US Army Corps
of Engineers ®
San Francisco District

PAJARO RIVER FLOOD RISK MANAGEMENT PROJECT
SANTA CRUZ AND MONTEREY COUNTIES
CALIFORNIA

HYDRAULICS
APPENDIX D NOVEMBER 2017

PAJARO RIVER, CALIFORNIA
PAJARO RIVER AND TRIBUTARIES
FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY REPORT – HYDRAULIC APPENDIX

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1 STUDY DESCRIPTION

1.1 Purpose and Scope

This Hydraulic Report documents the hydraulic analysis performed to support the Pajaro River Flood Damage Reduction Study and has been prepared to meet the intention of the new USACE SMART Planning process – Specific, Measurable, Attainable, Risk-informed and Timely. This report is to present the data, methods, and results of the HEC-RAS 5.03 hydraulic modeling study conducted to support the formulation and NED selection of flood control measures proposed for the Pajaro River and Corralitos and Salsipuedes Creeks.

1.2 Project Datum

Horizontal coordinates used for the Pajaro River hydraulic models are in California State Plane Zone 3 in US Survey Foot. All Vertical coordinates, elevations, presented are referenced to the North American Vertical Datum 1988 (NAVD88) in US Survey Foot.

1.3 Study Approach

The Pajaro River and Salsipuedes Creek Levees were constructed in the late 1940's and documented in the as-built plans reported in US Engineer Office (1949). The hydraulic calculations supporting the hydraulic design profile are provided in the US Engineer Office (1945). According to the report, the Pajaro and Salsipuedes Levees were designed to contain a 2% (1/50) ACE flood with a three foot freeboard allowance. At the time the project was constructed, the 2% chance flood corresponded to a discharge of 19,000 cubic feet per second (cfs) above the confluence of Salsipuedes Creek, and 22,000 cfs below that point for the Pajaro River. According to the as-built's the Manning's roughness coefficient along the Pajaro River was specified as 0.025 from the mouth to Thurwachter Bridge, and 0.035 from Thurwachter Bridge to the upstream end of project near Murphy's Crossing. The Manning's roughness coefficient along Salsipuedes Creek was specified as 0.035. Current flood frequency of the Pajaro River and Salsipuedes Creek indicates that these discharges of 19,000 and 22,000 presented in the USACE 1945 report correspond to frequencies on the order of the 10% or 1/10 Annual Chance Exceedance (ACE).

Using existing data this study used the HEC-RAS 5.0.3 (RAS) as a combined 1- and 2-dimensional hydraulic model to map flood impacts and formulate solutions to known flooding issues in the study area. RAS 1-D was used to model the channel and 2-D was used to model the floodplain. This effort included the development of the Without-Project model as the baseline condition along with the evaluation of a focused array of alternatives to evaluate With-Project features that will lead to the selection of a NED Plan.

The Existing Condition, or Without-Project model was needed to delineate floodplains in order to help generate flood damages for economic evaluation. The combined 1D/2-D RAS model was used to model breaches and overtopping along the existing main flood control channels of the system directly connected to the floodplain areas determining water surface Depths, and

Elevations, and hydrographs into the floodplain areas. Flood depths as well as stage-discharge-frequency curves derived from HEC-RAS output were used in performing risk analysis of the various without project and no action conditions. Floodplains were delineated for the 50% (1/2) Annual Chance Exceedance (ACE), 20% (1/5) ACE, 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.4% (1/250) ACE, and 0.2% (1/500) ACE events. The analysis was limited to flooding within the basin from levee breaches and does not include localized flooding from localized rainfall runoff from smaller streams and drainages. Though there exists many different flood possibilities for a particular areas within the basin, floodplain delineations presented in this study are based on breaches at each reach as described in Figure 1.

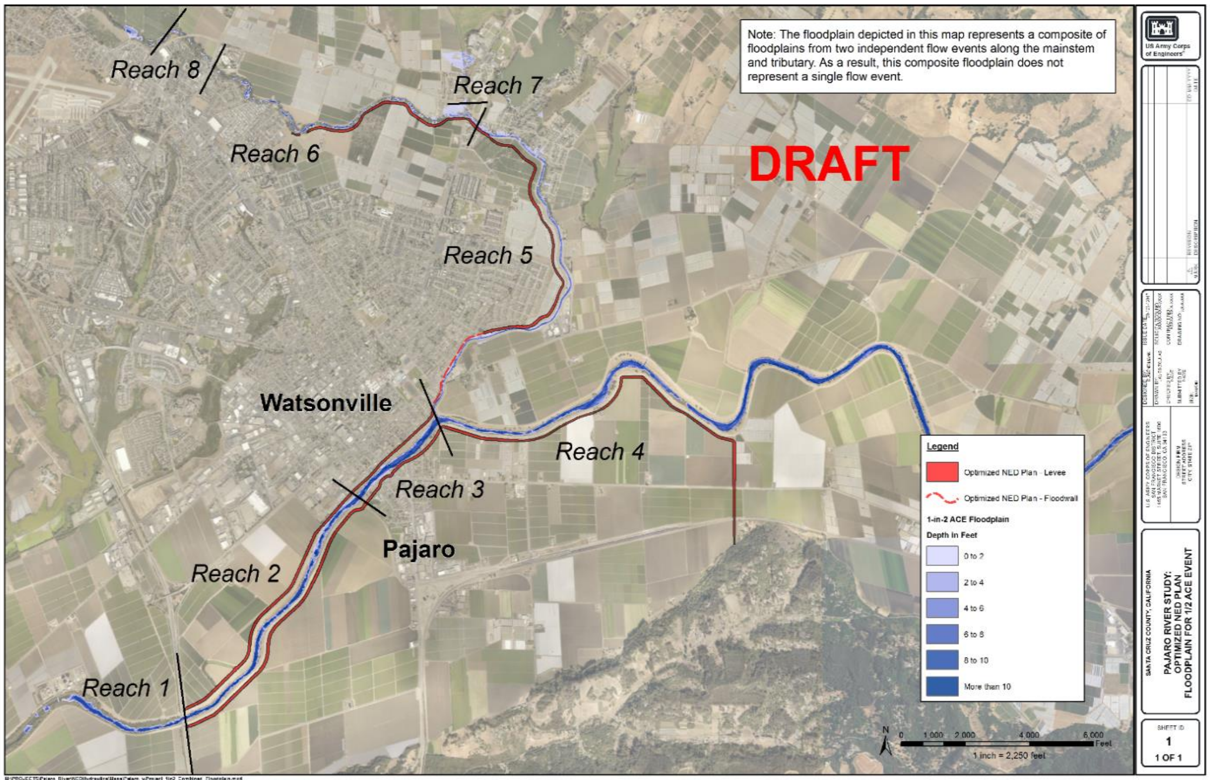


Figure 1 Delineated Reaches

2 HYDROLOGIC AND HYDRAULIC MODELING

2.1 Hydrologic Assumptions

The hydrologic data used for this effort was from the 1997 Report of the Santa Cruz County Pajaro River Basin Hydrologic Engineering Report. The USACE study used the statistical analysis of peak flood frequencies based on the stream flow data described. For Pajaro River flows USGS gage number 11159000, Pajaro River at Chittenden, CA was selected. Pajaro River at Chittenden has a drainage area of 1,188 Square miles. Corralitos Creek flows are selected from the USGS gage at Freedom (gage number 11159200). Corralitos Creek at Freedom has a drainage area of 27.8 Sq. mi. With a drainage area of 19.6 square miles, Salsipuedes Creek inflow to College Lake is not gaged and uses routed outflows from a HEC-1 analysis. The computation of College Lake outflows is provided in the 1997 report were utilized for this study and the computed peak outflows are provided in Table 3. .

Tables 1, and 2 shows the peak flood frequency flows obtained from the USACE's Hydrologic Engineering Report for the Pajaro River Basin. Figures 2, 3, and 4 represent the shape of the hydrographs used in the unsteady flow analysis.

Early is this study the local sponsor stated that Corralitos and Pajaro River storm events were not coincident. To verify this assumption a correlation coefficient was calculated. A correlation coefficient is a measure of the interdependence of two random variables that range in value from -1 to +1, indicating perfect negative correlation at -1, absence of correlation at zero, and perfect positive correlation at +1. To calculate the coefficient of correlation peak flow values of each gauge was edited to include daily peaks that occur within 2 days of each other. Using statistical analysis the calculated Pearson Product-Moment Correlation Coefficient (r) equaled 0.61with the Coefficient of correlation, also called the coefficient of Determination, (R^2) equaling 37%. These values equate to a mild uphill (positive) linear relationship with a weak goodness of fit. For this effort the storm hydrographs were assumed to be separate uncorrelated events.

Table 1 Summary of Pajaro Peak Flood Flows

Recurrence Interval	Discharge, cfs
50% (1/2 ACE)	3,200
20% (1/5 ACE)	10,800
10% (1/10 ACE)	16,900
4% (1/25 ACE)	26,800
2% (1/50 ACE)	35,200
1% (1/100 ACE)	43,500
0.4% (1/250 ACE)	50,000
0.2% (1/500 ACE)	62,500

Table 2 Summary of Corralitos Peak Flood Flows

Recurrence Interval	Discharge
50% (1/2 ACE)	1,050
20% (1/5 ACE)	2,400
10% (1/10 ACE)	3,500
4% (1/25 ACE)	4,650
2% (1/50 ACE)	6,100
1% (1/100 ACE)	7,200
0.4% (1/250 ACE)	9,600

Table 3 Summary of College Lake Peak Flood Outflow

Recurrence Interval	Discharge

20% (1/5 ACE)	(-80) 20*
10% (1/10 ACE)	(-30) 20*
4% (1/25 ACE)	570
2% (1/50 ACE)	720
1% (1/100 ACE)	1220
0.2% (1/500 ACE)	3000

* A minimum flow value of 20 cfs was added to the model to keep the hydraulic model wet for computational stability in order to evaluate left bank tributary features.

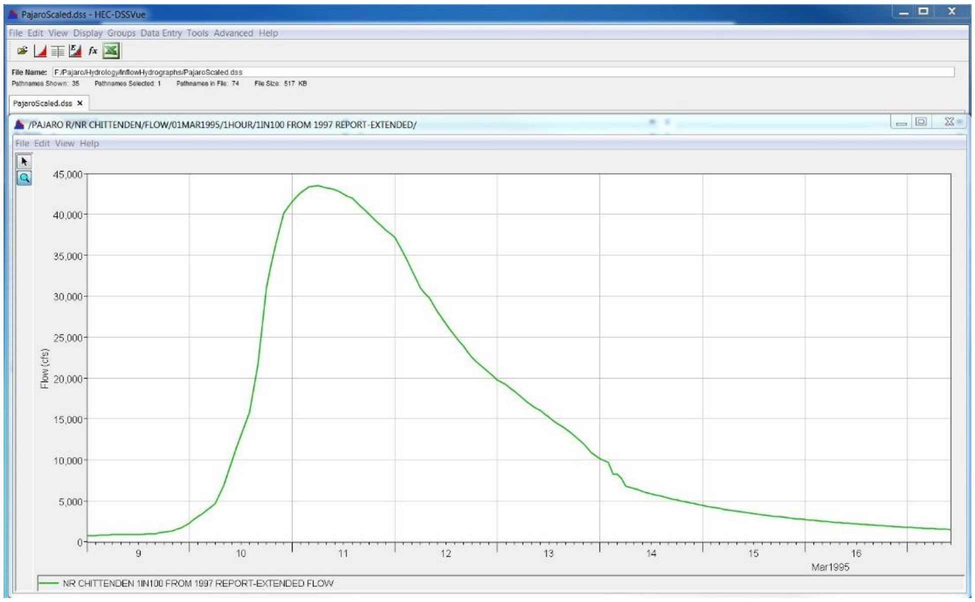


Figure 1. 1% Model Inflow Hydrograph- Pajaro River at Chittenden

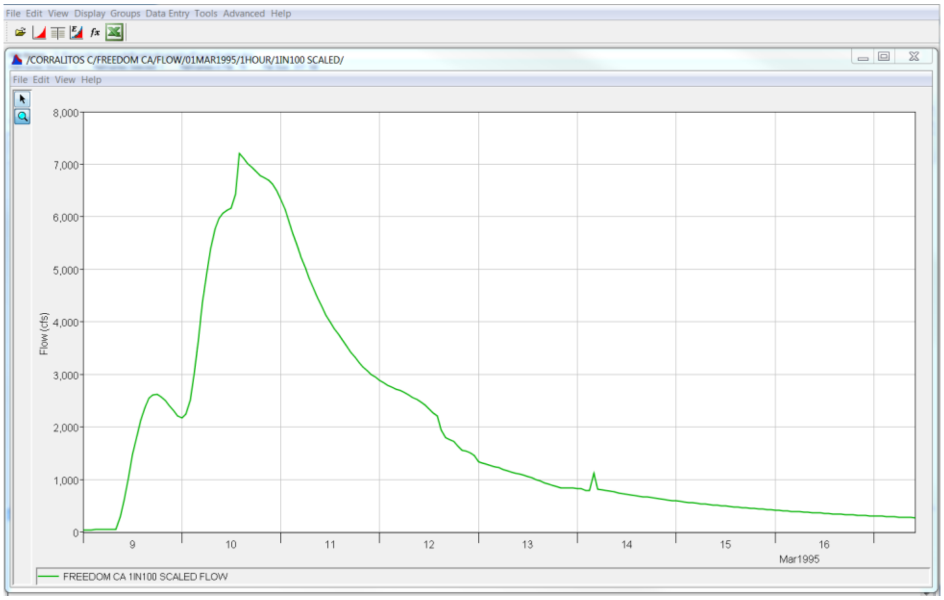


Figure 2. 1% Model Inflow Hydrograph- Corralitos Creek at Freedom

2.1 HEC-RAS Conversion and Compilation

Previous modeling efforts for the Pajaro River and Corralitos and Salsipuedes Creeks were undertaken using an earlier version of HEC-RAS 1-D. The model originated from an earlier effort RAS effort that was not geo-referenced and used a vertical datum of NGVD 1929. This original model was geo-referenced using the bridges as reference and distance to the downstream cross section as a key. Vertical datum conversion was completed using data supplied by the Santa Cruz County Public works Department with a conversion value of +2.72 feet. Table 4 below verifies the datum elevation change. Additional cross sections for the Pajaro River were supplemented cross sections extracted from models created by Northwest Hydraulics Consultants. The accuracy of cross sections of the Pajaro River were based on ground survey methods. Additional cross sections for Corralitos, College Lake, and Salsipuedes Creek were supplemented from RAS models developed for the College Lake Multi-Objective Management Effort conducted by CBEC for Santa Clara County. No information was available to assess the accuracy of the Corralitos and Salsipuedes Creek cross sections.

Table 4 National Geodetic Survey Benchmark Elevations

PID	Latitude	Longitude	Designation	Elevation (Feet) NAVD88	Elevation (Feet) NGVD29	Change (Feet)
GU2239	36.89867	-121.728	T 1236	36.55	33.84	2.71
GU2242	36.89556	-121.746	M 20 RESET	30.95	28.24	2.71
GU2245	36.90333	-121.755	V 1236	35.95	33.24	2.71
GU2246	36.90528	-121.758	W 1236	27.07	24.36	2.71
GU2248	36.90028	-121.778	X 1236	20.53	17.81	2.72
GU4187	36.91944	-121.748	W 16	43.79	41.07	2.72
GU4216	36.93917	-121.771	GAGING STATION	110.25	107.53	2.72

2.2 Terrain Model

The terrain model used for this study was downloaded from the National Elevation Dataset (NED) merged and projected to California State Plane 3, Survey Feet with a vertical projection of NAVD88 Feet. This raster product is assembled by the U.S. Geological Survey.
NED is designed to provide National elevation data in a seamless form with consistent datum's, elevation unit, and coordinate reference system. The NED assembly process involves edge matching and mosaicking elevation data into NED layers. Seamless NED layers for this region are available at the one third arc-second (approximately 10 meters) resolution. For more information please go to (http://ned.usgs.gov/about.html).

2.3 Existing Condition Model

The Existing Condition, or Without-Project model was created to delineate floodplains needed to generate flood damages for economic evaluation. The 1-D model was compiled, adjusted and run for model stability prior to addition of the 2-D features. Once the exemptions where corrected and the model ran stable in steady and unsteady the 2-D features were added. The 1-D was connected to the 2-D floodplain using the Lateral Structures tool in HEC-RAS. For the Existing Condition model lateral structure elements used existing levee heights that were supplied from the National Levee Dataset (NLD) and natural ground from the NED (see Figure 4). Existing HEC-RAS model roughness values were used with a letter dated February 10, 2003 from the California Regional Water Quality Control Board Central Coast Region. 2D model roughness was imported from the National Geospatial Asset Land Use Cover version NLCD 2011 Land Cover (2011 Edition, amended 2014) that can be downloaded at the URL https://www.mrlc.gov/nlcd2011.php.

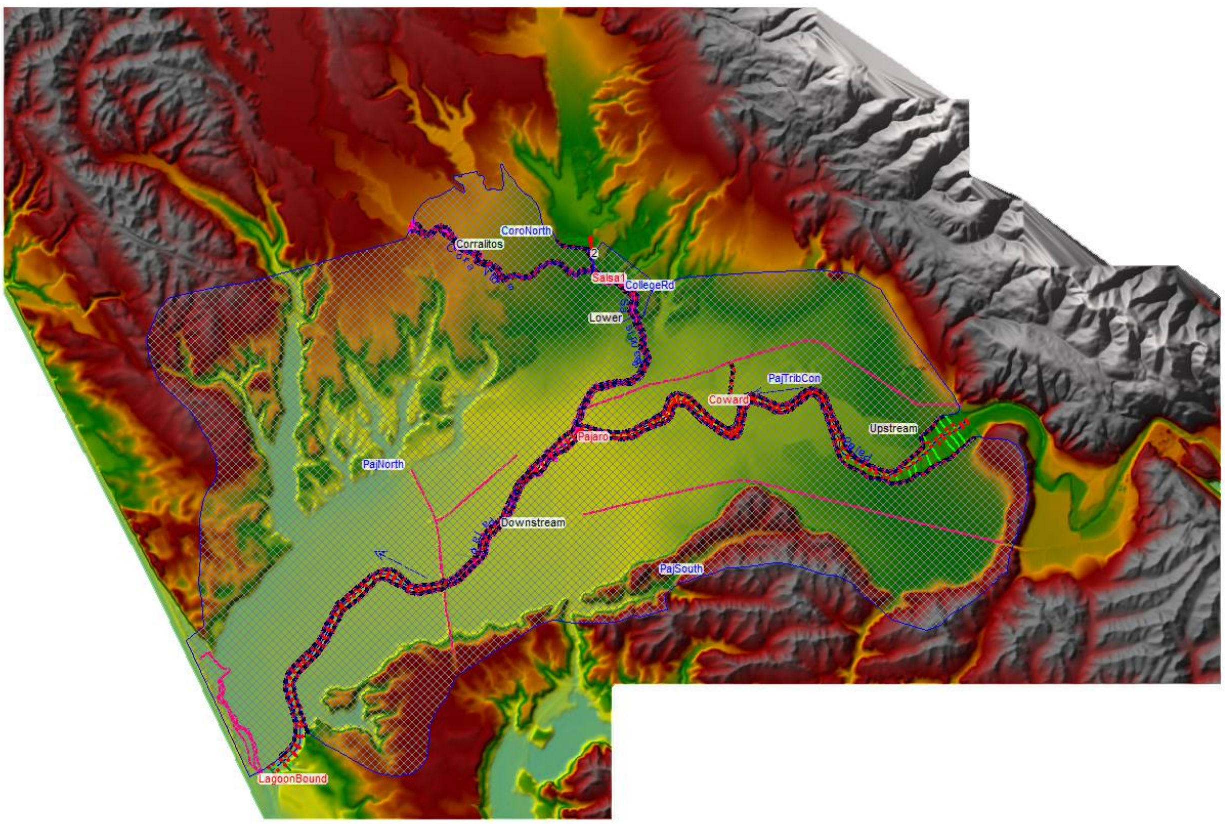


Figure 3. Existing Condition Model Extent

The combined 1D/2-D RAS model was used to model breaches and overtopping along the existing main flood control channels of the system directly connected to the floodplain areas determining water surface depths, and elevations, and stage/flow hydrographs into the floodplain areas. The existing condition model was calibrated using known flood events equating to a bank full 4% ACE storm event. Note: Normal Depth was selected for the downstream boundary condition for this and all other Pajaro RAS model runs as tidal signal did not affect calculated water surface elevations.

A breach length of 100 foot was chosen as a breach width and used consistently throughout this modeling effort. The 100 foot breach was selected through the application of the procedure outlined in the Sacramento District Document titled "Development of Levee Breach Parameters for HEC-RAS Application," dated August of 2013. Further study and effort beyond the scope of this study is needed if a more robust approach is desired for determination of the breach width.

Time to failure is the time it takes the levee to fail once the water surface elevation reaches the trigger elevation. Sensitivity analysis in other studies has demonstrated that hydrographs through failures change little with respect to the time to failure. Because it is not very sensitive to this parameter, levees were failed instantaneously.

Other breach parameters include the final breach invert elevation, and the breach weir coefficient. The final breach invert elevation was set as that of the landside ground elevation, and the weir flow coefficient was set at 2.6. Again, further study and effort beyond the scope of this study is needed if a more robust approach is desired determination of these parameters.

From this calibrated model simulated breaches were created at each index location for every reach. The 8 flood frequency models were run for each breach location and the resultant floodplains were then composited into one "n-year" floodplain for economic analysis. Please see Figure 5 for an example of the resultant composited floodplains used to establish economic damages. The composite floodplain is the highest stage from all the breach scenarios for a given frequency event.

