Hazen

Big Cypress Basin Model Update

40715-010

Task 5: BCB Design Events Simulation

May 12, 2025

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List of Acronyms

Abbreviation	Definition
BCB	Big Cypress Basin
CIMSS	Cooperative Institute of Meteorological Satellite Studies
CSS	Corkscrew Swamp Sanctuary
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
FDEP	Florida Department of Environmental Protection
FPLOS	Flood Protection Level of Service
GW	Ground Water
LiDAR	Light Detection and Ranging
NAVD88	North American Vertical Datum of 1988
NDBC	National Data Buoy Center
NOAA	National Oceanic and Atmospheric Administration
OC	Open Channel Flow
OL	Overland Flow
QA/QC	Quality Assurance and Quality Control
SFWMD	South Florida Water Management District
SW	Surface Water
SZ	Saturated Zone
USGS	United States Geological Survey
UZ	Unsaturated Zone

Executive Summary

The Big Cypress Basin (BCB) office of the South Florida Water Management District (SFWMD) operates over 140 miles of canals and water control structures, managing flood control in wet seasons and protecting water supply and the environment in dry seasons. The BCB Model Update aims to enhance the hydrologic and hydraulic model by updating software, reflecting changes in infrastructure, using new coastal survey data, and expanding the model domain to better capture the basin's boundaries. These updates will improve the model's ability to perform simulations.

BCB models were reviewed, and latest datasets included in the BCB Model Update after QA/QC. The update features new network additions, structures, cross sections, and survey data. The saturated zone contains the Water Table, Lower Tamiami, and Sandstone aquifers, as well as the confining units between them.

Data on stages, flows, gate operations, rainfall, evapotranspiration, and groundwater levels were collected and QA/QC'd from SFWMD, USGS, and county databases within the study domain.

The calibrated BCB Long-term Model (Run 76, as of August 2024) served as the basis for constructing and evaluating model performance during Hurricane Irma. Both models were built using DHI 2022 Update 1 version and MIKE HYDRO; the domain encompasses approximately 1,000 square miles and includes four 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.

The BCB Irma Model integrates several modifications to the calibrated BCB Model. These modifications include updated rainfall data, structural operations, canal backfilling, additions to the 1D network, and other minor edits. The calibration model has been refined to evaluate its performance under hydrologic conditions before, during, and after Hurricane Irma. Input from SFWMD was incorporated, resulting in various improvements to the BCB Irma model. This report details the updated version of the BCB Irma model.

The simulation period for the BCB Irma Model spans from August 27, 2017, to October 31, 2017. The evaluation of the BCB Irma Model under Hurricane Irma's hydrologic conditions used the same calibration criteria and targets as the long-term BCB model, along with additional metrics and targets specific to storm event simulations. The calibration criteria and targets are provided in the table on the following page. The calibration metrics utilized in the long-term model and applied for evaluation in the storm event model include Mean Error (ME), Cumulative Flow Error (CFE), Correlation (R), and Nash-Sutcliffe (NS). Additional metrics used in the evaluation of the model consist of peak stages and peak flows, time to peak stages and flows for surface water, as well as peak water levels and time to peak for groundwater.

Calibration metrics have been classified into 3 major criteria (Good, Acceptable, and Less Acceptable).

Observation Station Type	Model Performance Criteria		
	Good	**Acceptable**	**Less Acceptable**
Major canal stations (Stations in SFWMD BCB Atlas)	IMEI ≤ + 0.5 ft	0.5ft < IMEI ≤ ±1.0ft	IMEI >1.0 ft
Surface water stage and shallow wells (computational layer = 1)	IMEI ≤ +0.75ft	+0.75ft < IMEI ≤ ± 1.5ft	IMEI >1.5 ft
Deeper wells (computational layer > 1)	IMEI ≤ ±1.5ft	1.5ft < IMEI ≤ ±2.5ft	IMEI >2.5 ft
Surface water flow (instantaneous)	IMEI ≤ 15%	15% < IMEI ≤ 30%	30% < IMEI
Surface water flow (non-zero values)	|CFE| ≤ 15%	15% < |CFE| ≤ 30%	30% < |CFE|
All stations	0.85 ≤ R 0.5 ≤ NS	0.7 ≤ R < 0.85 0≤ NS < 0.5	R< 0.7 NS < 0
All SW stage stations	Difference in stages at peak and time to peak		
All SW flow stations	Difference in flows at peak and time to peak		
All GW stations	Difference in groundwater levels at peak and time to peak		

Evaluation of the BCB Irma Model showed:

- 83% of primary surface water stations met “Acceptable” criteria for ME, with 53% meeting the “Good” criteria.
- Among secondary surface water stations, 84% met “Acceptable” criteria, and 42% met “Good” criteria.
- For groundwater stations in Layer 1, 82% met “Acceptable” criteria, and 69% met “Good” criteria.
- For wells below Layer 1, 85% met “Acceptable” criteria, and 59% met “Good” criteria.
- Cumulative Flow Error values and water balances at BCB structures were mostly reasonable.

A key component of a storm event model is its behavior at peak (difference in magnitude and timing, compared to the observed). Peak surface water level residuals ranged from 0.1 ft to 2.3 ft. Time to peak difference at critical structures was less than 2 hours. Groundwater level residuals ranged from 0.01 ft to 4.3 ft. Time to peak residuals in saturated zone was less than 1 day for most stations, which is reasonable since the observed data are daily.

The BCB Irma Model is a regional-scale model that has met the established evaluation criteria and was deemed reasonable for providing a basis for design events evaluation. The BCB design event models were derived from the BCB Irma Model, with adjustments to rainfall parameters and structural operations. The model's domain includes all essential contributing areas and infrastructure, and its parametrization based on BCB Irma Model results is considered sufficiently accurate for use in design events.

The District relies on six (6) formal performance metrics (PMs) to evaluate the FPLOS provided by the primary water management infrastructure. These metrics are:

- PM #1 Maximum Stage in Primary Canals.
- PM #2 Maximum Daily Discharge Capacity through the Primary Canals.
- PM #3 – Structure Performance assessment due to sea level rise.
- Peak Storm Runoff evaluation due to sea level rise.
- PM #5 Frequency of Flooding.
- PM #6 Duration of Flooding.

This project included an evaluation of PM #1, 2, 5 and 6.

Most water bodies exhibited a flood protection level of service (FPLOS) below 5 years when assessed strictly on the criterion that no part of the canal banks is overtopped. Consequently, the major sub-basins were given both localized and overall FPLOS ratings. The overall FPLOS rating for the sub-basins was 10 years, except for Estero Bay and Faka Union, which received 5-yr and 25-yr FPLOS ratings, respectively.

Large areas of Cocohatchee, Faka Union, Fakahatchee, Henderson/Belle Meade, and parts of Golden Gate and Trafford sub-basins remain undeveloped and lack stormwater management systems, leading to significant flooding. In contrast, East Naples, Estero Bay, and some Coastal basins have effective drainage systems, as shown in the PM #5 assessment.

PM #6 results were consistent with PM #5. Eastern, southern, and some central basin areas in red experience prolonged flooding (>420 hours), especially in wetlands and near water management structures. Western urban areas show shorter floods (<72 hours); coastal zones and areas near water control structures have moderate flood durations (72-240 hours). Flood duration correlates with land use patterns; developed areas drain faster than natural and conservation areas.

Please note that due to variations in computational resources and environments across different machines, minor differences in model results are expected when the model is run. These small discrepancies are expected and generally do not impact the overall validity of the findings.

1. Introduction and Background

The Big Cypress Basin (BCB) in Florida is a unique and ecologically significant region characterized by its diverse ecosystems, intricate hydrology, and susceptibility to extreme weather events. As climate change intensifies, the frequency and intensity of storm events are projected to increase, posing significant challenges to the region's water management systems and natural habitats. Effective planning and management of stormwater runoff are essential to mitigate flooding, protect water quality, and safeguard the area's rich biodiversity.

This report focuses on the simulation of design storm events within the BCB, utilizing advanced modeling techniques to assess the impacts of various storm scenarios on hydrological responses. By integrating historical weather data, topographical information, and land use patterns, this effort aims to provide a comprehensive understanding of how different storm conditions influence watershed dynamics and flood risk. The findings will not only enhance our understanding of the BCB region's hydrological behavior but will also inform decision-makers and stakeholders about effective strategies for stormwater management, infrastructure development, and conservation efforts.

This report aims to give useful information on climate resilience in Florida's fragile ecosystems through detailed analysis and models, ultimately supporting sustainable progress and protecting the environment amid changing weather patterns.

The South Florida Water Management District (District) is carrying out a comprehensive review of its regional water management infrastructure to assess the current flood protection level of service (FPLOS) provided. The FPLOS indicates the degree of protection offered by the water management facilities within a watershed under both present and future conditions, with future FPLOS considering sea level rise and anticipated development. This data can assist local governments, the SFWMD, and other state and federal agencies in identifying areas needing improvements or upgrades to water management facilities, determining the responsible entities for these enhancements, and securing the necessary funding and technical resources to support these initiatives. However, this scope of work includes simulation of current (2020) conditions, and evaluation of FPLOS in response to 5-, 10-, 25- and 100-yr design rainfall.

This report is the last in a series of documentation describing the modeling efforts to update the BCB model calibration and application, including FPLOS assessment. A complete understanding for the reader should include this report, along with the previous reports and the included references in this report. It is important to note that this scope does not include the typical FPLOS assessment performed by the SFWMD; rather the scope of this project includes development of long-term model, calibration to long-term conditions, evaluation of model performance to Hurricane Irma, calibration of the short-term simulation to Hurricane Irma, and assessment of the model performance and FPLOS under current conditions and design events.

2. Flood protection Level Of Service (FPLOS) Performance Metrics (PM)

2.1 Description of the PMs

The District relies on six (6) formal performance metrics (PMs) to evaluate the FPLOS provided by the primary water management infrastructure. These metrics are defined briefly in this section.

- PM #1 Maximum Stage in Primary Canals – This represents the highest water level profile within the primary canal system. The profile is created for various design storm events (5-yr, 10-yr, 25-yr, and 100-yr). The most severe design storm that remains within the canal boundaries determines the FPLOS for the primary canal system.

- PM #2 Maximum Daily Discharge Capacity through the Primary Canals – PM #2 represents the maximum discharge capacity across the primary canal network. This discharge is measured as a flow, weighted by the contributing drainage area, expressed in cubic feet per second per square mile of the contributing area for the 25-yr design event. Tidal influences are mitigated by applying a 12-hour moving average to the discharge. While this report refers to the peak of the 25-yr net discharge hydrographs as the calculated discharge capacity, the actual capacity of the canal segment is the net discharge corresponding to the most significant design flood event that can be contained within the canal banks, based on the outcomes of the 5-yr, 10-yr, 25-yr and 100-yr events.

- PM #3 – Structure Performance – Impact of Sea Level Rise – This metric evaluates the operational capacity of a tidal structure. It is similar to the static design condition considered in the initial design but assesses the structure's flow under various storm surge events and different sea level rise scenarios.

- PM #4 Peak Storm Runoff – Impact of Sea Level Rise – PM #4 represents the maximum conveyance capacity of a watershed at the tidal structure for various design storms. It illustrates the peak conveyance (measured as a 12-hour moving average) for a given design storm and specific tidal boundary condition. This metric assesses the system's performance under extreme stress conditions and can be used to determine if these conditions exceed the design limits. When evaluating this PM, it is presumed that the design rainfall and design storm surge occur simultaneously or with a temporal gap that maximizes stress on the structure.

- PM #5 Frequency of Flooding – This performance measure assesses the elevations or depths of overland flooding across various design storms, including 5-yr, 10-yr, 25-yr, and 100-yr events. The resulting flood depths and elevations can be compared to the elevations of constructed features like buildings and roadways, where data are available. For the BCB FPLOS evaluation, flood inundation maps were created using model outputs for each storm event.

- PM #6 Duration of Flooding quantifies how long flooding lasts at key points within a watershed. For this study, we mapped the time flood levels stayed above a certain depth across the entire area using 2-D gridded model output files.

The SOW for the BCB Update model includes only PM #1, 2, 5 and 6. So this TM discusses the 4 PMs evaluated under this project.

2.2 Results Presentation

The SFMWD FPLOS metrics require comprehensive analyses of the model outcomes. This report outlines modeling analyses for PM metrics presentation, including 1-dimensional channel hydraulics, 2-dimensional overland flooding, and, to a lesser extent, 3-dimensional groundwater flow. These components are integrated within the model and numerically linked during the simulation but produce output at various timesteps, constrained by practical file size considerations. The MIKE 1D channel hydraulics simulation, MIKE SHE 2D overland flow, and MIKE SHE 3D groundwater flow utilize different time steps for computation and result printing. Consequently, some variations in the evaluation of results across the PMs should be anticipated.

The results of MIKE HYDRO 1-D stages and flows were utilized to evaluate PM #1 and PM #2. Occasionally, the operation of structures leads to temporary spikes, which produce stages and flows indicative of numerical oscillations rather than actual conditions. To mitigate this, 12-hour moving averages were applied. Additionally, 2-D overland depth maps were employed to assess PM #5 and PM #6, corresponding to the maximum flooding depth and duration, respectively.

The default setting in MIKE HYDRO assumes that the flow direction is positive as chainage numbers increase downstream. Consequently, in PM #1 plots, 'flow direction' identifies the left and right sides of the river by viewing along the main flow direction. Typically, the left and right banks are observed by looking towards decreasing chainage numbers.

Additionally, it is important to mention that the BCB FPLOS design event models were executed for the simulation period spanning from 8/27/2017 to 10/30/2017. However, these dates are arbitrary and do not reflect real hydrologic conditions for that time frame.

3. Base Conditions (Current) Model Setup

This part of the report details how datasets and processes were adjusted based on the model calibrated to Hurricane Irma. It focuses on the modifications made to the Irma model; details on the Irma model will be found in the previous reports under this project.

The current conditions model was set up for about 2 months 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.

3.1 Design Rainfall

The BCB Irma Model served as the foundation for the design events model and utilized NEXRAD data from Hurricane Irma (September 10-12, 2017). In simulating the design storm, the rainfall from Hurricane Irma was substituted with rainfall representing the spatially distributed 3-day design storm distribution provided by SFWMD in their Environmental Resource Permit, (SFWMD, 2014). This was used to create rainfall hyetographs. The cumulative percentage for the 72-hr rainfall distribution shown in **Table 3-1** and the plot is shown in Figure 3-1. 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.

Table 3-1. SFWMD 72-hr Rainfall Distribution

Time (Hours)	Cumulative Percentage of peak 24-Hr Rainfall	Cumulative Percentage of 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%

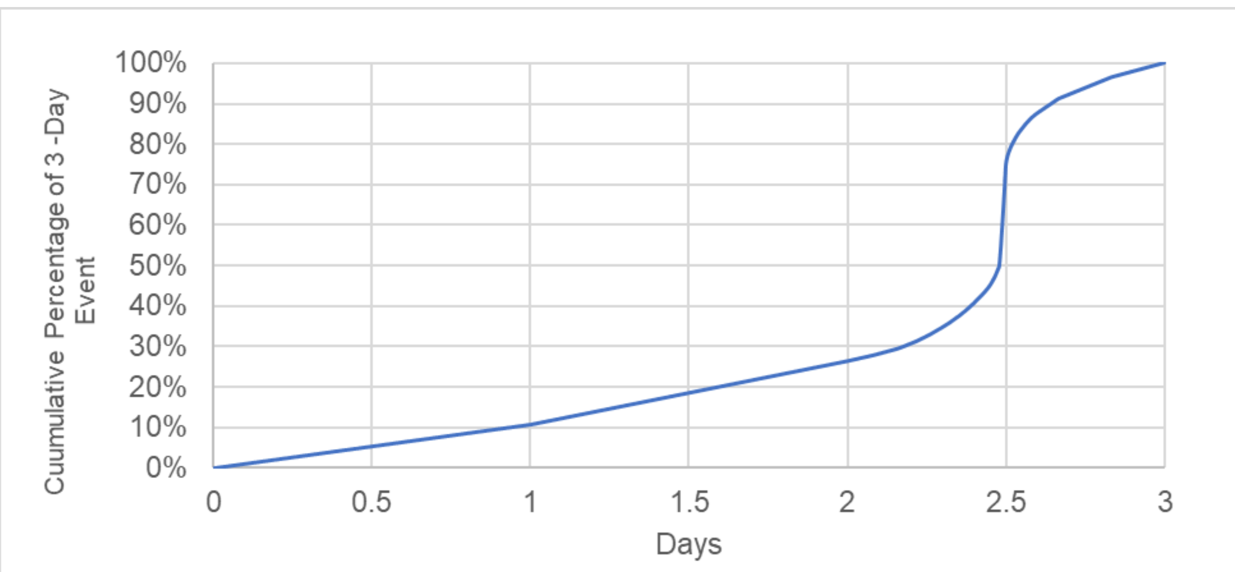

Figure 3-1. Unit Rainfall Distribution for 72-hr Rainfall Event

Rainfall depths were sourced from the National Oceanic and Atmospheric Administration (NOAA) Rainfall Atlas ASCII Grid data. According to NOAA documentation, the raster datasets of rainfall precipitation frequency were derived through spatial interpolation of point precipitation frequency estimates. These estimates were determined using regional statistics from the annual maximum data series.

Flood events that are anticipated to be equaled or surpassed at least once during a specific recurrence interval (5-, 10-, 25-, or 100-yr period) were chosen as design events. These flood occurrences, labeled as the 5-, 10-, 25- or 100-yr floods, correspond to annual probabilities of 20%, 10%, 4%, and 1%, respectively, of being equaled or exceeded within any given year.

The original NOAA grid provided total precipitation inches, in 2 km x 2 km grids. For the BCB design events simulations, 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 3-2** provides the cumulative depth of rainfall over the simulation period for the BCB Model Domain.

Table 3-2. **Summary Statistics for Rainfall for design storms (inches of depth)**

Design Event	Cumulative Rainfall Depth (inches)
5-year	7.36
10-year	8.81
25-year	10.95
100-year	14.63

3.2 Structure Operations

The hydrology of BCB is governed by natural areas, and discharge control structures, which manage water levels in drainage areas. These structures operate on calendar-based schedules. The operating schedules embody the management strategies of the BCB water management system, with a focus on optimizing flood control and supporting other core missions of SFWMD. Typically, lake regulation schedules reach their lowest and highest points at the start and end of the wet season to prepare for flood control and water supply needs, respectively. Despite this, the regulation schedules are flexible guidelines managed by SFWMD using the best professional judgment to oversee BCB's water resources. These schedules were incorporated into the model as breakpoint data.

Additionally, there are structures operated by the local governmental agencies, and their operations are not well-known. **Table 3-3** summarizes the operations coded for the design event simulations. The modeling efforts for this task were based on the modeling performed for Hurricane Irma. The modifications made were as follows:

- Rainfall: Hurricane Irma observed rainfall was replaced by design event rainfall.
- Structure operations: The observed data used in Hurricane Irma structure operations were replaced by regulatory schedules, and wet and dry season operations as seen in Table 3-3, as documented in BCB Atlas (SFWMD, 2020).
- Structure operations control rules were prioritized in this order; new construction (if any), emergency operation, historical gate levels (which was only used in BCB Irma Model, not the design events), wet/dry season rules, and unchanged.

Table 3-3. Water Level for Wet/Dry Season Used for Structure Operations

Structure	Wet		Dry	
	Close (ft, NAVD)	Open (ft, NAVD)	Min (ft, NAVD)	Close (ft, NAVD)
ARN Amil D-500	-	6.25	-	7.25
ARS Amil D-710	-	6.23	-	7.23
CC-1	2.70	5.40	4.00	5.40
CC-2	6.70	9.00	7.50	9.00
CC-3	8.74	10.24	9.74	10.74
Cork 3	10.71	12.71	11.71	13.71
Cork 1	7.69	9.69	10.19	11.19
Cork 2	8.70	10.20	10.20	11.20
CR951N Gate	9.21	10.71	9.71	11.71
CR951S	5.69	6.69	6.69	7.69

Structure	Wet		Dry	
	Close (ft, NAVD)	Open (ft, NAVD)	Min (ft, NAVD)	Close (ft, NAVD)
CYP1 Gate	8.20	8.70	8.45	9.20
FU-3	4.86	5.36	5.11	5.86
FU-4	9.17	10.17	9.67	11.17
FU-5	11.18	12.18	12.18	13.18
FU-6	13.18	13.68	13.42	14.18
FU-7	15.94	16.94	16.94	17.44
GG-1	1.73	2.73	3.23	3.93
GG-2	3.96	4.71	4.46	5.21
GG-3	6.30	7.00	7.19	7.89
GG-4	7.70	8.70	8.50	9.20
GG-5	9.18	9.68	9.43	10.18
GG-6	13.49	15.19	14.69	15.49
GG-7	12.09	13.49	13.49	14.49
HC-1 Flap Gate	4.20	4.45	4.20	4.95
HC-1 Gate	2.70	4.20	4.20	4.70
HC-2	6.69	8.69	8.19	9.69
I75-1 Gate	4.91	5.41	5.16	5.91
I75-2 Gate	6.72	7.22	6.97	7.72
I75-3 Gate	7.74	8.74	8.24	9.74

Structure	Wet		Dry	
	Close (ft, NAVD)	Open (ft, NAVD)	Min (ft, NAVD)	Close (ft, NAVD)
Merritt-1	5.20	9.00	7.70	9.00
MIL-2	4.87	5.37	5.12	5.87
MIL-3	7.70	8.70	8.40	9.30
S487_Pump	4.90	9.00	5.20	9.00
Twin Eagles Gate	9.70	11.20	11.20	12.20
CUR_1	7.70	9.70	10.20	11.20

3.3 Initial and Boundary Conditions

SFWMD recommended using the 50th percentile of the wet season for initial conditions in the model. This date was selected as August 27th, 2017; the procedure used is detailed below.

3.3.1 Initial Conditions

A previous deliverable in the project involved a long-term simulation of the BCB system, which was utilized to select a date and extract initial conditions as agreed with the SFWMD. For this purpose, the proprietary tool (Time Statistics) developed by Lago Consulting was utilized. This tool computes statistical parameters of grid cell values in time-varying dfs0, dfs2, and dfs3 files. Similar to “TxStat” in the Mike Zero Toolbox, it provides seasonal averages and includes an option for specifying percentiles.

The water table levels during late August and early September 2017 were determined to reasonably represent the 50th percentile. After reviewing the temporal dataset, 8/27/2017 was selected as the optimal date to represent the average conditions of the wet season within the BCB Model domain, a decision approved by the SFWMD. For both 1D channel flow and 2D overland flow, this date was chosen as the initial condition to accurately represent the hydrologic conditions across the entire model domain. Screenshots of some of the dates that were examined are provided below.

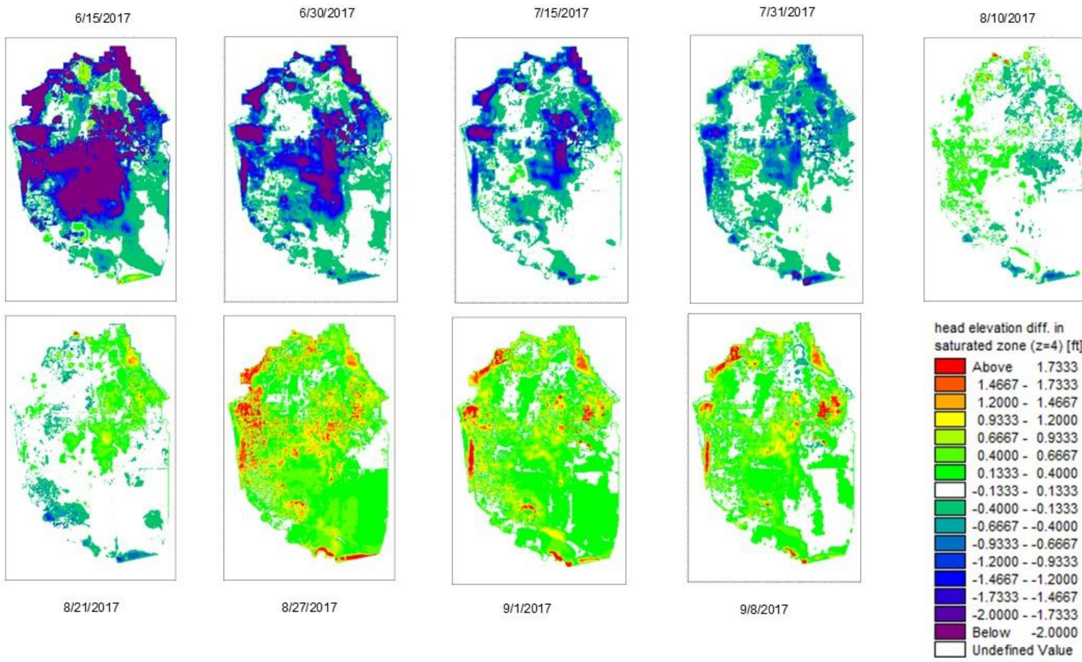

Figure 3-2. Sample dates considered for initial conditions during selection

3.3.2 Boundary Conditions

Tidal boundary conditions from storm surges, varying across different return periods, were utilized. These storm surge datasets, initially generated for the BCB FPLOS model, were applied to canals based on their proximity, as shown in **Table 3-4**.

Table 3-4. Boundary conditions time series data and their corresponding canals applied to**.**

Time series file	BC location to apply it
COCO1	Imperial River and Cocohatchee canal
GG1	Goldengate Main canal
HC1	Lely Canal, Henderson Creek, Fakaunion canal, and others.

4. Evaluation Of BCB Design Events Model performance for FPLOS PMs

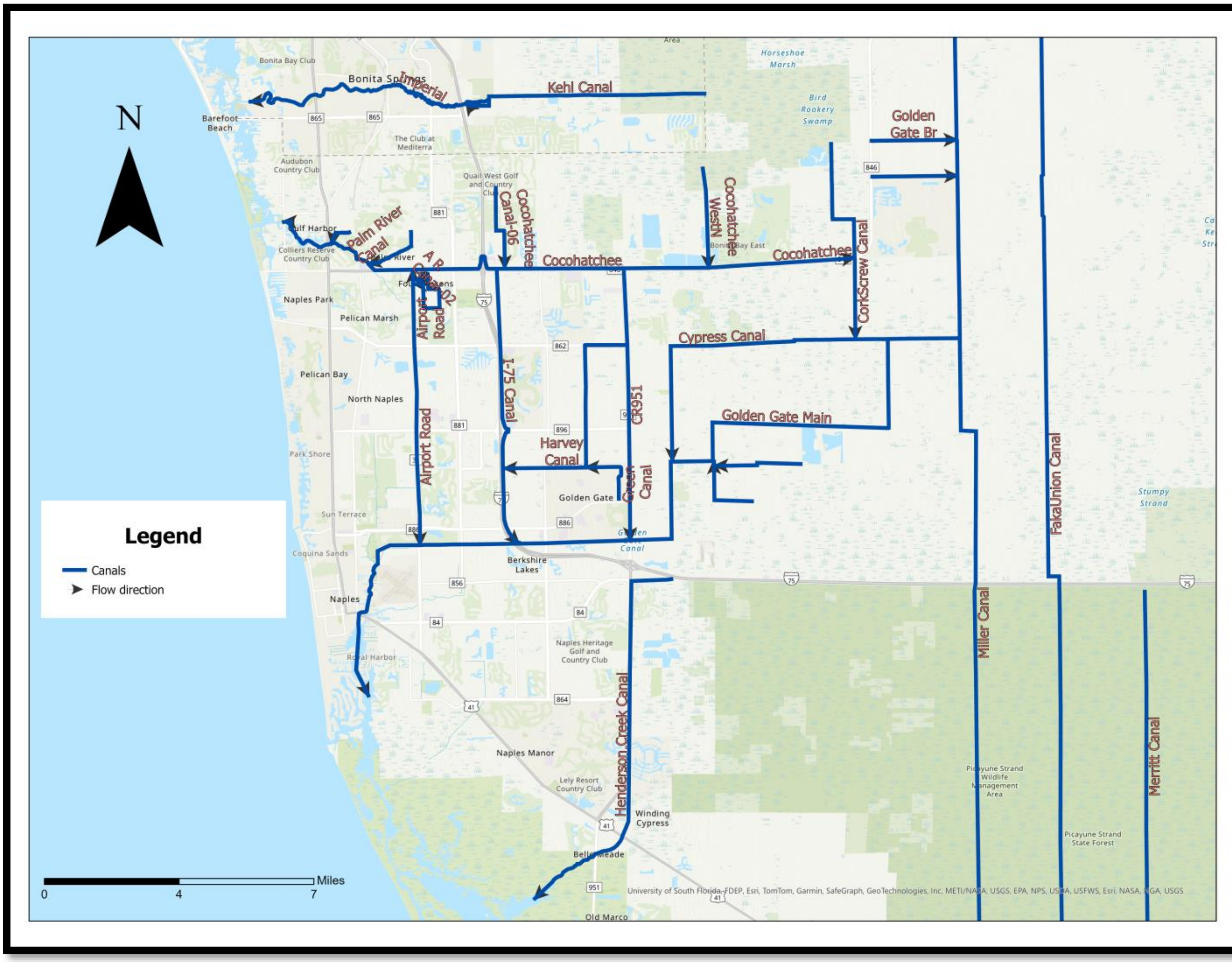

Figure 4-1. Major canals in BCB model domain

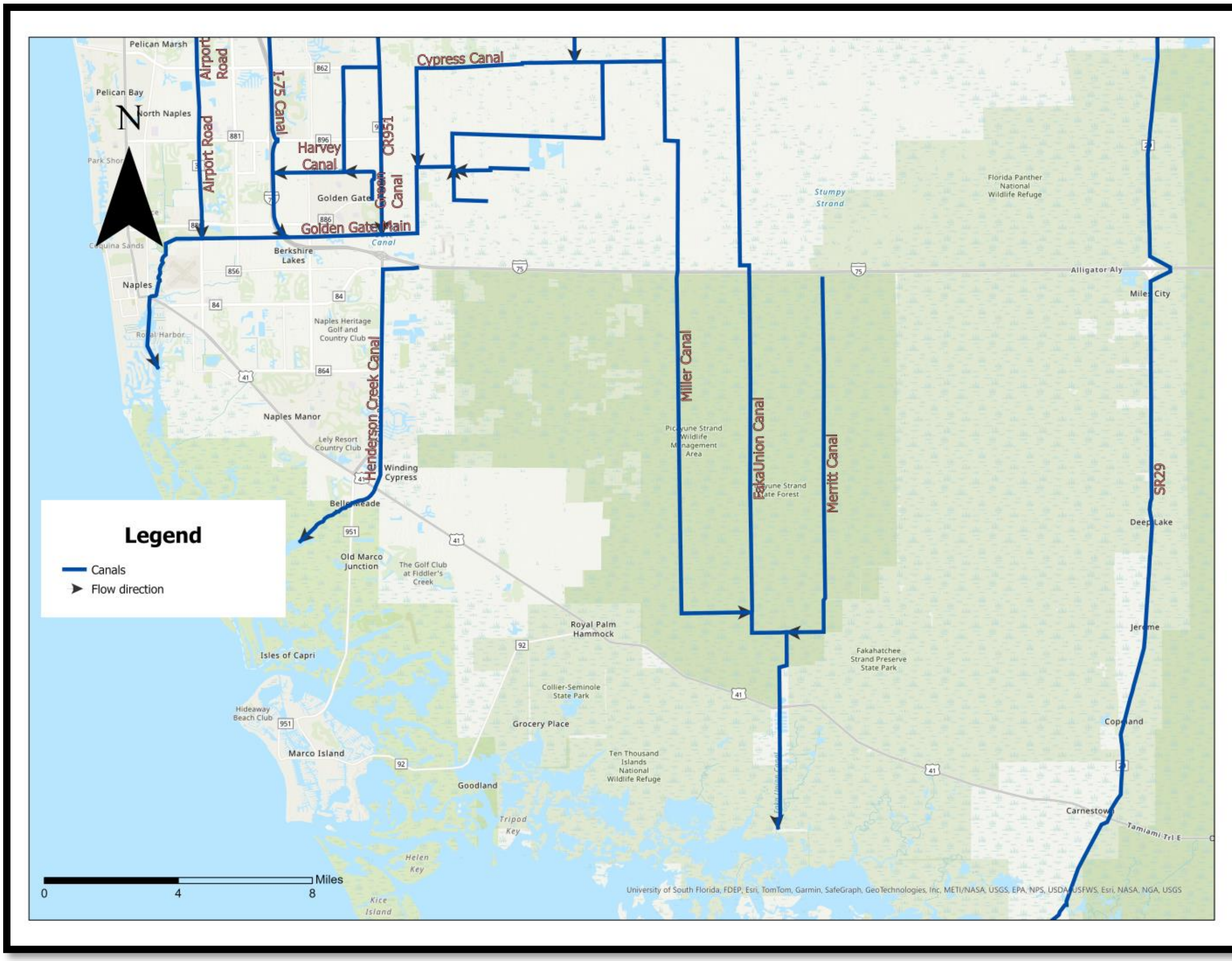

Figure 4-2. Major canals in BCB model domain

4.1 PM #1

This section presents the peak stage profile within the primary canal system (seen in, **Figure 4-1** and **Figure 4-2)** and its tributaries and some secondary system. The profiles were developed for various design storms (5-, 10-, 25-, and 100-yr events). The largest design storm that remains within the canal banks determines the Flood Protection Level of Service (FPLOS) for the primary canal system. To assess this performance measure under current conditions, instantaneous peak stage profiles were generated for all primary canals within or bordering the watersheds. The figures also depict significant roadway landmarks, control structures, and primary canal junctions. The nomenclature for the canals and structures follows the MIKE HYDRO notation (Parsons and Taylor Engineering, 2018).

Bank elevations on the profile plots were derived from MIKE HYDRO cross-section data. Generally, the canal segments evaluated under PM #1 do not achieve greater than a 5-yr.

FPLOS when strictly interpreting every cross-section and bank. However, as the water levels at these segments reflect local conditions and cross-sections were derived from LiDAR, which does not penetrate water, FPLOS was assessed under local conditions and overall. The overall rating considered the canals in the entire sub-basin, the magnitude of exceedance, whether one or both banks were exceeded, and the frequency of exceedance. **Table 4-1** show a summary of the PM1 evaluations whilst **Table 4-2** presents the peak flow for all four storm design events at BCB control structures.

- Airport Rd

The water profile for Airport Road Canal in **Figure 4-3** shows an increase in water levels from the beginning until culvert ARN-00-S0130 and a gradual downstream decrease in water level across all return period events, with the 100-yr event showing the highest water levels (above 12 feet) and the 5-yr event showing the lowest (approximately 6.9 feet at the downstream end). Water levels for all return periods remain below the left bank throughout all the reach, but there are a few areas where all water levels from the return period events exceed the right bank elevation.

Airport Road Canal partially meets flood protection levels of service. All events remain below the left bank throughout the reach, but there are various locations where all return periods exceed the right bank elevation.

- Airport Rd Canal 1

The water profile for Airport Road Canal 1 in **Figure 4-4** shows that water levels gradually decrease from upstream to downstream. The 100-yr event shows the highest water level (around 13 feet upstream), while the 5-yr event shows the lowest (around 10.5 feet downstream). The water levels for all return periods remain above both left and right bank elevations throughout most of the reach, except for a section between Road - Summit Pl. and Road - Winterview Dr. where water levels drop below both banks. Towards the end of the reach, water levels stay below the right bank while remaining above the left bank.

Airport Road Canal 1 does not meet flood protection levels of service for any design events. All events exceed both banks throughout most of the reach.

- Airport Rd Canal 2

The water profile in **Figure 4-5** for Airport Road Canal 2 shows that the water levels remain nearly constant throughout the complete reach, with very minimal variation. The 100-yr event maintains approximately 13 feet, the 25-yr event stays around 12.70 feet, the 10-yr event remains at about 12.5 feet, and the 5-yr event holds steady at approximately 12.16 feet. Water levels for all return periods consistently exceed both left and right bank elevations throughout most of the reach.

Airport Road Canal 2 does not meet flood protection levels of service for any design events (5-, 10-, 25-, and 100-yr) as water levels consistently exceed both left and right bank elevations throughout most of the reach, with minimal variation in water levels.

- Cocohatchee Canal 1

Figure 4-6 shows the water profile for Cocohatchee Canal 1; it shows minimal variation in water levels across the complete reach, with a slight decrease from upstream to downstream. The 100-yr event shows the most notable change, decreasing from 7.3 feet at the upstream end to 6.8 feet at the downstream end, while the other return periods (5-, 10-, and 25-yr events) maintain nearly constant water levels throughout the reach. Water levels for all return periods stay above both bank elevations throughout the entire reach.

Cocohatchee Canal 1 does not meet flood protection levels service for any design events (5-, 10-, 25-, and 100-yr) as water levels stay above both bank elevations throughout the entire reach, indicating potential flooding concerns for even smaller storm events.

- Cocohatchee Canal 6

The water profile for Cocohatchee Canal 6 in **Figure 4-7** shows a significant decrease in water levels from upstream to downstream. The 100-yr event shows the highest water level, decreasing from 16.13 feet at the upstream end to 13.31 feet downstream, while the 5-yr event shows the lowest levels, dropping from 15.17 feet to 11.61 feet. Water levels for the 100-yr event exceed both bank elevations at the upstream end, while other events approach but generally remain below the right bank but above the left bank elevations at the end of upstream. All events stay below both banks throughout most of the reach, except near the upstream end.

Cocohatchee Canal 6 partially meets flood protection levels service for 5- and 10-yr design events. All design events exceed bank elevations at the upstream end (first 600 feet) until Weir - QuailWest_Outfall, with the 100-yr event exceeding both banks and other events exceeding the left bank. However, all events stay below both banks throughout the remainder of the reach.

- Cocohatchee Canal

The water profile for Cocohatchee Canal in **Figure 4-8** shows a decrease in water levels from upstream to downstream. The 100-yr event shows the highest water level, starting at 14.64 feet at the upstream end and dropping significantly to 6.79 feet at the downstream end. Similarly, the 25-yr event decreases from 14.22 feet to 5.87 feet, the 10-yr event from 13.38 feet to 5.32 feet, and the 5-yr event from 13.12 feet to 4.92 feet. Notable drops in water levels occur around Culvert - CRB-00-S0260 and Weir - CC-1 Spillway. Water levels for the 100-yr event exceed the right bank elevations in some upstream sections and both banks near Culvert – Coles Br, while lower return period events generally remain below both banks except in a few small specific locations.

Cocohatchee canal does not meet flood protection service levels for any design events but partially for the 5-, 10-, and 25-yr events, which generally remain below both banks throughout most of the reach, while the 100-yr event exceeds right bank elevations in some upstream sections and both banks near Culvert - Col-esBr. Some localized exceedances occur for all events in locations around Road – NW 25th Ave. and Culvert – Coles Br.

- Cocohatchee Canal West N

Figure 4-9 shows the water profile for Cocohatchee Canal West N; it shows a gradual rise in the beginning and afterwards fall in water levels across the canal reach. Water levels for all return periods remain below both bank elevations throughout most of the reach.

Cocohatchee Canal West N does not meet flood protection service levels for any design events. However, it meets the protection service levels partially for all design events (5-, 10-, 25-, and 100-yr) as water levels remain mostly below both bank elevations throughout the reach, with some exceptions around Weir - Coco West N and downstream areas.

- Corkscrew Canal

The water profile in **Figure 4-10** for Corkscrew Canal shows a gradual decrease in water levels from upstream to downstream across the entire reach. A notable drop in water levels occurs around the middle section of the canal, particularly near Culvert - CCB-00-S0110 and Road - 33rd Ave NW. Water levels for all return periods alternate between exceeding and falling below both bank elevations throughout the reach, particularly in sections between structures.

Corkscrew Canal does not meet flood protection service levels for any return period as water levels alternate between exceeding and falling below both bank elevations throughout the reach, with consistent overtopping occurring in multiple locations.

- CR 951 Canal

The water profile for CR951 Canal in **Figure 4-11** shows a gradual decrease in water levels from upstream to downstream. The 100-yr event shows the highest water level, starting at 14.76 feet at the upstream end and decreasing significantly to 11.57 feet at the downstream end. Similarly, the 25-yr event decreases from 14.42 feet to 11.32 feet, the 10-yr event from 14.19 feet to 11.12 feet, and the 5-yr event from 13.95 feet to 10.93 feet. Water levels for all return periods remain below both bank elevations throughout most of the reach, though there are a few sections where the water level approaches or exceeds the right bank elevation.

CR951 Canal mostly meets flood protection levels for all return periods (5-, 10-, 25-, and 100-yr) throughout the reach, with only a few localized areas where water levels approach or exceed the right bank elevation. It provides the best protection level service for the 5- and 10-yr design events.

