SAN BRUNO CREEK TIDEGATES

– CERTIFICATION FEASIBILITY –

Prepared for

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1. INTRODUCTION

1.1 Background and Purpose

This report evaluates the feasibility for the FEMA certification of the San Bruno Creek tide-gate structure at the mouth of San Bruno Creek. It describes the results of a two-dimensional hydraulic analysis of coastal flood events, and includes an evaluation of the potential benefits to the local communities if the tidegates were to be accredited by FEMA. The intent is that if sufficient benefits exist to the community, and the condition of the structure is such that it provides protection against the design flood event as specified by FEMA, then the County could pursue an application to FEMA in the future for certifying the gates. A FEMA certified and accredited structure would have significant beneficial impacts to the special flood hazard area designation of the areas impacted by the operations of the tidegate.

San Bruno Creek runs through the City of San Bruno, in San Mateo County. The lower reaches of the creek run west and north of property owned by the San Francisco International Airport (SFO) as shown on Figure 1-1. San Bruno Creek's watershed which encompasses an area of approximately 4.5 square miles is also shown on the Figure. The creek collects runoff from the City's storm drain system and discharges it into San Francisco Bay via tidegates at the mouth of the creek, as shown on Figure 1-2.

The tidegate structure itself consists of four, 5-feet diameter circular pipes with flap gates on the downstream side as shown on Figure 1-3. It is under the jurisdiction of the San Bruno Creek Flood Control Zone, which was created by the San Mateo County Flood Control District in 1967.

In the last two rounds of mapping by FEMA (1984 and 2012), FEMA did not conduct detailed Flood Insurance Studies (FIS) for San Bruno. The 1984 FIRM does not show any flood designations for the lower reaches of the watershed, while the updated 2012 FIRM shows the lower reaches of the watershed marked as *Zone D* (areas with possible, but undetermined flood hazard) as shown on Figure 1-4. As a result of this mapping, accreditation of the tidegates was never sought by the County.

As part of a region-wide update of Flood Insurance Rate Maps (FIRM), FEMA embarked upon the *California Coastal Analysis and Mapping Project (CCAMP)* for San Francisco Bay, and recently released the Preliminary FIRMs for San Mateo County (FEMA 2016). These studies focused on updating coastal hazards, and in most cases did not analyze hazards from riverine sources. The recent maps for San Mateo County, shown on Figure 1-5 indicate the lower reaches of the San Bruno Creek watershed as being within a SFHA (Zone AE), which would necessitate the mandatory purchase of flood insurance for the residents in the communities affected by the map changes. The community that is affected the most is the Belle Air neighborhood (see Figure 1-5). The FIRM shows approximately 340 parcels within Belle Air neighborhood as susceptible to flooding during the 1% annual chance base flood event and an additional 119 parcels susceptible to flooding during the 0.2% annual chance base flood event.

Two observations were apparent in the recent mapping:

- Since no shoreline exists in the immediate vicinity of communities marked in the new SFHA (it is substantially removed from the source of coastal flooding), it appears that the extent of flooding has been determined by projecting the Base Flood Elevation (BFE) across contiguous area(s) that are topographically below the BFE, irrespective of overland flow distance and flood duration.

- The mapping shows that flooding within this area is driven by elevated water levels within San Francisco Bay that enter San Bruno Creek, unimpeded by the tidegate, and flow over the banks until they intersect higher ground. This is based on FEMA's current guidance policy, wherein all non-accredited structures are removed from their flood-mapping analyses, and the projected coastal surge event is assumed to continue past the structure.

The analyses described in this report address the above two factors; a detailed two-dimensional hydrodynamic analysis was conducted to assess the influence of tidally varying water levels in San Francisco Bay, and various combinations of riverine flow and tidal stage in the Bay were evaluated to assess the amount of potential flooding and the benefits of having an operational (certified) tidegate.

1.2 Scope of Work

The following tasks were performed for this scope of work.

1.2.1 Develop a Hydrodynamic / Hydraulic Model and Conduct Simulations

This task included the development of a numerical model utilizing the xp2D dynamically linked 1-D/2-D modules of XP-SWMM, and conducting several simulations to assess the benefits of accreditation of the tide-gates. Specific tasks included

- A comparison of the effects of including the tide-gates in the analysis of the 100-yr coastal surge event versus FEMA's current assumption that the gates are removed from the analysis.
- Simulating a range of fluvial flows, utilizing the 1-D/2-D approach, that coincide with different Bay water levels as suggested by FEMA (in lieu of coincident frequency analyses) to assess the flood capacity of the San Bruno Channel. The results for San Bruno Creek from the recently completed San Bruno Creek Colma Creek Resiliency Study (M&N 2015) were used to develop combinations of design storms and tide levels that are representative of: (1) the FEMA 1% occurrence flood event, and (2) the County's 25-year flood event design criteria for San Bruno Creek.
- Evaluating the limitations of the tide-gate structure's flow capacity, by identifying inundation areas and depth of flooding where flooding occurs upstream of the tide-gates on North Channel and Cupid Row Canal.

1.2.2 Assess Potential for Certification of Tidegate Structure

This task included:

- Assessing the current condition of the gates and gate structure,
- Reviewing and summarizing available prior geotechnical data for the structure,
- Collecting supplemental topographic survey data in the vicinity of the tide-gate, open-channel sections, and roadways adjacent to the structure.
- Describing the FEMA-required steps to analyze the structure such that an application for certification can be prepared.

1.2.3 Feasibility Report

This task included documenting assumptions and model parameters, summarizing the findings and results of the analysis, preparing a report, and presenting recommendations to the County. It includes the identification of deficiencies in the structure, and compares the potential benefits of replacing the tide-gate structure versus moving forward with certifying the existing structure.

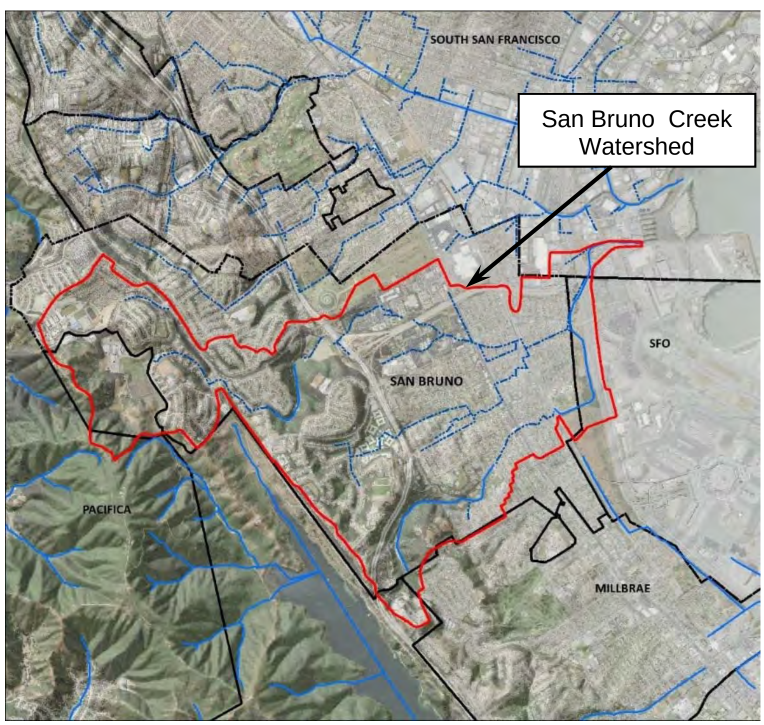
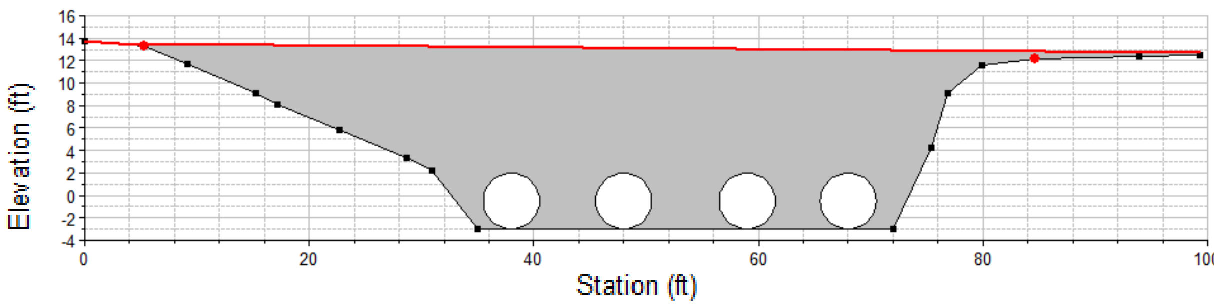


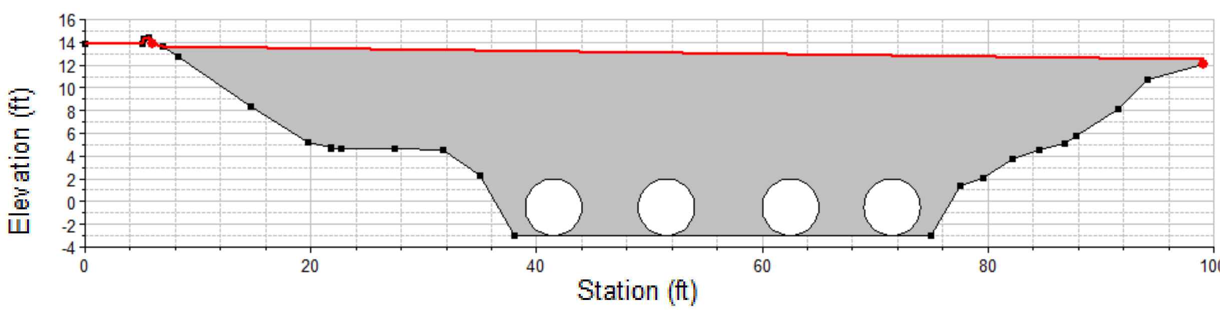
Figure 1-1: San Bruno Creek Location Map and Watershed Boundary

Figure 1-2: Vicinity Map

Downstream (Left), Upstream (Right) & Flapgate (Middle)



Cross-Section Upstream of Tidegate



Cross-Section Downstream of Tidegate

Figure 1-3: San Bruno Creek Tidegate Structure

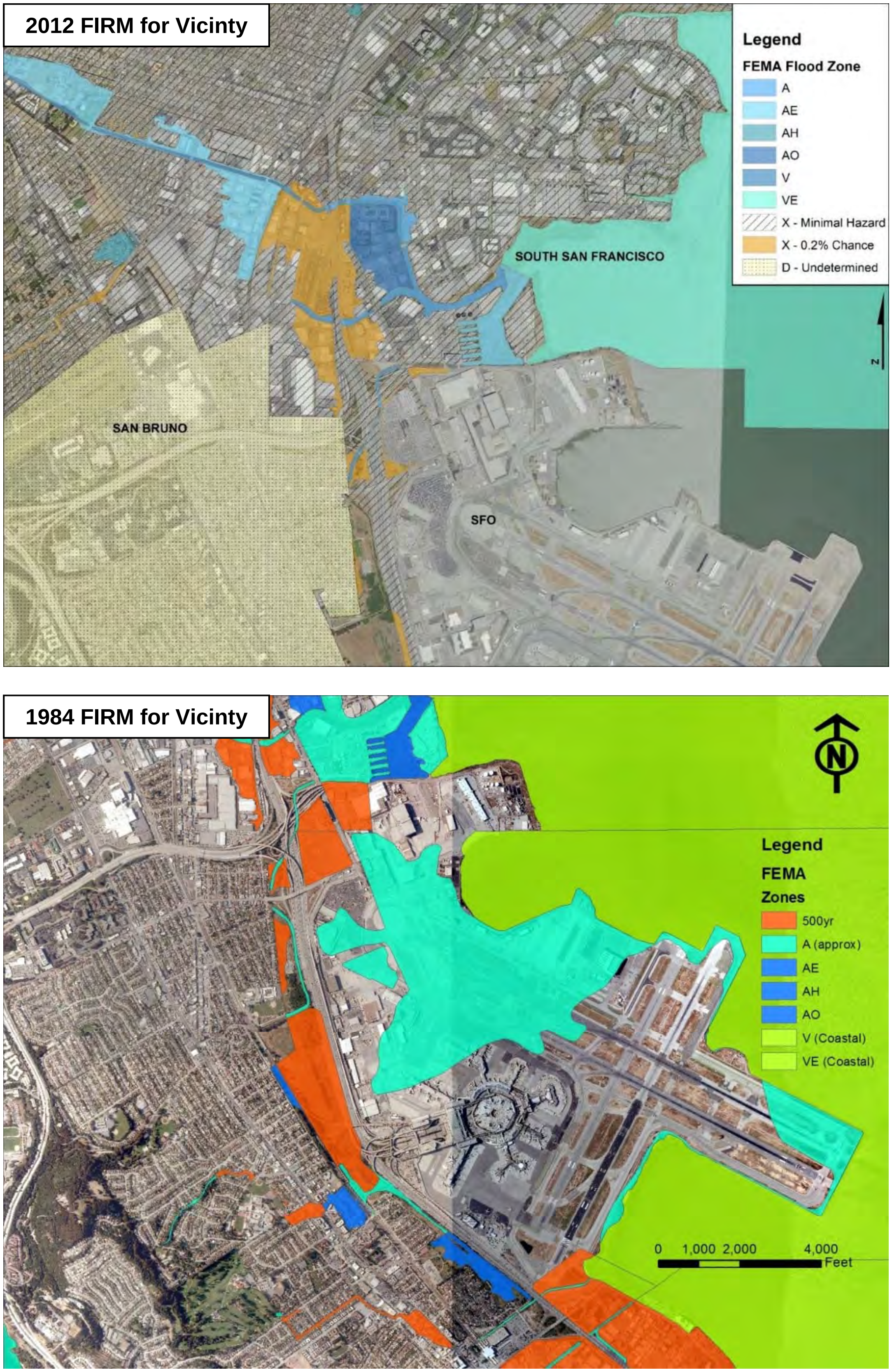


Figure 1-4: Superseded Flood Insurance Rate Maps for Vicinity

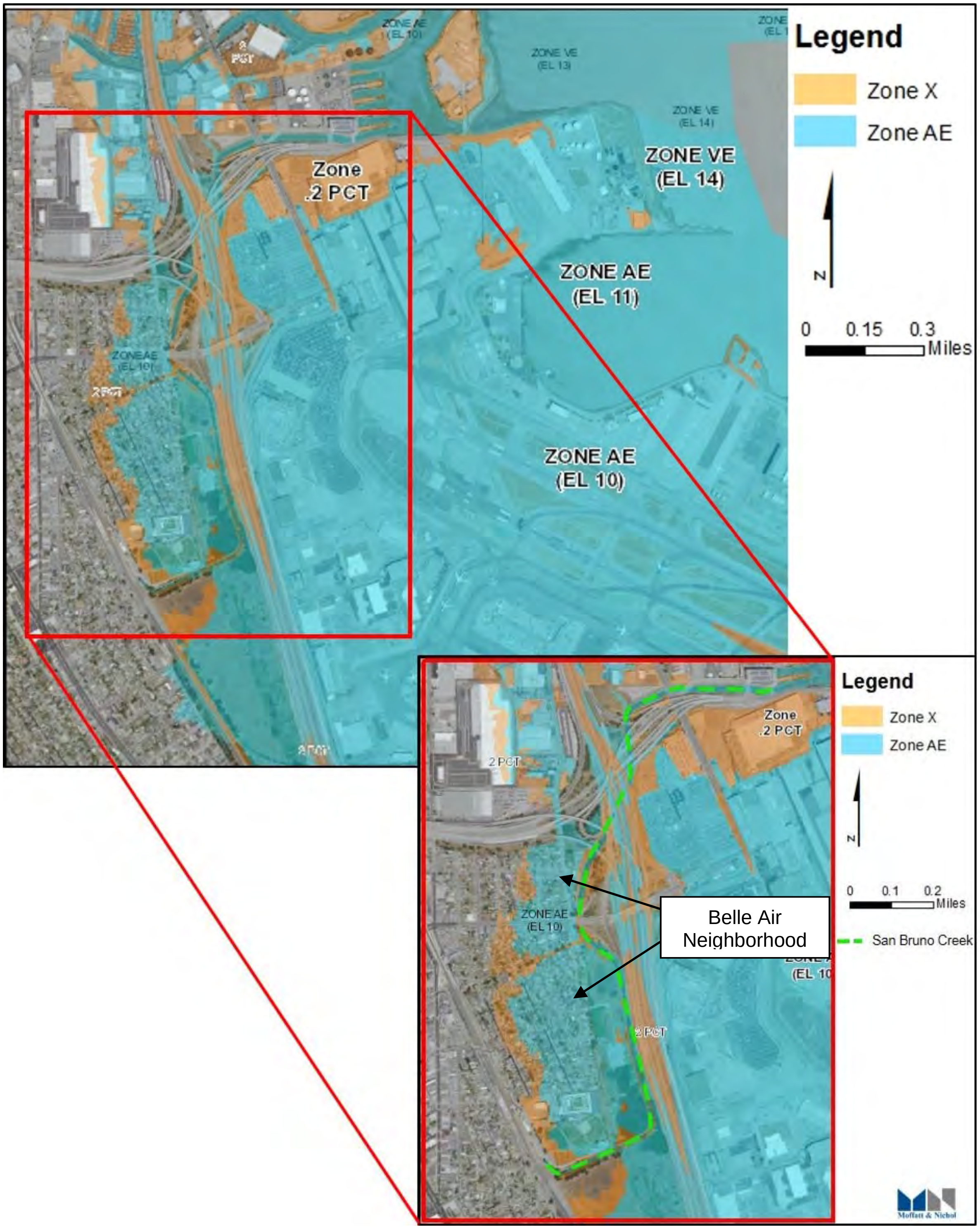


Figure 1-5: Preliminary FEMA Flood Insurance Rate Map for Vicinity (FEMA 2016)

2. EXISTING CONDITIONS

San Bruno Creek collects runoff from the City of San Bruno, a drainage area of approximately 4.5 square miles, which lies south of the Colma Creek drainage basin. Most of the San Bruno Creek watershed drains through pipes to the City's storm drain system. East of the Caltrain tracks, the creek turns into an open channel referred to as the Cupid Row Canal. The Canal flows east-west for about 1000 feet and then turns northwards, within SFO-owned property adjacent to Hwy 101. The channel passes under Hwy 101 through culverts and continues west along the border with San Francisco International Airport. The stretch of channel between Hwy 101 and the tidegate structure is referred to as the North Channel. The majority of the open channel, which runs for approximately 1.75 miles before discharging into San Francisco Bay through a tidegate structure, is composed of bare earth with vegetated banks.

The San Bruno Creek outlet to San Francisco Bay, which is though the San Bruno Tidegate Structure, is approximately 1,400 feet to the south of the Colma Creek outlet (see Figure 1-2). The tide gate structure where the North Channel exits to San Francisco Bay consists of four, 5-feet diameter circular pipes with flap gates on the downstream side. Drawings suggest that the channel and tide gate structure were designed for the 25-year return period flow of 1,100 cfs with the tidal elevation at 6.8 ft NAVD88, which is Mean Higher High Water at the site (MHHW) (SMCFCD, 1965).

The City of San Bruno recently completed a city-wide Storm Drain Master Plan (SDMP) study of the hydrology and hydraulics of the existing storm drain system (GHD, 2014). The design capacity of the City's storm drain system was the 25-year design storm event and the hydraulic analysis was done using Bentley's SewerGEMS software.

The SDMP determined that there are multiple capacity deficiencies in the system and identified six Priority 1 improvements and seven Priority 2 improvements that are recommended for the storm drain system (GHD, 2014). The SDMP also recommended the rehabilitation of the San Bruno Creek tide gate to restore full functionality at the discharge point of San Bruno Creek into the San Francisco Bay.

2.1 FEMA Flood Insurance Studies

FEMA performed a county-wide FIS for San Mateo County in 2012, which included the cities of San Bruno and South San Francisco. The City of San Bruno was mapped as Flood Zone D, which represents areas with possible but undetermined flood hazards. The two open channel sections of lower San Bruno Creek, Cupid Row Canal, and North Channel were part of Unincorporated San Mateo County on the previous FIRMs. The channels are designated as Flood Hazard Zone A and were not studied in detail by FEMA. Figure 1-4 shows the extents of the flood hazard areas that were previously assigned to SFO and vicinity in the 1984 and 2012 San Mateo Unincorporated County FIRMs.

This map has been superseded by the current effective San Mateo County FIRM. The recent FEMA Coastal Flood Hazard Studies for the San Francisco Bay also included the County of San Mateo. These BFE results (Figure 1-5) were computed during the CCSF Coastal Flood Hazard Study using transect-based wave runup calculations (BakerAECOM, 2013). Input wave and water level data for the transect model was obtained from a Bay-wide modeling effort that directly simulated the water level variation and wind-wave generation for a 54-year period (Danish Hydraulic Institute (DHI), 2011). Accompanying documentation indicates that a BFE of +10.4 ft NAVD88 was adopted for analysis purposes in the development of the draft work maps.

The FEMA studies also did not assess riverine flooding within the San Bruno Creek watershed. However, a significant portion of the lower San Bruno Creek watershed, east of the Caltrain tracks, is now shown as Flood Hazard Zone AE with a flood elevation of +10' NAVD on the draft FIRMs (FEMA FIRMs round the BFE to the nearest whole foot). This region of the City of San Bruno, between the Caltrain tracks and Hwy-101 and south of Interstate-380, is known as the Belle Air neighborhood. Since no shoreline exists in the immediate vicinity of this interior flood area, which is substantially removed from the source of coastal flooding, it appears that the extent of flooding has been determined by projecting the BFE across any contiguous area(s) that are topographically below the BFE, irrespective of overland flow distance.

For the portion of the Belle Air neighborhood that is north of San Bruno Avenue and Pine Street (Figure 1-2), the source of flood waters is overtopping of the south bank of Navigable Slough which has low spots below the BFE. The flood volume reaches Belle Air via the flow path along Shaw Road, then underneath I-380, and ultimately along 7th Avenue.

For the portion of the Belle Air neighborhood that is south of San Bruno Avenue and Pine Street, there are two sources of coastal flooding that are mapped on the draft FIRMs. The first source is the lack of any control structure at the mouth of San Bruno Creek, because FEMA's approach is to remove any non-certified structures. The second source of coastal flooding is the overtopping of Highway 101 through SFO (SFO's non-certified levees are removed from their analysis) and from the coastline south of SFO at Millbrae Avenue.

2.2 Topography

High-resolution LiDAR data is available covering the general topography of the area. The San Mateo County Flood Control District provided a LiDAR digital elevation model (DEM) surface for the entire watershed and the creek (referenced to NAVD88) with a horizontal resolution of 5 ft. as shown on Figure 2-1.

The collection of additional survey data was necessary to facilitate the development of the hydraulic model for the open channel portion of lower San Bruno Creek. Therefore, survey transect data was collected by Meridian Surveying along the open channel sections of Cupid Row Canal and North Channel, with the cross-sections spaced at roughly equal increments along the channel. The in-channel data was used to augment the high resolution LiDAR data sets that were available from SFO and the County of San Mateo. The field survey was performed by Meridian on 11/14/2014 and 11/17/2014, and the extent of the survey points are illustrated in Figure 2-1. Comparison of the in-channel data with the LiDAR data indicates that the two data sets are very consistent. Detailed elevation data from both sources is provided in Appendix A.

Figure 2-2 shows the elevations in the vicinity of the tidegate. Over the tidegate structure runs a road and trail that connects the SFO Long Term Parking Lot and San Francisco Bay Trail to North Access Road. Survey points were also collected along the edge of the tidegate structure. The lower plot of Figure 2-2 shows a roadway profile. A more detailed plan view of the structure can be found in Appendix A. The average elevation of the road is +12.7 feet NAVD88 with a minimum elevation of +12.2 feet NAVD88. For the FEMA BFE of +10.4 feet NAVD88, the structure has a freeboard of approximately 2 feet. Waves are not expected to impact the tidegate because the reclaimed peninsula used as a Sam Trans parking area, located east of the structure, intercepts wave action from the Bay.

Bank elevations along the North Channel are generally above the 1% annual chance stillwater elevation (SWEL) of +10.4 feet NAVD88. There is a short stretch of area along the southern bank of North Channel that has a maximum elevation of approximately +10.1 feet NAVD88 near the SFO parking lot. Correspondence with the Airport has indicated that flooding of this parking lot does occasionally occur from high water elevations within San Bruno Creek.

Bank elevations along Cupid Row Canal average approximately +11 feet NAVD88 along the northern portion of Belle Air neighborhood. Bank elevations along Cupid Row Canal just west of the airport property have elevations of +10 feet NAVD88 or less. This is the area that serves as the source of coastal flooding in the modeling effort. Elevations increase moving south along Cupid Row Canal with the highest bank elevation of greater than +13 feet NAVD88 along the northern bank on the east-west portion of the open channel.

2.3 Hydrology

2.3.1 San Francisco Bay

Water levels at the project site are dominated by a mixed semi-diurnal tide where two unequal highs and lows occur each tidal day. The shoreline near the project site is approximately equidistant to the two closest long-term active National Oceanic and Atmospheric Administration (NOAA 2014) tide gauges (Alameda and Redwood City). However, both are over 10 miles away and not representative of the tides at the project site because a change in tidal elevation occurs due to the narrowing of the Bay as the tide propagates southward. This narrowing results in amplification of the tide as it moves into the South Bay. NOAA has established tidal harmonic constituents at several closer locations based on short-term deployments that bound the tidal datums at the project site. The constituents and derived datum referenced to MLLW were available at Oyster Point, 3 miles north of the project site, and the San Mateo Bridge, approximately 7 miles east-southeast from SFO. Due to the location of these gauges in relation to SFO, the tidal range and NAVD88 datum conversion at SFO is expected to be between those of the two gauges.

Tidal planes estimated for the tidegate are shown in Table 2-1 below. The MHW elevation estimated was subsequently used in some of the simulations. Although the 100-year return period water level for the study area was computed to be slightly lower than FEMA’s BFE of +10.4’ NAVD, a decision was made to use the FEMA BFE so comparisons of flooding would not be biased downward.

Table 2-1: Tidal Datums in Project Vicinity

Tidal Plane	9414392 Oyster Point[1]		9414458 San Mateo Bridge[2]	
	MLLW (feet)	NAVD88 (feet)	MLLW (feet)	NAVD88 (feet)
MHHW	+7.18	+6.73	+7.72	+6.92
MHW	+6.54	+6.09	+7.09	+6.29
MTL	+3.84	+3.39	+4.14	+3.34
MSL	+3.77	+3.32	+4.11	+3.31
MLW	+1.14	+0.69	+1.19	+0.39
MLLW	+0.00	-0.45	+0.00	-0.80
NAVD	+0.45	0.00	+0.80	0.00

[1] MLLW to NAVD conversion based on Tucker & Associates Survey at Oyster Pt Marina

[2] MLLW to NAVD conversion based on USGS 2005 Survey

2.3.2 San Bruno Creek

As part of the San Bruno Creek/Colma Creek Resiliency Study (M&N 2015), San Bruno Creek discharges were estimated by calibrating the hydrologic model to the design storm of interest. Since there was no available stream gauge data for San Bruno Creek, and due to the close proximity to the Colma Creek watershed, the same rainfall pattern and losses that were used for the Colma Creek calibration analysis were applied to the San Bruno watershed. Because the City of San Bruno's Storm Drain Master Plan (GHD 2014) utilized a different methodology, their flows were slightly lower than the flows estimated by the M&N team (M&N 2015) as shown in Table 2-2 below. However, the flows presented are consistent with those produced by the San Mateo County Flood Control District's *San Bruno Creek Flood Control Zone* report from 1965 (Wilsey, Ham & Blair, 1965).

Table 2-2 : Estimated San Bruno Creek Discharges[4] (Schaaf & Wheeler, 2015)

Discharge Point	Frequency					
	2yr (cfs)	5yr (cfs)	10yr (cfs)	25yr (cfs)	50yr (cfs)	100yr (cfs)
Cupid Row Canal @ Lions Park	80	140	200[3]	250[2]	300	350
San Bruno Channel @ San Bruno Ave	380	630	910	1,130	1,330	1,520
San Bruno Channel d/s Highway 101	480	810	1,160	1,440[1]	1,710	1,960

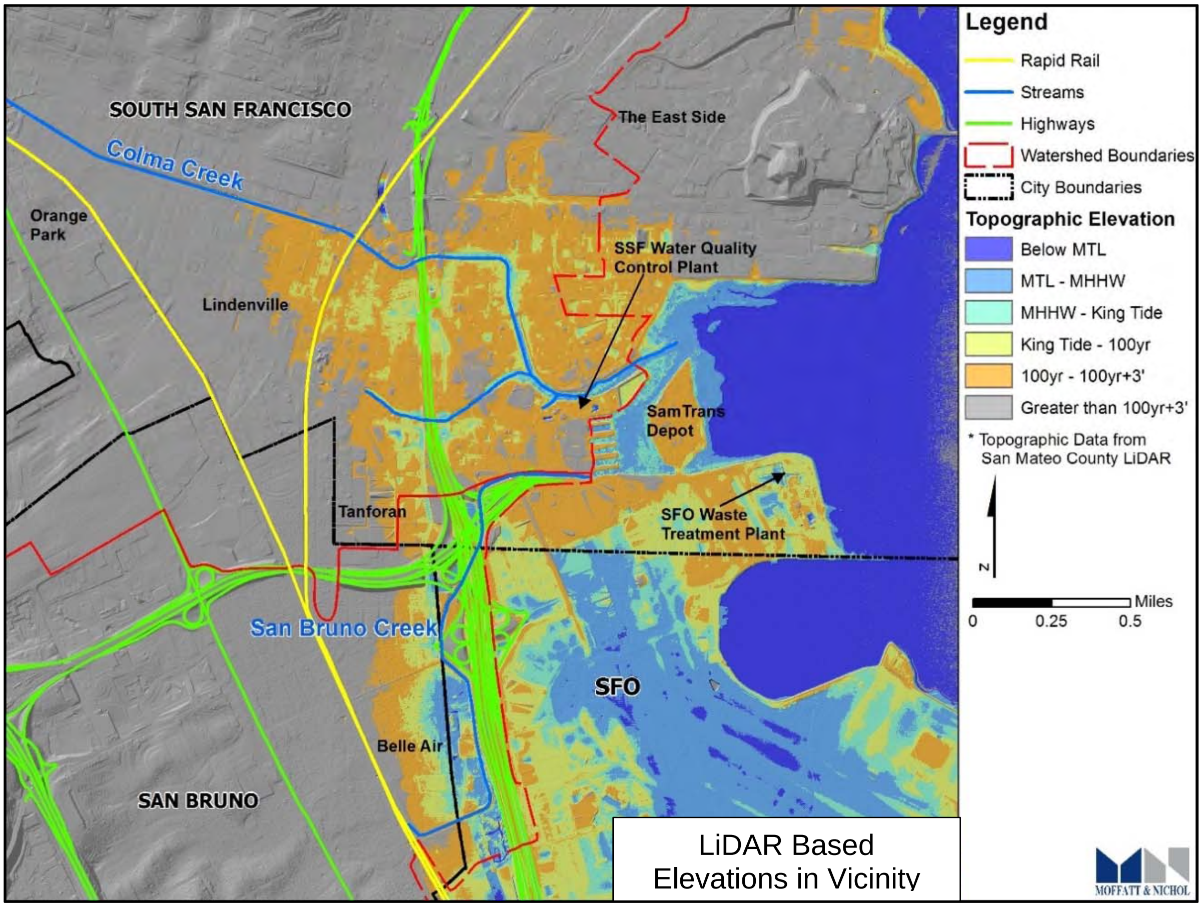
1. The San Mateo County Flood Control district calculated the 25 year discharge at 1,100 cfs for this location in 1965 (Wilsey, Ham, & Blair 1965).
2. Design drawings from the San Mateo County Flood Control District show that the channel was designed for 250 cfs (Wilsey, Ham, & Blair 1965).
3. The "Recovery Action Plan for the San Francisco Garter Snake" estimated the 10 year flow in Cupid Row Canal at 165 cfs (LSA Associates, 2008).
4. Discharges are in accumulated flow rates.