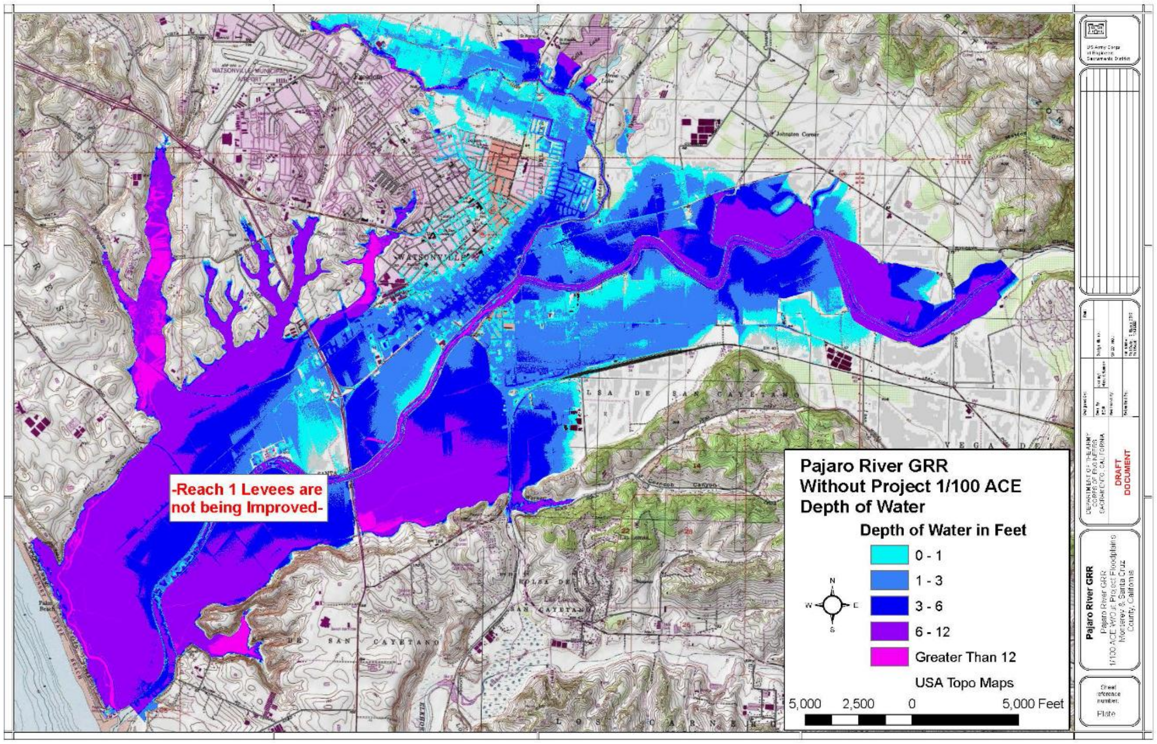
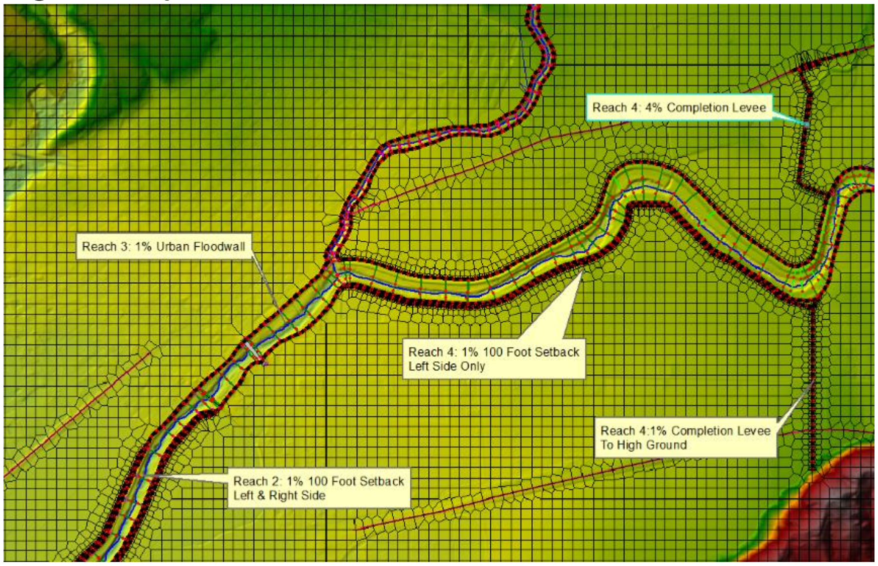


Figure 4. 1% ACE Composited Floodplain

2.4 With Project Condition Models

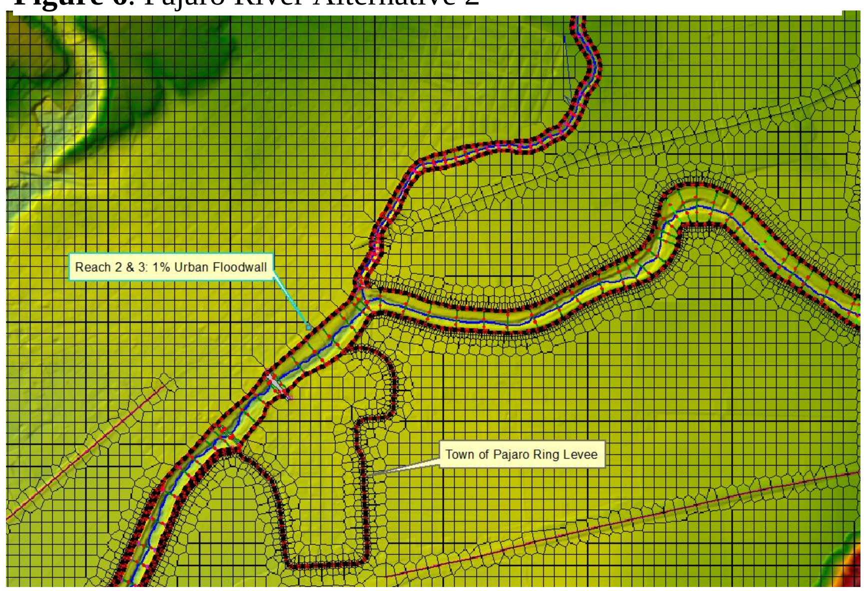
From a hydraulic perspective, measures to reduce the probability of inundation generally fall into four categories: levee improvements, upstream transitory storage, diversions, and combinations of these features. Of these features, it was determined that the first increment would be some amount of levee improvement and this is the base for combining additional measures to become the alternatives. Based on preliminary analyses, the other measures did not show significant reductions in stage or flow, had the potential to create hydraulic impacts, or had very large real estate requirements. Even with some of these additional measures, the stages and flows were not reduced enough to eliminate the need for levee improvements or deemed too costly for the damages prevented. For the purposes of the current study the four following alternatives were selected for consideration.

Figure 5. Pajaro River Alternative 1



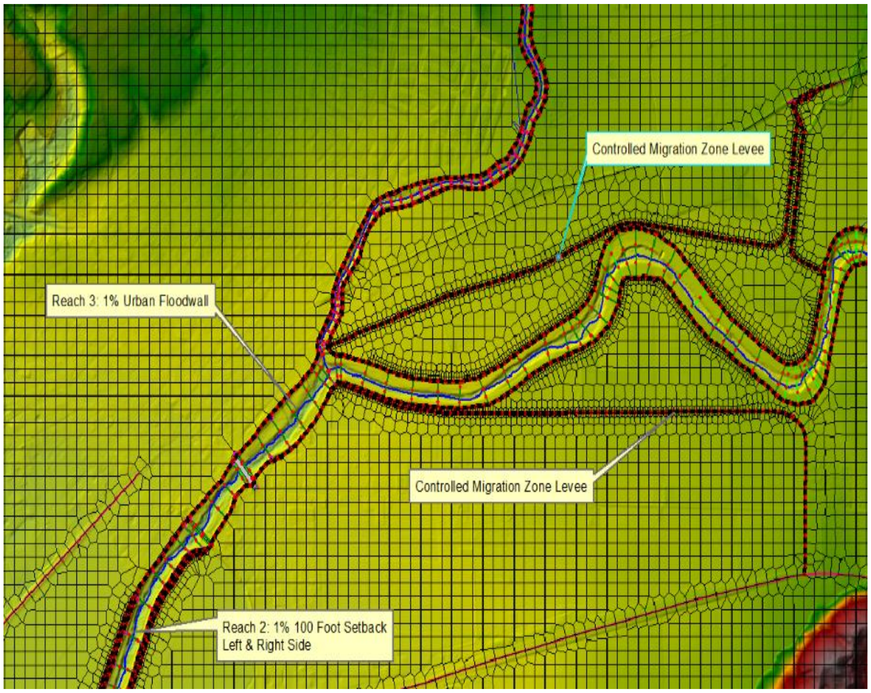
Alternative 1 primarily calls for levee improvements and levee setback measures that do not change the low flow channel geometry characteristics. These levee improvements involve the construction of levee remediation measures to address concerns such as seepage, slope stability, potential overtopping, and erosion. Features included in Alternative 1 focused on improvements along the Pajaro River. They included the construction of a 100 foot left and right levee along reach 2, an urban floodwall along the left and right banks of reach 3, a levee setback along the left bank of reach 4 along with reach 4 completion levees. All measures listed here were designed to contain the 1% event with 90% assurance.

Figure 6. Pajaro River Alternative 2



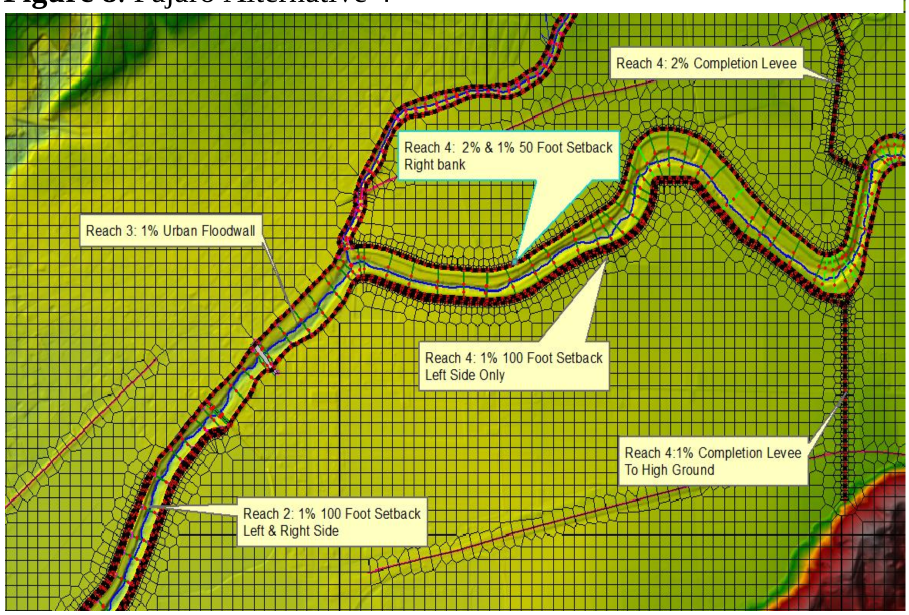
The primary features of Alternative 2 was the construction of a ring levee around the town of Pajaro and levee remediation along the right side of reaches 2 & 3. As with alternative 1 there are no change the low flow channel geometry characteristics. The smaller of the two ring levee features evaluated is shown in Figure 6

Figure 7. Pajaro River Alternative 3



Alternative 3 (Figure 7) also included the 100 foot setback levees on the left and right banks of Reach 2, floodwalls in urban areas of Reach 3, an Optimized Channel Migration Zone (CMZ) in Reach 4 left and Right bank, with 4% (1/25) ACE with 90% assurance in Reach 4 right bank where land used is primarily agricultural.

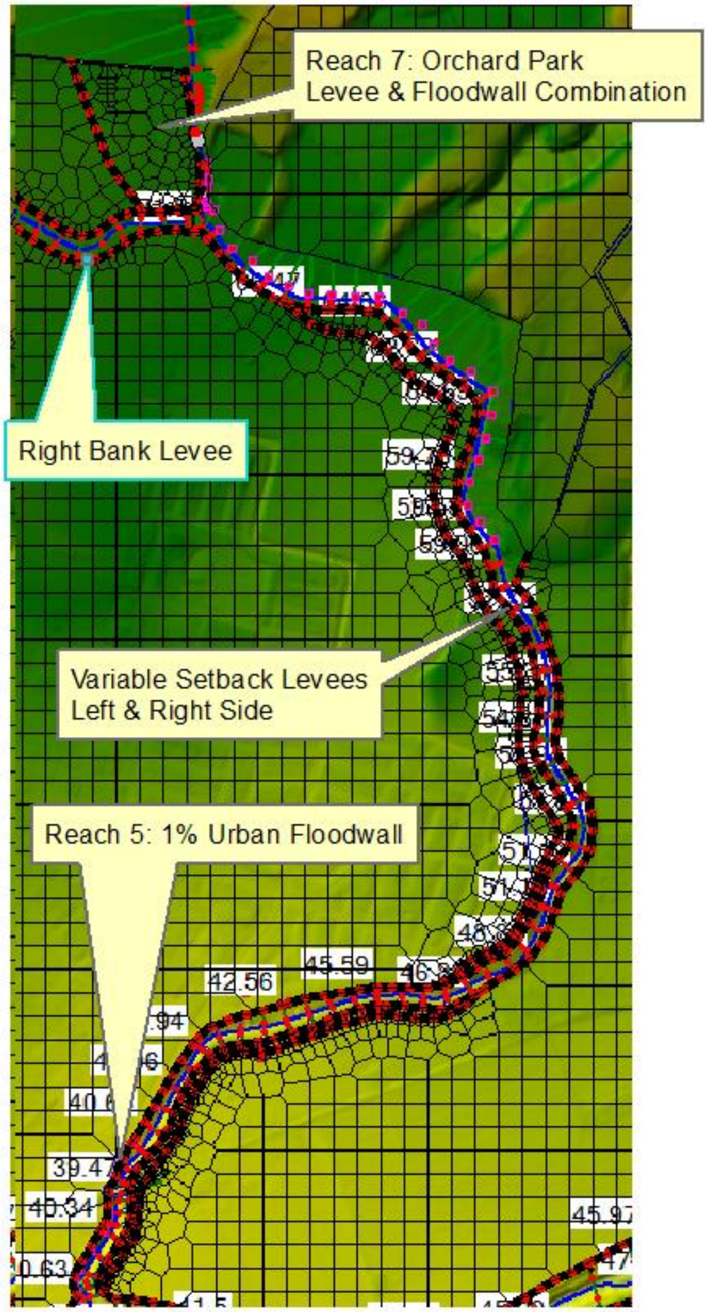
Figure 8. Pajaro Alternative 4



Pajaro Alternative 4 (Figure 8) included 100-foot setback levees on reaches 2 and 4 left bank, floodwalls in urban areas in Reach 3, 50 foot setback completion Levee with 1% or 2% ACE with 90% assurance in Reach 4 right bank.

There were also four alternatives for the Tributary Alternatives. Alternative 5 features include floodwalls in urban areas on Reach 5 right; variable setback levees along agricultural areas on the left and right of Reaches 5 and 6, levee and floodwall combination around Orchard Park (Reach 7) and levee on the left of Reach 8.

Figure 9. Tributary Alternative 6



Alternative 6 included a floodwall in urban areas on Reach 5 right, variable setback levees along agricultural areas on Reaches 5 left and right and variable setback levee on Reach 6 right, plus a Ring Levee around Orchid Park (Reach 7). Which was very similar to Alternative 5.

Alternative 7 included a floodwall in urban areas on Reach 5 right, a variable setback levee along agricultural areas along Reach 5 left and right, Optimized Channel Migration Zone (CMZ) levees along Reach 6 left and right; levee along Reach 8 left (about 225 feet wide).

Alternative 8 was the he same as alternative 7 above except for the exclusion of the Optimized Channel Mitigation Zone levee on Reach 6 left. Please see the Civil Appendix for more detailed view of these Features and alignments.

2.5 The National Economic Development/Recommended Plan

Based on an evaluation of the alternatives, the National Economic Development (NED) alternative was selected. The NED plan was then selected as the Recommended Plan
The NED plan includes features along the Left Bank tributary to mitigate for the marginal increase in flood stage.. These features included a Levee or Floodwall feature designed to the 4% (1/25 ACE) event with 50% assurance These features listed below are to be constructed to contain the 1% with a minimum of 90% assurance unless otherwise identified::

- Reach 2 - New 100 foot setback levee left & right bank
- Reach 3 - Rebuild existing levees and floodwalls
- Reach 5 - New Floodwall on the existing right bank levee
- Reach 5 - New Mitigation Floodwall to contain up to the 4% event with 50% assurance
- Reach 5 - New Levee with setback ranging from 100 to 225 feet
- Reach 5 - Rebuild Levees in Place
- Reach 6 – New Levee with setback ranging from 50 to 75 feet
- Reach 6 - New Mitigation Levee to contain up to the 4% event with 50% assurance.

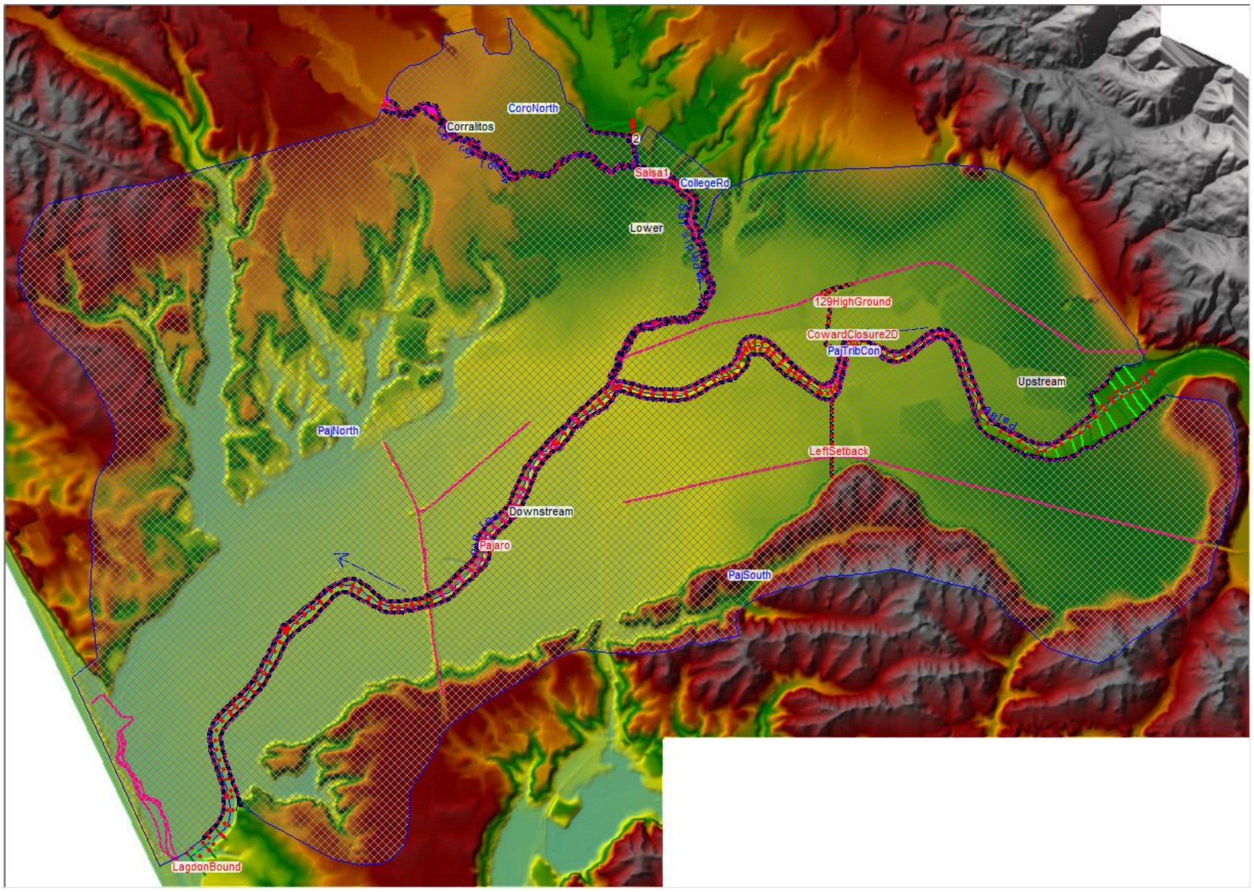


Figure 6. Tentatively Selected Plan Model Extent

2.6 Water Surface Profiles

The HEC-RAS model was used to develop water surface profiles and floodplains for the Pajaro River, and Corralitos and Salsipuedes Creek within the project area. Plan and profiles data shown are for the Recommended Plan only and were generated from the hydrologic data contained in the 1997 Report of the Santa Cruz County Pajaro River Basin Hydrologic Engineering Report. Please see the following Plates for profiles and Recommended Plan floodplains.

3 EROSION

Erosion is the removal of sediment, rocks, cobble, vegetation and general deterioration of a bank or a levee due to the power of water, often measured by shear stress and velocity. The probability of erosion eroding a levee resulting in failure can present a significant flood risk.

The primary concern about erosion related flood risk in the project area is in areas experiencing higher velocities than the bank material or levees can resist, resulting in levee failure. While there may be erosion occurring on the smaller tributaries in the project area, it is assumed that any repairs would be incorporated into current designs with limited added costs, would not involve large quantities of rock, and would not have specific designs called out.

The plan for erosion management features is ongoing; more analysis is expected to provide greater insight. Erosion repairs are expected to be part of all alternative and refinement efforts will continue during the PED phase. Existing erosion conditions in the project area are being designed by a separate multidisciplinary Erosion Protection analysis was developed for this study that contains addition information.

To estimate the probability of channel erosion an analysis was conducted to estimate velocities of the project reaches. Plate 25 illustrates the without project velocities for Pajaro at 1/25 ACE bank full from Pajaro River only flows. Plate 26 represents the with- project conditions on the Pajaro River with all the selected features in place. Plate 27 represents the impact to velocity from the differences in the without project condition versus the with- project condition. Plate 27 illustrates the velocities within reach 2 are reduced for bank full flows along with marginal increases in velocity within the channel of reach 3 and 4. Plate 28 illustrates the estimated extent, location, and type of channel lining material needed to maintain a stable channel. Calculations for material lining is based ERDC TN-EMRRP-SR-29 guidance labeled Stability Thresholds for Stream Restoration materials by Craig Fischenich et al.

3.1 Sedimentation and Channel Stability

Please see Attachment B "Channel Erosion Potential in the Lower Pajaro River Flood Control Project for previous sediment review and analysis. The summary of results are listed below. The results were based on an evaluation of Alternative 2A which is not directly comparable to the recommended plan. However, the general results of the assessment provide insight into the considerations of the recommended plan.

1. The shear stresses calculated for all conditions (existing, preferred plan and alternatives to the preferred plan) during the 1% (1/100) ACE flood are considerably higher (orders of magnitude) at meander apexes than the critical shear stress (resistance threshold) for bare soil or alluvial materials. Maximum predicted shear stresses range from 0.65 lb/ft2 on meander apexes under existing conditions, to 0.50 lb/ft2 with a bench on one side of the channel. It is thus essential that some type of protective cover is applied and maintained on the outside banks, floodplain, floodplain terraces and levees at meander apexes under any flood management plan. Exposed earth banks within the low flow channel may also be prone to erosion that could undermine more resistant upper banks.

2. Design shear stresses at meander apexes during the 1% (1/100) ACE flood under all potential channel conditions exceed those that can be overcome by the lowest classes of turf cover (Class C turf) or by short native grasses. Thus, denser and more resistant vegetative covers, such as willows planted from live stakes, are required on meander bends, regardless of which alternative is selected. 3. Predicted peak shear stresses in straight reaches are approximately two thirds of shear stresses at meander apexes. Predicted shear stresses in the straight sections of channel of 20-22 N/m2 or pascals (0.42 - 0.46 lb/ft2) greatly exceed the critical shear stress of bare soil or alluvial material, so protective bank cover is necessary to

prevent erosion. Predicted shear stresses for straight reaches are in the range where continuous grass or turf covers should be able to overcome shear stress, once fully established. Non-vegetated parts of the channel (e.g. below the low flow channel top) may be eroded during high flows, undermining more resistant upper banks.

4. The results suggest that benching will reduce shear stresses during flows that inundate the lower floodplain.

5. The highest bank shear stresses predicted by the model for existing conditions under the 1% (1/100) ACE flood are 31 pascals (0.65 lb/ft2). These stresses are found in the upstream meander bend, on the Santa Cruz side of the river. This stress suggests a design shear stress of 1.49 lb/ft2, allowing for factors of safety and for temporary fluctuations in stress. This stress level can be resisted by Class A and B turf, 6-inch rip rap, live brush mattresses (once established), and live willow stakes.

