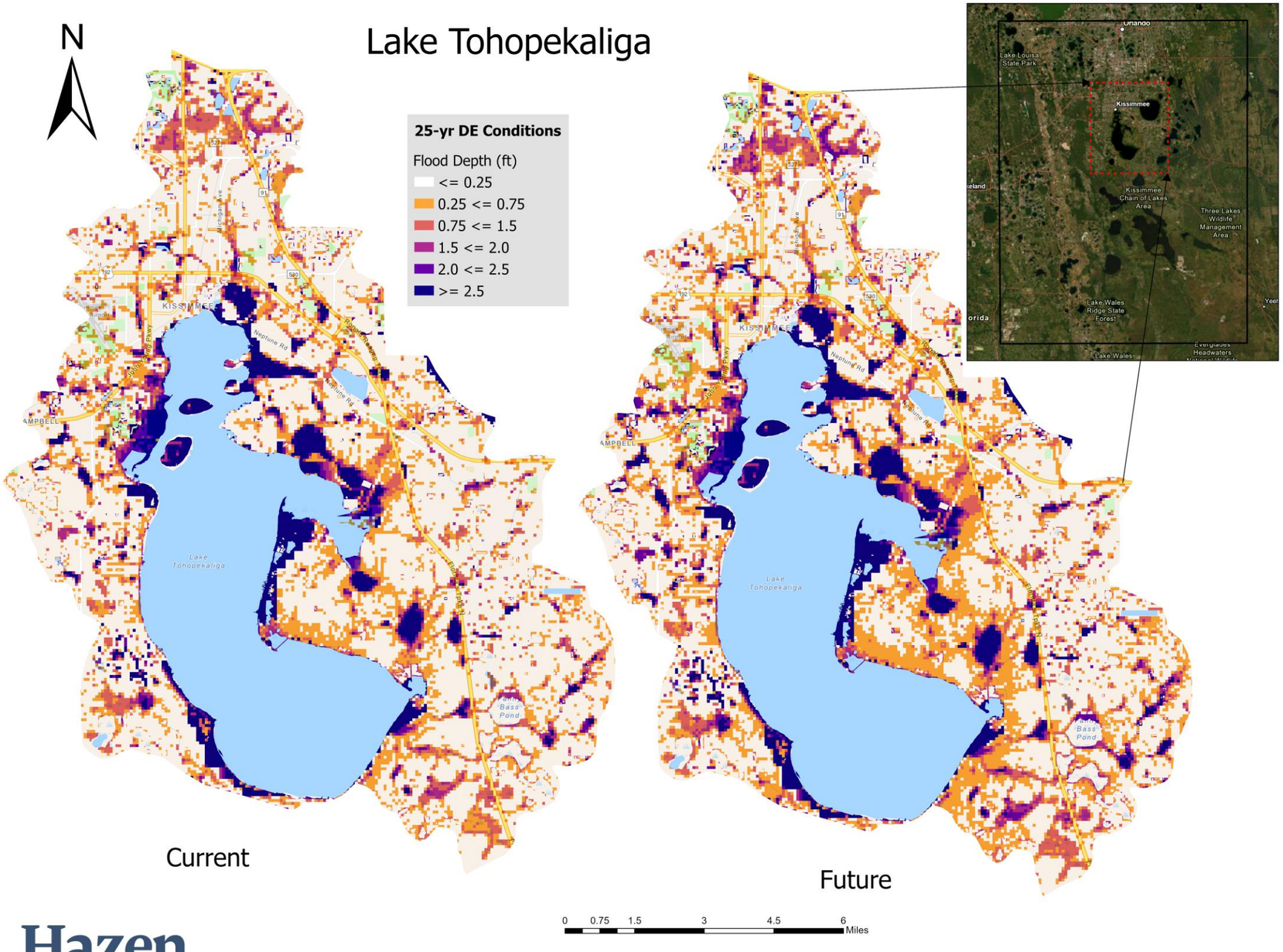


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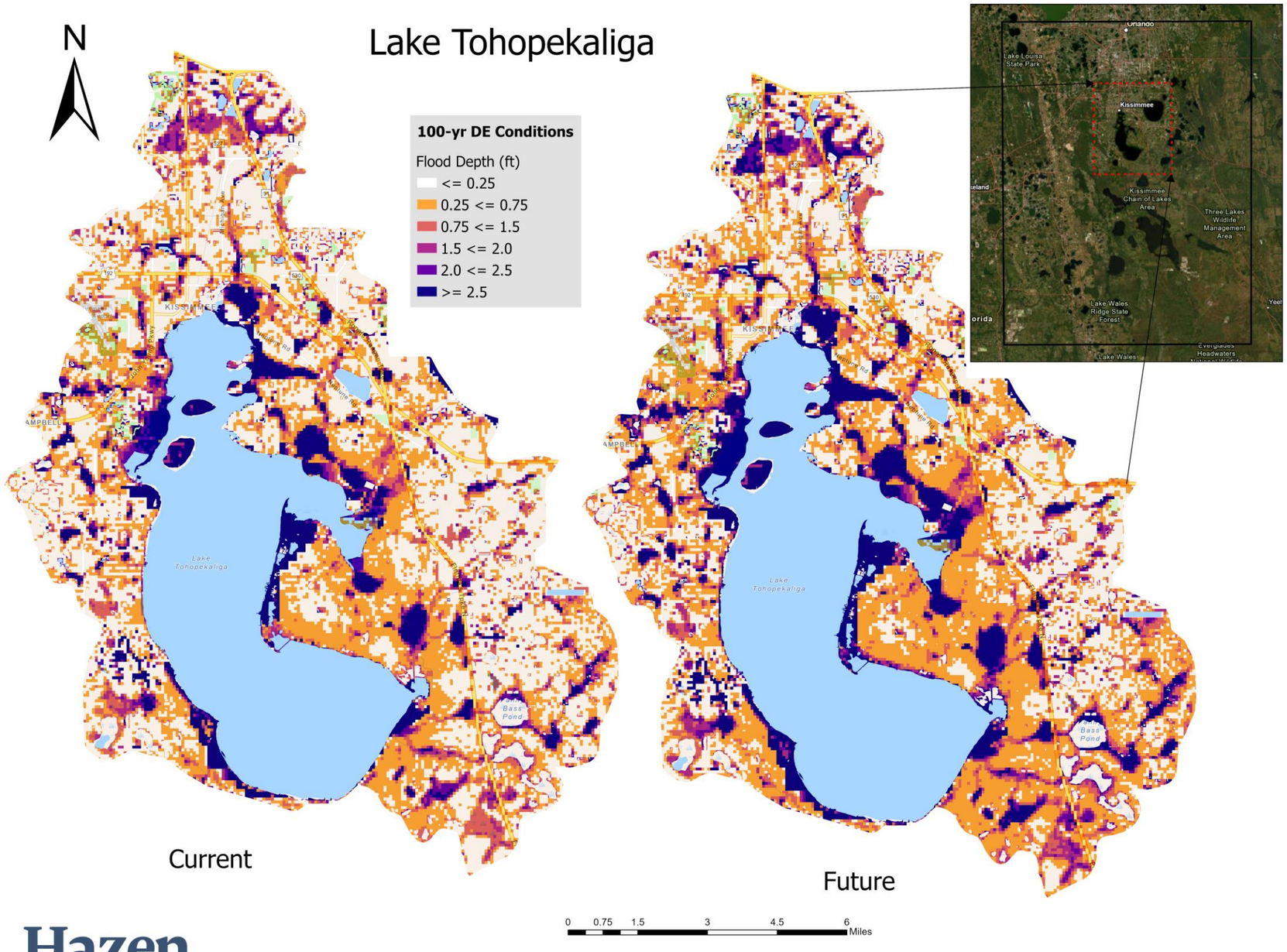
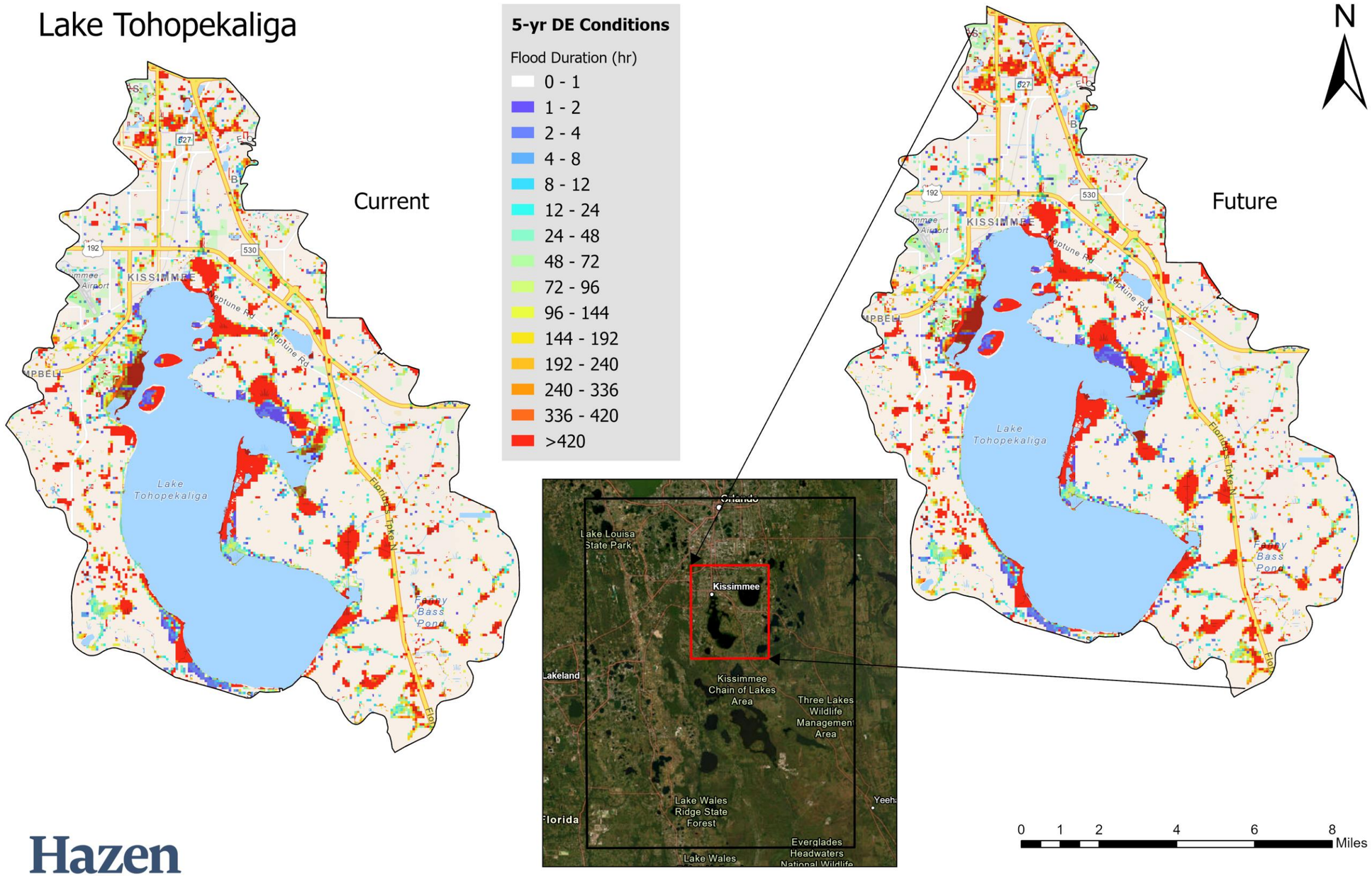
Figure 12-61: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA



Hazen

Figure 12-62: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in Lake Toho LMA



