Introduction

This technical memorandum outlines the development of a 2-Dimensional hydraulic model constructed to simulate a range of fluvial and coastal event conditions in the Hwy 101 corridor to support the Comprehensive Climate Adaptation and Implementation Plan (CAIP) for the Eureka-Arcata Corridor. The goal of this phase of the modeling exercise was to simulate existing conditions to characterize in-channel and floodplain hydraulics within the project area. The results will help guide the formulation of adaptation strategies and establish a baseline for comparison for forthcoming vulnerability assessments and future conditions scenarios.

The following sections provide a detailed overview of the hydrologic analyses used to define boundary conditions and develop the hydraulic model, including data inputs, assumptions, and methodologies. Additionally, salient model results are presented, highlighting key in-channel, floodplain and infrastructure hydraulic characteristics. All values provided in this memo are in English units and vertical elevations are referenced to the North American Vertical Datum of 1988 (NAVD88), unless otherwise noted. All geospatial data associated with the 2D hydraulic model were projected in the NAD 1983 NSRS 2007 State Plane California I FIPS 0401 (US Feet) following coordinate.

Methods

The key steps of the modeling exercise included: i) creating high-resolution terrain surfaces of the existing channel, drainage network and floodplains from a combination of LiDAR and field survey data, ii) 2D modeling of existing terrain and hydraulic structures, and iii) analysis of key existing ground (EG) hydraulic results (e.g., inundation extent & duration, flow velocity, depth, and water surface elevations) over a suite of design boundary conditions to quantify existing hydraulic conditions the Eureka-Arcata Corridor.

Project Area, Modeling Domain and Extents

The CAIP Project Area, hereafter referred to as the "Project Area", is outlined in the black dashed line in Figure 1. The model domain, which encompasses the Project Area, extends roughly 3,000 ft north of the 14th Bridge in Arcata to West Grant Road in Eureka. The western and eastern modeling extents roughly follow the high ground centerline of the Samoa Peninsula and Myrtle Ave/Old Arcata Road, respectively. The model also extends substantially south and eastward to include large portions of Ryan Creek, Freshwater Creek and Jacoby Creek (Figure 1). Although not within the CAIP project boundary, areas west of Hwy 101 were included in the model domain to facilitate integration of related forthcoming projects and future modelling efforts.

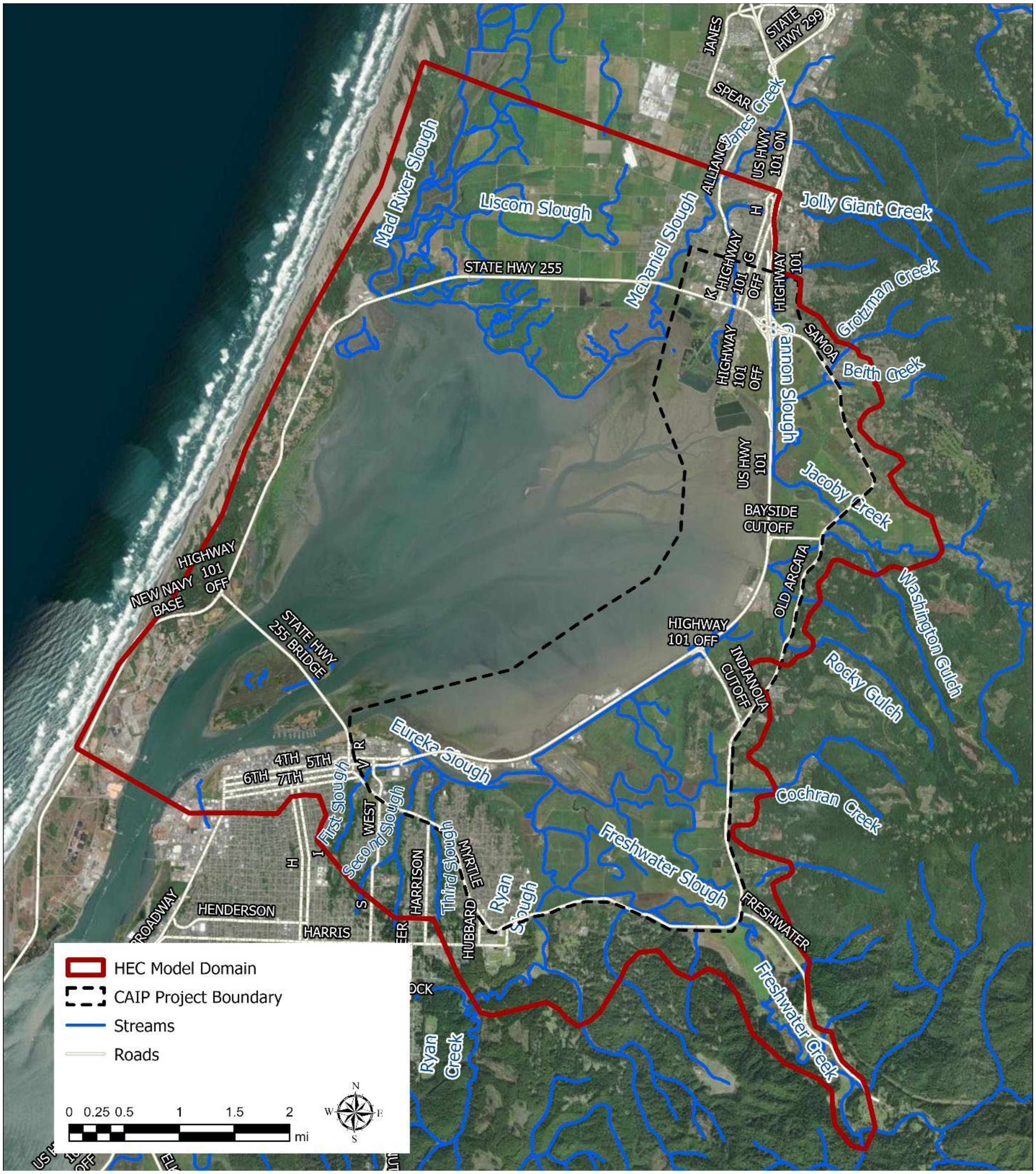


Figure 1. Model domain illustrating boundary condition locations and major streams and road infrastructure in the project area.

Topographic Data & Terrain Surfaces

A high-resolution digital elevation model (DEM) for the existing conditions was constructed using a combination of Light Detection and Ranging (LiDAR) and field survey data. More specifically, the base terrain surface was created using a USGS CoNED Topobathy 3.28 ft (1m) LiDAR DEM compiled in 2020 (OCM Partners, 2024). Additional field-surveyed terrain data were mosaicked on top of the base terrain surface from the following earlier projects:

- Hydraulic Modeling to Support the Sea Level Rise Adaptation Plan for Humboldt Bay Transportation Infrastructure (Phase 1) Project, Humboldt County, CA (NHE, 2021):
    - Topobathymetric surfaces of Eureka, Fay and Freshwater Sloughs, which were based on bathymetric surveys of 30 cross-sections in Eureka, Freshwater and Fay Sloughs to support Humboldt County's Jacob Avenue Levee Study (GMA, 2015).
    - Levee crest survey points of the Jacobs Avenue Levee, which were based on a topographic survey conducted by Humboldt County (County, 2016).
- City of Arcata - Sea Level Rise Vulnerability Assessment and Capital Improvement Project (CIP) Adaptation Plan (GHD, 2024):
    - Topobathymetric surface of the South Jacoby Creek Floodplain Restoration Project
    - Topobathymetric surface of the Jacoby Creek channel
- Cochran Creek Fish Passage and Channel Restoration Project (NHE 2018):
    - Topobathymetric surface of the Cochran Creek restoration designs, including channel bathymetry, culvert and tide gate surveys and floodplain and levee recontouring.
- Humboldt Bay Trail South and North (GHD, 2016 & 2022):
    - Design elevations and alignments of Humboldt Bay Trail North and South
    - Design elevations and alignments of the North Pacific Railroad line between Eureka and Arcata.

Because the base LiDAR DEM was hydro-flattened, we enforced mapped drainages and slough channels by "burning" or lowering DEM cells to reflect realistic channel dimensions and ensure water is routed correctly in the model. We estimated slough channel depth from channel top width using a hydraulic geometry relationship developed from dry channels in the LiDAR DEM (Figure 2).

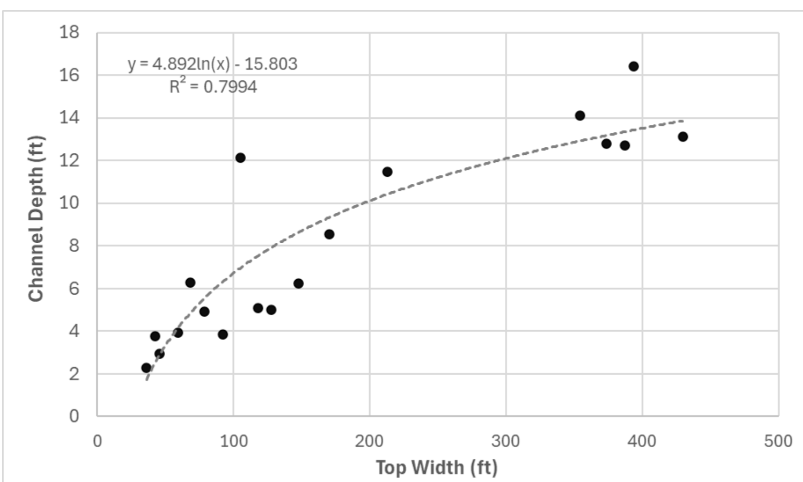


Figure 2. Hydraulic geometry relationship between channel top width and channel depth developed for slough channels within the project area.

Due to the fact that LiDAR DEM's are prone to false ground shots in areas of dense vegetation, we burned through erroneous high ground portions of channels and ditches throughout the project area to ensure proper connectivity and routing. Where necessary, ditch elevations proximal to culverts and tide gates were lowered to match the invert elevations of the associated hydraulic structure.

Hydrologic Analyses

The hydrologic analyses described below were conducted in order to establish upstream and tidal boundary conditions to support 2D model construction. The hydrologic computations included: (1) a determination of appropriate peak flows and baseflows for contributing tributaries; (2) development of a generalized hydrograph to create unsteady freshwater boundary conditions, and (3) generation of downstream boundary conditions from representative tidal series containing target extreme high-water level events and spring tide level.

Peak Flow Analysis

Table 1 shows peak flow estimates determined for the 2-yr, 10-yr and 100-yr events at 19 locations within the model domain. Peak flows were estimated using the USGS StreamStats application based on regional regression equations (Gotvald, 2012). Following FEMA (2016) guidelines, peak flows from all secondary tributaries listed in Table 1 were estimated via a flow-balancing procedure using regional regression results above and below the confluence with the mainstem. Cochran, Redmond and Myrtle Avenue Creek peak flows were determined by scaling the Fay Slough peak-flows by watershed area ratios. Winter base flow estimates for each tributary were scaled by drainage area ratio from the Little River near Trinidad station (USGS 11481200) winter base flow estimate (243 cfs; NHE, 2021).

Table 1. Summary of baseflow and flood-frequency estimates for all fluvial boundary conditions in the CAIP project area.

Hydrologic Group	Upstream Boundary Condition	Basin Area (mi^2)	Baseflow (cfs)	Peak-flow estimates (cfs) by return 2-yr 50%	10-yr 10%	100-yr 1%
Eureka Slough Tribs	Freshwater Creek	31.5	0	2,190	5,140	9,110
	Wood Creek	0.6	0	30	70	130
	Spears Rd Creek	1	0	50	120	210
	Ryan Slough	15.3	0	750	1,820	3,250
	Myrtle Ave Creek	0.7	0	51	128	241
	Redmond Creek	0.7	0	51	128	241
	Cochran Creek	1.5	0	109	274	517
	Third Slough	2.2	0	100	230	400
	Second Slough	1	0	40	110	200
Rocky Gulch Tribs	Rocky Gulch	1.8	0	139	367	694
	Washington Gulch	1.03	0	70	179	333
Jacoby Creek	Jacoby Creek	16.9	0	1,330	3,100	5,500
Gannon Slough Tribs	North Jacoby Creek	1.1	0	66	170	170
	Beith Creek	1.4	0	96	243	446
	Grotzman Creek	1.1	0	78	204	383
	Campbell Creek	0.8	0	63	172	331
Jolly Giant	Jolly Giant	0.8	0	66	179	342
Janes Creek	Janes Creek	2.1	0	158	416	787
Mad River Slough	Mad River Slough	8.3	0	501	1,310	2,460

Table 2 provides an example of the differencing procedure for Eureka Slough and its tributary channels.

Table 2. Summary of flood-frequency estimates for Eureka Slough and its tributaries. Values for the Eureka Slough tributaries listed in Table 1 were derived from differencing the values listed below in a downstream direction.

Tributary and Location	Basin Area (mi^2)	Flood-Frequency Estimate (cfs) by return interval and exceedance probability 2-yr 50%	10-yr 10%	100-yr 1%
Freshwater Slough downstream Myrtle Ave	31.5	2,190	5,140	9,110
Freshwater Slough downstream Wood Creek	32.1	2,220	5,210	9,240
Freshwater Slough downstream Spears Rd Creek	33.1	2,270	5,330	9,450
Freshwater Slough downstream Ryan Slough	48.4	3,020	7,150	12,700
Freshwater Slough downstream Fay Slough	53.2	3,230	7,680	13,700
Eureka Slough downstream Harrison Ave Creek	55.4	3,330	7,910	14,100
Eureka Slough upstream Eureka Slough Bridge	56.4	3,370	8,020	14,300

Generalized Hydrograph

Unsteady flow conditions at all fluvial boundaries were simulated using a generalized hydrograph, which was scaled from the January 13, 2024, event hydrograph at Little River. This event represented the highest recorded flow since 2000 and exhibited a hydrograph shape consistent with other isolated events at this location, as well as with the event hydrograph for

Jacoby Creek on the same date (Figure 3). Little River, the nearest USGS-gaged river to the project watersheds, shows a similar hydrograph shape and timing to volunteer-estimated streamflows at Jacoby Creek during the event. The generalized hydrograph was then scaled to match the peak flows shown in Table 1 and each hydrograph started and ended at winter baseflow.

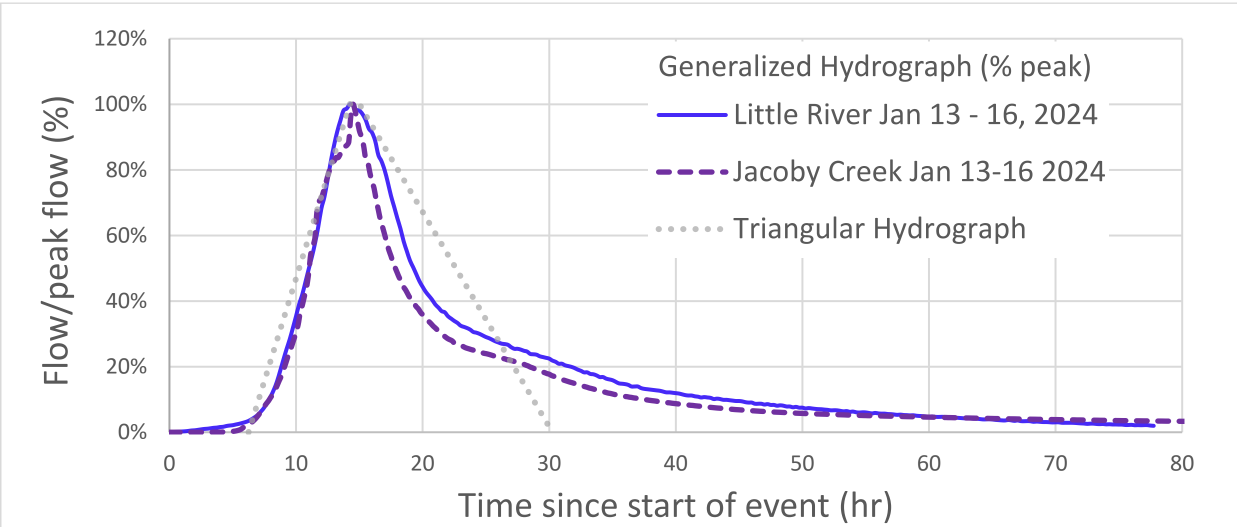


Figure 3. Generalized hydrograph used as a boundary condition for fluvial flows in the HEC-RAS model, based on the January 13, 2024 storm event at Little River (solid blue line). The Jacoby Creek hydrograph is represented by the dashed purple line, while a reference 24-hour triangular hydrograph is shown as the dotted red line.

Figure 3 shows a 24-hour triangular hydrograph for reference, as this approach is commonly used to represent the temporal distribution of streamflow. The total volumetric flow estimated by the triangular hydrograph closely matched the 3-day flow volume from the January 13 event at Jacoby Creek, with less than a 1% difference. However, it underestimated the 3-day flow volume at Little River by 15%. Given the potential focus on assessing longer-term drainage patterns, the measured hydrograph from Little River was used for a more accurate evaluation of potential drainage impacts during high flow events. Little River, with a watershed area of 40 square miles, has a larger contributing area than any other tributary in the study area, except for Freshwater and Eureka Sloughs.

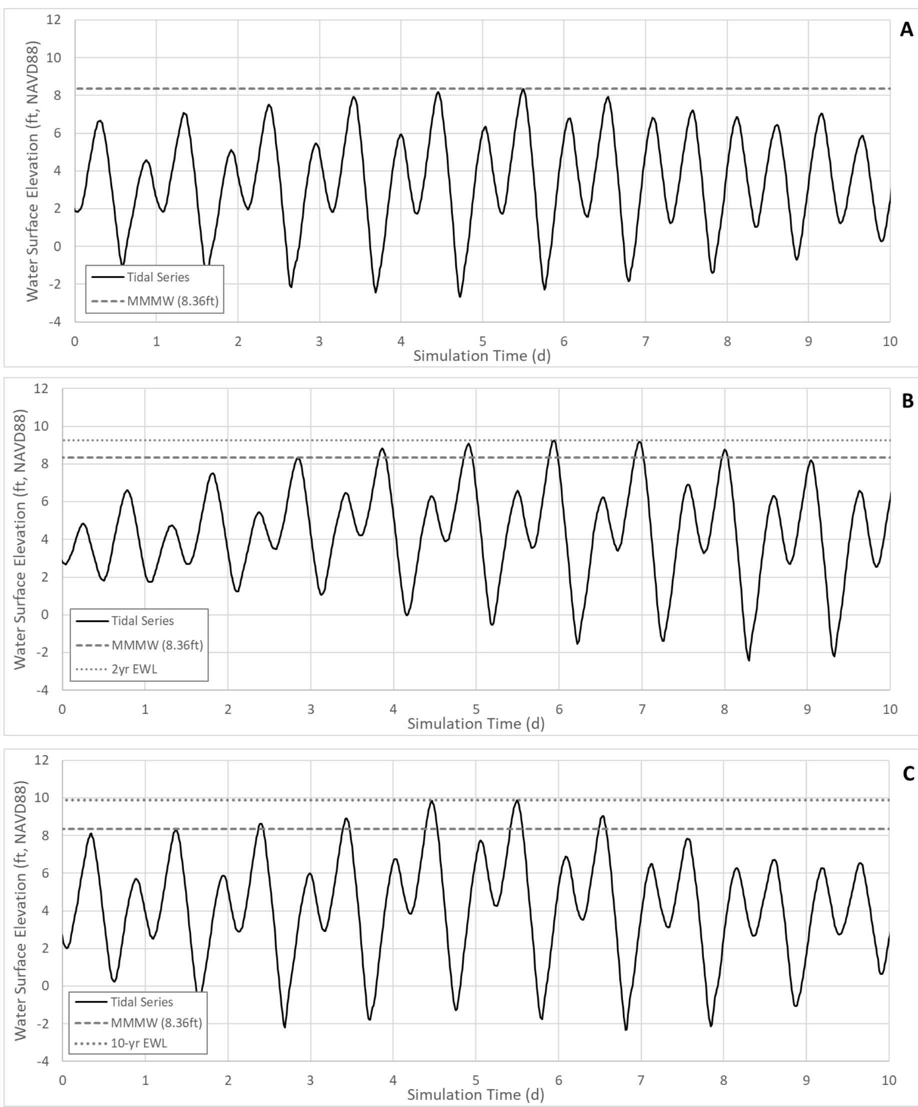
Coastal Extreme Water-Level Event and Mean Monthly Maximum Water Estimates

The downstream Boundary Conditions (BCs) for the 2D-Model consisted of representative tidal series containing the target extreme high-water level events and spring tide level. Spring tide was represented as the mean monthly maximum water (MMMW) tide level. All extreme high-water levels and representative tidal series were derived from the Humboldt Bay sea-level rise 2D model (NHE, 2015). We used the predicted 100-yr 15-min water levels extracted at the grid cell closest to the downstream end of 2D-Model (grid L = 707). Table 3 lists the estimated tidal datums and extreme high-water level probabilities for existing conditions (Year 2023) at the downstream BC. A declustering approach, detailed in NHE (2021), was used to generate representative tidal time-series data of extreme high-water level events and spring tide level.

Table 3. Summary of tidal levels and annual extreme high-water level probability estimates extracted from the NHE (2015) 2D model results (grid L = 707) for Year 2023 at the 2D CAIP model downstream BC.

Tidal Value or Percent Probability of Exceedance	Return Interval (yr)	Predicted Water Levels for Year 2023 (ft)
Mean monthly maximum water (MMMW)	--	8.36
50	2	9.26
10	10	9.89
1	100	10.88

Figure 4 illustrates the representative tidal series for the MMMW, 2-, 10- and 100-yr events used for the 2D-Model downstream BCs. The predicted tidal series in Figure 4D contains both the 100-yr and 500-yr extreme high-water level events (500-yr event, provided for reference, is the highest peak water surface elevation near the sixth day of simulation in Figure 4D).



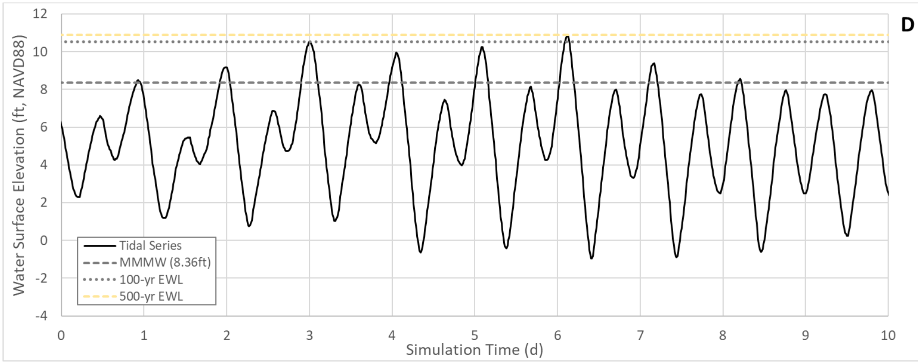


Figure 4. Representative tidal series for the 1D-Model downstream boundary condition for spring tide (mean monthly maximum water (MMMW)) (A) and a 2-yr (B), 10-yr (C) and 100-yr and 500-yr coastal storm (D).

Joint Probability Analysis

The Project Area is prone to flooding from coastal surges and riverine floods, which can occur separately or together, causing combined flood impacts. In the U.S. Pacific Coast region, including the Project Area, the storm systems causing coastal surges differ from those causing riverine floods, and are typically assumed to be independent (FEMA, 2005).

To confirm this assumption, NHE (2021) analyzed annual peak flows of the Eel River and Little River against the maximum daily tide at Crescent City. The data, including peak discharges from the Eel River at Scotia (USGS 11477000) and the Little River near Trinidad (USGS 11481200), and the coinciding tide levels from the Crescent City tide gauge (NOAA 94119750), were compared to flood probabilities and extreme high-water event probabilities.

The analysis indicated no simultaneous coastal and riverine events exceeding 10-year probabilities, suggesting that large coastal and fluvial events are independent. This independence simplifies the calculation of combined flood event probabilities in the Project Area, allowing future compound frequencies to be estimated by multiplying the probabilities of individual coastal and riverine events. NHE's compound frequency analysis also found that coastal water levels were typically between MHHW and the 2-year extreme event during most annual peak flows. This suggests that using mean monthly maximum water levels as a boundary condition for riverine floods is a valid approach, and that the 2-year level provides a reasonable conservative estimate.

Hydraulic Analyses

This section outlines the hydraulic analyses conducted for existing conditions over a range of event conditions in the CAIP project area. All hydraulic analyses were conducted via the U.S. Army Corps of Engineers' (COE) HEC-RAS River Analysis System Version 6.5 (COE, 2021), which solves the 2D (depth-averaged) Saint Venant shallow water equations. Reference can be made to the HEC-RAS manual (COE, 2016) for information specific to 2-dimensional hydraulic modeling.

The 2D solution algorithm requires the following: i) 2D computational mesh, ii) digital elevation model (terrain), iii) land cover dataset (Manning's roughness coefficient), iv) hydraulic table properties for 2D computational cells and cell faces, and v) boundary conditions (time-series of tidal elevations and riverine inflows).

Computational Mesh

The 2D computational mesh was generated using a combination of breaklines and refinement regions to ensure appropriate cells sizes and to ensure that cell face orientation is perpendicular to flow. Selecting an optimal cell size for 2D computational meshes in HEC-RAS is an iterative process dependent on flow velocities, terrain complexity, and model spatial extent. HEC-RAS pre-processes the terrain to create detailed cross-sections, describing hydraulic properties at each cell face (e.g., elevation, area, volume, wetted perimeter, and roughness). Cells can be partially dry with correct water volume based on underlying terrain, enabling larger computational cell sizes while still accurately capturing underlying terrain features. Larger cell sizes compute water surface values farther apart, averaging the water surface slope over longer distances. Rapidly varying slopes necessitate smaller cell sizes in specific areas.

The flexibility of the HEC-RAS 2D mesh allows varying cell size, shape, and orientation throughout the model domain, which is crucial for capturing high ground features and ensuring efficient run-times. After some iteration, we selected a base cell size of 200 x 200 ft. This was further refined in the channels and tributaries, as well as along levees, roads, hydraulic structures and select drainage ditches using refinement regions and breaklines (Figure 5). Cell sizes in primary channels of interest were chosen to ensure a minimum of 5-10 cells across the bankfull channel. More specifically, we selected a 25 x 25 ft cell size in the main and slough channels, which was enforced with a refinement region that extended from the top of left bank to top of right bank. Additional breaklines (15-50 ft cell size) were added on the centerlines of roads, levees and other important infrastructure to better define high ground areas and key terrain features. Numerous smaller drainage features were captured via dual top of bank breaklines (cell sizes 10-25ft). Finally, additional refinement regions were added to select floodplain areas to better represent the drainage characteristics of key locations (Figure 5).

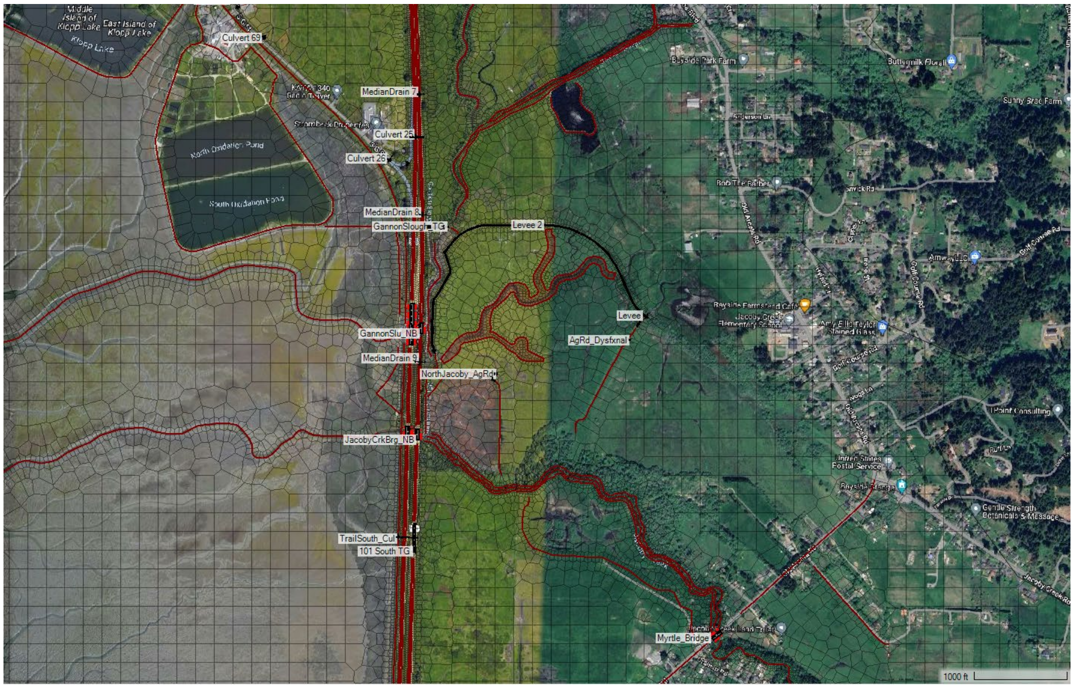


Figure 5. Example of computational mesh for the existing conditions CAIP 2D model illustrating refined 25 ft cells in the channels and coarser scale floodplain cells (50 – 200 ft cells). Red lines represent breaklines and refinement regions while black cells represent computational mesh.

Computational Settings

Computational time-steps were chosen to ensure adherence to the Courant condition and to achieve a balance between numerical accuracy and computational time. A variable time-step method was chosen wherein the model monitors Courant numbers and dynamically adjusts the time-step to ensure the Courant condition is met. Sensitivity analyses identified a maximum Courant number of 2 as the optimal threshold, minimizing computation time while ensuring hydraulic results stabilized.

HEC-RAS can solve either the Diffusive Wave Equations or the Full Momentum Equations (i.e., full Saint Venant or Shallow Water Equations) to route flows in the model. Preliminary model development runs employed the Diffusive Wave equations as this affords shorter model run times. Final production runs of existing conditions utilized the Shallow Water Equations, Eulerian-Lagrangian Method. This approach provided greater accuracy in capturing tidal dynamics, super-elevation in meander bends, and improved simulations of velocity distributions and water surface elevations near hydraulic structures and other key design features.

Hydraulic Structures

The existing conditions model includes a total of 166 hydraulic structures, consisting of 9 bridges, 49 tide gates, and 108 culverts (Table 4; Figure 6). All culverts and tide gates were modeled as Storage Area/2D connections (SA/2D) within the 2D computational mesh, and the standard 2D equation domain was used to solve for structure overflow. Invert elevations and other relevant dimensions for each culvert and tide gate were sourced from various Caltrans structure databases and supplemented by existing structure inventories and previous modeling efforts from GHD (2024), CPH & MLA (2022), and USFWS (2007).

Table 4. Summary of hydraulic structures included in the 2D model.

Type	Count
Bridge	9
Culvert	108
Tide Gate	47
Tide Gate w/ Flashboards	2
Total	**166**

Relevant structure parameters (i.e. dimensions, coordinates, and invert elevations) were categorized as follows and summarized in Table 5:

- Field Surveyed: Parameters were directly measured in the field using survey equipment.
- GHD (2024): Parameters were estimated from a draft model provided by GHD (2024), though the source of structure parameter measurements is unclear.
- Not Inventoried – Assumed: Parameters were assumed based on the dimensions of nearby surveyed structures and professional judgment. This category includes structures that were either uninventoried or mapped without recording the target parameter.
- Field Estimated: Parameters were estimated from photos or field observations without the use of survey equipment.

Table 5. Summary of relevant physical attributes of culvert and tide gates in the model domain.

	Dimensions				Invert Elevations				XY Corrdinates			
Structure	Field Surveyed	GHD (2024)	Not Inventoried - Assumed	Field Estimated	Field Surveyed	GHD (2024)	Not Inventoried - Assumed	Field Estimated	Field Surveyed	GHD (2024)	Not Inventoried - Assumed	Field Estimated
Culverts	60	16	24	8	20	16	65	7	59	16	33	0
Tide Gates	31	12	4	0	13	12	22	0	33	12	2	0

Many private tide gates and culverts visible in aerial photography and terrain surfaces were not included in the cited inventories. When these structures were crucial for accurate drainage simulation, they were added to the model, with their dimensions assumed to match nearby inventoried structures. In cases where structure materials, diameters, or dimensions were recorded, but invert elevations were missing, the structure inverts were assumed to align with the adjacent channel bed.

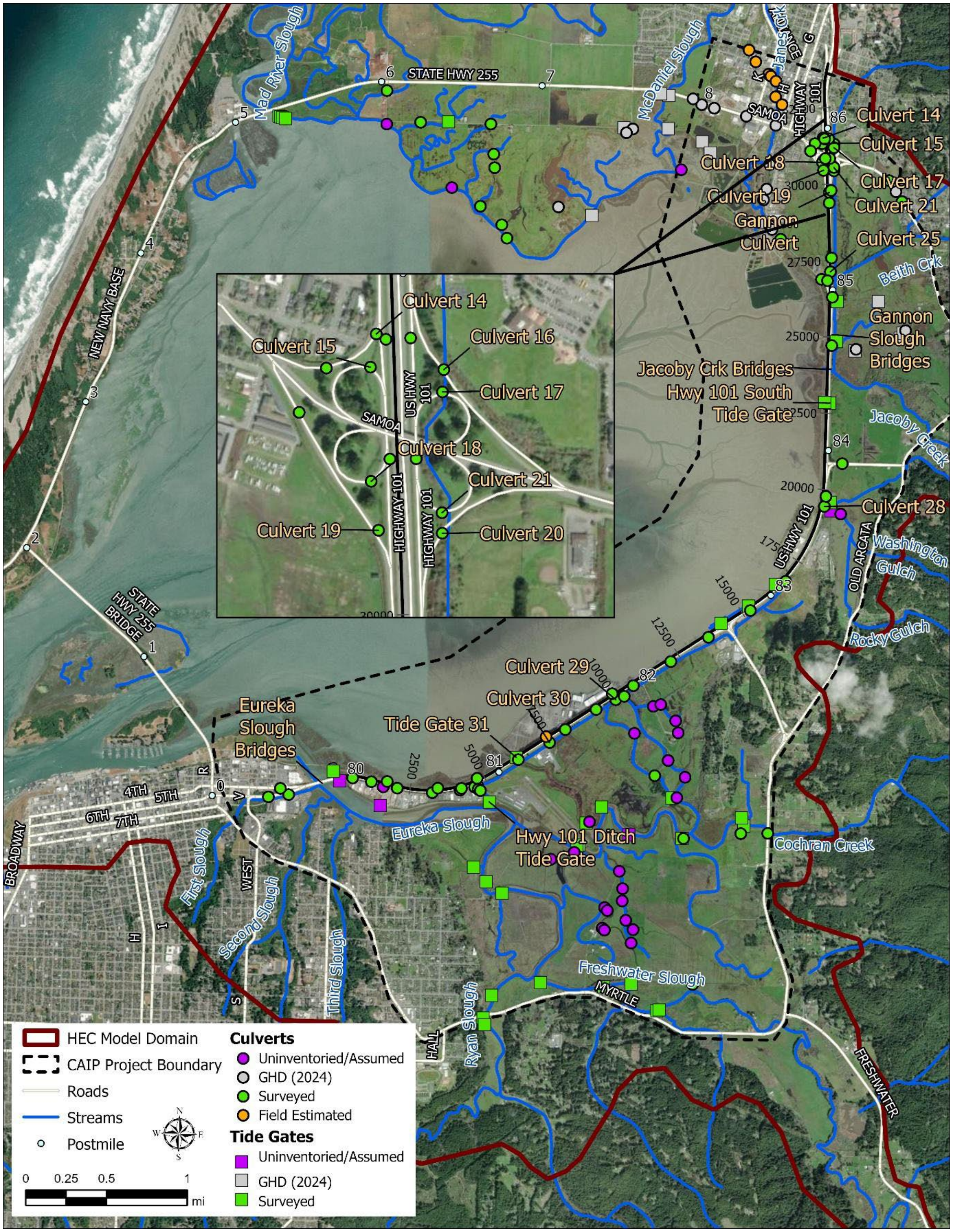


Figure 6. Locations of tide gates, culverts and bridges included in the 2D CAIP model. Orange labels represent priority structures included in the detailed hydraulic analysis outlined in the results section.

The Hwy 101 tide gates and the tide gate associated with the Cochran Creek restoration project were modeled with auxiliary doors to allow for muted tides. To accurately simulate these auxiliary doors in the 2D HEC model, they were represented as separate tide gates with flap gates not allowing positive flow to prevent overestimation of drainage. Levees were modeled either as weir structures or using breaklines in combination with the standard 2D equation domain.

The nine bridge crossings along Highway 101 and Route 255, including all bridge decks, solid railings, and piers, were incorporated into the model (Table 6; Figure 6). Key bridge attributes, such as lower chord and deck elevations, pier widths, and locations, were estimated using a combination of photographs, as-built plan sets, and field measurements provided by Caltrans. However, many of the plan sets were outdated and challenging to interpret, and no digital survey data was available. Consequently, hydraulic results at these bridge crossings should be refined with recent as-built survey data when it becomes available. The "Standard Step" approach for solving the energy equation and the "Pressure and/or Weir Flow" methods were chosen to simulate low and high flows through the bridges, respectively. A total of 10 bridges within the modeling domain were excluded from the model because: i) survey data describing their physical dimensions were unavailable and ii) their hydraulic impacts to the E-A Hwy 101 Corridor were deemed minor and localized outside of project area (see bridges with "Burned" listed as the Data Source in Table 6).

Table 6. Summary of bridges inside the 2D model domain. Data Source listed as "burned" represents a bridge that was not explicitly modeled – instead the channel was burned through the bridge deck.

Road Type	Site Name	Bridge #	Feature Crossed	Data Source
State Hwy Bridges	Eureka Slough	04-0022R (NB)	Eureka Slough	Caltrans
	Eureka Slough	04-0022L (SB)	Eureka Slough	Caltrans
	Jacoby Creek	04-0023R (NB)	Jacoby Creek	Caltrans
	Jacoby Creek	04-0313L (SB)	Jacoby Creek	Caltrans
	Gannon Slough	04-0024R (NB)	Gannon Slough	Caltrans
	Gannon Slough	04-0024L (SB)	Gannon Slough	Caltrans
	McDaniel Slough	04-0222	McDaniel Slough	GHD (2024)
	255 Bridge W	--	Unnamed Slough	GHD (2024)
	Mad River Slough	04-0257	Mad River Slough	Burned
Local Road Bridges	Old Arcata Rd Bridge	04C0182	Jacoby Creek	GHD (2024)
	Howard Heights Rd	04C0049	Freshwater Creek	Burned
	Myrtle Ave	04C0083	Ryan Slough	Burned
	Myrtle Ave	04C0177	Freshwater Creek	Burned
	Devoy Rd	04C0213	Freshwater Slough	Burned
	Jackson Ranch Rd	--	Liscomb Slough	Burned
	Myrtle Ave	--	Freshwater Crk Overflow	Burned
Railroad/ Trail Bridges	Humboldt Trail South	--	Jacoby Creek	Burned
	Humboldt Trail South	--	Gannon Slough	Burned
	Humboldt Trail South	--	Eureka Slough	Burned

Manning’s Roughness Coefficient

Manning's roughness coefficients (*n* values) were determined based on prior modeling experience, professional judgment and field observations. Model *n* values for floodplain areas were derived from conventional tabulated values (COE, 2024). Roughness values for mud-dominated slough channels ranged from 0.03-0.035, while upstream mud/gravel riverine values ranged from 0.04 – 0.055. According to GHD, Jacoby Creek roughness values may be as high as 0.15. This is consistent with very dense in-channel vegetation and is in accordance with calibrated roughness values in the nearby Elk River. For now, the Jacoby Creek *n* values are ~0.055, but future modeling refinement may adjust this value.

For all tide gate structures, *n* values were set to 0.012 - 0.015 for concrete box structures and 0.013 - 0.02 for round culverts. Culvert entrance and exit loss coefficients were set to 0.5 and 1.0, respectively.

Boundary Conditions

The 2D HEC model was forced with a combination of external and internal boundary conditions (BCs) grouped into three “Event Scenarios”:

- <u>Event Scenario I</u>: fluvial flooding from 2-, 10- and 100-yr peak flows – coupled with a representative spring tide (MMMW) at the downstream boundary condition (range ~2.29 - 8.36 ft). This scenario was designed to simulate typical spring tidal maximum conditions, with model simulation periods set to 100 hours.
- <u>Event Scenario II</u>: fluvial flooding from 2-, 10- and 100-yr peak flows – coupled with a 2-yr coastal flood event (range ~2.41 – 9.26 ft). This scenario aimed to represent the likely joint occurrence of an extreme coastal event and significant fluvial flooding, with model simulation periods set to 100 hours.
- <u>Event Scenario III</u>: coastal extreme high-water levels for the 2-, 10- and 100-yr events at the downstream boundary condition – coupled with winter baseflow conditions at all fluvial boundary conditions. This scenario was intended to simulate extreme coastal events in the absence of fluvial flooding, with model simulation periods set to 72 hrs.

Select scenarios from three event conditions were modeled under existing conditions, along with two OPC (2024) Intermediate Sea Level Rise (SLR) projections: 0.82 ft and 3.12 ft. Only model results from Event Conditions I and II are presented here. Results for Event Condition III may be considered during the vulnerability assessment phase, pending the integration of outcomes from ongoing coastal modeling efforts. In all scenarios, fluvial hydrograph peaks were simulated using a generalized hydrograph (Figure 3) and synchronized with the peak tidal boundary conditions. Several tributary flows were represented as internal boundary conditions.

Table 7. Summary of event conditions and scenarios used in the existing conditions CAIP model. Compound frequency estimated based on product of fluvial and tidal probabilities.

Event Condition	Scenario	Tidal Boundary Condition	Fluvial Boundary Condition	Sea Level Rise (ft)	Probability (% Chance per Year) 2023
I	1	MMMW[1]	2-yr	0	50%
	2	MMMW	2-yr	0.82	
	3	MMMW	2-yr	3.12	
	4	MMMW	10-yr	0	10%
	5	MMMW	10-yr	0.82	
	6	MMMW	10-yr	3.12	
	7	MMMW	100-yr	0	1%
	8	MMMW	100-yr	0.82	
	9	MMMW	100-yr	3.12	
II	10	2-yr[2]	2-yr	0	25%
	11	2-yr	2-yr	0.82	
	12	2-yr	2-yr	3.12	
	13	2-yr	10-yr	0	5%
	14	2-yr	10-yr	0.82	
	15	2-yr	10-yr	3.12	
	16	2-yr	100-yr	0	0.50%
	17	2-yr	100-yr	0.82	
	18	2-yr	100-yr	3.12	

[1] Peak of MMMW tide = 8.36 ft

[2] Peak of coastal 2-yr storm = 9.26 ft

Model Validation

Validation data for large storm events within the Project Area is limited. To our knowledge, no datasets exist with surveyed high water mark data for either fluvial or coastal storms, or extreme tidal events. This data gap precludes a robust model calibration and validation process. However, numerous photographs of King Tide events across the Bay offer a basis for qualitative validation.

