Ventura County
Watershed Protection District

Santa Clara River Levee (SCR-1)
NLD FC System ID No. 3805010085

Ventura County, California

Interim Risk Reduction Measures Plan

February 2015

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Prepared for:

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Prepared by:

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1.0 INTRODUCTION

The development of this Interim Risk Reduction Measures Plan (IRRMP) is in response to identified levee safety issues associated with the Santa Clara River Levee (SCR-1) upstream of Interstate Highway 101. The IRRMP describes the subject levee system, considers potential consequences associated with levee failure scenarios, and identifies, evaluates and recommends potential interim actions to address risks. The IRRMP also specifies an implementation schedule and costs associated with the recommended plan. The IRRMP is prepared in accordance with U.S. Army Corps of Engineers (Corps) guidelines, Engineering and Construction Bulletin 2012-1 (USACE 2012). Interim risk reduction measures (IRRMs) are actions implemented to reduce inundation risks posed by a levee system for which levee safety issues have been identified while more long-term and comprehensive inundation risk reduction and management solutions are being pursued (USACE 2012). The likelihood and consequences of inundation in a leveed area define the "inundation risk." Consequences are tied to the potential loss of life and economic and environmental losses should a levee fail. In establishing IRRMs, the prevention of loss of life is paramount, followed by prevention of high economic or environmental losses. The likelihood of failure along SCR-1 was determined by analyzing four failure modes specified in Corps guidelines (USACE 2012) that could cause inundation in a given levee system. Based on the analysis of failure modes, this IRRMP identifies the six failure scenarios most likely to occur along the SCR-1 levee system and the risks associated with them.

2.0 SYSTEM DESCRIPTION AND PURPOSE

2.3 Location

SCR-1 is located in the city of Oxnard, in Ventura County, California. It is 4.72 miles long and is located along the southeast bank of the Santa Clara River between Interstate Highway 101 and Saticoy (Figures 1 and 2). In the National Levee Database (NLD), it is listed as Santa Clara River 1, No. 3805010085. Figure 3 shows the effective Federal Emergency Management Agency (FEMA) floodplain associated with SCR-1, dated January 2010.

2.4 Main Features

SCR-1 was originally designed and constructed by the Corps to control the designated standard project flood discharge of 225,000 cubic feet per second (cfs) from the 1,600-square-mile Santa Clara River Watershed (USACE 1958a). The final design is documented in two Design Memorandums (DM): DM #1 for Hydrology (USACE 1958a) and a General Design Memorandum, GDM #2 (USACE 1958b) as well as final construction plans (USACE 1961). The project was constructed in 1961. The levee height varies from approximately 4 to 13 feet. The compacted fill embankment has a top width of 18 feet, and the levee embankment slopes are 2 feet horizontal distance to 1 foot vertical distance (2H:1V) on both the landside and the riverside. The riverside of the embankment has a 1.5- to 2-foot-thick rock revetment that has been concreted in the vicinity of highway bridges. The rock revetment extends from the top of the embankment to varying depths below the levee toe. The depth of rock revetment below the levee toe is referred to herein as the "toedown." The reasoning for the varying rock revetment depths (toedown) is described in the GDM (USACE 1958b). A board of consultants provided recommendations for the configuration of the rock revetment. The following is an excerpt from the GDM:

> The board of consultants recommend that (a) instead of a levee with a deep toe-down (the toe-down would extend 12 feet below the streambed), where a 200-foot berm of undisturbed granular streambed material exists between the levee and the main-stream channel, the depth of the toe-down be extended only 5 feet below the top elevation of this undisturbed material or (b) in the absence of this undisturbed material and at locations subject to direct attack by streamflow, groins extending 150 feet into the stream and spaced 225 feet apart – with slight deflection in the downstream direction – be built.

Based on the 2005 survey of the channel and comparison with the original toedown elevations in the final construction plans (USACE 1961), the existing condition of the revetment toedown varies from approximately 0 to 10 feet below the river streambed between Highway 101 (Station 244+46) and a point approximately 9,000 linear feet upstream (Station 335+00). Upstream of this point (Station 335+00) to Highway 118 (approximate Station 440+00), the existing toedown varies from approximately 2 feet below the streambed to approximately 18 feet above the streambed. Between Highway 118 and the upstream end of the levee, the existing toedown varies from approximately 1 foot below the streambed to approximately 18 feet above the streambed. In this reach, the bottom of the rock groins and levee toe are also above the current Santa Clara River streambed (see Section 3.1 of this report for the spatial extent of existing rock groins). If the existing groins fail to restrain the flow within the main channel, the levee would potentially be undercut. The rock groins recommended in the GDM were constructed to divert flows away from the levee rock revetment. In addition, a weighted stone toe section along the levee toedown was designed to launch into the river to protect the rock revetment from being undermined.

The recent Corps periodic inspection report on the levee (USACE 2011) rated SCR-1 as “unacceptable”. This unacceptable rating resulted in the levee system being put on “inactive” status in the Corps PL 84-99 Program, thus making the levee system ineligible for federal funding for repairs if damaged during a flood event. The Ventura County Watershed Protection District (VCWPD) is seeking reinstatement of PL 84-99 eligibility by developing and executing a System-Wide Improvement Framework (SWIF) Plan to correct complex deficiencies, and this IRRMP would be part of that process.

2.5 Ownership and Purpose

SCR-1 is owned and operated by the VCWPD. The levee protects property to the south and east from flood inundation that would affect a variety of local land uses, including residential, agricultural, commercial, and industrial.

Figure 1 – Santa Clara River Levee (SCR-1) Vicinity Map

Figure 2 – Santa Clara River Levee (SCR-1) Location Map

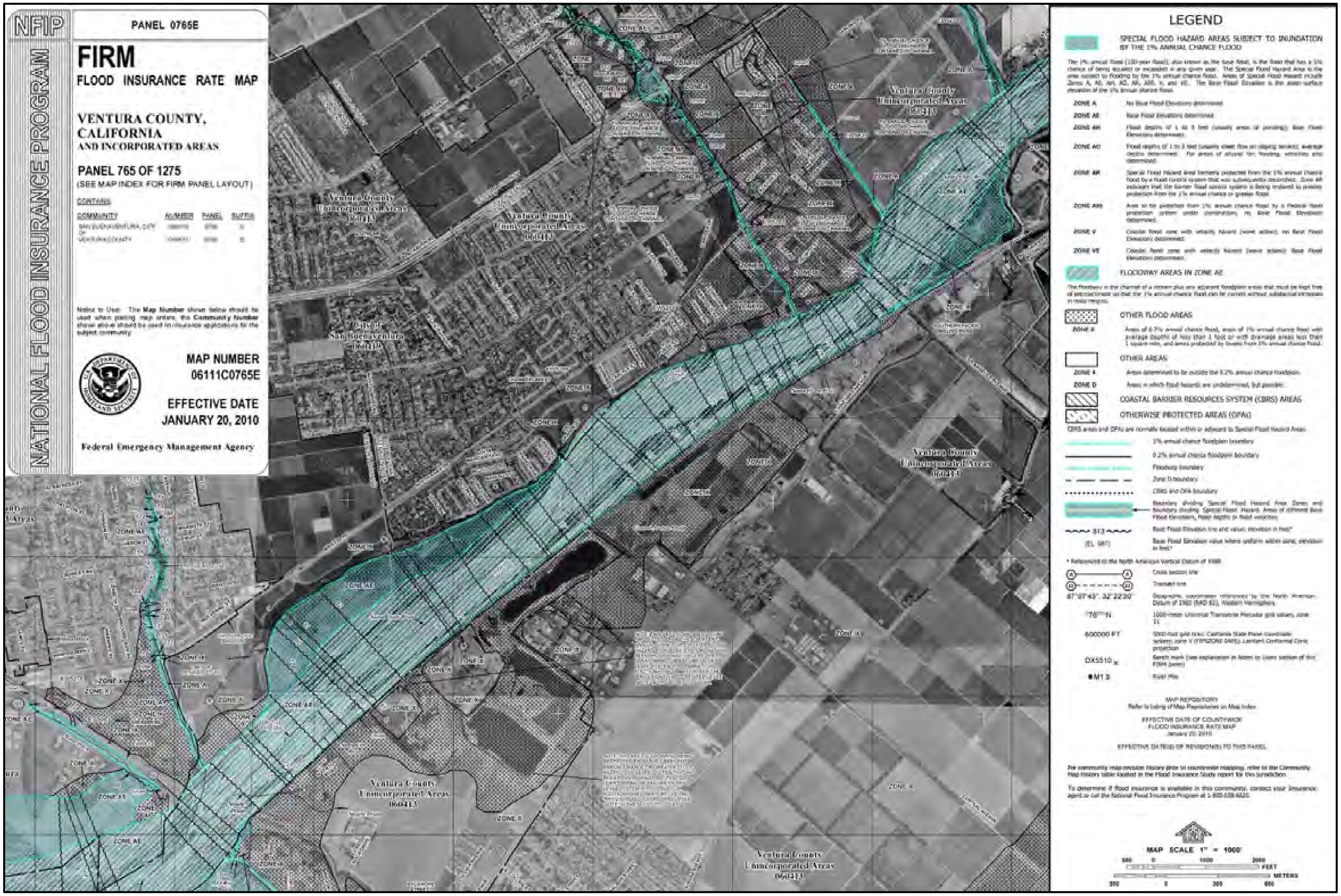


Figure 3 – FEMA Digital Flood Insurance Rate Map

2.6 IRRM Hydraulic Modeling

The hydraulic modeling used for the development of this IRRMP was originally derived from the Corps' Hydraulic Engineering Center River Analysis System (HEC-RAS) model of the Santa Clara River from July 2009. This model was modified to contain two scenarios. One scenario was a "with levee" geometry that included SCR-1 and the future improvements to the SCR-3 levee system located just downstream of Hwy 101. The other scenario was a "without levee" geometry that removed SCR-1 and left the original Corps model mostly unchanged downstream of SCR-1 (reflecting the absence of SCR-3). The geometries of both scenarios had identical additional extended cross sections added from the Provisionally Accredited Levee (PAL) report (VCWPD 2009) HEC-RAS model along the SCR-1 reach. These cross sections were based on the same topographic source as the original Corps model, a 2005 light detection and ranging (LiDAR) generated surface.

Due to extensive ineffective flow areas present in the geometry of the "without levee" scenario, the water surface elevations changed minimally from those in the "with levee" scenario. However, the "with levee" scenario reflects the reality that should a failure mode occur during a flood event, the water surfaces will reflect the levee functioning in place until it has a specific failure. Therefore, for the purposes of this IRRM plan, the "with levee" scenario was used for determining water surface elevations and flooding extents.

For floodplain mapping in Section 4, water surface elevations (WSE) from the HEC-RAS model were used to determine the flood footprint for a given breach scenario, beginning at the upstream end of a breach. Flooding upstream of the breach landside of the levee was mapped using the WSE at the breach. Water depth, both average and maximum, were derived by statistical analysis of a flood depth surface for a given flood footprint. As a conservative assumption, the gravel mines on the landside of the levee (approximate Station 298+00 to 320+00 and Station 326+00 to 350+00) were not included in this water depth analysis, which could limit or store flood waters that would occur in an area of a breach. For levee system malfunction scenarios involving a penetration, flooding was determined by the volume of water capable of exiting the penetration with the head pressure of the riverside WSE. The calculated water volume was mapped over the affected area on the landside of the levee with a depth of 1 foot, due to dispersion and sheet flow. Landside ground elevations higher than the actual WSE on the riverside of the penetration and the local gravel pits were not included in the flooded areas.

3.0 POTENTIAL FAILURE MODES

An overview of the potential failure modes for SCR-1 is provided in the following subsections. Historical references were documented and compared with the historical migration of the Santa Clara River to identify where the failures are likely to occur.

3.1 Breach Prior to Overtopping

A breach prior to overtopping is a scenario in which the levee embankment fails with no overtopping of the levee crest. This is a critical scenario to consider in that it is an emergency situation that may not be anticipated in time to warn the public. A review of the past performance of the SCR-1 system highlights the potential for a breach prior to overtopping.

The floods of January and February 1969 were the most damaging floods of record along the Santa Clara River in Ventura County. The estimated peak discharge of the 1969 flood is 165,000 cfs, and the flood frequency was estimated to be more than 50 years. The following is an excerpt from a Corps report pertaining to the reach from Highway 118 to Highway 101 (USACE 1969):

> The only significant damage that occurred in this reach during the January flood was damage to the revetment of an existing levee constructed by the Corps of Engineers. February flood flows washed out about 500 feet of State Route 118 Bridge, damaged agricultural property and utilities, and severely damaged flood-control improvements constructed by the Corps of Engineers. The flood eroded the south bank near the existing Corps levee, damaging some groins; then deflected, ricocheted from the State Route 118 Bridge, and returned to the south bank – where the flood flows cut in close to the Corps levee, bounced off to the north bank, and carved a long arch. The flood flows then deflected to the south bank where they undercut the toe protection on the Corps levee, causing the failure of about 2,000 feet of levee and eroding the ground behind the levee for a distance of about 100 feet.