Hazen

Figure 12-63: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions

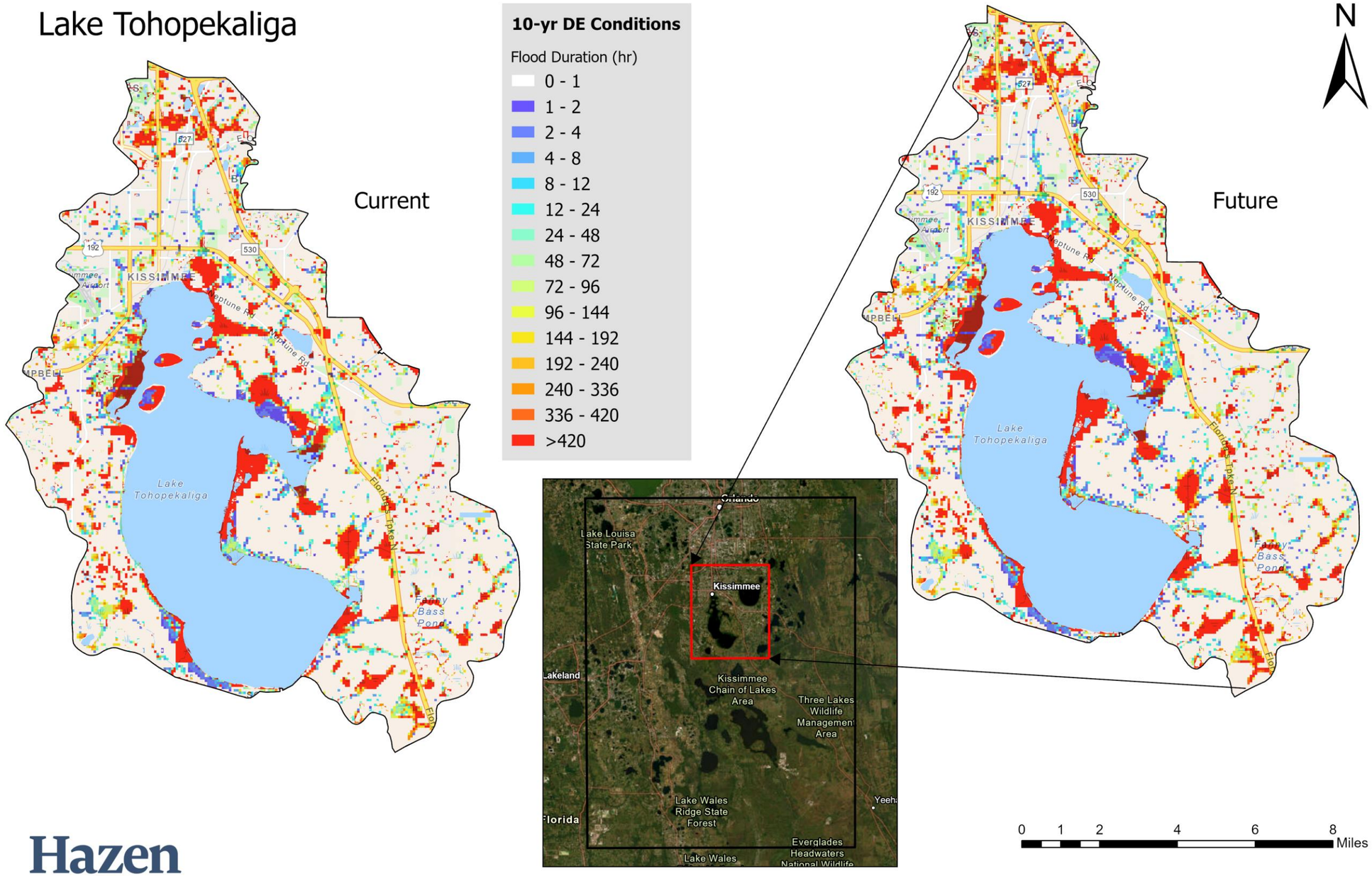


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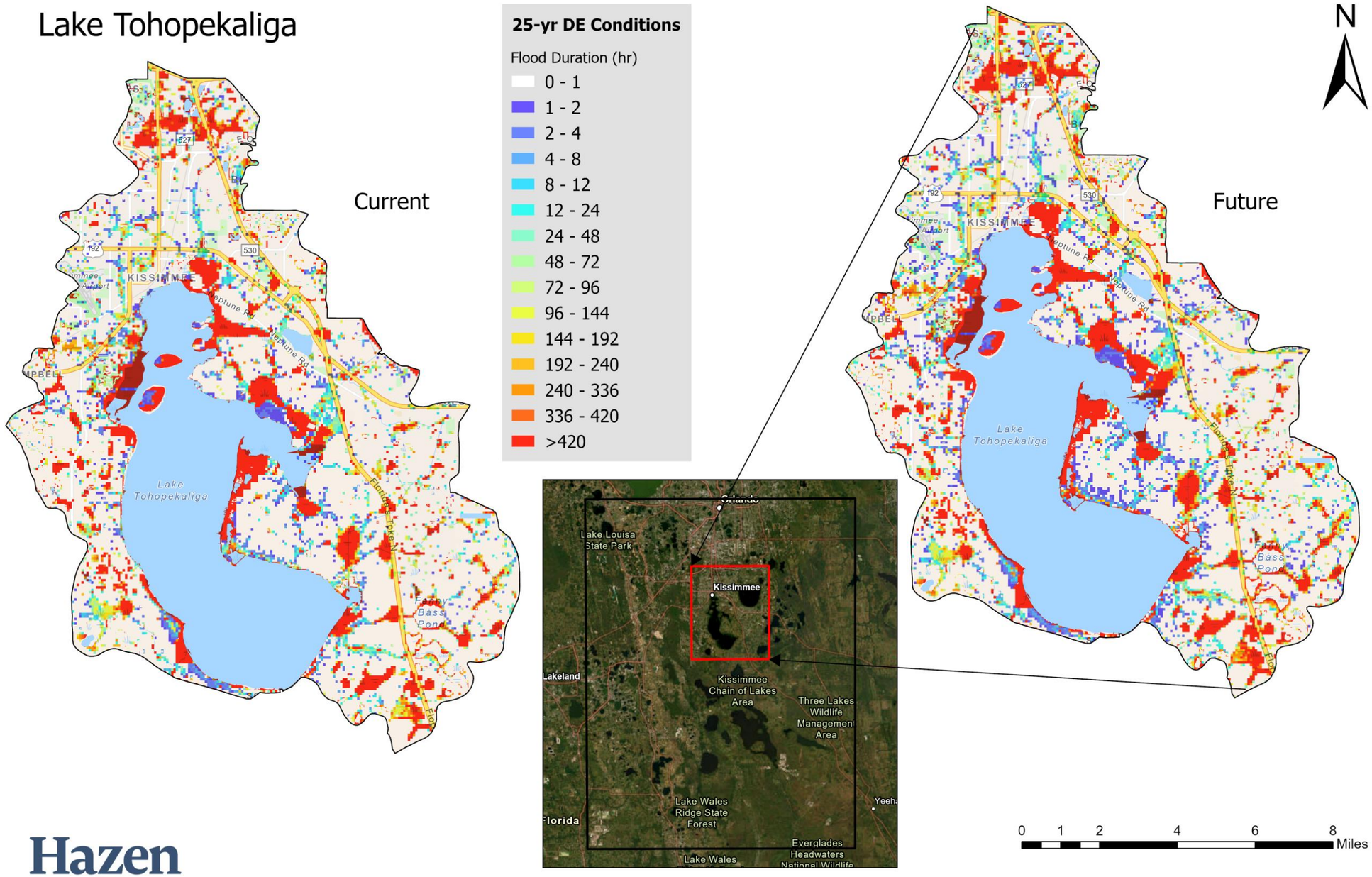
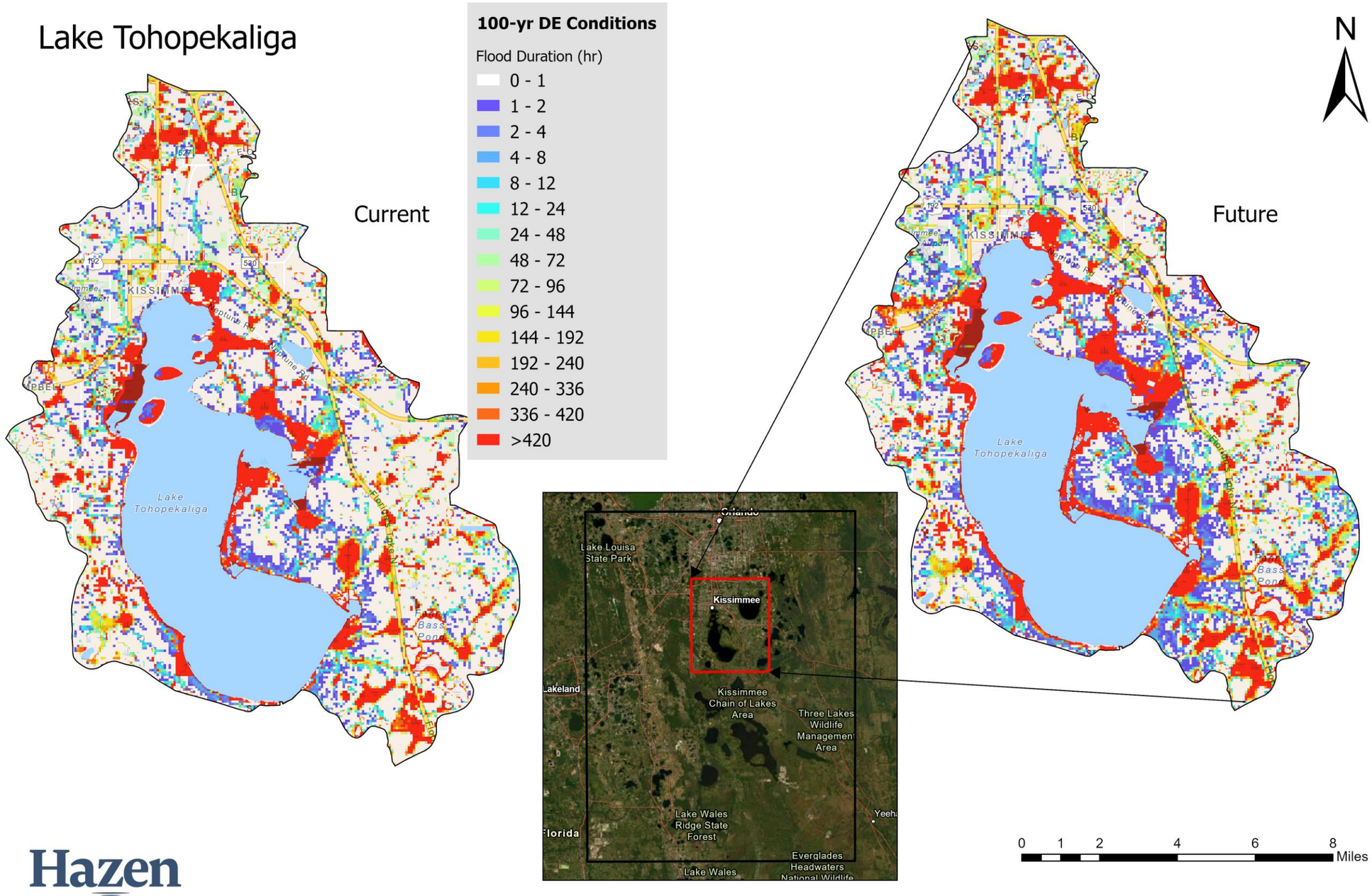
Figure 12-64: Flood Duration Maps for 10-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions



Hazen

Figure 12-65: Flood Duration Maps for 25-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions



Hazen

Figure 12-66: Flood Duration Maps for 5-yr 3-day Design Storm for Lake Tohopekaliga LMA under Current and Future Conditions