6. The highest predicted shear stress under the USACE Alternative 2A for the same site under the 1% (1/100) ACE flood is 27 pascals (0.56 lb/ft2), 13% lower than under existing conditions. This stress yields a design stress of 1.3 lb/ft2. Though less than under existing conditions and so providing a greater safety margin for use of the materials outlined above, this reduction in expected shear stress does not expand the range of bank protection materials that can be safely used.

7. The predicted peak shear stress… (8-foot deep bench on the Santa Cruz side of the river) is 24 pascals at the upper meander bend (0.5 lb/ft2) with a design stress of 1.15 lb/ft2. This is 23% lower than the predicted shear stresses under existing conditions and 10% less than under USACE Alternative 2A. In addition to the materials that are suitable for existing conditions or USACE Alternative 2A, this stress is within the range that can also be stabilized by the use of long native grasses.

3.2 Wind-Wave

Wind waves were not considered in detail because other recent studies indicated that the fetch distance and depths are insufficient for the the development of significant waves runup and overtopping. Levee overtopping ocould erode out the back side of the levee, which would reduce the levee stability which could eventually lead to a breach. The reaches under consideration are narrow and generally do not have long fetch lengths, and therefore will most likely not be able to generate large waves. Based on these reasons, the geotechnical fragility curves will probably change little, once wind-wave action is considered.

4 RISK AND UNCERTAINTY ANALYSIS

The flood risk faced by the communities of Pajaro in Monterey County, California and Watsonville in Santa Cruz County under the future without-project condition. Has been quantitatively characterized by examining the chance of flooding (i.e., how often the area can be expected to flood) and the consequences of flooding (i.e., who and what are expected to be impacted) using HEC-FDA. All inputs for Risk and Uncertainty were developed using the guidance in Engineering Manual (EM) 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies, USACE, 1996.

4.1 Index Points

Hydraulic results are available at each cross section and grid in the HEC-RAS model. For economic purposes, a single point is needed to represent each reach and is often referred to as an index point. The levees within the study reach are already separated by a waterway, are further divided into reaches represented by similar geotechnical conditions, as described in the geotechnical report. Each reach is represented by a single index point located at the same position as the geotechnical fragility curve.

Data is generated at representative index points within each reach and are used to estimate project performance statistics under both without-project and with-project conditions. The engineering data is also used in conjunction with economic data to estimate expected damages and benefits. Both sets of results are then used together to describe the flood risk in the study area. The index points used for the project are listed in Table 4-1.

Table 4-1: Pajaro Index Points

Index Point	Source of Flooding	Bank	Economic Impact Area
1	Pajaro River	Left	Downstream of HWY 1
2	Pajaro River	Right	Downstream of HWY 1
3	Pajaro River	Right	Upstream of HWY 1
4	Pajaro River	Right	Area between Salsipuedes Creek & Pajaro River
5	Pajaro River	Left	Upstream of HWY 1
8	Pajaro River	Left	Upstream of HWY 1
7L	Corralitos Creek	Left	North of Lakeview Road
7R	Corralitos Creek	Right	Upstream of HWY 1
10	Salsipuedes Creek	Left	Area between Salsipuedes Creek and Pajaro River

Hydraulic data were generated as inputs for the risk analysis to be performed by Economics using HEC-FDA The Hydrologic Engineering Center's Flood Damage Assessment (HEC-

FDA) is the principal tool used by the Corps to calculate flood damage risks. The HEC-FDA model performs the Monte Carlo random sampling of the discharge-frequency, stage-discharge, stage-probability of failure, and damage-stage relationships, and their respective uncertainty distributions. The primary outputs of HEC-FDA are expected annual damage (EAD) and project performance statistics. Project performance statistics include the annual exceedance probability (AEP), or the expected annual probability of flooding in any given year, the long-term risk of flooding over a 10-, 25-, or 50-year period, and the conditional non-exceedance (CNP) probability for specific events (the probability of passing specific flood events).

4.2 Hydrologic and Hydraulic Uncertainty

Engineering uncertainty used in the economic modeling is located in Attachment 6 and can also be found in the HEC-FDA models. The two main engineering uncertainties are:

- In-channel discharge uncertainty, known as the Discharge-Probability Function and,
- In-channel stage uncertainty, better known as the Stage-Discharge Uncertainty.

Discharge Uncertainty/ Discharge-Probability Function used in HEC-FDA was calculated using the Equivalent Record Length (ERL) information provided by USACE engineers in the 1997 Hydrology Report. The HEC-FDA program uses the ERL to compute uncertainty in discharge for a range of exceedance probability events. Uncertainty in the discharge versus frequency curves developed from gauged data for the Pajaro River at Chittenden and Corralitos Creek at Freedom is based on a systematic record length of 57 and 40 years, respectively. Longer ERLs imply less uncertainty in discharge.

Uncertainty in the flow-frequency curve for Pajaro River below its confluence with Salsipuedes Creek is based on an equivalent record length equal to 95 percent of the systematic record length for the Pajaro River at Chittenden (0.95 x 57 years= 54 years). The systematic record length at Chittenden was reduced to account for uncertainty due to contributing flows from Salsipuedes Creek, which represent a relatively small percentage (less than 10%) of contributing flows from the Pajaro River at Chittenden and above its confluence with Salsipuedes Creek. Uncertainty in the flow-frequency curve for Salsipuedes Creek at its confluence with the Pajaro River is based on an equivalent record length equal to 75 percent of the systematic record length for Corralitos Creek at Freedom (0.75 x 40 years= 30 years). The systematic record length was reduced to account for uncertainties introduced by routing flows from Corralitos and Salsipuedes Creeks through College Lake.

Stage Uncertainty/Stage-Discharge Uncertainty values were selected From Table 5-2 of EM 1110-2-1619. Selecting “Fair” Manning’s n Value Reliability and a Cross Section Based on a Field Survey the Standard Deviation in feet from Table 5-2 in stage was selected as 0.7.
Due to the relative uncertainty in hydrology another approach was used to estimate stage uncertainty by estimating the upper and lower bounds of a stage. For this effort the 1% containment model geometry was used and 1% discharges were varied by +20% and -20%. Please see Plate 29. The range between the upper and lower limit of these water stages were then used to estimate the standard deviation of stage uncertainty. Using equation 5-7 Pajaro 1% contain standard deviation can be estimated as:

$$S = {}^{E_{mean}}/_{4} \quad \text{or} \quad S = {}^{2.51}/_{4} \text{ or } S = 0.63 \text{ or Round up to } 0.7$$

For this effort a uniform standard deviation of 0.7 feet from table 5-2 was applied.

4.3 Performance Evaluation

Future without-project annual exceedance probability (AEP) was computed on a reach/index point-specific basis using the HEC-FDA model. The HEC-FDA model integrates the hydrologic, hydraulic, geotechnical and economic relationships with uncertainty to create exceedance probability-damage functions with uncertainty.

The annual exceedance probability (AEP) represents the percent chance of a target stage being exceeded in any given year, thereby causing flooding and subsequent significant property damage. The annual exceedance probability results for each damage area are computed by HEC-FDA based on specific engineering data: frequency-stage curve, equivalent record length, and top-of-bank stage.

The AEP results were used to establish the future without-project expected annual damages (EAD) to determine economic benefits and evaluate performance of the alternatives. Table 6-4 shows the results of the levee performance evaluation for each index point in the project area. The future without project condition is included in Table 6-4 because it is the basis of comparison for the alternatives; this is discussed in greater detail in Section 2.2. More information about the economic benefits and expected annual damages can be found in the economic appendix. Tables 4-2 and 4-3 display the engineering performance statistics for the without and with-project conditions.

Table 4-2: Without Project Engineering Performance Statistics

Without Project Engineering Performance Statistics							
System	**EIA**	**AEP**	**Assurance**				
			10%	**4%**	**2%**	**1%**	**0.2%**
Pajaro River	**A**	8.50%	72%	31%	11%	3%	1%
	B	7.30%	78%	37%	14%	4%	1%
	C	6.40%	83%	38%	13%	3%	1%
	D	8.60%	72%	28%	8%	2%	1%
	E	5.90%	87%	39%	12%	3%	1%
	F	N/A	N/A	N/A	N/A	N/A	N/A
Tributaries	**A**	N/A	N/A	N/A	N/A	N/A	N/A
	B	23%	4%	1%	1%	1%	1%
	C	N/A	N/A	N/A	N/A	N/A	N/A
	D	23%	4%	1%	1%	1%	1%
	E	25%	58%	28%	14%	6%	1%
	F	46%	1%	1%	1%	1%	1%

Table 4-3: With-Project Engineering Performance Statistics

With Project Engineering Performance Statistics							
System	**EIA**	**AEP**	**Assurance**				
			10%	**4%**	**2%**	**1%**	**0.2%**
Pajaro River	**A**	8.50%	72%	3100%	1100%	300%	100%
	B	7.30%	78%	99%	97%	87%	43%
	C	0.40%	99%	99%	97%	86%	40%
	D	0.50%	99%	39%	12%	3%	1%
	E	5.90%	87%				
	F	N/A	N/A	N/A	N/A	N/A	N/A
Tributaries	**A**	N/A	N/A	N/A	N/A	N/A	N/A
	B	7%	78%	37%	14%	4%	1%
	C	N/A	N/A	N/A	N/A	N/A	N/A
	D	0.3%	99%	99%	99%	89%	61%
	E	25%	58%	28%	14%	6%	1%
	F	2%	99%	90%	63%	30%	4%

4.4 Considerations and Assumptions

The results of the Risk Analysis are affected by technical considerations and assumptions regarding the input to HEC-FDA. For example, the geotechnical studies developed relationships that characterized the reliability of the levees, which were utilized assess the probability of levee failures in the FDA models, . Perhaps the most significant assumption is the failure methodology, which can significantly influence simulated flood flows. The methodology was chosen to provide a conservative and consistent simulation of potential flooding extent for system-wide hydraulic and economic evaluations. It does not necessarily represent conditions that would occur during an actual flood event, when flood fighting and other emergency actions are likely to take place.

As discussed previously there are several without project and no action conditions to consider. At this stage of the study only the alternatives described are being considered. As the study progresses and refinements are made to the engineering inputs (hydraulics, hydrology, geotechnical), these conditions may be updated and the conditions not yet considered will also be studied.

5 RESIDUAL RISK

Residual risk is the risk of being inundated after the selected alternative has been implemented which can include residual risk associated with the project features, residual risk from physical conditions not related to project features, and residual risk from an event exceeding the design of the system. Residual flood risk after completion of the selected plan would vary throughout the study area.

5.1 Residual Flooding

Please see the composite floodplain inundation mapping on plates 40 through 47 for resultant project floodplains. Consequences of flows exceeding the discharge capacity on the project have the potential for additional erosion and scour not previously identified.

5.2 Climate Change

The plan for Climate Change analysis is ongoing; more analysis is expected to provide greater insight. Erosion repairs are expected to be part of all alternatives in some fashion and refinement efforts will continue beyond the Tentatively Selected Plan (TSP) milestone.

5.2.1 Sea Level Change

Sea level elevations are expected to increase during the project life of any project selected for flood control on the Pajaro River. There are several estimates for sea level rise over the next 50 to 100 years.

The current (2017) project design assumes a starting water surface elevation in the tidal zone of the Pajaro River of Mean Tide and Normal Depth was used as the downstream boundary condition (Plate 30). The project design has been checked for sensitivity to starting water surface elevations. The starting water surface elevation can be as high as 11.0 feet stages are impacted within the improvement reach of the recommended plan. The current levee design elevation at the downstream elevation for Alternative 2A with bench excavation is 15.5 feet NGVD 1929. The Table below presents the estimated increase in sea level for a project life of 50 years (2015 to 2065) using the Sea-Level Change Curve Calculator from the USACE Climate Preparedness and Resilience webpage at http://corpsclimate.us/ccaceslcurves.cfm. Using the Monterey, CA gauge the estimated changes in sea level for the Low, Medium and High conditions are listed in Table 2.

Table 2. Estimated Relative Sea Level Change

Estimated Relative Sea Level Change
from 2015 To 2065 Pajaro River
9413450, Monterey, CA
NOAA's Regional Rate: 0.00449 feet/yr
All values are expressed in feet

Year	USACE Low	USACE Int	USACE High
2015	0.00	0.00	0.00
2020	0.02	0.05	0.12
2025	0.05	0.10	0.25
2030	0.07	0.15	0.41
2035	0.09	0.21	0.58
2040	0.11	0.27	0.77
2045	0.14	0.34	0.98
2050	0.16	0.41	1.21
2055	0.18	0.49	1.46
2060	0.20	0.57	1.72
2065	0.23	0.65	2.00

Even with the most conservative estimate of NRC Curve Three, the estimated 1% tide will not exceed 7.1 feet NGVD 1929 and mean higher high water will not exceed 5.1 feet NGVD 1929. Neither of these estimates would impact the performance of the proposed improvements in the recommended plan..

NRC Curve information: http://corpsclimate.us/ccaceslcurves.cfm (Monterey, CA)

5.2.2 Inland Climate Change

The plan for Inland Climate Change analysis is ongoing; more analysis is expected to provide greater insight. Erosion repairs are expected to be part of all alternatives in some fashion and refinement efforts will continue beyond the Tentatively Selected Plan (TSP) milestone.

5.3 Interior Drainage

The two major considerations for interior drainage for the recommended plan are drainage of local rainfall runoff and drainage of floods that exceed the project design capacity. The two sections below describe current designs for interior drainage facilities..

5.3.1 Pajaro River

Interior drainage facilities for localized rainfall runoff are already incorporated into the existing levees. Therefore, it is unlikely that additional features would be economically justified.

Past flood events have overtopped or caused failure of the existing Pajaro River levees. Flood waters from these events were trapped behind the existing project levees. The result was an increase in expected flood damages and the need to breach project levees on the Monterey side of the Pajaro River upstream from Highway 1 so that flooded areas could drain back into the Pajaro River. The flood event of 1995 overtopped levees in Monterey County upstream from Salsipuedes Creek. Flood waters damaged farmland and flooded the Town of Pajaro. Overbank flows continued to Highway 1 where they were stopped by the highway embankment. The floodwaters backed up into the Town of Pajaro causing additional damages. The flooded area was eventually drained into the Pajaro River by breaching the levees near Highway 1.

Model simulations of the Pajaro River Flood Control Project were performed to evaluate the minimum time required to drain each of the six storage areas through twin six-foot drainage culverts and through four six-foot drainage culverts (see Figure below for locations). For these simulations, the initial water surface elevations in each storage area were set to a maximum by assuming the storage areas to be completely filled. The starting water surface elevations in the storage areas were determined by identifying the lowest point on the ground surrounding the storage areas using a DEM of the local topography. The lowest point identified could have been based on the elevation of the top of the bounding levee for some storage areas or the low point on the landside of the bounding terrain for other storage areas. The storage area was defined as drained once the water level was less than one foot above the culvert invert elevation. A low flow condition was assumed in the river during these simulations. This low flow condition was adapted from the base flow of 10% (1/10) ACE hydrograph, which was around 400 cfs. The starting water surface elevation, depth of the drained water and minimum time required to drain

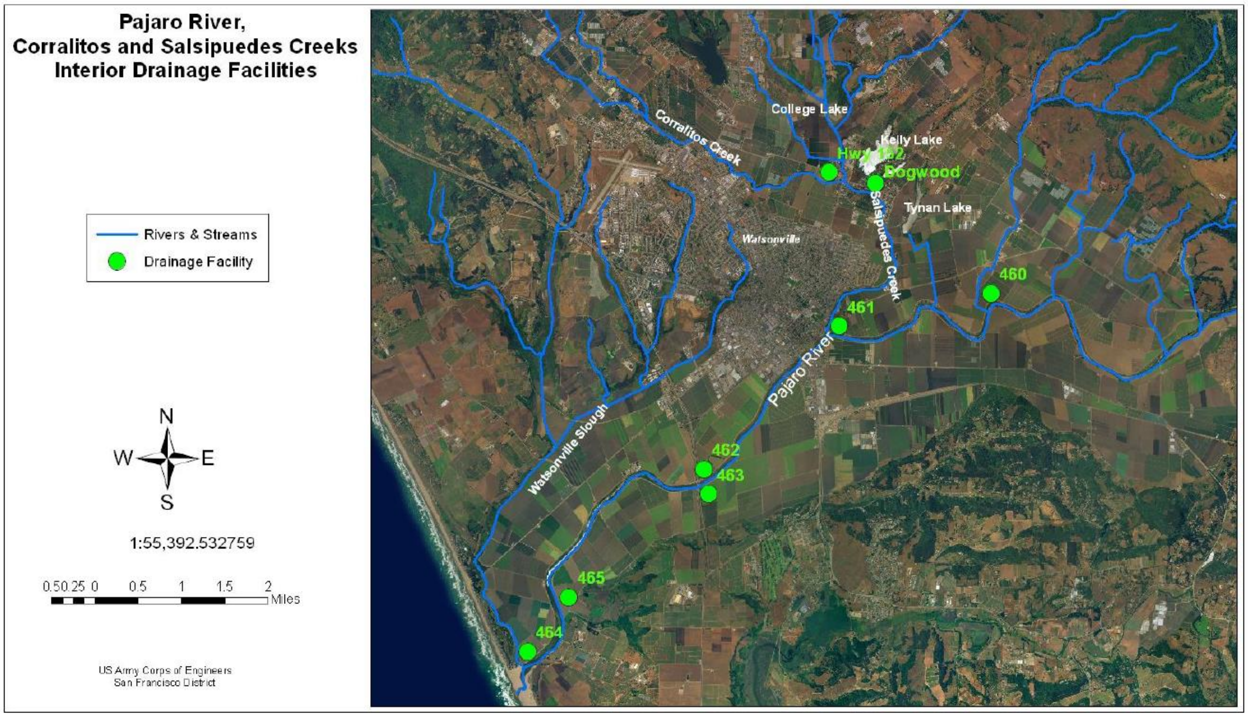


Figure 7. Interior Drainage Facilities

for each interior storage area were summarized in Table 3 presented below. Additional information on the interior drainage analysis for this report section is available from the Corps of Engineers San Francisco District.

Table 3. Interior Drainage Facility Requirements

Storage Area	Location	Starting WSEL (ft)	Depth Drained (ft)	2-6-foot Diameter Culverts Minimum Time to Drain (days)	4-6-foot Diameter Culverts Minimum Time to Drain (days)
460	Coward\Pajaro	42	4	3	2
461	Salsipuedes\Pajaro	34	4.2	3	2
462	Right Pajaro\Hwy 1	22	4.5	4	2
463	Left Pajaro\Hwy 1	26	9	10	5
464	Left Pajaro downstream	10	6	2	1
465	Right Pajaro downstream	6	3	2	1

5.3.2 Salsipuedes and Corralitos Creeks

Levee and floodwall construction on the left banks of Salsipuedes and Corralitos Creeks will create potential flood ponding in the Dogwood Lane area and the area immediately upstream from Highway 152 for localized rainfall runoff. Table 3 below presents the current drainage facilities designed for this area. The drainage facilities at this location will require more detailed design during PED. The culvert sizes below should be adequate based engineering judgment. Also the facilities presented below should be adequate for current project cost estimates of the recommended plan..

Table 4. Interior Drainage Facility Requirements Salsipuedes and Corralitos Creeks

Location	Facility Requirements
Salsipuedes Creek - Highway 152	2 – 3 foot diameter concrete pipes set at elevation 62.0 Feet NGVD 1929
Salsipuedes Creek at Dogwood Lane	2 – 3 foot diameter concrete pipes set at elevation 53.5 Feet NGVD 1929

5.4 Levee Superiority

According to ETL 1110-2-299, "Overtopping of Flood Control Levees and Floodwalls," two design types can be used to control initial overtopping. The first is the use of different levee heights relative to the design water surface from reach to reach to force overtopping in a desired location. The second design uses notches, openings, or weirs in the structure. The inverts for these features are at or above a design water surface elevation but below the neighboring top of levee. Examples are railroad or road crossings of levees and rock weirs.

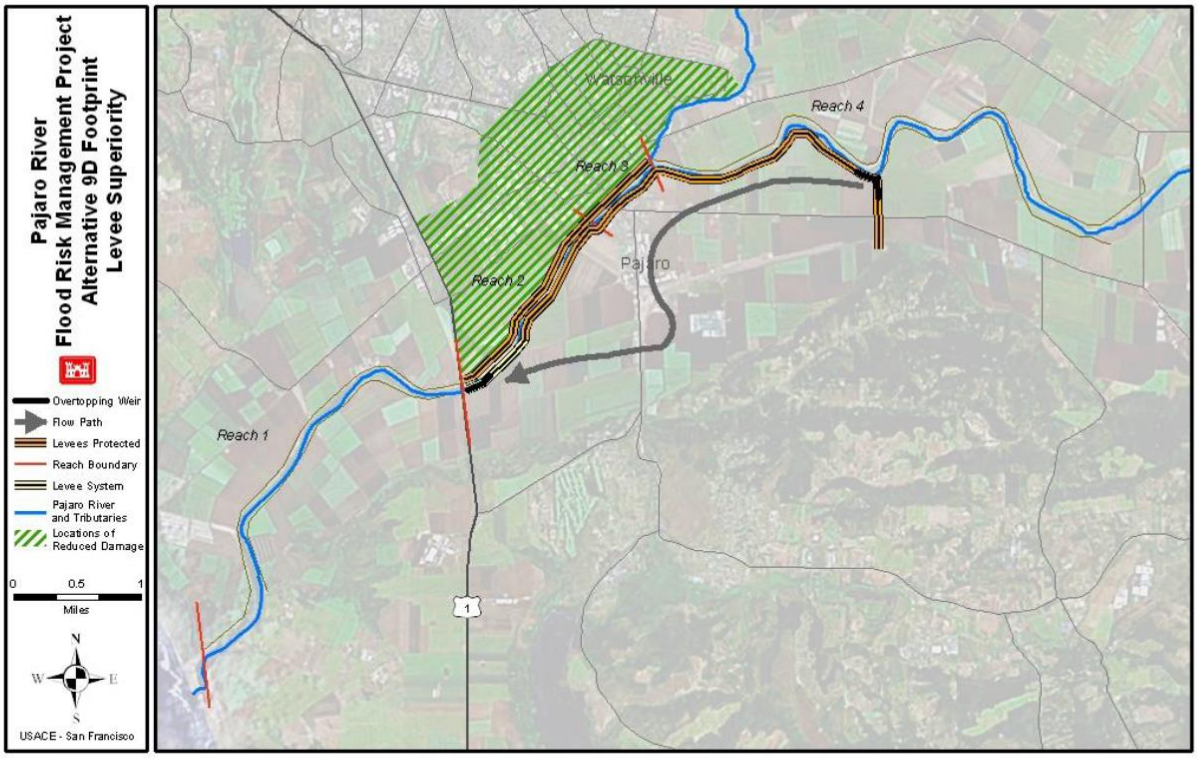


Figure 8. Proposed Overtopping Locations

As seen in Figure 6 overtopping locations were investigated for one location in Reach 4 along Alternative 9D. The upstream end of the Reach 4 levee where it joins the tie back levee was the site selected for levee superiority. Placing levee superiority at this location will provide some protection to the Alternative 9D levee system and protection to the City of Watsonville. Levee superiority plans were designed for newly constructed levees only. Modifications to the Santa Cruz county levees upstream from Salsipuedes Creek or the Monterey County levees upstream from the Alternative 9D tie back levee were not considered. Modifications to Reach 1 levees were not considered. These locations are not part of the planned construction and are not part of current design plans or estimates.