To qualitatively validate the model, we compared a subset of these photographs from various King Tides to model results for the MMMW & 2-year scenario. The MMMW peaks at 8.36 feet, which is roughly equivalent to a King Tide event in the Bay (see Figure 13 for photo locations). We focused on validation photos taken on the Bay side of Hwy 101 to minimize the influence of the 2-year fluvial event, as the King Tide photos were generally captured when fluvial flows were below a 2-year event threshold. Specifically, we referenced photos from King Tides on December 23 & 24, 2022, and January 11, 2024, where peak water levels at the North Spit Gage (NOAA Station 9418767) were 8.95 feet, 8.65 feet, and 8.4 feet, respectively.

Comparing the King Tide photos with 3D renderings of model results for the MMMW & 2-year event shows that the model accurately captured water surface elevations, flooding extents, and overtopping (Figures 7 – 12).

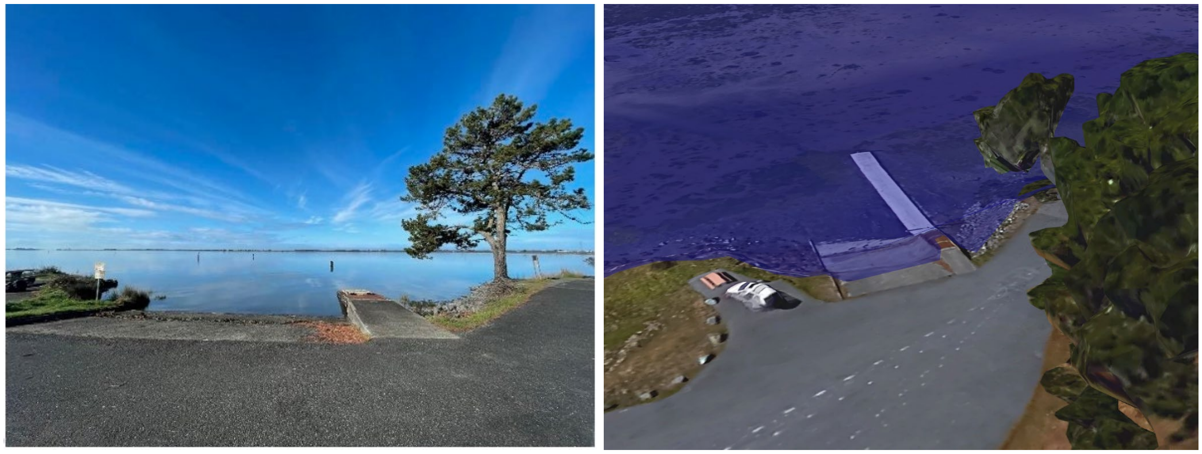


Figure 7. Photo of January 11, 2024 King Tide (left) and MMMW & 2-yr model result (right) at the I Street Boat Ramp, Arcata. Photo credit: City of Arcata.

Figure 8. Photo of January 11, 2024 King Tide (left) and MMMW & 2-yr model result (right) facing north on I Street, Arcata. Photo credit: City of Arcata.

Figure 9. Photo of December 23, 2022 King Tide (left) and MMMW & 2-yr model result (right) facing north at Butcher Slough at South H Street, Arcata. Photo credit: https://www.coastal.ca.gov/kingtides/gallery.html.

Figure 10. Photo of January 11, 2024 King Tide (left) and MMMW & 2-yr model result (right) facing northeast at McDaniel Slough, Arcata. Photo credit: City of Arcata.

Figure 11. Photo of January 11, 2024 King Tide (left) and MMMW & 2-yr model result (right) facing east along Jackson Ranch Rd, Arcata. Note: the model result on right indicates overtopping of Jackson Ranch Road, but inundation is not fully rendered due to larger computational mesh cell sizes in this area. The photo suggests more water may be flooding into the field to the right of the road than the model shows. This particular King Tide was 8.95 ft at the North Spit Station which is roughly 0.6 ft higher than modeled. Photo credit: https://www.coastal.ca.gov/kingtides/gallery.html.

Figure 12. Photo of December 24, 2022 King Tide (left) and MMMW & 2-yr model result (right) facing north at the Former Sierra Pacific Industries lumber mill on Mad River Slough, Manila. Note: peak tide at North Spit was ~8.65 ft. Photo credit: https://www.coastal.ca.gov/kingtides/gallery.html.

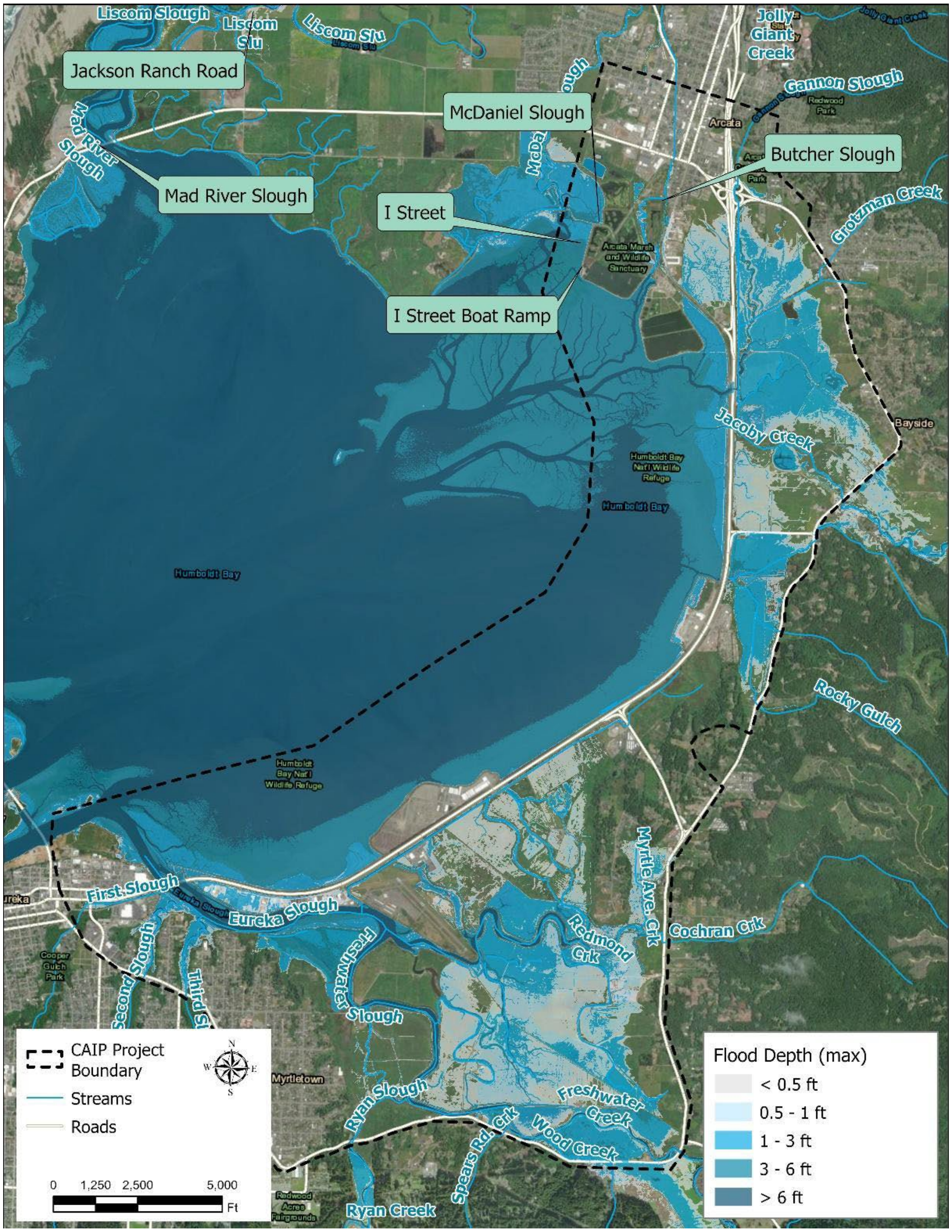


Figure 13. Locations of King Tide model validation photos overlaid on modeled flood depth during the MMMW & 2-yr event scenario.

Results

The following sections provide a brief overview of hydraulic model results under existing conditions across select event scenarios (Table 7), incorporating two OPC (2024) intermediate sea level rise (SLR) projections. These results are intended to support technical discussions and inform future modeling and vulnerability assessment efforts related to SLR impacts.

Flood Cells

Similar to GHD (2021), our floodplain analysis divides the project area into individual "flood cells", which represent hydraulically distinct zones within the floodplain (Figure 14). Typically enclosed by levees or other barriers, flood cells are often designed to contain or control floodwaters. Each cell is managed separately to mitigate flood risks, and a breach in any part of its perimeter can lead to inundation within that cell. Generally speaking, flood cells exclude major tributaries and slough channels, focusing instead on providing insights into floodplain hydraulics. The delineation of these cells is provisional and may be refined further to better support the upcoming vulnerability assessment.

Figure 14. Overview of delineated flood cells within the CAIP Project Area (subject to revision).

Flood Depths

Model results, depicted in Figures A-1 – A-10, show the maximum flood depths for all event scenarios currently modeled. These depths are also summarized for each Flood Cell within the Project Area in Tables 8 and 9. The data indicates that Flood Cells in the Freshwater, Fay, and Eureka Slough areas (e.g. Cells C1, C2, D, H, and E), as well as the north Jacoby Creek floodplain (Cell M2) and Gannon Slough West (Cell N2) areas typically experienced the deepest flooding across modeled events. Conversely, Cells with commercial land use and water control structures, like berms and levees (Cells K, R, A1, and A2), often had the shallowest flood depths. Notably, Flood Cell J remained dry until the most severe sea level rise scenario (3.12 ft), due to the fact that it is bounded by high ground features and the potential for a missing culvert at the Indianola Cutoff linking it to Cell A. Additionally, a comparison of Cells E and F indicates that flood depths in Cell E are relatively insensitive to fluvial storm magnitude (due to protective high ground features), whereas Cell F experiences a rather large increase in flood depth during the Q100 fluvial storms. Flood Cells directly adjacent to larger tributaries such as Jacoby Creek and Freshwater Creek, were particularly sensitive to fluvial flow magnitude. Conversely, cells on the west side of Hwy 101 (e.g. Cells Q & R) and those far removed from a major tributary (e.g. Cells I & O) are relatively insensitive to fluvial storm magnitude. A detailed interpretation of these results and their implications for infrastructure will be addressed in the upcoming vulnerability and risk assessment analysis.

Table 8. Mean maximum flood depths (ft) in all flood cells within the CAIP Project Area for all scenarios in Event Conditions I. Maximum depths are listed parenthetically.

Flood Cell	Flood Depths (ft)								
	MMMW & Q2	MMMW & Q2 + 0.82ftSLR	MMMW & Q2 +3.12ftSLR	MMMW & Q10	MMMW & Q10 + 0.82ftSLR	MMMW & Q10 + 3.12ftSLR	MMMW & Q100	MMMW & Q100 + 0.82ftSLR	MMMW & Q100 + 3.12ftSLR
Cell A	0.45 (7.3)	0.97 (8.1)	3.73 (11.3)	0.59 (7.6)	1.21 (8.3)	3.99 (11.6)	1.16 (8.3)	1.84 (9.1)	4.7 (12.3)
Cell A1	0.8 (4)	1.07 (9)	2.26 (8.8)	1.01 (5)	0.35 (5.8)	2.56 (9)	0.39 (5.7)	0.46 (6.5)	3.27 (9.8)
Cell A2	0.5 (8)	0.54 (8.8)	2.43 (8.7)	0.26 (5)	0.78 (5.8)	2.63 (9)	0.75 (5.7)	1.18 (6.5)	3.2 (9.8)
Cell A3	1.56 (4.7)	1.85 (5.2)	4.75 (8.4)	1.56 (4.6)	2.07 (5.4)	5.02 (8.6)	2.05 (5.3)	2.7 (6.1)	5.72 (9.4)
Cell B	0.68 (3.6)	0.72 (3.7)	1.3 (4.4)	1.42 (4.5)	1.55 (4.6)	2.28 (5.4)	2.66 (5.8)	2.74 (5.9)	3.4 (6.7)
Cell C1	0.7 (8.1)	0.77 (8.1)	2.83 (9)	2.34 (8.5)	2.47 (8.7)	4.51 (10.6)	4.55 (10.7)	4.62 (10.8)	5.38 (11.5)
Cell C2	1.48 (6.3)	1.67 (6.4)	2.66 (6.5)	3.83 (7.3)	3.86 (7.3)	3.99 (7.4)	4.38 (7.9)	4.39 (7.9)	4.59 (8.2)
Cell D	2.52 (7.9)	2.58 (8)	2.77 (8)	3.61 (8.8)	3.62 (8.9)	3.66 (8.9)	4.19 (9.4)	4.19 (9.4)	4.25 (9.5)
Cell E	3.1 (5.8)	3.4 (6.1)	3.9 (5.7)	4.07 (5.9)	4.13 (6)	4.34 (6.2)	4.55 (6.4)	4.58 (6.5)	4.86 (6.8)
Cell F	0.53 (4.2)	0.24 (4.8)	2.82 (5.8)	1.34 (4.3)	2.03 (5)	4.75 (7.8)	5.43 (8.3)	5.45 (8.4)	5.63 (8.6)
Cell G1	0.37 (3.3)	0.31 (4.5)	4.7 (9)	1.18 (5.5)	1.78 (6.1)	5.06 (9.4)	4.21 (8.5)	4.46 (8.8)	5.45 (9.7)
Cell G2	NA	0.2 (1.1)	2.36 (7.6)	0.59 (6.6)	0.34 (6.8)	2.67 (8)	1.95 (7.5)	2.15 (7.7)	3.02 (8.5)
Cell H	2.86 (9.2)	3.75 (10)	4.72 (10.7)	3.02 (8.9)	3.83 (9.7)	4.78 (10.7)	3.1 (9)	3.85 (9.8)	4.99 (10.9)
Cell I	1.1 (9.3)	1.81 (10.1)	3.34 (12.1)	1.13 (9.3)	1.85 (10.1)	3.35 (12.1)	1.14 (9.3)	1.86 (10.1)	3.36 (12.1)
Cell J	NA	NA	1.54 (4.2)	NA	NA	2.51 (5.5)	NA	NA	3.27 (6.4)
Cell K	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cell L	1.96 (4.5)	2.33 (4.9)	3.51 (6.2)	3.06 (5.8)	3.21 (5.9)	4.19 (6.9)	4.15 (6.9)	4.34 (7.1)	5.51 (8.3)
Cell L2	0.72 (4.3)	0.93 (4.7)	1.96 (5.7)	1.57 (5.3)	1.7 (5.5)	2.67 (6.5)	2.65 (6.4)	2.83 (6.6)	3.98 (7.8)
Cell M	0.66 (6.3)	0.86 (6.4)	2.57 (6.9)	2.22 (6.6)	2.44 (6.8)	3.75 (8.2)	3.72 (8.1)	3.92 (8.3)	5.06 (9.5)
Cell M2	1.64 (9.5)	1.74 (9.6)	3.79 (11.7)	3.6 (11.5)	3.69 (11.6)	4.85 (12.8)	4.79 (12.7)	4.86 (12.8)	5.8 (13.8)
Cell N	1.11 (4.6)	1.2 (4.7)	3.19 (7)	3.01 (6.8)	3.1 (6.9)	4.26 (8.1)	4.21 (8.1)	4.28 (8.1)	5.22 (9.1)
Cell N2	1.26 (6.3)	1.38 (6.4)	3.59 (7.7)	3.42 (7.5)	3.51 (7.6)	4.7 (8.9)	4.65 (8.8)	4.73 (8.9)	5.69 (9.9)
Cell O	0.51 (4.2)	0.51 (4.2)	0.34 (2.2)	0.45 (3.8)	0.46 (3.8)	0.83 (4.1)	1.13 (5)	1.15 (5.1)	1.34 (5.1)
Cell P	0.76 (4.6)	0.91 (5.3)	4.53 (8.7)	2.27 (5.5)	2.38 (5.5)	4.57 (8.7)	2.99 (6.4)	3.03 (6.5)	4.65 (8.8)
Cell Q	1.35 (4.6)	1.96 (5.4)	2.87 (7.8)	1.33 (4.4)	1.95 (5.2)	2.87 (7.8)	1.34 (4.4)	1.95 (5.2)	2.87 (7.8)
Cell R	NA	0.15 (0.4)	1.9 (6.7)	NA	0.13 (0.4)	1.93 (6.7)	NA	0.14 (0.4)	2.03 (6.9)

Table 9. Mean maximum flood depths (ft) in all flood cells within the CAIP Project Area for all scenarios in Event Conditions II. Maximum depths are listed parenthetically.

Flood Cell	Flood Depths (ft)								
	C2 & Q2	C2 & Q2 + 0.82ft SLR	C2 & Q2 + 3.12ft SLR	C2 & Q10	C2 & Q10 + 0.82ft SLR	C2 & Q10 + 3.12ft SLR	C2 & Q100	C2 & Q100 + 0.82ft SLR	C2 & Q100 + 3.12ft SLR
Cell A	1.15 (8.2)	2 (9.4)	6.92 (14.7)	1.4 (8.6)	2.24 (9.6)	7.07 (14.9)	1.92 (9.3)	2.76 (10.4)	7.23 (14.9)
Cell A1	0.52 (5.7)	0.59 (10.2)	5.79 (15.5)	0.31 (6)	0.65 (7)	5.88 (15.7)	0.53 (10.1)	0.83 (11.2)	5.91 (12.3)
Cell A2	0.71 (5.7)	1.17 (10)	5.32 (15.3)	0.94 (6)	1.28 (7)	5.48 (15.5)	1.24 (9.9)	1.67 (11)	5.64 (12.3)
Cell A3	2 (5.3)	2.9 (6.4)	7.91 (11.7)	2.24 (5.6)	3.14 (6.6)	8.07 (11.9)	2.8 (6.3)	3.75 (7.4)	8.22 (12)
Cell B	0.71 (3.7)	0.79 (3.8)	5.06 (8.4)	1.52 (4.6)	1.61 (4.7)	5.4 (8.7)	2.72 (5.9)	2.81 (6)	5.65 (9)
Cell C1	0.77 (7)	1.04 (8.1)	5.89 (12.4)	2.5 (8.7)	2.88 (8.9)	6.22 (12.8)	4.65 (11.2)	4.86 (11.4)	6.46 (12.6)
Cell C2	1.7 (6.3)	2.08 (6.5)	4.04 (8)	3.9 (7.3)	3.93 (7.4)	4.46 (8.3)	4.4 (8)	4.41 (8.1)	4.95 (8.7)
Cell D	2.59 (7.8)	2.66 (8.1)	3.39 (8.9)	3.66 (8.9)	3.67 (8.9)	3.87 (9.3)	4.18 (9.6)	4.19 (9.6)	4.47 (9.7)
Cell E	3.42 (5.2)	3.68 (6.4)	4.81 (7.6)	4.17 (6)	4.22 (6.1)	5.14 (8)	4.57 (7.3)	4.62 (7.4)	5.39 (7.3)
Cell F	0.3 (3.6)	1.06 (5.2)	5.82 (9.4)	2.33 (5.3)	3.24 (6.2)	6.16 (9.7)	5.46 (9)	5.49 (9)	6.39 (9.4)
Cell G1	0.34 (4.4)	1.73 (6.3)	6.04 (10.6)	2 (6.3)	3.36 (7.7)	6.38 (10.9)	4.51 (9.1)	4.82 (9.4)	6.62 (10.9)
Cell G2	0.2 (1.6)	1.07 (7)	3.6 (9.9)	0.49 (6.8)	1.33 (7)	3.9 (10.3)	2.2 (8.9)	2.46 (9.2)	4.13 (9.4)
Cell H	3.86 (9.8)	4.31 (10.6)	5.67 (12)	3.94 (9.8)	4.39 (10.3)	5.95 (12.3)	3.96 (10.2)	4.42 (10.7)	6.17 (12.2)
Cell I	1.93 (10.1)	2.55 (10.9)	4.02 (12.9)	1.96 (10.2)	2.58 (10.9)	4.04 (12.9)	1.97 (10.2)	2.57 (11)	4.05 (12.9)
Cell J	NA	NA	5.58 (9)	NA	NA	5.7 (9.1)	NA	NA	5.78 (9.1)
Cell K	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cell L	2.47 (5.1)	2.89 (5.5)	4.53 (7.2)	3.27 (6)	3.45 (6.2)	5.63 (8.4)	4.34 (7.1)	4.6 (7.3)	6.35 (9.2)
Cell L2	0.97 (4.6)	1.39 (5.3)	2.96 (7)	1.77 (5.5)	1.95 (5.7)	4.13 (8.2)	2.84 (6.9)	3.1 (7.1)	4.81 (8.7)
Cell M	0.8 (6.2)	1.08 (6.5)	4.03 (8.7)	2.56 (6.9)	2.94 (7.3)	5.19 (9.8)	3.91 (8.5)	4.16 (8.8)	5.9 (10.4)
Cell M2	1.77 (9.6)	2.21 (10)	5.44 (13.4)	3.78 (11.7)	3.96 (11.9)	5.91 (13.9)	4.88 (12.8)	5.1 (13.1)	6.37 (14.4)
Cell N	1.23 (4.8)	1.64 (5.3)	4.85 (8.7)	3.19 (7)	3.37 (7.2)	5.32 (9.2)	4.32 (8.2)	4.51 (8.4)	5.79 (9.7)
Cell N2	1.41 (5.5)	1.95 (7.1)	5.28 (10.6)	3.6 (7.7)	3.79 (7.9)	5.79 (11.1)	4.75 (10)	4.97 (10.3)	6.27 (10.4)
Cell O	0.46 (2.2)	0.5 (4.2)	1.09 (4.2)	0.49 (3.8)	0.56 (3.8)	1.17 (5.2)	1.17 (5.7)	1.23 (5.7)	1.13 (5.1)
Cell P	0.84 (5.4)	2.23 (7.2)	5.33 (10.3)	2.39 (5.8)	3.18 (6.8)	5.34 (10.4)	3.02 (7)	3.49 (7.8)	5.34 (9.8)
Cell Q	2.01 (5.3)	1.6 (6.2)	3.43 (8.6)	2.02 (5.3)	1.54 (6.1)	3.43 (8.6)	2.02 (5.5)	1.57 (6.2)	3.41 (8.6)
Cell R	0.22 (0.9)	0.47 (4)	4.57 (9.5)	0.22 (0.9)	0.46 (4)	4.73 (9.6)	0.22 (1)	0.48 (4)	4.89 (10)

Tables 10 & 11 summarize the percent of the total area of each Flood Cell within a range of flood depths for two event scenarios (MMMW & Q100 and MMMW & Q100 + 3.12ft SLR) and help add additional nuance to the flood depth results. For instance, despite Cell H showing significant flood depths, Table 10 reveals that only 43.7% of the Cell was wet during the MMMW & Q100 event. In contrast, Cells E and N2, while also experiencing deep flooding (i.e. high percent areas > 3ft in depth), had over 90% of their total area submerged, as indicated by Table 10. Additionally, Table 10 underscores that even in the absence of SLR, low-lying Flood Cells adjacent to Freshwater Creek and Slough are among the most heavily inundated, with more than 90% of their areas underwater and substantial portions exceeding depths of 3 ft (e.g. C1, C2, D, E, F, & G1).

Table 10. Percent of total area at different flood depth ranges for all flood cells in the Project Area during the MMMW & Q100 event scenario.

Flood Cell	Flood Depths							
	<0.5 ft	0.5-1 ft	1-2 ft	2-3 ft	>3 ft	% AOI Wet	Total Inundated Area (ac)	Area of AOI (ac)
Cell A	10.22	15.77	28.83	8.83	2.51	66.17	405.23	612.42
Cell A1	1.82	0.26	0.25	0.13	0.9	3.35	1.74	51.91
Cell A2	6.96	12.67	4.27	0.78	1.38	26.06	26.63	102.15
Cell A3	7.24	8.41	22.61	25.6	14.72	78.58	15.67	19.94
Cell B	1.09	1.37	4.85	13.99	8.73	30.03	60.14	200.27
Cell C1	0.49	0.54	1.34	2.71	85.49	90.57	791.36	873.74
Cell C2	1.19	2.04	6.28	9.87	78.44	97.82	71.54	73.13
Cell D	0.92	2.32	5.95	8.82	78.28	96.3	68.11	70.73
Cell E	5.69	3.44	6.77	6.46	68.67	91.02	10.04	11.03
Cell F	1.17	1.14	2.19	2.79	84.56	91.85	33.09	36.03
Cell G1	0.34	0.42	1.06	1.19	95.25	98.26	129.04	131.32
Cell G2	0.65	0.93	3.07	3.64	0.7	8.99	29.03	322.98
Cell H	1.08	1.19	3.94	12.42	25.08	43.71	85.68	196.01
Cell I	1.94	4.05	7.52	0.63	2.7	16.84	57.5	341.48
Cell J	0	0	0	0	0	0	0	173.75
Cell K	0	0	0	0	0	0	0	21.39
Cell L	4.87	3.02	4.3	6.06	54.69	72.94	65.75	90.15
Cell L2	1.4	1.61	7.18	24.48	18.75	53.42	16.89	31.63
Cell M	4.54	3.35	7.23	9.24	59.14	83.5	160.44	192.15
Cell M2	3.03	3.07	7.96	6.53	53.87	74.45	292.56	392.94
Cell N	2.09	1.71	4.98	11.4	72.82	92.98	115.37	124.08
Cell N2	1.47	1.93	3.81	3.32	86.77	97.3	26.95	27.7
Cell O	8.06	8.73	14.77	4.73	3.13	39.41	28.41	72.09
Cell P	5.59	5.2	7	10.55	27.85	56.18	116.07	206.6
Cell Q	1.87	1.84	9.92	1.25	0.19	15.07	5.08	33.7
Cell R	0	0	0	0	0	0	0	74.64

A comparison of Tables 10 and 11 indicates that some Flood Cells exhibit greater sensitivity to sea level rise (SLR) than others. Notably, Flood Cells A1, A2, A3, J, Q, and R experience a significant increase in both the percentage of area submerged under more than 3 feet of water and the total inundated area with 3.12 feet of SLR. This heightened sensitivity is largely attributed to the overtopping of protective features, such as levees, at this SLR level. In contrast, many Flood Cells show relatively low sensitivity to an intermediate SLR level of 3.12

feet (e.g., C1, C2, D, F, K, and O), as demonstrated by the minimal increase in inundated area with rising SLR (see Tables 10 & 11).

Table 11. Percent of total area at different flood depth ranges for all flood cells in the Project Area during the MMMW & Q100 + 3.12 ft SLR event scenario.

Flood Cell	Flood Depths							
	<0.5ft	0.5-1ft	1-2ft	2-3ft	>3ft	% AOI Wet	Total Inundated Area (ac)	Area of AOI (ac)
Cell A	1.25	1.57	4.45	6.17	71.36	84.81	519.41	612.42
Cell A1	0.76	0.87	7.42	26.87	58.29	94.2	48.9	51.91
Cell A2	3.23	4.09	15.35	20.14	50.84	93.64	95.66	102.15
Cell A3	1.06	1.22	3.21	4.33	87.44	97.26	19.39	19.94
Cell B	0.91	1.01	2.76	5.85	21.03	31.56	63.21	200.27
Cell C1	0.48	0.46	1.03	1.39	87.95	91.3	797.73	873.74
Cell C2	1.11	1.91	5.87	9.26	79.88	98.03	71.69	73.13
Cell D	0.92	2.27	5.7	8.62	78.86	96.37	68.16	70.73
Cell E	4.46	5.28	7.15	6.28	70.96	94.13	10.38	11.03
Cell F	1.16	1.21	2.18	2.52	85.29	92.36	33.28	36.03
Cell G1	0.26	0.33	0.68	1.05	96.68	99.01	130.03	131.32
Cell G2	0.24	0.35	1.42	2.78	4.97	9.77	31.55	322.98
Cell H	0.94	1.01	1.99	2.45	40.98	47.38	92.86	196.01
Cell I	2.03	2.33	4.17	3.99	15.81	28.33	96.75	341.48
Cell J	2.2	2.3	5.36	8.62	26.07	44.56	77.43	173.75
Cell K	0	0	0	0	0	0	0	21.39
Cell L	3.51	2.4	3.46	3.12	62.49	74.99	67.61	90.15
Cell L2	0.89	0.93	2.55	4.21	47.43	56.02	17.72	31.63
Cell M	3.21	2.62	5.41	6.1	71.06	88.4	169.86	192.15
Cell M2	2.71	2.38	6.64	5.24	59.03	76	298.65	392.94
Cell N	1.21	1.52	2.79	4.6	84.47	94.59	117.36	124.08
Cell N2	0.51	0.89	3.32	3.85	90.25	98.83	27.37	27.7
Cell O	8.17	7.4	16.14	6.35	3.27	41.32	29.79	72.09
Cell P	2.43	3.18	7.24	10.59	48.43	71.87	148.48	206.6
Cell Q	4.53	5.03	10.25	12.14	28.71	60.66	20.45	33.7
Cell R	3.82	6	36.36	35.04	14.18	95.4	71.21	74.64

Duration of Inundation

The model results, presented in Figures B-1 – B-10 in Appendix B, provide a comprehensive overview of flooding duration across the Project Area for all modeled scenarios where overtopping of Hwy 101 occurred, while Tables 12 & 13 summarize the inundation times for each Flood Cell. During the smallest event (MMMW & Q2), the Project Area generally experiences mild to moderate flooding duration (lasting approximately 1 – 36 hours). For the MMMW & Q10 event, the duration nearly doubles in many regions and Flood Cells (e.g. Cells C1, C2 and D; Table 12) and continues to increase with larger events (Tables 12 & 13 and Figures B-1 – B-10).

Flood Cells E, H, and L not only exhibit some of the deepest floodwaters (Tables 8 & 9) but also endure the longest periods of inundation, indicating higher flood risk and inefficient drainage. Extended flooding in Cell L, in particular, results from a combination of factors: i) overbank flow from Rocky Gulch, ii) a confining downstream levee, and iii) an undocumented culvert penetrating the downstream levee, modeled with estimated dimensions and elevations, and potentially equipped with a tide gate. Flood Cells E, H, and L remain inundated for nearly the entire simulation period across all modeled 2-year coastal event scenarios (Table 13). Both Cells E and H are surrounded by substantial levees with significant upstream breaches, allowing large volumes of floodwater to enter during 2-yr coastal storms. However, downstream levees and undersized tide gates or culverts restrict outflow, leading to prolonged flood durations within these cells. In contrast, at 3.12 feet of sea level rise, Cells C2, O and G2 consistently exhibit minimal flood depths and durations. Notably, Cells C1, C2, D and N2 all exhibit significant flood depths and extensive areas of inundation. However, these cells also demonstrate shorter durations of inundation, suggesting they are characterized by good hydrologic connectivity and efficient drainage.

Table 12. Mean duration of inundation (hrs) for flooded areas within each Flood Cell within the CAIP Project Area for all scenarios in Event Condition I (total simulation time 100 hrs).

Flood Cell	Inundation Duration (hrs)								
	MMMW & Q2	MMMW & Q2 +0.82ftSLR	MMMW& Q2 +3.12ftSLR	MMMW& Q10	MMMW & Q10+ 0.82ftSLR	MMMW& Q10+ 3.12ftSLR	MMMW& Q100	MMMW & Q100+ 0.82ftSLR	MMMW& Q100+ 3.12ftSLR
Cell A	72.5	83.25	100.25	77	83.5	100.25	80.5	83.75	100.25
Cell A1	83.25	81	100.25	79	58.75	100.25	61.5	54.75	100.25
Cell A2	61	69.5	100.25	57	74.75	100.25	68.25	74.25	96.75
Cell A3	76.75	83.25	100.25	77.25	83.25	100.25	76.75	82.5	100.25
Cell B	68	72.75	88	76.25	85.75	89.25	88.25	89.5	90
Cell C1	30	36.5	85.5	71.75	82.5	86.5	85.75	86.75	87.25
Cell C2	16	18.5	70	33	34.75	81	49.25	56	83.75
Cell D	19.25	20.5	81.25	35	36.5	82.75	47.5	49	84
Cell E	79.5	90.25	99	80.5	90	99	71.25	89.25	98.75
Cell F	85	29.75	86.5	35.25	44.25	86.75	57	59	87.5
Cell G1	31.5	35.25	87.25	61.5	82.75	87.25	86.25	86.5	87.75
Cell G2	NA	82.25	86	4	28.75	86.5	38.25	42.75	86.5
Cell H	89.25	92	100.25	89.25	92	100.25	89.5	92	100.25
Cell I	27.5	65.5	98.25	27.5	65.5	98.25	27.5	65.5	98.25
Cell J	NA	NA	58.75	NA	NA	68.5	NA	NA	81.75
Cell K	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cell L	83.25	88.5	97	89.75	90.25	96.75	91	91.25	96.5
Cell L2	43.75	54.25	96.25	56.5	66.5	96.25	68.75	72.25	90.25
Cell M	39.5	46.25	85.75	68.75	74.75	87	83.5	87.25	88.25
Cell M2	36	50	89.5	57.5	61.25	90	69.25	72	91
Cell N	30.75	33.75	86.5	48.25	51.75	88.25	60.5	63.25	89.75
Cell N2	30.25	33.75	87.75	54.75	58.5	89.25	64.25	69.75	90
Cell O	10.75	10.75	27.25	9.25	9.25	29	12.5	12.5	15
Cell P	59.25	69	87	79.5	84	87	82.25	85.25	87.25
Cell Q	51.75	65.75	86.5	51.75	65.75	86.5	51.75	65.75	86.5
Cell R	NA	85.5	85.25	NA	85.25	85.25	NA	85.5	85.25

It is important to note that the mean duration values in Tables 12 & 13 can obscure underlying flood patterns. For example, the duration of the C2 & Q2 storm in Cell G2 is 80.5 hrs, but when sea level rise is introduced (C2 & Q2 + 0.82ft SLR), the mean inundation period actually decreased to 35.25 hrs. The longer inundation period under the no-SLR scenario is due to the fact that only about 1% of the cell is flooded, with water pooling in shallow, poorly drained depressions. In contrast, with 0.82 ft of sea level rise, the extent of inundation increases to approximately 10%, accompanied by deeper water (over 1 ft) and improved drainage, as the flooding is no longer restricted to isolated shallow depressions.

These complex patterns indicate varied flood response characteristics across the flood cells, which will be explored further in the forthcoming vulnerability and risk assessment.

Table 13. Mean duration of inundation (hrs) for flooded areas in all flood cells within the CAIP Project Area for all scenarios in Event Condition II (total simulation time 100 hrs).

Flood Cell	Inundation Duration (hrs)								
	C2 & Q2	C2 & Q2 + 0.82ftSLR	C2 & Q2 + 3.12ftSLR	C2 & Q10	C2 & Q10 + 0.82ftSLR	C2 & Q10 + 3.12ftSLR	C2 & Q100	C2 & Q100 +0.82ftSLR	C2 & Q100 +3.12ftSLR
Cell A	79.75	81	96	79.75	81	96	79.75	81	96
Cell A1	65.5	52.25	96	51.5	52.5	96	51.75	54.75	96
Cell A2	58.5	76.75	96	70	77	96	69.75	76.75	96
Cell A3	76.75	81	96	77	81.25	96	77	81.25	96
Cell B	71	83.5	82.5	84.5	84.75	84.25	85.25	85.5	85.5
Cell C1	35.5	68	82.5	80.5	81.75	83.25	82.5	82.75	83.75
Cell C2	19.75	33	83.5	36.25	47	83.75	57	60.5	84.5
Cell D	19.5	54.5	91	37.5	57	91	48	60	91
Cell E	90.75	92.25	96	90.5	92.25	96	90	91.75	96
Cell F	31	55.25	90.25	44.5	58.25	90.25	59.75	62.5	90.5
Cell G1	55.25	81.5	91.25	81.25	82	91.25	82.5	82.75	91.25
Cell G2	80.5	35.25	83.25	28.25	56.5	83.5	42.25	58.25	83.75
Cell H	91.5	92.75	96	91.5	92.75	96	91.75	92.75	96
Cell I	78	79.25	90.5	78	79.25	90.5	78	79.25	90.5
Cell J	NA	NA	82	NA	NA	82	NA	NA	82
Cell K	NA	NA	NA	NA	NA	NA	NA	NA	NA
Cell L	84.5	84.75	92.5	86	86.25	92.25	87	87.25	92.25
Cell L2	56.75	79.75	92	65.75	82.25	91.75	78.75	84	91.75
Cell M	45.75	61	82.5	74.75	82.5	83.25	83.75	84	84
Cell M2	55.25	62	90.5	61	74.5	90.5	70.5	83	90.5
Cell N	34	58	83.25	55.5	64.5	84.75	64	70.5	86
Cell N2	33.75	59.75	85.5	59.75	70.25	86.75	69	76.75	87.5
Cell O	10.75	10.75	76.75	8.75	9	53.5	12.5	12.75	27
Cell P	69.25	81.5	87.5	80.5	81.75	88.25	81.5	82	88
Cell Q	78	81	83.25	78	81.25	83.25	78	81.25	83.25
Cell R	81	51.25	82.5	81.25	51.5	82.5	81	51.25	82.5

Inundation Extents

Tables 14 and 15 summarize the percentage of the total area inundated in each Flood Cell for event conditions I and II, respectively. The greatest extent of inundation occurs in flood cells near upper Freshwater Creek (e.g. Cells C1, C2, D, and E), which lack substantial levee protection. Although Cell E is surrounded by a levee, a significant breach at the upstream end allows floodwaters to enter, leading to larger flooding extents. Similarly, large portions of Flood Cells adjacent to Rocky Gulch, Washington Gulch, and Jacoby Creek (e.g., Cells L, L2, M, M2, N, and N2) experienced extensive inundation, even under moderate fluvial flows and without sea level rise (SLR).

In contrast, Flood cells in higher elevation areas (e.g. Cells B, G2 and K) and those protected by intact levees and high ground features (e.g. Cells R, J, and A2) were more resilient to SLR and generally exhibited smaller inundation extents, particularly with < 3 ft of SLR. Cells closer to the Bay (e.g. Cells A1 & A2), particularly those west of Highway 101 (e.g. Cells R and Q), were more affected by coastal conditions and sea level rise. Conversely, cells located farther inland and near inflowing stream channels are more influenced by fluvial flood events. For example, Cell O, which experiences overbank flooding from Campbell Creek, shows a much greater percentage increase in inundated area as fluvial peak flows rise (Q2 vs. Q100) compared to increases in sea level rise levels.

Table 14. Percent of the total area inundated in all flood cells within the CAIP Project Area for all scenarios in Event Condition I.

Flood Cell	Percent of Total Area Inundated								
	MMMW & Q2	MMMW & Q2 + 0.82ftSLR	MMMW & Q2 + 3.12ftSLR	MMMW & Q10	MMMW & Q10 + 0.82ftSLR	MMMW & Q10 + 3.12ftSLR	MMMW & Q100	MMMW & Q100 + 0.82ftSLR	MMMW & Q100 + 3.12ftSLR
Cell A	46.2	64.8	82.0	55.3	67.3	82.9	66.2	72.5	84.8
Cell A1	1.5	2.5	92.5	1.4	3.7	93.1	3.4	21.1	94.2
Cell A2	6.7	22.0	89.0	12.9	27.0	90.3	26.1	42.2	93.6
Cell A3	67.7	75.2	95.0	67.9	79.6	95.7	78.6	86.2	97.3
Cell B	20.7	21.5	25.8	26.4	26.9	29.2	30.0	30.2	31.6
Cell C1	65.9	70.6	88.6	83.7	88.0	90.4	90.6	90.6	91.3
Cell C2	87.3	88.7	91.7	96.3	96.4	96.6	97.8	97.8	98.0
Cell D	88.3	88.6	89.5	94.6	94.6	94.7	96.3	96.3	96.4
Cell E	77.0	79.2	84.2	85.0	85.5	88.8	91.0	91.3	94.1
Cell F	0.2	31.9	85.8	79.1	82.8	90.3	91.8	91.9	92.4
Cell G1	11.4	53.5	98.6	95.2	96.0	98.8	98.3	98.4	99.0
Cell G2	0.0	0.2	9.3	0.9	3.1	9.6	9.0	9.2	9.8
Cell H	42.2	43.9	46.9	42.5	44.2	47.1	43.7	45.1	47.4
Cell I	16.7	19.2	28.2	16.8	19.3	28.3	16.8	19.4	28.3
Cell J	0.0	0.0	34.0	0.0	0.0	40.6	0.0	0.0	44.6
Cell K	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cell L	61.0	63.2	67.1	69.2	69.6	72.3	72.9	73.3	75.0
Cell L2	34.3	43.3	51.5	50.2	50.7	53.4	53.4	53.8	56.0
Cell M	60.1	63.7	76.8	74.9	76.4	83.3	83.5	84.3	88.4
Cell M2	63.0	63.3	69.6	70.7	71.0	73.6	74.5	74.6	76.0
Cell N	64.4	66.6	88.9	88.9	89.2	92.3	93.0	93.1	94.6
Cell N2	84.7	85.4	93.5	92.8	93.2	97.4	97.3	97.5	98.8
Cell O	1.8	1.8	7.5	10.5	11.3	18.7	39.4	39.6	41.3
Cell P	36.2	39.6	71.1	48.1	50.1	71.3	56.2	57.1	71.9
Cell Q	15.2	19.7	60.6	15.0	19.7	60.6	15.1	19.8	60.7
Cell R	0.0	0.1	94.4	0.0	0.2	94.6	0.0	0.2	95.4

Table 15. Percent of the total area inundated in all flood cells within the CAIP Project Area for all scenarios in Event Condition II.

