Attachment D

SANTA RITA CREEK FLOOD STUDY

Prepared for:

Monterey County
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EXECUTIVE SUMMARY

A comprehensive Flood Study of Santa Rita Creek was undertaken to identify the underlying causes of frequent, damaging flooding in the Bolsa Knolls community and to develop a range of mitigation strategies to reduce future flood risk. Key findings and recommendations from the study are summarized below:

- **Chronic Flooding History:** Flooding along Santa Rita Creek has been a persistent issue for nearly a century, with documented impacts dating back to the 1930s and more recent events occurring in 2014, 2017, 2021, 2022, 2023, and 2025. Reports from local news outlets confirm that flooding has repeatedly affected streets, yards, and, at times, residential structures throughout Bolsa Knolls.

- **Hydrologic Analysis:** Watershed-scale hydrologic modeling was conducted to generate runoff hydrographs for recent storm events (February 13, 2025; December 27, 2022; and February 20, 2017) as well as for 10- and 100-year design storms. Resulting peak flow estimates along Santa Rita Creek at Russell Road were 210, 290, 360, 550, and 1,190 cubic feet per second (cfs), respectively.

- **Hydraulic Analysis:** Hydraulic modeling representing existing conditions indicates that Santa Rita Creek begins to overtop its banks at flow rates as low as approximately 80 cfs through the low-density developed area at the upstream end of Bolsa Knolls and closer to 100 cfs through the medium-density residential area further downstream. Contributing factors to this limited conveyance capacity include a relatively small channel cross-section, pockets of dense channel vegetation, minimal adjacent undeveloped floodplain, and constrictions at the Paul Avenue and Rogge Road crossings where sediment accumulation further reduces capacity. As the creek channel capacity is exceeded, flow is routed through the streets and residential properties of Bolsa Knolls, with the hydraulic model predicting that inundation extents would impact approximately 60 and 80 residential structures in the 10- and 100-year flood events respectively.

- **Recommended Improvements:** The study identifies several targeted improvements to reduce flood risk in Bolsa Knolls:

    1. Enlargement of the culverts at Paul Avenue and Rogge Road.
    2. Sediment removal at culvert crossings.
    3. Selective vegetation management within key channel segments (reliant on the acquisition of permits).
    4. Construction of upstream sediment retention and flow detention basins.

Four project alternatives have been developed, each combining different configurations of these elements, including varying sizes and locations of detention/sediment basins.

- **Flood Reduction Benefits:** Hydraulic modeling of the proposed improvements indicates that the overtopping threshold of the creek channel through the medium-density residential area can be increased from 100 cfs to approximately 190 cfs. Modeled benefits include a reduction in peak water surface elevations by up to 2 feet and removal of up to 20 parcels from the flood extents associated with events like the February 13, 2025, storm. Additionally, if significant upstream detention volume can be provided, flood elevations and inundation areas throughout the study reach could be further reduced for larger events up to and including the 100-year flood.

1 INTRODUCTION

Santa Rita Creek, locally known as Little Bear Creek, originates in the Gabilan Range at the northern edge of Monterey County and flows southwest through the community of Bolsa Knolls before continuing west towards the Salinas River. At the downstream end of Bolsa Knolls, the creek watershed area is 3.7 square miles in size and has been extensively developed for agricultural and residential use. These factors result in significant runoff rates from the watershed, while the creek's limited conveyance capacity through Bolsa Knolls, combined with development encroaching into the natural floodplain, has resulted in frequent, damaging flooding within the community. This study investigates the underlying causes of flooding in Bolsa Knolls and evaluates a range of mitigation strategies to reduce future flood risks.

1.1 Study Objectives and Approach

The primary objective of this study is to develop and evaluate a set of feasible project alternatives to reduce flood risk in Bolsa Knolls. The study provides technical analyses and cost-benefit insights to help Monterey County select an informed and effective course of action.

Achieving this goal first requires a clear understanding of the underlying causes of flooding in the area. To this end, a hydrologic model of the Santa Rita Creek watershed was developed to simulate runoff volumes and peak flow rates generated during storm events. The resulting flood flow hydrographs were then routed through a detailed hydraulic model of Santa Rita Creek and its floodplain to identify conveyance constraints and assess flood extents under a range of storm scenarios.

This modeling framework informed an opportunity and constraints analysis used to screen and refine a suite of potential flood mitigation strategies. Four project alternatives were ultimately developed; each composed of implementable actions designed to be cost-effective and responsive to site-specific conditions. These alternatives were tested within the hydraulic model to evaluate their potential flood reduction benefits across a range of storm events.

Beyond flood reduction performance, each alternative was evaluated based on additional criteria such as relative cost, permitting complexity, and implementation feasibility. These evaluations together form a comprehensive decision-support tool to assist the County in selecting a preferred alternative for flood risk reduction in Bolsa Knolls.

1.2 Study Reach

The defined study reach of Santa Rita Creek extends through the Bolsa Knolls community for a length of approximately 5,600 feet from Russell Road at the downstream end to the limit of residential development at the upstream end. The study reach is shown in **Figure 1-1.**

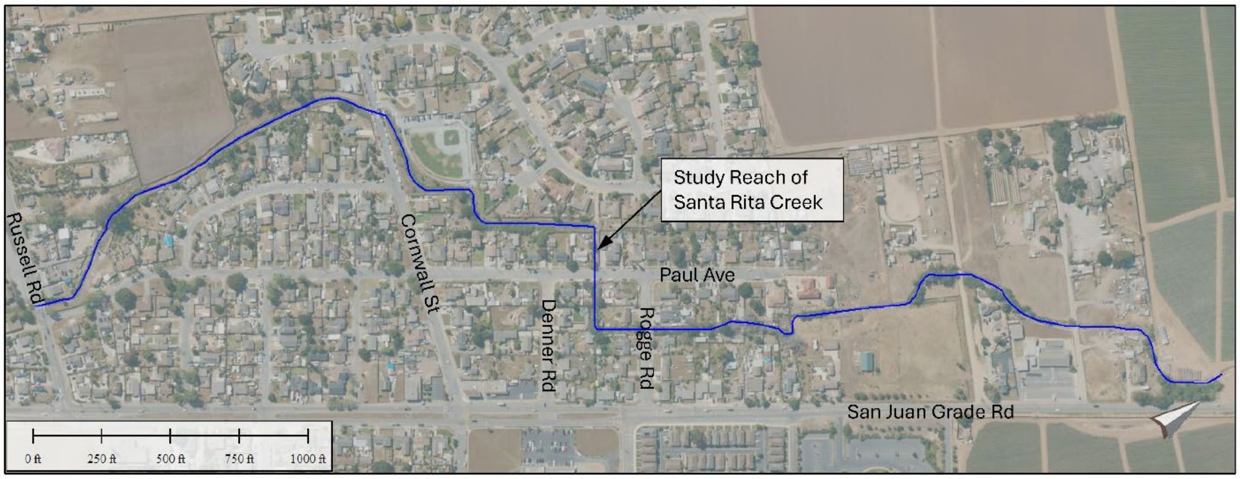

Figure 1-1 Map of study reach. Note that north is pointing to the right.

Although the study focuses on resolving flooding along this specific creek segment, additional project elements such as sediment retention and flow detention basins are considered further upstream. Similarly, the hydrologic and hydraulic models extend beyond the defined study reach to account for upstream and downstream influences, ensuring that the broader watershed context is incorporated into project evaluation.

Flooding History

A review of historical flooding was completed using reports from local news outlets. Articles dating as far back as the 1930s describe flooding impacts in the Santa Rita neighborhood of the City of Salinas, downstream of the current study area, indicating that flood issues have long affected Santa Rita Creek, even before later development in Bolsa Knolls. Examples of past flooding are illustrated in **Figure 1-2**, showing both high flood flow rates and overbank flooding in vicinity of what is now North Main Street.

Figure 1-2 Flooding near Highway 101 (now Main Street) circa 1940s

More recent flooding within the study area has been documented in the years 2014, 2017, 2021, 2022, 2023, and 2025 (this list is derived from the news reports that were identified and may not include all notable flood events that occurred across this time frame). These events, as reported in the local media, highlight frequent and widespread flooding within Bolsa Knolls. Reports include qualitative descriptions corroborated by photos and videos showing inundated streets, residential yards, and, in several cases, damage to homes. Specific examples are compiled in **Figure 1-3**.

Figure 1-3 Example images of past flooding events. additional photos with dates and locations noted are included as part of **Appendix C**.

Unfortunately, no stream gauge records, high water marks, or inundation mapping from these past flood events have been identified. As a result, the flood impacts could not be directly tied to specific flow rates or recurrence intervals (e.g., 5- or 10-year events). Nevertheless, information from these reports was used as reference material to calibrate the hydrologic and hydraulic models developed for this study through examination of the captured flood depths and extents. This allowed the model output to be used to approximate flood extents and potential impacts corresponding to several historical storm events as well as hypothetical design storms.

2 HYDROLOGIC MODELING

2.1 Purpose

Hydrologic modeling of the Santa Rita Creek watershed was conducted to generate flood event hydrographs for input into the hydraulic model of the creek. This analysis also contextualizes historical flooding by estimating the relative scale of past storm events.

2.2 Watershed Characteristics

Santa Rita Creek drains approximately 14 square miles at its confluence with Alisal Slough. However, this study focuses on the 4.8-square-mile portion located upstream of U.S. Highway 101 (**Figure 2-1**). Land use in the upper watershed is predominantly agricultural and residential, with smaller areas of commercial development, a golf course upstream of the study reach, and undeveloped rangeland along the northern boundary. Soil types across the upper watershed include Chualar loams and Santa Ynez fine sandy loams as the dominant classes, along with pockets of Gloria sandy loams, Arroyo Seco gravelly loams, Arnold loamy sands, Placentia sandy loams, and Elkhorn fine sandy loams. A more complete description of the soils covering the watershed is included in the *Approaches and Practices to Enhance Conditions in the Santa Rita Watershed* (Balance Hydrologics, 2017).

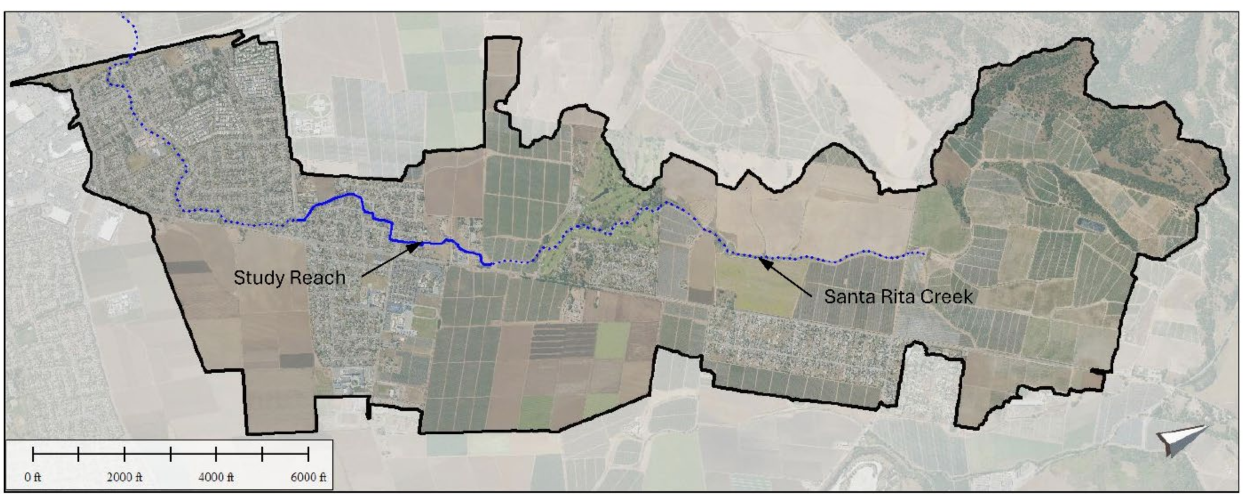

Figure 2-1 Santa Rita Creek Watershed at Highway 101. Note that north is pointing to the right.

2.3 Gauged Rainfall Data

Rainfall data were obtained from three nearby gauges maintained by the Monterey County Water Resources Agency (MCWRA), as shown in **Figure 2-2**. Among them, the Natividad gauge was deemed most representative of the watershed due to its proximity and similar elevation. Rainfall records were collected from 2007 through 2025 (with the Dunes Colony gauge beginning operation in 2021).

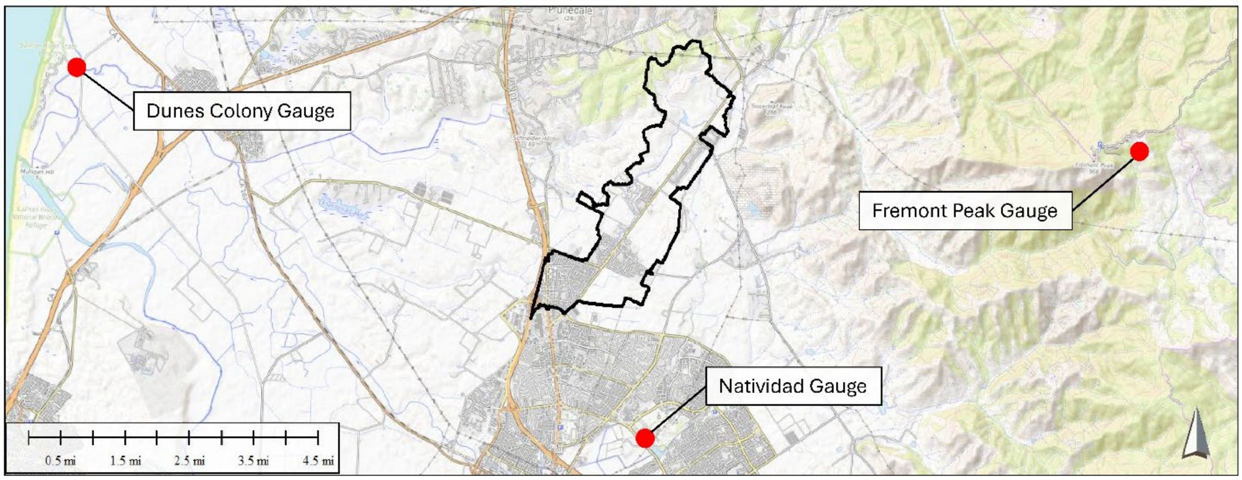

Figure 2-2 Rain gauge locations in relation to the Upper Santa Rita Creek watershed

Figure 2-3 illustrates the annual cumulative precipitation at each gauge, highlighting year-to-year variability and elevation-driven differences in rainfall depth.

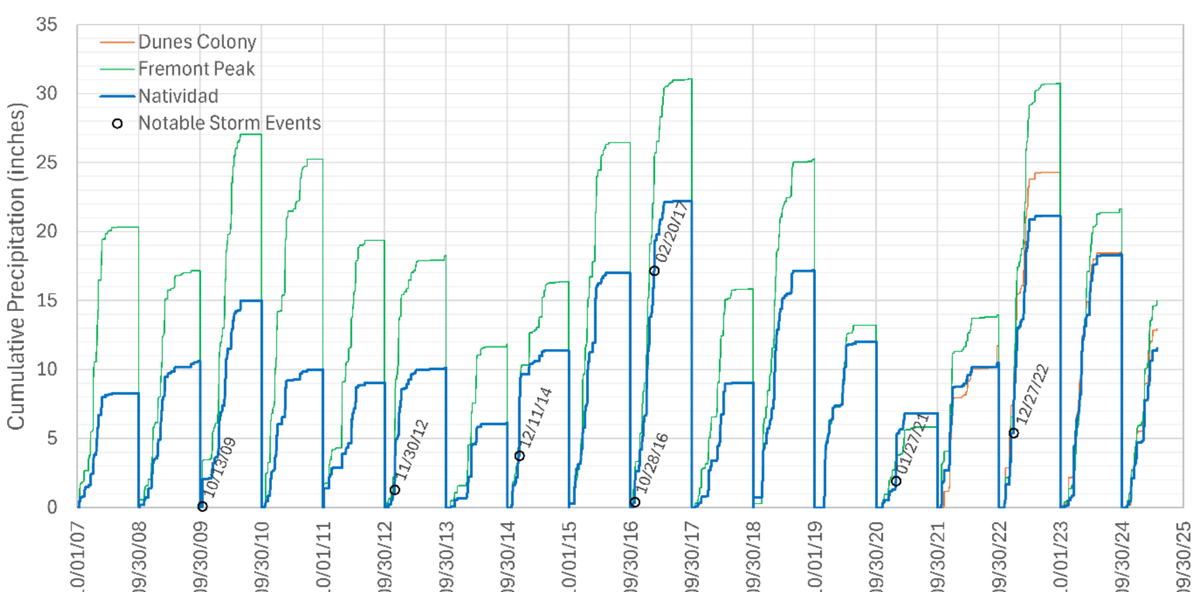

Figure 2-3 Gauged cumulative annual precipitation and significant storm events

A statistical analysis of precipitation at the Natividad gauge identified storm events used for model calibration and contextualized the magnitude of past flood events. **Table 2-1** ranks the largest storm events based on total depth over durations ranging from one to 24 hours and compares those values to 10- and 100-year design storm depths from the Atlas 14 precipitation frequency estimates compiled by the National Oceanic and Atmospheric Administration.

Table 2-1 Ranked storm depths at the Natividad gauge

	1 hour		3 hour		6 hour		12 hour		24 hour	
Rank	Date	Depth	Date	Depth	Date	Depth	Date	Depth	Date	Depth
-	-	*inches*	-	*inches*	-	*inches*	-	*inches*	-	*inches*
1st	12/11/14	0.72	12/11/14	1.44	12/11/14	2.44	12/12/14	3.72	12/12/14	3.96
2nd	03/05/16	0.68	12/27/22	1.32	11/30/12	2.08	11/30/12	2.32	01/28/21	2.56
3rd	11/30/12	0.60	11/30/12	1.24	12/27/22	1.84	12/27/22	2.32	12/27/22	2.36
4th	12/27/22	0.56	10/28/16	1.12	10/13/09	1.40	02/20/17	1.96	11/30/12	2.32
5th	12/19/23	0.56	02/01/24	1.02	02/20/17	1.40	10/13/09	1.88	12/14/21	2.24
6th	12/20/23	0.53	01/31/24	1.00	10/25/21	1.40	01/27/21	1.84	02/20/17	2.12
7th	10/28/16	0.48	01/19/16	0.96	01/19/16	1.40	10/28/16	1.80	03/10/23	2.08
8th	02/06/09	0.48	03/05/16	0.92	10/28/16	1.40	03/10/23	1.68	10/13/09	2.00
100yr Design Storm	-	1.38	-	2.15	-	2.84	-	3.74	-	4.92
10yr Design Storm	-	0.87	-	1.34	-	1.78	-	2.34	-	3.08

2.4 Hydrologic Modeling Approach

Hydrologic modeling was completed using the Curve Number runoff methodology implemented within the U.S. Army Corps of Engineers HEC-HMS modeling platform (version 4.12). Simulated scenarios include the 10- and 100-year, 24-hour design storms as well as the February 20, 2017; December 27, 2022; and February 13, 2025, calibration storm events.

These calibration events were selected based on variability in total depth, peak intensity, and antecedent conditions. The December 2022 storm was among the largest in the record (see **Table 2-1**), with documentation of flood impacts in Bolsa Knolls. The February 2017 event, although less intense, followed a sequence of prior storms that contributed to elevated runoff rates. The February 2025 event was relatively modest in magnitude (1.5 inches of total storm depth) but was included to represent a threshold storm that initiated flooding.

2.5 Key Parameters and Assumptions

Several of the most important assumptions and parameters used in the hydrologic model are summarized below:

SUB-WATERSHED DELINEATION: Sub-watersheds were delineated using 2017 LiDAR data compiled by the United States Geological Survey (USGS), subdivision as-built plans, aerial imagery, and field reconnaissance. Sub-watershed boundaries and areas are shown in **Figure 2-4**, with key attributes are summarized in **Table 2-2.**

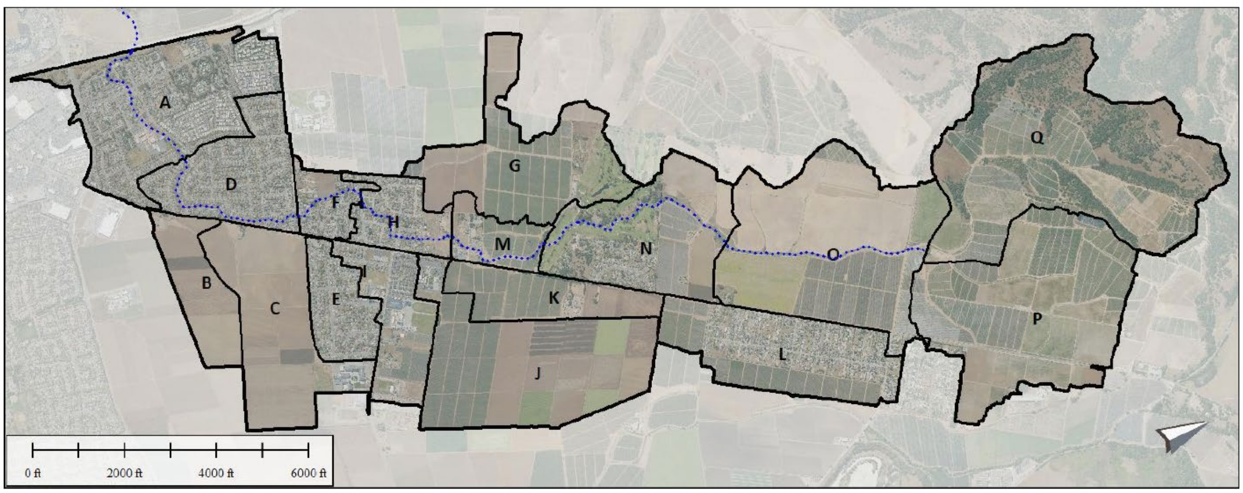

Figure 2-4 Delineated sub-watersheds

CURVE NUMBERS: Composite Curve Numbers were developed based on hydrologic soil groups as provided in the Soil Survey Geographic Database (SSURGO) from the Natural Resources Conservation Service and land cover data provided in the 2023 National Land Cover Dataset (NLCD) form the USGS. Impervious surfaces were incorporated using the NLCD 2023 Fractional Impervious Surface dataset. Curve Numbers assigned to agricultural areas assumed 70 percent coverage of plastic mulch, increasing runoff potential by 10 percent within those areas. A summary of composite Curve Numbers by sub-watershed is included in **Table 2-2**.

Table 2-2 Summary of key model parameters by sub-watershed

Watershed	Area	Rainfall Depths		Curve Num.	Initial Abs.	Time Lag
		100yr 24hr	10yr 24hr			
-	*acres*	*in*	*in*	-	*in*	*min*
A	251	4.42	2.76	77	0.15	20
B	82	4.44	2.78	79	0.13	39
C	194	4.48	2.80	78	0.14	41
D	138	4.48	2.80	72	0.19	20
E	71	4.58	2.88	76	0.15	20
F	43	4.57	2.86	69	0.23	20
G	203	4.78	2.99	79	0.13	34
H	68	4.61	2.88	69	0.22	20
I	99	4.63	2.90	77	0.15	20
J	268	4.78	3.00	80	0.13	60
K	94	4.81	3.02	78	0.14	33
L	183	5.15	3.24	75	0.17	38
M	53	4.76	2.99	72	0.19	20
N	200	4.92	3.08	70	0.21	36
O	333	5.17	3.25	80	0.12	31
P	334	5.46	3.44	81	0.12	40
Q	441	5.45	3.43	68	0.23	54

Note: Curve Numbers and Initial Abstractions presented for Antecedent Moisture Condition 2.0.

ANTECEDENT MOISTURE CONDITIONS: Antecedent Moisture Conditions (AMC) values were assigned to each storm event and used to adjust Curve Numbers to account for the degree of saturation across the watershed at the initiation of rainfall. AMC values ranged from 1.0 (dry) to 3.0 (wet). 10- and 100-year design storm events were assigned an AMC value of 2.0, assuming moderately saturated watershed conditions. The February 20, 2017; December 27, 2022; and February 13, 2025 calibration storm events were assigned AMCs of 3.0, 1.5, and 2.0 respectively to reflect the preceding rainfall before each event as illustrated in **Figure 2-5**. The Curve Number values assigned for different Antecedent Moisture Conditions are included in **Appendix A**.

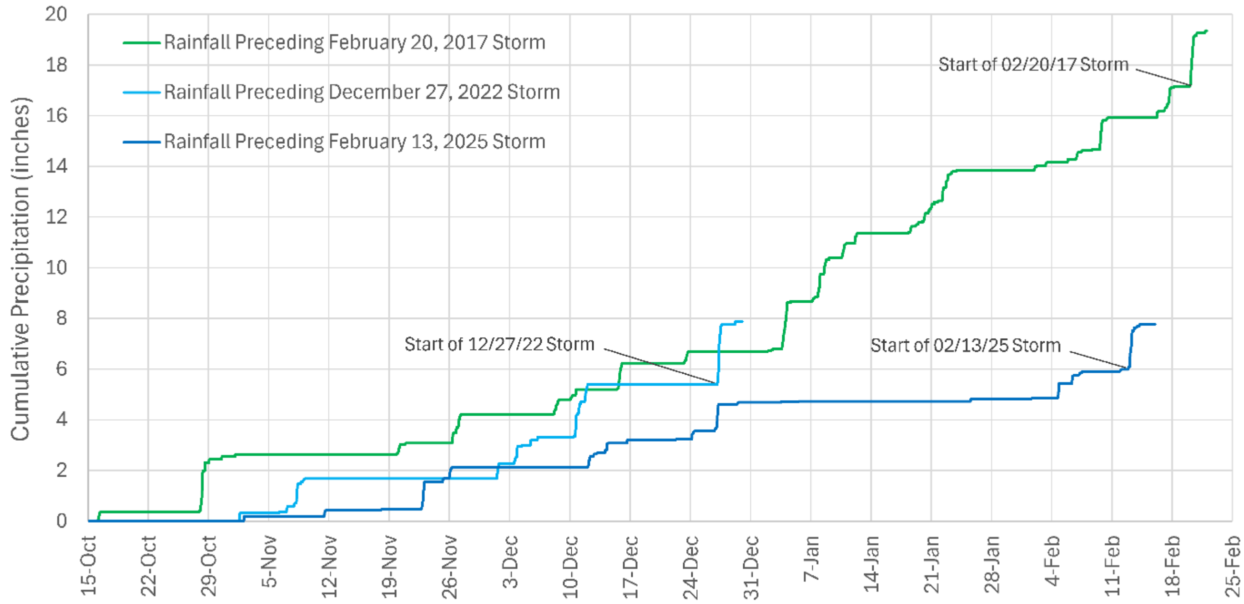

Figure 2-5 Cumulative rainfall preceding calibration storm events

LAG TIME: Lag time was calculated using a modified U.S. Army Corps of Engineers basin lag equation, incorporating slope, flow path length, roughness, and centroid distances. Calculated lag times are summarized in **Table 2-2**, with supporting input parameters in **Appendix A.**

RAINFALL: Calibration event rainfall data were derived from the Natividad gauge, processed into uniform 15-minute intervals, and applied consistently across all sub-watersheds. Design storm depths were based on NOAA Atlas 14 precipitation frequency estimates at each sub-watershed and distributed using an SCS Type I storm pattern. Cumulative precipitation plots for all modeled events are presented in **Figure 2-6**.

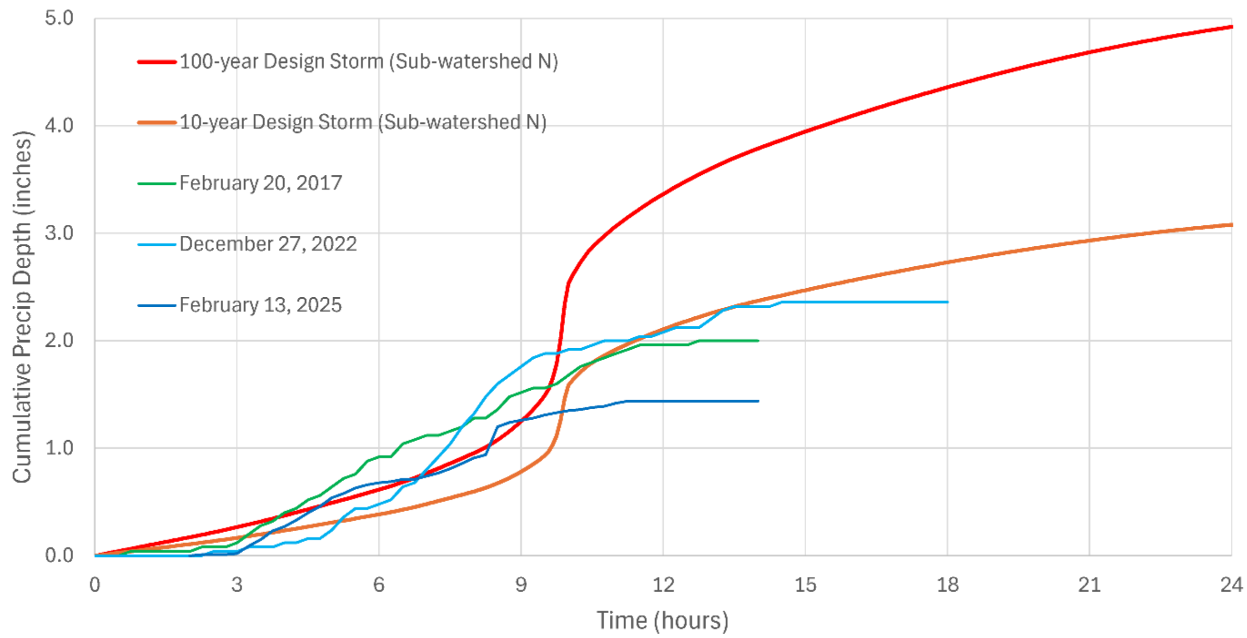

Figure 2-6 Cumulative precipitation across calibration and design storm events

REACH ROUTING: Runoff from the sub-watersheds was combined using routing reaches to represent channel connectivity. Routing used the Muskingum-Cunge method, with modeled parameters summarized in **Appendix A**. The HEC-HMS model schematic including the routing reach elements is shown in **Figure 2-7**.

BASIN ROUTING: Two relatively large detention basins were identified within the upper watershed at the locations shown in Figure 2-8. A review of aerial photographs indicates the embankment defining the middle reach basin was frequently breached and most notably during the February 20, 2017, and December 27, 2022, storm events, and as a result was not considered in the model. However, the larger upper reach basin was included in the model and was parameterized with a storage volume of 37 acre-feet at the basin spillway elevation. The outlet works include twin 36-inch diameter corrugated metal pipes (CMPs) and a 30-foot crest length overflow spillway.

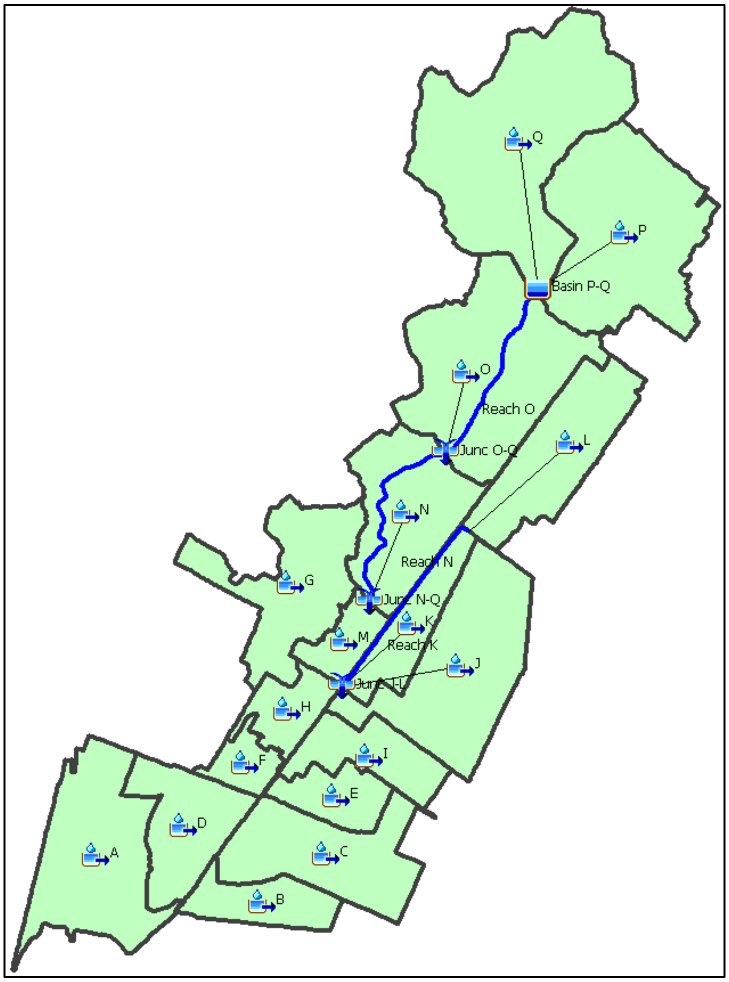

Figure 2-7 HEC-HMS model schematic

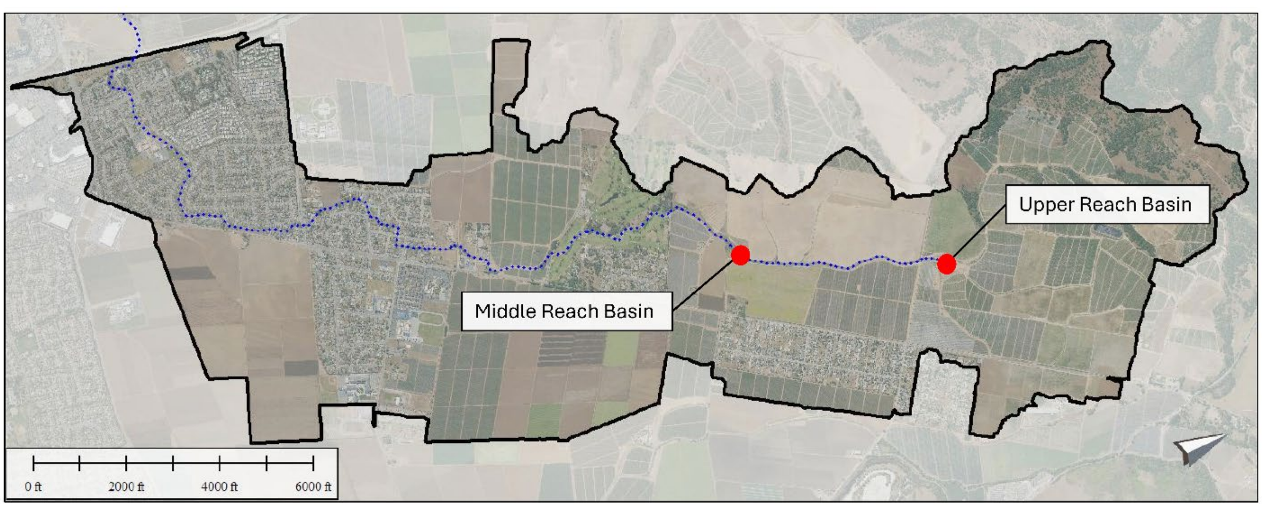

Figure 2-8 Existing basin locations

2.6 Calibration

In the absence of actual streamflow gauging data, calibration of the hydrologic modeling was conducted using observed flood photographs and videos from past events. Key calibration parameters included Curve Numbers, antecedent moisture conditions, and a coefficient used to calculate lag time. Calibration results are compared with observed flood conditions in the Existing Conditions Hydraulic Modeling discussion in **Section 3**.

2.7 Results

Hydrographs for design and calibration storm events routed to the study reach are included in **Appendix B**. **Figure 2-9** presents summed hydrographs discharging to the study reach. These totals do not account for flow attenuation due to channel storage and phasing, which are captured subsequently in the hydraulic model.

These results underscore three primary factors that govern peak runoff rates from the watershed: total storm depth, peak storm intensity, and antecedent moisture conditions. For instance, although the December 27, 2022, storm produced both greater rainfall totals and higher peak intensities than the February 20, 2017, event, drier antecedent conditions in 2022, resulted in lower peak flow rates. In contrast, the 10- and 100-year design storms assume conservatively high storm depths, intense rainfall rates, and moderately saturated soil conditions, factors that together are projected to generate significantly higher peak flow rates than those observed during the calibration storm events.

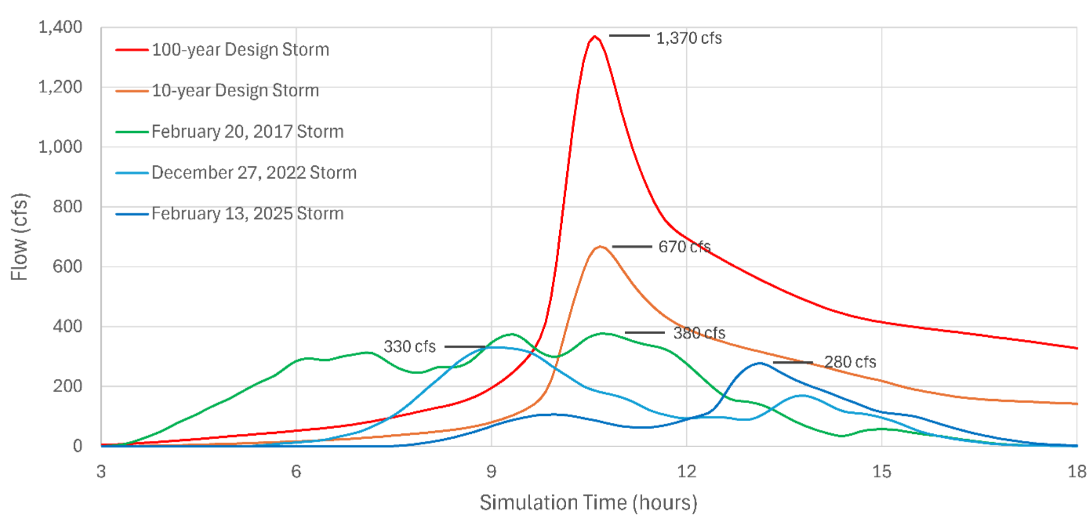

Figure 2-9 Summed hydrographs discharging to the study reach

Another important outcome of the modeling is the predicted performance of the existing upstream basin in attenuating peak flows. The model predicts the basin will lower 100-year peak flow rates by 58 percent, and 10-year peak flow rates by 45 percent at the basin outlet. Less peak flow attenuation, on the order of 10 to 20 percent, is predicted for the calibration storm events. A comparison of inflow and outflow hydrographs from the existing upstream basin is included as **Figure 2-10**.

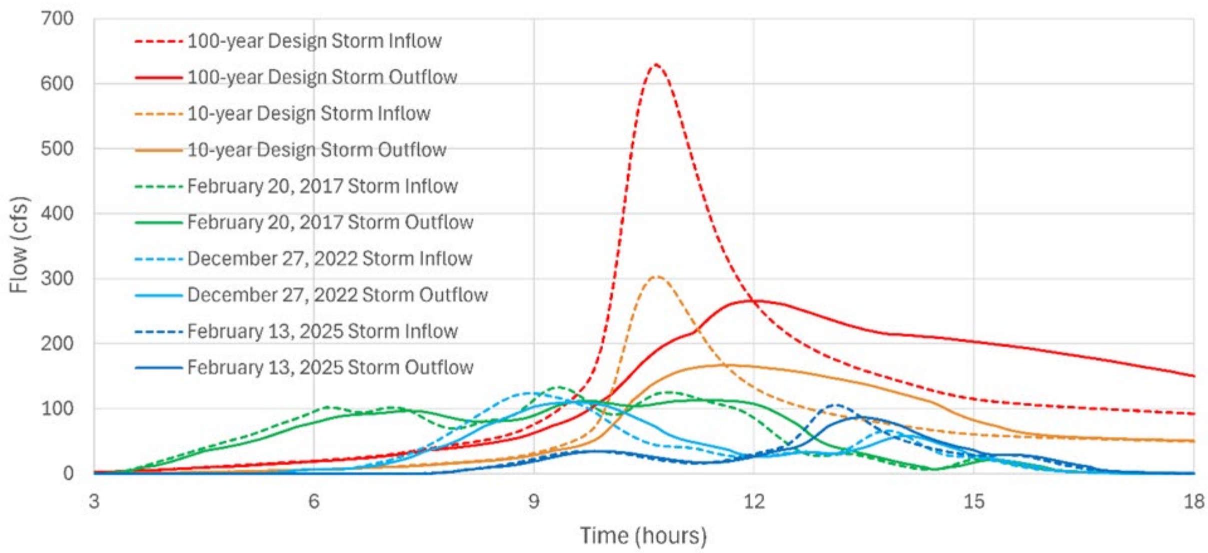

Figure 2-10 Modeled peak flow attenuation through the existing upstream basin

Finally, the results from the current hydrologic model can be compared to those from earlier analyses, including the FEMA Flood Insurance Study (completed in the late 1970s) and the Master Drainage Plan for the Santa Rita Creek Watershed (Monterey County Surveyors, 1972). All three studies report peak flow rates at a shared point of concentration, downstream at the Main Street crossing. The current model estimates a 100-year peak flow of 1,310 cfs (after routing the hydrographs through the hydraulic model), compared to 465 cfs reported by FEMA and 992 cfs from the Master Drainage Plan.

3 EXISTING CONDITIONS HYDRAULIC MODELING

3.1 Purpose

Hydraulic modeling of the study reach of Santa Rita Creek was developed to:

- Estimate the existing flood hazard through Bolsa Knolls across a range of storm events.
- Identify channel constrictions and evaluate the impact of other factors such as vegetation and sedimentation on conveyance capacity.
- Provide a tool to assess the benefits of potential project alternatives and a baseline condition that those alternatives can be quantitatively compared against.

3.2 Channel Characteristics

Through Bolsa Knolls, Santa Rita Creek consists of a single-thread channel with a typical top width of 20 to 30 feet, an average depth of 5 to 6 feet, and a longitudinal slope of approximately 0.5 percent. Cross sections at representative locations along the study reach are shown on **Figure 3-1** and are presented in comparison to the deeper and wider cross sections along the downstream reach that is known to flood less frequently. Historical aerial imagery indicates the creek channel runs along its pre-development alignment from Russell Road to Cornwall Street, however, the channel has been re-aligned along most of the upstream segment of the study reach to include four relatively short-radius 90-degree channel bends.

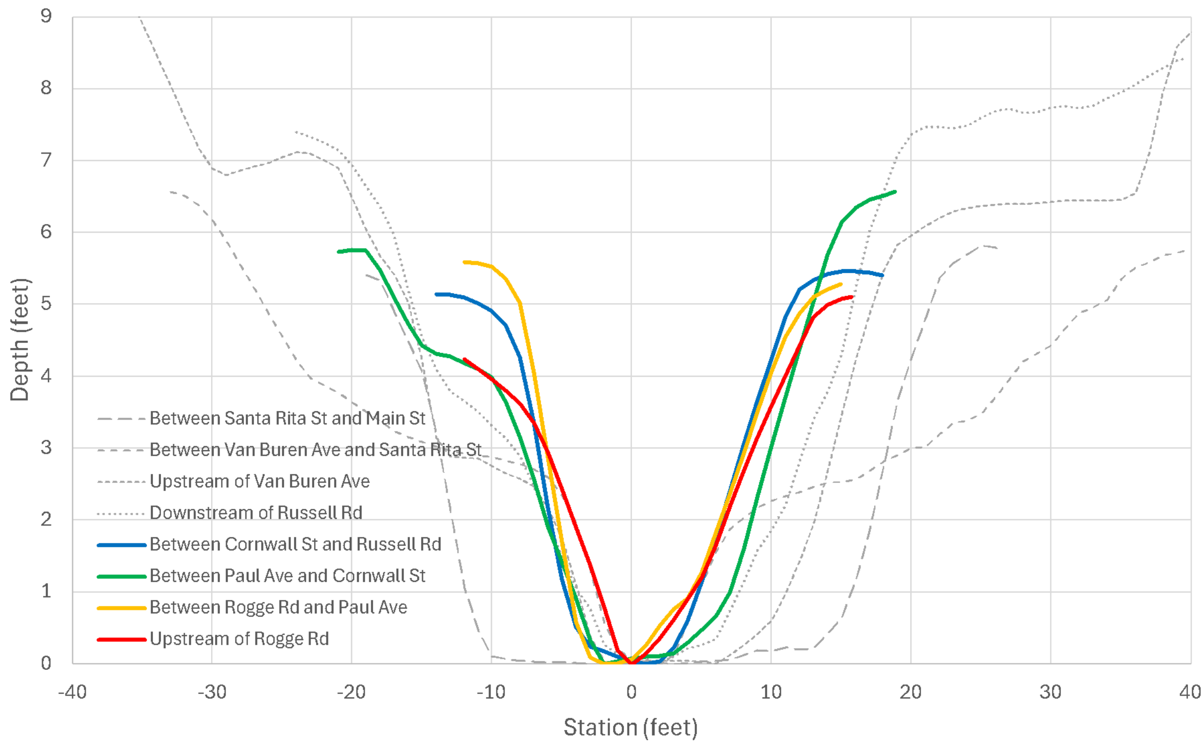

Figure 3-1 Example cross sections. Dashed sections are downstream of the study reach.

Channel crossings were identified at nine locations along the study reach as shown in **Figure 3-2**. These crossings vary greatly in size from the twin 5-foot span by 10-foot box culverts at Cornwall Street to a pair of partially obstructed 2.5 to 3.5-foot diameter pipe culverts located along a private drive near the upstream end of the study reach.

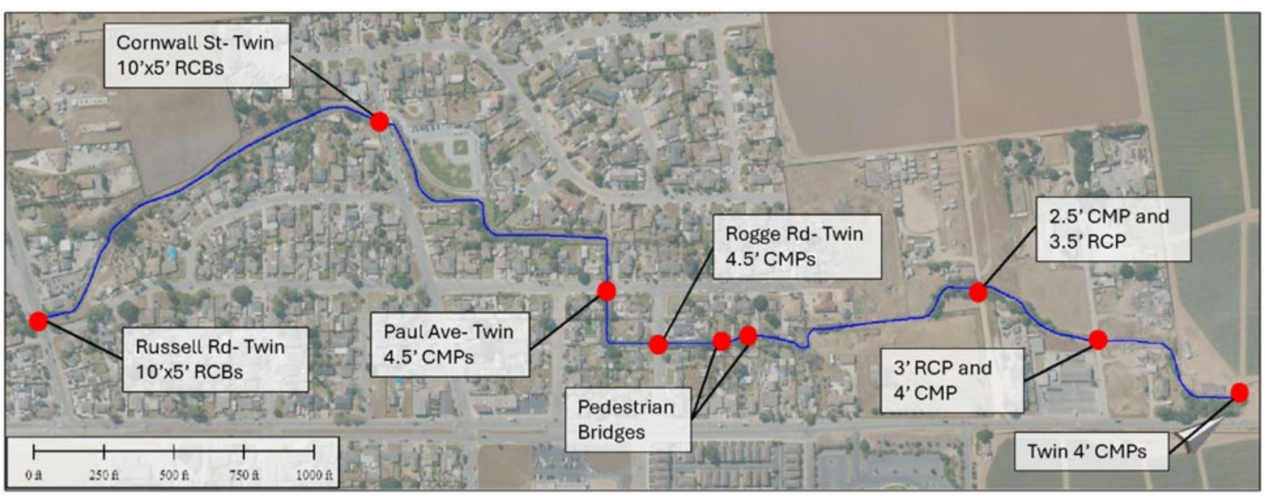

Figure 3-2 Channel crossing locations

Sediment accumulation has impacted conveyance in some areas, particularly at culvert inlets. While a comparison between late-1970s survey data (collected for the FEMA Flood Insurance Study) and 2017 USGS LiDAR suggests an average rise of no more than 1 to 2 feet in channel invert elevation, localized sediment buildup at crossings has been more significant, up to 2 to 3 feet in some locations. **Figure 3-3** and **Figure 3-4** illustrate sediment conditions before and after the February 13, 2025, storm event.

Figure 3-3 Sediment before and after the February 13, 2025, storm event at the downstream face of the culverts at Rogge Road

Vegetation presents a similar constraint to flow, particularly upstream of Cornwall Street. Aerial imagery review and field reconnaissance indicate denser patches of vegetation along this segment compared to the more open downstream reach**. Figure 3-5** provides a photographic comparison.

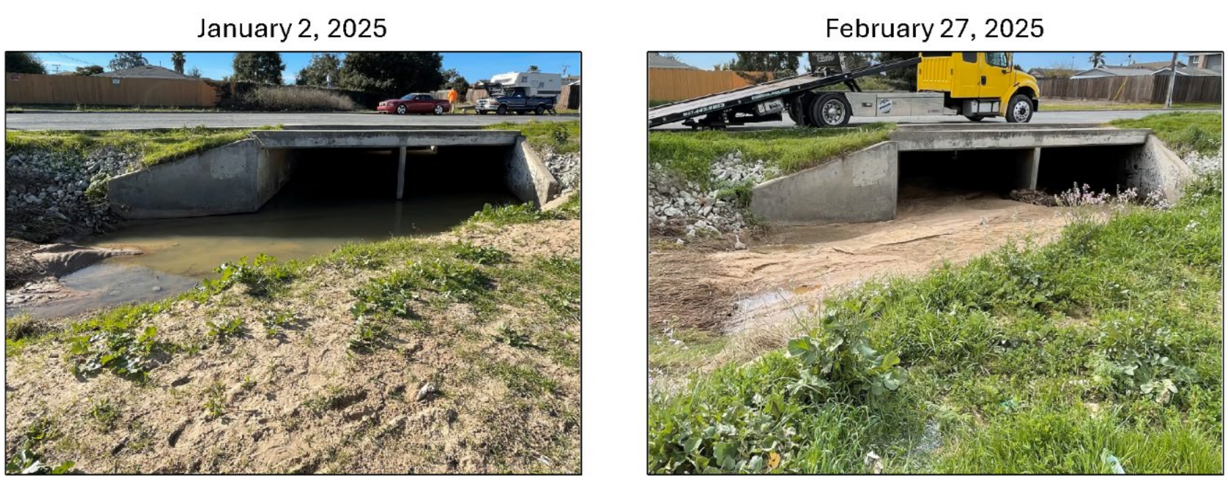

Figure 3-4 Sediment before and after the February 13, 2025, storm event at the upstream face of the culverts at Cornwall Street

The floodplain adjacent to the creek channel is largely developed along the study reach except for the section along Ferrasci Park that was restored to provide a naturalized floodplain bench. In some areas, the floodplain is as wide as 700 feet and relatively flat, allowing overtopping flood flows to spread widely, affecting streets, yards, and residential structures.

Downstream from Cornwall Street | Upstream from Cornwall Street

Figure 3-5 Example comparison of the differences in vegetation density across the study reach

3.3 Approach

Hydraulic modeling of Santa Rita Creek was completed using the U.S. Army Corps of Engineers HEC-RAS modeling platform (version 6.6) utilizing the two-dimensional hydrodynamic routing ability of the model. Existing condition model scenarios include the 10- and 100-year design storm events as well as the February 20, 2017; December 27, 2022; and February 13, 2025, calibration storm events, consistent with the previously discussed hydrologic modeling effort.

3.4 Key Parameters and Assumptions

Several of the most important assumptions and parameters used in the hydraulic model are summarized below:

MODEL DOMAIN: The model domain extends downstream to Highway 101 and upstream into the Salinas Golf and Country Club allowing for the assessment of flooding impacts both within and beyond the Bolsa Knolls study area. Additionally, by extending the model further in the upstream direction, a location is provided to evaluate proposed upstream detention alternatives. The full model domain is shown in **Figure 3-6**.

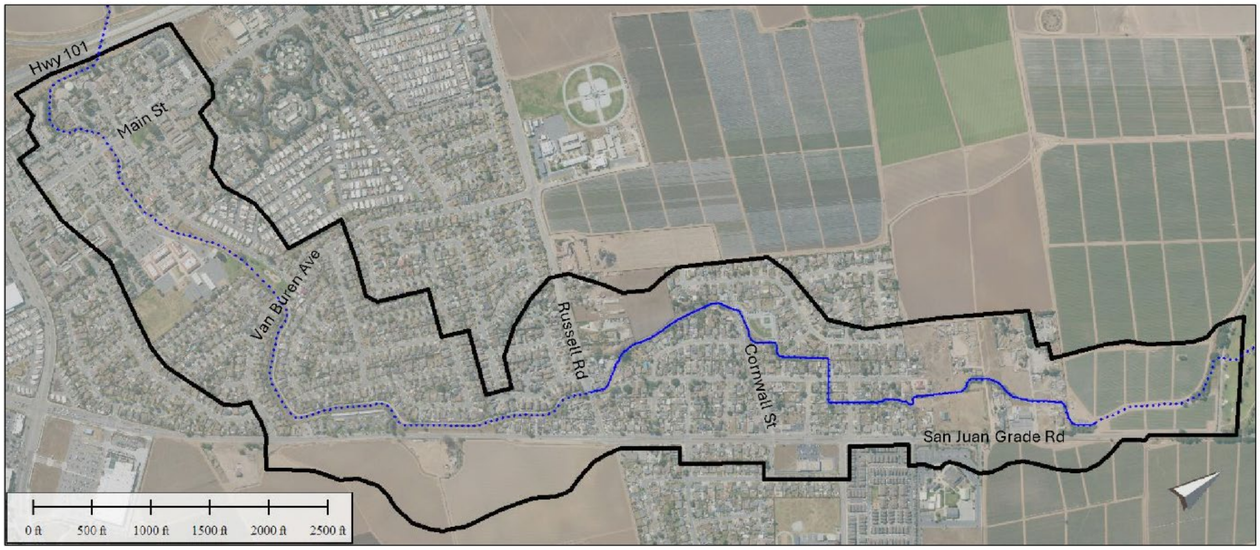

Figure 3-6 Hydraulic model domain

TERRAIN: The terrain, shown across the study reach in **Figure 3-7**, was derived primarily from the 2017 USGS LiDAR data and edited in select locations obscured by vegetation to reflect observations made in the field. Elevations near the crossing locations were also edited to reflect surveyed spot elevation data collected as part of this study.