The original construction, completed in 1961, contained 41 groins from Station 330+00 to Station 392+00 and from Station 443+00 to Station 470+00. After the 1969 flood damage, the Corps performed the following repairs, which were completed in 1971:

- **Station 311+00 to Station 332+00,** Restored 2,100 linear feet of levee embankment with deeper rock revetment where the levee embankment had failed.
- **Station 330+00 to Station 344+50,** Repaired 7 of the original 41 groins.
- **Station 246+00 to Station 330+00 and Station 421+80 to Station 436+80,** Added 35 new groins.

A total of 75 groins are now in place along the SCR-1 reach from Station 246+00 to Station 470+00. Erosion along the levee toe was reported during the 1969 flood from approximately Station 410+00 to Station 430+00 and additional groin damage was reported somewhere in the vicinity of Station 452+00 to Station 470+00. There was no further detail on the extent of this additional damage.

Floods in 1983, with a peak estimated discharge of 100,000 cfs and an associated frequency of approximately 25 years, caused additional damage to the riverside tips of five groins between Station 321+00 and Station 333+00 in the vicinity of the 1969 levee failure (Station 311+00 to Station 332+00). The damage to the groins was likely resulted from the low-flow channel impinging and washing out the top portion of the groin tips. The County of Ventura repaired and restored these groins, which included one of the original 1961 groins and four of those added in

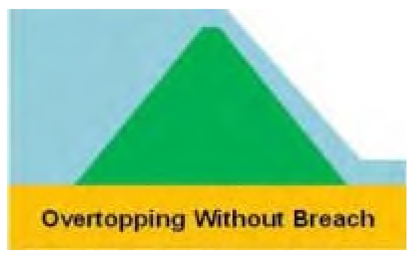
1971. The repair included the removal of approximately 2 feet of existing rock and placement of 2-ton rock riprap back to the original design dimensions and backfilling with uncompacted fill. This is the only known non-Corps stone that was added to the system.

This history of required flood repairs indicates that the levee is vulnerable to failure prior to overtopping in three areas:

- **Station 311+00 to Station 348+00.** Six groins, on a southerly bend upstream and on the north bank of Santa Clara River, directs flow toward this levee location on a path that currently mirrors that of the 1969 flood. This was the levee segment that failed in the 1969 flood.
- **Station 410+00 to Station 430+00.** Aerial photographs show the river being directed toward the levee by the alignment of the Los Angeles Avenue Bridge (SR 118), where river encroachment was reported in 1969 (USACE 1969).
- **Station 452+00 to Station 486+00.** The 1969 Corps report indicated erosion near the levee with some unspecified damage to groins, which appeared to be due to another bend, upstream and on the north bank, as the river entered the leveed reach, directing flow toward the levee upstream of the Los Angeles Avenue Bridge (SR 118).

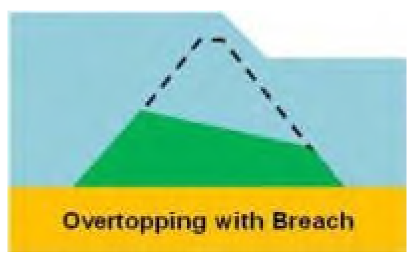
The river currently exhibits the same geomorphic and flow pattern at all three locations (outside river bends), making them susceptible to future damages.

3.2 Overtopping without Breach



Overtopping without breach is a scenario in which the levee embankment stays in place and does not fail when the crest is overtopped. The landside embankment of SCR-1 consists of a sandy soil with no erosion protection that would be highly susceptible to erosion due to overtopping flows. This would lead to subsequent levee failure. Therefore it is assumed that this failure mode would not occur; rather an overtopping with breach failure would occur.

3.3 Overtopping with Breach



Overtopping with breach is a scenario in which the levee embankment fails once the crest is overtopped. It is expected that any overtopping flows will cause a breach in the levee embankment of SCR-1. The hydraulic models of SCR-1 indicate that overtopping will most likely occur near the downstream reach of the levee. From Station 244+00 to Station 304+00, the levee would be overtopped by approximately the 200-year event. Because this reach would overtop by the higher frequency flood events rather than other reaches, it is considered the critical reach for overtopping. Upstream of this reach, from Station 304+00 to Station 354+00 and from Station 434+00 to Station 463+00, overtopping would occur at approximately the 500-year event. Beyond these stations, overtopping would occur at less frequent events: those larger than the 500-year event.

3.4 Malfunction of Levee System Components

A review of the levee system components identified several penetrations that require action by officials to maintain the integrity of the levee during a flood event. Table 1 details these penetrations. Potential malfunctions are varied, ranging from a flap gate that is immobile due to rust, jammed open due to debris, or chained open or a failure to implement a manual closure during a flood event. Sufficient inspection and maintenance of the penetrations prior to and during large flood events would be necessary to avoid the malfunctions. It is also required for a levee operator to be familiar with the system, closure structures, necessary tools and/or materials to operate the closure structures before flood events. The individual penetration features and their potential threat are summarized in Table 1.

Table 1 – Summary of Levee Penetrations

River Station	Description	Assessment
480+00	Side Drain No. 1, 42-inch-diameter RCP and flap gate (on landside)	Invert is located at the 500-year WSE. Minimal threat.
442+00	Side Drain No. 2, 48-inch-diameter RCP and flap gate (on landside) just upstream of Los Angeles Avenue	Invert is located at the 200-year WSE. Minimal threat.
422+25	Commercial drain from asphalt plant (not found in December 9, 2008, field inspection)	Located on top of the levee and buried by additional soil above the 500-year WSE. Minimal threat
410+60	Side Drain No. 3, 48-inch-diameter RCP and flap gate (on landside)	Invert is located above the 200-year WSE. Minimal threat.
385+77	12-inch-diameter metal pipe commercial drain from process plant	Invert located at the 100-year WSE and has no known closure structure. Small diameter opening. Minimal threat.
351+50	Central Avenue Drain, two 72-inch-diameter RCP with flap gates (on riverside)	Large-diameter openings located below the 50-year WSE. Potential threat.
316+60	Side Drain No. 4, 48-inch-diameter RCP and flap gate (on landside)	Large-diameter openings located below the 50-year WSE. Potential threat.
282+00	Side Drain No. 6, 48-inch-diameter RCP and flap gate (on landside)	Large-diameter openings located below the 50-year WSE. Potential threat.
246+20	Stroube Drain, Unit I, sluice gate structure and 10-foot-wide by 8-foot-high RCB	Requires manual operation. Potential threat.

RCB = reinforced-concrete box
RCP = reinforced-concrete pipe
WSE = water surface elevation

The most significant threats due to malfunction of the levee system components are the following:

- Two 72-inch-diameter reinforced-concrete pipes (RCPs) and flap gates at Station 351+50
- A 48-inch-diameter RCP and flap gate at Station 316+60
- A 48-inch-diameter RCP and flap gate at Station 282+00
- A sluice gate structure at Station 246+20

4.0 CONSEQUENCES OF FAILURES

Based on the identification of the potential failure modes and their likelihood of occurrence, six failure scenarios have been identified for SCR-1. For each of these scenarios, a potential area of inundation has been determined using the results of the IRRMP hydraulic modeling. Property damages and the population at risk associated with each of these inundation areas has been determined by the output from a HEC-Flood Damage Reduction Analysis (FDA) model and FEMA Hazus data.

The potential inundation area mapping methodologies for the different failure scenarios fell into two categories: those that mapped assumed breaches or overtopping failures and those that mapped assumed levee penetrations failures. For breach or overtopping failures (Failures 1, 2, 3, 4, and X), inundation extents were mapped by projecting the channel WSEs of the HEC-RAS hydraulic modeling across the floodplain behind SCR-1 along the breach and areas downstream of the breach down to Highway 101. For areas upstream of the breach, the WSE from the upstream end of the breach was extended upstream on the landside of the levee to determine the backwater ponding limit. For levee penetration failures (Failures 5a, 5b, 5c, and 6), inundation extents were determined by calculating the volume of water that would exit to the landside through the penetration, based on the size of the opening and a hydraulic head difference between riverside and landside of the levee. This exited water on the landside was assumed to not exceed a depth of 1 foot as it would spread and disperse around the penetration. That volume at 1 foot depth was mapped following the contours of the landside of the levee downstream from the penetration. The upstream limit of this mapped area was determined by extending the landside WSE (1 foot above existing ground at the penetration) upstream.

A flood damage model was developed in HEC-FDA, which uses a detailed structure inventory and hydraulic and hydrologic information to estimate flood damages. The output of the HEC-FDA model provides estimated flood damages for each of eight flood event frequencies (2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year), and this information can be sorted to determine damages by location. The locations that were used apply to the cross sections from which HEC-FDA obtains information about water surface profiles. For each of the failure scenarios, certain cross sections (or locations) were determined to be inundated; therefore, only the damages for those cross sections were summed. Additionally, expected annual damages were determined based on the simplified Expected Annual Damage Calculation for each failure scenario.

From the output of HEC-FDA the number of structures that would be inundated during each frequency event can be derived. The total number of inundated structures was sorted by location, in the same manner as the total damages, to determine the total count for each failure scenario. The number of structures was then divided into residential and non-residential structures to calculate the affected population with the use of FEMA Hazus, the Hazards USA geographic information database and modeling system. The HAZUS databases contain populations for both day and night in Ventura County, which were then divided by the total number of households in the county to get an average population per residence (day and night). The average HAZUS values per residence, which are estimated to be 0.93 during the day and 2.98 during the night, were then multiplied by the number of inundated structures to calculate the total affected population.

4.1 Failure 1 – Breach from Station 311+00 to Station 348+00

This failure is expected to be a "breach prior to overtopping" failure. In general, the landside toe along this reach is at a lower elevation than the riverside toe (see Attachment 1). A failure of the levee embankment would lead to inundation of the adjacent floodplain. The riverside toe becomes hydraulically loaded at the 50- to 100-year storm events. However, the damage history indicates that river impingement for the 25-year event leads to groin damage and levee breaches. In this reach, a levee breach would cause inundation of the residential area between two local detention basins from approximately Station 300+00 to Station 325+00. This flood extent and larger flood event extents are shown in Figure 4. Table 2 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages.

Table 2 – Failure 1 Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
25-year	0.9	3.1	233	$543,000
50-year	2.0	14.2	308	$14,666,000
100-year	2.8	17.8	505	$56,043,000
200-year	3.9	21.4	691	$127,272,000
500-year	9.6	43.0	1,072	$307,674,000
Expected Annual Damage				$1,633,000

The damages associated with each storm event are based on the number and type of affected structures. Table 3 provides the details of the affected structures associated with the damages identified in the previous table. The counts of residential and non-residential structures are approximate.

Table 3 – Failure 1 Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
25-year	21	0	None	26	73
50-year	316	3	1 school	484	1,104
100-year	682	17	1 school	1,411	2,382
200-year	853	31	1 school	3,191	4,429
500-year	1,214	72	1 school	5,861	6,067