2.4 Geotechnical Conditions

The San Bruno Creek tidegate is located on land reclaimed from San Francisco Bay. Placement of artificial fill in the area took place between 1930 and 1975. The tidegate, however, appears to have been placed along the alignment of the 1880 levee, which suggests that it may not build on reclaimed land but a surface of harder material. The USGS Geologic Map for the San Francisco South 7.5' Quadrangle, an excerpt of which is shown in Figure 2-3, shows the area containing the tidegate structure classified as 'Sandstone and Shale' (KJsk).

In 2013, the Moffatt & Nichol – AGS Joint Venture (JV) performed a geotechnical investigation for the SFIA. The analysis found that soil within the vicinity of San Bruno Creek tidegate is underlain by Franciscan Sedimentary Rocks. During this investigation, a boring was taken approximately 350 feet southwest of the tidegate. The boring log indicated that the first 8 feet of material down from the surface consist of a sandy lean or silty clay with some sand and gravel. From a depth of 8 feet to 12 feet, the log showed sandstone (Franciscan Formation). Refusal occurred at approximately 12 feet below the ground surface.

The information derived from the boring log indicates the presence of a harder surface underlying fill material in the vicinity of San Bruno Creek tidegate. This implies that any remaining settlement of the gate is minimal and additional work would not cause a significant amount of additional settlement.



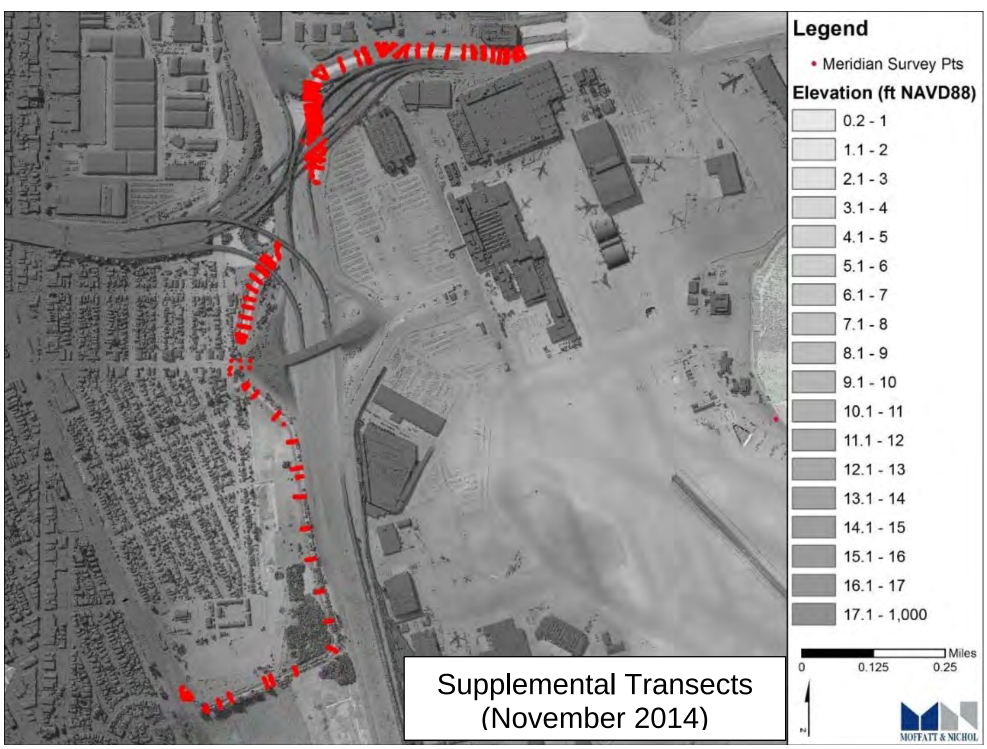


Figure 2-1: Topography in Study Area

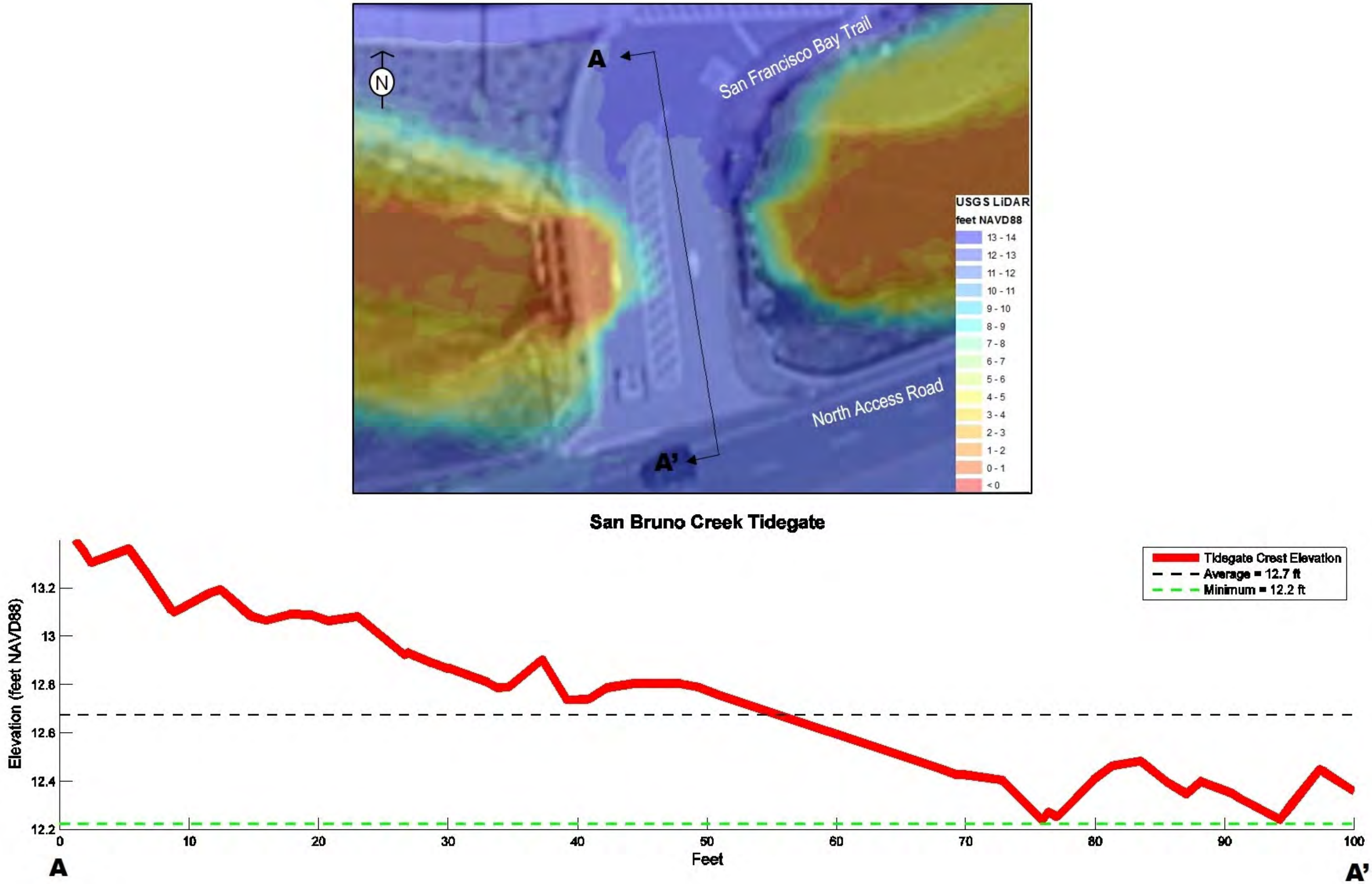


Figure 2-2: Elevations in Vicinity of Tidegate

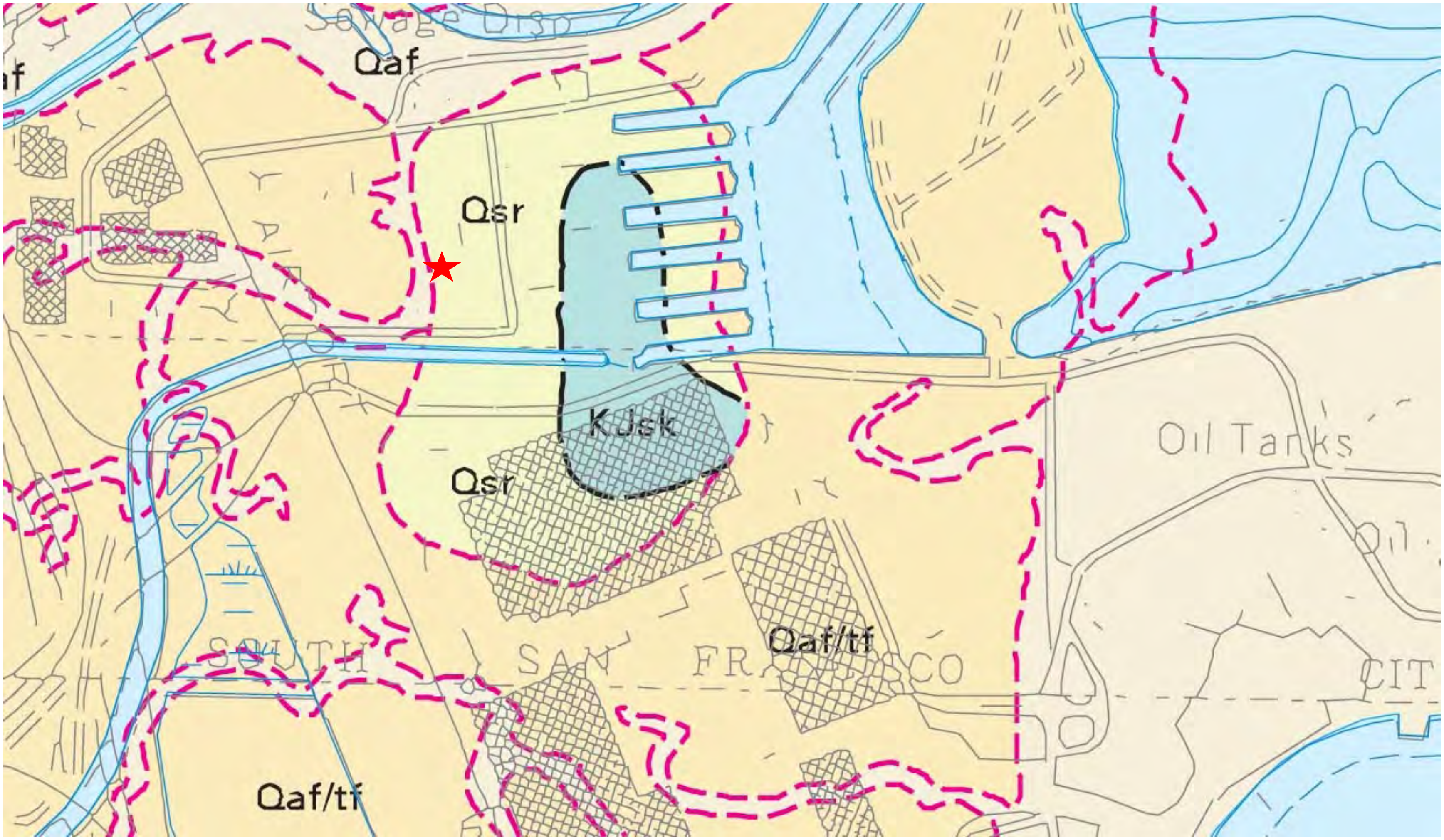


Figure 2-3: Excerpt from South San Francisco Quadrangle Geologic Map (USGS 1998)

3. HYDRAULIC ANALYSIS

The hydraulic analysis performed for this Scope of Work is intended to address two issues related to the performance of the San Bruno Creek tidegate. Part I focuses on evaluating the performance of the tidegate structure during coastal flooding events equivalent to the FEMA 1% annual chance base flood event. Part II of the hydraulic analysis is to evaluate the performance of the tidegate structure under different combinations of riverine flood and coastal flood conditions. Further details on each simulation are provided in Section 0 - Model Simulations.

3.1 Model Description

The XPSWMM is a fully dynamic hydraulic and hydrologic modeling software that combines 1D calculations for upstream to downstream flow with 2D overland flow calculations. It utilizes the EPA Stormwater Management Model (SWMM) 1D analytical engine for running rainfall-runoff simulations for a single event or long-term simulations of runoff quantity and quality. SWMM simulates runoff from sub-catchment areas and routes it through systems of pipes, channels, pumps, and storage devices. The XP-SWMM also incorporates a 2D analytical module for the routing of surface flood flows, based on the TUFLOW program. The TUFLOW mode has the ability to dynamically link to the 1D network of the SWMM engine, i.e. the 2D and 1D domains are linked to form one model.

The XPSWMM model was used to conduct this analysis. It is one of the models approved by FEMA under the National Flood Insurance Program (NFIP) for two-dimensional (2D) flood modeling and mapping.

3.2 Model Development and Domain

The XPSWMM 1D/2D integrated hydraulic model was developed to cover the entire project area. 1D nodes and links were used to simulate model components such as tidegates at San Bruno Creek, culverts on San Bruno Creek, Colma Creek and Navigable Slough. A 2D surface model was developed to represent both the floodplain and open channels. The 1D and 2D components are dynamically linked, and the surface flows in the 2D model can be routed through the 1D model.

In order to model both the base flood and 0.2% annual chance flood, the model domain was setup to cover the entire possible floodplain under the peak elevation of +12 feet NAVD88 during the 0.2% chance flood, and not limited to the City boundaries. Therefore, the developed model covers the SFIA, part of the City of South San Francisco, City of Millbrae, and the City of San Bruno. The model captures the bay shoreline and extends further east to cover all the areas with a ground elevation below FEMA's 0.2% SWEL (+12 feet NAVD88). The original topographic data from the FEMA Coastal Hazard Study was utilized to determine the extent of the +12 feet NAVD88 contour line. Figure 3-1 presents the outline of the model domain (yellow polygon), overlaid with jurisdictional boundaries in black solid line.

3.3 Model Setup

The hydrodynamic model is based on the same topographic/bathymetry data used by FEMA for FIRM purposes and the model is set up such that the 1% and 0.2% annual chance floods can be appropriately applied at the model boundary. The following subsections discuss model setup, input parameters and calibration factors used in this modeling effort.

3.3.1 Boundary Conditions

Offshore Boundary Condition: In order to develop tail water conditions during extreme SWELs, the 54-year water level time series developed for FEMA's "Regional Coastal Hazard Modeling Study for North and Central San Francisco Bay" (DHI 2011) were utilized. This study developed hourly water levels between 1956 and 2009 along the entire San Francisco Bay coast. Figure 3-2 shows twelve model points where time series of water surface elevations were available (DHI, 2011). The three storm time series with the highest water elevations were extracted from the 54-year time series and are presented in Figure 3-3. The figure shows that the highest water level was measured in January 1983. However, the February 1998 event has the longest duration of elevated water levels. Since the duration of the elevated water levels affects flood depth and extents, the most conservative approach would be to utilize the time series with the longest duration of elevated water level. Therefore, the February 1998 event was selected as the prototype time series and the time series was then elevated such that the peak elevation reaches the FEMA's 1% SWEL (+10.4' NAVD88) for the 1% annual chance flood unsteady modeling and FEMA's 0.2% SWEL (+12.0' NAVD88) for the 0.2% flood unsteady modeling. The resulting time series shown in Figure 3-4 were applied at the model offshore boundary.

Initial Boundary Condition: The model simulation started at Mean Higher High Water (MHHW), approximately +6.8' NAVD88, and the initial water elevation for the entire modeling domain was also set to the MHHW. The overall duration of water level higher than +9' NAVD88 is about 4 hours for the 1% annual chance flood and 6 hours for the 0.2% annual chance flood.

Flow Boundary Conditions: In the unsteady modeling, riverine hydrographs from San Bruno Creek were applied at flood boundaries. The San Bruno Creek/Colma Creek Resiliency Study mentioned that SCS Type I 24-hour distribution was used as design rainfall event in the City of San Bruno. The same hyetograph has been also used in the Storm Drain Master Plan study of the City of San Bruno (GHD 2014)). Similar pattern was assumed for the hydrograph at San Bruno Creek in this study since no hydrology study has been performed for the watershed. The unit time SCS Type I hyetograph applied in this study is presented in Figure 3-5.

The unsteady flows were applied at three locations as indicated in Table 2-2 (from upstream to downstream): 1) Cupid Row Canal at Lions Park; 2) San Bruno Creek at San Bruno Avenue; and 3) San Bruno Creek at Highway 101. Since the design capacity of the City's storm drain system is a 25-year storm, four frequencies of design storms (2-year, 5-year, 10-year and 25-year) were modeled in this study, combined with different offshore boundary conditions (see Table 3-4 for details). The three locations where the unsteady flood flows applied are showed in Figure 3-6.

3.3.2 Topography Data

The XPSWMM model was developed based on the digital terrain model (DTM) developed by Baker/AECOM and was provided to the City of San Bruno via data request for the appeal. It is the same topographic data used in FEMA's coastal hazard analysis. The DTM data is received in raster format with a 10 feet grid resolution. Based on the information provided in Topographic Data Development report (Baker/AECOM 2012), the San Francisco Coastal LiDAR Project dataset collected in 2010 by USGS is the basis for the 10 feet resolution DEM of the San Mateo County.

In the XPSWMM model, the 2D model grid was developed based on the DTM. The 10 feet resolution DTM provided by Baker/AECOM is sufficient to developing the 2D grids with a 15 feet grid size. Figure 3-7 shows the topographic data received from Baker/AECOM.

The only revisions to the topographic data are the San Bruno Creek tidegates and levees along the SFO since these structure are not currently FEMA certified and are, therefore, not included in the hydrodynamic analysis. Their potential positive impacts in flood defending are not considered in this study. Detailed discussion is in Section 3.3.3.

3.3.3 Tidegates, Levees and Other 1D Structures

San Bruno Creek Tidegate

The San Bruno Creek tidegate near North Access Road is close to its exit to the Bay. The tidegate structure consists of four 5-foot diameter circular culverts with one-way flapgate on the downstream side, shown in **Error! Reference source not found.** and **Error! Reference source not found.**. The San Bruno Creek Tidegate is not currently FEMA certified, hence, it is excluded in the model for this analysis. As no clear guidance on tidegate structure removal is found, the following two options of tidegate removal were simulated:

1) Remove the entire tidegate structure by connecting open channels from both ends of the gate; and
2) Leave the tidegate structure in place and leave all flapgates open.

San Francisco International Airport Levees

The City borders with the SFO on the east. The SFO encompasses approximately 8 miles of San Francisco Bay shoreline. The 8 mile shoreline was divided into a number of reaches with different shoreline and levee structures (Figure 3-8). Similar to the San Bruno Creek Tidegate, the SFO levee system is not a FEMA accredited structure. Therefore, all these levees were removed in the model analysis.

Other 1D Structures

There are total of 16 structures/links modeled as 1D structure in the model. The sizes of these tidegates and culverts are based on as-built drawings, previous reports, and communications with County staffs. Site visits were also conducted to confirm dimensions used in the model. Figure 3-9 and Figure 3-10 show the location of these 1D structures/links, and their shapes and sizes are listed in Table 3-1.

3.3.4 Buildings and Other Types of Obstructions

The XPSWMM model covers significant amount of urban development areas. Buildings such as commercial buildings and residential houses were carefully treated in the model setup. Literature review of prior studies for FEMA flood mapping and hydrodynamic modeling with XPSWMM indicate that the buildings were often treated as:

- Buildings with large foot print were set as inactive cells; and
- High density residential subarea with smaller building footprints were set as active cells with high roughness.

Smith (2012) conducted a study on the influence of building treatment in the numerical models on flooding. He compared different ways of building treatment in TUFLOW and DHI MIKE models, and also measured velocity fields around the buildings in physical models. He concluded that “the best way to treat buildings in numerical models was to either remove the computational grids under the building footprint completely or to increase the elevation of the building footprint to be above the maximum expected flood height”. His conclusions agree well with the floodplain analysis and mapping guidance prepared by Dewberry (2008) for larger buildings. The Dewberry’s guidance on modeling buildings for floodplain mapping using HEC-

RAS unsteady model states: "Modeling Buildings: Accounted for through the use of Manning's n adjustments (general case) or blocked obstructions (extreme case)." In this appeal study, buildings with a footprint larger than 10,000 square feet were treated as inactive cells.

Table 3-1: Table List of 1D Structures in the XPSWMM Model

No.	Stream	Location	Shape	Dimension (feet)	No. of Barrels
1	San Bruno Creek	Creek exit to the San Francisco Bay	circular	5**	4
2	San Bruno Creek	At crossing of San Francisco Bay Trail (downstream side)	rectangular	53' by 11'	1
3	San Bruno Creek	At crossing of San Francisco Bay Trail (upstream side)	rectangular	55' by 10'	1
4	San Bruno Creek	At crossing of S. Airport Boulevard	rectangular	50' by 12'	1
5	San Bruno Creek	At crossing of Highway 101	rectangular	10' by 8'	4
6	San Bruno Creek	At crossing of San Bruno Avenue	circular	3.5	3
7	Millbrae Creek	Channel exit to the San Francisco Bay	rectangular	12' by 10'	2
8	Millbrae Creek	At crossing of Highway 101	rectangular	10' by 6'	3
9	Millbrae Creek	At crossing of Aviator Avenue	rectangular	10' by 6'	3
10*	Navigable Slough	At crossing of S. Airport Boulevard	circular	8'	1
10*	Navigable Slough	At crossing of S. Airport Boulevard	circular	5'	1
11*	Navigable Slough	At crossing of Highway 101	rectangular	10' by 6'	1
11*	Navigable Slough	At crossing of Highway 101	rectangular	5' by 4'	1
12	Colma Creek	At crossing of Utah Avenue	rectangular	80' by 13'	1
13	Colma Creek	At crossing of S Airport Boulevard	rectangular	94' by 15'	1
14	Colma Creek	At crossing of Highway 101	rectangular	72' by 14'	1
15	Colma Creek	At crossing of Produce Avenue	rectangular	70' by 15'	1
16	Colma Creek	At crossing of Caltrain	rectangular	70' by 15'	1

** Multiple links with different sizes*

*** Diameter for circular culverts*

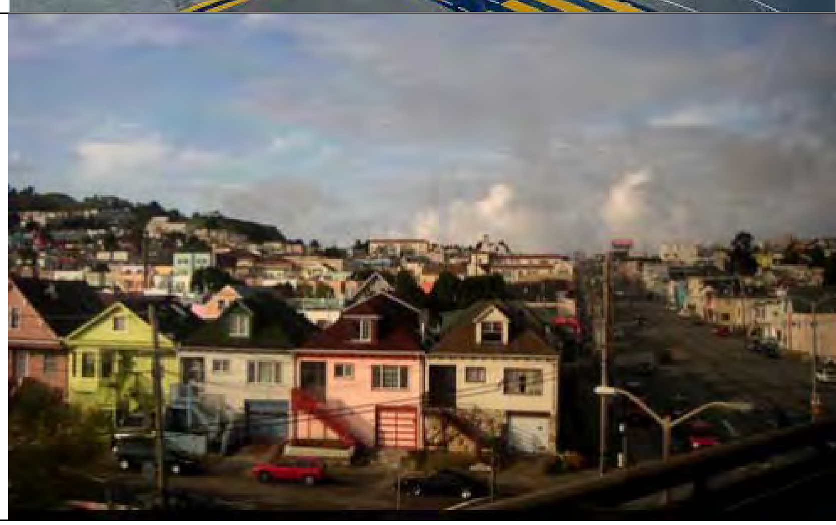
3.3.5 Bottom Roughness

In the XPSWMM hydrodynamic model, the primary calibration parameter is the roughness coefficient or the Manning's n value. In this appeal study, Manning's n values were assigned based on different land uses. The various types of land uses are listed in**Error! Reference source not found.** with photos and assigned Manning's n values. Manning's n values were assigned based on literature reviews and past similar hydraulic modeling experience as calibration data were not available for the project area. The high density urban area with numerous buildings/houses was assigned with a Manning's n value of 0.12 to include the building effects during the flood passages.

Table 3-2: Typical Land Uses and Manning's n Values

Land Uses	Photos	Manning's n
Creeks, Open Channels and Sloughs (including overbank)		0.04
Vegetated Open Area (grass, marsh with scattered trees)		0.04

Table 3-3: Typical Land Uses and Manning's n Values (continued)

Land Uses	Photos	Manning's n
Paved Roads		0.015
Airport Runway		0.015
High Density Urban Area (e.g. Belle Air)		0.12

3.4 Model Simulations

As mentioned in the first paragraph of Section 3, unsteady XPSWMM hydraulic analyses are simulated in two parts to evaluate two conditions described in subsequent paragraphs.

Part I: The first condition is to evaluate the effect of the tidegate structure during coastal flooding events equivalent to the FEMA 1% annual chance base flood event. No riverine discharge is included in Part I analyses. FEMA's analysis applied a steady state ("bathtub") approach in mapping flood zones under the base flood elevation (BFE). FEMA's methodology of mapping all areas below the BFEs contiguous to a flooding source, regardless of the duration of elevated water levels and terrain changes. Part I of this hydraulic analysis is to use an unsteady state methodology by applying a tidal series discussed in Section 3.3.1 at its offshore boundary. The proposed methodology considers the duration of the high tide and effects of the topography such as topo, buildings, roughness coefficients, and etc. In order to accomplish this, nine hours of hourly water levels from February 1998 event was raised to match FEMA's 1% SWEL (+10.4' NAVD88). Three scenarios are modeled: two options of tidegate removal discussed in Tidegates, Levees and Other 1D Structures (see Section 3.3.3) and one scenario of the tigegate functional condition.

1) FEMA 1% annual chance flood with open flapgates at all tidegates including San Bruno Creek tidegate
2) FEMA 1% annual chance flood with the tidegate structure removed for all tidegate structures including that in San Bruno Creek
3) FEMA 1% annual chance flood with functional flapgates at San Bruno Creek tidegate (assumed a FEMA certified tidegate structure)
4) FEMA 1% annual chance flood with functional flapgates at San Bruno Creek tidegate and Navigable slough tidegate (assumed FEMA certified tidegate structures)

Part II: The second part of the hydraulic analysis is to evaluate the performance of the tidegate structure under different combinations of riverine flood and coastal flood conditions. These simulations also used the tidal cycle from the February 1998 high tide event, but the entire tide cycle is vertically adjusted such that the peak elevation matches three different high tide conditions (MHHW, 10% SWEL and 1% SWEL). The tidegate structure was fully operational in all scenarios simulated; however, it should be noted that the tidegate structures does not affect the riverine flood discharge since it consists of one-way flapgates.

A coincident frequency analysis between San Francisco Bay SWEL and measured discharge values from Colma Creek was performed as part of the Colma Creek/ San Bruno Creek Resiliency Study (Moffatt & Nichol & AGS JV, 2015). Since no measurements exist for San Bruno Creek, discharges from Colma Creek and coinciding tail water conditions were compared to determine if there exists a relationship between higher discharges and tidal residuals. Tidal residuals are defined as the difference in *measured* water level and *predicted* water levels. Water level elevations occur as a combination of the astronomical cycle (predictable, varying water level elevations due to combined effects of the gravitational forces exerted by the Moon and Sun and the rotation of the Earth) and meteorological effects (storm surge, wind setup, El Niño). These meteorological effects can be significant and are referred to as tidal residuals.

The analysis showed that relatively high daily mean discharges (between 50 and 100 cfs) are associated with higher tidal residuals. However, when the daily mean discharge exceeds 100 cfs, the analysis showed that there is very little correlation between daily mean discharge and high tidal residuals. Because the discharge values applied to this analysis consist of high event discharges, which will exceed 100 cfs, the combinations of discharge and tail water conditions are analyzed as independent of one another. Table 3-4 summarizes the scenarios simulated in this modeling effort.