Flood Cell	Percent of Total Area Inundated								
	C2 & Q2	C2 & Q2 + 0.82ftSLR	C2 & Q2 + 3.12ftSLR	C2 & Q10	C2 & Q10 + 0.82ftSLR	C2 & Q10 + 3.12ftSLR	C2 & Q100	C2 & Q100 + 0.82ftSLR	C2 & Q100 + 3.12ftSLR
Cell A	67.1	73.6	86.8	69.2	75.1	86.8	73.0	78.0	86.9
Cell A1	3.0	25.4	97.9	6.6	36.6	98.0	24.7	69.4	97.8
Cell A2	24.9	53.9	97.5	31.5	62.0	97.6	43.2	70.8	98.0
Cell A3	78.4	87.1	99.6	82.3	88.9	99.7	86.5	91.3	99.8
Cell B	21.2	22.2	34.0	26.7	27.2	34.5	30.1	30.4	34.8
Cell C1	73.2	82.9	91.6	88.4	88.9	91.9	90.6	90.8	92.1
Cell C2	88.8	90.4	95.5	96.5	96.6	97.2	97.6	97.6	98.4
Cell D	88.8	88.9	91.0	94.8	94.8	95.0	96.2	96.2	96.7
Cell E	80.3	81.8	93.8	85.8	86.6	95.6	90.9	91.5	96.6
Cell F	42.2	77.5	92.3	84.1	86.9	93.0	91.4	91.5	93.8
Cell G1	66.5	96.2	99.2	96.3	97.7	99.3	98.4	98.6	99.3
Cell G2	0.9	6.3	10.0	4.4	8.0	10.1	9.2	9.5	10.2
Cell H	44.3	45.7	48.3	44.6	45.9	48.7	45.3	46.1	49.0
Cell I	19.6	22.6	30.9	19.7	22.6	31.0	19.8	22.7	31.0
Cell J	0.0	0.0	50.2	0.0	0.0	50.4	0.0	0.0	50.5
Cell K	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cell L	63.8	65.3	70.0	69.8	70.3	74.6	73.3	73.7	76.4
Cell L2	46.4	49.2	54.0	50.9	51.5	56.2	53.7	54.2	57.5
Cell M	63.0	66.9	84.3	77.1	79.2	88.3	84.3	85.4	90.7
Cell M2	63.4	64.7	73.2	71.3	71.8	75.3	74.6	74.9	76.8
Cell N	67.3	77.8	93.1	89.5	90.0	94.2	93.2	93.7	95.3
Cell N2	85.5	87.7	98.3	93.6	94.4	98.8	97.5	97.9	99.2
Cell O	1.6	1.8	15.6	12.6	13.9	28.1	39.9	40.6	40.0
Cell P	39.4	53.5	75.2	50.5	59.4	75.3	56.8	62.5	76.2
Cell Q	20.1	32.1	67.8	20.1	32.3	67.8	20.1	32.5	68.0
Cell R	0.6	34.5	98.4	0.7	34.4	98.6	0.7	35.0	98.2

Infrastructure

The following section presents a detailed overview of post-processed hydraulic model results for the Eureka-Arcata Corridor, specifically focusing on the southbound and northbound travel lanes of Highway 101, as well as a selection of the 166 bridges, culverts, and tide gates within the model domain. Special emphasis is placed on the hydraulic structures located within the Highway 101 right-of-way. These findings aim to inform discussions on key impact thresholds and associated vulnerability assessments.

Tide Gates and Culverts

A total of 16 Caltrans culverts and tide gates located within the Eureka-Arcata Hwy 101 Corridor (Figure 6) were included in the detailed hydraulic analysis outlined in the following section. Of these 16 structures, five crossed both Hwy 101 northbound and southbound travel lanes (Tables 16 & 17). Among these, only the South Hwy 101 Tide Gate and Culvert 28 were directly hydraulically connected to Humboldt Bay (i.e. tailwater controlled directly by Bay tidal fluctuations). The remaining culverts and tide gates were primarily associated with drainage ditches paralleling Hwy 101 or ditches that crossed under on/off ramps or access roads proximal to Hwy 101.

The hydraulic performance of the 16 structures was analyzed over the suite of 18 event conditions, focusing on the following key metrics:

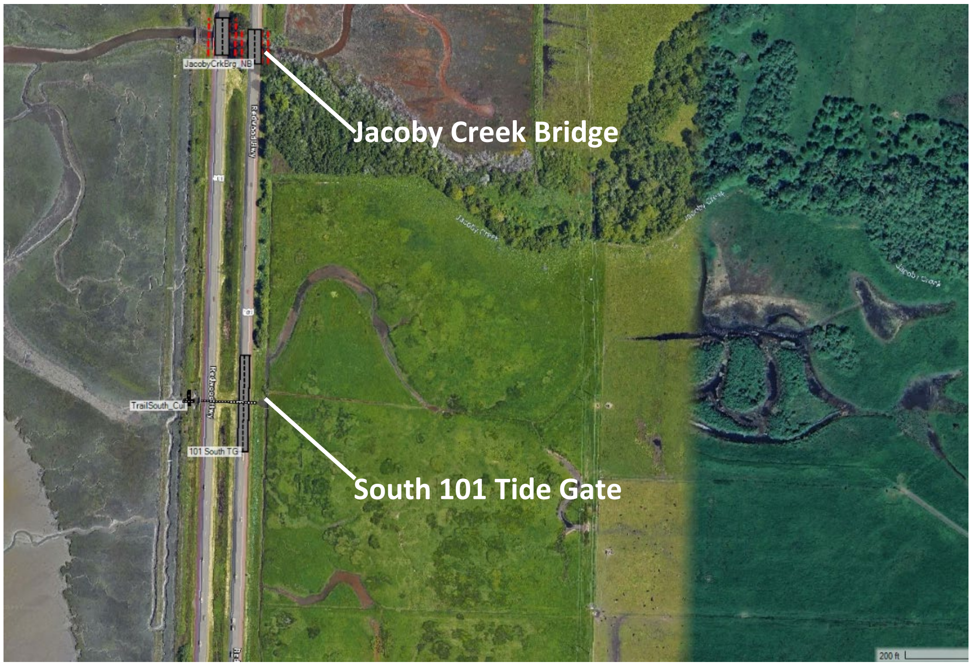
- Drainage Window: The total time available for draining upstream catchments. Rising sea levels are expected to significantly reduce this time in the Project Area, impacting drainage efficiency.
- Drainage Volume: The total water volume drained through the tide gate during the simulation. Sea level rise is anticipated to reduce conveyance capacity due higher tailwater elevations. This data may be used to calculate a drainage efficiency ratio, a metric useful for assessing vulnerability by comparing the total volume drained to the total storage volume in the upstream drainage area.
- Maximum Headwater Elevation: The peak upstream water surface elevation during the simulation. This can be evaluated against the maximum allowable headwater elevation, defined either by the headwater-to-culvert diameter (Hw/D) ratio (typically 1.2 to 1.5, depending on culvert size and design storm) or by setting it 1-2 feet below the low shoulder elevation of the adjacent road surface for a Q100 storm (NYSDOT, 2021). For this analysis, the headwater threshold was set 1 ft below the edge of travel lane on the headwater side.
- Maximum Tailwater Elevation: The peak downstream water surface elevation during the simulation. This can be evaluated against the maximum allowable tailwater elevation, which is commonly set at 1-2 ft below the low shoulder elevation of the adjacent road surface. For this analysis, the tailwater threshold was set 1 foot below the edge of travel lane on the tailwater side.
- Overtopping Duration: The total time during which the headwater or tailwater elevations exceeded one or more of the following thresholds: i) Maximum Allowable Headwater/Tailwater Elevation, ii) road centerline elevation, or iii) road centerline elevation plus a defined threshold depth of water. For this analysis, the threshold overtopping depth was set at 4 inches above the road centerline.

Hydraulic results for the selected structures are summarized in Tables 16 - 19. Performance plots illustrating the maximum headwater and tailwater elevations relative to the road centerline, as well as the maximum allowable headwater/tailwater elevations for each crossing structure, are shown in Figures C-1 – C-16 in Appendix C. Although this section provides only a high-level overview, a detailed analysis is included for a representative subset of structures to highlight key findings. A comprehensive evaluation of the hydraulic performance for all structures, focusing specifically on vulnerabilities related to sea level rise, will be conducted in the next phase of the project.

Culverts & Tide Gates that Cross Hwy 101

Only Culvert 28 and the South Hwy 101 Tide Gate are directly connected to Humboldt Bay (refer to Figure 6 for structure locations). As such, they possessed unique hydraulics that were directly affected by rising tailwater elevations associated with sea level rise. Consequently, the "drainage window" performance metric was particularly relevant for these structures. Tables 14 and 15 demonstrate that rising sea levels (0.82 & 3.12 ft of SLR) shorten the drainage window due to tidal dampening and higher low tide levels, limiting the time the tide gate can remain open to allow effective drainage. For instance, during the MMMW & Q10 event scenario, a 3.12 ft increase in sea level shortens the drainage window for the South Highway 101 Tide Gate (Figure 15) by approximately 22%, from 77 hours to 60 hours during a ~100 hour model run. Culvert 28 experiences even greater reductions in drainage window of ~30%. These changes not only decrease the time available for effective drainage but also increasingly impair passage for vulnerable aquatic organisms, highlighting the broader ecological impacts of reduced drainage capacity. Other culverts, such as the Gannon Culvert, exhibit more complex hydraulics. For instance, during many modeled coastal 2-year storms, the drainage window increases with rising SLR, and some scenarios show negative net drainage volumes - indicating net upstream movement of floodwater through the culvert due to elevated tailwater levels (Tables 16 & 17). The hydraulics in the Gannon Culvert are largely explained by its connection between Flood Cells P and N2, where Cell P (upstream) is inundated only during more extreme events (e.g., MMMW + 3.12 feet SLR and C2 + 0.82 or 3.12 feet SLR). Consequently, headwater elevations exceed tailwater elevations only during these extreme conditions, leading to positive drainage volumes and longer drainage windows with increasing SLR.

A further examination or the South Hwy 101 Tide Gate performance plot indicates that the centerline elevation of both southbound and northbound Hwy 101 as well as the maximum allowable headwater/tailwater elevations are only exceeded in model runs including 3.12 ft of SLR (Tables 14 & 15 and Figure C-16). The elevation of the edge of travel lane on the headwater side of Hwy 101 is approximately 12.6 feet. With a maximum allowable headwater elevation set at 1 ft below the edge of travel lane (11.6 ft), Figure C-16 indicates this threshold is exceeded for roughly 8 hrs during the C2 & 100-year storm with 3.12 ft of SLR. The centerline of northbound Hwy 101 proximal to the structure (headwater side) is not overtopped during any of the modeled event conditions.



Aside from fluvial 100-yr events, tailwater elevations at this structure are generally higher than headwater elevations under model runs, indicating the tide gate and adjacent road is generally more prone to overtopping from coastal storms. As depicted in Figure C-16, at 3.12 feet of SLR, the maximum allowable tailwater and the centerline elevations of southbound Hwy 101 are exceeded for durations ranging from 2.25 to 14.25 hours, depending on the event condition. However, neither the northbound nor southbound road centerlines are overtopped by more than 4 inches of water in any of the modeled scenarios.

As evidenced in Tables 16 and 17 and Figures C-12 – C-16, the maximum allowable headwater/tailwater and adjacent road centerlines were not substantially overtopped for any culverts or tide gates crossing under Hwy 101 except under the 3.12 ft SLR simulations. Tide Gate 31 (Figure C-15) exhibited the longest duration of overtopping of the adjacent Hwy 101 centerline – 19 hrs from elevated tailwaters during the C2 & Q100 + 3.12 ft SLR (Table 17).

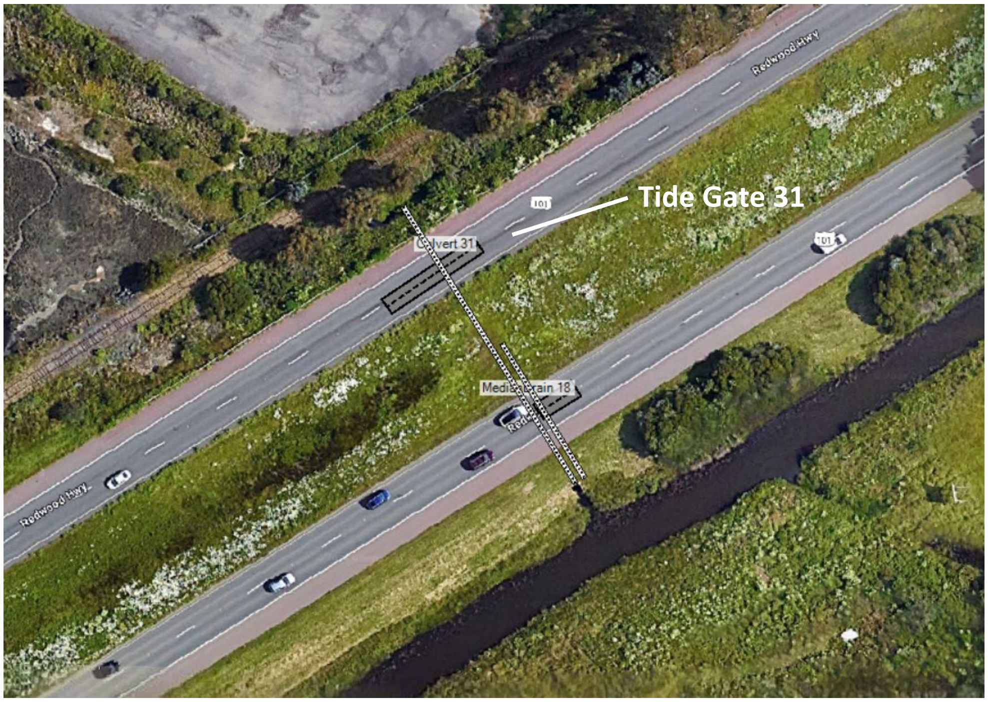
Culverts & Tide Gates Adjacent to Hwy 101

Of the 11 remaining culverts and tide gates that do not directly intersect Highway 101 (Tables 18 - 21; Figures C-1 – C-11), Culverts 21, 29, and 30 exhibit the highest frequency, depth, and duration of overtopping. These structures not only exceed the maximum allowable headwater/tailwater elevations but also frequently overtop the adjacent road centerlines. For example, during nearly all coastal 2-year event conditions and sea level rise (SLR) scenarios, the road centerlines near Culverts 29 and 30 are overtopped (Figures C-9 & C-10). At 3.12 feet of SLR, the adjacent roads remain inundated for almost the entire 100 hr simulation period. Both culverts are situated at lower elevation crossings along the Highway 101 ditch that drains the southwestern portion of the highway between postmiles approximately 80.8 and 82.68 (Figure 6 & 16).

In contrast, Culverts 14 through 16, located near the northern boundary of the project area at the Highway 101-Samoa Interchange (Figure 6), show the lowest frequency, depth, and duration of overtopping for both road centerlines and maximum allowable headwater/tailwater elevations (Figures C-1 – C-3).

The hydraulic performance differences between these structures are primarily driven by: (i) their proximity to the Bay (i.e., tailwater controls) and the presence of internal high ground features like protective levees, (ii) the size and presence of inflowing tributaries, (iii) the dimensions of each structure, and (iv) the interaction between fluvial peak flood timing and tidal dynamics. For instance, Culvert 14 is located far from the Bay, lacks a direct connection to a tributary, and is upstream of several high ground features and culverts that dampen tidal influences (Figure 6). As a result, it remains relatively unaffected by sea level rise until coastal 2-year storms occur in conjunction with 3.12 feet of SLR.

In general, culverts adequately sized to handle flow demand, are characterized by nearly equal headwater and tailwater elevations as the culvert is not acting as a significant hydraulic control. Conversely, if the culvert is undersized or partially obstructed, it can create a restriction that results in higher headwater elevations compared to tailwater elevations, especially during high flow events. That being said, culvert hydraulics in the Project Area are complicated by many factors. For instance, culverts with tidally influenced tailwaters that drain areas surrounded by levees or high ground can exhibit significant differences between headwater and tailwater elevations. These features can impede flow, resulting in backwatering and elevated headwater levels. In other cases, structures draining areas protected by high ground features are characterized by much greater tailwater elevations. Tide Gate 31 serves as an example, as it drains a ditch located between the elevated areas of the Humboldt Bay Trail and southbound Highway 101 (Figure 17). Here, tailwater elevations consistently exceed headwater levels, primarily because the drainage ditch remains dry in all simulations except the 3.12 feet SLR model runs. Beyond this point, tidal conditions overtop Highway 101, filling the drainage ditch and raising headwater elevations.



Additionally, culverts situated in channels with minimal gradients are more susceptible to tidal influences than to gravity-driven flow. As a result, when tailwater elevations increase rapidly during high tides, the tailwater can easily surpass the headwater levels. These dynamics highlight the complex interactions between structure sizing, tidal conditions, hydrologic connectivity and surrounding topographic features in the Project Area.

Table 16. Performance metrics for culverts and tide gates that cross Hwy 101 during Event Condition I scenarios.

	Event Condition					I				
	Scenario #	1	2	3	4	5	6	7	8	9
	Tidal Boundary Condition	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
South Hwy 101 TG	Drainage Window (hrs)	62	55	50.8	76.5	71	60.2	84	79.8	70
	Volume (ac-ft)	63.61	82.41	267.43	314.96	339.55	442.96	561.31	570.29	622.85
	Max Headwater (ft)	6.47	6.73	8.66	8.3	8.53	9.9	9.88	10.07	11.25
	Max Tailwater (ft)	8.61	9.42	11.63	8.62	9.43	11.63	8.86	9.44	11.64
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)			3.75			3.75			3.75
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Gannon Culvert	Drainage Window (hrs)	57	62.2	83.5	67.5	63.5	67.2	63.2	58.2	56.5
	Volume (ac-ft)	-62.32	-63.02	306.8	-112.87	-106.04	119.78	-160.15	-165.85	-58.99
	Max Headwater (ft)	6.9	6.93	11.34	8.35	8.49	11.39	9.21	9.27	11.51
	Max Tailwater (ft)	7.49	7.61	9.88	9.69	9.78	11.01	10.96	11.03	12.01
	Duration HW Exceed Max Allowed (hrs)			2.5			2.75			3.25
	Duration TW Exceed Max Allowed (hrs)						3	4.25	4.75	8
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									1
Culvert 25	Drainage Window (hrs)	48.5	56.2	71.5	45.8	42	57	42.5	40	48.2
	Volume (ac-ft)	-0.86	0.22	181.43	-64.94	-80.69	0.76	-255.52	-266.52	-232.46
	Max Headwater (ft)	7.52	7.64	11.19	9.56	9.61	11.33	10.1	10.13	11.52
	Max Tailwater (ft)	7.5	7.63	9.88	9.69	9.78	11	10.95	11.02	12
	Duration HW Exceed Max Allowed (hrs)									0.5
	Duration TW Exceed Max Allowed (hrs)									4.75
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 28	Drainage Window (hrs)	90.8	83.8	63.8	96.8	88.2	68.8	98	93.8	77.2
	Volume (ac-ft)	750	712.98	396.02	1242.55	1197.52	937.66	1923.51	1882.4	1628.8
	Max Headwater (ft)	8.59	8.85	9.23	8.61	8.88	9.89	9.8	10.02	11.21
	Max Tailwater (ft)	8.61	9.4	11.59	8.62	9.4	11.6	8.64	9.42	11.61
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)			1.5			1.5			2
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Tide Gate 31	Drainage Window (hrs)			63.8			65.2			60.2
	Volume (ac-ft)			67.52			68.34			67.07
	Max Headwater (ft)			9.05			9.06			9.07
	Max Tailwater (ft)	4.16	4.97	8.27	4.51	5.29	8.55	5.24	6.04	9.29
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)									4.5
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									

Table 17. Performance metrics for culverts and tide gates that cross Hwy 101 during Event Condition II scenarios.

	Event Condition	II								
	Scenario #	10	11	12	13	14	15	16	17	18
	Tidal Boundary Condition	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
South Hwy 101 TG	Drainage Window (hrs)	55	50.8	51	69	64.8	60.2	74.8	72.2	65.5
	Volume (ac-ft)	84.96	122.32	381.9	359.4	386.35	540.2	565.34	585.77	631.8
	Max Headwater (ft)	6.66	7.01	10.18	8.65	9.05	11.37	10.06	10.31	12.09
	Max Tailwater (ft)	9.5	10.29	12.48	9.51	10.28	12.48	9.59	10.3	12.5
	Duration HW Exceed Max Allowed (hrs)									8
	Duration TW Exceed Max Allowed (hrs)			13.25			13.5			14.25
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)			2			2.25			2.25
Gannon Culvert	Drainage Window (hrs)	47.8	75	69	51.8	63.8	56	51.8	54.8	50.2
	Volume (ac-ft)	-57.57	178.44	239.02	-104.7	37.46	105.07	-153.2	-70.94	-26.03
	Max Headwater (ft)	6.94	8.37	12.41	8.5	9.46	12.43	9.24	9.88	12.52
	Max Tailwater (ft)	7.65	8.2	11.58	9.87	10.07	12.1	11.06	11.28	12.6
	Duration HW Exceed Max Allowed (hrs)			11.75			11.75			12.5
	Duration TW Exceed Max Allowed (hrs)			7.5			11	5	5.75	15.5
	Duration HW Exceed Road Top (hrs)			3.5			3.75			4
	Duration TW Exceed Road Top (hrs)						0.75			4
Culvert 25	Drainage Window (hrs)	51.5	49.5	61.2	43.8	47.5	50.5	38.2	42.5	46.8
	Volume (ac-ft)	-0.55	-1.76	262.07	-86.74	-88.61	89.37	-271.8	-271.8	-117.2
	Max Headwater (ft)	7.67	8.21	12.38	9.66	9.78	12.41	10.14	10.24	12.5
	Max Tailwater (ft)	7.66	8.2	11.58	9.87	10.07	12.1	11.05	11.27	12.6
	Duration HW Exceed Max Allowed (hrs)			6			6.25			6.75
	Duration TW Exceed Max Allowed (hrs)			2.75			6		2.75	10.25
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									0.75
Culvert 28	Drainage Window (hrs)	80	73	56	82.2	77.5	65	86.8	82.8	70.2
	Volume (ac-ft)	691.22	617.51	383.99	1188.2	1097.9	1098.2	1836.5	1767.6	1494.7
	Max Headwater (ft)	8.86	8.99	10.17	8.97	9.17	11.36	9.99	10.25	12.07
	Max Tailwater (ft)	9.49	10.25	12.44	9.49	10.25	12.45	9.51	10.26	12.47
	Duration HW Exceed Max Allowed (hrs)									8.25
	Duration TW Exceed Max Allowed (hrs)			10			10			10.25
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Tide Gate 31	Drainage Window (hrs)		0.5	78.5		0.5	76.5		0.5	75.8
	Volume (ac-ft)		0.03	221.59		0.03	200.91		0.03	178.48
	Max Headwater (ft)		2.83	11.54		2.87	11.69		2.92	11.88
	Max Tailwater (ft)	5.19	6.24	11.54	5.51	6.53	11.7	6.14	7.18	11.88
	Duration HW Exceed Max Allowed (hrs)			15.25			18.5			23.25
	Duration TW Exceed Max Allowed (hrs)			39.25			51.75			57.5
	Duration HW Exceed Road Top (hrs)						1.25			2
	Duration TW Exceed Road Top (hrs)			11			14.25			19

Table 18. Performance metrics for culverts and tide gates adjacent to Hwy 101 during Event Condition I scenarios.

	Event Condition	**I**								
	Scenario #	1	2	3	4	5	6	7	8	9
	Tidal Boundary Condition	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
Culvert 14	Drainage Window (hrs)			44.5			47.2	46.5	40	49.5
	Volume (ac-ft)			0.52			0.17	0.56	0.54	0.23
	Max Headwater (ft)	8.4	8.4	11.34	8.4	8.4	11.4	9.24	9.16	11.51
	Max Tailwater (ft)	8.2	8.2	11.36	8.2	8.2	11.38	9.23	9.16	11.51
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)									
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 15	Drainage Window (hrs)			53	66.8	2.8	48.8	57.2	66.2	51
	Volume (ac-ft)			1.06	0.11	-0.23	1.26	0.53	0.7	1.11
	Max Headwater (ft)	8	8	11.35	8.27	8.33	11.39	9.04	9.16	11.52
	Max Tailwater (ft)	7.9	7.9	11.35	8.26	8.34	11.4	9.04	9.15	11.51
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)									
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 16	Drainage Window (hrs)	100.5	100.5	100.2	100.5	100.5	100.5	100.5	100.5	100.5
	Volume (ac-ft)	115.87	115.87	115.53	239.26	239.24	235.67	361.61	361.14	357.64
	Max Headwater (ft)	10.04	10.04	10.41	12.31	12.31	12.67	13.42	13.42	13.46
	Max Tailwater (ft)	9.65	9.65	10.27	11.51	11.51	12.15	12.77	12.78	12.84
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)									
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 17	Drainage Window (hrs)	100.5	100.5	99.8	100.5	100.5	100.2	100.5	100.5	100.2
	Volume (ac-ft)	115.9	115.95	115.11	239.24	239.61	236.59	373.09	371.76	361.77
	Max Headwater (ft)	9.54	9.54	10.22	11.39	11.4	12.11	12.73	12.75	12.82
	Max Tailwater (ft)	9.34	9.34	10.14	10.99	11.04	11.85	12.4	12.42	12.55
	Duration HW Exceed Max Allowed (hrs)							2.25	2.75	3.75
	Duration TW Exceed Max Allowed (hrs)						0.75	6.5	6.5	7.5
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 18	Drainage Window (hrs)			47.2	16.5	21.2	46.8	30.2	41.8	56.2
	Volume (ac-ft)			-0.7		0.3	-0.33	1.29	0.58	4.04
	Max Headwater (ft)	6.9	6.9	11.35	8.26	8.34	11.39	9.03	9.15	11.51
	Max Tailwater (ft)	6.57	6.57	11.35	8.26	8.32	11.4	9.02	9.14	11.51
	Duration HW Exceed Max Allowed (hrs)			4.25			4.75			6
	Duration TW Exceed Max Allowed (hrs)			1.5			1.75			2.25
	Duration HW Exceed Road Top (hrs)									0.25
	Duration TW Exceed Road Top (hrs)									0.25

Table 19. Performance metrics for all culverts and tide gates adjacent to Hwy 101 during Event Condition II scenarios.

	Event Condition	II								
	Scenario #	10	11	12	13	14	15	16	17	18
	Tidal Boundary Condition	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
Culvert 14	Drainage Window (hrs)			65.5		46.2	59.5	48.8	75.2	59
	Volume (ac-ft)			0.35		0.42	-1.05	0.45	0.96	-0.15
	Max Headwater (ft)	8.4	8.4	12.38	8.4	9.43	12.39	9.16	9.83	12.52
	Max Tailwater (ft)	8.2	8.2	12.39	8.2	9.41	12.41	9.16	9.84	12.52
	Duration HW Exceed Max Allowed (hrs)			3.75			3.25			4.25
	Duration TW Exceed Max Allowed (hrs)			1			1			2.25
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 15	Drainage Window (hrs)		17	67.2	50.2	26.8	60.5	52.2	66.2	66.5
	Volume (ac-ft)		-0.13	2.74	-0.11	-0.13	-0.87	0.49	0.63	-0.66
	Max Headwater (ft)	8	8.45	12.39	8.34	9.42	12.41	9.13	9.83	12.53
	Max Tailwater (ft)	7.9	8.45	12.39	8.34	9.41	12.41	9.12	9.84	12.53
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)			5.75			5.75			6.5
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 16	Drainage Window (hrs)	96.2	96.2	95.2	96.2	96.2	95	96.2	96.2	96
	Volume (ac-ft)	114.19	114.19	112.84	242.49	242.05	229.51	359.17	358.06	351.58
	Max Headwater (ft)	10.04	10.04	11.67	12.38	12.4	12.89	13.43	13.44	13.48
	Max Tailwater (ft)	9.65	9.66	11.61	11.59	11.7	12.54	12.78	12.8	12.93
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)									
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 17	Drainage Window (hrs)	96.2	96.2	92.8	96.2	96.2	94.2	96.2	96.2	95.2
	Volume (ac-ft)	114.22	114.21	111.79	242.9	241.99	229.57	369.18	366.05	351.23
	Max Headwater (ft)	9.54	9.55	11.6	11.49	11.62	12.52	12.75	12.77	12.91
	Max Tailwater (ft)	9.34	9.35	11.6	11.13	11.27	12.35	12.43	12.46	12.74
	Duration HW Exceed Max Allowed (hrs)							2.75	3.25	4.75
	Duration TW Exceed Max Allowed (hrs)						4.5	6.5	7	10.25
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 18	Drainage Window (hrs)		29.5	60.8	40	39.5	59.8	59.2	59.2	71
	Volume (ac-ft)		-0.24	3.63	-0.34	1.27	-0.14	0.38	1.16	8.96
	Max Headwater (ft)	6.9	8.44	12.4	8.34	9.42	12.41	9.12	9.83	12.53
	Max Tailwater (ft)	6.57	8.44	12.4	8.32	9.41	12.42	9.1	9.83	12.51
	Duration HW Exceed Max Allowed (hrs)			16.5			17.5			18.75
	Duration TW Exceed Max Allowed (hrs)			9.75			10.25			11
	Duration HW Exceed Road Top (hrs)			6			6.25			6.5
	Duration TW Exceed Road Top (hrs)			6			6.25			6.5

Table 20. Performance metrics for all culverts and tide gates adjacent to Hwy 101 during Event Condition I scenarios.

	Event Condition	I								
	Scenario #	1	2	3	4	5	6	7	8	9
	Tidal Boundary Condition	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
Culvert 19	Drainage Window (hrs)	17	16.8	49.2	29.5	29.8	50.8	34.8	36.8	54
	Volume (ac-ft)	0.35	0.34	0.78	0.53	0.6	0.97	-2.42	-1.33	0.35
	Max Headwater (ft)	6.81	6.84	11.35	8.25	8.33	11.39	9.02	9.14	11.51
	Max Tailwater (ft)	6.8	6.83	11.35	8.25	8.33	11.4	9.03	9.14	11.5
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)			0.75			0.75			0.75
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 20	Drainage Window (hrs)	17	16.8	49.2	29.5	29.8	50.8	34.8	36.8	54
	Volume (ac-ft)	0.35	0.34	0.78	0.53	0.6	0.97	-2.42	-1.33	0.35
	Max Headwater (ft)	6.81	6.84	11.35	8.25	8.33	11.39	9.02	9.14	11.51
	Max Tailwater (ft)	6.8	6.83	11.35	8.25	8.33	11.4	9.03	9.14	11.5
	Duration HW Exceed Max Allowed (hrs)			1.25			1.75			2
	Duration TW Exceed Max Allowed (hrs)			3.5			4.25			4.75
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									
Culvert 21	Drainage Window (hrs)	100.5	100.5	97.2	100.5	100.5	98	100.5	100.5	99.2
	Volume (ac-ft)	115.98	115.4	114.91	241.9	242.02	235.92	390.1	389.82	369.99
	Max Headwater (ft)	8.38	8.38	9.98	10.42	10.53	11.69	12.23	12.25	12.42
	Max Tailwater (ft)	8.2	8.2	9.92	10.1	10.21	11.39	11.69	11.74	12.15
	Duration HW Exceed Max Allowed (hrs)			2.25	7.5	8	14.5	19.75	20.25	27.25
	Duration TW Exceed Max Allowed (hrs)						3.25	7.5	7.5	8.75
	Duration HW Exceed Road Top (hrs)						2.25	6.75	7.25	8.25
	Duration TW Exceed Road Top (hrs)							3.75	4.25	6.25
Culvert 29	Drainage Window (hrs)	53	62.2	46.5	48.2	63.5	52.5	67.2	47.8	52.5
	Volume (ac-ft)	44.08	81.68	-2.42	44.66	67.35	1.78	80.74	5.86	2.4
	Max Headwater (ft)	4.16	5.02	8.28	4.51	5.32	8.56	5.28	6.07	9.29
	Max Tailwater (ft)	4.16	5.01	8.28	4.51	5.31	8.55	5.27	6.07	9.29
	Duration HW Exceed Max Allowed (hrs)			100.25			100.25		66.5	100.25
	Duration TW Exceed Max Allowed (hrs)			100.25			100.25		65.75	100.25
	Duration HW Exceed Road Top (hrs)			95.5			95.5			95.5
	Duration TW Exceed Road Top (hrs)			95.5			95.5			95.5
Culvert 30	Drainage Window (hrs)	58	82.5	83.5	51	79.8	77.8	78.2	79.8	72
	Volume (ac-ft)	158.85	381.06	116	186.44	365.69	84.39	332.12	395.12	72.55
	Max Headwater (ft)	4.16	4.98	8.27	4.5	5.3	8.55	5.25	6.06	9.29
	Max Tailwater (ft)	4.16	4.98	8.27	4.51	5.28	8.55	5.24	6.05	9.28
	Duration HW Exceed Max Allowed (hrs)		6	100.25		49.5	100.25	39.75	75.5	100.25
	Duration TW Exceed Max Allowed (hrs)		4.5	100.25		41.25	100.25	36.25	73.5	100.25
	Duration HW Exceed Road Top (hrs)			97.75			97.75			98
	Duration TW Exceed Road Top (hrs)			97.75			97.75			97.75
Tide Gate - Hwy 101 Ditch	Drainage Window (hrs)	39.2	50.8	66	53.5	46	67.8	60.5	51.2	70.5
	Volume (ac-ft)	217.73	518.67	1793.95	413.86	567.92	1814.63	691.46	841.22	1902.2
	Max Headwater (ft)	4.16	4.97	8.27	4.78	5.28	8.55	5.3	6.04	9.26
	Max Tailwater (ft)	8.54	9.32	10.83	8.61	9.39	10.86	8.61	9.41	10.9
	Duration HW Exceed Max Allowed (hrs)									
	Duration TW Exceed Max Allowed (hrs)			2			2.25			2.75
	Duration HW Exceed Road Top (hrs)									
	Duration TW Exceed Road Top (hrs)									

Table 21, Performance metrics for all culverts and tide gates adjacent t0 Hwy 101 during Event Condition II scenarios.

	Event Condition					**II**				
	Scenario #	10	11	12	13	14	15	16	17	18
	Tidal Boundary Condition	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
Culvert 19	Drainage Window (hrs)	50.8	42.2	64.5	31.5	49.8	60.5	12.2	61.2	74
	Volume (ac-ft)	0.27	0.58	1.25	0.27	1.22	-4.68	-0.56	2.77	8.82
	Max Headwater (ft)	6.8	8.45	12.41	8.33	9.41	12.42	9.1	9.83	12.5
	Max Tailwater (ft)	6.83	8.44	12.42	8.33	9.41	12.43	9.12	9.83	12.49
	Duration HW Exceed Max Allowed (hrs)			4.5			4.5			5.25
	Duration TW Exceed Max Allowed (hrs)			8.25			8.5			8.5
	Duration HW Exceed Road Top (hrs)									0.5
	Duration TW Exceed Road Top (hrs)									0.5
Culvert 20	Drainage Window (hrs)	50.8	42.2	64.5	31.5	49.8	60.5	12.2	61.2	74
	Volume (ac-ft)	0.27	0.58	1.25	0.27	1.22	-4.68	-0.56	2.77	8.82
	Max Headwater (ft)	6.8	8.45	12.41	8.33	9.41	12.42	9.1	9.83	12.5
	Max Tailwater (ft)	6.83	8.44	12.42	8.33	9.41	12.43	9.12	9.83	12.49
	Duration HW Exceed Max Allowed (hrs)			9.5			9.75			10
	Duration TW Exceed Max Allowed (hrs)			15.5			15.75			16.5
	Duration HW Exceed Road Top (hrs)			5			5.25			5.5
	Duration TW Exceed Road Top (hrs)			4.75			5.25			5.25
Culvert 21	Drainage Window (hrs)	96.2	96.2	89.8	96.2	96.2	91.5	96.2	96.2	94.8
	Volume (ac-ft)	114.55	114.11	112.87	244.94	245.1	230.38	386.81	382.29	348.28
	Max Headwater (ft)	8.39	8.58	11.6	10.66	10.89	12.29	12.26	12.3	12.67
	Max Tailwater (ft)	8.21	8.48	11.59	10.33	10.56	12.15	11.75	11.84	12.62
	Duration HW Exceed Max Allowed (hrs)			16	8.75	9.25	22.5	19.5	21.25	36
	Duration TW Exceed Max Allowed (hrs)			5.25			9.5	7.75	8.25	15.5
	Duration HW Exceed Road Top (hrs)			1.25			6.25	7.25	7.5	12.25
	Duration TW Exceed Road Top (hrs)			1			5.75	4.25	5	10.25
Culvert 29	Drainage Window (hrs)	57.8	55.8	47.8	52	47.5	49	47.2	43.8	44.8
	Volume (ac-ft)	57.99	-14.64	-2.91	29.65	-29.91	3.65	0.9	-37.06	-6.08
	Max Headwater (ft)	5.25	6.25	11.54	5.55	6.55	11.69	6.17	7.2	11.87
	Max Tailwater (ft)	5.24	6.26	11.53	5.54	6.54	11.7	6.17	7.2	11.88
	Duration HW Exceed Max Allowed (hrs)		56	96	12.25	58	96	60.75	75.5	96
	Duration TW Exceed Max Allowed (hrs)		56.25	96	11.75	58.25	96	60.5	77	96
	Duration HW Exceed Road Top (hrs)			96		2.25	96		39.5	96
	Duration TW Exceed Road Top (hrs)			96		2.5	96		39.75	96
Culvert 30	Drainage Window (hrs)	73.8	76.5	57	72	78	49.8	74.5	80.8	54.8
	Volume (ac-ft)	374.67	403.96	30.22	346.43	325.19	9.5	355.92	248.2	22.2
	Max Headwater (ft)	5.21	6.25	11.54	5.53	6.54	11.7	6.16	7.2	11.88
	Max Tailwater (ft)	5.19	6.24	11.54	5.51	6.53	11.7	6.14	7.19	11.88
	Duration HW Exceed Max Allowed (hrs)	22.25	77	96	55.25	79.25	96	72.25	79.5	96
	Duration TW Exceed Max Allowed (hrs)	17.75	74.25	96	48.25	79	96	71.75	79.25	96
	Duration HW Exceed Road Top (hrs)		2.25	96		14.75	96		61.5	96
	Duration TW Exceed Road Top (hrs)		1	96		10.75	96		57.75	96
Tide Gate - Hwy 101 Ditch	Drainage Window (hrs)	36.8	37.8	57.8	30.5	36.2	58.5	33.2	38.2	61.2
	Volume (ac-ft)	396.4	651.41	1682.5	407.24	678.45	1675.4	589.53	858.12	1700.3
	Max Headwater (ft)	5.19	6.22	11.55	5.51	6.53	11.7	6.13	7.17	11.88
	Max Tailwater (ft)	9.44	10.02	11.56	9.5	10.08	11.71	9.51	10.11	11.89
	Duration HW Exceed Max Allowed (hrs)			6			7.75			12.75
	Duration TW Exceed Max Allowed (hrs)			11.5			12.5			13.75
	Duration HW Exceed Road Top (hrs)						0.75			1.75
	Duration TW Exceed Road Top (hrs)						1			1.5

Bridges

The bridge performance plots (Figure D-1 – D6) indicates that maximum headwater elevations for northbound bridges often exceed tailwater elevations – particularly with increasing sea level rise. This suggests reduced drainage efficiency and heightened backwater effects due to elevated tailwater levels during high tides as sea levels rise. The direct hydraulic connection of the Hwy 101 bridges to the Bay and concomitant strong tidal influence is also evidenced by the pronounced and consistent inverse relationship between drainage window and sea level rise. For example, during the MMMW & Q100 event, a 3.12 ft increase in sea level led to an approximately 20% reduction in the drainage window at most Highway 101 bridges due to higher tailwater conditions. Among the structures evaluated, the northbound Gannon Slough Bridge appears the most vulnerable to the effects of rising sea levels, while the southbound and northbound Eureka Slough Bridges exhibit the highest resilience. The following sections provide a more detailed assessment of each bridge.

For the purposes of this memo, the terms "lower chord" and "soffit" are used interchangeably to refer to the lowest structural element of a bridge relevant for hydraulic analysis. This element represents the lowest elevation of any major component member of the bridge superstructure, and is critical for assessing potential hydraulic impacts, such as clearance during high water events or risk of submergence under extreme conditions.

Gannon Slough Bridges

Tables 24 & 25 and the bridge performance plot (Figure D-1) indicate that the lower chord of both Gannon Slough Bridges are engaged by floodwaters during 2-year coastal storms combined with sea level rise. Under existing conditions (i.e. without SLR), the northbound bridge soffit is engaged under the combined C2 Q100 event. The upstream soffit is almost engaged under the relatively moderate C2 & Q2 event scenario, becomes moderately submerged in most 0.82 ft SLR scenarios, and is fully submerged across all 3.12 ft SLR scenarios. The low elevation of the bridge deck itself is inundated during all 3.12 ft SLR events, with flood depths ranging from 0.04 to 1.24 ft and durations lasting from 0.75 to 9.75 hours. Flood depths over the bridge deck exceeded the 4-inch impact threshold for 0.25 to 7.25 hours in most 3.12 ft SLR scenarios. Figure D-1 also highlights inadequate freeboard at this bridge, as the maximum headwater/tailwater elevation (1 ft lower than the soffit) is exceeded or very nearly exceeded over nearly all modeled events scenarios – even those without SLR.

Significant soffit engagement and submergence suggest the bridge will likely experience reduced flow conveyance, increased backwater effects, and elevated hydraulic pressures with increasing SLR. Elevated hydrodynamic pressures—particularly when the water reaches the soffit or overtops the bridge deck during storm surges or river flooding—can lead to structural fatigue and compromise the bridge's long-term integrity. Moreover, the lack of adequate freeboard makes the bridge vulnerable to debris flows and blockages (e.g. large woody debris, vegetation, or man-made materials lodging against the soffit or piers). These obstructions not only threaten damage to the bridge’s structural components but also increase hydrodynamic drag, intensifying the forces exerted on the structure and raising the risk of damage and failure.

In a tidal environment with direct exposure to coastal storms and wind waves, insufficient freeboard also increases the likelihood of wave loading during storm events and high tides. Waves repeatedly striking the underside of the bridge deck accelerate wear and tear on both the deck and its foundation. Additionally, as the bridge soffit becomes submerged, the risk of scour around piers and abutments is heightened due to increased flow velocities, turbulence, and shear stress. This combination of factors can compromise the foundation of the bridge, further exacerbating the risk of structural failure.

Tables 24 and 25, along with Figure D-2, indicate that under MMMW scenarios, the lower chord and deck of the southbound Gannon Slough Bridge remain unaffected by flooding until sea level rise reaches 3.12 feet. However, during 2-year coastal storms, the bridge's lower chord and deck become increasingly vulnerable to SLR, as evidenced by soffit submergence at 0.82 feet of SLR and significant deck submergence at 3.12 feet of SLR. Similar to the northbound bridge, limited freeboard is observed during 2-year coastal storms even without SLR, with maximum allowable headwater/tailwater elevations exceeded during all C2 events. At 3.12 feet of SLR, the low elevation of the bridge deck is inundated during most events, with flood depths ranging from 0.02 to 0.94 feet and durations lasting 0.5 to 6.75 hours. Notably, flood depths over the bridge deck exceeded the 4-inch impact threshold for approximately 4.25 hours during all 2-year coastal storms with 3.12 feet of SLR.