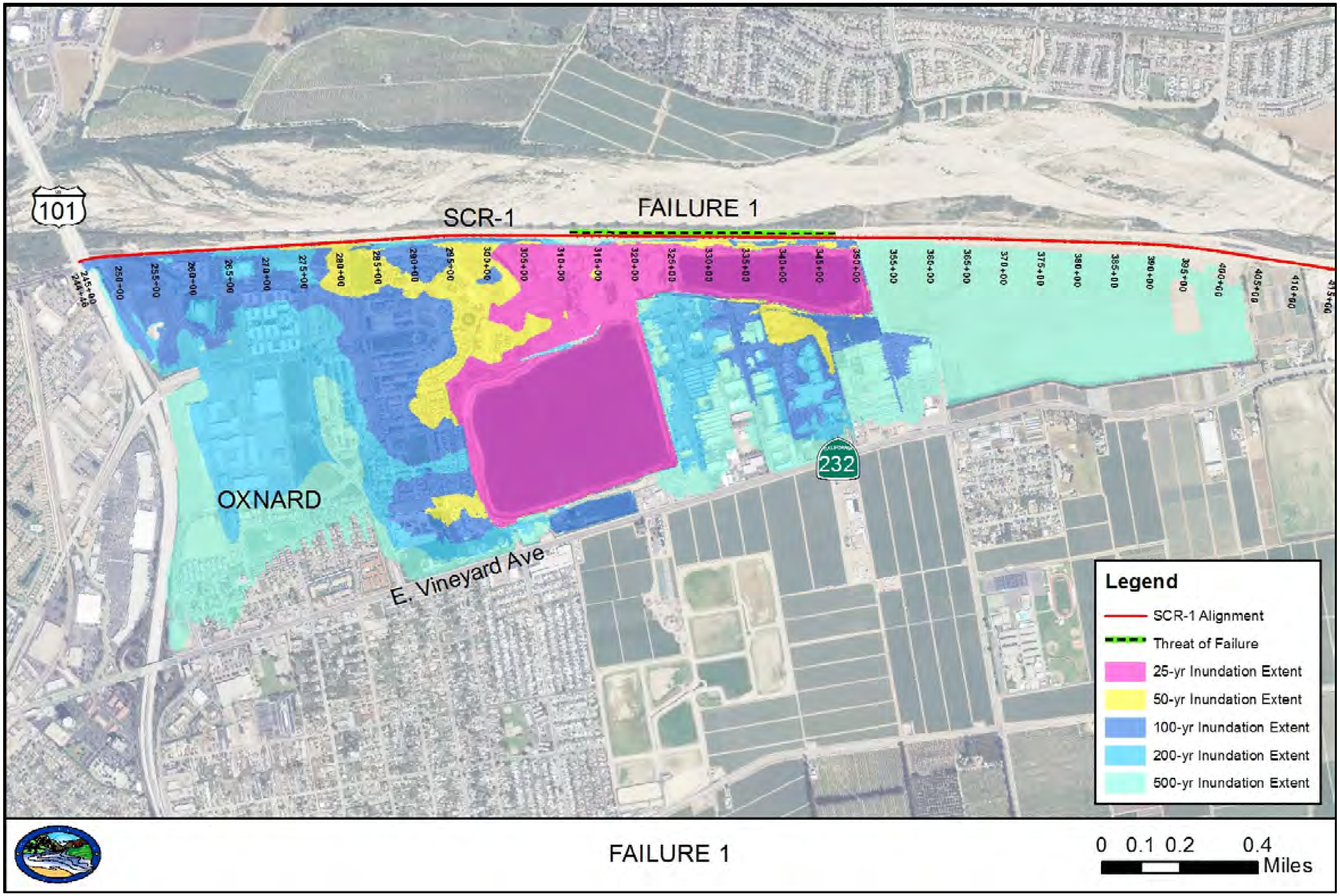


Figure 4 – Failure 1 Inundation Areas

4.2 Failure 2 – Breach from Station 410+00 to Station 430+00

This failure is expected to be a "breach prior to overtopping" failure. In general, the landside toe along this reach is at a higher elevation than the riverside toe (see Attachment 1). A failure of the levee embankment would lead to inundation of the adjacent floodplain only once the landside toe elevation is exceeded. The riverside toe becomes hydraulically loaded at the 200-year storm event. However, the damage history indicates that river impingement for the 25-year event leads to groin damage and levee breaches. The relatively high ground on the landside will largely protect the area from flood inundation. There is a potential for significant bank erosion along the asphalt plant property, which accounts for most of the expected damages. There are no significant inundation areas for this failure (Figure 5). There are no known permanent residential or non-residential structures within the flooded area, therefore there are no HEC-FDA economic damages.

4.3 Failure 3 – Breach from Station 452+00 to Station 486+00

This failure is expected to be a "breach prior to overtopping" failure. In general, the landside toe along this reach is at a lower elevation than the riverside toe in the lower portion of the reach (Station 453+00 to Station 457+00) and upper portion of the reach (Station 469+00 to Station 477+00) (see Attachment 1). A failure of the levee embankment in the lower reach would lead to inundation of the adjacent floodplain. The riverside toe becomes hydraulically loaded at the 100- to 200-year storm event in the lower reach; in the upper reach, the loading occurs beyond the 500-year event. However, the damage history indicates that river impingement for the 25-year event leads to groin damage and levee breaches in the upper reach (Station 465+00 to Station 486+00). The expected inundation areas are shown in Figure 5. The area consists primarily of non-productive agricultural fields and related structures, which account for the minimal if any damages. There are no known permanent residential or non-residential structures within the flooded area, therefore there are no HEC-FDA economic damages.

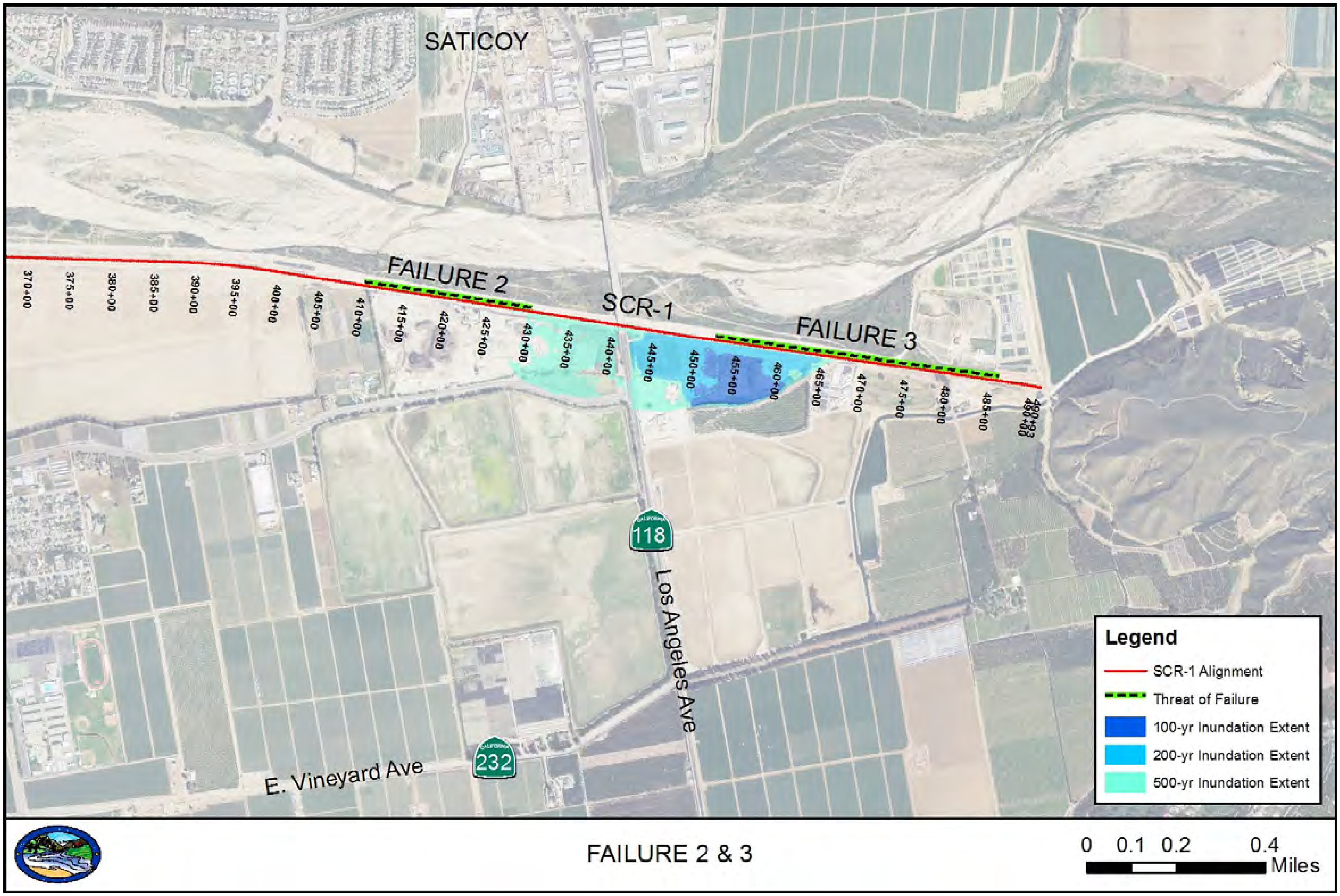


Figure 5 – Failures 2 and 3 Inundation Areas

4.4 Failure 4 – Overtopping from Station 244+00 to Station 304+00

This failure is expected to be an "overtopping with breach" failure. During such a scenario, the storm frequency expected to overtop the levee is a 200-year event, which occurs as the result of continuous storms in the region over days or weeks, with a culminating event resulting in a peak surge. In such circumstances, flood fighting agencies have warning that such a large event is coming. Despite their best efforts, the river may overtop and breach the levee, inundating the adjacent floodplain area north of Highway 101. The flood inundation extent is shown in Figure 6. In this reach, a levee overtopping would cause inundation of the residential and commercial areas from Station 245+00 to approximately Station 350+00. Table 4 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages.

Table 4 – Failure 4 Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
200-year	3.9	21.4	691	$127,272,000
500-year	9.6	43.0	1,072	$307,674,000
Expected Annual Damage				$971,000

Table 5 provides the details of the affected structures that are associated with the damages identified in the previous table. The counts of residential and non-residential structures are approximate.

Table 5 – Failure 4 Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
200-year	853	31	1 school	3,191	4,429
500-year	1,214	72	1 school	5,861	6,067

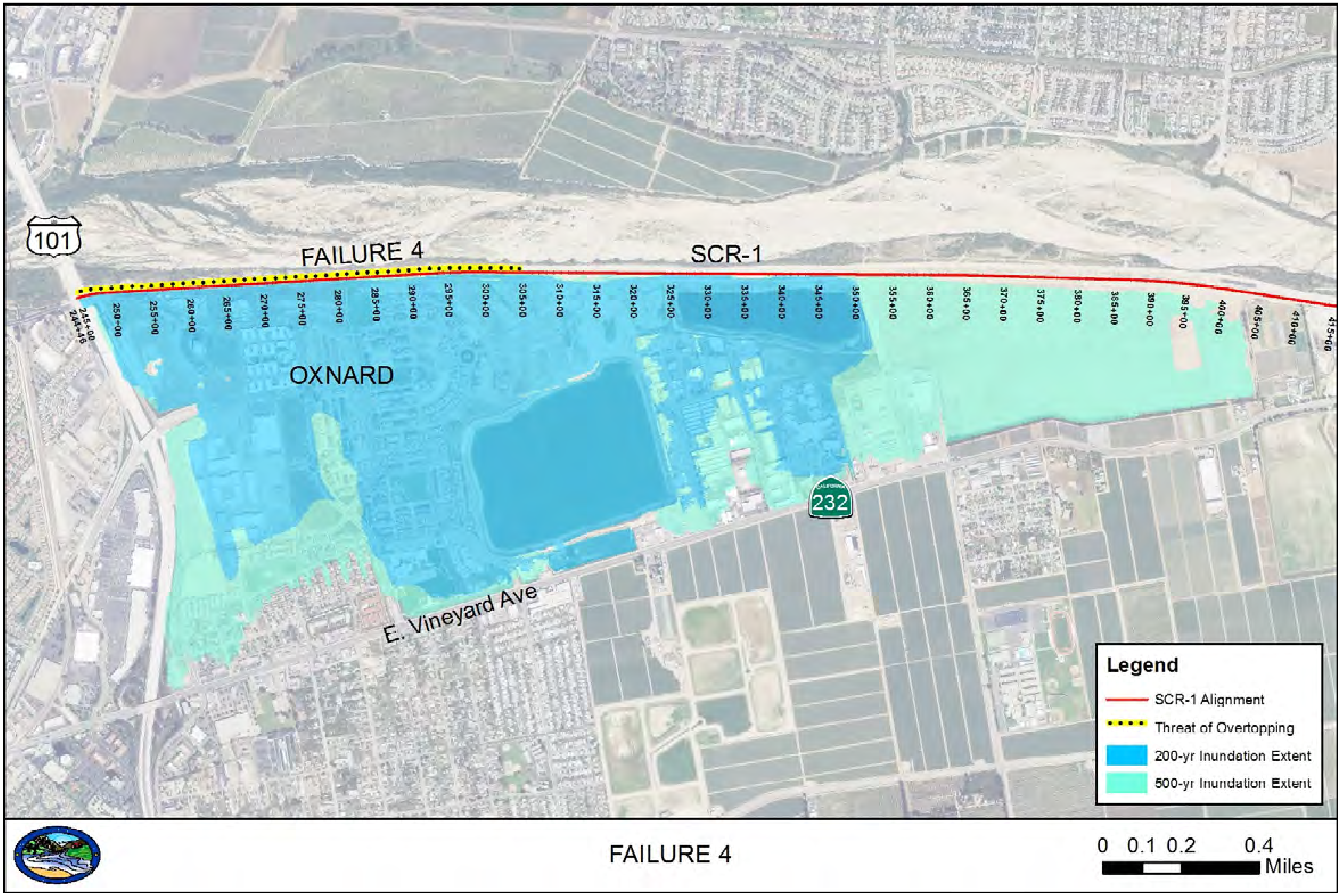


Figure 6 – Failure 4 Inundation Areas

4.5 Failure 5 – Storm Drain Failure

Failure 5 consists of a set of failures that are expected to be "malfunction of levee system components" failure type. The three levee system components evaluated in Failure 5 are penetrations by storm drains that are assumed to fail to close.

4.5.1 Failure 5a – Central Avenue Drain

The first penetration (Failure 5a) is the Central Avenue Drain, which consists of two 72-inch-diameter RCPs with flap gates at Station 351+50 (on riverside), a riverside opening elevation of 92 feet, and a landside opening elevation of 93.5 feet, allowing a 25- to 50-year flood event to begin flooding the landside of the levee. The culverts lead to a large drainage channel with elevated banks that contain flow depths resulting from the 100- to 200-year flood event. Beyond Vineyard Avenue, the storm drain continues as an underground culvert to Central Avenue and then becomes an open channel at Rio Mesa High School. The inundation area from the Santa Clara River through the Central Avenue Drain is shown in Figure 7. Flow from the 200- and 500-year events would spill over the channel into the low areas to the north and south. There would be no significant damage or affected population in any of these flooded areas.