Table 3-4: XPSWMM Model Runs

Part	Scenario	Description	SWEL (ft NAVD88)	San Bruno Creek Discharge (cfs)			Tidegate
				@ Cupid Row	@ San Bruno Ave	@ Hwy 101	
I	1	1% SWEL & Removed Structure	+10.4	-			Removed
	2	1% SWEL & Open Flapgates	+10.4	-			Open
	3	1% SWEL & Functional Tidegates	+10.4	-			Operational
II	4	1% SWEL + 2-year Discharge	+10.4	80	300	100	Operational
	5	1% SWEL + 25-year Discharge	+10.4	250	880	310	Operational
	6	10% SWEL + 25-year Discharge	+9.1	250	880	310	Operational
	7	MHHW + 25-year Discharge	+6.4	250	880	310	Operational
	8	10% SWEL + 2-year Discharge	+9.1	80	300	100	Operational
	9	10% SWEL + 5-year Discharge	+9.1	140	490	180	Operational
	10	10% SWEL + 10-year Discharge	+9.1	200	710	250	Operational

Figure 3-1: XPSWMM Model Domain

Figure 3-2: Locations of Available Time Series from 1956 to 2009 (DHI, 2011)

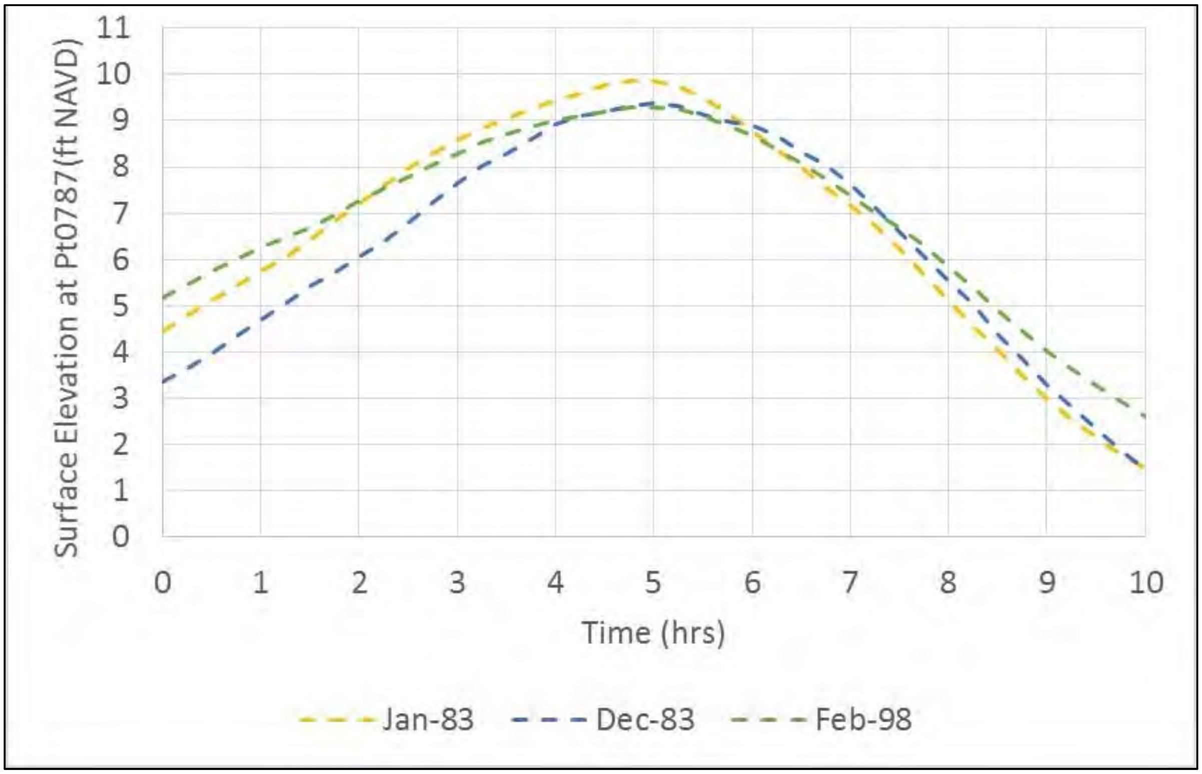


Figure 3-3: Time Series of Three Highest Water Level Events from 1956 to 2009

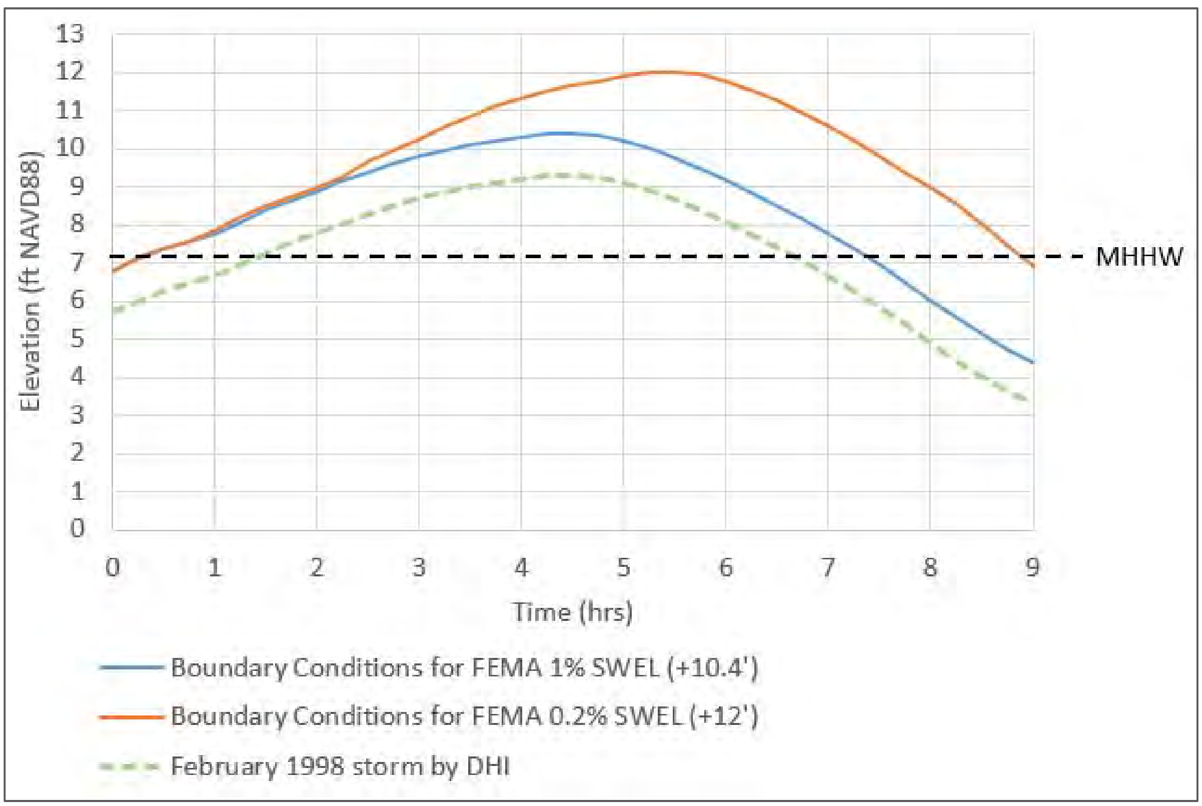


Figure 3-4: Boundary Conditions Developed for Unsteady Modeling

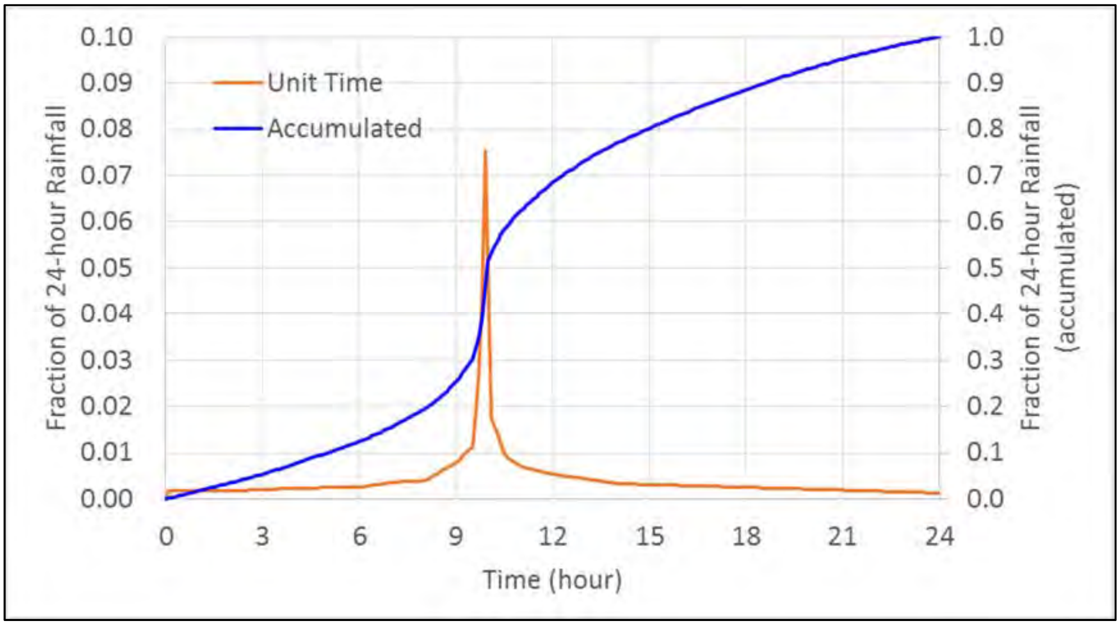


Figure 3-5: SCS Type I 24-hour Hyetograph (Moffatt & Nichol & AGS JV, 2015)

Figure 3-6: Flow Boundary Locations

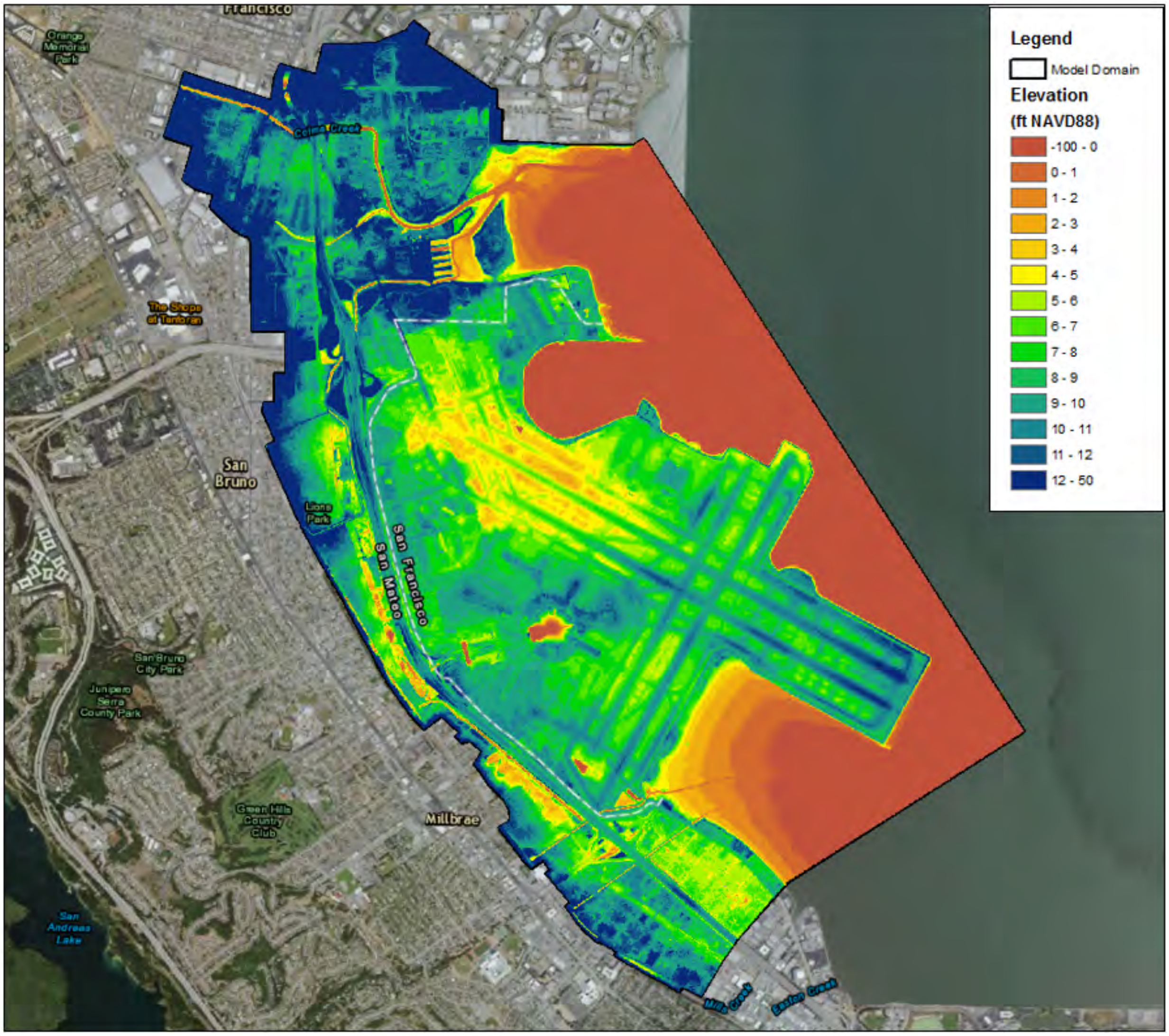


Figure 3-7: Model Topographic Data (Provided by Baker/AECOM)

Figure 3-8: San Francisco Airport Shore Protection Structures

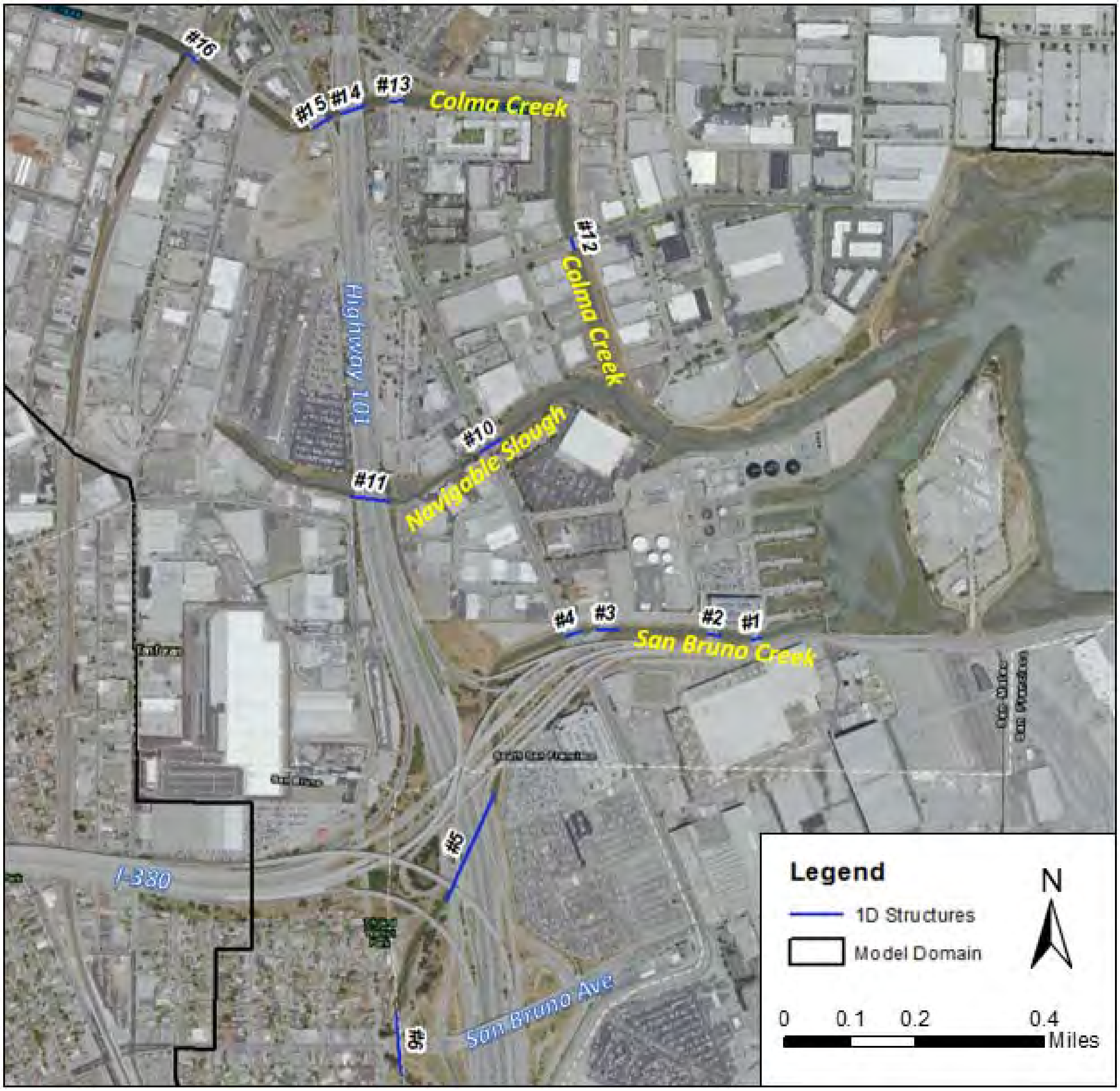


Figure 3-9: Map of 1D Structures Modeled along Colma Creek, Navigable Slough and San Bruno Creek (Shown in Blue)

Figure 3-10: Map of 1D Structures Modeled along Millbrae Creek (Shown in Blue)

4. ANALYSIS RESULTS

The following section describes results of the hydraulic analyses for the simulations described in the previous section.

4.1 Part I (Use of 2-D Unsteady Flow Model) Results

The initial phase of the hydraulic analysis is meant to compare the flood extents of the FEMA flood hazard areas with the hydraulic modeling results with a different methodology described in Section 3. The modeling effort followed FEMA's approach closely except that it used a different methodology which considers the duration of elevated water levels in the Bay and topographic features of the floodplains whereas FEMA's methodology did not account these as described in Section 0.

During the San Bruno Creek/ Colma Creek Resiliency Study, flooding sources for Belle Air neighborhood were evaluated through modeling and DTM elevation checks. The following section discusses flood sources/paths (shown in Figure 4-1) that were identified to have led to inundation of portions of the Belle Air neighborhood. The inundation areas in Belle Air neighborhood is divided into two sub-areas separated by Pine Street:

1) Areas of the neighborhood north of Pine Street will experience flooding from flood waters overtopping banks of Navigable Slough, traveling south along Shaw Road and crossing under I-380 to 7th Avenue. There appears to be low spots below the BFE along the south bank of Navigable Slough. Just north of Pine Street, there is an area of slightly higher elevation that is thought to prevent the flood water from Navigable Slough to travel farther south.

2) Areas of the neighborhood south of Pine Street will experience flood due to three potential sources:

a) SFIA. Flood waters would enter the SFO property since SFO is protected by a non-accredited levee system which was removed in the coastal flood hazard analysis allowing the airport to flood as if water could enter the area unrestricted. The flood waters would first fill the topographic basin where the airport is located to an elevation of approximately +9.4 feet NAVD88, after which the flood waters would overtop Hwy-101, then get into the Belle Air neighborhood.

b) Cupid Row Canal. There are low spots on banks along the Cupid Row Canal, allowing water to overflow the bank, flow north for approximately 12,000 feet along an overland flow-path adjacent to Hwy 101 and then enter into the Belle Air neighborhood. Based on modeling results, this would occur before northward flowing flood water coming from Hwy-101 discussed in the next bullet.

c) Millbrae. The low coastline of Millbrae located south of the SFO would allow flood waters to enter and flow northward along Hwy 101, and then into the Belle Air neighborhood.

The comparison of the two scenarios is shown in Figure 4-2. When comparing the two figures shown in Figure 4-2, only the area denoted in blue on the FEMA flood hazard map (left) should be considered. Comparing the flood extents in the Belle Air neighborhood, shown in the zoomed maps to the right of each picture, it is evident that using a scientifically more accurate methodology has a significant impact on the area shown as inundated during the 1% annual chance base flood (shaded blue). All of the flooding south of San Bruno Avenue and Pine Street is alleviated.

The results of the modeling scenarios with the tidegate structure in place but not operational (open flapgates) and the complete removal of the tidegate structure were compared (Scenarios 1 and 2). The modeling found that there is no difference in flood extent for the two scenarios during the 1% annual chance flood hazard.

Figure 4-3 compares the model scenarios of the non-operational tidegate and the operational tidegate (Scenarios 2 and 3) in San Bruno Creek to see if the current tidegate structure will reduce flooding within Belle Air neighborhood. The figure shows that there is minimal difference between the two scenarios. However, flooding in the Belle Air neighborhood is alleviated when a tidegate structure is included in Navigable Slough, as shown in Figure 4-4.

4.2 Part II (Combined Coastal and Fluvial Flows) Results

Part II of the modeling effort involved joint effects of elevated Bay water levels (tail water) with San Bruno Creek discharge. Seven scenarios, as presented in Table 3-4, were modeled to evaluate the performance of the tidegate structure. The results of these simulations are depicted in Figure 4-5 through Figure 4-8.

Figure 4-5 compares the two simulations conducted with the 1% annual chance SWEL (+10.4 feet NAVD88). Both a 2-year and 25-year discharge were run and neither flow was able to pass the tidegate at this tail water condition. Flooding within Belle Air neighborhood is evident in both scenarios.

Figure 4-6 shows two other tail water conditions (MHHW and 10% SWEL) for the 25-year discharge. Again, the 25-year discharge does not pass the tidegate structure in either scenario, even when the tail water condition is taken as +6.4 feet NAVD88, approximately MHHW at this location. Flood water do appear to overtop the banks of Cupid Row Canal and enter the southern portion of Belle Air neighborhood.

Figure 4-7 depicts the only two scenarios simulated in which the flow did pass the tidegate structure. Both scenarios have a tail water condition of the 10% annual chance SWEL of +9.1 feet NAVD88. Scenario 8 shows the 2-year discharge and Scenario 9 shows the 5-year discharge. Even though the flow passes the tidegate structure, some flooding does occur due to low areas along Cupid Row Canal. The 10-year discharge, shown in Figure 4-8 paired with the same tail water condition, does not pass the tidegate.

It should be noted that discharge values will not influence the flooding in Belle Air neighborhood north of San Bruno Ave and Pine Street because the flood source for this area comes from Navigable Slough, not San Bruno Creek. However, the Belle Air neighborhood south of San Bruno Ave and Pine Street experiences flooding in all but one scenario (Scenario 8 for the 10% SWEL with the 2-year discharge). Scenario 9, showing the 10% SWEL with the 5-year discharge, does show some flooding, however, the depth is on the order of inches. Other scenarios show that flooding depths in Belle Air neighborhood are generally between 0 and 1.5 feet.

Figure 4-1: Flood Sources and Paths of Belle Air Neighborhood

(Inundation Area by 1% Annual Chance Flood Shaded in Light Blue)

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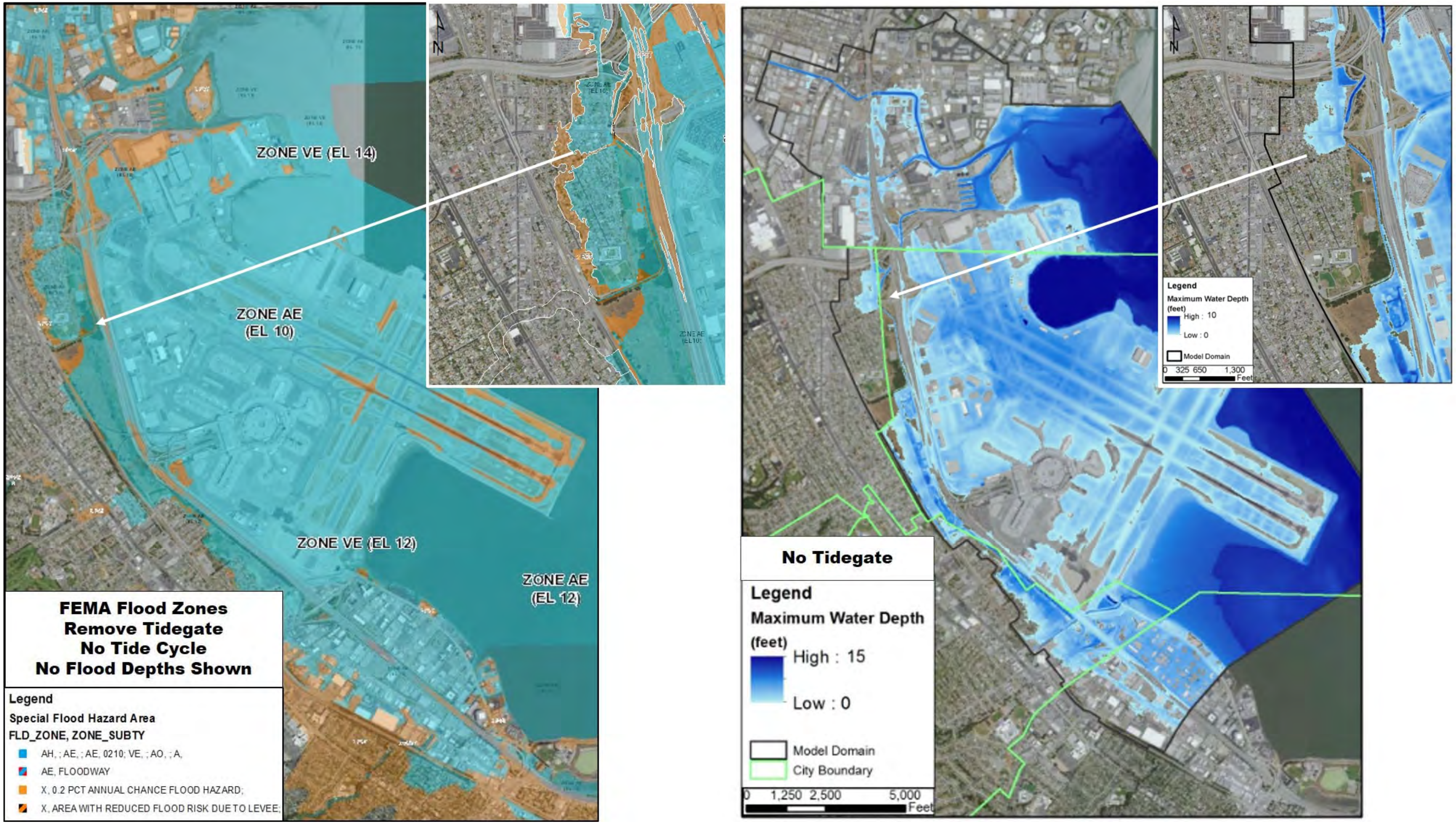


Figure 4-2: Comparison of Flood Hazard Areas for Preliminary FIRM (left) and for 2-D Unsteady Modeling (right) for the 1% Annual Chance Flood

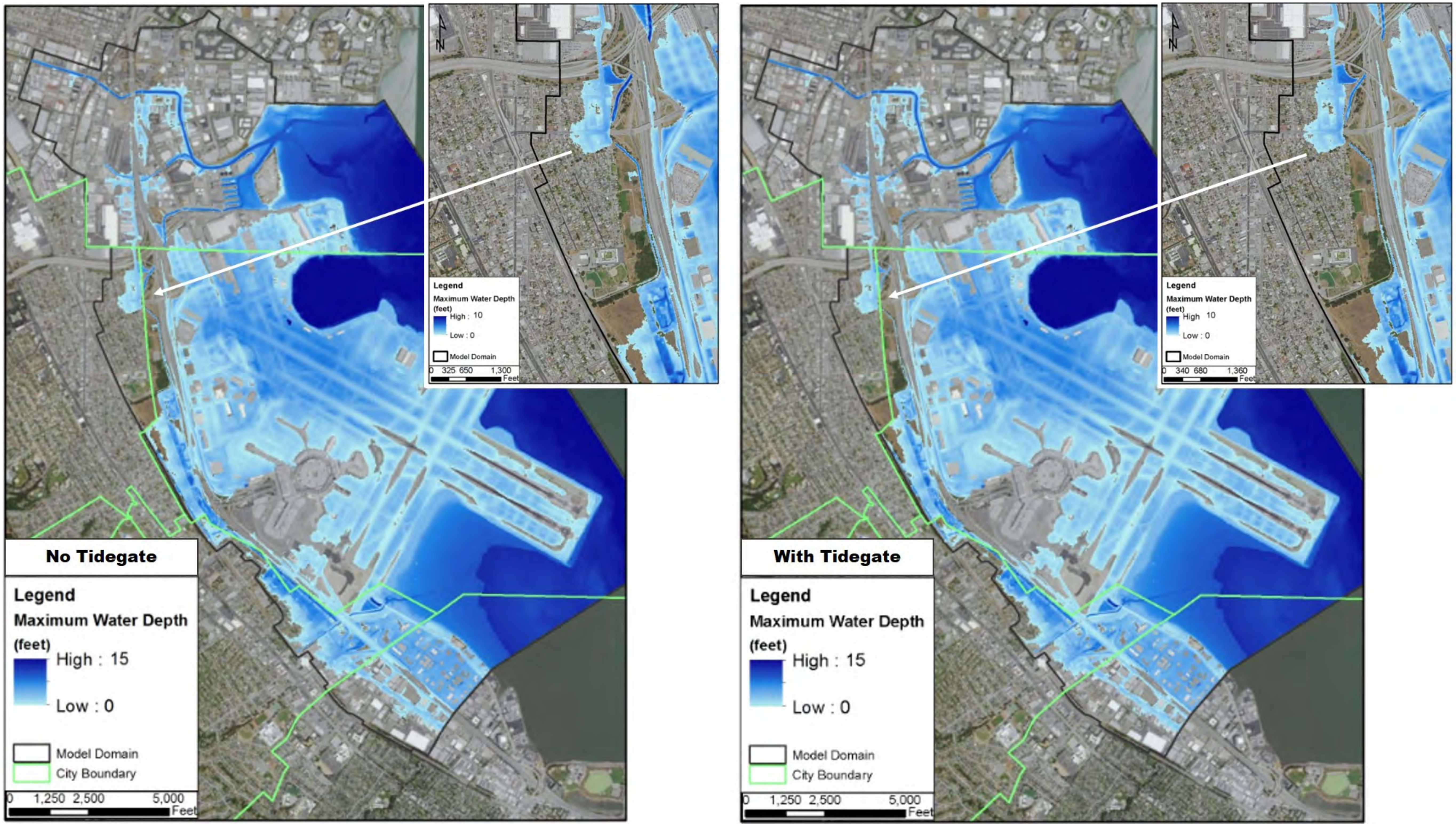
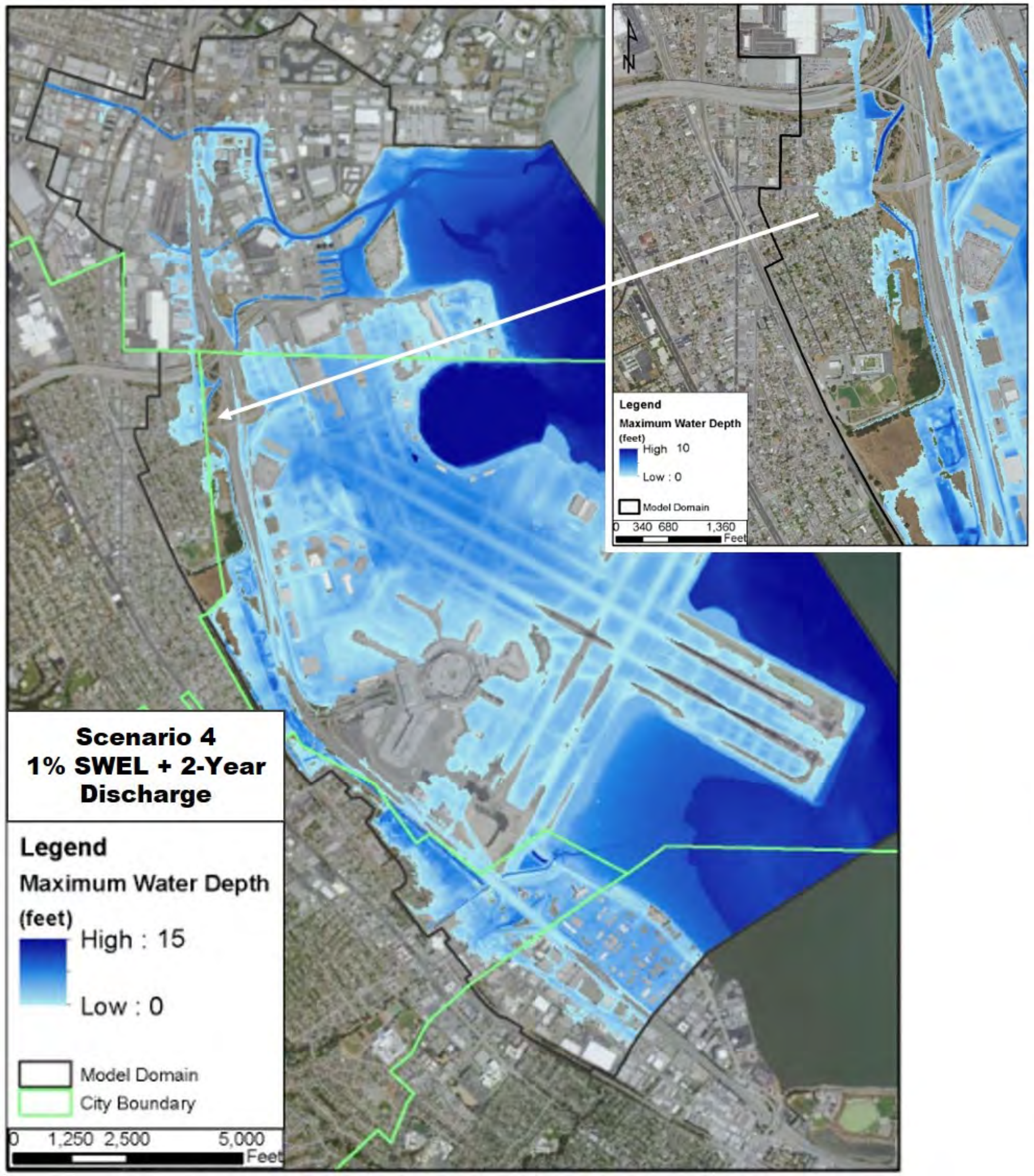


Figure 4-3: Comparison of Flood Areas Between Non-operational Tidegate (left) and Operational Tidegate (right) for the 1% Annual Chance Base Flood