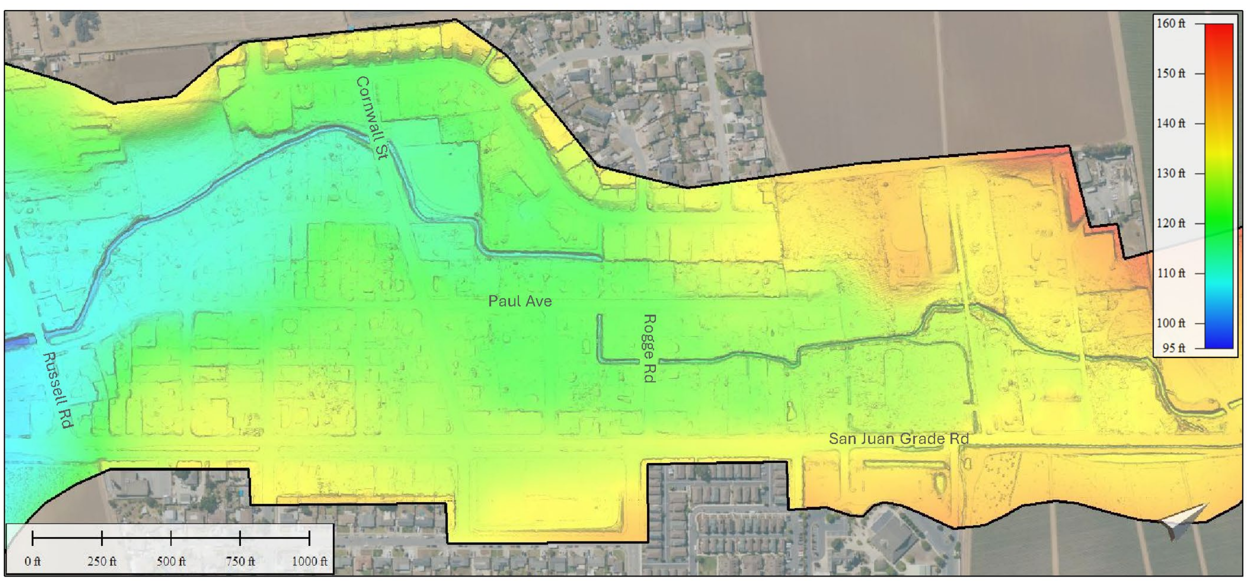

Figure 3-7 Terrain across the study reach

CALCULATION CELLS: Model cells along the creek channel were typically defined with dimensions of 5-feet by 5-feet and cells along overbank areas were set to 20-feet by 20-feet, with more than 70,000 cells covering the model domain. Breaklines were used throughout the model to capture high points in the terrain and to align cell faces perpendicular to flow along the channels. An example of the cell spacing and alignments is shown in **Figure 3-8**.

Figure 3-8 Example cell spacing and alignment

BOUNDARY CONDITIONS: Flow hydrographs generated by the hydrologic model were loaded at 12 inflow boundary locations shown in **Figure 3-9**. An outflow boundary condition was applied at the downstream end of the model near Highway 101 and defined with a normal depth slope of 0.3 percent.

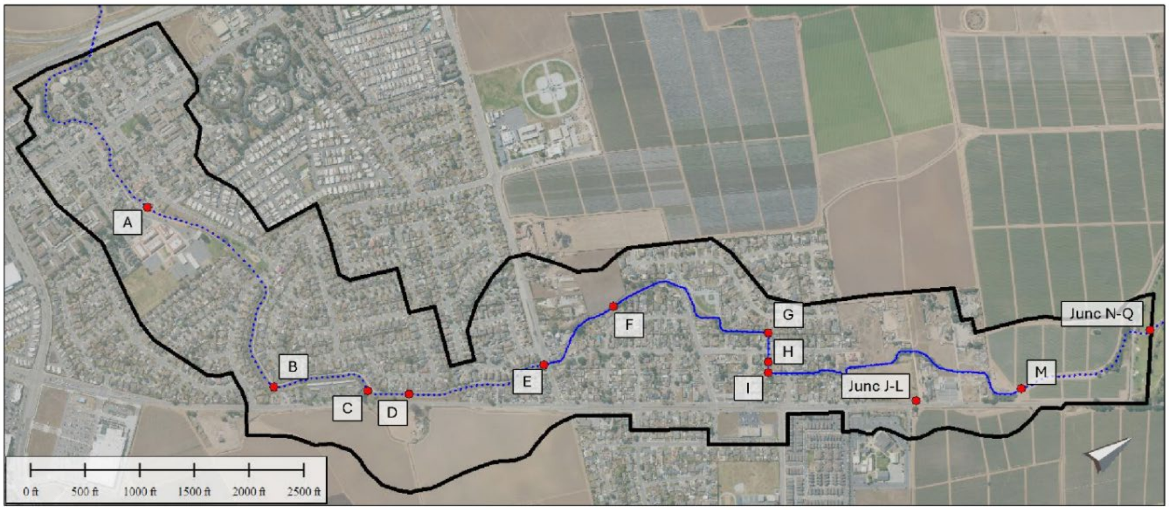

Figure 3-9 **Inflow boundary locations**. Labels reference sub-watersheds described in hydrologic modeling section.

CHANNEL AND FLOODPLAIN ROUGHNESS: Manning's 'n' values were defined across the model domain by zone with channel areas set to between 0.03 and 0.06 and overbank areas set to between 0.05 and 0.075. Streets were set to 0.03 and structures, such as buildings, were set to a very high value of 0.50 to represent their inherit obstruction to flow. Mapped 'n' values across the study reach are shown in **Figure 3-10**.

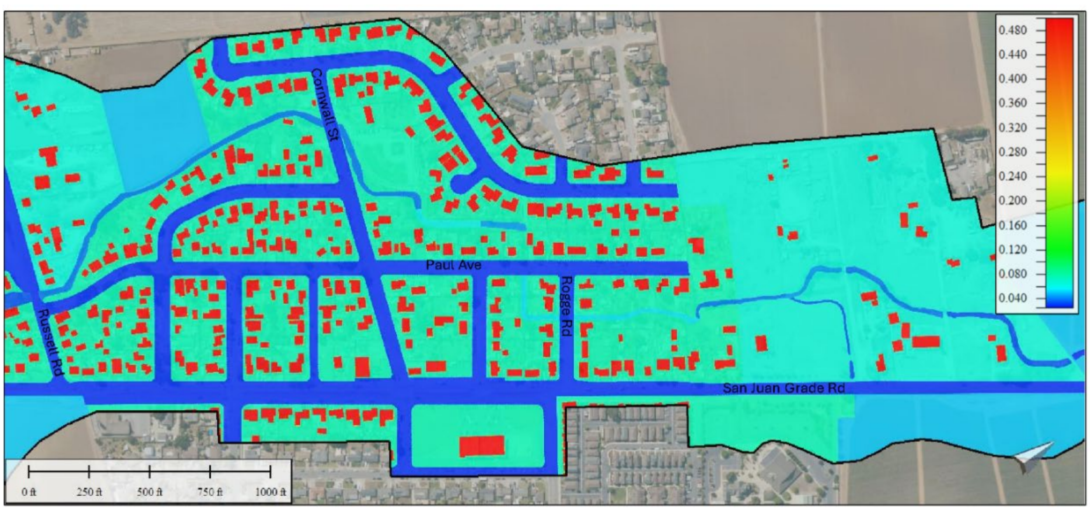

Figure 3-10 **Manning's 'n' values along the study reach**

CHANNEL CROSSINGS: 16 bridge and culvert crossings were included in the model as Storage/2D Area Connections at locations shown on **Figure 3-11**. The crossings were parameterized using survey data and field measurements, with sediment levels based on measurements and survey data collected in January and February 2025.

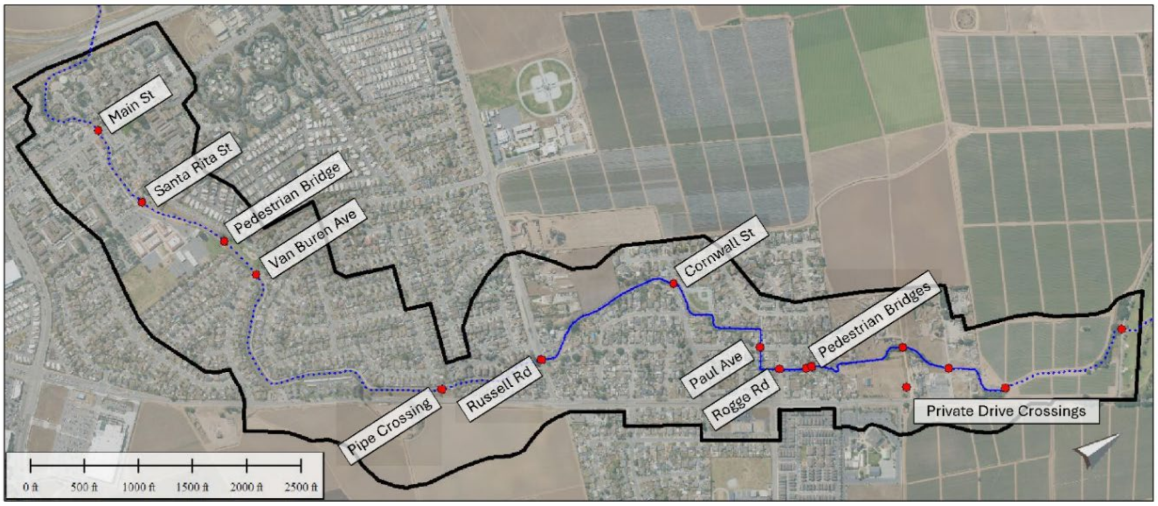

Figure 3-11 Modeled channel crossing locations

3.5 Calibration

Model calibration was conducted through visual comparison of simulated flood extents with photographic and video evidence from past storm events. Adjustments to Manning's '*n*' values were the primary means of calibration. Example comparisons between observed and modeled results are included below.

Figure 3-12 illustrates conditions for the February 20, 2017, flood event at the Paul Avenue crossing with depth of flood waters at the vehicles as comparison points.

Figure 3-12 Comparison between observed and modeled results at the Paul Avenue Crossing during the February 20, 2017, flood event

Figure 3-13 shows the flooding observed in the vicinity of Denner Road during the December 27, 2022, event with lateral extent of the inundation used as a primary metric.

Figure 3-13 Comparison between observed and modeled results at Denner Road during the December 27, 2022, flood event

Lastly, **Figure 3-14** shows flooding along Cornwall Street as observed during the February 13, 2025, storm with the encroachment of flooding on the residential structures as important indicators.

Figure 3-14 Comparison between observed and modeled results at Cornwall Street during the February 13, 2025, flood event

Additional calibration results are included as **Appendix C**.

3.6 Results

Large scale flood depth and water surface elevation plots generated by the existing conditions hydraulic model are included as **Appendix D**. Despite attenuation of peak flow rates in the large existing detention basin upstream of the hydraulic model domain, peak flow rates in the creek clearly exceed the channel capacity at numerous locations for all three of the calibration storms examined. The modeled flood flow hydrographs at the Rogge Road and Russell Road crossings are illustrated in **Figure 3-15** and show that the10-year design storm is predicted to generate peak flow rates at those locations of 500 and 550 cfs respectively, roughly 200 cfs higher than that of the largest calibration storm (February 20, 2017). Naturally, given the much larger total rainfall, coupled with limited upstream detention capacity, the 100-year design storm results in much higher predicted peak flow rates. At 1,030 and 1,190 cfs respectively, the 100-year flow rate is over twice the magnitude of the 10-year event.

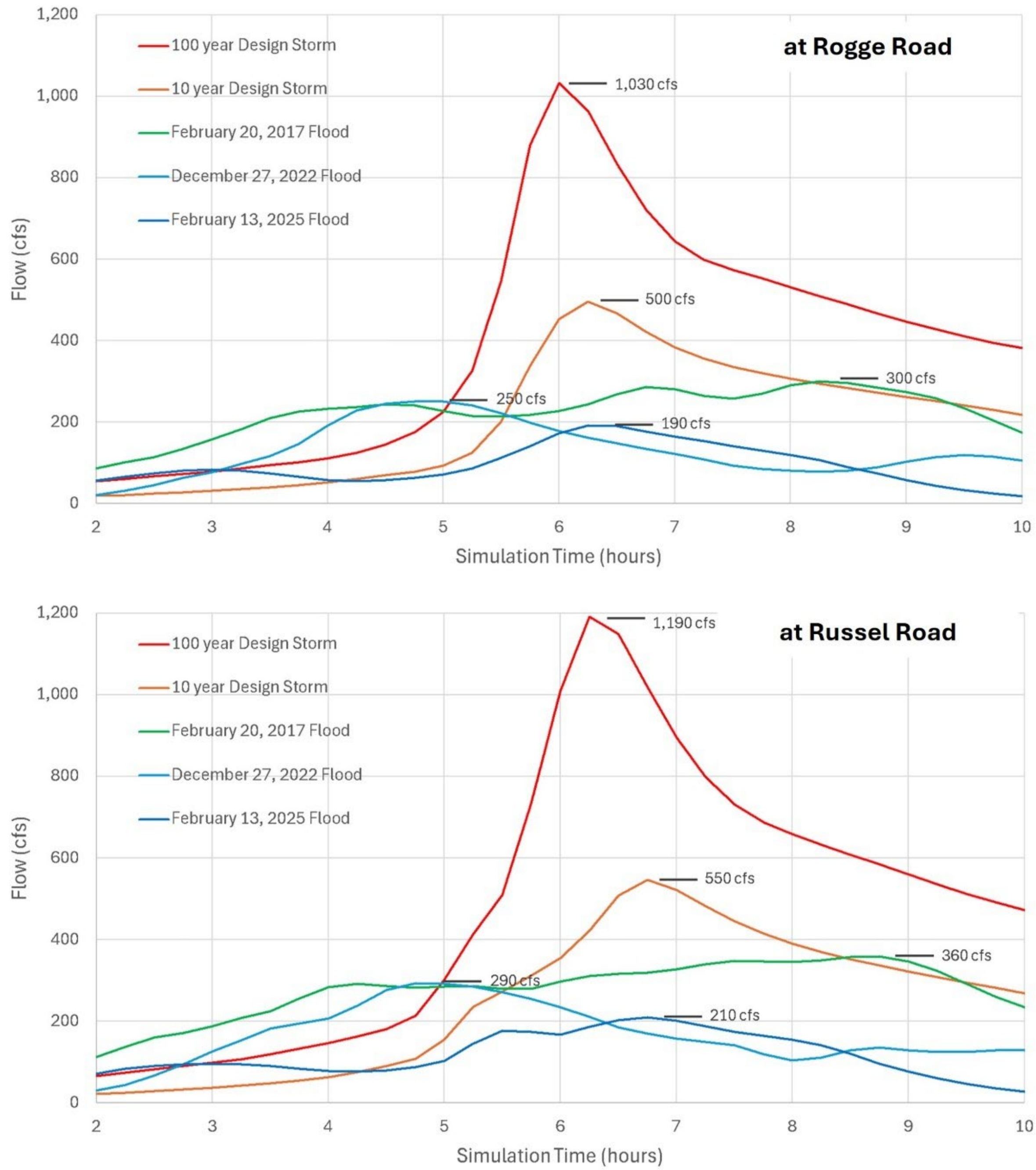

Figure 3-15 Existing conditions routed hydrographs at Rogge Road

Flooding is estimated to be initiated at the upstream edge of the study reach, where a tributary channel (draining Sub-watersheds J-L) overtops its banks at flows exceeding 20 cfs. Santa Rita Creek itself overtops just downstream from the confluence with this tributary at approximately 80 cfs. Within the medium-density residential neighborhood, overtopping first occurs 300 to 400 feet upstream from Rogge Road at flow rates above

100 cfs. These results are consistent with the Master Drainage Plan Santa Rita Creek Watershed (Monterey County Surveyors, 1972) that estimated a channel capacity along this reach, and all others downstream to Russell Road, of ±100 cfs. These characteristics of flooding within Bolsa Knolls for the observed calibration storms are illustrated in the model output plots in **Figure 3-16**.

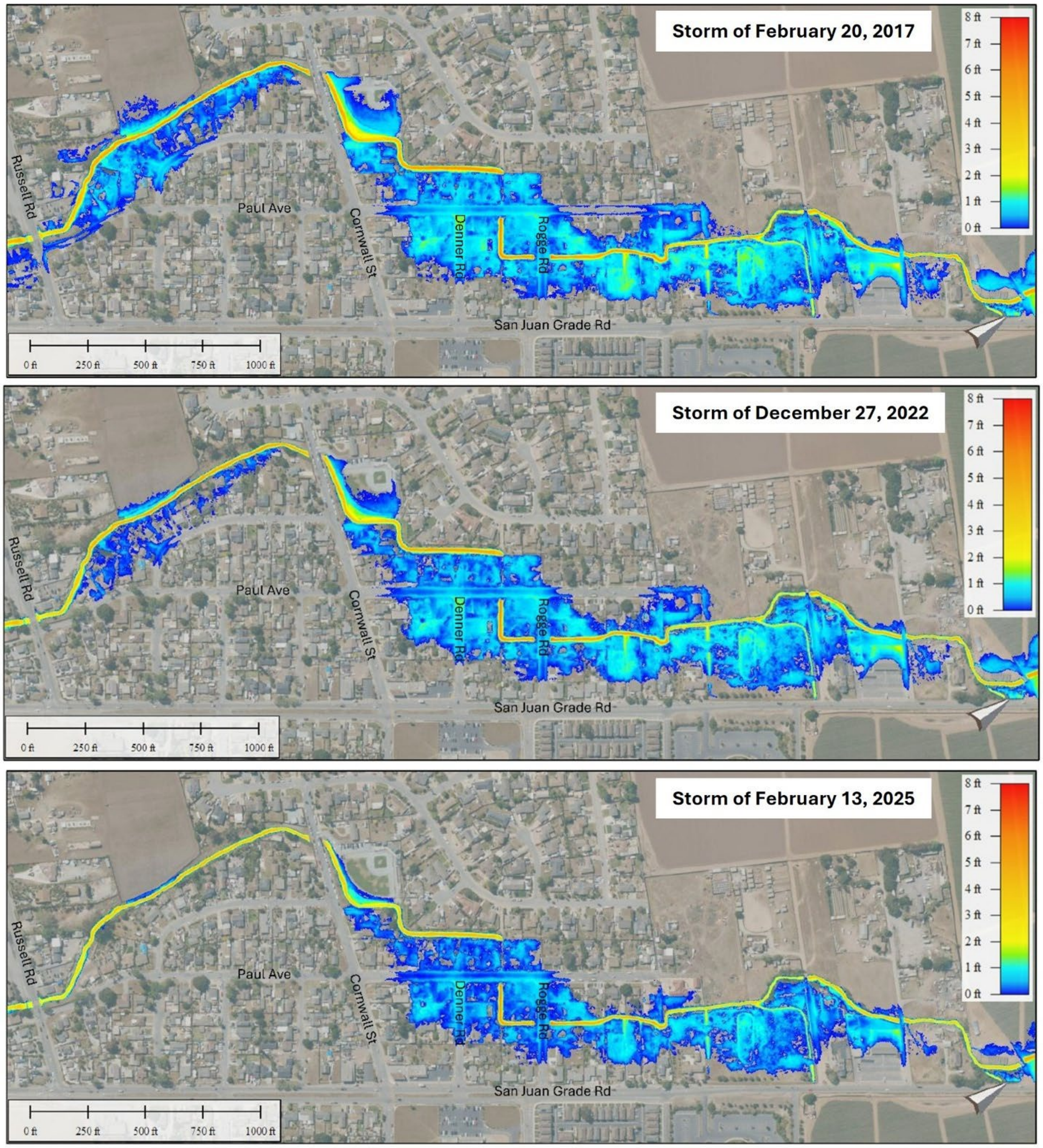

Figure 3-16 Existing conditions flood depths for historical calibration storms

Further downstream, the twin 4.5-foot diameter CMP crossings at Rogge Road and Paul Avenue present a constriction to flow with conveyance capacities of only 110 and 130 cfs respectively. By comparison the twin 10-foot high by 5-foot span reinforced concrete box (RCB) crossings at Cornwall Street and Russell Road are estimated to have capacities of 420 and 460 cfs respectively, though the modeling also shows that the channel between the two crossings would experience overtopping at multiple locations as flows approach 250 cfs.

Flooding extents and depths for the 10- and 100-year design floods are naturally markedly larger as shown in **Figure 3-17**.

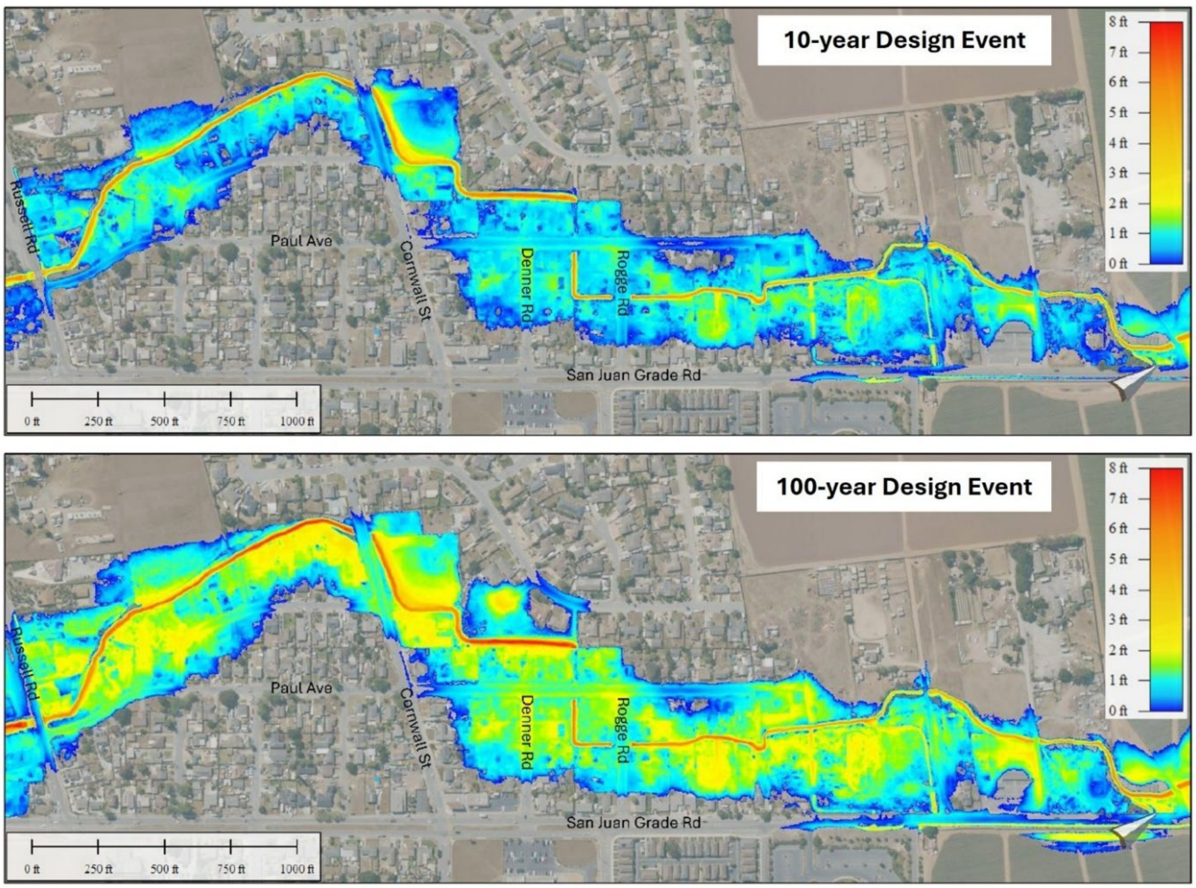

Figure 3-17 10- and 100-year design storm model output for existing conditions

Flows that overtop the channel banks are modeled to typically disperse across the relatively flat, developed floodplain before slowly draining back to the channel as flood levels recede. Two indicators of flood impacts across the floodplain are summarized in **Table 3-1** including inundation area along the study reach and residential structures (i.e., houses) fully or partially within the modeled flood extents (not necessarily at finish floor elevations which were not surveyed for this study).

Table 3-1 Existing flood overtopping impacts along the study reach across modeled storm events

Flood Event	Inundated Area	Impacted Structures
-	*acres*	-
100 year	44	81
10 year	34	61
February 20, 2017	24	48
December 27, 2022	20	43
February 13, 2025	15	30

A comparison between modeled 100-year flood extents and FEMA's mapped floodplain is shown in **Figure 3-18**. While the modeled extents generally indicate greater flood impacts, the FEMA 100-year water surface elevations are locally higher near Rogge Road and Paul Avenue. Upstream of Cornwall Street, the FEMA floodplain aligns more closely with the February 13, 2025, event than with the modeled 100-year scenario. However, in the Denner Road area, flooding observed and modeled from the relatively small February 2025 flood event extends significantly beyond the limits of the FEMA mapping.

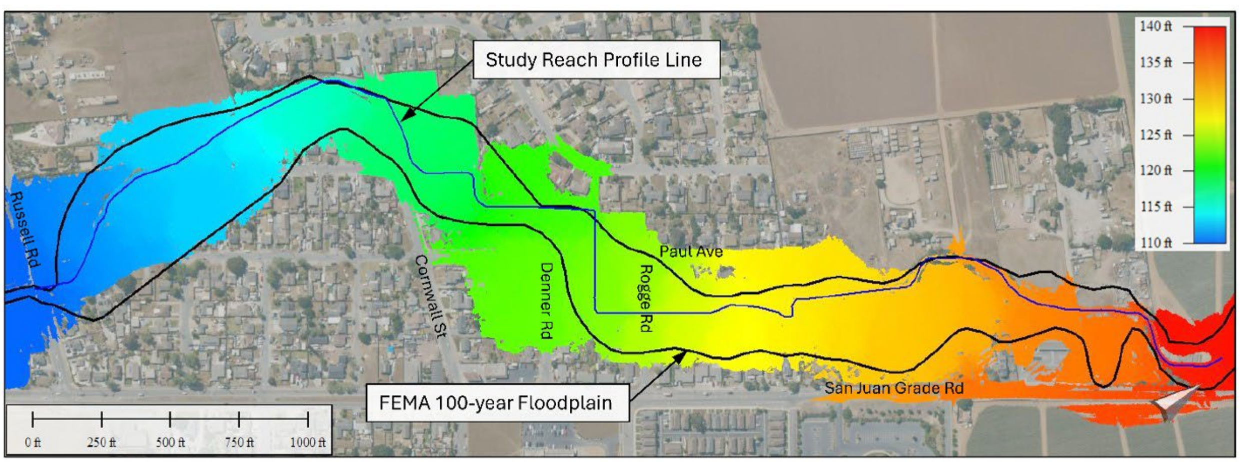

Figure 3-18 Modeled existing 100-year event compared to current FEMA mapping

The same comparison with respect to the flood profile comparison illustrated in **Figure 3-19**.

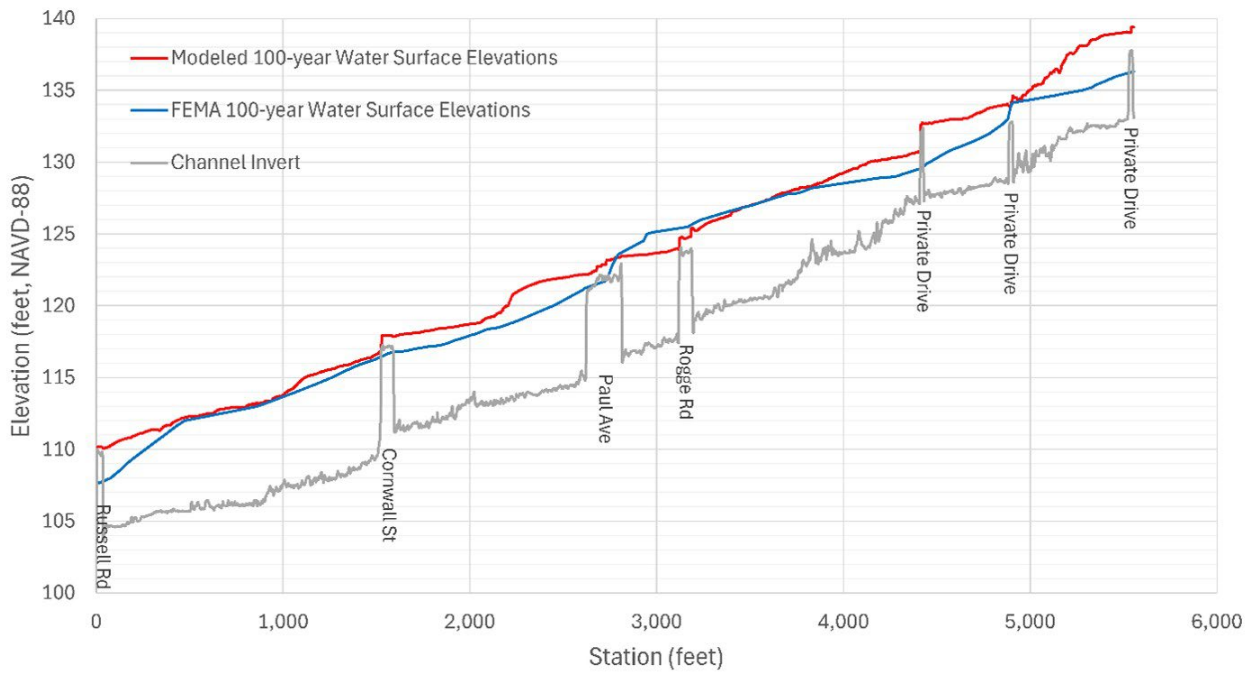

Figure 3-19 Currently effective FEMA vs existing hydraulic model 100-year flood profiles

4 PROJECT ALTERNATIVES

Several project alternatives, each combining different elements and actions, were developed that have the potential to minimize flood risk and impacts along the study reach of Santa Rita Creek. The selection of viable project components was informed by background data collected during this study, field investigations, and hydraulic model results. These inputs were used to identify key opportunities and constraints, which in turn guided the formulation of appropriate project elements and management actions. Ultimately, four project alternatives were assembled and evaluated using the hydrologic and hydraulic models developed for this study, enabling a comparative analysis of the respective costs and benefits of each alternative.

4.1 Opportunities and Constraints

Several opportunities have been identified to reduce flood risk in the Bolsa Knolls area:

> *Scale and frequency of impacts*: The frequent flood events affecting Bolsa Knolls are relatively minor in scale, though not in impact. As a result, they can be addressed with targeted, small-scale interventions rather than larger infrastructure projects typically required for more severe events like the 100-year flood.
>
> *Localized channel constrictions*: Hydraulic modeling has identified specific locations where channel capacity is restricted due to factors such as overgrown vegetation, sediment buildup at crossings, and undersized culverts, particularly at Rogge Road and Paul Avenue. These localized issues can be addressed more feasibly than larger, reach-wide improvements.
>
> *Un-urbanized upstream land*: Many areas upstream from the study reach remain largely unurbanized and consist of agricultural land. While valuable, it remains feasible to acquire land at a usable scale needed to site project elements.

While there are promising opportunities, several constraints could complicate efforts to mitigate flood risk:

> *Limitations in mitigating large events*: Although smaller, frequent flood events can be addressed, the existing limited channel conveyance capacity poses a significant challenge for managing larger storm events, such as the 100-year flood. Effective mitigation would likely require a comprehensive reconfiguration of the creek corridor and/or the development of substantial upstream detention capacity.

Constrained creek corridor: In many areas, residential landscaping, fences, and even structures extend up to the channel top of bank. This limits the potential for in-channel or floodplain improvements, such as widening or the construction of floodplain benches.

Private property: Much of Santa Rita Creek within the study reach traverses private property and several of these have no known drainage easements. Implementing certain project elements will necessitate extensive coordination and outreach, increasing the complexity of both design and implementation.

Environmental permitting requirements: Permitting projects within the creek corridor has the potential to be costly and time-intensive, particularly for any projects with larger footprints or impacts on sensitive habitats. These regulatory hurdles may constrain the scope and timeline of potential flood mitigation measures.

4.2 Project Elements and Management Actions

Several project elements and management actions have been identified that can provide implementable and cost-effective flood mitigation benefits across the study reach.

4.3 Culverted Crossings

Hydraulic modeling indicates that the crossings at Paul Avenue and Rogge Road are undersized relative to the conveyance capacity of the adjacent channel segments and downstream crossings at Cornwall Street and Russell Road. These undersized culverts create localized flow restrictions and reduce the efficiency of floodwater conveyance through this portion of the study reach. Importantly, both crossings are located within existing public rights-of-way or drainage easements, making them viable locations for targeted infrastructure improvements.

The existing Paul Avenue crossing consists of twin 4.5-foot CMPs extending 190 feet along the roadway and continuing downstream between two residential parcels. Photographs taken at the upstream and downstream faces of the culverts and a plan view of the culvert alignments are included in **Figure 4-1**. Site constraints include both vertical limitations, due to minimal ground cover above the culverts, and horizontal constraints, as the culverts are confined between residential fences approximately 25 feet apart. The

San Juan Acres Tract Map dated 1946 indicates that the downstream segment of the culverts lies within a 25-foot-wide drainage easement.

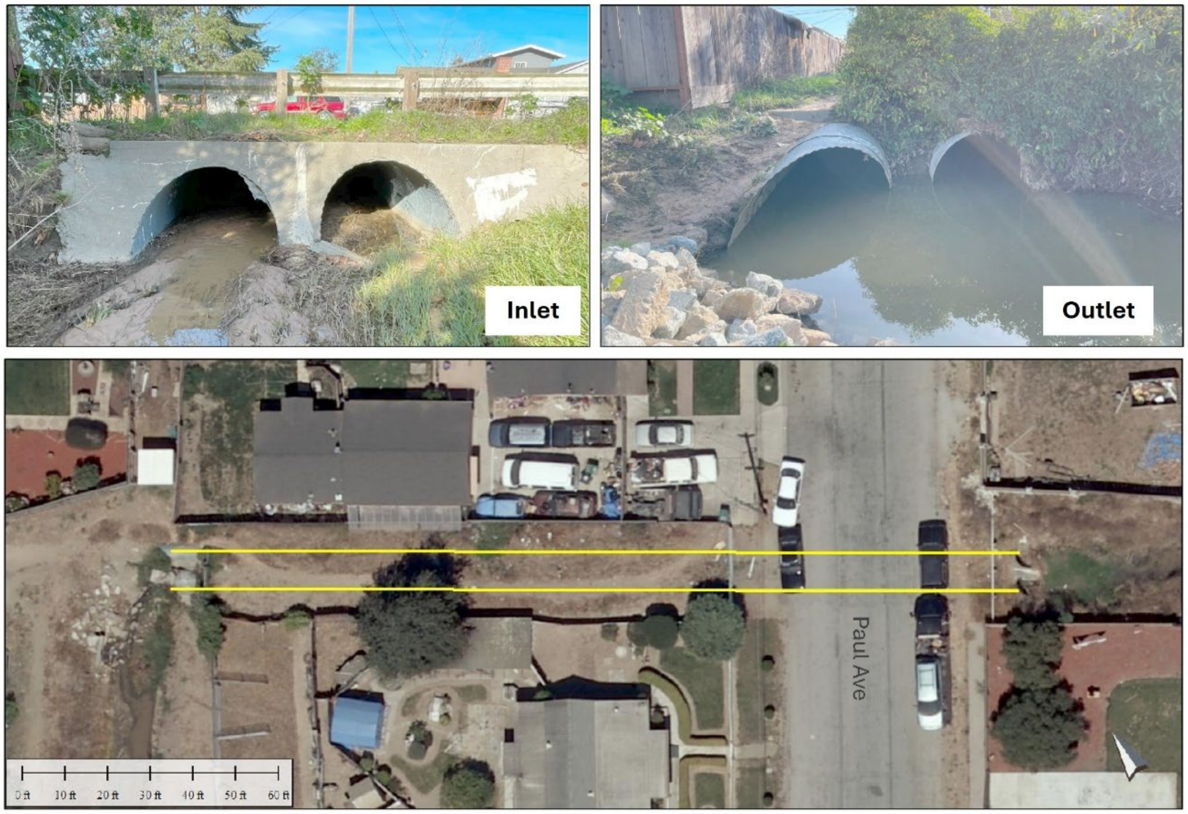

Figure 4-1 Photographs and plan view location of existing culverts at Paul Avenue

A conveyance enhancement project element would replace the existing CMPs with twin by 8-foot-wide by 5-foot-high reinforced concrete box culverts (RCBs). While these are smaller than those at Cornwall Street and Russell Road, they would be sized to fit within the available 25-foot easement. Should subsequent detailed engineering analyses demonstrate that twin 10-foot by 5-foot RCBs can be accommodated within the easement, that larger size would be preferable to further improve capacity. The invert elevations of the new culverts would need to be lowered below existing grade to maintain the necessary cover, as shown in the culvert section views included as **Figure 4-2**.

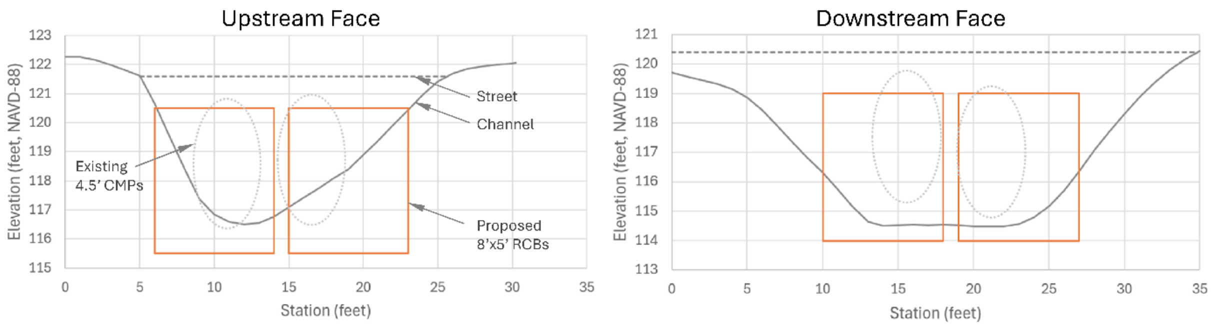

Figure 4-2 Proposed culvert sections at Paul Avenue

The Rogge Road crossing also comprises twin 4.5-foot CMPs, spanning 76 feet across the right-of-way and potentially extending into 25-foot drainage easements on either side. Photographic documentation and a plan view of the alignment are presented in **Figure 4-3**. As with Paul Avenue, ground cover above the culverts is limited, presenting a vertical constraint for any replacement design.

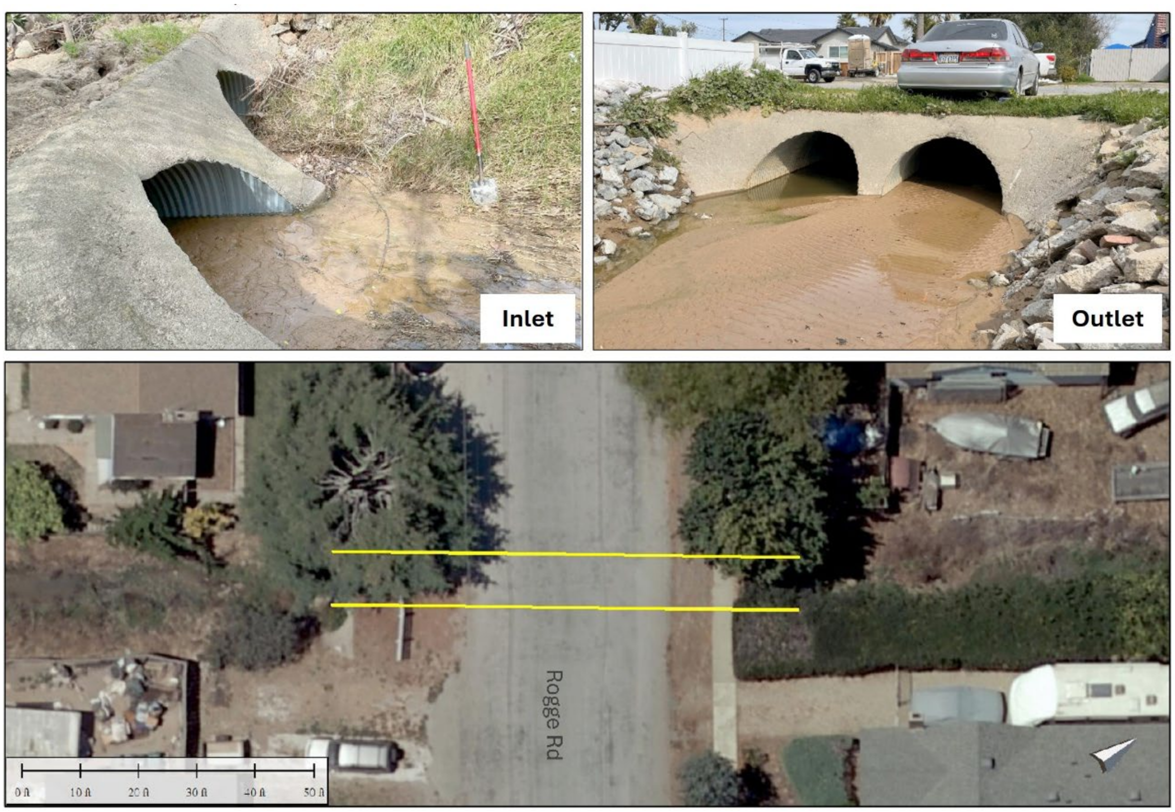

Figure 4-3 Photographs and plan view of existing culverts at Rogge Road

Similar to the proposed Paul Avenue upgrade, the proposed improvement at Rogge Road would involve replacing the existing CMPs with twin 8-foot span by 5-foot rise RCBs. These will also require installation below existing grade to ensure sufficient ground cover, as illustrated in the culvert section drawings included as **Figure 4-4**.

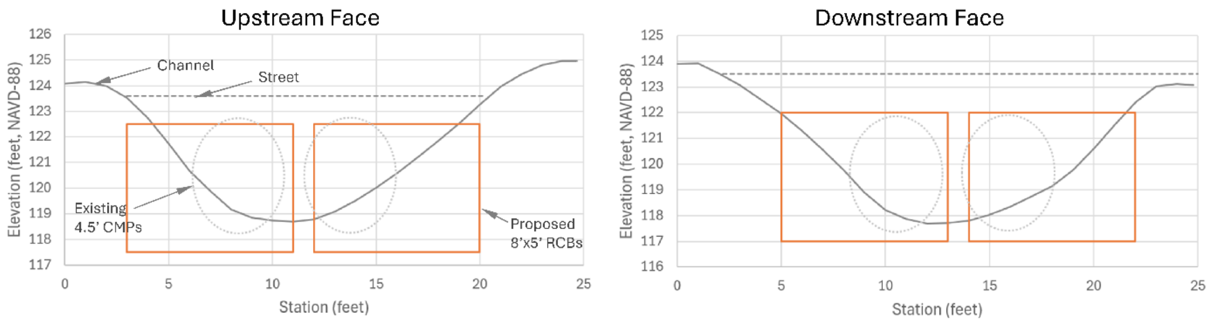

Figure 4-4 Proposed culvert sections at Rogge Road

Improvements at the Paul Avenue and Rogge Road crossings would entail a standard infrastructure improvement project. Costs would largely be driven by the purchase and installation of the RCBs, construction of concrete headwalls, and refinishing surfaces above the culverts with total construction costs for both crossings estimated in the $500,000 to $1,000,000 range.

4.4 Sediment Removal at Crossings

Sediment deposition has been most pronounced, and most consequential in terms of flood risk, at culverted crossing locations. Field observations indicate minimal sediment accumulation at the Russell Road crossing; therefore, no action is currently recommended at this site. In contrast, significant sediment buildup is present at the Paul Avenue and Rogge Road crossings, limiting conveyance capacity. However, if the aforementioned culvert replacement projects proceed at these locations, sediment removal would be completed as part of the initial improvements.

Upstream of Rogge Road, several culverts along private access roads show partial sediment infill. These crossings are assumed to be the maintenance responsibility of the respective property owners.

At Cornwall Street, the County has conducted recurring sediment removal, and this location remains a candidate for near-term maintenance. Current sediment conditions at Cornwall Street are estimated to reduce conveyance capacity to approximately 420 cfs before overtopping onto the roadway occurs. However, even at lower flows there is a backwater created at the crossing that impacts the more constrained channel reach along and upstream of Ferrasci Park. Targeted sediment removal at this location would improve conveyance and reduce overtopping risk.

It is also important to note that if additional sediment management practices are not implemented upstream of the study reach (i.e. upstream of Bolsa Knolls), an ongoing need for sediment removal at downstream channel crossings should be anticipated.

County staff indicated that past sediment removal projects focused on a single crossing location can cost up to $50,000 to implement.

4.5 Targeted Vegetation Management

Field observations and aerial imagery have identified approximately 1,100 feet of channel, distributed across three reaches, with relatively dense vegetation growth (**Figure 4-5**). These areas have been parameterized in the hydraulic model with elevated Manning's 'n' values ranging from 0.055 to 0.06, reflecting increased surface roughness that reduces conveyance capacity and contributes to localized overtopping.

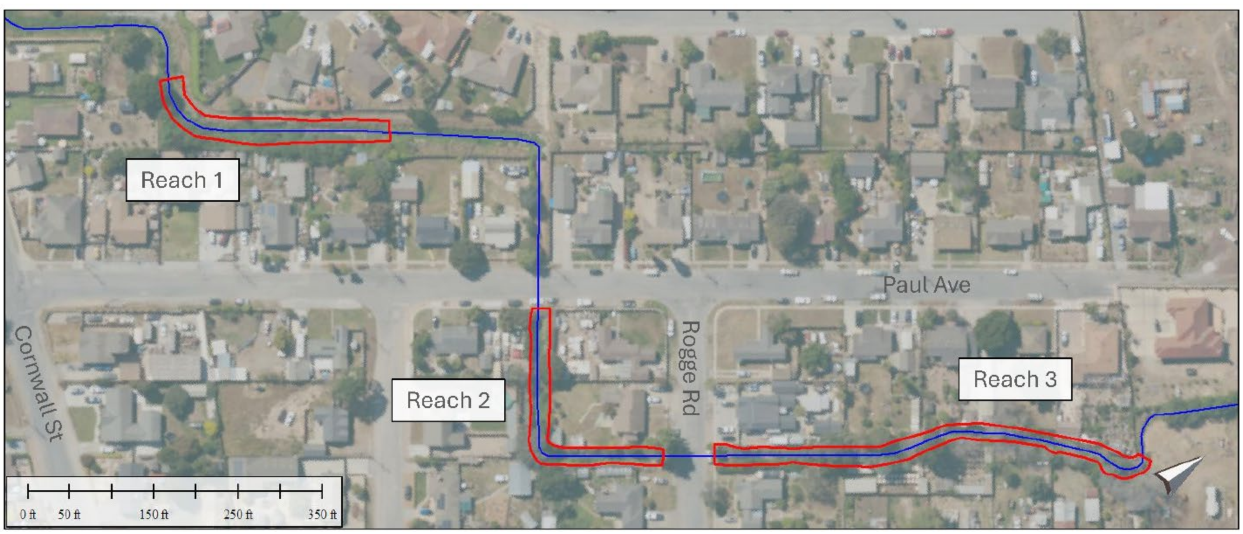

Figure 4-5 Targeted reaches for vegetation management

The proposed vegetation management strategy would involve selective thinning rather than complete clearing. Vegetation that impedes flow would be removed to improve channel conveyance while maintaining riparian habitat function. Current vegetation conditions are illustrated in **Figure 4-6**, while **Figure 4-7** provides representative photographs from the reach downstream of Cornwall Street, illustrating the intended outcomes of vegetation thinning.

Figure 4-6 Existing vegetation within targeted reaches

Costs for a crew to thin the vegetation would be relatively small, in the $50,000 to $150,000 range depending on the number of sites and extent of the work involved. Soft costs such as permitting the project and coordinating with the multiple private property owners that would be impacted by the project could potentially significantly increase the costs of this project element.

Figure 4-7 Examples of targeted outcomes from vegetation management activities

4.6 Upstream Sediment Retention

Sediment loading to the study reach has been a persistent and significant issue, as evidenced by the County's recurring sediment removal operations at channel crossings and by field observations during and after the February 13, 2025, storm event. Comparative observations of sediment accumulation before and after the event are presented in **Figure 3-3** and **Figure 3-4**, while additional photographs illustrating turbidity and sediment deposits are included in **Figure 4-8**

Figure 4-8 High turbidity and sediment deposits observed during the February 13, 2025, storm event

The primary source of sediment is almost certainly agricultural lands upstream in the watershed, particularly fields that are bare or situated on steeper slopes. An example of observed erosion from one such field, likely caused by the February 2025 event, is shown in **Figure 4-9**.

Figure 4-9 Example of a sediment source in the upper watershed

Several existing basins have been identified across the upper watershed, many of which appear to have been designed for the purpose of sediment retention. However, their performance is inconsistent and limited by factors such as storage capacity relative to inflow volumes, the configuration of outlets, and the maintenance frequency. Field evidence from the February 2025 storm event suggests that these facilities are insufficient to mitigate sediment transport to the study reach, and that additional retention volume would be needed to reduce downstream impacts.

Additional sediment retention could be achieved either through distributed retention facilities, with smaller basins located at individual agricultural operations, or through one or more regional facilities. While a single, consolidated basin would typically offer greater sediment capture efficiency and ease of maintenance, it would require a higher initial investment from the County. A hybrid approach, combining a large basin along the creek with distributed basins throughout the upper watershed, may be the most effective solution, given the likely infeasibility of achieving optimal sediment control through a single strategy.

There are several potential locations for a regional basin along Santa Rita Creek between the study reach and Hebert Road. However, selecting a basin as close as possible to the study reach would maximize the contributing drainage area and improve sediment capture efficiency. A basin located just upstream of the study reach and downstream of the golf course would have the added benefit of capturing sediment from Sub-watersheds K and L.

To illustrate the scale and function of a potential sediment retention facility, a conceptual basin was simulated immediately upstream of the study reach. This concept, shown in **Figure 4-10**, includes 8 acre-feet of sediment retention storage (also known as "dead pool" storage) and an additional 13 acre-feet of active flow control storage with an outlet consisting of triple 36-inch reinforced concrete pipes (RCPs), for a total storage volume of 21 acre-feet. The total footprint of this conceptual sediment basin would be 3.4 acres. The total earthwork volume to construct the facility would be approximately 35,000 cubic yards.

The cost of implementing a sediment retention basin project similar to the example provided would vary significantly depending on several factors, including site location, property acquisition, project scale, whether storage volume can be achieved through berm construction versus excavation, and also environmental mitigation requirements. However, initial capital costs are expected to be in the millions of dollars. In addition to these up-front expenses, ongoing maintenance costs for periodic sediment removal would also need to be factored into the long-term operations budget.

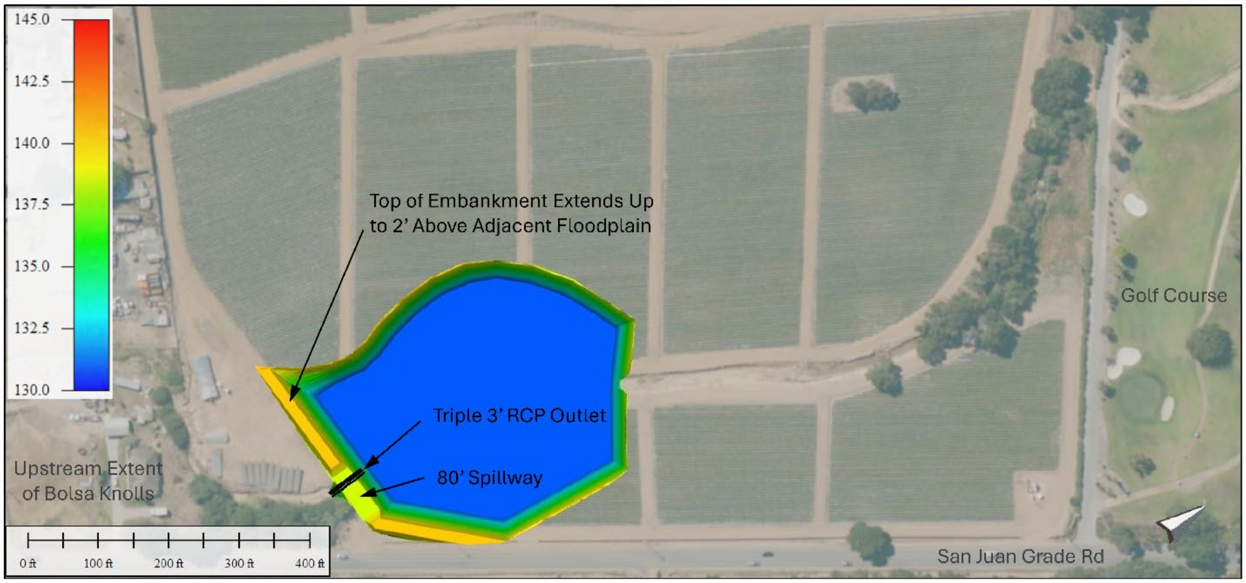

Figure 4-10 Conceptual sediment retention basin

4.7 Upstream Detention

While the previously described project elements would minimize localized constrictions along the study reach and conveyance deficiencies, they are insufficient on their own to eliminate flooding from large storm events. Reducing peak flows entering the study reach through upstream detention remains the most cost-effective strategy for providing broader flood protection benefits.

Similar to the sediment retention basin concept, detention volume could potentially be accommodated at various locations along Santa Rita Creek upstream of the study reach, extending as far as Herbert Road. Additional opportunities may also exist for smaller, distributed detention basins along tributary channels draining individual sub-watersheds such as G, J, K, and L.

To illustrate the potential range of benefits and scale, two detention basin concepts were simulated for a site located immediately upstream of the study reach. These concepts include total storage volumes of 35 and 90 acre-feet and serve as examples of how detention capacity could be implemented.

The smaller basin concept, shown in **Figure 4-11**, includes 5 acre-feet of sediment retention storage and 30 acre-feet of active flow control detention storage, for a total capacity of 35 acre-feet. The basin would be drained through a 6-foot wide, by 4-foot-tall reinforced concrete box (RCB) culvert. The basin footprint would be approximately 6.9 acres, with an estimated 51,000 cubic yards of earthwork required for construction.