4.5.2 Failure 5b – Side Drain No.4

The second penetration (Failure 5b) is Side Drain No. 4, which consists of a 48-inch-diameter RCP with a flap gate (on landside). The culvert is at a riverside elevation of 89 feet, with a landside elevation of 91 feet that places the opening at approximately the 50-year water surface elevation. The culvert opens directly to the landside of the levee at Station 316+60, just upstream of the northern extent of the newly developed residential area. The topography indicates that during a 100-year event, water entering this area through the culvert will begin to inundate the residential area, generally flowing south. The volume of water expected to inundate this area was calculated for each event on the basis of the head pressure exerted by the water surface elevation of the river. It was generally assumed that this volume of water could not accumulate to depths much greater than a foot without flowing downriver or being dispersed to other areas as sheet flow. Failure of the flap gate will lead to interior drainage flooding in residential neighborhoods from approximate Station 277+00 to Station 316+60, as shown in Figure 8. Table 6 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages. Table 7 provides the approximate number of structures and populations affected.

Table 6 – Failure 5b Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
25-year	NA	NA	0	$0
50-year	NA	NA	0	$0
100-year	~1	~1 to 2	9	$0
200-year	~1	~1 to 2	34	$956,000
500-year	~1	~1 to 2	106	$5,783,000
Expected Annual Damage				$12,500

NA = not applicable

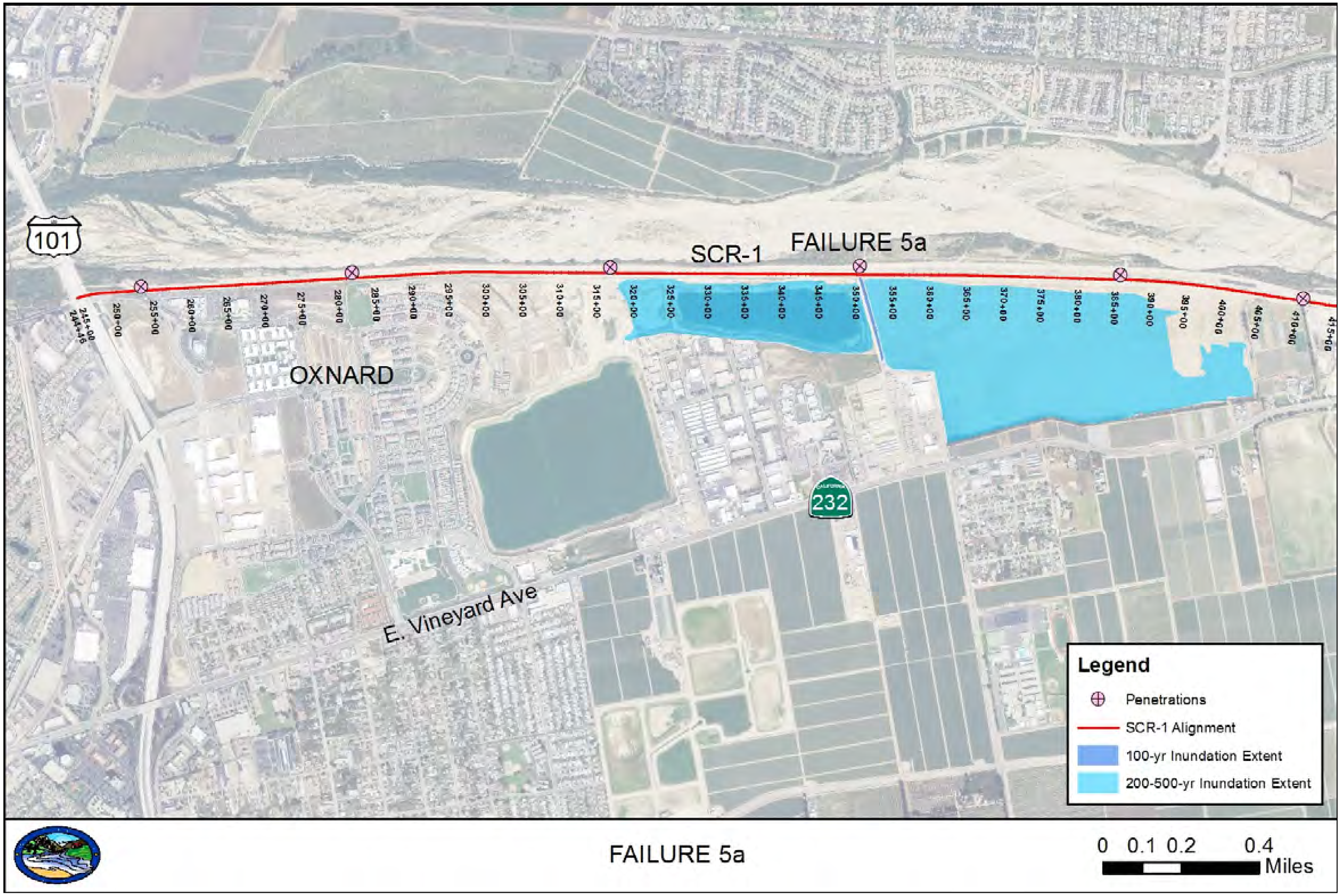


Figure 7 – Failure 5a Inundation Areas

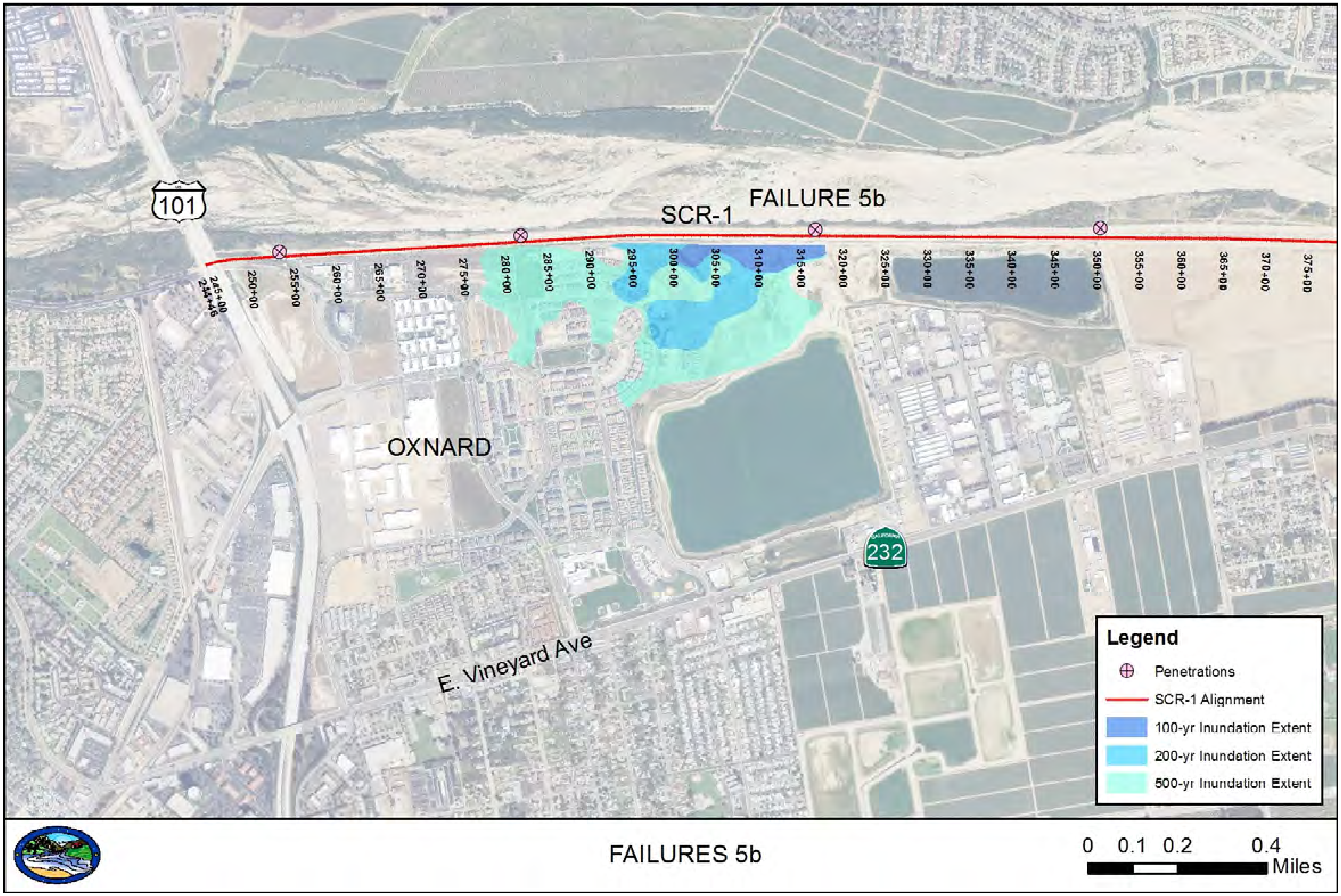


Figure 8 – Failure 5b Inundation Areas

Table 7 – Failure 5b Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
25-year	0	0	None	0	0
50-year	0	0	None	0	0
100-year	0	0	None	0	0
200-year	49	0	None	61	171
500-year	250	0	None	314	873

4.5.3 Failure 5c – Side Drain No.6

The third penetration (Failure 5c) is Side Drain No. 6, which consists of a 48-inch-diameter RCP with a flap gate (on landside). The riverside culvert is at an elevation of 74 feet, and the landside culvert is at an elevation of 78 feet, which indicates that flooding would occur only in events close to the 50-year flood. The culvert leads and opens directly to the landside of the levee at Station 282+00, into recreational and vacant open spaces surrounded by residential lots, with commercial spaces downstream. The volume of water expected to inundate this area was calculated for each event on the basis of the head pressure exerted by the water surface elevation of the river. It was generally assumed that this volume of water could not accumulate to depths much greater than a foot without flowing downriver or being dispersed to other areas as sheet flow. The likely flooded areas are shown in Figure 9. Table 8 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages. Table 9 provides the approximate number of structures and populations affected.

Table 8 – Failure 5c Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
25-year	NA	NA	0	$0
50-year	~1	~1 to 2	35	$14,196,000
100-year	~1	~1 to 2	90	$18,384,000
200-year	~1	~1 to 2	151	$21,610,000
500-year	~1	~1 to 2	291	$38,316,000
Expected Annual Damage				$494,700

NA = not applicable

Table 9 – Failure 5c Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
25-year	0	0	None	0	0
50-year	53	0	None	587	1,635
100-year	79	9	None	619	1,726
200-year	192	10	None	761	2,120
500-year	529	12	None	1,319	3,675