Figure 4-4: Flood Extents with Operational Tidegate at San Bruno Creek and Navigable Slough



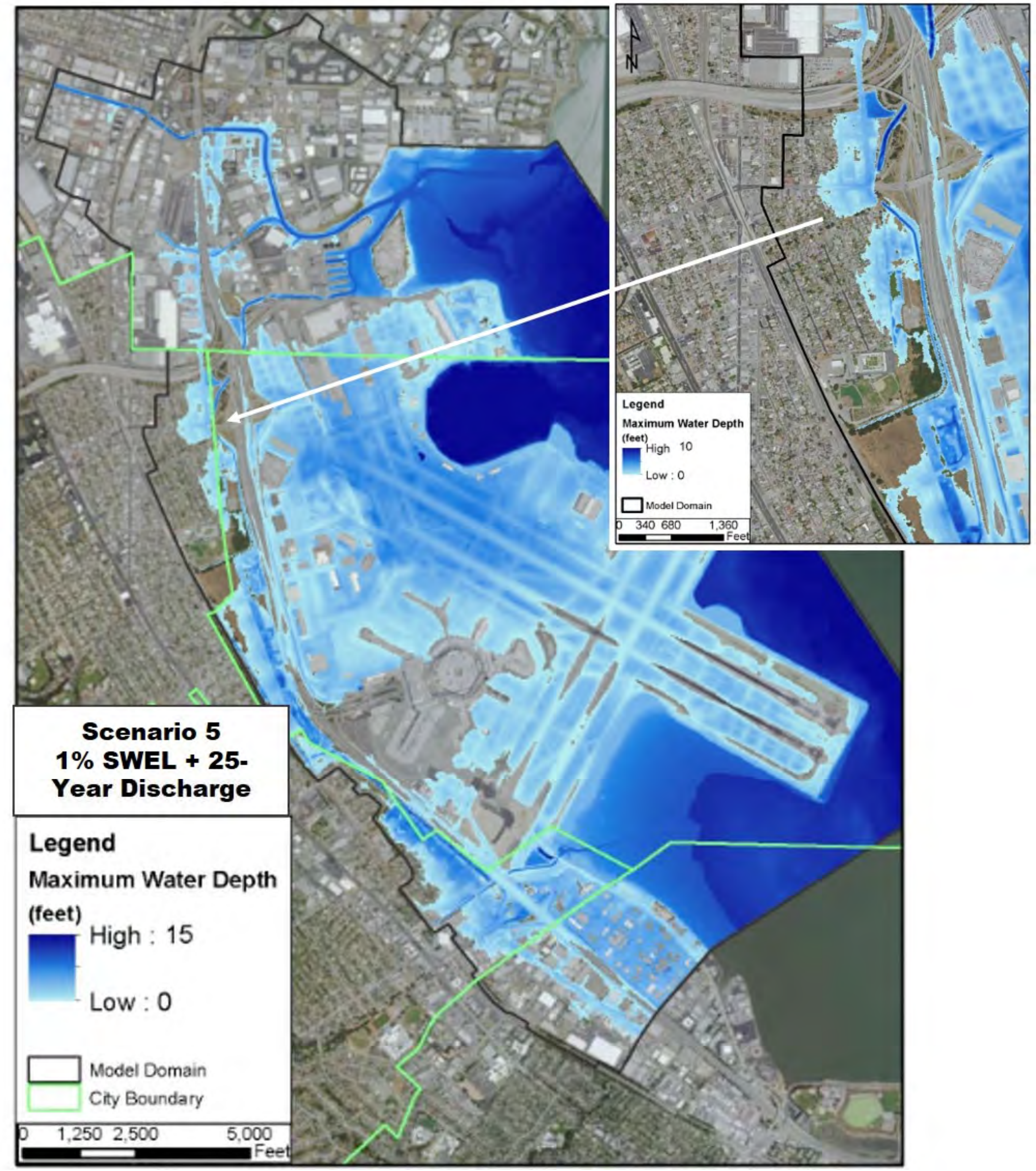


Figure 4-5: 1% Annual Chance SWEL Tailwater Condition with 2-yr Discharge (left) and 25-yr Discharge (right)

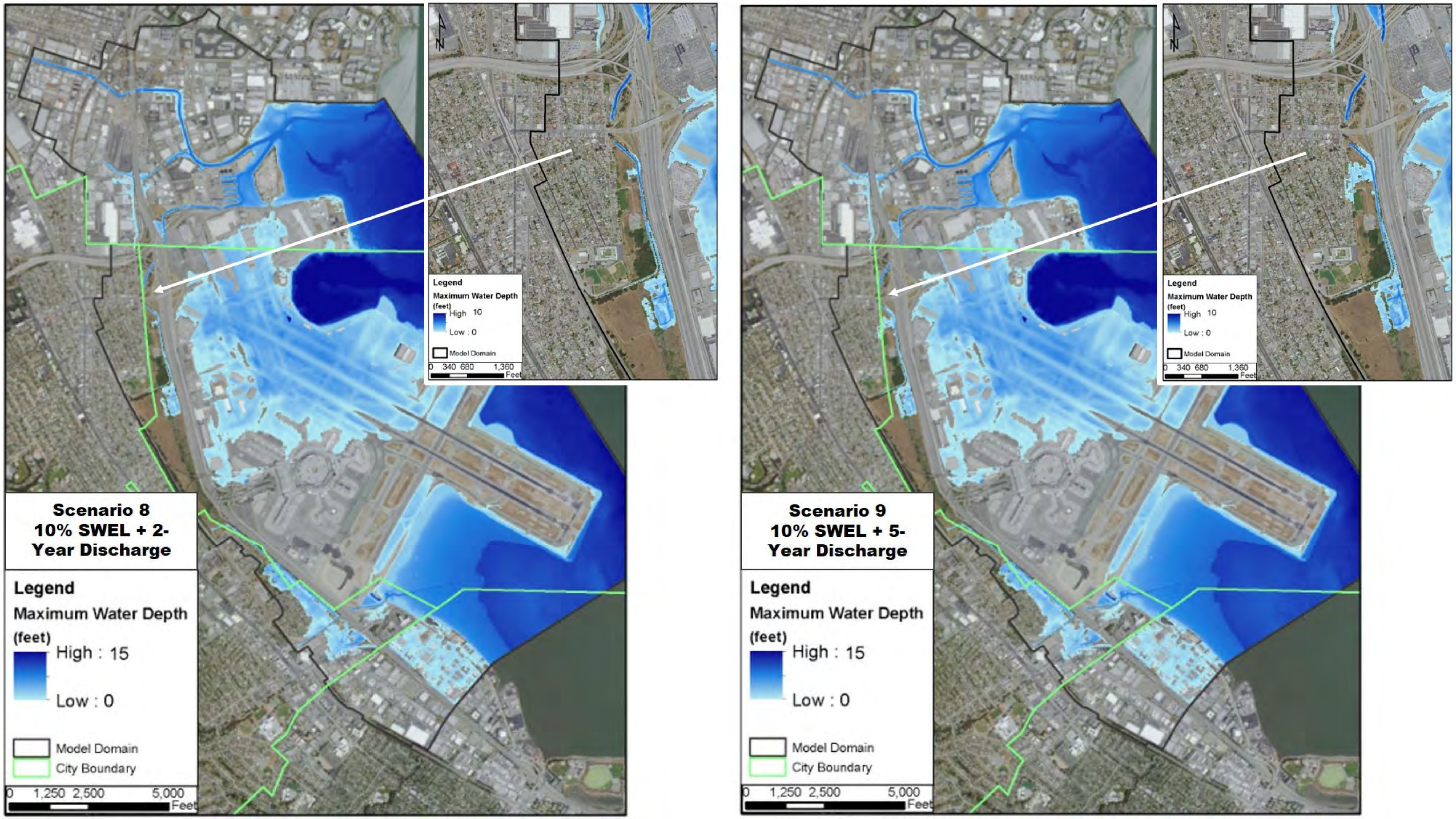


Figure 4-6: 10% Annual Chance SWEL Tailwater Condition with 2-Year Discharge (left) and 5-year Discharge (right)

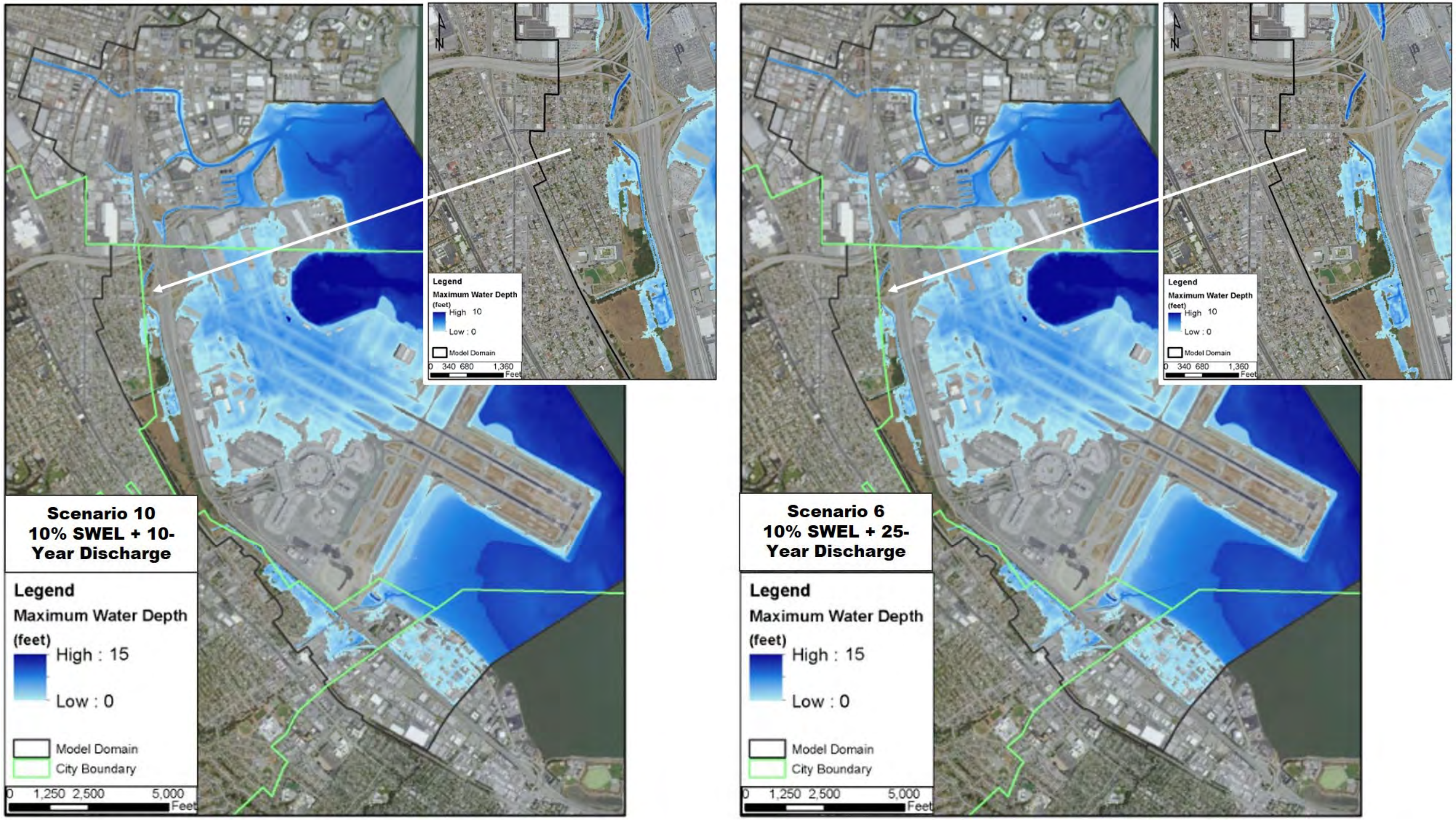


Figure 4-7: 10% Annual Chance SWEL Tailwater Condition with 10-Year Discharge (left) and 25-year Discharge (right)

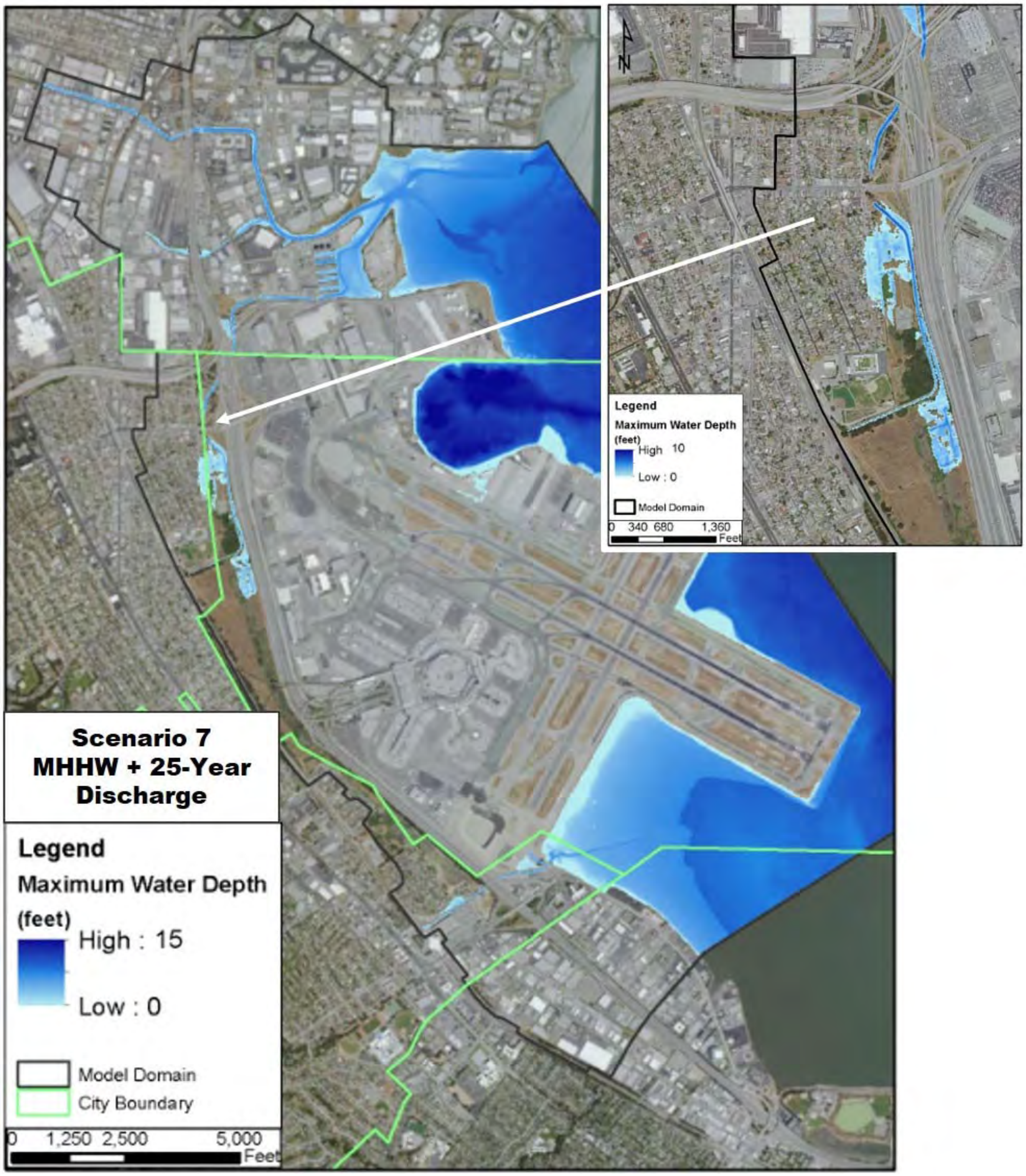


Figure 4-8: MHHW Tailwater Condition with a 25-Year Discharge

5. CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the hydraulic analyses, the following conclusions and recommendations were developed for San Bruno Creek.

5.1 Part I Conclusions – Use of 2-D Unsteady Flow Model

Part I of the hydraulic analyses examined the coastal flooding component of the FEMA flood hazard area mapping in the vicinity of San Bruno Creek. The following conclusions are drawn from this modeling effort.

5.1.1 Methodology of analyses has significant impact on flood extents.

Examining the FEMA flood hazard area map, which appears to extend the base flood elevation within San Francisco Bay to the intersection with the topography of the same elevation, the duration of elevated water levels within the Bay and topographic features are not considered. By using an unsteady state 2D model and applying a tidal cycle to the elevated water levels, the analysis shows that flooding will not reach as far inland as shown on the FEMA's preliminary flood hazard areas. Specifically, the Belle Air neighborhood south of San Bruno Ave and Pine Street is shown to have significantly reduced flooding.

Currently, there are approximately 340 parcels within Belle Air neighborhood located within the FEMA flood hazard area. Using a scientifically more accurate methodology reduces the number of parcels to approximately 102 parcels within the flood hazard area.

5.1.2 No difference is seen in the "Open Flapgates" and "Removed Tidegate" scenarios.

Because the tidegate structures have not been accredited, FEMA methodology dictates that flood hazard mapping be conducted by removing the entire structure from the analysis and treating the outlet into San Francisco Bay as an open channel. FEMA will allow the structure to remain in place, but be non-operational, if the structure can be shown to survive the 1% annual chance base flood. This condition will allow the presence of the structure to provide incremental benefits to the flood protection. Part I of the hydraulic analysis examined whether allowing the structure to remain in place, but opening the flapgates to allow flow into San Bruno Creek, would alter the extents of flooding. No difference was shown to exist between the scenario with the tidegate present and flapgates open and the scenario with the whole structure removed.

5.1.3 Including the operational tidegate into the flood hazard mapping will not reduce flooding seen in south Belle Air neighborhood.

Compared to the scenario with no tidegate structure present but the water elevated during a typical tidal cycle (Scenario 2), there is little difference in flood extent with the operational tidegate structure (Scenario 3). The same number of parcels (102) are within the flood extents in both scenarios.

5.1.4 Constructing a tidegate at Navigable Slough would reduce flooding in North Belle Air neighborhood.

The model scenario depicting the San Bruno Creek tidegate as operational and a constructed tidegate at Navigable Slough (at the Caltrains tracks) results in no flooding within Belle Air neighborhood.

5.2 Part II Conclusions – Combined Coastal and Fluvial Flows

Part II of the hydraulic analyses analyzed the performance of the San Bruno Creek tidegate by combining extreme coastal SWELs as tail water conditions with riverine storm discharges in San Bruno Creek. Based on this analysis, the following conclusions can be drawn:

5.2.1 The tidegate structure cannot pass the 25-year return period discharge.

The 25-year return period riverine discharge was paired with the 1% and 10% annual chance coastal SWELs as well as the MHHW tail water condition. None of these SWELs allowed for the riverine flow to exit the tidegate, causing flow to collect within the channel. This indicates that the tail water level is too high for the riverine flow to discharge to the Bay. Flooding in south Belle Air neighborhood resulted in all three scenarios.

5.2.2 The tidegate cannot pass a 2-year riverine storm discharge during the 1% annual chance coastal SWEL

Even for a 2-year return period riverine flow, the lowest of the riverine storm discharges modeled, there was backwater and overflow at low bank elevations.

5.2.3 The tidegate can only pass flows less than the 5-year return period riverine discharge during the 10% annual chance coastal SWEL

For the 10% annual chance coastal SWEL, the tidegate could only pass storms less than the 5-year return period discharge. The scenario of the 10% annual chance coastal SWEL and 10-year riverine discharge did not pass the tidegate.

In summary, the hydraulic analysis conducted in Part 1 shows that there exists very little benefit to trying to certify the San Bruno Creek tidegate structure (see Figure 4-3). The presence of the structure is not shown to reduce the number of residents required to have flood insurance under the NFIP compared to the scenario with no tidegate structure.

5.3 Tidegate Certification Requirements

While FEMA has stringent design criteria and certification related guidance for levees and floodwalls, it does not have specific standards for the evaluation of a tidegate or other coastal structures that affect flooding. Tidegates are treated as miscellaneous coastal structures that are identified and evaluated for Flood Insurance Studies using historical evidence, readily-available data and engineering judgement for determining their influence on coastal hazards mapping.

In the absence of specific criteria for such structures, FEMA adopted the US Army Corps of Engineers' Technical Report CERC-89-15, "Criteria for Evaluating Coastal Flood-Protection Structures" in 1991. The proposed criteria establish the conditions, procedures, and standards under which coastal flood protection structures would he credited on NFIP Flood Insurance Rate Maps as providing protection from the base flood.

Direction from the FEMA Administrator to Region Directors of FEMA (FEMA 1990), along with the relevant criteria is included in Appendix B. The actual Form MT-2 which would have to be completed to apply for a LOMR is included in Appendix C.

Based on the basic data requirements for the evaluation of such coastal structures, the following information is expected to be necessary

- Type, location, basic layout, and crest elevation of structure;
- Dominant site particulars (e.g. local water depth, tide, surge, and wave conditions, erosion rate, sediment characteristics and geotechnical conditions, debris hazards, and ice climate);
- Construction materials and present integrity;
- Historical record for structure, including construction data, maintenance plan, responsible party, repairs after storm episodes;
- Clear indication of effectiveness or ineffectiveness;

Mapping of areas protected by coastal flood protection structures

A listing of the required criteria for evaluation of the structure is provided below. The guidance attached in Appendix B provides additional details.

A. General

Coastal flood protection structures should meet, and continue to meet, minimum design and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria established by 44 CFR Part 60.3.

B. Design Criteria (specified in guidance)

1. Design Parameters:
    i. Design water Levels
    ii. Wave Conditions
    iii. Breaking Wave Forces
2. Minimum Freeboard
3. Toe Protection
4. Backfill Protection
5. Structural Stability (Minimum Water Level)
    i. Geotechnical analyses
    ii. Engineering analyses
6. Structural Stability (Critical Water Level)
    i. Geotechnical analyses
    ii. Engineering analyses
7. Material Adequacy
8. Ice and Impact Alignment (where appropriate)
9. Structure Plan Alignment
10. Other Design Criteria

C. Adverse Impact Evaluation

D. Community and/or State Review

E. Maintenance Plans and Criteria

F. Certification Requirements

Data and analyses submitted to support that a given coastal flood protection structure complies with the structural design requirements set forth in paragraphs (B) (1) through (10) above must be certified by a registered professional engineer.

Additionally, because FEMA has not standardized the criteria necessary for evaluating miscellaneous structures such as tidegates, FEMA may request additional information to be presented for accreditation. For example, a recent restudy of the Chesapeake Bay required a tidegate structure to be evaluated for the 1% annual chance discharge paired with a MHW. Though not specifically studied in this hydraulic analysis, based on the results of the 25-year return period discharge paired with a MHHW tail water condition, it can be inferred that the storm drainage system does not have enough capacity for the 100-year return period discharge, even at a lower water level.

5.4 Additional Recommendations

The results of Part II of the hydraulic analysis show that flooding occurs from San Bruno Creek even with a 5-year return period discharge when the tidegate is operational (Figure 4-6) due to deficiencies in both the channel capacity and the tidegate structure. To improve the functionality of the system, improvements can be made that include raising the low-spots along the bank of the open channel, raising the tidegates so that they discharge at a higher elevation, or increasing the size of tidegate structure.

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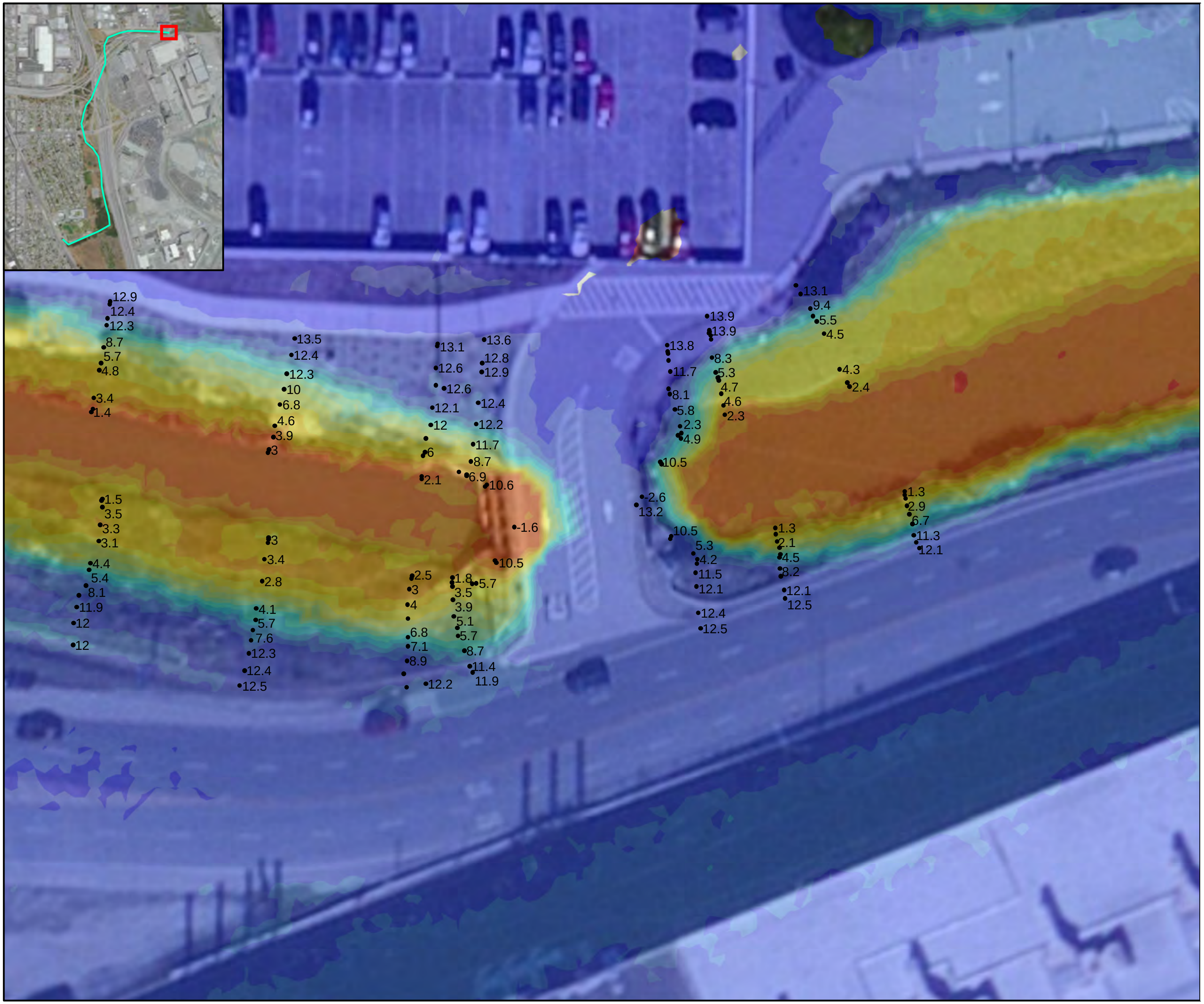
APPENDIX A
San Bruno Creek Supplemental Surveys

(Meridian Surveying Engineering, Inc. 2014)



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Feet

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Elevation

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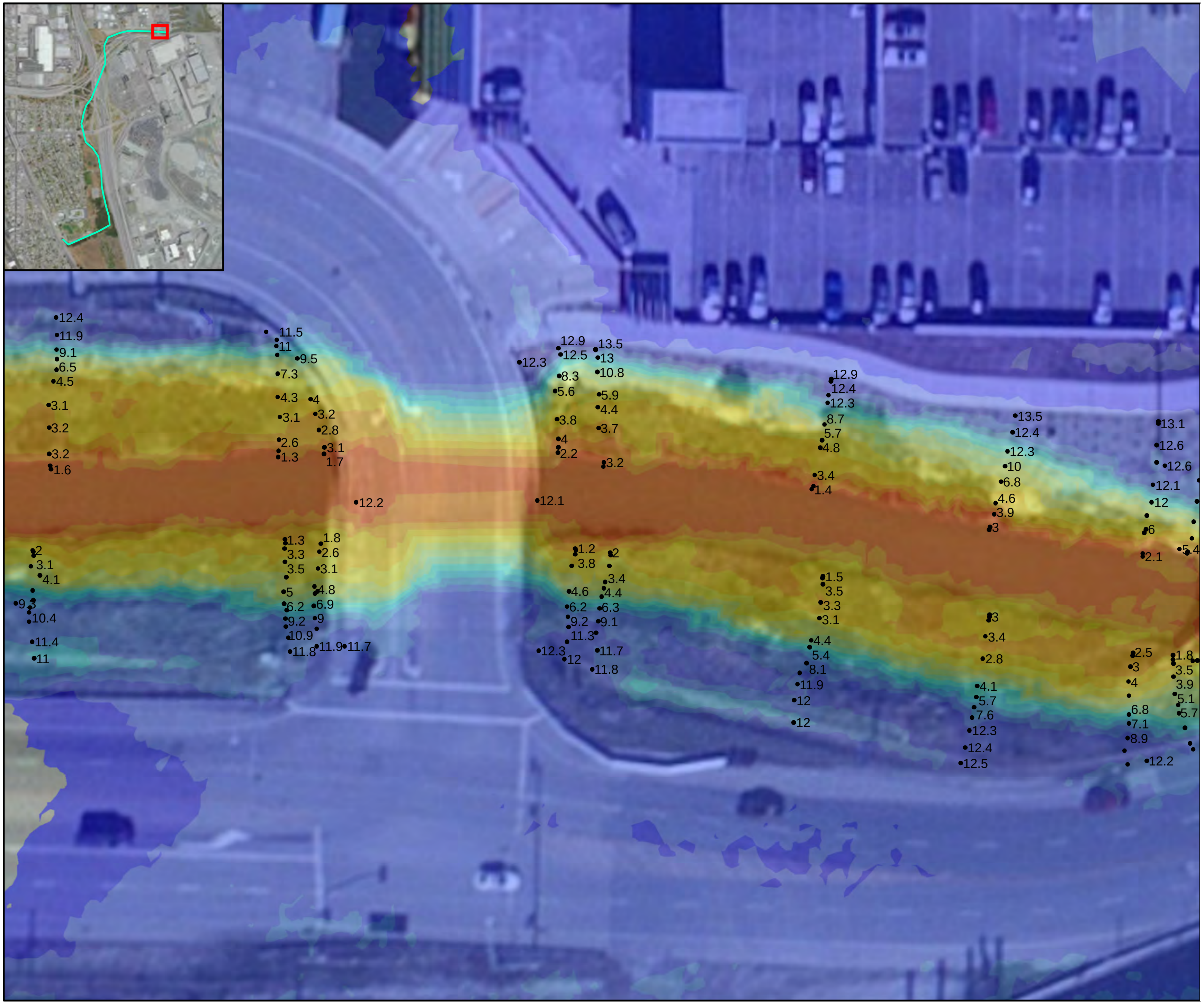