- Cypress canal

The water profile for Cypress Canal in **Figure 4-12** shows a gradual decrease in water levels from upstream to downstream. The 100-yr event demonstrates the highest water level, starting at 13.33 feet at the upstream end and decreasing to 12.04 feet at the downstream end. Similarly, the 25-yr event decreases from 13.09 feet to 11.80 feet, the 10-yr event from 12.91 feet to 11.63 feet, and the 5-yr event from 12.73 feet to 11.48 feet. The left and right bank elevations show significant variations throughout the reach, with several peaks and dips, particularly around Road - 13th St NW and Weir - Cyp-1. Water levels for all return periods fluctuate between exceeding and falling below both bank elevations throughout the reach, with several sections where water levels exceed the right bank, particularly between Road - 8th St NE and Road - 13th St NW, while remaining below the left bank in most locations.

Cypress Canal does not meet flood protection service levels for any of the design events. The 5- and 10-yr events generally remain below banks throughout 50% of the canal, while the 25- and 100-yr events exceed bank elevations in most of the locations.

- Faka Union

The water profile in **Figure 4-13** for the Faka Union Canal shows an initial spike due to model instability then, the water levels quickly stabilize and then gradually decrease downstream, with the 100-yr event dropping from about 20 feet to 6.77 feet, the 25-yr event from 19 feet to 5.84 feet, the 10-yr event from 18.72 feet to 5.30 feet, and the 5-yr event from 18.62 feet to 5.20 feet. The left and right bank elevations show considerable variation throughout the reach, with several peaks and valleys, particularly in the middle portion. Water levels exceed both bank elevations in several locations, most notably near Weir - FU-3 and Gate - FU-4, while mostly remaining below bank elevations in other sections.

Faka Union Canal does not meet flood protection service levels for any return period (5-, 10-, 25-, and 100-yr) in the upstream portion where there's a significant spike. However, it partially meets protection levels in the areas from Weir - FU-7 to Road – Golden Gate Blvd E, where all events water levels remain below both bank elevations.

- Golden Gate 9

The water profile for Golden Gate 9 canal in **Figure 4-14** shows minimal variation in water levels across the entire reach. The 100-yr event shows the highest water level, maintaining a nearly constant level around 12.15 feet throughout the reach. Similarly, the 25-yr event remains steady at approximately 11.92 feet, the 10-yr event at 11.75 feet, and the 5-yr event shows a slight decrease from 11.64 feet to 10.54 feet. Water levels for all return periods remain below both bank elevations throughout the entire reach, except around the areas around Road – SW 23rd St.

Golden Gate 9 does not meet flood protection service levels for all design events since all water levels exceeded both banks around Road – SW 23rd St but remain below banks elsewhere.

- Golden Gate 10

The water profile for the Golden Gate 10 canal in **Figure 4-15** shows minimal variation in water levels across the entire reach. The 100-yr event demonstrates the highest water level, maintaining a nearly constant level throughout the reach. Similarly, the 25-yr, 10-yr, and 5-yr event remains steady throughout the reach. Water levels for all return periods remain below both bank elevations throughout most of the reach, except where all design events exceed the left bank in the first half of the reach.

Golden Gate 10 does not meet flood protection service levels for all design, since all events exceed the left bank in the first half of the reach but remain below banks elsewhere.

- Golden Gate 12

The water profile for the Golden Gate 12 canal captured in **Figure 4-16** shows a consistent decrease in water levels from upstream to downstream. The 100-yr event demonstrates the highest water levels, decreasing from 16.34 feet at the upstream end to 16.18 feet at the downstream end. Similarly, the 25-yr event decreases from 16.00 feet to 15.65 feet, the 10-yr event from 15.60 feet to 15.27 feet, and the 5-yr event shows the lowest levels, dropping from 15.01 feet to 14.65 feet. Water levels for all return periods remain below both bank elevations throughout most of the reach except in a small area where 100-yr event exceeds the right bank, with the higher separation between water levels and bank elevations occurring at the downstream end of the canal.

Golden Gate 12 meets flood protection levels for all return periods (5-, 10-, 25-, and 100-yr) as water levels remain below both bank elevations in most of the entire reach.

- Golden Gate Br

Figure 4-17 shows the water profile for Golden Gate Br; this shows a gradual decrease in water levels from upstream to downstream across the reach. Water levels for all return periods remain below both bank elevations for most of the reach.

Golden Gate Br partially meets flood protection service levels as water levels exceeded the left bank in some small portion of the reach.

- Golden Gate Main

The water profile for Golden Gate Main Canal in **Figure 4-18** shows a significant decrease in water levels from upstream to downstream. The 100-yr event shows the highest water levels, decreasing from 19.14 feet at the upstream end to 6.77 feet at the downstream end. Similarly, the 25-yr event decreases from 18.51 feet to 5.86 feet, the 10-yr event from 17.90 feet to 5.23 feet, and the 5-yr event from 17.24 feet to 4.84 feet. The left and right bank elevations show considerable variation throughout the reach, with several notable spikes and dips, particularly around Road - Golden Gate Blvd E and Airport Pulling Rd N, and Weir **-** GG6. Water levels for all return periods remain above both bank elevations throughout most of the reach, except for a few locations where some events approach or slightly exceed bank elevations, particularly near some road crossings.

Golden Gate Main canal does not meet flood protection levels for any return period (5-, 10-, 25-, or 100-yr) as water levels remain above both bank elevations throughout most of the reach, with only a few locations where water levels stay below banks near road crossings.

- Green Canal

Figure 4-19 shows the water profile in Green Canal; this shows minimal variation in water levels across the reach. The 100-yr event demonstrates the highest water surface elevations, maintaining a constant level. Similarly, all other events remain steady throughout the reach. Water levels for all return periods remain below both bank elevations throughout most of the reach, except for some locations where the water levels exceed the bank elevations.

Green Canal does not meet flood protection service levels for events throughout the reach.

- Harvey Canal

The water profile in **Figure 4-20** for Harvey Canal shows minimal to no variation in water levels across the entire reach. The water levels for all design events stayed between 10 and 11 feet. Water levels for all return periods remain below both bank elevations throughout the entire reach, except the area around Bridge - GCB-00-S0100 and around the chainage of 18700 downstream, where the water level exceeds one or both banks.

Harvey Canal does not meet flood protection service levels for all events. The water levels for all events exceed bank elevations near Bridge - GCB-00-S0100.

- Henderson Creek Canal

The water profile for Henderson Creek Canal in **Figure 4-21** shows a gradual decrease in water levels from upstream to downstream across the reach. The 100-yr event shows the highest water levels, starting at 11.027 feet at the upstream end, maintaining relatively high levels until around Road - Veronawalk Blvd, where it begins to decrease significantly, eventually dropping to 6.77 feet at the downstream end. Similarly, the 25-yr event decreases from 10.68 feet to 5.86 feet, the 10-yr event from 10.45 feet to 5.31 feet, and the 5-yr event from 10.27 feet to 4.88 feet. Water levels for all return periods generally remain below both bank elevations throughout most of the reach. However, there are some locations where these events approach or exceed either one of both bank elevations, particularly around Road - Tamiami Trl E and after the chainage 45000.

Henderson Creek Canal does not meets flood protection service levels for all return periods (5-, 10-, 25-, and 100-yr). Water levels generally remain below both or one of the banks throughout most of the reach but exceed or approach bank elevations at a few specific locations: around Road - Tamiami Trl E and after chainage 45800.

- I-75 Canal

Figure 4-22 captures the water profile for the I-75 Canal; this shows a gradual increase in water levels from upstream to downstream. Water levels for all return periods generally remain below both bank elevations throughout most of the reach, though there are some locations where some of the events exceed either one (mostly right bank) or both bank elevations, particularly around Road - Shady Oaks Ln.

I-75 Canal mostly meets flood protection levels for all return periods (5-, 10-, 25-, and 100-yr) throughout the reach, with only localized exceedances around Road - Shady Oaks Ln, where almost all design events exceed both bank elevations.

- Kehl Canal

The water profile for Kehl Canal shows a gradual decrease in water levels from upstream to downstream; captured in **Figure 4-23**. The 100-yr event shows the highest water surface elevations, decreasing from 16.66 feet at the upstream end (near Pionner Rd) to 11.16 feet at the downstream end. Similarly, the 25-yr event decreases from 16.43 feet to 10.67 feet, the 10-yr event from 16.26 feet to 10.23 feet, and the 5-yr event from 16.14 feet to 9.73 feet. Water levels for all return periods remain above both bank elevations throughout most of the reach, though there are some locations where some of the events approach bank elevations or exceed one of the banks, particularly in the middle portions of the canal. After Culvert - Imperial-BonGRBR towards downstream, all events stay below both banks.

Kehl Canal partially meets flood protection service levels for all return periods (5-, 10-, 25-, and 100-yr) but only in the downstream section after Culvert - Imperial-BonGRBR. The upstream and middle portions do not meet protection levels as water levels exceed bank elevations.

- Miller Canal

The water profile for Miller Canal in **Figure 4-24** shows a gradual decrease in water levels from upstream to downstream. The 100-yr event shows the highest water level, decreasing from 13.69 feet at the upstream end to 5.52 feet at the downstream end. Similarly, the 25-yr event decreases from 13.40 feet to 5.16 feet, the 10-yr event from 13.15 feet to 4.82 feet, and the 5-yr event from 12.95 feet to 4.56 feet. Multiple structures, including Bridge - MLC-00-S140, Gate - MLC-L2 branch, Weir - Miller - 2, and Road – SE 28th Ave influence the water surface profiles. Water levels for all return periods generally remain above both bank elevations throughout most of the reach. However, there are some locations where the water level approaches bank elevations or stays below banks, particularly around Road - Golden Gate Blvd E and Road - SE 28th Ave.

Miller Canal does not fully meet flood protection levels for any return period but partially meets protection levels for all events (5-, 10-, 25-, and 100-yr) near Road - Golden Gate Blvd E, where water levels stay below bank elevations.

- Palm River Canal

The water profile along Palm River Canal shows a constant water level until Road – Cypress Way E and then a gradual decrease in water levels from upstream to downstream; this is in **Figure 4-25**. The 100-yr event demonstrates the highest water surface elevations, decreasing from 10.13 feet at the upstream end to 8.69 feet at the downstream end. Similarly, the 25-yr event decreases from 9.48 feet to 7.97 feet, the 10-yr event from 8.84 feet to 7.37 feet, and the 5-yr event from 8.28 feet to 6.87 feet. Three notable structures influence the water level profiles: Road - Cypress Way E, Road - Palm River Blvd, and Gate - PALM-00-S010Q. Water levels for all return periods exceed both bank elevations throughout the entire reach.

Palm River Canal does not meet flood protection levels of service for any return periods (5-, 10-, 25-, or 100-yr events) as all water levels exceed both bank elevations throughout the entire reach.

Table 4-1. Summary of PM #1 Evaluation

Sub-basin Name	Local FPLOS	Overall FPLOS	Comments
Estero Bay	<5	5	This sub-basin which has the City of Bonita Springs, and the Imperial River has water levels for all design events above banks in most locations
Cocohatchee	<5	10	Cocohatchee provides a 25-yr FPLOS for most of the main canal. For Airport Rd Canal, water levels remain below left bank for almost all of the events, For Palm River, the water levels exceed the banks for all events
Golden Gate Main + Trafford	<5	10	Water levels in Golden Gate Main Canal exceed the bank elevations in most locations, however the tributaries of Golden Gate canal are able to contain the water within banks along most of its length. CR951 meets FPLOS for all events except few localized areas. All design events exceed the banks in many locations along Cypress Canal. Harvey and Green canals meet the 25-yr FPLOS generally. I75 canal meets FPLOS for all events except for localized areas
Henderson-BelleMeade	<5	25	The left and right banks of Henderson Creek contain the water levels at most locations except upstream of HC1, near Tamiami Trl
Faka Union	<5	10	Faka Union canal water levels stay within banks except near Alligator Alley and Tamiami Trl. Miller Canal water levels exceed the banks at most locations.

It should be noted that there are some isolated cases of the hydrograph from a smaller event exceeding the bigger event in the PM #1 profiles – for example, at Kehl Canal, FU-1 Canal, the stage from the 5-yr event exceeds the stage from the 100-yr event. On examination of the time series data from the model results, it is evident that this happens for a single time step only, and the stages return to what is expected. This is an instability that was not investigated further due to time constraints.

Table 4-2. Summary of peak stages at control structures

Structure	Canal	Chainage	Peak Stages (ft)			
			5 yr	10 yr	25 yr	100 yr
ARN Amill D-500	AirportRdN	4593	10.81	11.23	11.61	12.00
Air 1	AirportRdS	11697	9.39	9.70	10.03	10.44
ARN Amill D-700	AirportRdS	22199	8.80	9.00	9.23	9.52
CC4	CocohatcheeEast	1499	13.19	13.36	13.56	13.81
CC3	CocohatcheeWest	26164	13.47	13.68	13.95	14.35
CC2	CocohatcheeWest	39202	10.36	10.95	11.59	12.17
CC1	CocohatcheeWest	50002	8.22	8.70	9.30	9.99
Cork2	CorkScrewCan	2789	14.53	14.73	14.98	15.35
Cork1	CorkScrewCan	18357	12.95	13.12	13.33	13.62
CR951S	CR951	35701	11.22	11.32	11.40	11.57
CR951N	CR951	10352	13.25	13.38	13.51	13.65
Cyp1	CypressCan	22472	12.38	12.54	12.70	12.93
FU7	FakaUnionCan	11155	17.82	18.11	18.60	19.68
FU6	FakaUnionCan	22741	15.63	16.31	16.97	17.69
FU5	FakaUnionCan	33630	14.48	15.01	15.49	16.08
FU4	FakaUnionCan	62444	12.12	12.30	12.51	13.52
FU3	FakaUnionCan	81397	10.39	10.72	10.87	11.20
FU2	FakaUnionCan	112051	7.31	7.54	7.84	8.10
FU1	FakaUnionCan	152754	4.16	4.44	4.77	5.21
GG 7	GoldenGateBr	7792	15.22	15.93	16.52	16.98
GG 6	GoldenGateMain	13452	16.06	16.62	17.22	17.81
GG 5	GoldenGateMain	30297	13.72	14.08	14.41	14.74
GG 4	GoldenGateMain	51430	12.72	12.89	13.07	13.31
GG 3	GoldenGateMain	109592	10.89	11.11	11.33	11.57
GG 2	GoldenGateMain	127116	9.33	9.61	9.88	10.13
GG 1	GoldenGateMain	140755	6.16	6.61	7.10	7.76
Harvey1	HarveyCan	8531	10.03	10.26	10.52	10.81
HC2	HendersonCr	15206	9.55	9.77	10.03	10.34
HC1	HendersonCr	40459	4.74	5.08	5.45	6.00
I751	I-75Can	37184	9.74	9.99	10.27	10.59
I752	I-75Can	21523	10.96	11.22	11.53	11.91
I753	I-75Can	10298	11.88	12.08	12.32	12.58
Kehl1	KehlCan	26001	13.17	13.52	14.13	14.98
Miller 3	MillerCan	465	12.92	13.14	13.37	13.66
Miller 2	MillerCan	36409	10.23	10.31	10.40	10.76
Miller 1	MillerCan	69377	6.76	6.95	7.13	7.43

4.1.1 Comparison Between Simulated Tailwater Stages and Storm Surge Tidal Boundary Conditions.

Generally, coastal boundary conditions are represented as stage hydrographs located on the downstream (tidal) side of the structure for different return periods, sea level rise projections, and planning horizons. The tidal boundary conditions in the BCB Update Model's design events simulations used stages generated by SFWMD (2017). The coastal boundary conditions for the SFWMD LOS pervious modeling effort (SFWMD, 2017) included the water levels downstream of the coastal tidal water control structures COCO1, GG1, and HC1, which are associated with the Cocohatchee, Golden Gate Main, and Henderson basins, respectively. These tidal structures are SFWMD water control structures, designed to discharge water from the interior watersheds to the ocean. They also prevent seawater from entering the upstream reaches of the canal when the gates are closed. SFWMD's process to produce the boundary condition hydrographs included frequency analysis of historical data to determine extreme stages for various return periods. A base storm hydrograph is then selected from the historical data and re-scaled to match the peak extreme value derived from the frequency analysis. Finally, offset values from sea level rise projections were added to the re-scaled hydrographs to generate the final boundary conditions. More details can be found in SFWMD (2017).

Figure 4-26 through **Figure 4-33** present time-series plots comparing tailwater stages at structures COCO1, GG1, HC1, and FU1 for both the 10-yr and 5-yr design events. These plots include the boundary conditions used, simulated stages at the tailwater of the structures and stages at the location where the boundary conditions were applied. As the boundary conditions were applied several miles downstream of the structures, this approach allows for an intuitive understanding of how the boundary conditions translate to water levels at the structures themselves.

The results indicate that the peak simulated water stages at the structures are consistently higher than the observed stages. However, the hydrograph sequences for both the 10-year and 5-year design events show a strong correlation, with the 10-year event exhibiting predictably higher stages. Notably, a 98% resemblance was observed between the simulated stages at the boundary condition chainage and the observed storm surge tidal boundary conditions for both design events, showing the impact of the application of the boundary conditions. This consistency explains the variations in canal profiles downstream of these structures, as depicted in the PM1 plots.