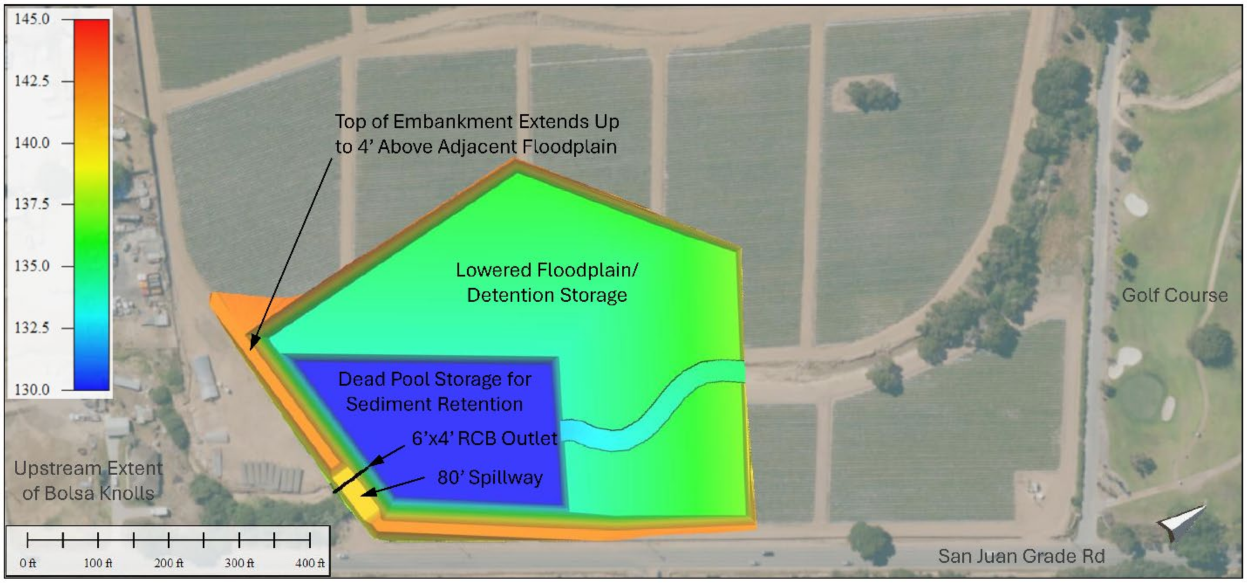

Figure 4-11 Conceptual 35-acre-foot detention basin

The larger basin concept, shown in **Figure 4-12**, would include 10 acre-feet of sediment retention storage and 80 acre-feet of active flow control detention storage, drained through a 3-foot wide by 4-foot tall RCB culvert. The total storage capacity would be 90 acre-feet, with a basin footprint of approximately 19.6 acres. Construction would require an earthwork volume of roughly 260,000 cubic yards. Notably, detention facilities exceeding 6 feet in height and 50 acre-feet in volume fall under the jurisdiction of the California Division of Safety of Dams (DSOD). This may make it preferable to distribute detention capacity at this scale across multiple smaller facilities to simplify implementation and permitting requirements.

As with sediment retention basin projects, the capital cost for detention facilities will vary significantly based on site location, land acquisition costs, project scale, whether storage can be achieved primarily through berm construction versus excavation, and the overall environmental mitigation requirements. However, preliminary capital costs are generally expected to range from several million to tens of millions of dollars. As with sediment retention basins, long-term operation and maintenance costs would have to be incorporated into overall project planning and operations budgeting.

4.8 Project Alternatives

Four project alternatives were developed by combining previously described project elements. These alternatives were evaluated using the hydraulic model prepared for this study to allow a comparative analysis of their respective flood mitigation benefits and associated costs.

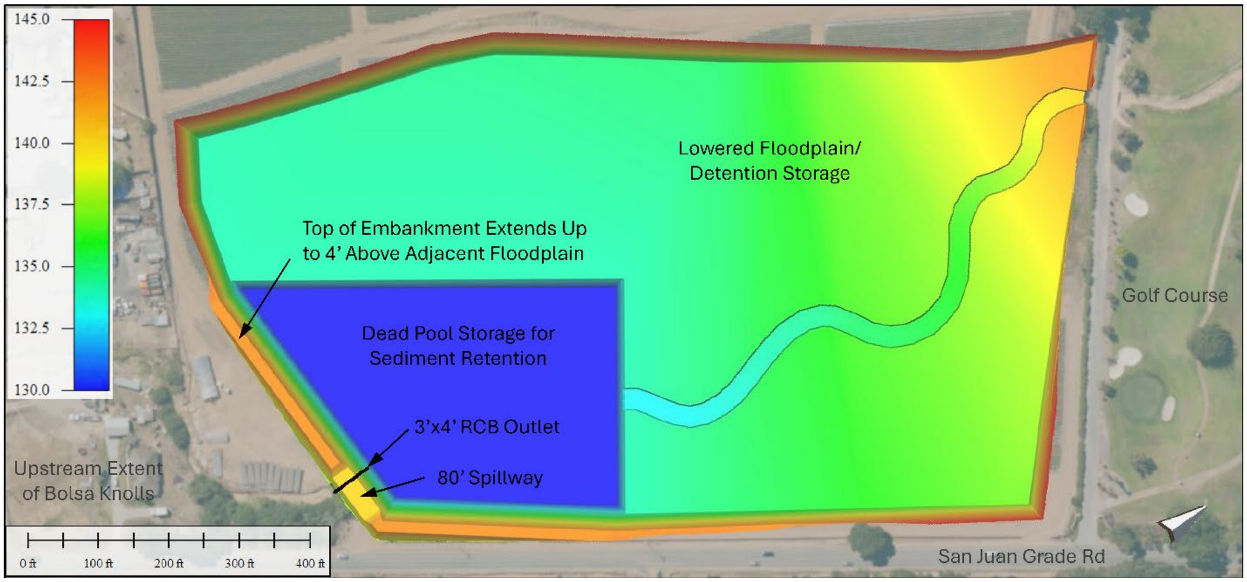

Figure 4-12 Conceptual 90-acre-foot detention basin

Alternative 1 includes all of the focused conveyance project elements paired with a single sediment retention basin project. Specific elements in this alternative include:

- Replacement of undersized culverts at the Paul Avenue and Rogge Road crossings.
- Sediment removal at the Cornwall Street crossing.
- Targeted vegetation thinning along 1,100 feet of channel.
- Construction of a 21-acre-foot sediment retention basin immediately upstream of the study reach.

Alternative 2 isolates conveyance improvements by excluding sediment management components and helps assess the consequences of not addressing upstream sediment loading. Specific components include:

- Replacement of culverts at Paul Avenue and Rogge Road, but with the culverts partially obstructed by additional sediment accumulations.
- Targeted vegetation thinning along 1,100 feet of channel.

Alternative 3 builds on Alternative 1 by including a larger dual-purpose basin that provides both sediment retention and peak flow attenuation rather than a smaller sediment-retention focused facility. Specific components include:

- Replacement of culverts at Paul Avenue and Rogge Road.
- Sediment removal at the Cornwall Street crossing.
- Targeted vegetation thinning along 1,100 feet of channel.
- Construction of a 35-acre-foot sediment and detention basin upstream of the study reach.

Alternative 4 is identical to Alternative 3, except it includes a significantly larger 90-acre-foot basin at the upstream end of the study reach. Specific elements in this alternative include:

- Replacement of culverts at Paul Avenue and Rogge Road.
- Sediment removal at the Cornwall Street crossing.
- Targeted vegetation thinning along 1,100 feet of channel.
- Construction of a 90-acre-foot sediment and detention basin upstream of the study reach.

4.9 Permitting Considerations

A discussion of the anticipated regulatory permitting requirements for the project is included as **Appendix E**. Permits will range with federal and State resource agencies, and will have their own challenges.

5 PROJECT ALTERNATIVES MODELING

5.1 Purpose

Hydraulic modeling of the project alternatives was completed to:

- Inform and optimize the configuration of project elements in order to maximize flood control benefits.
- Provide a quantitative estimate of flood reduction performance to support comparison with the previously identified costs and implementation considerations.

5.2 Approach

The existing conditions, hydraulic model was adapted to simulate each of the four project alternatives with modifications made to reflect the improvements associated with Alternatives 1 through 4. Simulations were conducted for two calibration events, the February 13, 2025, and December 27, 2022, flood events, as well as for the 10- and 100-year design storms. The February 20, 2017, flood was not modeled, as its hydraulic characteristics were found to be similar to those of the December 2022 event.

5.3 Alternative 1

To simulate Alternative 1, the model was updated to include the twin 8-foot by 5-foot RCBs within the Storage/2D Area Connections defining the Paul Avenue and Rogge Road crossings. Culverts were filled to depths of 0.8 and 0.6 feet at the Paul Avenue and Rogge Road crossings respectively to account for the culverts being placed below existing channel invert elevations. Associated culvert parameters including Manning's 'n' values, entrance loss coefficients, and FHWA Chart Numbers were revised to reflect the proposed configurations.

Targeted vegetation thinning was represented by reducing Manning's 'n' values along the affected reaches from a range of 0.055 to 0.06 to a uniform value of 0.045.

Sediment removal at the Cornwall Street crossing was accounted for by setting sediment depths in the culverts to zero.

The grading plan developed for the 21-acre-foot sediment retention basin was used to update the model terrain, and breaklines were adjusted to ensure topographic features and flow paths were accurately captured. Basin outflows were modeled using a Storage/2D Area Connection with triple 36-inch RCPs, sized to convey flows up to 130 cfs (the peak flow from the February 2025 event at this location). A spillway was included to pass the full 100-year flow with one foot of freeboard. The dead pool volume of the conceptual basin was filled at the start of each simulation to an elevation matching the culvert invert.

5.4 Alternative 2

Alternative 2 incorporates the same culvert and vegetation improvements as Alternative 1. However, an additional 1-foot of sediment was assumed in the Paul Avenue and Rogge Road culverts to reflect the absence of upstream sediment controls.

5.5 Alternative 3

The Alternative 3 simulation was parameterized consistent with Alternative 1, but terrain and Storage/2D Area connection elements were modified to reflect a 35-acre-foot basin. The basin outlet was modeled as a 6-foot by 4-foot RCB sized to convey the 10-year peak flow. Higher flows were routed over a spillway designed to pass the 100-year event with one foot of freeboard.

5.6 Alternative 4

Alternative 4 builds on Alternative 3 but was modified to reflect a 90-acre-foot basin. The outlet was modeled using a 3-foot by 4-foot RCB designed to pass the 10-year peak flow, with overflow directed to a spillway capable of passing the 100-year event with one foot of freeboard. While the increased storage provides opportunities to reconfigure the outlet to better control 100-year flows, doing so would reduce benefits for more frequent, lower-magnitude events (10-year magnitude and smaller).

5.7 Results

Model output in the form of depth, water surface elevation, and change in water surface elevation plots for the 10- and 100-year design storm events and the February 20, 2017; December 27, 2022; and February 13, 2025, historical storm events are included as **Appendix F**. A high-level summary of these results is provided through comparisons of modeled inundation area and number of impacted structures included as **Figure 5-1** and **Figure 5-2**. A detailed discussion of results for each project alternative is included below.

Table 5-1 Modeled inundation areas over the study reach across project and flood event scenarios.

Flood Event	Inundated Area				
	Existing	Alt 1	Alt 2	Alt 3	Alt 4
-	*acres*	*acres*	*acres*	*acres*	*acres*
100 year	44	43	43	40	35
10 year	34	31	33	23	14
December 27, 2022	20	11	16	8	6
February 13, 2025	15	7	9	7	5

Table 5-2 Number of impacted residential structures over the study reach across project and flood event scenarios

Flood Event	Impacted Structures				
	Existing	Alt 1	Alt 2	Alt 3	Alt 4
-	-	-	-	-	-
100 year	81	81	81	78	70
10 year	61	58	61	49	33
December 27, 2022	43	20	41	8	4
February 13, 2025	30	7	12	6	4

5.8 Alternative 1

As discussed previously, Alternative 1 incorporates a set of targeted conveyance improvements designed to address localized flooding along the study reach. These improvements were configured to focus flood impact benefits on smaller, more frequent storm events, while their effectiveness diminishes for larger events such as the 10- and 100-year design storms. The inclusion of a sediment retention basin, although not specifically designed to reduce peak flows, results in a decrease in peak flow rates at the upstream end of the study reach by approximately 3 to 17 percent across the range of storm events evaluated but again noting that the largest relative peak flow reductions are associated with smaller storm events.

Model results indicate that Alternative 1 would eliminate overtopping during storms similar to the February 13, 2025, flood event along the channel reach through the medium-density residential area. Channel overtopping is still predicted in the low-density residential area at the upstream end of the study reach with excess flood flows conveyed across the overbank area toward Rogge Road. These outcomes are illustrated in **Figure 5-1**, which compares modeled flood depths for Alternative 1 with the existing conditions inundation extents for the February 2025 event. Additionally, **Figure 5-2** shows the corresponding change in water surface elevations, with reductions of up to 3.1 feet predicted upstream of the Paul Avenue crossing.

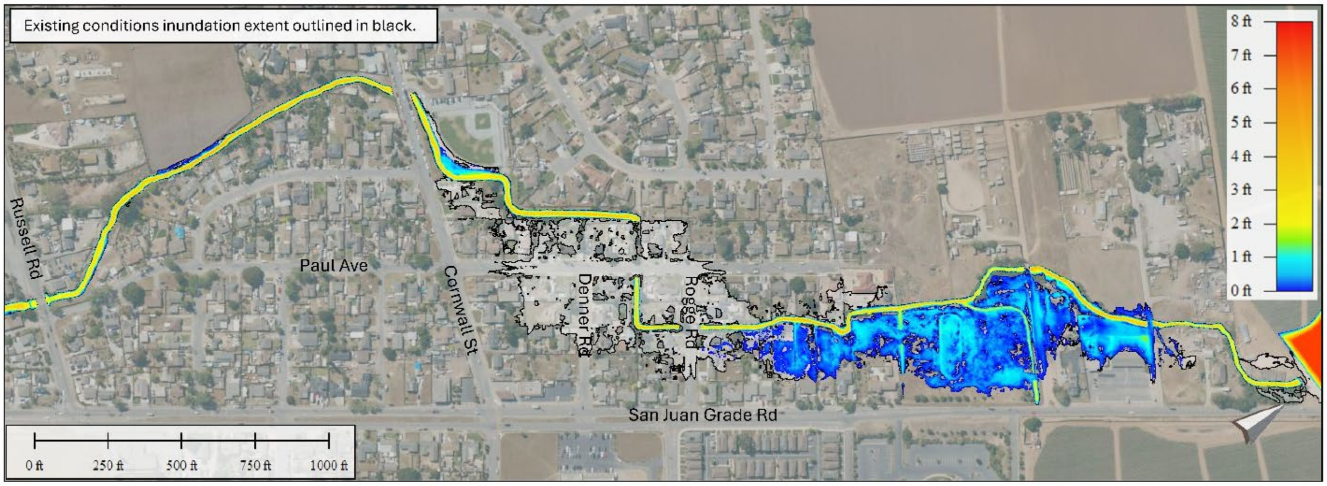

Figure 5-1 Modeled Alternative 1 flood depths compared to existing conditions inundation extents for the February 13, 2025, flood event

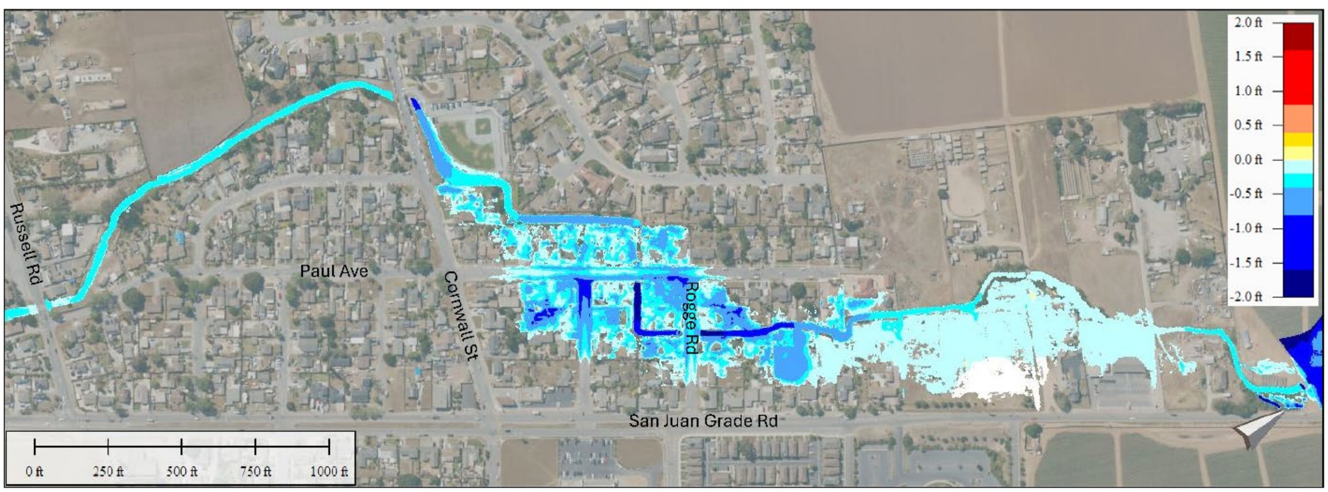

Figure 5-2 Modeled change in peak water surface elevations resulting from Alternative 1 during the February 13, 2025, flood event. Note shades of blue depict lowered water surface elevations and shades of red increased water surface elevations.

For the December 27, 2022, flood event, the Alternative 1 modeling predicts significantly reduced, but not entirely eliminated, overtopping along the upper segment of the study reach. Downstream from Cornwall Street, however, channel overtopping would be eliminated as shown on **Figure 5-3**. Modeled water surface elevations are substantially lowered throughout the reach as shown on **Figure 5-4.**, with reductions of up to 2.8 feet predicted upstream of the Paul Avenue crossing.

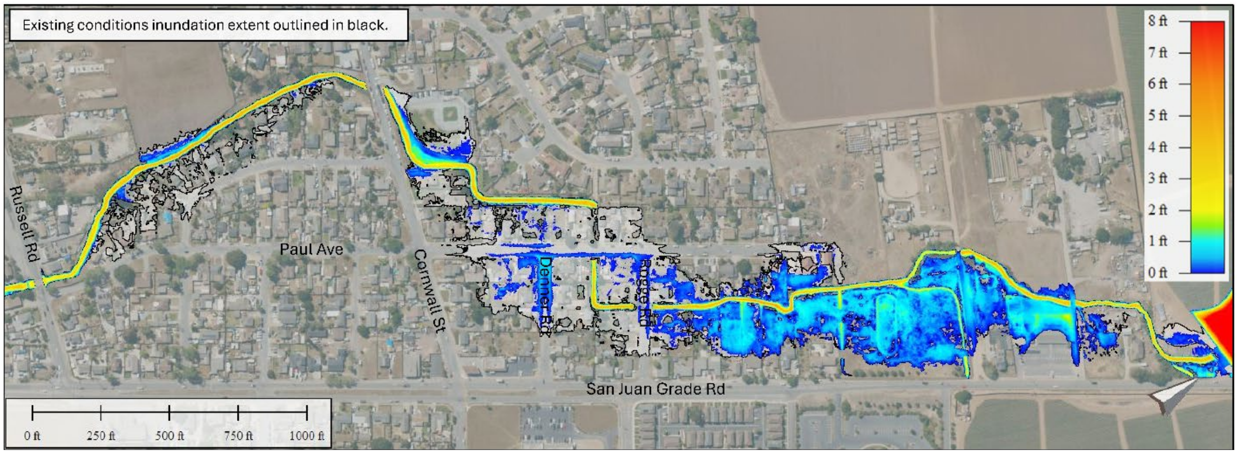

Figure 5-3 Modeled Alternative 1 flood depths compared to existing conditions inundation extents for the December 22, 2022, flood event

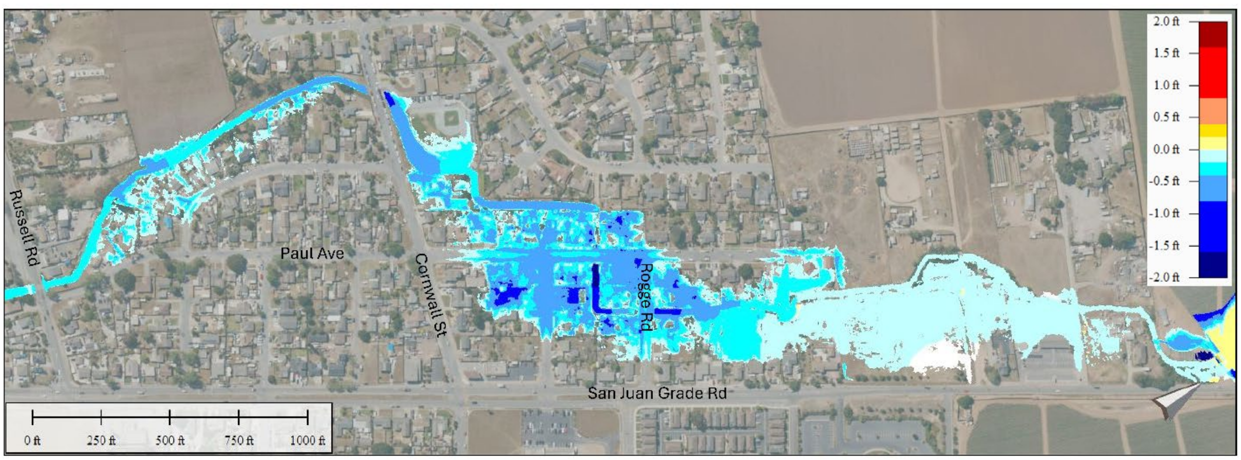

Figure 5-4 Modeled change in peak water surface elevations resulting from Alternative 1 during the December 22, 2022, flood event

For larger, less frequent events such as the 10- and 100-year design storms, flood reduction benefits would be more modest. Water surface elevations would be reduced by several tenths of a foot, as illustrated in **Figure 5-5** and **Figure 5-6**. Importantly, no adverse downstream impacts are predicted beyond Russell Road for any of the modeled scenarios (i.e. the modeling predicts no increase in peak flow rate or inundation area down to Highway 101).

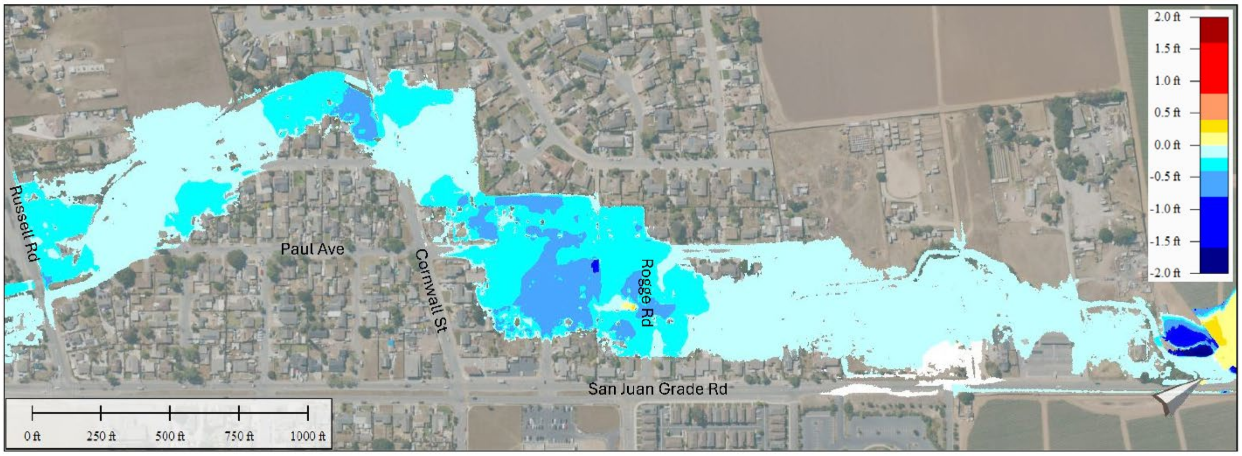

Figure 5-5 Modeled change in peak water surface elevations resulting from Alternative 1 during the 10-year flood event

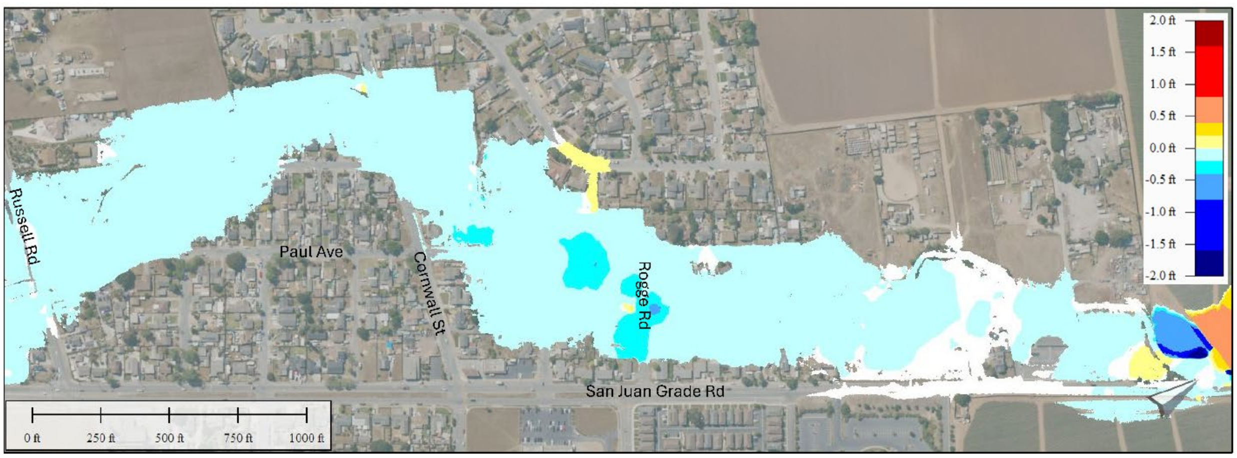

Figure 5-6 Modeled change in peak water surface elevations resulting from Alternative 1 during the 100-year flood event

5.9 Alternative 2

Modeling of the removal of sediment management elements as per Alternative 2 results in reduced predicted flood control benefits compared to those predicted with Alternative 1. Nevertheless, modeling of the February 13, 2025, flood event shows that Alternative 2 could still substantially reduce overtopping through the medium-density residential area, with only minor flooding predicted, as illustrated in **Figure 5-7**. Similar reductions in performance relative to Alternative 1 are observed for the December 2022 event as well as the 10- and 100-year design storms.

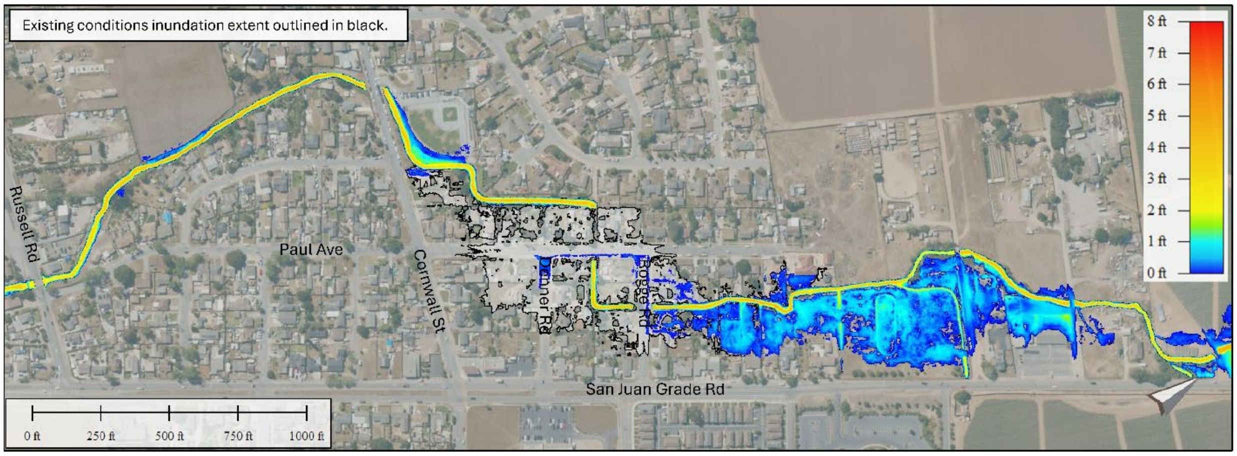

Figure 5-7 Modeled Alternative 2 flood depths compared to existing conditions inundation extents for the February 13, 2025, flood event

Unlike Alternative 1, Alternative 2 is predicted to cause slight increases in water surface elevations, up to 0.09 feet, between Paul Avenue and Highway 101 during the February 2025 flood event. These increases are shown in the change in water surface elevation plot included as **Figure 5-8**. Similar effects are predicted for the December 2022 event. These increases can be attributed to less runoff stored in the reduced floodplain inundation area within the study reach, which would otherwise contribute to peak flow attenuation. While this effect also occurs with Alternative 1, the modeling indicates it would be offset by the attenuation benefits provided by the sediment retention basin in that alternative.

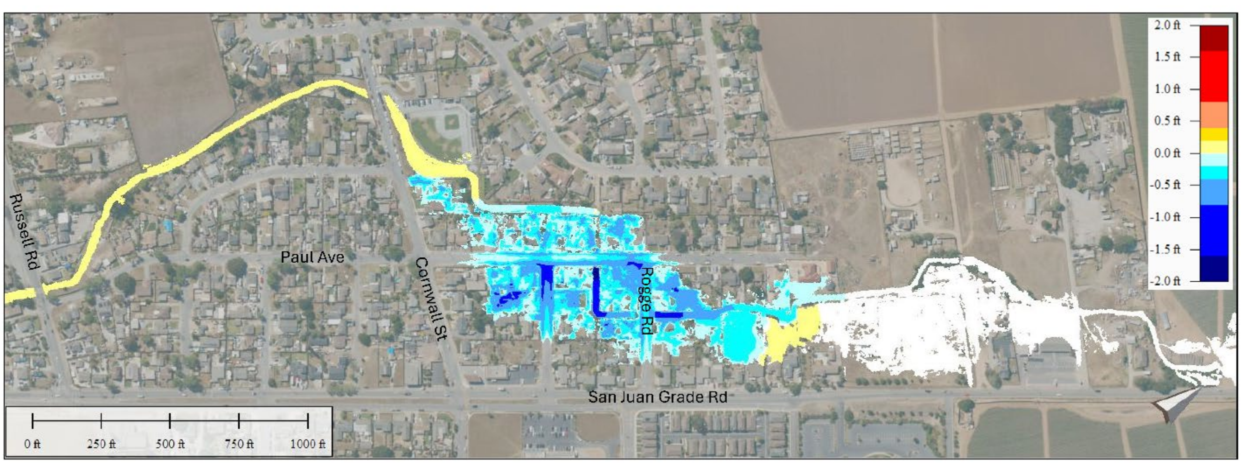

Figure 5-8 Modeled change in peak water surface elevations resulting from Alternative 2 during the February 13, 2025, flood event

5.10 Alternative 3

The 35-acre-foot basin included with Alternative 3 was configured in the modeling to lower peak flow rates at the upstream end of the study reach by 27, 27, 35, and 22 percent for the February 13, 2025; December 27, 2022; and 10- and 100-year design storm events respectively. These reductions result in enhanced flood mitigation benefits throughout the study reach, including for the larger 10- and 100-year events. The associated decreases in water surface elevations are illustrated in **Figure 5-9** and **Figure 5-10**. Importantly, the model predicts no adverse downstream flood impacts beyond the study reach under any of the flood scenarios evaluated.

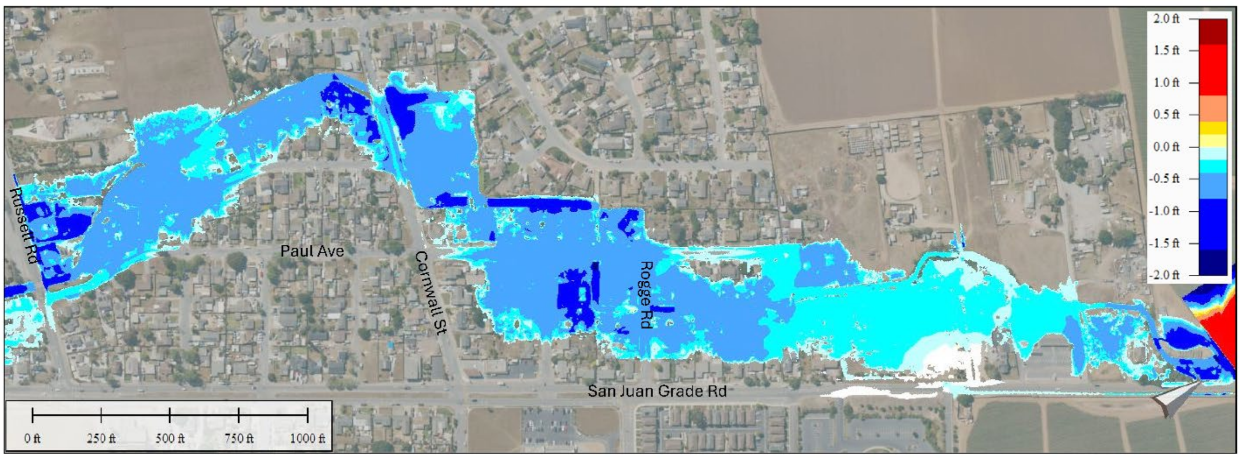

Figure 5-9 Modeled change in peak water surface elevations resulting from Alternative 3 during the 10-year flood event

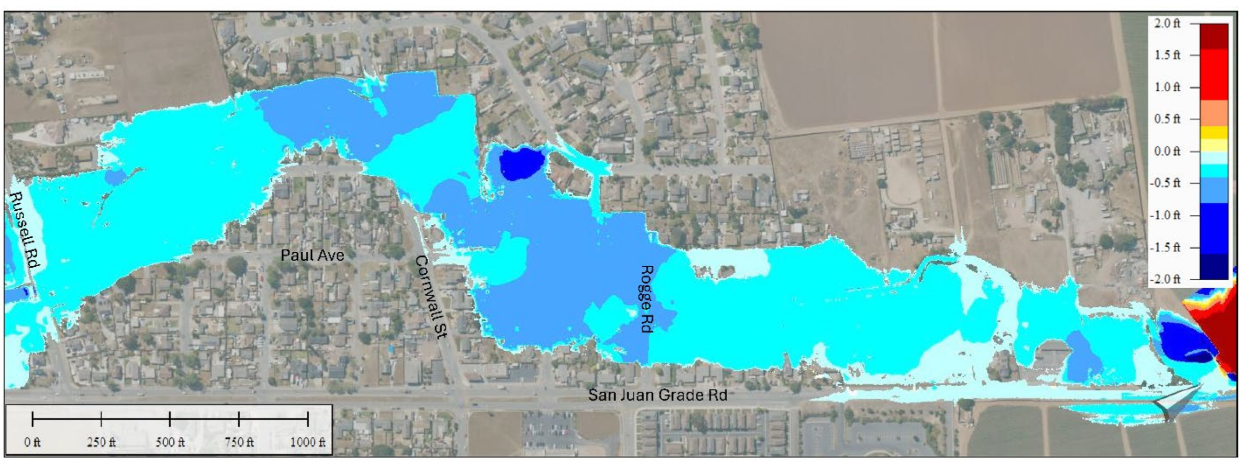

Figure 5-10 Modeled change in peak water surface elevations resulting from Alternative 3 during the 100-year flood event

5.11 Alternative 4

The 90-acre-foot basin included with Alternative 3 is modeled to lower peak flow rates at the upstream end of the study reach by 44, 66, and 65 percent for the February 13, 2025; December 27, 2022; and 10- and 100-year design storm events respectively. These additional reductions in peak flow rates are predicted to be large enough to eliminate channel overtopping through the medium-density residential portion of the study reach for the February 2025 and December 2022 events and significantly reduce the modeled inundation extents during the 10-year flood event as illustrated by **Figure 5-11**. Change in water surface elevation plots for the 10- and 100-year flood events are included as **Figure 5-12** and **Figure 5-13**.

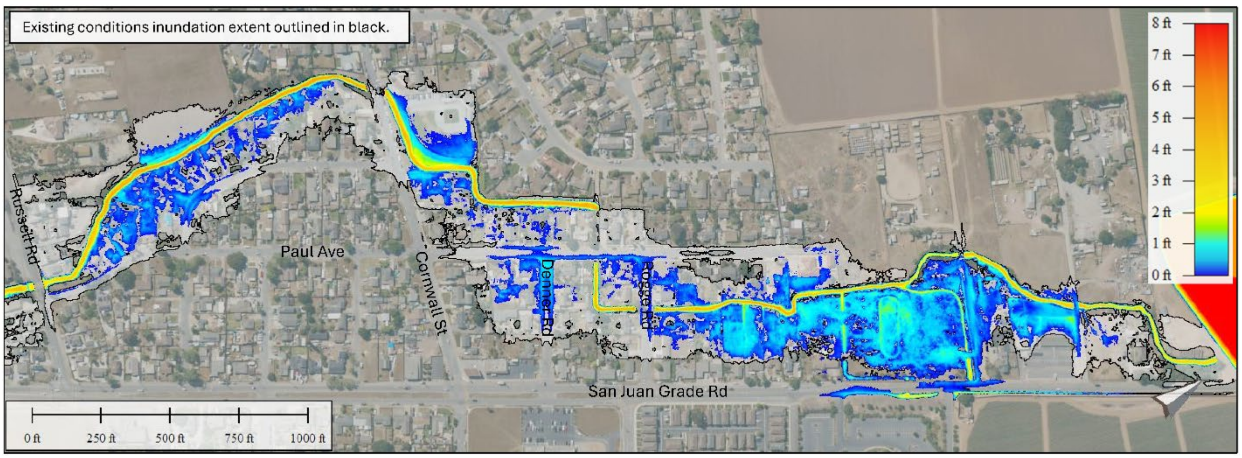

Figure 5-11 Modeled Alternative 4 flood depths compared to existing conditions inundation extents for the 10-year flood event

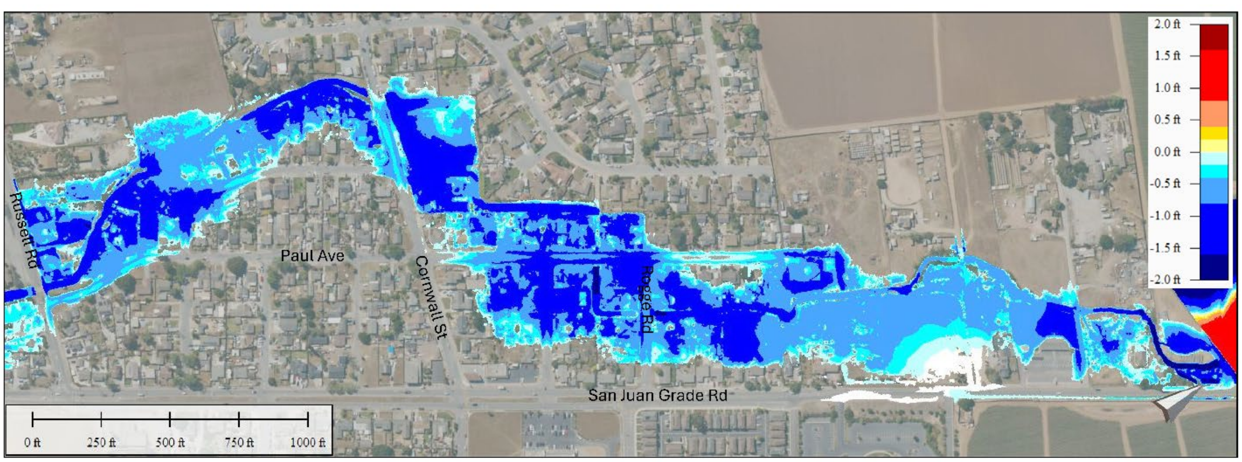

Figure 5-12 Modeled change in peak water surface elevations resulting from Alternative 4 during the 10-year flood event

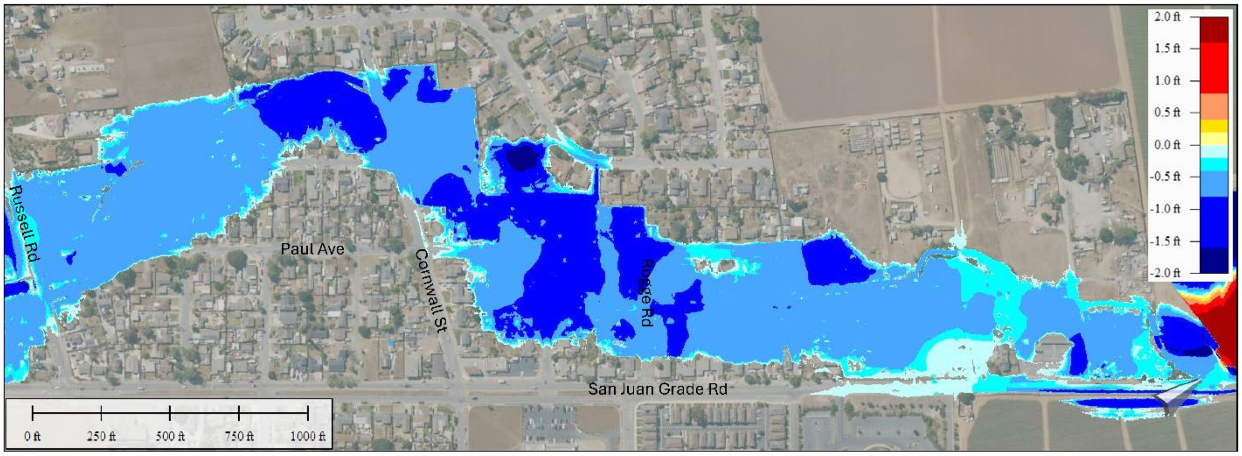

Figure 5-13 Modeled change in peak water surface elevations resulting from Alternative 4 during the 100-year flood event

6 CONCLUSIONS

This Flood Study of Santa Rita Creek provides a comprehensive analysis of flood hazards affecting the Bolsa Knolls community and evaluates a range of potential improvements to reduce flood risk. Modeling results confirm that even relatively small storm events frequently exceed the limited conveyance capacity of the channel, resulting in widespread overbank flooding. Contributing factors include undersized culverts, sediment deposition, pockets of dense in-channel vegetation, and development of the adjacent floodplain.

The study identifies a set of implementable and scalable project elements including culvert replacements, sediment removal, vegetation thinning, and construction of upstream basins that can be combined in different configurations to reduce flood impacts. Four project alternatives were developed to represent a range of strategies varying in scope and cost.

Hydraulic modeling of these alternatives demonstrates that targeted conveyance improvements, such as culvert replacements and vegetation thinning, provide meaningful benefits during smaller, more frequent storm events, while upstream detention basins are essential to mitigate flooding from larger storms.

The results of this study provide a data-driven foundation for selecting a preferred project alternative and inform future planning, design, and permitting. While implementation of any alternative will involve trade-offs in terms of cost, permitting, and feasibility, the study clearly shows that a combination of localized conveyance improvements and upstream storage provides the most effective strategy to reduce flood risk in Bolsa Knolls.

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APPENDICES

APPENDIX A

Hydrologic Model Input Parameters

Hydrologic Model Input: Area, Rainfall Depth, Curve Number, and Initial Abstraction Parameters

		Rainfall Depths		Curve Numbers					Initial Abstractions				
Watershed	**Area**	**100yr 24hr**	**10yr 24hr**	**AMC 1.0**	**AMC 1.5**	**AMC 2.0**	**AMC 2.5**	**AMC 3.0**	**AMC 1.0**	**AMC 1.5**	**AMC 2.0**	**AMC 2.5**	**AMC 3.0**
-	*acres*	*in*	*in*	-	-	-	-	-	-	-	-	-	-
A	251	4.42	2.76	65	71	77	81	86	0.26	0.20	0.15	0.11	0.08
B	82	4.44	2.78	62	71	79	84	89	0.31	0.20	0.13	0.09	0.06
C	194	4.48	2.80	61	70	78	83	89	0.32	0.21	0.14	0.10	0.06
D	138	4.48	2.80	61	66	72	77	83	0.32	0.25	0.19	0.15	0.10
E	71	4.58	2.88	62	69	76	82	87	0.30	0.22	0.15	0.11	0.07
F	43	4.57	2.86	54	61	69	75	81	0.43	0.32	0.23	0.17	0.12
G	203	4.78	2.99	62	71	79	84	89	0.31	0.21	0.13	0.09	0.06
H	68	4.61	2.88	54	62	69	76	82	0.42	0.31	0.22	0.16	0.11
I	99	4.63	2.90	62	70	77	83	88	0.30	0.21	0.15	0.11	0.07
J	268	4.78	3.00	63	63	80	85	90	0.29	0.29	0.13	0.09	0.06
K	94	4.81	3.02	61	61	78	84	89	0.32	0.32	0.14	0.10	0.06
L	183	5.15	3.24	57	66	75	81	87	0.37	0.25	0.17	0.12	0.08
M	53	4.76	2.99	55	64	72	78	84	0.41	0.28	0.19	0.14	0.09
N	200	4.92	3.08	53	62	70	77	83	0.45	0.31	0.21	0.15	0.10
O	333	5.17	3.25	64	73	80	86	90	0.28	0.19	0.12	0.08	0.05
P	334	5.46	3.44	65	74	81	86	91	0.26	0.18	0.12	0.08	0.05
Q	441	5.45	3.43	51	60	68	75	81	0.48	0.33	0.23	0.17	0.12

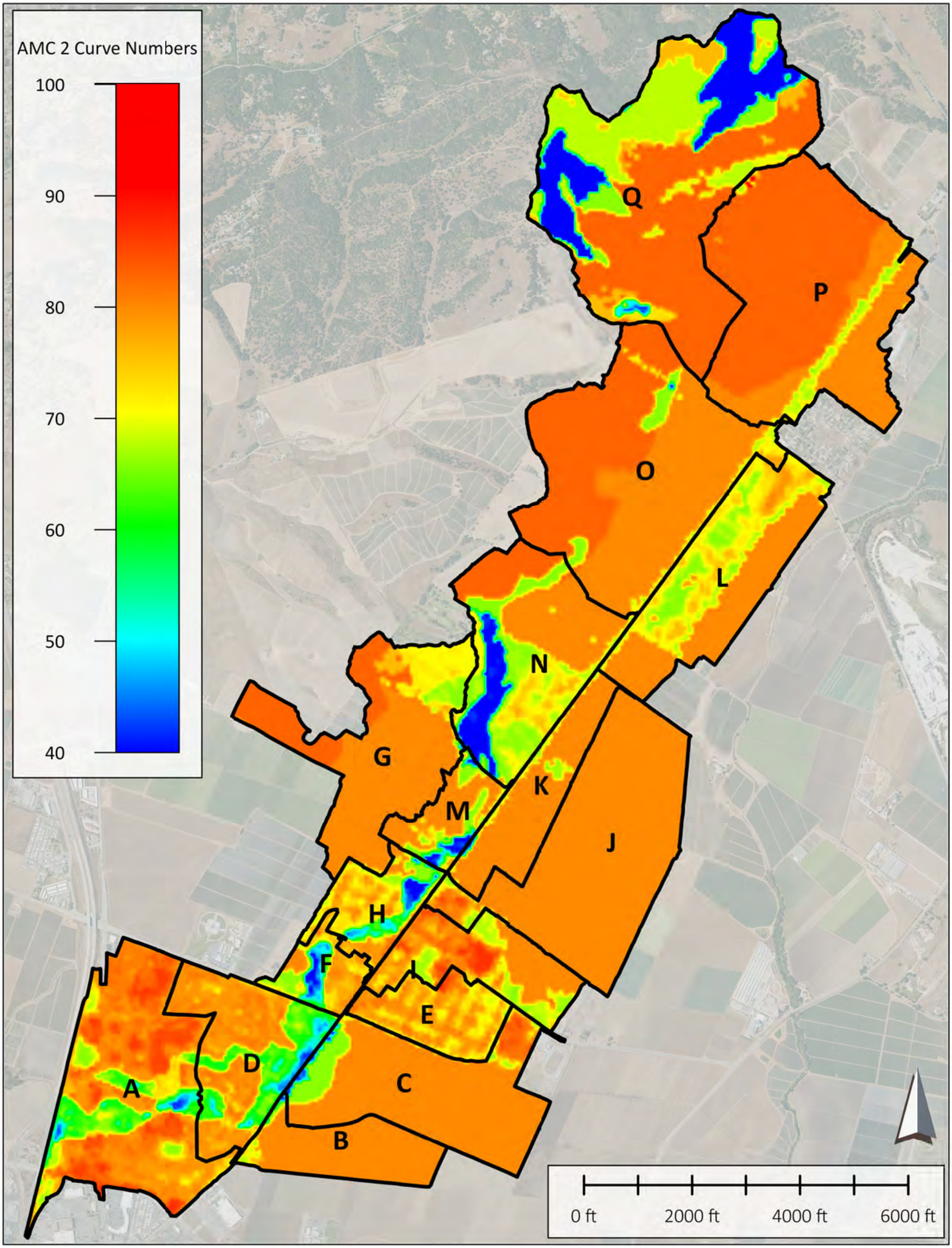
Hydrologic Model Input: Time Lag Parameters

Watershed	Length	Centroid L	U/S Elev	D/S Elev	Slope	Basin N	Time Lag
-	*mi*	*mi*	*ft*	*ft*	*ft/mi*	-	*min*
A	0.9537	0.4235	121	66	58	0.026	20
B	0.9592	0.5524	134	92	44	0.059	39
C	1.1388	0.5905	148	95	47	0.057	41
D	0.9418	0.4131	131	81	53	0.025	20
E	0.7712	0.3079	148	116	41	0.027	20
F	0.4535	0.1376	124	104	44	0.033	20
G	1.0398	0.4708	214	138	73	0.058	34
H	0.6724	0.1941	150	111	58	0.034	20
I	0.8281	0.3096	158	124	41	0.040	20
J	1.4975	0.9943	185	131	36	0.059	60
K	0.9738	0.3985	183	131	53	0.059	33
L	1.2244	0.6738	225	178	38	0.047	38
M	0.5398	0.2546	169	128	76	0.055	20
N	1.1667	0.5136	190	141	42	0.051	36
O	1.1451	0.4428	325	168	137	0.059	31
P	1.3484	0.7478	378	201	131	0.059	40
Q	1.7113	0.9148	492	198	172	0.070	54

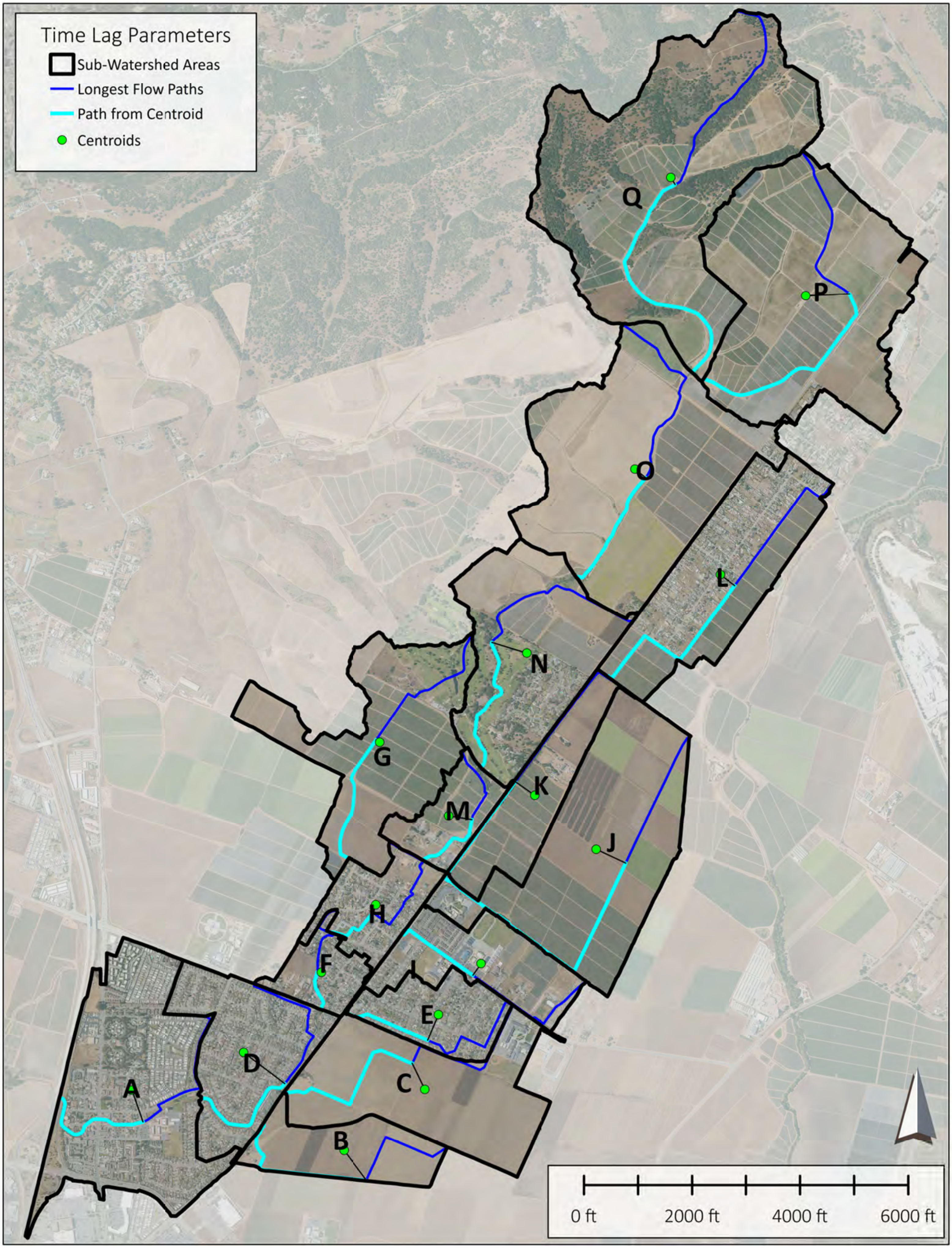
Time Lag = 1728 x 'Basin N' x (('Length' x 'Centroid Length') / ('Slope'$^{0.5}$))$^{0.38}$