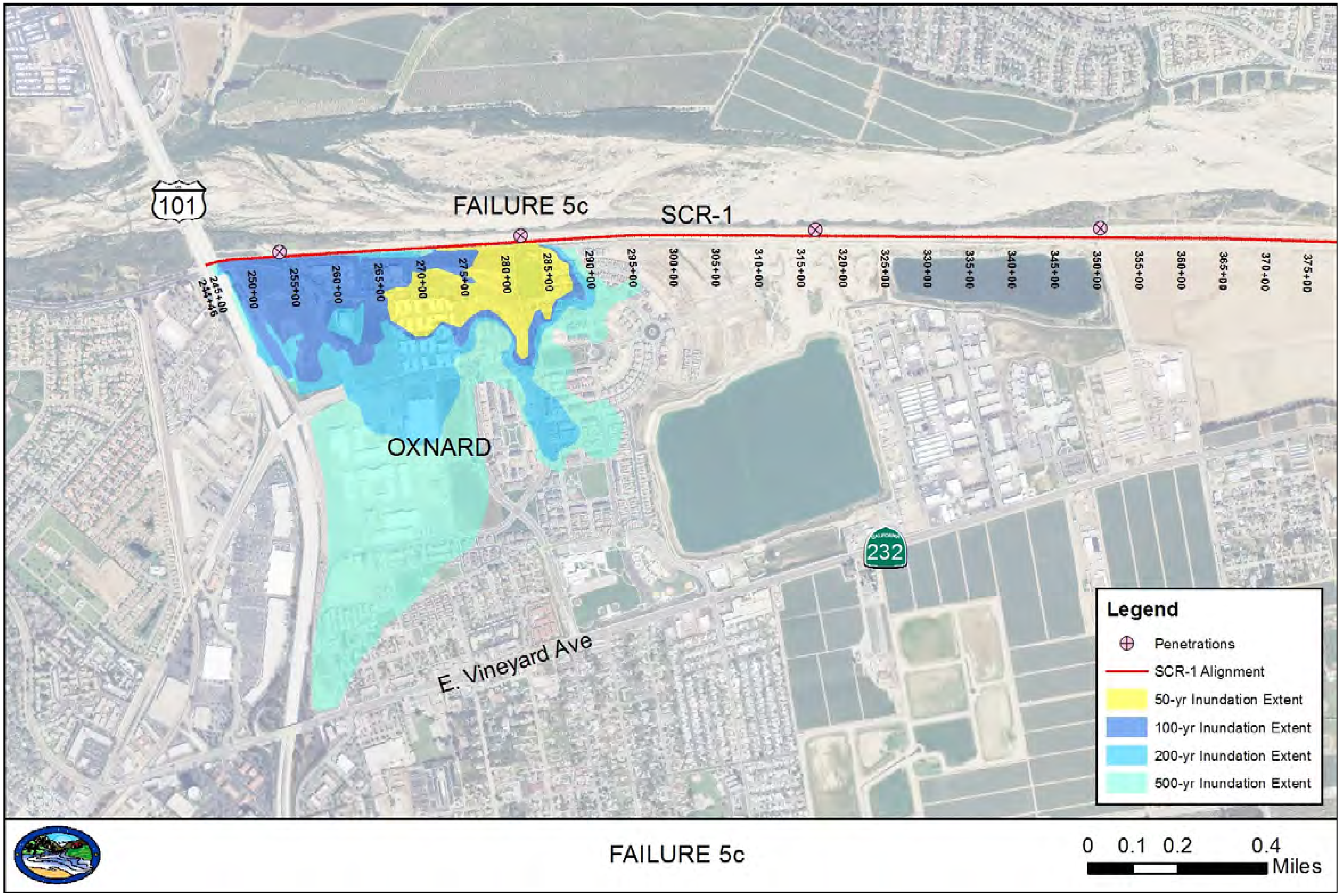


Figure 9 – Failure 5c Inundation Areas

4.6 Failure 6 – Sluice Gate Structure Failure at Station 246+20

This failure is a "malfunction of levee system components" failure type. The penetration structure is Stroube Drain, Unit I, which consists of a sluice gate structure in a 10-foot-wide by 8-foot-high reinforced-concrete box (RCB). Although the closure structure, located at Station 246+20, is just above the water surface elevation of the 5-year event (at 65 feet), the landside elevation of 76 feet and the surrounding topography indicate that backwater flooding would not occur until the 50-year event, when the head pressure would build enough to cause storm drains to back up and overflow. The topography indicates that floodwater will flow south along Ventura Road and pond under the Highway 101 Bridge. In larger events, the volume of water capable of entering the area could potentially inundate lower lying areas along the north side of Highway 101, as shown in Figure 10. Table 10 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages. Large flood depths are due to ponding under and beside the Highway 101 Bridge. Table 11 provides the approximate number of structures and populations affected.

Table 10 – Failure 6 Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
25-year	NA	NA	0	$0
50-year	~1 to 2	~6	3	$0
100-year	~1 to 2	~13	26	$148,000
200-year	~1 to 2	~16	57	$4,628,000
500-year	~1 to 2	~18	142	$16,936,000
Expected Annual Damage				$45,000

NA = not applicable

Table 11 – Failure 6 Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
25-year	0	0	None	0	0
50-year	0	0	None	0	0
100-year	0	1	None	96	0
200-year	0	8	None	277	0
500-year	18	11	None	1,654	1,963

4.7 Failure X – Breach from Station 352+00 to Station 402+00

This failure is expected to be a "breach prior to overtopping" failure. Although Failure X was not considered in the IRRMP analysis as it is not a likely failure scenario, VCWPD is interested in information about a potential failure along this reach for internal purposes. In general, the landside toe along this reach is at a lower elevation than the riverside toe (see Attachment 1). A failure of the levee embankment would lead to inundation of the adjacent floodplain, which consists of a large depressed spreading ground managed by the United Water Conservation District (UWCD). The riverside toe becomes hydraulically loaded at the 25-year event near Station 352+00 to the 200-year event at Station 378+00. There is no known historical damage for this reach of the levee. In this reach, a levee breach would cause inundation of the depressed area, which is contained by high banks on all sides. Flooding beyond the spreading ground would only occur around the 200-year event. The flood extents are shown in Figure 11.

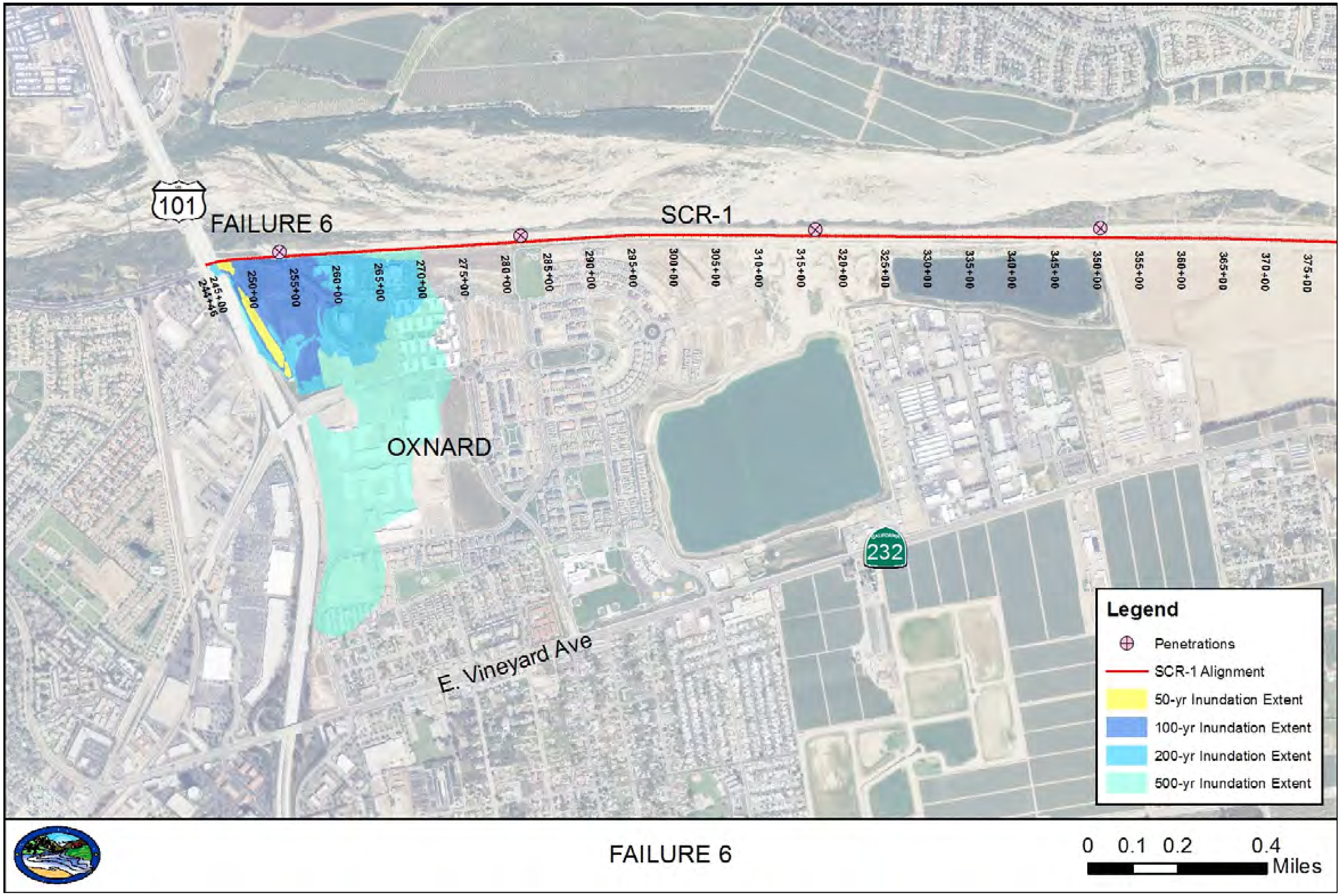


Figure 10 – Failure 6 Inundation Areas

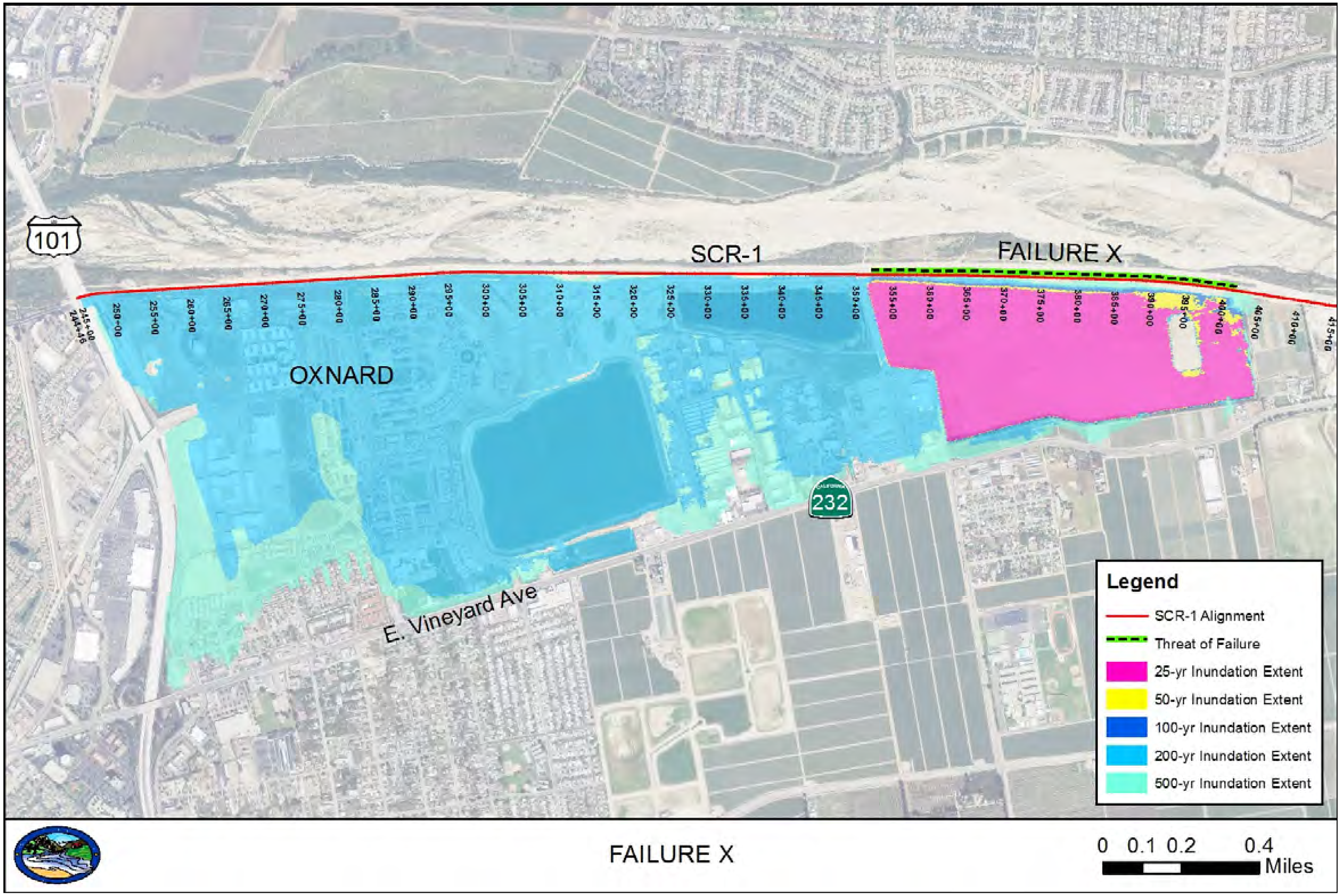


Figure 11 – Failure X Inundation Areas

Table 12 provides the expected flood depths in the adjacent floodplain at a range of flood events and the associated damages. Table 13 provides the approximate number of structures and populations affected.

Table 12 – Failure X Affected Area

Frequency	Average Depth (feet)	Maximum Depth (feet)	Area Flooded (acres)	Damages
25-year	8.6	29.6	170	$0
50-year	10.8	31.6	179	$0
100-year	13.3	33.9	185	$0
200-year	6.9	36.0	924	$129,679,000
500-year	9.4	43.0	1,106	$307,674,000
Expected Annual Damage				$980,000

Table 13 – Failure X Affected Structures and Populations

Frequency	Residential	Non-Residential	Critical Facilities	Population at Risk (Day)	Population at Risk (Night)
25-year	0	0	None	0	0
50-year	0	0	None	0	0
100-year	0	0	None	0	0
200-year	853	37	1 school	3,271	4,429
500-year	1,214	72	1 school	5,861	6,067

5.0 POTENTIAL IRRMS

Potential IRRMs were developed on the basis of the considerations discussed in *Engineering and Construction Bulletin No. 2012-1* (USACE 2012). IRRMs were developed first for the failure scenarios associated with the "breach prior to overtopping" failure mode. Additional IRRMs were then developed for the "overtopping with breach" and the "malfunction of levee system components" failure.

5.1 Breach Prior to Overtopping

A "breach prior to overtopping" is a critical failure mode because it has the least warning (not predictable) of all the failure modes. Of the failure scenarios, Failure 1 is the most significant because it is associated with the most significant damages and the most population at risk. To provide interim risk reduction for Failure 1, potential measures were identified (Table 14). Development of IRRMs focused on addressing risks associated with Failure 1 given the magnitude of consequences, however these measures could also reduce risks associated with Failures 2 and 3.