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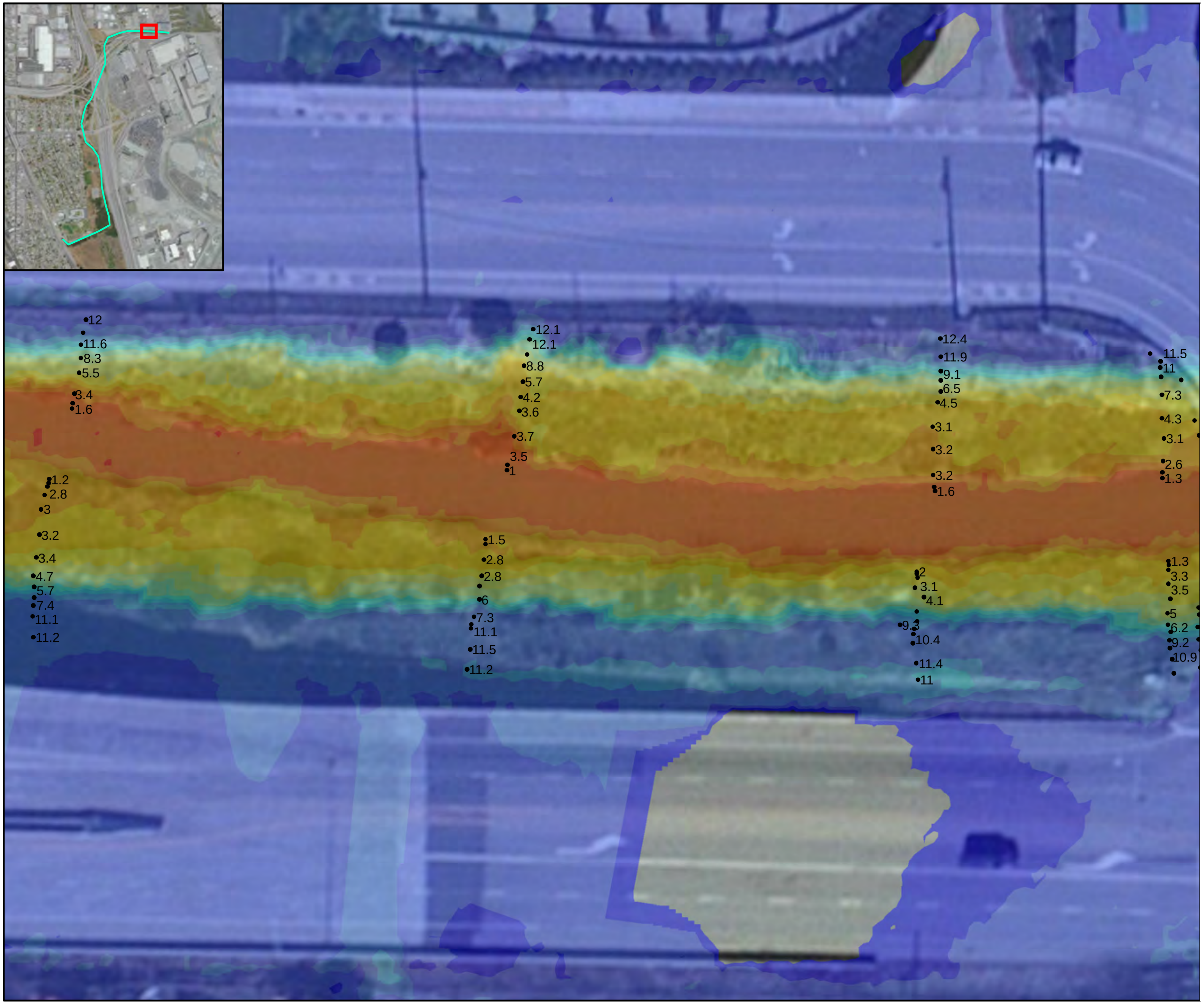

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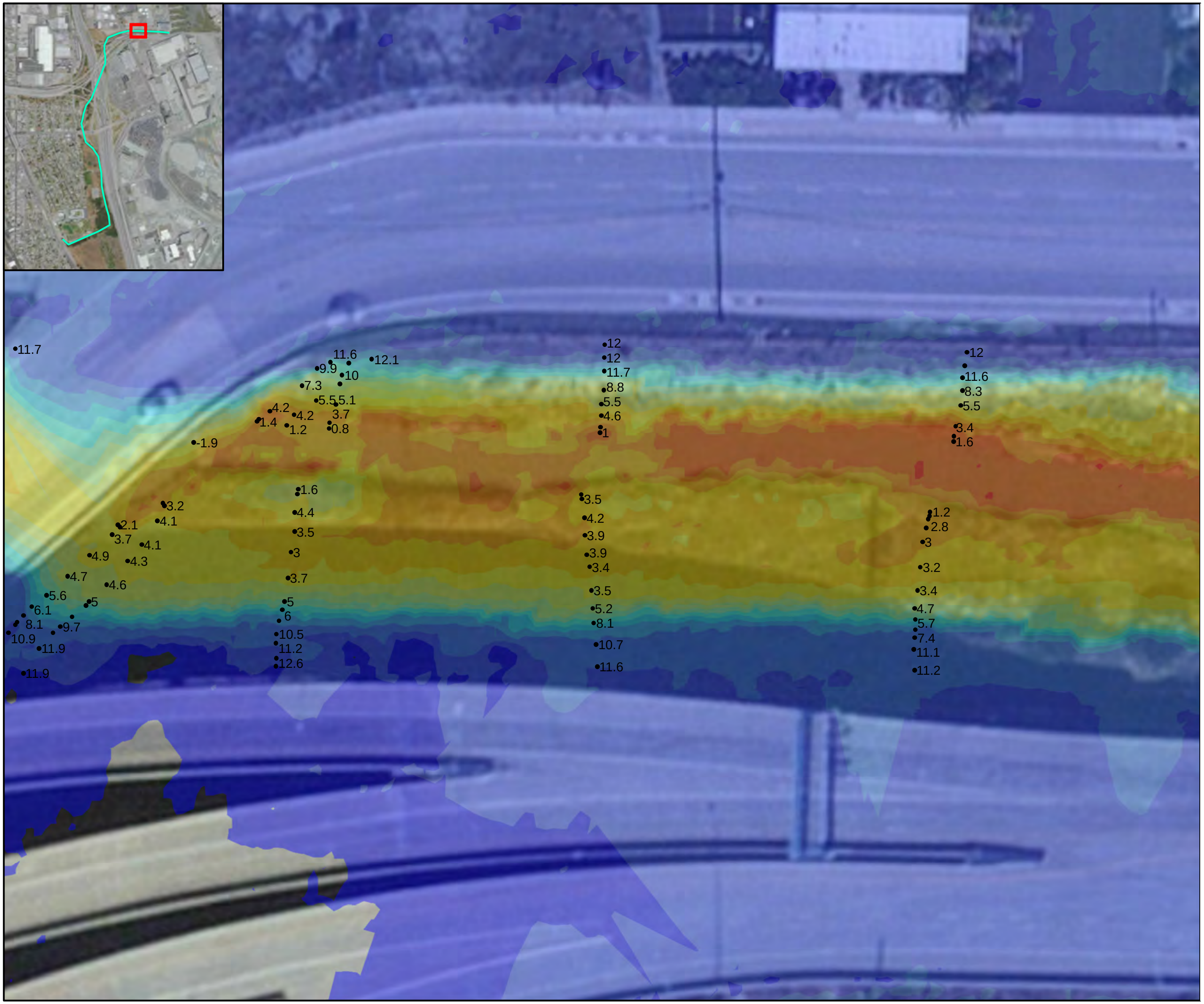

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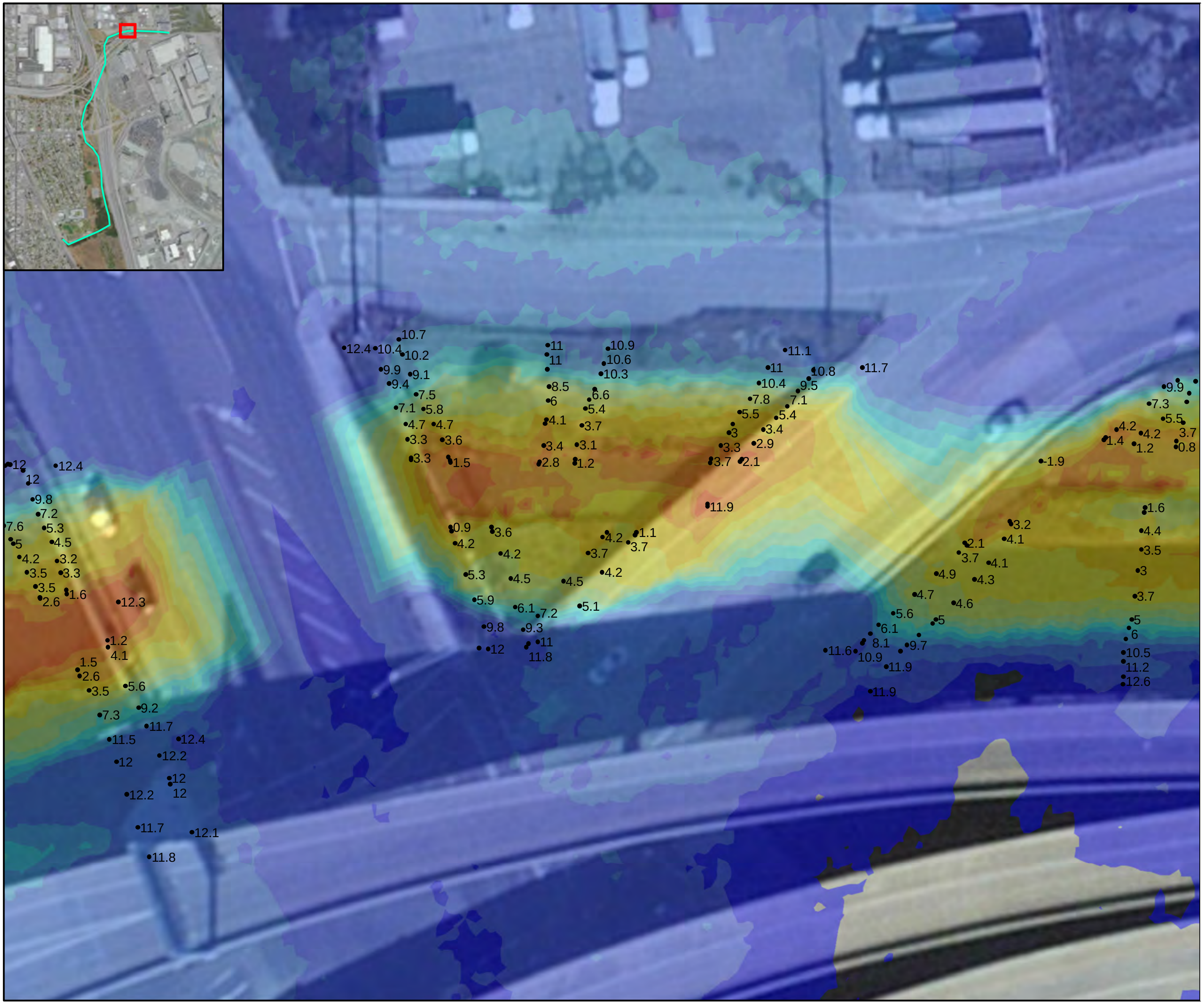

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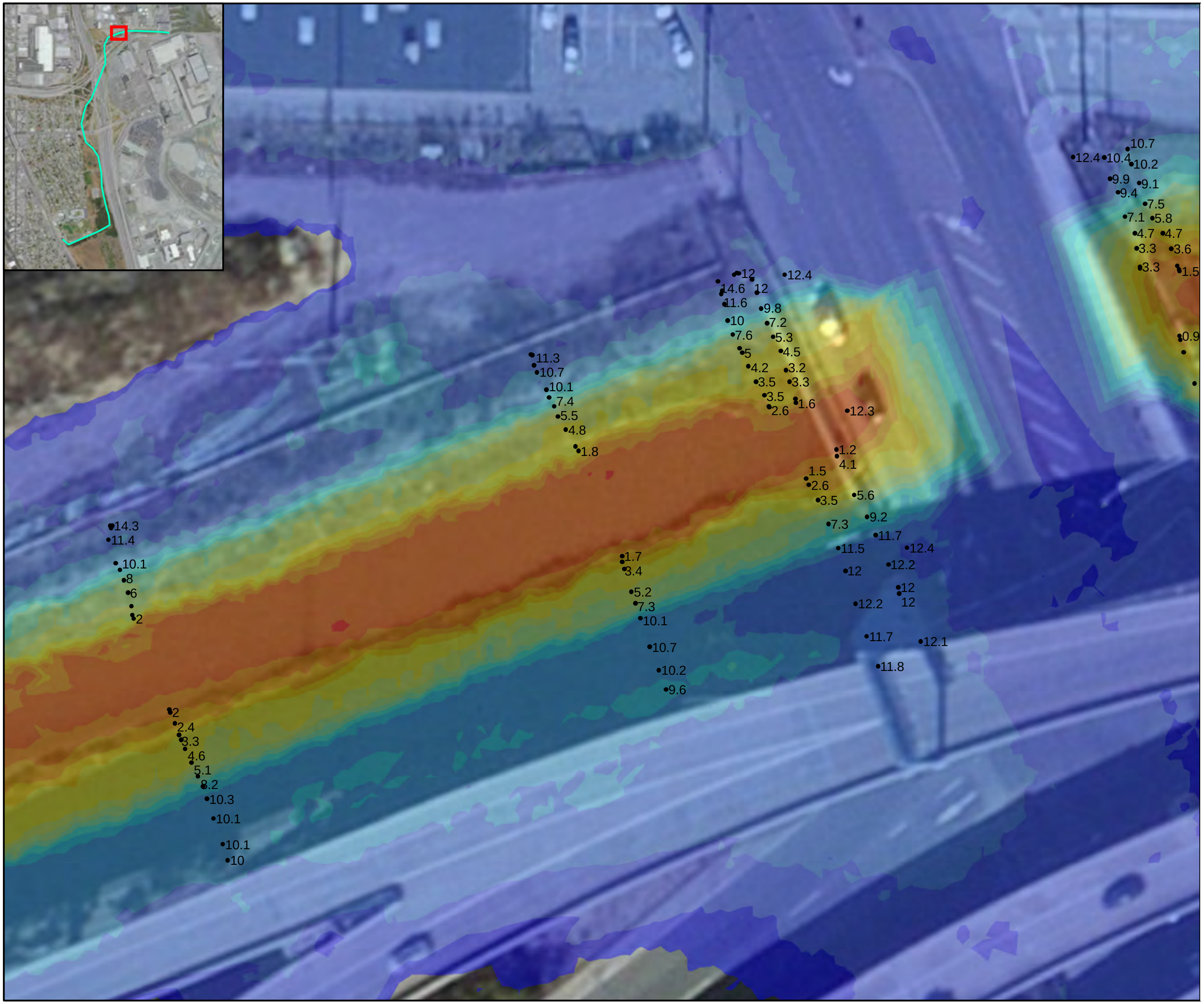

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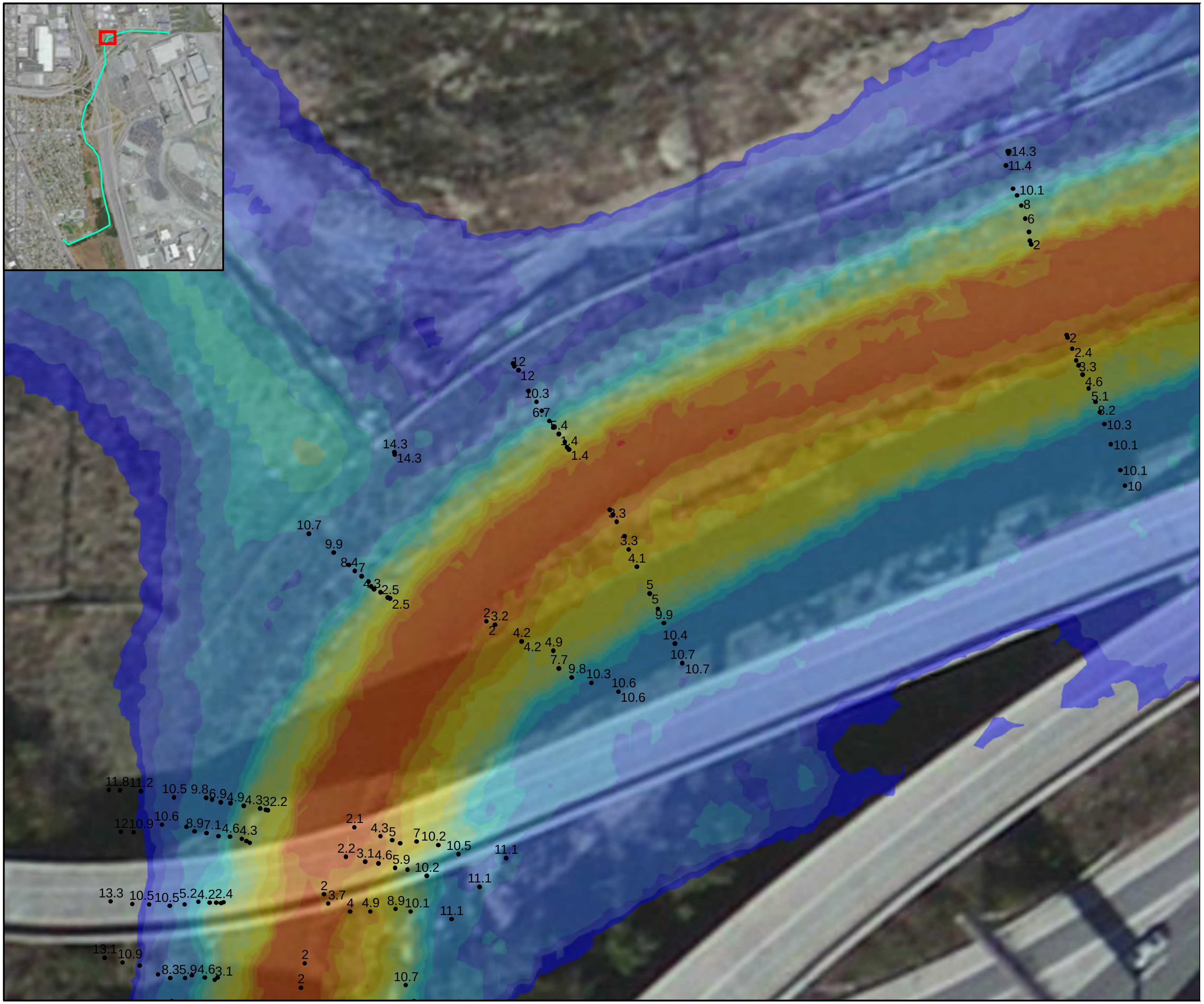

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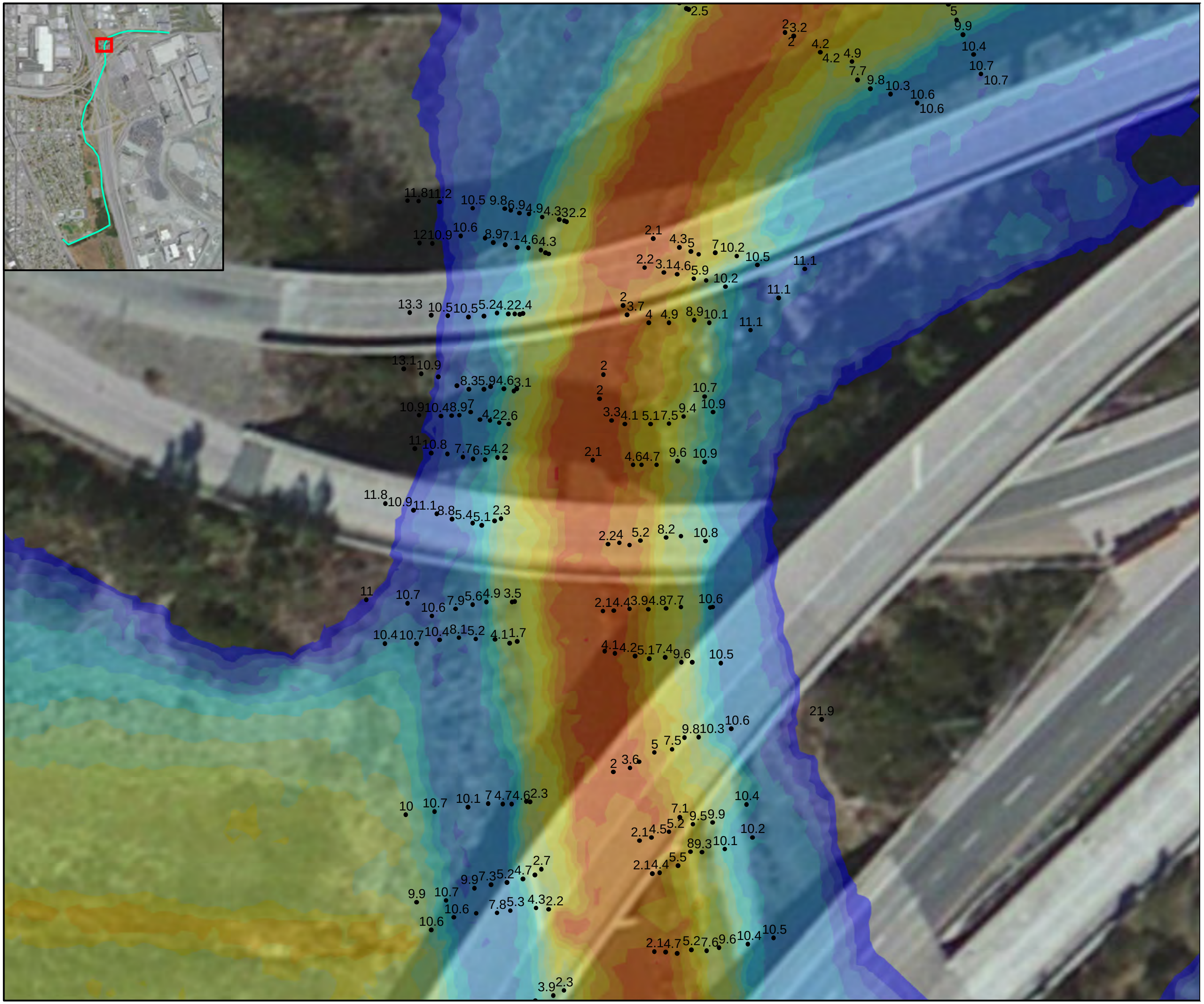

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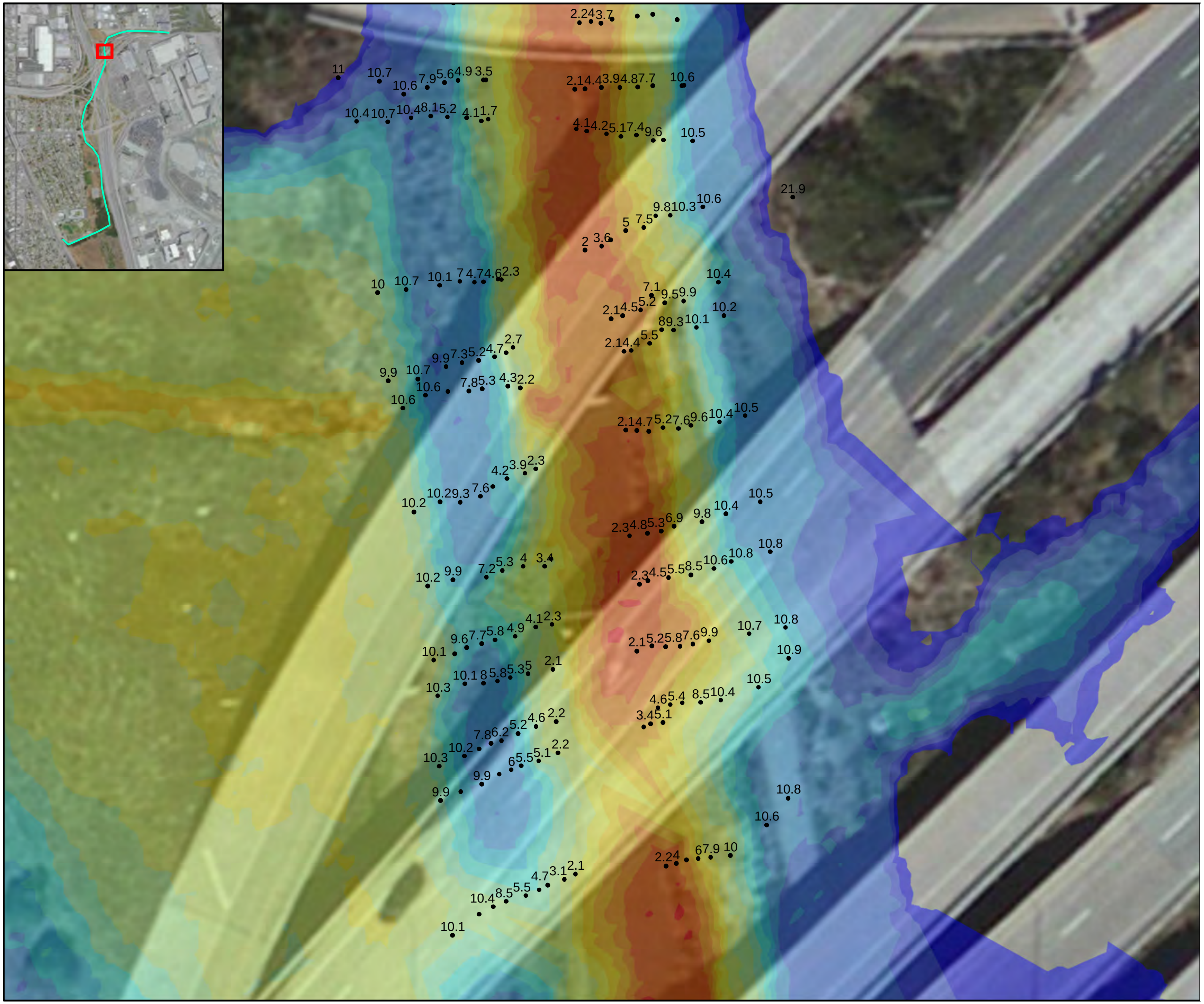

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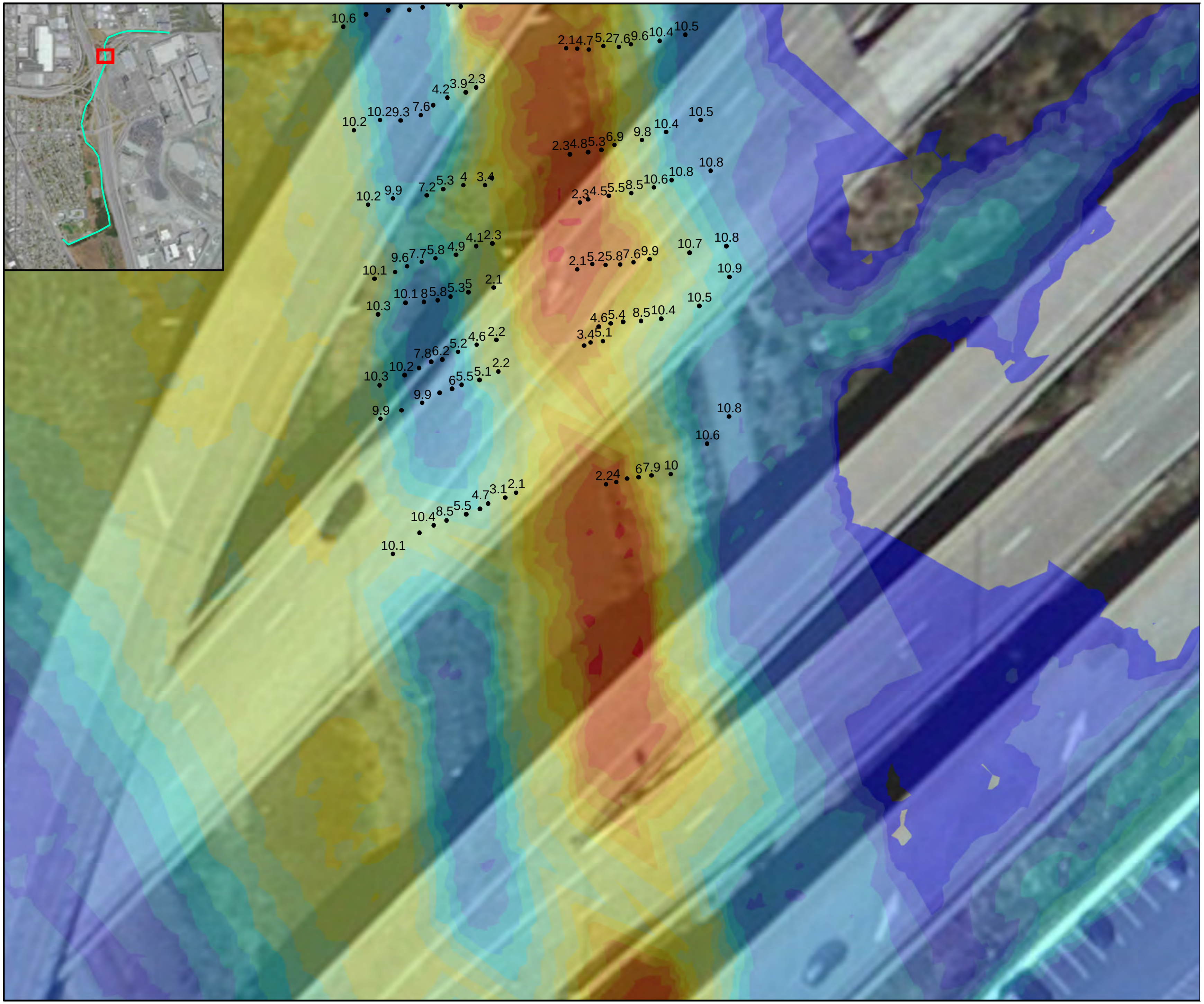

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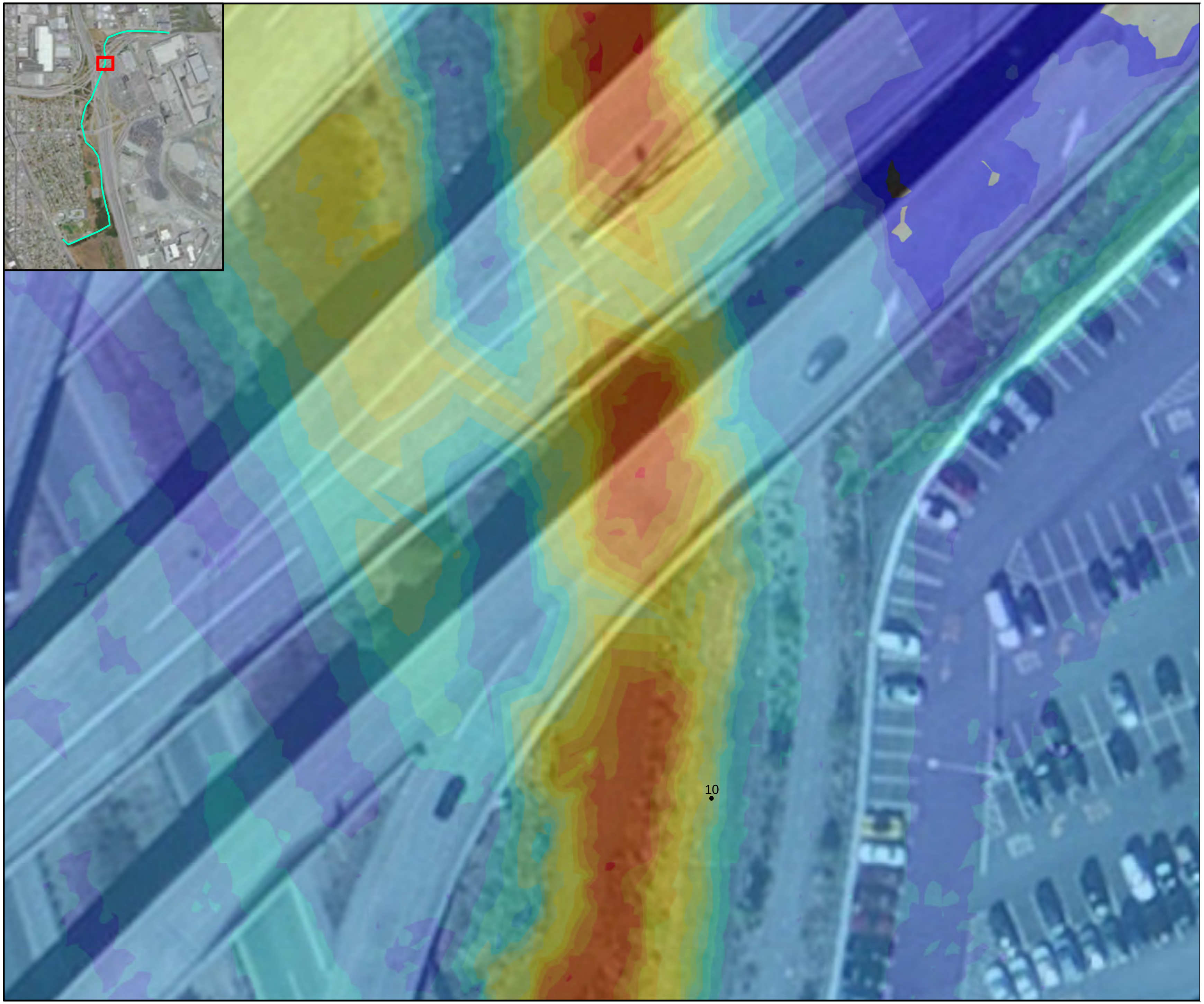