Hydrologic Model Input: Routing Parameters

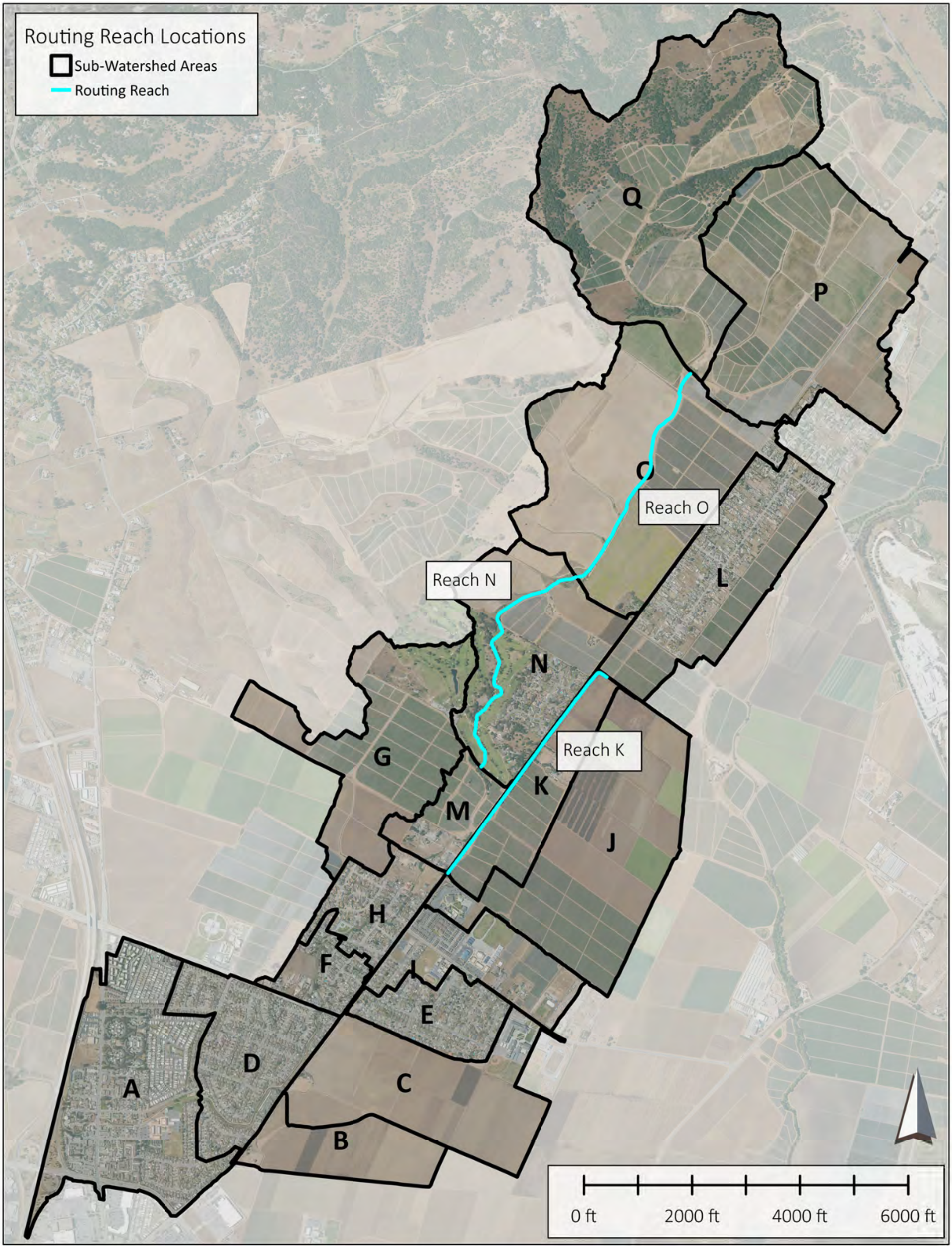
Location	Length	Slope	Mannings n	Index Flow	Shape	Width	Side Slope
-	*ft*	*ft/ft*	-	*cfs*	-	*ft*	-
Reach K	4,820	0.009	0.040	100	trap	3	2.0
Reach N	4,970	0.005	0.045	300	trap	8	2.5
Reach O	4,460	0.005	0.045	250	trap	8	2.5



AMC 2 Curve Numbers
100
90
80
70
60
50
40
Q
P
O
L
N
G
K
M
J
H
F
I
E
D
A
C
B
0 ft
2000 ft
4000 ft
6000 ft



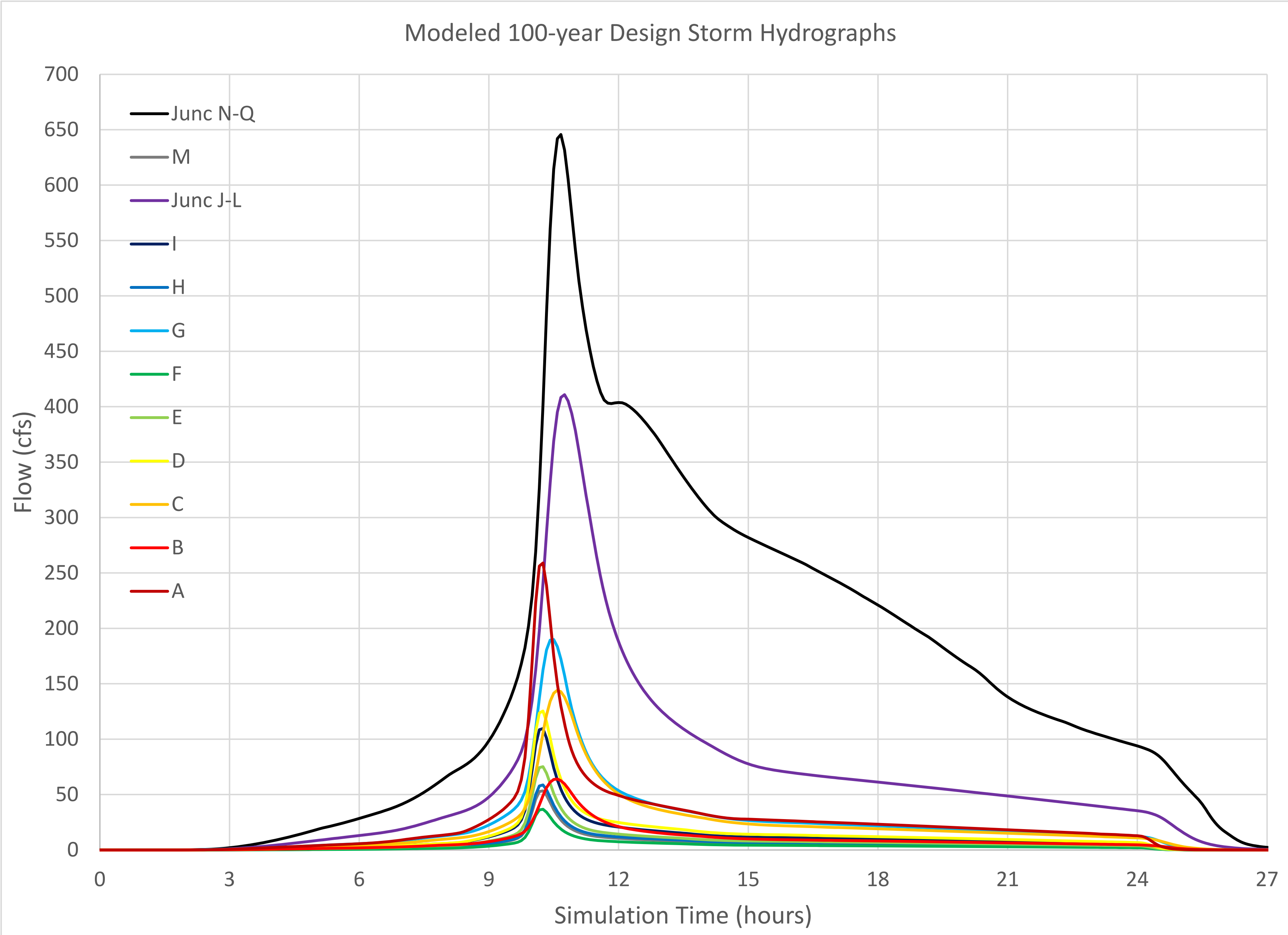
Time Lag Parameters
Sub-Watershed Areas
Longest Flow Paths
Path from Centroid
Centroids
Q
P
O
L
N
G
K
M
J
H
F
I
E
D
C
A
B
0 ft
2000 ft
4000 ft
6000 ft



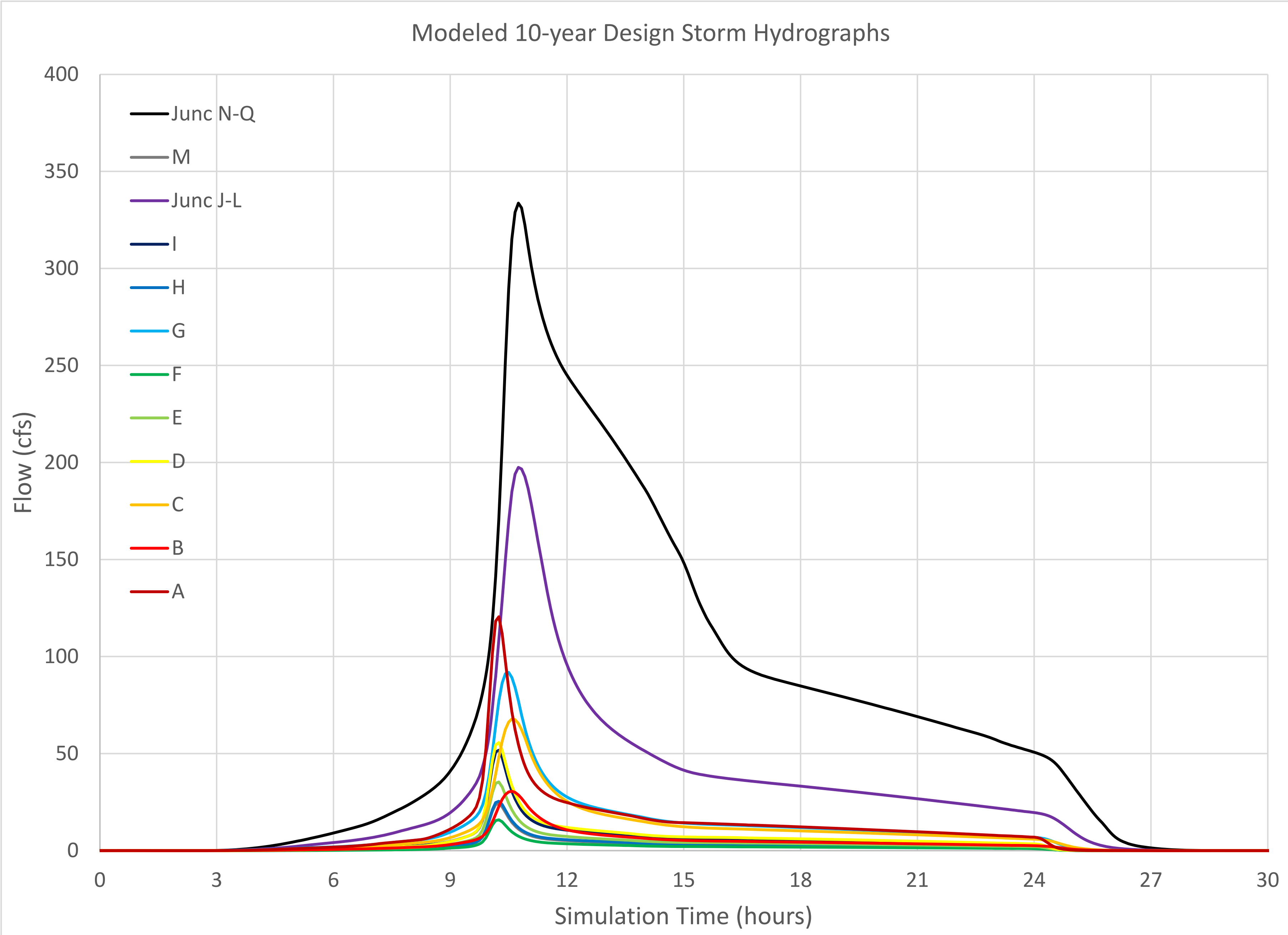
Routing Reach Locations
Sub-Watershed Areas
Routing Reach
Q
P
O
Reach O
Reach N
L
N
G
Reach K
K
M
J
H
F
I
E
D
C
A
B
0 ft
2000 ft
4000 ft
6000 ft

APPENDIX B

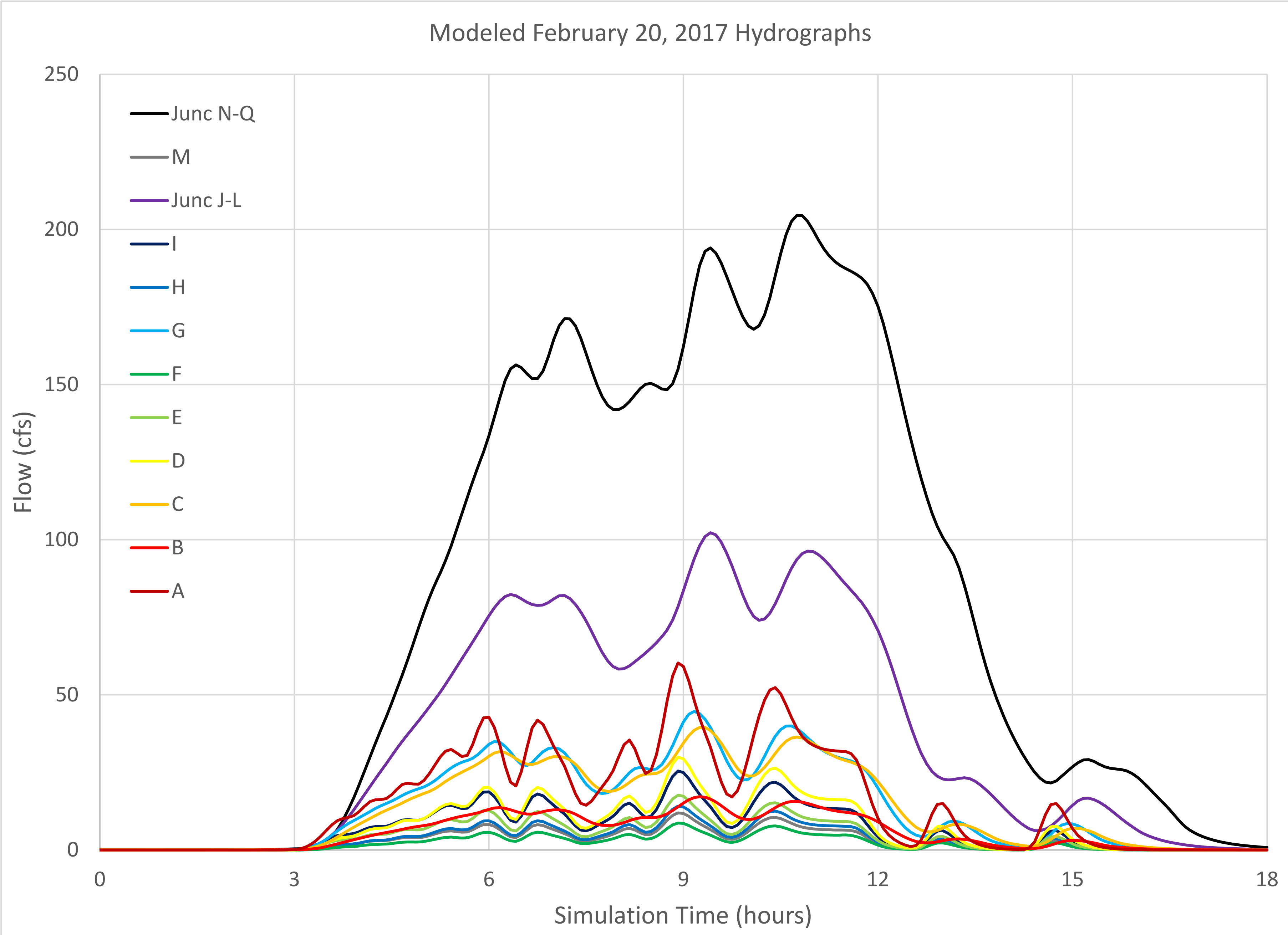
Modeled Hydrographs

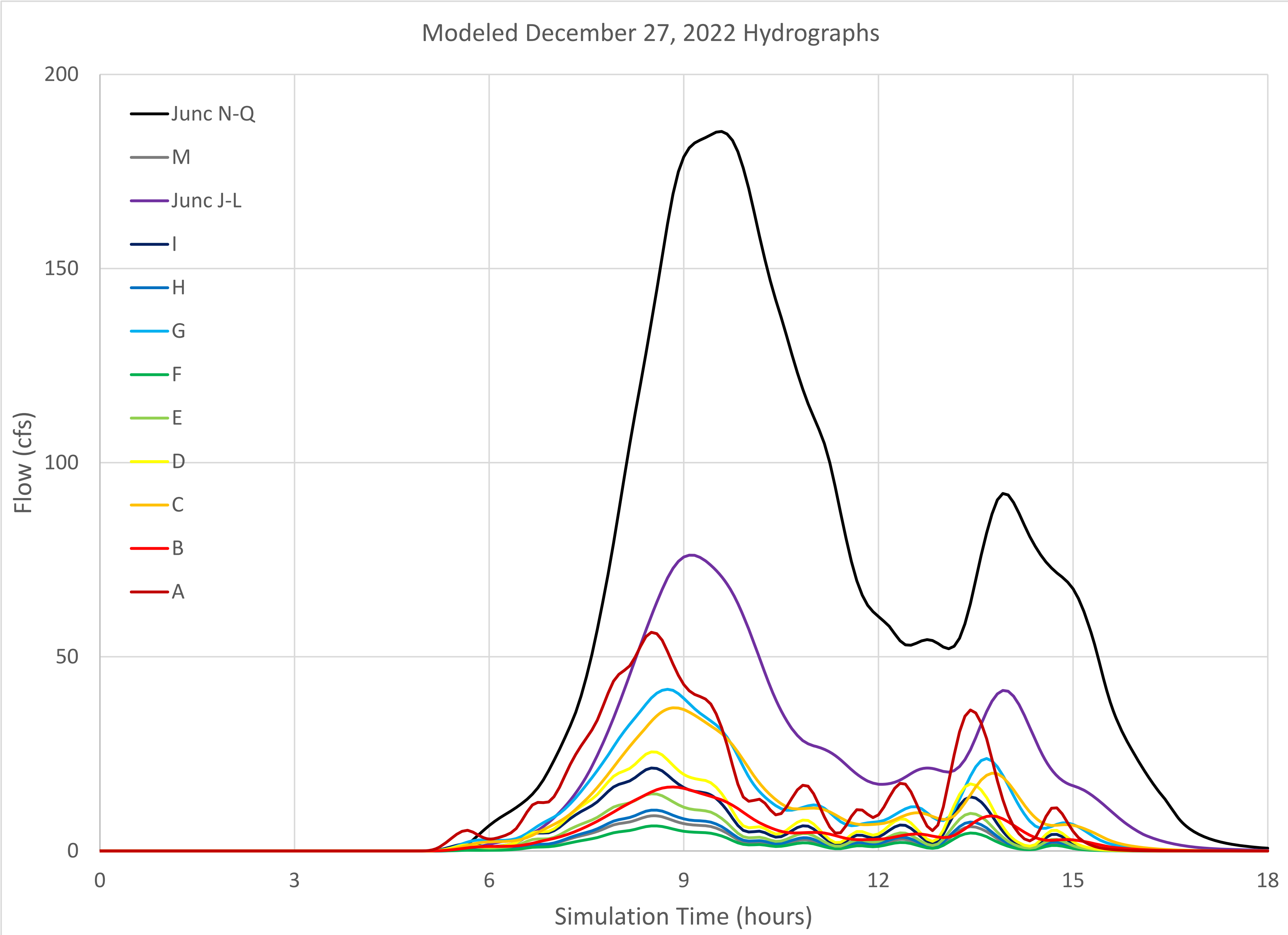


Modeled 100-year Design Storm Hydrographs
Flow (cfs)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
0
3
6
9
12
15
18
21
24
27
Simulation Time (hours)
Junc N-Q
M
Junc J-L
I
H
G
F
E
D
C
B
A

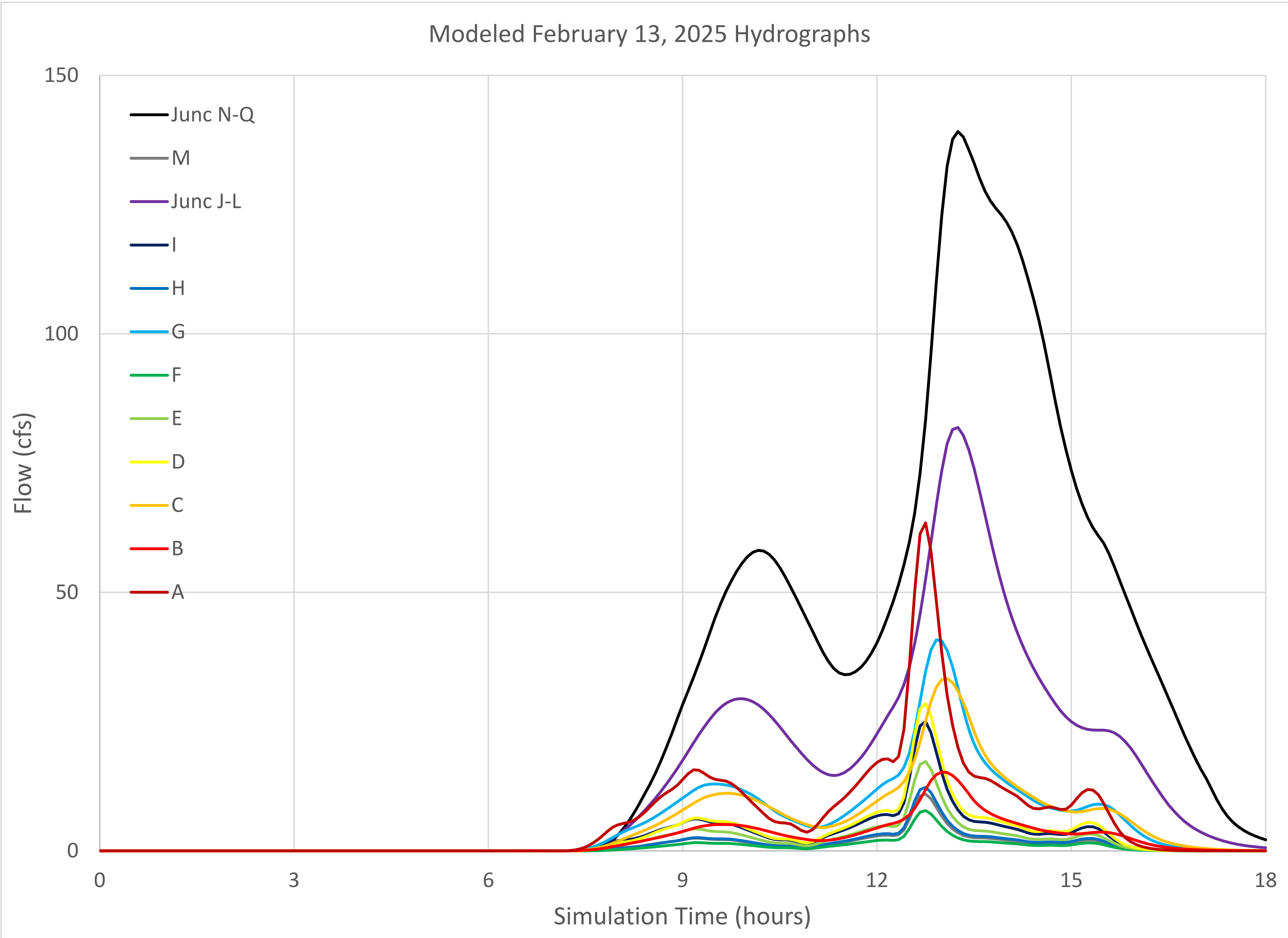


Modeled 10-year Design Storm Hydrographs
Flow (cfs)
400
350
300
250
200
150
100
50
0
0
3
6
9
12
15
18
21
24
27
30
Simulation Time (hours)
Junc N-Q
M
Junc J-L
I
H
G
F
E
D
C
B
A


Modeled February 20, 2017 Hydrographs
Flow (cfs)
250
200
150
100
50
0
0
3
6
9
12
15
18
Simulation Time (hours)
Junc N-Q
M
Junc J-L
I
H
G
F
E
D
C
B
A



Modeled December 27, 2022 Hydrographs
Flow (cfs)
200
150
100
50
0
0
3
6
9
12
15
18
Simulation Time (hours)
Junc N-Q
M
Junc J-L
I
H
G
F
E
D
C
B
A

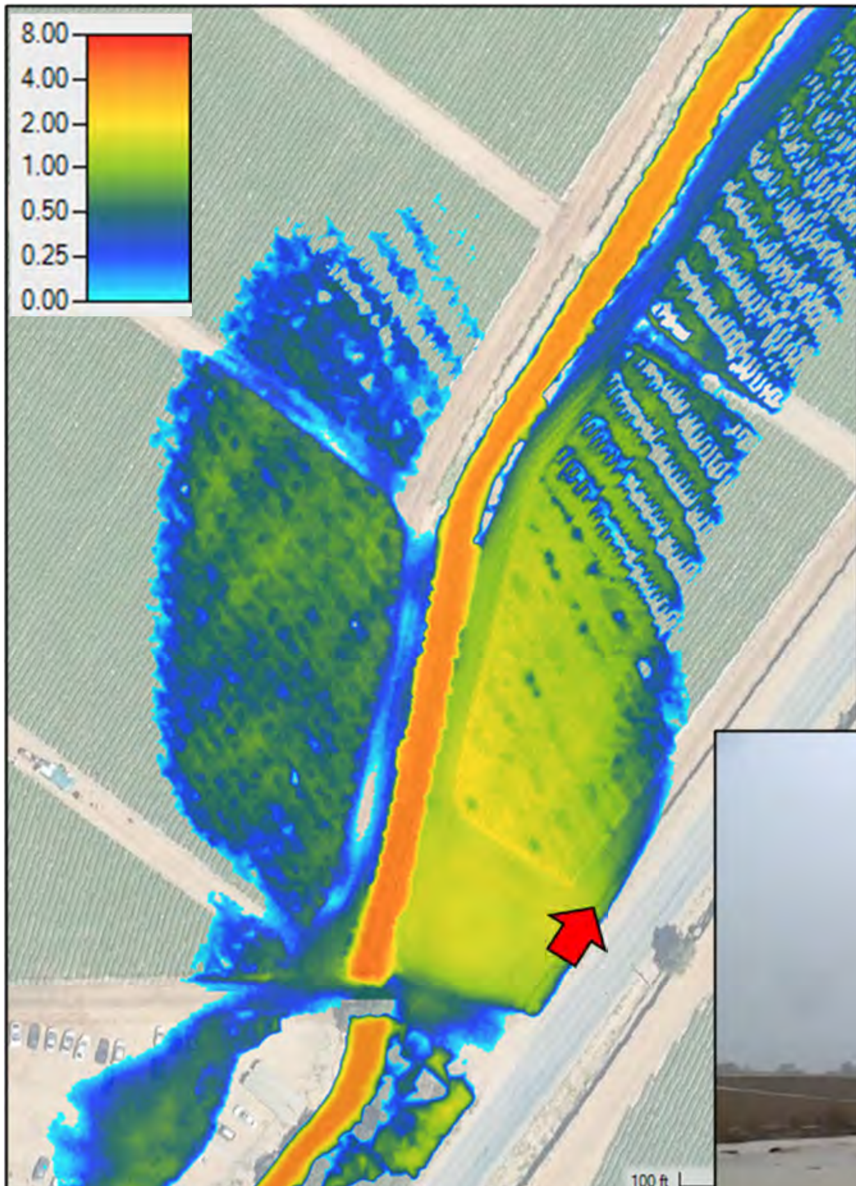


Modeled February 13, 2025 Hydrographs
Flow (cfs)
150
100
50
0
0
3
6
9
12
15
18
Simulation Time (hours)
Junc N-Q
M
Junc J-L
I
H
G
F
E
D
C
B
A

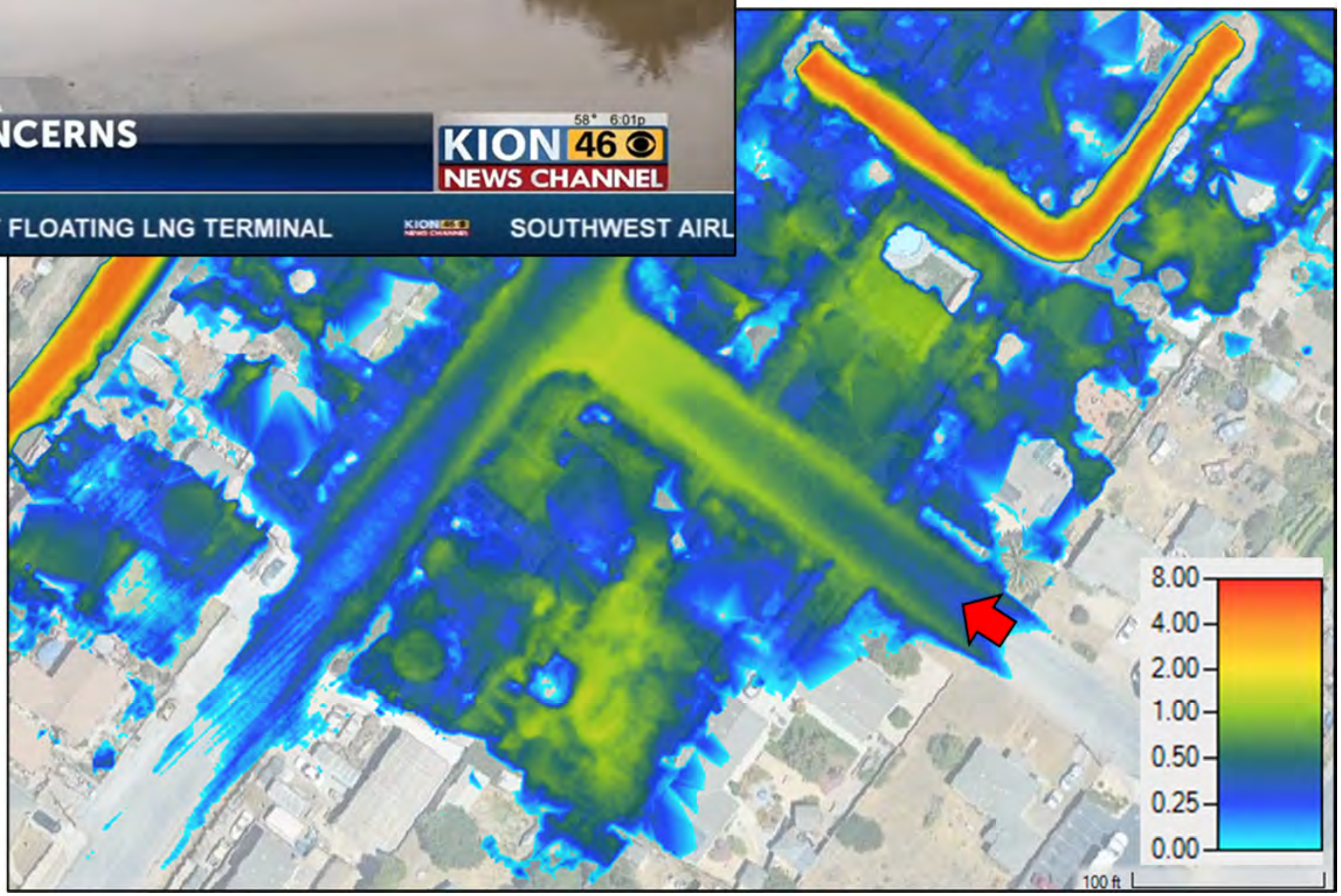
APPENDIX C

Model Calibration Images

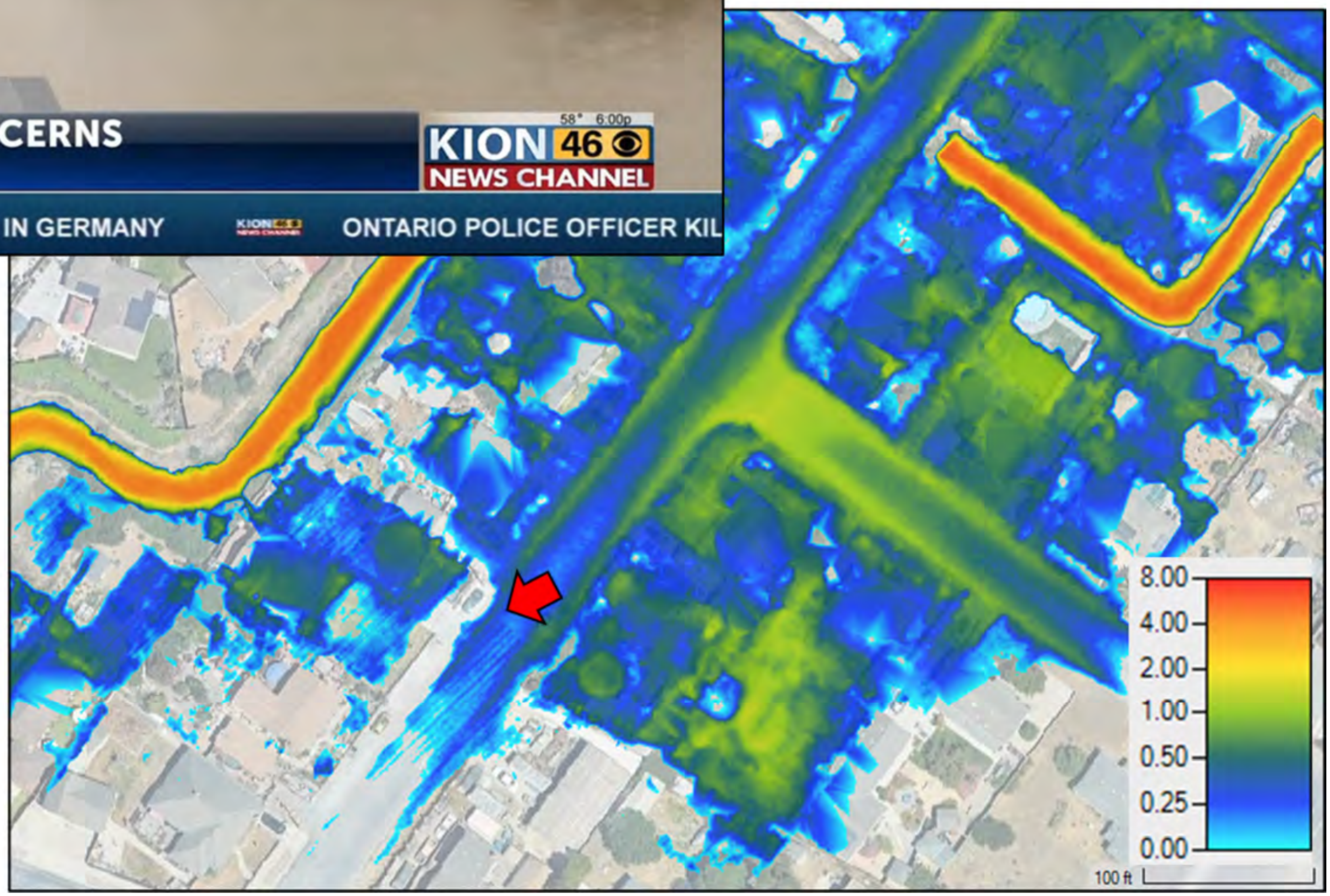
Flood Depths at the Upstream End of the Study Reach, December 27, 2022



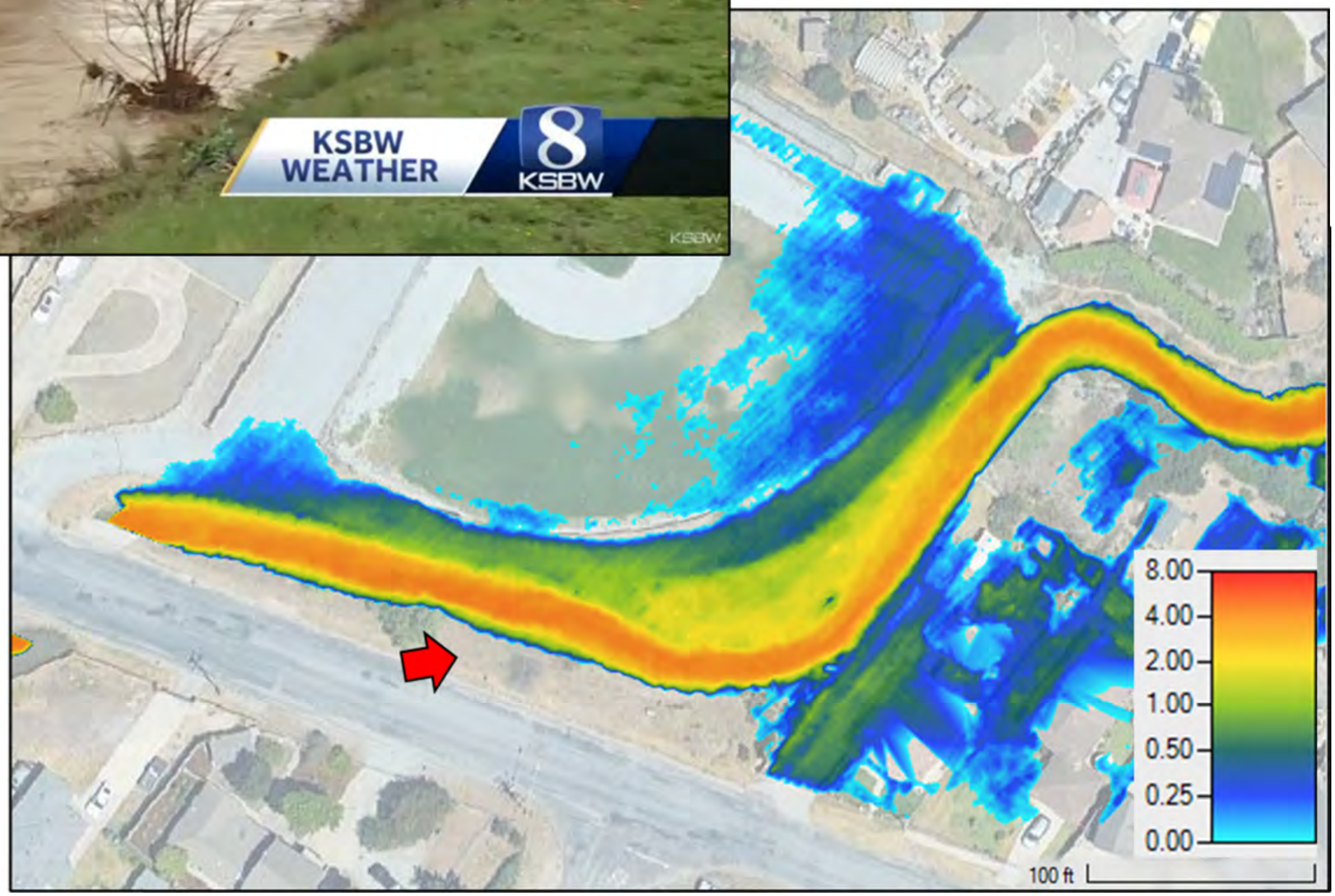
Flood Depths at Denner Road, December 27, 2022



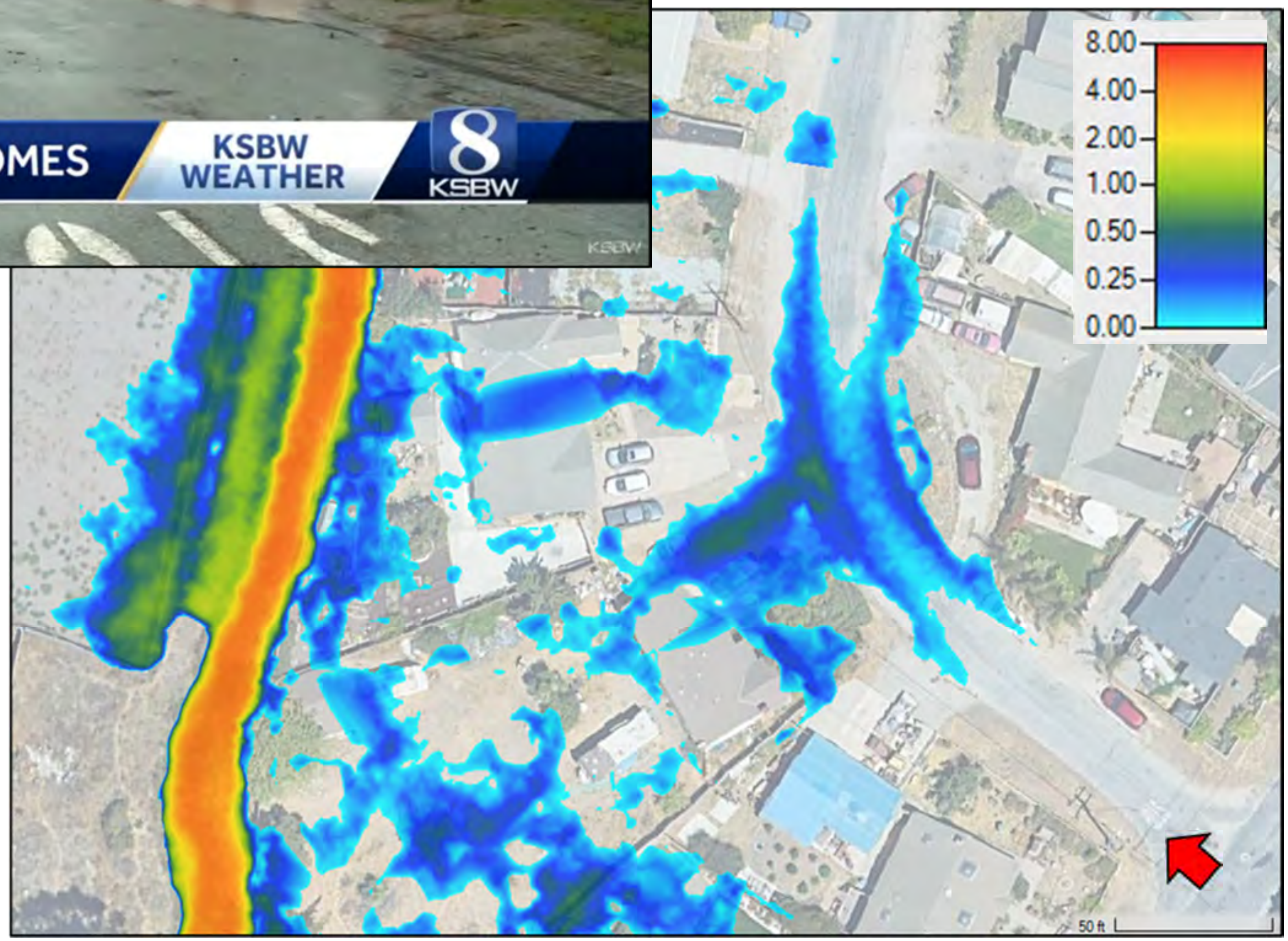
Flood Depths at Paul Avenue, December 27, 2022



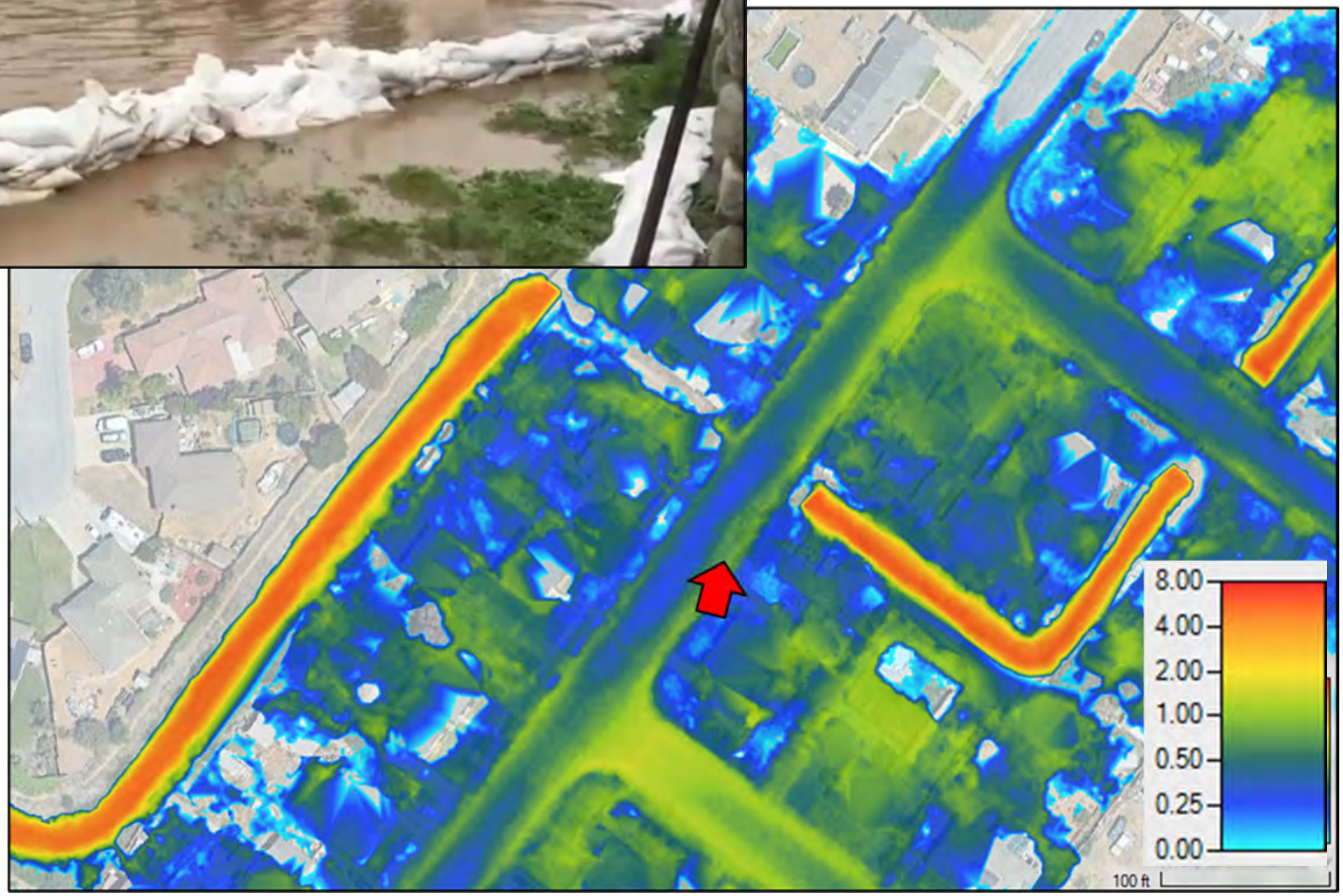
Flood Depths at Ferrasci Park, December 27, 2022



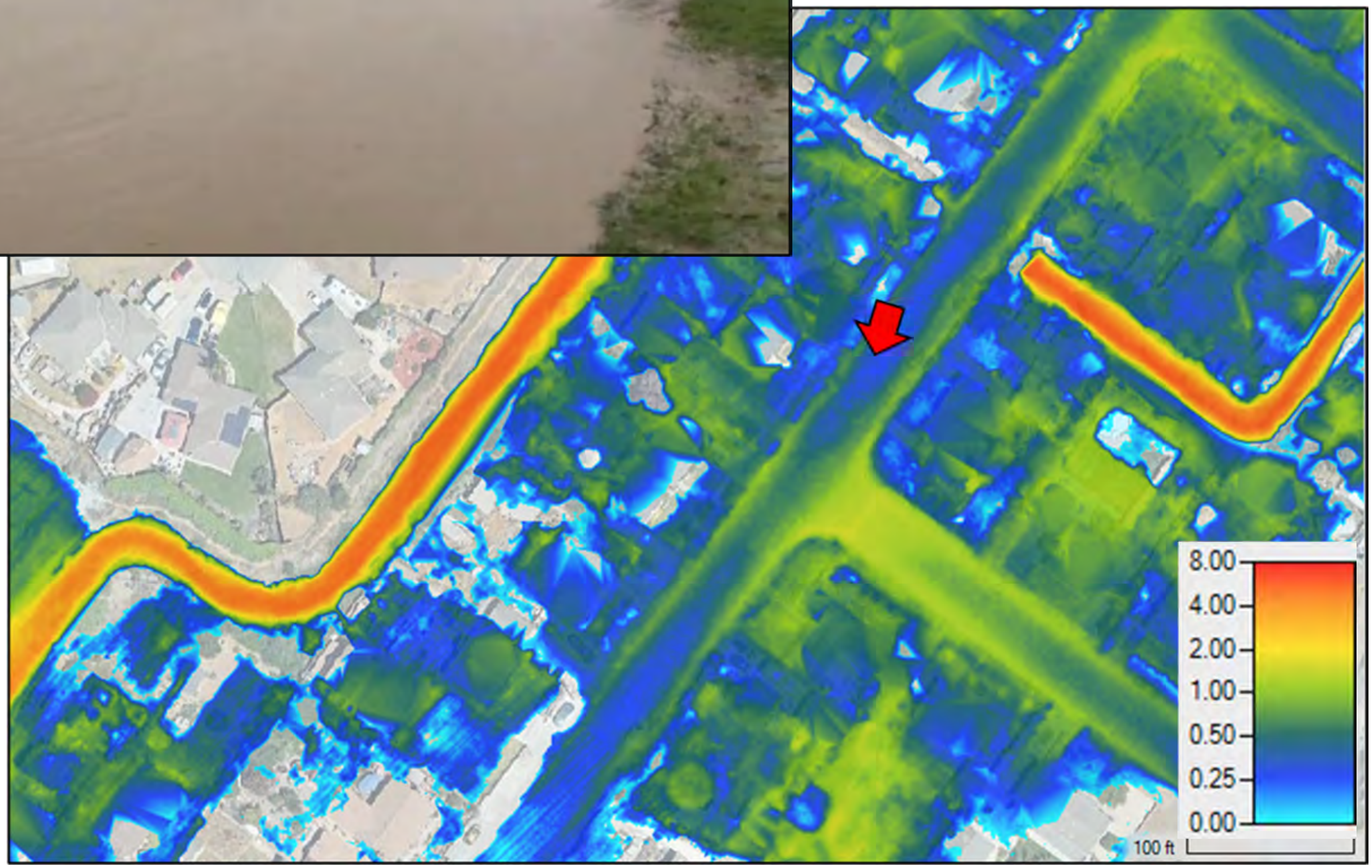
Flood Depths at England Avenue, December 27, 2022