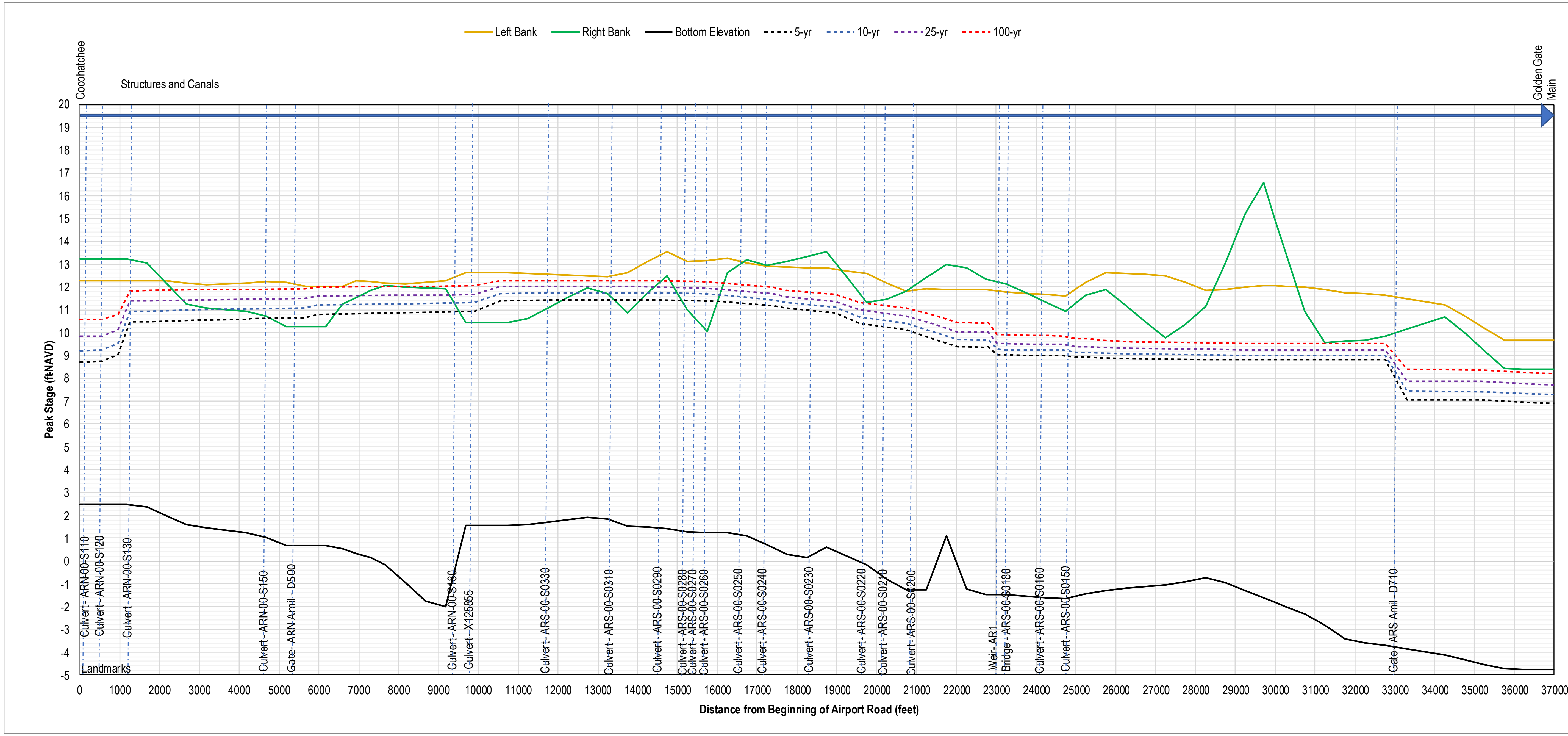

Figure 4-3. Water Profile for Airport Road

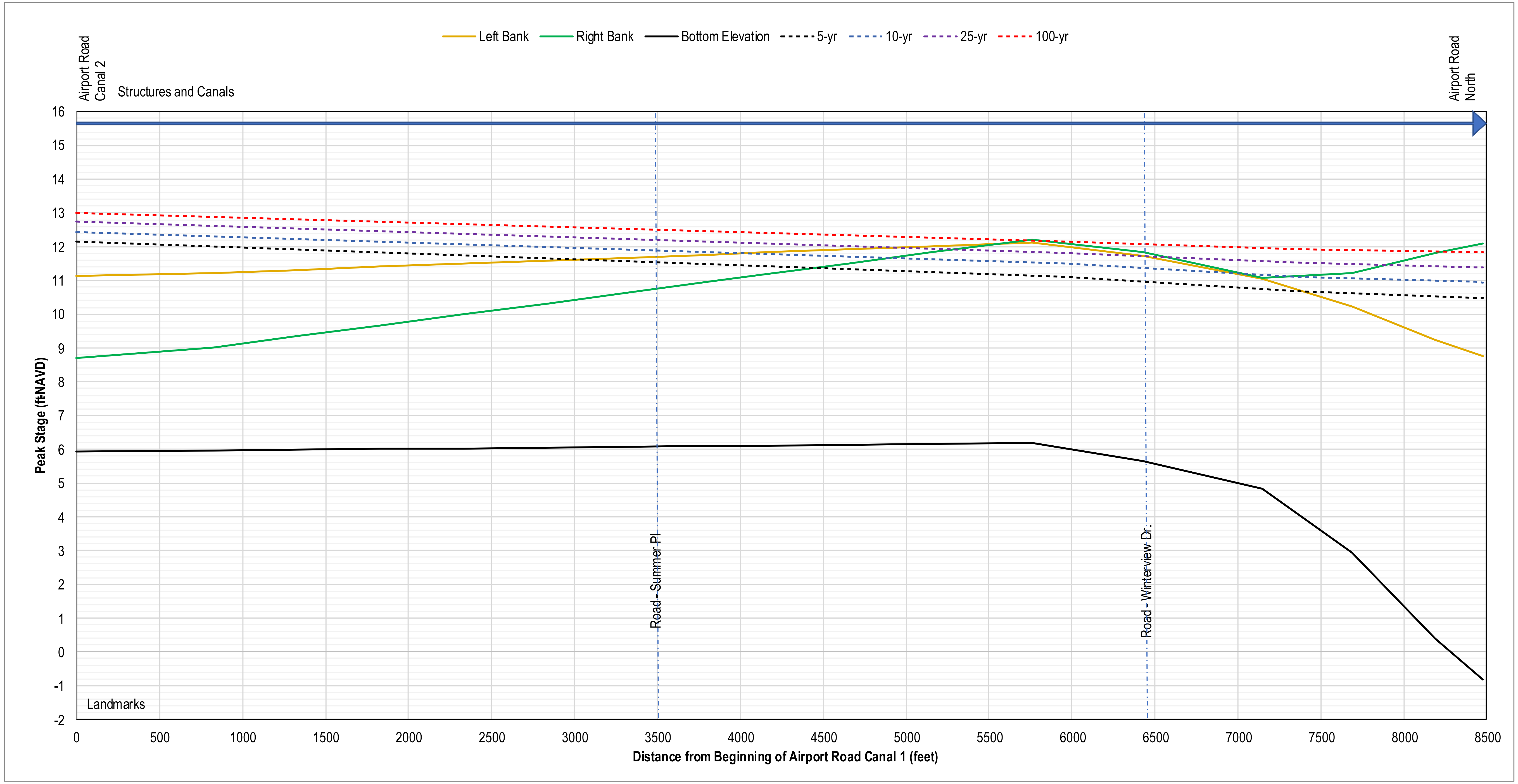

Figure 4-4. Water Profile for Airport Road Canal 1

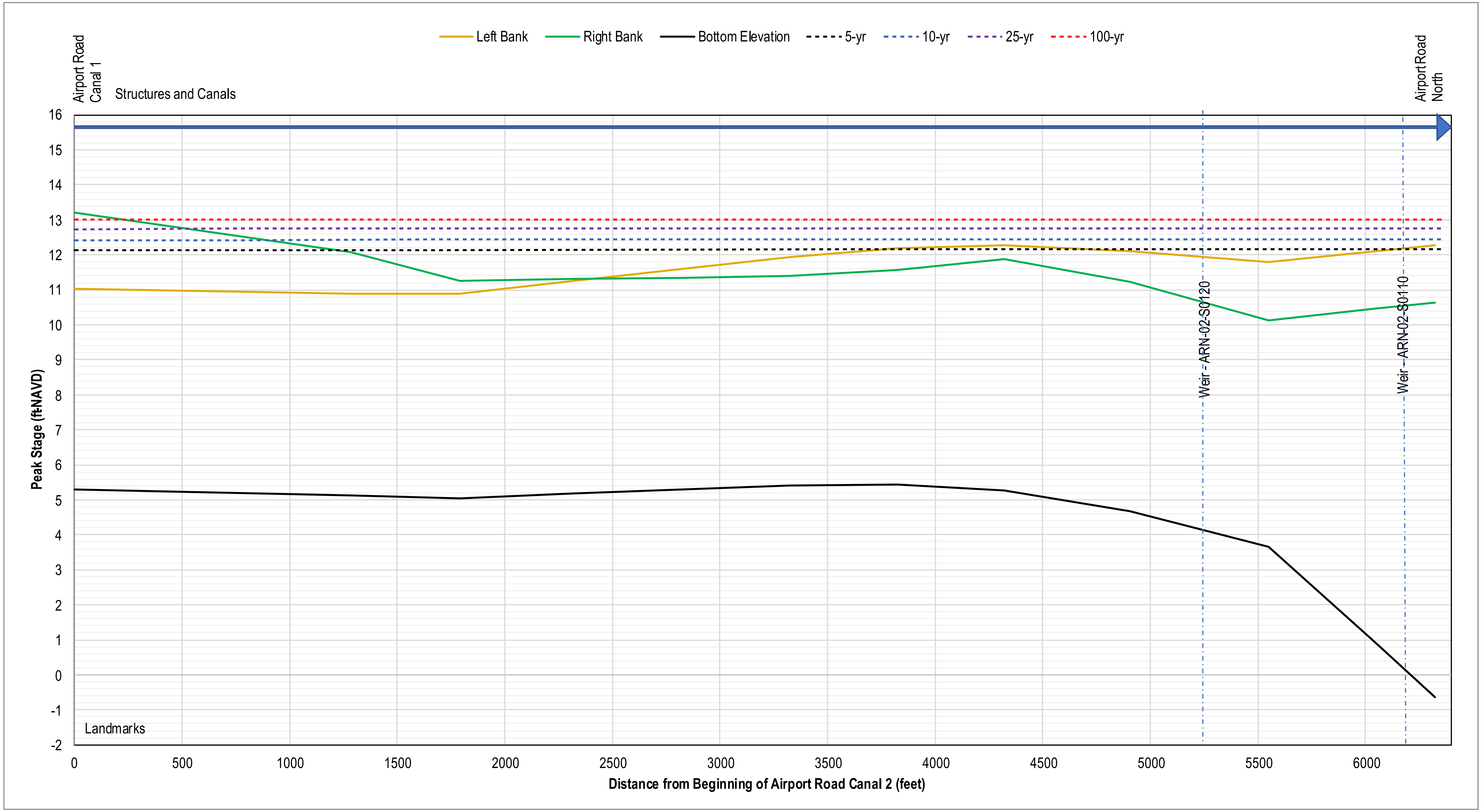

Figure 4-5. Water Profile for Airport Road Canal 2

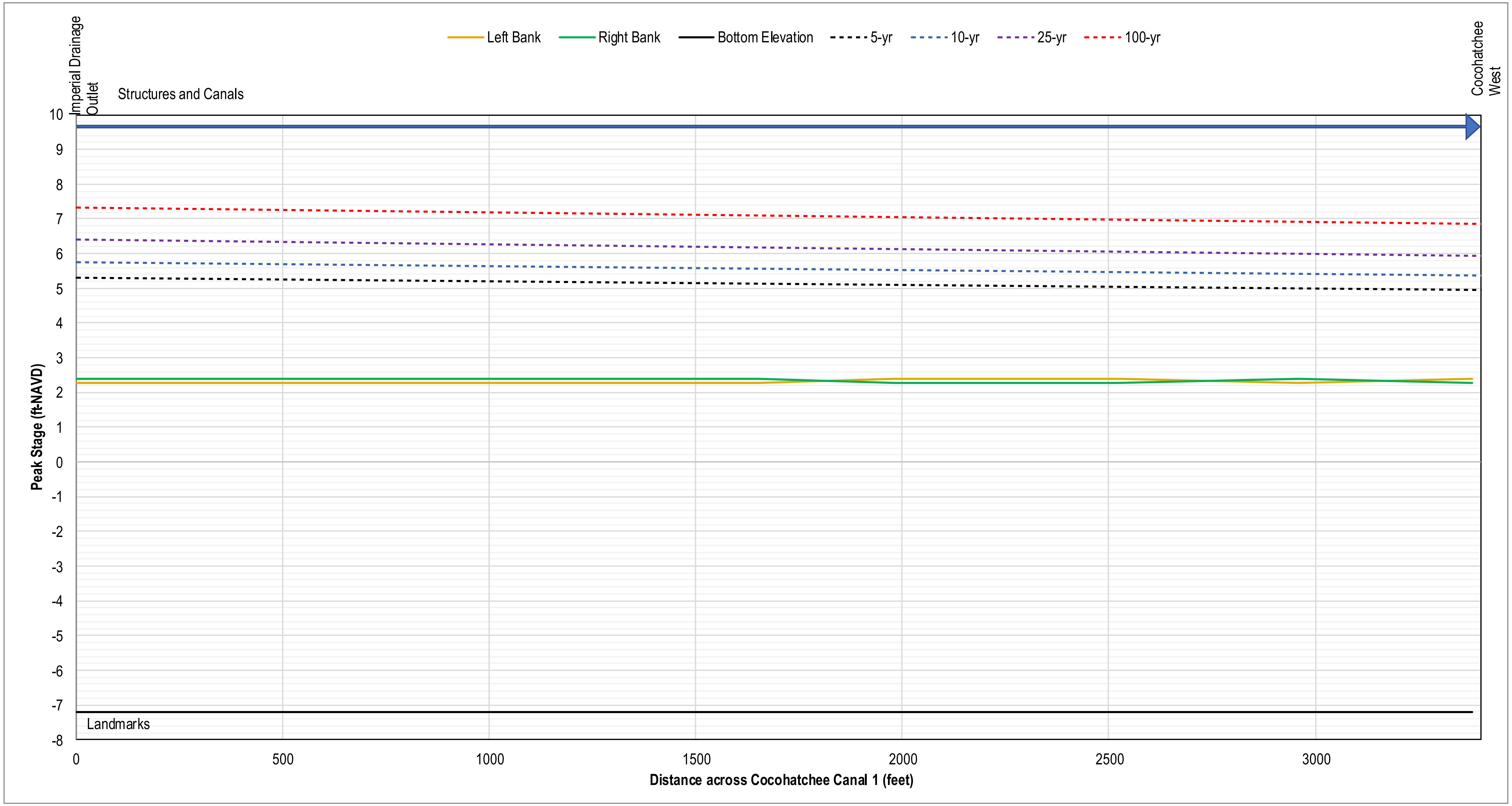

Figure 4-6. Water Profile for Cocohatchee Canal 1

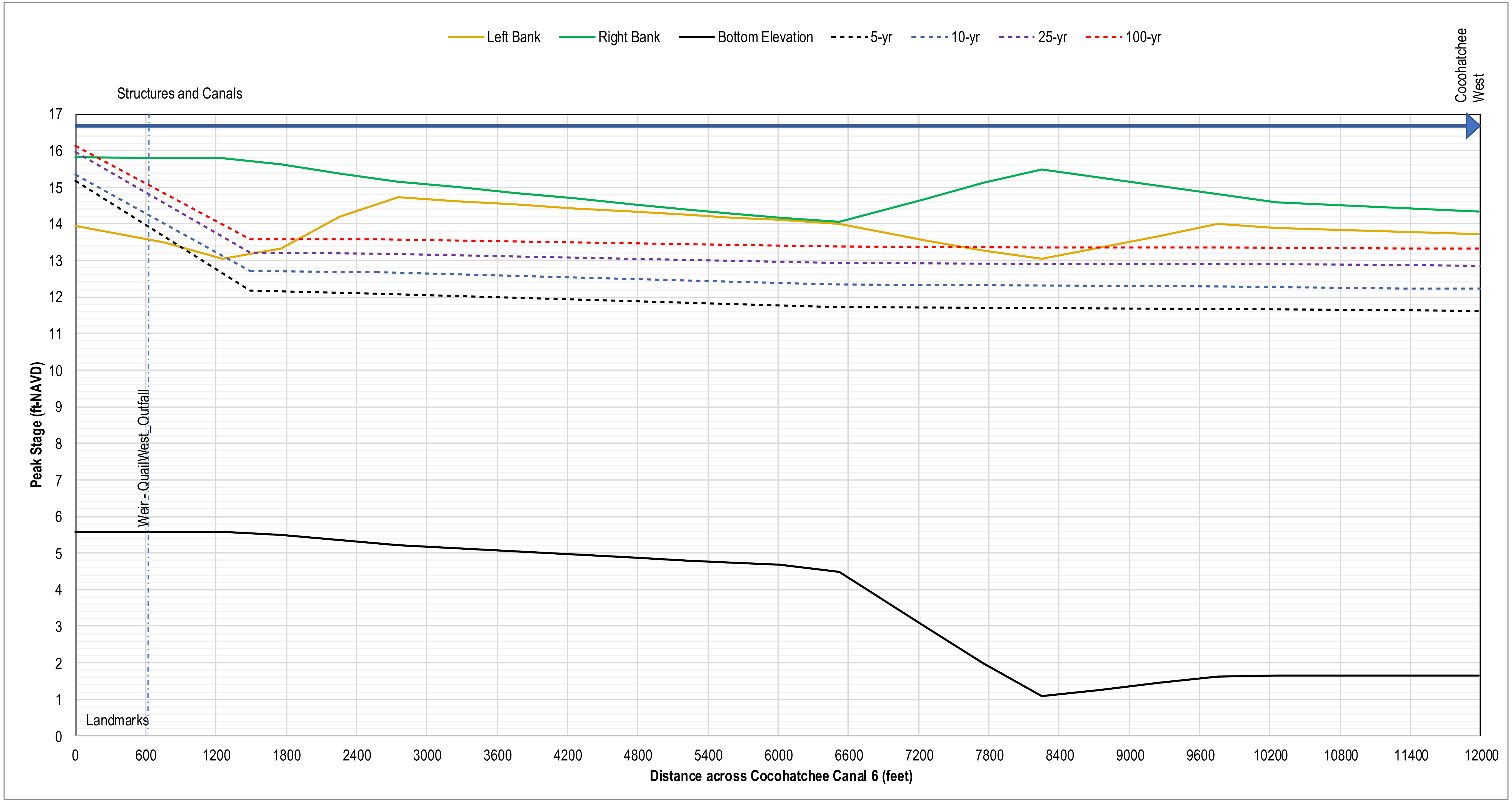

Figure 4-7. Water Profile for Cocohatchee Canal 6

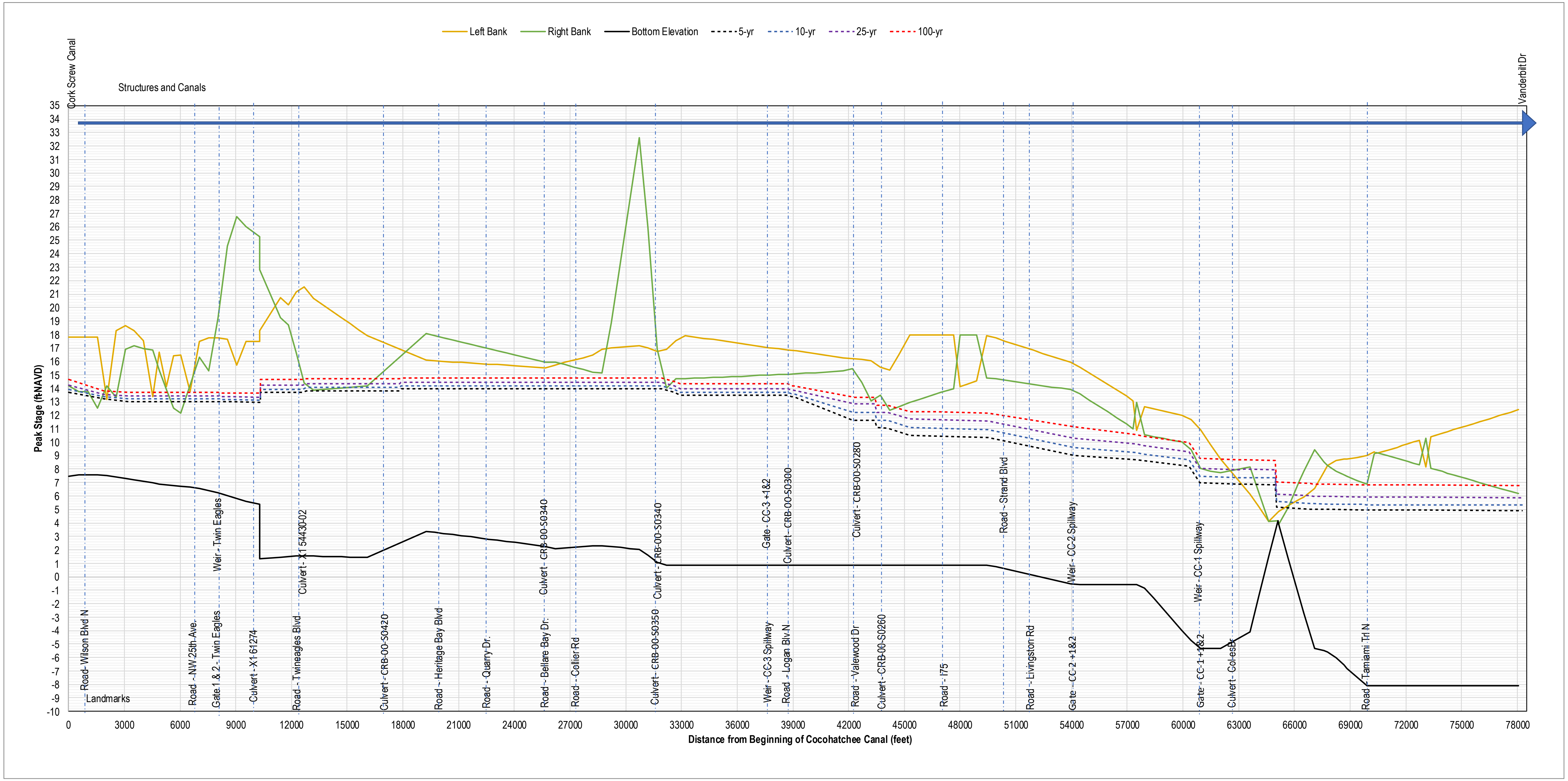

Figure 4-8. Water Profile for Cocohatchee Canal

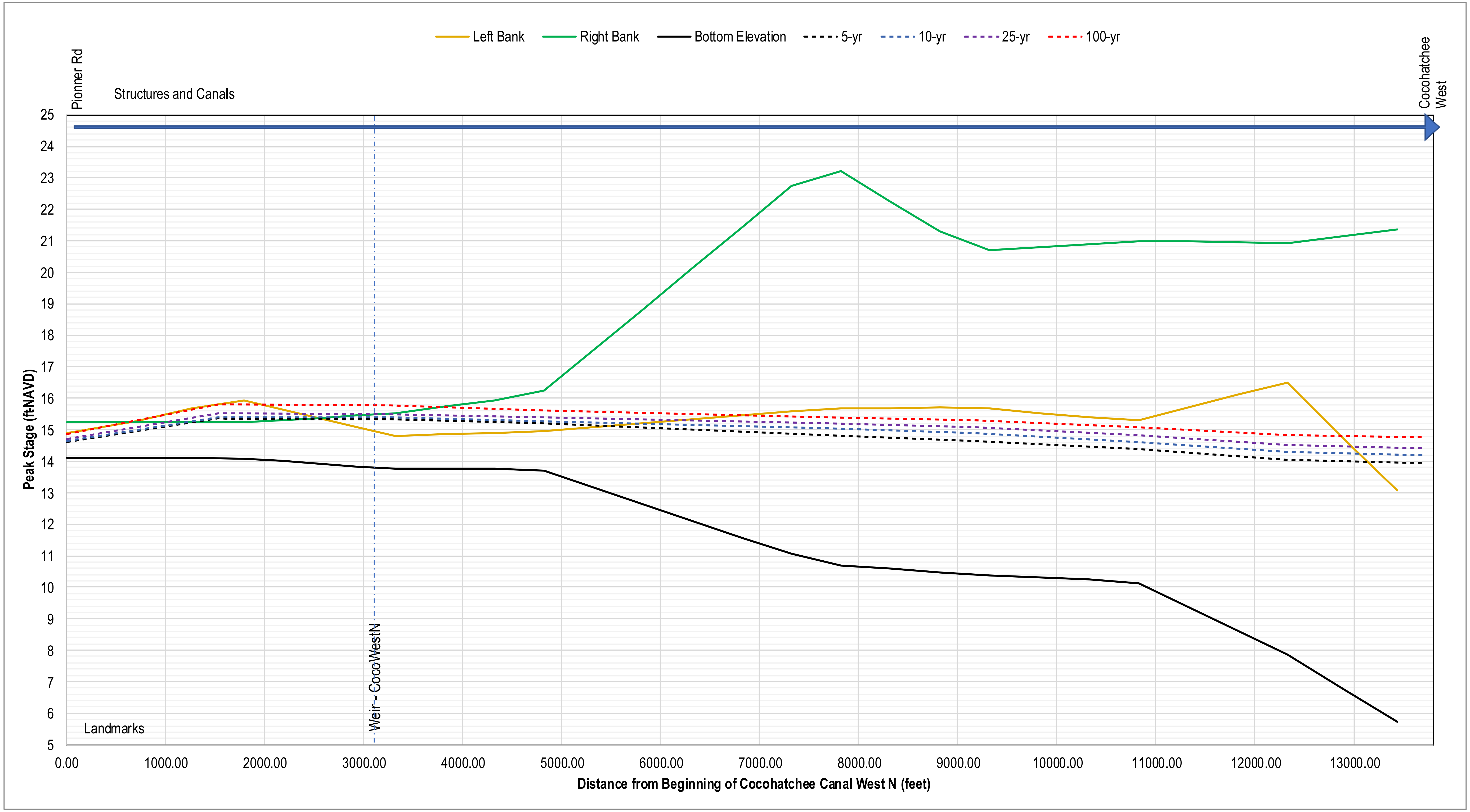

Figure 4-9. Water Profile for Cocohatchee Canal West North

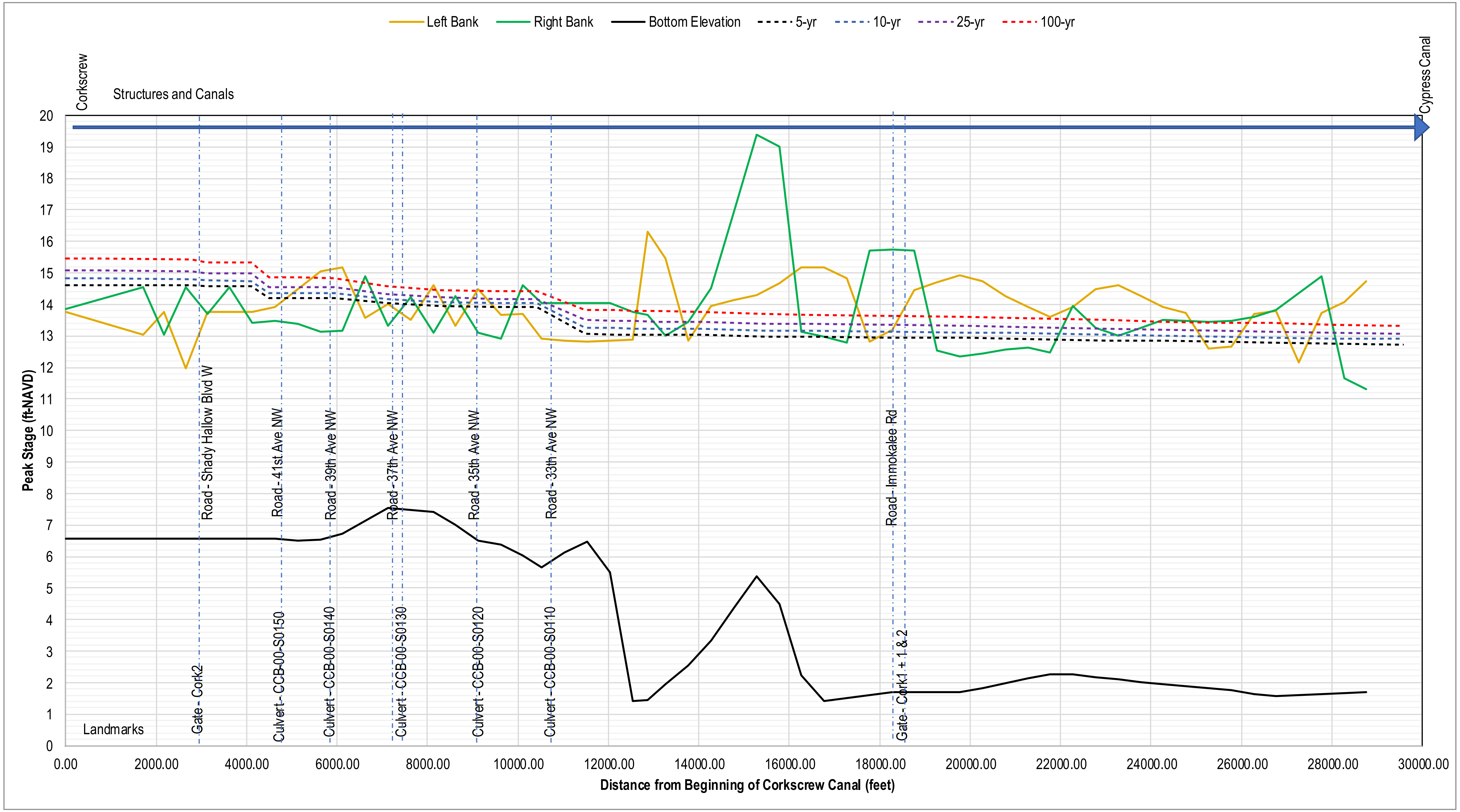

Figure 4-10. Water Profile for Corkscrew Canal

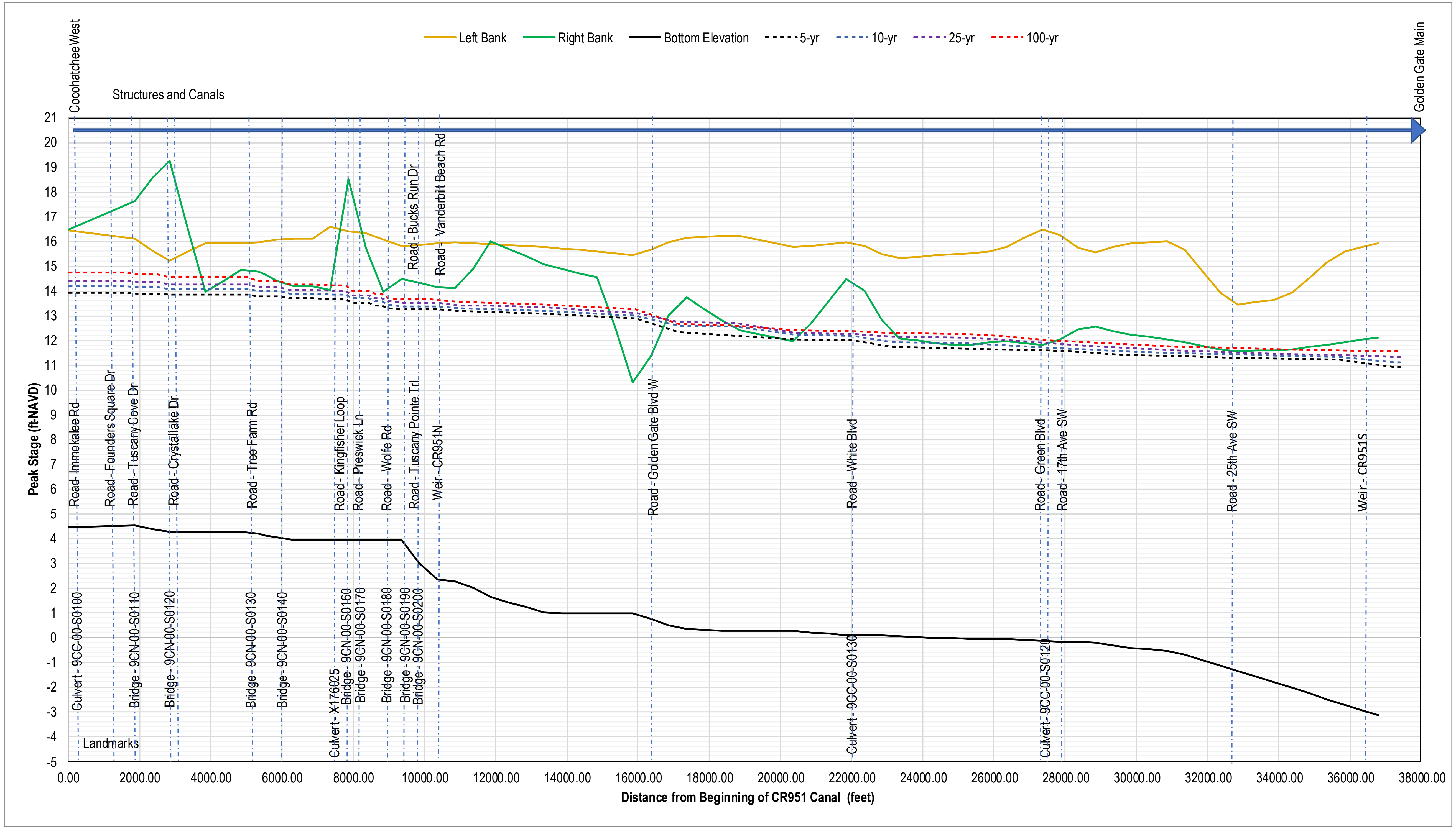

Figure 4-11. Water Profile for CR951 Canal

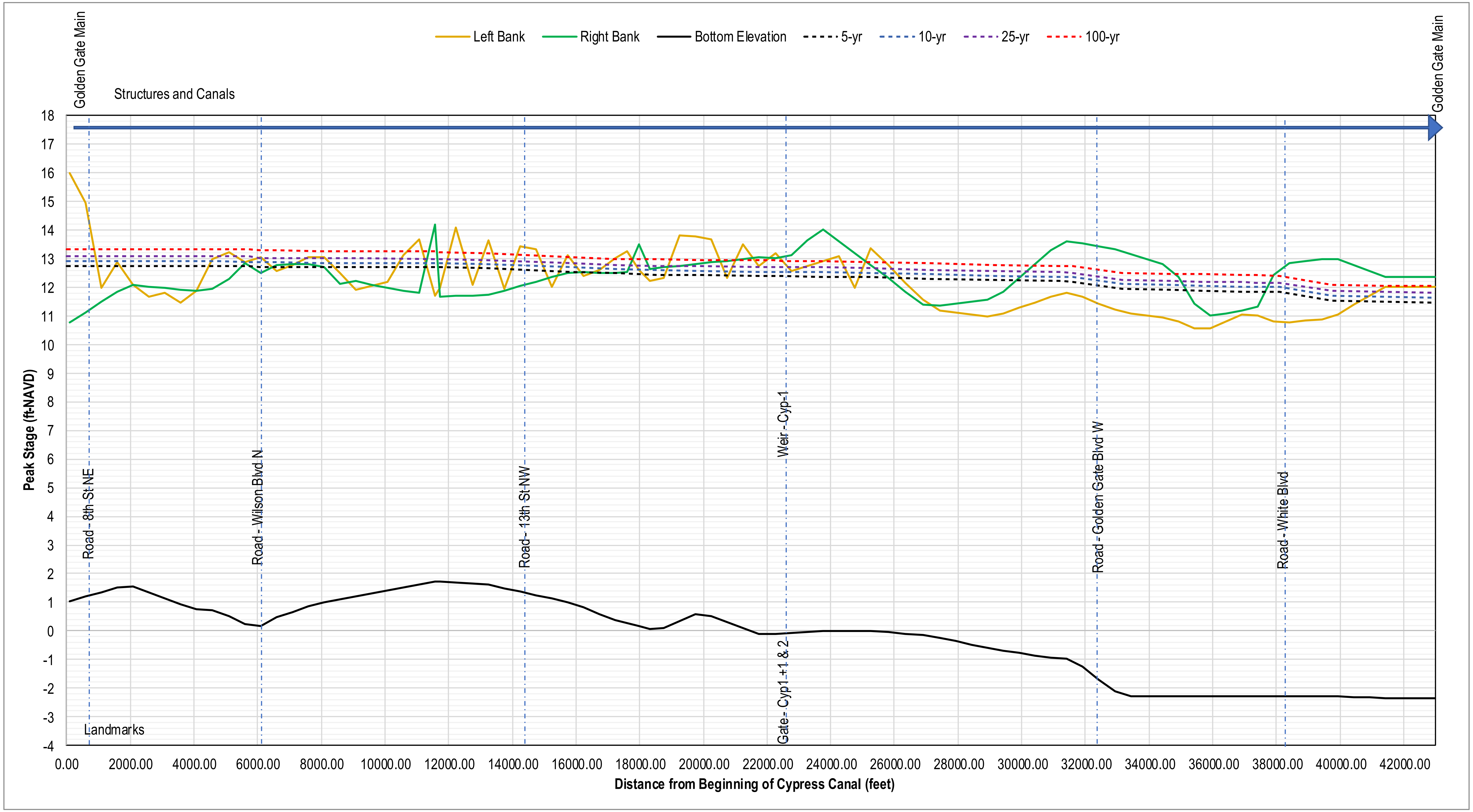

Figure 4-12. Water Profile for Cypress Canal

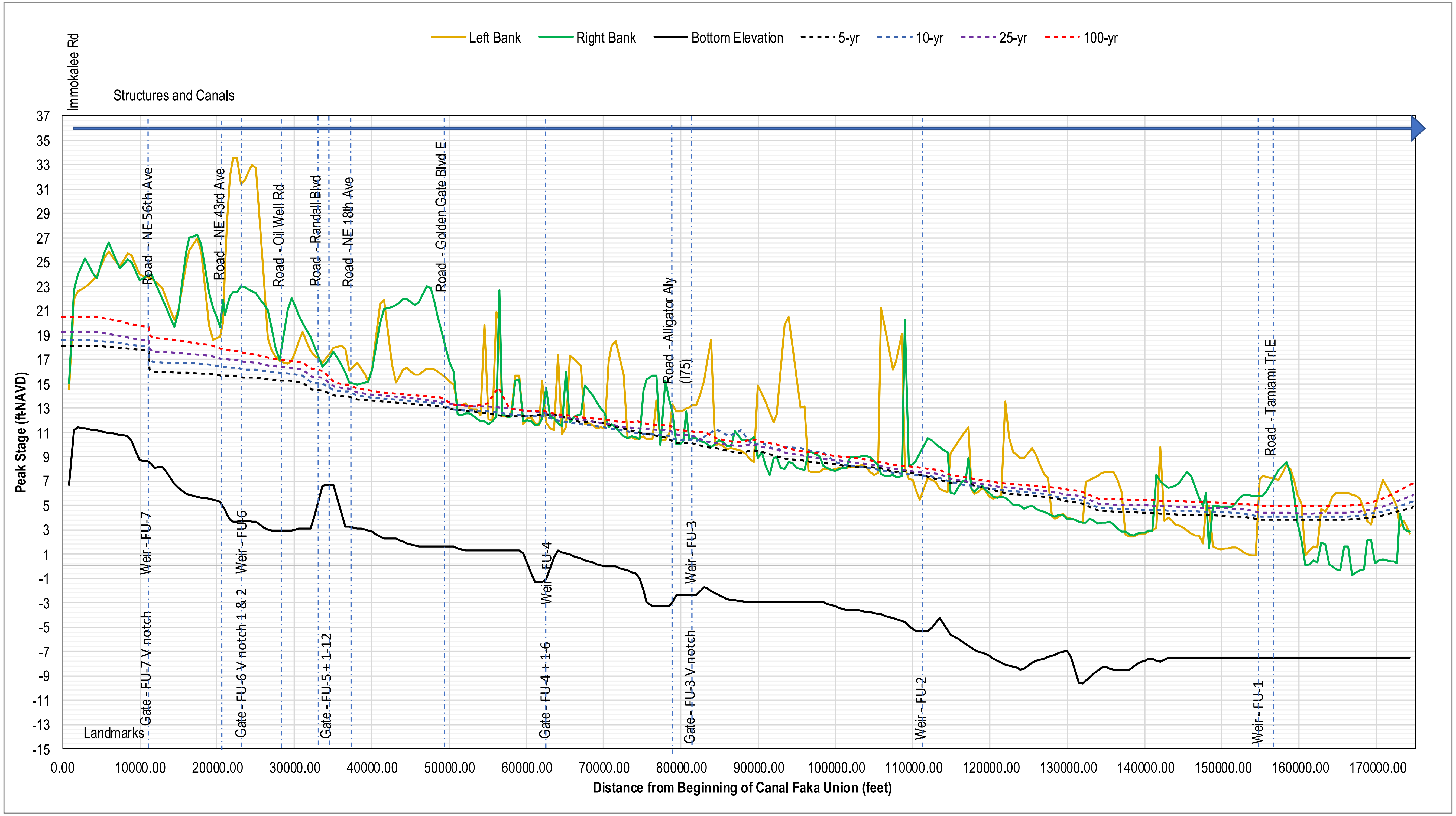

Figure 4-13. Water Profile for Faka Union Canal

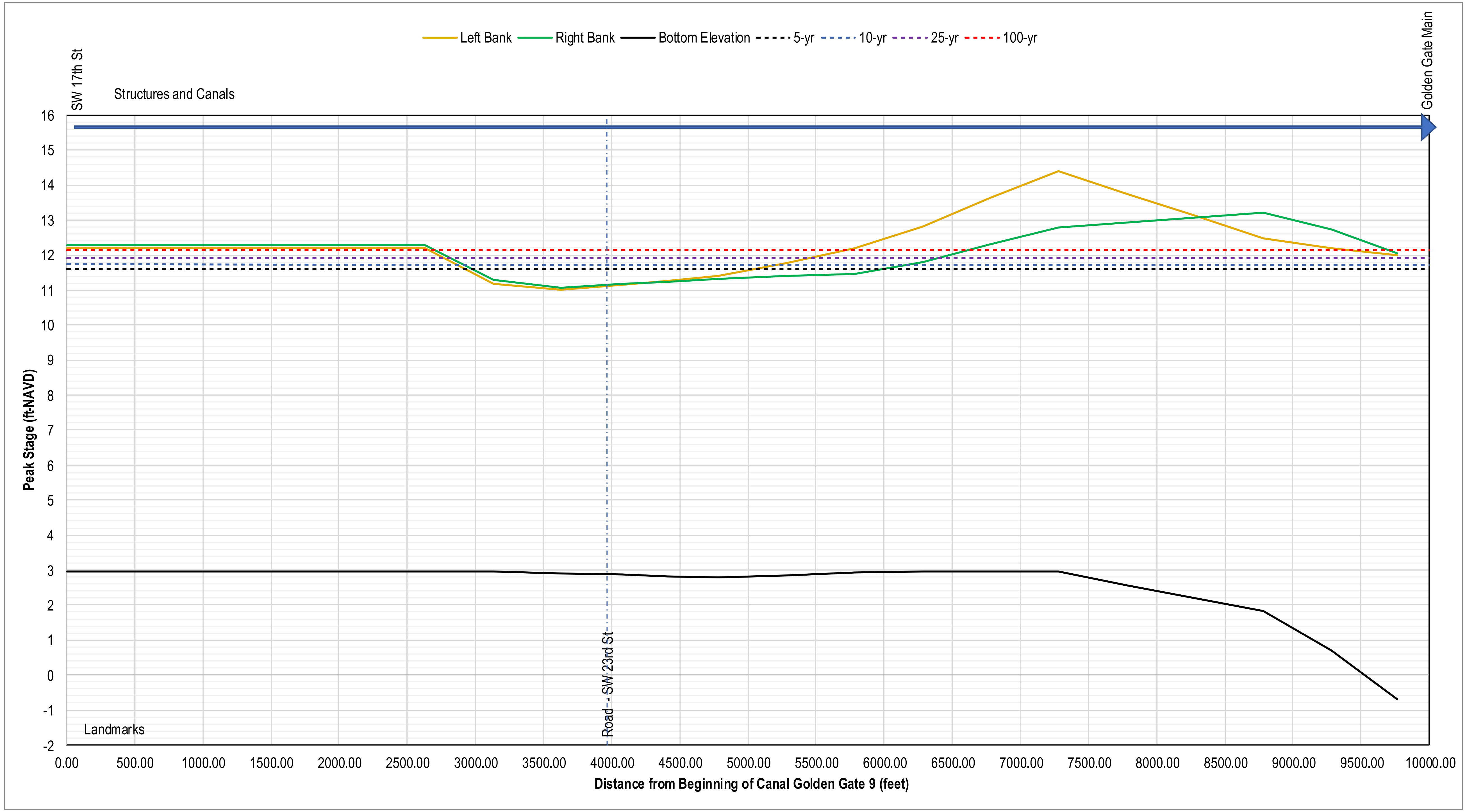

Figure 4-14. Water Profile for Golden Gate 9 Canal

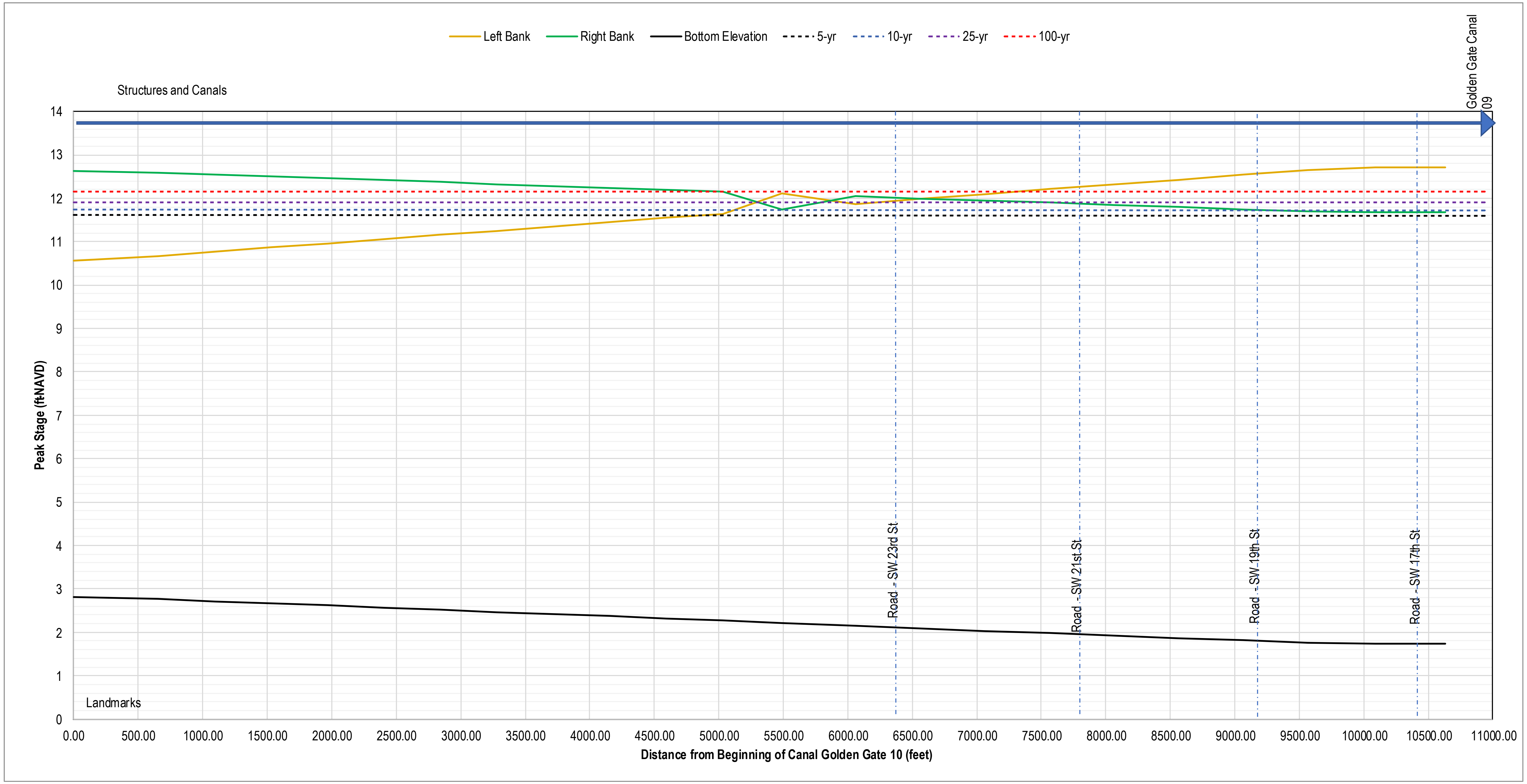

Figure 4-15. Water Profile for Golden Gate 10 Canal

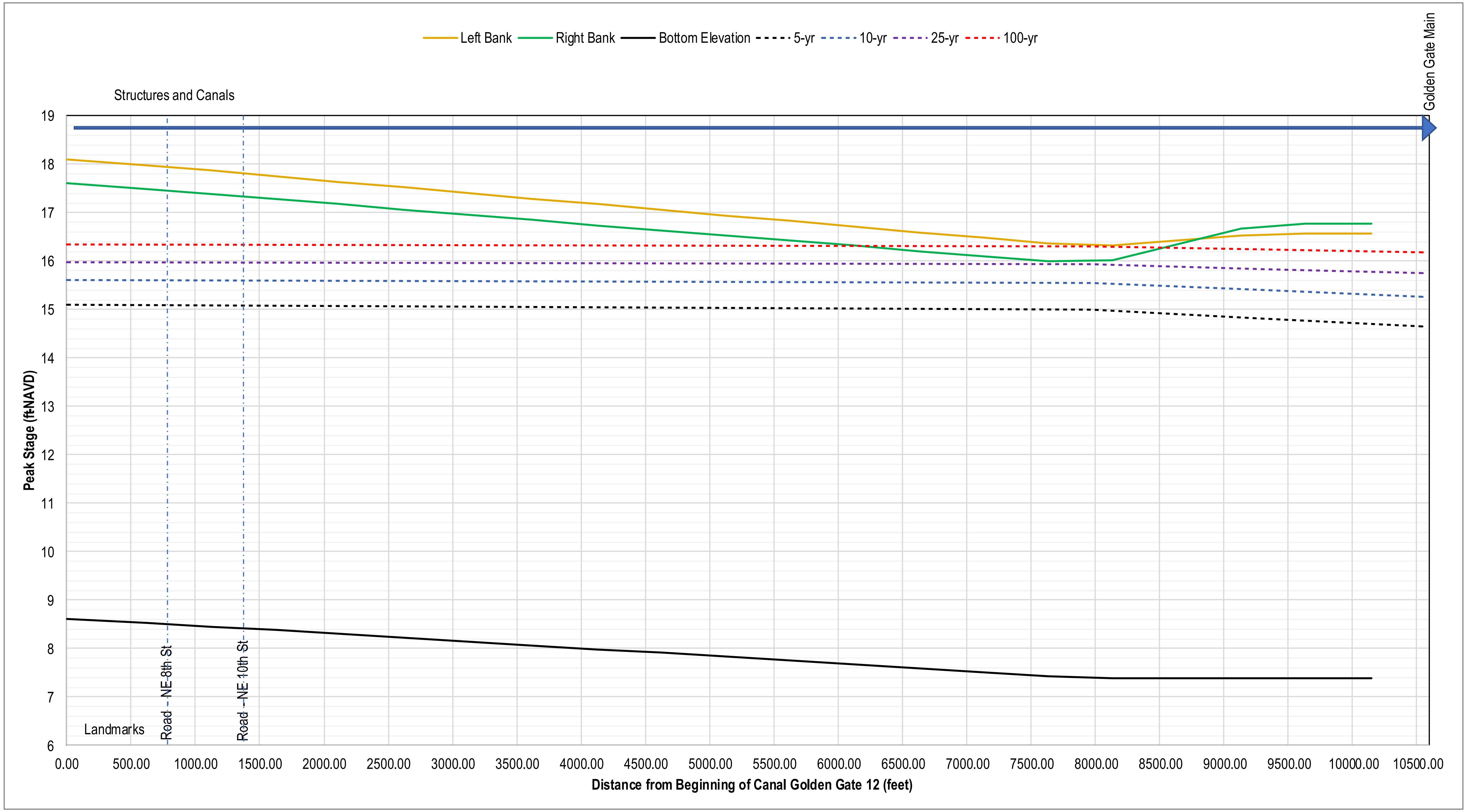

Figure 4-16. Water Profile for Golden Gate 12 Canal

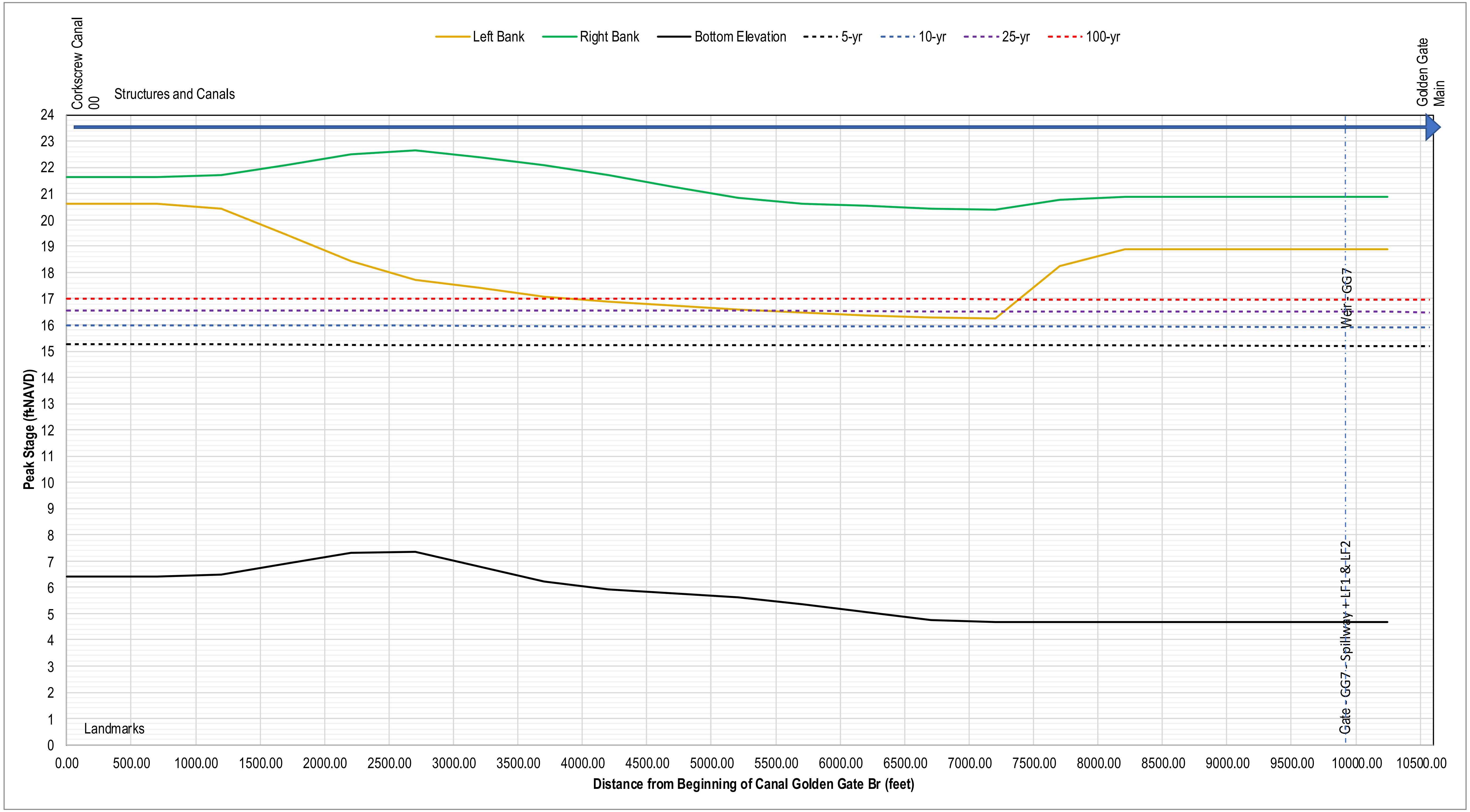

Figure 4-17. Water Profile for Golden Gate Br Canal