Although the southbound Gannon Slough Bridge appears to be moderately less vulnerable to coastal and fluvial storms compared to the northbound bridge, the limited freeboard and the risk of soffit submergence during certain scenarios—such as those coastal storm with 0.82 ft of sea level rise and above—indicate that the bridge is still at significant risk for structural impairment and may face frequent closures in the future.

Figure 27 presents a hydrograph depicting tidally influenced headwater elevations relative to the soffit and bridge deck of the northbound Gannon Slough Bridge during the most extreme existing condition scenario (2-year coastal storm & Q100), as well as two sea level rise scenarios (0.82 ft and 3.12 ft). It is evident that overtopping of the bridge deck occurs near peak high tide throughout much of the C2 & Q100 storm with 3.12 ft of SLR. Additionally, there is inadequate freeboard during higher high tides associated with 2-year coastal storm events, indicating a heightened risk of debris blockage, structural fatigue, and compromised hydraulic performance under future sea level rise conditions.

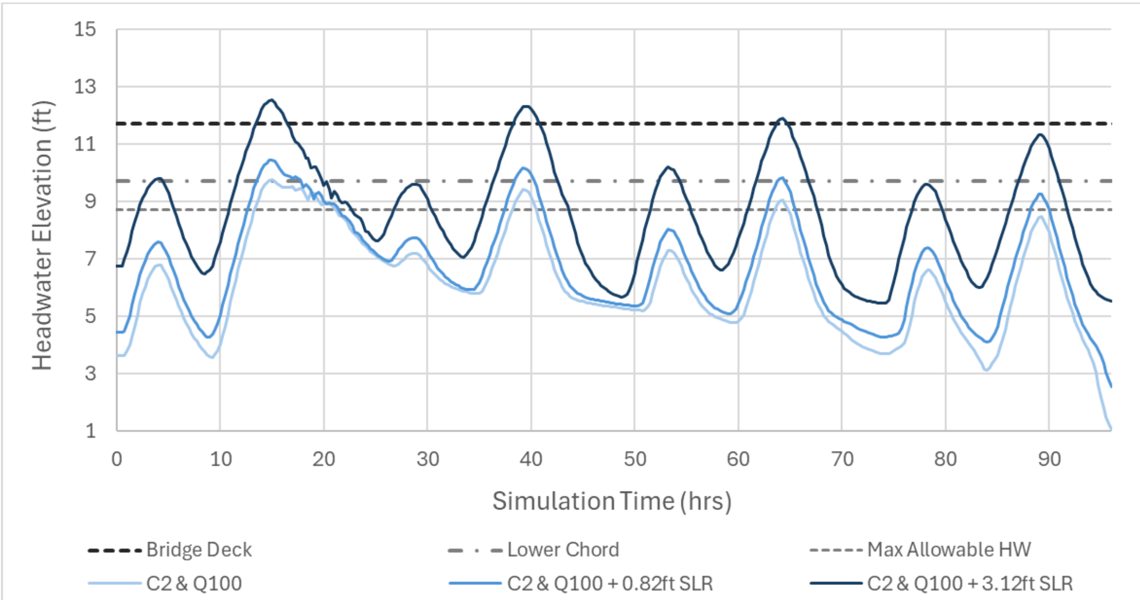


Figure 27. Headwater elevations over a range of event conditions relative to the lower chord and deck of the northbound Gannon Slough Bridge.

Figures 28 and 29 below illustrate flow velocities over a range of event conditions involving a 2-yr coastal storm and varying fluvial flood magnitudes for the northbound and southbound Gannon Slough Bridges, respectively. Maximum velocities for both bridges reach approximately 5-6 ft/s and occur during the modeled C2 & Q100 sea level rise scenarios. Although a comprehensive bridge and pier scour analysis is beyond the scope of this study, these velocity plots can serve as reference points when qualitatively assessing scouring flows in the context of future storm and sea level rise scenarios.

It is important to note, however, that these bridge velocities represent general flow conditions and do not capture the higher localized velocities associated with specific bridge hydraulics, such as flow acceleration around bridge piers (e.g. horseshoe vortices), through narrow openings, under low-clearance structures or eddies forming downstream of abutments. These localized accelerated velocities are important considerations, as they may significantly exceed the general flow velocities reported here, amplifying erosion, sediment transport, and the potential for structural damages to bridges.

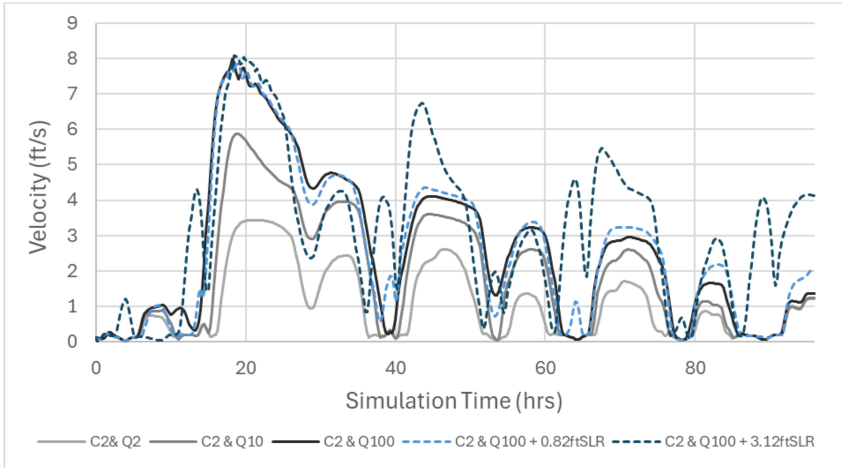


Figure 28. Flow velocities proximal to the northbound Gannon Slough Bridge over a suite of event conditions.

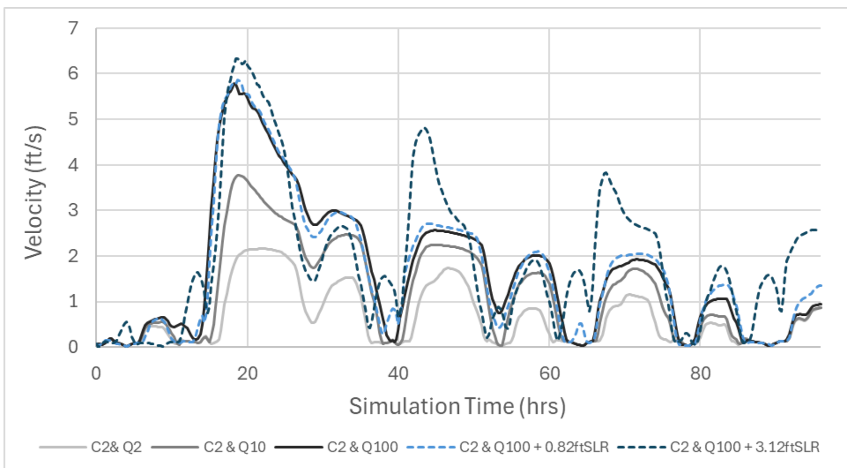


Figure 29. Flow velocities proximal to the southbound Gannon Slough Bridge over a suite of event conditions.

Jacoby Creek Bridges

Tables 24 & 25 and Figures 30, D-3 & D-4 indicate that the decks of the Jacoby Creek Bridges (northbound and southbound) are not overtopped in any of the modeled scenarios. However, during the C2 & Q100 event with 3.12 ft of SLR, headwater elevations come within roughly 2 inches of the deck's lowest point. The soffits of both bridges are submerged in all scenarios involving a 3.12 ft of sea level rise.

Due to its closer proximity to the Bay and lower soffit elevation, the southbound bridge's lower chords are submerged during all modeled 2-year coastal storm events with 0.82 feet of SLR. Overall, the Jacoby Creek Bridges generally maintain more freeboard across the modeled events compared to the Gannon Slough Bridges. Nonetheless, under current conditions (without SLR), the maximum allowable headwater/tailwater elevations are exceeded at the southbound bridge during the C2 & Q2 and higher events. With 0.82 ft of SLR, the maximum allowable headwater/tailwater elevations are exceeded at the southbound bridge across all modeled event scenarios, indicating limited capacity to accommodate future sea level rise.

While the northbound bridge provides slightly more freeboard, it also exceeds the maximum allowable headwater/tailwater elevation during numerous coastal storm events with 0.82 ft of SLR, further emphasizing the limited resiliency of both bridges to future sea level rise.

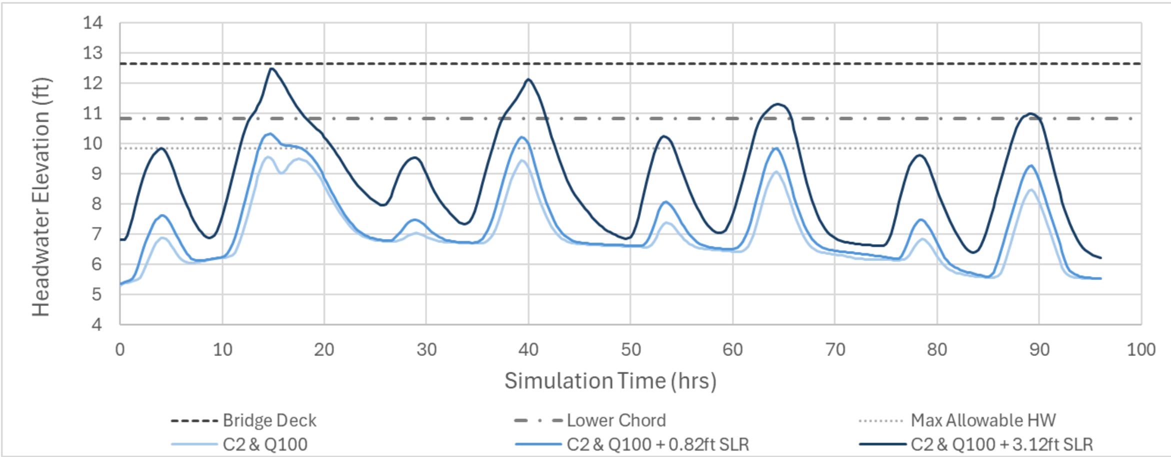


Figure 30. Headwater elevations over a range of event conditions relative to the lower chord and deck of the northbound Jacoby Creek Bridge.

Figures 31 and 32 below illustrate flow velocities over select event conditions involving a 2-yr coastal storm and varying fluvial flood magnitudes for the northbound and southbound Jacoby Creek Bridges, respectively. Maximum velocities for both bridges reach approximately 5-6.5 ft/s and occur during the modeled sea level rise scenarios. Although a comprehensive bridge and pier scour analysis is beyond the scope of this study, these velocity plots offer qualitative insights for evaluating potential scouring risks under future storm and sea level rise conditions.

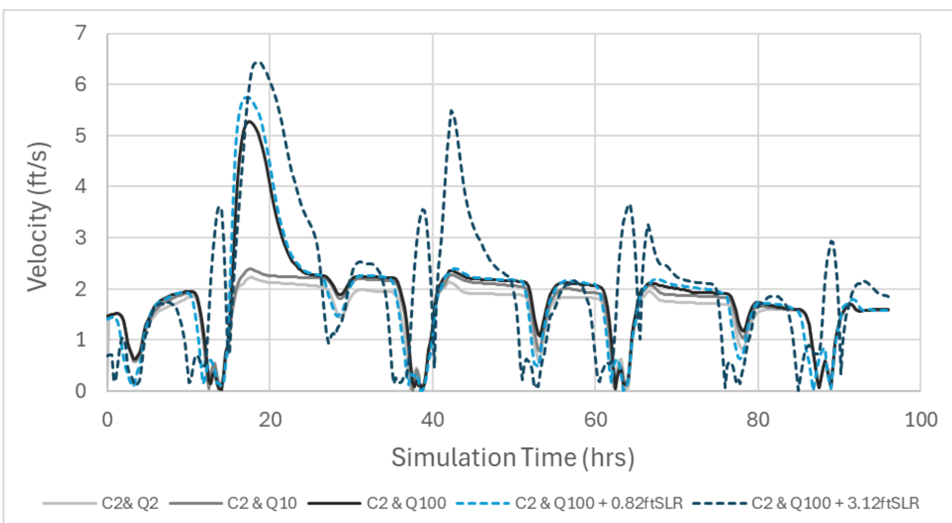


Figure 31. Flow velocities proximal to the northbound Jacoby Creek Bridge over a suite of event conditions.

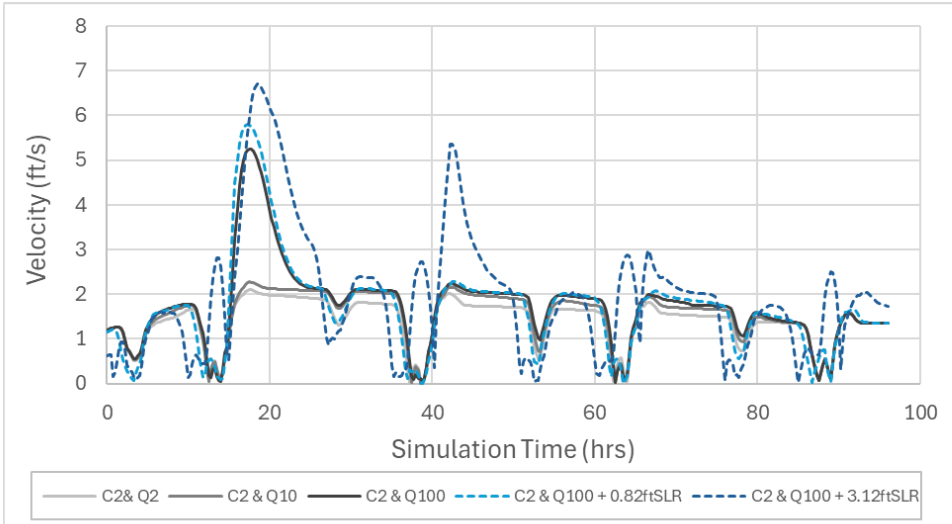


Figure 32. Flow velocities proximal to the southbound Jacoby Creek Bridge over a suite of event conditions.

Eureka Slough Bridges

The lower chords and decks of the southbound and northbound Eureka Slough Bridges are unaffected by any currently modeled flood events (Tables 24 & 25 and Figures D-5 & D-6). As shown in Figures 33 & 39, there is significant freeboard across all scenarios, indicating that the Eureka Slough Bridges are unlikely to be impacted except under the most extreme combinations of sea level rise and coastal or fluvial storm conditions.

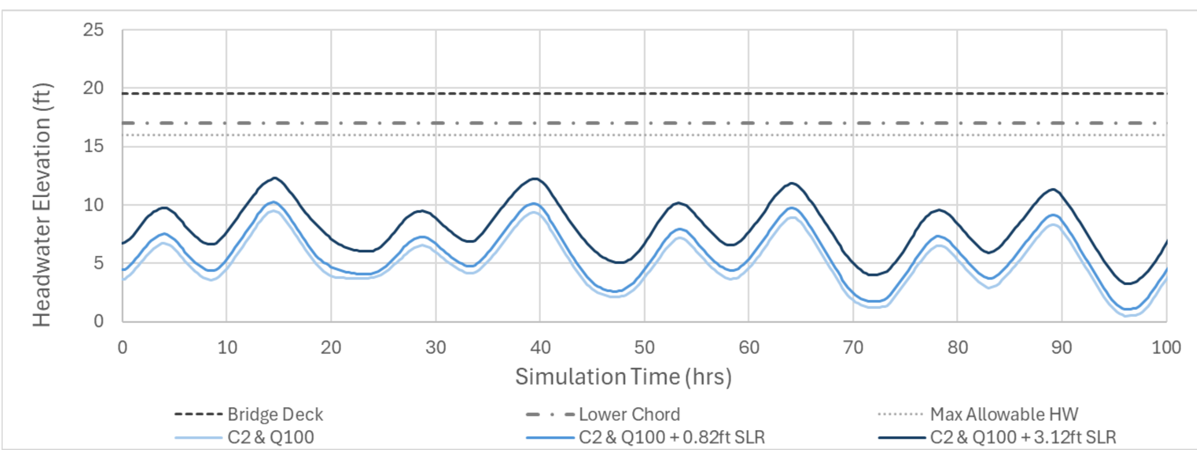


Figure 33. Headwater elevations over a range of event conditions relative to the lower chord and deck of the northbound Eureka Slough Bridge.

Figures 34 and 35 below illustrate flow velocities over select event conditions involving a 2-yr coastal storm and varying fluvial flood magnitudes for the northbound and southbound Eureka Slough Bridges, respectively. Maximum velocities for both bridges are relatively low (~ 2 – 3 ft/s) and occur during the modeled sea level rise scenarios. Although a detailed scour analysis for the bridges and piers is beyond the scope of this study, these velocity plots serve as reference points for qualitatively assessing scouring potential in the context of future storms and rising sea levels.

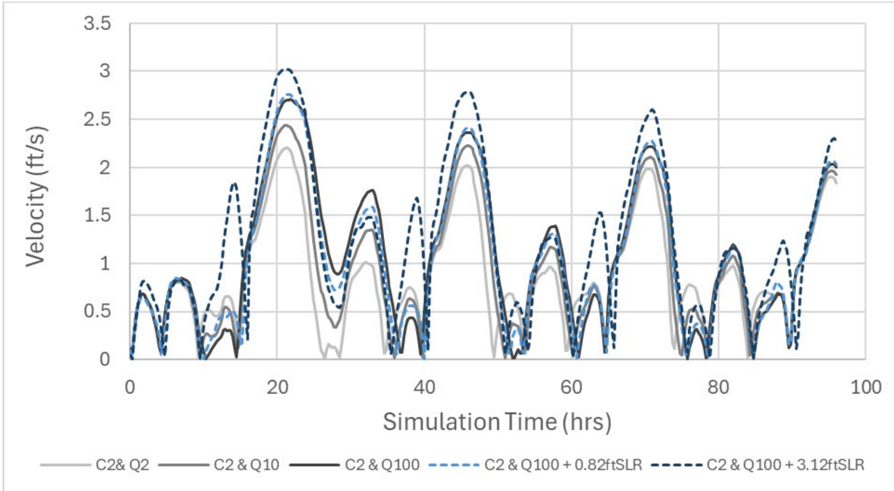


Figure 34. Flow velocities proximal to the northbound Eureka Slough Bridge over a suite of event conditions.

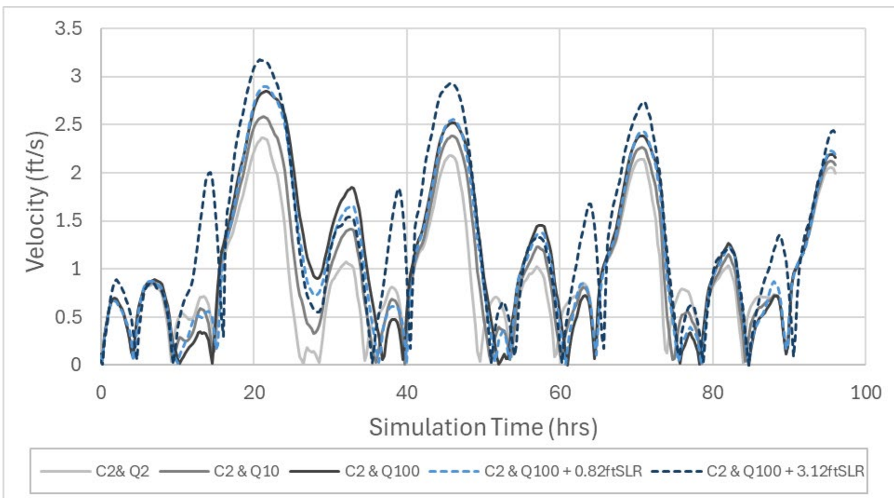


Figure 35. Flow velocities proximal to the southbound Eureka Slough Bridge over a suite of event conditions.

Table 24. Performance metrics for all bridges in the Eureka-Arcata Hwy 101 Corridor Event Condition I scenarios.

	Event Condition					I				
	Scenario #	1	2	3	4	5	6	7	8	9
	Tidal Boundary Condition	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW	MMMW
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
	Drainage Window (hrs)	81	76.8	66	86.5	82.8	69.2	91	89.8	72.5
	Volume (ac-ft)	1421	1415	2007	3220	3211	3453	5835	5783	5451
	Max Headwater (ft)	8.62	9.42	11.47	8.63	9.43	11.48	9.36	9.73	11.71
	Max Tailwater (ft)	8.62	9.42	10.87	8.63	9.42	10.9	8.76	9.48	11.66
	Max Overtop HW/TW Depth (ft)			0.18			0.19			0.42
	Duration HW Exceed 4" Depth Over Deck (hrs)									0.75
Gannon Slough Bridge - Northbound	Duration TW Exceed 4" Depth Over Deck (hrs)									0.25
	Duration HW Exceed Low Deck Elev. (hrs)			1			1			2
	Duration TW Exceed Low Deck Elev. (hrs)									0.75
	Duration HW Exceed Lower Chord (hrs)			15			15.5		0.25	17.75
	Duration TW Exceed Lower Chord (hrs)			14.75			14.75			15
	Duration HW Exceed Max Allowable (hrs)		3.25	30.75		3.25	32.75	6.5	8.75	36.5
	Duration TW Exceed Max Allowable (hrs)		3.25	30.25		3.25	30.5	0.5	3.5	31.5
	Drainage Window (hrs)	80.8	76.2	66	86.5	82.8	69.2	91	89.8	72.5
	Volume (ac-ft)	1420	1414	1998	3219	3210	3444	5834	5781	5446
	Max Headwater (ft)	8.62	9.42	11.55	8.63	9.43	11.55	8.76	9.49	11.66
	Max Tailwater (ft)	8.62	9.42	11.51	8.63	9.43	11.52	8.75	9.47	11.66
	Max Overtop HW/TW Depth (ft)									0.08
	Duration HW Exceed 4" Depth Over Deck (hrs)									
Gannon Slough Bridge - Southbound	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									0.5
	Duration TW Exceed Low Deck Elev. (hrs)									0.5
	Duration HW Exceed Lower Chord (hrs)			14			14.25			14.25
	Duration TW Exceed Lower Chord (hrs)			13.75			14.25			14.25
	Duration HW Exceed Max Allowable (hrs)		2.75	29		2.75	29.5		3	29.75
	Duration TW Exceed Max Allowable (hrs)		2.75	29		2.75	29.5		3	29.5
	Drainage Window (hrs)	99.8	96.8	76	100	97.8	77.2	100.5	98.8	80.2
	Volume (ac-ft)	1012	1011	810	1182	1181	1045	1544	1579	1785
	Max Headwater (ft)	8.66	9.44	11.24	8.66	9.44	11.27	9.27	9.48	11.66
	Max Tailwater (ft)	8.64	9.43	11.02	8.65	9.43	11.08	8.88	9.45	11.65
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
Jacoby Creek Bridge - Northbound	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)			3			3.25			4.5
	Duration TW Exceed Lower Chord (hrs)			2.5			3.25			3.5
	Duration HW Exceed Max Allowable (hrs)			13			13.5			17
	Duration TW Exceed Max Allowable (hrs)			12.75			13			14
	Drainage Window (hrs)	99.8	96.8	76	100	97.8	77.2	100.5	98.2	80.2
	Volume (ac-ft)	1012	1011	812	1182	1181	1047	1544	1579	1786
	Max Headwater (ft)	8.64	9.43	11.48	8.65	9.43	11.5	8.87	9.45	11.65
	Max Tailwater (ft)	8.63	9.43	11.25	8.64	9.43	11.29	8.69	9.45	11.65
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
Jacoby Creek Bridge - Southbound	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)			15.5			15.5			16.5
	Duration TW Exceed Lower Chord (hrs)			15.25			15.25			16
	Duration HW Exceed Max Allowable (hrs)		3.25	30.5		3.5	31	2	6.75	35
	Duration TW Exceed Max Allowable (hrs)		3	30.25		3.25	30.75		5.5	34.5
	Drainage Window (hrs)	61.2	59.2	58	70.2	66.5	61.2	80.5	74.5	65.2
	Volume (ac-ft)	5568	5305	3392	10783	10404	8215	17978	17445	15588
	Max Headwater (ft)	8.53	9.33	11.4	8.56	9.36	11.41	8.57	9.37	11.43
	Max Tailwater (ft)	8.53	9.33	11.39	8.56	9.36	11.4	8.57	9.37	11.42
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
Eureka Slough Bridge - Northbound	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)									
	Duration TW Exceed Lower Chord (hrs)									
	Duration HW Exceed Max Allowable (hrs)									
	Duration TW Exceed Max Allowable (hrs)									
	Drainage Window (hrs)	61	59.2	58	70	66	61.2	80	74.5	65.2
	Volume (ac-ft)	5565	5303	3390	10781	10401	8212	17976	17443	15586
	Max Headwater (ft)	8.53	9.33	11.42	8.56	9.36	11.43	8.57	9.37	11.44
	Max Tailwater (ft)	8.53	9.33	11.4	8.56	9.36	11.41	8.57	9.37	11.43
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
Eureka Slough Bridge - Southbound	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)									
	Duration TW Exceed Lower Chord (hrs)									
	Duration HW Exceed Max Allowable (hrs)									
	Duration TW Exceed Max Allowable (hrs)									

Table 25. Performance metrics for all bridges in the Eureka-Arcata Hwy 101 Corridor Event Condition II scenarios.

	Event Condition	II								
	Scenario #	10	11	12	13	14	15	16	17	18
	Tidal Boundary Condition	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr	2-yr
	Fluvial Boundary Condition	2-yr	2-yr	2-yr	10-yr	10-yr	10-yr	100-yr	100-yr	100-yr
	Sea Level Rise (ft)	0	0.82	3.12	0	0.82	3.12	0	0.82	3.12
Gannon Slough Bridge - Northbound	Drainage Window (hrs)	73	69.5	58.8	77.8	73.8	63	85.2	79	64.8
	Volume (ac-ft)	1419	1628	1918	3328	3438	3202	5750	5675	5022
	Max Headwater (ft)	9.5	10.22	12.31	9.51	10.23	12.36	9.75	10.45	12.53
	Max Tailwater (ft)	9.49	10.11	11.6	9.5	10.12	12.01	9.56	10.32	12.5
	Max Overtop HW/TW Depth (ft)			1.02			1.07			1.24
	Duration HW Exceed 4" Depth Over Deck (hrs)			6			6.75			7.25
	Duration TW Exceed 4" Depth Over Deck (hrs)						2.25			3.25
	Duration HW Exceed Low Deck Elev. (hrs)			8.25			9.25			9.75
	Duration TW Exceed Low Deck Elev. (hrs)			2.5			3.75			4.5
	Duration HW Exceed Lower Chord (hrs)		4	23.25		4	24	0.5	6.75	26.25
	Duration TW Exceed Lower Chord (hrs)		4	22		4	22.25		4.5	22.25
	Duration HW Exceed Max Allowable (hrs)	5.75	11.5	40.5	5.75	12	42	11.25	17	46.25
	Duration TW Exceed Max Allowable (hrs)	5.75	11.5	38.5	5.75	11.5	38.75	6	12.25	39.25
Gannon Slough Bridge - Southbound	Drainage Window (hrs)	72.8	69.5	59.2	77.8	73.5	63.2	85.2	78.8	64.8
	Volume (ac-ft)	1419	1628	1898	3328	3438	3176	5750	5676	5009
	Max Headwater (ft)	9.5	10.25	12.41	9.51	10.25	12.43	9.56	10.32	12.52
	Max Tailwater (ft)	9.5	10.24	12.37	9.51	10.24	12.4	9.54	10.32	12.52
	Max Overtop HW/TW Depth (ft)			0.83			0.85			0.94
	Duration HW Exceed 4" Depth Over Deck (hrs)			4.25			4.5			4.5
	Duration TW Exceed 4" Depth Over Deck (hrs)			3.75			4			4.25
	Duration HW Exceed Low Deck Elev. (hrs)			6.25			6.75			6.75
	Duration TW Exceed Low Deck Elev. (hrs)			6.25			6.5			6.5
	Duration HW Exceed Lower Chord (hrs)		4	21.75		4	22		4	22
	Duration TW Exceed Lower Chord (hrs)		3.75	21.5		3.75	21.5		4	21.75
	Duration HW Exceed Max Allowable (hrs)	5.25	10.75	38.25	5.25	10.75	38.5	5.5	11	38.75
	Duration TW Exceed Max Allowable (hrs)	5.25	10.75	38.25	5.25	10.75	38.5	5.25	11	38.5
Jacoby Creek Bridge - Northbound	Drainage Window (hrs)	89.5	86.8	66	90.2	87.8	68.5	91.2	88.2	70.5
	Volume (ac-ft)	981	997	208	1158	1169	705	1568	1674	1590
	Max Headwater (ft)	9.53	10.29	11.94	9.53	10.29	12.09	9.55	10.33	12.48
	Max Tailwater (ft)	9.51	10.28	11.58	9.52	10.29	11.97	9.53	10.31	12.46
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)			12.5			13			14.5
	Duration TW Exceed Lower Chord (hrs)			11.25			11.75			12.5
	Duration HW Exceed Max Allowable (hrs)		3.5	21.25		3.5	22.75		6	25.75
	Duration TW Exceed Max Allowable (hrs)		3.25	21		3.25	21.5		3.75	22.75
Jacoby Creek Bridge - Southbound	Drainage Window (hrs)	89.5	86.8	66	90.2	87.8	68.5	91.2	88.2	70.5
	Volume (ac-ft)	981	997	272	1158	1169	763	1568	1674	1628
	Max Headwater (ft)	9.51	10.28	12.36	9.52	10.29	12.39	9.53	10.31	12.5
	Max Tailwater (ft)	9.51	10.28	11.96	9.52	10.29	12.1	9.53	10.3	12.48
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)		4.5	22.5		4.5	22.75		5	25.25
	Duration TW Exceed Lower Chord (hrs)		4.5	22.25		4.5	22.5		5	23.25
	Duration HW Exceed Max Allowable (hrs)	5.75	12.25	38.75	5.75	12.25	40.5	9.5	16	44
	Duration TW Exceed Max Allowable (hrs)	5.75	12	38.75	5.75	12.25	39.75	8.25	15.5	43.25
Eureka Slough Bridge - Northbound	Drainage Window (hrs)	59.5	58	56.5	67.2	65.5	60.5	72.5	69.5	62.8
	Volume (ac-ft)	5671	5173	6725	10942	10142	12230	17587	16855	19064
	Max Headwater (ft)	9.45	10.2	12.2	9.48	10.22	12.22	9.49	10.23	12.24
	Max Tailwater (ft)	9.45	10.19	12.19	9.48	10.21	12.2	9.49	10.22	12.22
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)									
	Duration TW Exceed Lower Chord (hrs)									
	Duration HW Exceed Max Allowable (hrs)									
	Duration TW Exceed Max Allowable (hrs)									
Eureka Slough Bridge - Southbound	Drainage Window (hrs)	59.5	57.8	56.5	67.2	65.2	60.8	72.5	69.5	62.8
	Volume (ac-ft)	5675	5177	6730	10946	10147	12235	17590	16859	19069
	Max Headwater (ft)	9.46	10.21	12.22	9.48	10.22	12.23	9.49	10.23	12.26
	Max Tailwater (ft)	9.45	10.2	12.2	9.48	10.22	12.22	9.49	10.23	12.24
	Max Overtop HW/TW Depth (ft)									
	Duration HW Exceed 4" Depth Over Deck (hrs)									
	Duration TW Exceed 4" Depth Over Deck (hrs)									
	Duration HW Exceed Low Deck Elev. (hrs)									
	Duration TW Exceed Low Deck Elev. (hrs)									
	Duration HW Exceed Lower Chord (hrs)									
	Duration TW Exceed Lower Chord (hrs)									
	Duration HW Exceed Max Allowable (hrs)									
	Duration TW Exceed Max Allowable (hrs)									

Roads

The following section details hydraulic results for the southbound and northbound travel lanes of Hwy 101. Similar analyses may be conducted for the various on/off ramps and local cross streets or connectors during the upcoming vulnerability assessment.

Highway 101

In the absence of sea level rise, Highway 101 is not overtopped in all modeled scenarios of coastal and riverine event conditions, except during 100-yr fluvial flood event (Table 22; Figures A1 – A10). In this case, water overtops the highway at the Samoa-101 interchange, regardless of the tidal boundary conditions (Table 22 and Figures 18 & A-3). It is important to note that this localized inundation may be at least partially attributable to limited topobathymetric and structure data for Campbell Creek and Gannon Slough, as well as the surrounding floodplains. Notably, the LiDAR DEM inaccurately represents the creek's geometry and the detention basin on the Arcata Sports Complex property. Future modeling should address this data gap through targeted field surveys.

It is also worth noting that, although the results of Event Conditions III are not covered as part of this report,

Table 22. Summary table indicating whether the west edge of travel lane, east edge of travel lane or centerline of the northbound and southbound lanes of Hwy 101 are overtopped.

Event Condition	Scenario	Tidal Boundary Condition	Fluvial Boundary Condition	Sea Level Rise (ft)	Hwy 101 Overtopping					
					Southbound West	Southbound Centerline	Southbound East	Northbound West	Northbound Centerline	Northbound East
I	1	MMMW[2]	2-yr	0						
	2	MMMW	2-yr	0.82						
	3	MMMW	2-yr	3.12	X	X	X	X	X	X
	4	MMMW	10-yr	0						
	5	MMMW	10-yr	0.82						
	6	MMMW	10-yr	3.12	X	X	X	X	X	X
	7	MMMW	100-yr	0			X[1]	X[1]	X[1]	X[1]
	8	MMMW	100-yr	0.82			X[1]	X[1]	X[1]	X[1]
	9	MMMW	100-yr	3.12	X	X	X	X	X	X
II	10	2-yr[3]	2-yr	0						
	11	2-yr	2-yr	0.82						
	12	2-yr	2-yr	3.12	X	X	X	X	X	X
	13	2-yr	10-yr	0						
	14	2-yr	10-yr	0.82						
	15	2-yr	10-yr	3.12	X	X	X	X	X	X
	16	2-yr	100-yr	0	X[1]	X[1]	X[1]	X[1]	X[1]	X[1]
	17	2-yr	100-yr	0.82	X[1]	X[1]	X[1]	X[1]	X[1]	X[1]
	18	2-yr	100-yr	3.12	X	X	X	X	X	X

[1] Hwy 101 - Samoa Interchange or median drain overtopping only
[2] Peak of MMMW tide = 8.36 ft
[3] Peak of coastal 2-yr storm = 9.26 ft

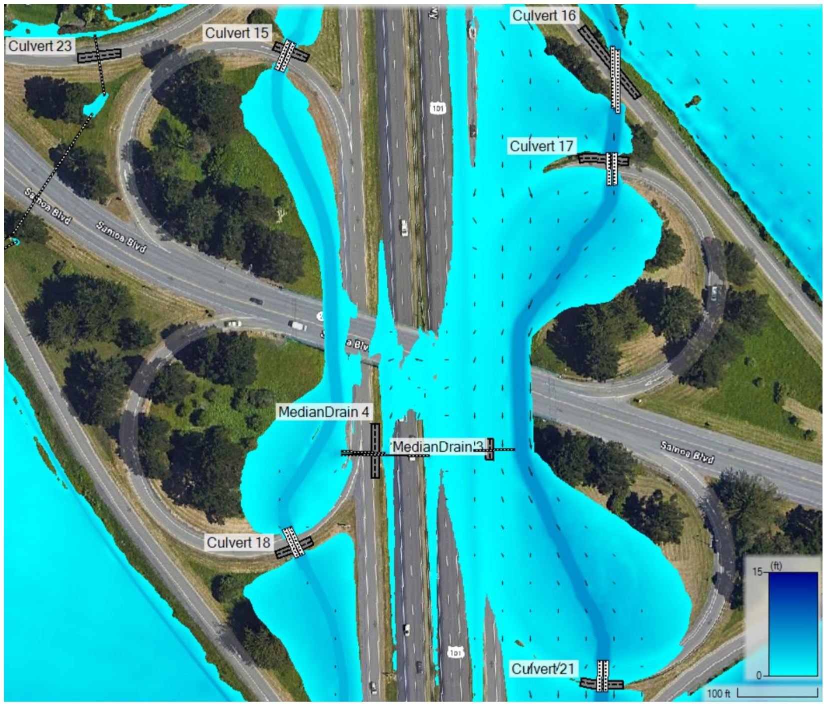


Figure 18. Overtopping at the Hwy 101-Samoa interchange during the MMMW & 100-yr fluvial flood + 3.12 ft sea level rise event scenario. Note, flooding in this area may be amplified by limited topobathymetric data in the Campbell Creek and Gannon Slough channels and adjacent floodplains.

Figure 19 shows that during the Q100 storm event, the average flood depth within the Samoa-Hwy 101 Interchange overtopping zone is less than 0.2 feet, with peak depths reaching up to 0.6 feet in the northbound lanes. The figure also indicates that both the southbound and northbound lanes of Hwy 101 experience inundation exceeding ~2.5 inches (0.2 feet) for approximately 2.75 and 4.5 hours, respectively. Although overtopping in this area may be amplified by potential inaccuracies in the LiDAR DEM, this analysis highlights the utility of 2D model outputs in assessing future conditions and identifying vulnerabilities in the context of impact thresholds.

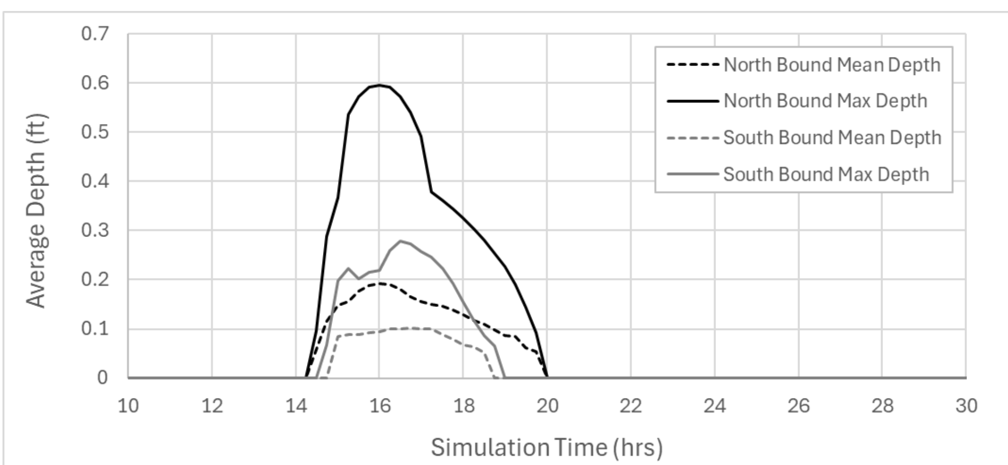


Figure 19. Hydrographs of mean and maximum flood depths for the northbound and southbound Hwy101 at the Samoa – Hwy 101 interchange during the MMMW & Q100 + 3.12 ft SLR event scenario.

Figures in Appendices A & B demonstrate how 2D model results can be post-processed to generate detailed maps of both overtopping depths (Appendix A) and the duration of Highway 101 centerline inundation above a specified depth corresponding to an asset impact threshold (Appendix B). For the analyses in Appendix B, a threshold depth of 4 inches was used to calculate inundation durations along the Highway 101 centerlines. These analyses and figures cover all event scenarios where either the southbound or northbound centerlines of Highway 101 were overtopped.

As shown in Table 22, Appendices A & B, and Figures 23 - 28, the southbound and northbound lanes of Highway 101 in the Eureka-Arcata Corridor do not experience significant overtopping in terms of depth or duration until sea level rise (SLR) reaches 3.12 ft. At this level, flood depths frequently exceed 1 ft, and overtopping durations often surpass 48 hours. Additionally, with 3.12 ft of SLR, the overtopping duration is significantly longer during a 2-year coastal storm than under MMMW tidal conditions—averaging approximately 32 hours for MMMW compared to 65 hours for the 2-year coastal event.

Figures 23 - 28 below present water surface elevation profiles along the entire length of the southbound and northbound lanes of Highway 101 within the Project Area, with station 0 ft corresponding to the eastern edge of the Eureka Slough Bridges. As previously noted, apart from the Highway 101-Samoa Interchange area, the highway remains unaffected by inundation until sea level rise reaches 3.12 ft. Both the southbound and northbound lanes are particularly vulnerable to flooding between stations 6,000 and 16,500 ft (roughly postmile 81.15 – 83.3; approximately between Murray Field and Bracut Industrial Park). While the northbound lanes are generally lower in elevation throughout this segment, the southbound lanes experience deeper flooding under MMMW conditions due to their closer proximity to Humboldt Bay, making them more susceptible to coastal flooding and the effects of SLR. Notably, high elevations of the Bay Trail and levee enclosing the Brainard mill site (~11.5 – 21.25 ft) offer added protection from coastal inundation for southbound Hwy 101 within the Murray Field to Bracut segment. This is evident in the reduced flood depths and lower overtopping risks for Hwy 101 in this area (Figures 23 – 24).

Table 23 summarizes the mean and maximum flood depths overtopping the centerlines of northbound and southbound Highway 101, as well as the total length of the roadway inundated. The results show that overtopping depths and inundation lengths are minor across all event scenarios when sea level rise (SLR) is ≤ 0.82 feet. However, during all modeled fluvial storm events with MMMW + 3.12 feet of SLR as the tidal boundary condition, the southbound centerline experiences significantly greater flood depths and longer inundated lengths compared to the northbound lanes, highlighting its greater vulnerability to tidal and SLR conditions.

In scenarios involving a coastal 2-year storm combined with 3.12 feet of sea level rise, both the northbound and southbound centerlines of Highway 101 are extensively inundated, with approximately 56% - 72% of the roadway length within the model domain submerged and mean water depths exceeding 1 foot along most of its length (Table 23). Under these extreme conditions, the northbound lanes experience generally higher mean and maximum flood depths due to lower road elevations between approximately 6,000 and 16,500 feet, making this segment more susceptible to deeper flooding compared to the southbound lanes (Figure 21).