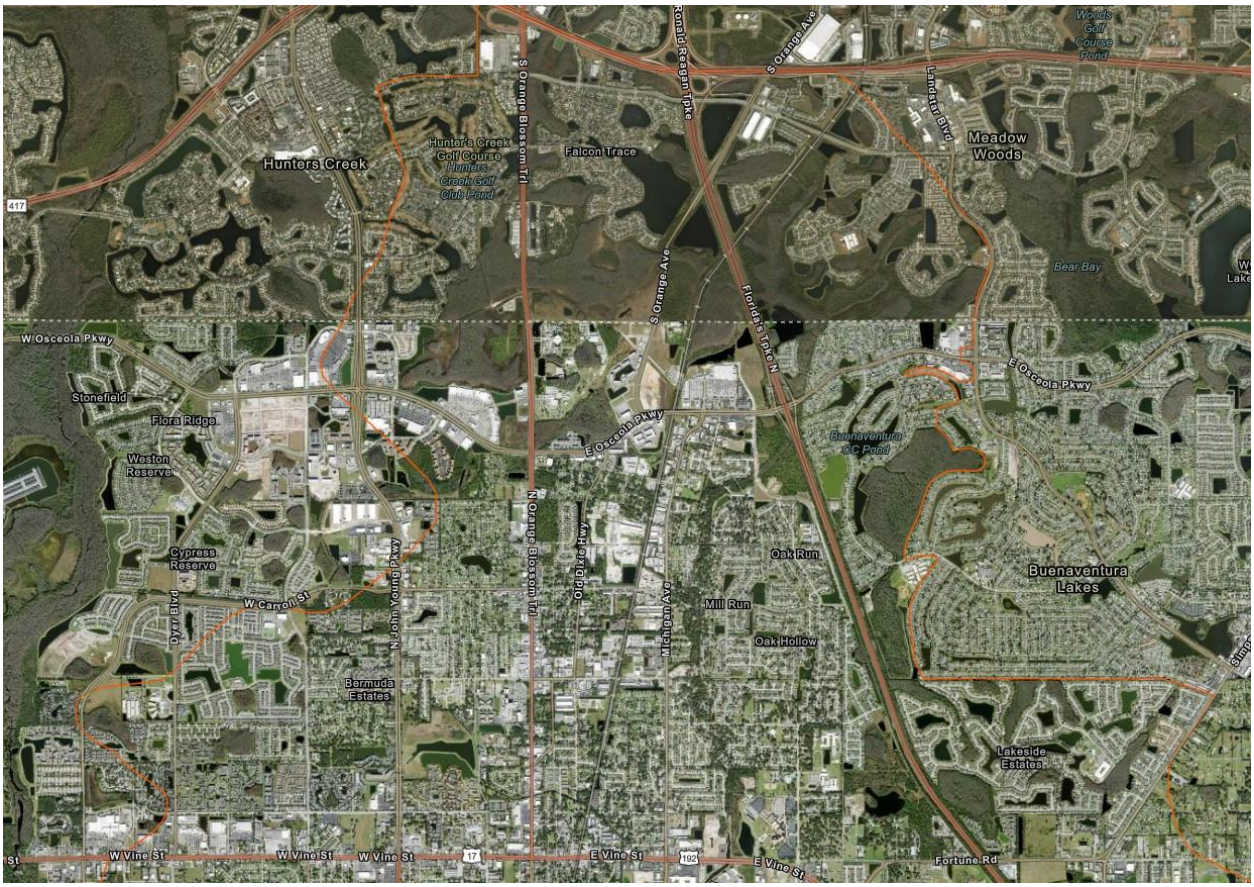


Figure 12-67: Aerial Image of Section 1 for Lake Toho LMA

Section 1 of Lake Tohopekaliga LMA as seen in **Figure 12-58** focuses on the areas north of Lake Tohopekaliga. During the 5-year design event, flood depths ranged between 0.25 and 0.75 feet in the northern parts of the section, with a slight increase in intensity under future conditions. As the design events intensified from the 5-year to the 100-year storm, flood depths progressively increased across the same areas. For all design events, future conditions consistently exhibited higher flood depths than current conditions.

PM6 demonstrated consistent patterns in flood depth and drainage times. Areas with higher flood depths during the 5-year design event experienced prolonged drainage times exceeding 420 hours, a trend that persisted across all design events. Meanwhile, regions with minimal drainage times during the 5-year event saw a gradual increase in duration as the design events intensified, culminating in an average drainage time of less than 4 hours. This progression highlights the dynamic relationship between flood depth and duration across varying event intensities. Future conditions had higher durations for the 100-year event.

Figure 12-68: Aerial Image of Section 2 for Lake Toho LMA

The area east of Lake Tohopekaliga and west of the Florida Turnpike experienced significant flood depths along much of the lake's banks and portions of its central area during all design events. In the surrounding areas, flooding became more pronounced as the intensity of the design events increased. Across all events, future conditions consistently exhibited greater flood depths than current conditions.

The flood duration along the lake banks aligned closely with the patterns of flood depth previously discussed. Small portions within central areas consistently exhibited prolonged flood durations exceeding 420 hours across all design events. As design events intensified, areas with initially low flood durations gradually increased, with the maximum observed duration remaining below 4 hours. Notably, future conditions consistently demonstrated higher flood durations compared to current conditions, pointing out their increased vulnerability.

Figure 12-69: Aerial Image of Section 3 for Lake Toho LMA

In this section (Figure 12-58), the banks of Lake Tohopekaliga experienced the most significant flooding across all design events. Consistent with the trends observed in previous sections, flooding within this area intensified with increasing storm severity, reaching its peak during the 100-year design event. For all events, future conditions showed a noticeable rise in flood depths compared to current conditions.

Flood duration was strongly correlated with areas of higher flood depths, with both metrics increasing progressively as design events increased. Future conditions exhibited longer flood durations compared to current conditions for the 25 and 100-year events, highlighting the heightened vulnerability under future scenario.

Figure 12-70: Aerial Image of Section 4 for Lake Toho LMA

Section 4 as seen in Figure 12-58 focuses on the southwest corner of Lake Tohopekaliga, where, as expected, the banks experienced the most significant flooding. Additionally, wetlands in the central and western parts of the section also saw notable flooding. As the intensity of the design events increased, flooding expanded outward from the areas of highest intensity. For all design events, future conditions consistently exhibited greater flood depths than current conditions.

Flood duration in this section was generally low, except for small areas along Lake Tohopekaliga's banks and the central region's wetlands. As design events intensified, the duration increased gradually, reflecting a predictable escalation with event severity. Differences between future and current conditions were minimal, but areas with prolonged durations remain particularly vulnerable across all design events.

Figure 12-71: Aerial Image of Section 5 for Lake Myrtle LMA

Like other sections of Lake Tohopekaliga, Section 5 shows the banks along the northwestern part of the lake experienced the highest flooding. While there was no significant difference in flood depths between current and future conditions across all design events for this section, flood depths consistently increased with the intensity of the design events.

PM6 aligned with trends observed across other sections within the LMA. Areas with greater flood depths, particularly along the banks, experienced prolonged flood durations exceeding 420 hours. These durations extended outward from the banks with increasing intensity, highlighting a strong correlation between depth and duration. Notably, aside from the 100-year design event, no significant differences in duration intensity existed between current and future conditions.