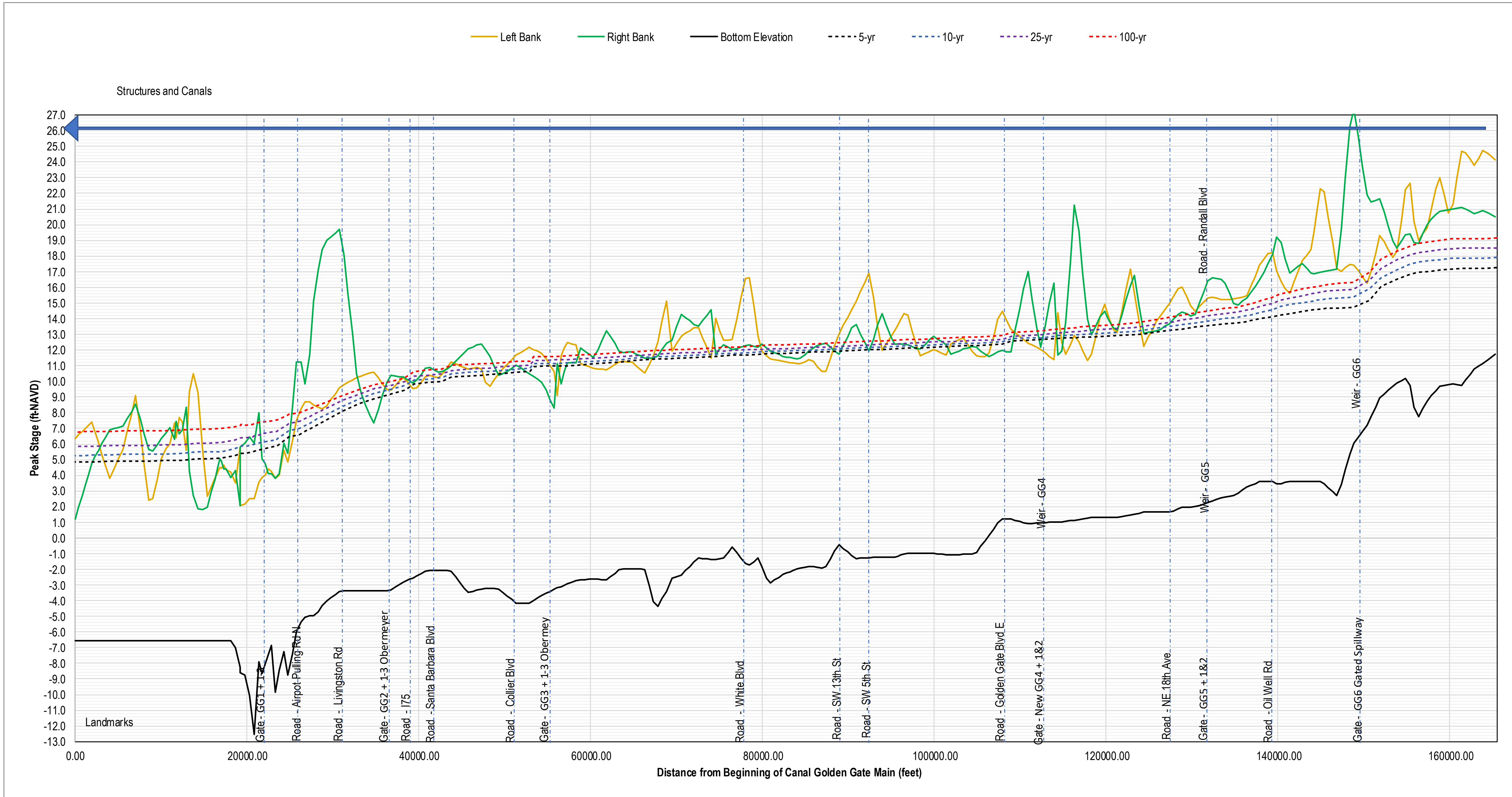

Figure 4-18. Water Profile for Golden Gate Main Canal

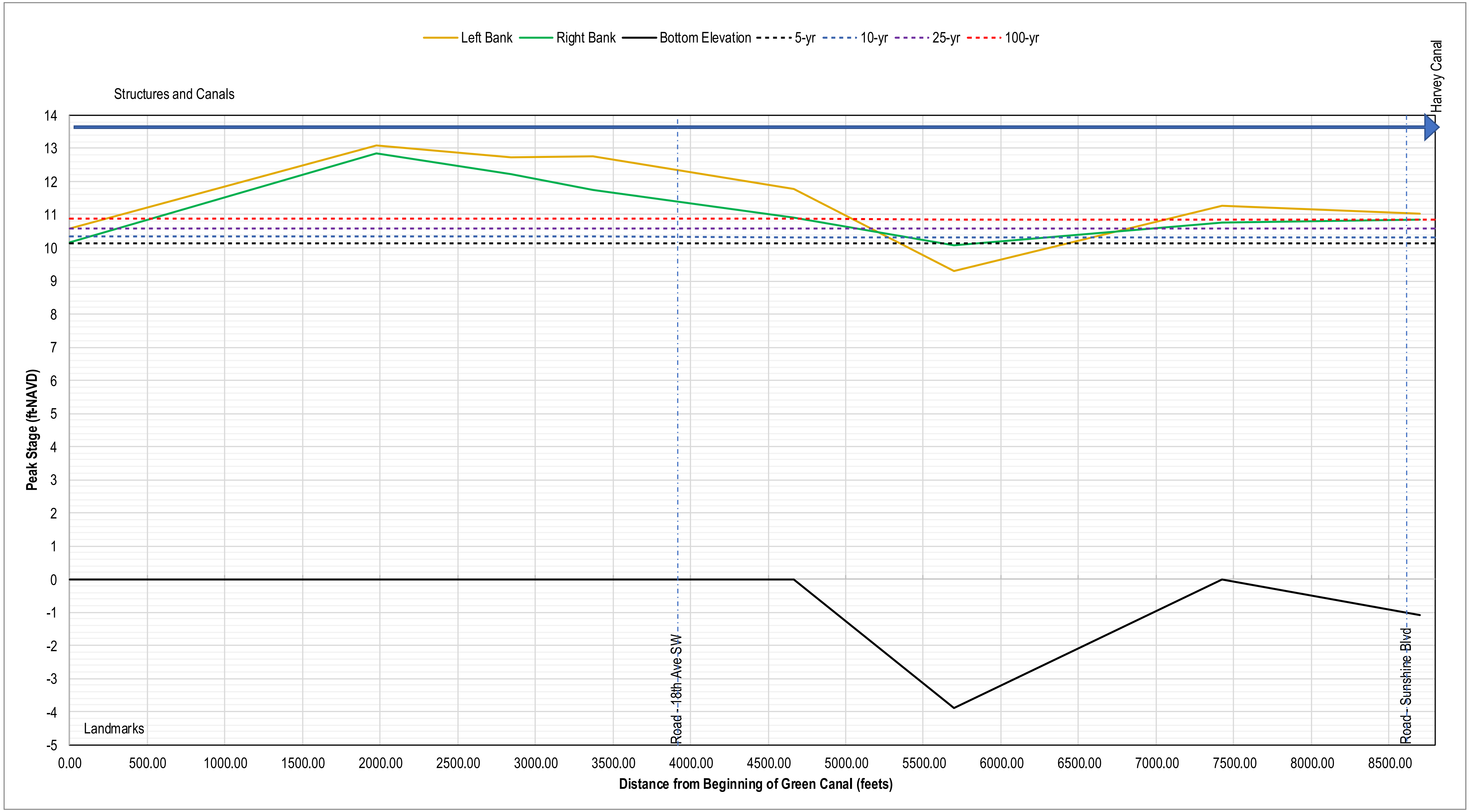

Figure 4-19. Water Profile for Green Canal

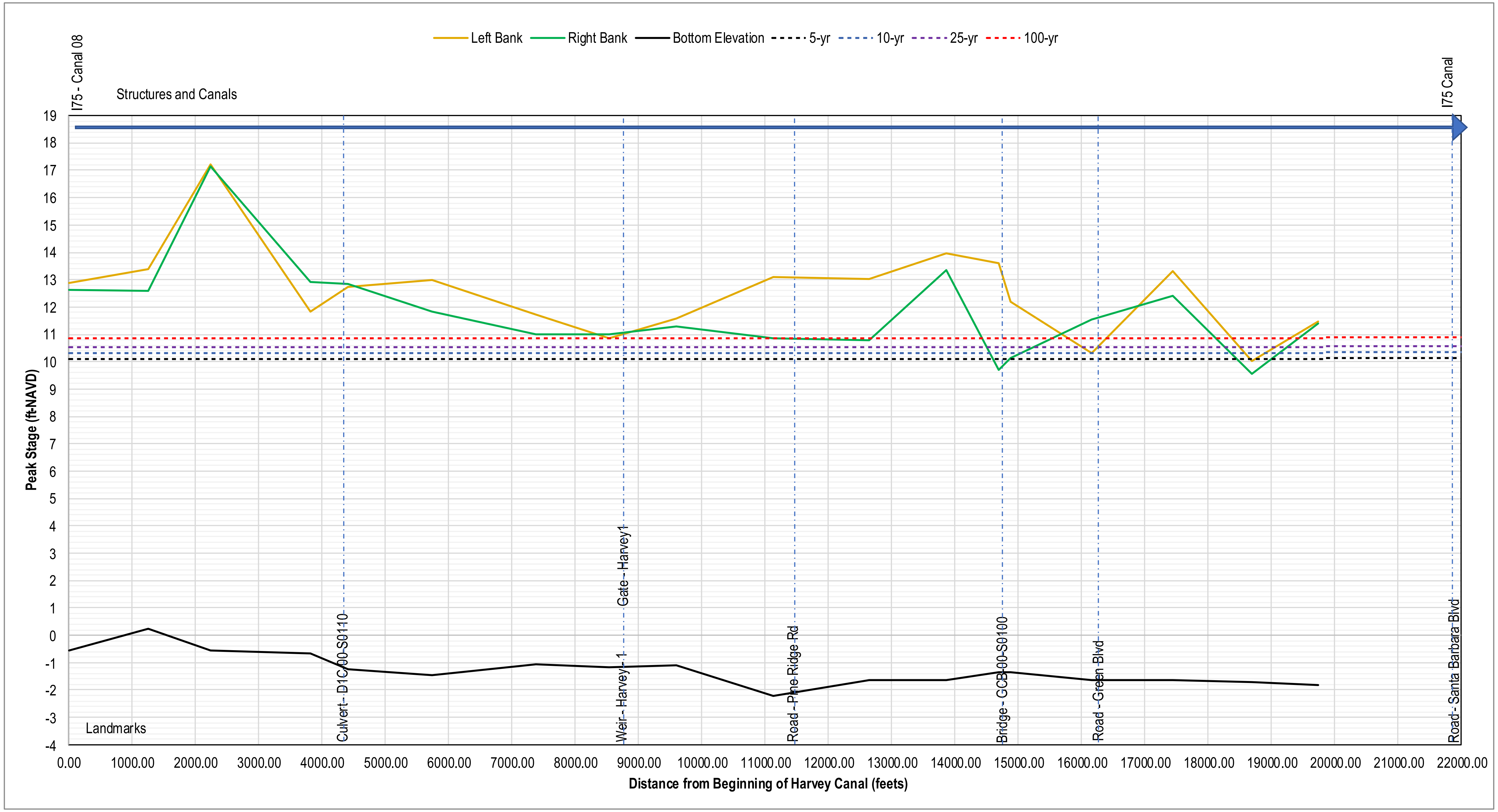

Figure 4-20. Water Profile for Harvey Canal

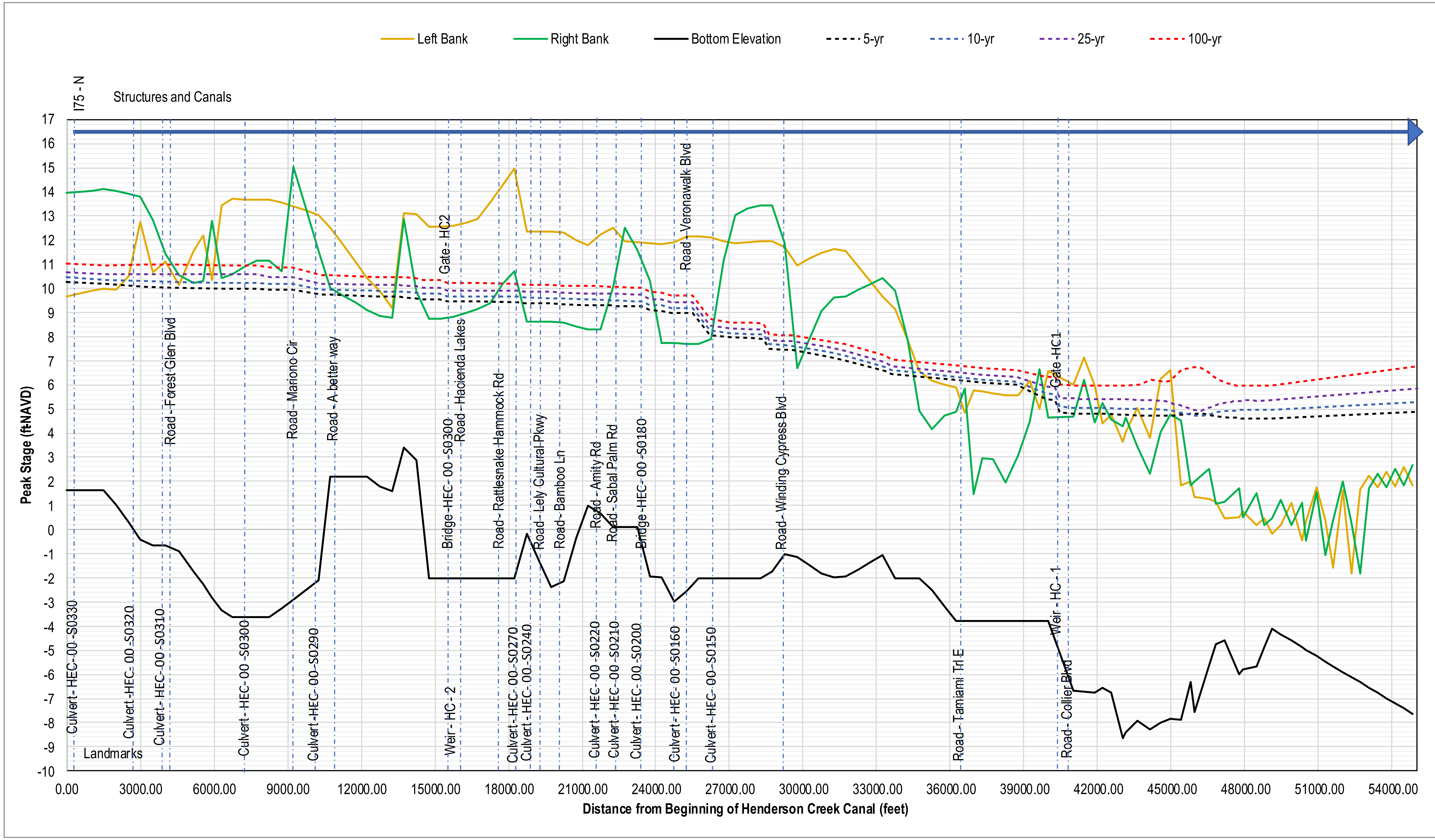

Figure 4-21. Water Profile for Henderson Creek Canal

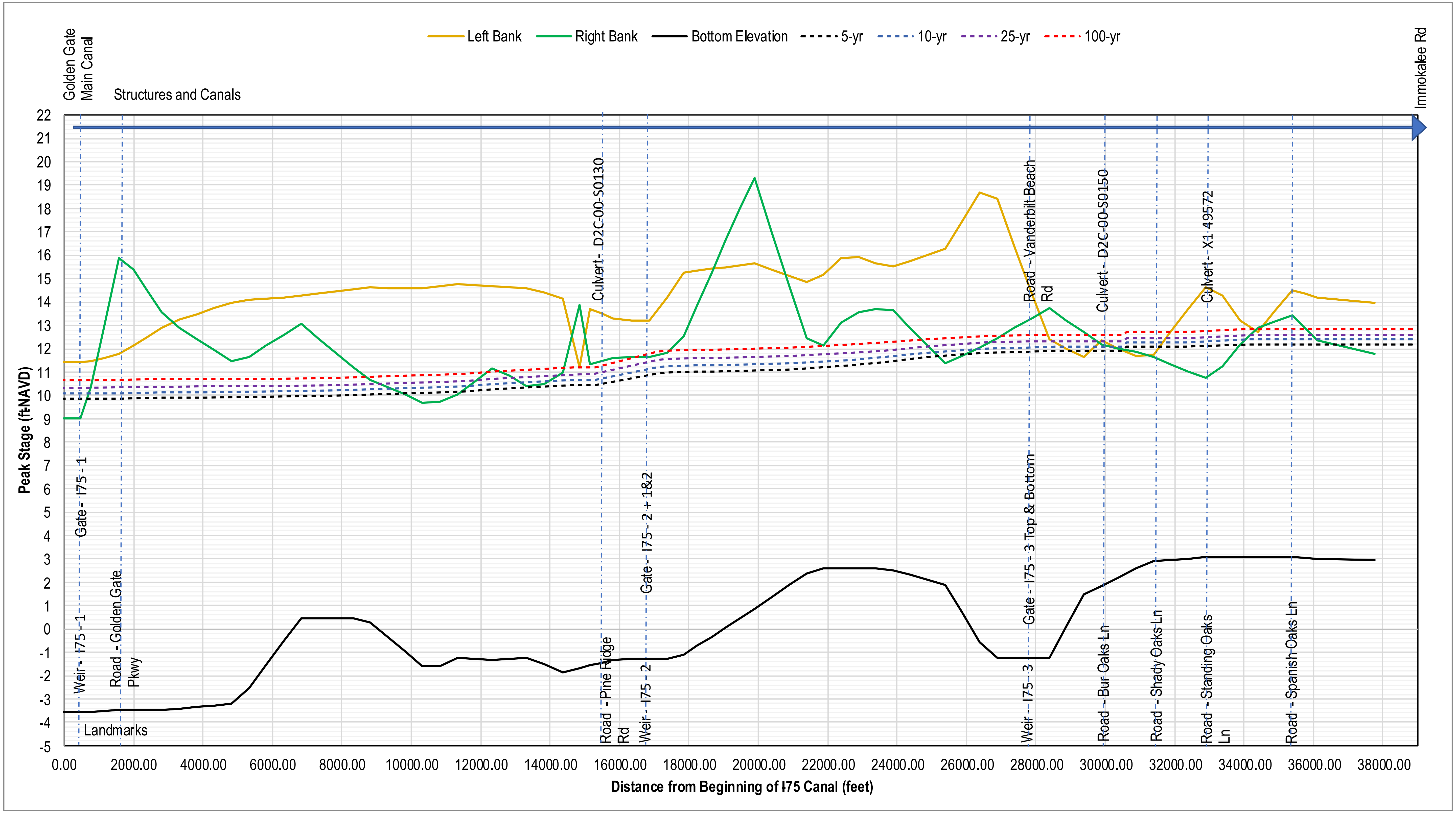

Figure 4-22. Water Profile for I-75 Canal

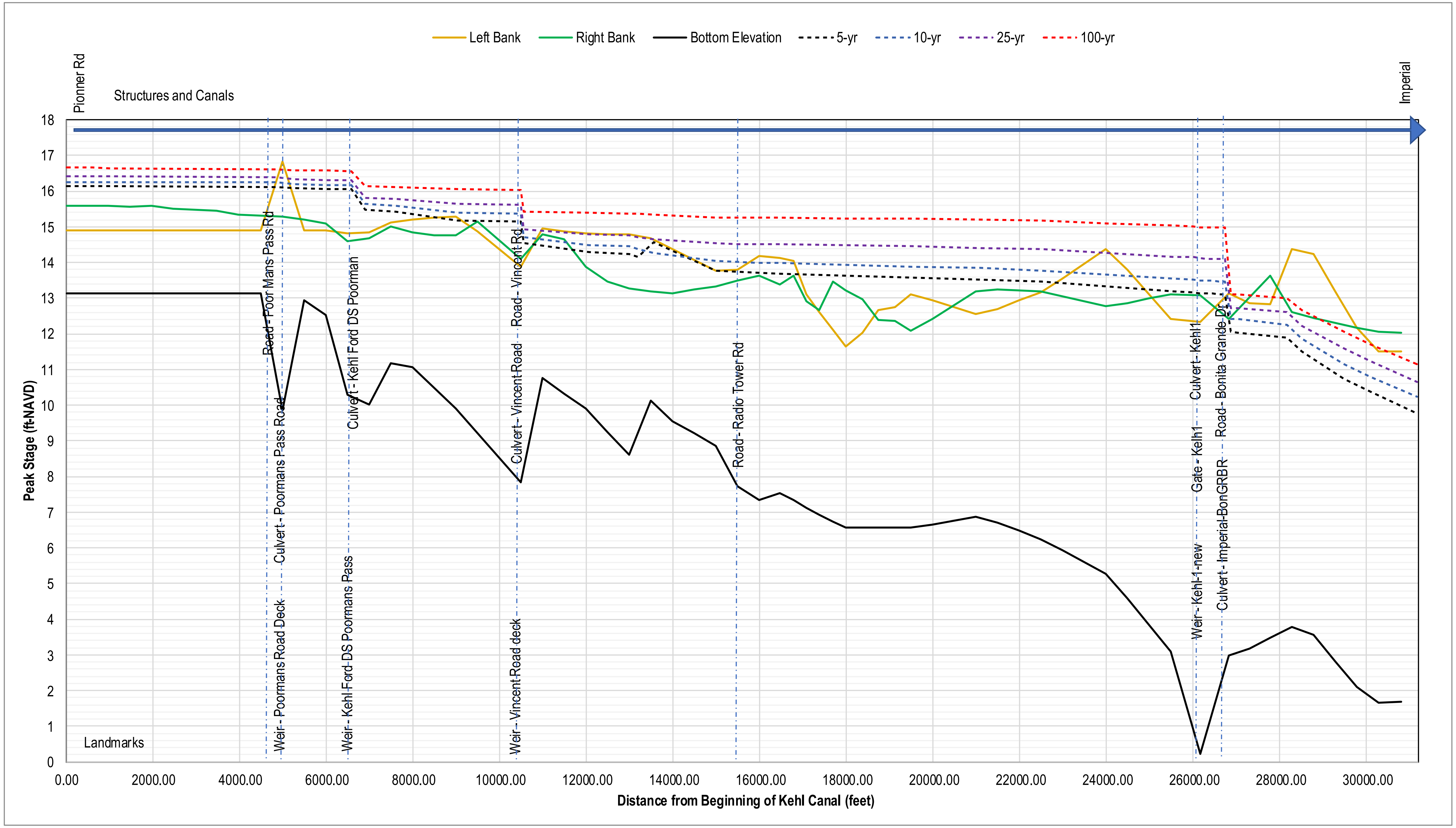

Figure 4-23. Water Profile for Kehl Canal

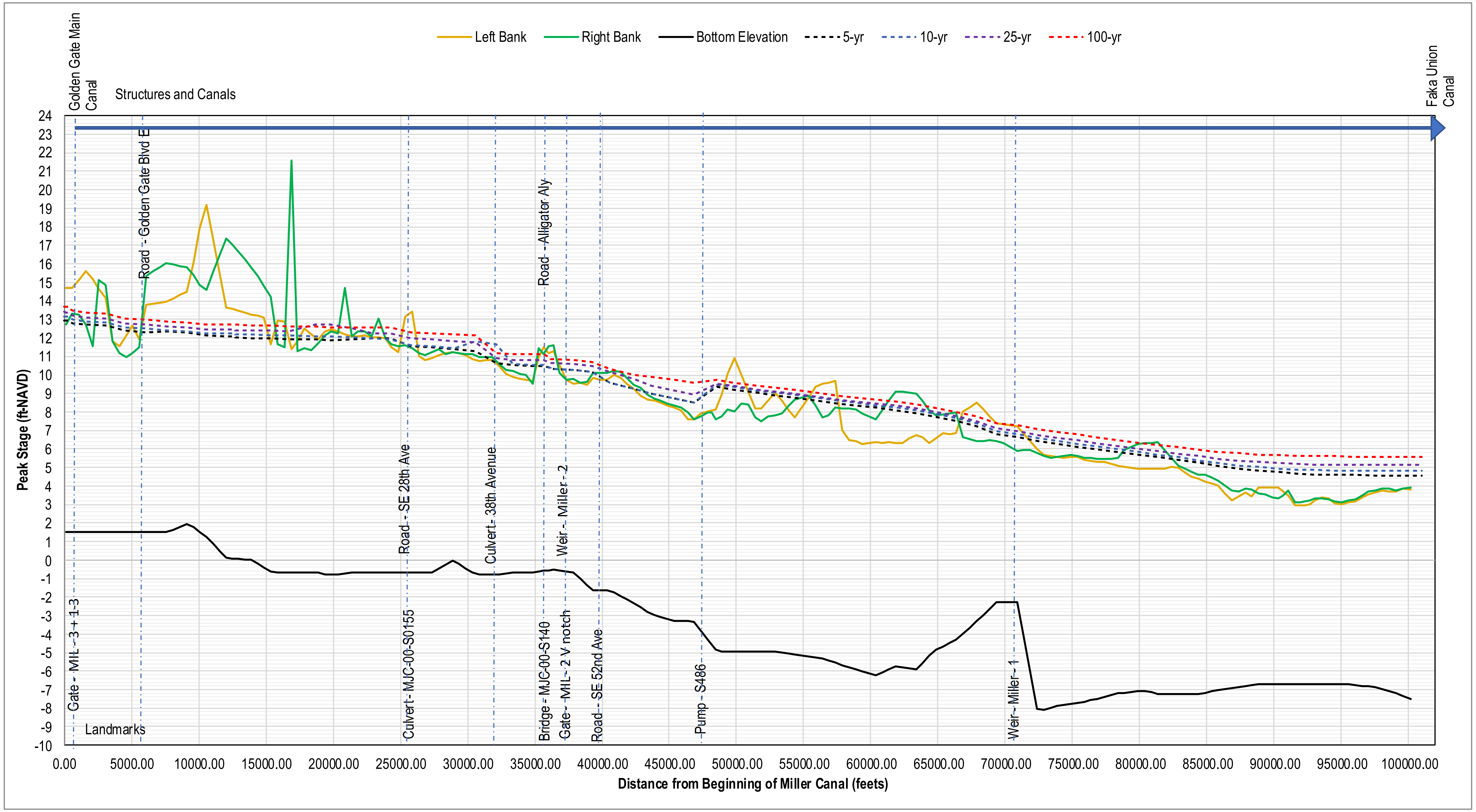

Figure 4-24. Water Profile for Miller Canal

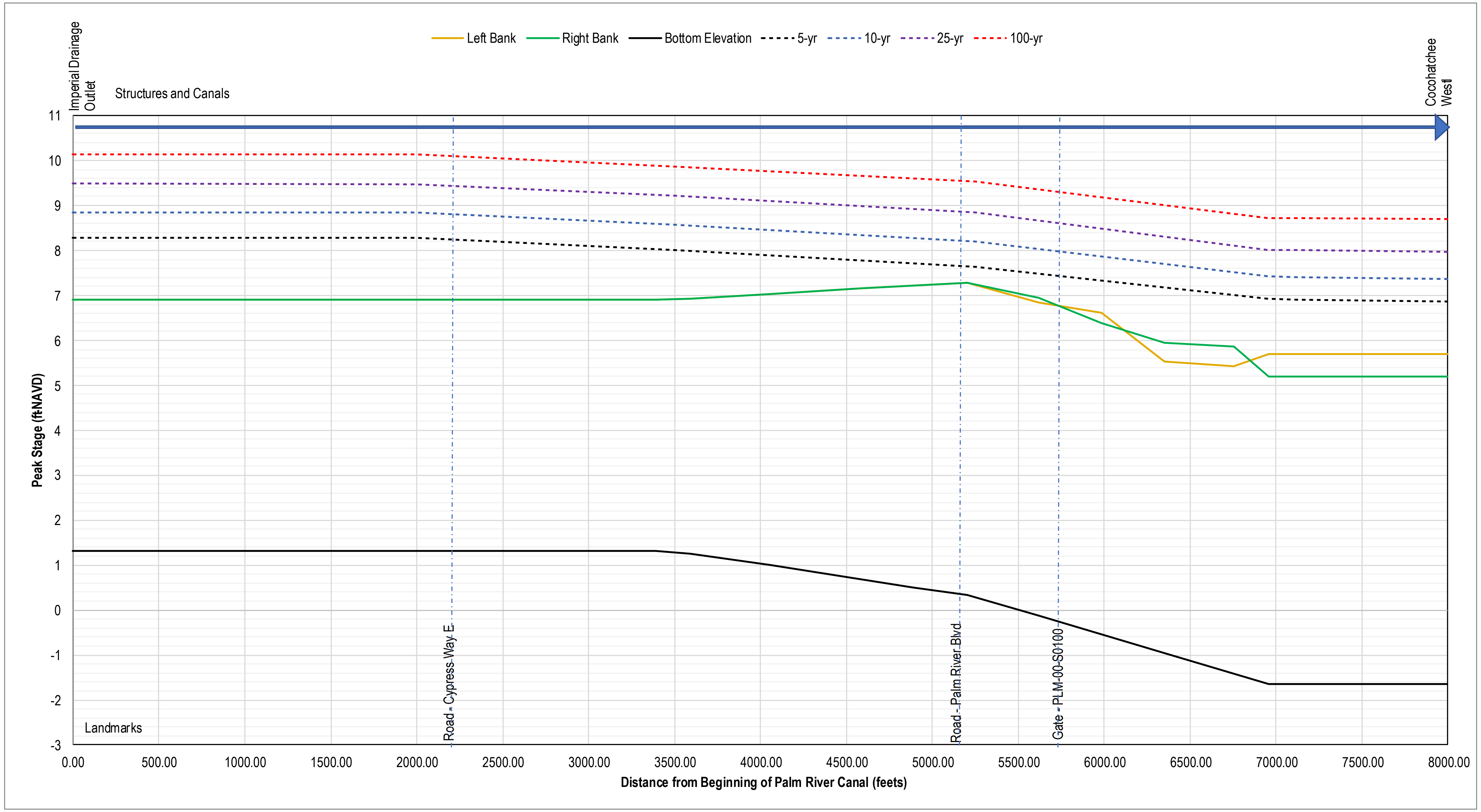

Figure 4-25. Water Profile for Palm River Canal

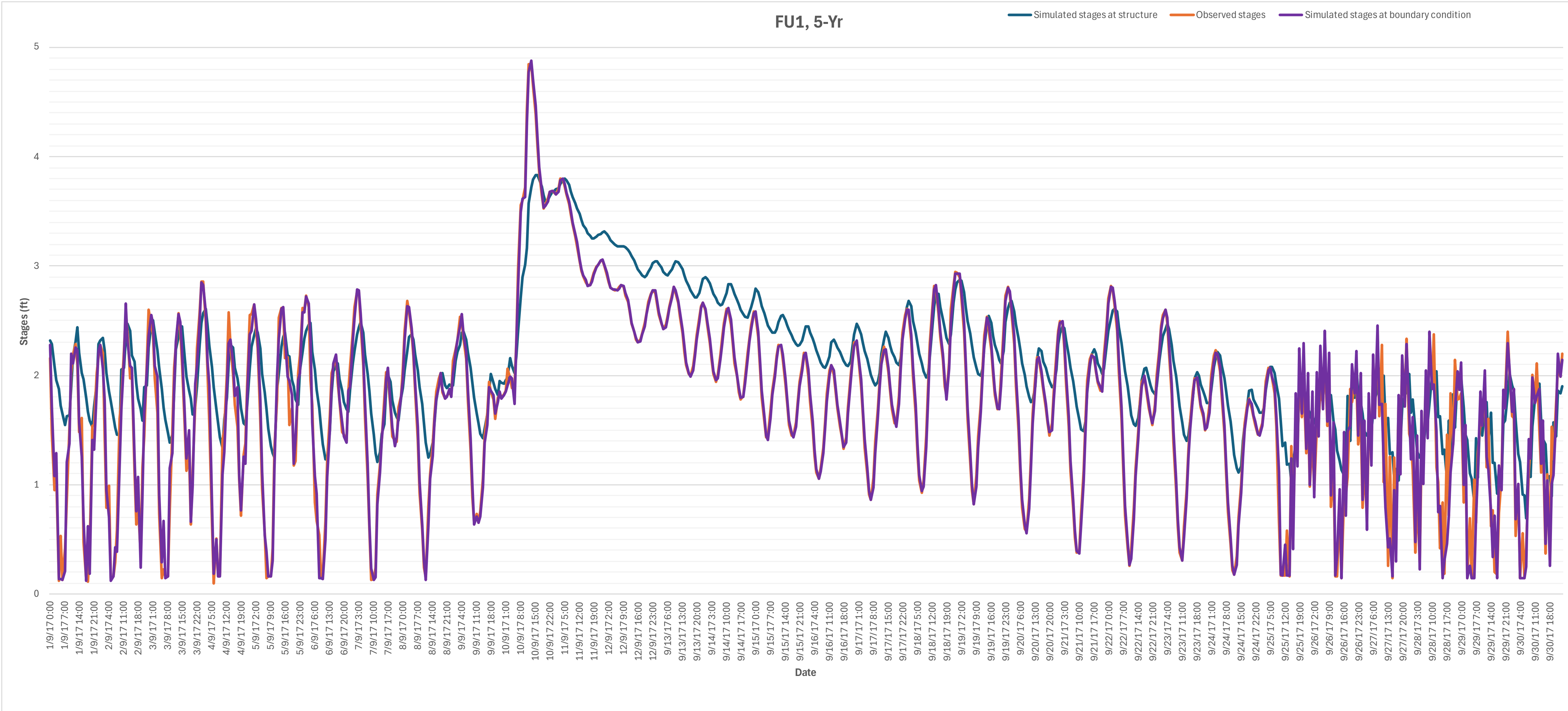

Figure 4-26. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 5-yr event at FU1.

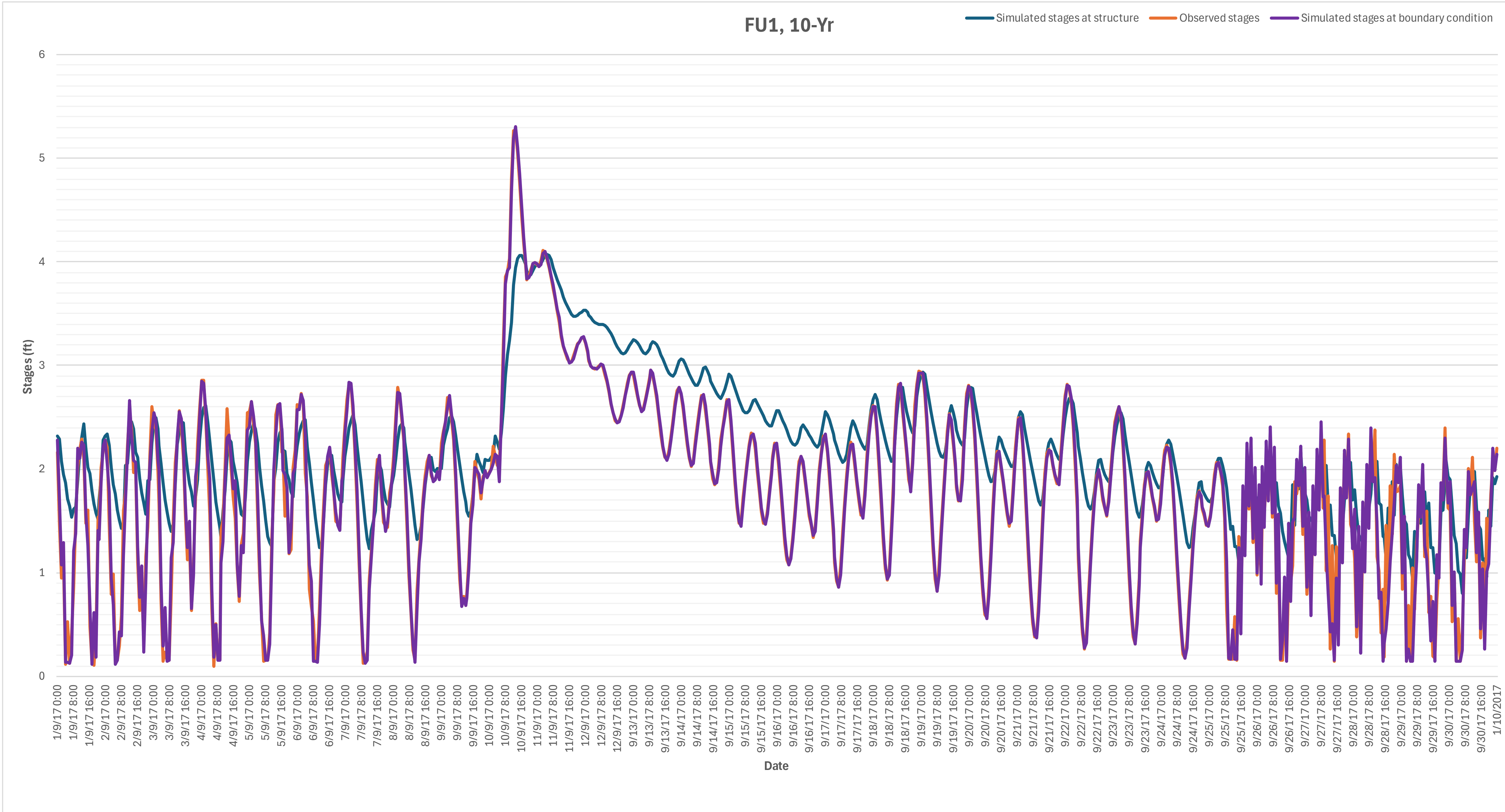

Figure 4-27. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 10-yr event at FU1.

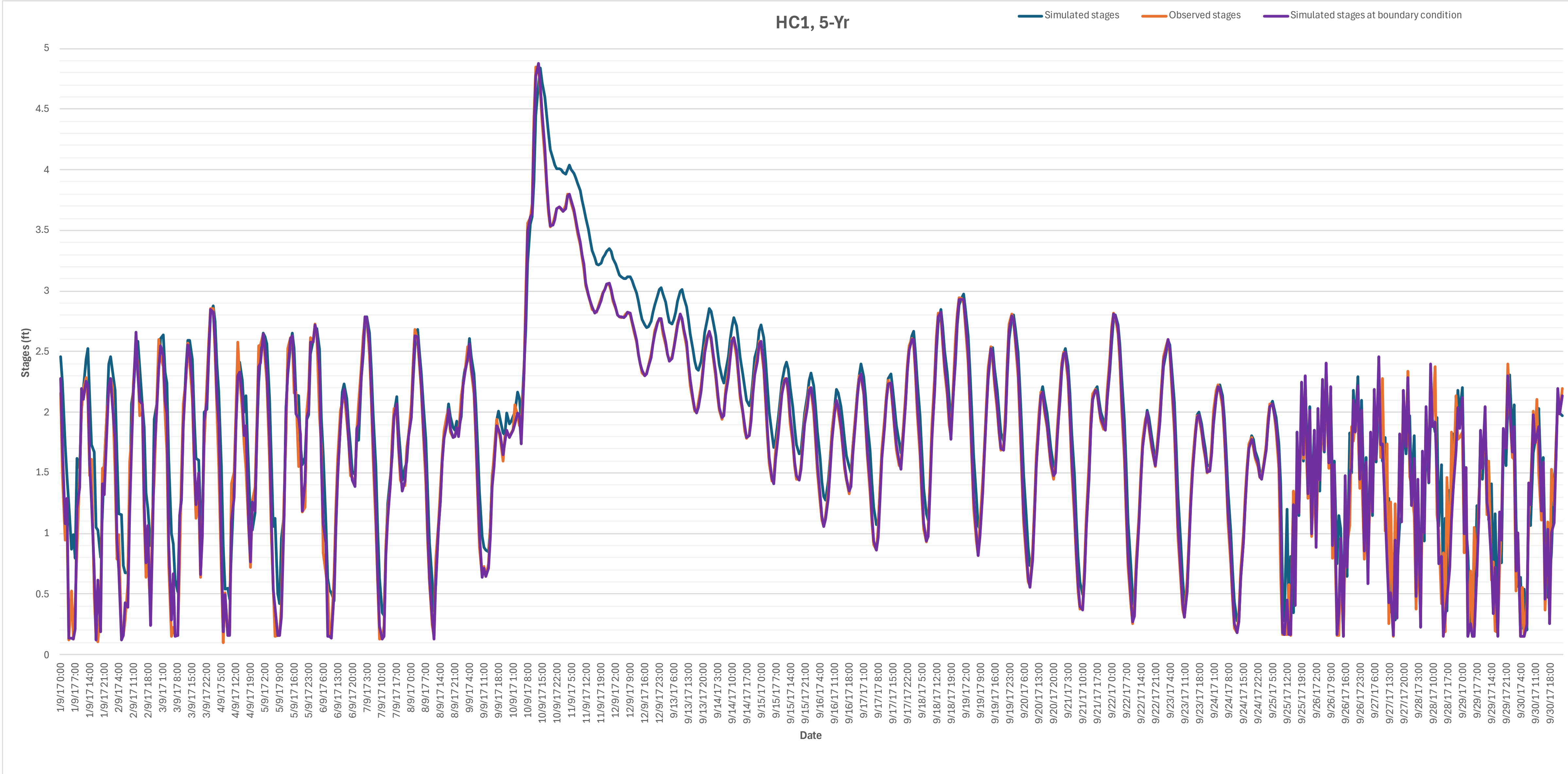

Figure 4-28. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 5-yr event at HC1.

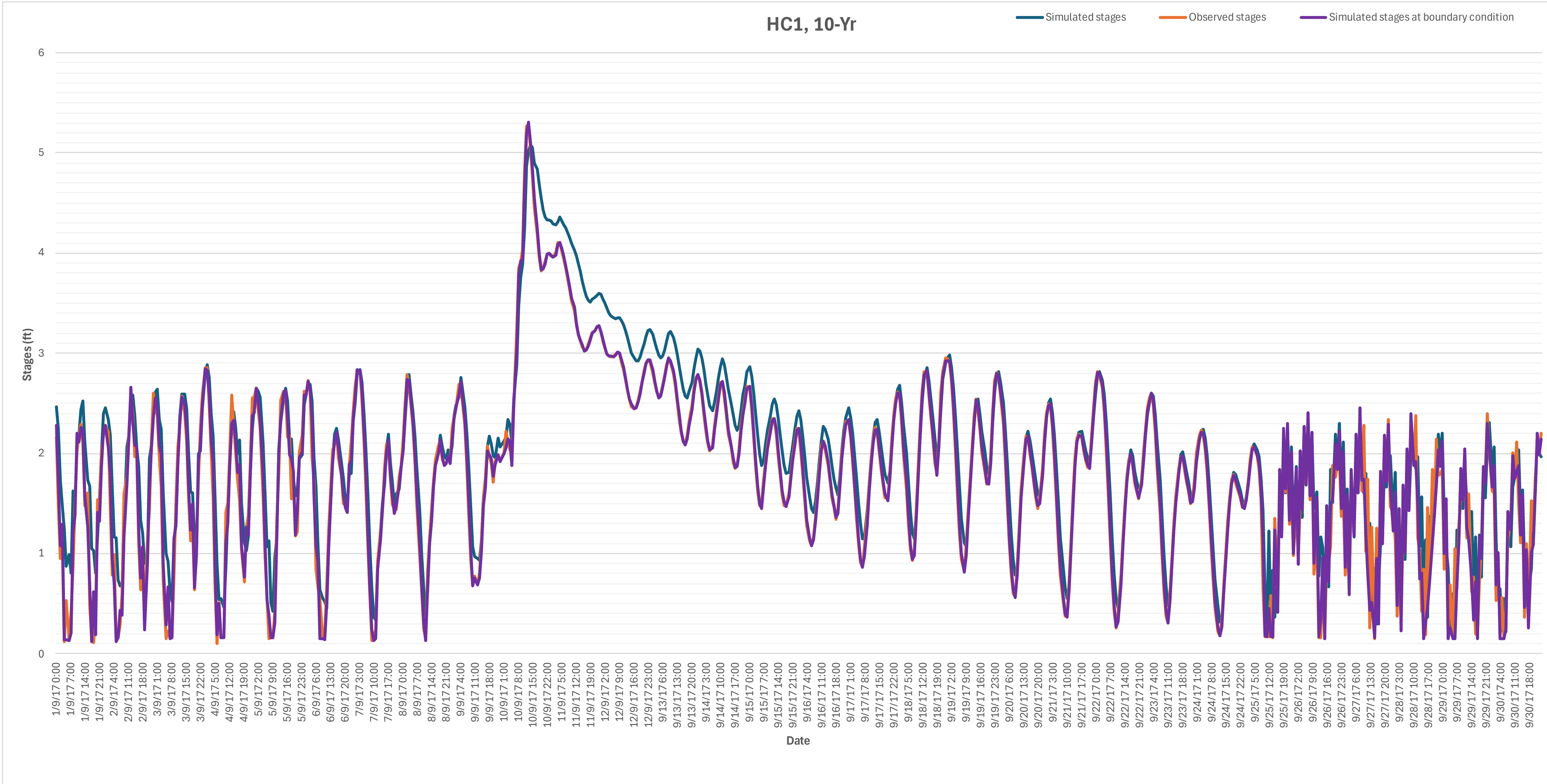

Figure 4-29. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 10-yr event at HC1.

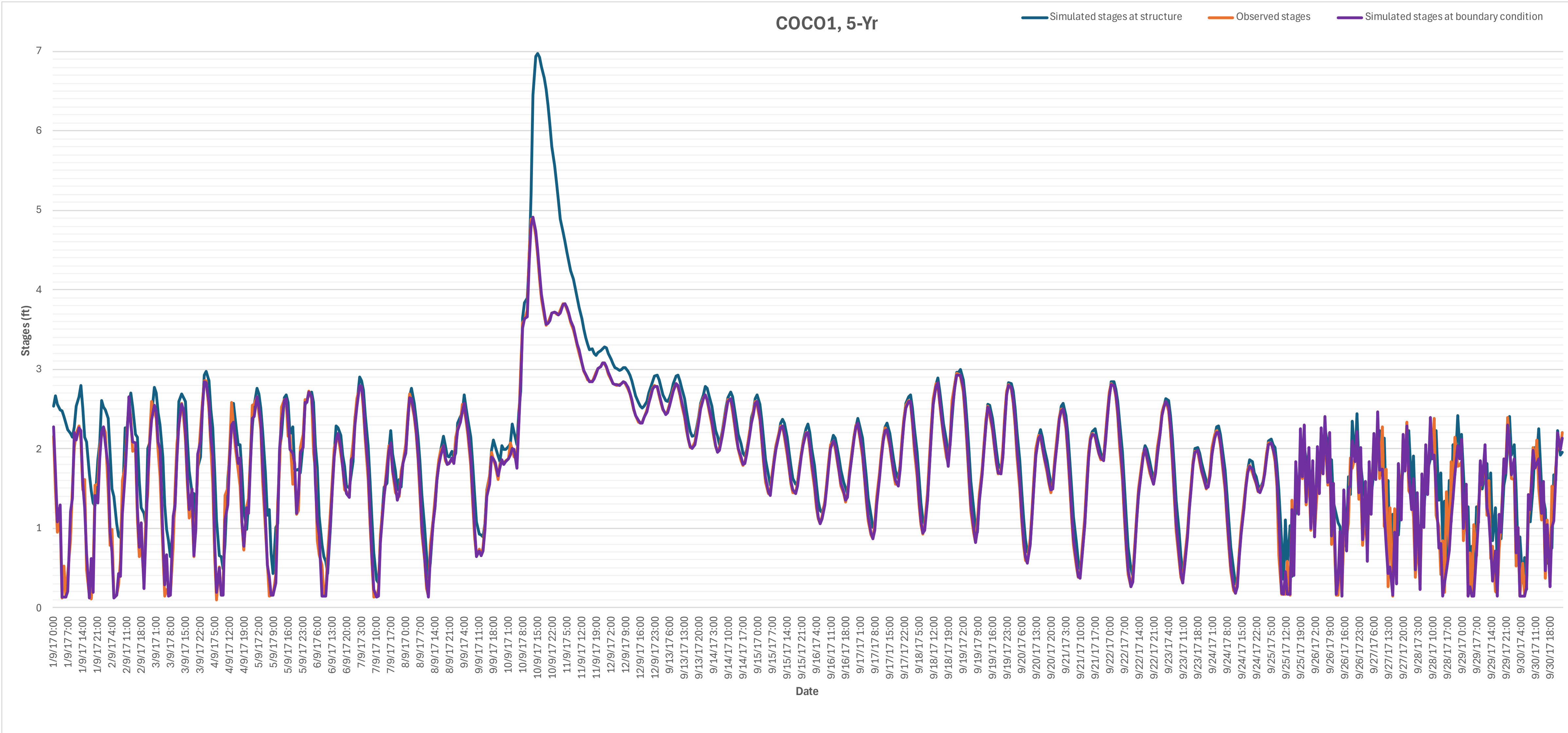

Figure 4-30. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 5-yr event at COCO1.