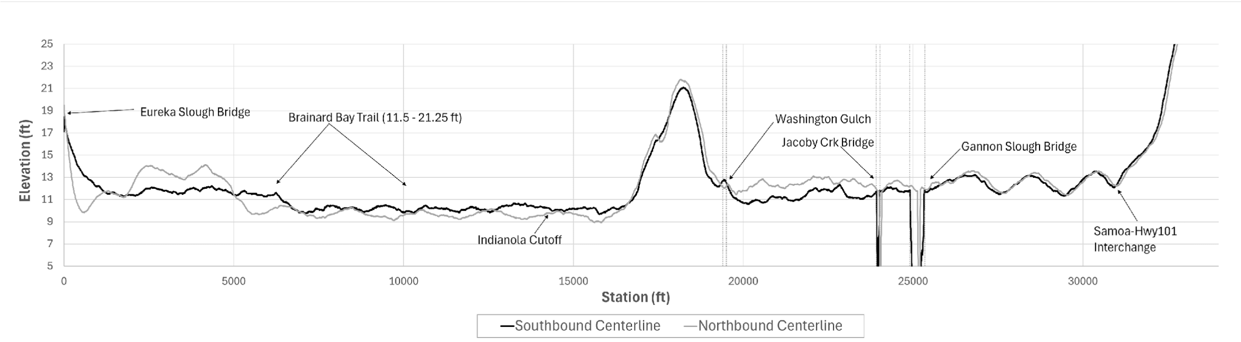


Figure 21. Road surface elevations for the centerlines of southbound and northbound Hwy 101 in the model domain.

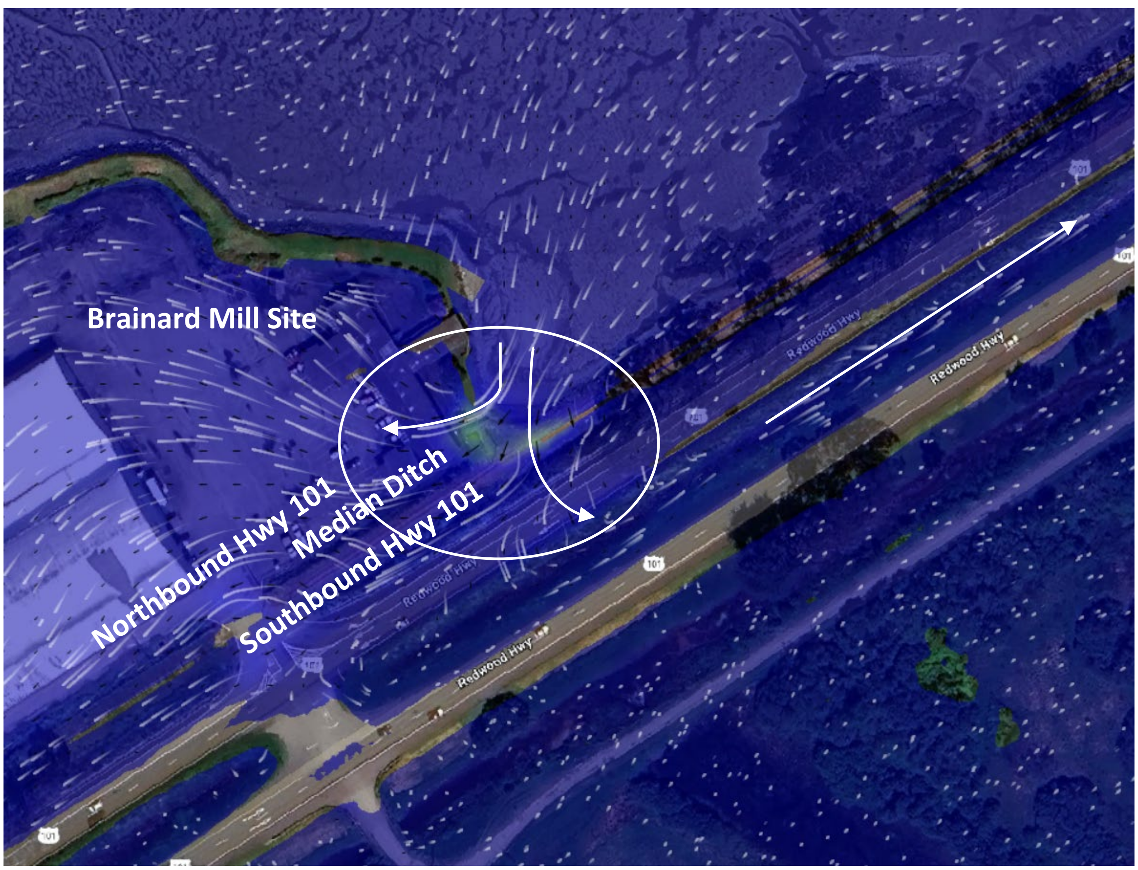
Although coastal conditions primarily drive flood dynamics within the Highway 101 Corridor, the influence of fluvial flooding is evident, especially in the northbound lanes, which are the first to be affected as floodwaters from the east reach the roadway. For instance, under MMMW + 3.12 feet SLR conditions, increasing fluvial flows from Q2 to Q100 result in roughly a 1-inch rise in mean flood depth and a 3.6-inch increase in maximum flood depth in the northbound lanes, along with an additional 2,162 feet of the roadway becoming inundated (Table 23). Similarly, during C2 + 3.12 feet SLR scenarios, higher fluvial flows cause the mean flood depth in the northbound lanes to increase by approximately 1.5 inches, with the total inundated length extending by 1,305 feet.

In contrast, the southbound lanes are minimally affected by fluvial conditions, showing little variation in mean or maximum flood depths or inundation extents as fluvial flows increase. While fluvial conditions disproportionately impact the northbound lanes, coastal flooding remains the dominant factor influencing flood dynamics throughout the corridor. This is evident in the significant increases in flood depths and inundation extents along both the northbound and southbound lanes as coastal storm magnitudes intensify. For example, comparing the MMMW & Q100 + 3.12 feet SLR scenario to the C2 & Q100 + 3.12 feet SLR scenario, inundation lengths and mean flood depths along the northbound centerline increase by approximately 4 and 7 times, respectively, highlighting the overriding influence of coastal conditions.

Table 23. Mean and maximum depths and total inundated lengths for the northbound and southbound Hwy 101 centerlines for all overtopping event conditions.

Hwy 101	Parameter	MMMW & Q100	MMMW & Q100 + 0.82 ft SLR	MMMW & Q2 + 3.12 ft SLR	MMMMW & Q10 + 3.12 ft SLR	MMMW & Q100 + 3.12 ft SLR	C2 & Q100	C2 & Q100 + 0.82 ft SLR	C2 & Q2 + 3.12 ft SLR	C2 & Q10 + 3.12 ft SLR	C2 & Q100 + 3.12 ft SLR
Northbound Centerline	Mean Depth (ft)	0.05	0.04	0.11	0.12	0.19	0.05	0.07	1.35	1.42	1.47
	Max Depth (ft)	0.09	0.11	0.25	0.26	0.55	0.12	0.16	3.10	3.10	3.16
	Length Inundated (ft)	52	105	1,870	1,925	4,031	108	144	19,017	19,378	20,323
Southbound Centerline	Mean Depth (ft)	0.00	0.00	0.43	0.42	0.42	0.02	0.03	1.11	1.16	1.24
	Max Depth (ft)	0.01	0.03	1.06	1.06	1.07	0.03	0.07	2.32	2.32	2.39
	Length Inundated (ft)	7	10	12,389	12,510	13,382	10	30	24,190	24,239	24,433

Flooding along the western travel lane of northbound Highway 101 between ~10,000 and 16,500 ft during MMMW + 3.12 ft SLR scenarios is primarily driven by coastal inundation. Waters from the Bay overtop the Bay Trail and southbound lanes, filling the median ditch that separates the two directions of travel. As the ditch becomes overwhelmed, it spills eastward, flowing across the northbound lanes. A substantial amount of coastal water entering the median ditch—and subsequently the Brainard mill site—comes from overtopping at a low-lying section of the Bay Trail near the northeastern corner of the mill site (highlighted in Figure 22). It is important to highlight that the Bay Trail's elevations in this area are based on design specifications, which may differ from actual as-built conditions. To ensure the accuracy of the hydraulic model, it is recommended that these elevations be verified using as-built survey data.



Figures 23 - 28 also highlight the fact that, once SLR reaches 3.12 ft, fluvial conditions become relatively inconsequential in the Hwy 101 Corridor (e.g. water surface elevations are similar between the C2 & Q2 + 3.12 ft SLR and C2 & Q100 + 3.12 ft SLR scenarios). Although fluvial flooding does influence the eastern side of northbound Hwy 101 and areas further inland, coastal conditions—particularly sea level rise—dominate the flooding dynamics within the Highway 101 Corridor.

A comprehensive interpretation of these results, along with their implications for road infrastructure, will be addressed in an upcoming vulnerability and risk assessment report.

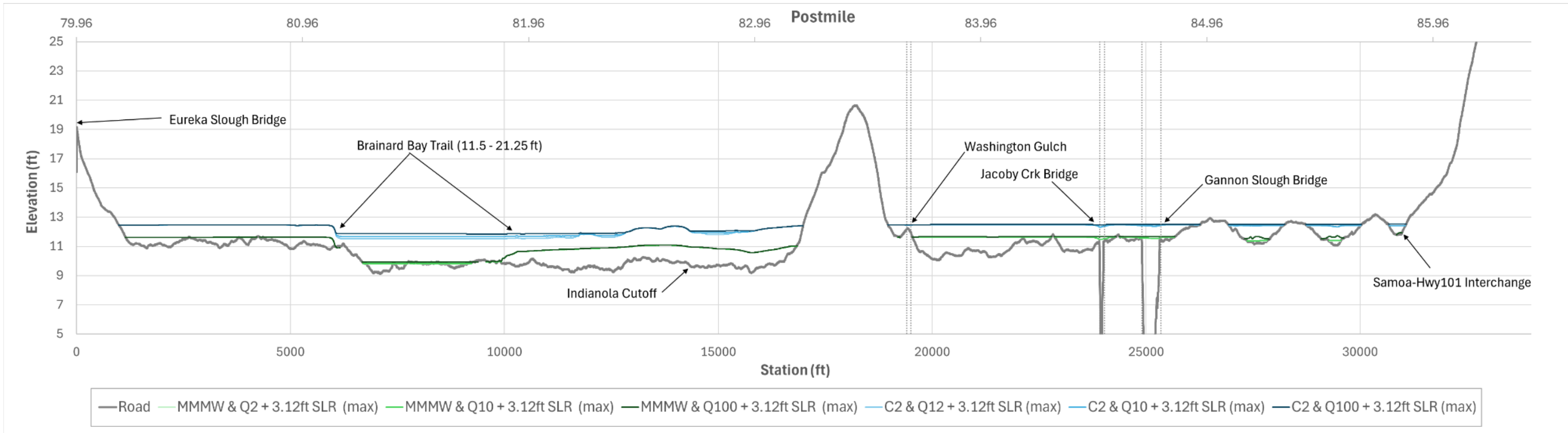


Figure 23. Water surface elevations over a range of modeled event conditions relative to the southbound Hwy 101 west edge of travel lane ground surface. Note, only those event conditions resulting in overtopping are displayed.

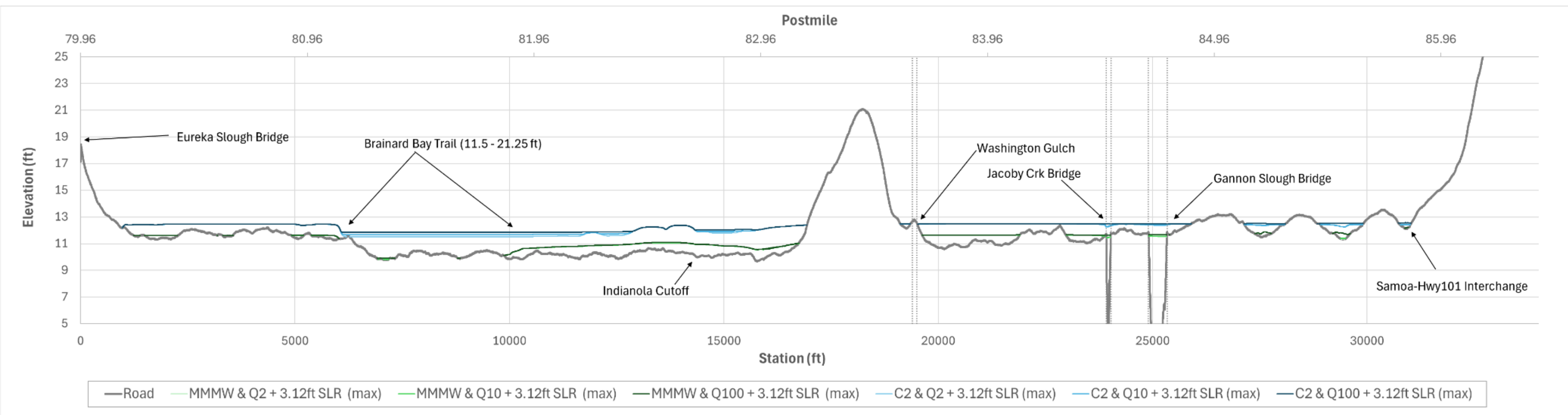


Figure 24. Water surface elevations over a range of modeled event conditions relative to the southbound Hwy 101 centerline ground surface. Note, only those event conditions resulting in overtopping are displayed.

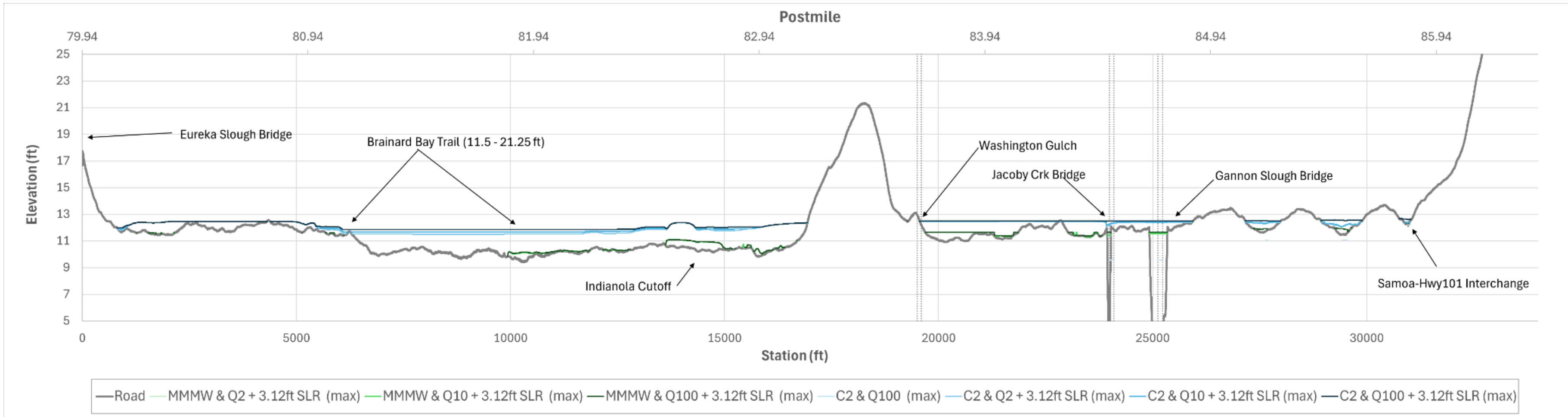


Figure 25. Water surface elevations over a range of modeled event conditions relative to the southbound Hwy 101 east edge of travel lane ground surface. Note, only those event conditions resulting in overtopping are displayed.

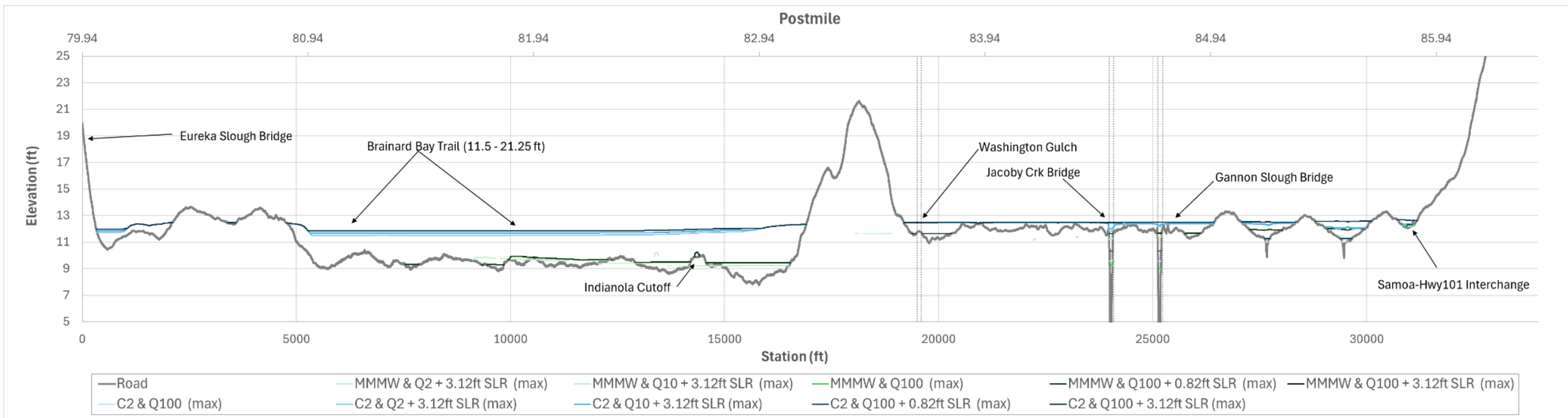


Figure 26. Water surface elevations over a range of modeled event conditions relative to the northbound Hwy 101 west edge of travel lane ground surface. Note, only those event conditions resulting in overtopping are displayed.

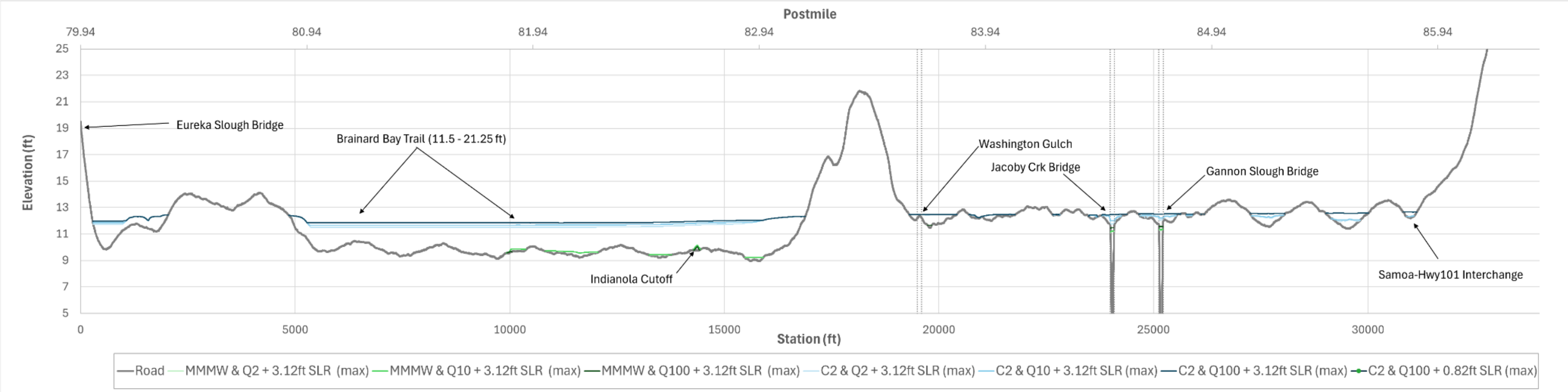


Figure 27. Water surface elevations over a range of modeled event conditions relative to the northbound Hwy 101 centerline ground surface. Note, only those event conditions resulting in overtopping are displayed.

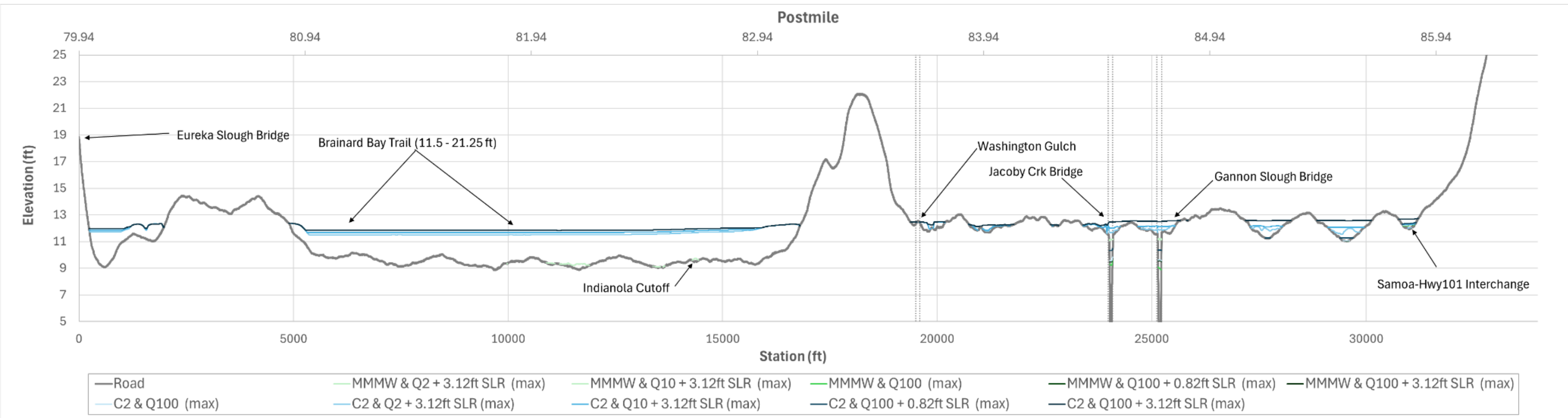


Figure 28. Water surface elevations over a range of modeled event conditions relative to the northbound Hwy 101 east edge of travel lane ground surface. Note, only those event conditions resulting in overtopping are displayed.

Summary

This technical memorandum outlines key findings from ongoing 2D hydraulic modeling efforts conducted to simulate fluvial and coastal event conditions within the Highway 101 corridor as part of the Comprehensive Climate Adaptation and Implementation Plan (CAIP) for the Eureka-Arcata Corridor. The primary objective was to evaluate existing hydraulic conditions across various flood scenarios, including those influenced by sea level rise, to inform vulnerability assessments and guide future adaptation strategies.

The analysis focused on two primary aspects: (1) Flood Cell Dynamics, highlighting flood exposures in specific floodplain areas, and (2) Infrastructure Vulnerabilities, examining the impacts of flooding on critical assets such as roads, culverts, tide gates, and bridges. Key findings from these analyses are summarized below.

Key Findings:

- <u>Flood Cells</u>:
    - The analysis identified several flood-prone areas, with the flood cells associated with Freshwater, Fay, and Eureka Sloughs exhibiting some of the deepest floodwaters and longest inundation durations. In contrast, areas characterized by commercial land use and engineered water control features, such as berms and levees, experienced shallower and less extensive flooding, particularly with SLR below 3.12 feet.
- <u>Highway 101 Flooding</u>:
    - **Limited Overtopping Under Current Conditions**: Without sea level rise (SLR), Highway 101 is not overtopped in any modeled coastal or fluvial scenarios except during the 100-year fluvial flood event, where localized overtopping occurs at the Samoa-Hwy 101 interchange. While this section is one of the most susceptible to flooding, particularly during high fluvial flows, flood depths in both the southbound and northbound lanes rarely exceed 0.5 feet and typically persist for less than 5 hours, even with 3.12 ft of SLR.
    - **Significant Overtopping with Increasing SLR**: With 3.12 feet of sea level rise (SLR), overtopping depths and durations along Highway 101 increase substantially, often exceeding 1 foot and lasting more than 48 hours during storm events. During coastal 2-year storms at this SLR level, over 55% of the Highway 101 corridor between Eureka and Arcata becomes inundated, with average overtopping depths exceeding 1.1 feet and durations reaching approximately 65 hours. These conditions indicate a high likelihood of frequent road closures and an increased risk of structural damage in the future.
    - **Most Flood-Prone Segments**: In addition to the Samoa-Hwy 101 interchange, both northbound and southbound lanes are particularly vulnerable to flooding

between stations 6,000 and 16,500 feet (approximately between Murray Field and Bracut Industrial Park). While northbound lanes are generally at lower elevations, southbound lanes are closer to Humboldt Bay and experience deeper flooding under MMMW conditions. A significant portion of the flooding between stations 10,000 and 16,500 feet is driven by coastal waters overtopping a low-lying section of the Bay Trail near the northeastern corner of the Brainard mill site. This coastal flood water quickly overwhelms the inboard ditch on the east side of the Bay Trail and the Highway 101 median ditch, causing floodwaters to spill eastward and inundate both the southbound and northbound lanes.

  - **Coastal Flooding Dominance**: Although fluvial flooding impacts areas farther inland, coastal conditions—particularly sea level rise (SLR)—are the primary drivers of flooding dynamics within the Highway 101 corridor. As SLR scenarios become more severe, both the extent and depth of inundation increase significantly, highlighting the need for robust coastal flood mitigation strategies. However, larger fluvial flood events also contribute to notable increases in road overtopping depths and inundation extents, particularly in the northbound lanes, which are the first to encounter fluvial floodwaters from the east. For instance, under MMMW + 3.12 feet SLR conditions, increasing fluvial flows from Q2 to Q100 leads to approximately a 1-inch rise in mean flood depth, a 3.6-inch increase in maximum depth, and an additional 2,162 feet of inundated roadway in the northbound lanes.

- <u>Bridge Vulnerabilities</u>: Critical infrastructure such as the Gannon Slough and Jacoby Creek Bridges face substantial risks under extreme flood events combined with intermediate SLR. The Gannon Slough Bridges are particularly vulnerable; even without SLR, the soffit of the northbound bridge is engaged by floodwaters during a combined 2-year coastal and 100-year fluvial storm. With 0.82 feet of SLR, the soffit becomes moderately submerged in most scenarios, and under 3.12 feet of SLR, it is fully submerged across all events. The bridge deck itself is overtopped in all 3.12 feet SLR scenarios, with flood depths ranging from 0.18 to 1.12 feet and durations lasting 0.75 to 9.5 hours. Both bridges also exhibit inadequate freeboard, indicating a high risk of structural impairment and frequent closures as SLR increases.

- <u>Tide Gate and Culvert Performance</u>: The South Hwy 101 Tide Gate and Culvert 28 face significant reductions in drainage capacity and exceed allowable headwater elevations in more severe flood scenarios combined with SLR. Similarly, culverts associated with Campbell Creek and Gannon Slough (e.g. Culverts 20 & 21), as well as the Highway 101 ditch (e.g. Culverts 29 & 30), exceed their maximum allowable headwater thresholds and overtop adjacent roadways even under current conditions. With 3.12 feet of SLR, these structures and their adjacent roadways experience prolonged flooding, increasing the likelihood of more frequent road closures and a higher risk of structural damage due to extended exposure to corrosive saltwater. Reduced drainage efficiency and shorter drainage windows also elevate risks of sustained inland flooding and changes in wetland habitat due to altered salinity, sediment transport and hydrology. These changes not only decrease the time available for effective drainage but also reduce habitat connectivity and hinder passage for vulnerable aquatic organisms, highlighting the broader ecological impacts of undersized or low-resiliency structures.

These findings emphasize the need to incorporate hydraulic modeling insights into future adaptation strategies, with a focus on refining hydraulic models and conducting targeted field surveys to close data gaps, especially regarding hydraulic structure details and topobathymetric data. These data gaps, along with additional uncertainties related to LiDAR accuracy, sea level rise projections, and peak flow estimates for upstream boundary conditions, present significant challenges. While a robust quantification of model uncertainty is beyond the scope of this analysis, future vulnerability assessments should qualitatively address these compound uncertainties.

Despite these uncertainties, the results underscore the value of 2D hydraulic modeling in assessing current conditions and anticipating future climate impacts, providing a solid foundation for infrastructure planning and the development of effective adaptation measures across the project area.

References

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Appendix A – Depth & Overtopping Maps

This appendix contains maps of flood depth and road overtopping depths for the Hwy 101 Corridor for all event scenarios during which either the southbound or northbound centerlines of Hwy 101 were overtopped.

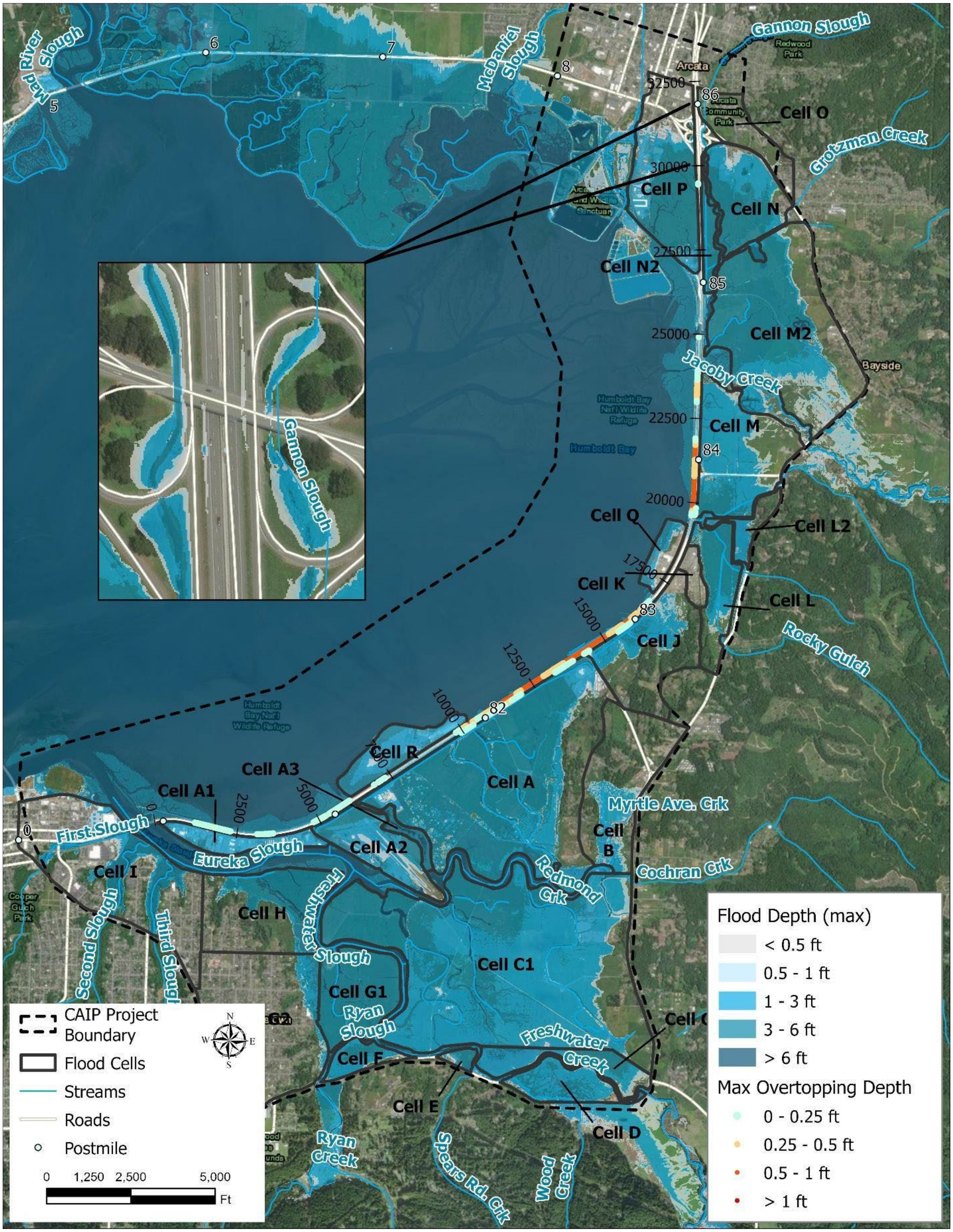

Figure A-36. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the MMMW and Q2 event scenario with 3.12 ft of sea level rise.

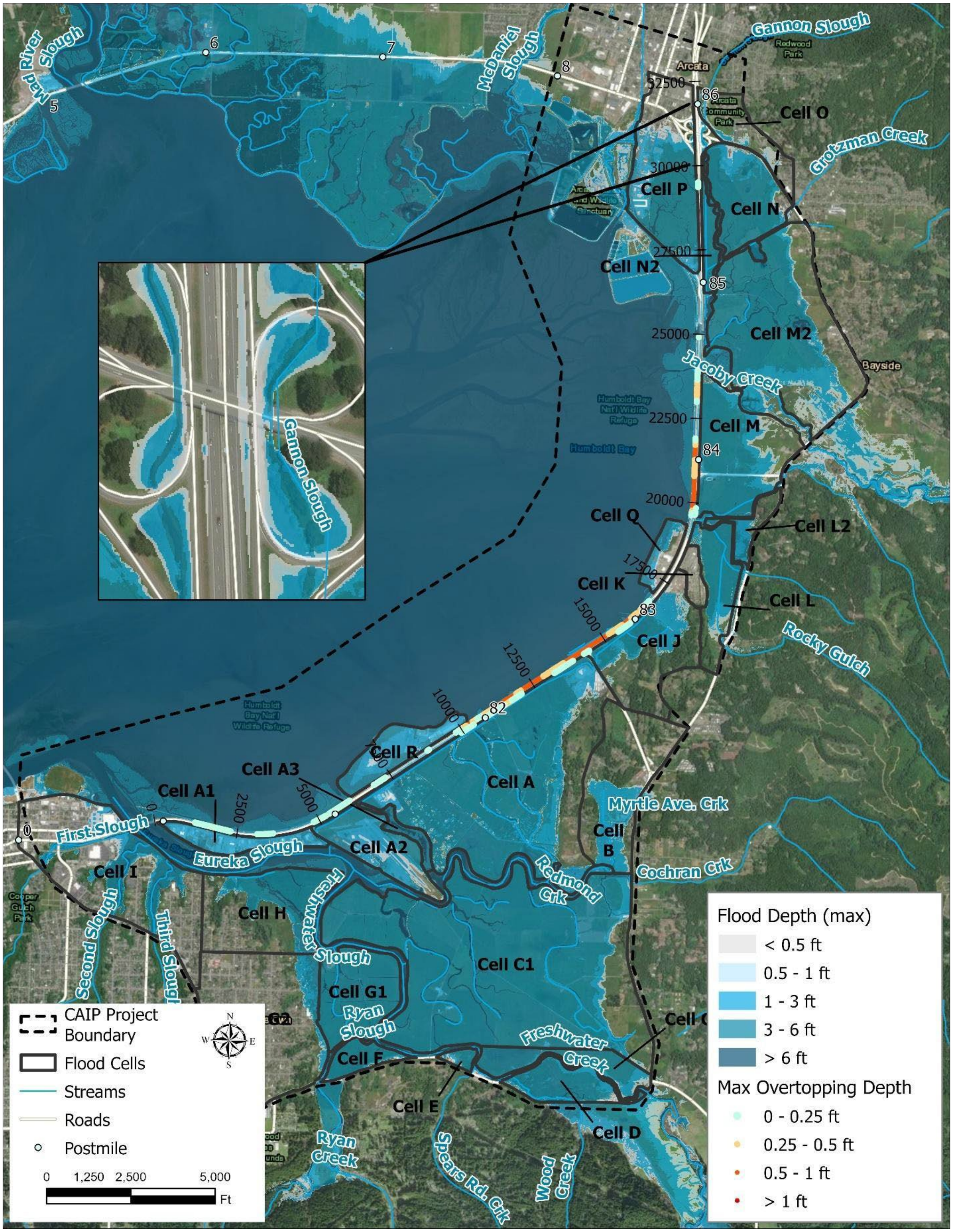

Figure A-37. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the MMMW & Q10 + 3.12 ft SLR event condition.

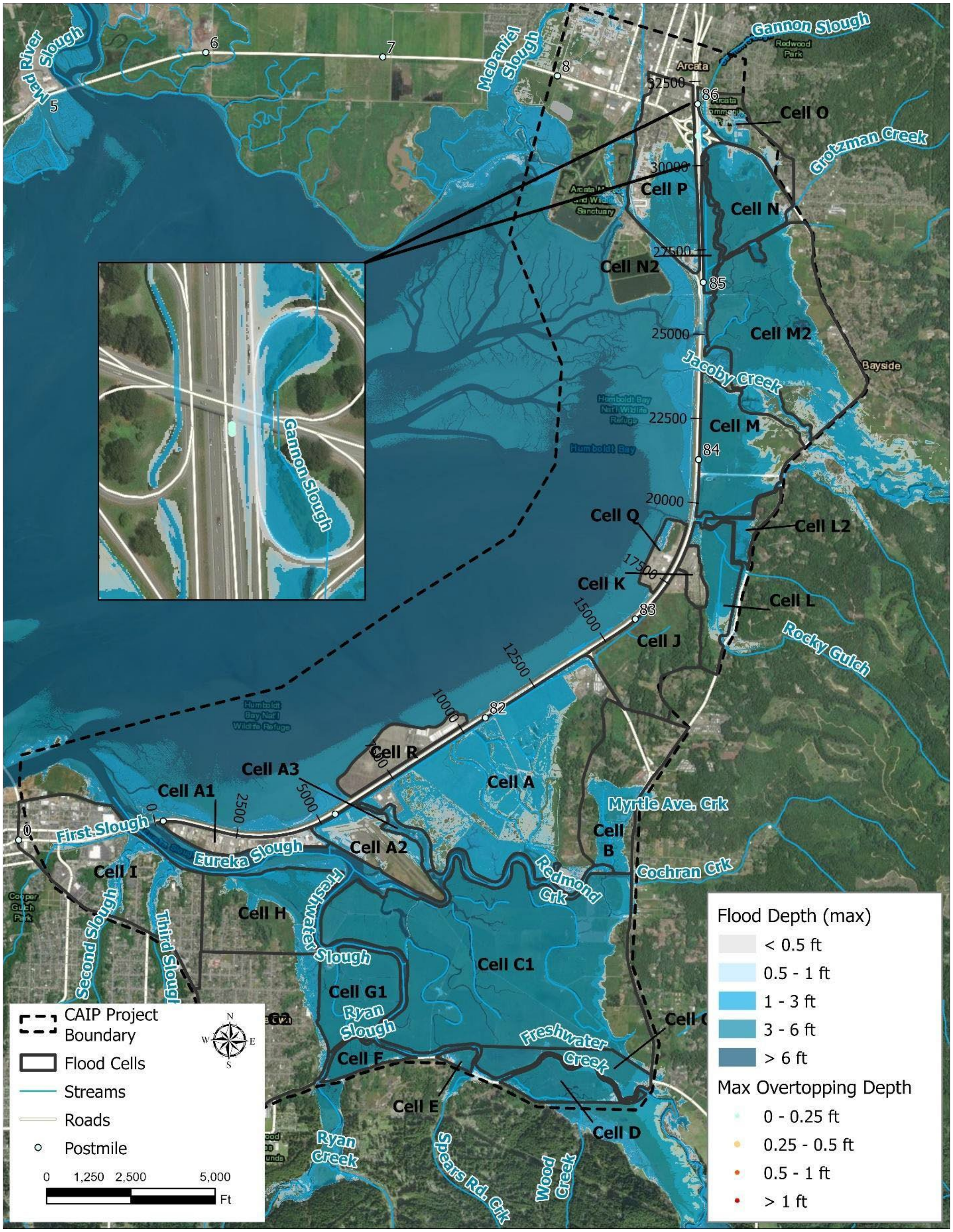

Figure A-3. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the MMMW & Q100 event condition.

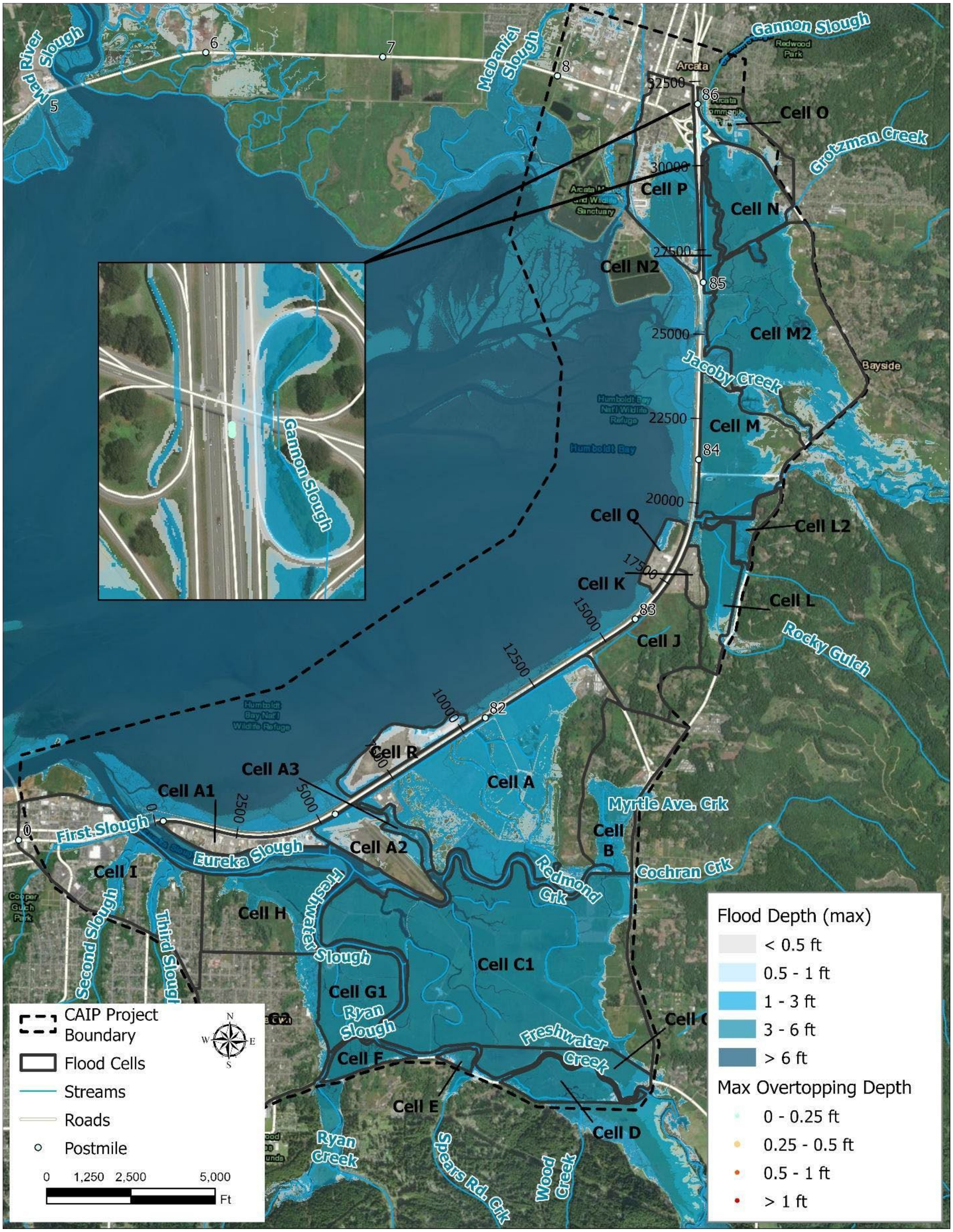

Figure A-4. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the MMMW and Q100 event scenario with 3.12 ft of SLR.