Flood Depths at Paul Avenue, February 20, 2017

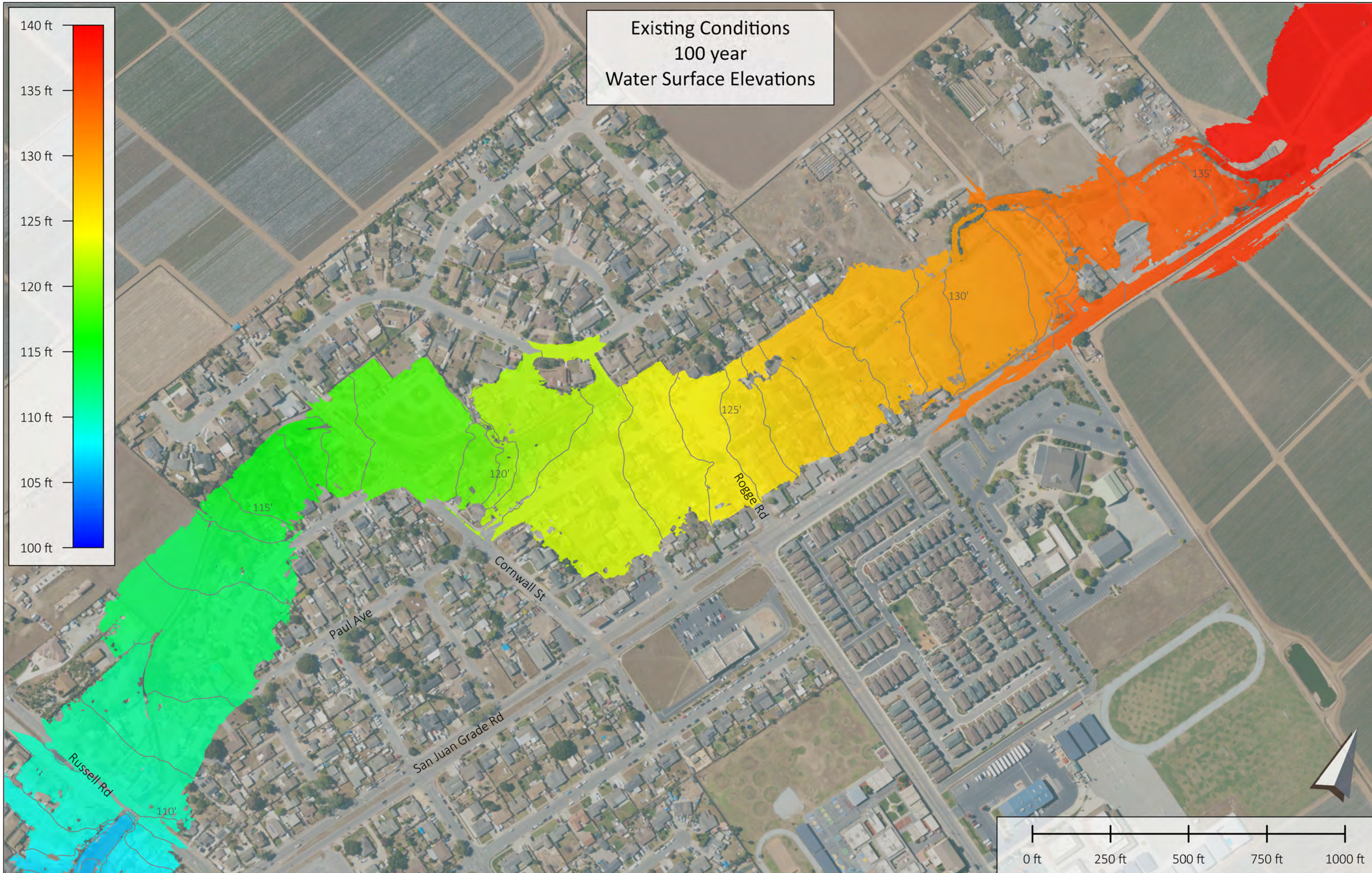


Flood Depths at Paul Avenue, February 20, 2017

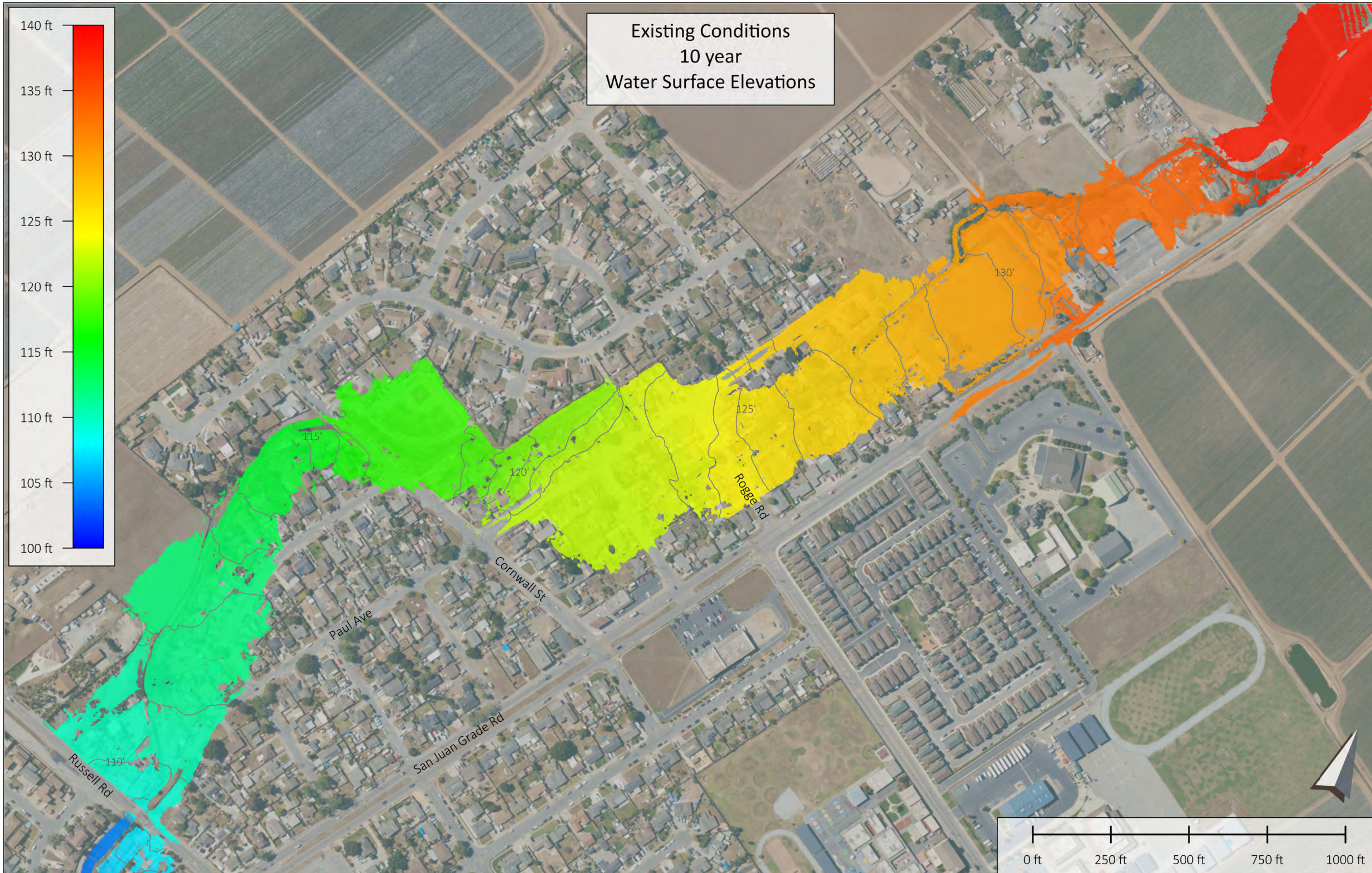


APPENDIX D

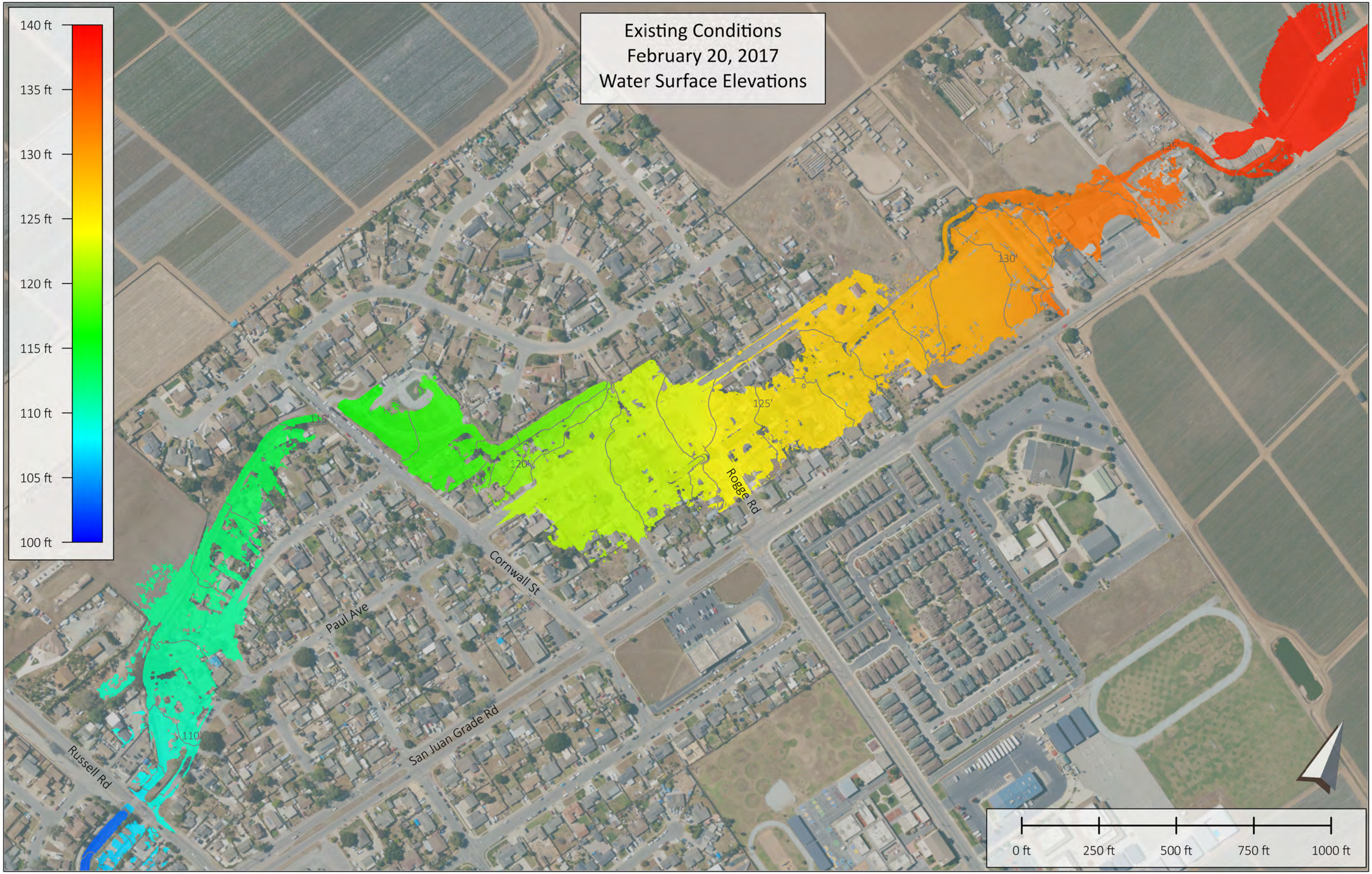
Existing Conditions Hydraulic Model Results

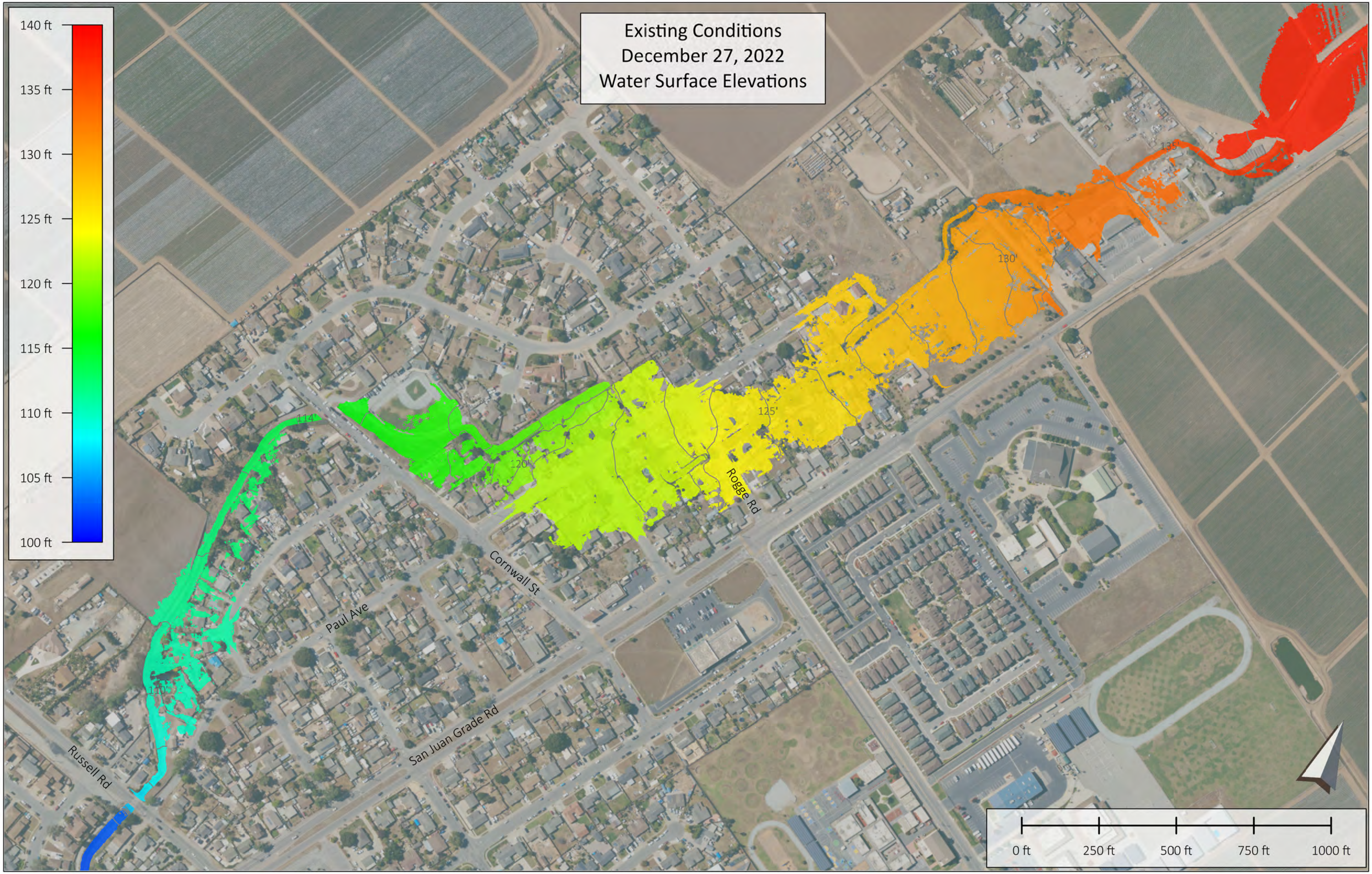


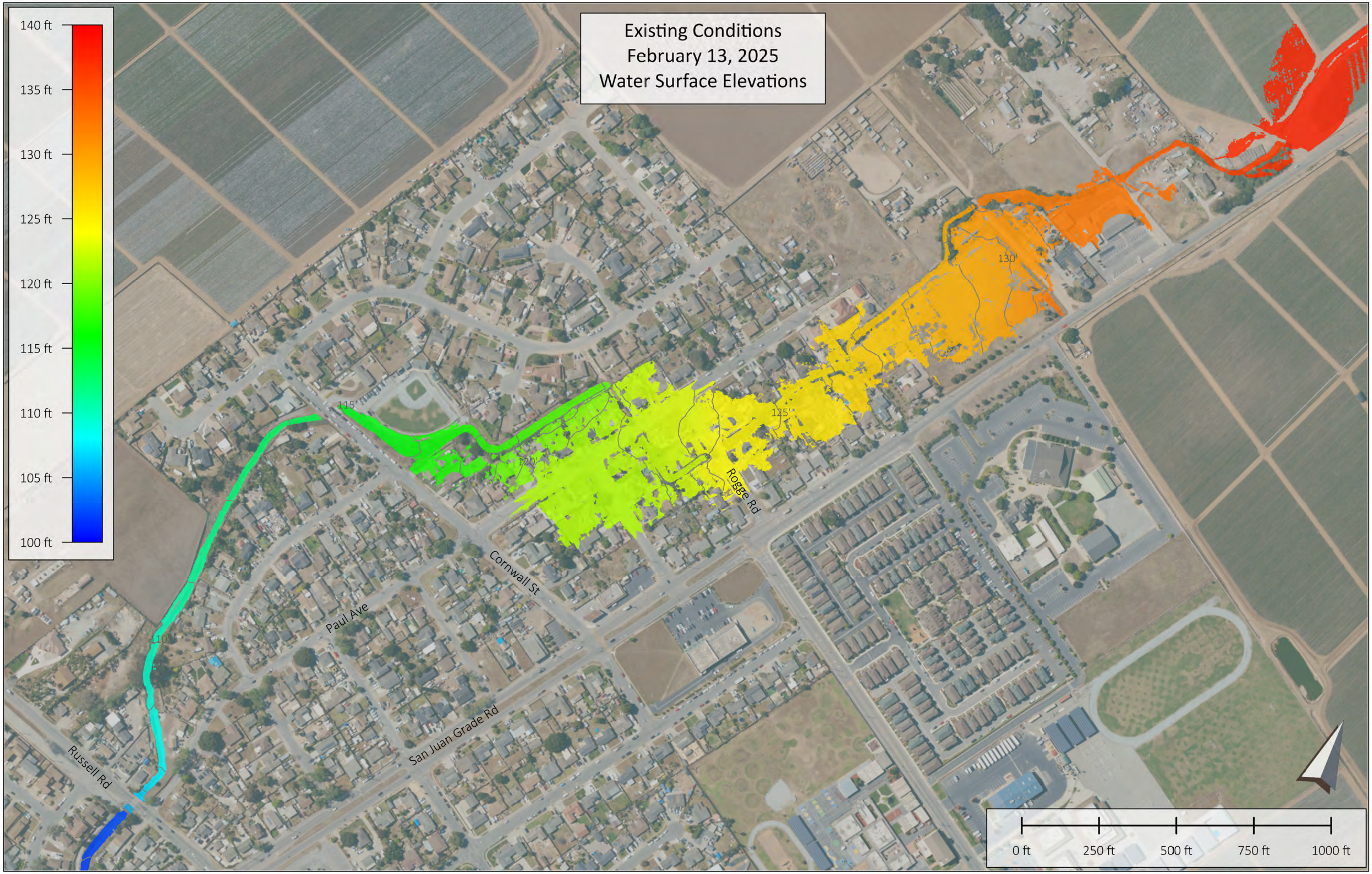
Existing Conditions
100 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



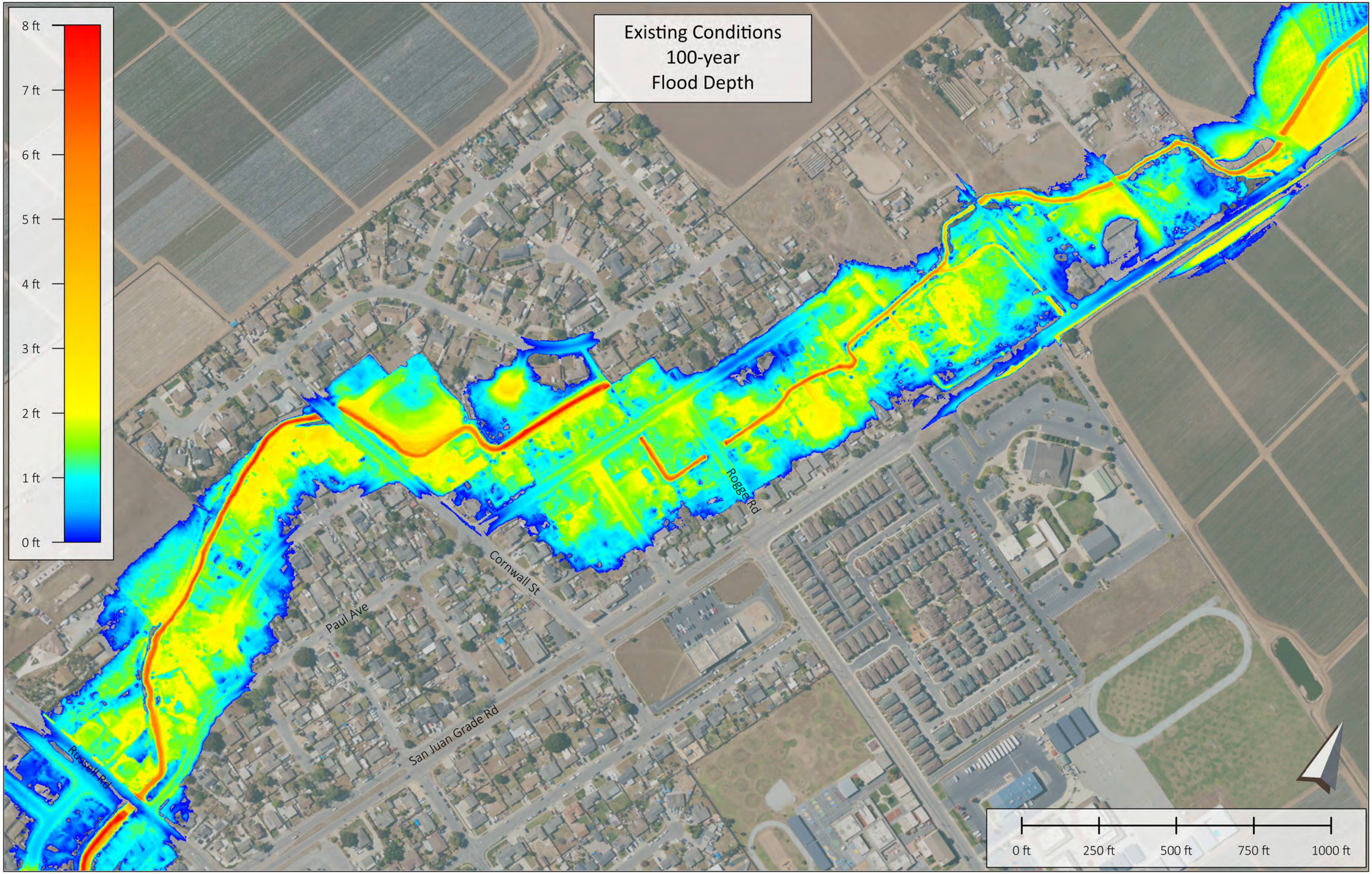
Existing Conditions
10 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Existing Conditions
February 20, 2017
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

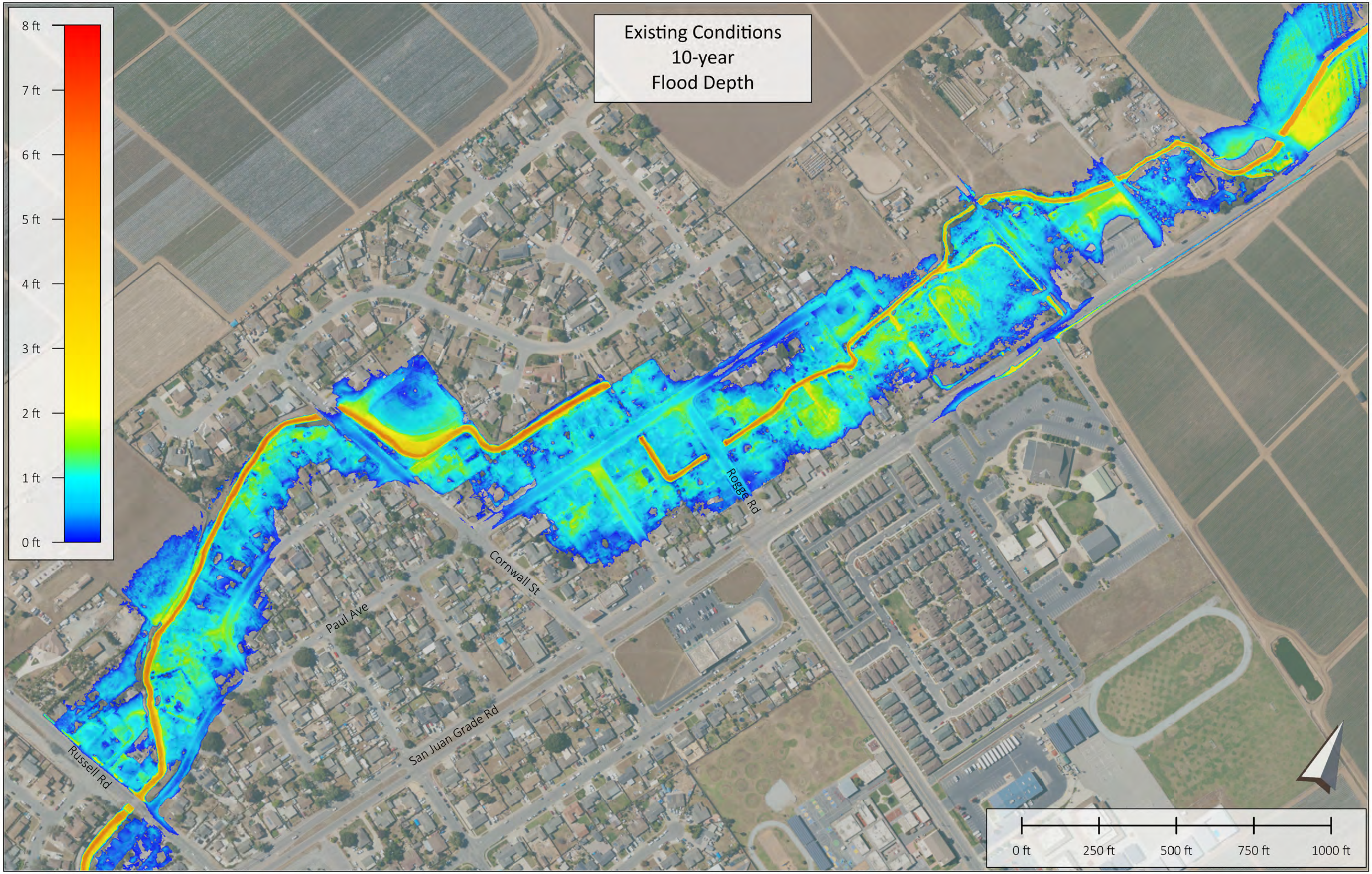

Existing Conditions
December 27, 2022
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
114'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

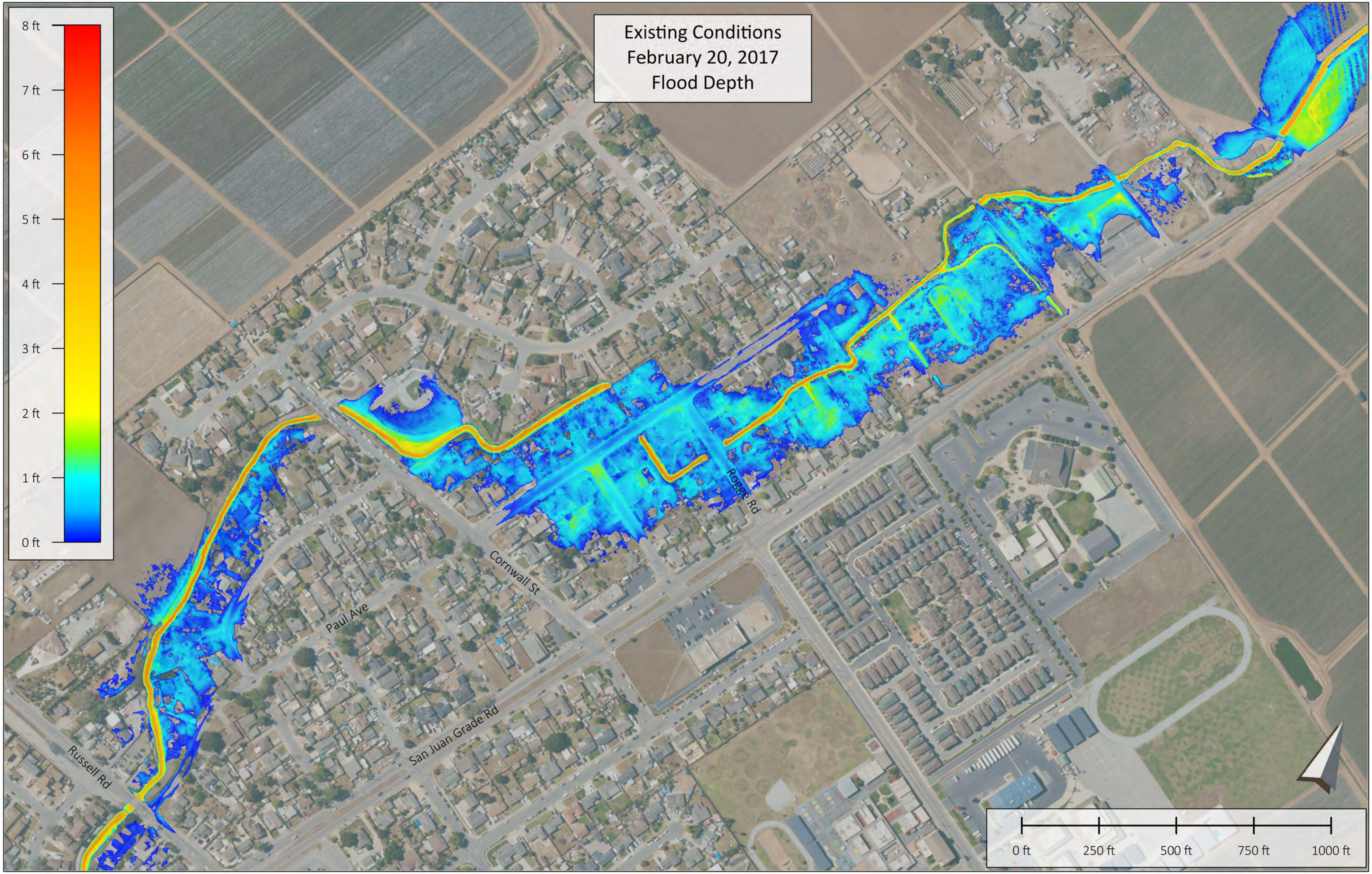


Existing Conditions
February 13, 2025
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

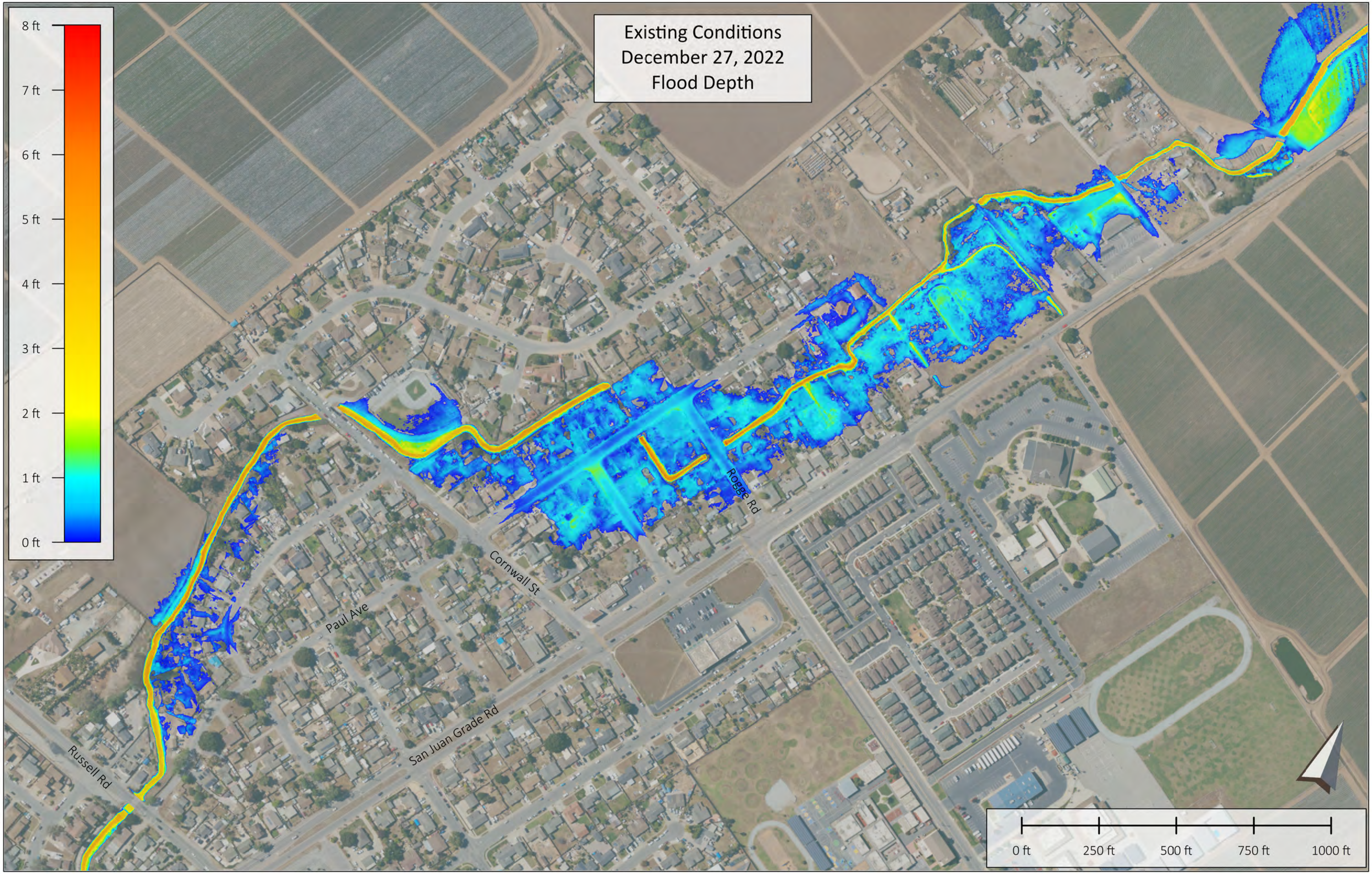


Existing Conditions
100-year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

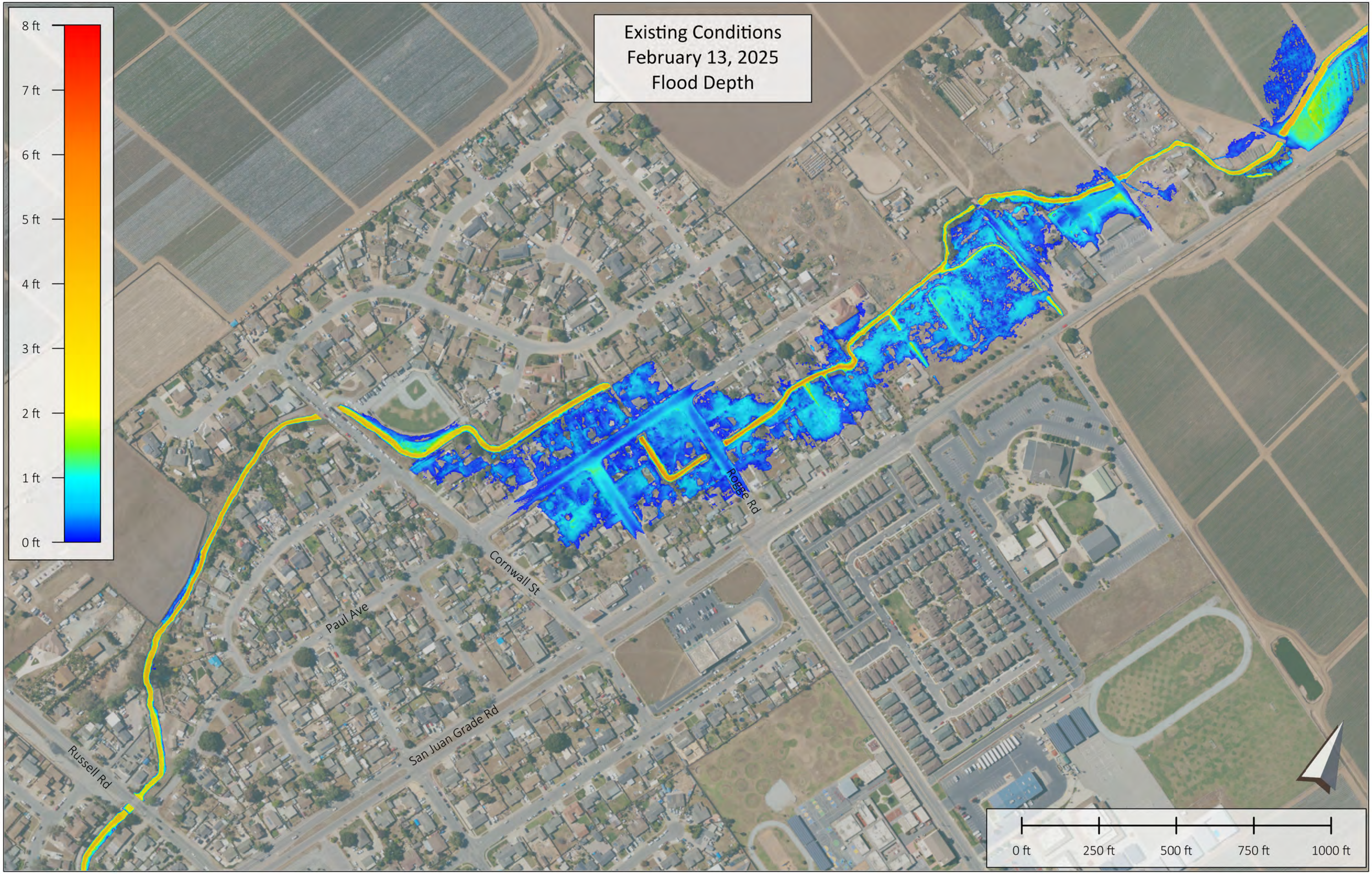

Existing Conditions
10-year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



Existing Conditions
February 20, 2017
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



Existing Conditions
December 27, 2022
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Existing Conditions
February 13, 2025
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

APPENDIX E

Permitting Discussion Memo

DATE: June 25, 2025

TO: Eric Riedner, P.E. – Civil Engineer/Hydrologist

FROM: Josh Harwayne, Denise Duffy & Associates, Inc.

RE: **Regulatory Permitting Discussion for the Santa Rita Creek Project**

Mr. Riedner,

The following provides a discussion on the anticipated regulatory permitting requirements for the Santa Rita Creek Project (project). Because each proposed project alternative would result in significant impacts to the bed and bank of Santa Rita Creek, the project would require multiple regulatory permits from state and federal agencies regardless of the chosen alternative. The following regulatory agencies regulate impacts to aquatic resources: the California Department of Fish and Wildlife, Central Coast Regional Water Quality Control Board, and Army Corps of Engineers. Additionally, three known California tiger salamander (*Ambystoma californiense,* CTS) breeding locations are located within 2.2km of the project site. CTS are listed as federal and state threatened under the Endangered Species Act and California Endangered Species Act, respectively.

The timelines for acquiring approvals from the following regulatory agencies vary, but typically take up to a year and require multiple rounds of comments and information requests from the regulatory agencies. Each permit would require an application fee, which ranges from hundreds to thousands of dollars per permit. The cost and timeline of acquiring regulatory permit approvals may be limiting to the project scope and budget.

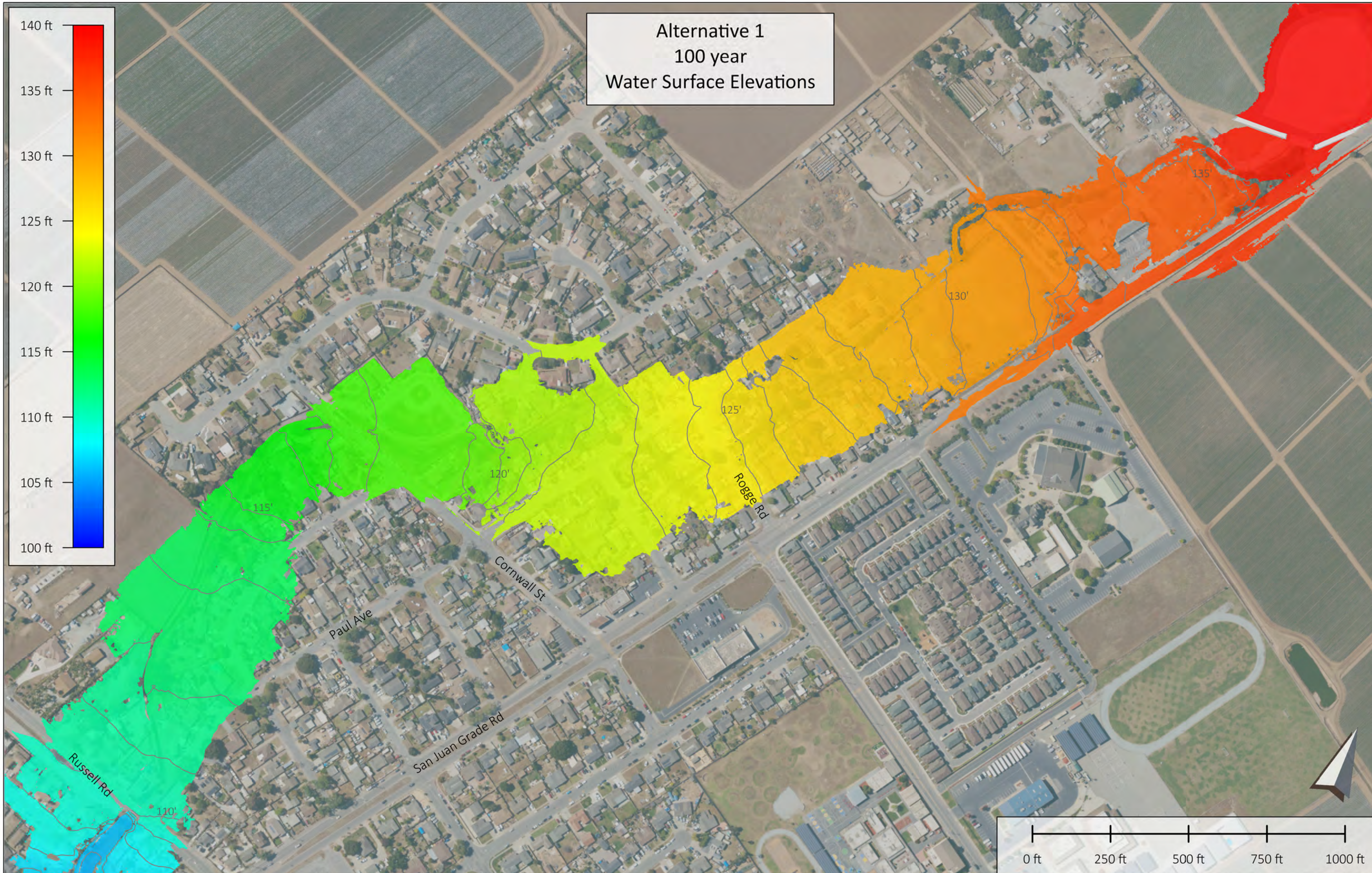
The following regulatory permits would likely be required for any chosen project alternative:

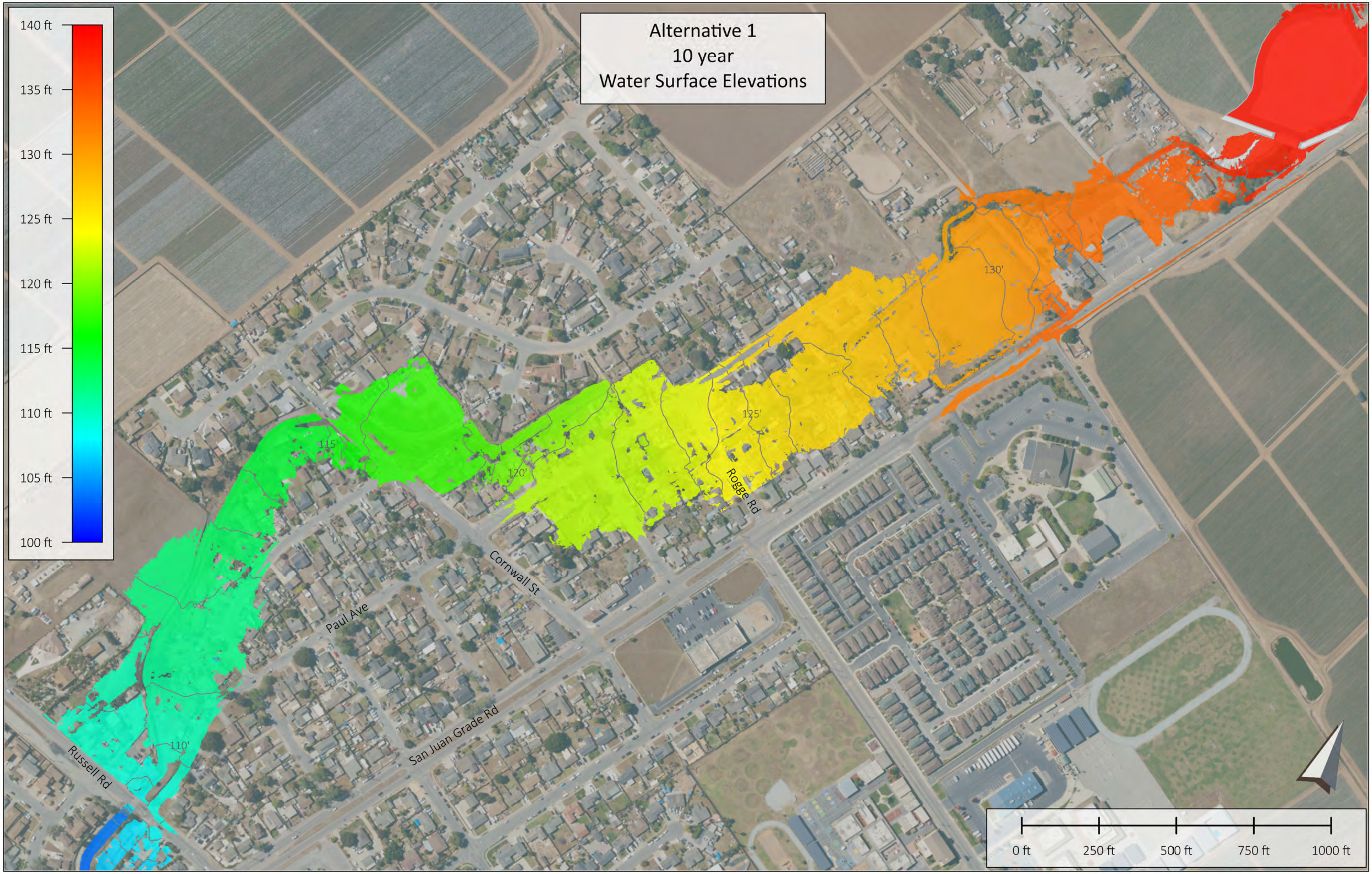
- ***United States Fish and Wildlife Service:*** Endangered Species Act Section 7 consultation or Section 10 incidental take authorization (includes a Habitat Conservation Plan [HCP]).
- ***California Department of Fish and Wildlife:*** California Endangered Species Act Section 2081(b) Incidental Take Permit and California Fish and Game Code Section 1602 Streambed Alteration Agreement.
- ***United States Army Corps of Engineers:*** Clean Water Act Section 404 Water Quality Certification.
- ***Central Coast Regional Water Quality Control Board:*** Clean Water Act Section 401 Water Quality Certification.

Please do not hesitate to reach out with any questions on the information outlined above.

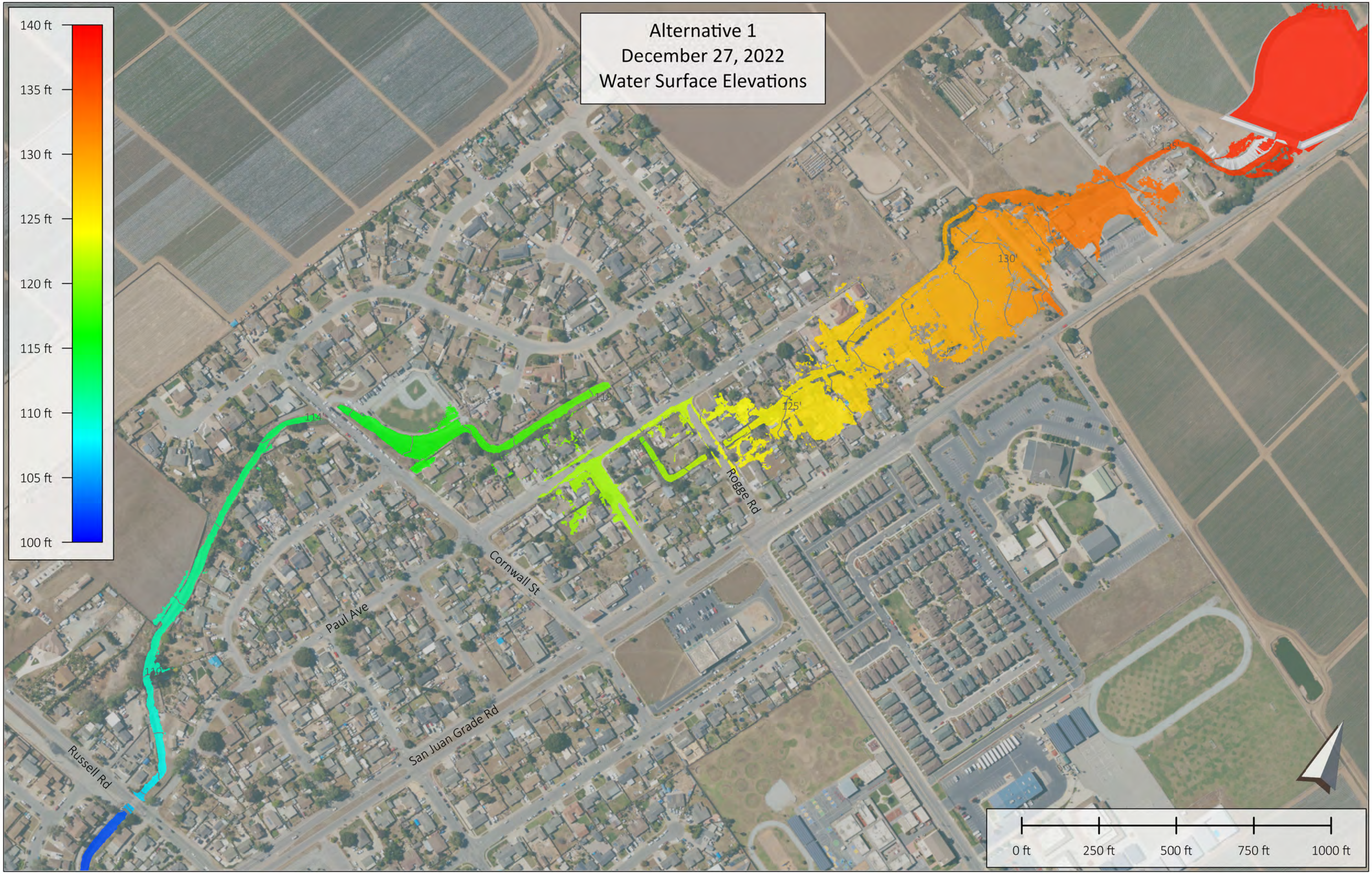
APPENDIX F

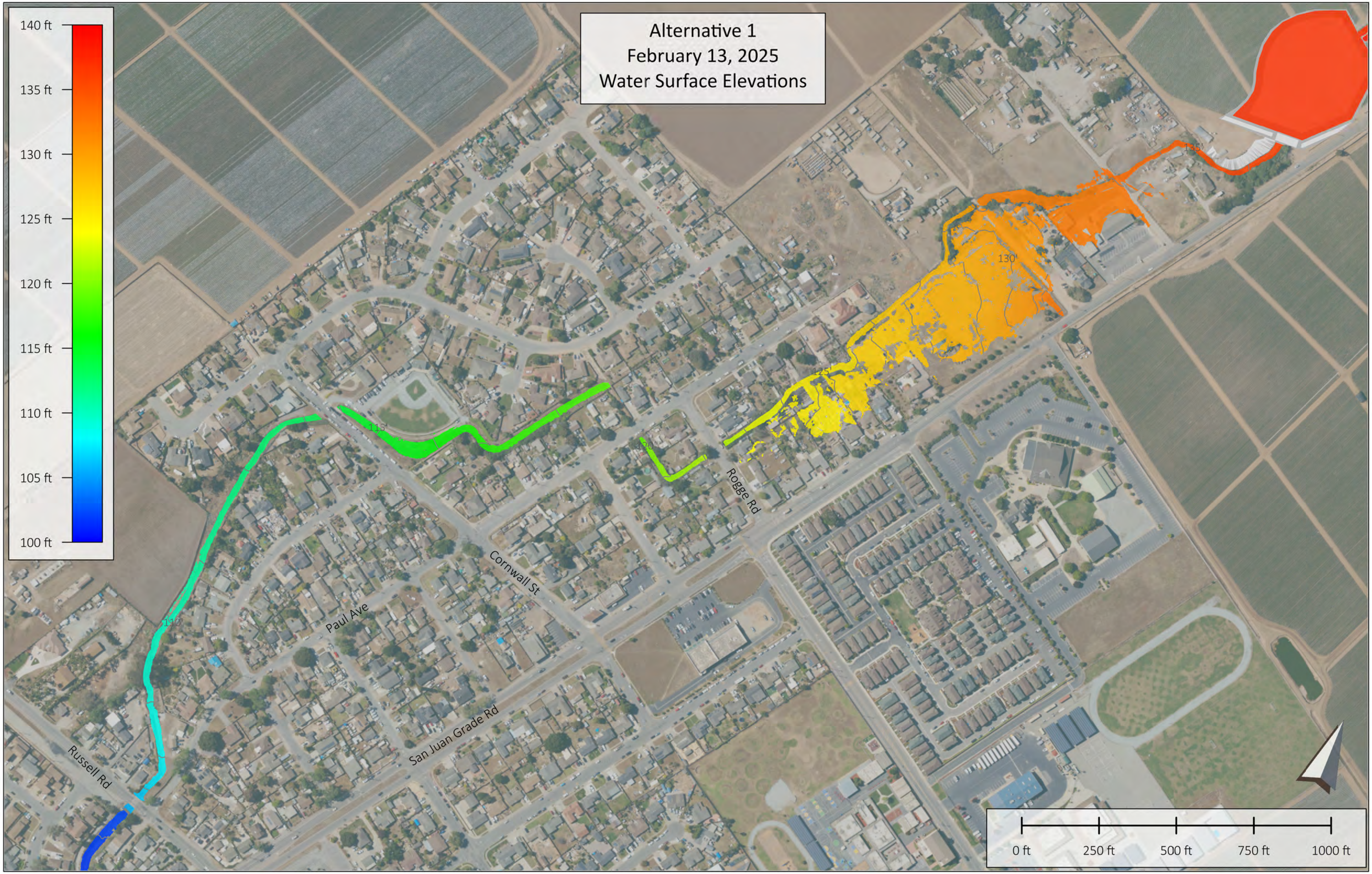
Alternatives Hydraulic Model Results


Alternative 1
100 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

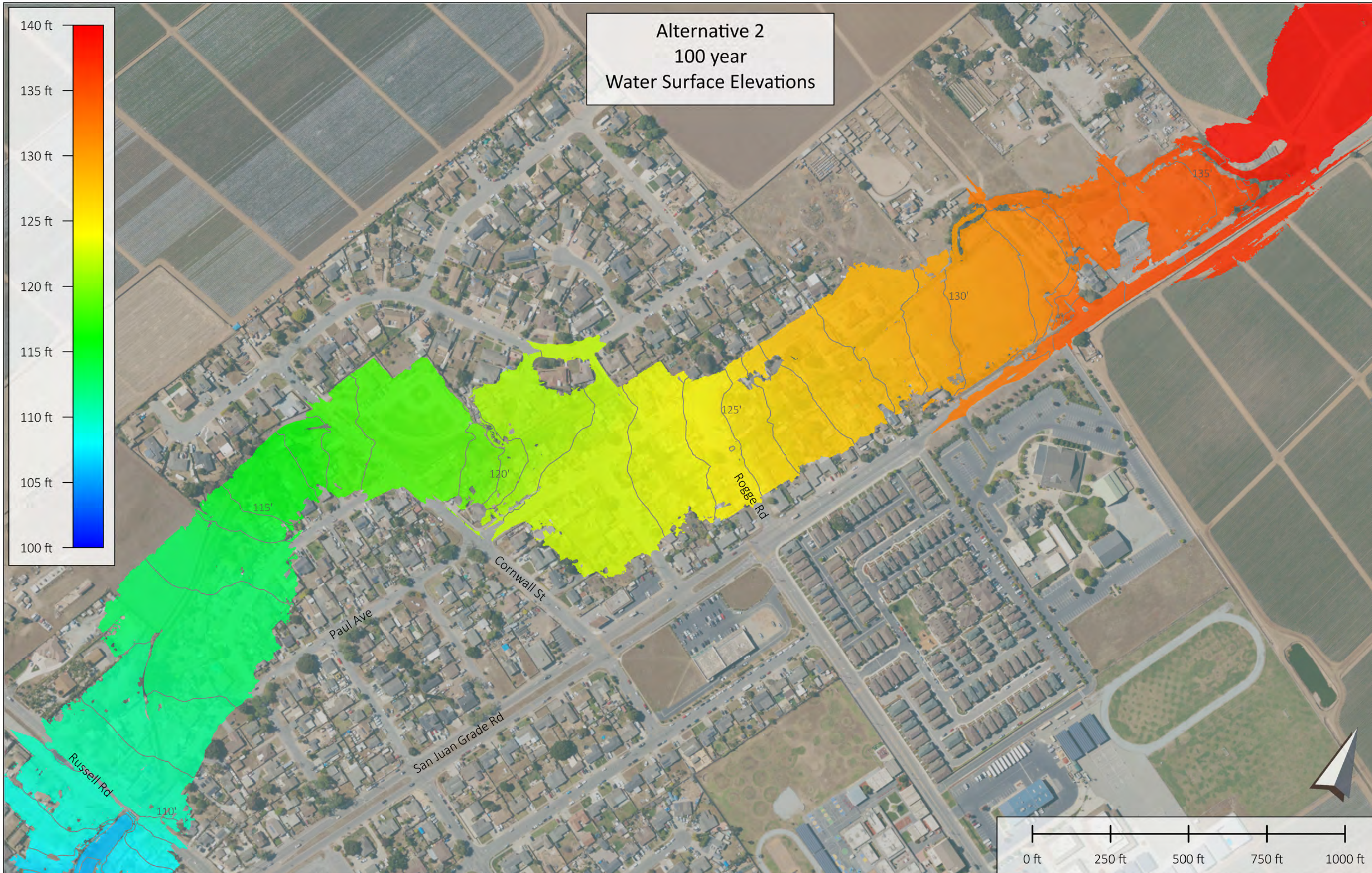


Alternative 1
10 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

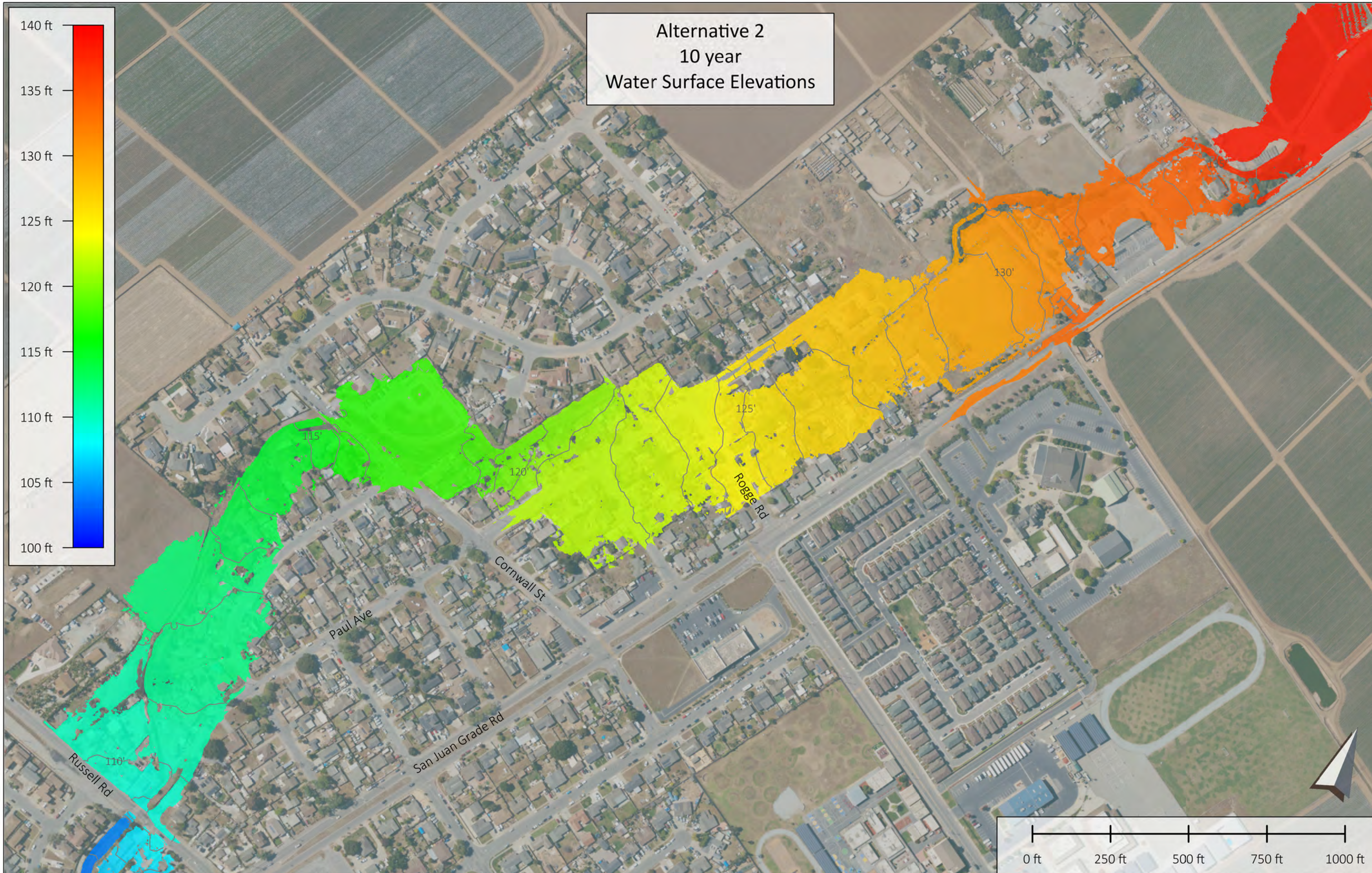

Alternative 1
December 27, 2022
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