6 REFERENCES

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HEC, January 2010. "HEC-RAS, River Analysis System, Users Manual," Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, CA.

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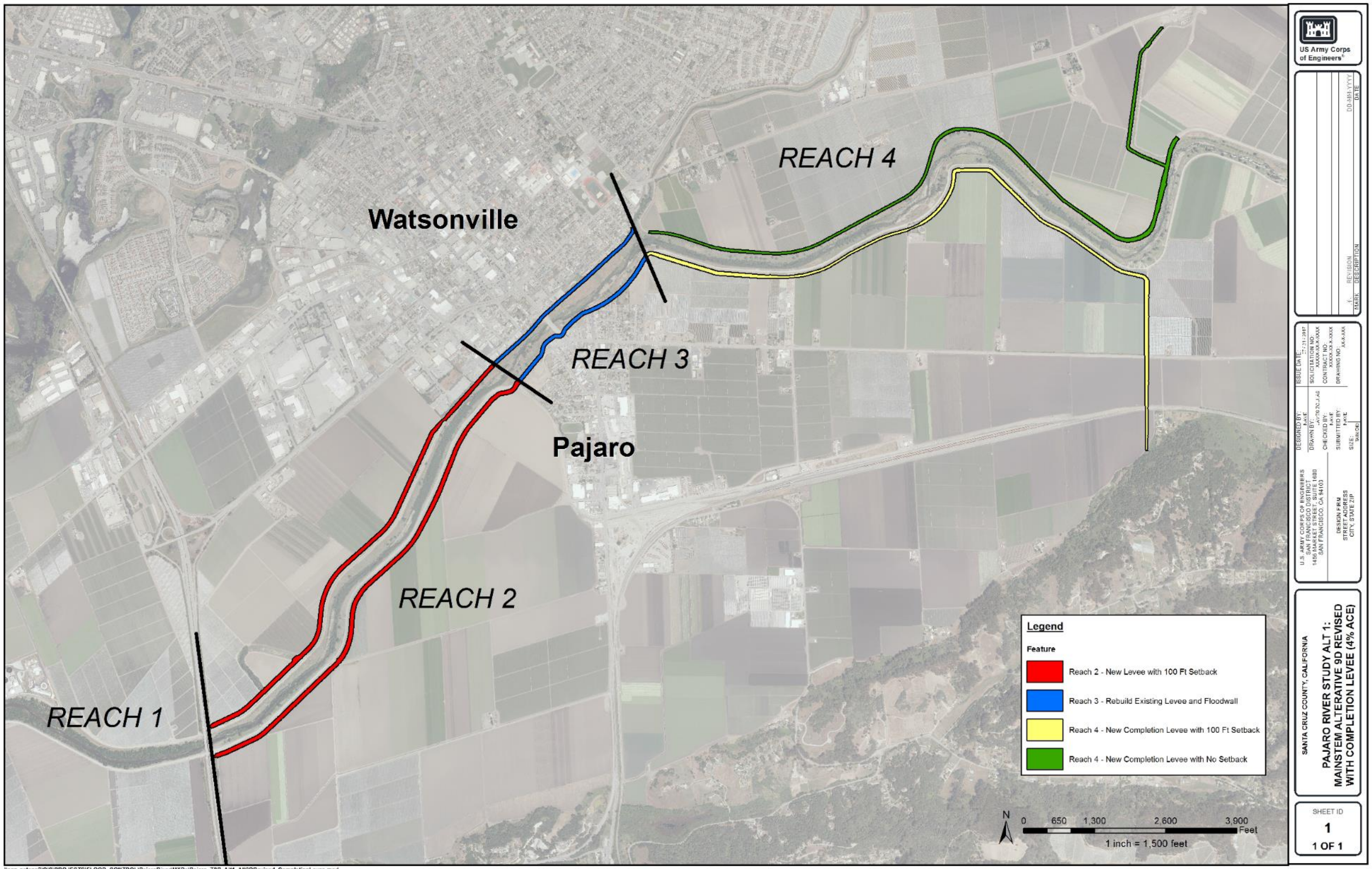