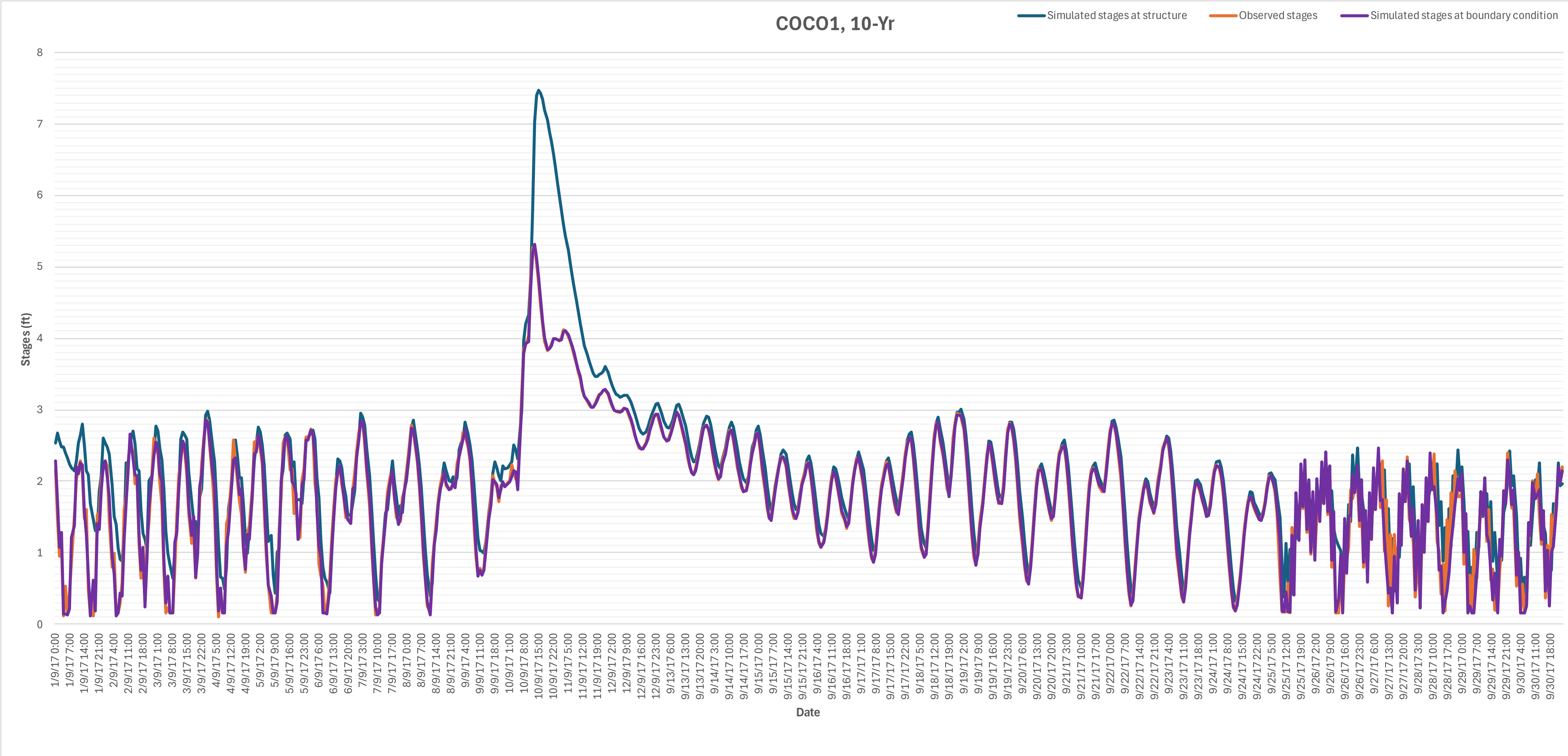

Figure 4-31. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 10-yr event at COCO1.

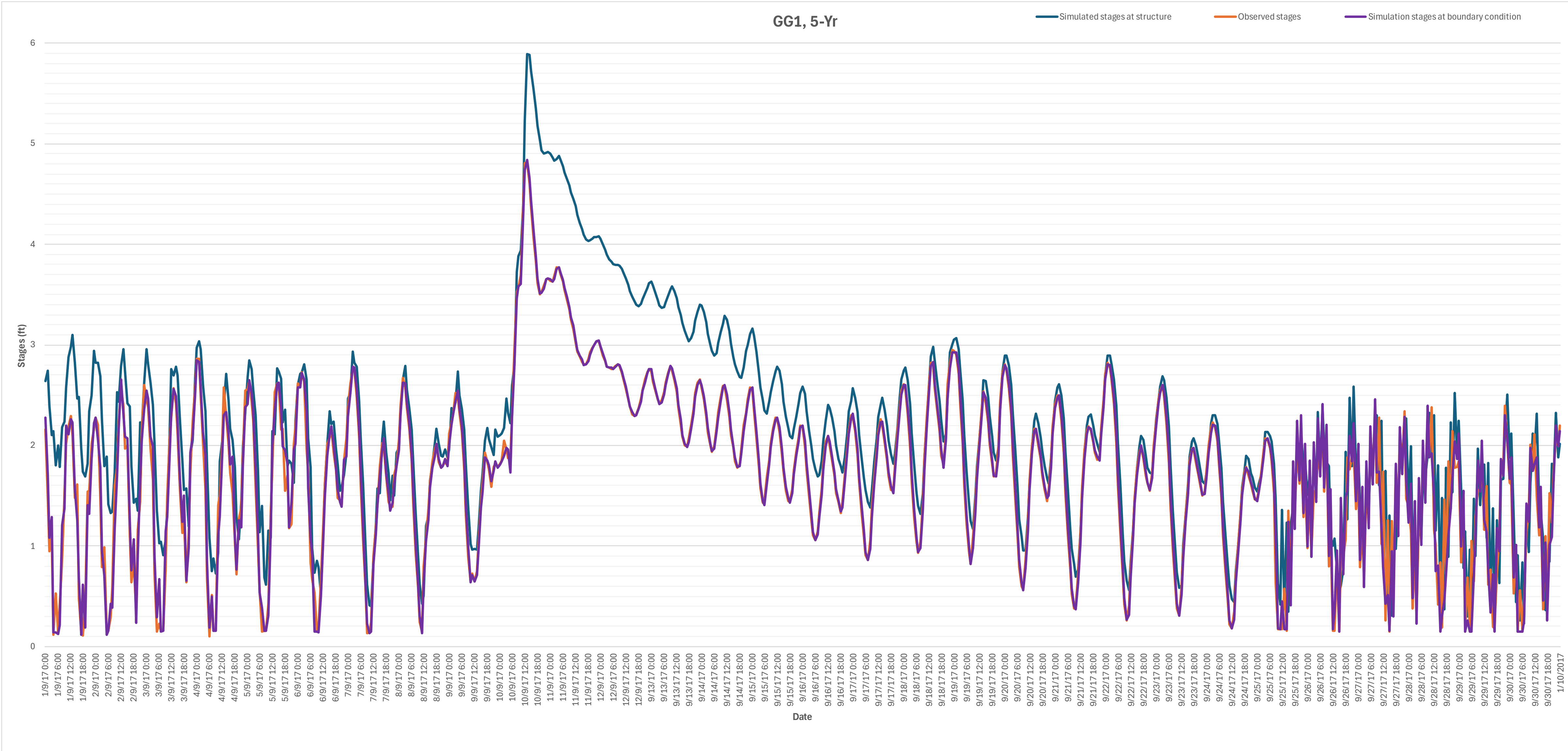

Figure 4-32. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 5-yr event at GG1.

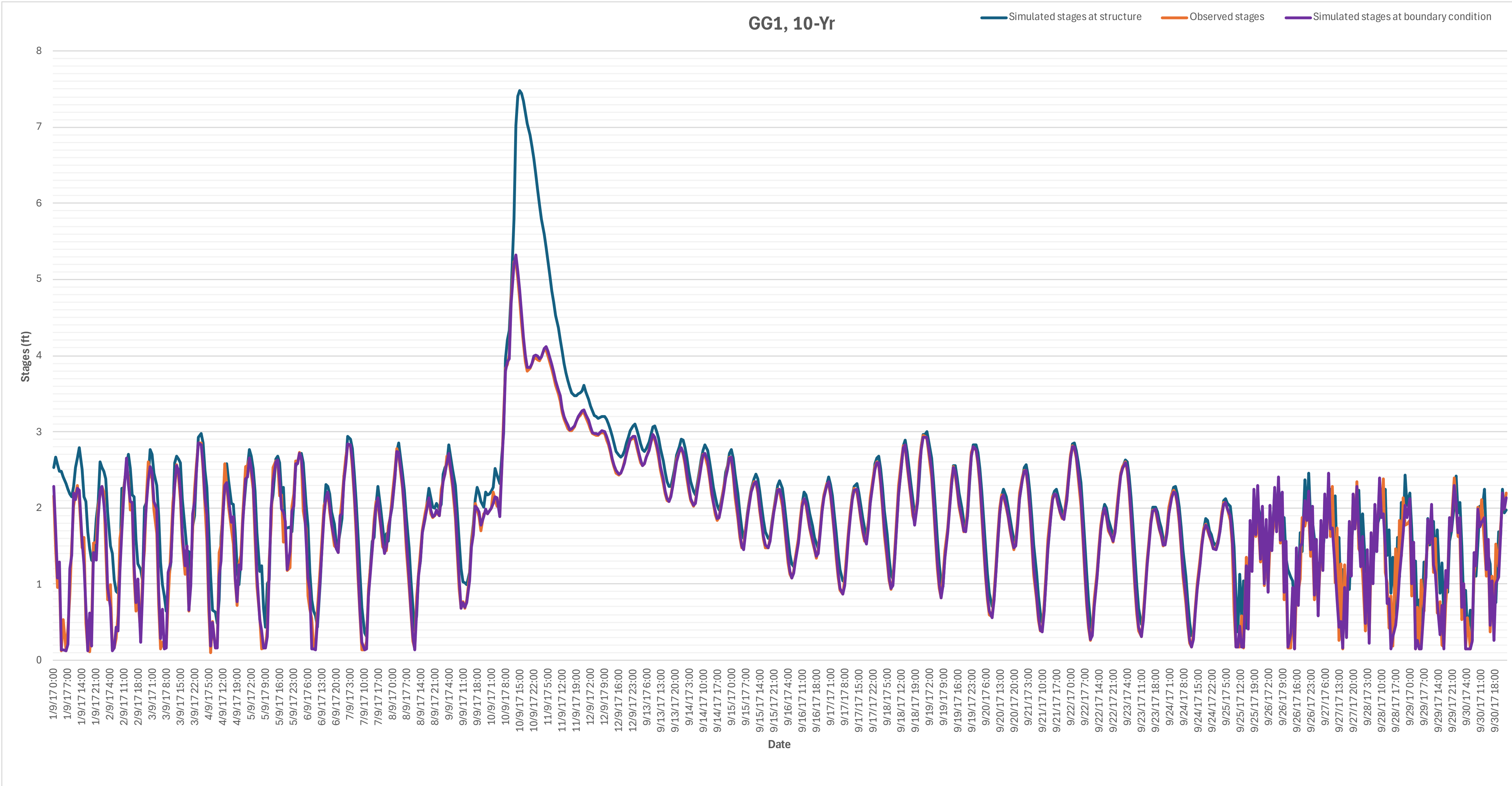

Figure 4-33. Simulated tailwater stages and storm surge tidal boundary conditions used in BCB model for the 10-yr event at GG1.

4.2 PM #2

PM 2, maximum discharge capacity is the highest flows through a canal in cubic feet per second (cfs) as a ratio of the area in square miles of their sub-watershed. Sub-watersheds were obtained from the District's Arc Hydro Enhanced Database (AHED). Some sub-watersheds were merged based on canal segments considering location of water control structures. **Table 4-3** details each canal segment analyzed, identified by its downstream structures. Table also shows the peak discharge at the control structures or available chainage preceding the structure for all the four design events.

For each segment, discharge capacity was calculated by:

- Firstly, finding the sum of all inflow and outflow hydrographs from the 25-year, 3-day design storm event model timeframe separately based on their inflow and outflow water control structures.
- 12-hr average was calculated for inflows and outflows respectively.
- The maximum 12-hr average for inflows was subtracted from outflows and divided by the area of the canal segment to get the maximum discharge capacity in cubic feet per second per square mile.

Figure 4-34 shows the water control catchments created from AHED sub-watersheds. **Figure 4-35** through **Figure 4-61** shows the 12-hr moving average for each canal segment for all design events. There were instances where the discharge capacity was negative, because of inflows exceeding outflows or due to timing mismatches in peak flows as the flood wave progresses downstream through the canal network.

For most canal segments, the discharge capacity generally increases as the return period grows. However, there are exceptions, such as Cypress1, CR951-S, COCO-3, FU-1 and FU-2, where the calculated discharge capacity for the 100-year storm is lower than for the 25-year storm or even the 5-year storm event at some instances. These discrepancies arise due to backwater effects and variations in the timing of peak flows during larger storms. In all of the above-mentioned canal segments, there were recorded multiple peaks. The second or third peak occurs when the downstream tailwater recedes, which allows stored water to be released.

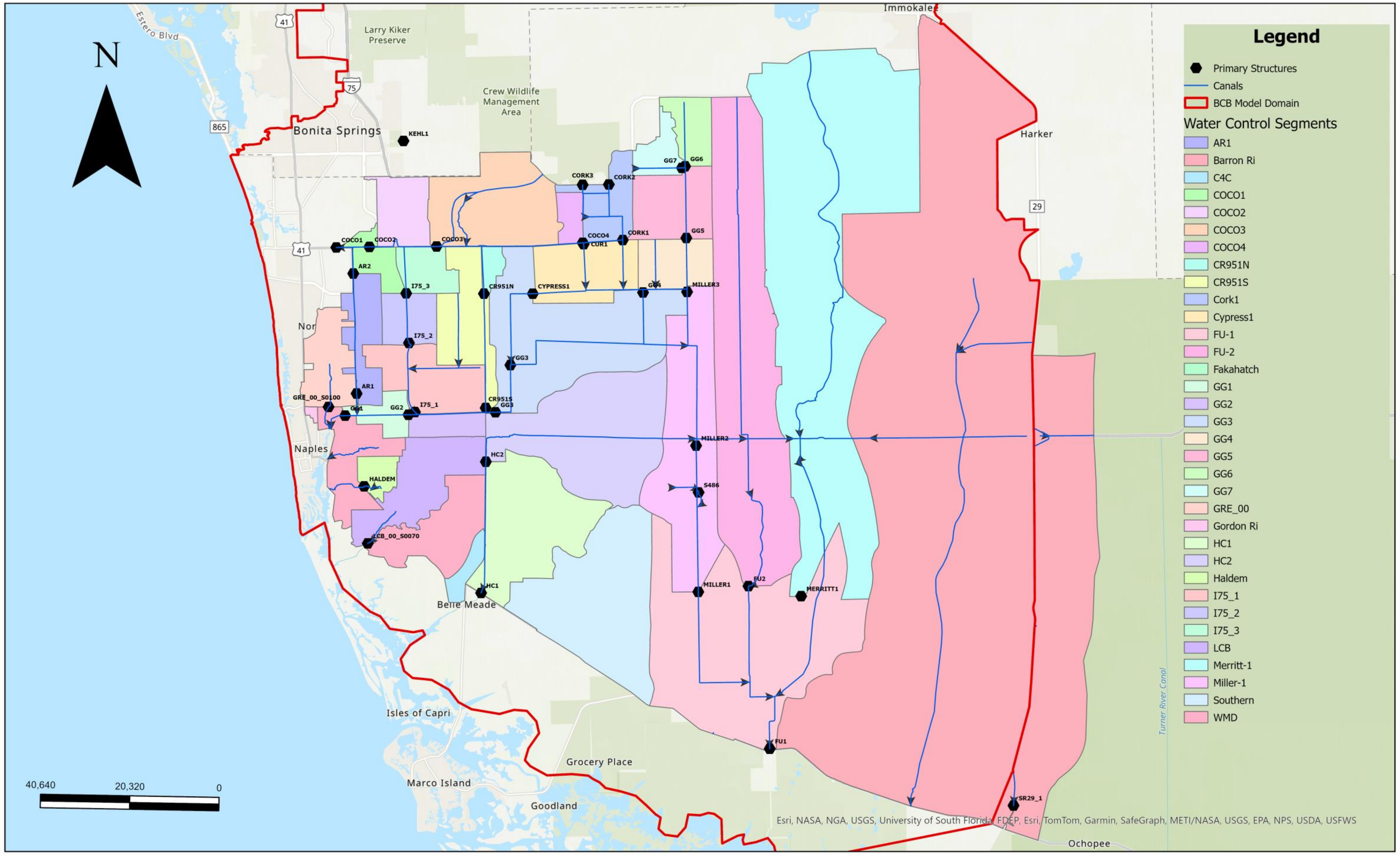

Figure 4-34. BCB Sub-watersheds used for calculating PM#2

Table 4-3. Control Catchments Inflow and Outflow Points and Discharge Capacity for 25-year Design Event

Structure/Segment	Inflow Point(s)	Outflow Points(s)	Total Area (mi2) Sub watersheds	25- Year (cfs/sq. mi.) Discharge Capacity
GG1	GG2, AR1	GG1	3.10	69.49
GG2	I75-1, GG3, CR951-S	GG2	3.15	24.86
GG3	CYPRESS1, GG4	GG3	29.18	13.88
GG4	GG5	GG4, MILLER3	6.10	17.63
GG5	GG6, GG7	GG5	8.91	81.50
GG6		GG6	4.36	143.06
GG7		GG7	3.35	62.56
Airport Rd		AR1, AR2	6.29	102.22
I75-1	I75-2	I75-1	9.8	28.67
I75-2	I75-3	I75-2	4.64	96.56
I75-3		I75-3	4.11	105.32
CR951-S	CR951-N	CR951-S	9.58	29.53
CR951-N		CR951-N	1.43	82.88
CYPRESS1	CUR1, CORK1	CYPRESS1	9.88	-48.67
CORK1	CORK2, CORK3	CORK1, CUR1, COCO4	6.27	92.19
COCO1	COCO2, AR2	COCO1	3.42	119.69
COCO2	COCO3	COCO2	6.12	84.06
COCO3	COCO4	COCO3	19.2	3.20
HC1	HC2	HC1	19.71	19.48
HC2		HC2	27.23	11.71
GRE_00_S0100 (Gordon)		GRE_00_S0100	7.02	147.49
FU1	Miller1, FU2, Merritt1	FU1	48.91	-33.37
FU2		FU2	48.82	54.19
Miller1	Miller3	Miller1	25.29	20.48
Merrit1		Merrit1	89.08	0.93
Halderman (HCB)		HCB	2.36	239.00
Lely Canal (LCB-00-S0100)		LCB	11.55	168.25

Table 4-4. Peak discharge rate at BCB control structures summary

Structure	Canal	Chainage	Peak Discharge (cfs)			
			5 yr	10 yr	25 yr	100 yr
ARN Amill D-500	AirportRdN	4763.78	271.29	260.62	268.09	283.60
Air 1	AirportRdS	11896.33	556.36	564.91	579.57	621.13
ARN Amill D-700	AirportRdS	22227.69	137.44	145.37	153.77	160.23
CC4	CocohatcheeEast	2181.76	142.84	149.53	157.68	170.08
CC3	CocohatcheeWest	27329.40	186.69	201.67	224.53	266.45
CC2	CocohatcheeWest	43779.53	667.91	748.38	852.17	908.70
CC1	CocohatcheeWest	50524.93	1341.62	1509.91	1746.99	2001.04
Cork2	CorkScrewCan	2952.76	129.87	153.24	178.88	216.38
Cork1	CorkScrewCan	18517.06	542.04	630.52	717.30	763.37
CR951S	CR951	36421.53	642.79	560.74	499.01	486.97
CR951N	CR951	10449.48	140.46	151.77	153.92	167.24
Cyp1	CypressCan	22565.62	413.33	462.58	683.14	520.41
FU7	FakaUnionCan	11237.84	314.15	427.15	615.22	910.73
FU6	FakaUnionCan	23018.37	674.17	917.54	1263.01	1674.57
FU5	FakaUnionCan	34569.06	972.63	1160.19	1324.80	1519.68
FU4	FakaUnionCan	62514.76	1777.52	1814.94	1885.51	2750.96
FU3	FakaUnionCan	81585.63	2273.91	2392.81	2496.81	2569.15
FU2	FakaUnionCan	112099.74	2552.43	3107.43	2708.57	2834.48
FU1	FakaUnionCan	154825.79	1981.41	2106.46	2248.17	2431.72
GG 7	GoldenGateBr	9946.10	199.00	230.20	267.63	303.59
GG 6	GoldenGateMain	14503.61	445.20	553.61	690.06	834.32
GG 5	GoldenGateMain	30757.87	1262.53	1525.66	1733.65	1952.42
GG 4	GoldenGateMain	52004.59	1007.14	1059.01	1109.51	1172.00
GG 3	GoldenGateMain	110106.36	1760.02	1812.90	1871.46	1936.29
GG 2	GoldenGateMain	127909.45	3176.44	9082.80	3351.91	4965.23
GG 1	GoldenGateMain	140798.88	3615.61	3831.70	3980.43	4280.55
Harvey1	HarveyCan	8841.86	14.28	13.24	11.81	13.48
HC2	HendersonCr	15369.09	304.88	310.13	319.35	342.28
HC1	HendersonCr	40354.33	610.71	723.26	737.47	825.22
I751	I-75Can	38152.89	1191.20	1246.75	1279.69	1309.43
I752	I-75Can	21653.54	1002.53	1011.75	1060.75	1139.60
I753	I-75Can	10400.26	429.07	431.48	481.93	526.56
Kehl1	KehlCan	26010.50	506.17	556.94	629.98	703.61
Miller 3	MillerCan	555.64	576.37	637.76	687.43	716.41
Miller 2	MillerCan	37237.53	992.55	1032.08	1812.36	1071.76
Miller 1	MillerCan	70275.59	1058.81	1127.85	1164.60	1249.88