Table 14 – IRRMs for Levee Breach Prior to Overtopping

Measure Number	Description of Measure
1	FWEEP
2	Update Hydrologic and Hydraulic Study
3	Update Scour Study
4	Develop Local NLD
5	GIS Performance Database
6	Develop Project Operations and Maintenance Manual
7	Visual Markers
8	Stockpile Materials
9	Diversion to Gravel Pit
10	Diversion to Upstream Reservoir
11	Launchable Rock
12	Improve Groins
13	Full Bank Protection

GIS = geographic information system
FWEEP = Flood Warning and Emergency Evacuation Plan
NLD = National Levee Database

Measure 1: Develop a FWEEP. The Flood Warning and Emergency Evacuation Plan (FWEEP) would be an enhanced version of the existing countywide plan. It would focus on the conditions that are specific to the SCR-1 reach. The cost of this measure would be $50,000.

Measure 2: Update Hydrologic and Hydraulic Studies. Implementation of a long-term solution requires an updated hydrologic and hydraulic analysis to use as the basis for design. Existing hydrology (VCWPD 2011) should be reviewed and the current HEC-RAS hydraulic model that was developed for the FEMA Flood Insurance Study (FIS) (FEMA 2010) or the Corps Watershed Study (USACE 2012) should be updated with available topographic mapping such that it is consistent with the requirements for a feasibility-level design. The cost of this measure would be $15,000.

Measure 3: Update Scour Study. Implementation of a long-term solution requires updated sediment-transport and scour analyses to support the freeboard, embankment protection, and embankment stability analyses. Analysis should be performed to provide insight into long-term system sediment discontinuity and expected future channel degradation if the current watershed and river management practices remain unchanged. After the analysis, appropriate local-scour equations will be used to predict single-event scour associated with specific storm events along SCR-1. The cost of this measure would be $80,000.

Measure 4: Develop Local National Levee Database (NLD). This measure would involve the development of a geographic information system (GIS) database to graphically display data specific to the SCR-1 system and make it available in a National Levee Database (NLD) format. IRRMs adopted for SCR-1 would be available in the NLD as data layers. Additionally equipment and material storage would be identified. The cost of this measure would be approximately $20,000.

Measure 5: GIS Performance Database. This measure would formalize the reporting and evaluation that would accompany the Visual Markers (Measure 7), as well as housing performance data to assess the level of loading experienced during each storm event and the performance of the levee. The information in this database would be reviewed as part of the preseason readiness, an activity to be included in the Project O&M Manual (Measure 6), and could be included as a layer in the Local NLD (Measure 4). The initial set-up cost of this measure would be approximately $10,000

Measure 6: Develop Supplemental O&M Manual. The existing operations and maintenance (O&M) manual (USACE 1963) was provided to the VCWPD by the Corps at completion of SCR-1 construction. The 1963 manual is appropriate for standard O&M practices but does not incorporate the performance history, which would highlight the reaches that are most at risk. As part of this measure, a supplemental O&M manual would be prepared to include performance history and IRRM measures specific to the SCR-1 system. The cost of this measure would be approximately $25,000.

Measure 7: Visual Markers. The critical feature to evaluate for the levee breach failure modes is the lateral extent of floodwaters and any active erosion of the groins. The recommended IRRM measure is the installation of markers adjacent to the groins, with one at the tip of each groin (a yellow marker), followed by one 50 feet from the tip toward the levee embankment (a red marker). The approximate recommended locations of the markers are indicated in Table 15.

Table 15 – Approximate Visual Marker Locations

Marker Stations*	Applicable Failure Scenarios
271+00	Levee Overtop Failure 4 (Station 244+00 to Station 304+00)
289+00	
299+00	
310+00	Levee Breach Failure 1 (Station 311+00 to Station 348+00)
320+00	
329+00	
341+00	
354+00	Levee Breach Failure X (Station 352+00 to Station 402+00)
370+00	
395+00	
438+00	Between Levee Breach Failures 2 and 3
442+00	

*Note: These locations may change to minimize disturbance to existing species and habitat

The measure assessment and associated actions are identified in Table 16.

Table 16 – Visual Marker Assessment

Visual Assessment of Markers	Action
Flow is at the yellow or red marker, but the marker and groin are stable.	Notify storm patrol to verify status of markers at a maximum of 60-minute intervals.
Flow is at the yellow marker, and the marker and groin are showing instability (i.e., the marker has rotated or is slumping due to undermining at the groin).	Storm patrol should be continuous at this location. Notify EOC that the situation is approaching a critical point and put at-risk population on alert. Mobilize emergency repair crews and supplies to the site. If failure occurs (yellow marker showing instability), expect 4 hours until breach.
Flow is at the red marker, and the marker and groin are showing instability (i.e., the marker has rotated or is slumping due to undermining at the groin).	Storm patrol to be continuous and emergency repair crews active at this location. Notify EOC to evacuate affected population. If failure occurs (yellow and red marker showing instability), expect 2 hours until breach.

EOC = Emergency Operations Center

This measure could be incorporated into both the FWEEP and the levee O&M Manual. It is assumed that the markers could be installed under the existing O&M permits for the system, and the estimated cost of the markers, including up-front permitting efforts and mitigation, would be approximately $50,000.

Measure 8: Stockpile Materials. Materials stockpiled on site would provide supplies to address failures immediately.

It is suggested that these materials consist of:

- ½ ton class rock (D50) of 28 inches (approximately 2,500 cubic yards),
- gravel stockpiles for maintaining transit paths on wet ground (approximately 2,000 cubic yards),
- multi-use plastic sheeting such as polyethylene (6mm thick),
- emergency lighting equipment for night flood fighting, and
- reliable communications system.

The rock was sized for an anticipated velocity of 17 feet per second and suitable for underwater placement (i.e., during the flood event). Preliminary calculations indicate that ½ ton class rock with a median diameter (D_{50}) of 28-inches would be suitable (D_{100} = 42-inches). Typical breach lengths along U.S. rivers have been reported to be approximately 500 feet (FEMA 2013). A reasonable preliminary scour estimate based on engineering judgment for a design flood is on the order of 10 feet. With an assumed design slope of 2H:1V and an additional 5 feet of embankment height yields a total rock volume of 2,175 cubic yards (58,725 cubic feet). Based on a unit price for riprap of $135 per cubic yards, the cost for the material for this measure would be $295,000.

The amount of gravel to maintain access for heavy equipment and fill in voids during rock placement was estimated based on a path 12 feet wide, 12-inches thick and 4,500 feet long, which yields a total gravel volume of 2,000 cubic yards. Based on a unit price for gravel of $40 per cubic yards, the cost for the material for this measure would be $80,000.

The rock and gravel stockpiles are assumed to be 5 feet high for frontend loader placement and loading. These stockpiles would require a base footprint of approximately 48,000 square feet.

The acreage of the potential sites for the stockpiled material includes the footprint of the stockpiled material and a 20-foot buffer for vehicles and personnel access. There are five potential sites (Figure 12).

- Site 1, which is located near Station 280+00, is a large undeveloped private property. It is located next to the levee as well as a City park. The parcel is 10.2 acres, with the stockpile site occupying 1.45 acre, 14 percent of the property.
- Site 2, which is located near Station 320+00, is private property that is part of one of the old gravel pit mines. It is located next to the levee, on two parcels under the same ownership. The combined parcels constitute 67.7 acres, with the stockpile site occupying 1.45 acre, 2 percent of the property.
- Site 3 is located near Station 352+00, on undeveloped County-owned property. It is set back from the levee beyond an adjacent gravel pit. Site 3 is farther from the levee than Sites 1 and 2, but it would be less susceptible to damage than the other two sites if the levee breaches quickly. The parcel is 6.1 acres, with the stockpile site occupying 1.45 acre, 24 percent of the property.
- Site 4 is located near Station 402+00, on County-owned land that is being used by the United Water Conservation District (UWCD). It is located next to the levee and adjacent to agricultural land. This area is undeveloped and is farther upstream than the other three sites, at a location with higher bank elevations and little history of flood damage. The parcel is 6.1 acres, with the stockpile site occupying 1.45 acre, 24 percent of the property.
- Site 5 is located on the north side of the river, opposite the levee, in Saticoy. It is currently used as a county stockpile site for flood defense materials as well as other uses. It is a large pre-existing site capable of stockpiling additional rock. Material would have to be transported upstream or downstream across Hwy 101 or Los Angeles Ave bridges for flood fighting efforts on the SCR-1. The stockyard size is approximately 9.8 acres, with the stockpile occupying 1.45 acre, 15 percent of the property.

Of the potential sites detailed above, Site 3 appears to be the best candidate for a stockpile location. It is in relative proximity to the levee, although not close enough to take immediate damage should a catastrophic levee failure occur. Site 3 also has easy access for large vehicles from three directions to public streets or county right of way, as well as at least one direct access route to the levee. The site is large enough to accommodate at least twice the proposed rock stockpile footprint should the space be required by flood fighting personnel. The property is County owned land, however the County may have an alternate plan for its use. In addition to Site 3, the County currently has an existing stockpile site (Site 5) for flood fighting and other uses on the north side of the river near Saticoy (Figure 13).

Stockpiled rock is best placed with heavy equipment such as dump trucks and front-end loaders that can place the rip-rap to the desired thickness at a given location in one lift. Otherwise, excavators with special purpose-use heads such as clamshells, or a thumb, are preferred for precision rip-rap placement. If possible, placed rip-rap should be from the bottom working to the top if possible to minimize rolling and/or segregation. The need or use of these materials and equipment may vary depending on the state of the scour and the condition of the levee once flood fighting is underway.

Materials and equipment on hand should equal the needs for the initial response, but keeping a current account of nearby quarries or other stockpile sites and their contact information for additional material needs, as well as local contractors and their contact information who can be called upon to bring additional vehicle or equipment resources to bear for flood fighting efforts, should be maintained. Preparedness for possible ongoing or larger flood fighting efforts is key. As storm patrols on the levee begin, monitoring the marker locations and status, stockpiled materials should be verified and necessary equipment on hand to mobilize should the markers begin to indicate scour of the groins is approaching a critical point requiring action.

Several rolls of multi-use plastic sheeting such as polyethylene (6mm thick) are also suggested to be readily available for aiding in erosion protection, seepage mitigation where water may back up for extended periods, and directing local flows away from the levee or transit paths.

Since flooding could occur during the night, emergency lighting for the stockpile yard as well as failure locations along the levee is recommended. There may be a need for up to 4 light stations at the stockpile yard and 4-6 light stations along the levee.

As with any emergency, reliable communications to coordinate efforts is a must. It is assumed that the County has adequate communication equipment and that personnel have been trained for this type of situation.

The available time to access and install the proposed additional stockpiled materials was assessed on the basis of the available hydrographs for the studied events. The time available once the 10-year water surface elevation has been reached is indicated in Table 17.

Table 17 – Time to Peak Volume Exceeding Water Surface Elevation of 10-Year Event

Frequency	Time from 10-Year WSE to Peak Volume (hours)	Time from Peak Volume to 10-Year WSE (hours)	Total Duration of Peak Volume (hours)
25-year	4	2	6
50-year	6	5	11
100-year	18	6	24
200-year	18	6	24
500-year	23	7	30

WSE = water surface elevation

Based on the available time, County crews are expected to have time to install the stockpiled material to fight the flood. The total estimated cost for this IRRM is $400,000 (does not include the cost for stockpile site land or improvements). Depending on the site location and the improvements needed, VCWPD has estimated another $200,000 would be required to implement this IRRM.