Plate 1. Alternative 1 Alt9D Revised Completion Levee

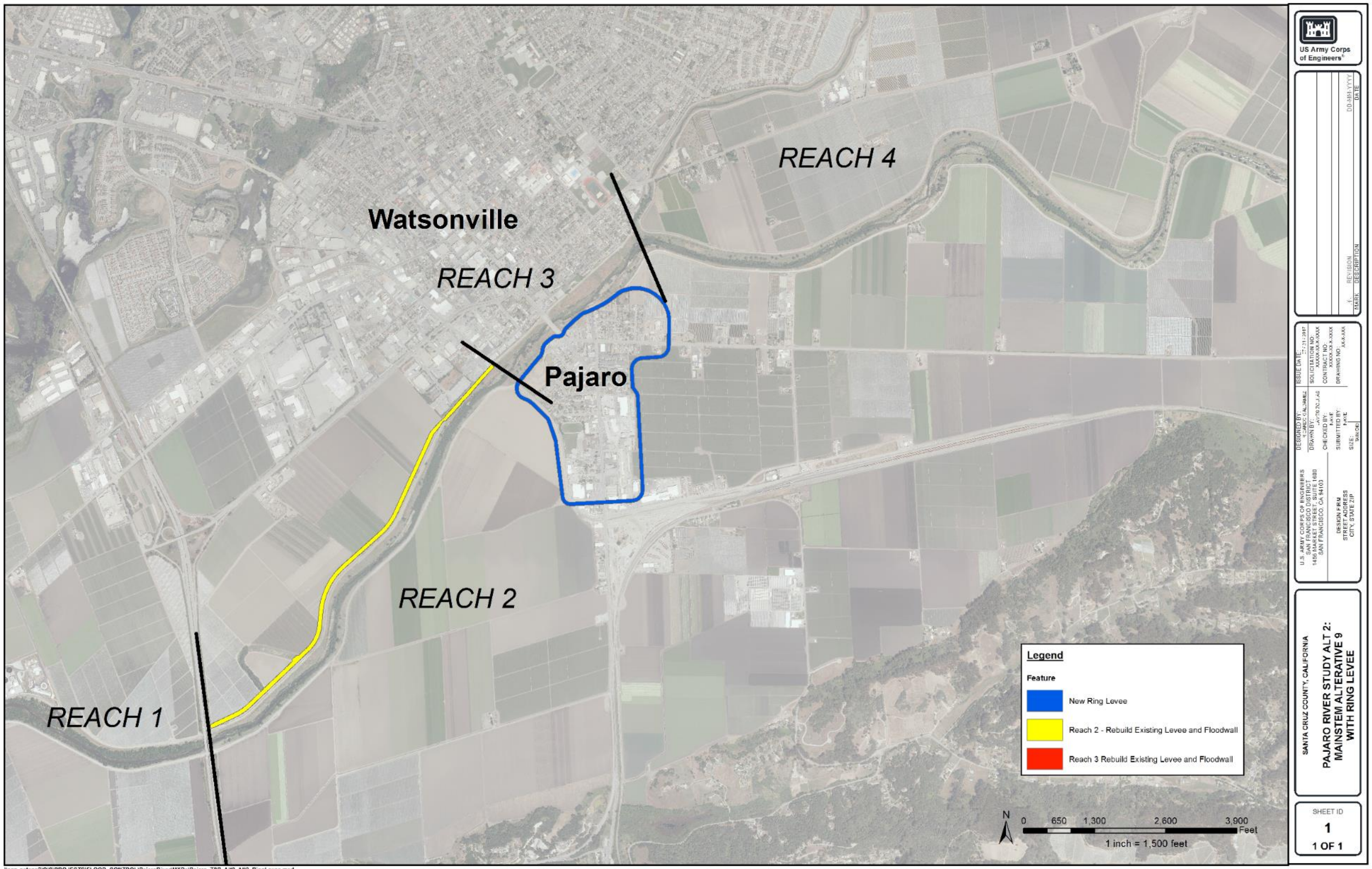


Plate 2. Alternative 2 Ring Levee

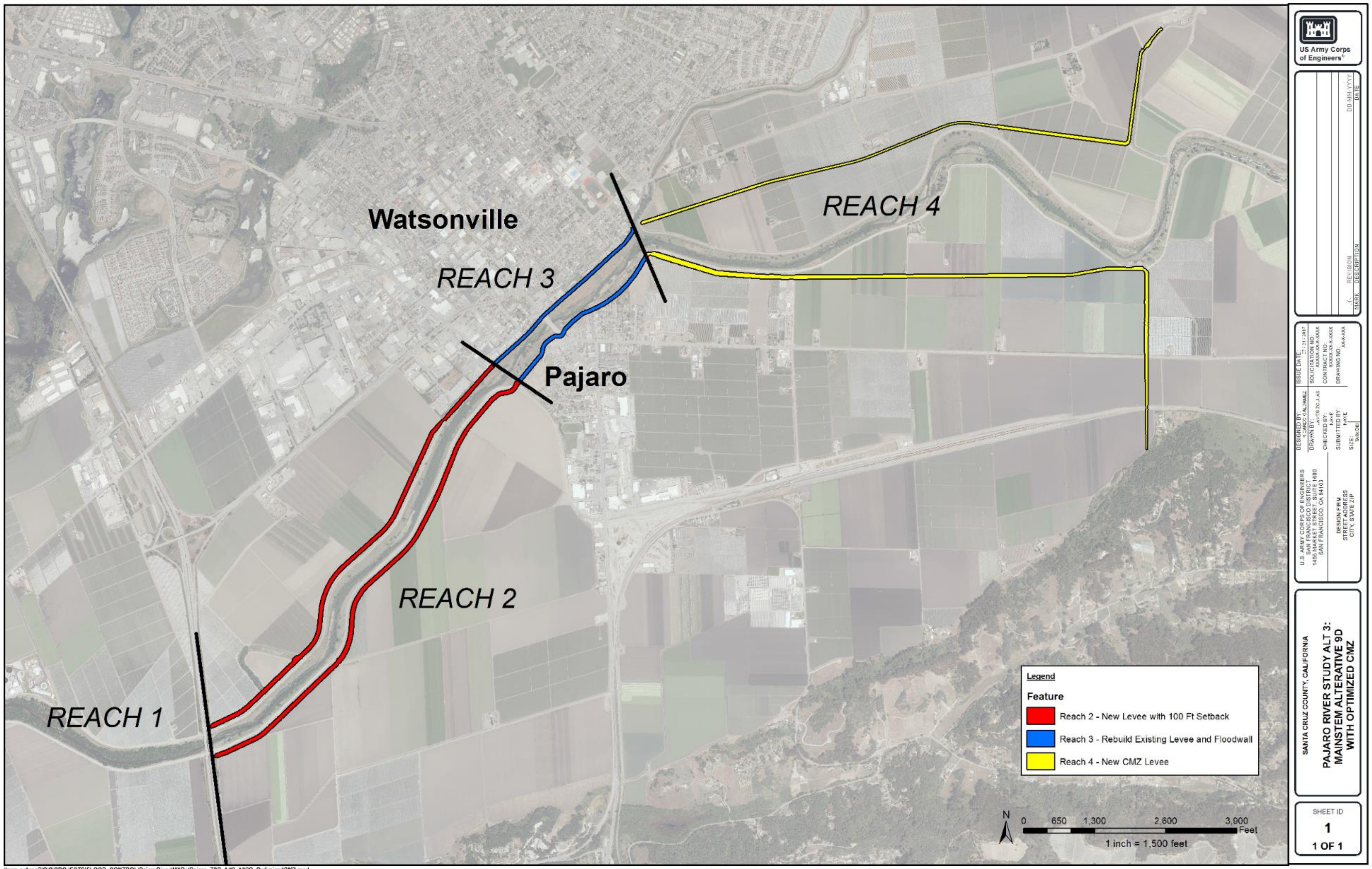


Plate 3. Alternative 3 Optimized CMZ

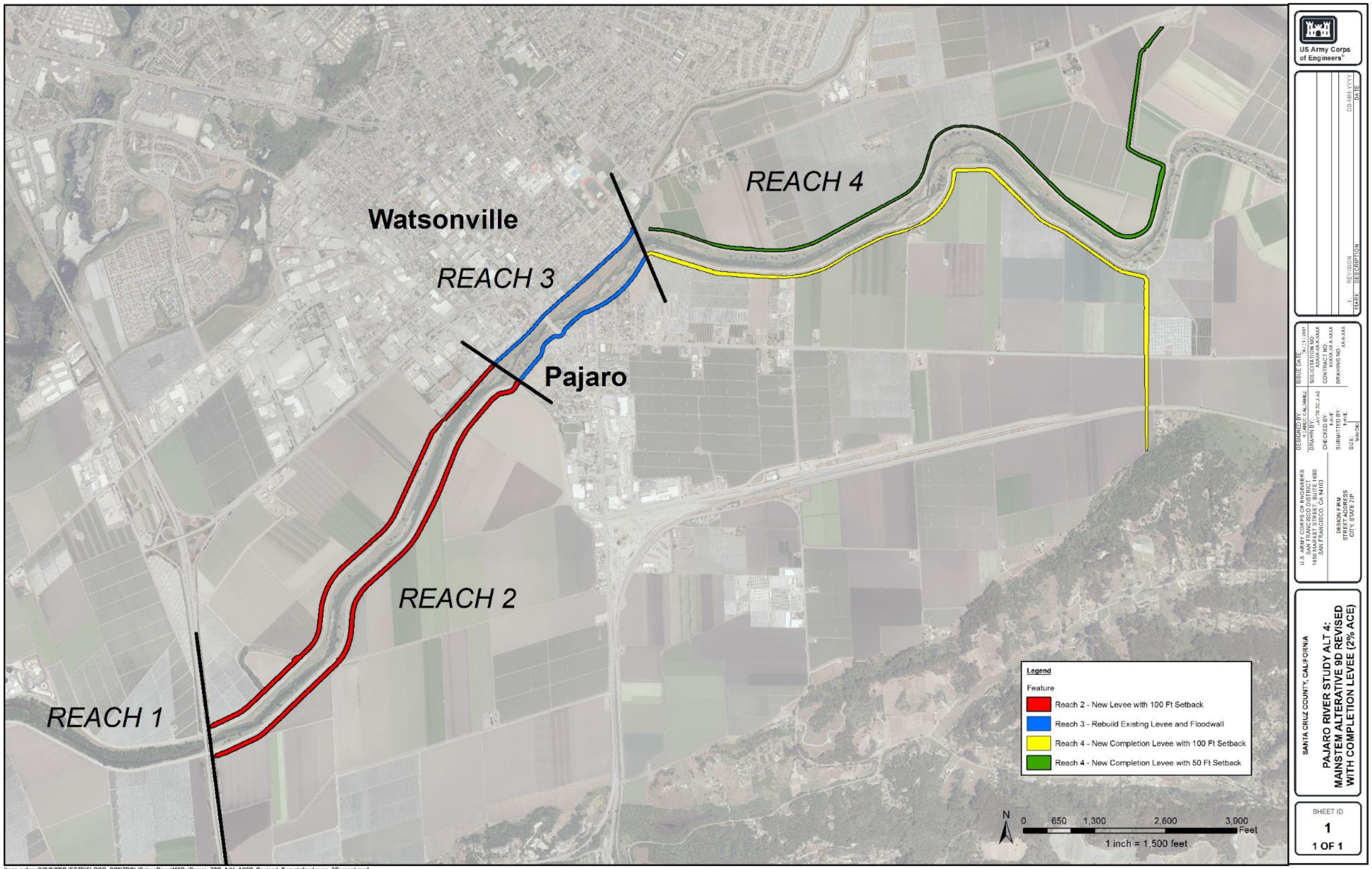


Plate 4. Alternative 4 Alt9D Revised 2% Completion Levee

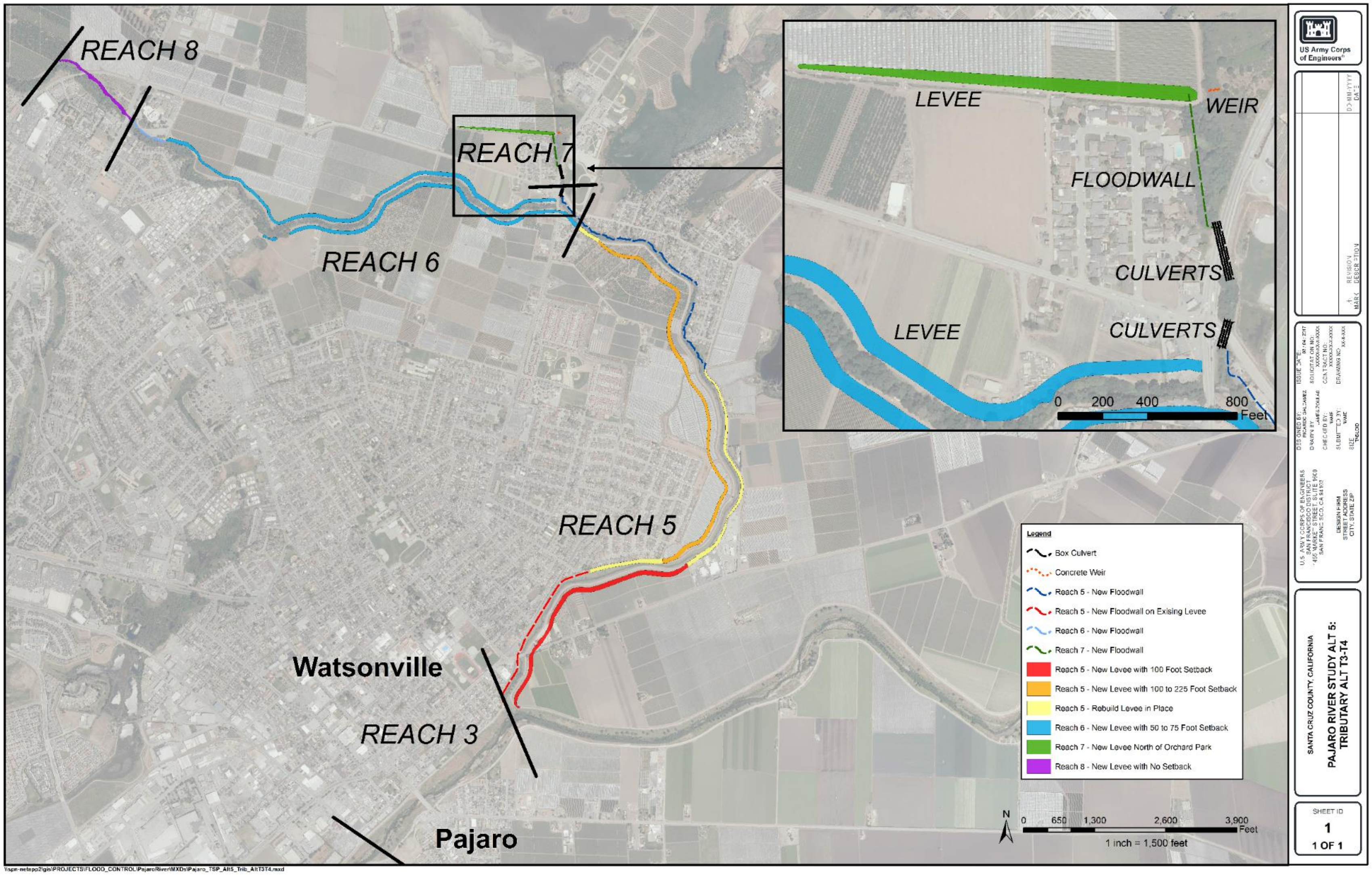


Plate 5. Alternative 5 Tributary Alt-T3T4

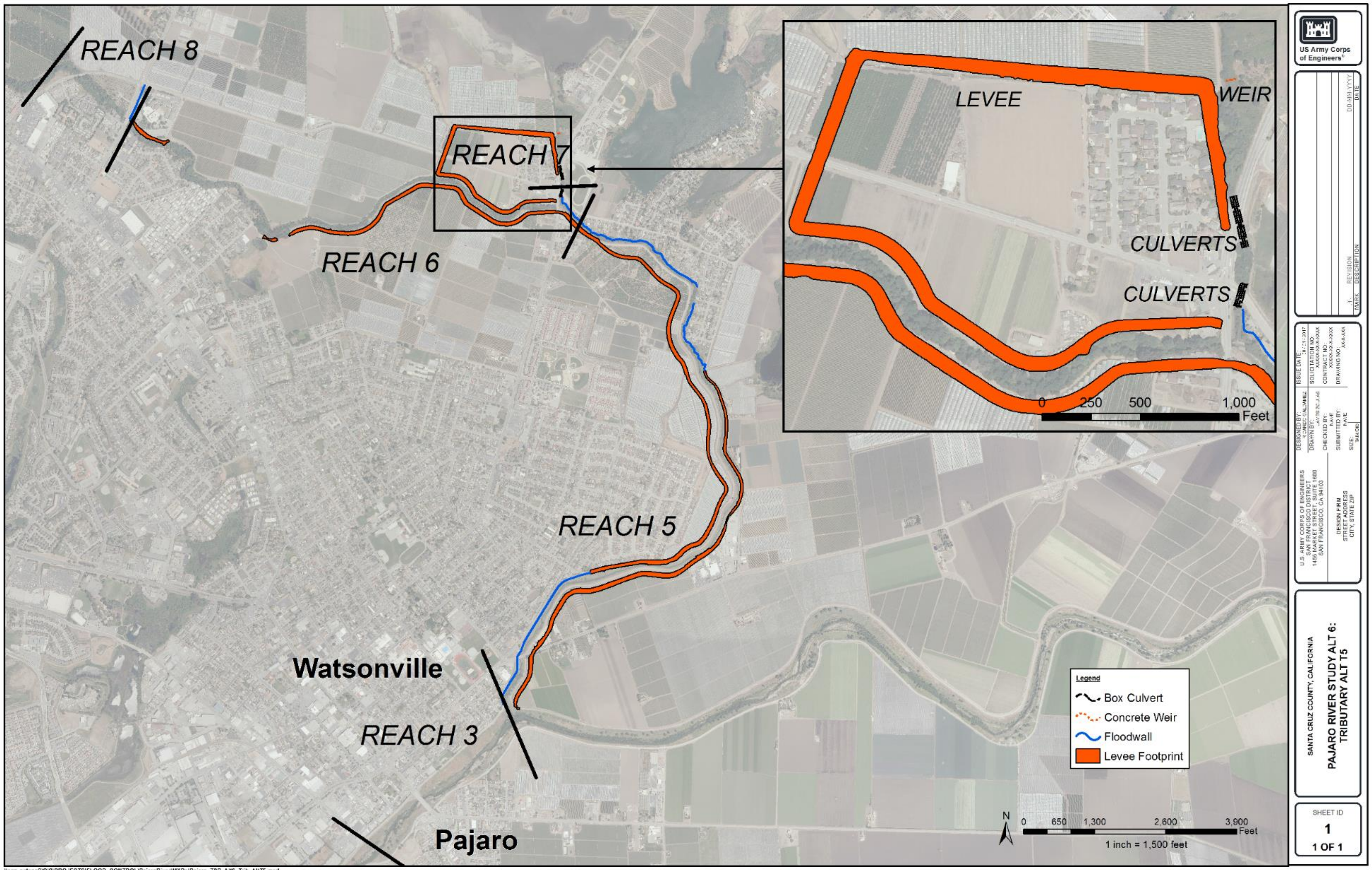


Plate 6. Alternative 6 Tributary Alternative T5

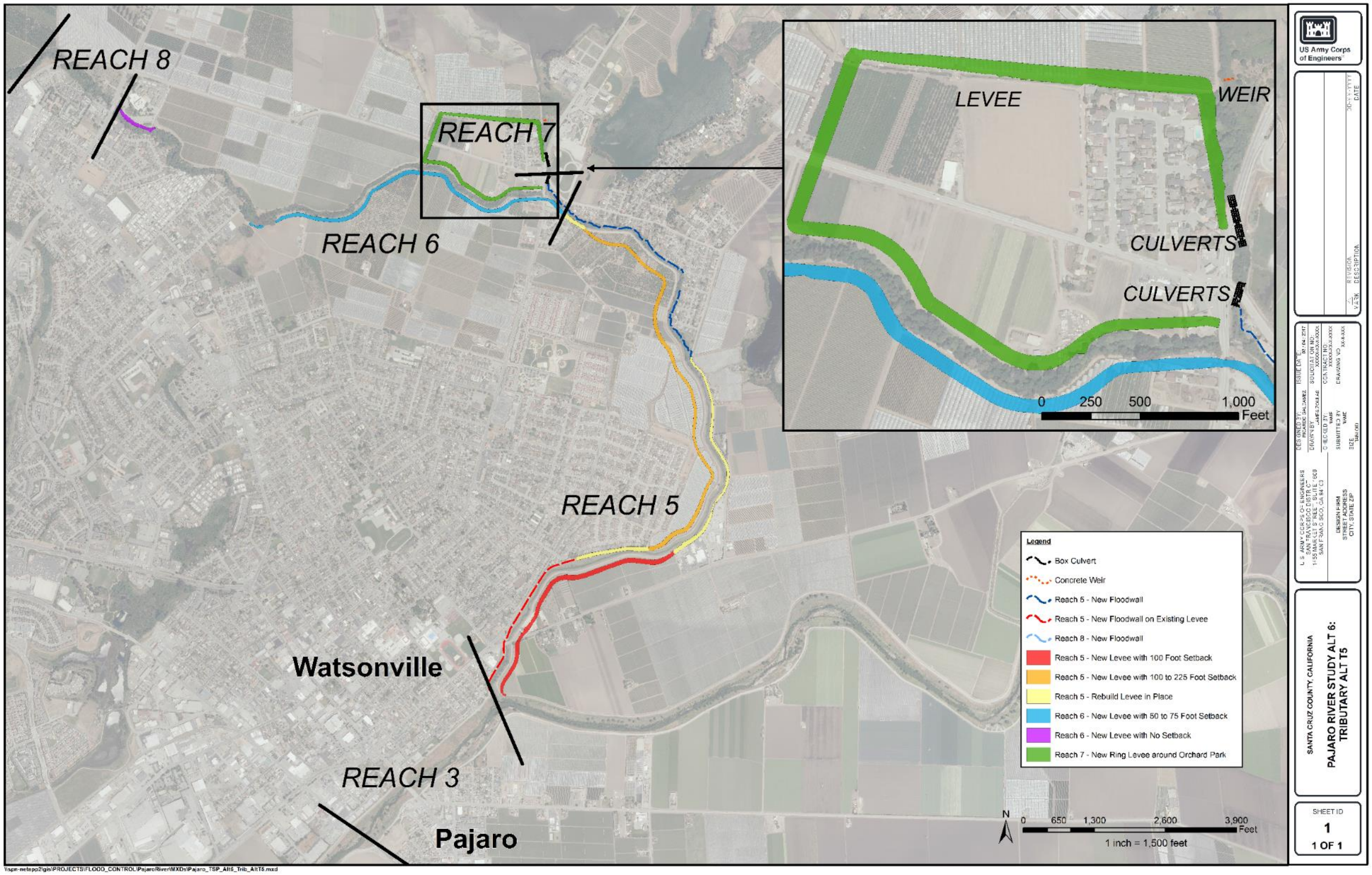


Plate 7. Alternative 6 Tributary Alternative T5 Version 2