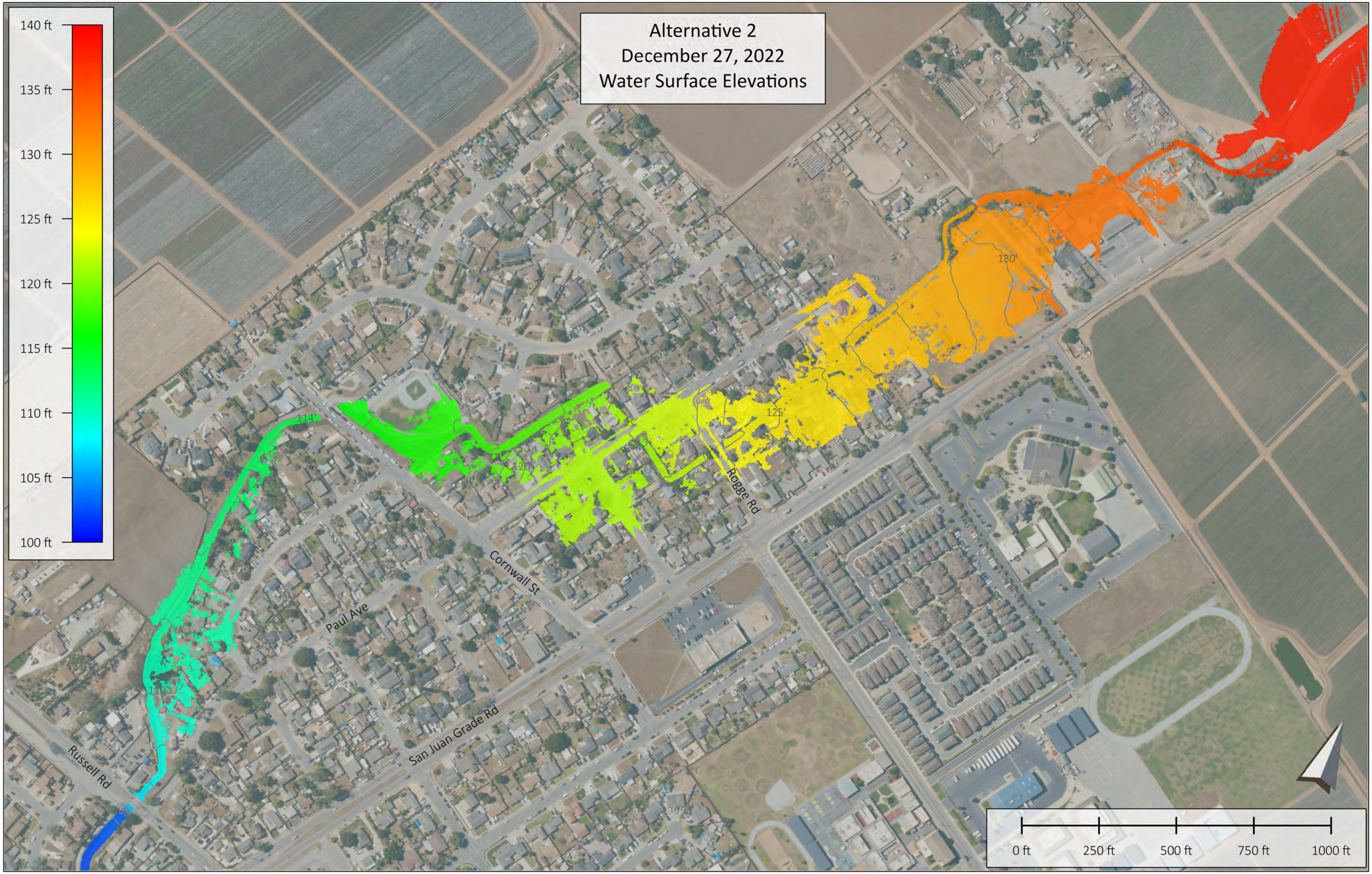


Alternative 1
February 13, 2025
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

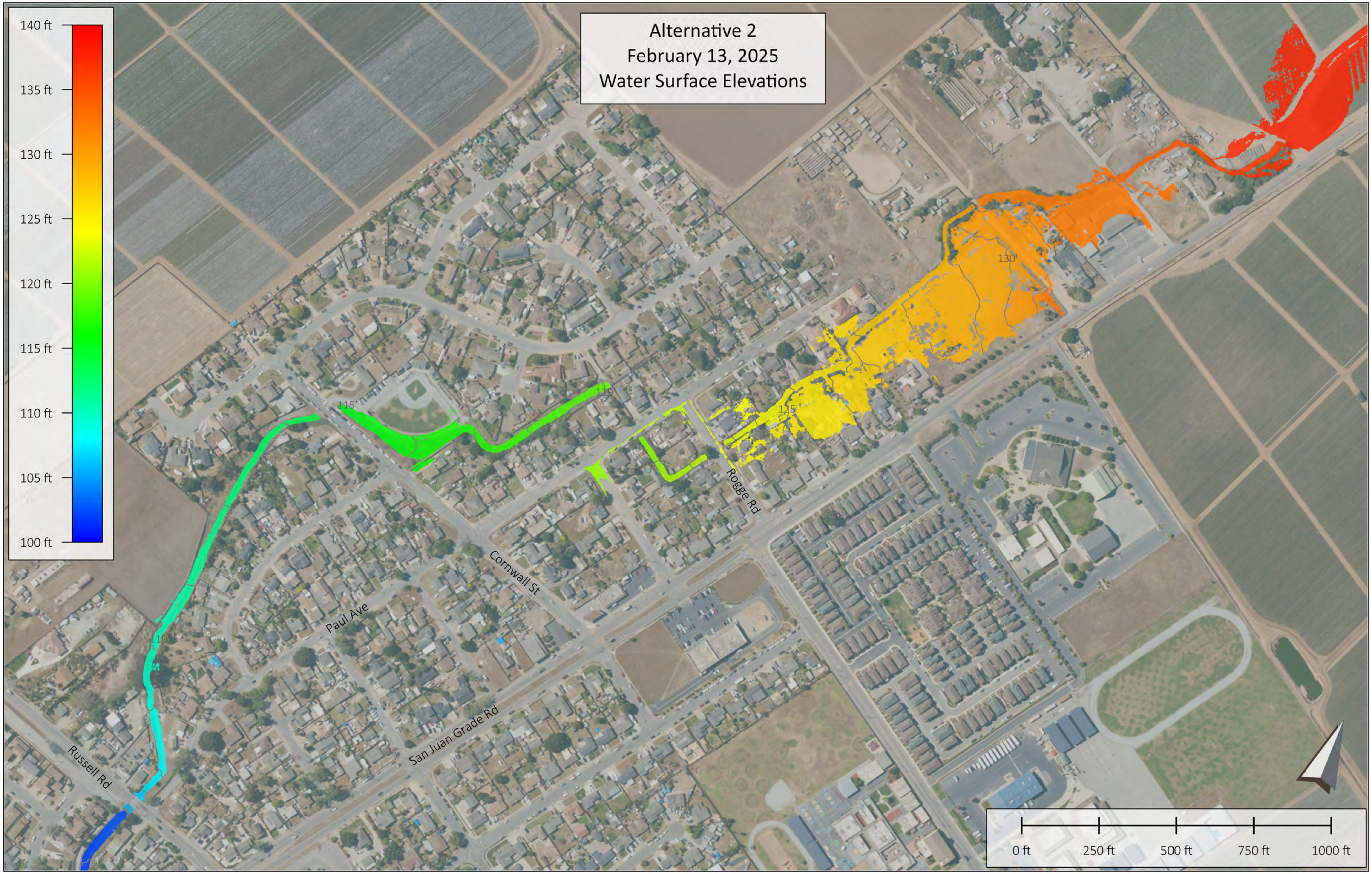


Alternative 2
100 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

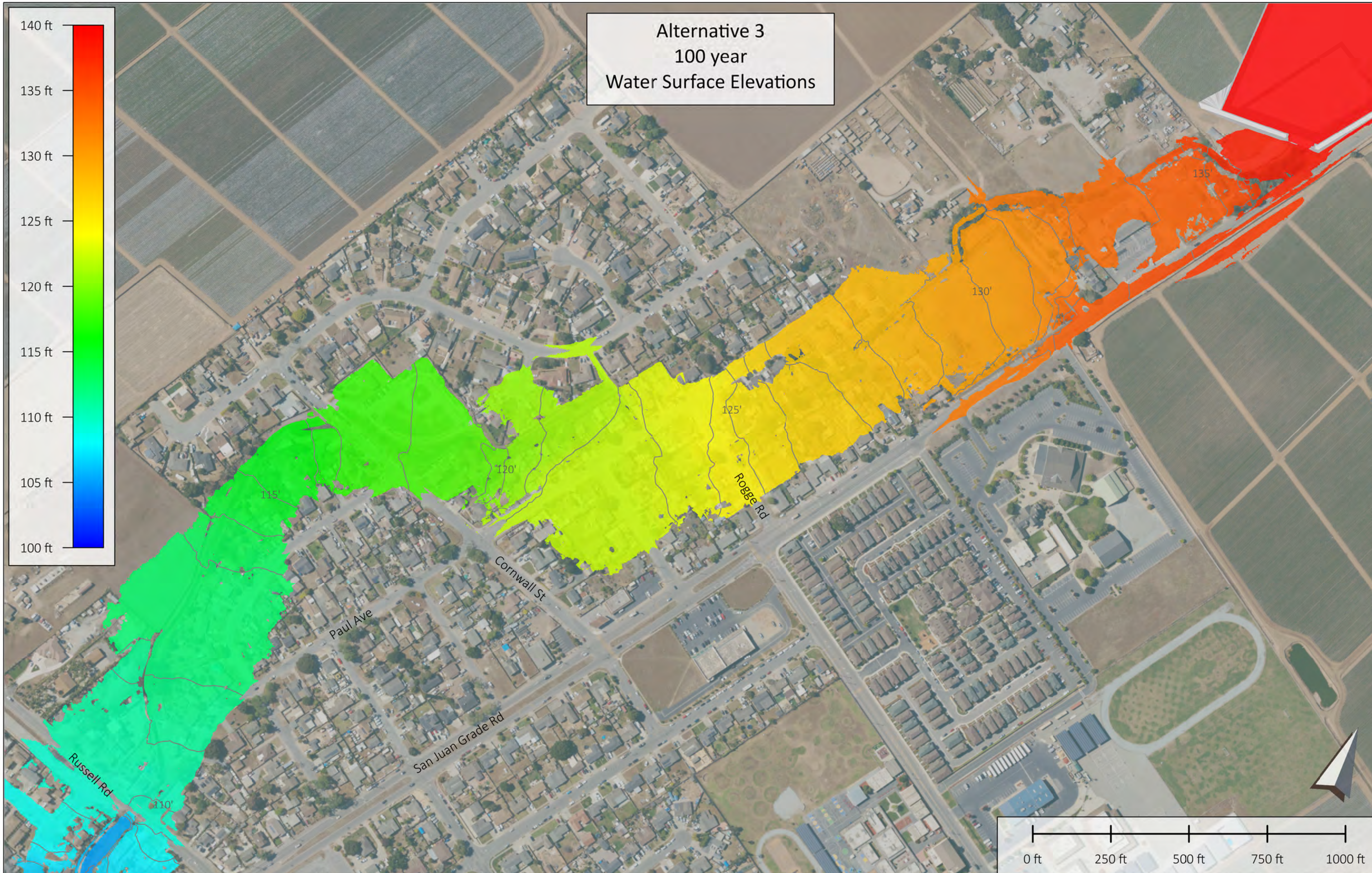

Alternative 2
10 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



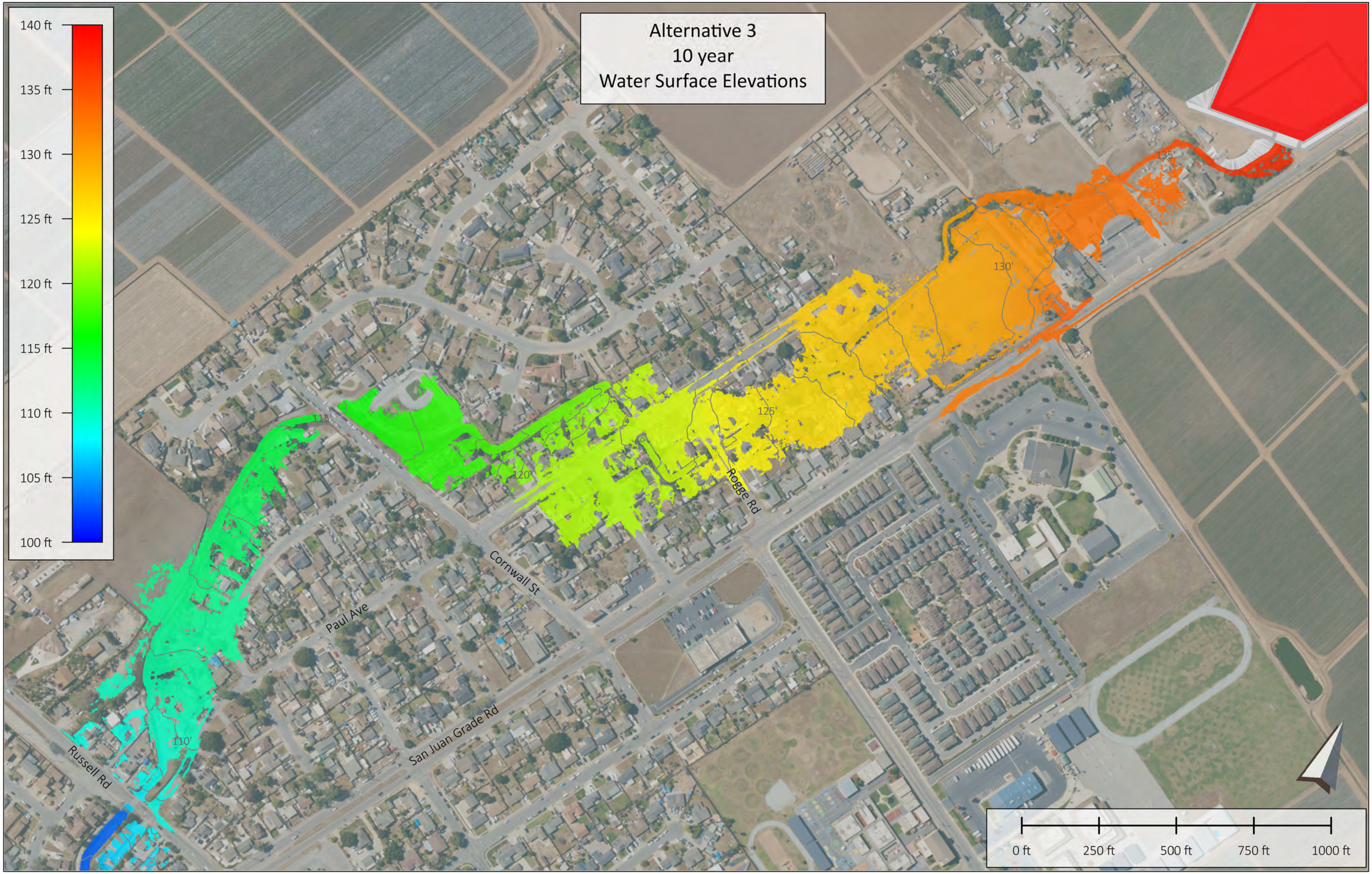
Alternative 2
December 27, 2022
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
114'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



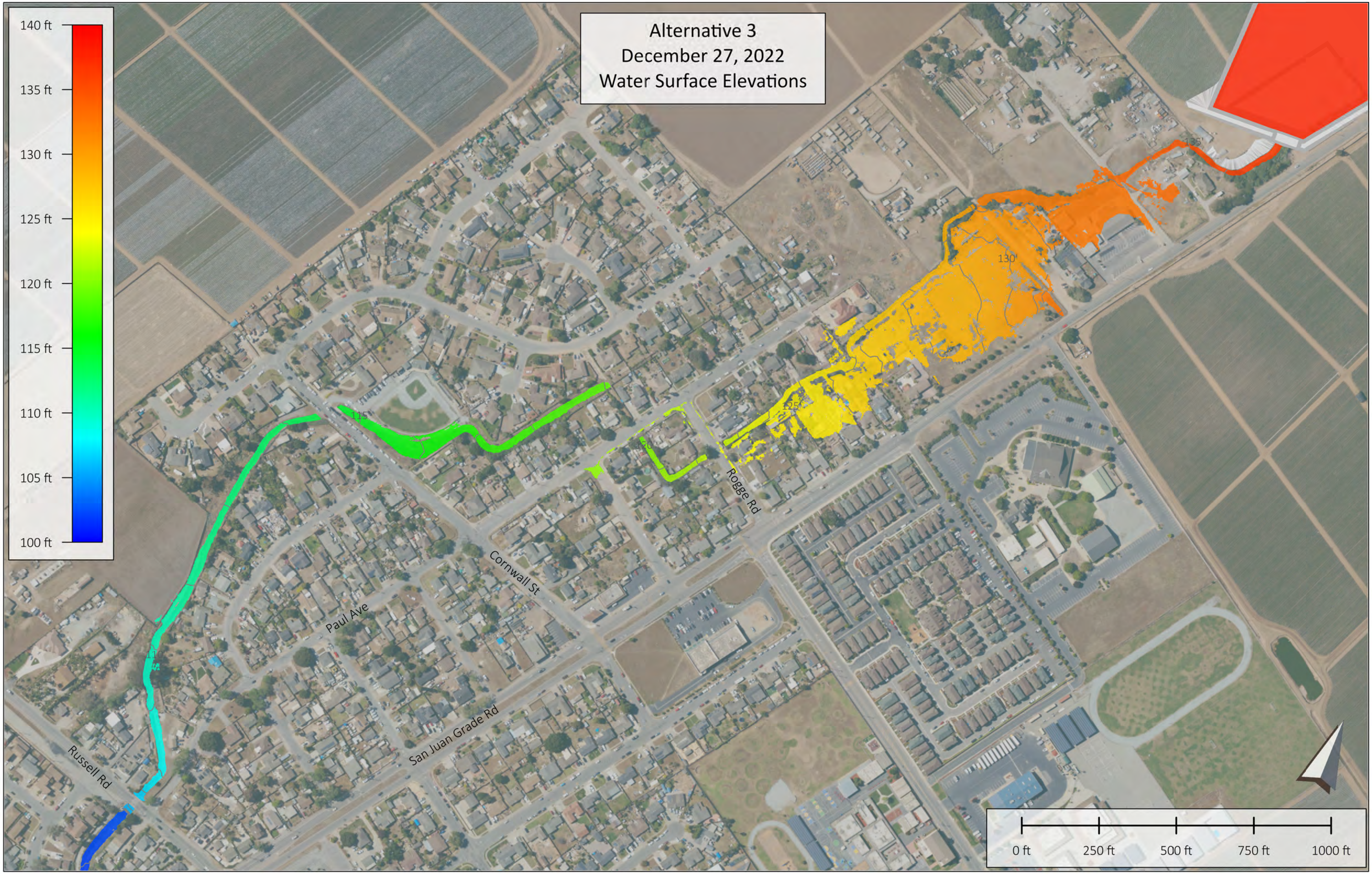
Alternative 2
February 13, 2025
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



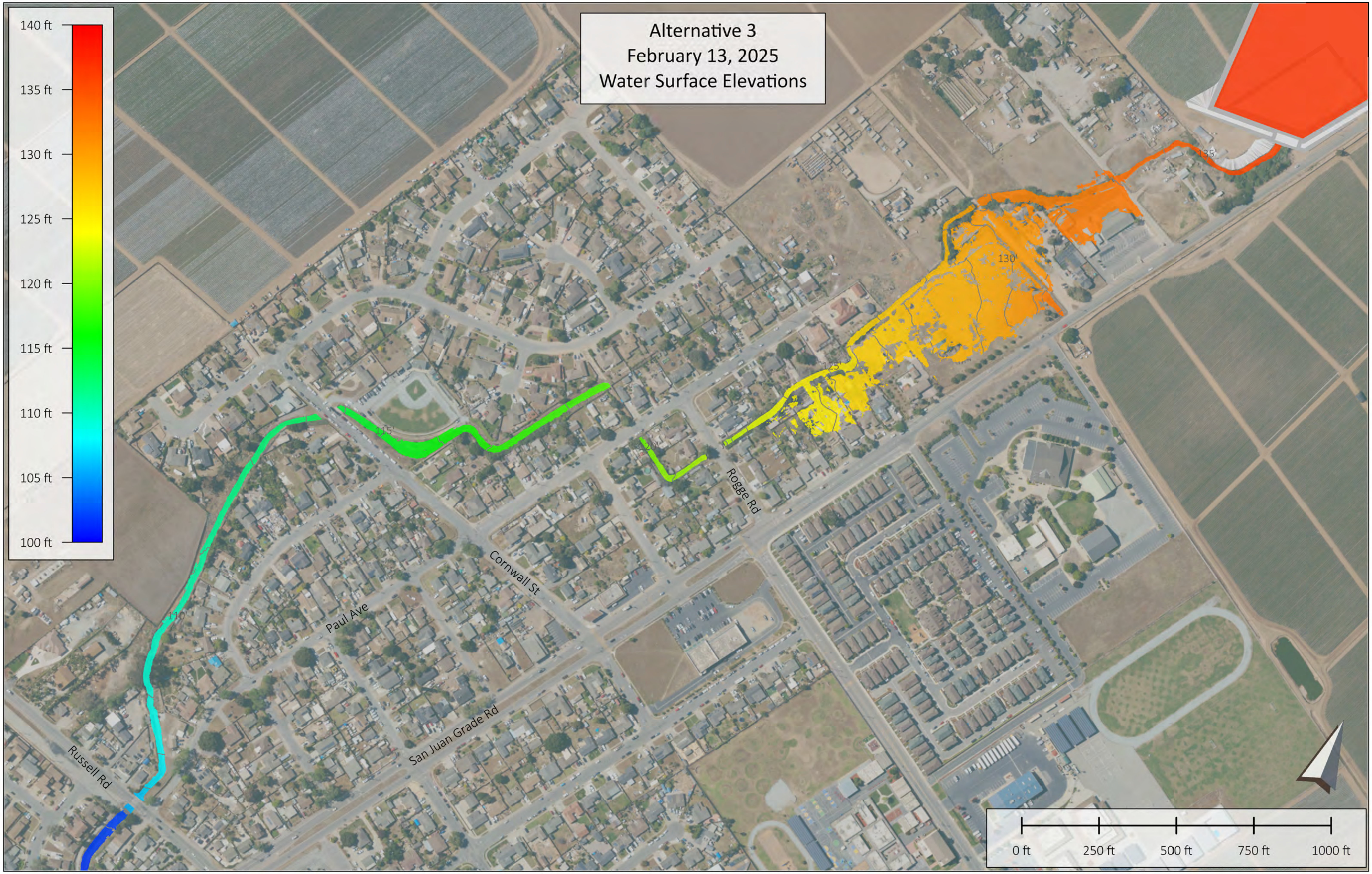
Alternative 3
100 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



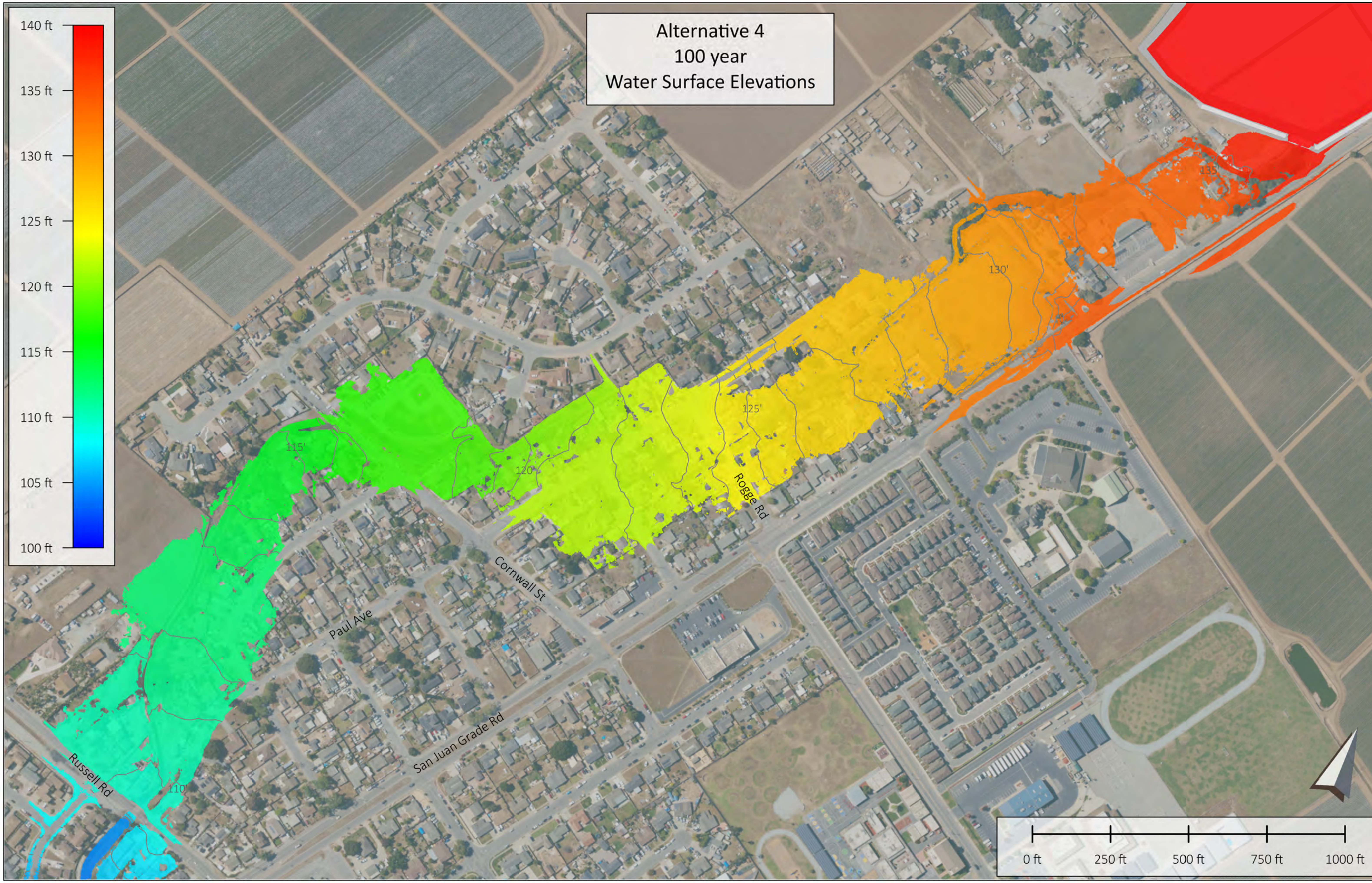
Alternative 3
10 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



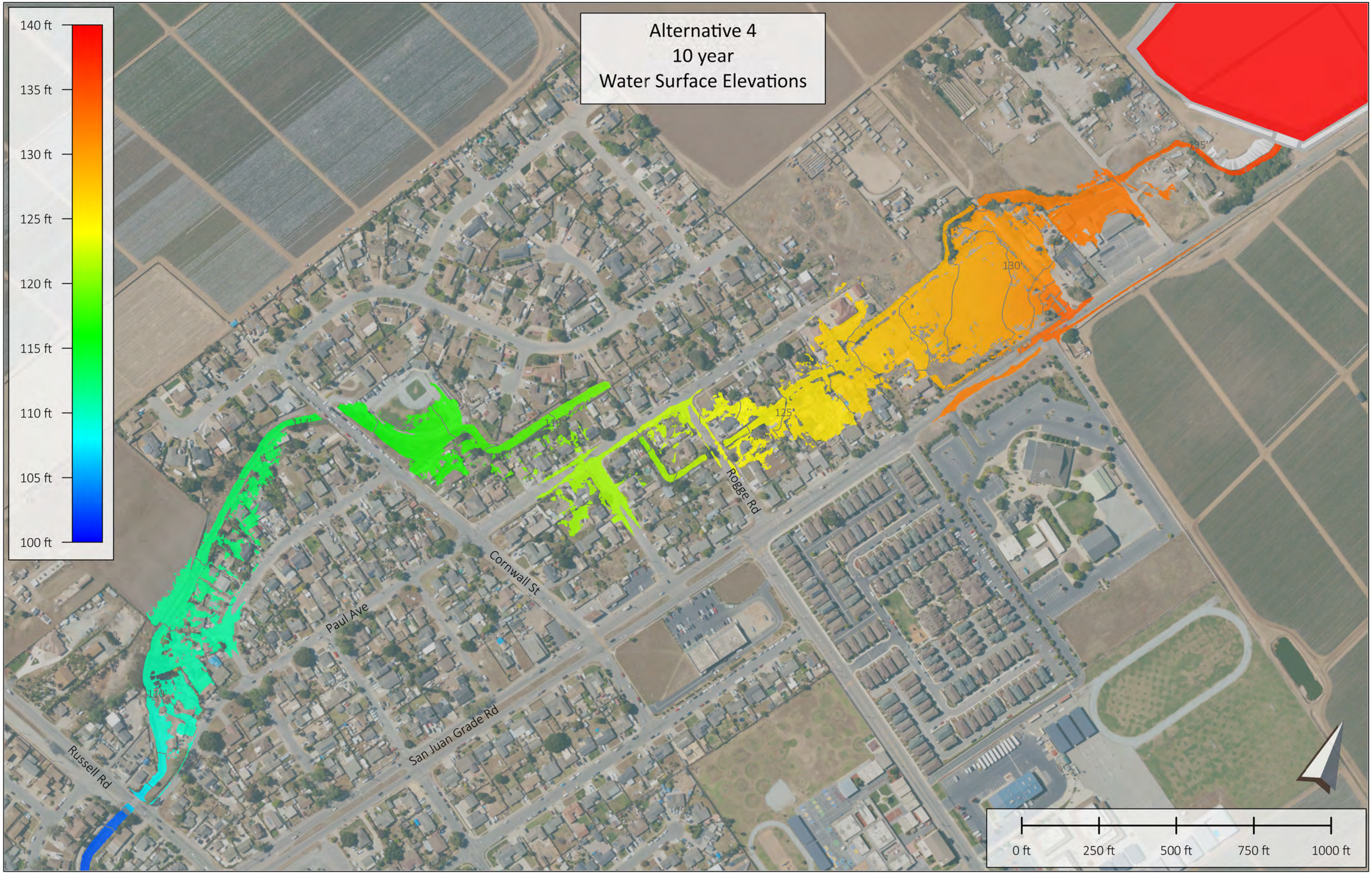
Alternative 3
December 27, 2022
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



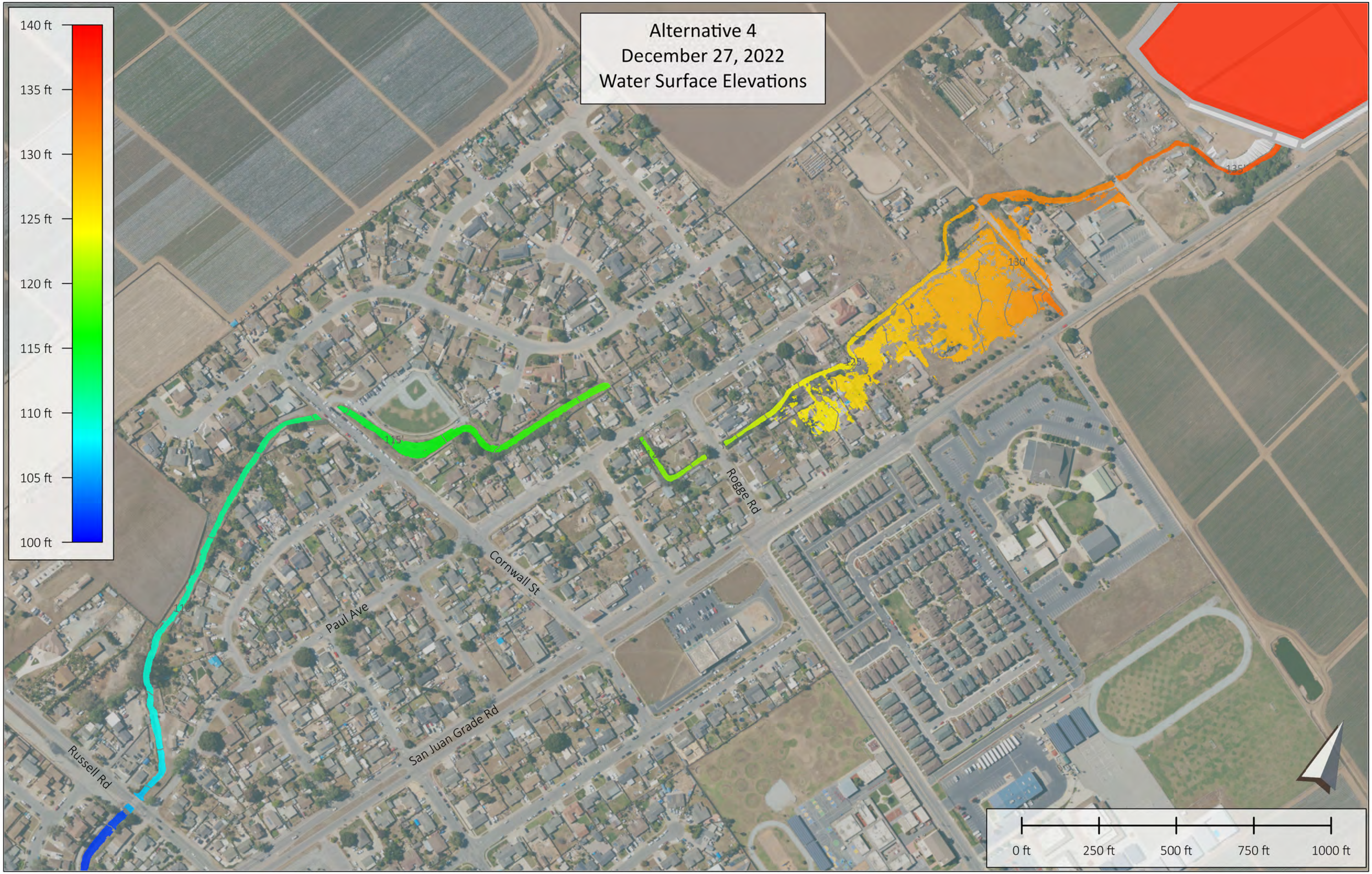
Alternative 3
February 13, 2025
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

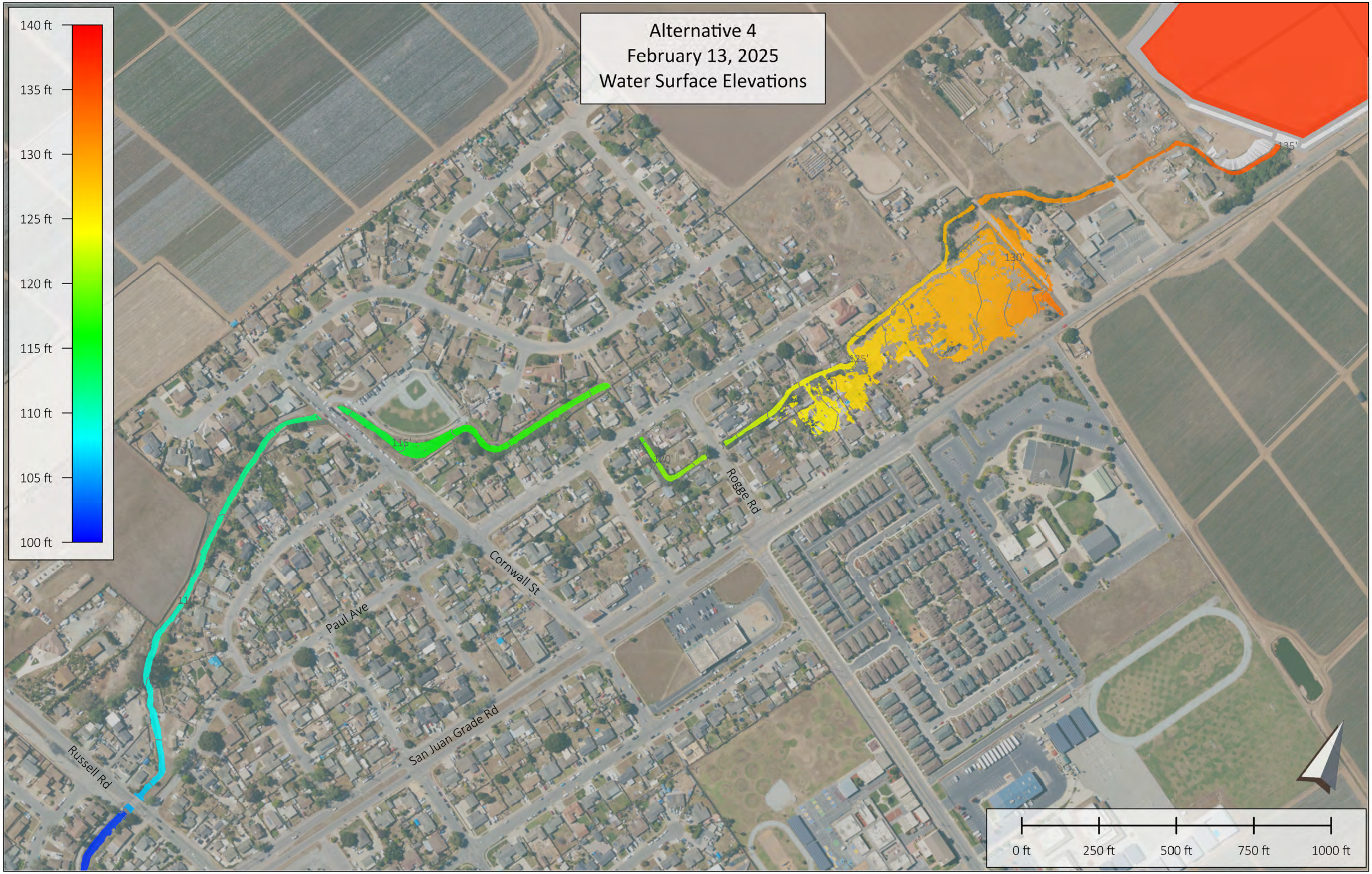


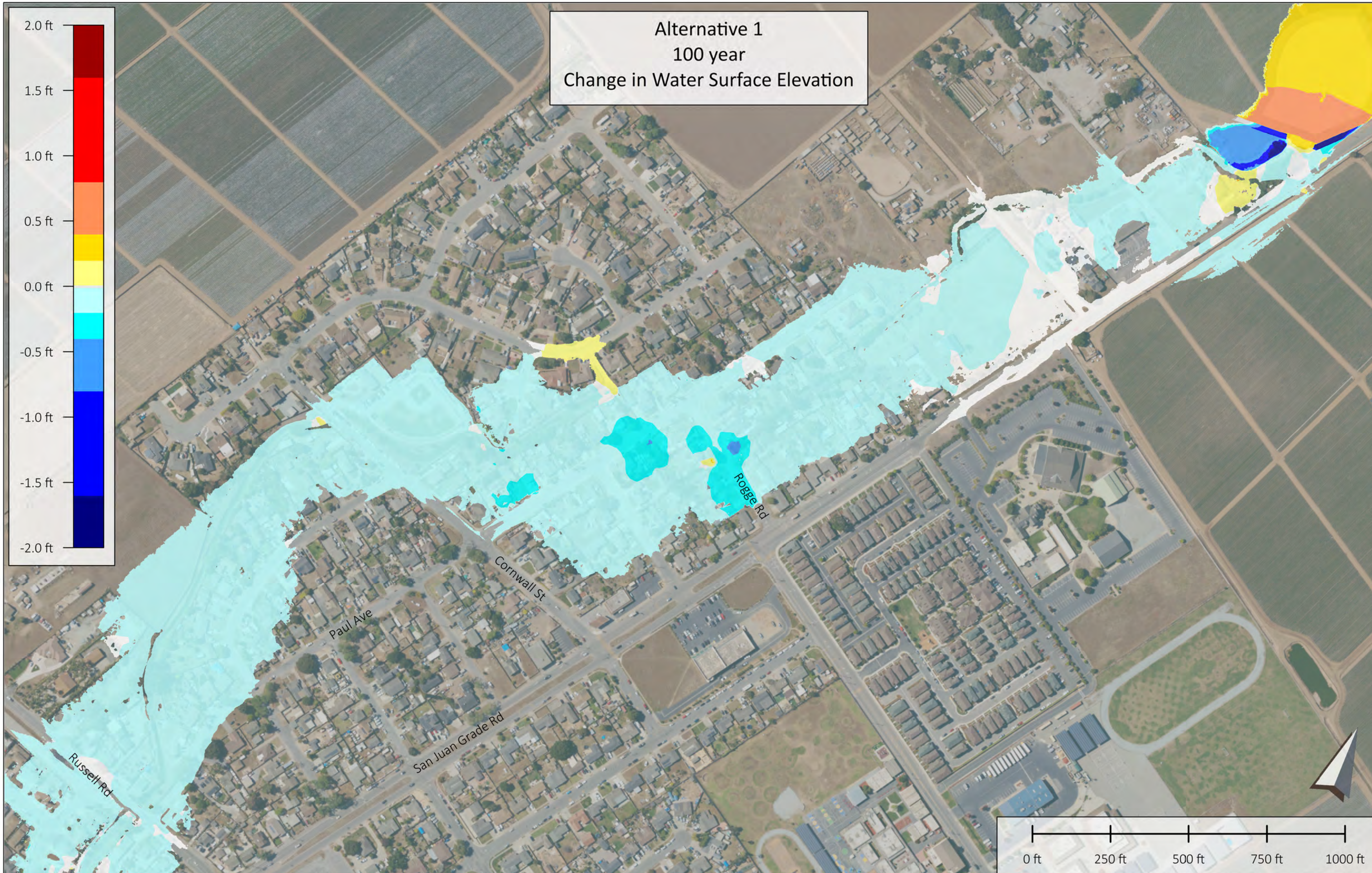
Alternative 4
100 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



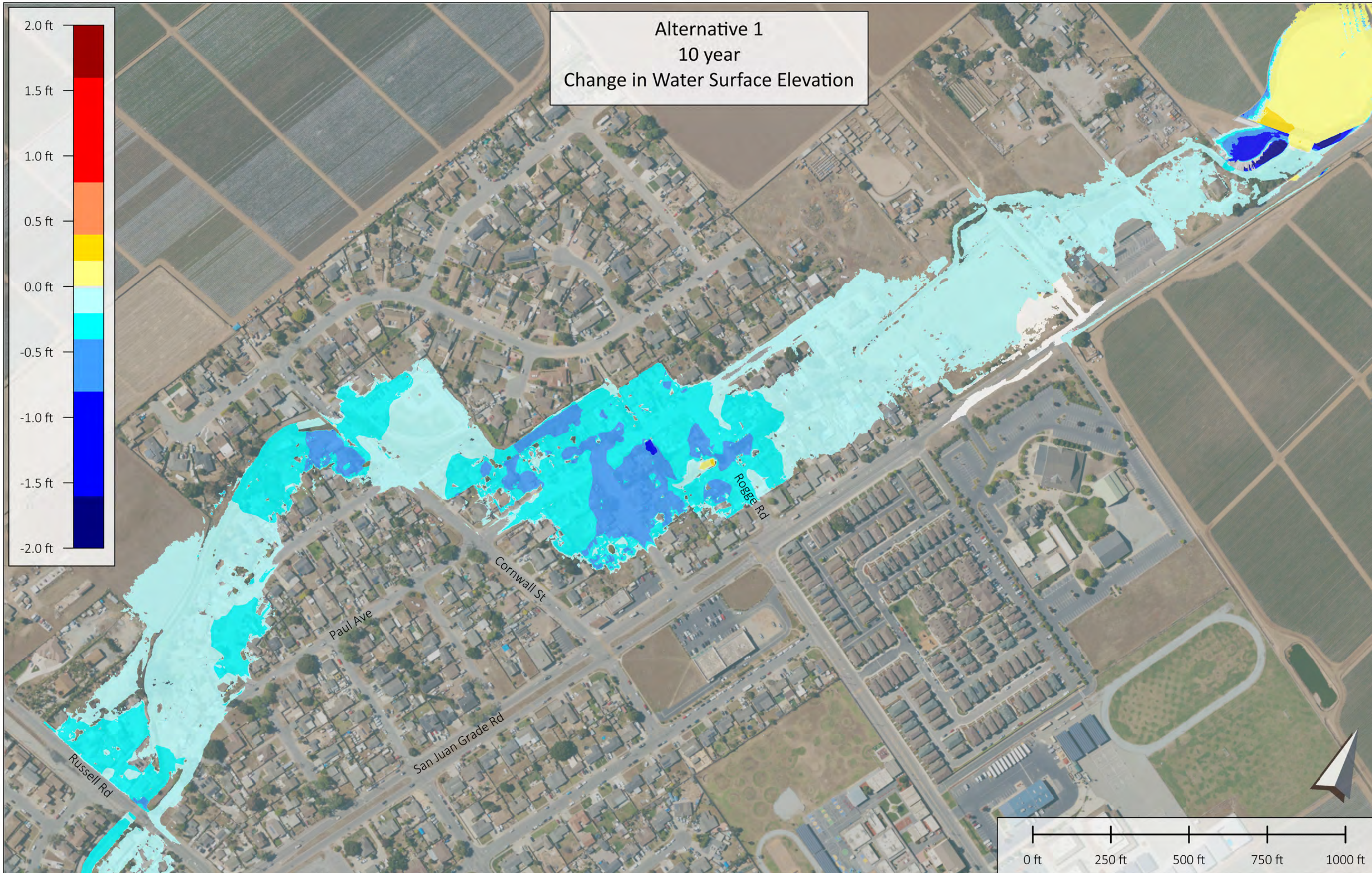
Alternative 4
10 year
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
119'
114'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

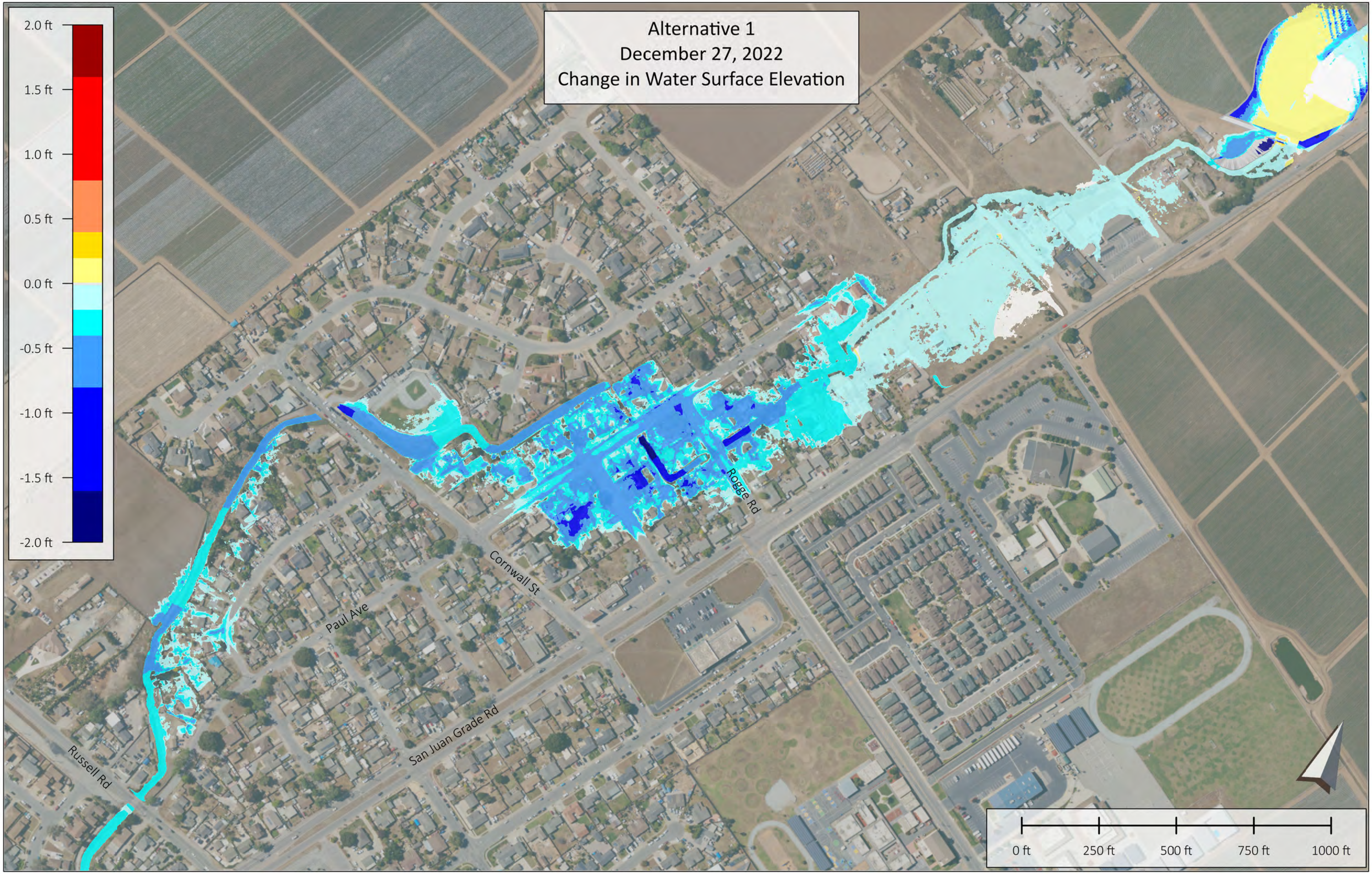

Alternative 4
December 27, 2022
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
115'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Alternative 4
February 13, 2025
Water Surface Elevations
140 ft
135 ft
130 ft
125 ft
120 ft
115 ft
110 ft
105 ft
100 ft
135'
130'
125'
120'
115'
110'
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