12.7 Lake Cypress, Hatchineha, Kissimmee LMA

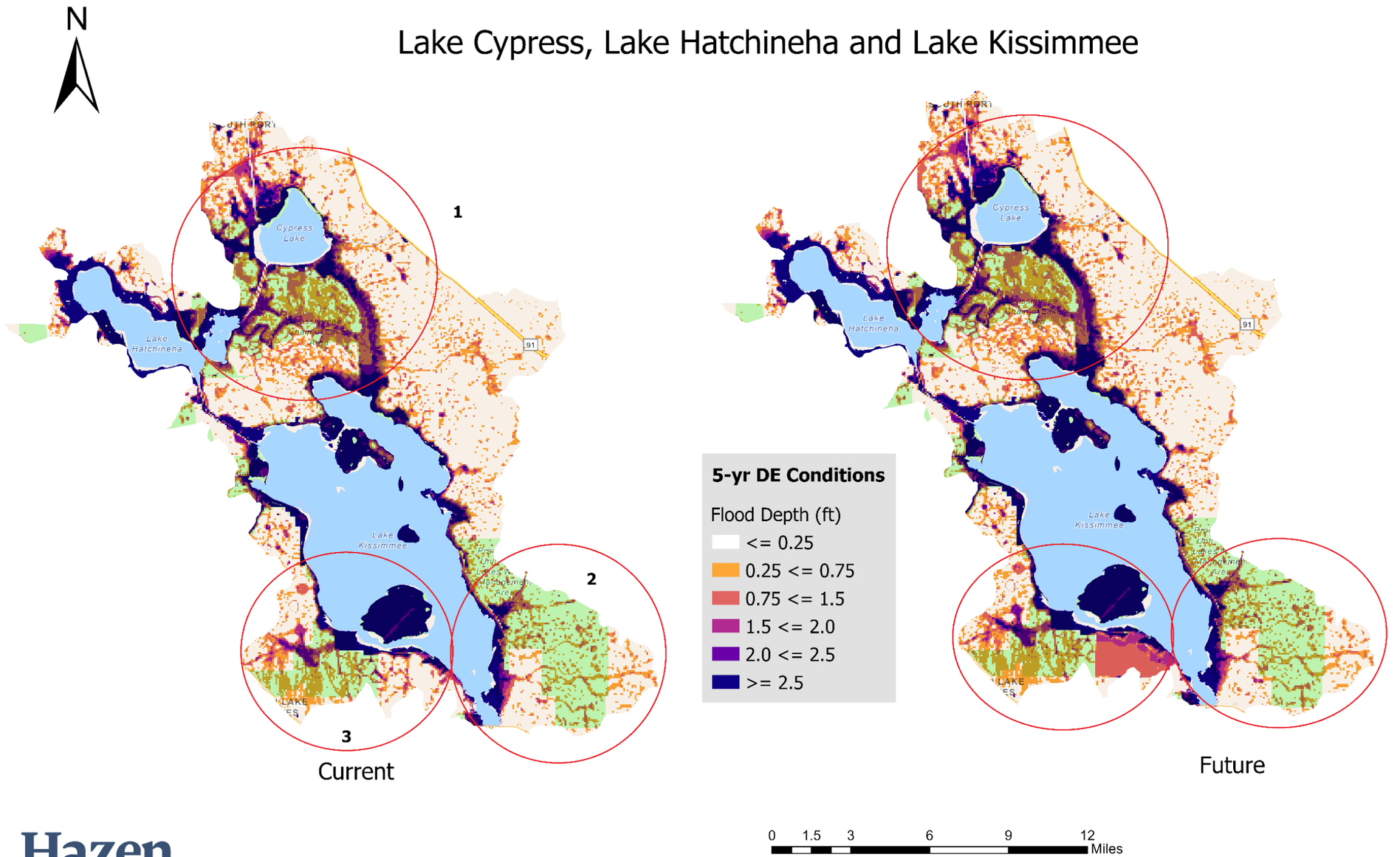
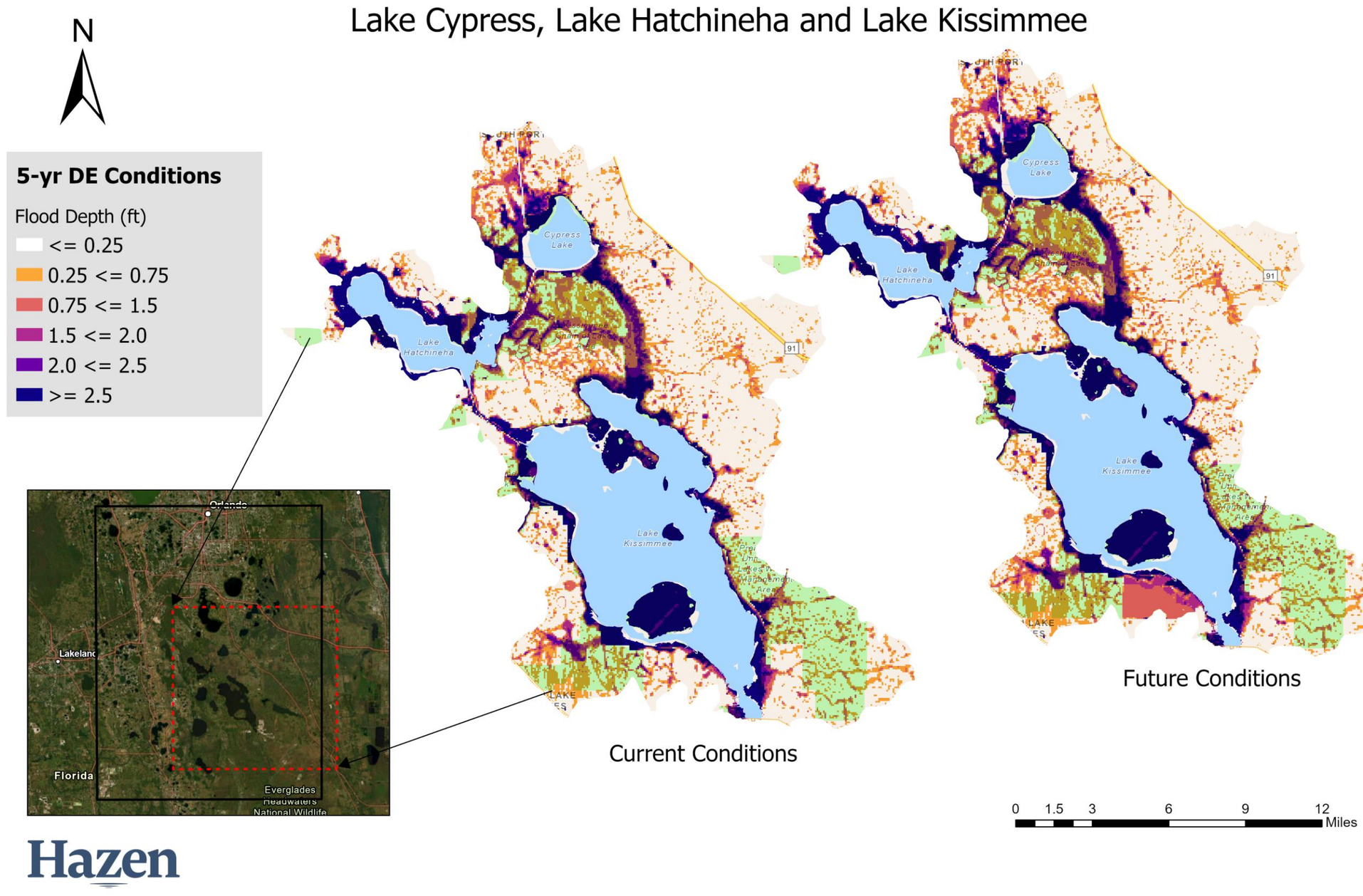
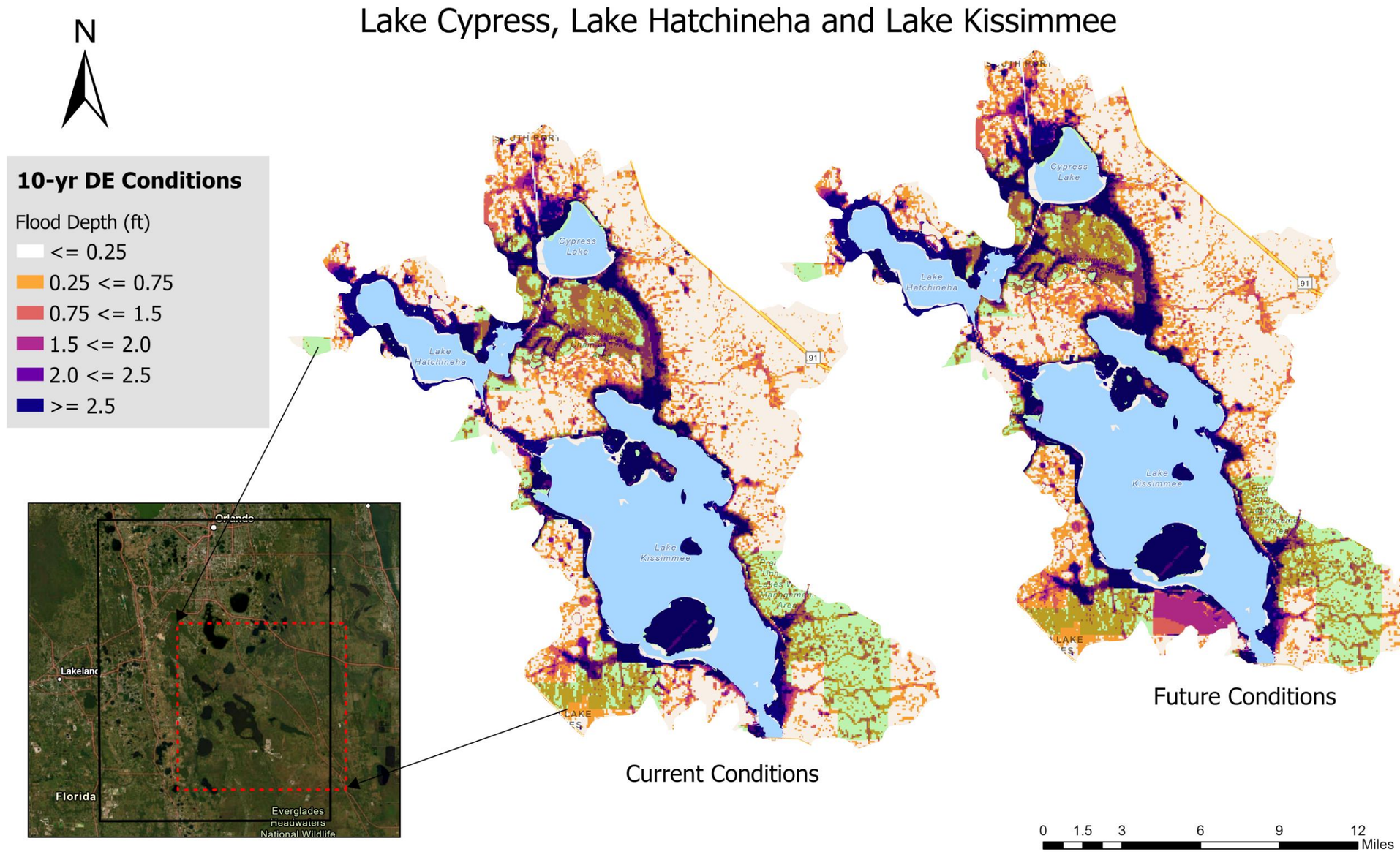


Figure 12-72: Lake Cypress, Hatchineha, and Kissimmee LMA in Sections



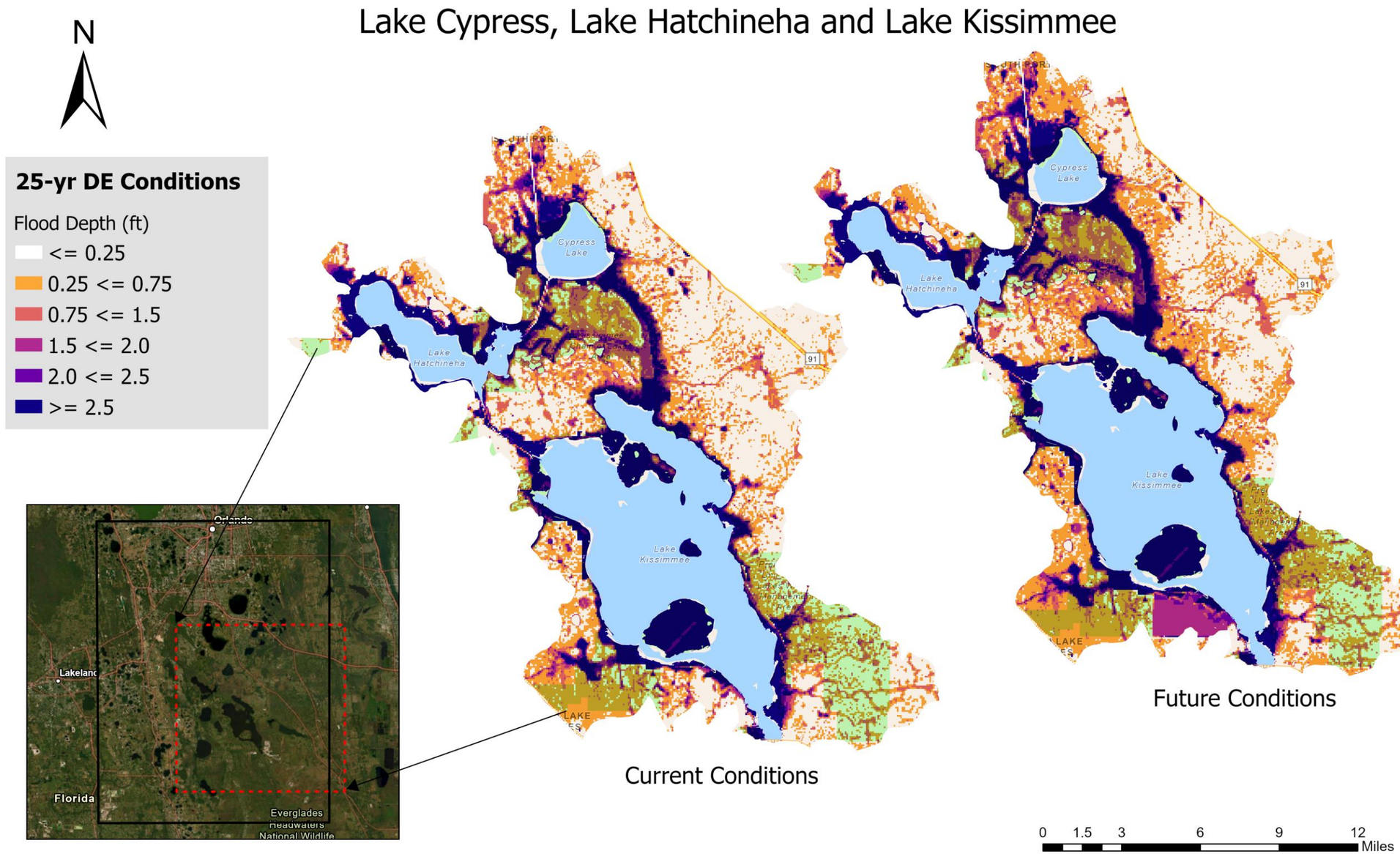
Hazen

Figure 12-73: Maximum Overland Flood Depth for 5-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