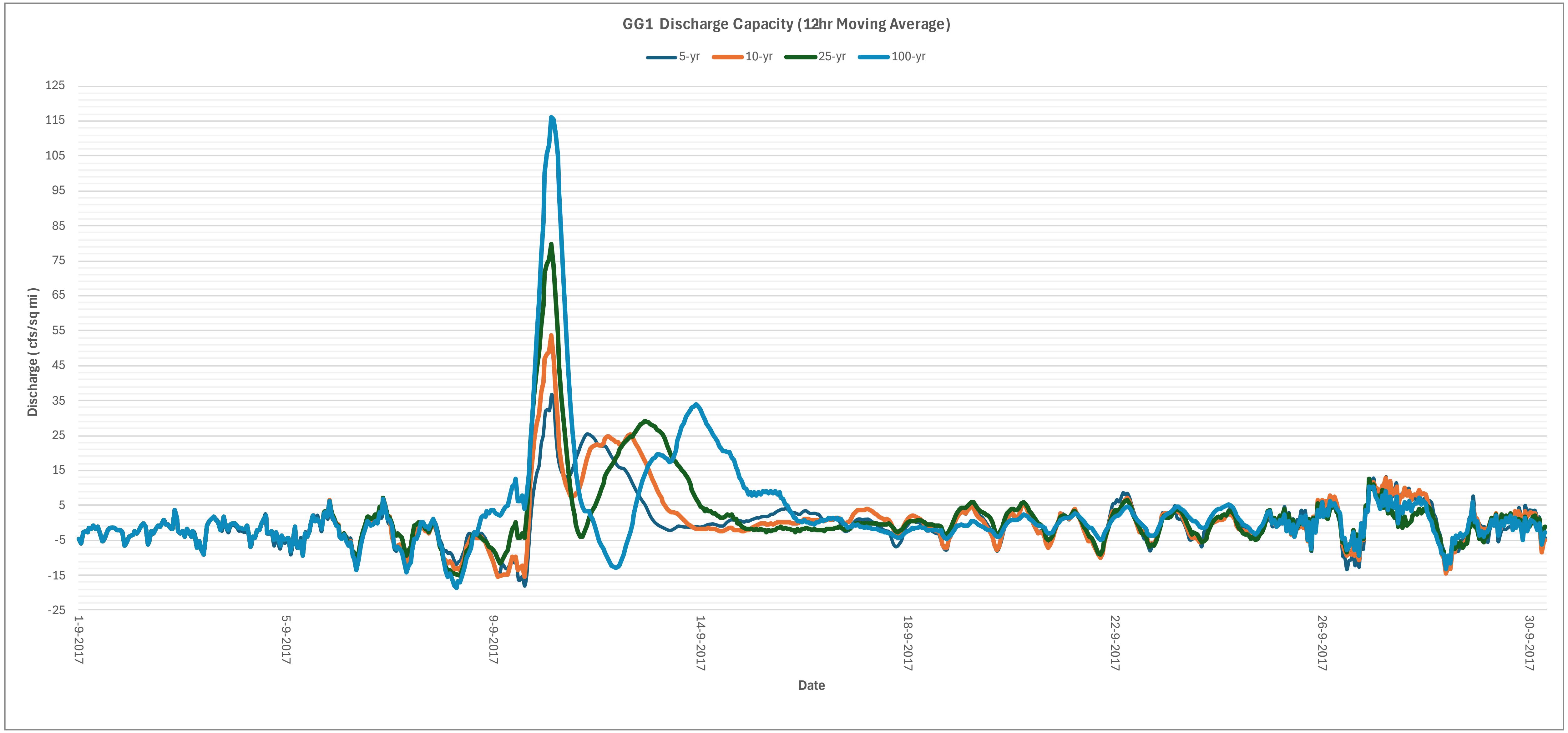

Figure 4-35. Discharge Capacity Hydrograph for GG1

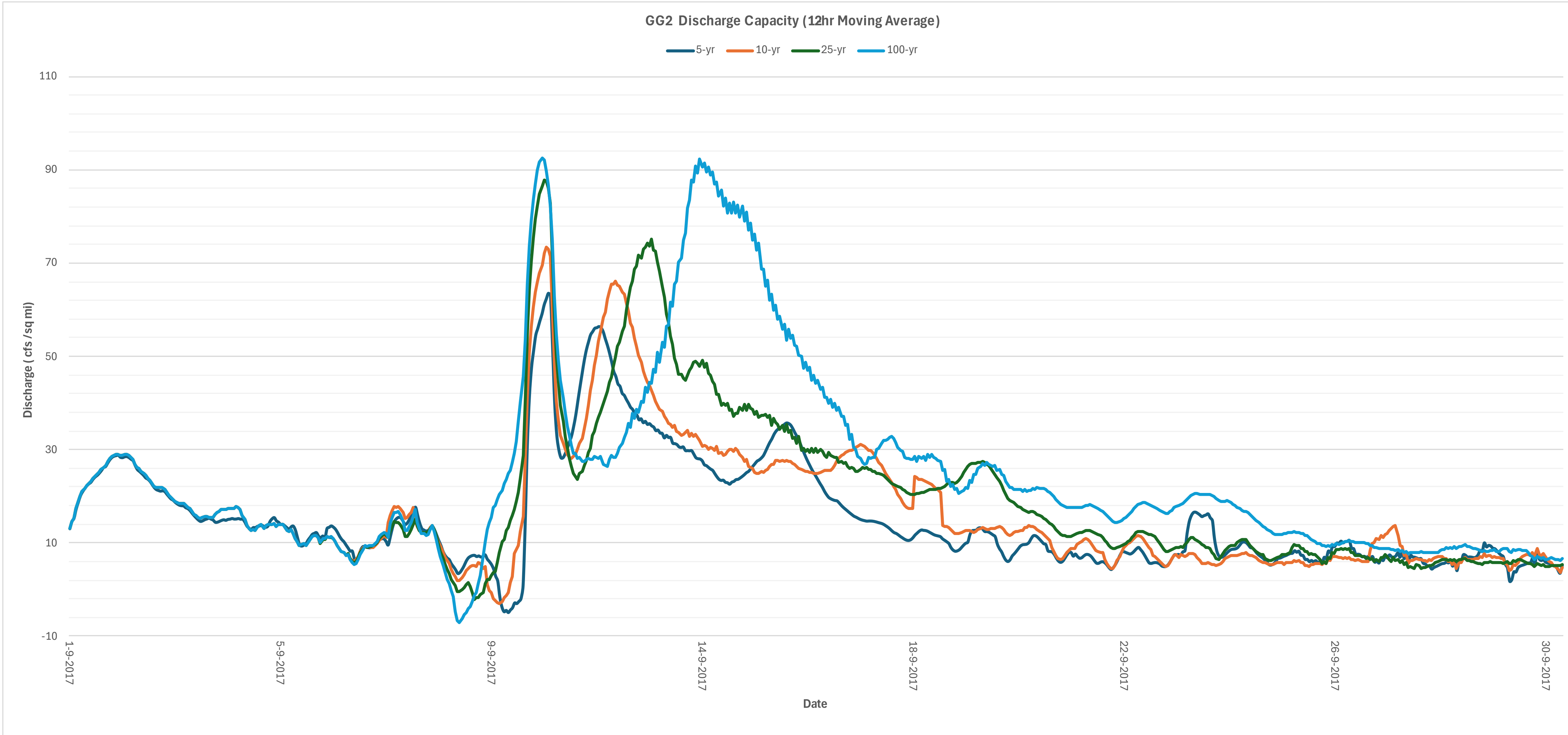

Figure 4-36. Discharge Capacity Hydrograph for GG2

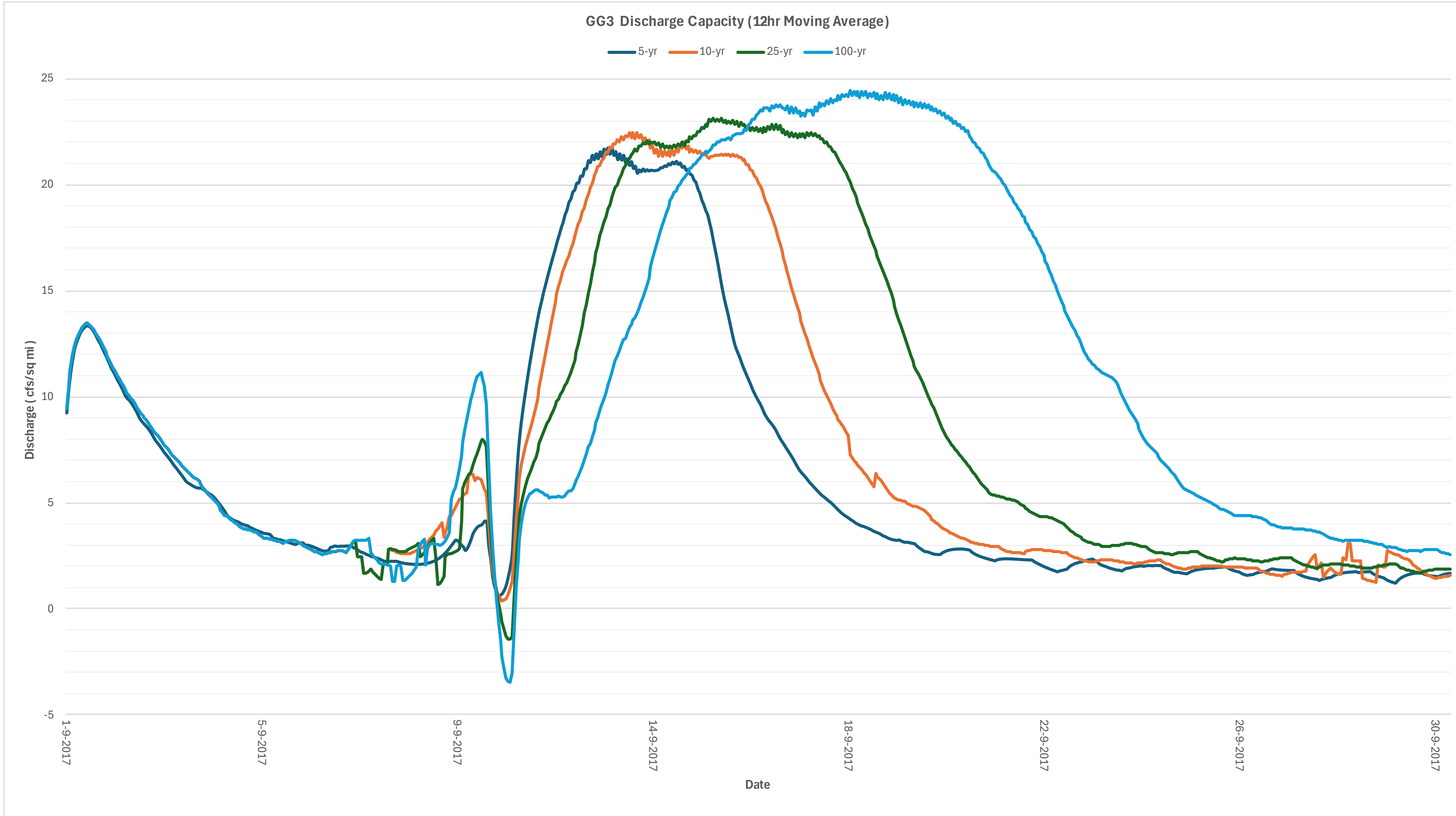

Figure 4-37. Discharge Capacity Hydrograph for GG3

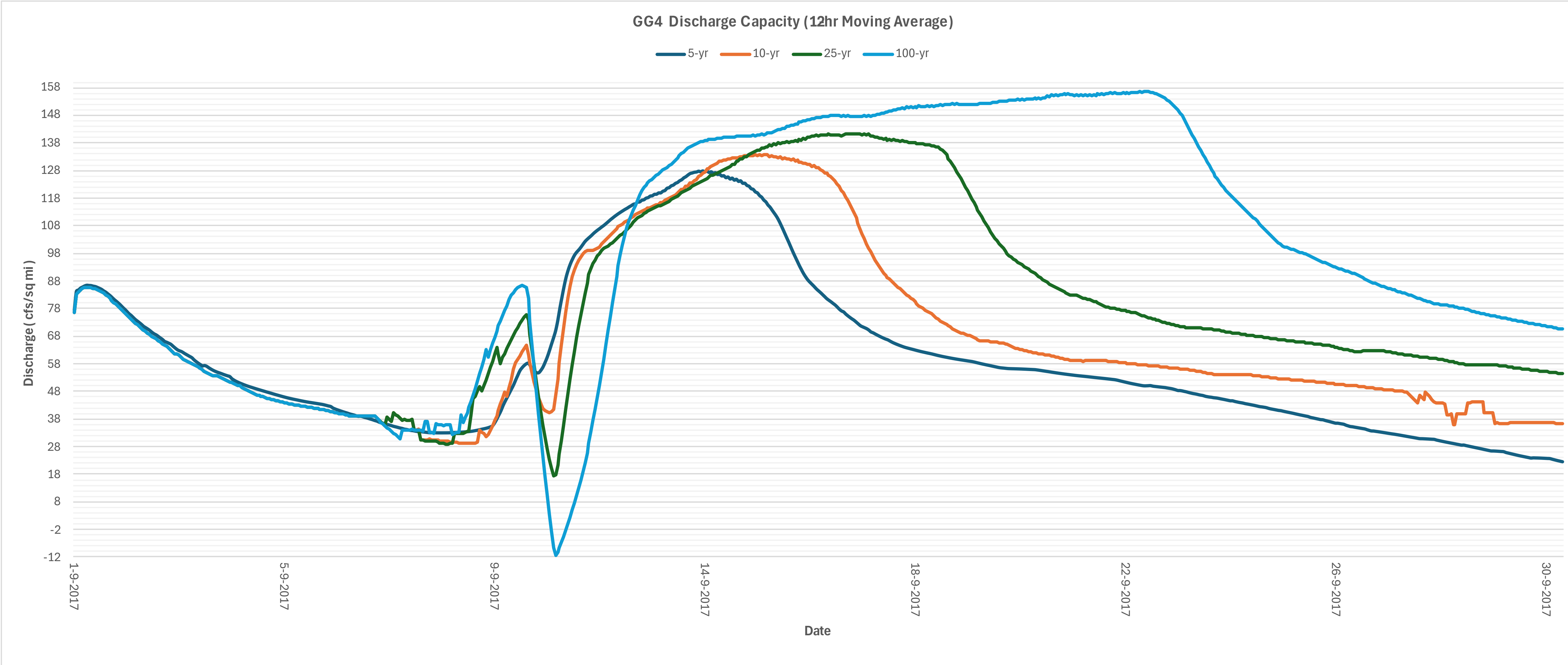

Figure 4-38. Discharge Capacity Hydrograph for GG4

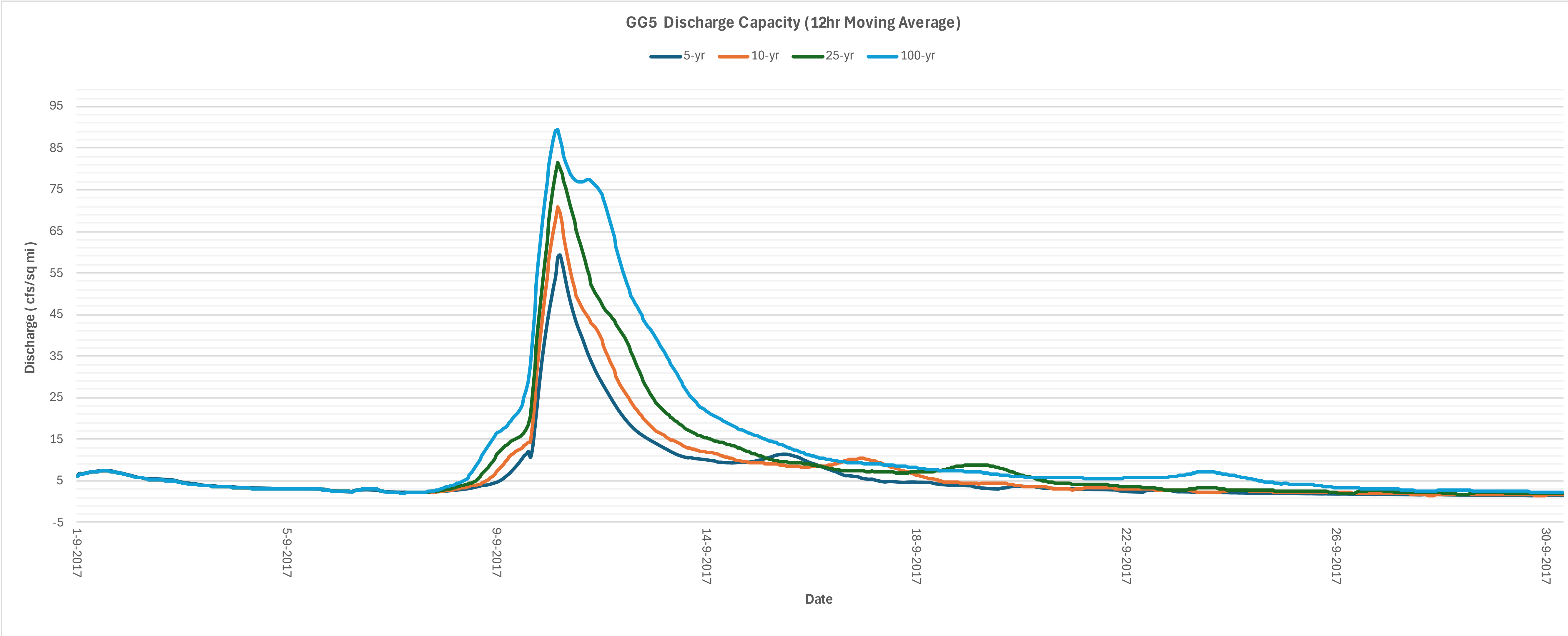

Figure 4-39. Discharge Capacity Hydrograph for GG5

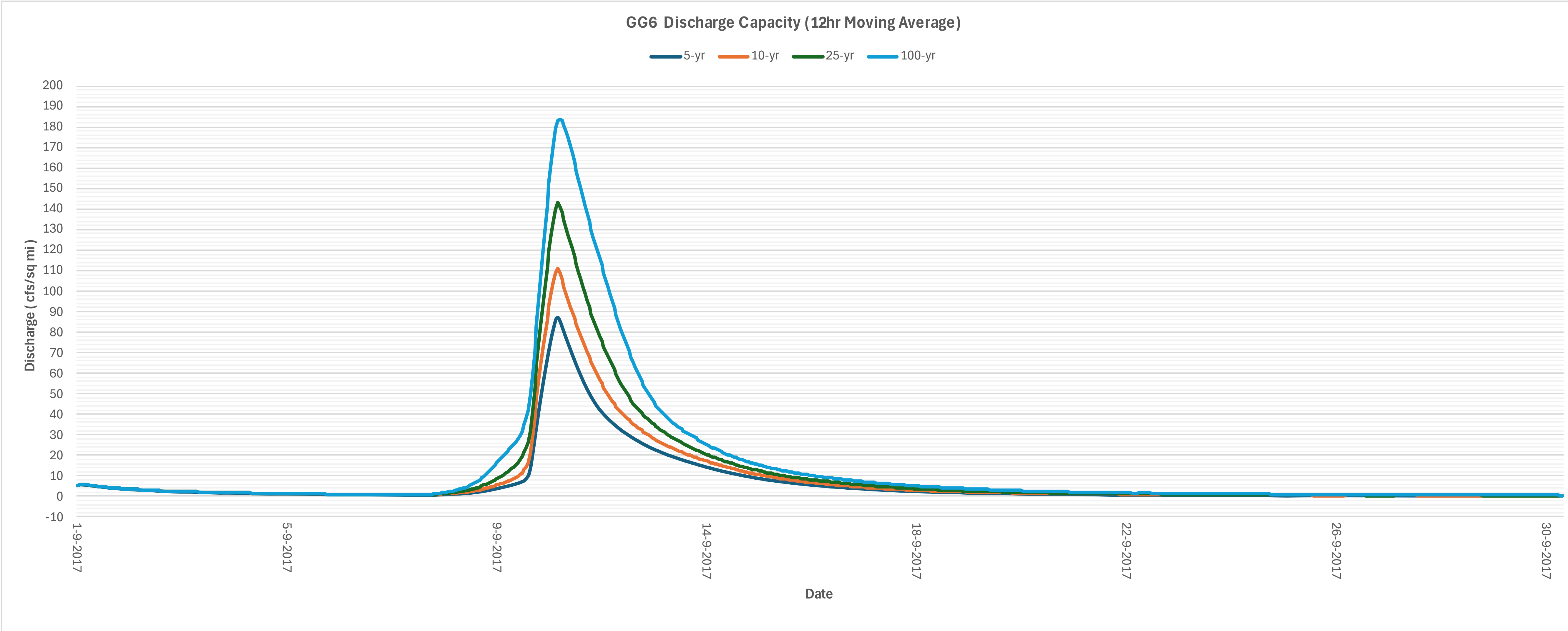

Figure 4-40. Discharge Capacity Hydrograph for GG6

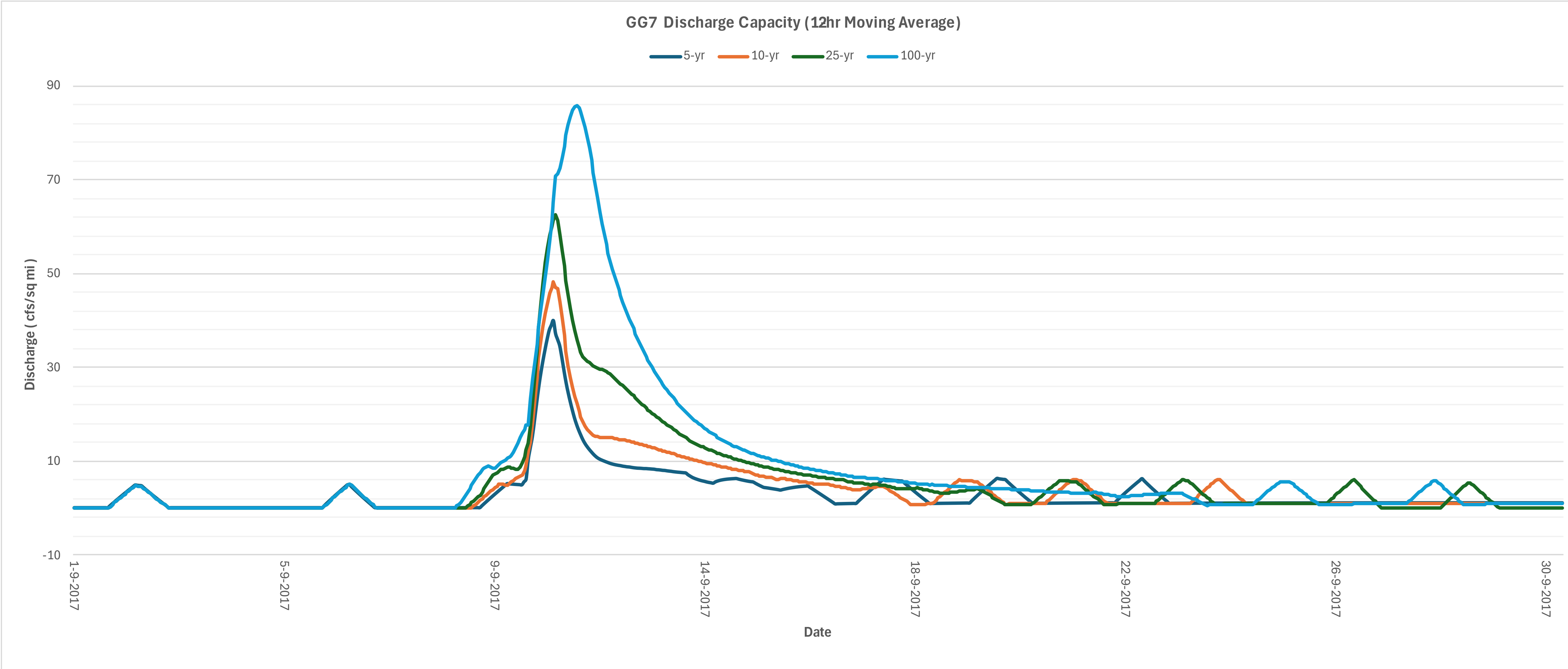

Figure 4-41. Discharge Capacity Hydrograph for GG7

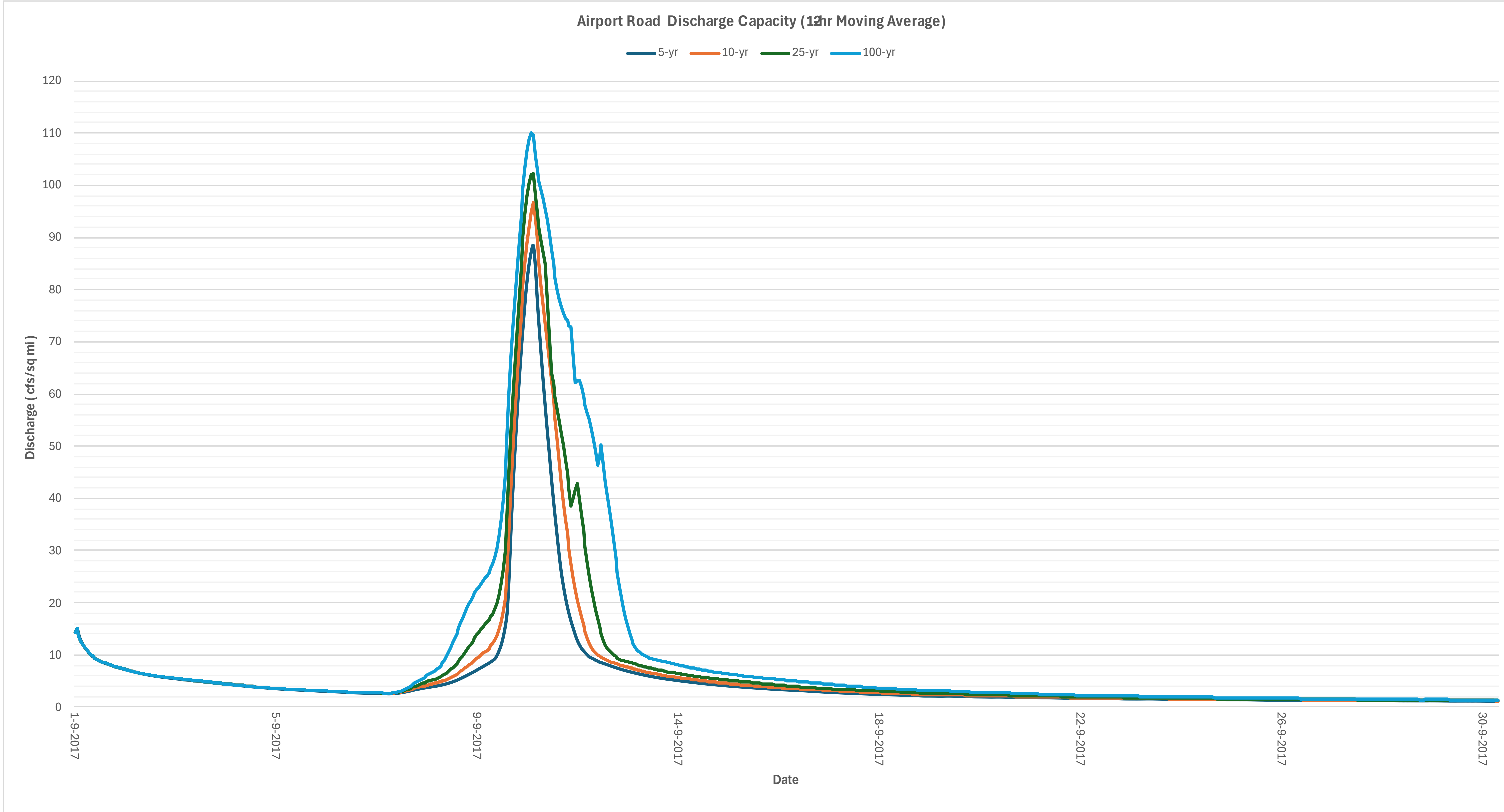

Figure 4-42. Discharge Capacity Hydrograph for Airport Road

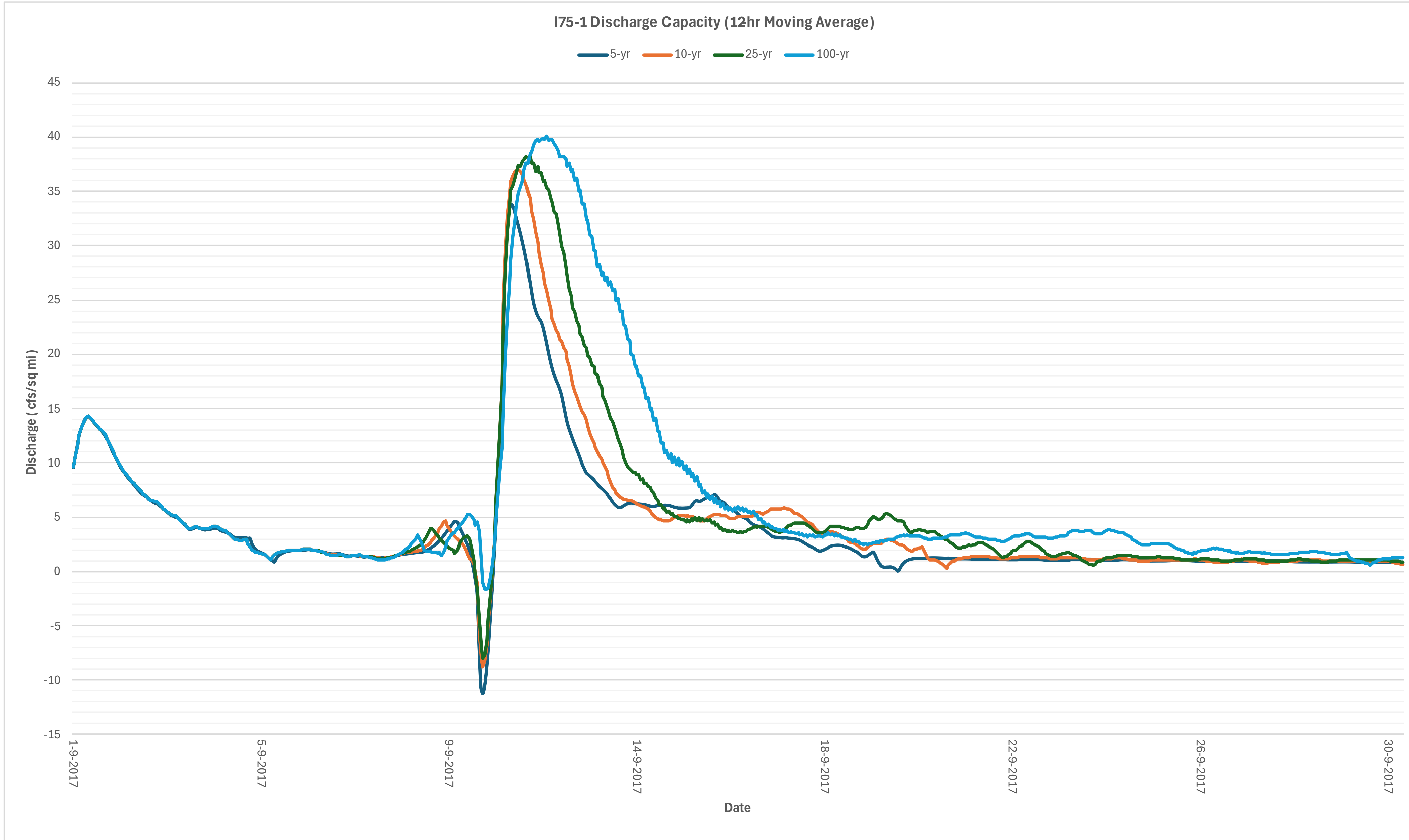

Figure 4-43. Discharge Capacity Hydrograph for I75-1

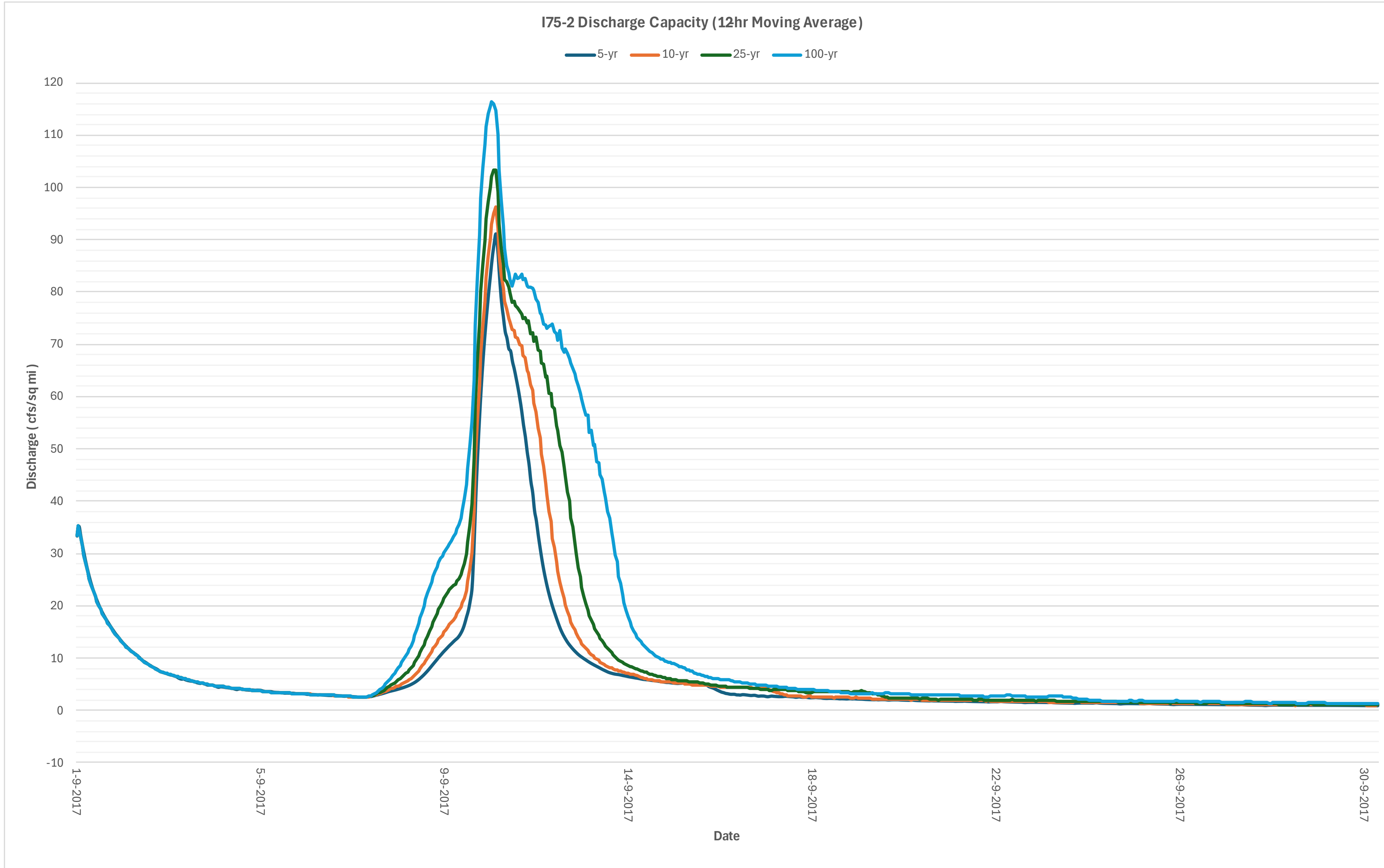

Figure 4-44. Discharge Capacity Hydrograph for I75-2

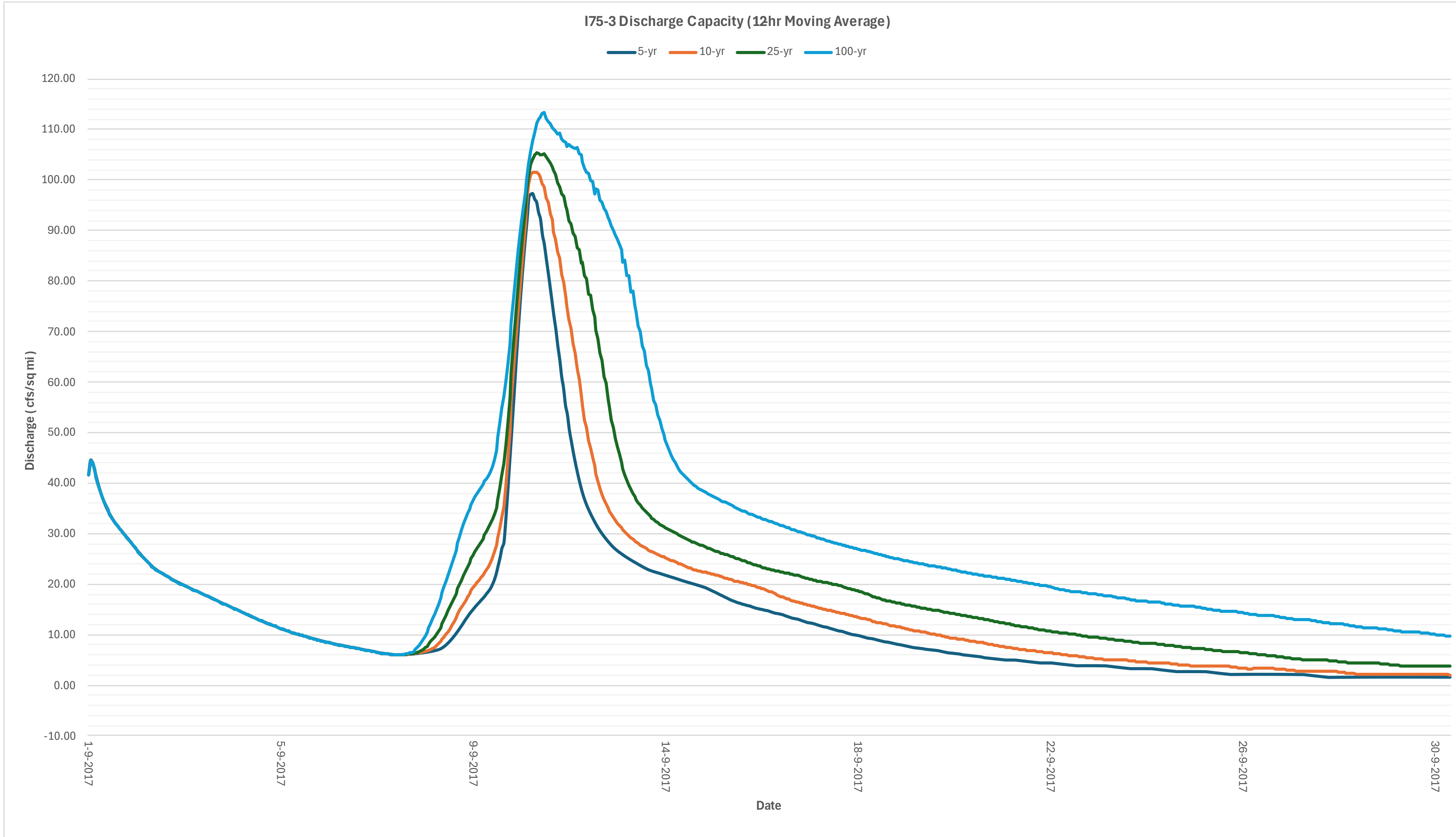

Figure 4-45. Discharge Capacity Hydrograph for I75-3

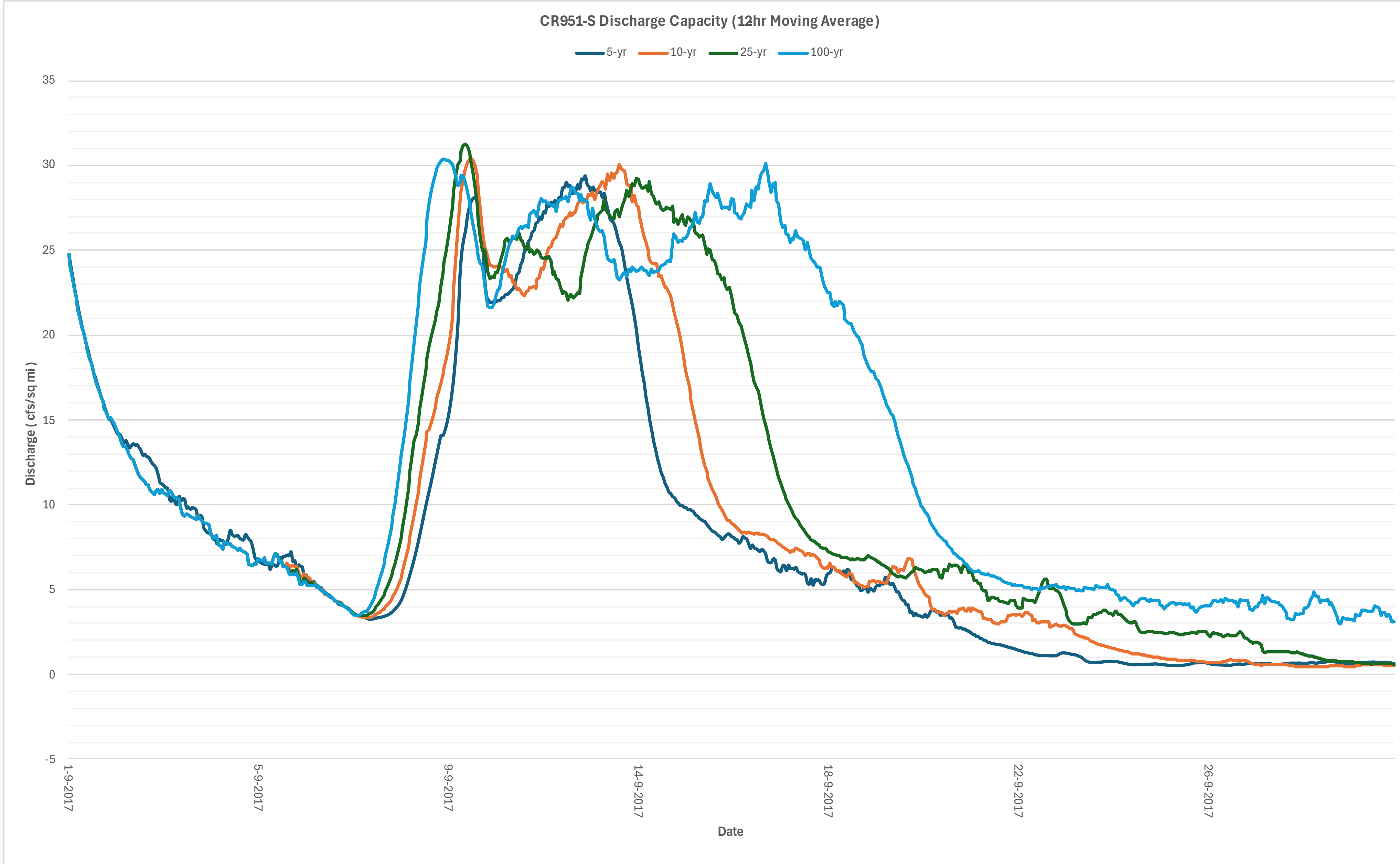

Figure 4-46. Discharge Capacity Hydrograph for CR951-S

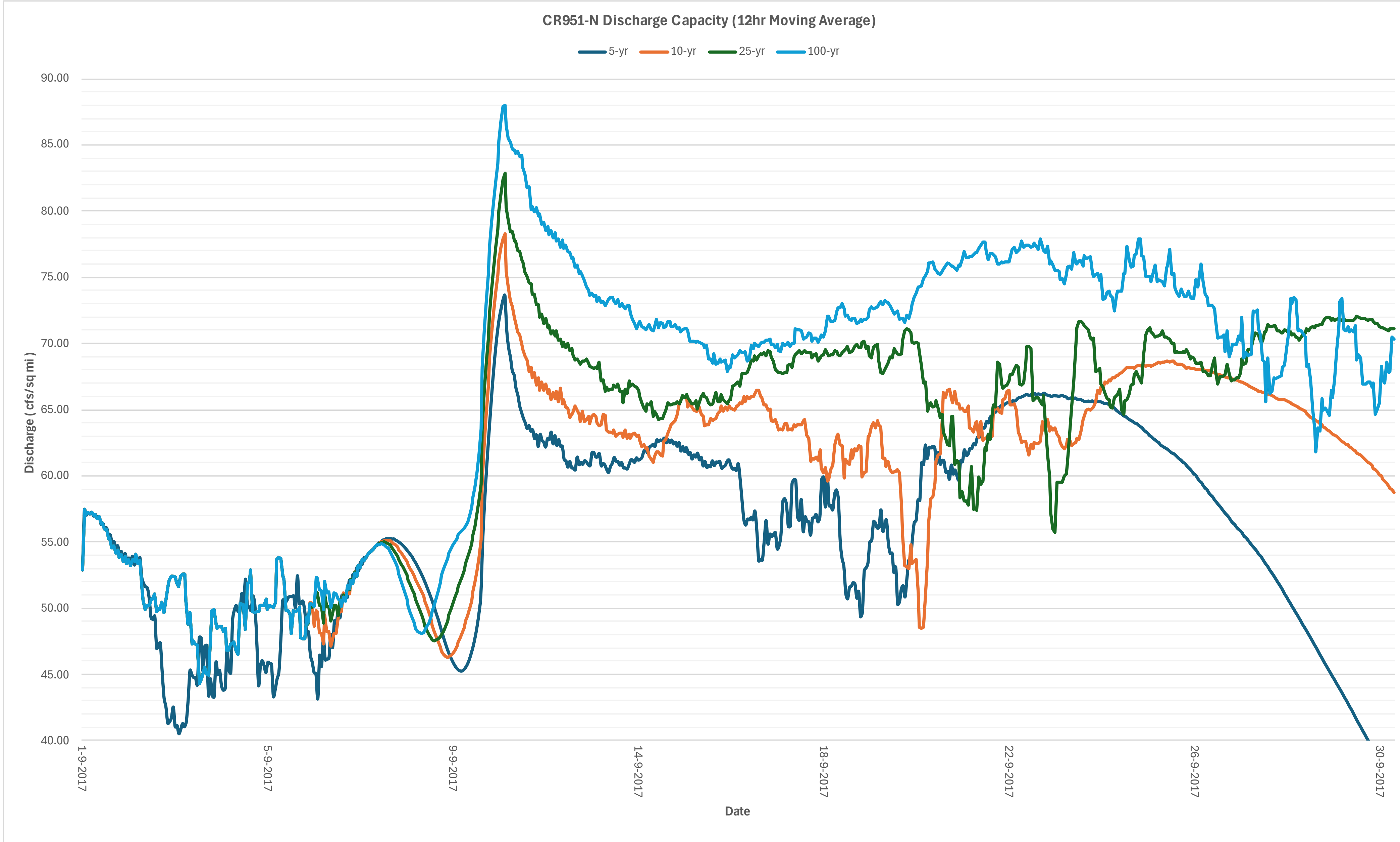

Figure 4-47. Discharge Capacity Hydrograph for CR951-N

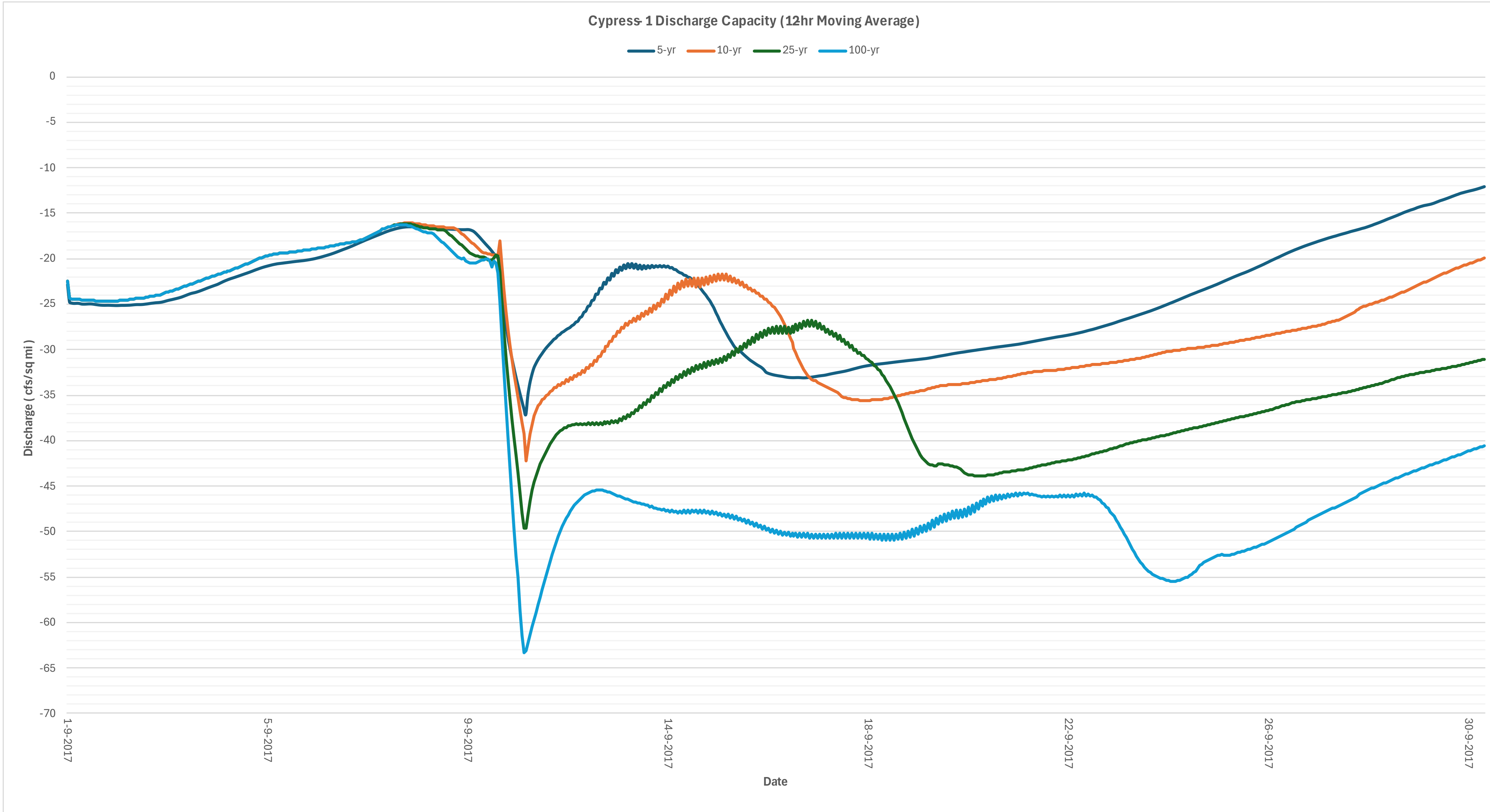

Figure 4-48. Discharge Capacity Hydrograph for Cypress 1

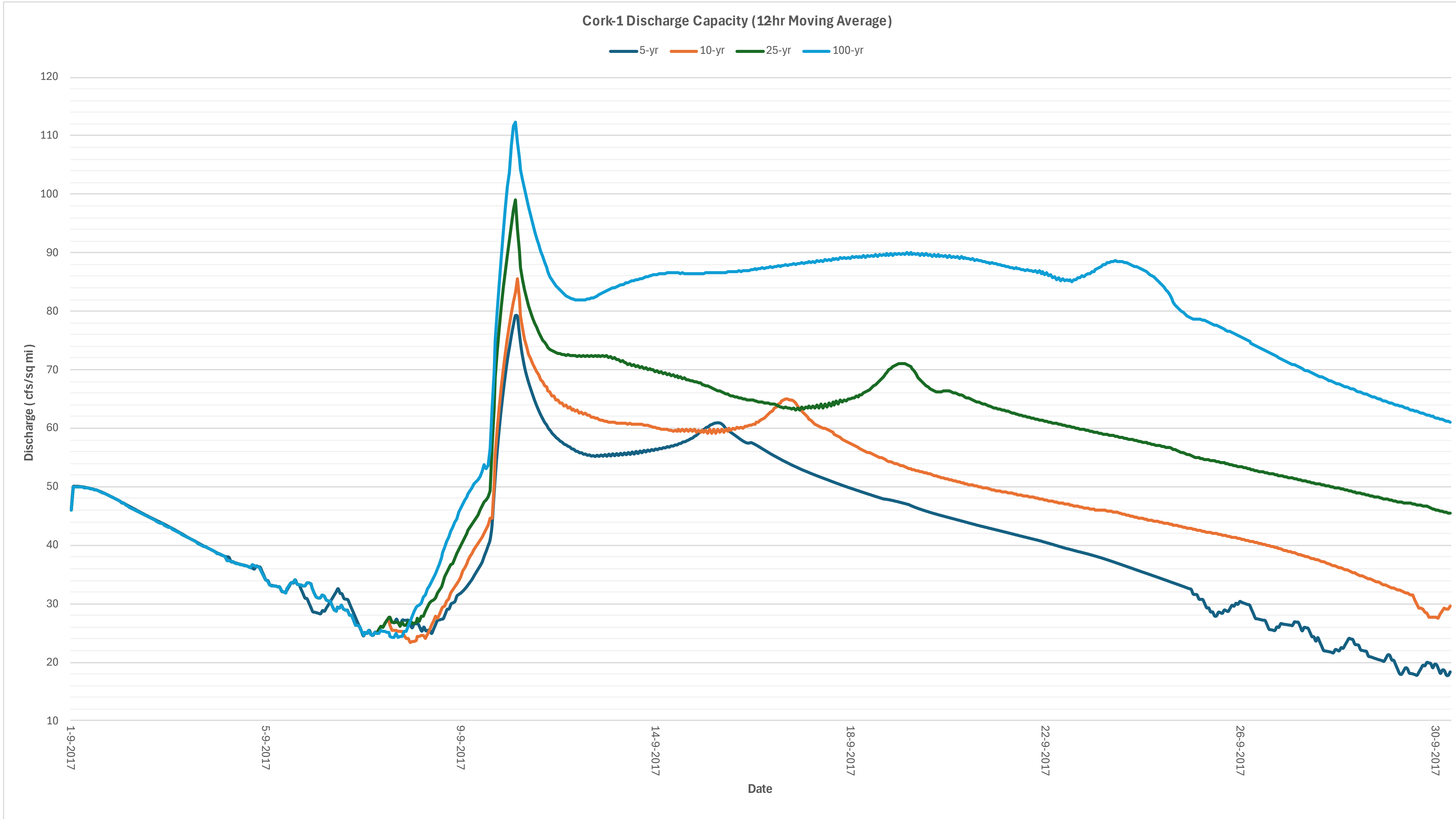

Figure 4-49. Discharge Capacity Hydrograph for Cork 1