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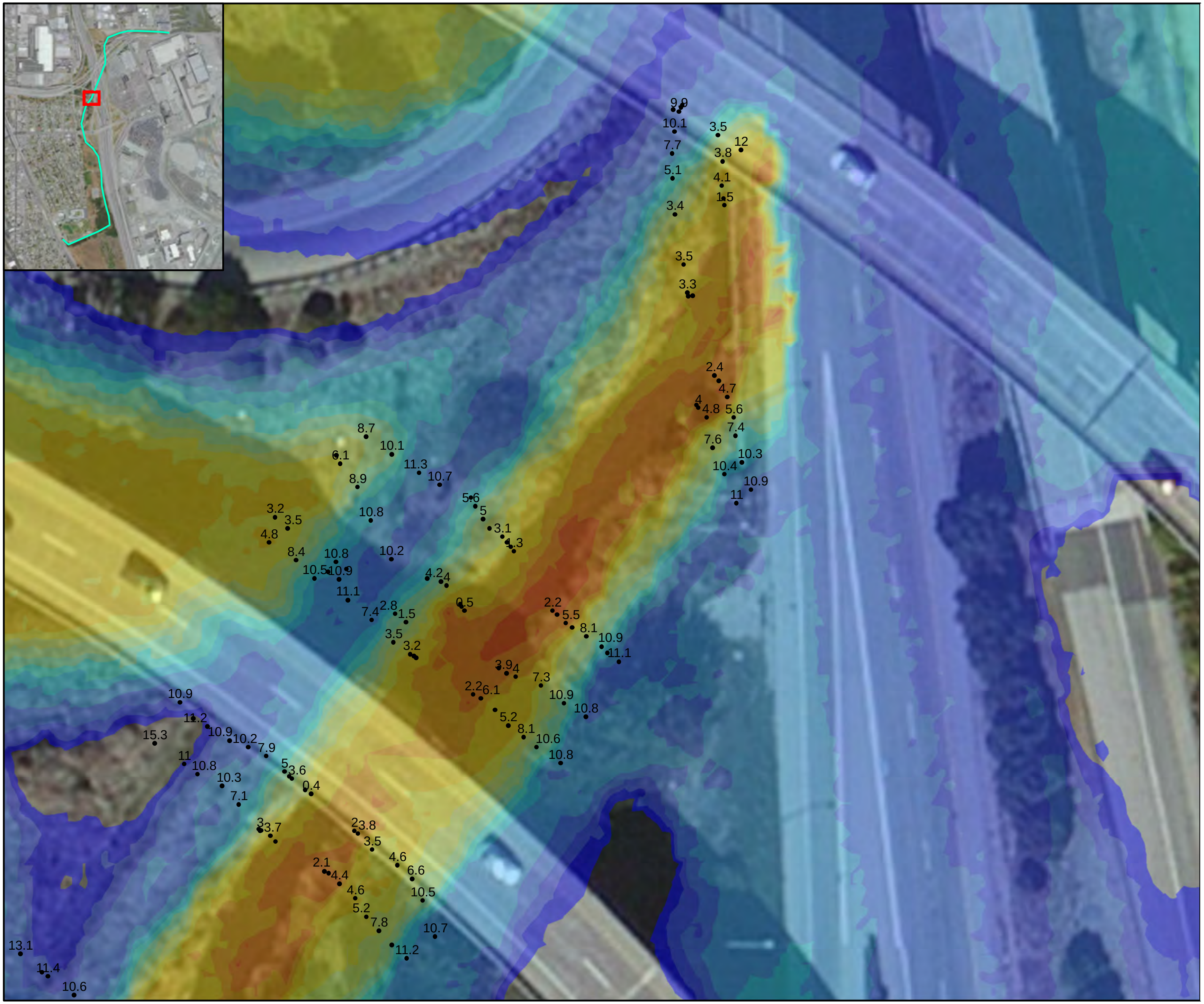

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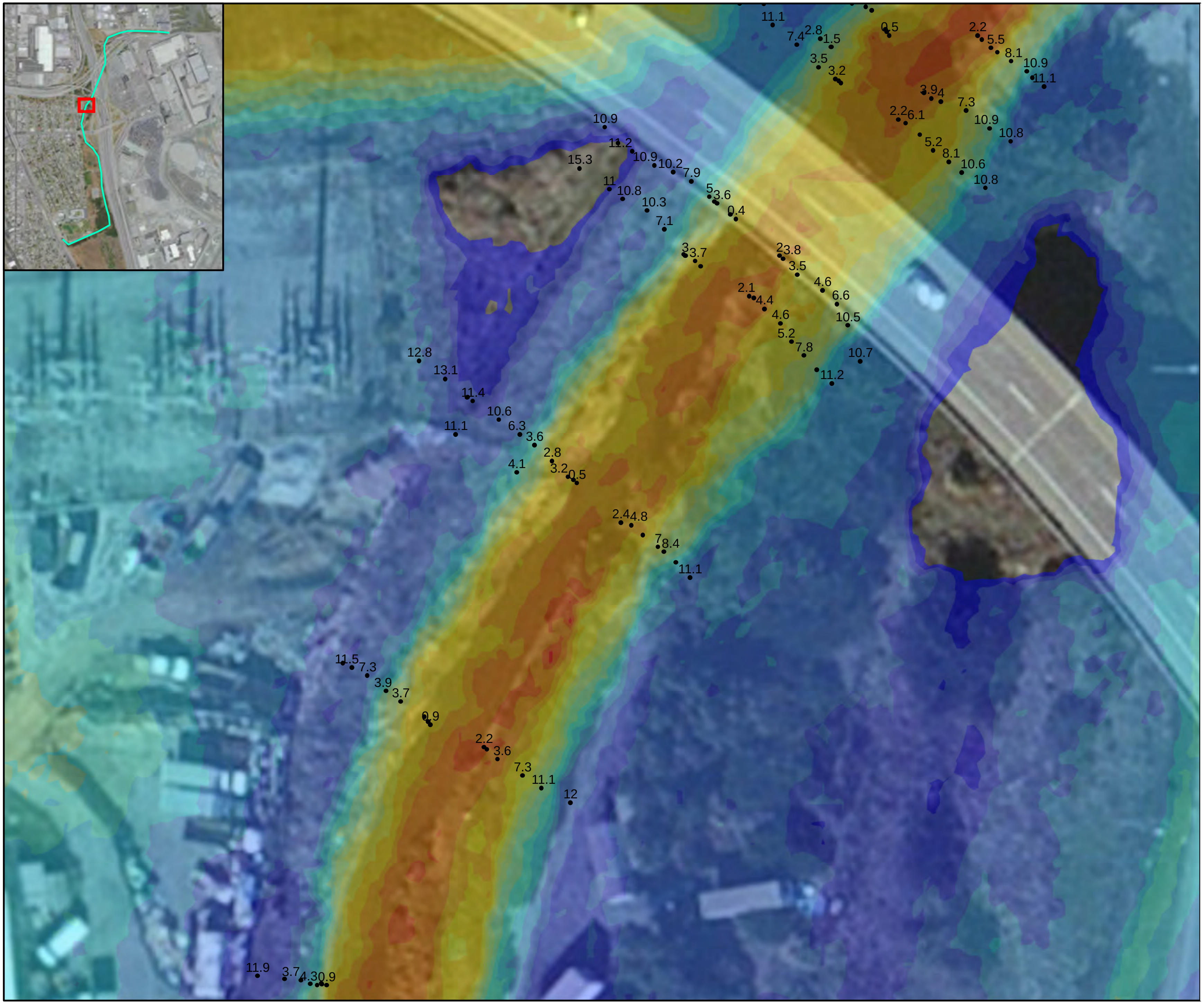

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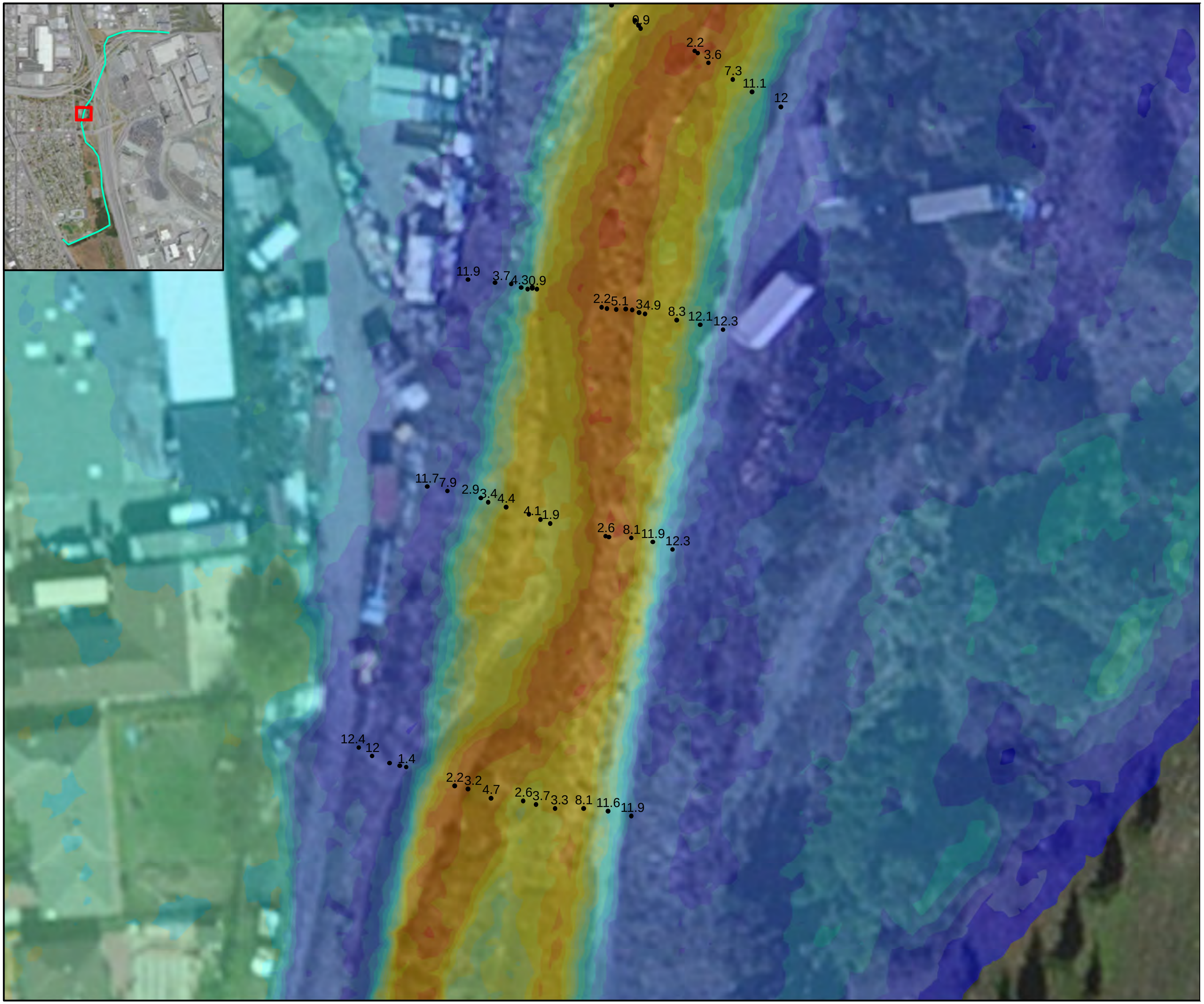

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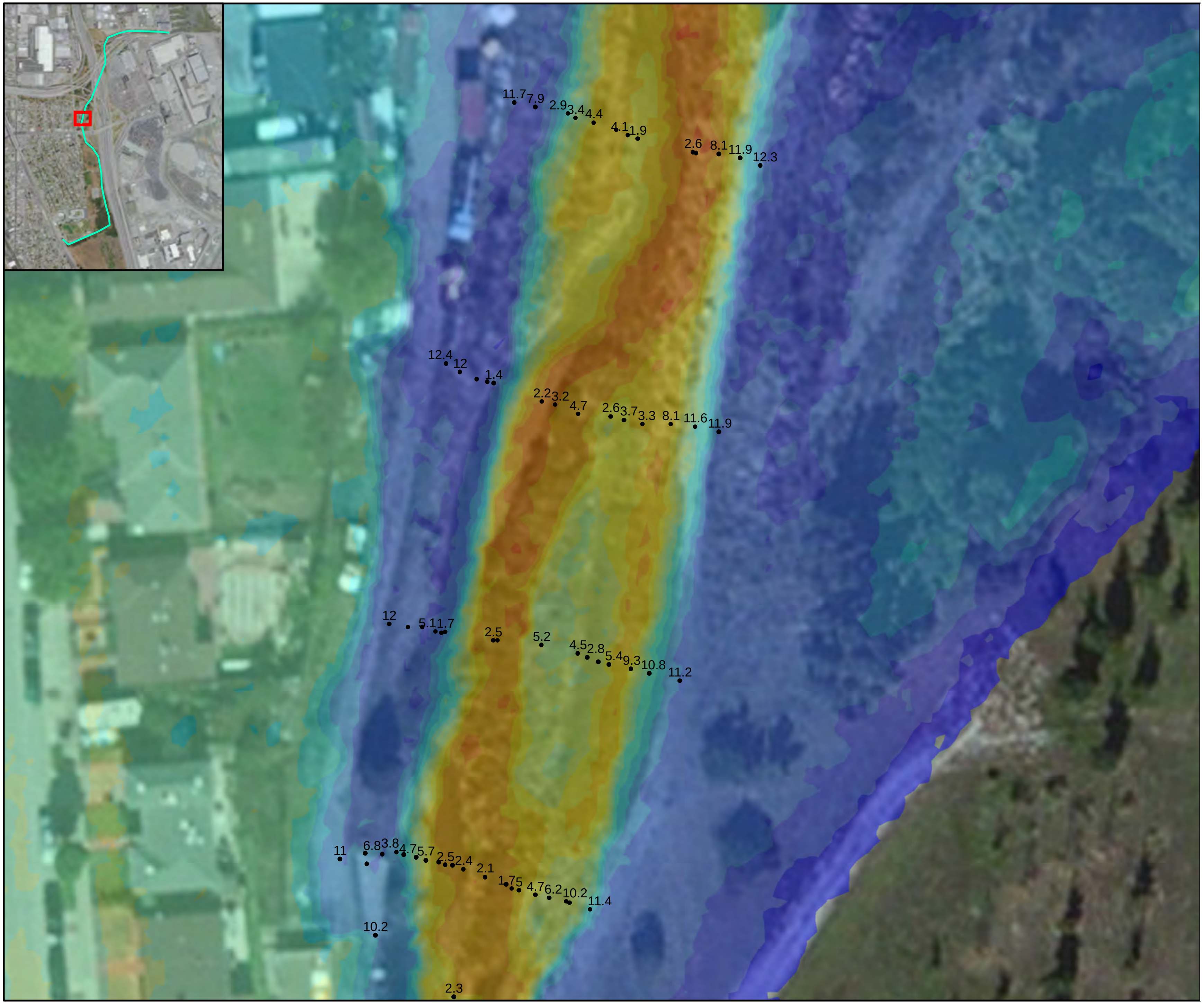

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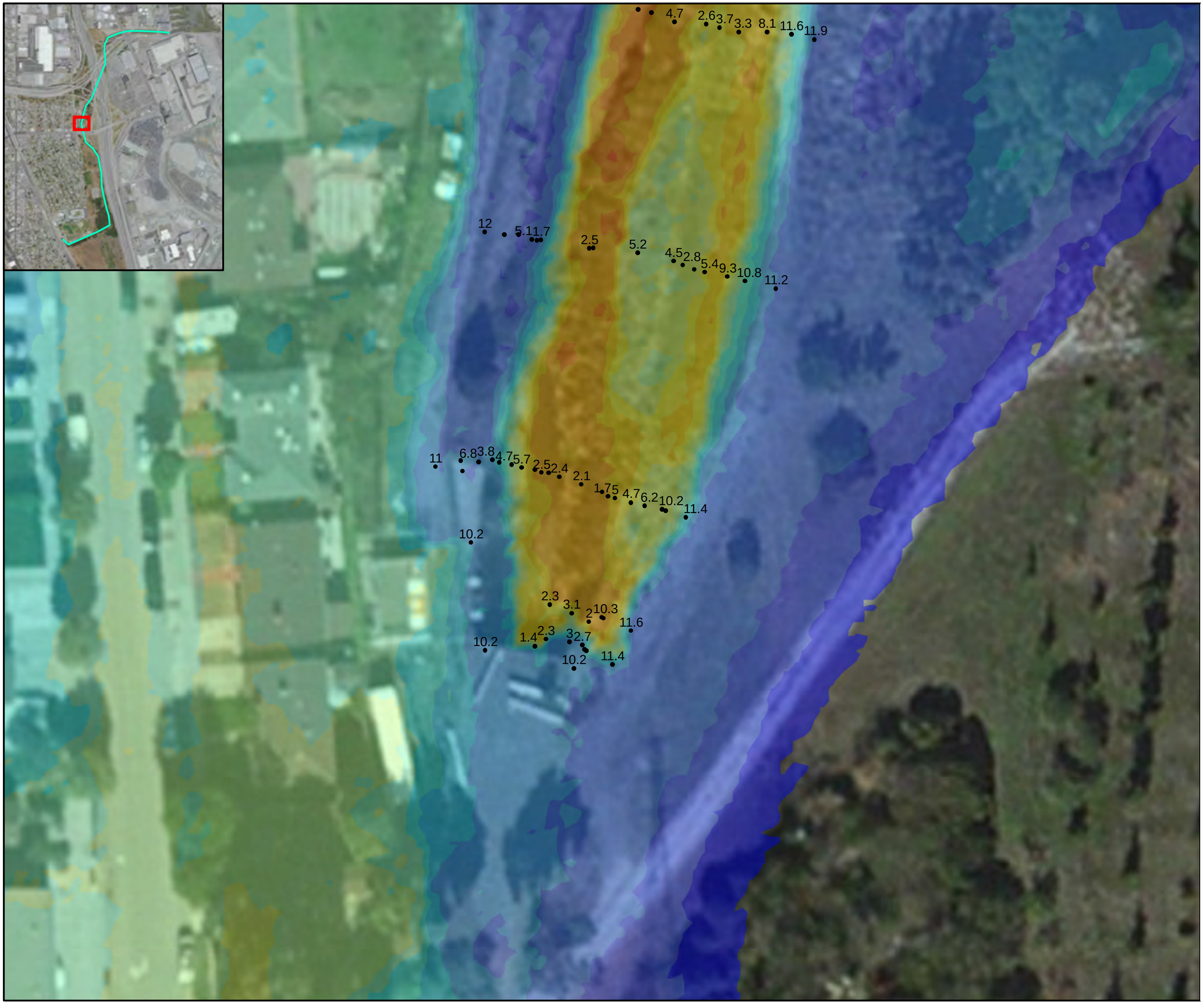

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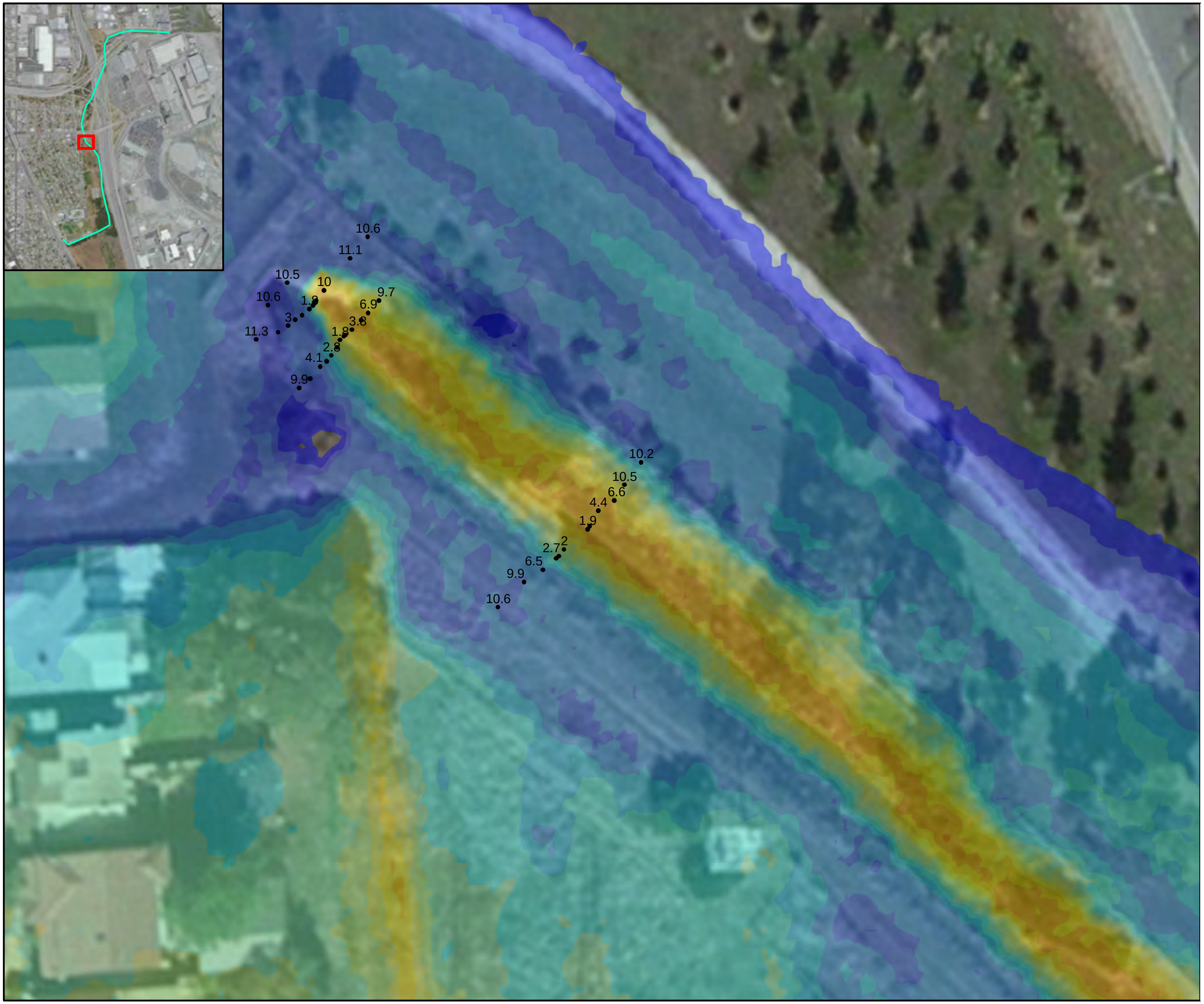

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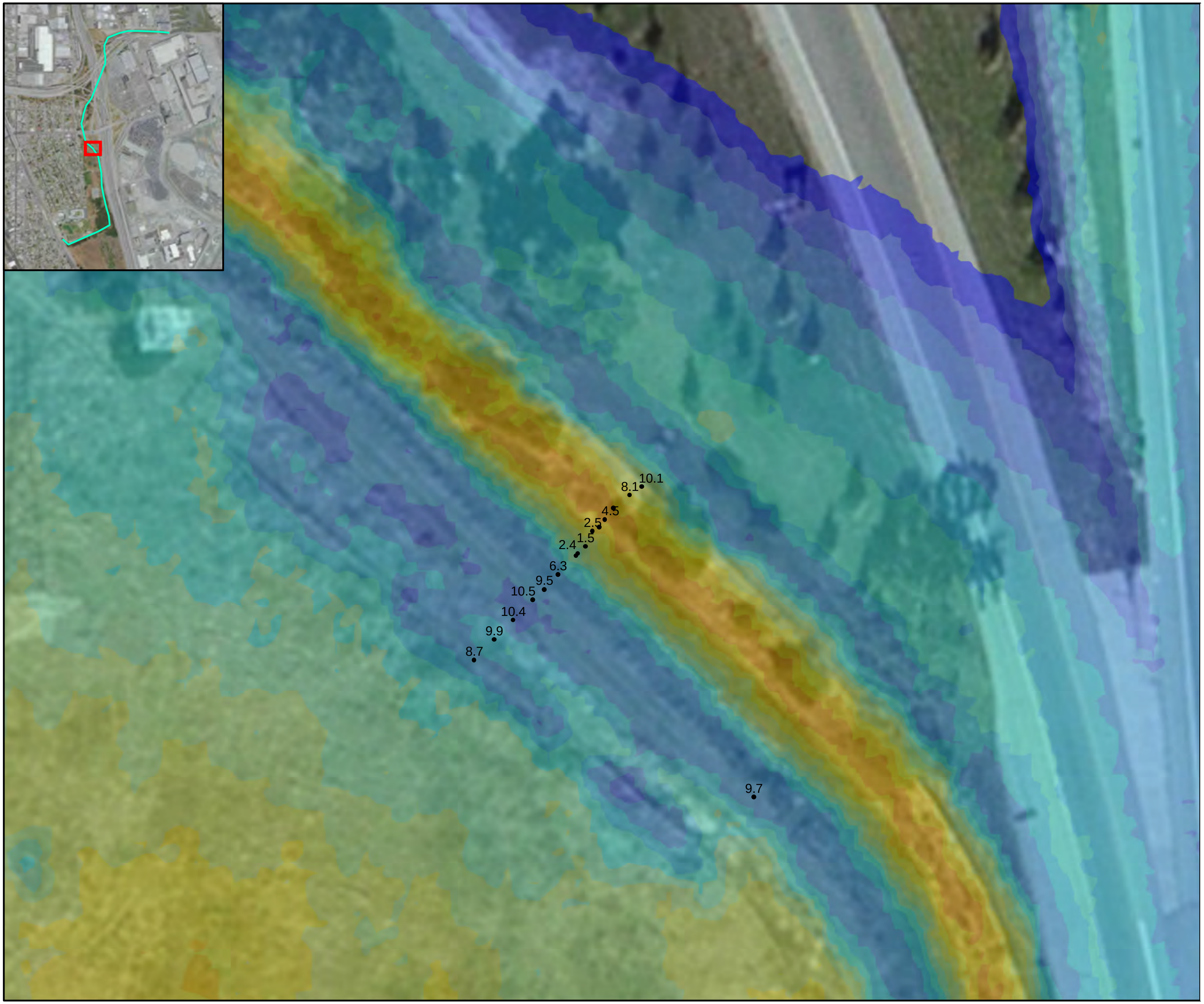

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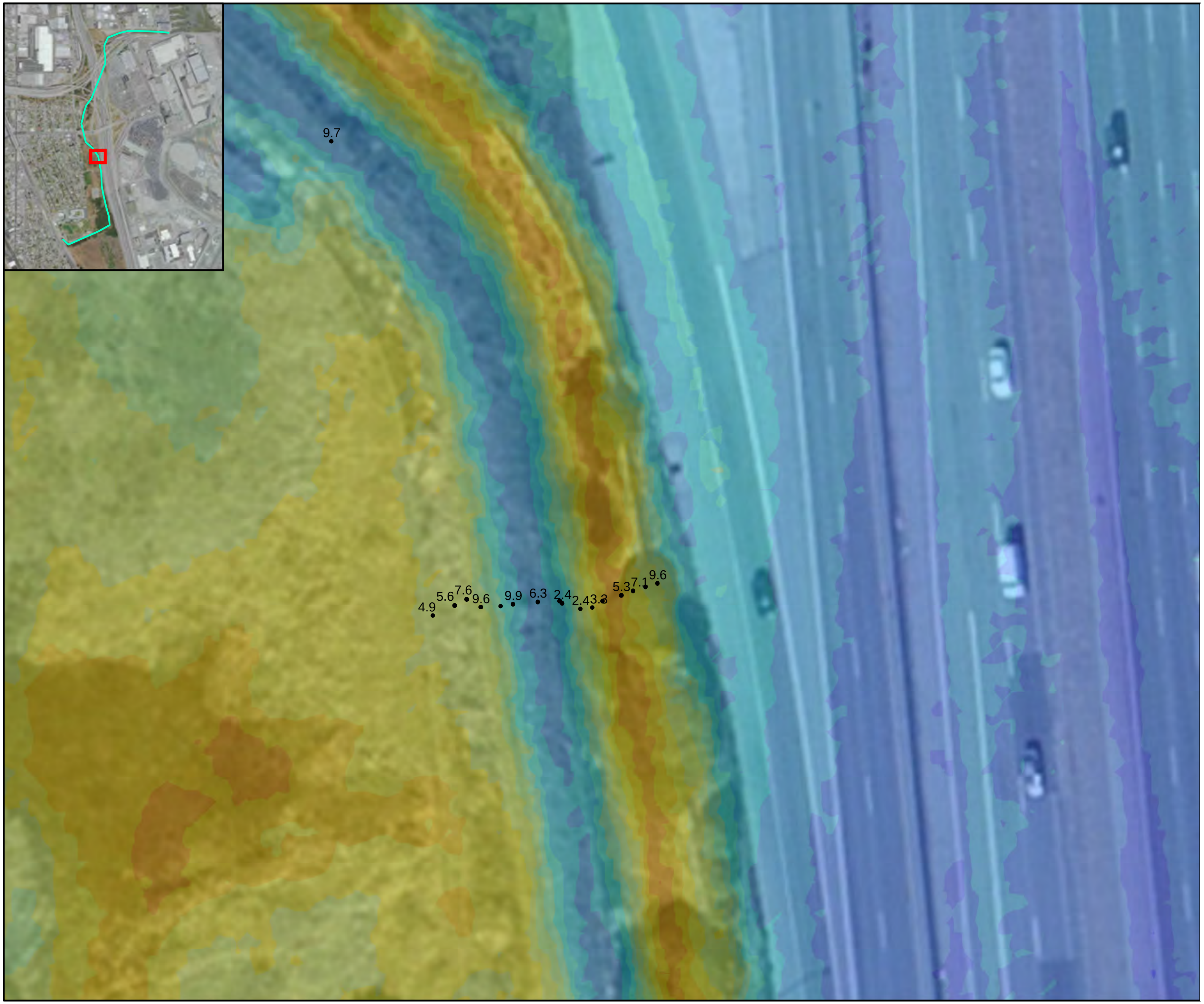