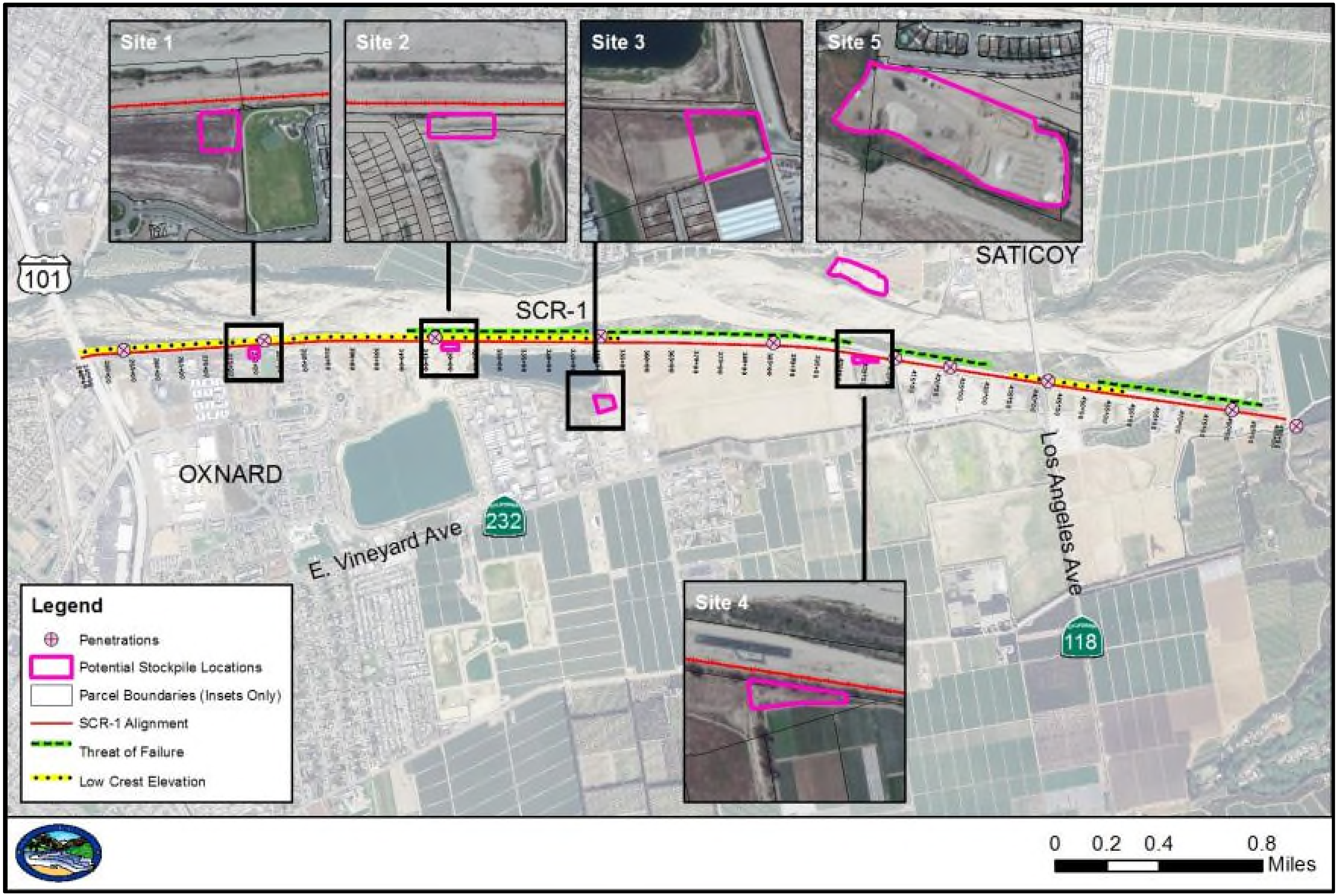


Figure 12 – SCR-1 Potential Stockpile Locations

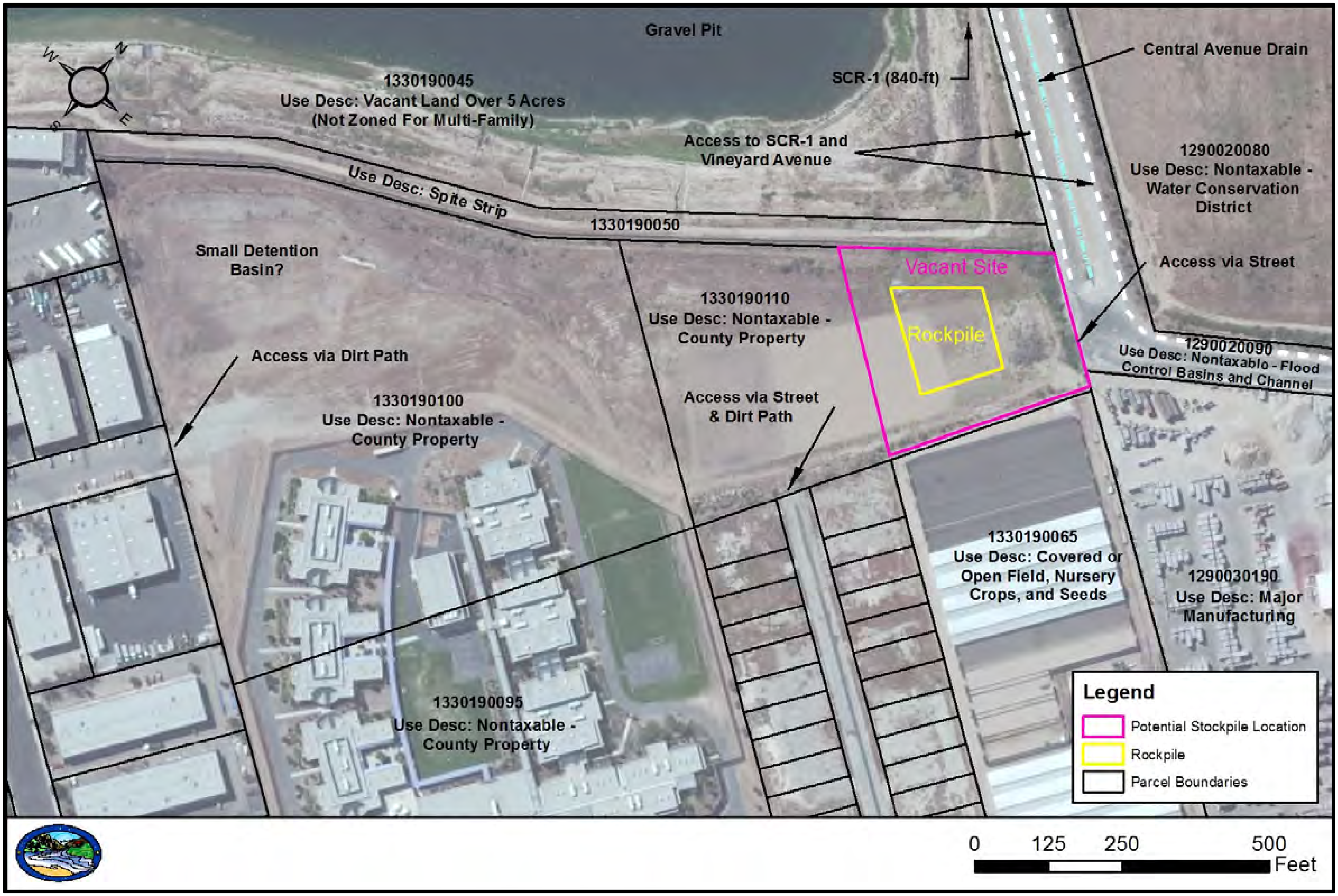


Figure 13 – Site 3: Recommended Stockpile Location

Measure 9: Diversion to Gravel Pit. SCR-1 is adjacent to two large privately owned gravel pit mines as well as a large spreading ground managed by the UWCD. With breach threats greatest downstream of these features, one potential measure is to use the pits and basin for peak water storage to relieve the pressure on the levee. This could be accomplished by installing diversion structures from the river into the spreading ground basin and connecting the basin to the other two downstream gravel pits. The total usable storage capacity of the settling basin and gravel pits is approximately 8,000 acre-feet. Diverting peak flows greater than those of the 10-year event produces the volumes in Table 18, which also indicates the percentage of the peak volume able to be detained. Also tabulated is the time required to reach the storage capacity once the 10-year-event water surface elevation has been exceeded and the duration of the exceedance of the 10-year event water surface elevation.

Table 18 – Peak Volumes Exceeding Water Surface Elevation of 10-Year Event

Frequency	Total Volume of Flow (acre-feet)	Amount of Flow Divertible to Storage	Time Required to Reach Storage Capacity (hours)	Duration of Exceedance of 10-Year WSE (hours)
25-year	8,168	100%	6	6
50-year	31,769	26%	4	11
100-year	71,796	11%	6	17
200-year	135,193	6%	5	24
500-year	254,146	3%	5	30

WSE = water surface elevation

Despite the potential storage on the landside of the levee, this measure is limited by the efficiency of diversion structures along the levee in diverting flow into these storage areas. If the lateral weir is considered as a diversion structure, the bottom of the weir needs to be as low as the 10-year flood water surface elevation so that any flow exceeding that of the 10-year event can be diverted. With this requirement, the weir dimensions must be 20 feet high and 3,500 feet long to divert the peak flow during the 25-year event. However, the closure structure that will cover this lateral weir opening is not feasible in terms of design and costs.

Tetra Tech recently designed closure structures (Tainter gate) for the Corps' Robles Diversion Dam, which included four 30-foot-wide by 12-foot-high openings for diversion. This same set of gates was used as a conceptual layout for Measure 9. This weir will be able to divert only the flow greater than the 50-year event because the bottom of the weir is only 12 feet below the top of the levee. With this structure, approximately 1 percent of the 100-year flood in excess of the 50-year event water surface elevation will be diverted. Based on the gate structures (four Tainter gates) and landside toe protection, the cost of this measure would be approximately $1,500,000.

The feasible structure (four 30- by 12-foot Tainter gates) would not provide considerable flood protection because the total amount of diverted flow would be low. Also, there are potential risks associated with tainter gates, including failure of mechanical components and landside protection. The gate system will need to be inspected and maintained prior to a large storm to ensure the gate is in operable conditions. The cost of this measure would be high. In addition, it would require considerable time to design, permit, and construct, making it difficult to implement as part of an interim plan.

Measure 10: Diversion to Upstream Reservoir. There are four major reservoirs within the Santa Clara River Watershed: the Bouquet Canyon, Pyramid, Castaic, and Piru Reservoirs (Figure 14). The Bouquet Canyon Reservoir is operated by the Los Angeles City Department of Water and Power. It provides important safety storage downstream from the San Andreas Fault for the water transported through the Los Angeles Aqueduct, as well as water from peak hydroelectric power generation at the San Francisquito Power Plants. The Pyramid and Castaic Reservoirs are part of the State Water Project (SWP) system and are operated by the California Department of Water Resources. The Pyramid Reservoir is located on Piru Creek, and the Castaic Reservoir is located on Castaic Creek, but the two are hydraulically connected. The Piru Reservoir is run by UWCD and located on Piru Creek downstream from the Pyramid Reservoir. UWCD's primary operational goals are groundwater recharge, public recreation, and power generation.

The Pyramid Reservoir is operated to provide water storage and flood protection and to provide water to Piru Creek and the Piru Reservoir, primarily for agriculture, groundwater recharge, and flow maintenance to support habitat. The Pyramid Reservoir releases approximately the natural inflows to Piru Creek. During significant rainfall events, when natural inflows are high, some of the flow is retained, either for later release at the request of downstream users in the watershed or for appropriation by the SWP for delivery to outside users via the Castaic Reservoir. The Piru Reservoir also receives runoff water from the local watershed. Water storage at the Piru Reservoir allows for strategic conservation releases aimed at recharging downstream groundwater basins and aquifers, which provide irrigation and drinking water, and ultimately help restrain the saltwater intrusion on the Oxnard Plain. Low-flow release volumes are also used by downstream landowners who hold riparian rights.

The reservoir drainage areas and capacities are summarized in Table 19. The total regulated drainage area of the four reservoirs is approximately 590 square miles. The Santa Clara River drainage area is 1,690 square miles approximately at the Pacific Ocean. The total regulated drainage area consists of only 35 percent of the entire Santa Clara River drainage area. None of the reservoirs are owned by VCWPD. An initial assessment of the effort that would be necessary to coordinate and obtain approval for a change in the reservoirs' operation to serve a goal (i.e., flood risk) outside of the current use (i.e., water supply and power generation) indicates that it would be a significant effort, particularly in terms of time. The potential time required to implement this type of change is outside the "interim" nature of the measures that are being considered in this IRRM plan. No cost was estimated for this measure.

Table 19 – Reservoirs in Santa Clara River Watershed

Reservoir	Owner	County	Stream	Drainage Area[1] (square miles)	Capacity[2] (acre-feet)
Bouquet Canyon	City of Los Angeles	Los Angeles	Bouquet Creek	13.6	36,500
Castaic	California Department of Water Resources	Los Angeles	Castaic Creek	153.7	323,700
Pyramid	California Department of Water Resources	Los Angeles	Piru Creek	293.0	171,200
Piru	United Water Conservation District	Ventura	Piru Creek	421.4	88,340