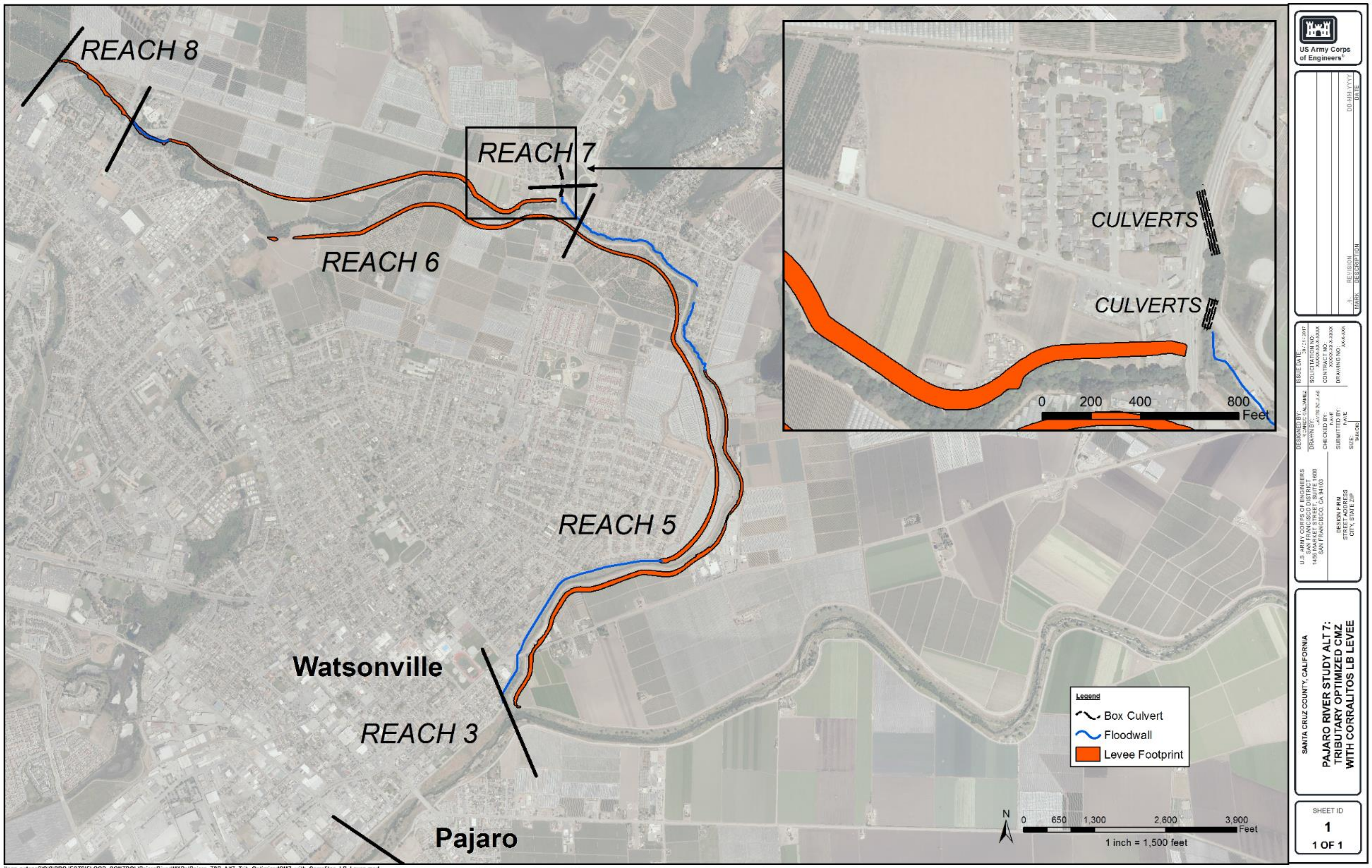


Plate 8. Alternative 7 Tributary Optimized CMZ with Corralitos Left Bank Levee

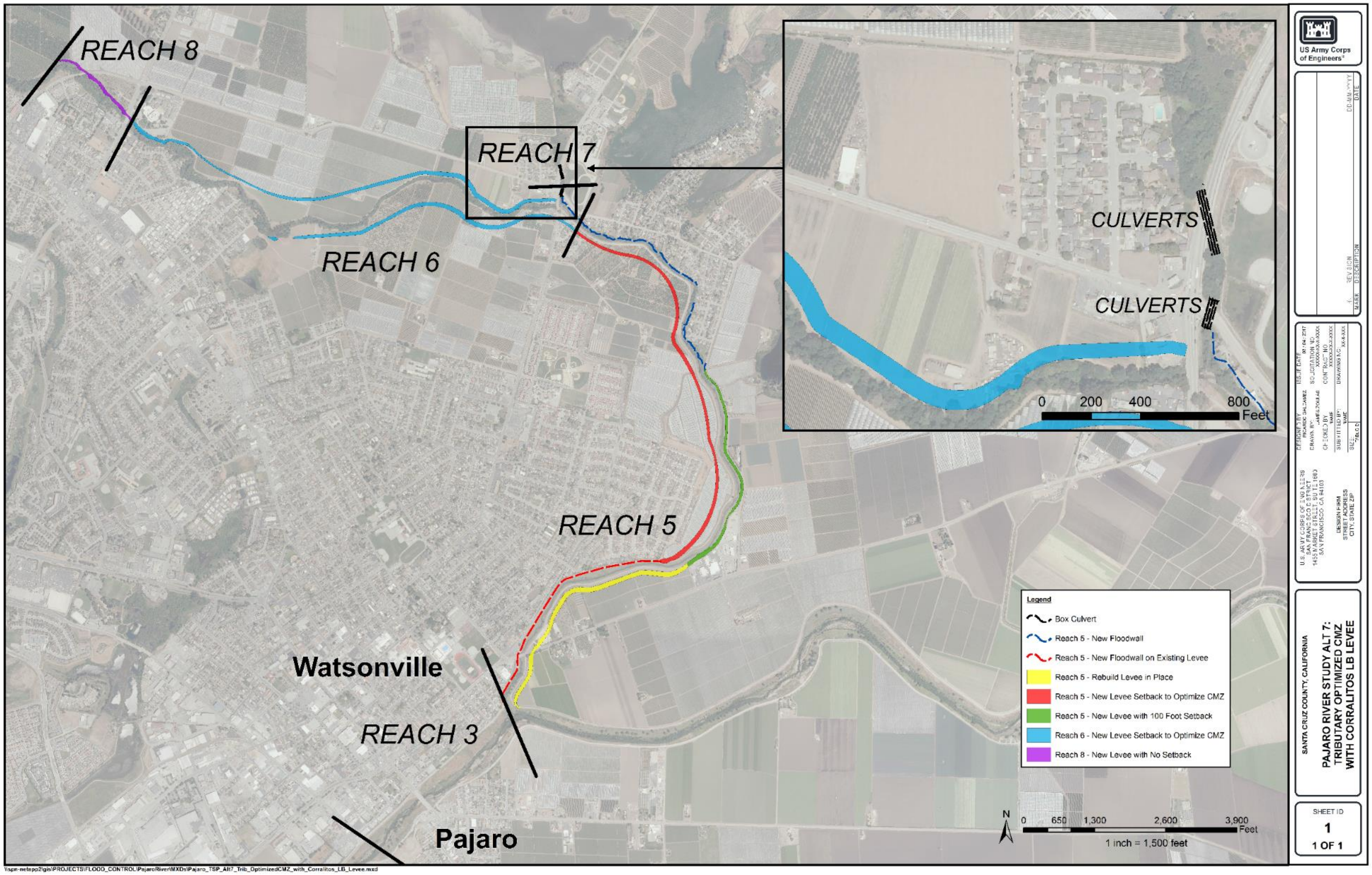


Plate 9. Alternative 7 Tributary Optimized CMZ with Corralitos Left Bank Levee Version 2

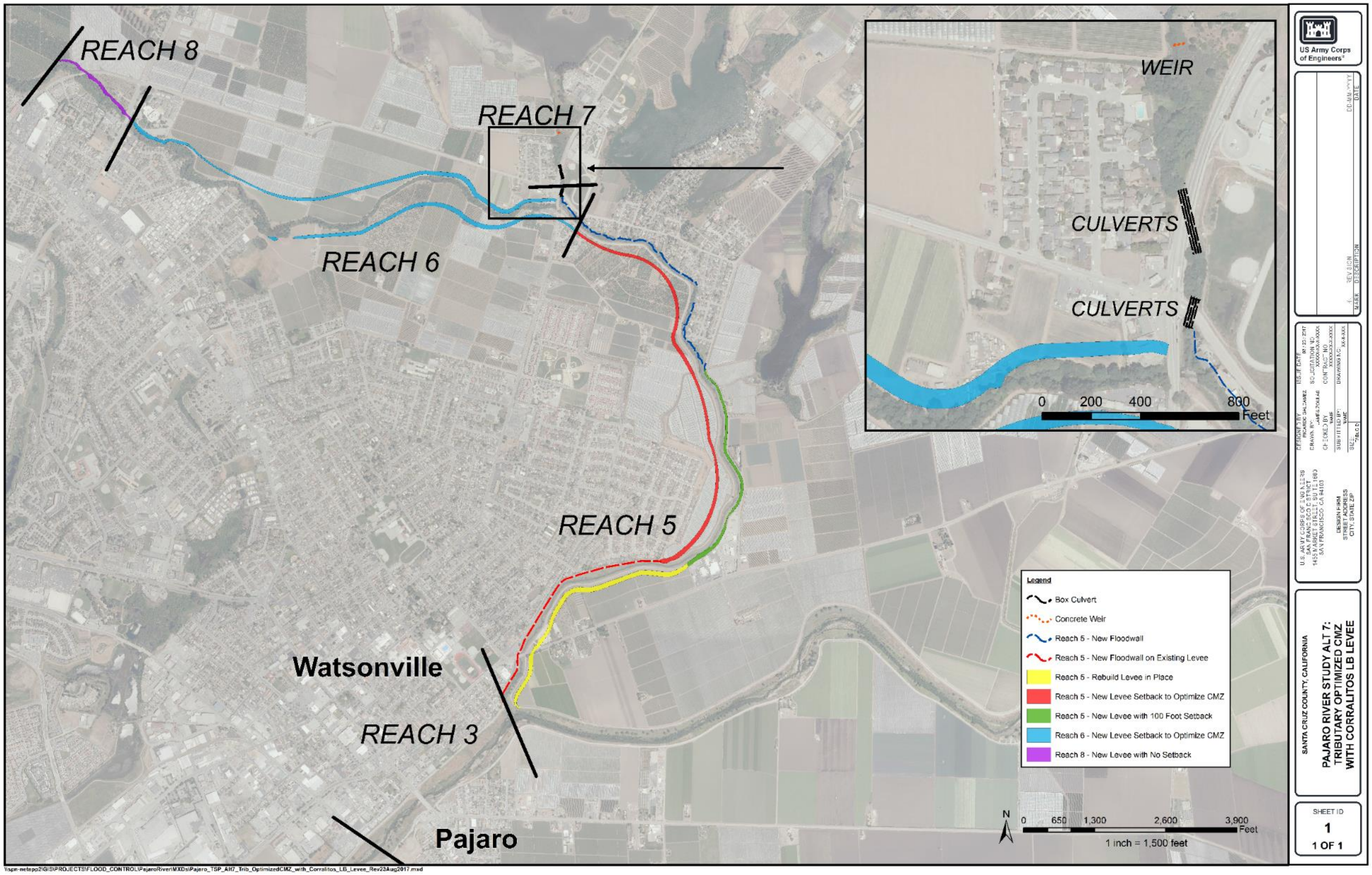


Plate 10. Alternative 7 Tributary Optimized CMZ with Corralitos Left Bank Levee Version 3

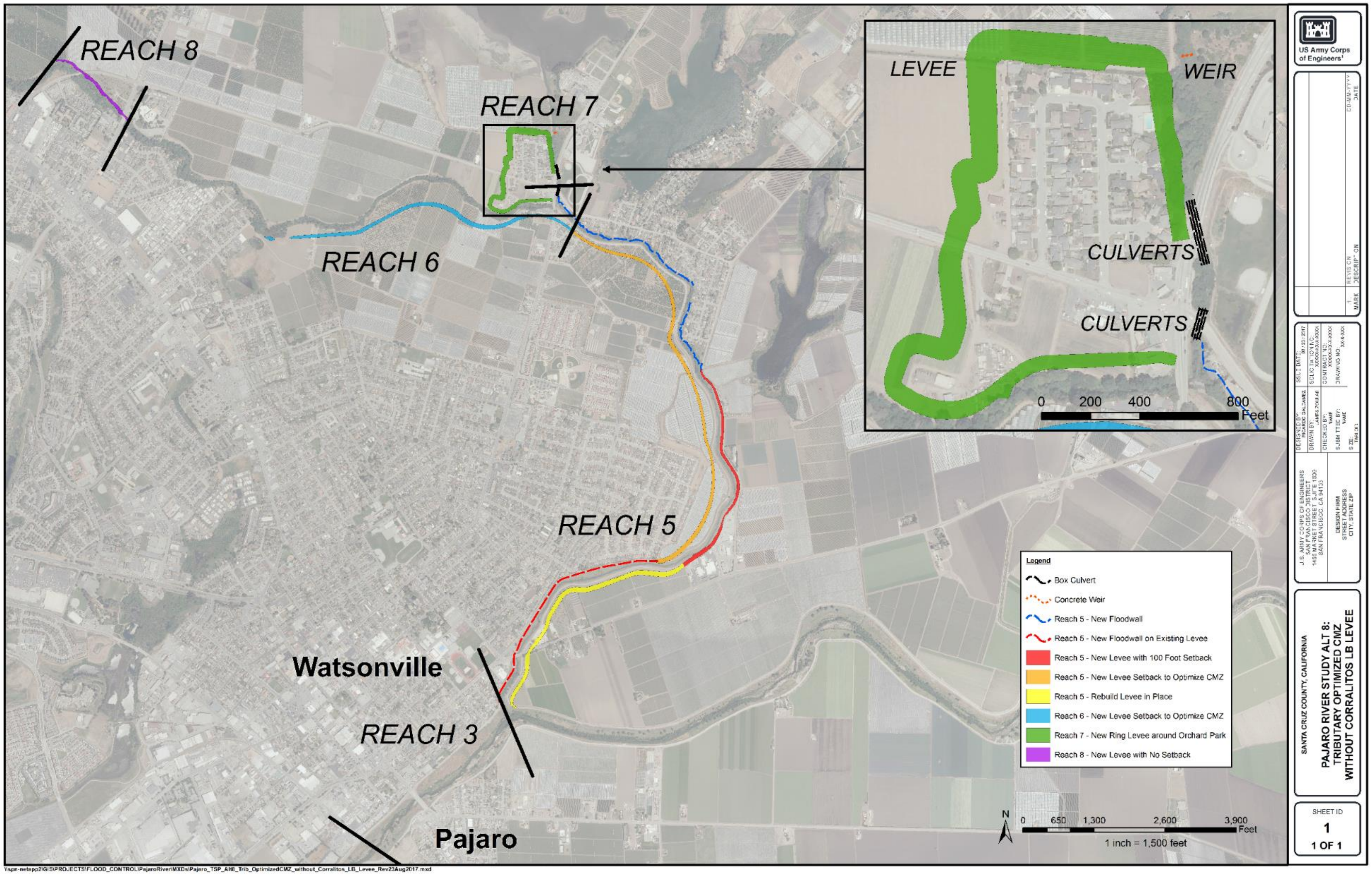


Plate 11. Alternative 8 Tributary Optimized CMZ without Corralitos Left Levee

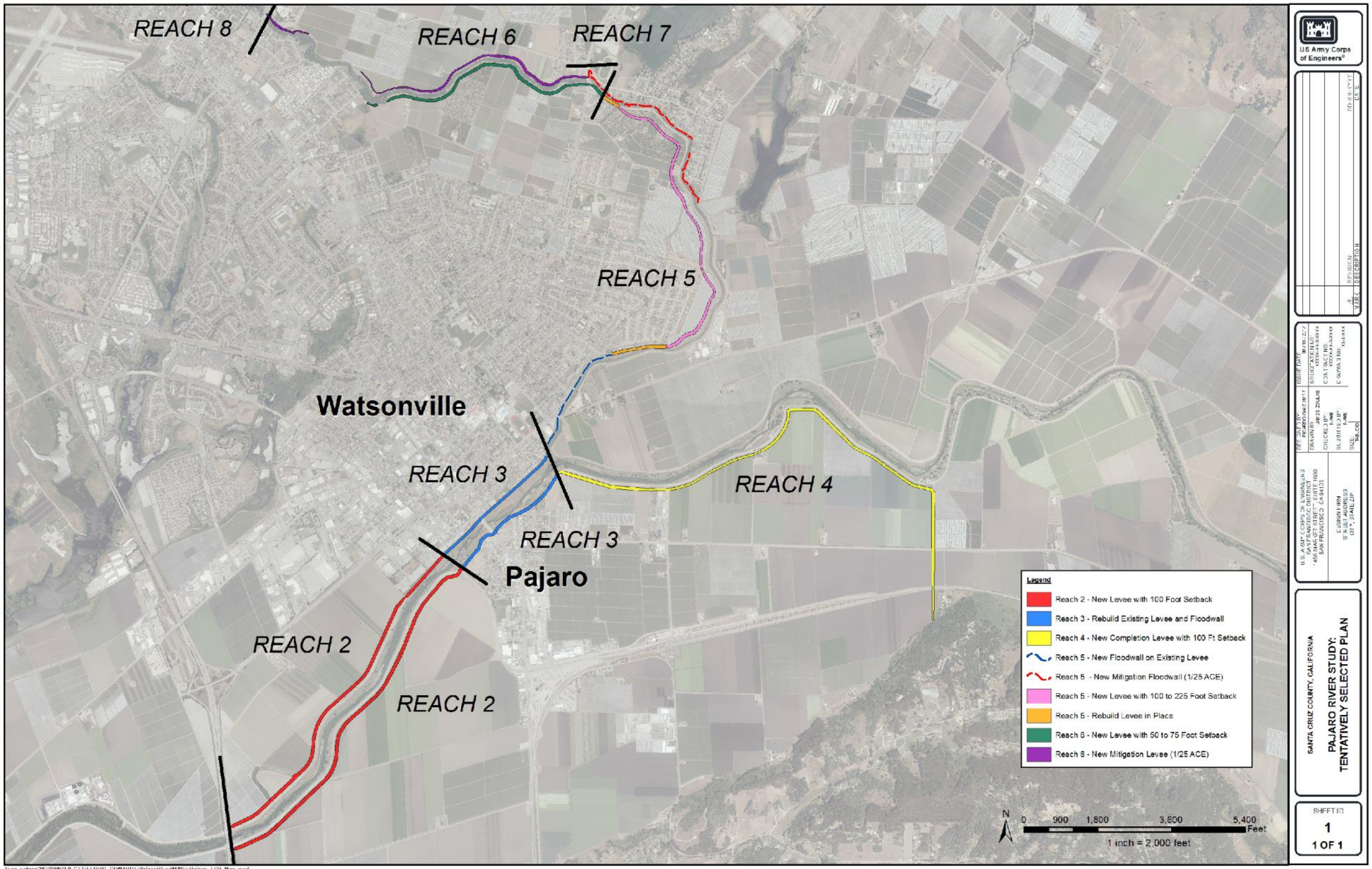


Plate 12. Recommended Plan (TSP)