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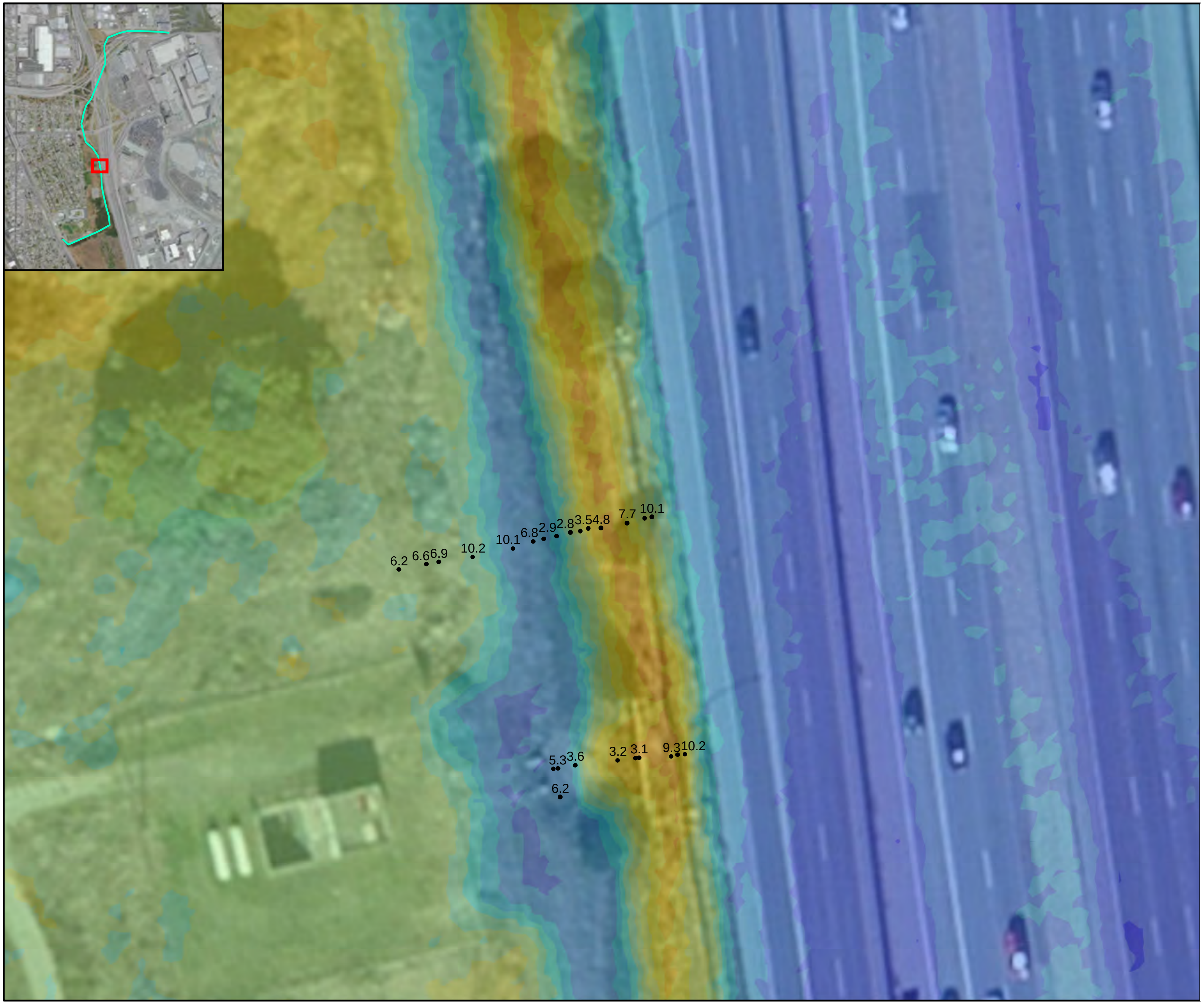

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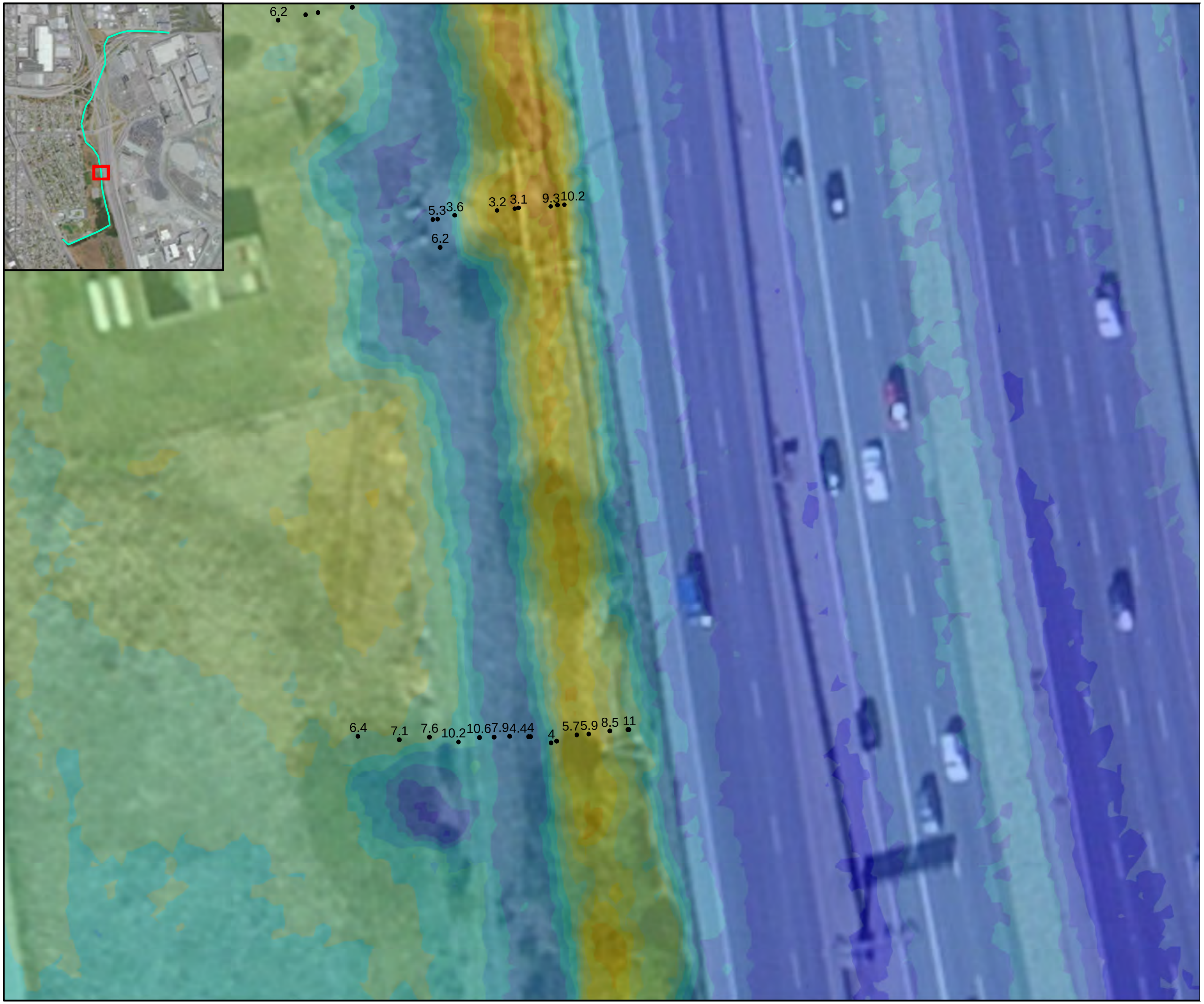

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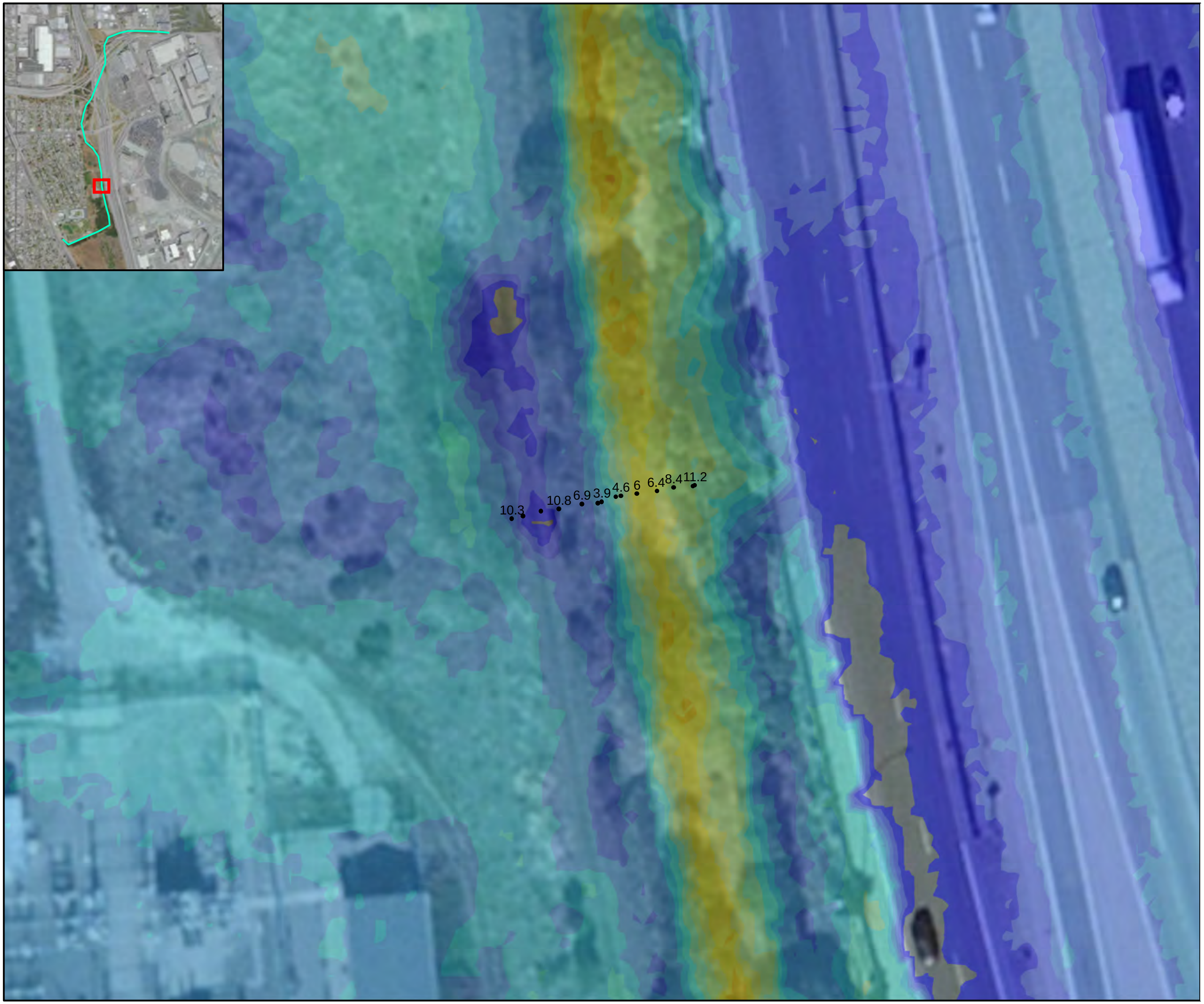

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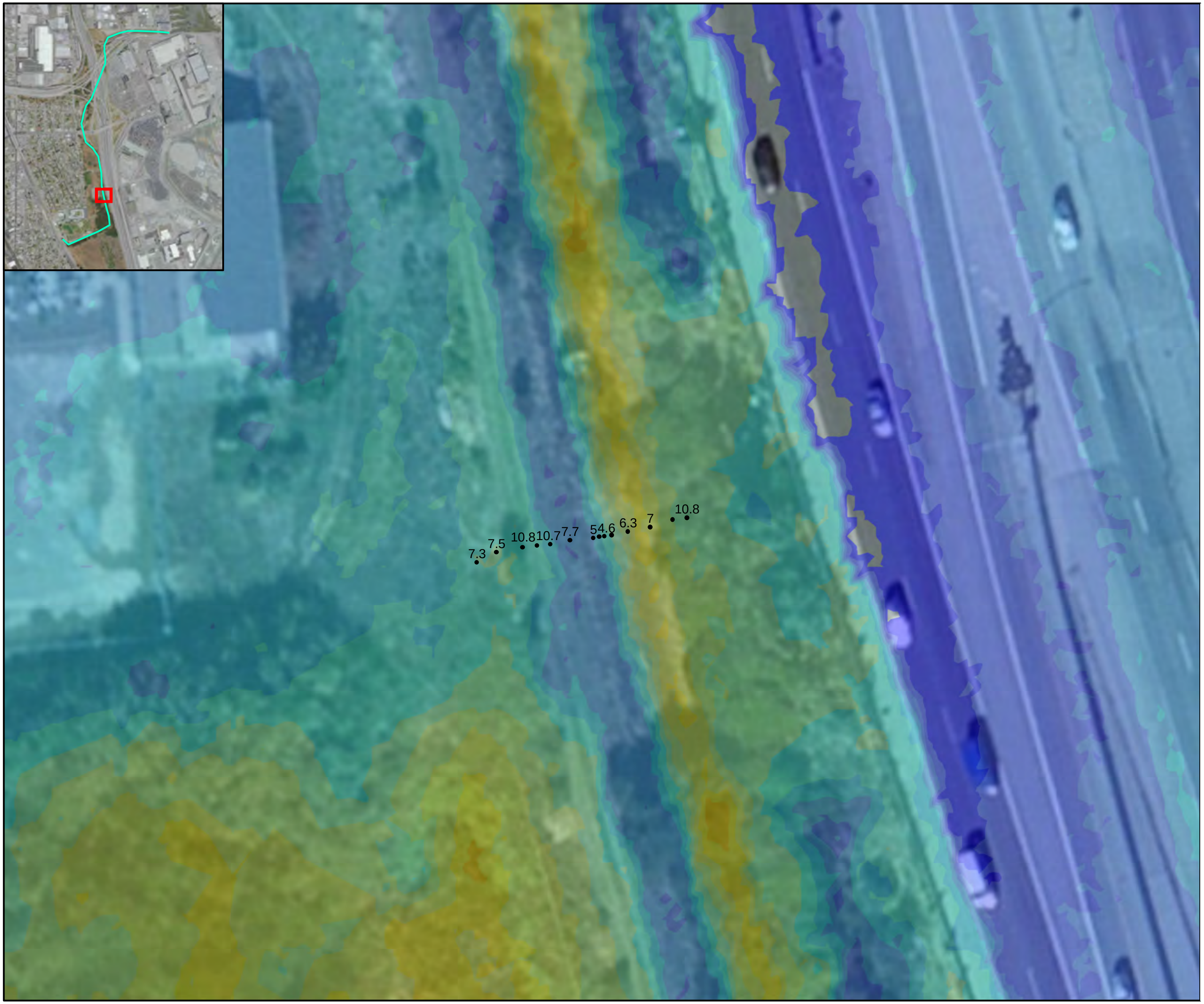

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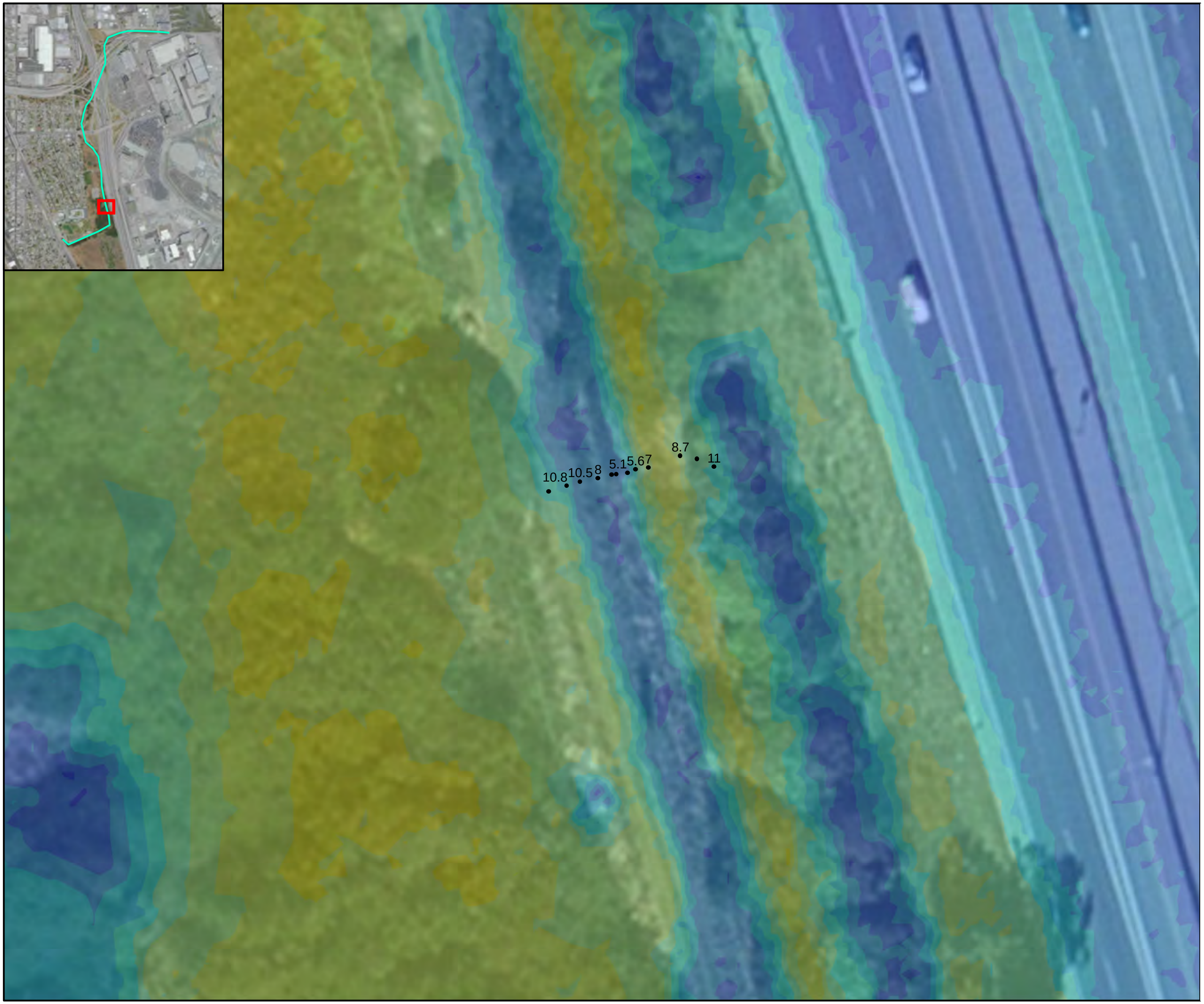

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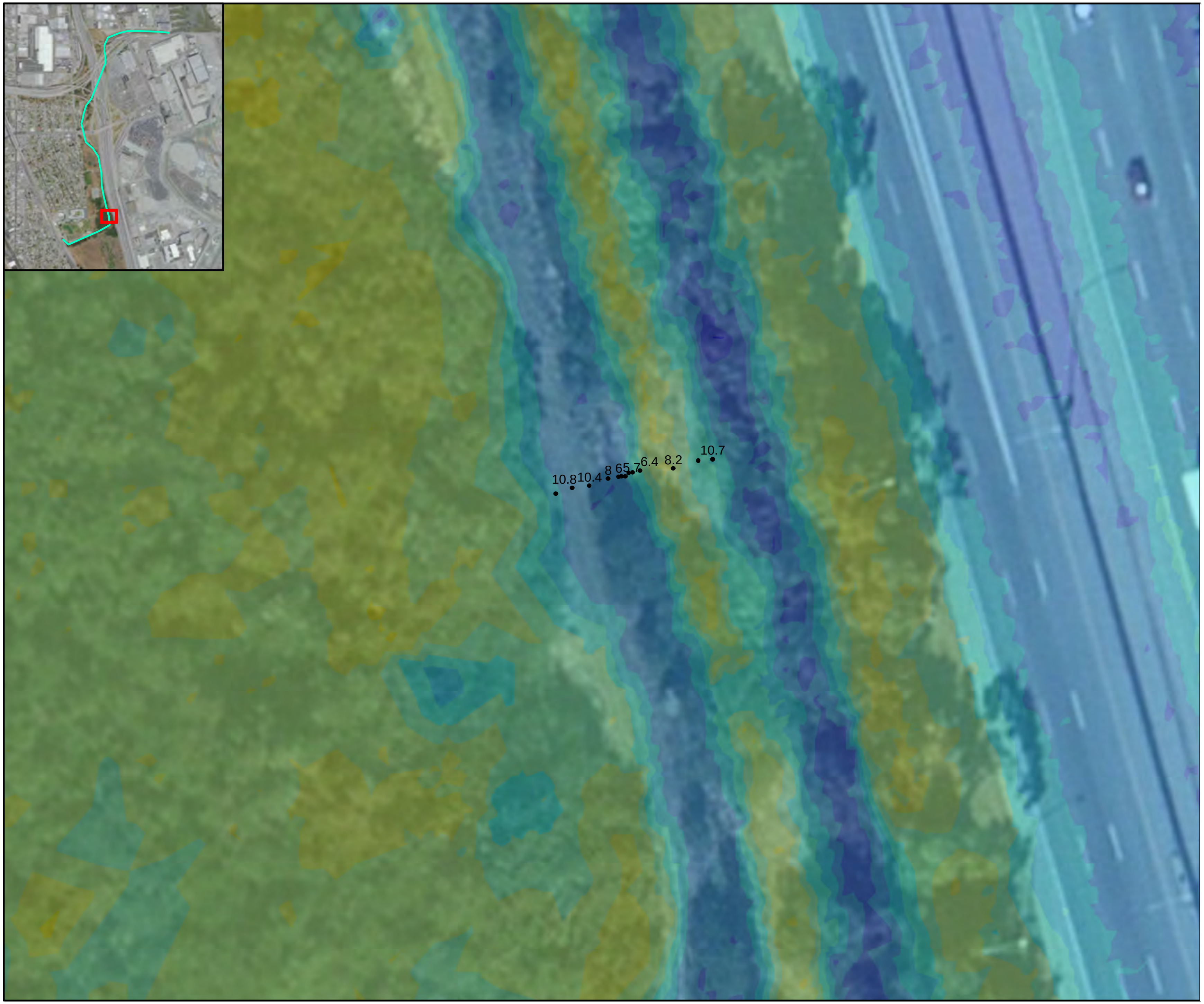

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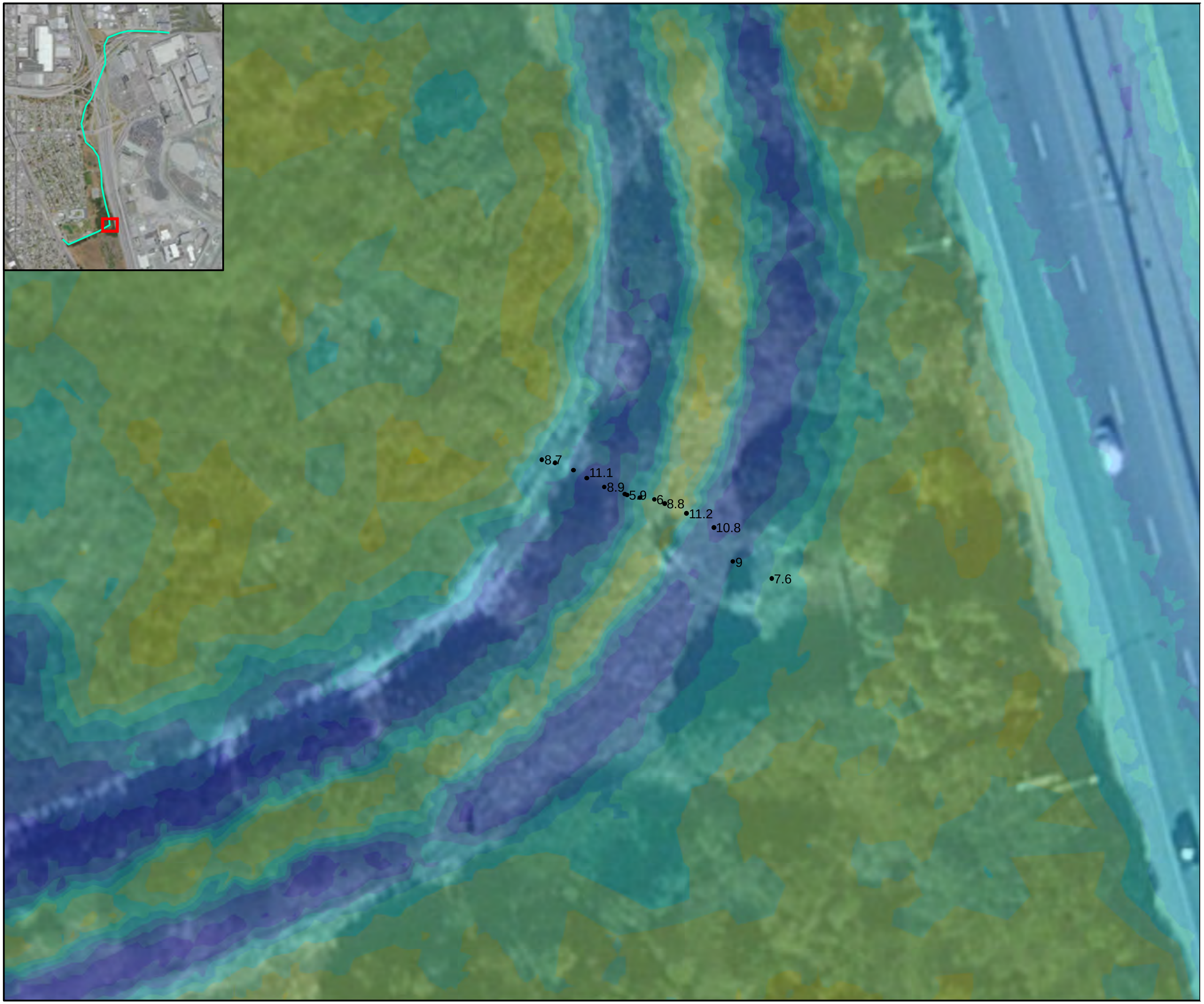

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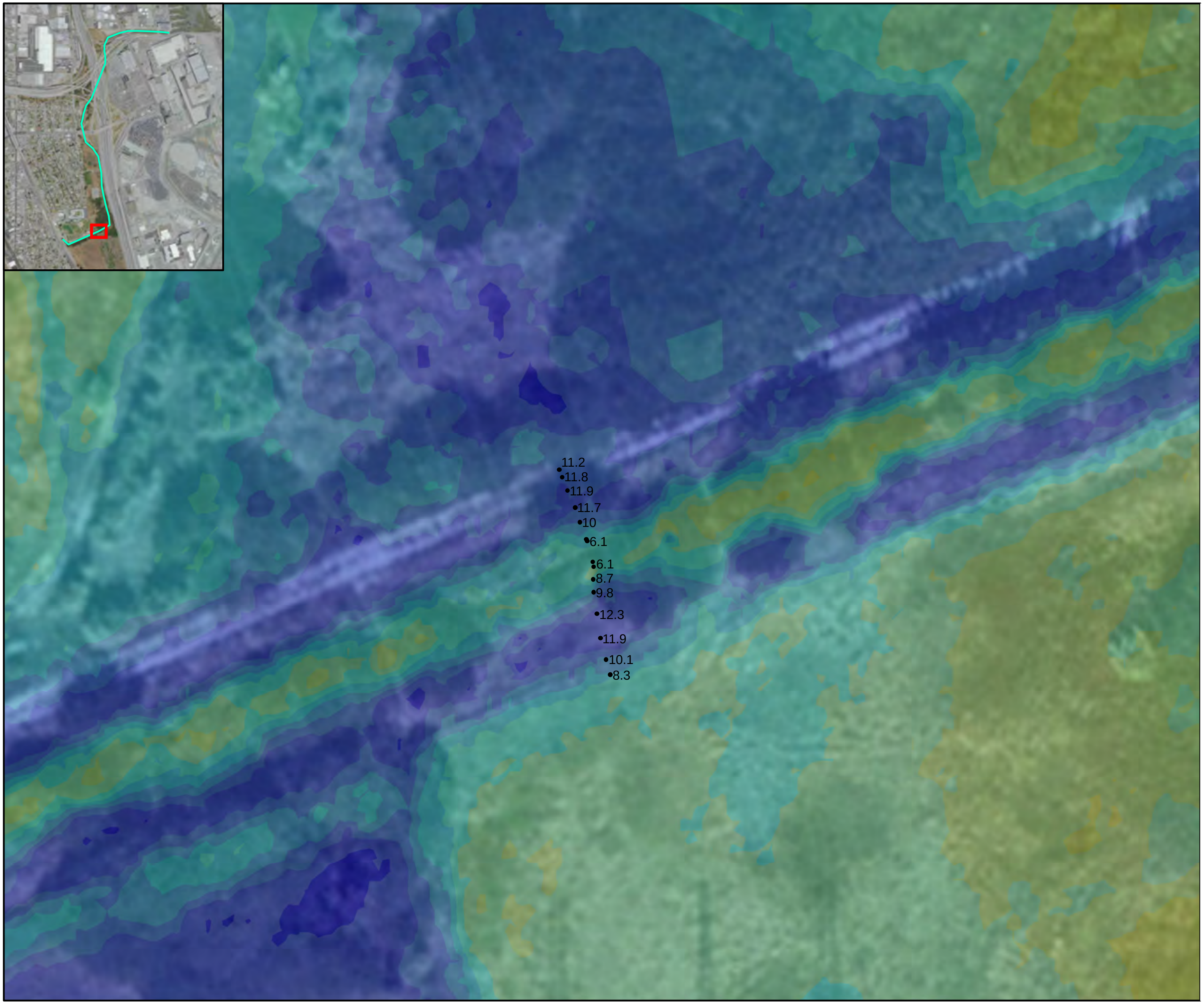