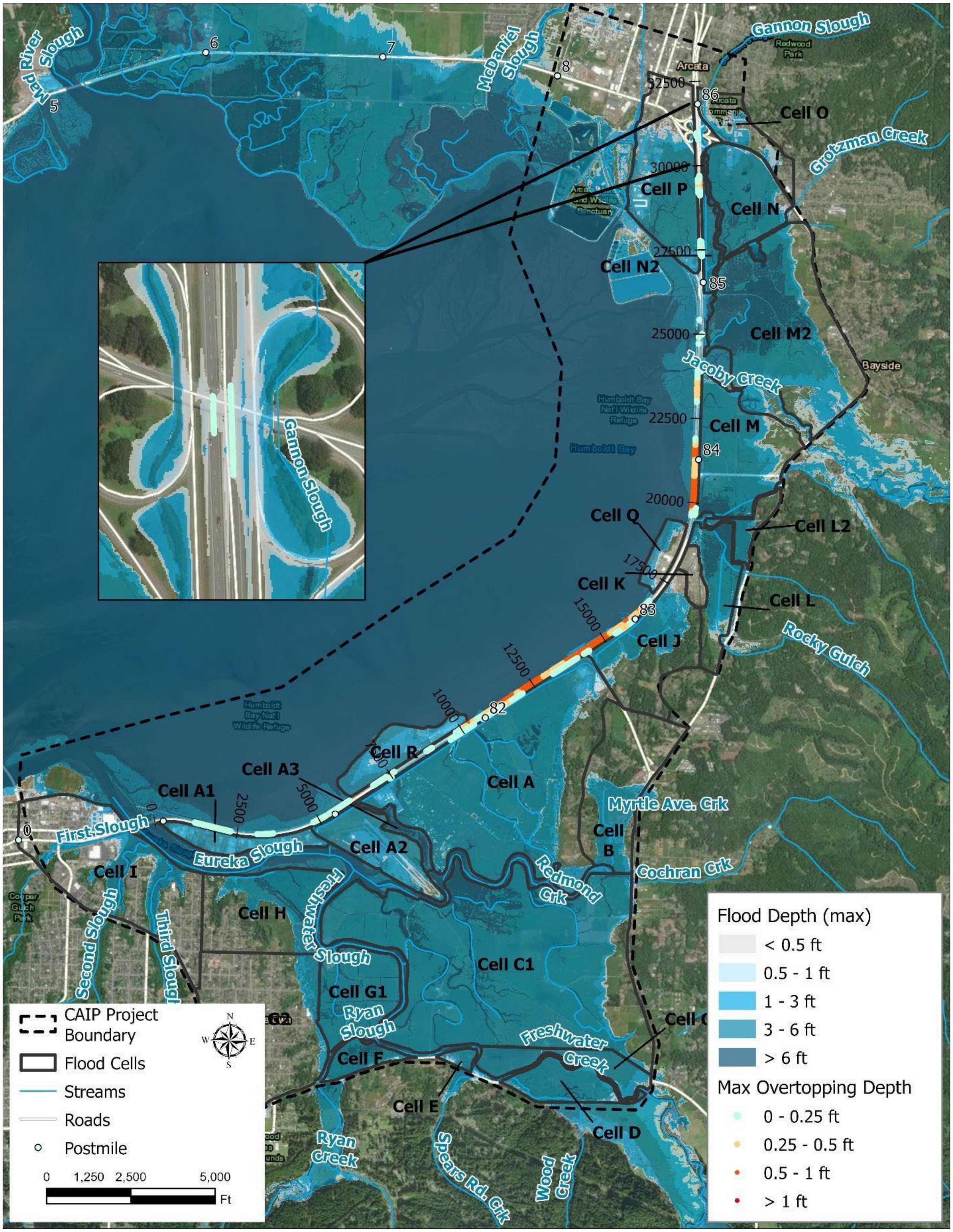

Figure A-5. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the MMMW and Q2 event scenario with 3.12 ft of sea level rise.

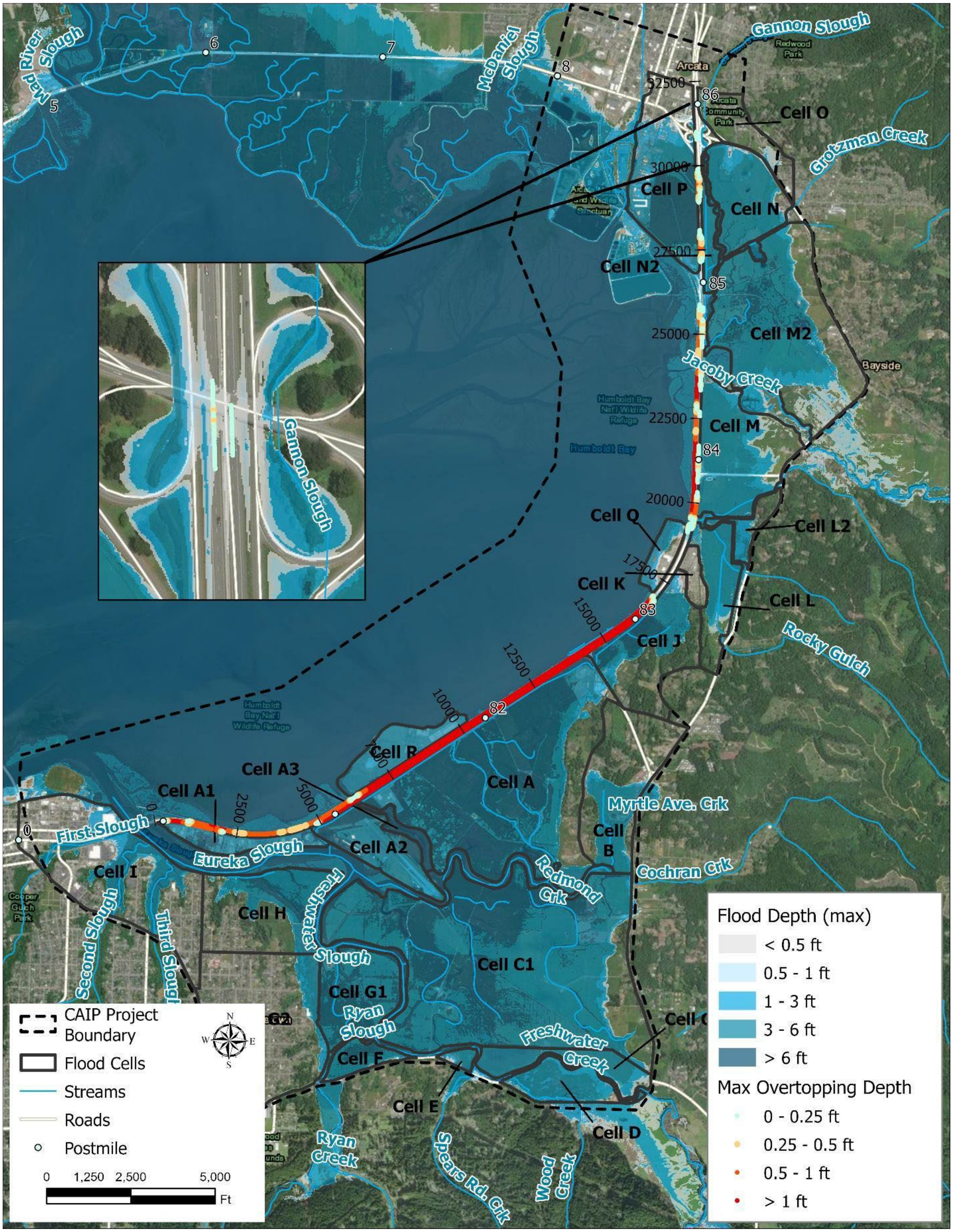

Figure A-6. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the coastal 2-yr and fluvial 2-yr event scenario with 3.12 ft of sea level rise.

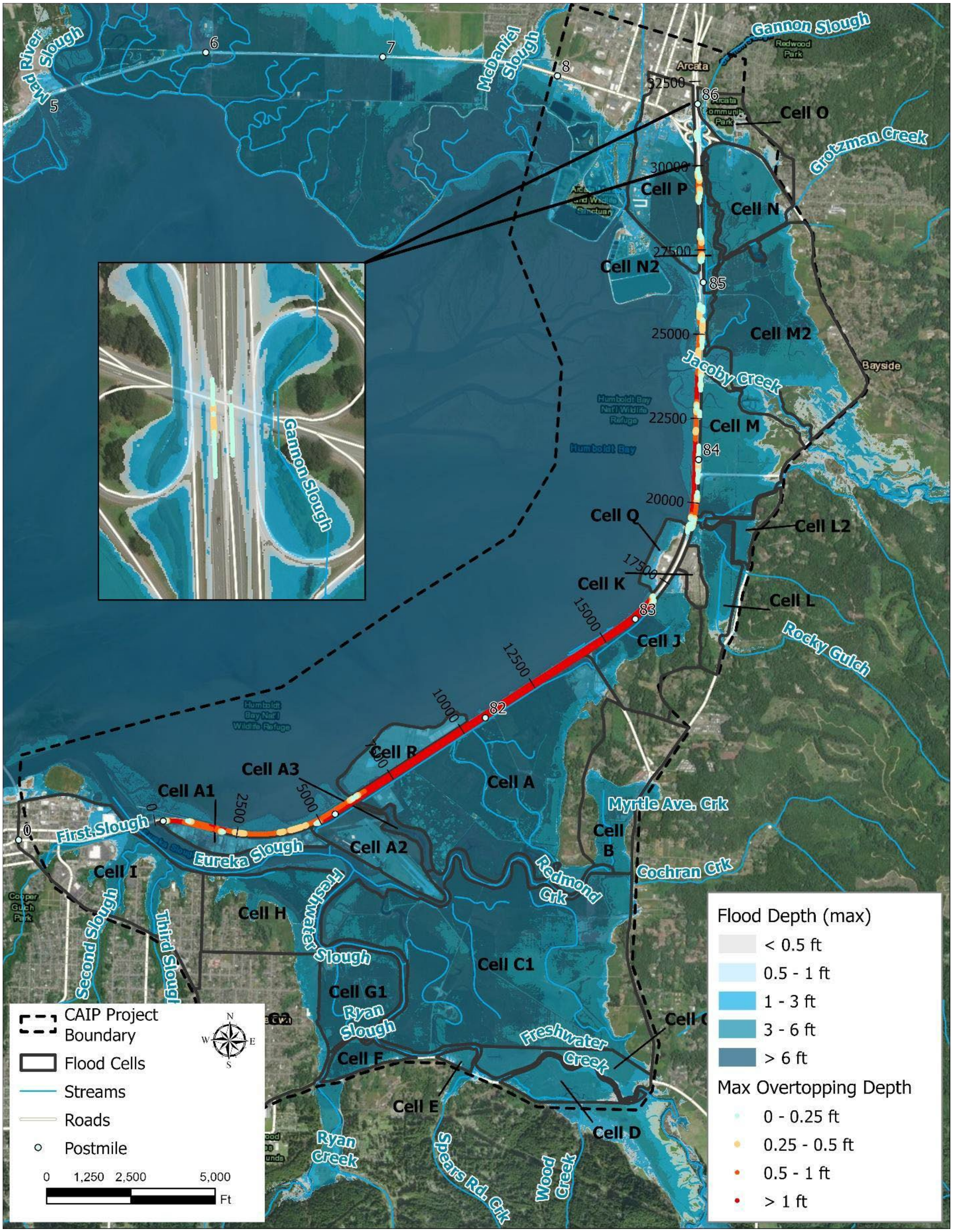

Figure A-7. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the coastal 2-yr and fluvial 10-yr event scenario with 3.12 ft of sea level rise.

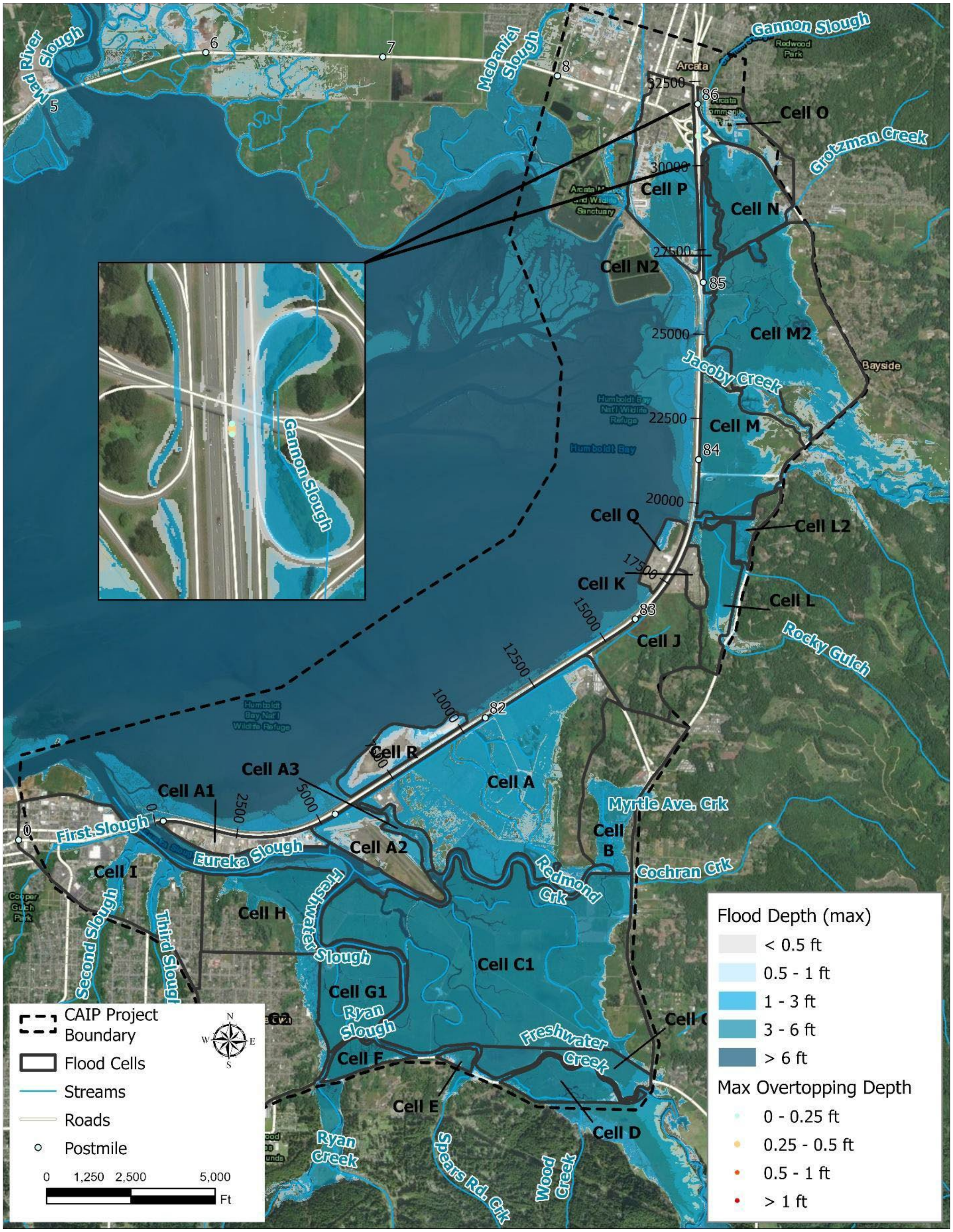

Figure A-8. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the coastal 2-yr and fluvial 100-yr event scenario.

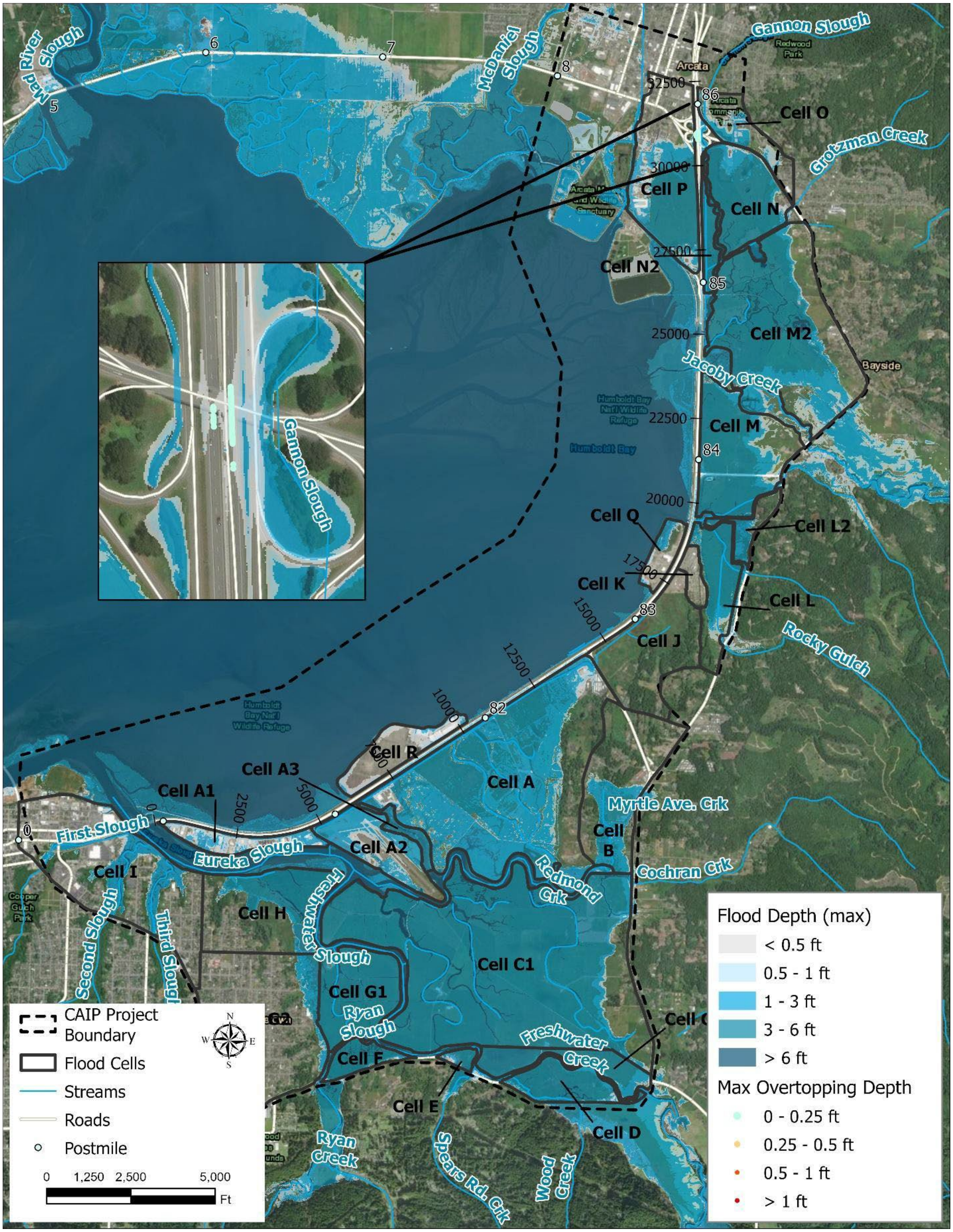

Figure A-9. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the coastal 2-yr and fluvial 100-yr event scenario with 0.82 ft of sea level rise.

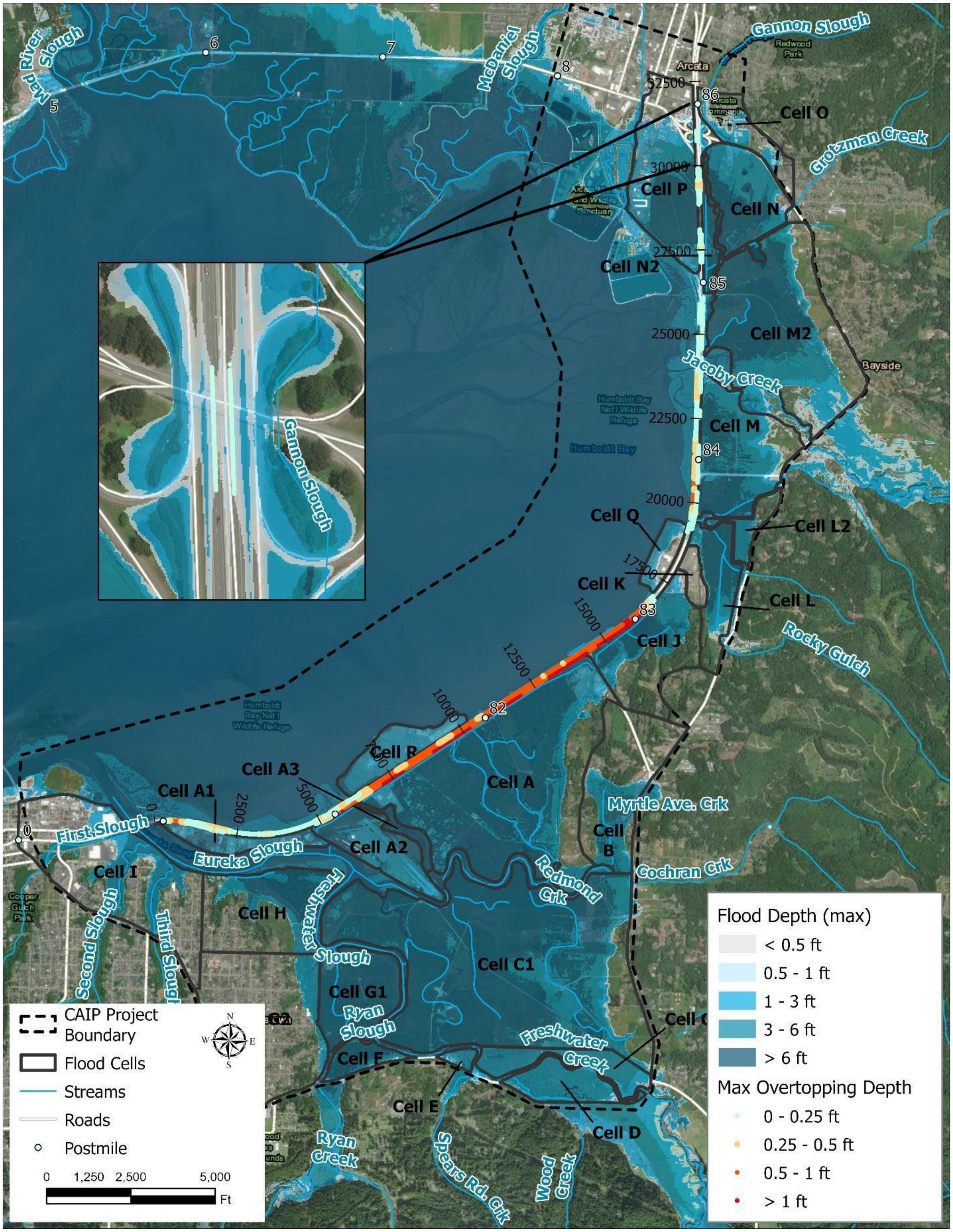

Figure A-10. Maximum flood depths across the model domain, including the maximum overtopping depths on the northbound and southbound lanes of Highway 101 during the coastal 2-yr and fluvial 100-yr event scenario with 3.12 ft of sea level rise.

Appendix B – Inundation Duration Maps

This appendix contains maps depicting flood inundation duration (in hours) and the duration during which road overtopping depths exceeded 4 inches along the Highway 101 Corridor for all event scenarios in which either the southbound or northbound centerlines of Highway 101 were overtopped.

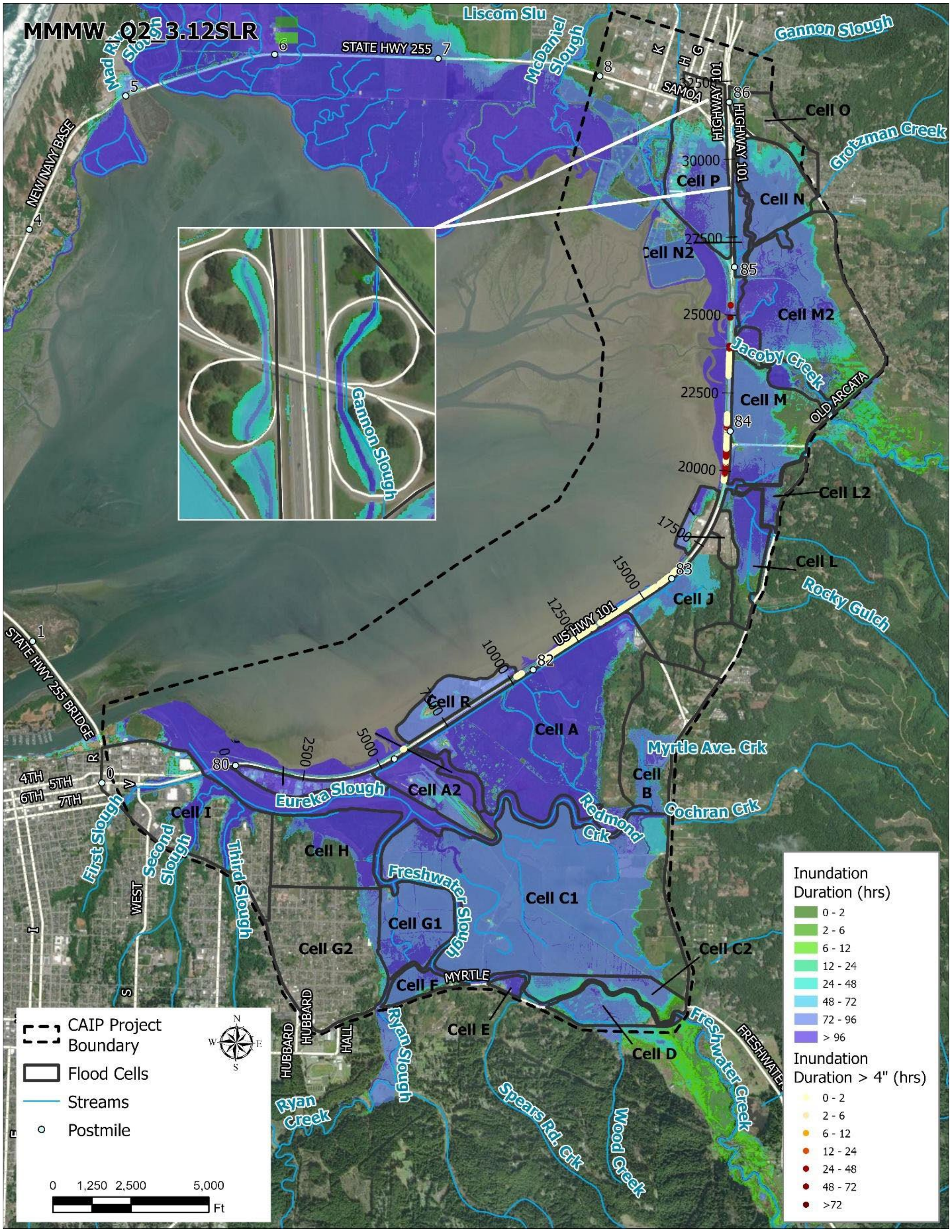

Figure B-1. Modeled inundation duration for the existing MMMW (8.36 ft) tidal boundary conditions, coupled with a 2-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

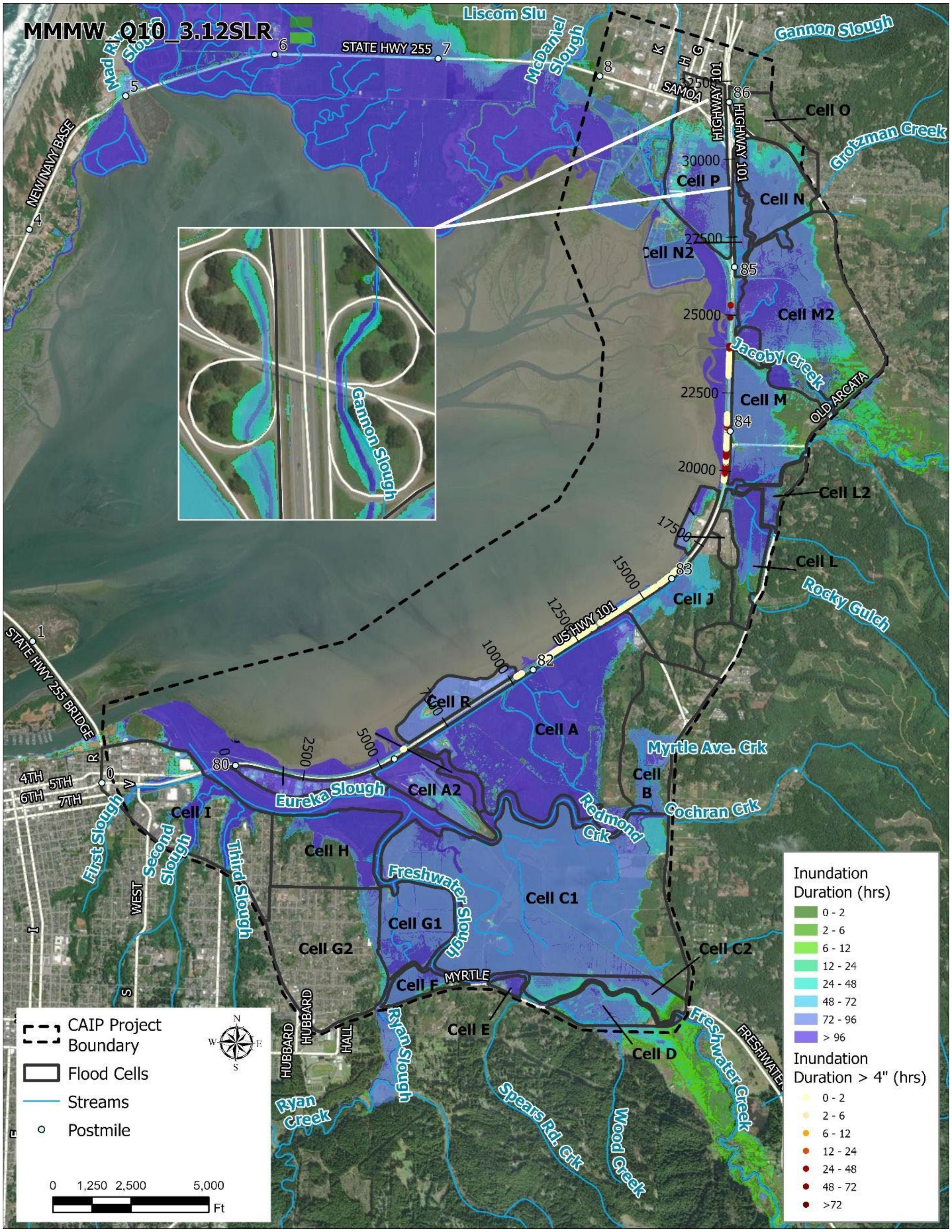

Figure B-2. Modeled inundation duration for the existing MMMW (8.36 ft) tidal boundary conditions, coupled with a 10-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

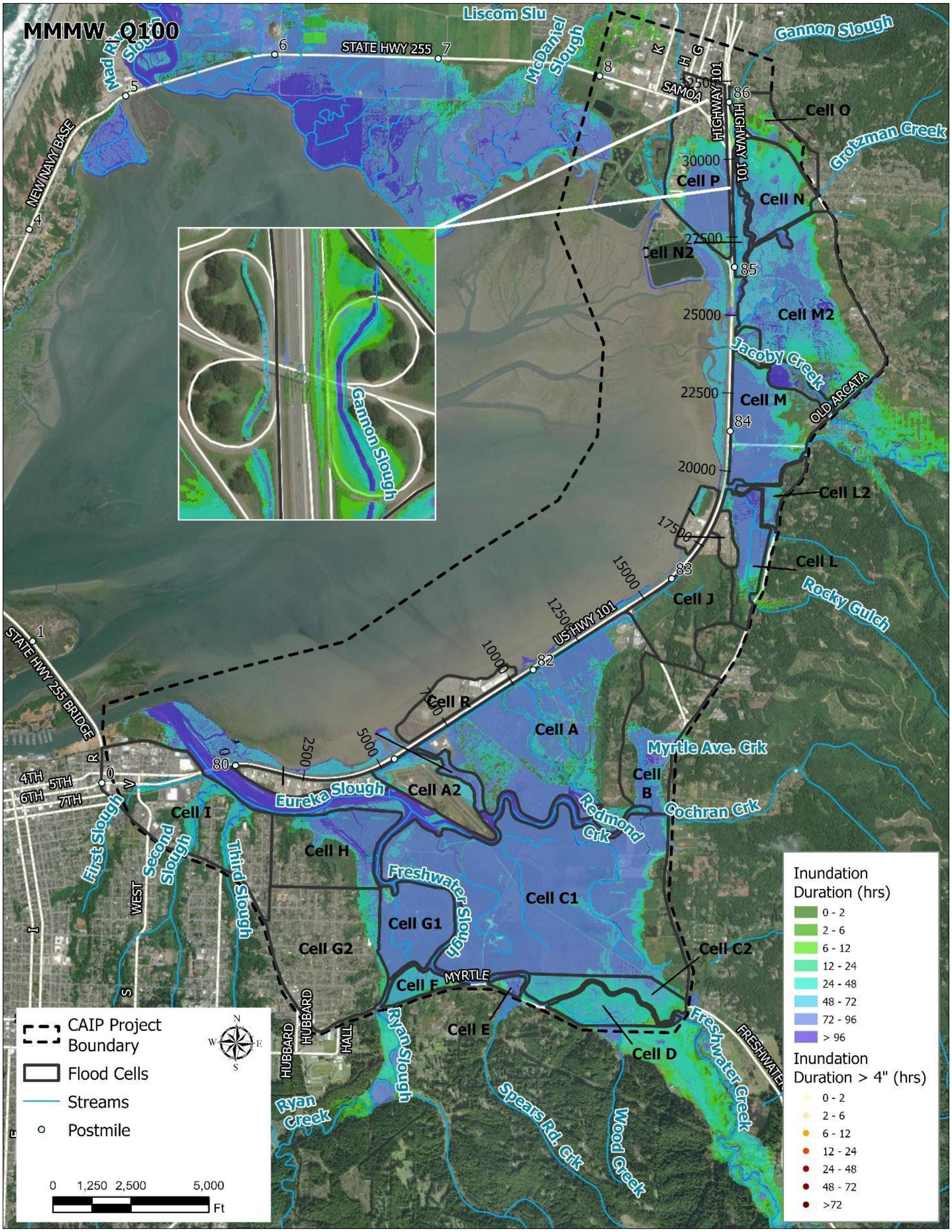

Figure B-3. Modeled inundation duration for the existing MMMW (8.36 ft) tidal boundary conditions, coupled with a 2-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

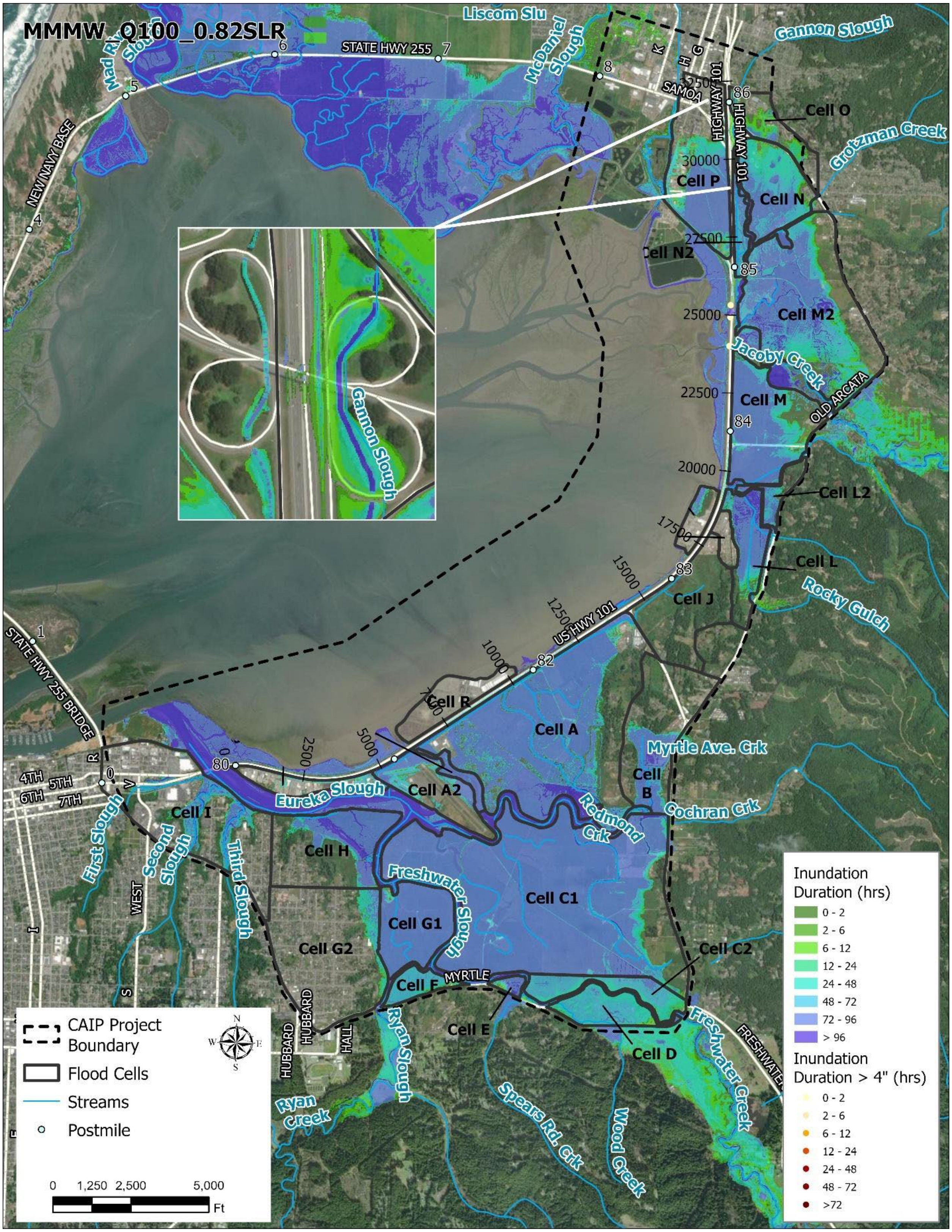

Figure B-4. Modeled inundation duration for the existing MMMW (8.36 ft) tidal boundary conditions, coupled with a 100-year fluvial flood event and 0.82 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

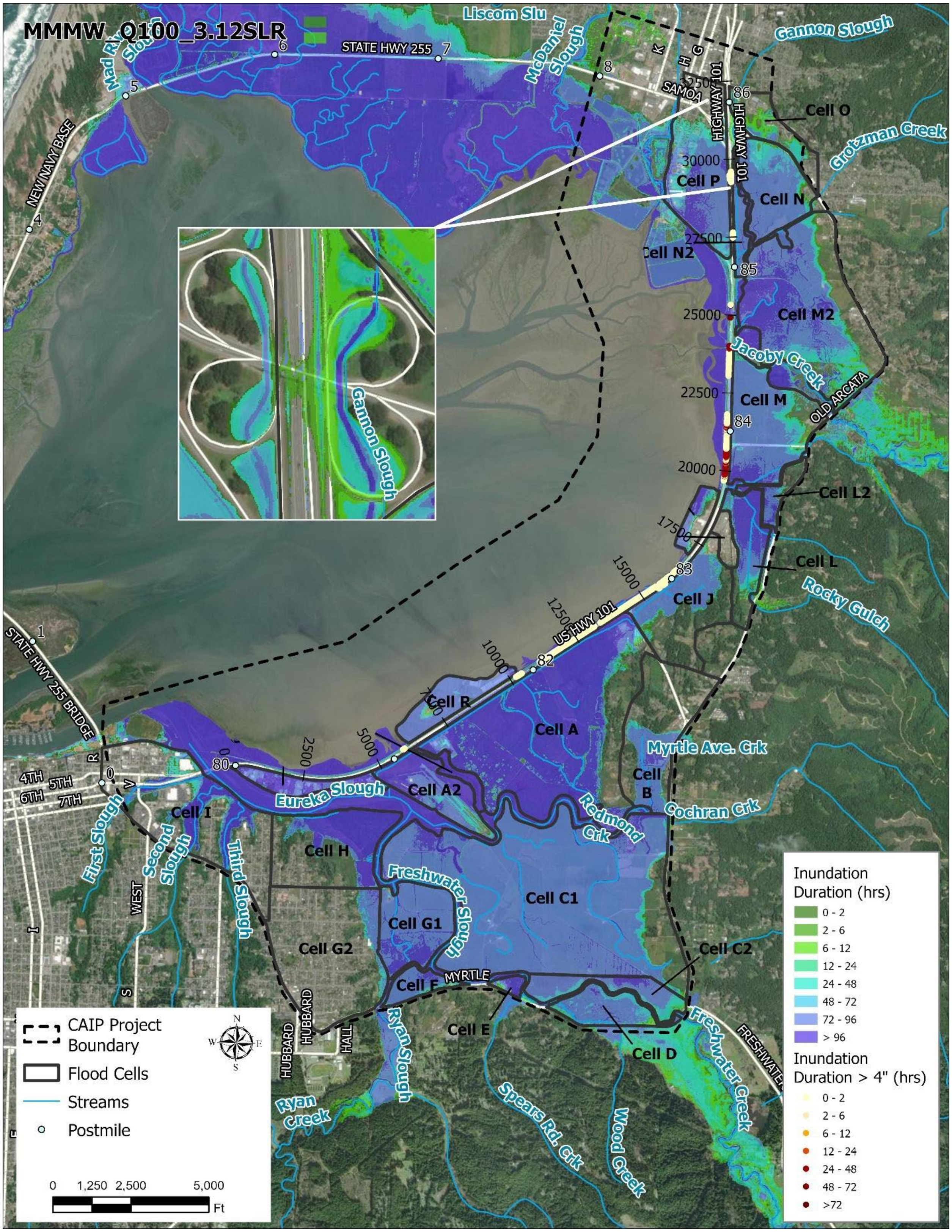

Figure 38. Modeled inundation duration for the existing MMMW (8.36 ft) tidal boundary conditions, coupled with a 100-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