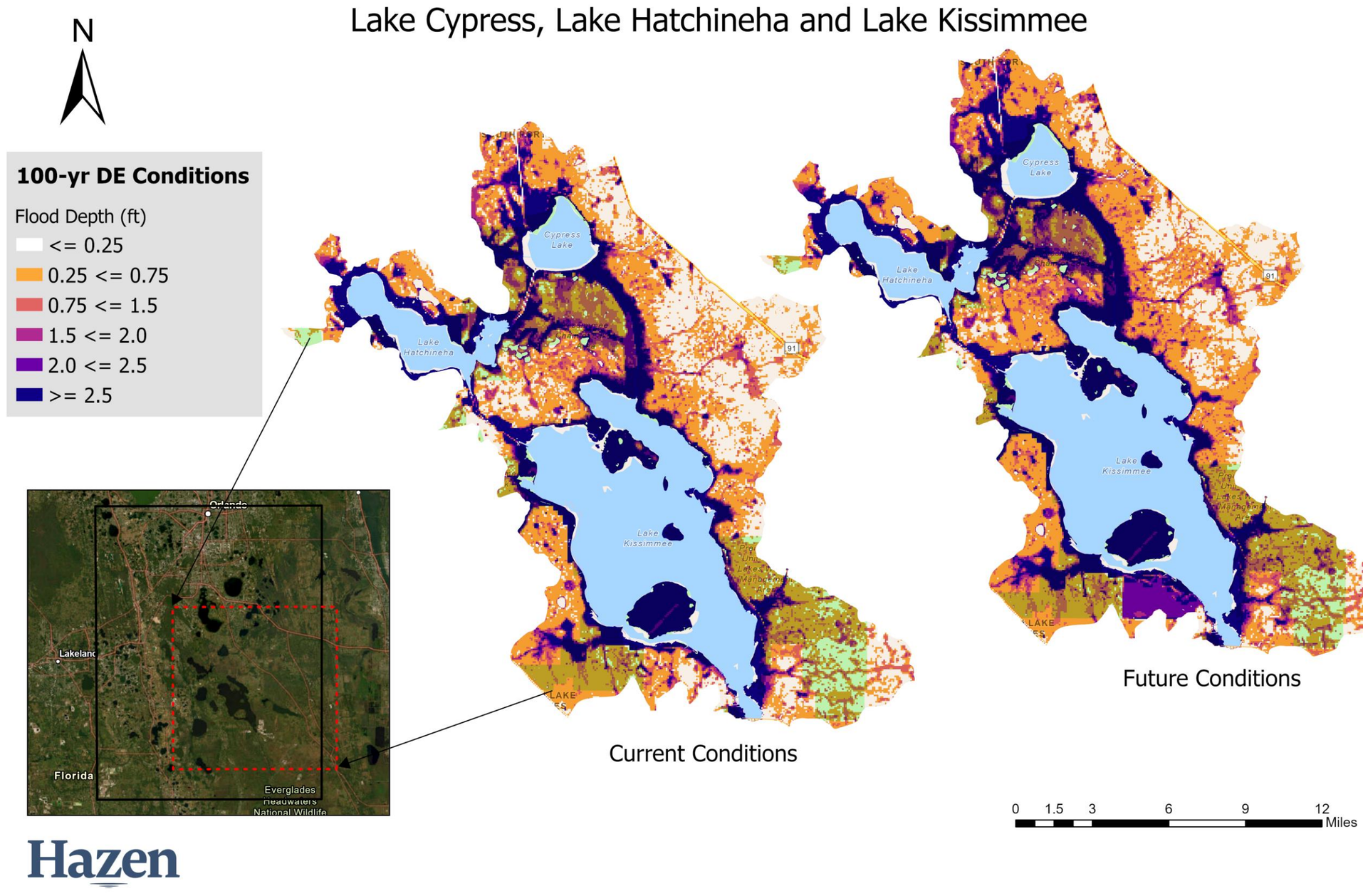
Hazen

Figure 12-74: Maximum Overland Flood Depth for 10-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



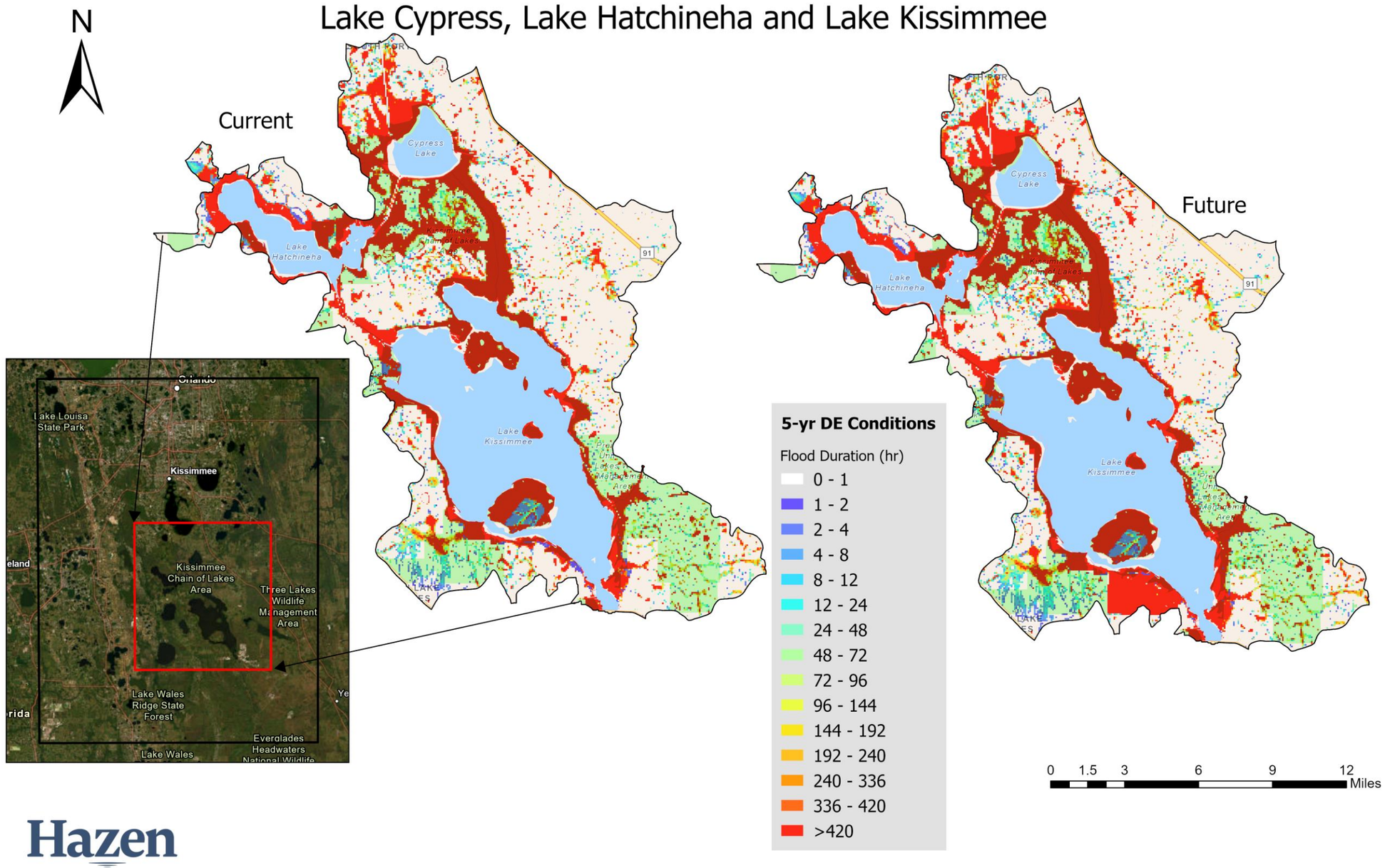
Hazen

Figure 12-75: Maximum Overland Flood Depth for 25-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



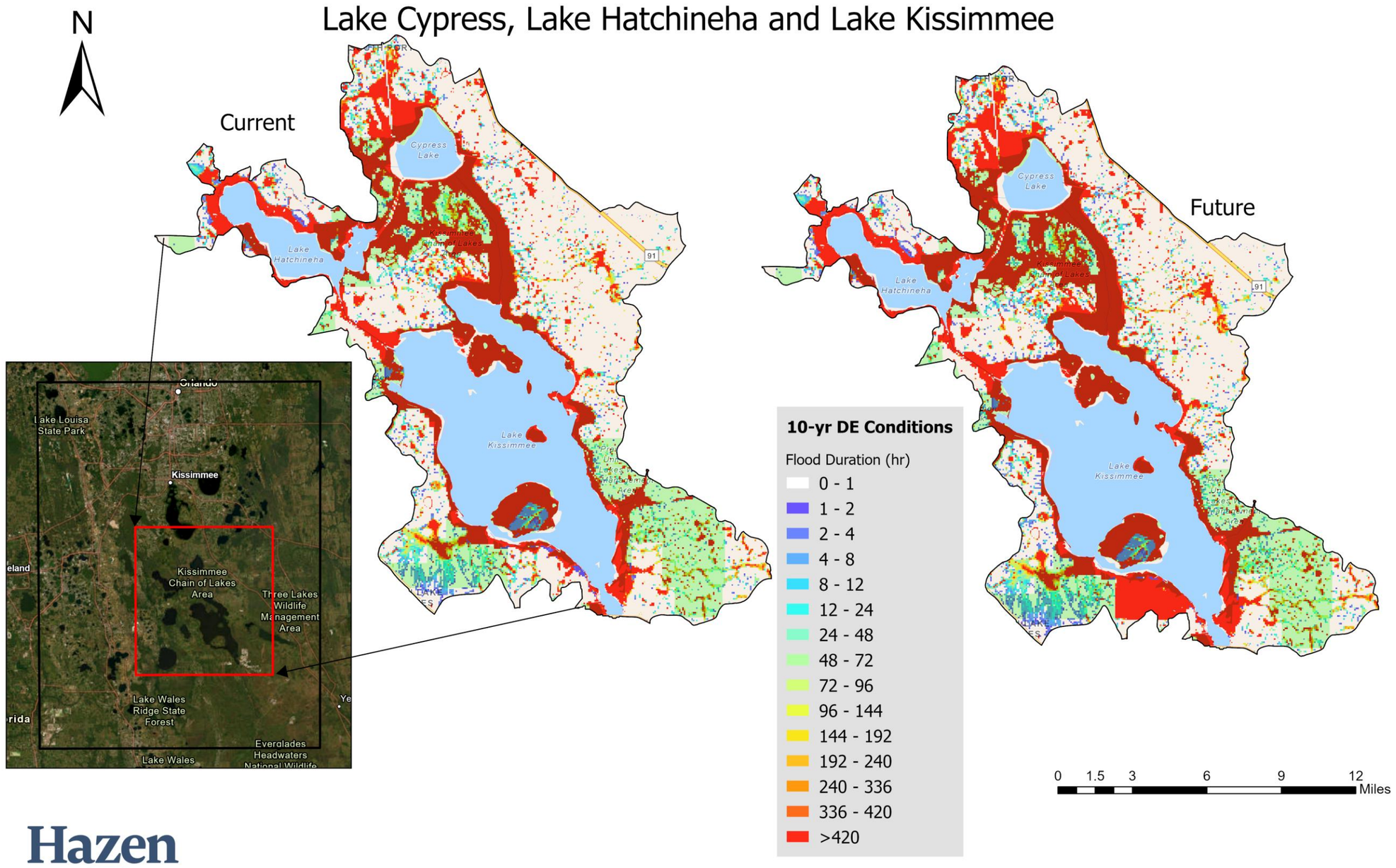
Hazen

Figure 12-76: Maximum Overland Flood Depth for 100-yr 3-Day Design Storm under Current and Future Conditions in lakes Kissimmee-Cypress-Hatchineha LMA