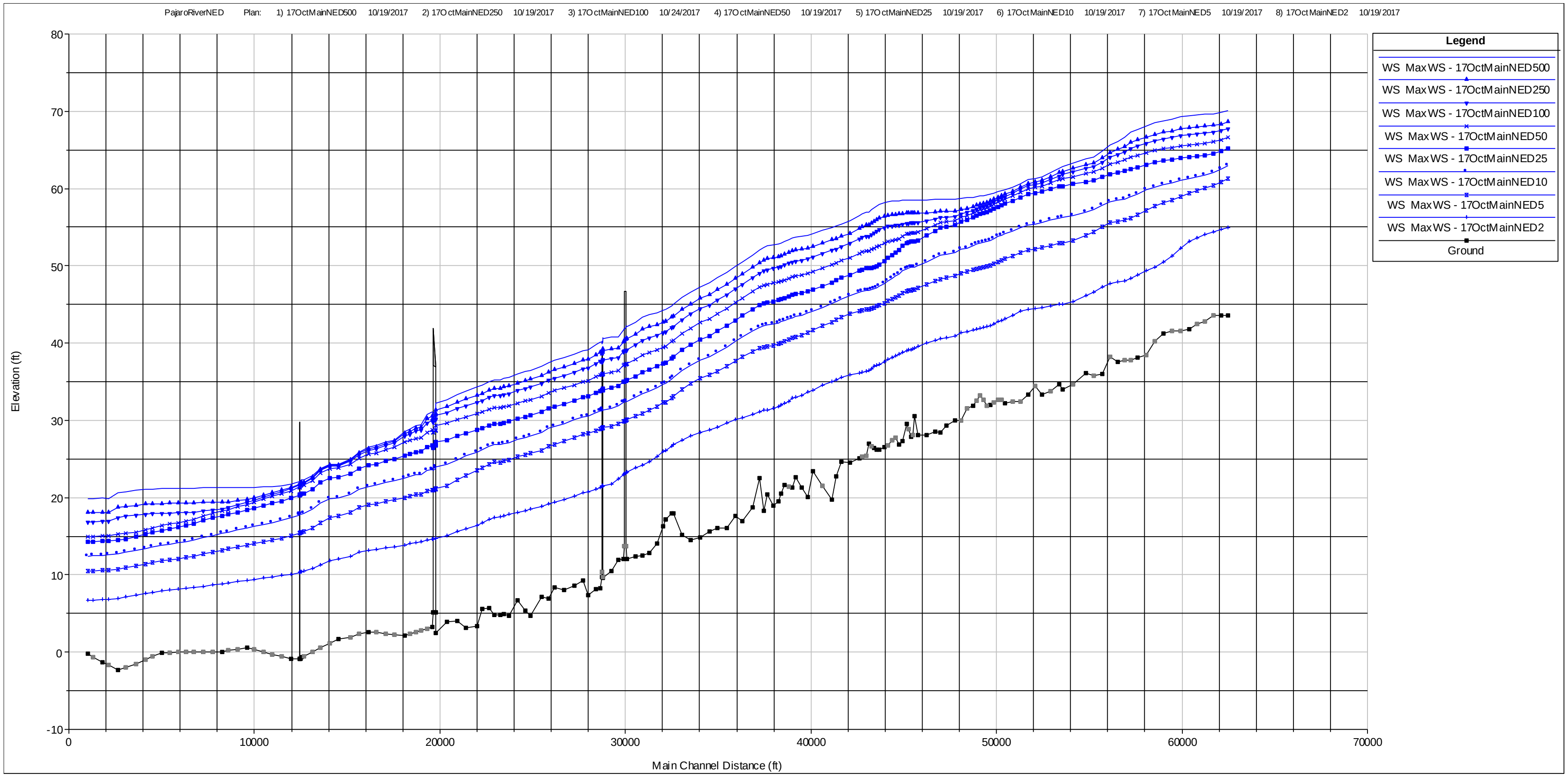


Plate 13. Pajaro River Recommended Plan Median Water Surface Profiles

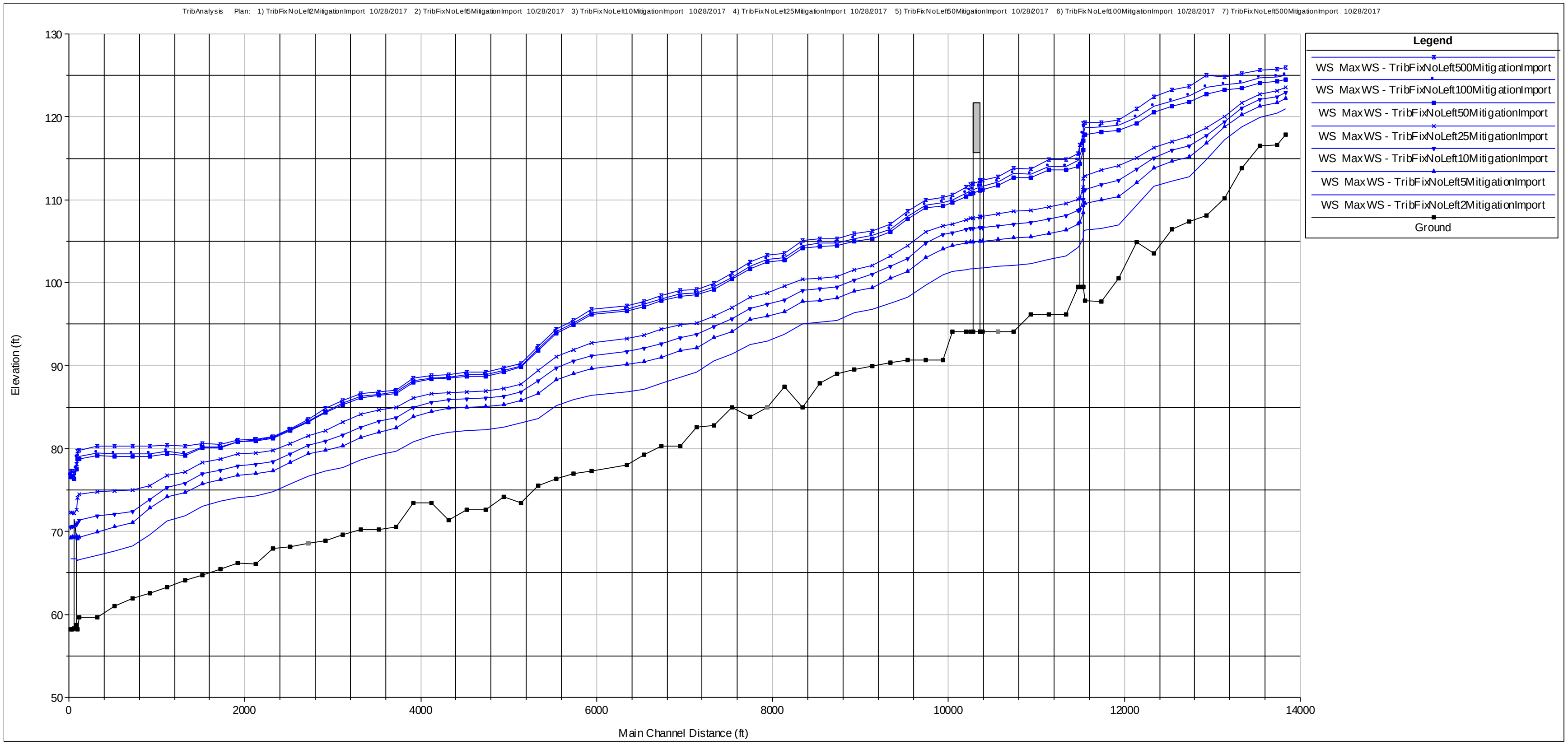


Plate 14. Corralitos Creek Recommended Plan Water Surface Profiles

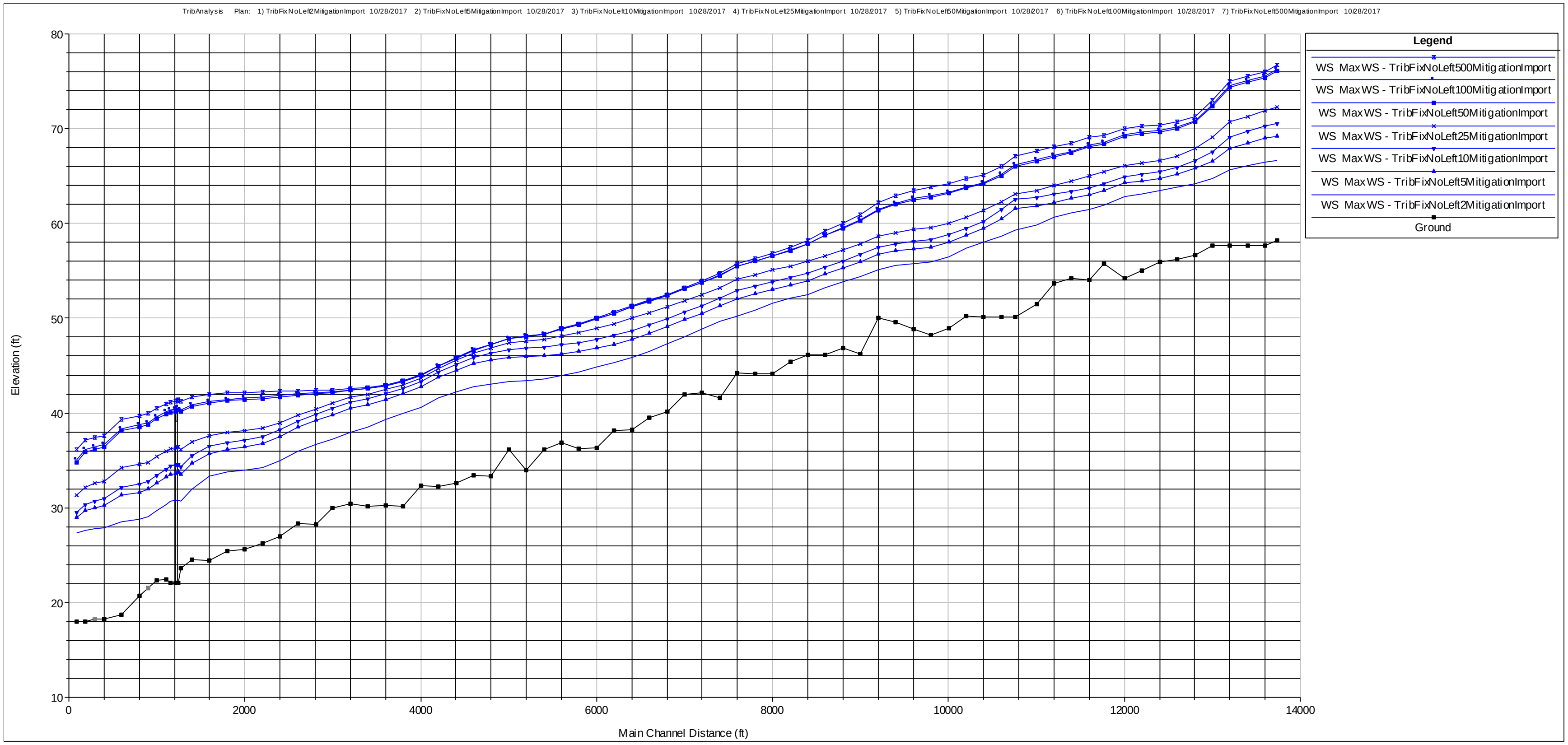


Plate 15. Salsipuedes Creek Recommended Plan Water Surface Profiles

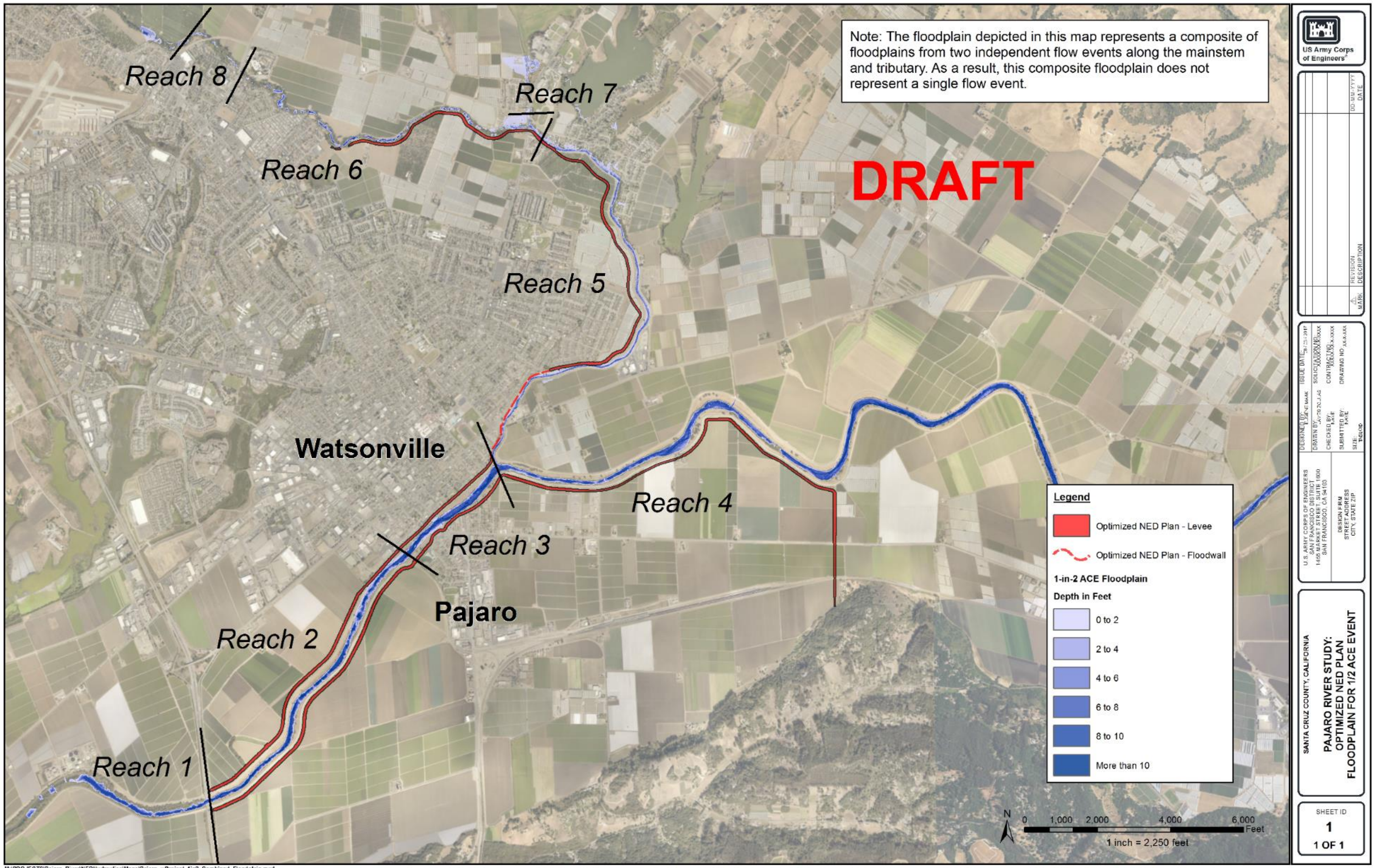


Plate 16. Recommended Plan 50% (1/2 ACE) Composite Floodplains

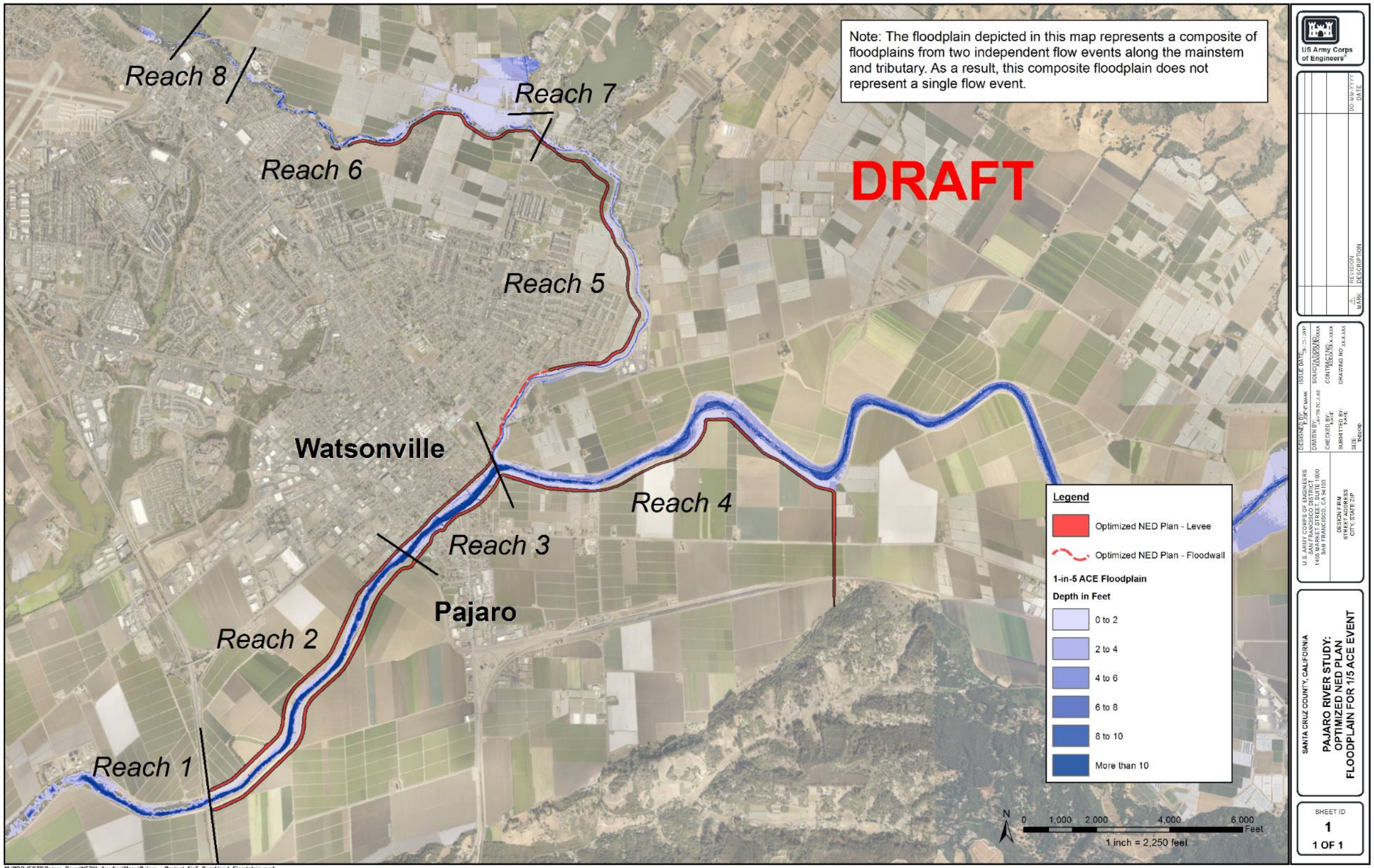


Plate 17. Recommended Plan 20% (1/5 ACE) Composite Floodplains

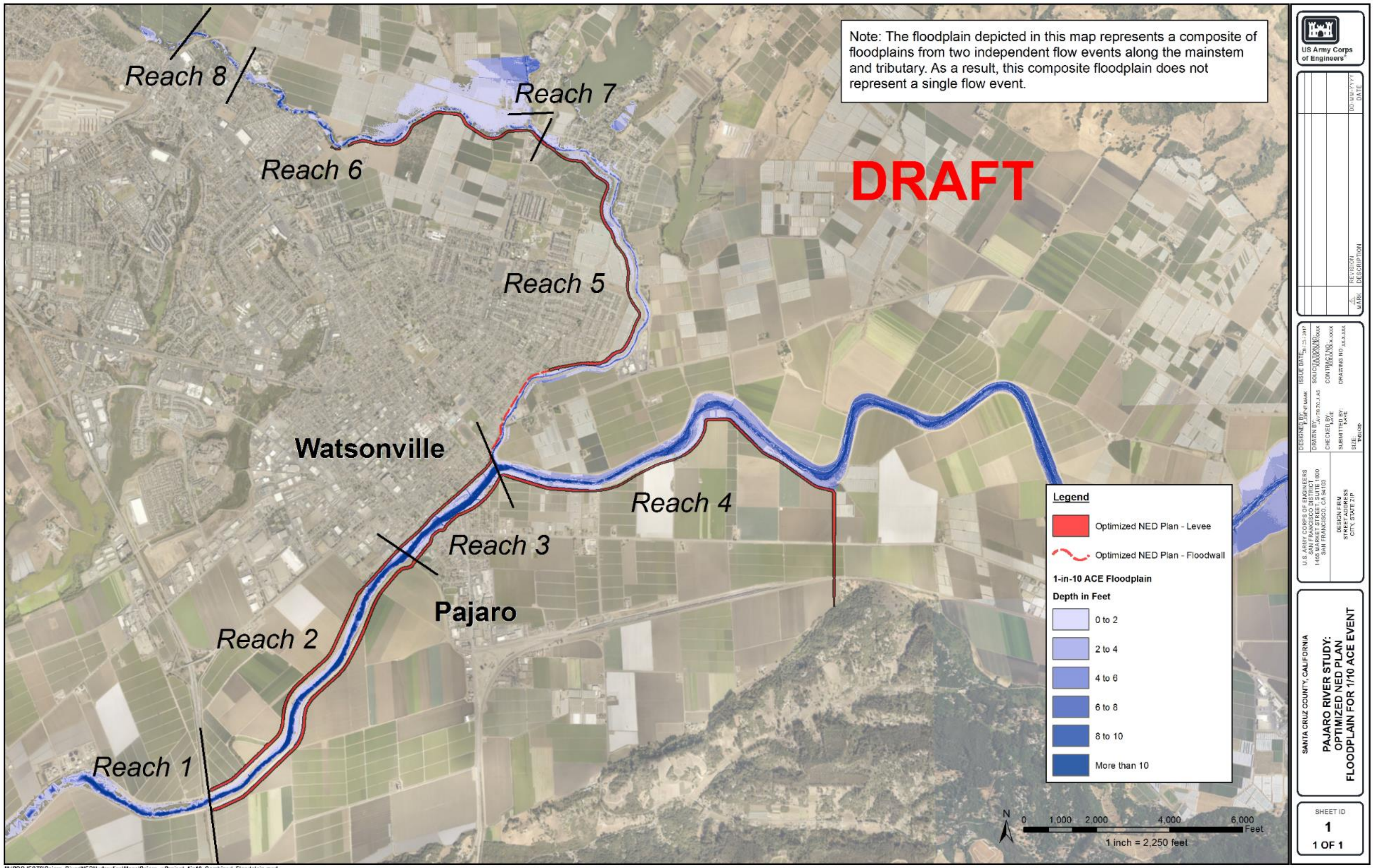


Plate 18. Recommended Plan 10% (1/10 ACE) Composite Floodplains

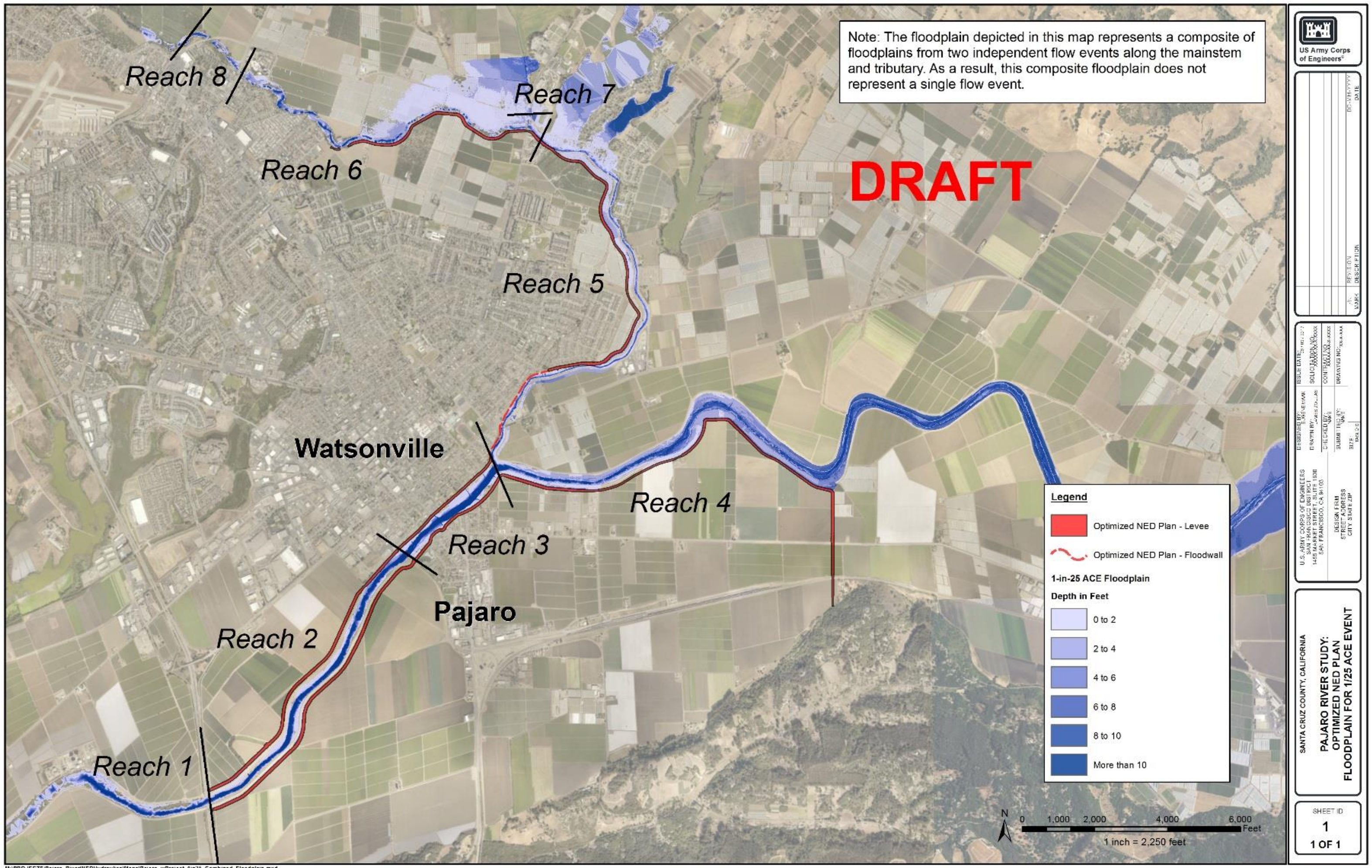


Plate 19. **Recommended Plan 4% (1/25 ACE) Composite Floodplains (Overtopping Only)**

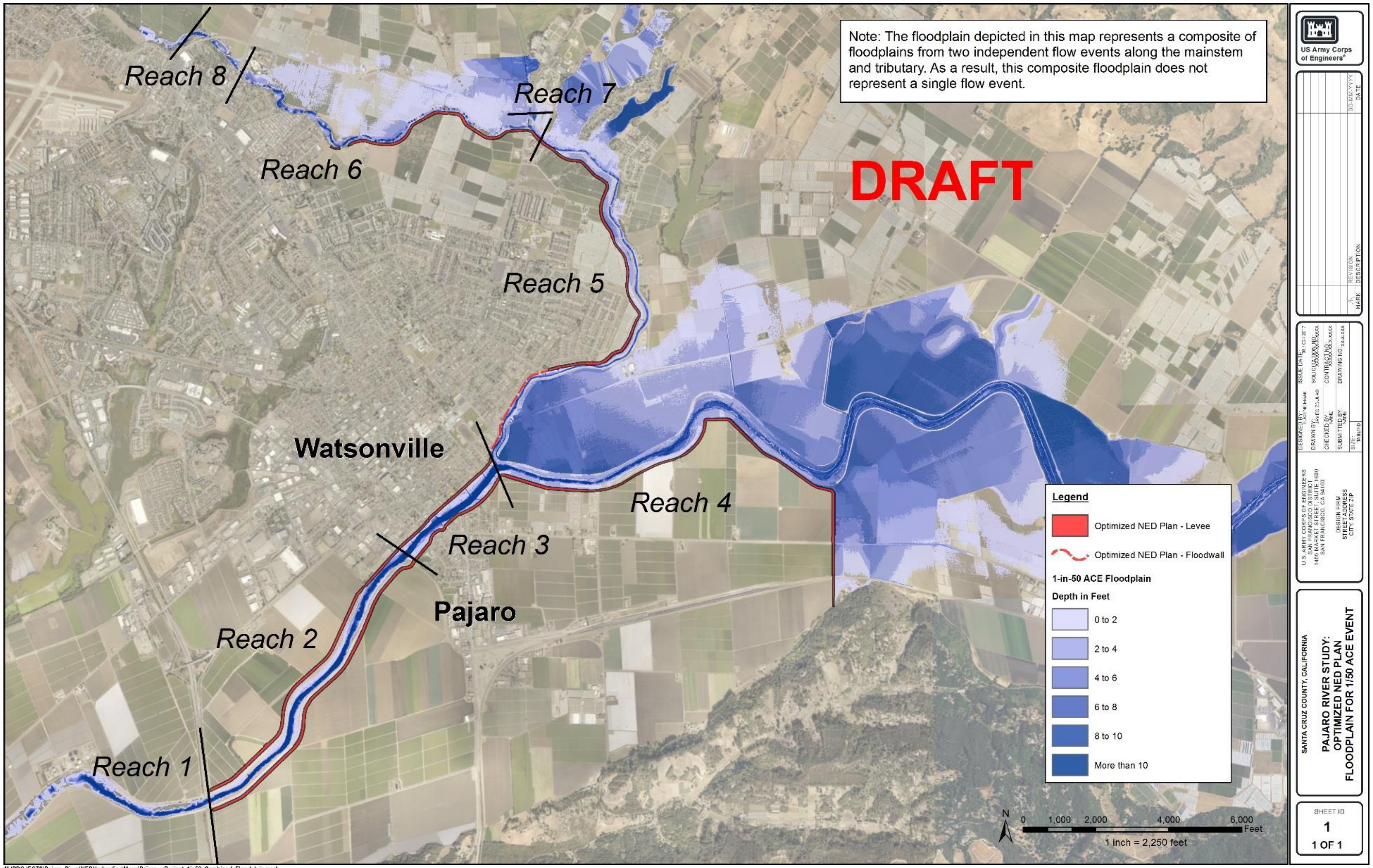


Plate 20. Recommended Plan 2% (1/50 ACE) Composite Floodplains (Overtopping Only)

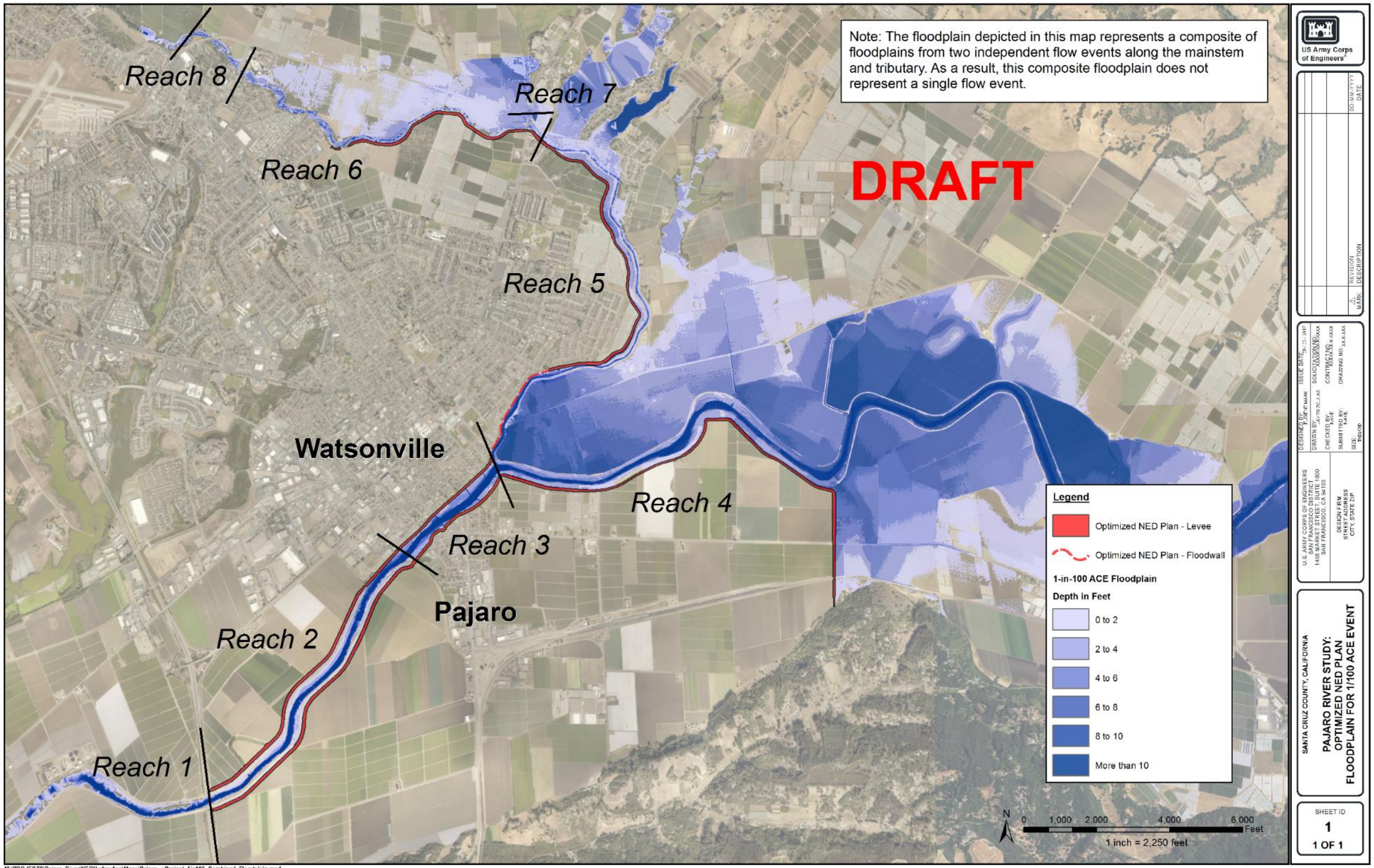


Plate 21. Recommended Plan 1% (1/100 ACE) Composite Floodplains (Overtopping Only)

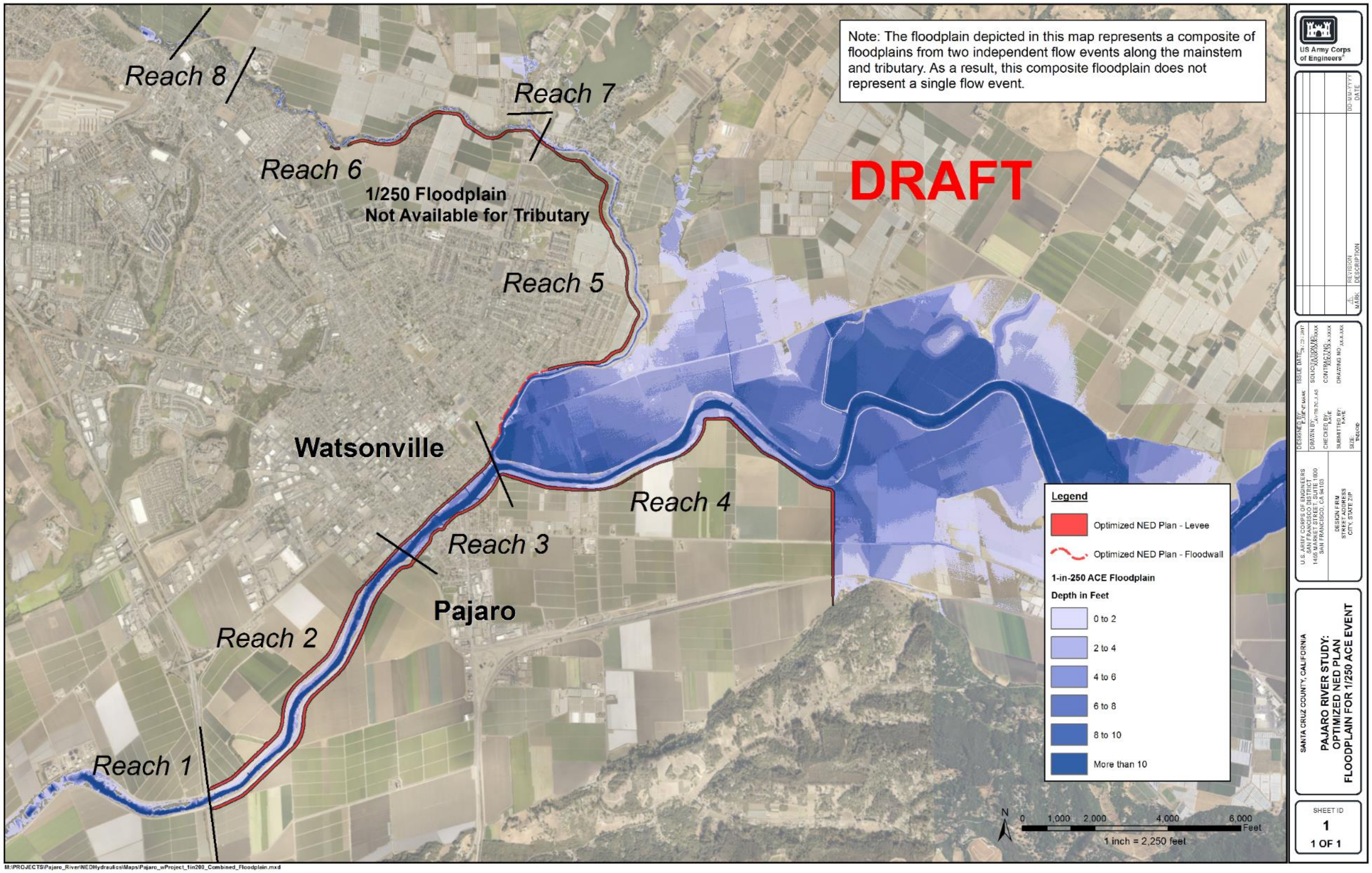


Plate 22. Recommended Plan 0.4% (1/250 ACE) Pajaro Floodplain (Overtopping Only)

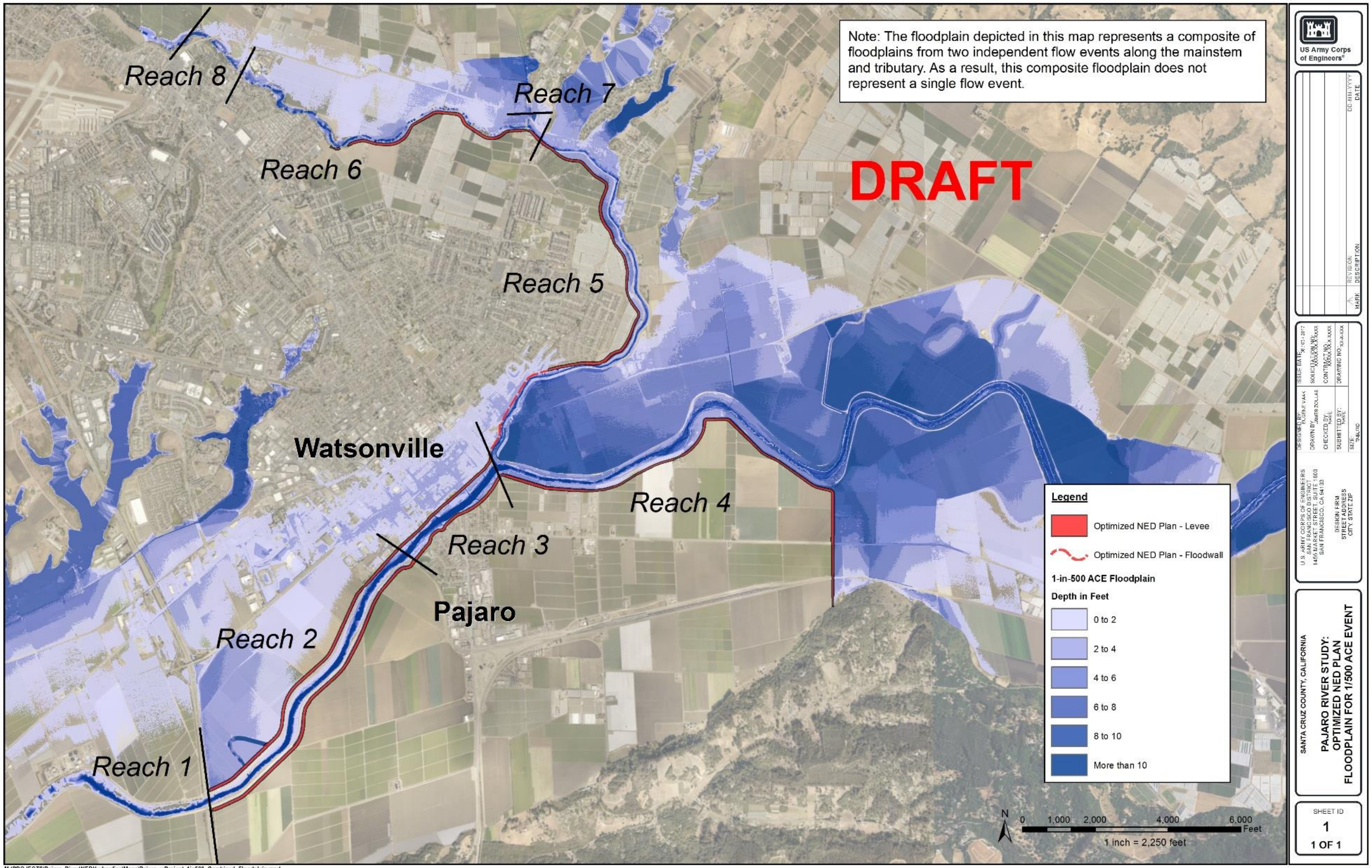


Plate 23. Recommended Plan 0.2% (1/500 ACE) Composite Floodplains (Overtopping Only)