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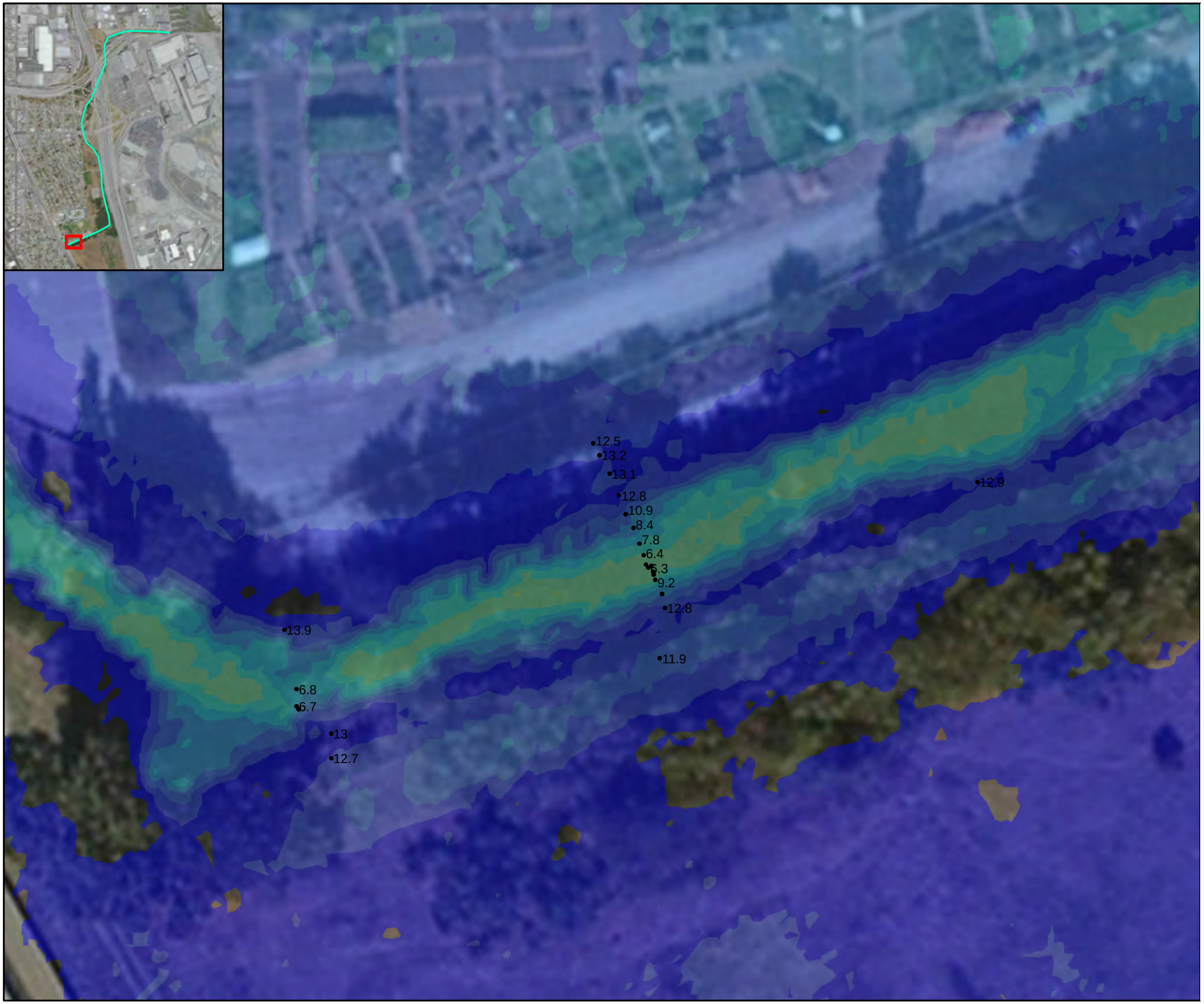

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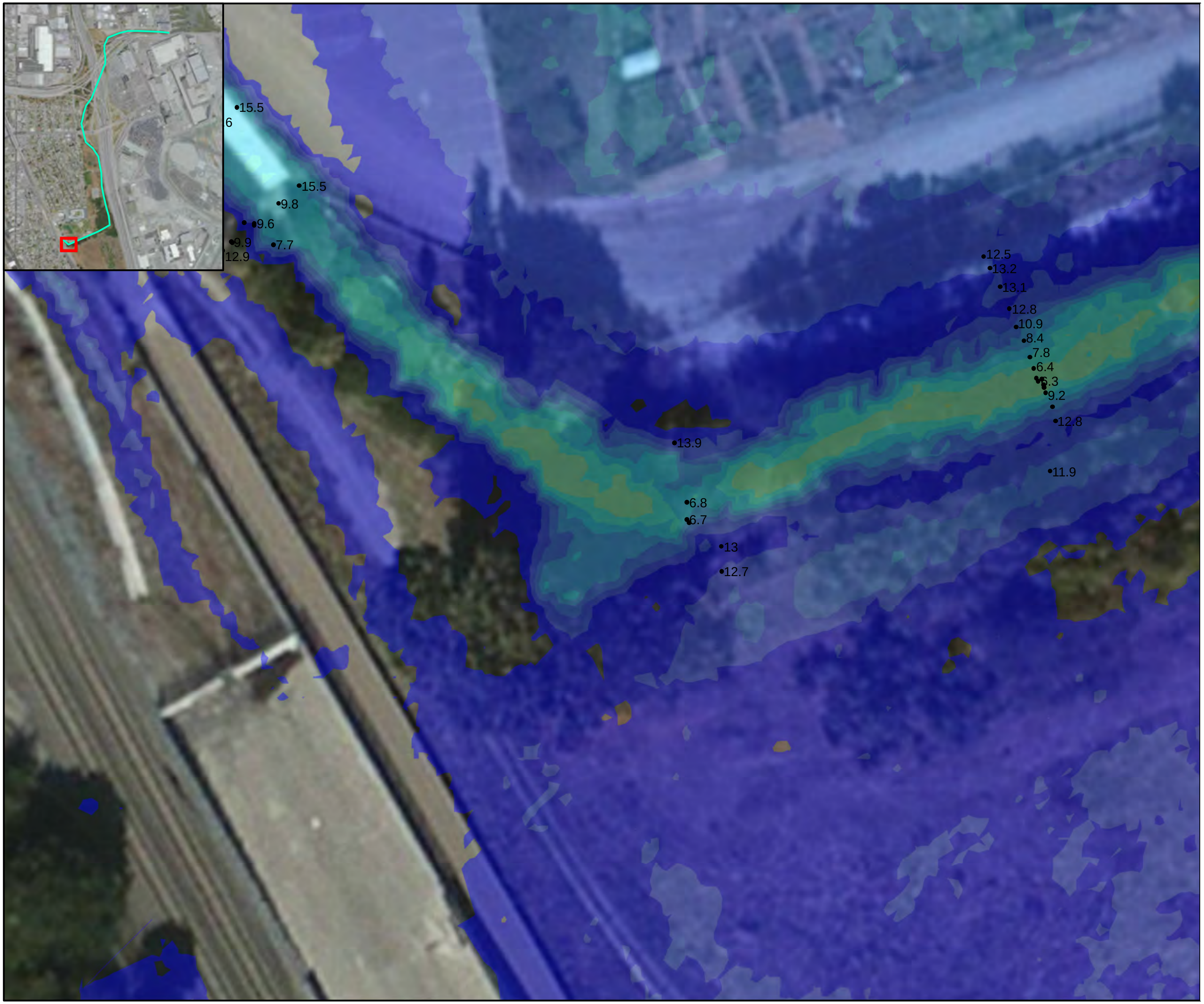

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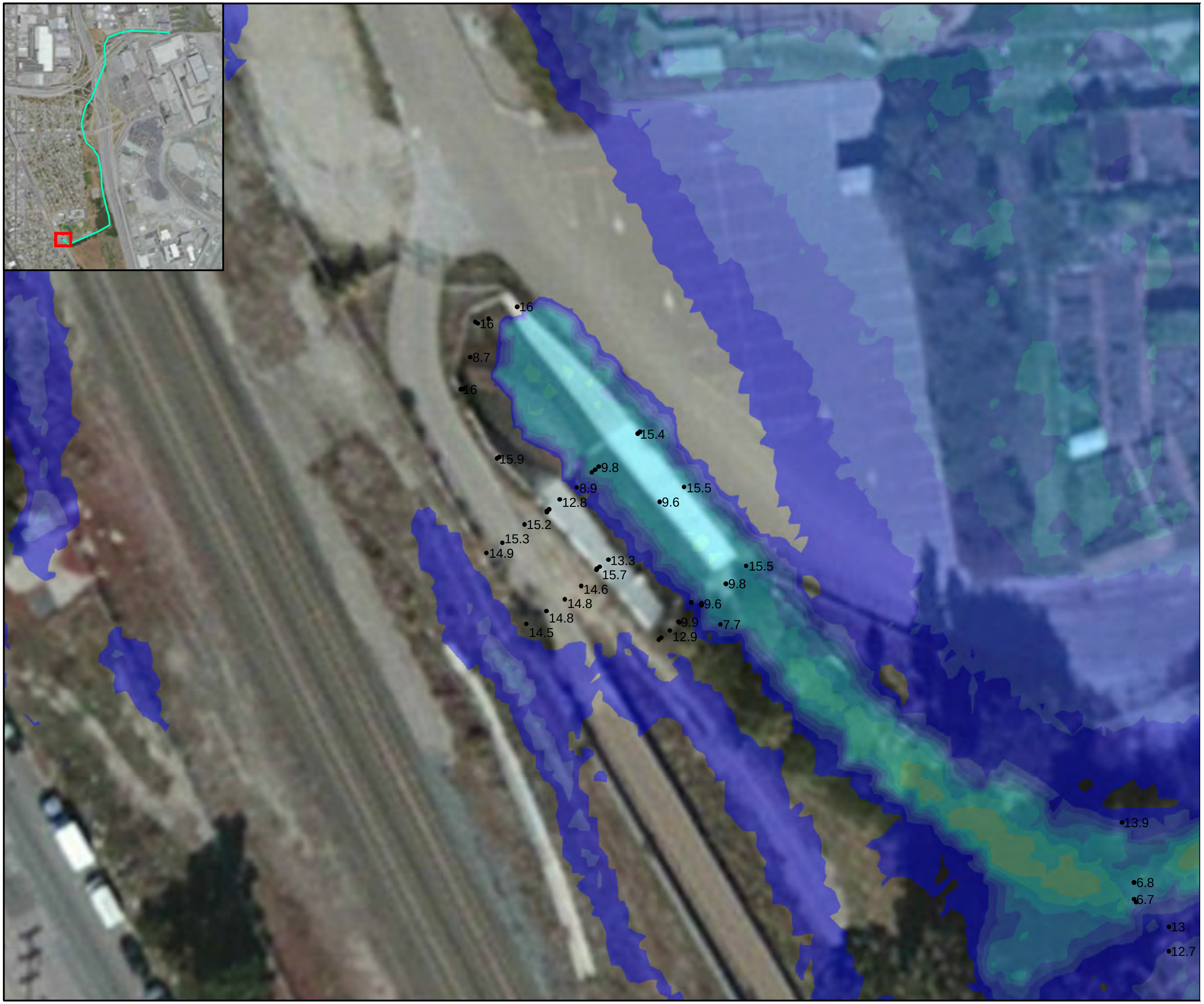

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APPENDIX B

Criteria for Evaluating Coastal Flood-Protection Structures

(FEMA 1990)

Federal Emergency Management Agency

Washington, D.C. 20472

APR 23 1990

MEMORANDUM FOR: FEMA REGIONAL DIRECTORS

FROM: Harold T. Duryee, Administrator
Federal Insurance Administration

SUBJECT: Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program (NFIP) Purposes

In order to better guide our staff, study contractors, and technical evaluation contractors, in the performance of flood insurance studies and in the review of flood map revision requests based on coastal structures, the Federal Insurance Administration has developed the attached proposed criteria statement. The proposed criteria would establish the conditions, procedures, and standards under which coastal flood protection structures would be credited on NFIP Flood Insurance Rate Maps as providing protection from the base flood.

It is our intention to issue these criteria as rulemaking during FY 1991. Any coments you have should be forwarded to the Office of Risk Assessment by May 25, 1990.

Also, attached is a copy of the Corps' Technical Report CERC-89-15, "Criteria for Evaluating Coastal Flood-Protection Structures" for your reference. CERC-89-15 was used as the basis for this proposed interim procedure.

Attachments

Criteria for Evaluating Coastal Flood Protection Structures

Background

Many property owners and communities along the U.S. coast are resorting to the construction of coastal flood control structures to protect existing or new development from potential damage associated with hurricanes and other major coastal storm events. Flooding and erosion caused by natural processes, sea level rise, and/or man-made influences are factors contributing to the decision to construct structures such as seawalls, revetments, bulkheads, and coastal levees/dikes. Although there is continued debate on the overall impact of these coastal structures, their construction and use requires that FEMA evaluate their effectiveness for reducing flood risk and their viability as an alternative to the non-structural flood loss reduction approaches required for community participation in the National Flood Insurance Program (NFIP).

The areas protected by coastal flood protection structures are frequently designated as Coastal High Hazard Areas (V zones) on the Flood Insurance Rate Maps (FIRMs) published by FEMA. FEMA is often requested to revise FIRMs to reflect the protection provided by a coastal structure against the base (100-year) flood. Because of the different types of coastal structures, materials, and construction methods, FEMA must perform a detailed review of these requests to assure that the structure is adequately designed and constructed to provide the stated level of protection, and to withstand the 100-year flooding event.

Part 65 of the NFIP regulations requires that any requester of a FIRM revision based on flood protection structures provide an analysis of the revised flood hazards, demonstrate and certify that the structure is designed and constructed for 100-year flooding conditions, and provide assurance that the structure will be maintained. Revision requests based on coastal structures are currently reviewed on a case-by-case basis using these regulations. A wide variation has been found in the quality of data submitted. Some possible reasons for this variation include the requester's inexperience or unfamiliarity with the different types of structures, the available design guidance, and/or the base (100-year) flood considered by the NFIP. In order to improve the quality of information submitted, and the ability of FEMA to review revision requests based on coastal structures, FEMA has decided to establish minimum design criteria that must be addressed in the request.

FEMA commissioned the U.S. Army Corps of Engineers, Waterways Experiment Station (WES), Coastal Engineering Research Center to identify or develope criteria for evaluating the effectiveness of all types of coastal flood protection structures in preventing or reducing damages and flooding from the 100-year event. This study identified and defined the different coastal structures that provide protection against flooding to property landward of the structure, and documented successful and unsuccessful cases for each structure type. The minimum criteria, considerations, and/or conditions applicable to the 100-year flooding event that are necessary for an evaluation of a coastal structure were also identified. The WES study recommended a procedure using these criteria to evaluate the adequacy, of a coastal flood protection structure to survive the 100-year flooding event, and to provide protection against flooding, wave runup and overtopping, wave forces, and erosion.

The WES Technical Report CERC-89-15 "Criteria for Evaluating Coastal Flood Protection Structures" was used as the basis for these critera. These criteria will also be used to resolve appeal challenges and in the conduct of flood insurance studies, when sufficient design and construction data are available.

Mapping of areas protected by coastal flood protection structures.

(a) General. For purposes of the NFIP, FEMA will only recognize in its flood hazard and risk mapping effort those coastal flood protection structures that meet, and continue to meet, minimum design and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria established by 44 CFR Part 60.3. Accordingly, this procedure describes the types of information FEMA needs to recognize, on NFIP maps, that a coastal flood protection structure provides protection from the base flood. This information must be supplied to FEMA by the community or other party seeking recognition of such a coastal flood protection structure at the time a flood risk study or restudy is conducted, when a map revision under the provision of Part 65 of this subchapter is sought based on a coastal flood protection structure, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure will perform in a flood event.

(b) Design Criteria. For coastal flood protection structures to be recognized by FEMA, sufficient evidence must be provided that adequate design, construction, and maintenance have been undertaken to provide reasonable assurance of durable protection from the base flood. The following requirements must be met:

(1) Design Parameters. A coastal flood protection structure must be designed using physical parameters that fully represent the base (100-year) flooding event, including the following:

(i) Design water levels evaluated should range from the mean low water level at the site to the 100-year stillwater surge elevation. The full range of elevations must be examined to determine the critical water level since the most severe conditions may not occur at either extreme.

(ii) Wave heights and periods must be calculated for each water level analyzed. At a minimum, significant wave height and periods should be used for "flexible" structures such as revetments, with larger wave height, up to the one-percent wave height (1.67 times the significant wave height), used for more rigid structures such as seawalls and bulkheads. The U.S. Army Corps of Engineers (COE) Shore Protection Manual (1984 or later edition), provides guidance and procedures for determining appropriate wave heights and periods.

(iii) Breaking wave forces under structure-perpendicular loading must be considered in the design unless it can be demonstrated that the structure will not be subject to breaking waves. The very high, short duration "shock" pressures must be used for low mass structures such as bulkheads, while only the secondary "non-shock" pressures need to be used for massive structures such as gravity seawalls. Analyses of the breaking wave forces using methods such as those identified in the COE report "Criteria for Evaluating Coastal Flood Protection Structures," (WES TR CERC-89-15) must be submitted.

(2) Minimum Freeboard. The minimum freeboard for coastal flood protection structures to be recognized on FEMA flood maps for protection against the storm surge component of the base flood shall be two feet above the 100-year stillwater surge elevation.

(3) Toe Protection. The loss of material and profile lowering seaward of the structure must be included in the design either through the incorporation of adequate toe protection or an evaluation of structural stability with potential scour equal to the maximum wave height on the structure. Engineering analyses such as those recommended in the COE's "Geotechnical Engineering in the Coastal Zone" (WES IR CERC-87-1), or "Design of Coastal Revetments, Seawalls, and Bulkheads" (COE EM 1110-2-1614) must be submitted for the toe protection, or an analysis of scour potential such as found in "Criteria for Evaluating Coastal Flood Protection Structures" (WES TR CERC-89-15) must be submitted.

(4) Backfill Protection. Engineering analyses of wave runup, overtopping, and transmission must be performed using methods provided in the COE report "Criteria for Evaluating Coastal Protection Structures" (WES TR CERC-89-15). Where the structure height is not sufficient to prevent overtopping and/or wave transmission, protection of the backfill must be included in the design. This should address prevention of loss of backfill material by rundown over the structure, by drainage landward, under, and laterally around the ends of the structure; as well as through joints, seams, or drainage openings in the structure.

(5) Structural Stability, Minimum Water Level. Analyses of the ability of the structures to resist the maximum loads associated with the minimum seaward water level, no wave action, saturated soil conditions behind the structure, and maximum toe scour must be submitted.

(i) For coastal dikes and revetments, a geotechnical analyses of potential failure in a landward direction by rotational gravity slip must be submitted.

(ii) For gravity and pile-support seawalls, engineering analyses of seaward sliding, of seaward overturning, and of foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.

(iii) For anchored bulkheads, engineering analyses of shear failure, moment failure, and the adequacy of the tiebacks and deadmen to resist the loadings must be submitted.

(6) Structural Stability - Critical Water Level. Analyses of the ability of the structure to resist the maximum loads associated with the critical water level, which may be any water level from the mean low water level to the 100-year stillwater elevation, including hydrostatic and hydrodynamic (wave) loads, saturated soil conditions behind the structure and maximum toe scour, must be submitted.

(i) For coastal dikes and revetments, geotechnical analyses of potential failure in a seaward direction by rotational gravity slip and of foundation failure due to inadequate bearing strength must be submitted.

(ii) For revetments, engineering analyses of the rock, riprap, or armor blocks' stability under wave action; uplift forces on the rock, riprap, or armor blocks; toe stability, and adequacy of the graded rock and geotechnical filters must be submitted.

(iii) For gravity and pile-supported seawalls, engineering analyses of landward sliding, of landward overturning, and of foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.

(iv) For anchored bulkheads, engineering analyses of shear and moment failure using "shock" pressures must be submitted.

(7) Material Adequacy. Documentation and/or analyses must be submitted that demonstrate that the materials used for the construction of the structure are adequate and suitable including life expectancy considerations, for the conditions that exist at the site.

(8) Ice and Impact Alignment. Where appropriate, analyses of ice and impact forces must be submitted.

(9) Structure Plan Alignment. A shore protection project should present a continuous structure with redundant return walls at frequent intervals to isolate locations of failure. Isolated structures or structures with a staggered alignment must submit analyses of the additional forces from concentrated, diffracted, and/or reflected wave energy on the different sections and ends.

(10) Other Design Criteria. FEMA will require that flood protection structures, regardless of type described above, be evaluated on the basis of how they may react structurally to applied forces. Therefore, analyses normally required of one structure type may also be required by another type which would react in a similar manner to applied forces. In unique situations, FEMA may require that other design criteria and analyses be submitted to show that the structure provides adequate protection. In such situations, sound engineering practice will be the standard on which FEMA will base its determinations. FEMA will provide the rationale for requiring any additional information.

(c) Adverse Impact Evaluation. All requests for flood map revisions based upon new or enlarged coastal flood control structures shall include an analysis of potential adverse impacts of the structure on flooding and erosion within, and adjacent, to the protected area.

(d) Community and/or State Review. For coastal flood protection structures to be recognized, evidence must be submitted to show that the design, maintenance, and impacts of the structures have been reviewed and approved by the affected communities and by any Federal, state or local agencies that have jurisdiction over flood control and coastal construction activities.

(e) Maintenance Plans and Criteria. For a coastal flood protection structures to be recognized as providing protection from the base flood, the structure must be maintained in accordance with an official adopted maintenance plan, and a copy of this plan must be provided to FEMA by the owner of the structure when recognition is being sought or when the plan for a previously recognized structure is revised in any manner. All maintenance activities must be under the jurisdiction of a Federal or state agency, an agency created by Federal or state law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability and overall integrity of the structure and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

(f) Certification Requirements. Data and analyses submitted to support that a given coastal flood protection structure complies with the structural design requirements set forth in paragraphs (b)(1) through (10) above must be certified by a registered professional engineer. Also, certified as-built plans of the structure must be submitted. Certifications are subject to the definition given at § 65.2 of 44 CFR Part 65. In lieu of these certification requirements, a Federal agency with responsibility for design of coastal flood protection structures may certify that the structure has been adequately designed and constructed to provide protection against the base flood.

APPENDIX C

FEMA MT-2 – Application Forms and Instructions for Conditional Letters of Map Revisions, and Letters of Map Revision

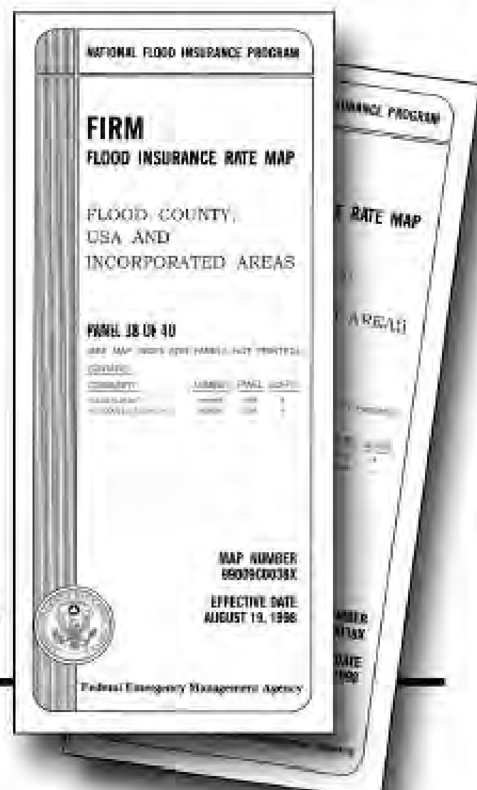
(FEMA)

MT-2

FEDERAL INSURANCE AND MITIGATION ADMINISTRATION
HAZARD MAPPING DIVISION

REVISIONS TO NATIONAL FLOOD INSURANCE PROGRAM MAPS

APPLICATION FORMS AND INSTRUCTIONS FOR CONDITIONAL LETTERS OF MAP REVISION, AND LETTERS OF MAP REVISION



MT-2
FEMA FORM 81-89 SERIES
SEPT 02

FEMA

Federal Emergency Management Agency

INSTRUCTIONS FOR COMPLETING THE COASTAL STRUCTURES FORM (FORM 5)

The Coastal Structures Form is to be completed when a revision to coastal flood hazard elevations and/or areas is requested based on coastal structures being credited as providing protection from the base flood. The purpose of the Coastal Structures Form is to ensure that the structure is designed and constructed to provide protection from the base flood without failing or causing an increase in flood hazards to adjacent areas. Refer to the *Consolidated Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix D: Guidance for Coastal Flooding Analyses and Mapping* which can be obtained from the Federal Emergency Management Agency's (FEMA's) Internet site at http://www.fema.gov/mit/tsd/dl_cgs.htm, for the criteria for evaluating flood protection structures.

If the coastal structure is a levee/floodwall, complete the Levee/Floodwall System section of the Riverine Structure Form (Form 3), in addition to this form. When the Coastal Structures Form is submitted, the Coastal Analysis Form (Form 4) should also be submitted.

Section A: Background

Information about the type of structure, the location, the material being used, and the age of the structure must be provided. Certified "as built" plans must also be provided. If these plans are not available, an explanation must be given with sketches of the general structure dimensions as described. If the structure design has been certified by a Federal agency to provide flood protection and withstand forces from the 1% annual chance (base) flood, the dates of the project completion and certification of the structure should be provided, and the remainder of the form does not need to be completed.

Section B: Design Criteria

Documentation must be provided that ensures a coastal structure is designed and constructed to withstand the wind and wave forces associated with the base flood. The minimum freeboard of the structure must be in compliance with National Flood Insurance Program (NFIP) Regulation 44 CFR Ch. 1, Section 65.10. Additional concerns include the impact to areas directly landward of the structure that may be subjected to overtopping and erosion along with possible failure of the structure due to undermining from the backside and the possible increase in erosion to unprotected properties at the ends of the structure. The evaluation of protection provided by sand dunes must follow the criteria outlined in NFIP Regulation 44 CFR Ch. 1, Section 65.11.

Section C: Adverse Impact Evaluation

If the structure is new, proposed, or modified, and will impact flooding and erosion for the areas adjacent to the structure, provide an explanation and documentation to support your conclusions.

Section D: Community and/or State Review

Provide documentation of Community and/or State review of the revision.

Section E: Certification

The licensed professional engineer and/or land surveyor should have a current license in the State where the affected communities are located. While the individual signing this form is not required to have obtained the supporting data or performed the analyses, he or she must have supervised and reviewed the work.

If the requester is a Federal agency who is responsible for the design and construction of flood control facilities, a letter stating that "the analyses submitted have been performed correctly and in accordance with sound engineering practices" may be submitted in lieu of certification by a registered professional engineer. Regarding the certification of completion of flood control facilities, a letter from the Federal agency certifying its completion and the flood frequency event to which the project protects may be submitted in lieu of this form.

FEDERAL EMERGENCY MANAGEMENT AGENCY

COASTAL STRUCTURES FORM

O.M.B. No. 3067-0148
Expires September 30, 2005

PAPERWORK REDUCTION ACT

Public reporting burden for this form is estimated to average 1 hour per response. The burden estimate includes the time for reviewing instructions, searching existing data sources, gathering and maintaining the needed data, and completing, reviewing, and submitting the form. You are not required to respond to this collection of information unless a valid OMB control number appears in the upper right corner of this form. Send comments regarding the accuracy of the burden estimate and any suggestions for reducing this burden to: Information Collections Management, Federal Emergency Management Agency, 500 C Street, SW, Washington DC 20472, Paperwork Reduction Project (3067-0148). Submission of the form is required to obtain or retain benefits under the National Flood Insurance Program. **Please do not send your completed survey to the above address.**

Flooding Source:
Note: Fill out one form for each flooding source studied

A. BACKGROUND

1. Name of structure (if applicable):
2. Structure location:
3. Type of structure (check one):

☐ Levee/Floodwall* ☐ Anchored Bulkhead ☐ Revetment ☐ Gravity Seawall

☐ Breakwater ☐ Pile supported seawall ☐ Other:

***Note:** If the coastal structure is a levee/floodwall, complete Section E of Form 3 (Riverine Structures Form). The remainder of this form does not need to be completed.

4. Material structure is composed of (check all that apply):

☐ Stone ☐ Earthen fill ☐ Concrete ☐ Steel

☐ Sand ☐ Other

5. The structure is (check one):

☐ New or proposed ☐ Existing ☐ Modification of existing structure

☐ Replacement structure of the same size and design as what was previously at the site

Describe in detail the existing structure and/or modifications being made to the structure and the purpose of the modifications:

If existing, please include date of construction:

6. Copies of certified "as-built" plans ☐ are ☐ are not attached. Attach all design analyses that apply.

If "as-built" plans are not available for submittal, please explain why and attach a sketch with general structure dimensions including: face slope, height, length, depth, and toe elevation referenced to the appropriate datum (e.g. NGVD 1929, NAVD 1988, etc.).

7. Has a Federal agency with responsibility for the design of coastal flood protection structures designed or certified that the structures have been adequately designed and constructed to provide protection against the 1%-annual-chance event?

☐ Yes ☐ No

If Yes, specify the name of the agency and dates of project completion and certification.

If Yes, then no other sections of this form need to be completed.

FEMA Form 81-89D, SEP 02 Coastal Structures Form MT-2 Form 5 Page 1 of 4

B. DESIGN CRITERIA

1. Design Parameters

   a. Were physical parameters representing the 1%-annual-chance event or greater used to design the coastal flood protection structure?

   ☐ Yes ☐ No

   b. The number of design water levels that were evaluated (number) range from the mean low water elevation of feet to the 1%-annual-chance stillwater surge elevation of feet. The critical water level is feet. The datum that these elevations are referenced to is (e.g.: NGVD 1929, NAVD 1988, etc.).

   Attach an explanation specifying which water levels and associated wave heights and periods were analyzed.

   c. Were breaking wave forces used to design the structure?

   ☐ Yes ☐ No If No, attach an explanation why they were not used for design.

2. Settlement

   a. What is the expected settlement rate at the site of the structure?

   Please attach a settlement analysis.

3. Freeboard

   a. Does the structure have 1 foot of freeboard above the height of the 1%-annual-chance wave-height elevation or maximum wave runup (whichever is greater)?

   ☐ Yes ☐ No

   b. Does the structure have freeboard of at least 2 feet above the 1% annual chance stillwater surge elevation?

   ☐ Yes ☐ No

4. Toe Protection

   Specify the type of toe protection:

   If no toe protection is provided, provide analysis of scour potential and attach an evaluation of structural stability performed with potential scour at the toe.

5. Backfill Protection

   Will the structure be overtopped during the 1%-annual-chance event? ☐ Yes ☐ No

   If the structure will be overtopped, attach an explanation of what measures are used to prevent the loss of backfill from rundown over the structure, drainage landward, under or laterally around the ends of the structure, or through seams and drainage openings in the structure.

6. Structural Stability - Minimum Water Level

   a. For coastal revetments, was a geotechnical analysis of potential failure in the landward direction by rotational gravity slip performed for maximum loads associated with minimum seaward water level, no wave action, saturated soil conditions behind the structure, and maximum toe scour?

   ☐ Yes ☐ No

   b. For gravity and pile-supported seawalls, were engineering analyses of landward sliding, landward overturning, and of foundation adequacy using maximum pressures developed in the sliding and overturning calculations performed?

   ☐ Yes ☐ No

   c. For anchored bulkheads, were engineering analyses performed for shear failure, moment failure, and adequacy of tiebacks and deadmen to resist loading under low-water conditions?

   ☐ Yes ☐ No

FEMA Form 81-89D, SEP 02 Coastal Structures Form MT-2 Form 5 Page 2 of 4

B. DESIGN CRITERIA (CONTINUED)

C. ADVERSE IMPACT EVALUATION

If the structure is new, proposed, or modified, will the structure impact flooding and erosion for areas adjacent to the structure?

☐ Yes ☐ No

If Yes, attach an explanation.

D. COMMUNITY AND/OR STATE REVIEW

Has the design, maintenance, and impact of the structure been reviewed and approved by the community, and any Federal, State, or local agencies having jurisdiction over flood control and coastal construction activities in the area the structure impacts?

☐ Yes ☐ No

If Yes, attach a list of agencies who have reviewed and approved the project.

If No, attach an explanation why review and approval by the appropriate community or agency has not been obtained.

E. CERTIFICATION

As a Professional Engineer, I certify that the above structures will withstand all hydraulic and wave forces associated with the 1% annual chance flood without significant structural degradation. All documents submitted in support of this request are correct to the best of my knowledge. I understand that any false statement may be punishable by fine or imprisonment under Title 18 of the United States Code, Section 1001.

Certifier's Name:

License No.: Exp. Date:

Company Name:

Telephone No.: Fax. No.:

Signature: ______________________________ Date:

Seal (optional)