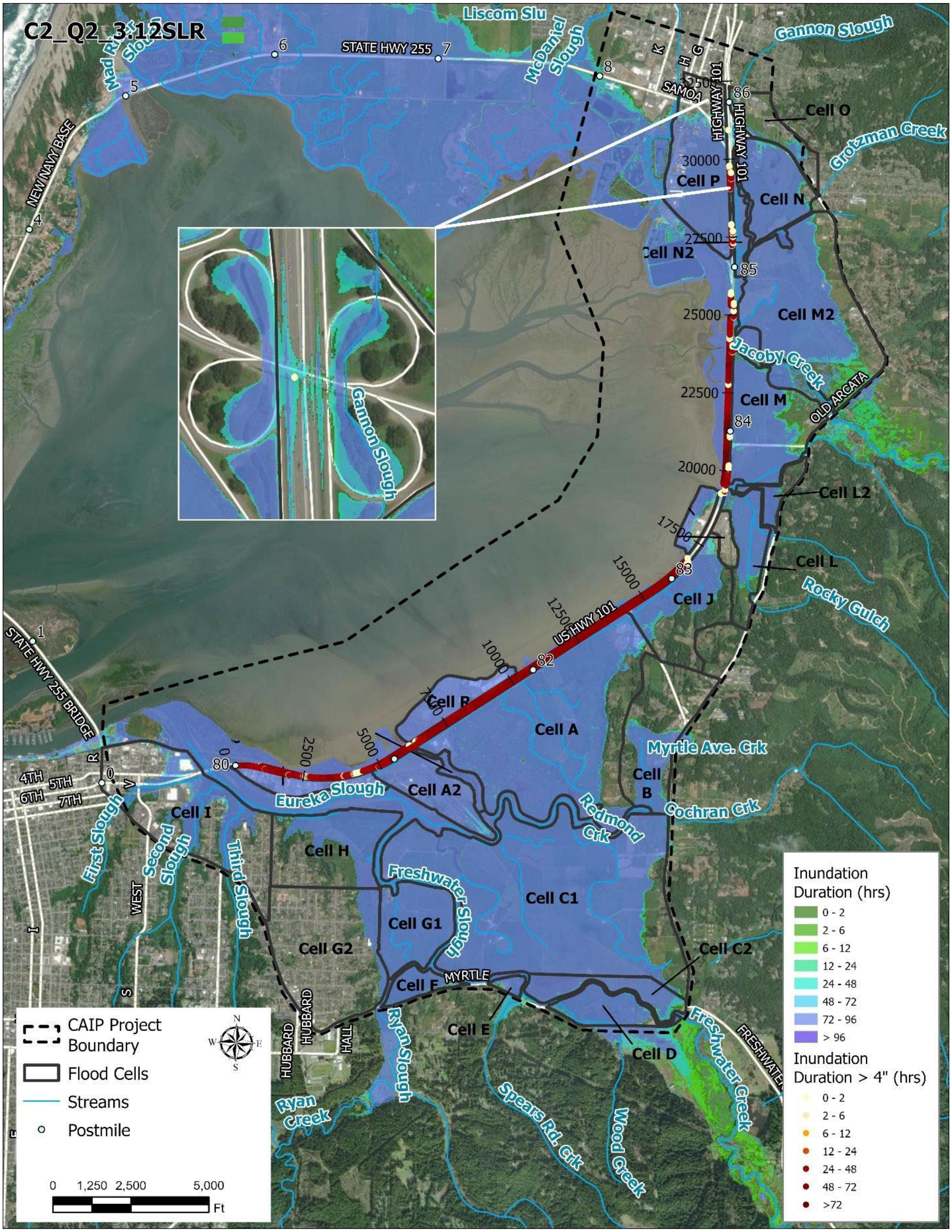

Figure B-6. Modeled inundation duration for the coastal 2-yr storm (9.26 ft), coupled with a 2-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

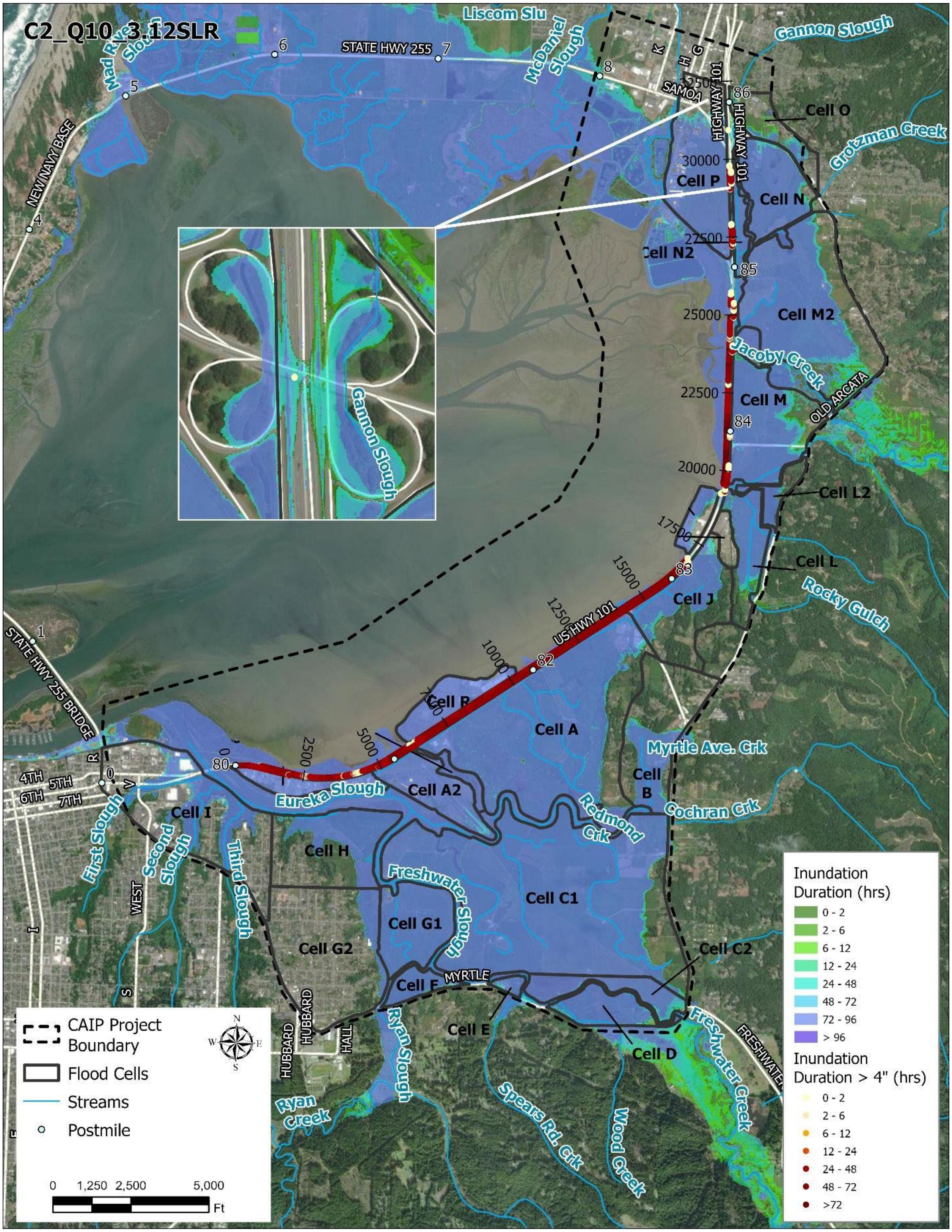

Figure B-7. Modeled inundation duration for the coastal 2-yr storm (9.26 ft), coupled with a 10-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

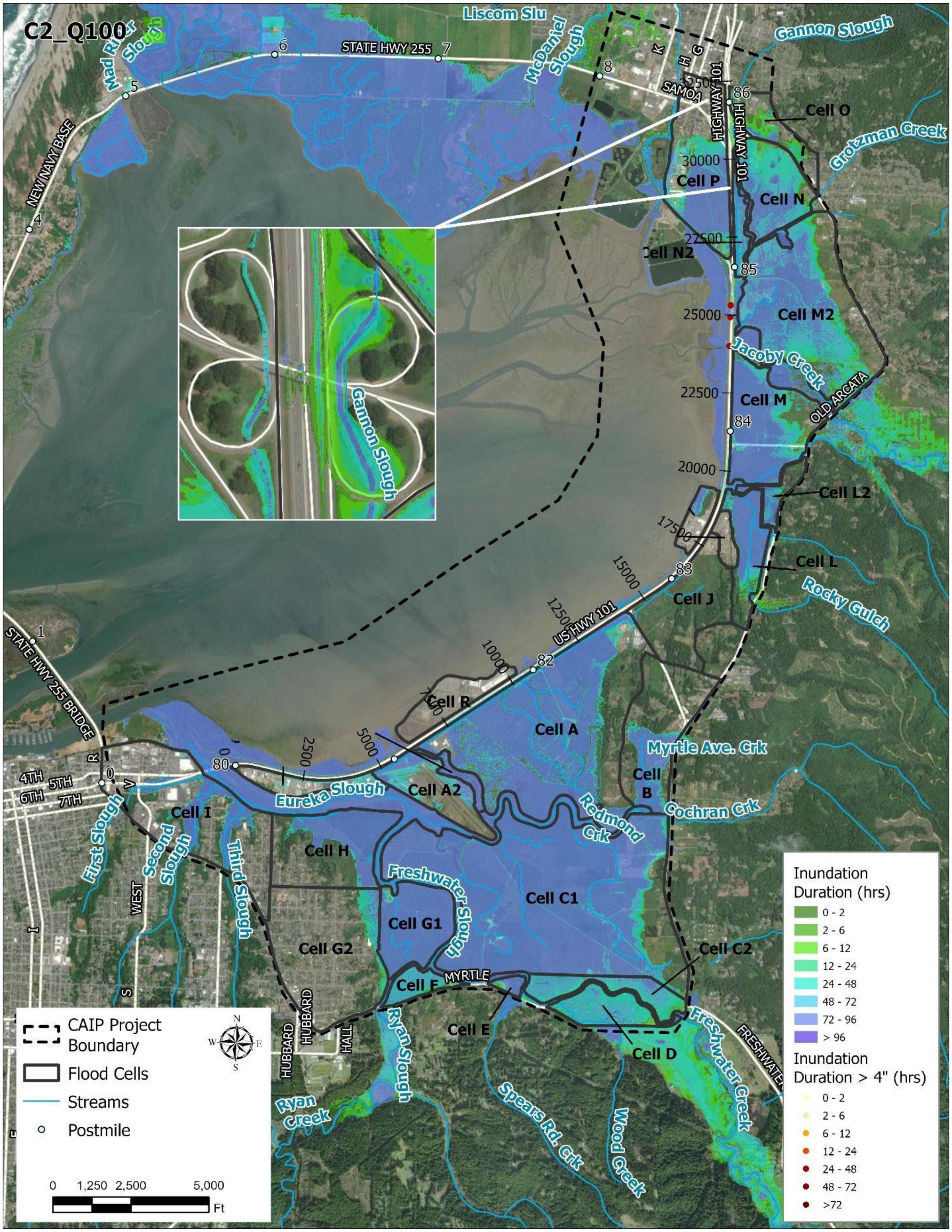

Figure B-8. Modeled inundation duration for the coastal 2-yr storm (9.26 ft), coupled with a 100-year fluvial flood event. The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

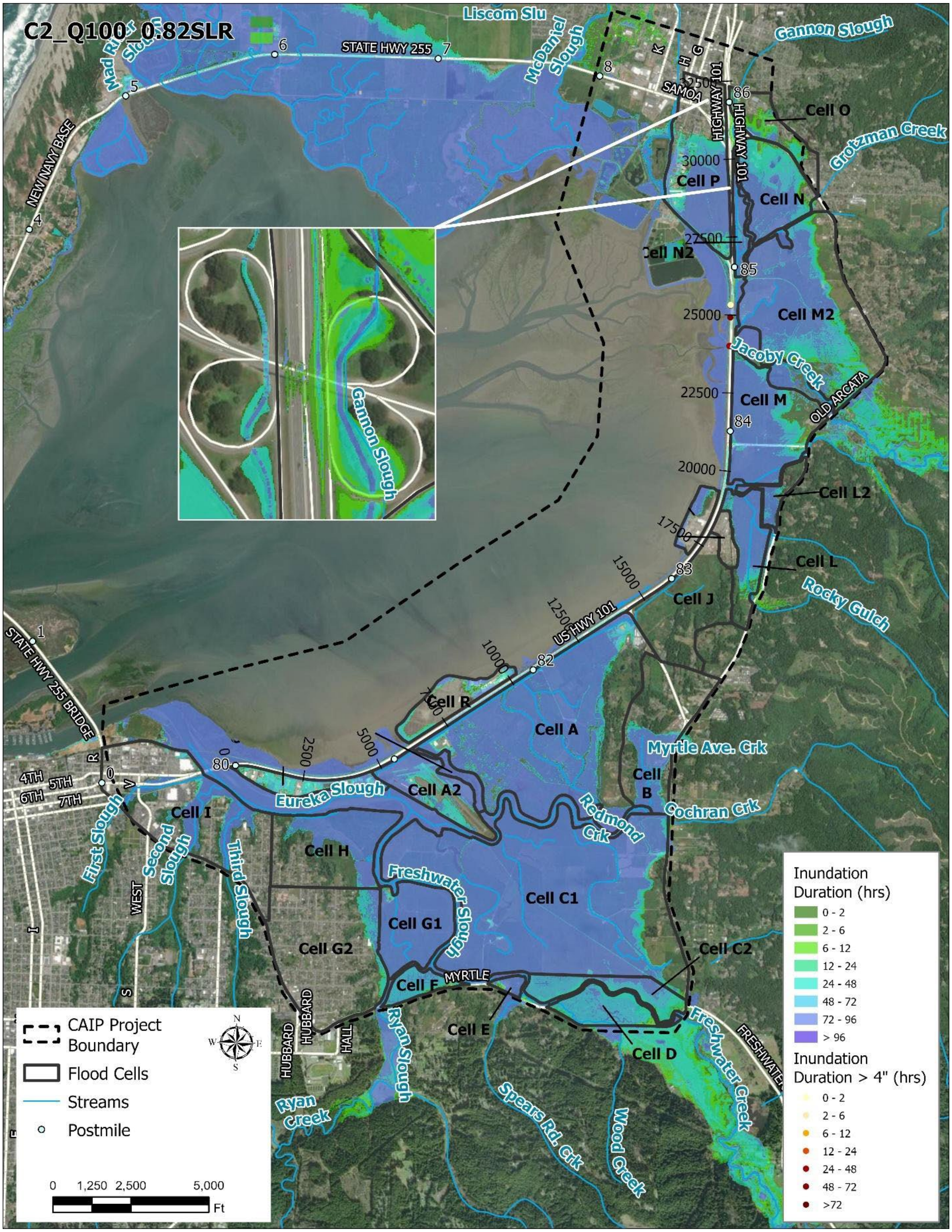

Figure B-9. Modeled inundation duration for the coastal 2-yr storm (9.26 ft), coupled with a 100-year fluvial flood event and 0.82 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

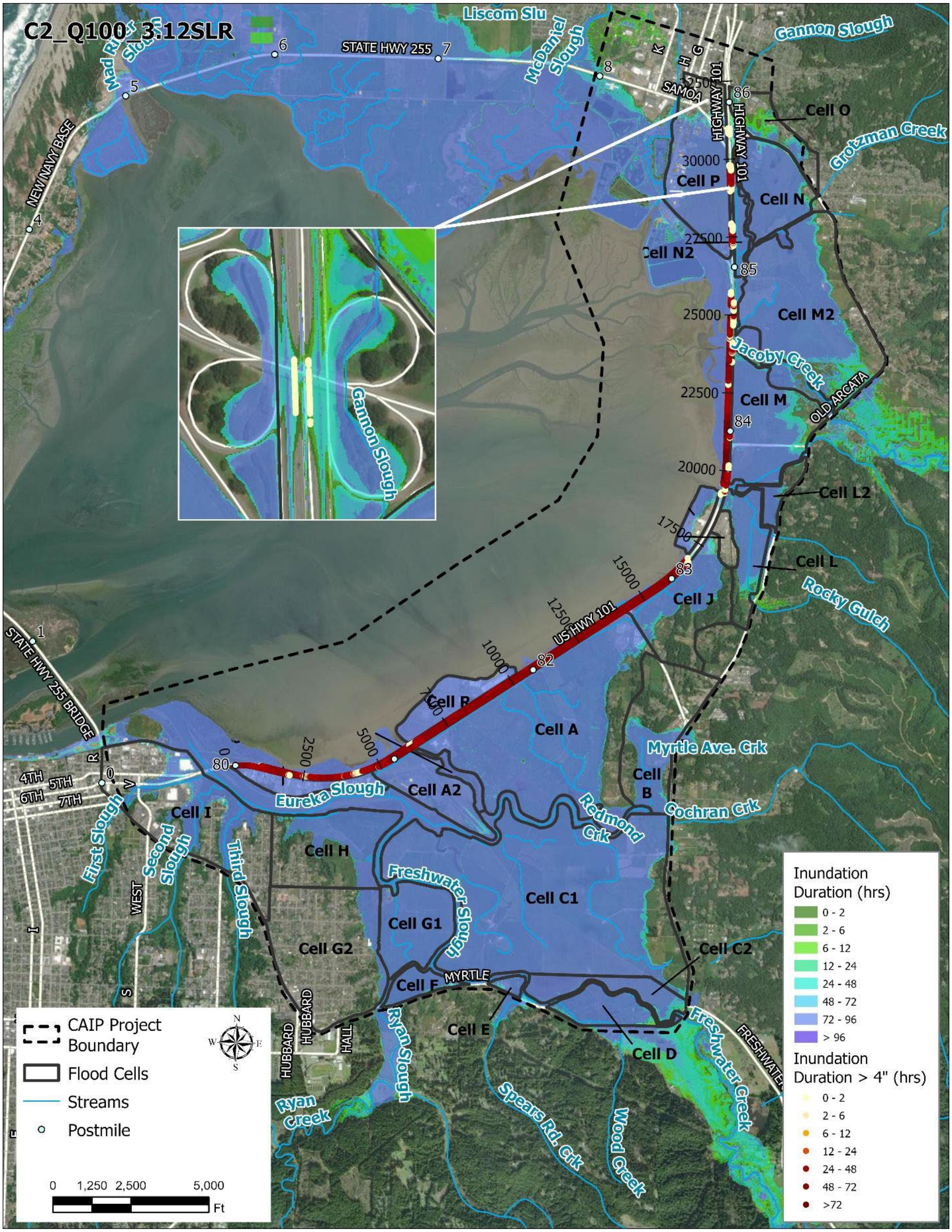

Figure B-10. Modeled inundation duration for the coastal 2-yr storm (9.26 ft), coupled with a 100-year fluvial flood event and 3.12 ft of sea level rise (SLR). The duration of road overtopping at the Highway 101 northbound and southbound centerlines, where water depths exceed 4 inches, is shown as yellow to red points.

Appendix C – Performance Plots: Culverts & Tide Gates

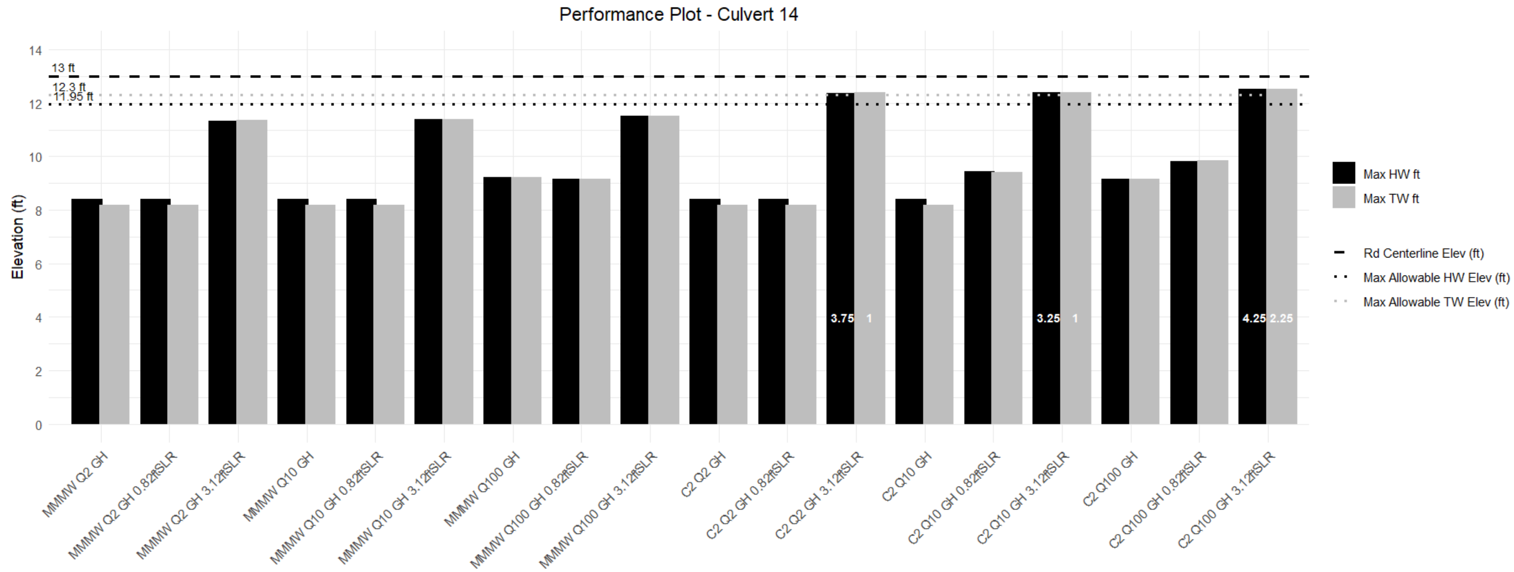

Figure 39. Performance plot for Culvert 14 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

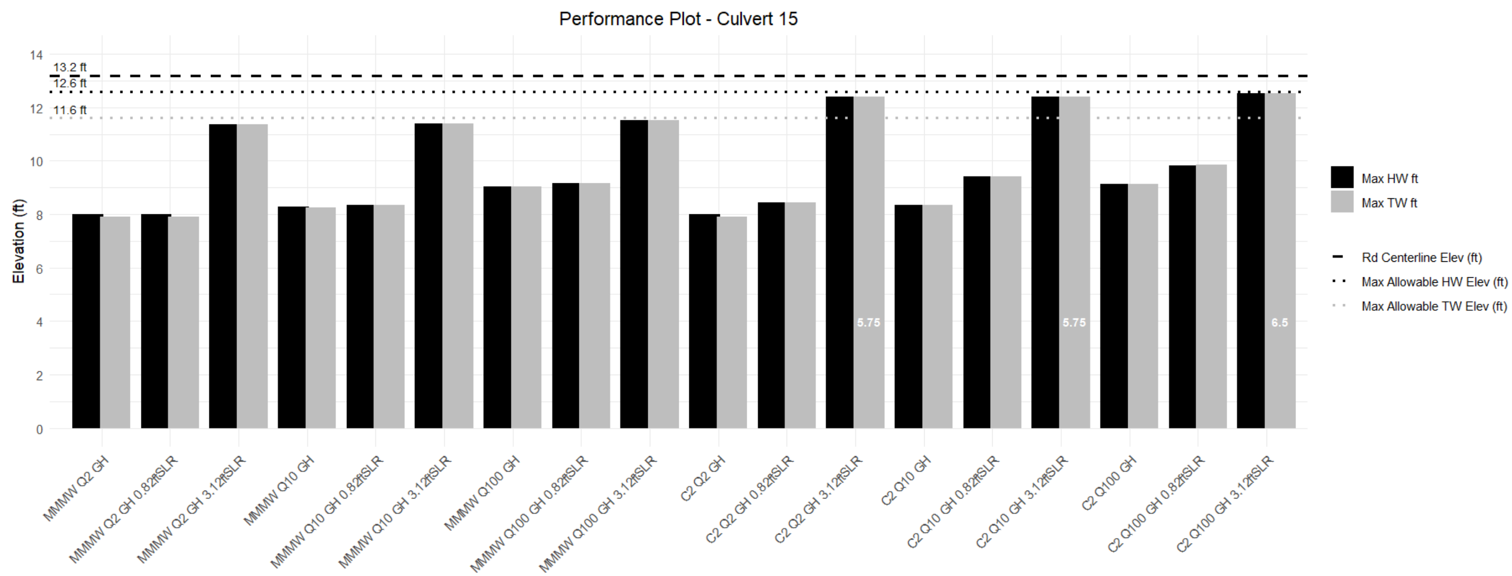

Figure 40. Performance plot for Culvert 15 over a suite of event conditions.

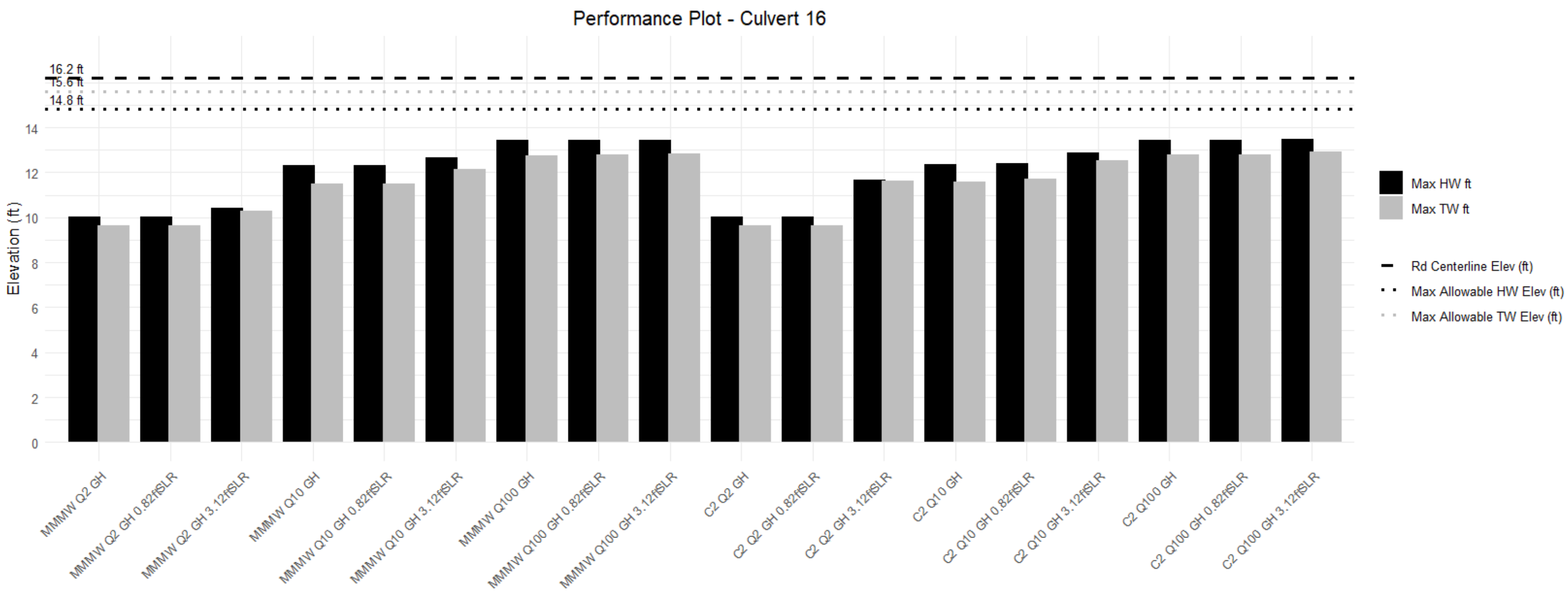

Figure C-3. Performance plot for Culvert 16 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

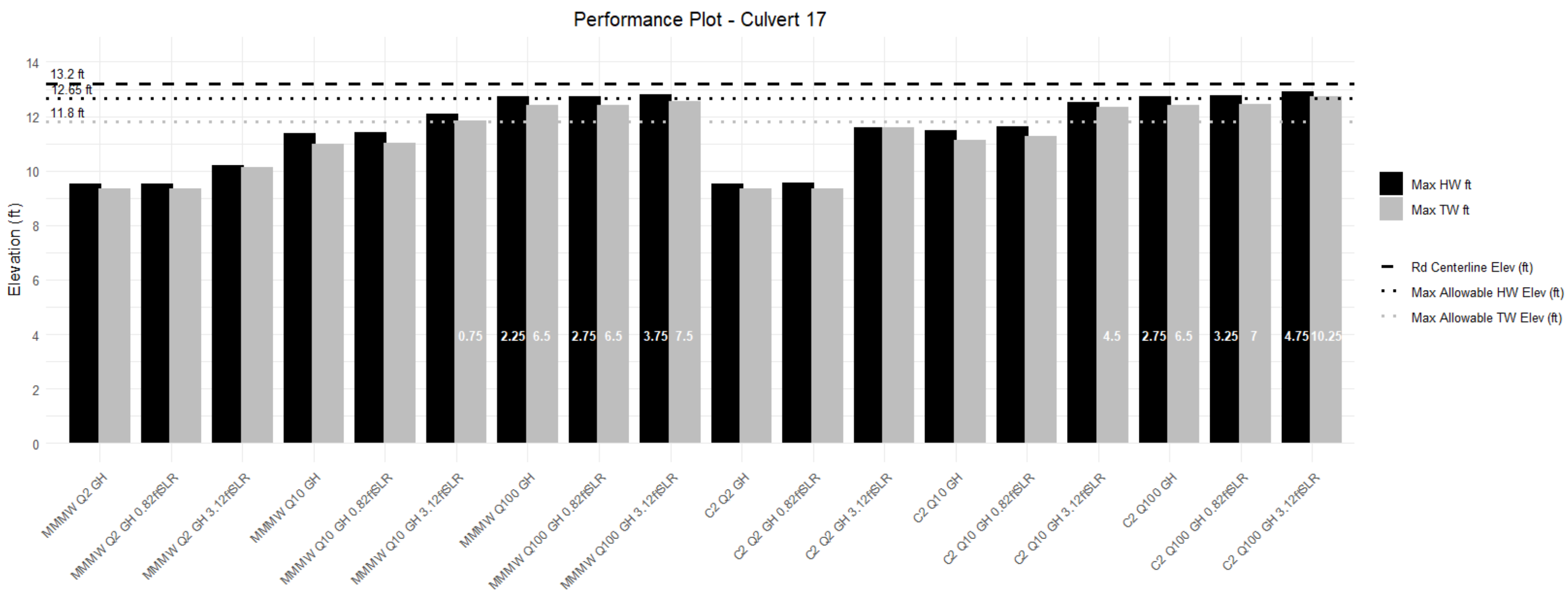

Figure C-4. Performance plot for Culvert 17 over a suite of event conditions.

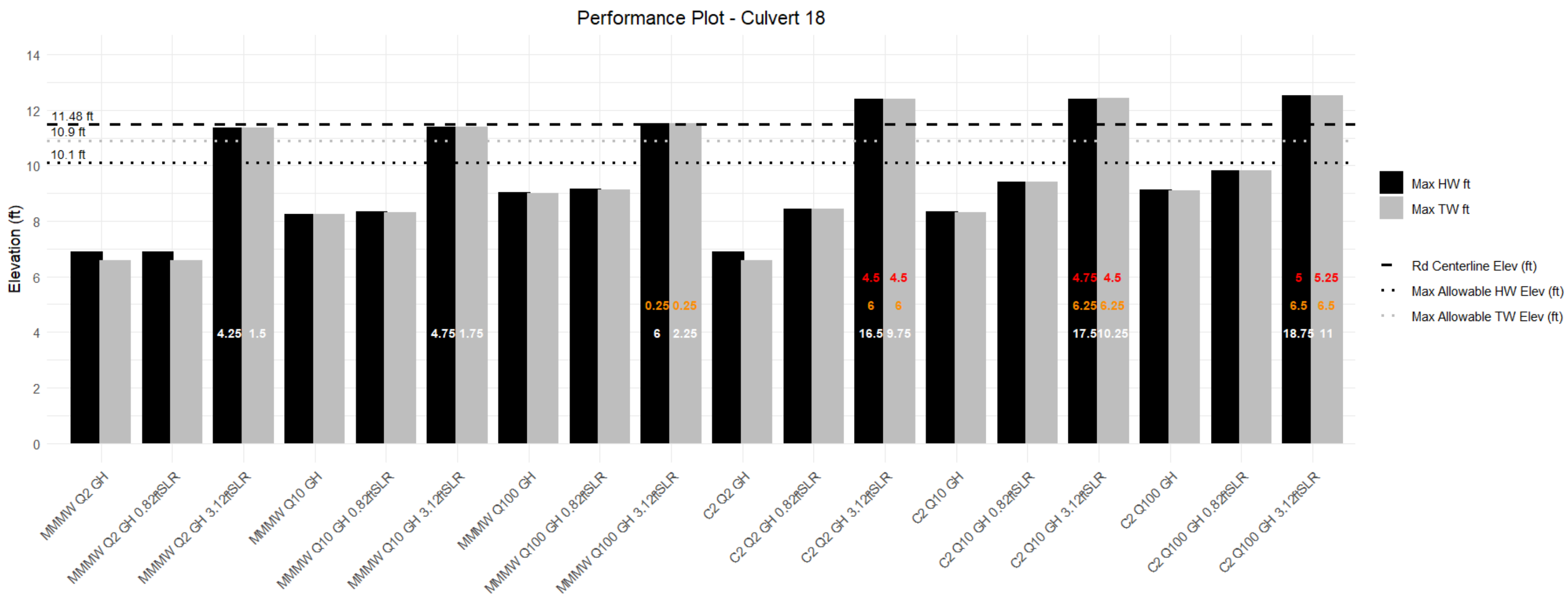

Figure C-5. Performance plot for Culvert 18 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

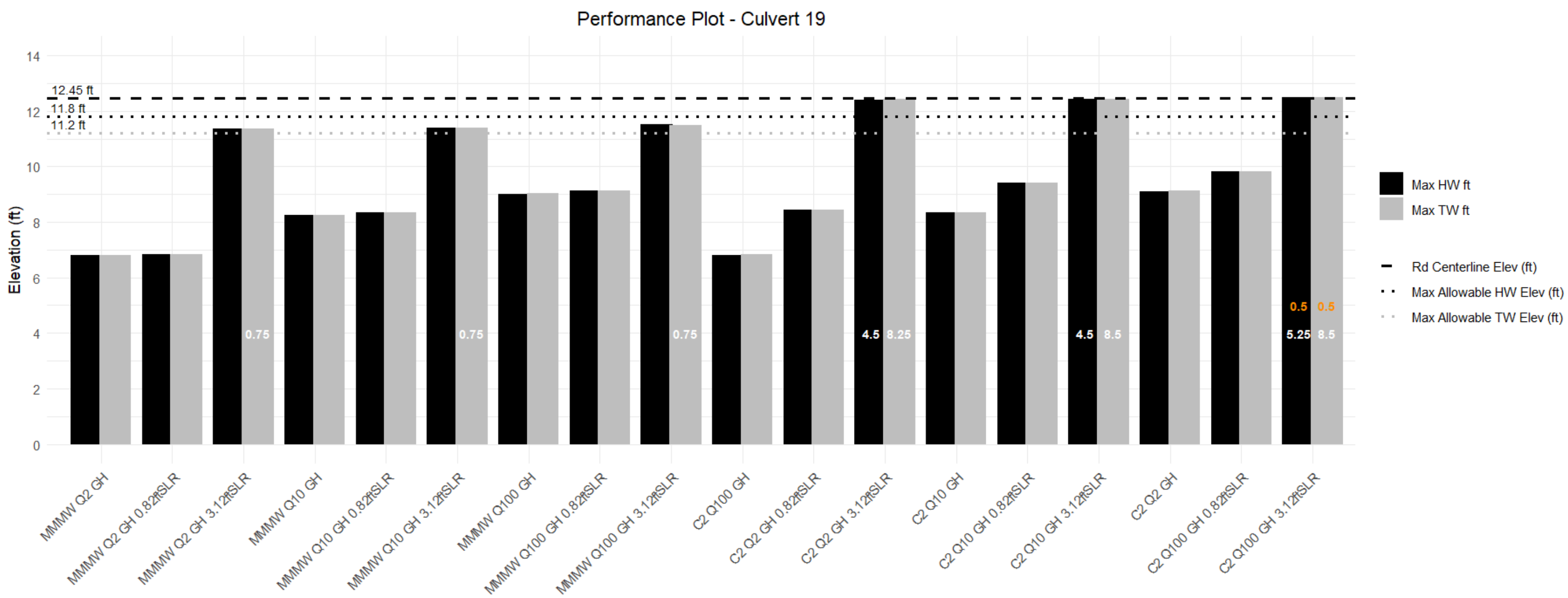

Figure C-6. Performance plot for Culvert 17 over a suite of event conditions.

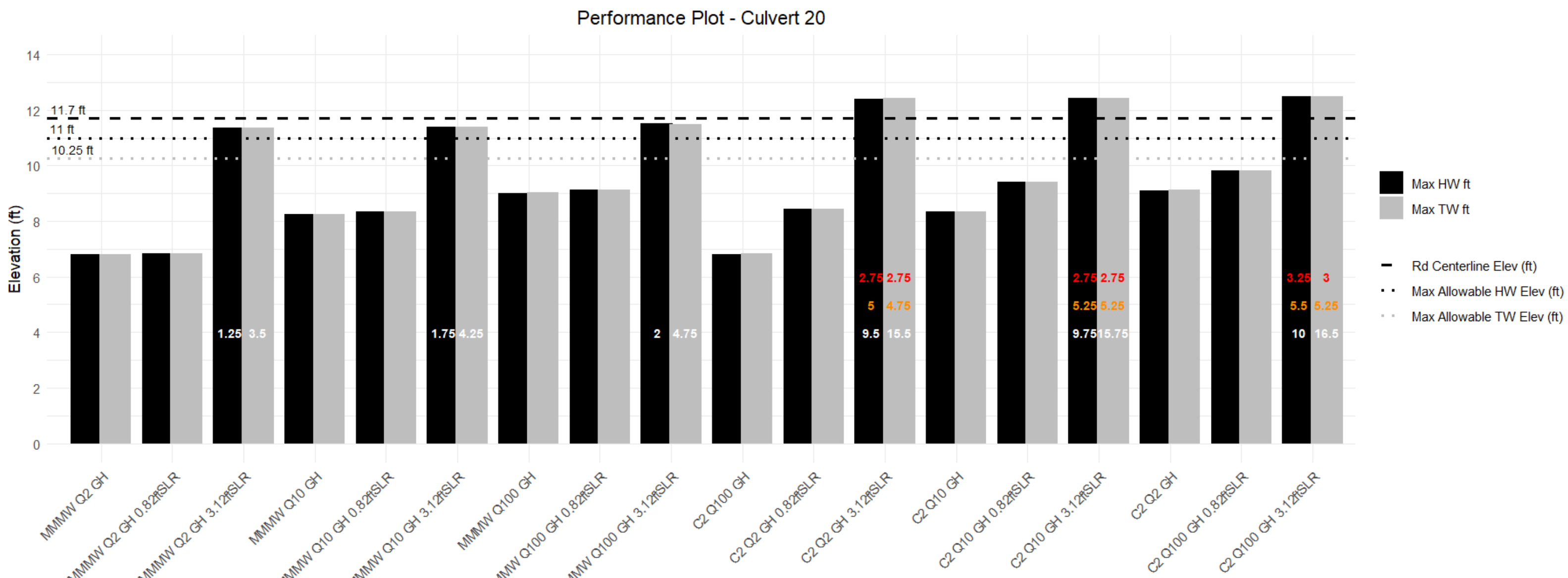

Figure C-7. Performance plot for Culvert 20 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

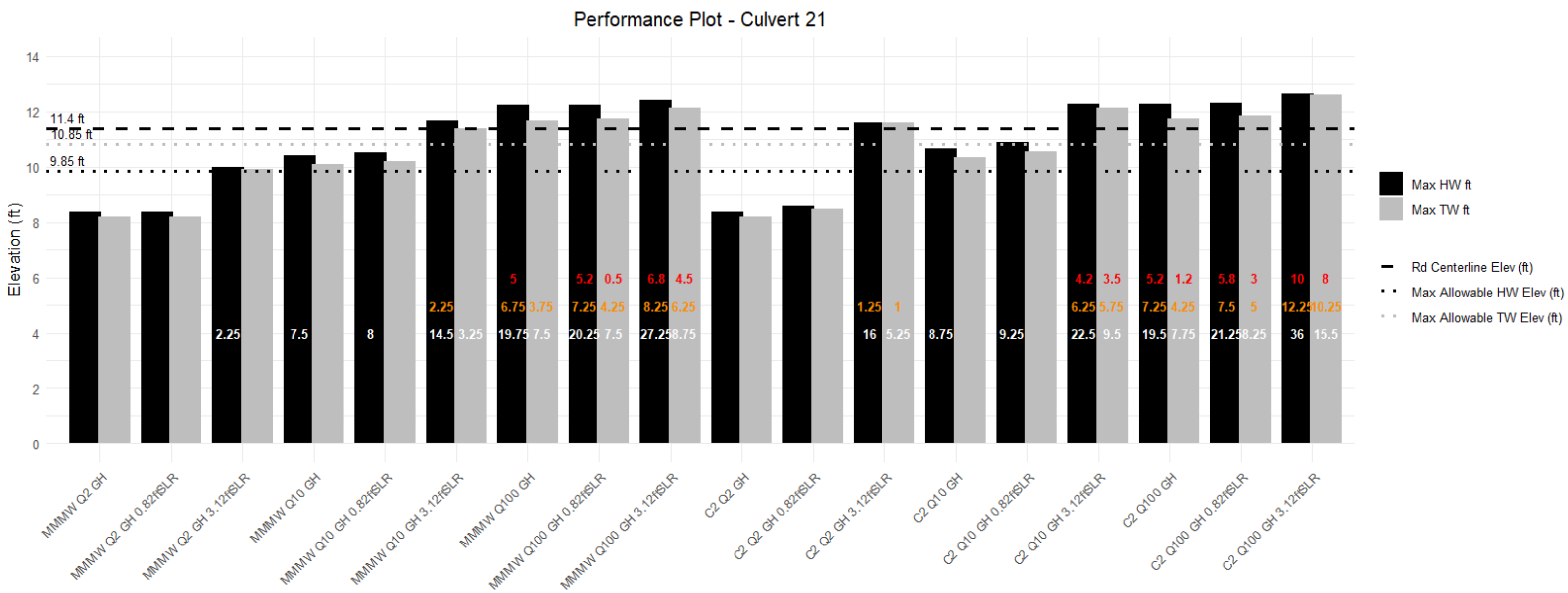

Figure C-8. Performance plot for Culvert 21 over a suite of event conditions.