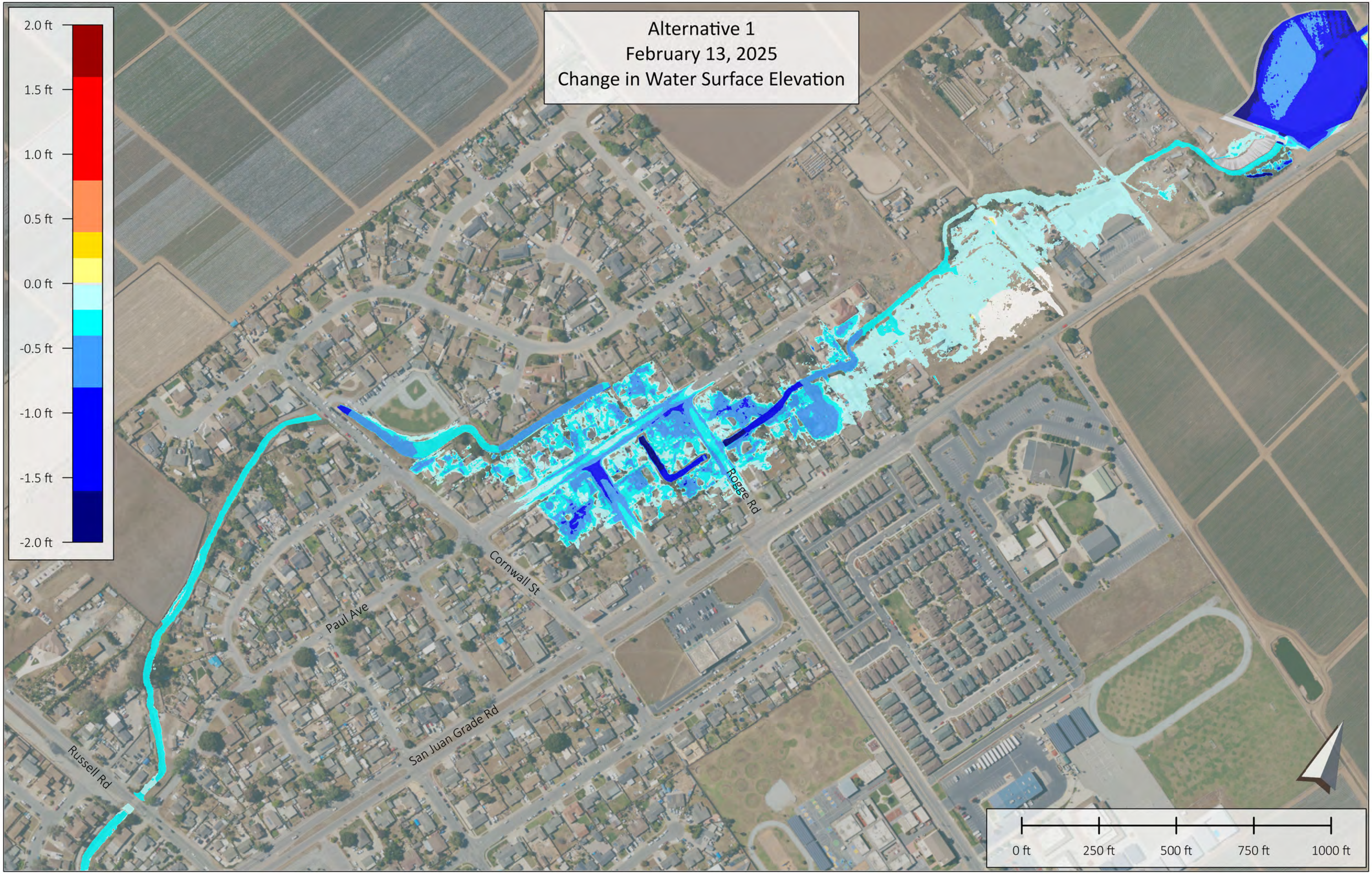


Alternative 1
100 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

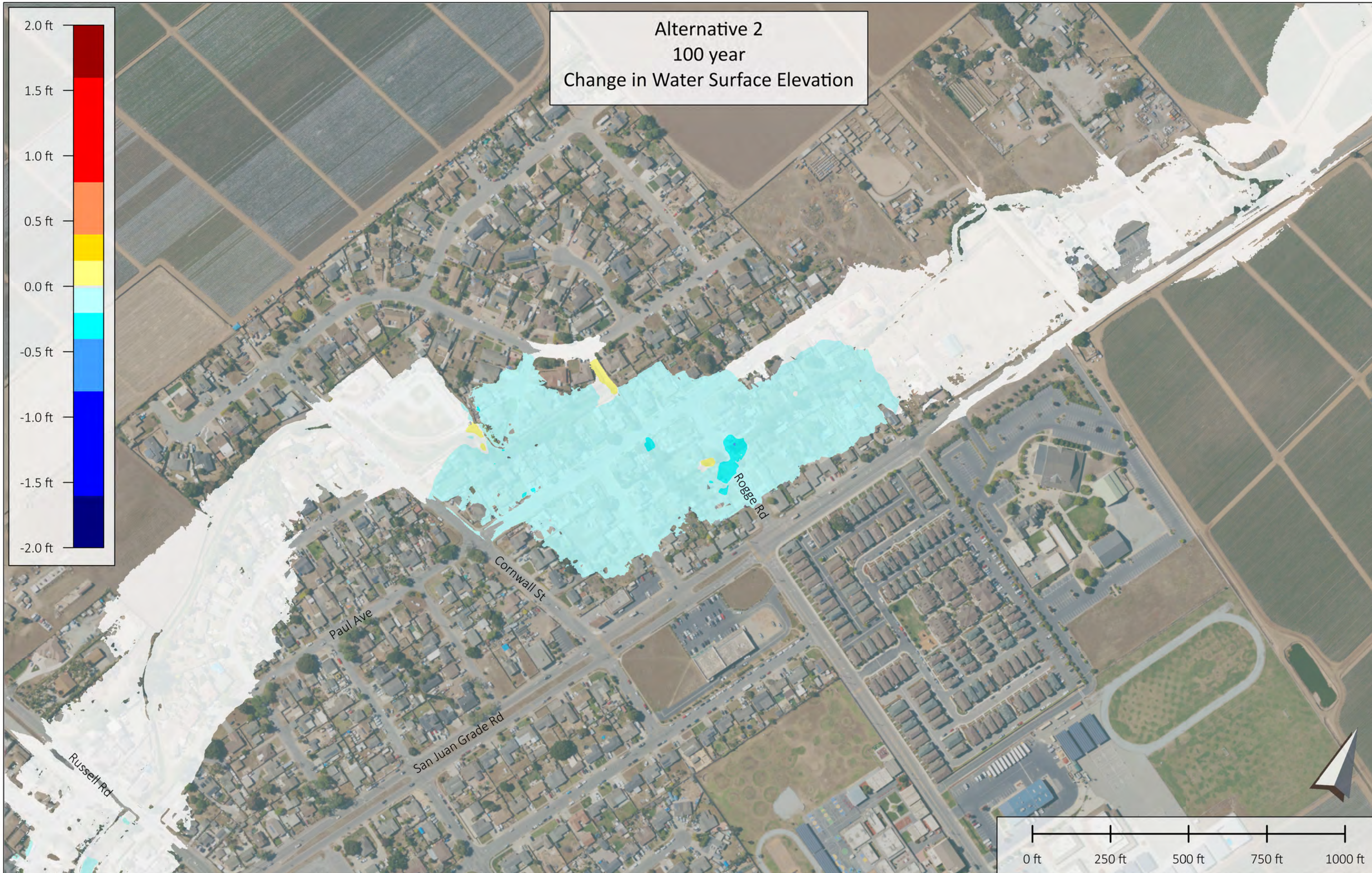

Alternative 1
10 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

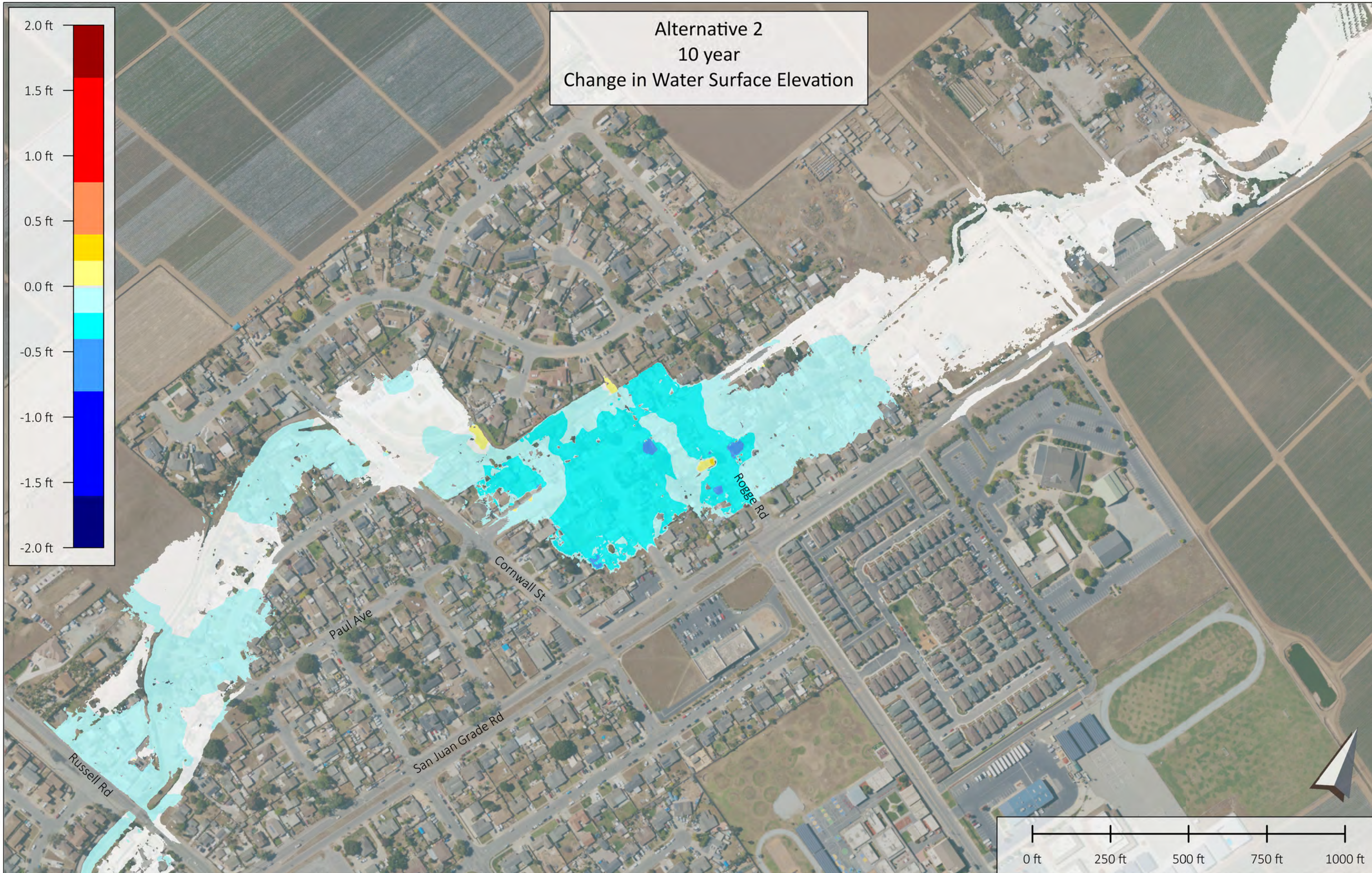


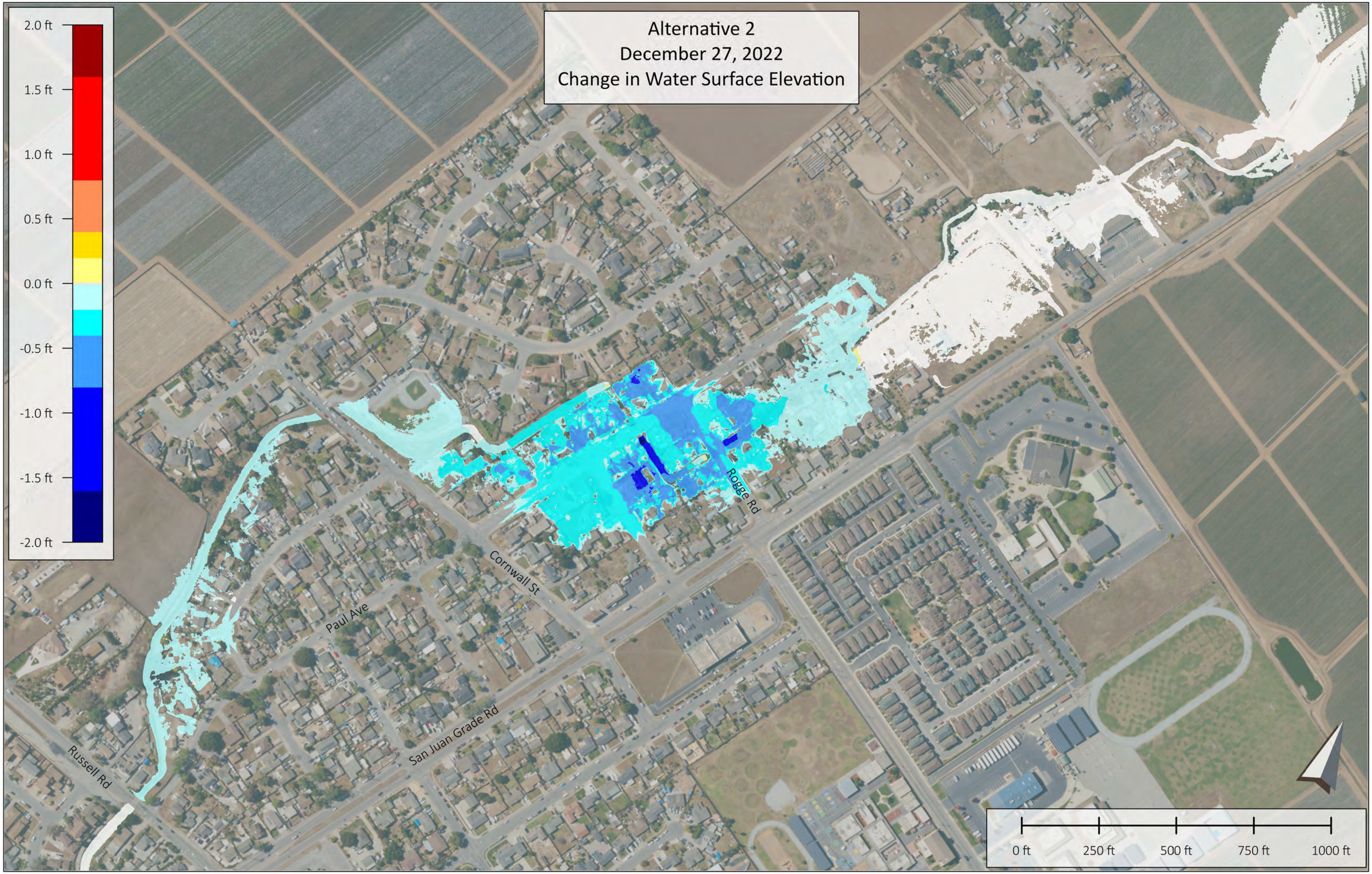
Alternative 1
December 27, 2022
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



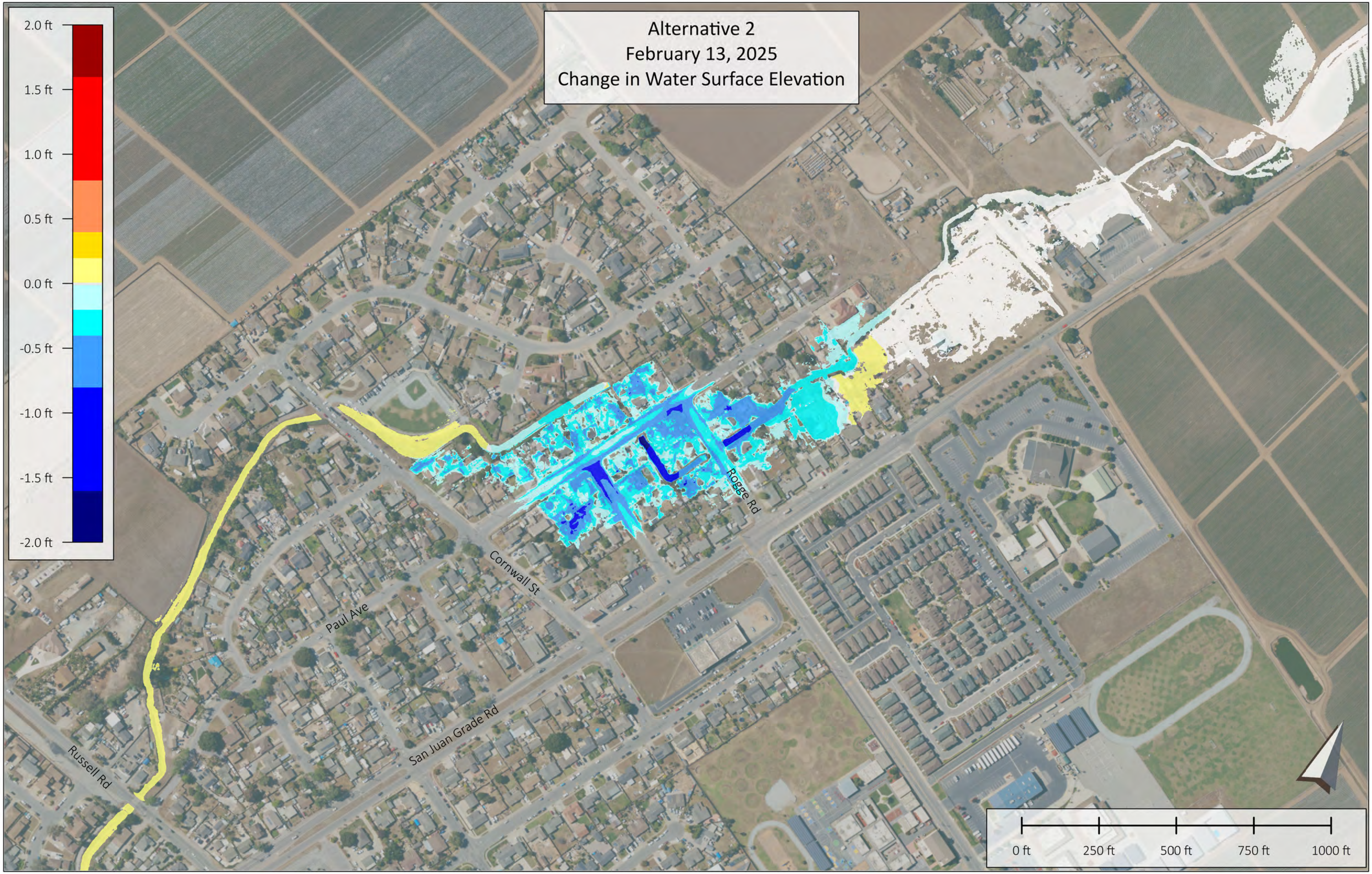
Alternative 1
February 13, 2025
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Alternative 2
100 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

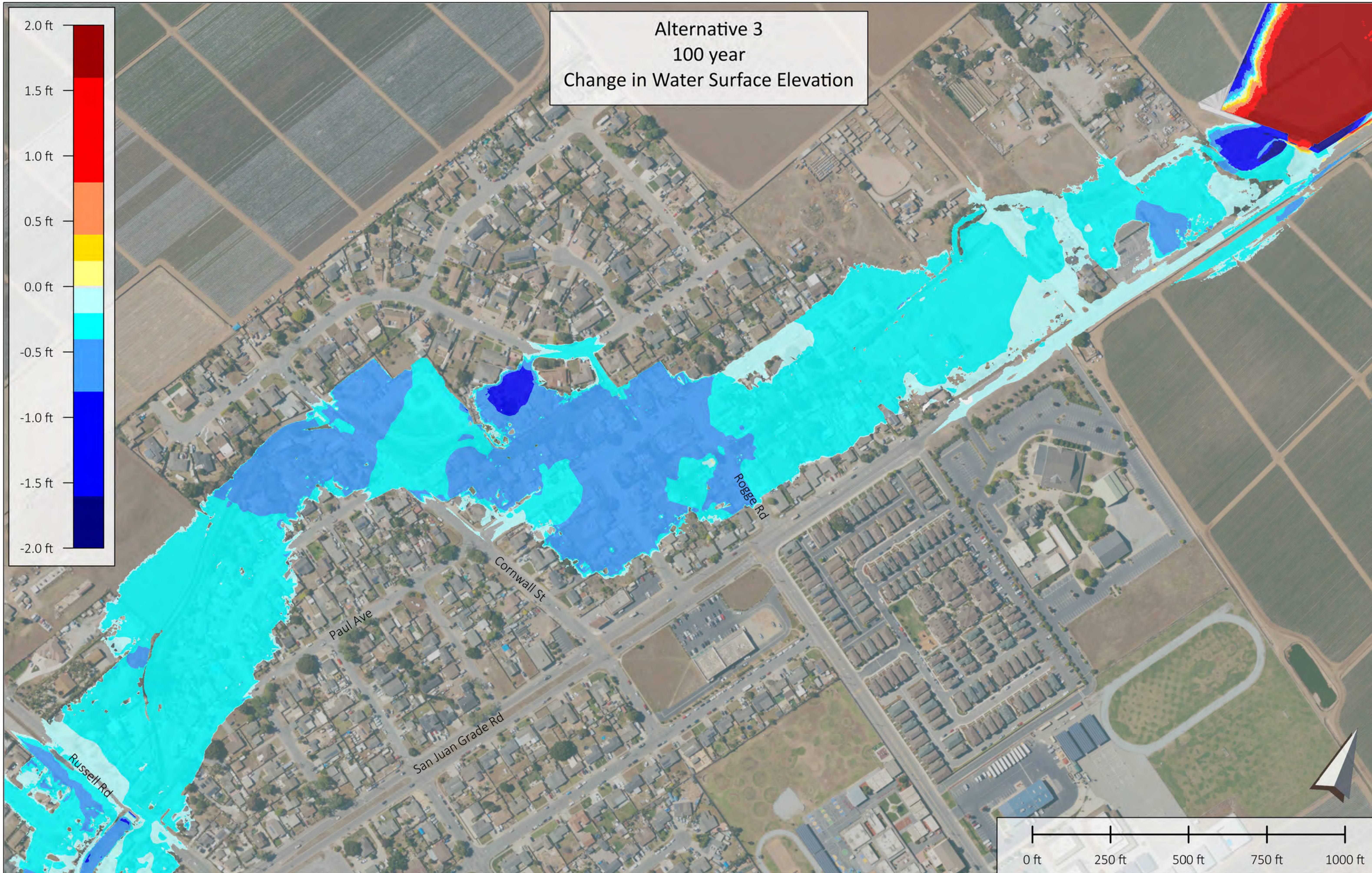

Alternative 2
10 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

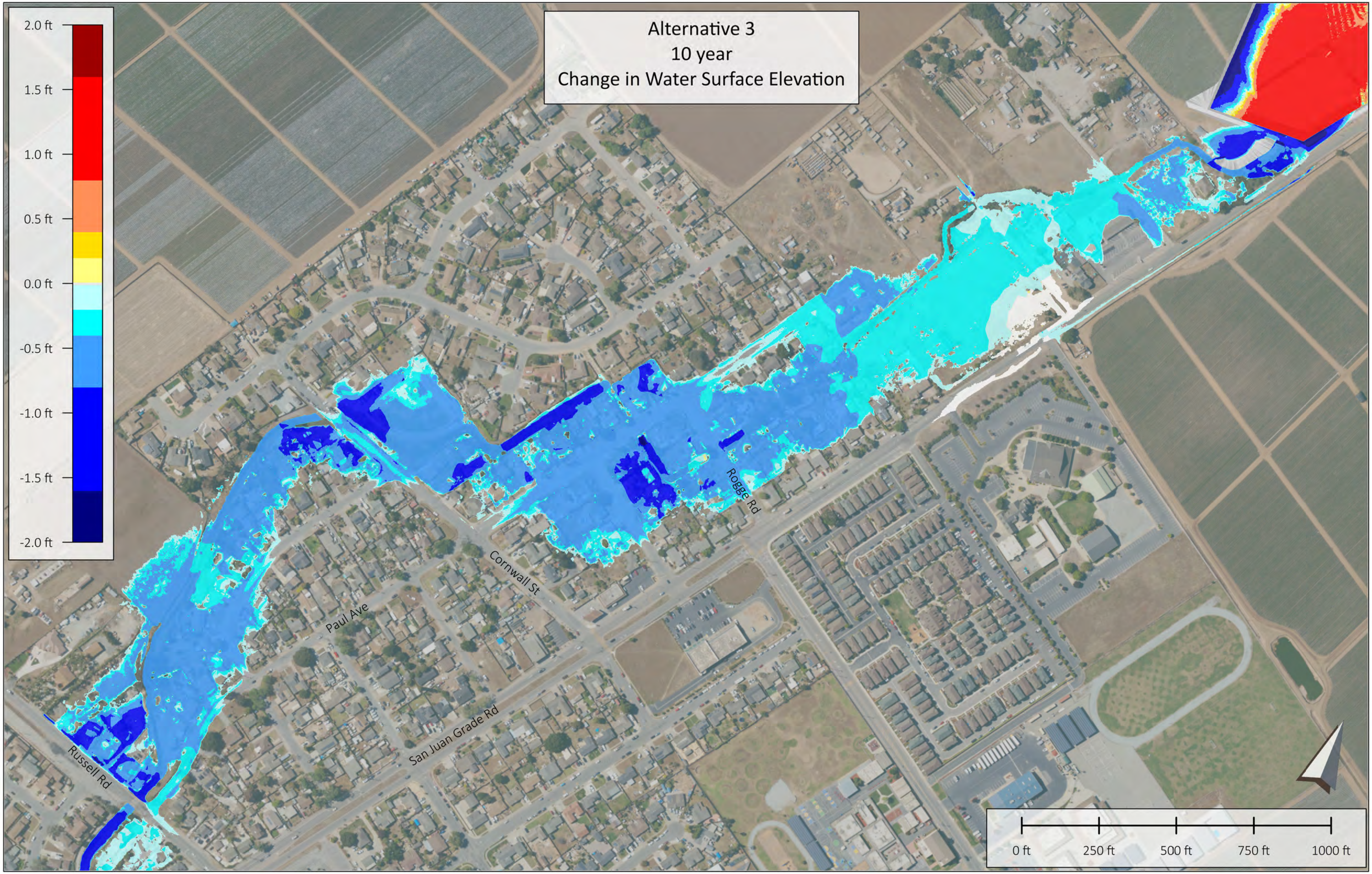


Alternative 2
December 27, 2022
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

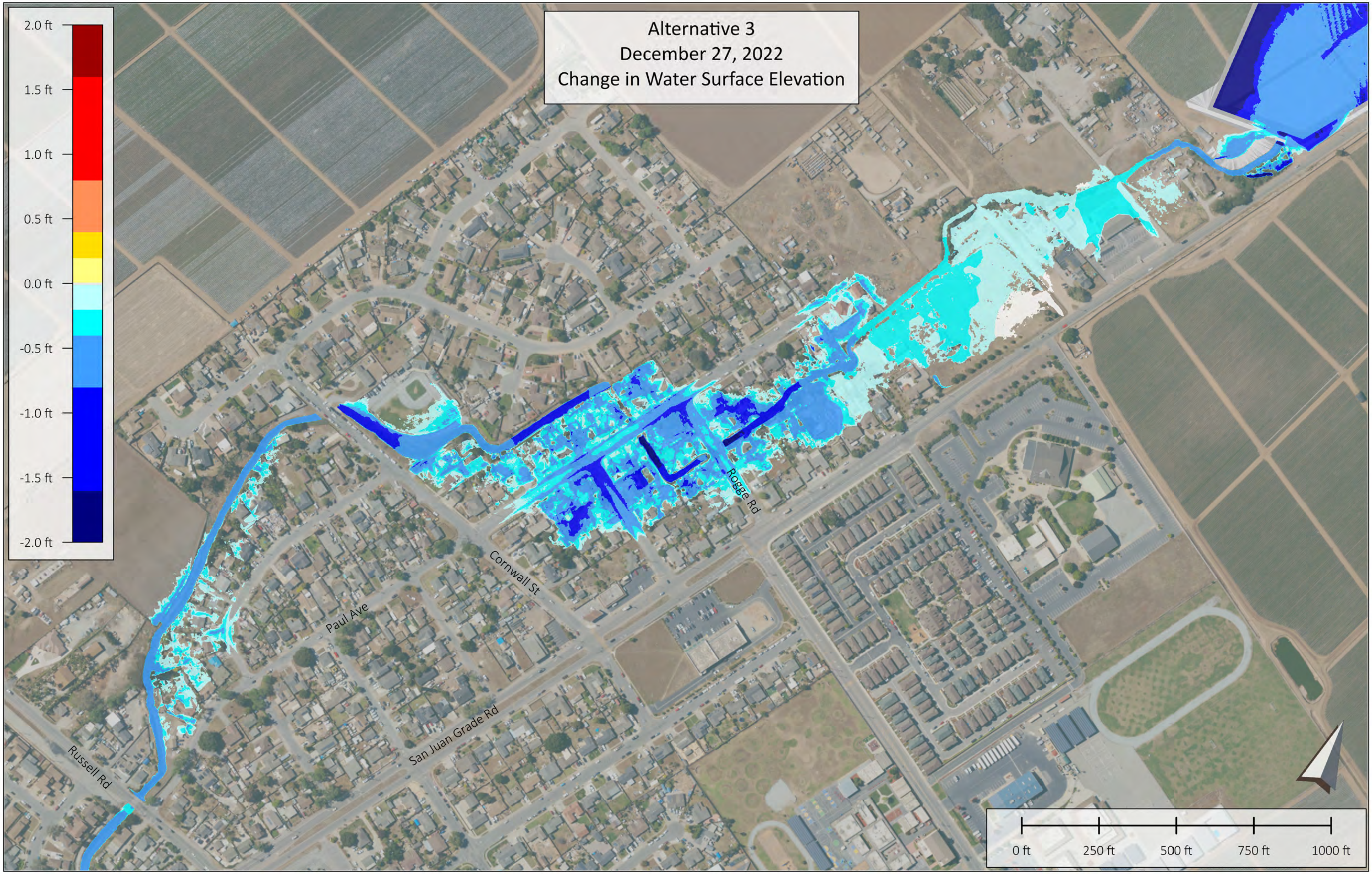


Alternative 2
February 13, 2025
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

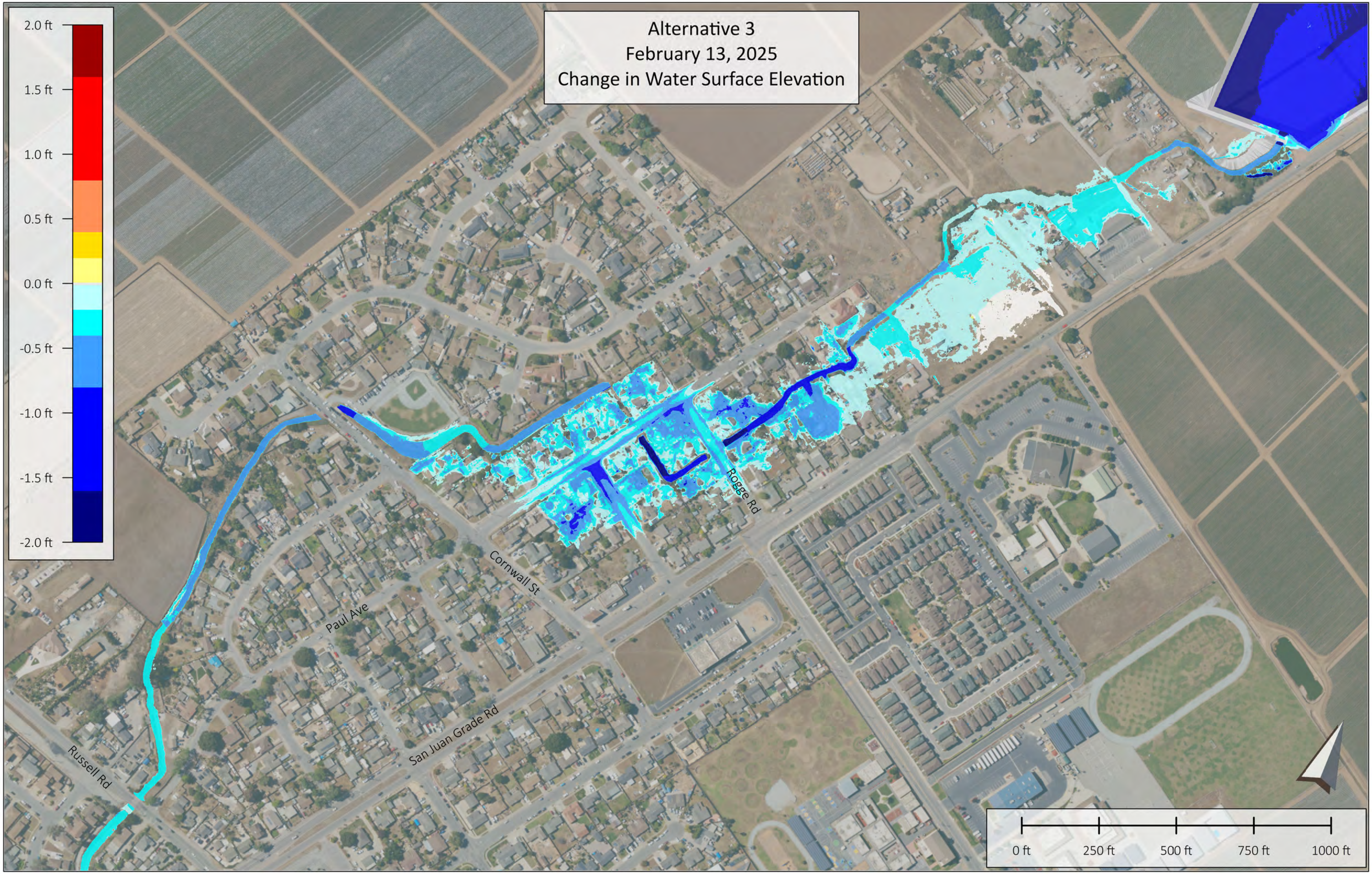

Alternative 3
100 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



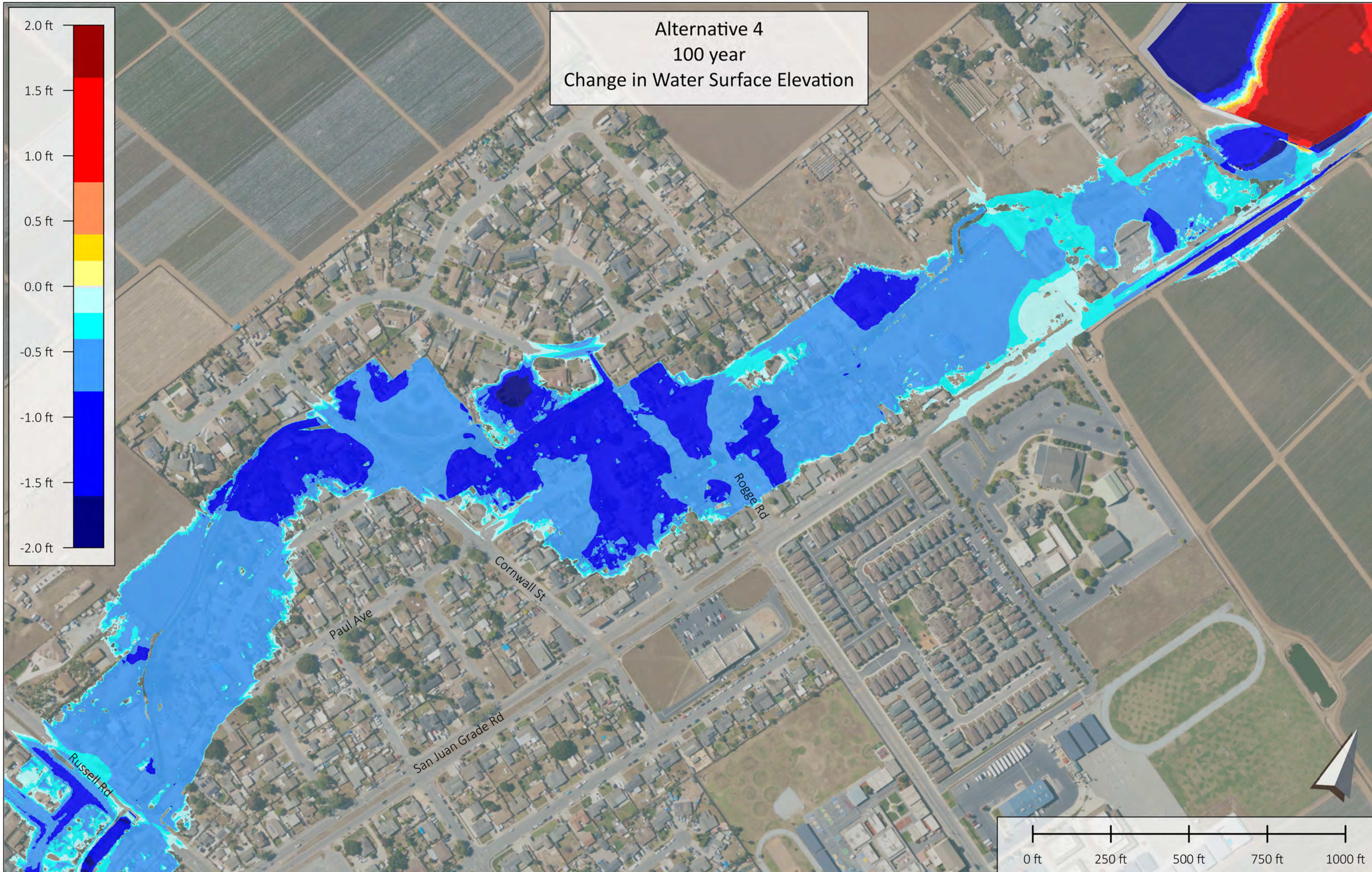
Alternative 3
10 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



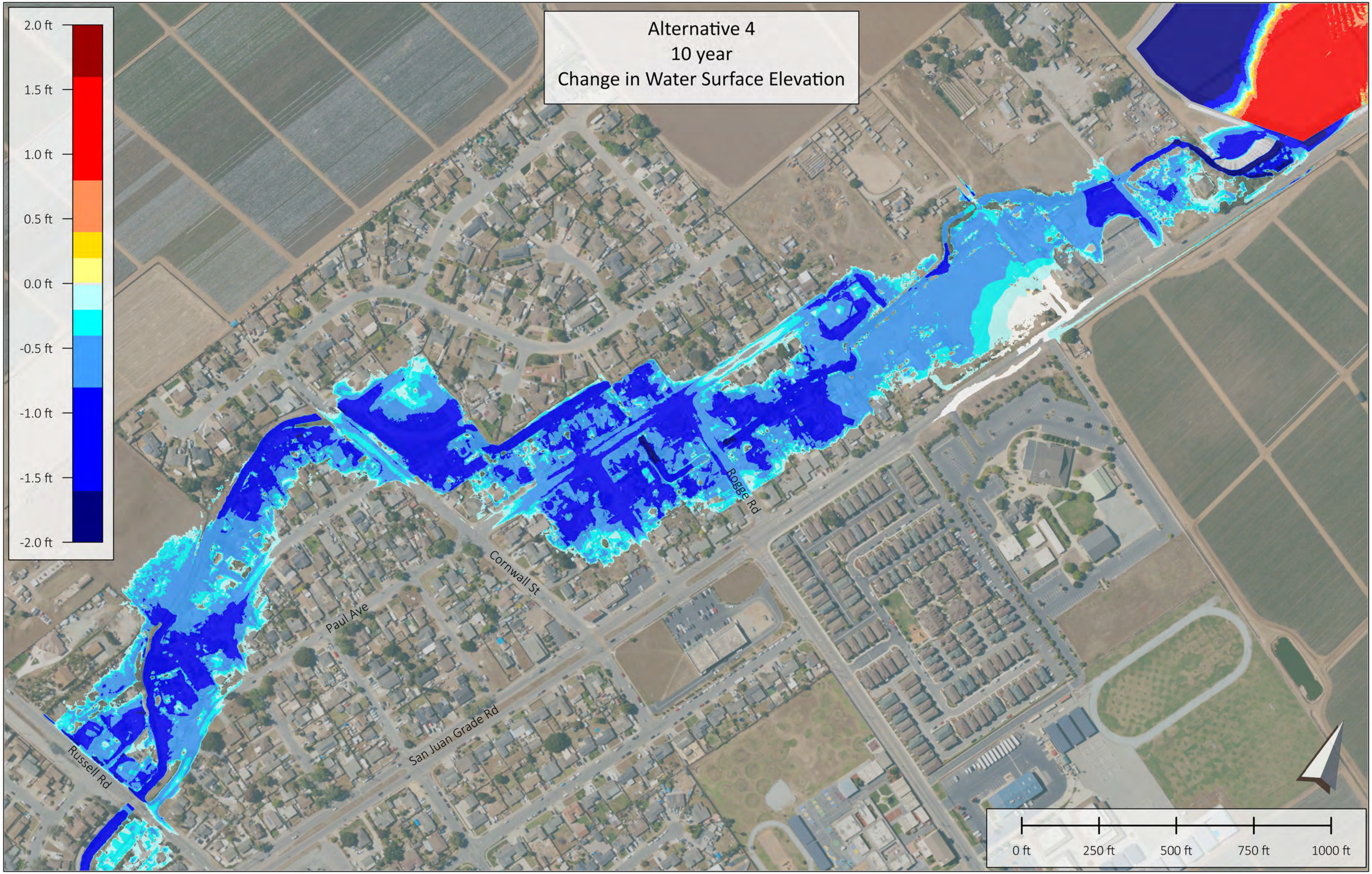
Alternative 3
December 27, 2022
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

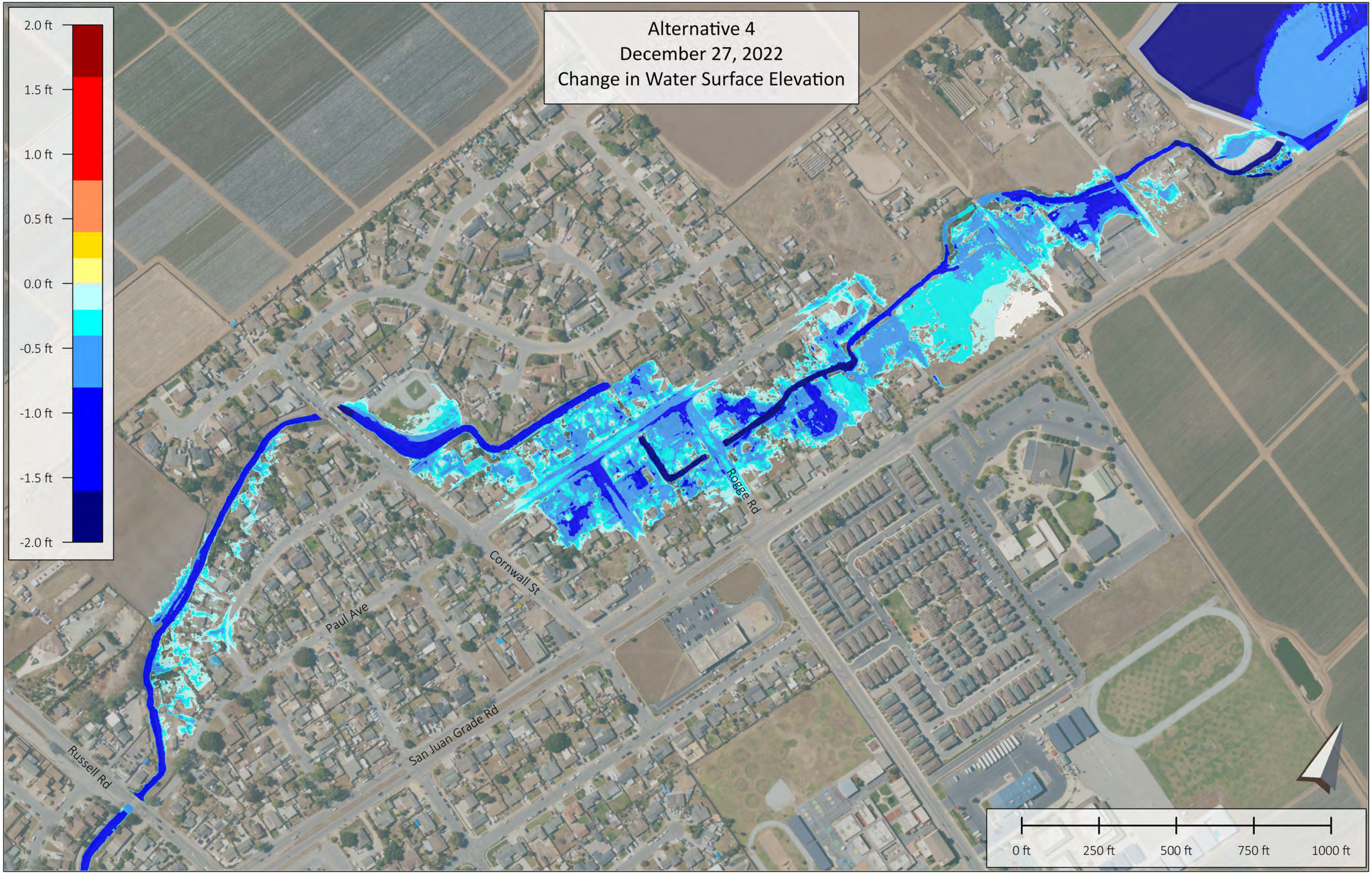


Alternative 3
February 13, 2025
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

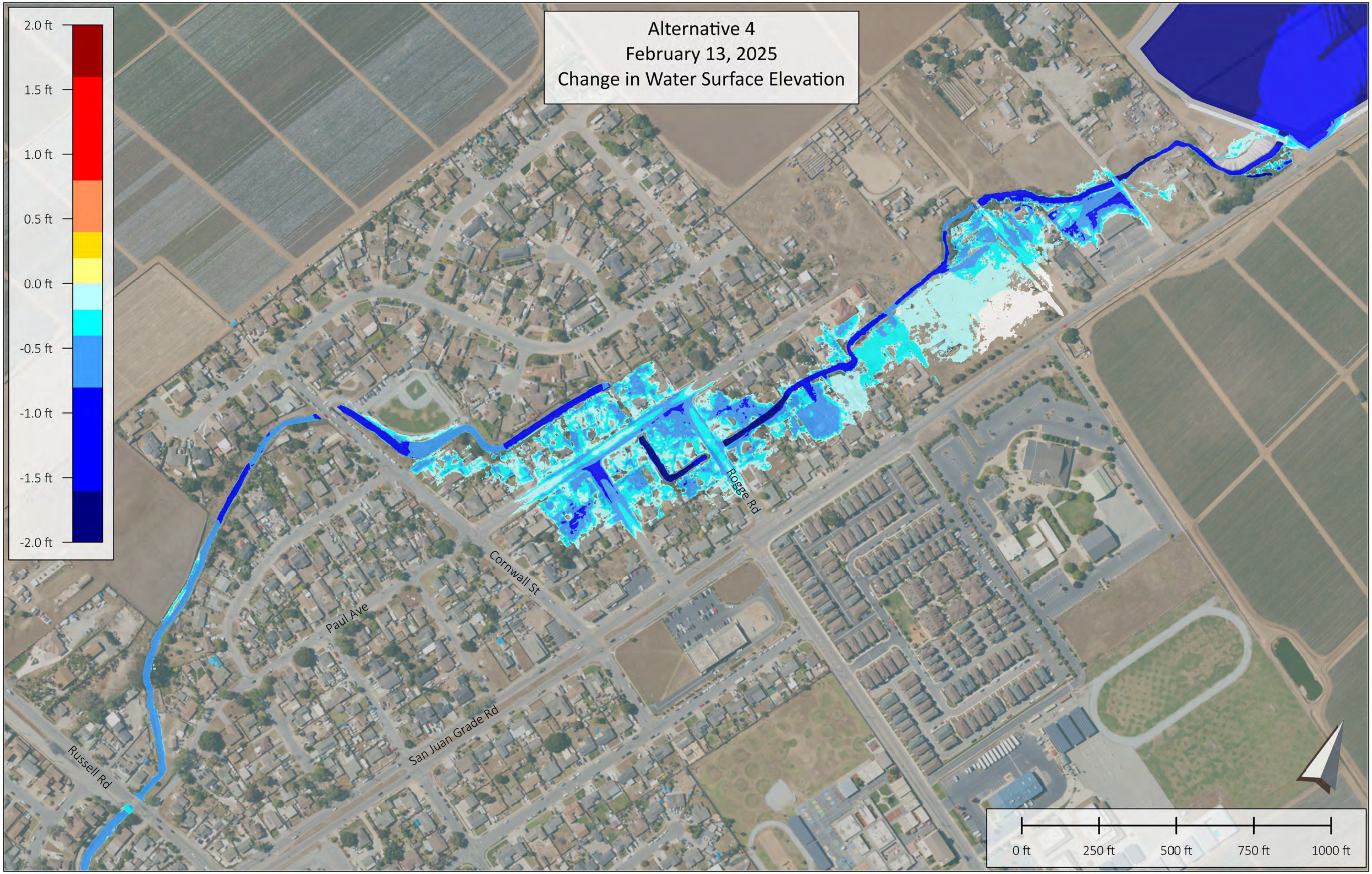


Alternative 4
100 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

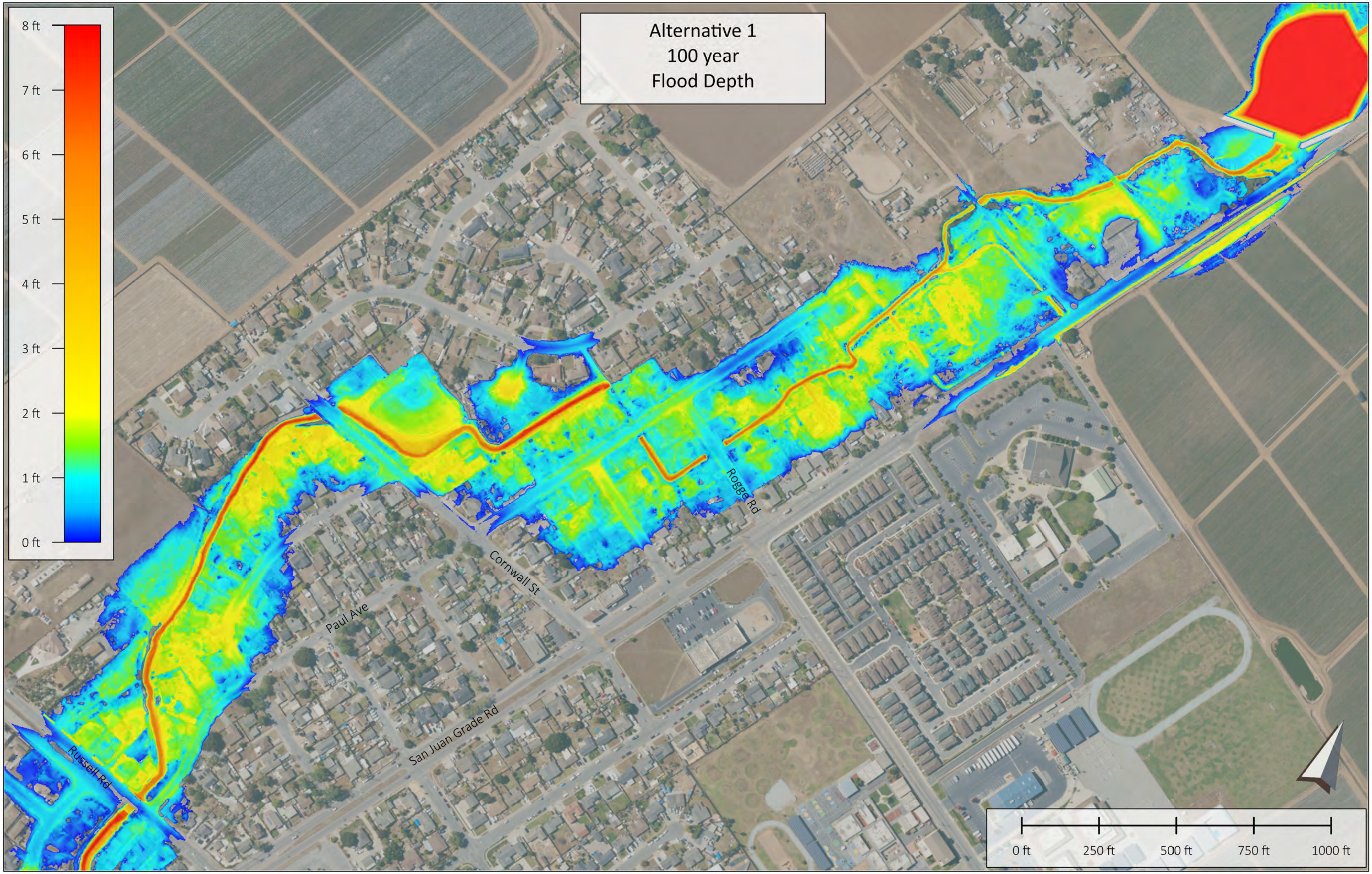

Alternative 4
10 year
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

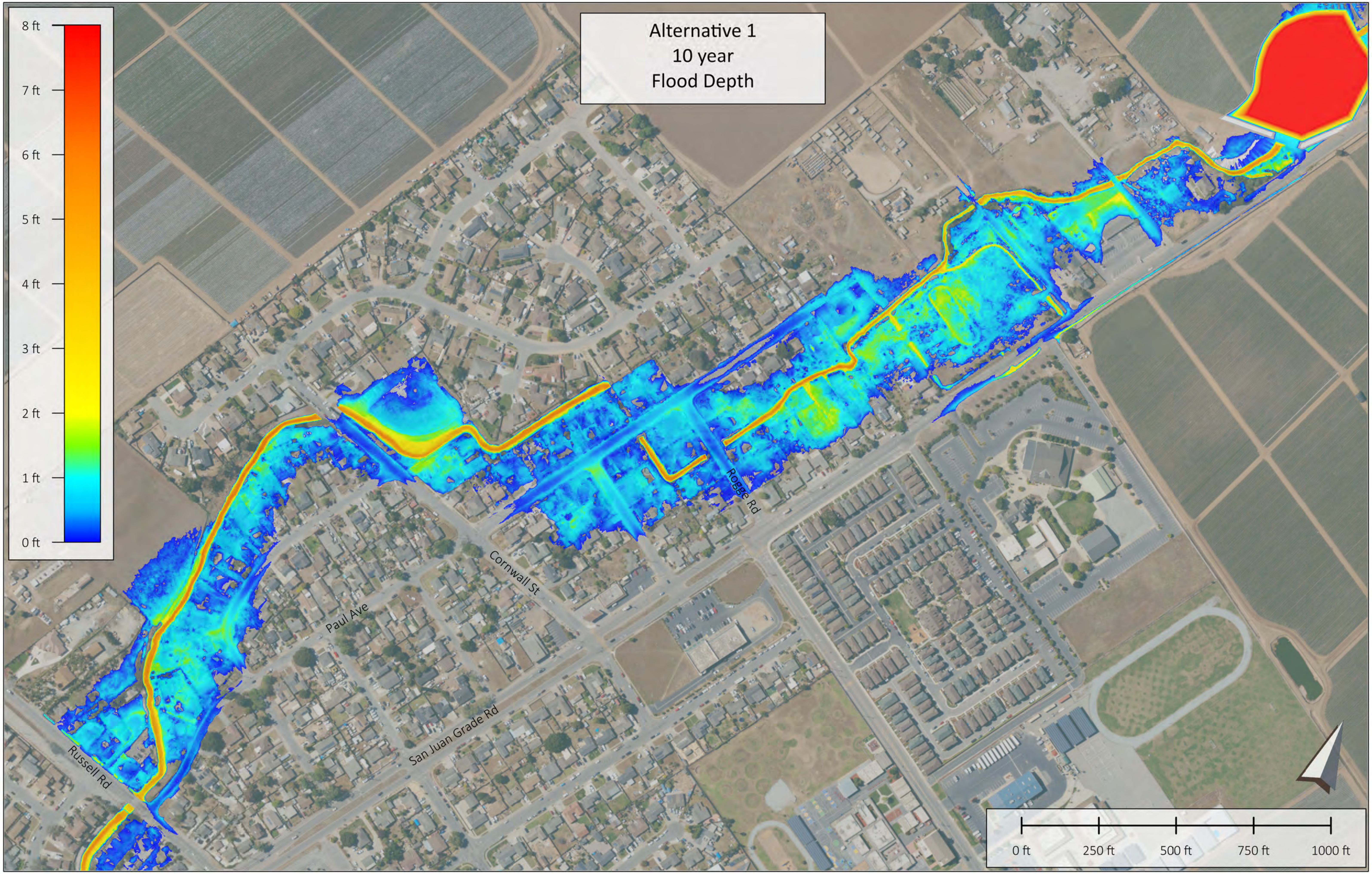


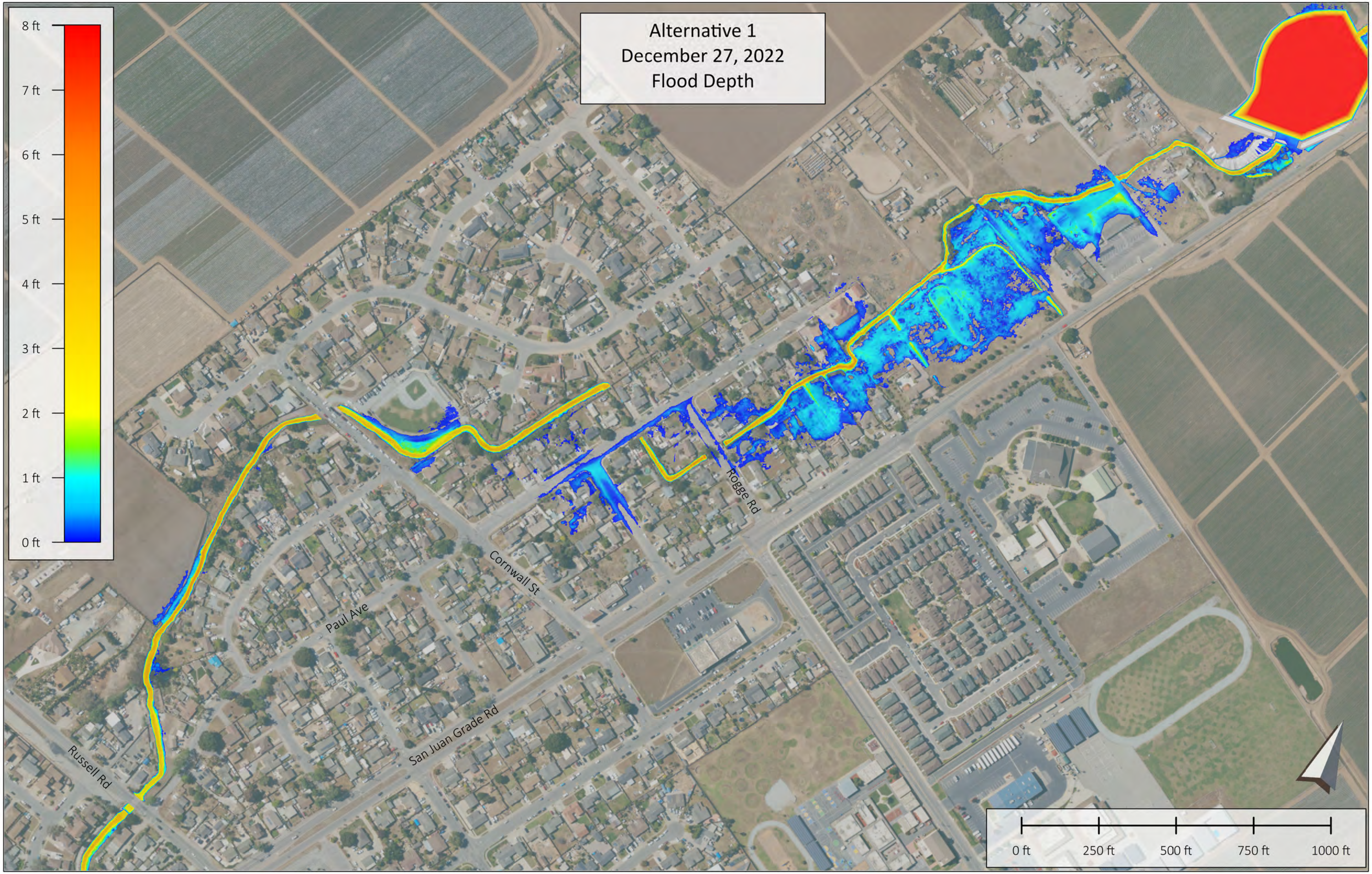
Alternative 4
December 27, 2022
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