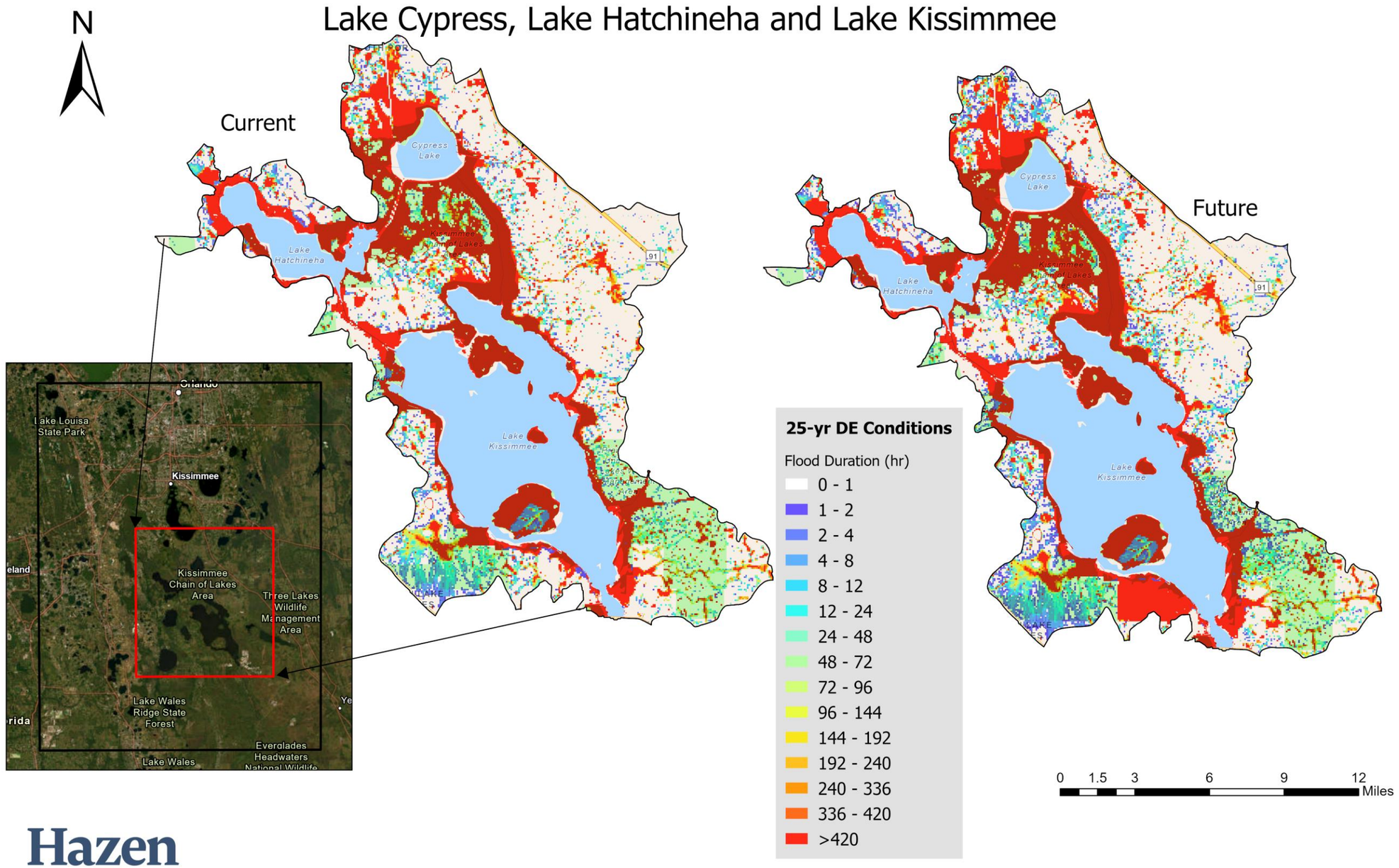
Hazen

Figure 12-77: Flood Duration Maps for 5-yr 3-day Design Storm for KCH System under Current and Future Conditions



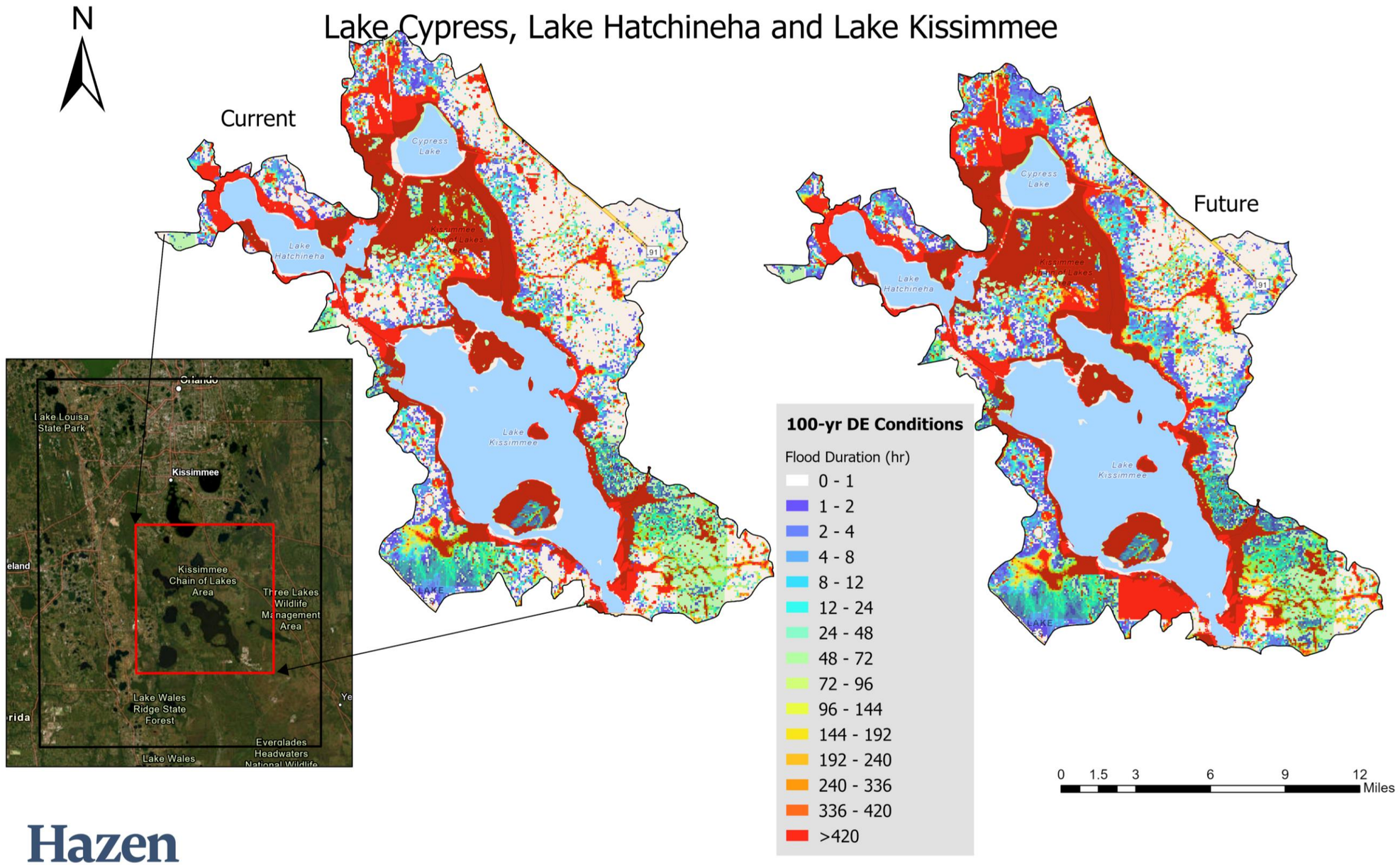
Hazen

Figure 12-78: Flood Duration Maps for 10-yr 3-day Design Storm for KCH System under Current and Future Conditions



Hazen

Figure 12-79: Flood Duration Maps for 25-yr 3-day Design Storm for KCH System under Current and Future Conditions



Hazen

Figure 12-80: Flood Duration Maps for 100-yr 3-day Design Storm for KCH System under Current and Future Conditions

Figure 12-81: Aerial Image of Section 1 for Lake Cypress, Hatchineha, Kissimmee LMA

For all design events, Cypress, Hatchineha, and Kissimmee LMA experienced significant flood depths along the lakes and canal banks. As a result, this analysis focuses on areas beyond the banks for these three sections seen in **Figure 12-72**.

Section 1 centers on the island between the three lakes, primarily surrounding the Kissimmee Chain of Lakes. In this area, flood depths gradually extended inward from the canal banks, with flooding becoming more pronounced as the design events intensified. A slight increase in flood intensity was observed when comparing current conditions to future scenarios.

This section exhibited significant flood durations, exceeding 420 hours, particularly from the canal banks extending inland. As the design events intensified toward the 100-year event, the Kissimmee Chain of Lakes experienced flood durations exceeding 420 hours across approximately 90% of the area. Regions farther from the lake banks began to show increasing durations, reaching up to 4 hours as the design events progressed. Additionally, future conditions consistently exhibited higher flood durations than current conditions, emphasizing the growing impact of intensified design events on the region's flood vulnerability.

Figure 12-82: Aerial Image of Section 2 for Lake Cypress, Hatchineha, Kissimmee LMA

Section 2 focuses on the area between Lakes Kissimmee, Jackson, and Marian. As in Section 1 above, flooding in this area intensified with increasing design events, progressing gradually from the banks further inland. Future conditions consistently exhibited higher flood intensities compared to current conditions.

Flood durations increased progressively with the increase of design events, with future conditions consistently showing higher durations than current conditions. Areas along the banks experienced the most prolonged durations, with the 5-year design event already surpassing 420 hours. For regions farther from the lake, flood durations rose to an average of 75 hours as design events became more severe, highlighting the expanding impact of flooding beyond the immediate banks under intensified conditions.

Figure 12-83: Aerial Image of Section 3 for Lake Cypress, Hatchineha, Kissimmee LMA

This section examines the southern portion of Lake Kissimmee, particularly around Otter Slough. During the 5-year design event, the central part of Otter Slough experienced higher flood depths, with little to no difference between current and future conditions in these areas. However, moving east of Otter Slough, only minimal flooding was observed under current conditions, while future conditions showed a notable increase in flood depth for the same area. Flood depths consistently rose with the intensity of design events, with a similar degree of difference between future and current conditions across all areas within this section.

Flood durations in this section mirror flood depth as stated above. During the 5-year design event, the middle portions of Otter Slough had durations exceeding 420 hours, consistent with areas of higher flood depths. Areas east of Otter Slough had minimal flood durations for current conditions, whereas the future had a sharp increase in flood durations.

As the design events intensified, flood durations expanded in extent and severity. Areas initially experiencing shorter durations began to observe averages of 24 hours or more under the 25-year and 100-year design events. Future conditions consistently revealed higher durations compared to current conditions, showing this section is vulnerable even for lower design events, but it’s more vulnerable to future conditions.

12.8 Unmanaged Watersheds

The unmanaged watersheds within the jurisdiction of the SFWMD refer to areas that drain into creeks not regulated by water control structures. These watersheds are equipped with stage and/or flow recorders that monitor surface water levels and flows; and were included in the calibration dataset for this project. The creeks subsequently flow into LMAs controlled by a primary SFWMD structure.

The unmanaged watersheds within the UKBFPLOS Study Area, including Boggy Creek, Reedy Creek, and Shingle Creek watersheds, were included in the LMA analysis. **Appendix M** contains PM #1 profiles, PM #5 and PM #6 maps for the entire model domain, as well as detailed maps for the USGS watersheds of Boggy, Shingle, and Reedy Creek. The sections assessing the FPLOS in each LMA in this report include calculations of the inundation depths and duration of inundation for these watersheds. This appendix also provides maps of the entire KB.