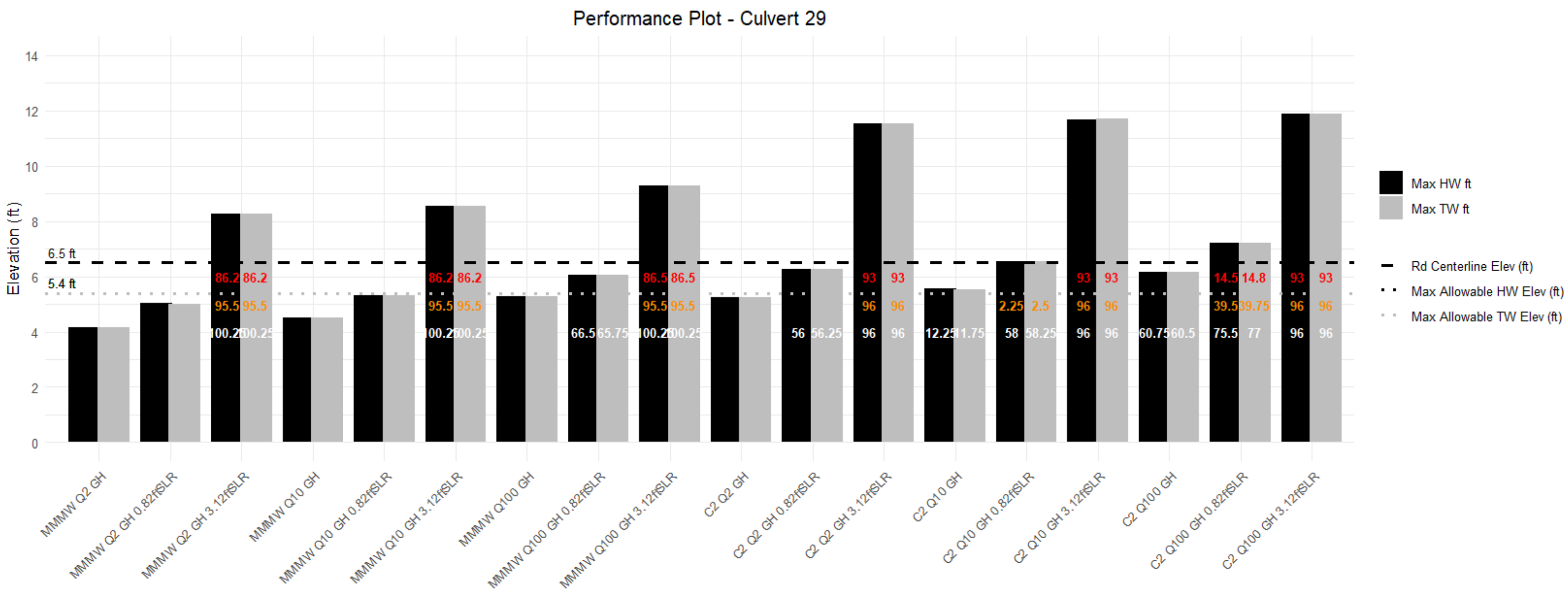

Figure C-9. Performance plot for Culvert 29 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

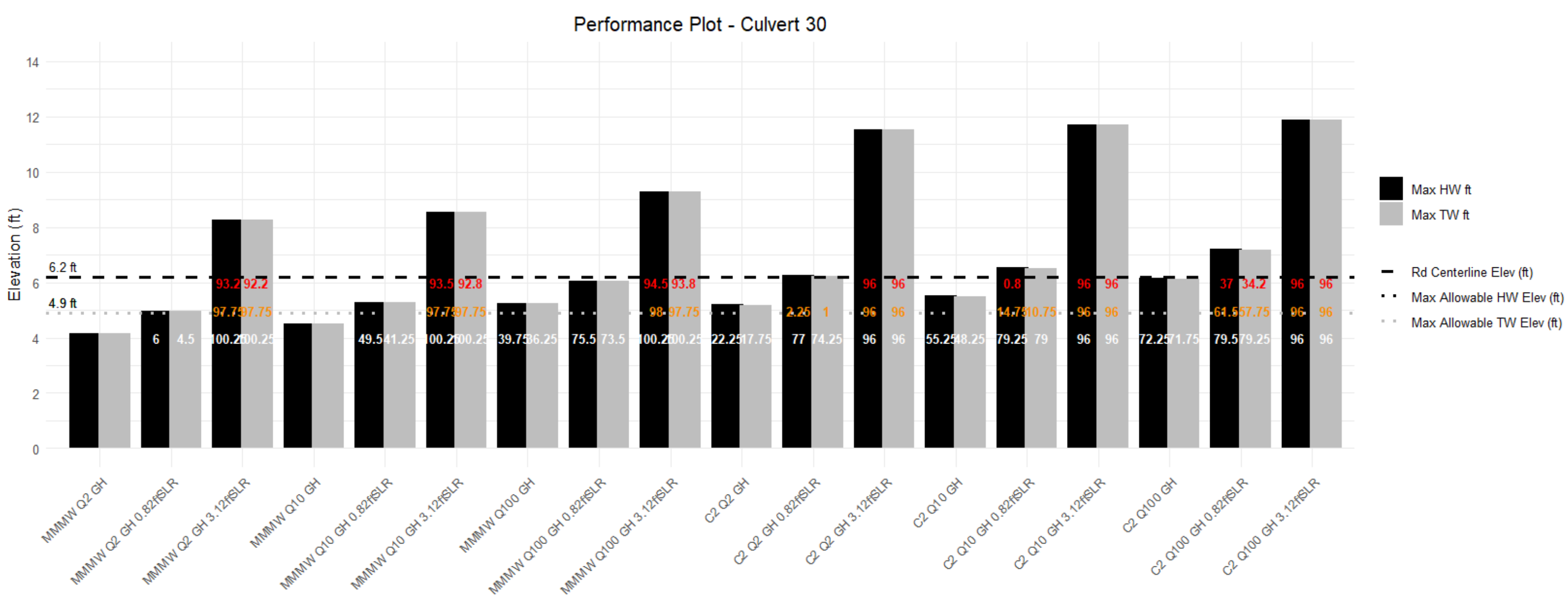

Figure C-10. Performance plot for Culvert 30 over a suite of event conditions.

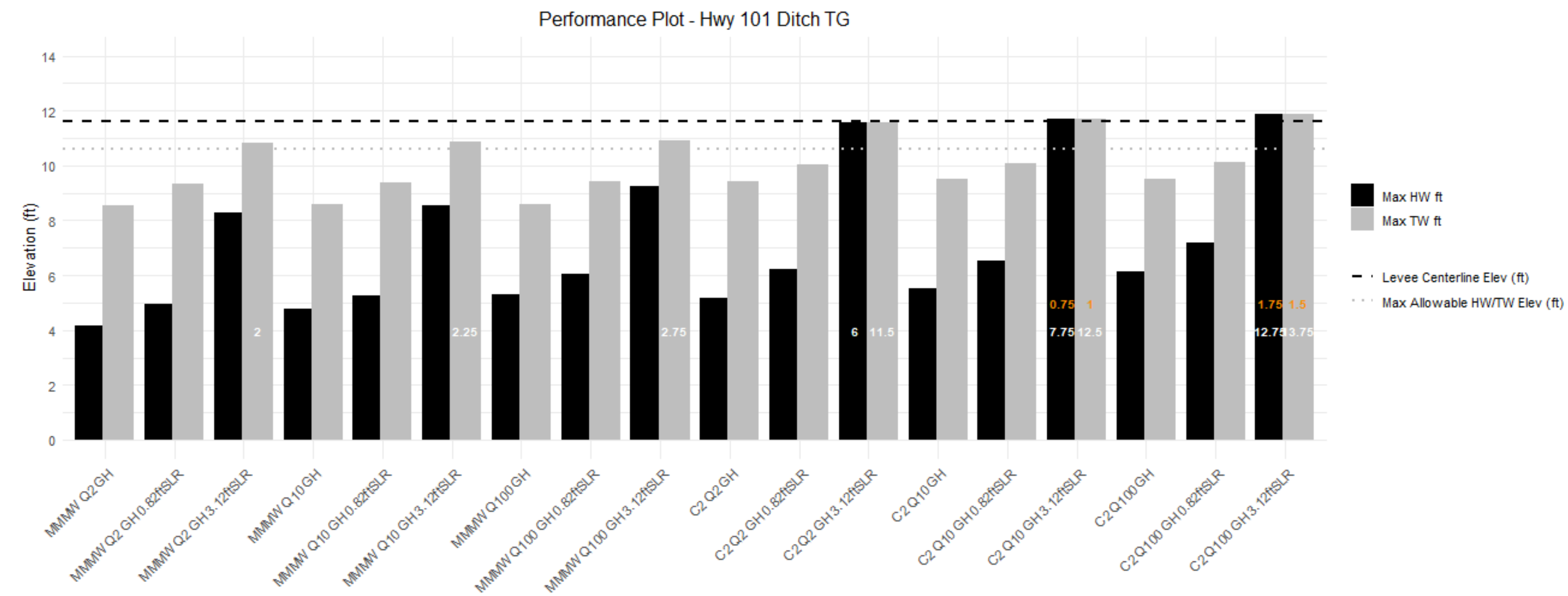

Figure C-11. Performance plot for the Hwy 101 Ditch Tide Gate over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

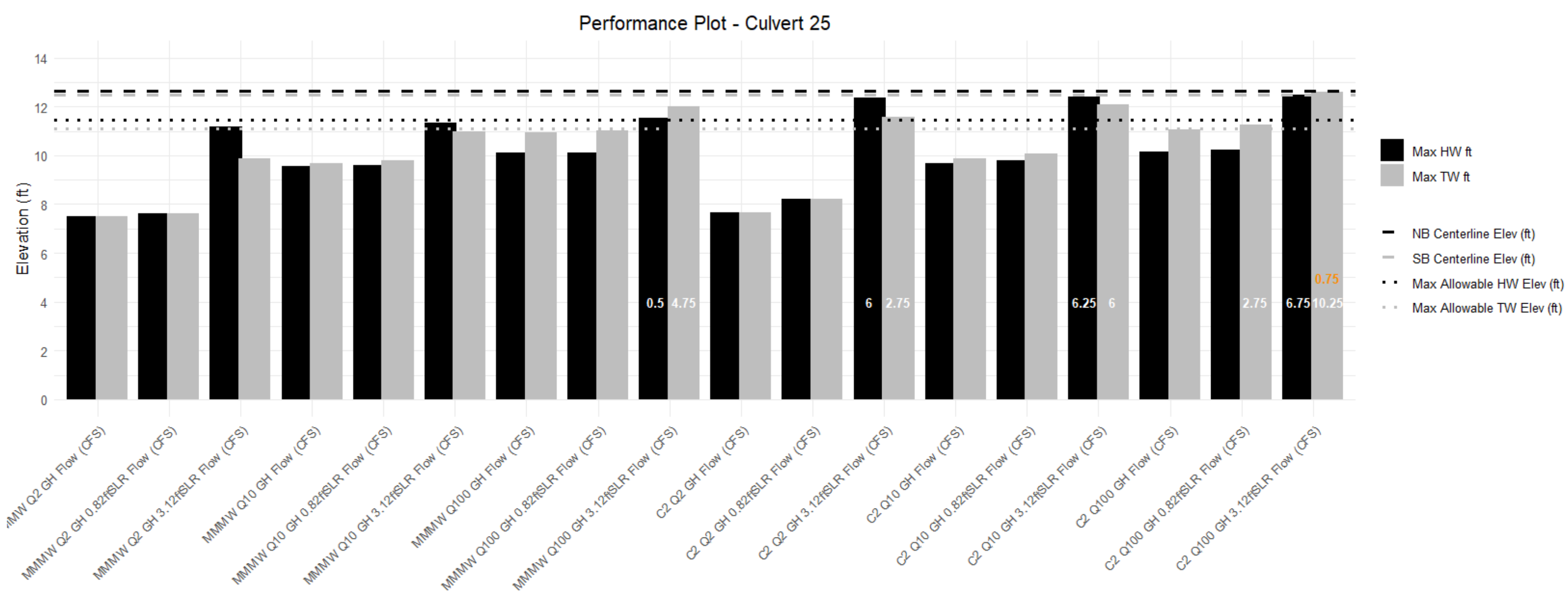

Figure C-12. Performance plot for Culvert 25 over a suite of event conditions.

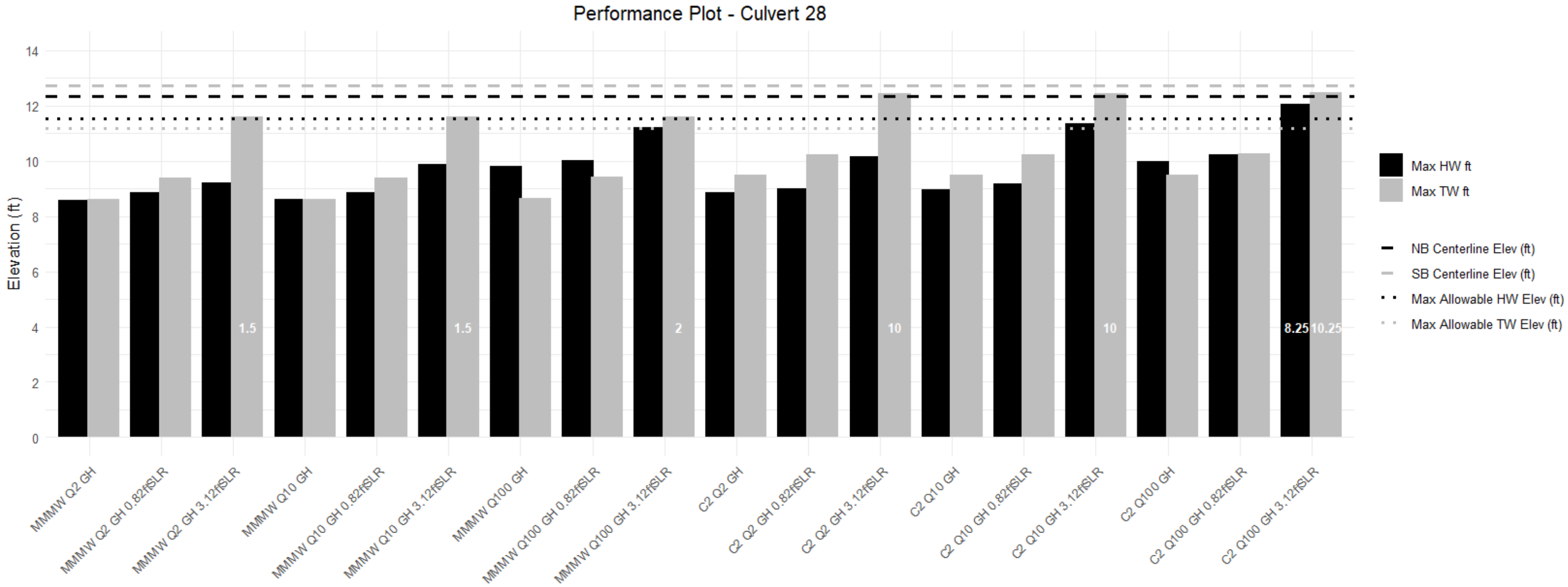

Figure C-13. Performance plot for Culvert 28 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

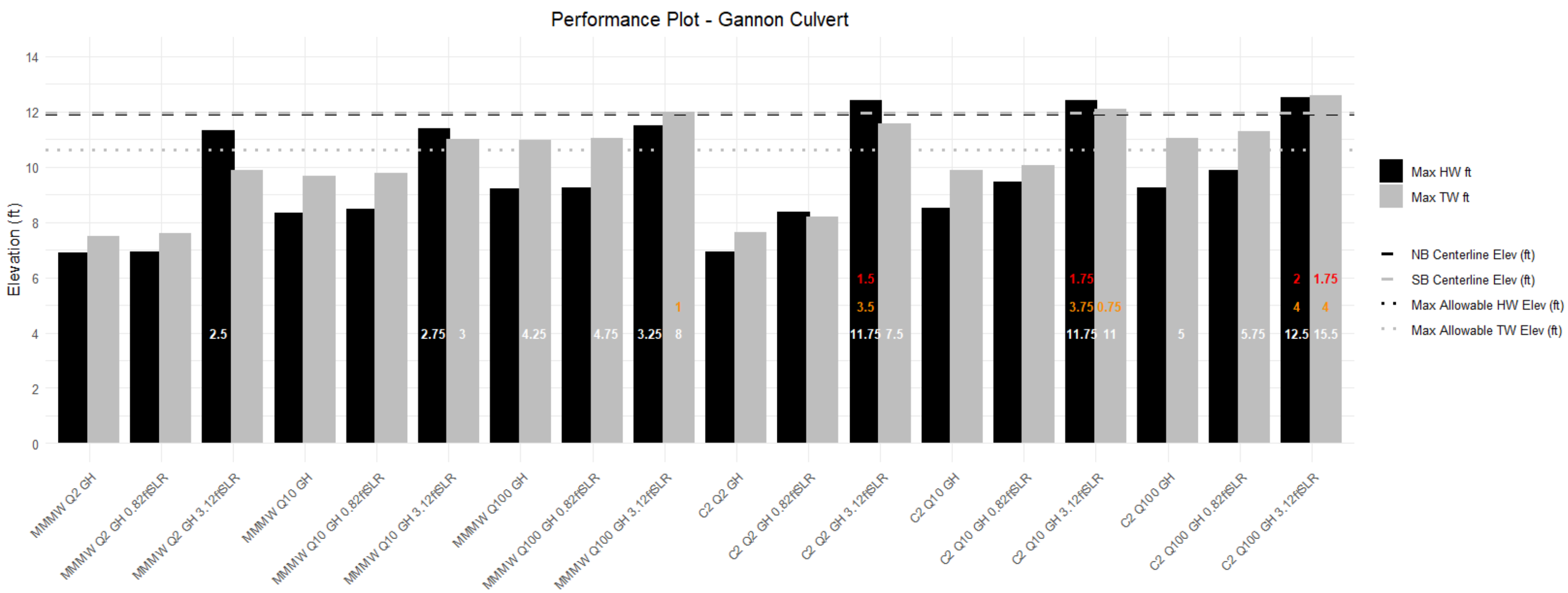

Figure C-14. Performance plot for Gannon Culvert over a suite of event conditions.

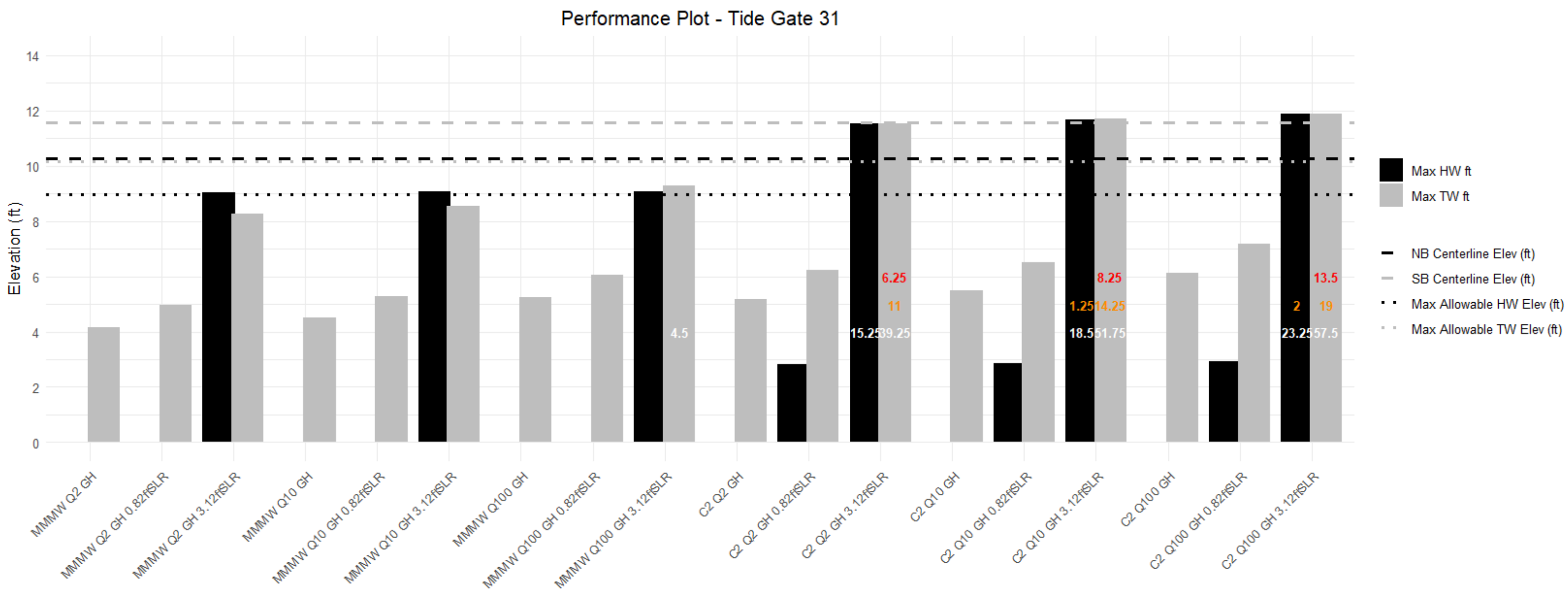

Figure C-15. Performance plot for Tide Gate 31 over a suite of event conditions. White, orange and red text represents the overtopping duration (hrs) of the maximum allowable headwater/tailwater elevation, adjacent road centerline and the threshold depth of 4 inches over the road centerline, respectively.

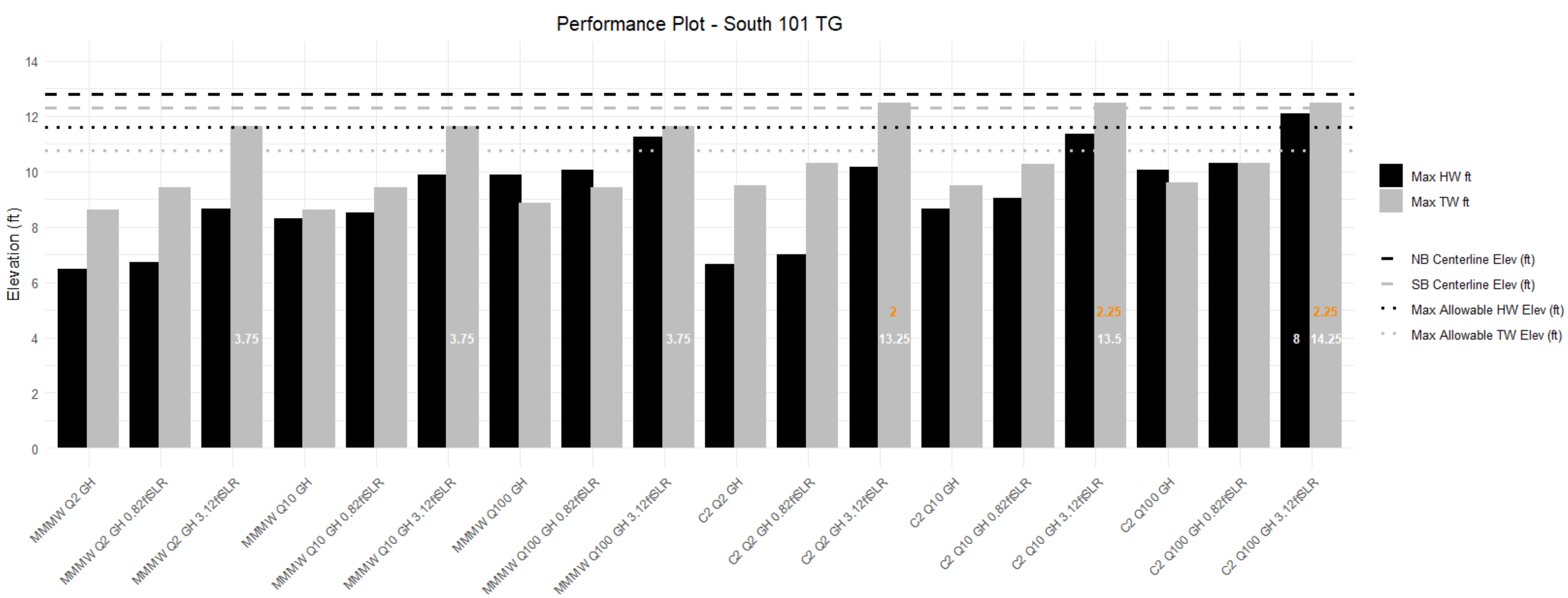

Figure C-16. Performance plot for101 South Tide Gate over a suite of event conditions.

Appendix D- Performance Plots: Bridges

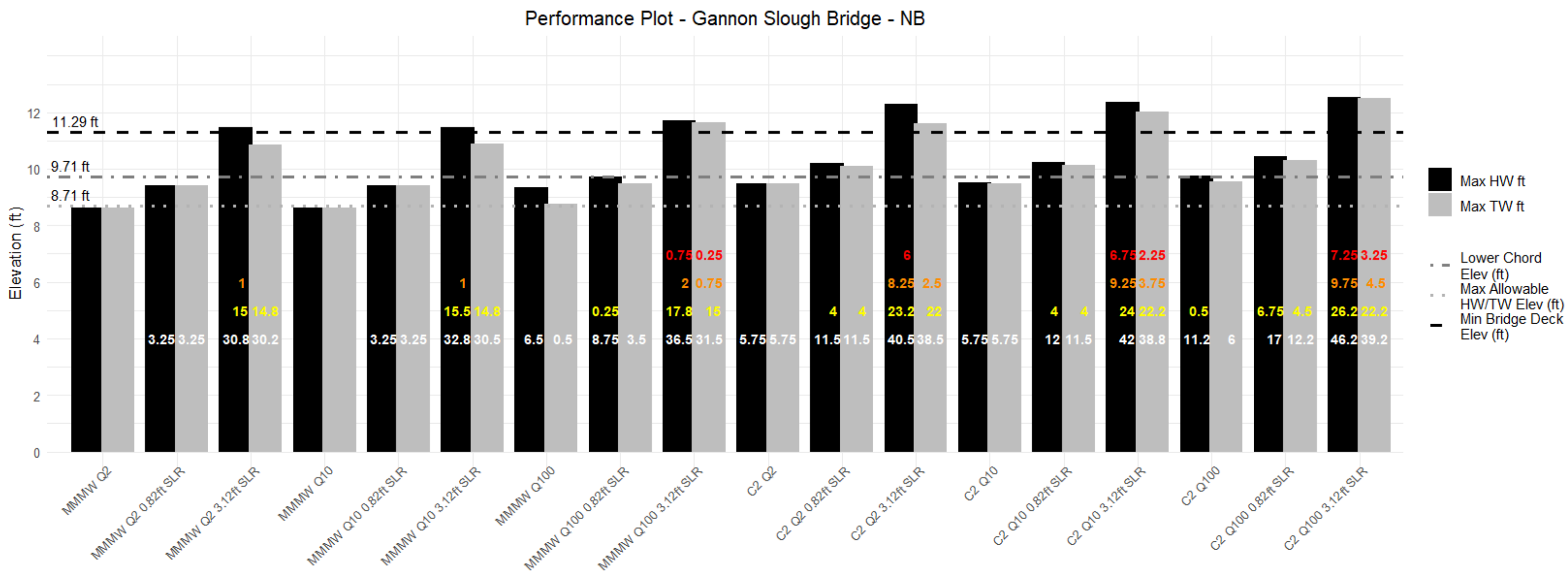

Figure D-1. Hydraulic performance plot of northbound Gannon Slough Bridge illustrating maximum headwater and tailwater (TW) relative to key bridge features. Red text represents the duration (hours) that the minimum deck elevation is inundated with greater than 4 inches of water, orange text represents the duration that the minimum bridge deck elevation is exceeded, yellow represents the duration the lower chord elevation is exceeded, and white text represents the duration the maximum allowable HW or TW is exceeded.

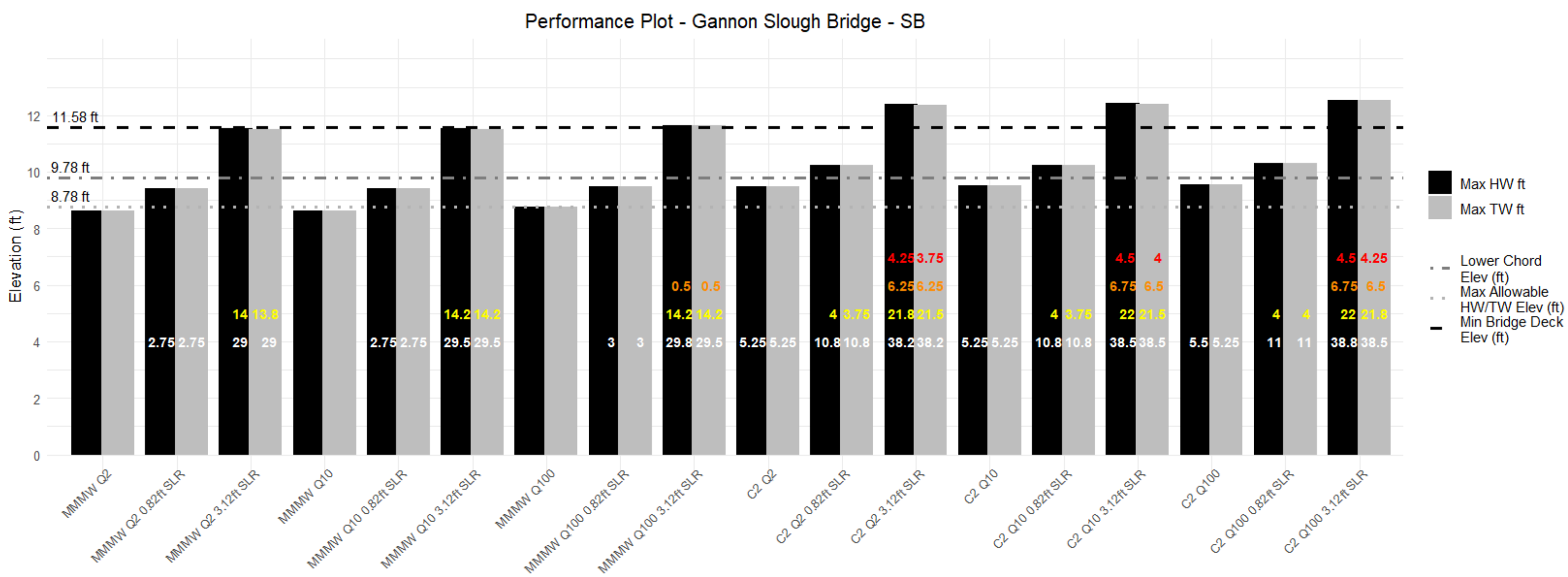

Figure D-2. Hydraulic performance plot of northbound Gannon Slough Bridge illustrating maximum headwater and tailwater (TW) relative to key bridge features.

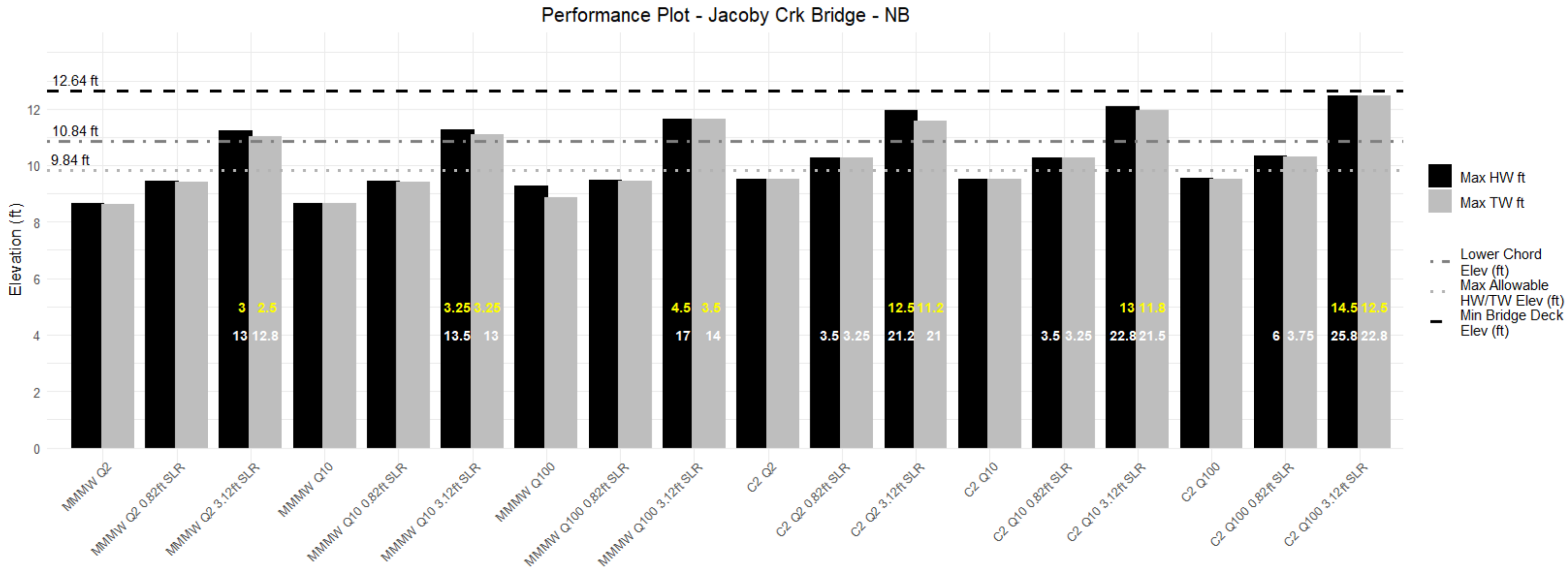

Figure D-3. Hydraulic performance plot of northbound Jacoby Creek Bridge illustrating maximum headwater and tailwater (TW) relative to key bridge features.

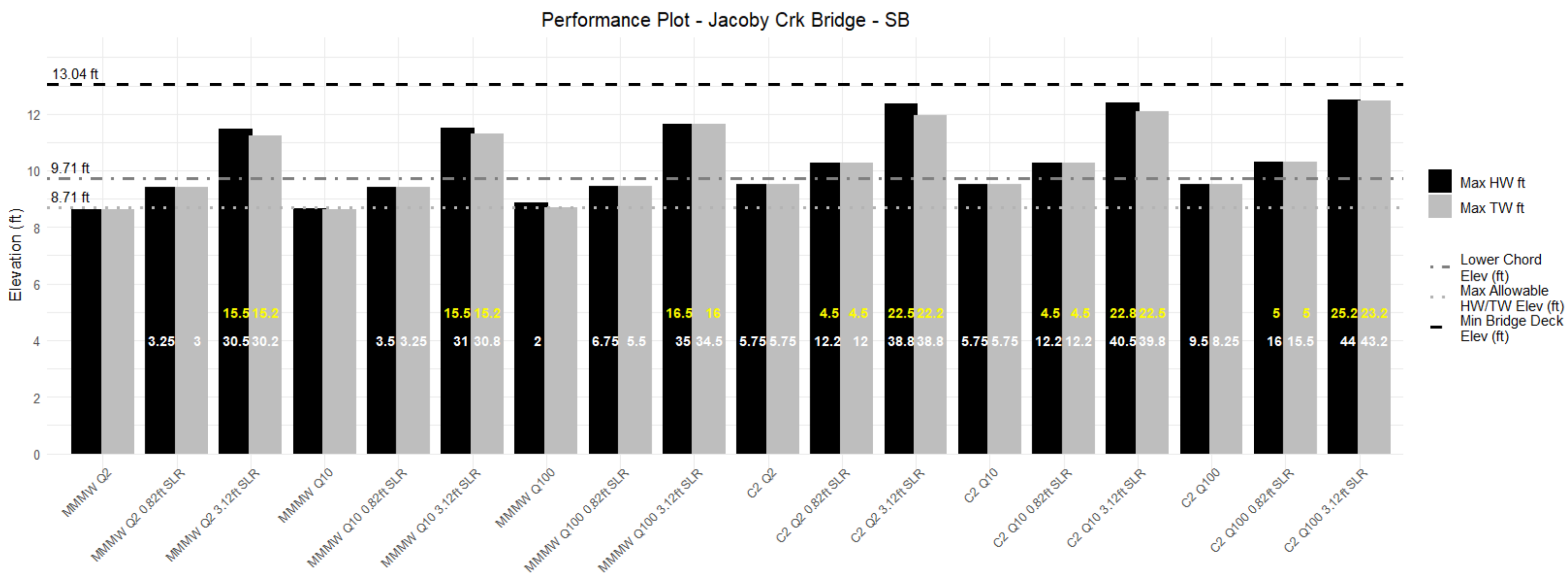

Figure D-4. Hydraulic performance plot of southbound Jacoby Creek Bridge illustrating maximum headwater and tailwater (TW) relative to key bridge features.

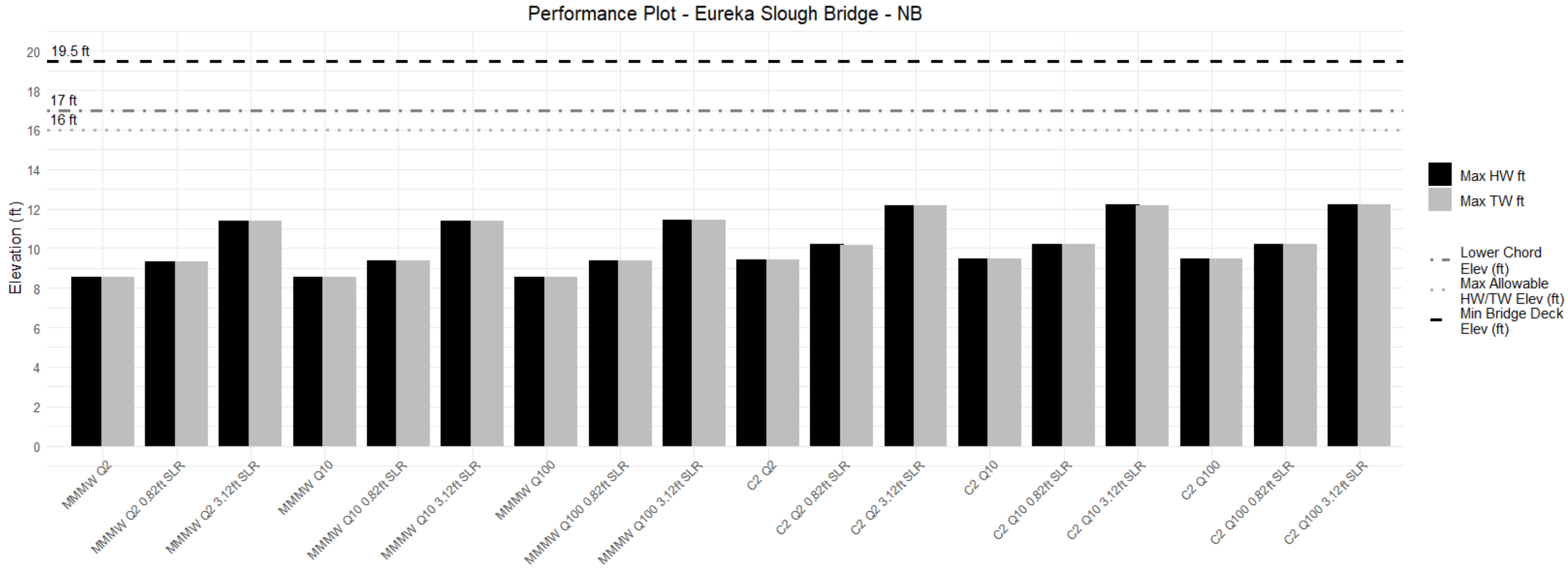

Figure D-5. Hydraulic performance plot of northbound Eureka Slough Bridge illustrating maximum headwater (HW) and tailwater (TW) relative to key bridge features.

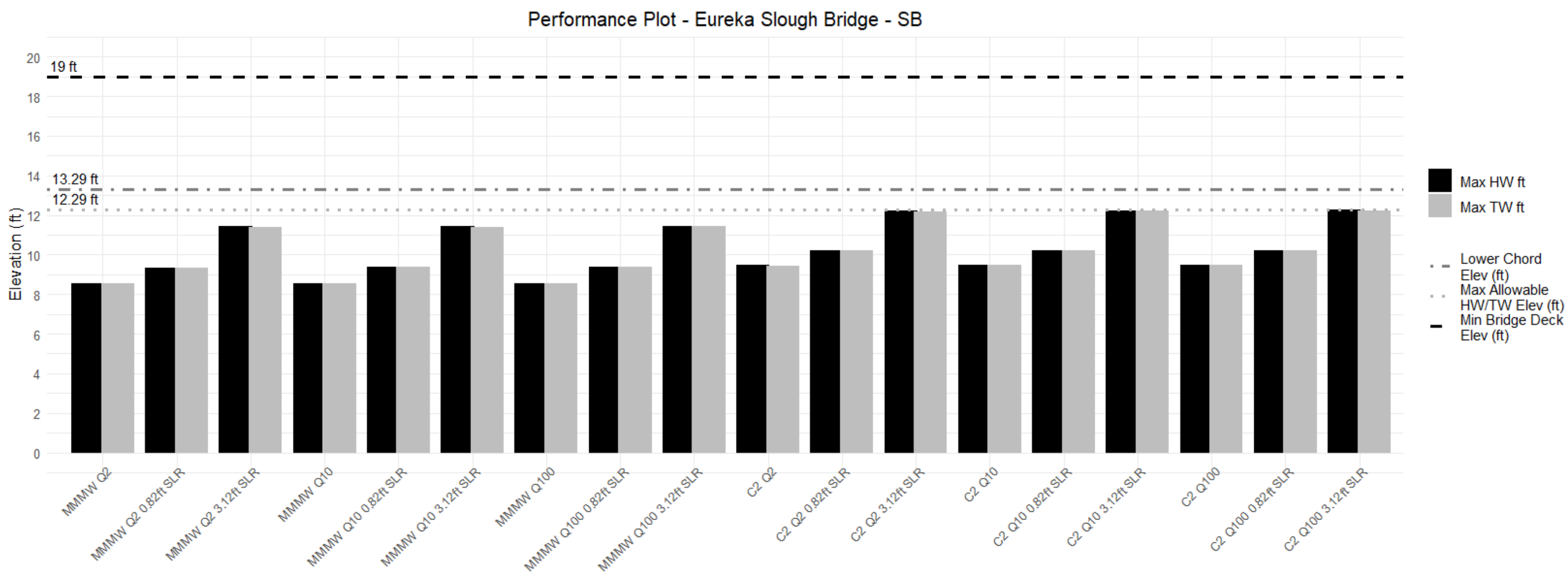

Figure D-6. Hydraulic performance plot of southbound Eureka Slough Bridge illustrating maximum headwater (HW) and tailwater (TW) relative to key bridge features.