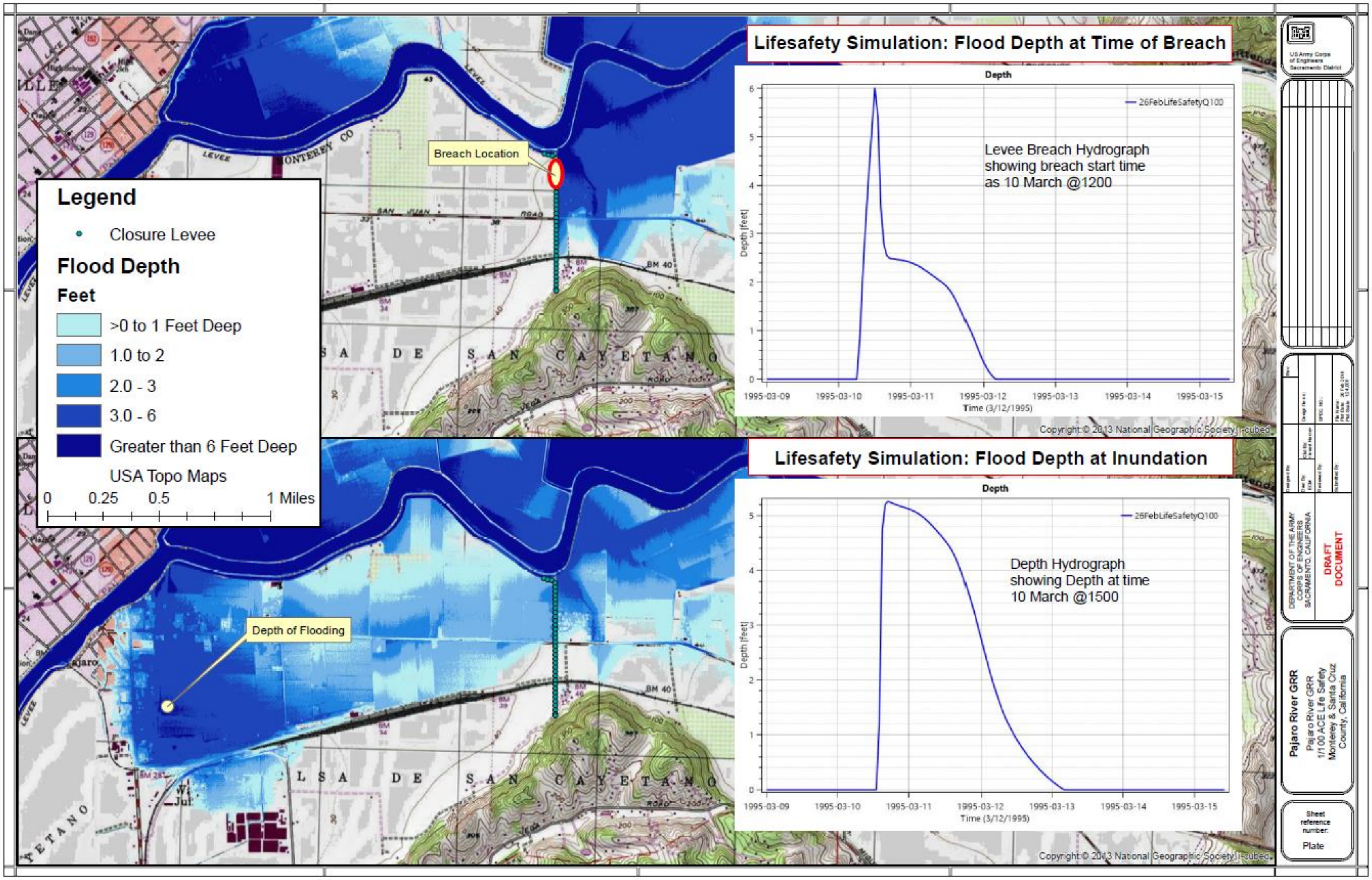


Plate 24. Life Safety Simulation

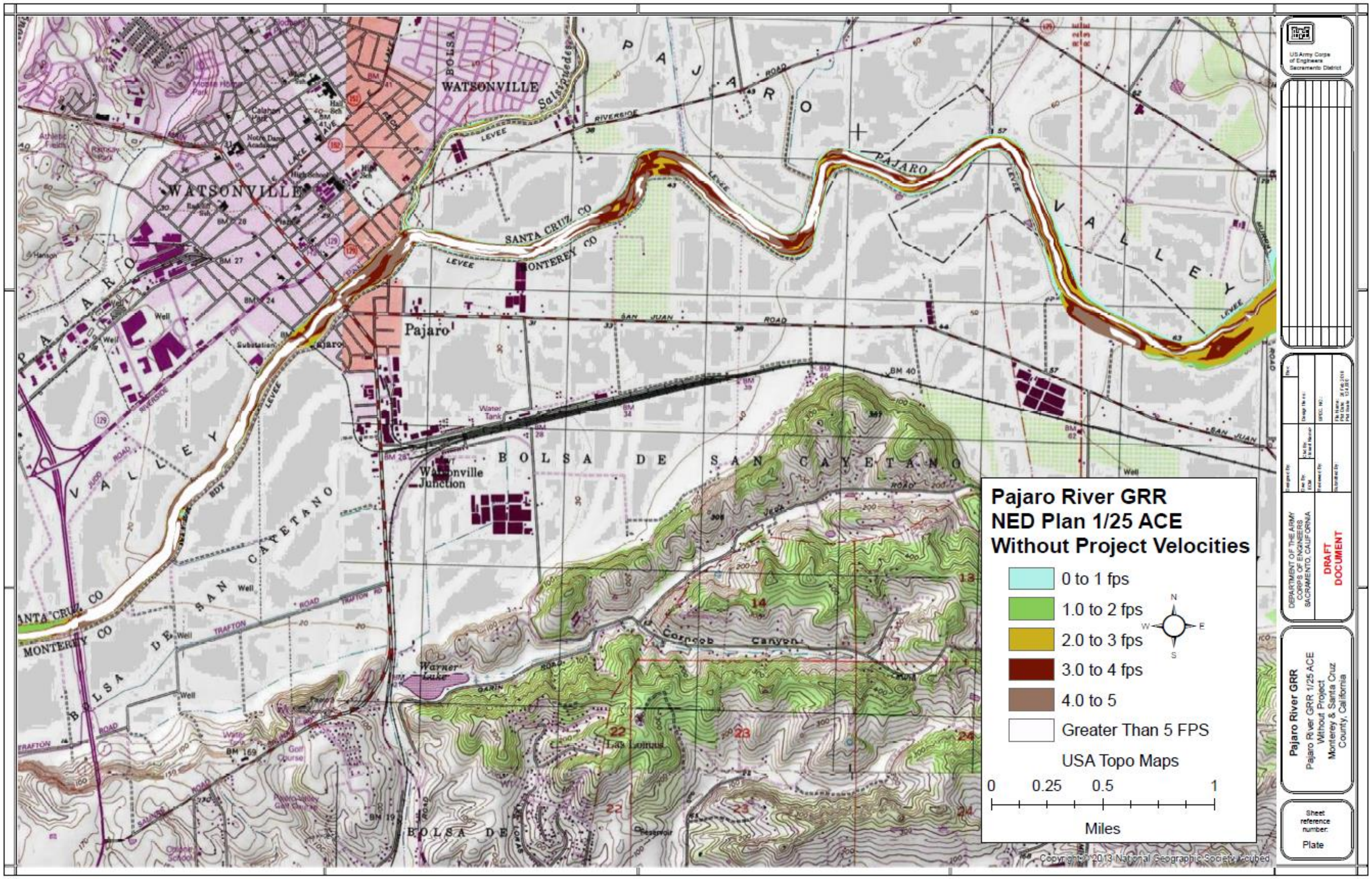


Plate 25. Without Project Bank Full Velocities

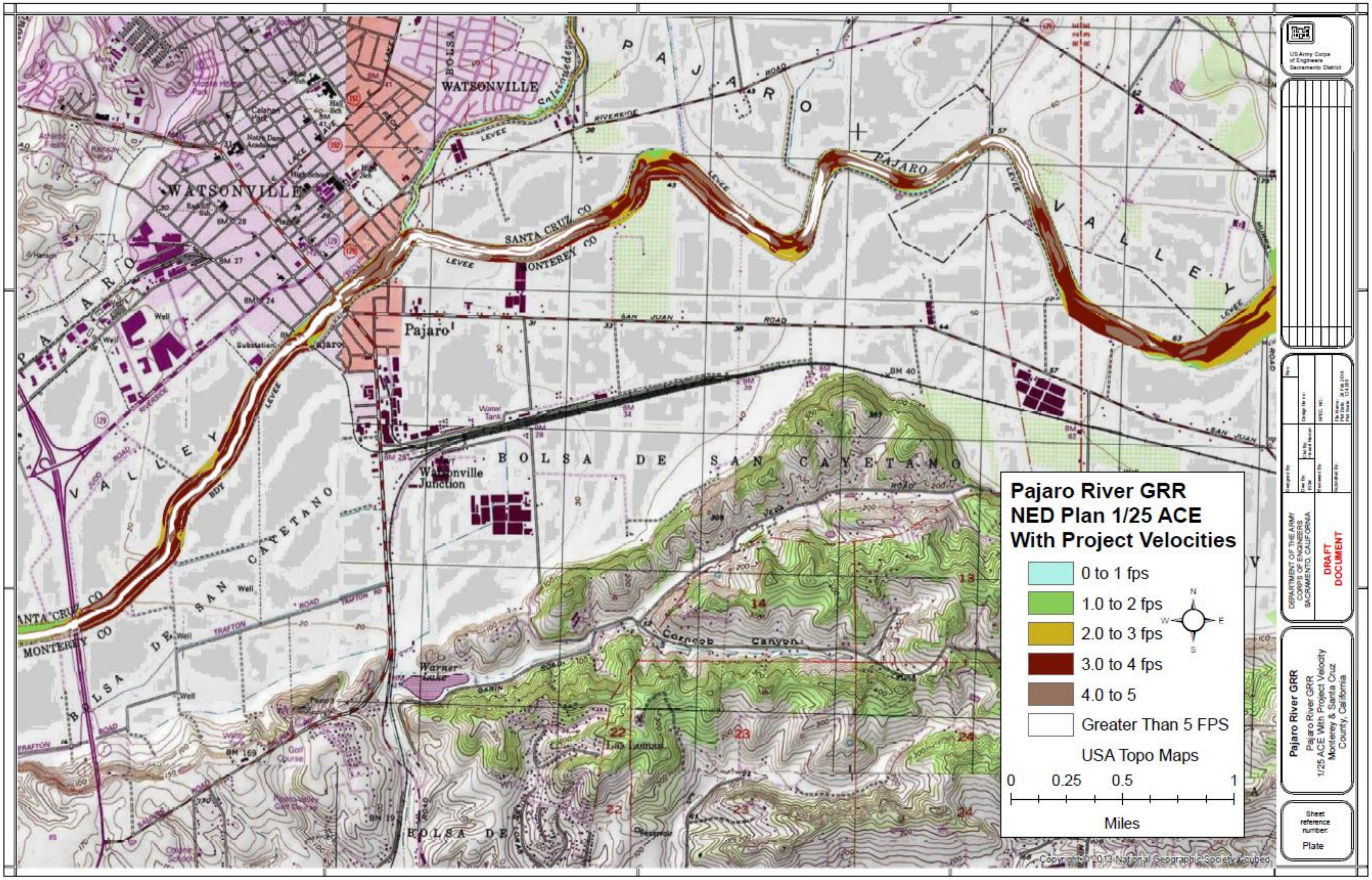


Plate 26. With Project Bank Full Velocities

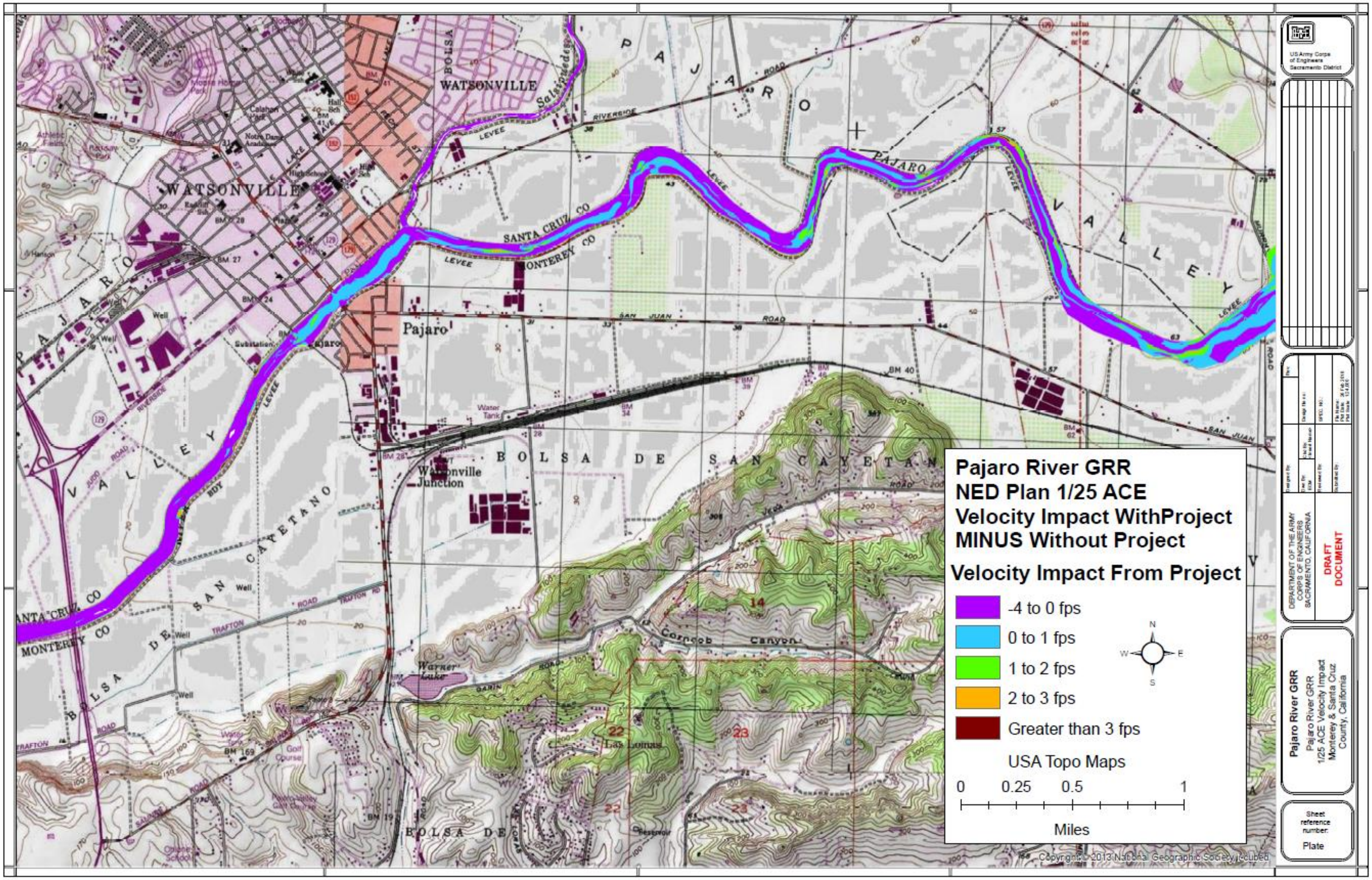


Plate 27. With Project Velocity Impacts

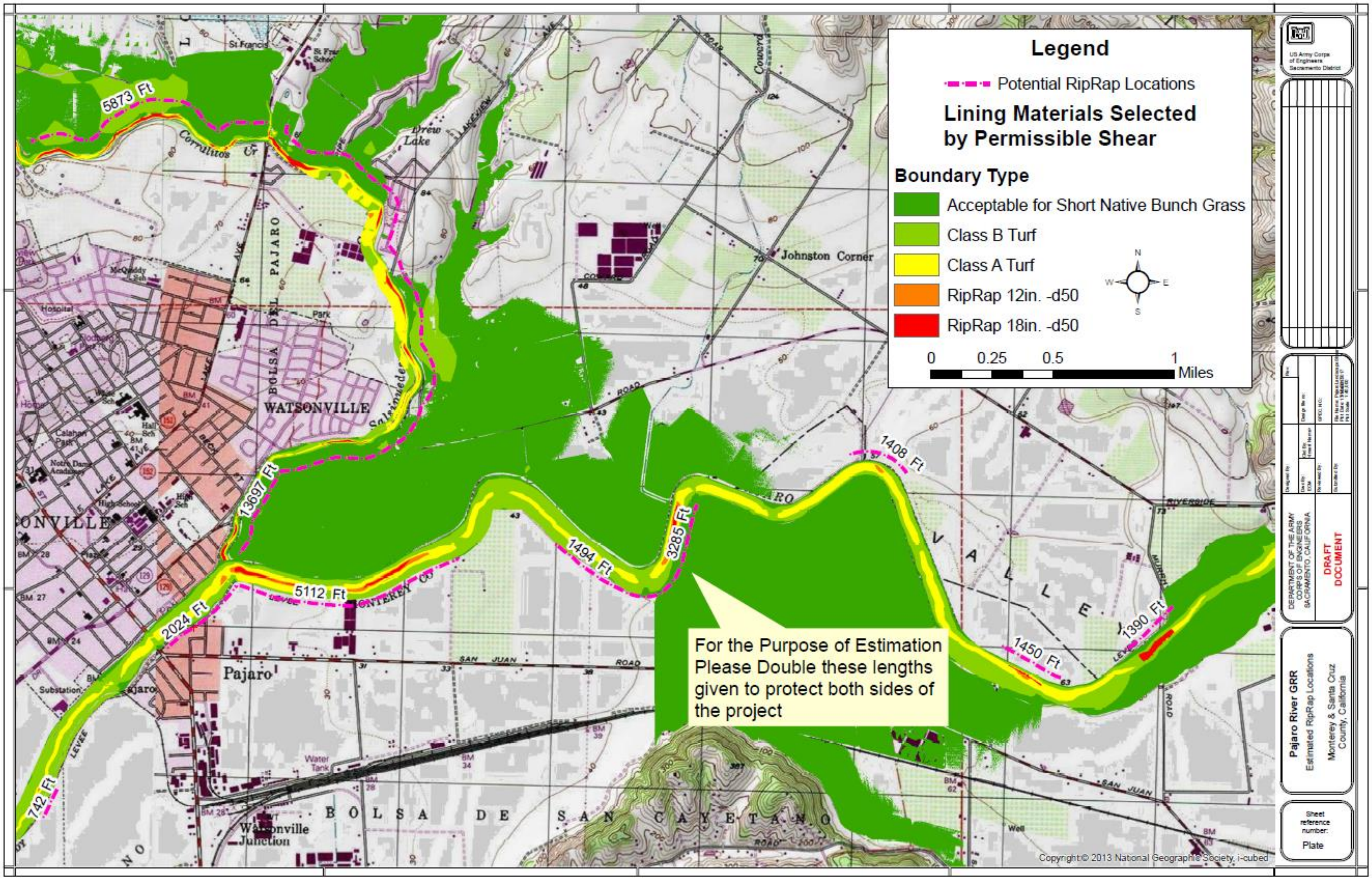


Plate 28. Estimated Channel Lining Materials for Stability

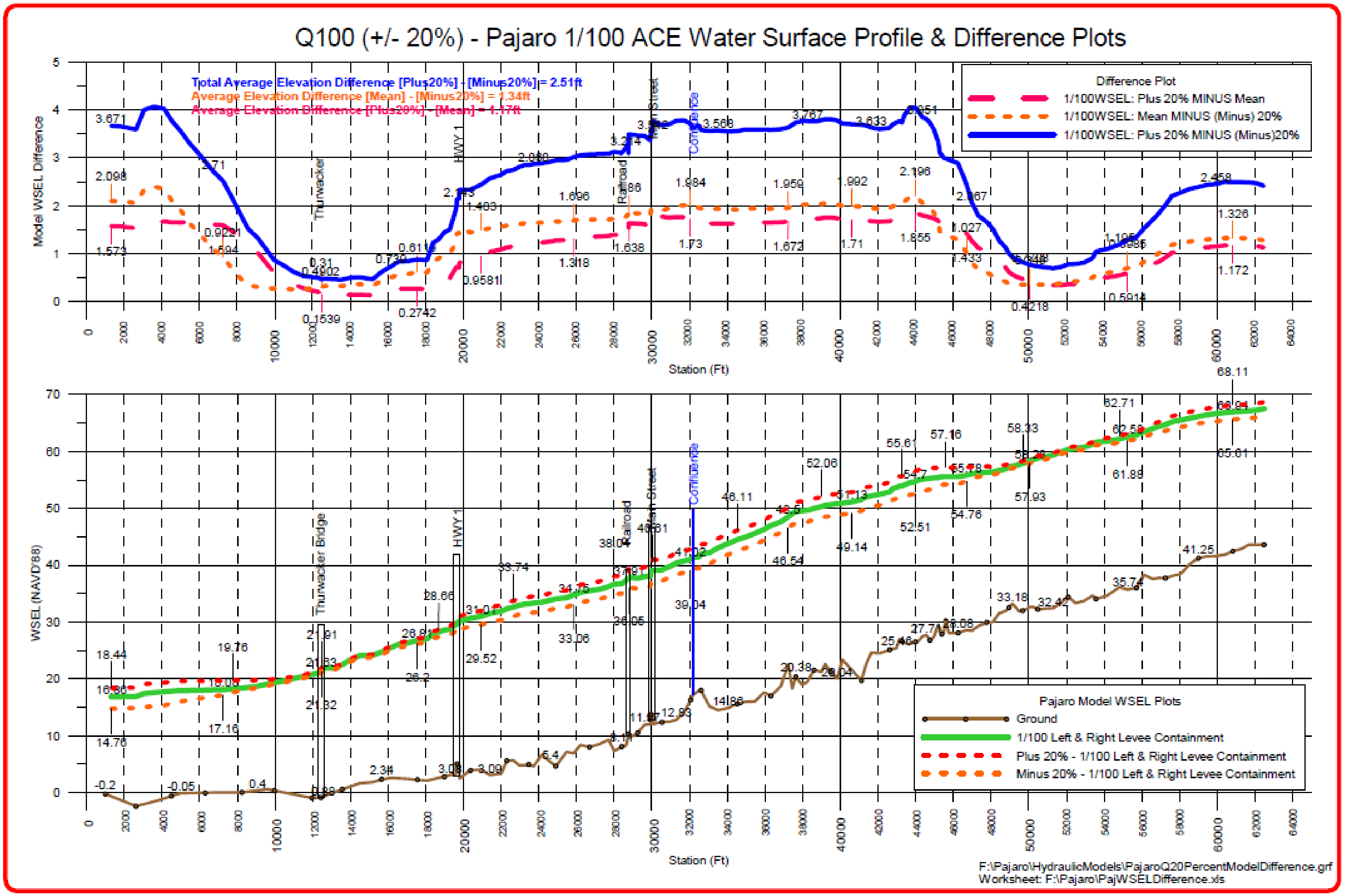
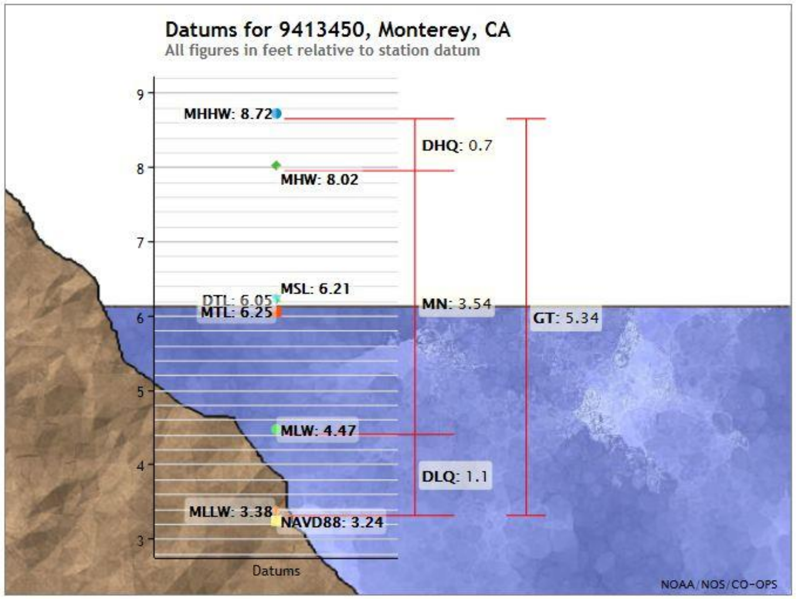


Plate 249. Sensitivity of stage to changes in flow.

Elevations on Station Datum

Station: 9413450, Monterey, CA
Status: Accepted (Oct 7 2011)
Units: Feet
T.M.: 120
Epoch: 1983-2001
Datum: STND

Datum	Value	Description
MHHW	8.72	Mean Higher-High Water
MHW	8.02	Mean High Water
MTL	6.25	Mean Tide Level
MSL	6.21	Mean Sea Level
DTL	6.05	Mean Diurnal Tide Level
MLW	4.47	Mean Low Water
MLLW	3.38	Mean Lower-Low Water
NAVD88	3.24	North American Vertical Datum of 1988
STND	0.00	Station Datum
GT	5.34	Great Diurnal Range
MN	3.54	Mean Range of Tide
DHQ	0.70	Mean Diurnal High Water Inequality
DLQ	1.10	Mean Diurnal Low Water Inequality
HWI	6.37	Greenwich High Water Interval (in hours)
LWI	12.42	Greenwich Low Water Interval (in hours)
Maximum	11.26	Highest Observed Water Level
Max Date & Time	01/27/1983 16:12	Highest Observed Water Level Date and Time
Minimum	0.99	Lowest Observed Water Level
Min Date & Time	01/11/2009 00:18	Lowest Observed Water Level Date and Time
HAT	10.40	Highest Astronomical Tide
HAT Date & Time	12/31/1986 17:30	HAT Date and Time
LAT	1.41	Lowest Astronomical Tide
LAT Date & Time	05/25/1990 13:12	LAT Date and Time



Showing datums for
9413450 Monterey, CA

Data Units ◉ Feet
○ Meters

Epoch ◉ Present (1983-2001)
○ Superseded (1960-1978)

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Plate 30. Elevations on Station Datum