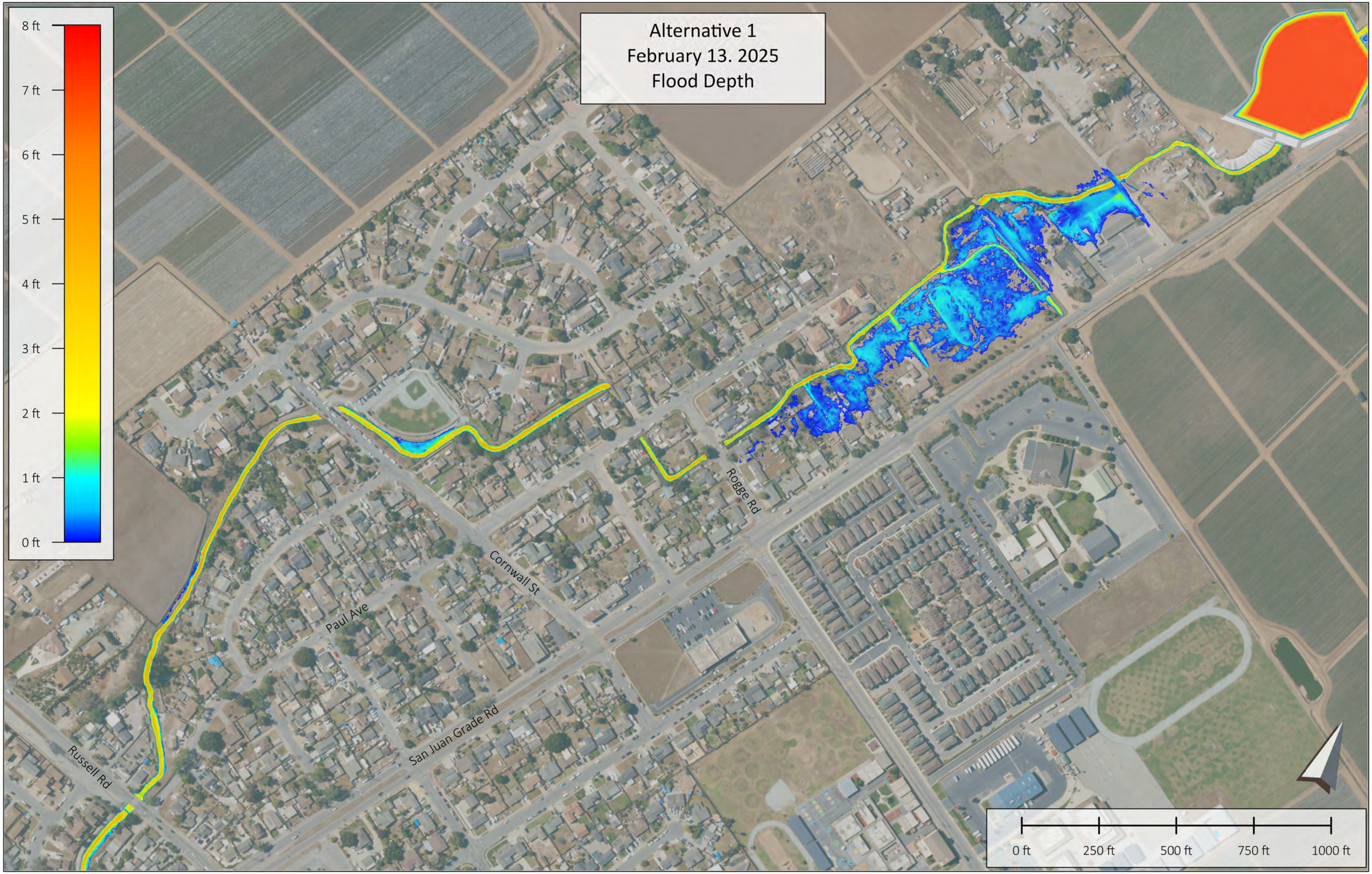


Alternative 4
February 13, 2025
Change in Water Surface Elevation
2.0 ft
1.5 ft
1.0 ft
0.5 ft
0.0 ft
-0.5 ft
-1.0 ft
-1.5 ft
-2.0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

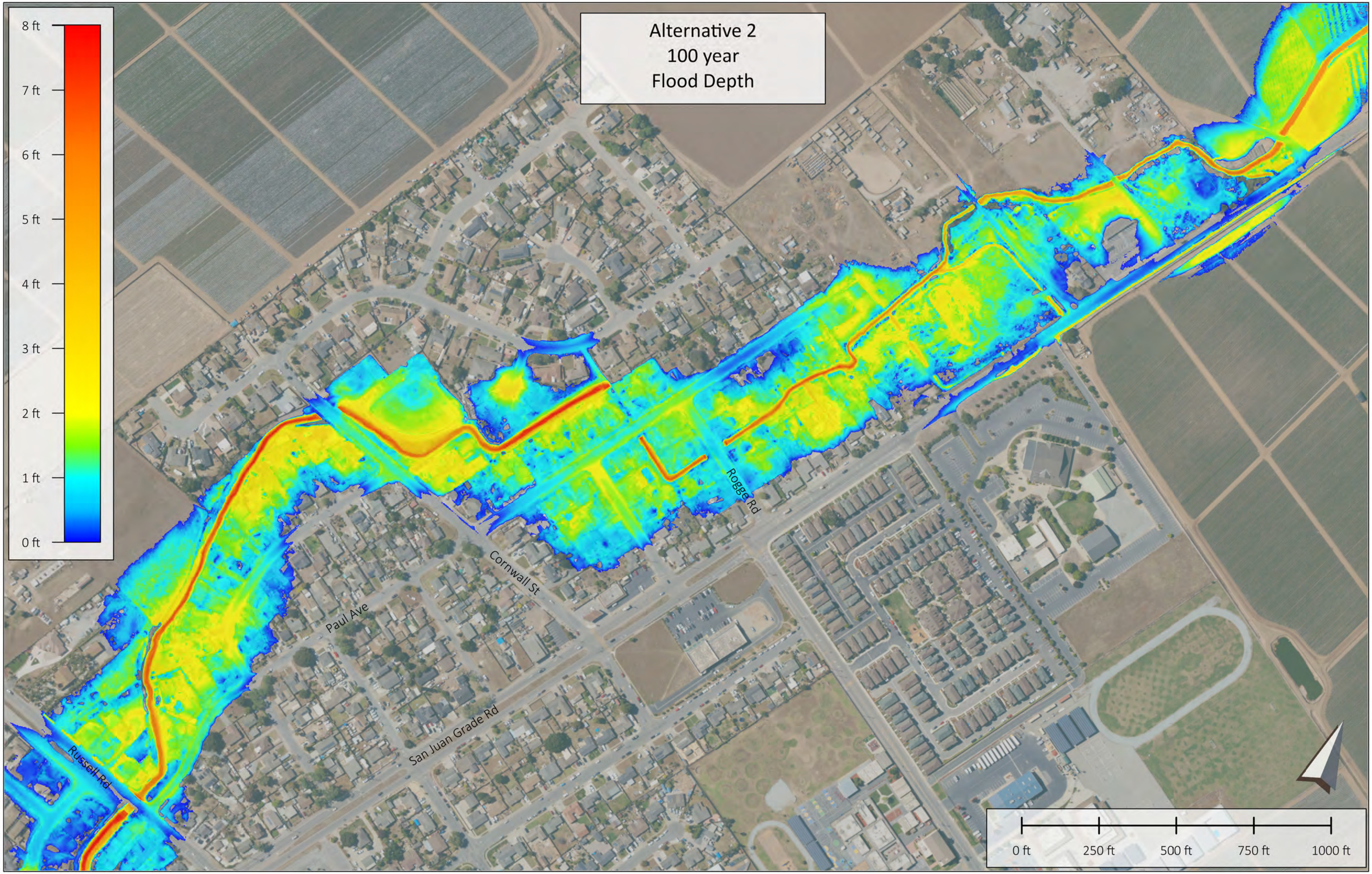

Alternative 1
100 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Alternative 1
10 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

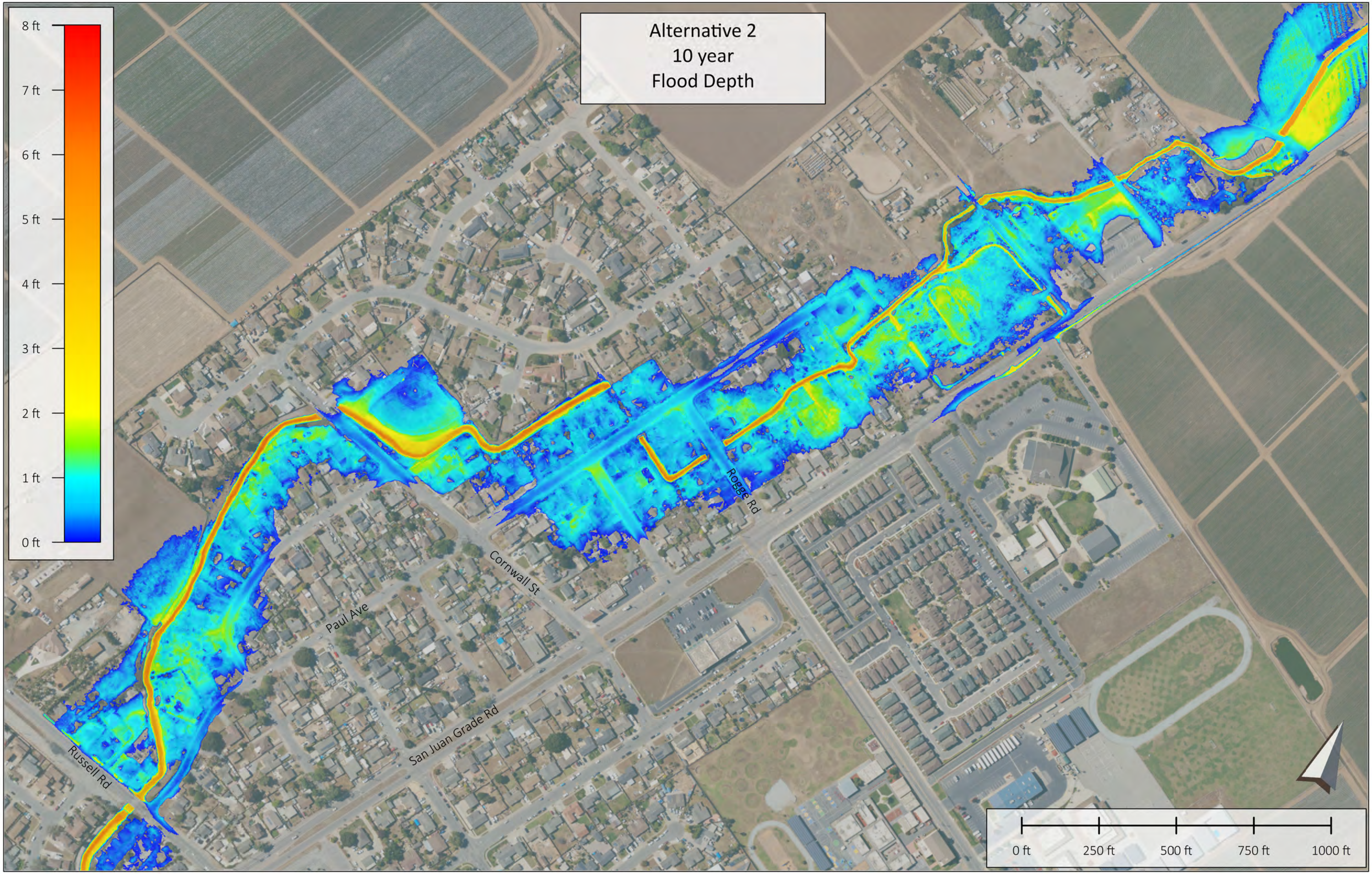

Alternative 1
December 27, 2022
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



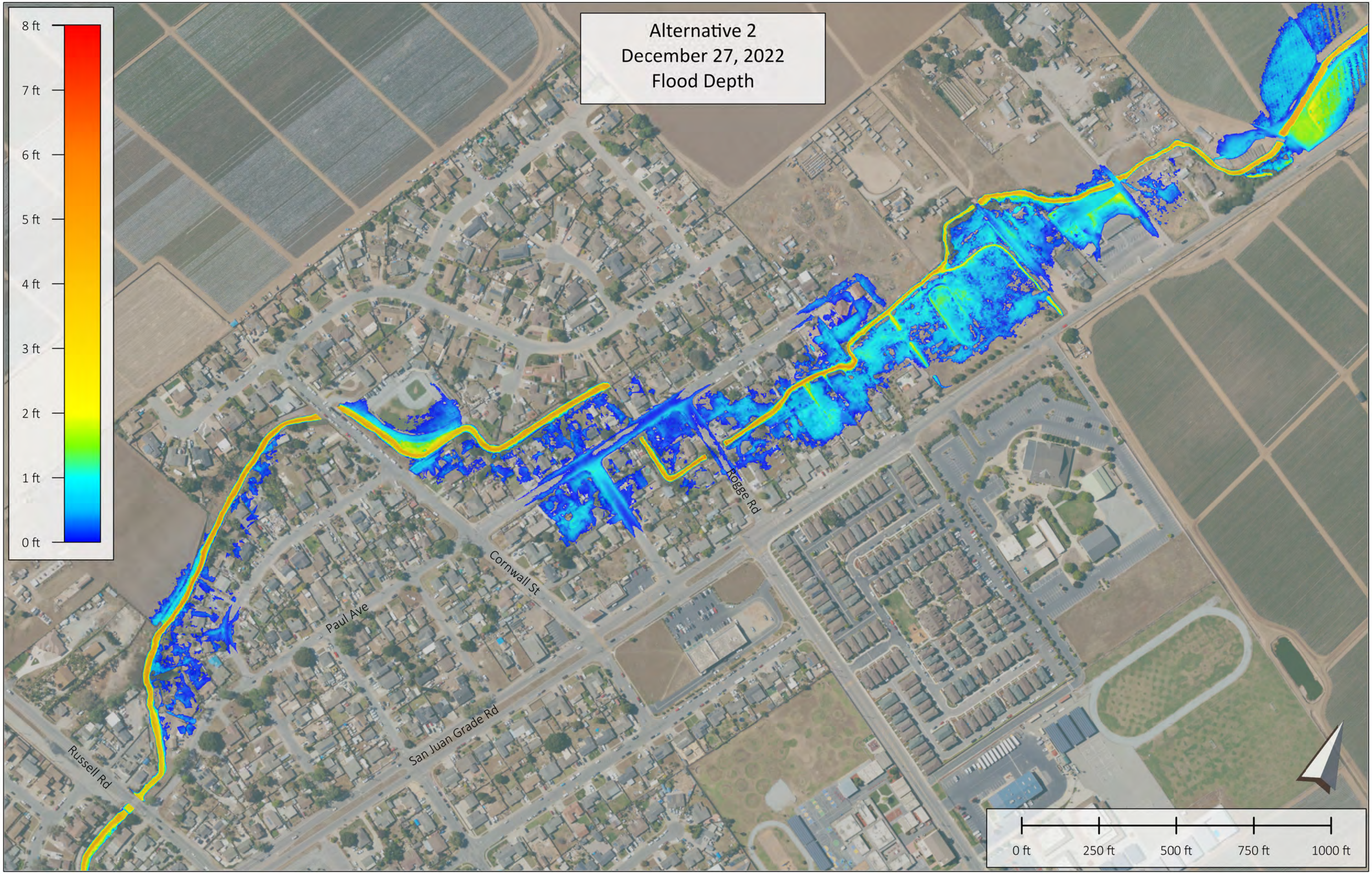
Alternative 1
February 13. 2025
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



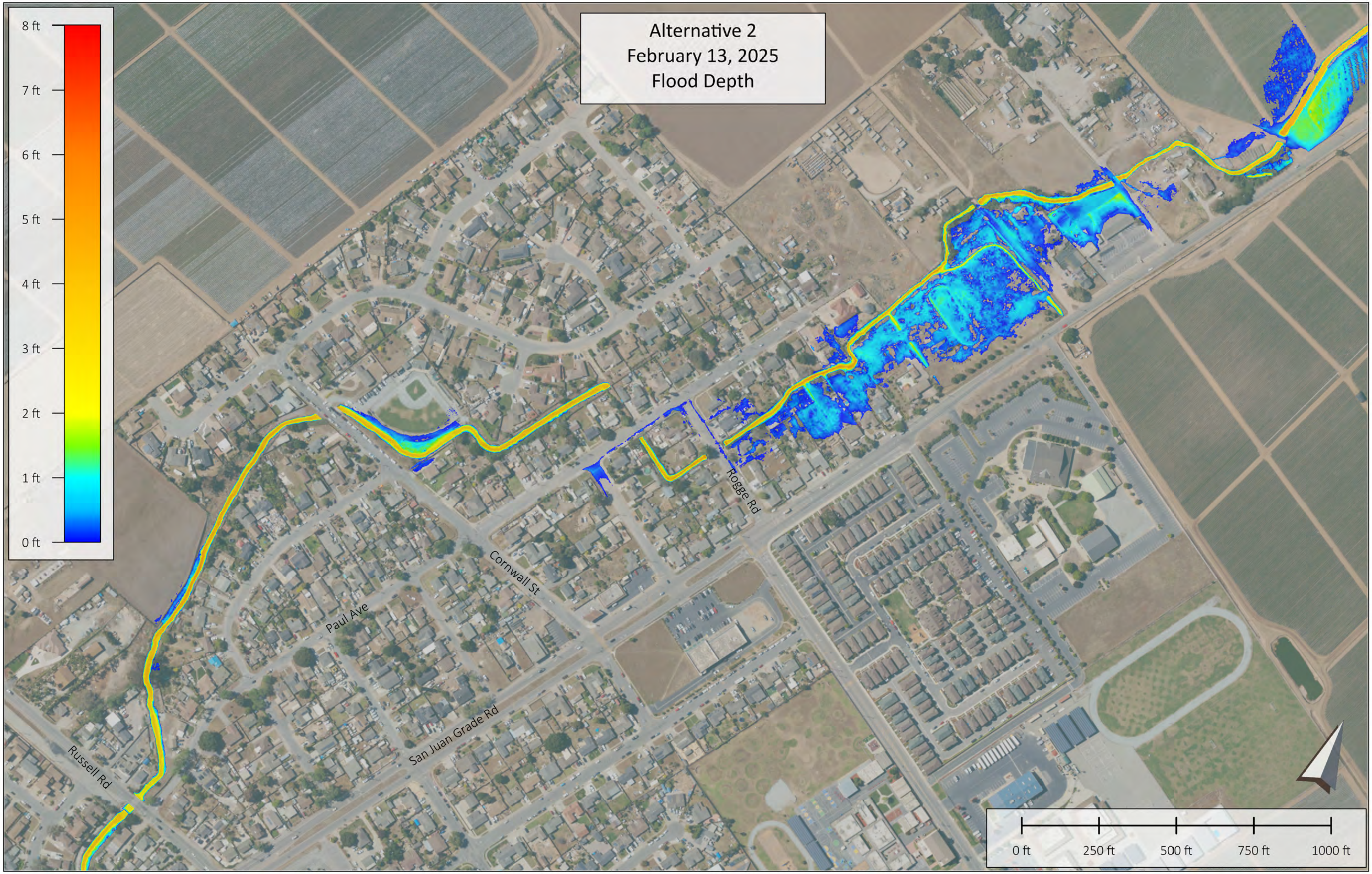
Alternative 2
100 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



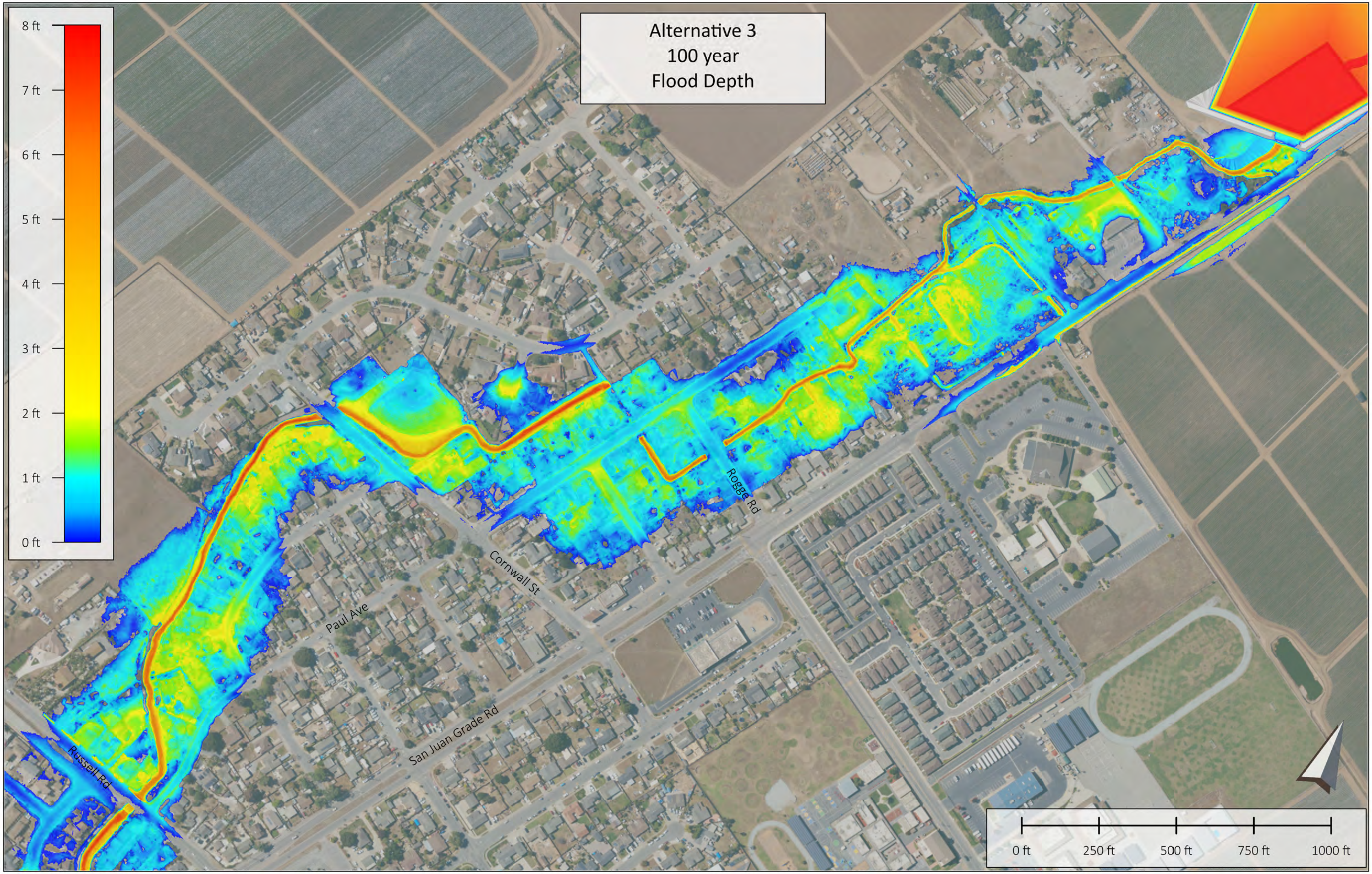
Alternative 2
10 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

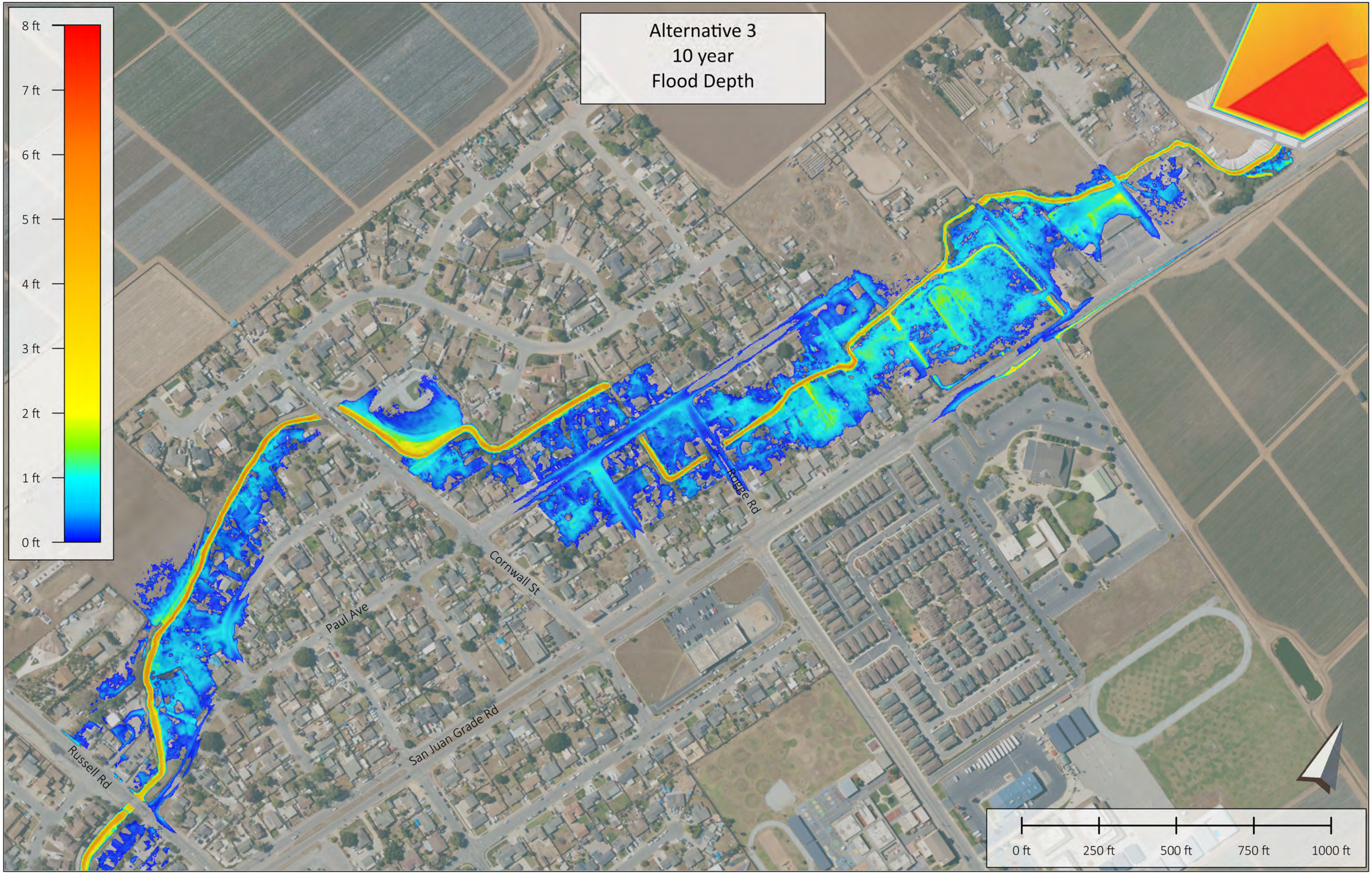


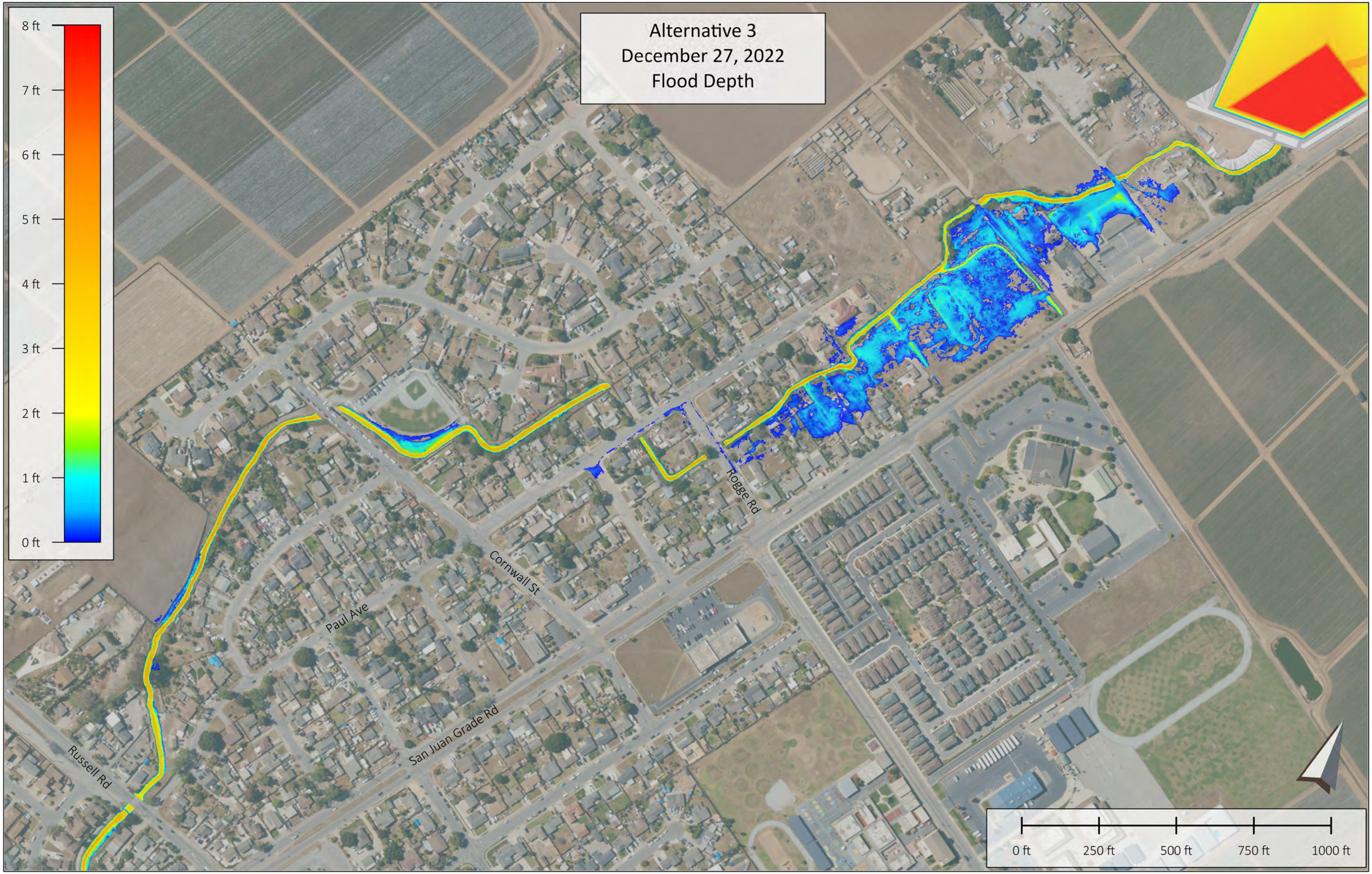
Alternative 2
December 27, 2022
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



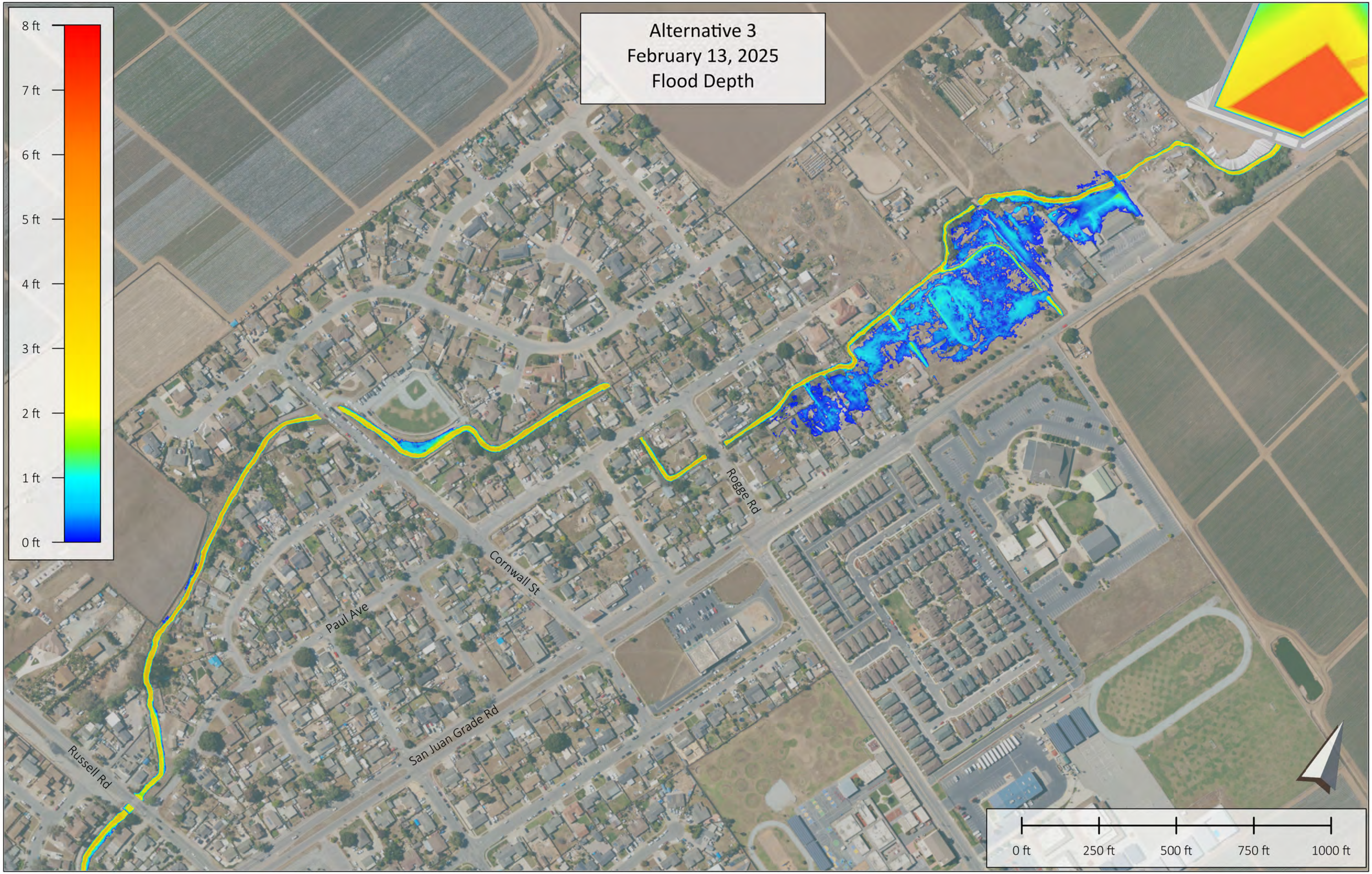
Alternative 2
February 13, 2025
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

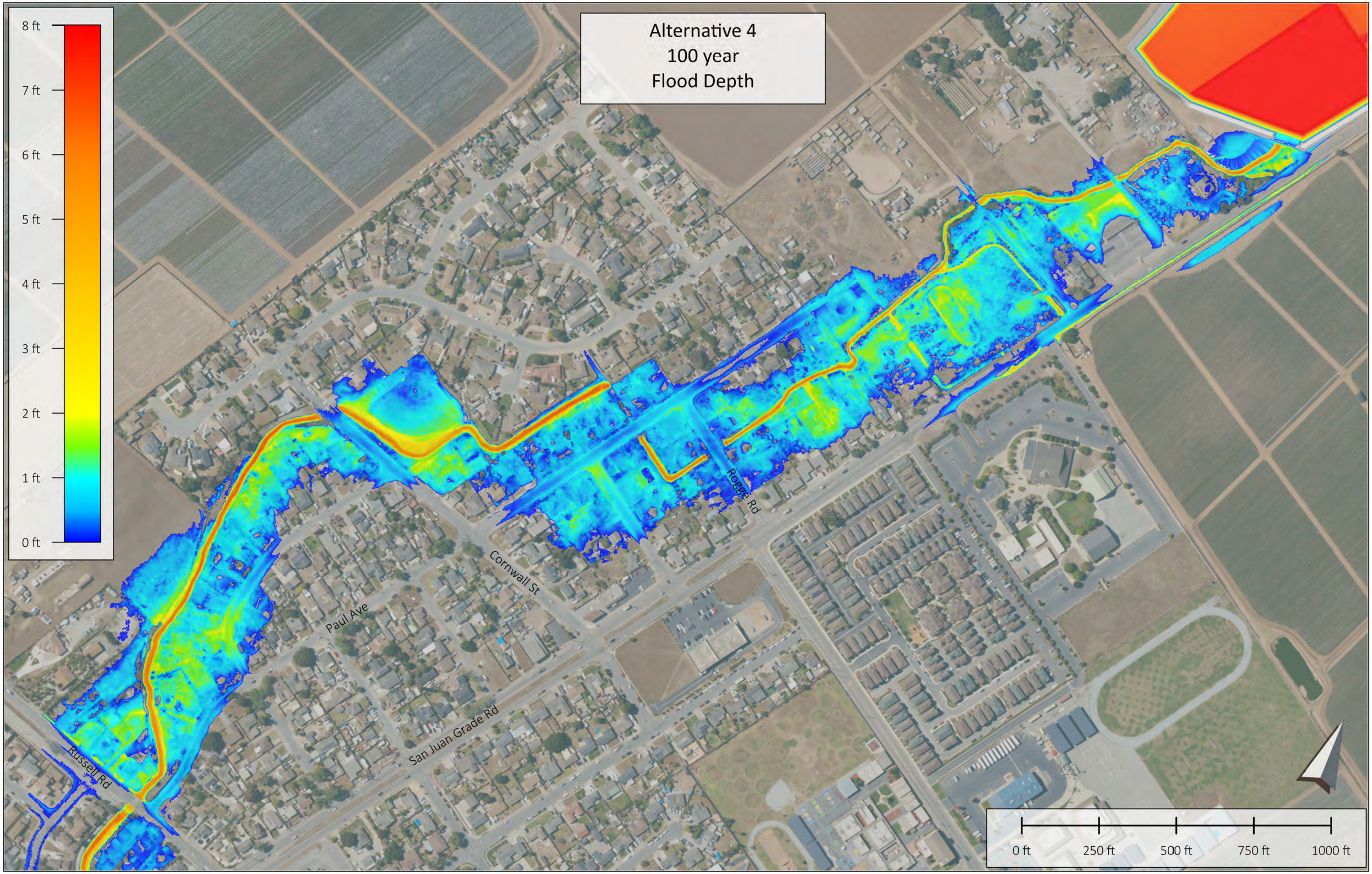

Alternative 3
100 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

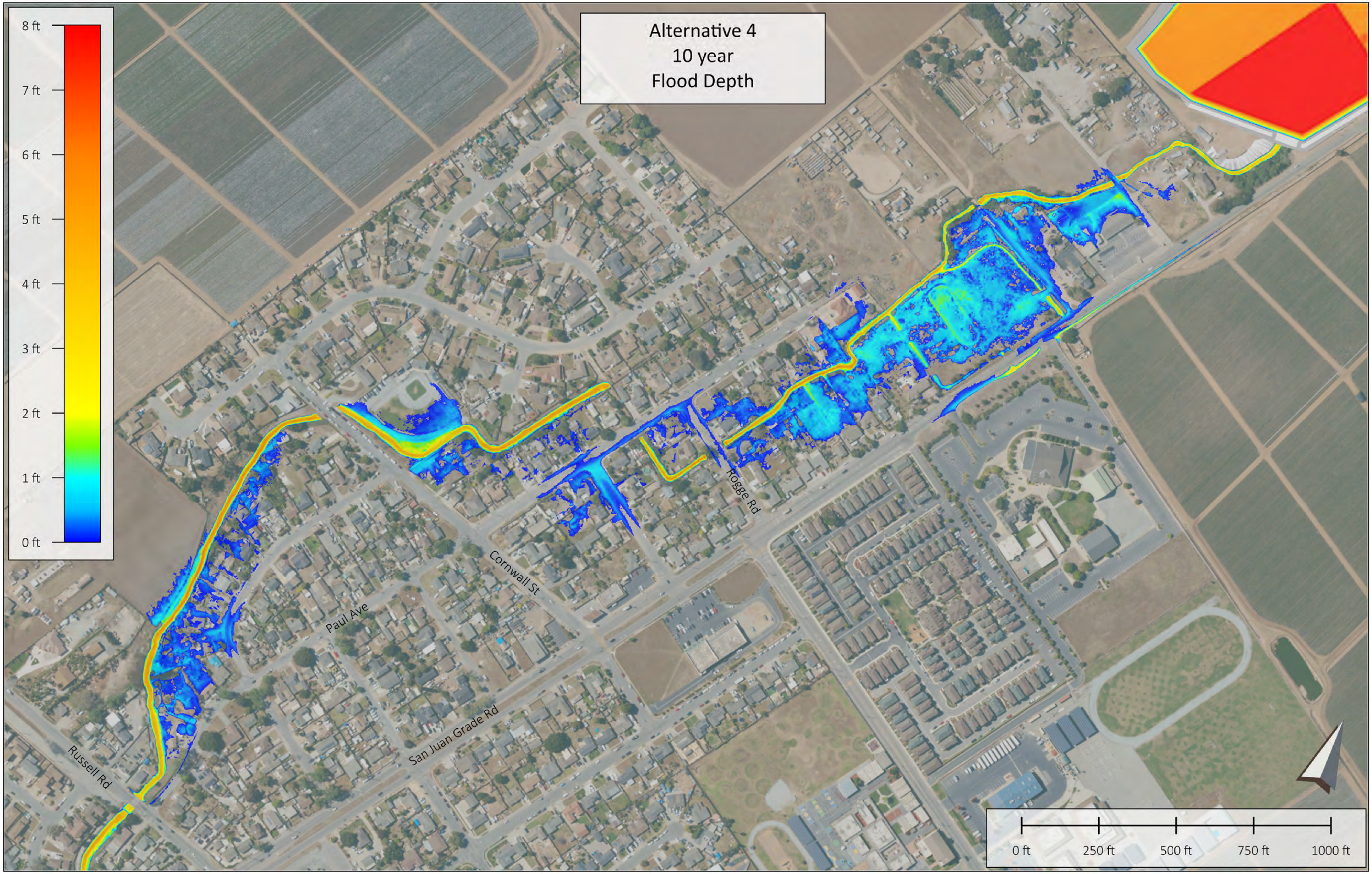

Alternative 3
10 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



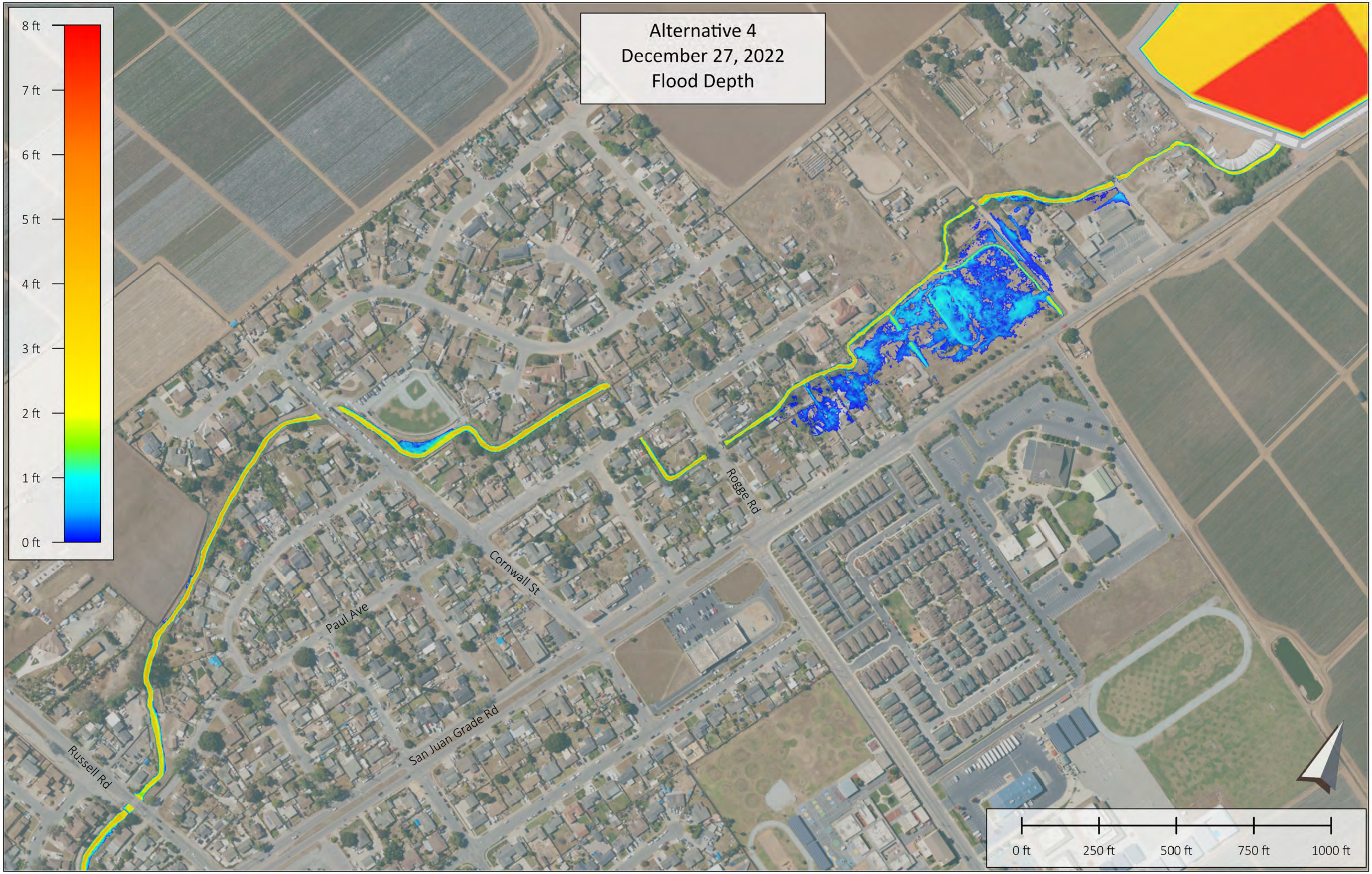
Alternative 3
December 27, 2022
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Alternative 3
February 13, 2025
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft

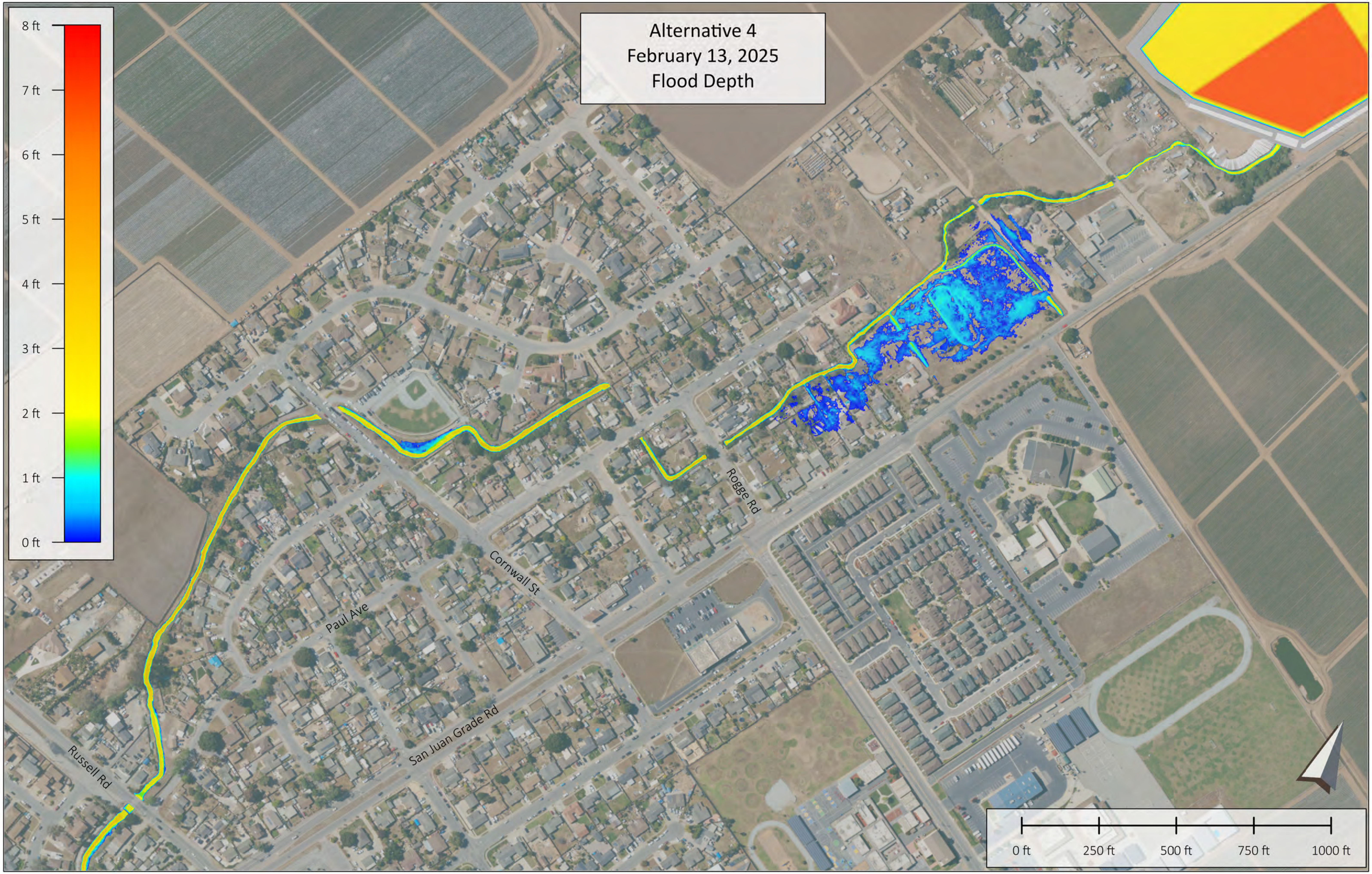

Alternative 4
100 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



Alternative 4
10 year
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft



Alternative 4
December 27, 2022
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft


Alternative 4
February 13, 2025
Flood Depth
8 ft
7 ft
6 ft
5 ft
4 ft
3 ft
2 ft
1 ft
0 ft
Rogge Rd
Cornwall St
Paul Ave
San Juan Grade Rd
Russell Rd
0 ft
250 ft
500 ft
750 ft
1000 ft