13. Summary and FPLOS

The UKB FPLOS Model Study Area covers 26 watersheds in the UKB, spanning roughly 1600 sq. mi., while the model domain extends to about 3560 sq. mi., accounting for hydrology and reducing boundary effects by representing surface and subsurface water exchanges. This domain crosses 3 water management districts and uses spatial datasets at 100 m resolution. The ECFTX model under the Central Florida Water Initiative (CFWI) provided hydrostratigraphy and saturated zone properties for the Surficial and Upper Floridan aquifers, with NAVD88 as the vertical datum. Calibration involved Hurricane Irma (Sept 9-11, 2017), and validation used the No Name storm (Oct 8-11, 2011). Calibration ensured accuracy in representing peak stages and flows, crucial for evaluating flood protection by existing infrastructure. Following successful calibration and independent event evaluation, the model served as the basis for base condition assessments.

Evaluation of the FPLOS for the UKB watersheds in central Florida was performed using an H&H model application of MIKE Suite of software was calibrated and validated and then modified for design storm analysis of existing and future conditions. The calibration and validation processes were extensive and thorough, given these complexities of the system:

- Highly managed surface water network
- Interconnected chain of lakes
- Coupled surface and groundwater
- Shifting drainage boundaries
- Groundwater fed lakes and wetlands, including by the deeper confined aquifer
- Unmonitored withdrawals and irrigation

Sensitivity analyses were performed with the pre-calibration version of the model. This was the basis for further research and analysis of the system and the results. The calibrated model was the basis of the 5-, 10-, 25- and 100-yr design storms used for a comparison of current conditions with future conditions. The current and future conditions models represent 2020 and 2070 respectively and included modifications with the best available data, increment 1 was not simulated. The base current conditions simulations were based on the calibrated model and included modifications to structure operations and design rainfall, initial and boundary conditions. The base future conditions model included modifications to account for new SFWMD DWM projects, higher rainfall and structure operations.

SFWMD Performance metrics (PM) provided guidelines for LOS evaluation in UKB. This was the framework for evaluation of the flood protection provided by the SFWMD's primary conveyance system. Evaluation was done for Lake Management Areas (LMAs) that group many watersheds, and an evaluation lens that has been used by the SFWMD. This also provided a meaningful way to assess the system as each LMA is separated from the other by 1 or more structures operated by SFWMD. The models give a better understanding of the potential issues that the LMAs may experience. Each PM addresses a different characteristic of the LMA's LOS from aspects of canal and structure conveyance

to flooding extent and duration. PM4 was not included in the scope of this project, but a modification of the LOS Program PM#4 was used to assess the FPLOS due to downstream constraints. In addition, volume of water above regulation schedule resulting from the design rainfall, and the time taken for the lakes to return to the regulatory schedules, was part of the assessment of the overall system within the LMA.

To focus the overall findings of each watershed, the simulated design storms identified as the LOS for all PMs were synthesized with primary focus on the PM #1, PM #5, and PM #6 since they provide a basin-level view of the flooding, conveyance capacity and ability to drain the system. Specifically, PM #1 focused on stages and overtopping of lakes and canal banks; and it determines the overall level of service for the LMA. Therefore, PM #1 decided the minimum LOS rating if it was the lowest. PM #5 and PM #6, focused on the inundation depth and duration, and are also critical for understanding flooding impacts and a key factor in assigning LOS. PM#5 and PM#6 metrics were used to assess the FPLOS differently for each LMA. However, PM #1 was used as the first level of assessment that determines the minimum LOS, as this considers infrastructure and direct overtopping. Additionally, for PM #1 assessment, main tributaries to the lakes were evaluated.

Table 13-1 summarizes the FPLOS for each LMA as evaluated using the guidelines in Section 9.1. The overall FPLOS rating is the following: less than 5-yr or 5- yr FPLOS rating or 5-yr for all the LMAs. However, it should be noted this is dependent on the thresholds used, which can be refined for UKB since this is the first inland FPLOS assessment for SFWMD. It is also important to note that included in the LMAs are several water bodies, and the FPLOS rating was assessment of the overall system within the LMA. The overall rating is summarized in **Table 13-2.**

The SFWMD performance metrics were effective in understanding the critical areas of the UKB watersheds that will be impacted by flooding caused by high-intensity storm events, groundwater rise and future development. In this FPLOS analysis, no watershed in UKB provides protection from flooding caused by high-intensity rainfall and high groundwater under current conditions, and as can be seen in the results future conditions increased these impacts, in general. In some watersheds, the DWM projects made the flood protection offered slightly better.

More value will be added to this FPLOS analysis in future SFWMD work efforts to conceptualize and prioritize potential flood mitigation projects and strategies. Valuable insight from this FPLOS study regarding the current and future potential flooding risks associated with each UKB watershed will be used to formulate preliminary mitigation strategies to help mitigate the effects of extreme rainfall and its associated hydrology. Preliminary cost estimates of these mitigation strategies performed as part of Phase II in SFWMD FPLOS, will be implemented into the UKB FPLOS model. This will help understand the effectiveness of the strategy to reduce flood inundation depths and durations, and to protect critical infrastructure. The FPLOS evaluated in this study provides a solid baseline to understand current and future hydrologic conditions expected in the UKB watersheds and to develop an adaptation strategy.

Table 13-1: Summary of FPLOS Rating in UKB for PMs 1, 5 and 6

Conditions	Performance Metric	Lake Alligator LMA	Lake Myrtle LMA	Lake Hart LMA	Lake Gentry LMA	Lake East Toho LMA	Lake Toho LMA	Lake Cypress LMA	Lake Hatchineha LMA	Lake Kissimmee LMA
Current	1	<5-yr	<5-yr	<5-yr	< 5-yr	25-yr	< 5-yr	< 5-yr	< 5-yr	25-yr
	5	5-yr	< 5-yr	10-yr	5-yr	10-yr	10-yr	10-yr	10-yr	5-yr
	6	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr
Future	1	<5-yr	<5-yr	<5-yr	<5-yr	25-yr	< 5-yr	< 5-yr	< 5-yr	25-yr
	5	5-yr	< 5-yr	10-yr	5-yr	10-yr	10-yr	10-yr	10-yr	5-yr
	6	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr	5-yr

Table 13-2: Overall FPLOS Rating in UKB

Model Conditions	Lake Alligator LMA	Lake Myrtle LMA	Lake Hart LMA	Lake Gentry LMA	Lake East Toho LMA	Lake Toho LMA	Lake Cypress LMA	Lake Hatchineha LMA	Lake Kissimmee LMA
Current	<5-yr	<5-yr	<5-yr	<5-yr	5-yr	<5-yr	<5-yr	<5-yr	5-yr
Future	<5-yr	<5-yr	<5-yr	<5-yr	5-yr	< 5-yr	< 5-yr	< 5-yr	5-yr

13.1 Current Conditions Analysis

- **Lake Alligator LMA:** Under current conditions, the lake is expected to perform below the 5-year return period for PM #1, meaning that the performance is relatively poor, with events expected to occur more frequently than every 5 years. However, for PM #5 and PM #6, the lake meets the 5-year return period, indicating a moderate level of resilience.
- **Lake Myrtle LMA:** This lake also shows poor performance for PM #1 under current conditions, with expectations below the 5-year return period. For PM #5, the performance was still below the 5-year return period. PM #6 shows the best performance, maintaining the 5-year return period consistently.
- **Lake Hart LMA:** Performance is generally better than the first two lakes, with PM #1 falling below the 5-year return period, but PM #5 shows improvements, reaching the 10-year return period, and PM #6 consistently at the 5-year return period.
- **Lake Gentry LMA:** Similar to Lake Hart, Lake Gentry exhibits performance below the 5-year return period for PM #1 but improves to maintain the 5-year return period for both PM #5 and PM #6.
- **Lake East Toho LMA:** This lake shows significant difference relative to other lakes, achieving the 25-year return period for PM #1, indicating a strong level of performance. PM #5 and PM #6 are at the 10-year and 5-year return periods, respectively.
- **Lake Toho LMA:** Similar to many lakes in UKB, this lake reaches the <5-year return period for PM #1, showing better performance in some of the locations along the banks. PM #5 and PM #6 are both consistent at the 10-year and 5-year return periods, respectively.
- **Lake Cypress LMA, Lake Hatchineha LMA, and Lake Kissimmee LMA:** These lakes show lower performance for PM #1, with return periods below 5 years; except Lake Kissimmee which met the 25-yr FPLOS or close to it. However, they all except Lake Kissimmee achieved the 10-year return period for PM #5 and the 5-year return period for PM #6 which achieved a 5-year return period for PM5, indicating moderate performance.

13.2 Future Conditions Analysis

The future conditions largely mirror the current conditions with only slight variations:

- **Lake Alligator LMA:** The performance remains unchanged in future conditions, with PM #1 still below the 5-year return period, PM #5 reaching the 5-year return period, and PM #6 maintaining the 5-year return period.
- **Lake Myrtle LMA:** Similar to the current conditions, the performance under future scenarios shows no significant improvement, with PM #1 and PM #5 still under the 5-year return period and PM #6 maintaining 5-year return periods.

- **Lake Hart LMA:** Under PM #1, PM #5 and PM #6, both lakes Hart and Mary Jane maintain the same performance levels in future as in current conditions.
- **Lake Gentry LMA:** Under PM #1, PM #5 and PM #6, Lake Gentry maintains the same performance levels in future as in current conditions.
- **Lake East Toho LMA:** The lakes in Lake East Toho LMA maintain a strong performance under future conditions, with PM #1 consistently achieving the 25-year return period. PM #5 and PM #6 remain at the 10-year and 5-year return periods, respectively.
- **Lake Toho LMA:** These lakes maintains the same performance under future conditions, with PM #1 consistently achieving the 5-year return period or lower. PM #5 and PM #6 remain at the 10-year and 5-year return periods, respectively
- **Lake Cypress LMA, Lake Hatchineha LMA, and Lake Kissimmee LMA:** These lakes show similar performance under future conditions as in the current state, with PM #1 at or below the 25-year return period and PM #5 and PM #6 between 5-yr and 10-yr for the various lakes in this LMA.
- **Table 13-3** suggests that certain lakes (East Toho LMA and Toho LMA) are expected to perform significantly better under both current and future conditions, with higher return periods indicating less frequent adverse events and better resilience. Other lakes, particularly Lake Alligator LMA and Lake Myrtle LMA, show consistently lower performance, which may warrant further investigation or intervention to improve their resilience to future conditions. The general stability across PM #5 and PM #6 across all lakes suggests a baseline level of resilience that could be built upon, particularly in improving the performance of PM #1 for the lakes that are currently underperforming.

Table 13-3 provides cumulative water volume data (in acre-feet) due to storms for various lakes under different design event (DE) scenarios (5-year, 10-year, 25-year, 100-year) under both current and future conditions. The following analysis highlights the trends and implications of this data:

- Lake Kissimmee (Kissimmee Branch): Under current conditions, the excess water volume due to storms increases significantly with the severity of the storm, ranging from 20,843.7 ac-ft for a 5-year DE to 41,193.6 ac-ft for a 100-year DE. This trend intensifies under future conditions, with the 100-year DE volume reaching 48,465.6 ac-ft, indicating that future storm events will likely contribute to even greater excess water volumes.
- Lake Alligator (Alligator_Lk Branch): This lake shows a more substantial increase in water volume under future conditions, especially for more severe storms. For example, the 25-year DE volume rises from 7,551.4 ac-ft under current conditions to 8,733.8 ac-ft under future conditions, and the 100-year DE volume nearly doubles from 11,860.4 ac-ft to 15,786.5 ac-ft. This suggests a significant vulnerability to larger storm events in the future.
- Lake Myrtle (LK_Myrtle Branch): This lake exhibits smaller cumulative volumes compared to others, but the increase in volume under future conditions is notable. The 25-year DE volume under future conditions (2,485.3 ac-ft) is significantly higher than under current conditions (556.0 ac-ft), indicating a considerable increase in stormwater retention challenges.