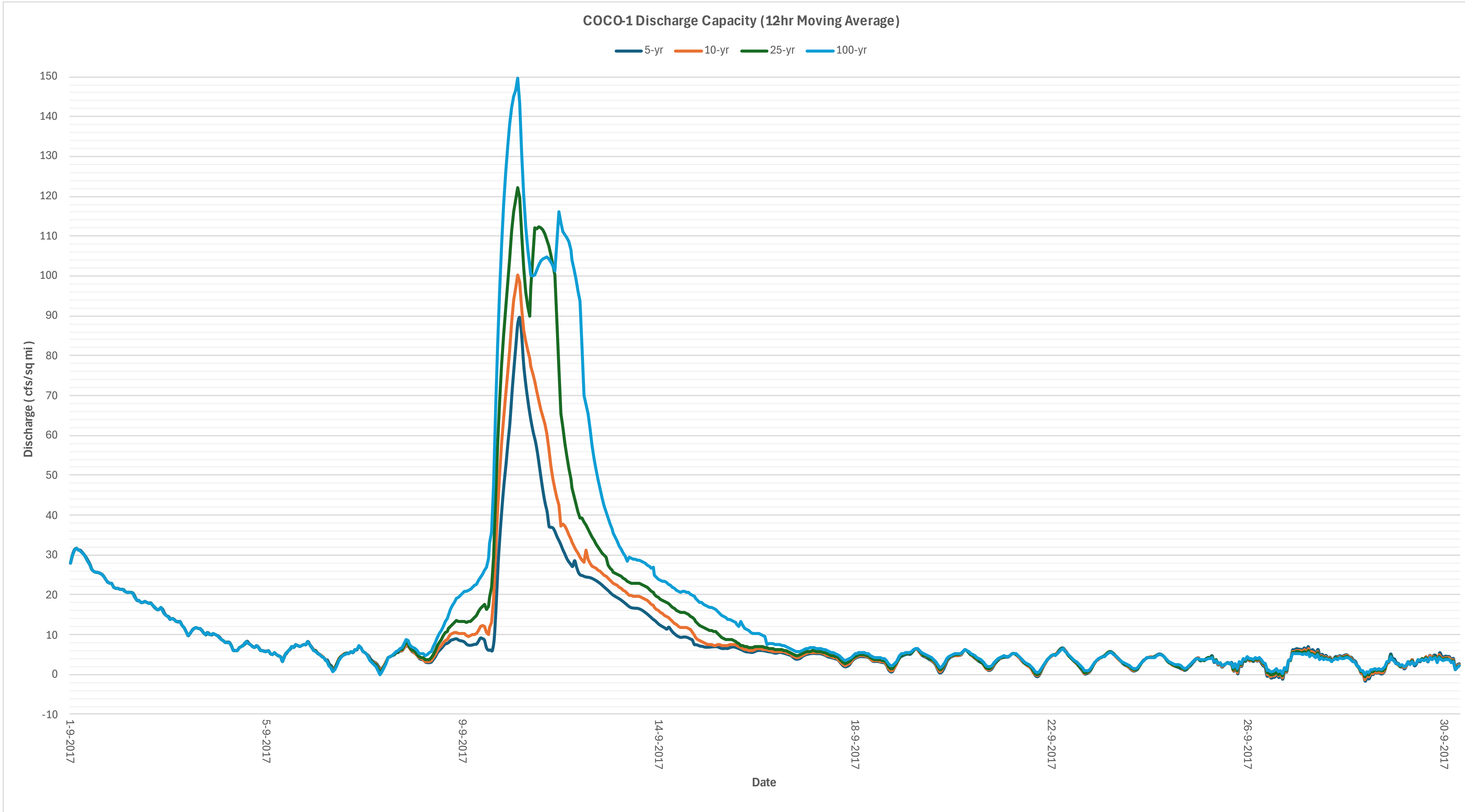

Figure 4-50. Discharge Capacity Hydrograph for COCO-1

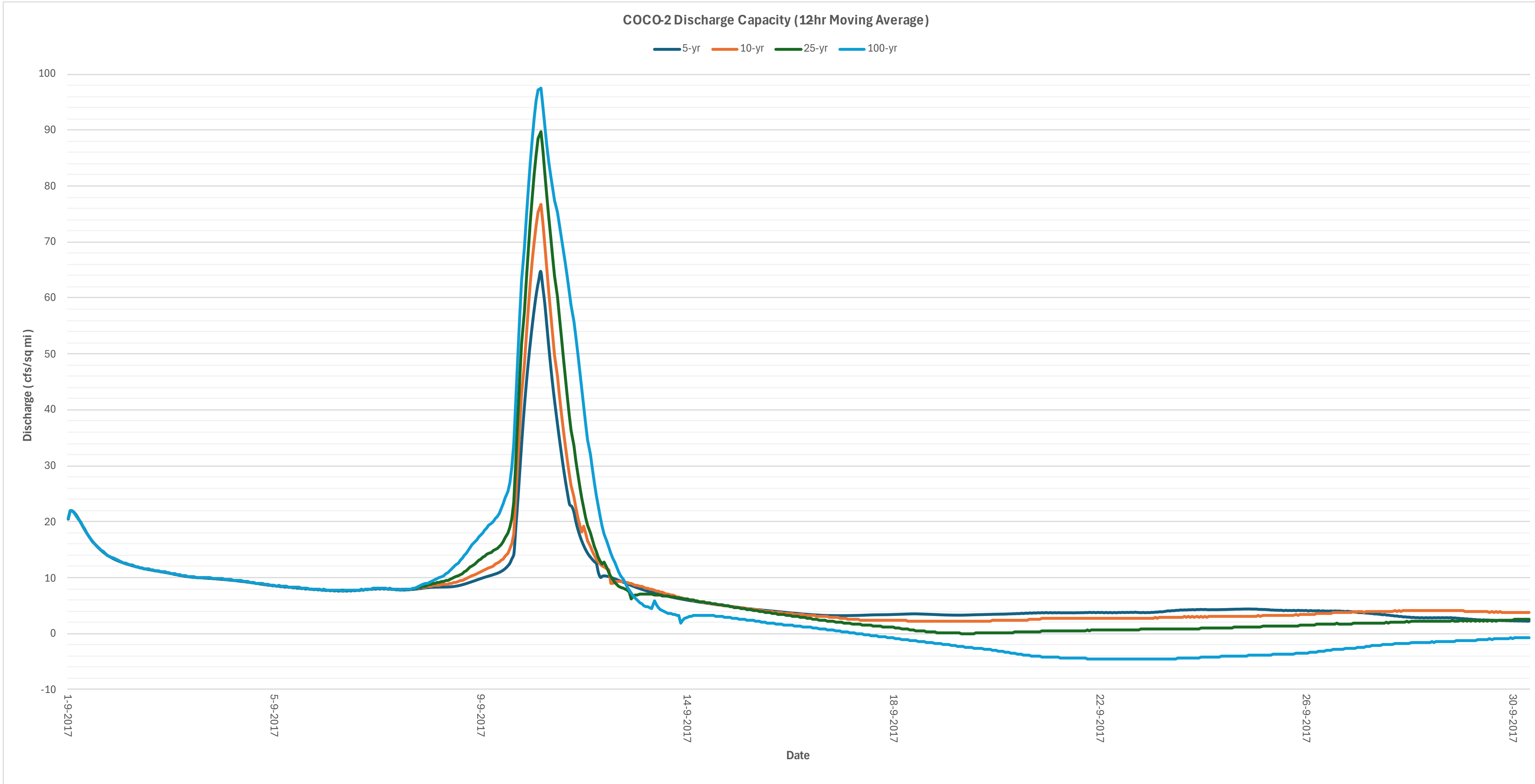

Figure 4-51. Discharge Capacity Hydrograph for COCO-2

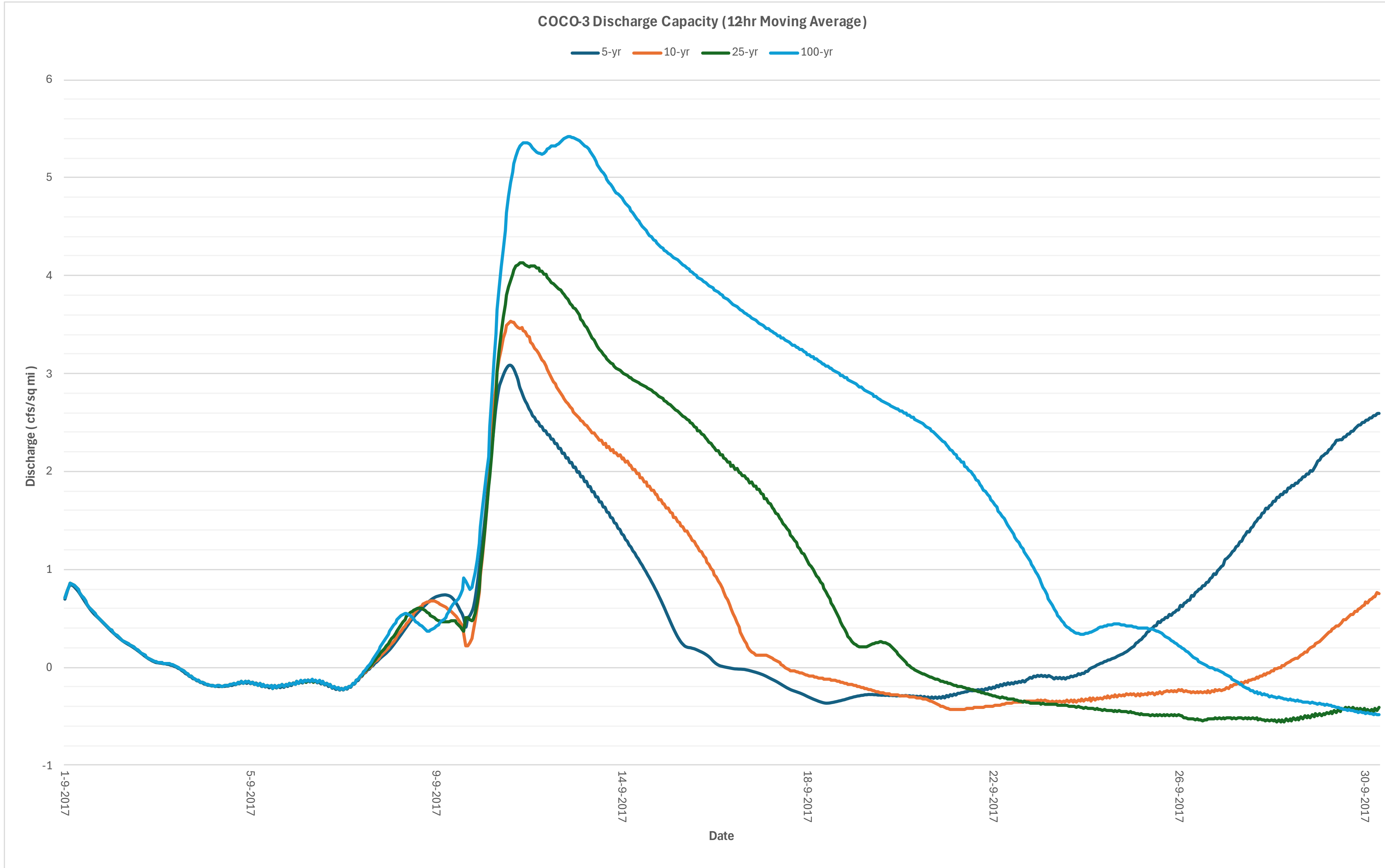

Figure 4-52. Discharge Capacity Hydrograph for COCO-3

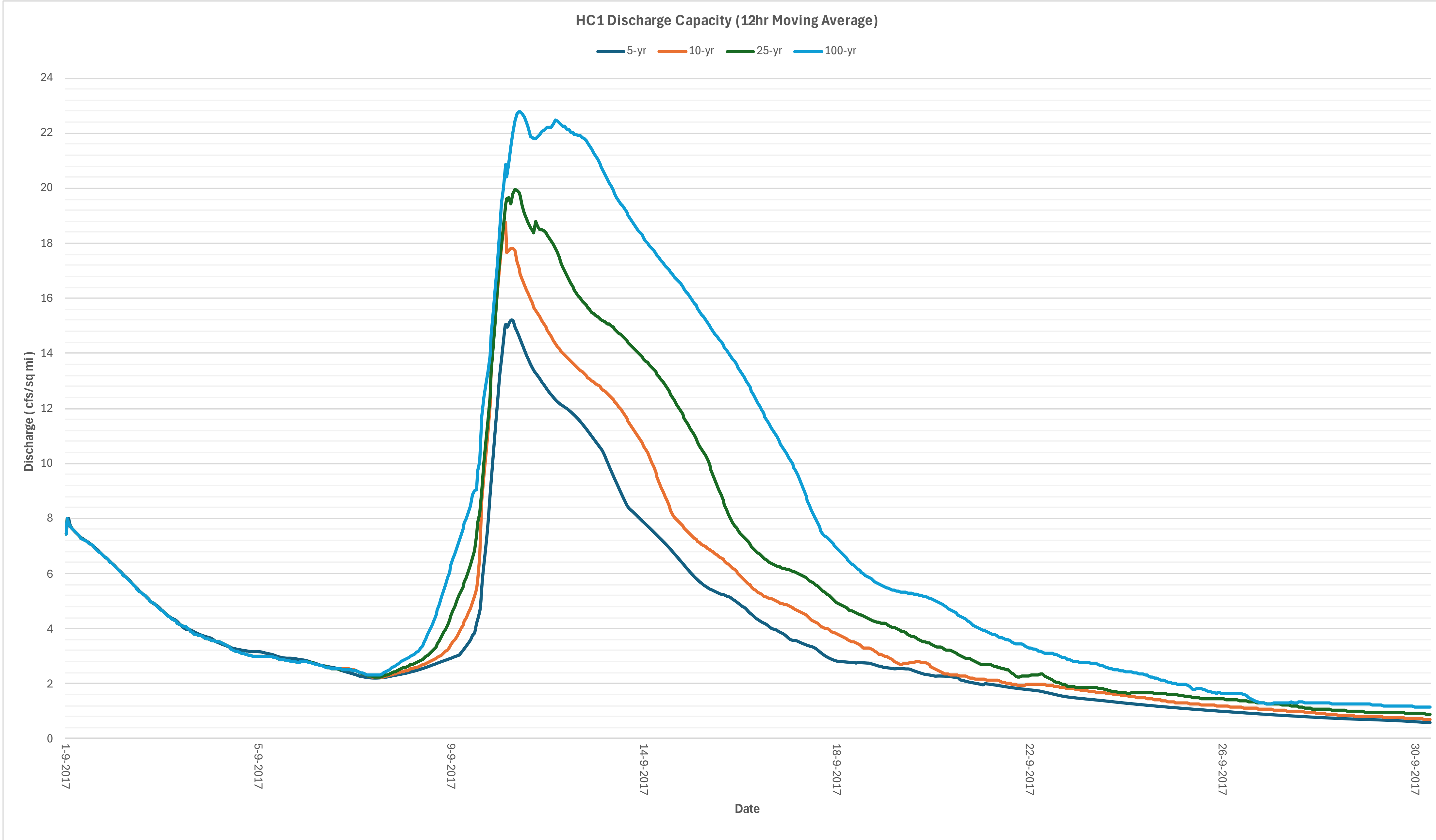

Figure 4-53. Discharge Capacity Hydrograph for HC1

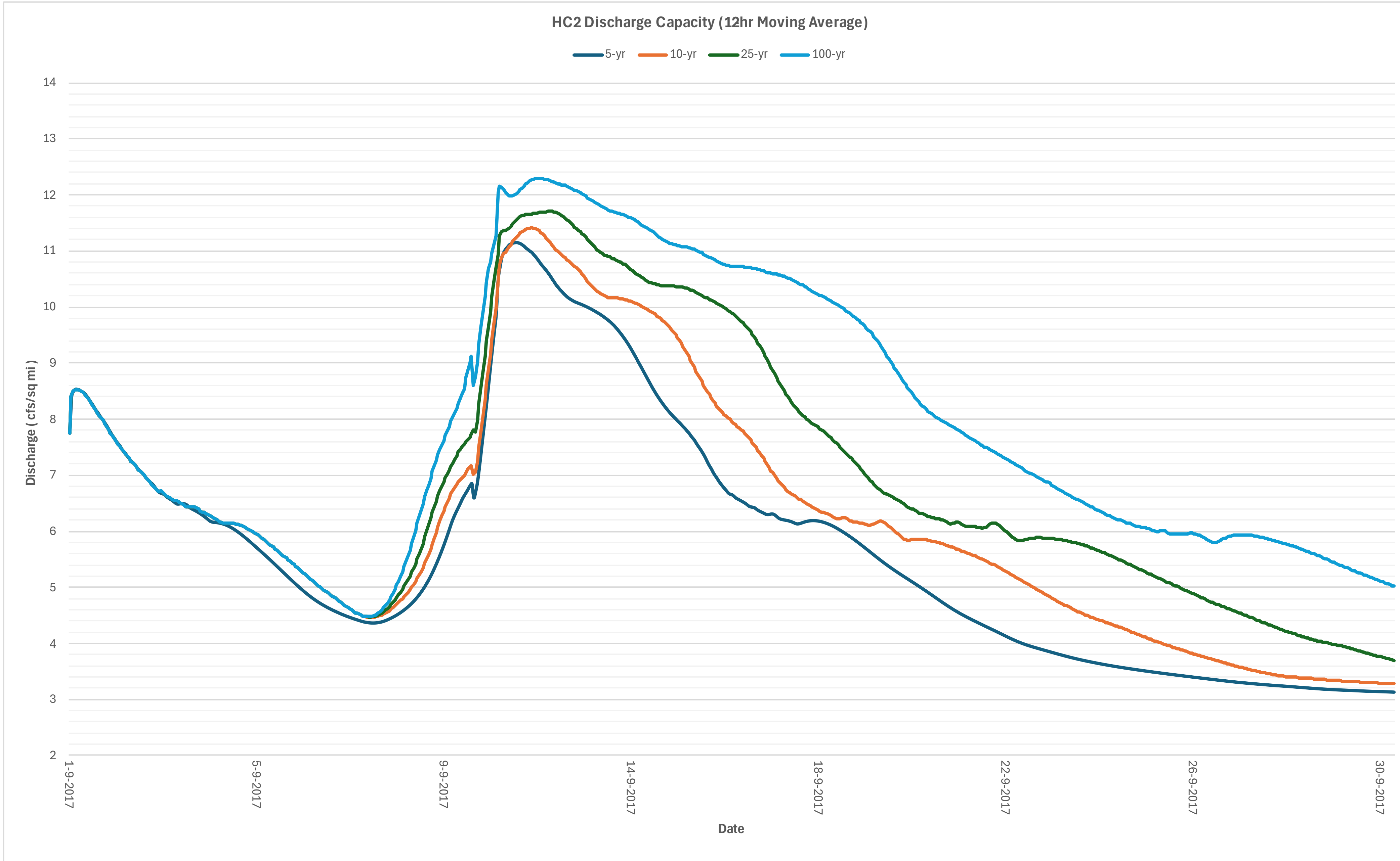

Figure 4-54. Discharge Capacity Hydrograph for HC2

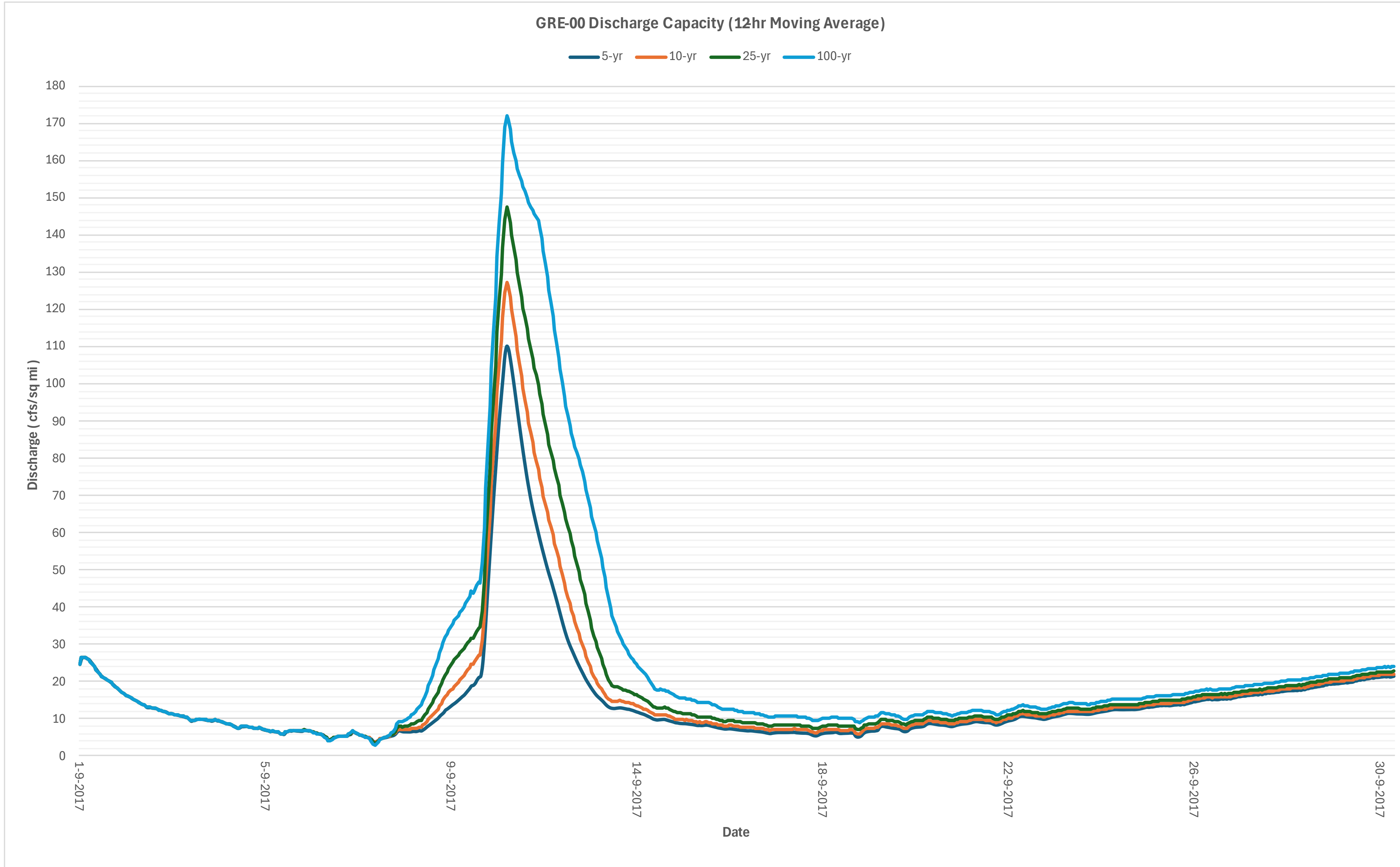

Figure 4-55. Discharge Capacity Hydrograph for GRE-00

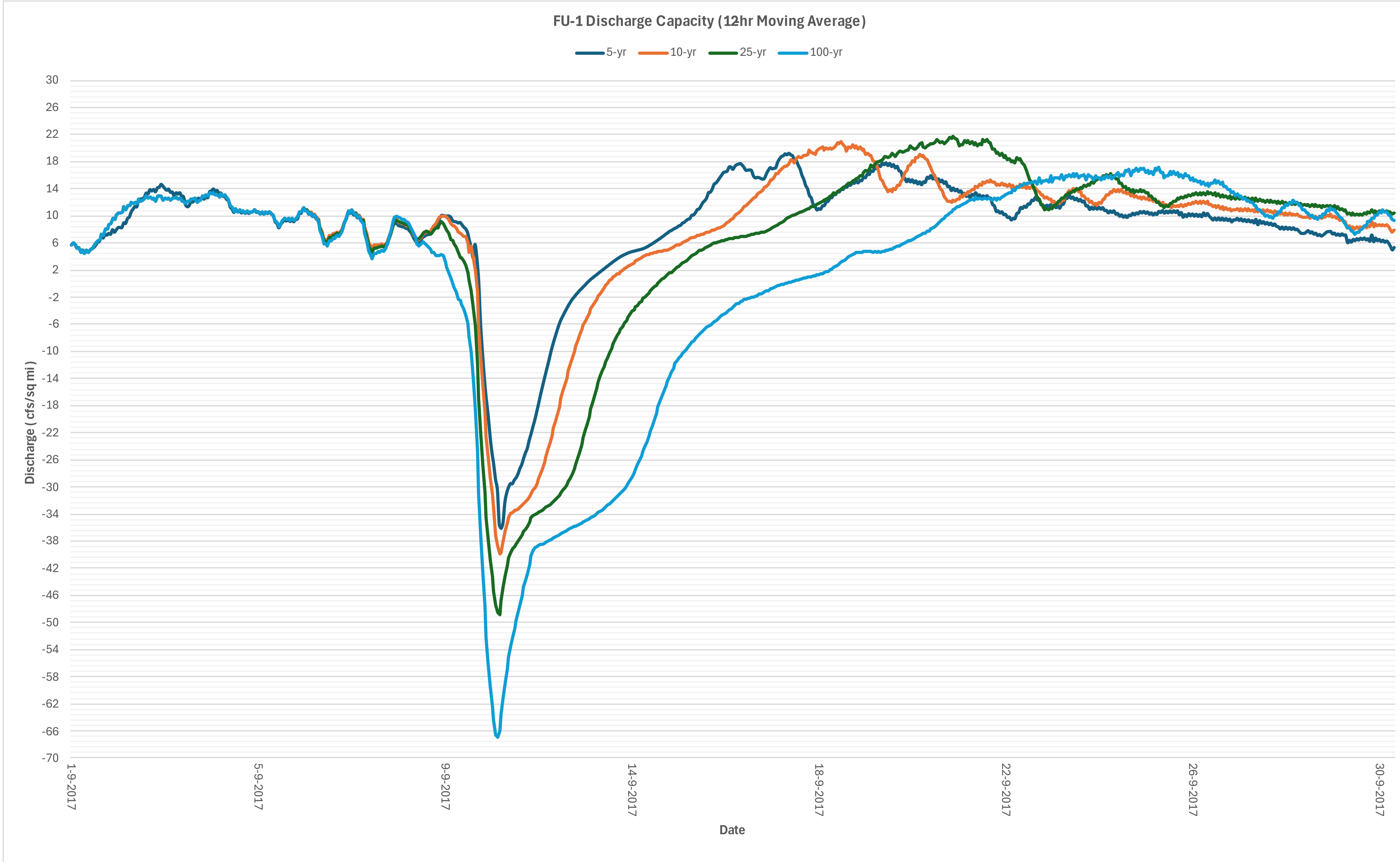

Figure 4-56. Discharge Capacity Hydrograph for FU-1

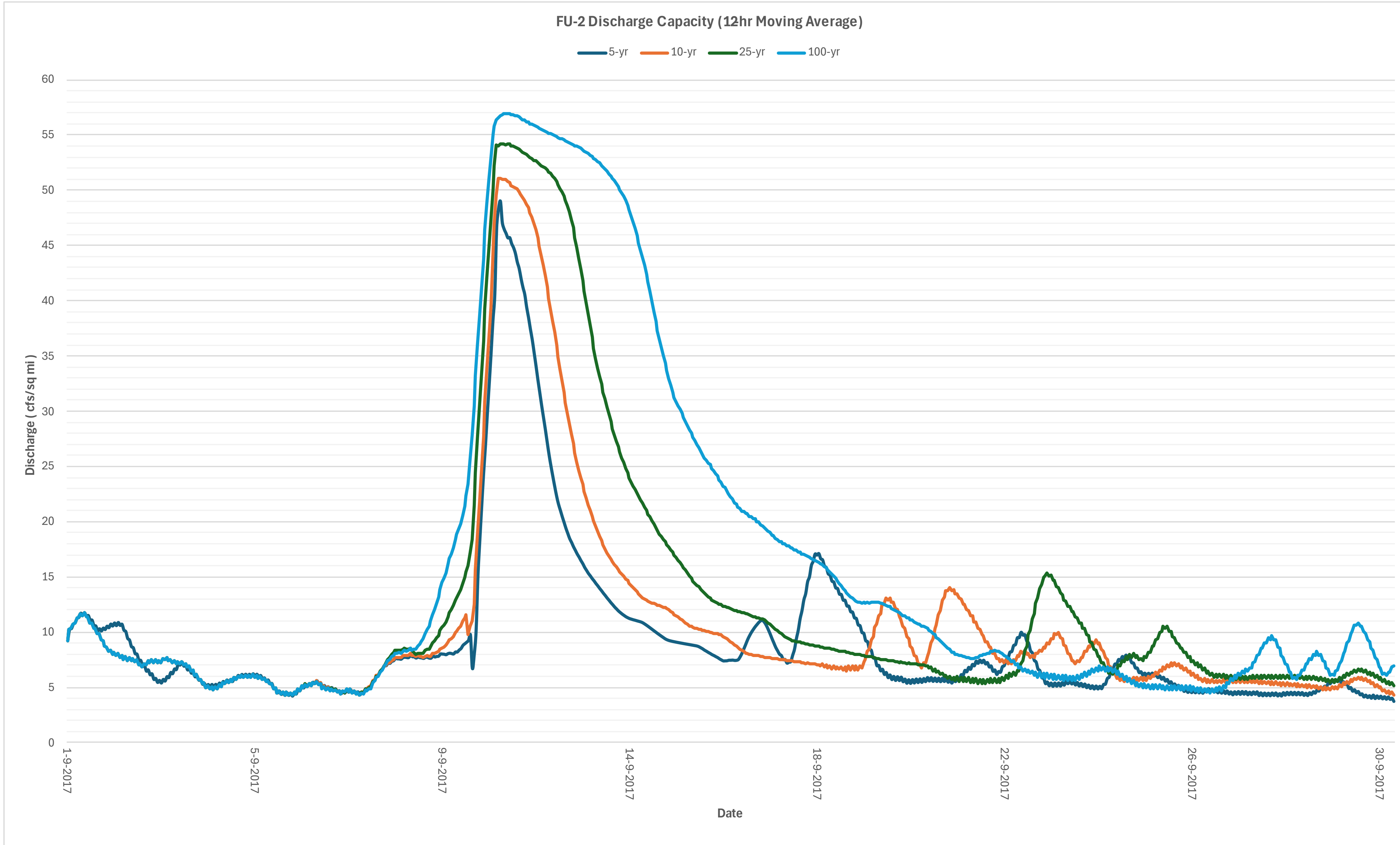

Figure 4-57. Discharge Capacity Hydrograph for FU-2

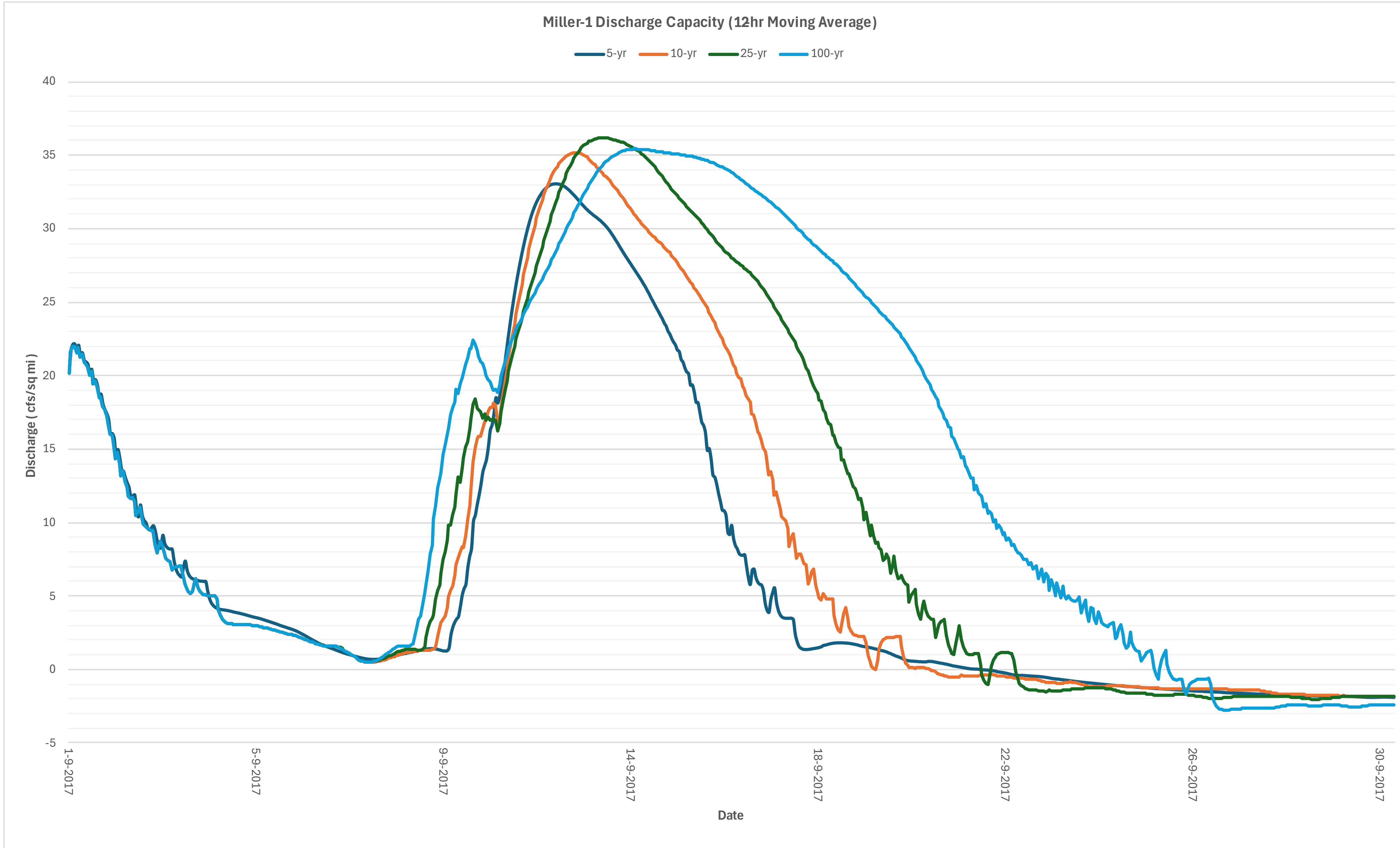

Figure 4-58. Discharge Capacity Hydrograph for Miller-1

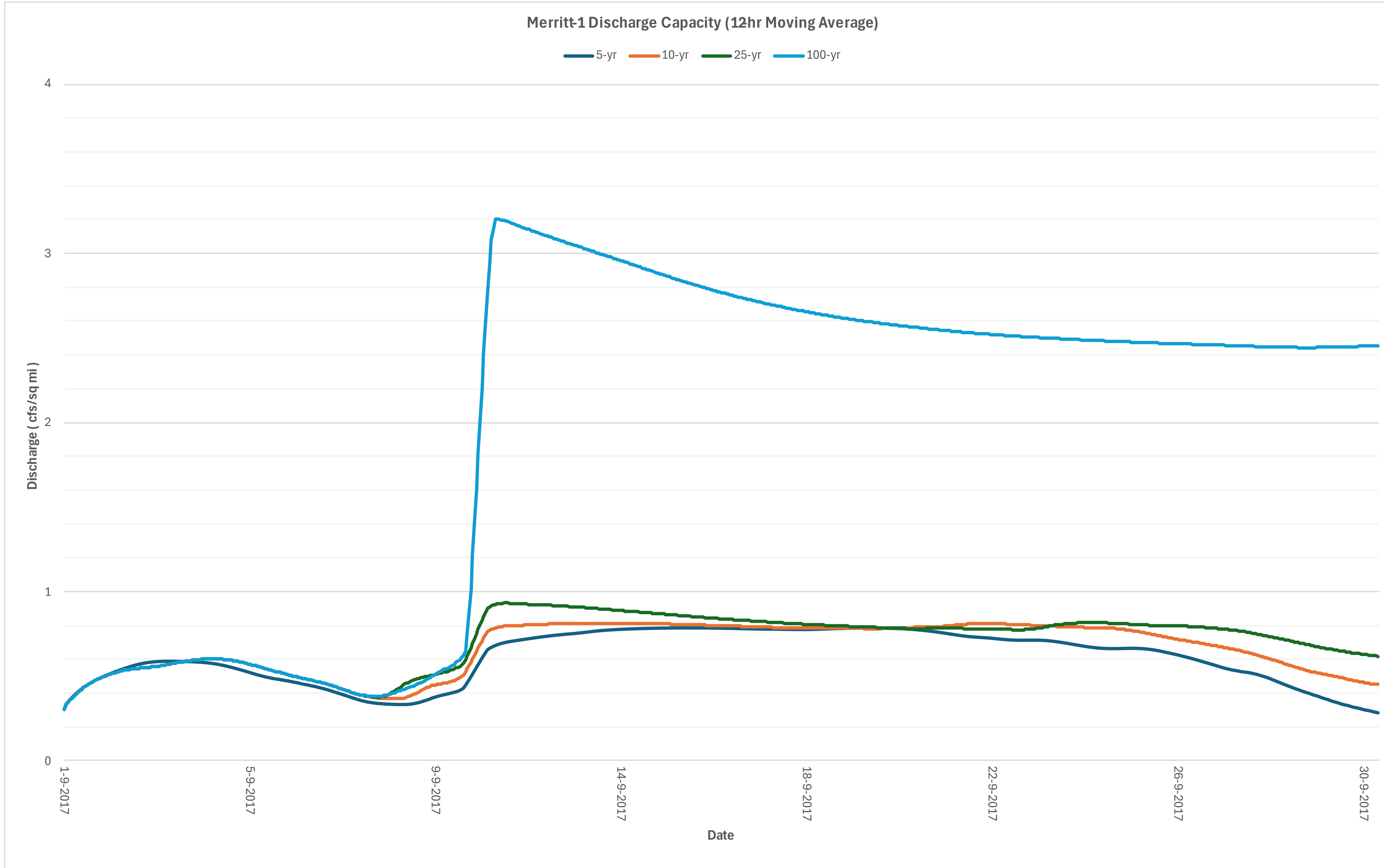

Figure 4-59. Discharge Capacity Hydrograph for Merritt-1

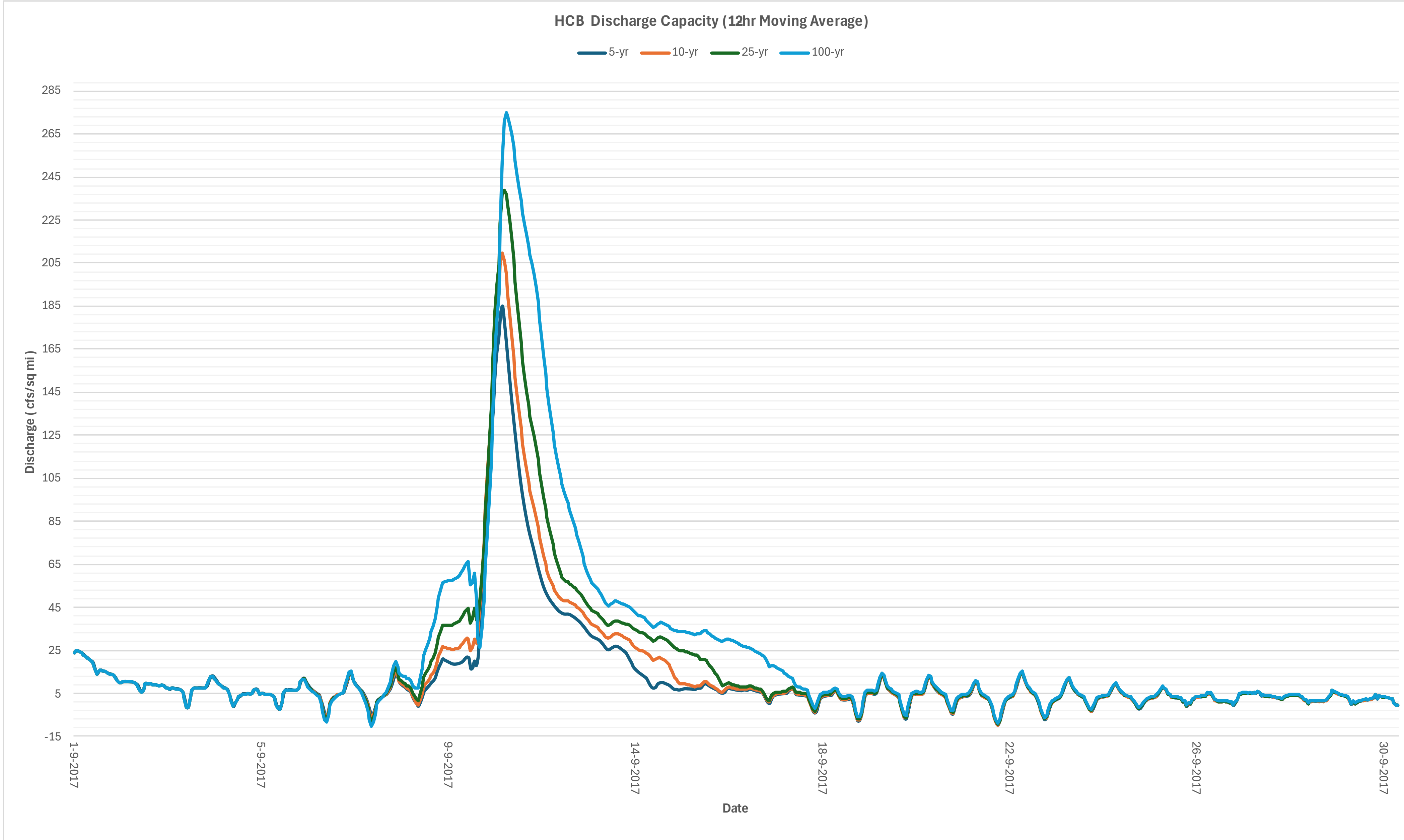

Figure 4-60. Discharge Capacity Hydrograph for Discharge Capacity Hydrograph for HCB

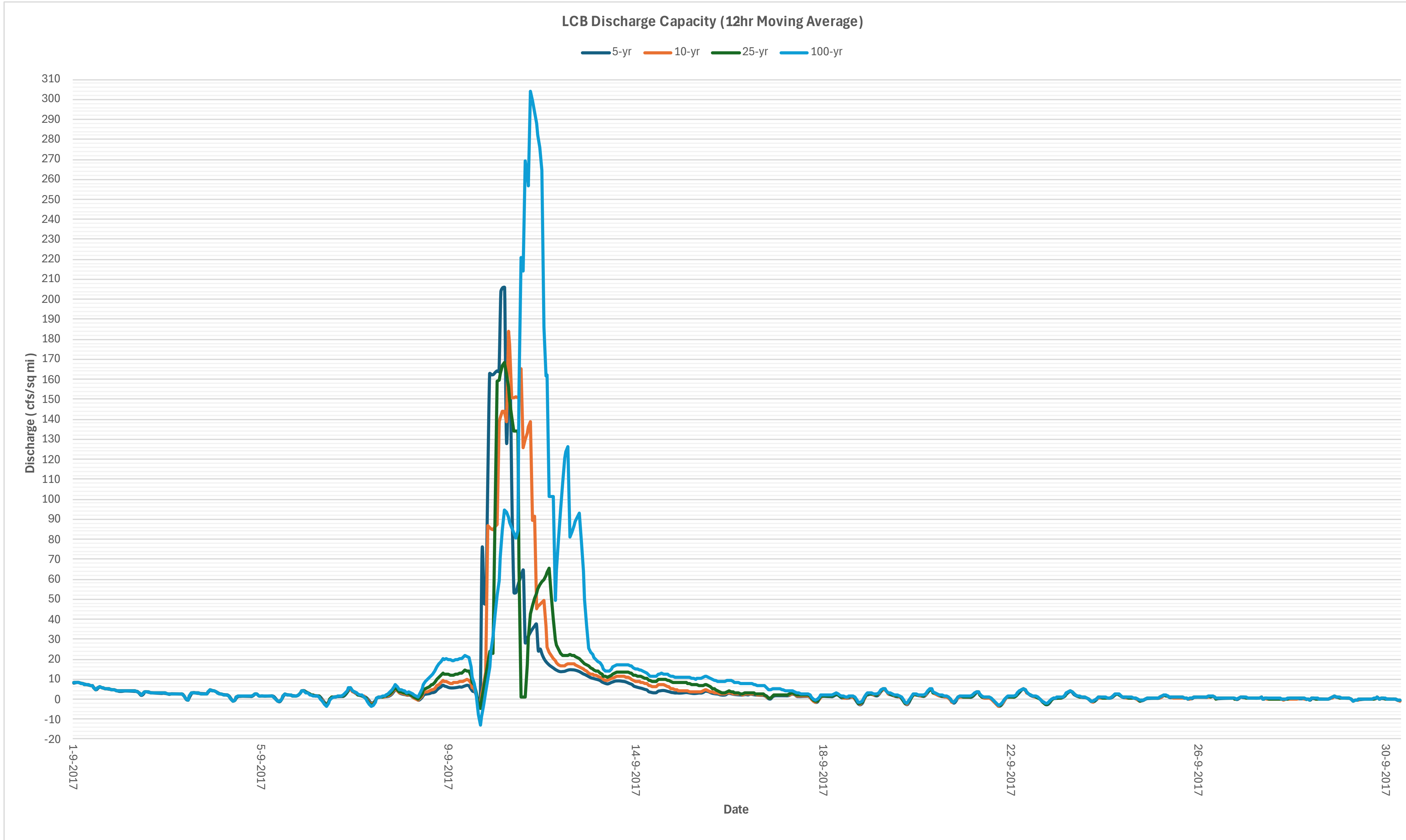

Figure 4-61. Discharge Capacity Hydrograph for LCB

4.3 PM #5

For PM #5, the inundation depths of overland water were evaluated for the design storms. The maps were compared with elevations of infrastructure. The gridded MIKE SHE output for the design storm events were converted to GIS format.