- Lake Hart (LK-Hart Branch) and Lake Mary Jane (Lk_Mary_Jane Branch): Both lakes show a steady increase in excess water volume with increasing storm severity. For Lake Hart, the 100-year DE volume under future conditions (8,608.0 ac-ft) is significantly higher than the current condition volume (6,199.7 ac-ft). Similarly, Lake Mary Jane sees its 100-year DE volume rise from 3,783.6 ac-ft under current conditions to 5,280.2 ac-ft in future scenarios.
- East Lake Tohopekaliga (Lower-E-Toho Branch): This lake shows a substantial increase in excess water volume across all storm scenarios. The 100-year DE volume jumps from 27,072.8 ac-ft under current conditions to 39,345.6 ac-ft under future conditions, indicating a marked increase in stormwater impact.
- Lake Gentry (LK_Gentry Branch): The data for Lake Gentry under future conditions shows significant increases in volume for lower return period events (e.g., 5-year DE), suggesting that even less severe storms will contribute to more excess water in the future.
- Lake Toho (Toho-main Branch): Similar to Lake Kissimmee, Lake Toho shows a substantial increase in excess water volumes, particularly under more severe future storm conditions. The 100-year DE volume increases from 37,976.2 ac-ft under current conditions to 51,813.0 ac-ft under future conditions, highlighting the lake's vulnerability to intense storm events.
- Cypress (Cypress_Lk Branch) and Lake Hatchineha (East_LK_Hatchineha Branch): Both lakes show significant increases in cumulative volumes, particularly for the 100-year DE scenario, where Lake Cypress sees an increase from 6,372.0 ac-ft under current conditions to 9,127.1 ac-ft under future conditions, and Lake Hatchineha from 2,406.9 ac-ft to 3,065.3 ac-ft.

Table 13-3 examines the peak water volume (in acre-feet) due to storms for various lakes under different DE scenarios. The data highlights the following:

- Lake Kissimmee (Kissimmee Branch): Under current conditions, the peak volumes range from 471,473.1 ac-ft for the 5-year DE to 502,379.1 ac-ft for the 100-year DE. Future conditions show a marked increase in these values, with the 100-year DE volume reaching 517,403.7 ac-ft. This suggests that peak stormwater volumes will significantly impact the lake, especially under future scenarios. This also shows that the K-C-H system that was designed to store water from the UKB until LKB flooding conditions are better and can receive water from UKB, is performing as was designed to.
- Lake Alligator (Alligator_Lk Branch): The peak volumes show a considerable increase under future conditions, particularly for severe storms. The 100-year DE peak volume rises from 41,496.7 ac-ft to 44,438.4 ac-ft, indicating that this lake will face higher maximum water levels during future storm events. The smaller volume increase from 5-yr to 100-yr is an increase of 7% in volume similar to Lake Kissimmee.
- Lake Myrtle (LK_Myrtle Branch): Although this lake shows lower peak volumes compared to others, the future conditions data indicates a steep increase. For example, the 100-year DE peak volume increases from 7,716.9 ac-ft under current conditions to 8,602.8 ac-ft in the future, suggesting growing challenges in managing peak stormwater volumes. This also shows the

inflow of water from the east, across from the surface water divide; this has been observed during many rain events.

- Lake Hart (LK-Hart Branch) and Lake Mary Jane (Lk_Mary_Jane Branch): Both lakes show rising peak volumes across all DE scenarios. Lake Hart's 100-year DE peak volume rises from 19,451.1 ac-ft under current conditions to 22,348.8 ac-ft under future conditions. Lake Mary Jane shows its 100-year DE peak volume increase from 9,570.6 ac-ft to 11,395.7 ac-ft, indicating a significant rise in stormwater levels.
- East Lake Tohopekaliga (Lower-E-Toho Branch): This lake shows a significant increase in peak volumes, with the 100-year DE volume rising from 139,714.8 ac-ft under current conditions to 156,725.1 ac-ft in the future, a 12% increase higher than the other watersheds. This suggests that peak stormwater volumes will become increasingly difficult to manage. This is critical because the Orlando International Airport, and other growing urban areas are drained by this watershed.
- Lake Gentry (LK_Gentry Branch): Unlike the other lakes, Lake Gentry shows only modest increases in peak volumes under future conditions. The 100-year DE peak volume rises from 27,583.2 ac-ft to 29,931.1 ac-ft, indicating about the same 7% increase in volume as other watersheds.
- Lake Toho (Toho-main Branch): Similar to Lake Kissimmee, Lake Toho exhibits substantial increases in peak volumes. The 100-year DE volume increases from 208,353.3 ac-ft under current conditions to 230,085.1 ac-ft under future conditions, highlighting the lake's growing vulnerability to extreme weather events. The increase represents about 10% increase in volume.
- Cypress (Cypress_Lk Branch) and Lake Hatchineha (East_LK_Hatchineha Branch): Both lakes see significant increases in peak volumes under future conditions. For example, Cypress sees its 100-year DE peak volume increase from 41,140.1 ac-ft to 44,977.1 ac-ft, while Lake Hatchineha's peak volume rises more modestly.

The analysis across both tables indicates a general trend of increasing excess water volumes in lakes due to storm events, with future conditions showing a significant exacerbation of this trend, due to rainfall being higher under the future conditions, than under the current conditions. The K-C-H system under current and future conditions show how important this system is, in terms of storing the water from the UKB, until the LKB flooding recedes. Lake Kissimmee, Lake Toho, and East Lake Tohopekaliga exhibit the growing challenges in stormwater management in future that would impact the system. Smaller lakes such as Lake Alligator and Lake Myrtle also demonstrate heightened vulnerability to shifting watershed boundaries on the east particularly during bigger storm events, which suggests that targeted mitigation measures will be necessary to mitigate these impacts.

The overall rating of the entire UKB is <5-yr. This is based on PM #1, PM #5 and PM #6 for all LMAs and assigning the rating based on the biggest rainfall that could be managed by all of the structures in the basin without inundation impacts.

Table 13-3. Excess Water Volume in Lakes due to the Storms for Simulation Period

Lake	Branch	Cumulative Volume in ac-ft due to the Storm			
		5-Yr DE Under Current Conditions	10-Yr DE Under Current Conditions	25-Yr DE Under Current Conditions	100-Yr DE Under Current Conditions
Lake Kissimmee	Kissimmee	20843.7	26847.5	36555.4	41193.6
Lake Alligator	Alligator_Lk	3444.5	5333.5	7551.4	11860.4
Lake Myrtle	LK_Myrtle	977.1	1311.1	556.0	903.1
Lake Hart	LK-Hart	1344.2	2148.9	3487.8	6199.7
Lake Mary Jane	Lk_Mary_Jane	922.8	1442.7	2228.1	3783.6
East Lake Tohopekaliga	Lower-E-Toho	7413.3	10196.4	15134.0	27072.8
Lake Gentry	LK_Gentry	2171.0	3203.1	4555.0	6909.2
Lake Toho	Toho-main	11634.6	16133.7	23861.6	37976.2
Cypress	Cypress_Lk	2296.3	3112.5	4464.7	6372.0
Lake Hatchineha	East_LK_Hatchineha	1093.1	1423.9	1970.5	2406.9
Lake	Branch				
		5-yr DE under future conditions	10-yr DE under future conditions	25-yr DE under future conditions	100-yr DE under future conditions
Lake Kissimmee	Kissimmee	24200.3	31900.0	37763.6	48465.6
Lake Alligator	Alligator_Lk	2655.0	4406.8	8733.8	15786.5
Lake Myrtle	LK_Myrtle	1160.8	1661.2	2485.3	3811.9
Lake Hart	LK-Hart	1849.6	2971.1	4956.3	8608.0

Lake Mary Jane	Lk_Mary_Jane	1253.2	1920.4	3066.8	5280.2

Lake	Branch	5-yr DE under future conditions	10-yr DE under future conditions	25-yr DE under future conditions	5-yr DE under future conditions
East Lake Tohopekaliga	Lower-E-Toho	9240.7	13277.5	21502.3	39345.6
Lake Gentry	LK_Gentry	461.6	823.5	1588.5	934.6
Cypress	Cypress_Lk	2904.8	4048.8	5590.5	9127.1
Lake Toho	Toho-main	14743.1	21068.9	31047.8	51813.0
Lake Hatchineha	East_LK_Hatchineha	1293.7	1733.2	2156.5	3065.3

Table 13-4. Excess Water Volume in Lakes due to the Storms at Peak

Lake	Branch	Cumulative Volume at Peak in ac-ft due to the Storm			
		5-Yr DE Under Current Conditions	10-Yr DE Under Current Conditions	25-Yr DE Under Current Conditions	100-Yr DE Under Current Conditions
Lake Kissimmee	Kissimmee	471473.1	478926.9	491854.7	502379.1
Lake Alligator	Alligator_Lk	33437.7	34726.9	36916.4	41496.7
Lake Myrtle	LK_Myrtle	4917.4	5510.3	6373.0	7716.9
Lake Hart	LK-Hart	14005.4	14923.3	16457.9	19451.1
Lake Mary Jane	Lk_Mary_Jane	6392.7	6948.8	7812.8	9570.6
East Lake Tohopekaliga	Lower-E-Toho	112661.3	116342.9	124029.8	139714.8
Lake Gentry	LK_Gentry	22517.6	23360.4	24779.4	27583.2
Lake Toho	Toho-main	175592.1	180728.8	190030.0	208353.3
Cypress	Cypress_Lk	36186.7	37237.1	39014.9	41140.1
Lake Hatchineha	East_LK_Hatchineha	13474.0	13890.7	14609.9	15407
Lake	**Branch**	**Cumulative Volume at Peak in ac-ft due to the Storm**			
		5-yr DE under future conditions	**10-yr DE under future conditions**	**25-yr DE under future conditions**	**100-yr DE under future conditions**
Lake Kissimmee	Kissimmee	481453.6	491749.0	499124.6	517403.7
Lake Alligator	Alligator_Lk	29840.5	31680.3	36209.1	44438.4
Lake Myrtle	LK_Myrtle	4537.6	5415.7	6610.4	8602.8
Lake Hart	LK-Hart	14570.5	15854.4	18087.0	22348.8

Lake	Branch	Cumulative Volume at Peak in ac-ft due to the Storm			
		5-yr DE under future conditions	10-yr DE under future conditions	25-yr DE under future conditions	100-yr DE under future conditions
Lake Mary Jane	Lk_Mary_Jane	6740.3	7472.9	8757.1	11395.7
East Lake Tohopekaliga	Lower-E-Toho	115165.2	121324.4	132828.4	156725.1
Lake Gentry	LK_Gentry	13227.7	13527.1	14425.3	14931.1
Cypress	Cypress_Lk	37820.3	39276.5	41036.1	44977.1
Lake Toho	Toho-main	181963.3	189546.5	202452.6	230085.1
Lake Hatchineha	East_LK_Hatchineha	37820.3	39276.5	41036.1	44977.1

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Appendix A: Surface Water and Groundwater Observation Datasets

Appendix B: Calibration/Validation Stations with DBKEYS

Appendix C: Surface Water and Groundwater Plots for Calibration Event

Appendix D: Surface Water and Groundwater Plots for Calibration Event – Storm Event Only

Appendix E: Statistics (09/10/2017 – 10/10/2017) for Calibration Locations

Appendix F: Surface Water and Groundwater Plots for Validation Event

Appendix G: Statistics (09/29/2011 – 10/27/2011) for Validation Locations

Appendix H: Water Budgets Terminology

Appendix I: Watershed level MIKE SHE Water Budgets for POS – Calibration and Validation

Appendix J: MIKE HYDRO Water Budgets – Calibration and Validation

Appendix K: UKB FPLOS Model Operational Logic

Appendix L: Water Budgets – Current and Future Conditions UKB FPLOS Models

Appendix M: Current and Future Conditions UKB FPLOS Models – Performance Measures for Ungaged Watersheds