In **Figure 4-62**, the 5-yr design event flood depth map for the BCB shows varying inundation patterns across the domain, with flood depths ranging from 0.25 to over 2.5 feet. The eastern and southern portions of the basin experience the most severe flooding, indicated by dark purple areas with depths exceeding 2.0 feet, particularly around the natural wetland areas. The central and western regions generally show moderate flooding between 0.75 to 1.5 feet (shown in orange to red), while coastal and developed areas in the west maintain lower flood depths of 0.25 to 0.75 feet (shown in light orange). Within the coastal area of north Naples, including Vanderbilt Beach, Vanderbilt Lakes, and Pelican Bay, flooding of more than 2 feet is observed. Notable flooding hotspots appear near hydraulic structures such as culverts, where depths frequently exceed 2.0 feet.

In **Figure 4-63**, the 10-yr design event flood depth map shows similar spatial patterns to the 5-yr event but with notably higher flood depths throughout the model domain. The southeastern and southern portions of the basin continue to experience the most severe flooding, with expanded areas of dark purple indicating depths exceeding 2.0-2.5 feet, particularly in natural wetland areas and around water management structures (such as culverts). The central region shows intensified flooding compared to the 5-yr event, with broader areas of red to brown coloring indicating depths of 1.5-2.0 feet. The western coastal areas, while still showing the lowest relative depths of 0.25-0.75 feet (light orange), exhibit slightly more extensive flooding coverage compared to the 5-yr scenario. In the northwestern coastal area, flood depths increase to over 2 feet, particularly in the Vanderbilt Beach region and portions of Pelican Bay, showing increased flood risk compared to the 5-yr event.

The 25-yr design event flood depth map in **Figure 4-64** shows the more severe flooding conditions than 5- and 10-yr return periods, with significantly expanded areas of deep flooding throughout the basin. The southern, southeastern and central portions show extensive dark purple and brown areas, showing flood depths exceeding 2.0-2.5 feet, with some locations reaching maximum depths near culvert structures. Mostly, these areas are natural wetlands. The central region shows broader areas of 1.5-2.0 feet flooding (shown in brown), while the western developed areas maintain relatively lower but increased flood depths of 0.75-1.5 feet compared to shorter return periods. However, some western coastal areas, particularly in low-lying areas of Vanderbilt Beach and Pelican Bay, experienced increased flooding with depths over 2 feet.

In **Figure 4-65**, the 100-yr design event flood depth map shows the most extreme flooding scenario, with substantially increased flood depths and inundation extent across all regions. The southern and eastern portions show extensive dark purple areas indicating widespread flooding exceeding 2.5 feet, while the central region displays predominantly brown to purple coloring representing depths of 1.5-2.5 feet. Even the typically less-impacted western coastal areas show increased flood depths of 0.75-1.5 feet, with some developed areas experiencing depths up to 2.0 feet. Some of the northwestern coastal areas near Vanderbilt Beach and Pelican Bay experience flood depths up to more than 2.5 feet. This event represents the most severe flooding conditions as expected, with significant implications for both natural and developed areas across the entire basin.

Extensive regions of Cocohatchee, Faka Union, Fakahatchee, Henderson/Belle Meade, as well as portions of the Golden Gate and Trafford sub-basins, remain in their natural and undeveloped state. As previously mentioned in this section, these areas do not have planned stormwater management systems. Consequently, they experience the greatest depths and extents of inundation. In contrast, East Naples, Estero Bay, and parts of the Coastal basins sub-basins are equipped with designed stormwater management systems that effectively drain these areas.

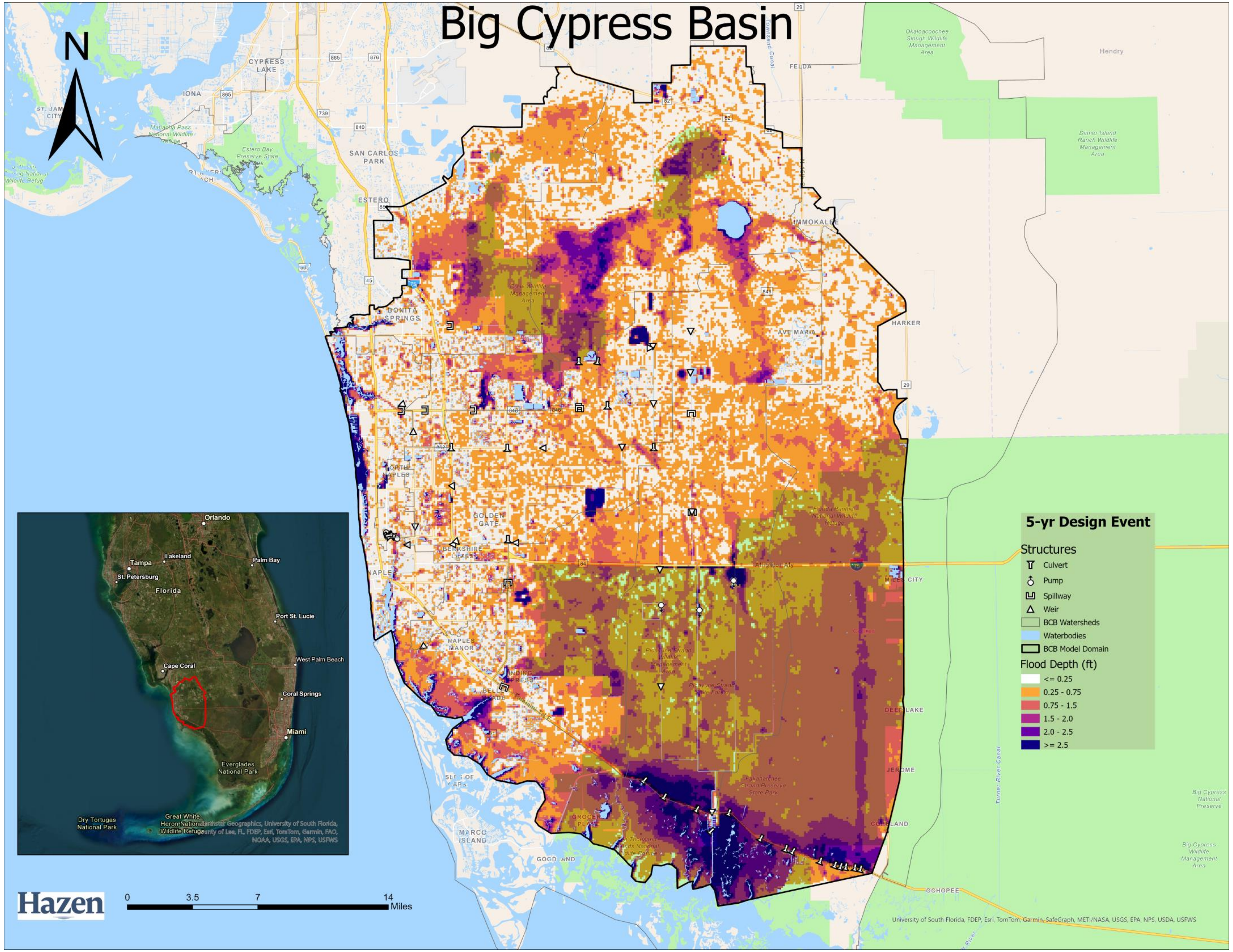

Figure 4-62. Flood Depths Map from 5-yr Design Event Simulation

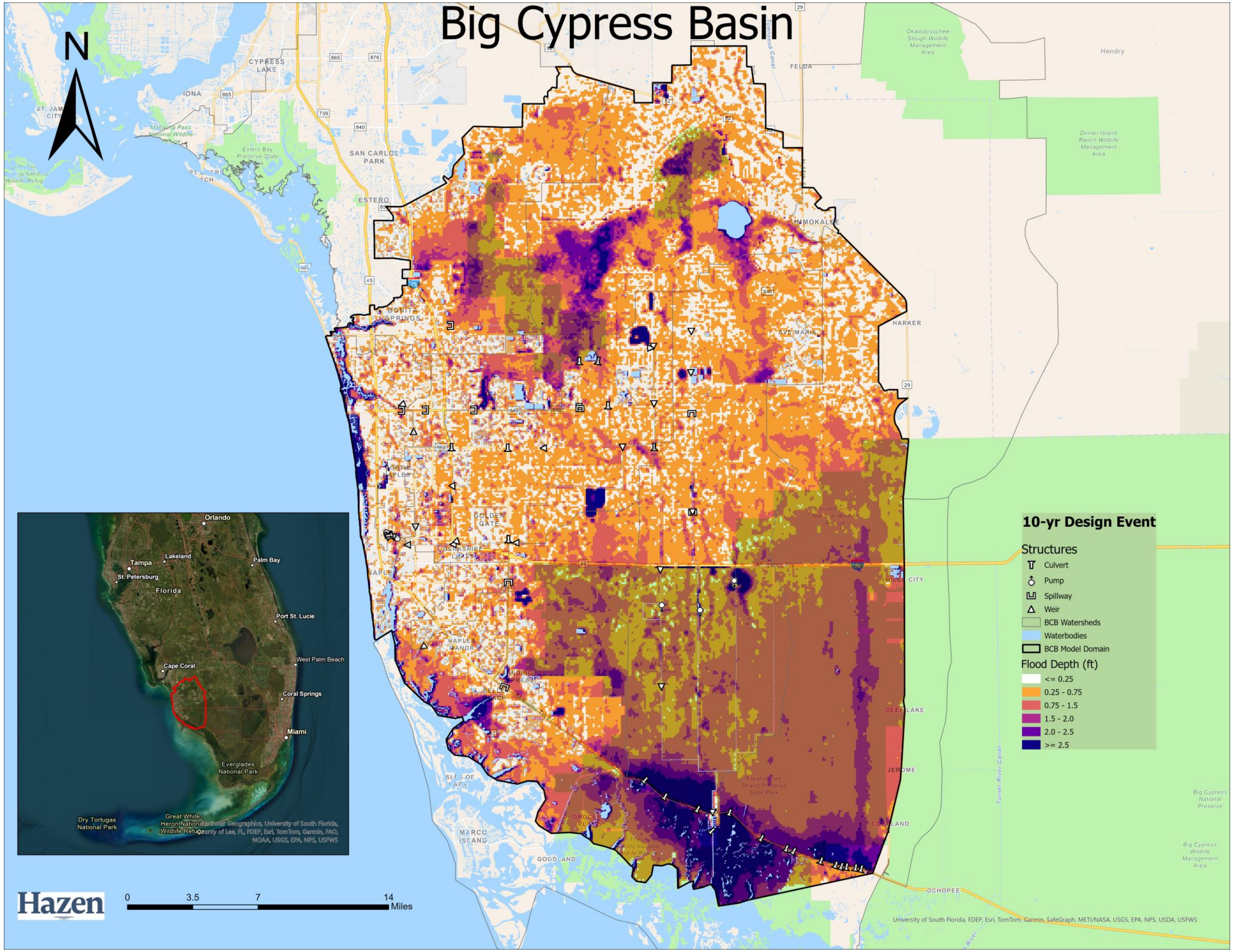

Figure 4-63. Flood Depths Map from 10-yr Design Event Simulation

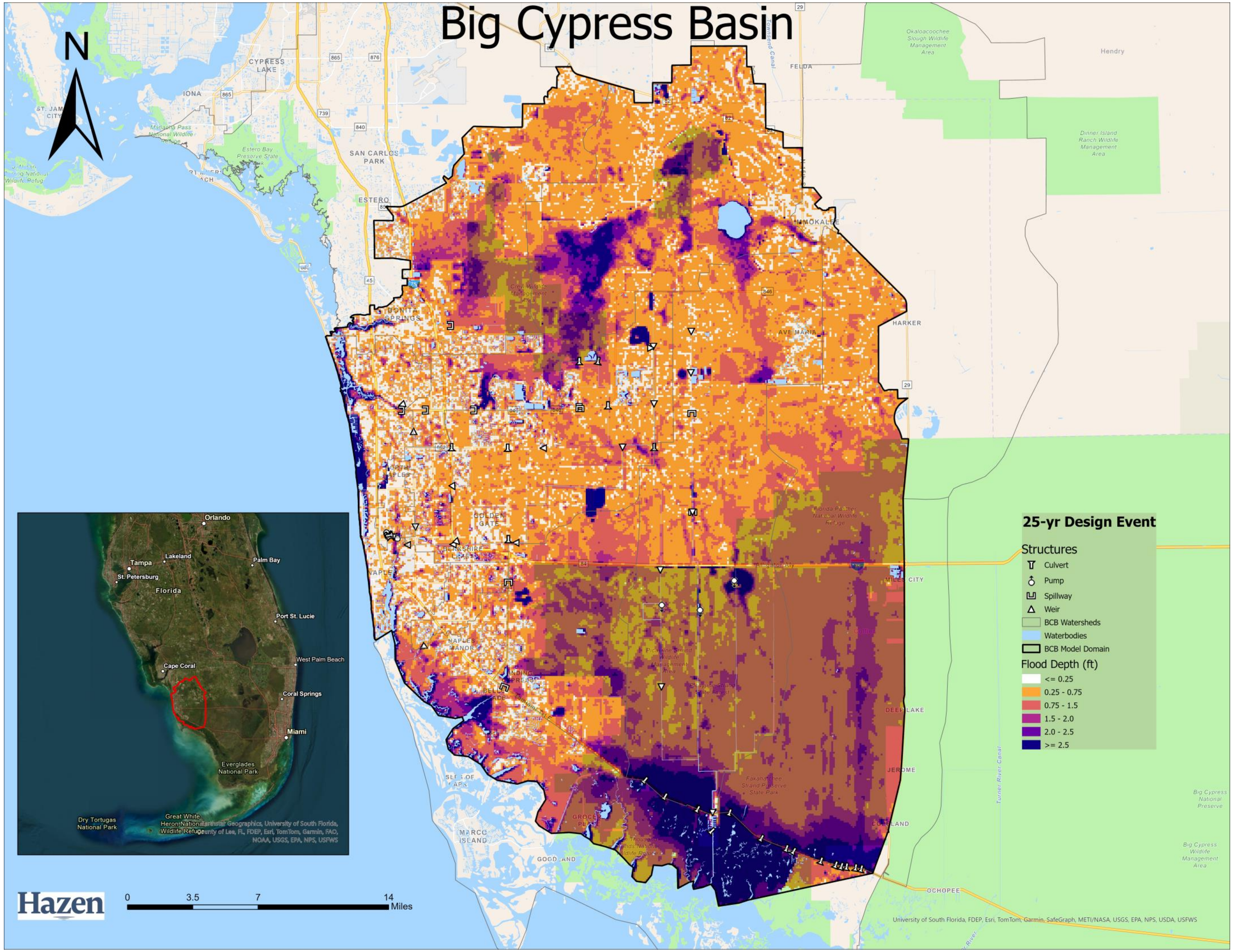

Figure 4-64. Flood Depths Map from 25-yr Design Event Simulation

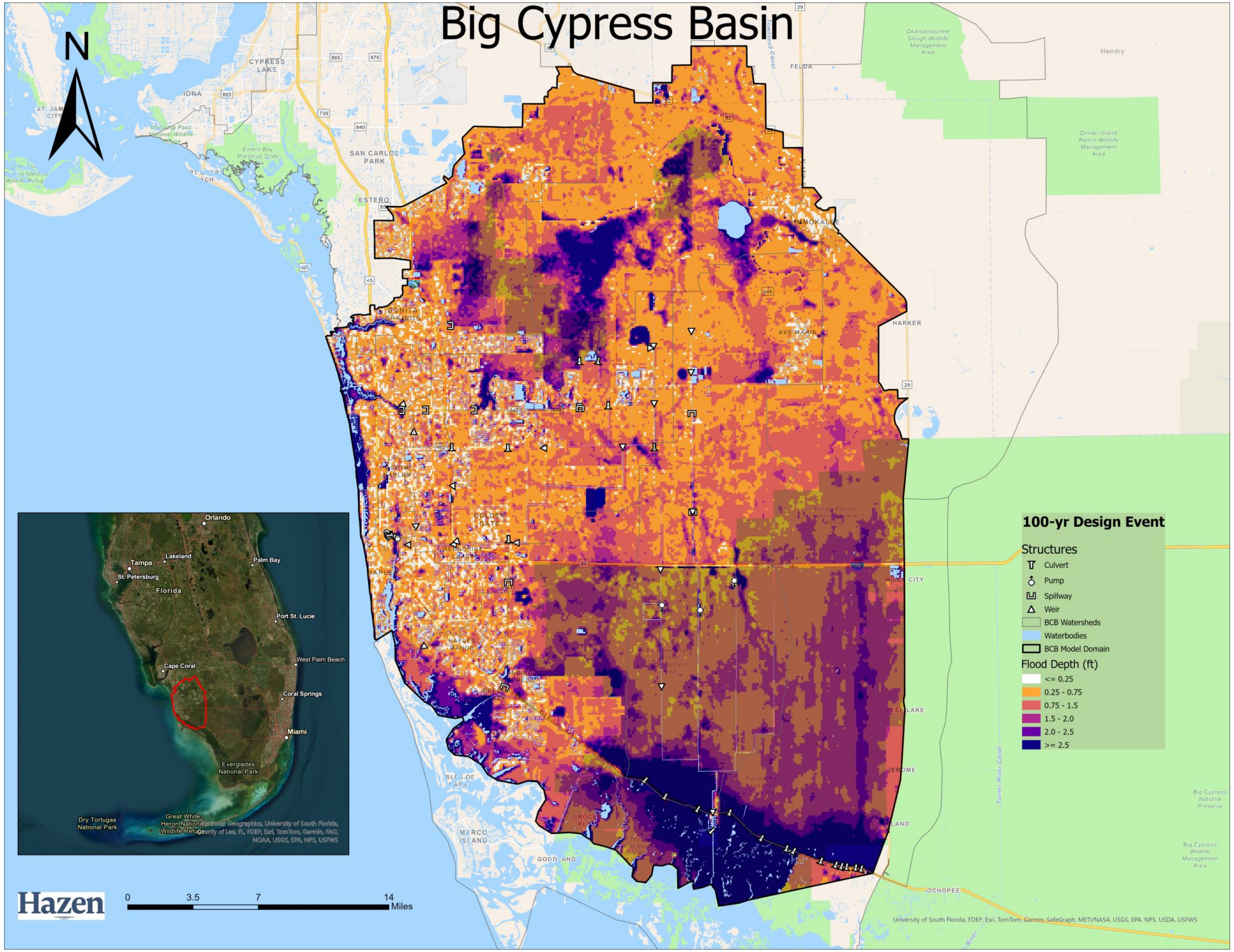

Figure 4-65. Flood Depths Map from 100-yr Design Event Simulation

4.4 PM #6

SFWMD FPLOS PM #6 quantifies the duration of flooding at specific locations of interest within the model domain. For this metric, potentially flood-prone locations associated with existing or future development were identified. The discussion of duration of inundation is done relative to each other. Since flash flooding duration is 6 hours, duration lower than that is considered not severe.

Figure 4-66 shows varying flood durations across the basin during a 5-yr design event. The eastern, southern, and some central portions of the basin, as shown in red, experience the most prolonged flooding (>420 hours), particularly in natural wetland areas and around some water management structures. The western urbanized areas generally show shorter flood durations (<72 hours), indicated by the lighter blue and green shades. Coastal zones and areas adjacent to water control structures exhibit moderate flood durations (72-240 hours), represented by yellow to orange color. The spatial distribution of flood duration appears to correlate with land use patterns, with developed areas showing faster drainage compared to natural and conservation areas where water is retained longer.

In **Figure 4-67**, the 10-yr design event flood duration map shows similar spatial patterns to the 5-yr event but with notably few more extensive and prolonged flooding periods. The eastern portion of the basin shows widespread areas of sustained inundation exceeding 420 hours (shown in red), particularly in natural wetland areas and conservation zones. The western urbanized regions maintain relatively shorter flood durations (<72 hours, shown in blue and light blue shades), though with slightly expanded areas of moderate flooding compared to the 5-yr event. Areas surrounding water control structures and the coastal zone display intermediate flood durations (72-240 hours, shown in yellow and orange).

Flood duration for the 25-yr design event in **Figure 4-68** shows the more severe flooding patterns compared to the 5- and 10-yr return periods, with significantly expanded areas of prolonged inundation. The eastern, southern and central portions of the basin are dominated by extensive areas of sustained flooding exceeding 420 hours (shown in red), encompassing both natural wetlands and some previously less-impacted areas. The western urbanized sections, while still showing relatively better drainage characteristics, exhibit increased areas of moderate flooding (72-240 hours) compared to the 5- and 10-yr events. The coastal regions and areas around water management structures show extended flood durations (240-420 hours, orange to red), indicating the reduced effectiveness of drainage systems during this more extreme event.

Figure 4-69 captures the flood duration for 100-yr design event; it shows the most extreme flooding scenario for the BCB, showing substantially increased inundation periods across all regions. The eastern and central portions display extensive areas of prolonged flooding exceeding 420 hours (dark red), with the red zones notably expanding into previously less affected areas. The western urbanized areas, while still maintaining better drainage characteristics, show increased areas of moderate to long duration flooding (72-420 hours) compared to shorter return periods, indicating potential drainage system overwhelm during this extreme event. Some of the coastal zones and areas surrounding water control structures show longer flood durations (240-420+ hours).

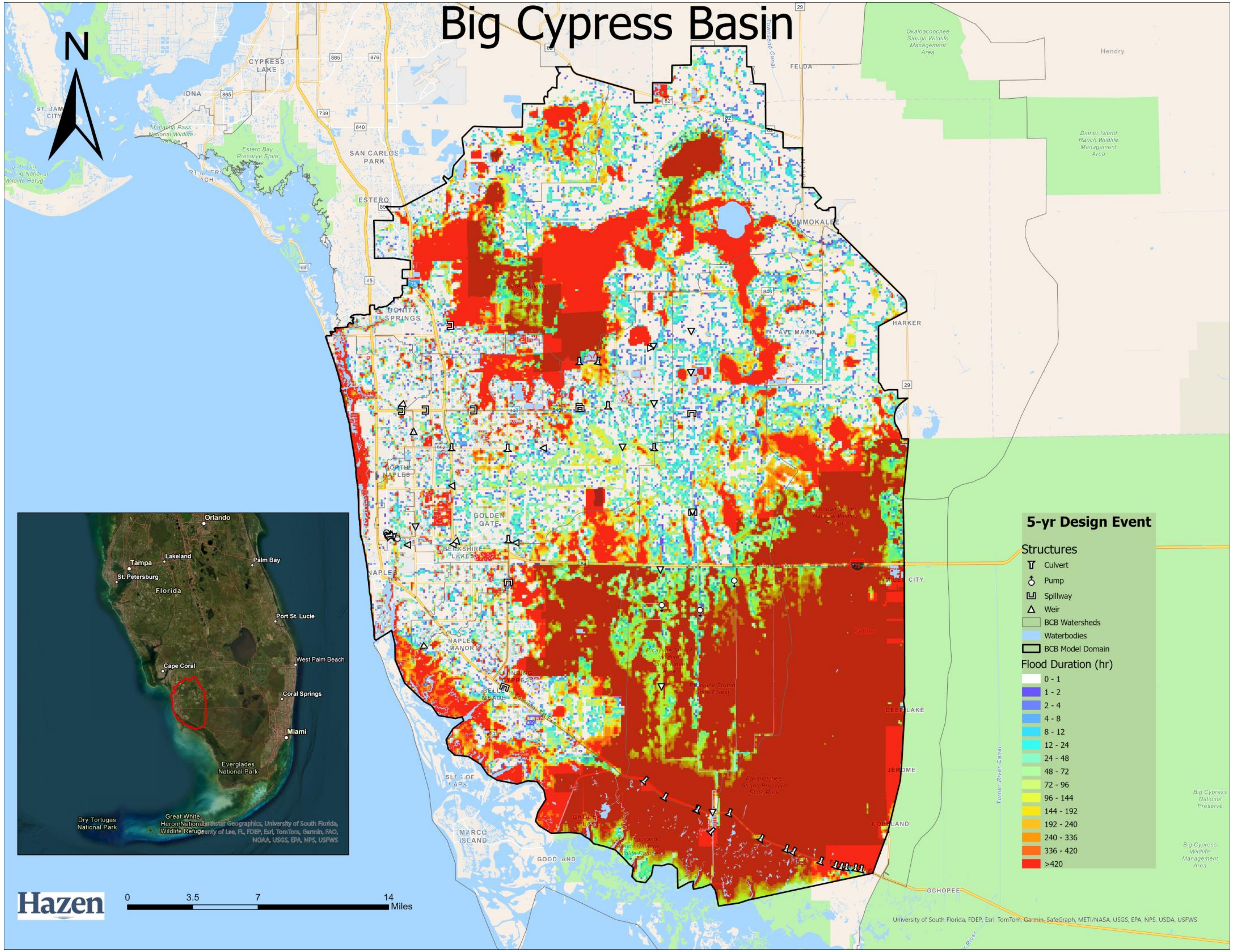

Figure 4-66. Flood Duration Map from 5-yr Design Event Simulation

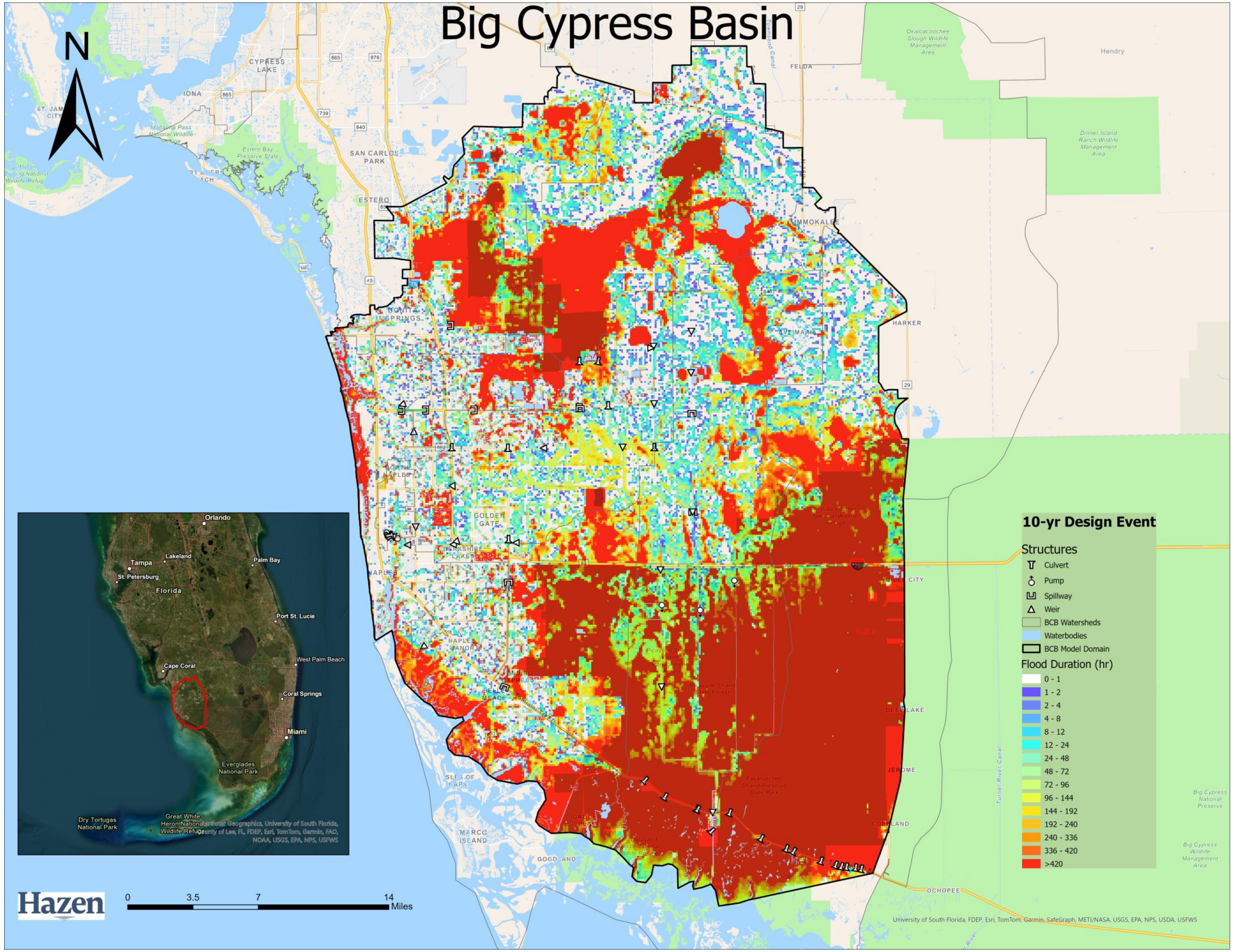

Figure 4-67. Flood Duration Map from 10-yr Design Event Simulation

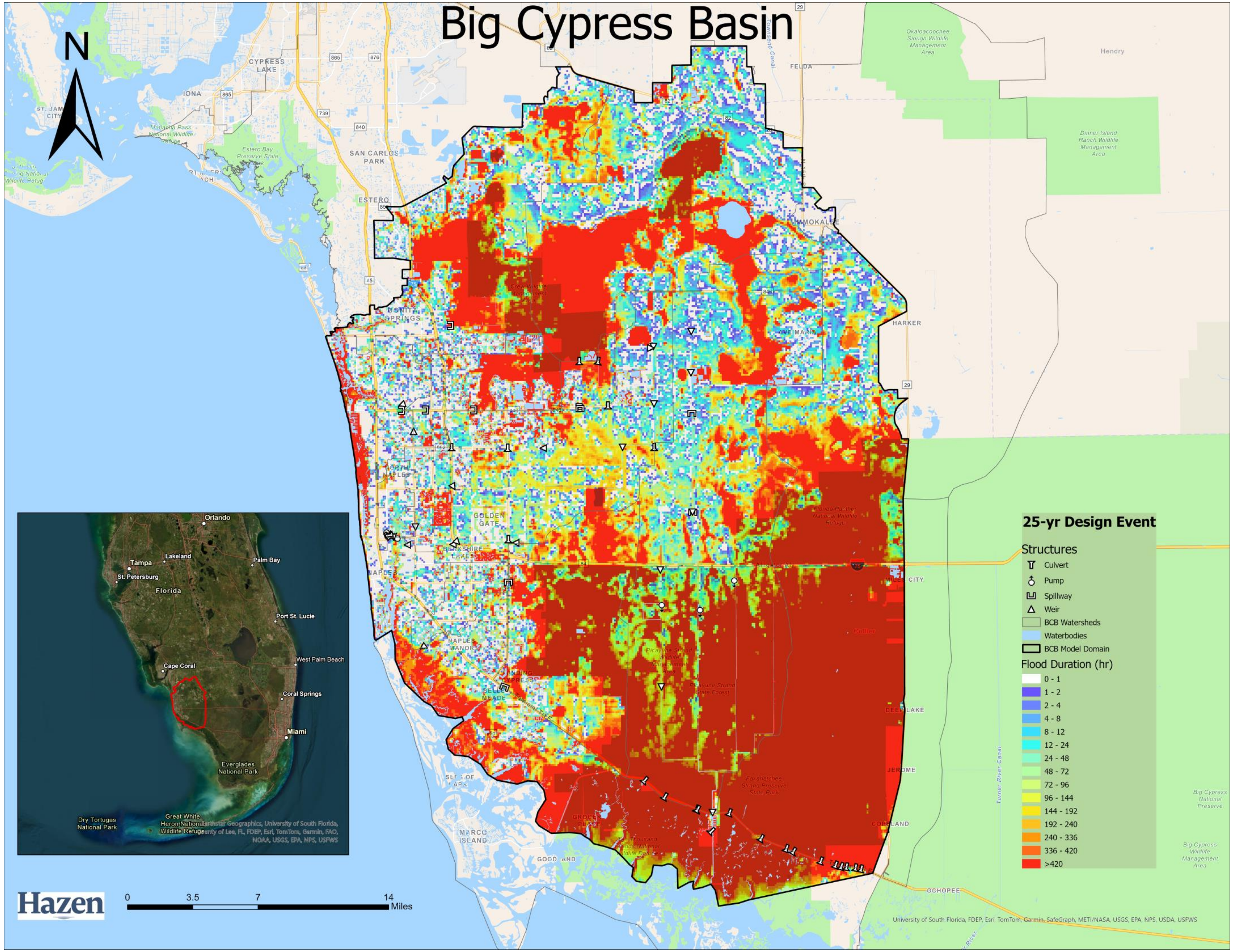

Figure 4-68. Flood Duration Map from 25-yr Design Event Simulation

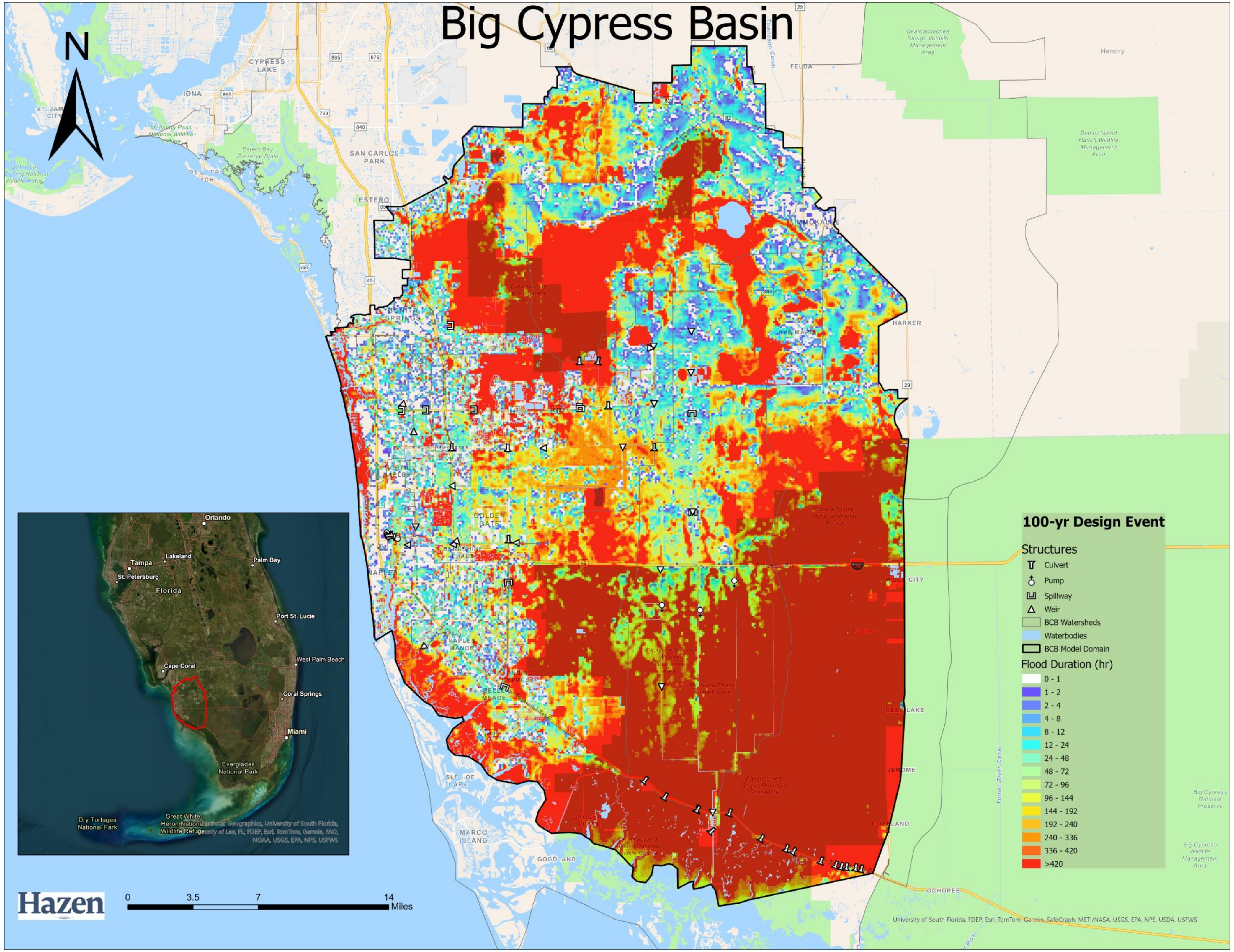

Figure 4-69. Flood Duration Map from 100-yr Design Event Simulation

5. Summary of BCB Design Events Simulations

The BCB design event models were derived from the BCB Irma Model, incorporating adjustments to rainfall parameters and structural operations. The model's domain encompasses all essential contributing areas and infrastructure, with parametrization based on BCB Irma Model results deemed sufficiently accurate for use in design events.

The evaluation of the hydrologic and hydraulic model for BCB, specifically concerning design events, constituted a thorough process that examined the model's performance and its appropriateness for regional-scale interpretations. The District relies on six (6) formal performance metrics (PMs) to evaluate the FPLOS provided by the primary water management infrastructure. These metrics are:

- PM #1 Maximum Stage in Primary Canals.
- PM #2 Maximum Daily Discharge Capacity through the Primary Canals.
- PM #3 – Structure Performance assessment due to sea level rise.
- Peak Storm Runoff evaluation due to sea level rise.
- PM #5 Frequency of Flooding.
- PM #6 Duration of Flooding.

This project included an evaluation of PM #1, 2, 5 and 6.

Overall, the evaluation aimed to ensure that the BCB design event H&H model could reliably simulate the impacts of extreme weather events and could be used for future planning, management, and mitigation strategies in the BCB region. The summary below highlights major structures and areas within the BCB Model domain, based on PM #1, 2, 5 and 6.

- Estero Bay sub-basin: This sub-basin that includes the Imperial River has water levels of all events exceeding the canal banks at most locations. So this sub-basin was assigned a 5-yr FPLOS rating.
- Cocohatchee sub-basin: The main canal overflows the banks at least in portions for all design events. However, the canals and waterways that flow into the Cocohatchee Canal offer at least a 10-yr FPLOS. Considering the good performance of Airport Rd Canal and poor performance of Palm River, this sub-basin is assigned a FPLOS rating of 10-yr.
- Golden Gate and Trafford sub-basins: Although the water levels in the main Golden Cate Canal exceed the banks in some locations even for the 5-yr event, the water levels in the tributaries flowing into it stay within banks for almost all events. I75 canal, Green canal, Harvey canal and CR951 Canal offer at least a 10-yr FPLOS or higher, but Cypress Canal does not perform well. Therefore, this sub-basin was assigned an overall FPLOS rating of 10-yr.
- Henderson Belle-Meade sub-basin: Henderson Creek Canal partially meets FPLOS for all return periods (5-, 10-, 25-, and 100-yr). Water levels generally remain below both or one of

the banks throughout most of the reach except along Tamiami Trl. As a sub-basin, it meets a 25-yr FPLOS.

- Faka Union sub-basin: Simulated water levels exceed the banks near FU-3 and FU-4 for all events, but the water levels stay within banks in the rest of this sub-basin. FPLOS offered along Miller Canal based on PM #1 is 5-yr, due to exceedance near the bridges.

Please note that due to variations in computational resources and environments across different machines, minor differences in model results are expected when the model is run. These small discrepancies are expected and generally do not impact the overall validity of the findings.

6. Recommendations for future

Understanding the gradients and interactions within the surface water system and groundwater is crucial for effective water management, flood forecasting, and infrastructure planning in the BCB. Accurate modeling can inform decision-makers about potential flood risks, necessary infrastructure improvements, and strategies to manage water resources during extreme weather events.

The calibrated BIB Irma model was used as a basis for design events simulations. The BCB Irma Model's ability to capture water surface gradients and interactions across the surface water system suggested that it is a tool that can be reliably used for understanding hydrodynamics in the context of storm events. This model was calibrated to one of the most severe and complex events to affect the BCB system. The model was calibrated to surface water stages and flows, groundwater levels and to the observed high-water marks. So the results of the design event simulation at a regional scale, presented in this model, are considered acceptable. However, as with any other modeling tool, continued refinement and validation of the model will further enhance its application for flood management and environmental planning in the region.

7. References

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iv. Lago Consulting and Services. *Time Statistics of dfs0/dfs2/dfs3 files*. https://www.lago-consulting.com/time-statistics-of-dfs2-dfs3-files.html

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vi. South Florida Water Management District Regulation Division, 2014. *Environmental Resource permit Information Manual*. 2014.

vii. South Florida Water Management District, Hydrology and Hydraulics Bureau. 2017. *Flood Protection Level of Service (LOS) Analysis for the Big Cypress Basin. Appendix C: Preparation of Boundary Conditions at the Tidal Structures*. Dated June 21, 2017.

viii. South Florida Water Management District Hydrology and Hydraulics Bureau and Big Cypress Basin Service Center. May 2023. Water Control Operations Atlas: Big Cypress Basin System Part 2: Descriptions.