1. Source: California Department of Water Resources, Division of Safety of Dams 2010.
2. Source: Aqua Terra 2009.

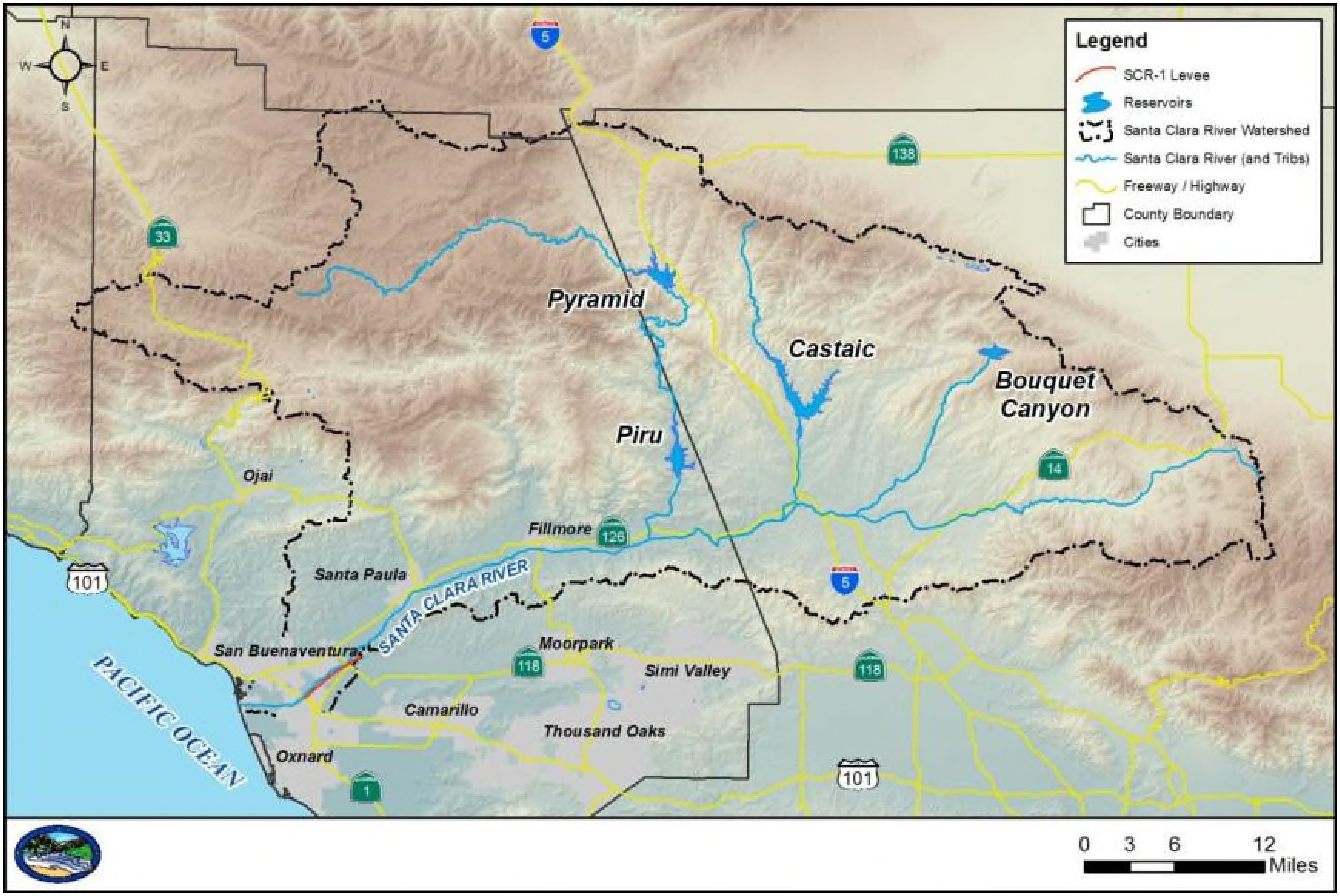


Figure 14 – Reservoir Locations

Measure 11: Launchable Rock. A partial protection measure is to install a launchable stone toe that would provide a degree of stability if the existing channel bank toe begins to be undermined. The critical elevation to consider is the elevation of the landside toe; instability above that elevation could result in inundation of the protected area. This measure considers placing the launch section at the same elevation as the landside toe. It is assumed that a 5-foot-high protected toe would provide a reasonable degree of protection for the interim condition. Following design guidelines in Engineer Manual 1110-2-1601 (USACE 1994), the launch section would be 20 feet wide measured out from the riverside face. Launchable rock would be provided along Failure Reach 1 (Station 311+00 to Station 332+00, Failure Reach 2 (Station 410+00 to Station 430+00), and Failure Reach 3 (Station 452+00 to Station 486+00). Table 20 provides the quantities and costs for Measure 10. Typical cross sections for each reach are shown in Figures 15, 16, and 17.

Table 20 – Quantities and Costs for Measure 11

Failure Number	Reach Length (feet)	Volume of Rock (cubic yards)	Construction Cost	Expected Annual Damages
1	2,100	7,778	$3,330,573	$1,633,000
2	2,000	7,408	$2,267,581	$0
3	3,400	12,593	$5,337,024	$0

Table 20 also shows the expected annual damages, as shown in Table 2. Because there are no FDA calculated economic damages associated with Failures 2 and 3, this measure would only include improvements along Failure Reach 1. This is a high-cost measure that would be difficult to design, permit, and construct on the expedited schedule associated with an interim plan.

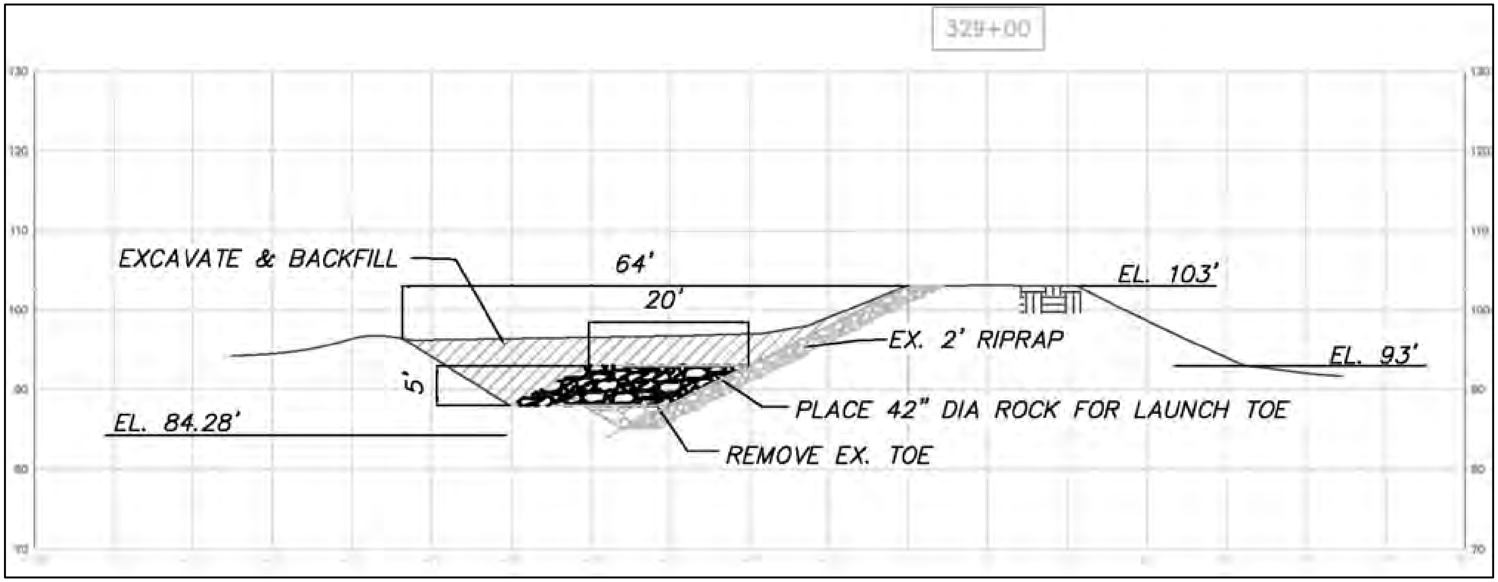


Figure 15 – Typical Cross Section of Measure 11, Failure Reach 1

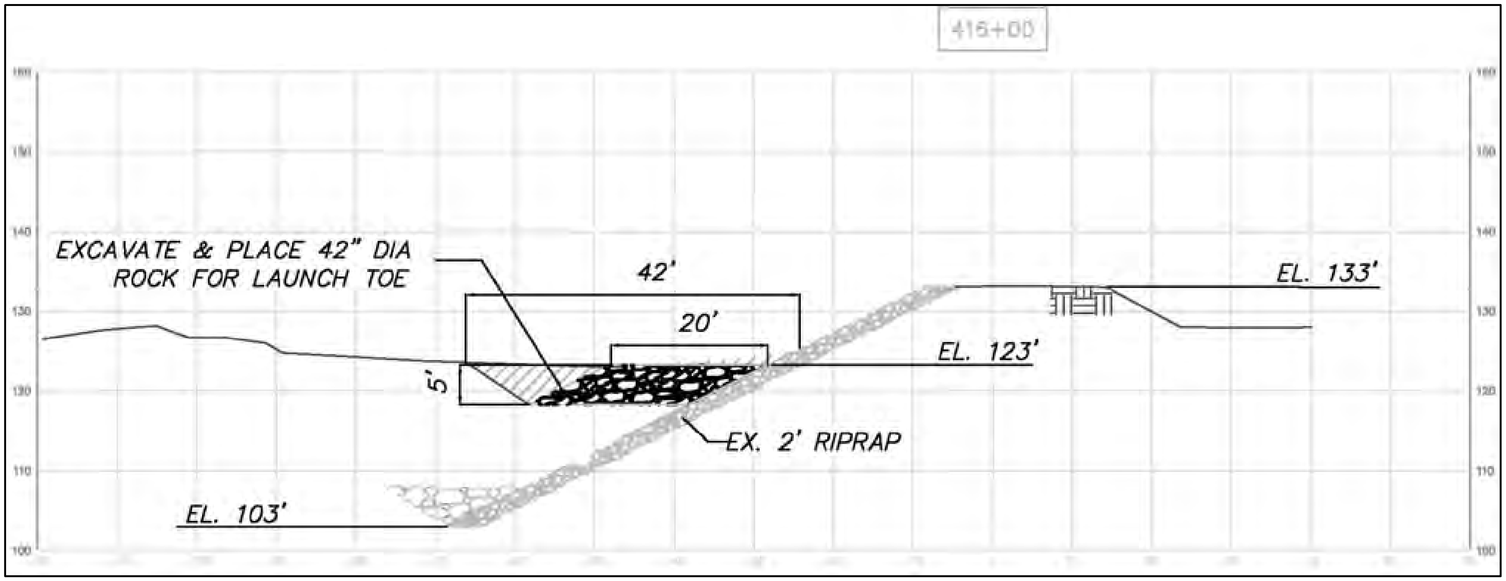


Figure 16 – Typical Cross Section of Measure 11, Failure Reach 2

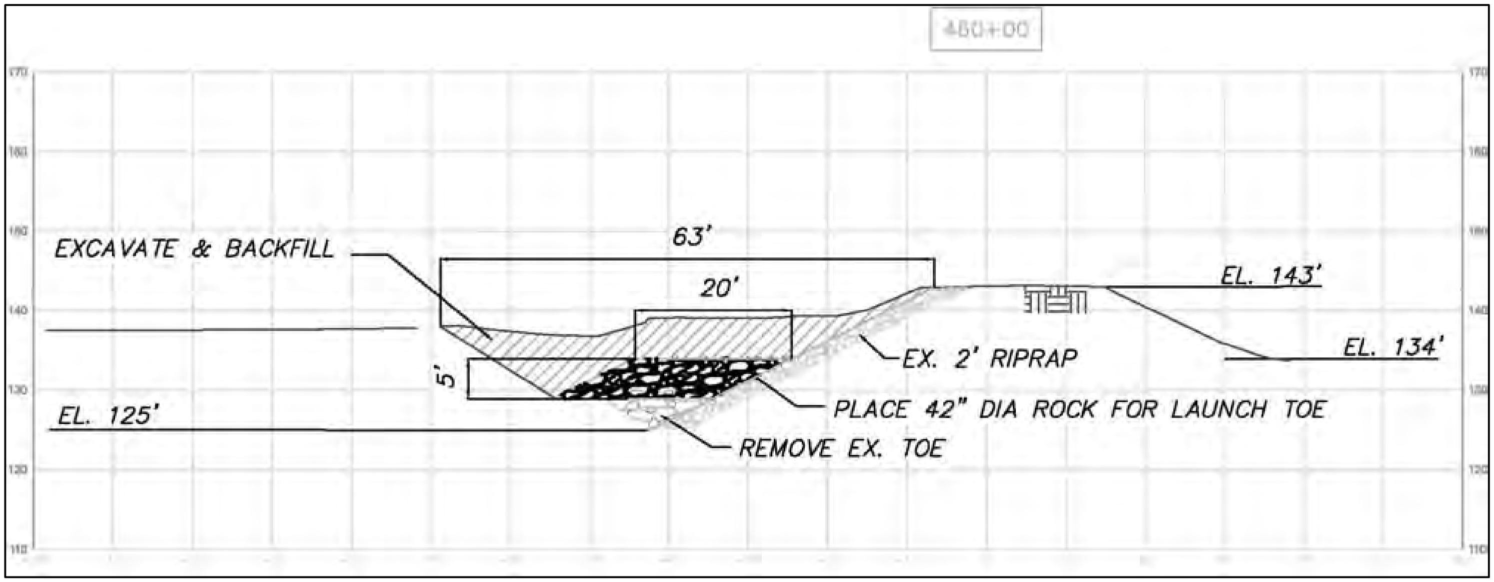


Figure 17 – Typical Cross Section of Measure 11, Failure Reach 3

Measure 12: Improve Groins. According to the available as-built drawings (USACE 1961, 1971), the existing groins are stabilized by rock with an unknown D_{50}. In past events, this rock was washed out and the bank was compromised. This measure includes rebuilding the groins within the failure reaches with larger rock. Hydraulic analysis has indicated that ½-ton rock riprap (D_{50} of 28 inches) would be adequate for the conditions of SCR-1. There are a total of 75 groins along SCR-1, with 19 of them located within the three failure reaches. Three typical groin cross sections were used to represent each of the three failure reaches. Table 21 compares the quantities and costs of rebuilding the groins within the individual reaches. Figure 18 shows a cut through the existing groin. Typical cross sections for each failure reach are shown in Figures 19, 20, and 21. It is important to note that the cost of mitigation ($150,000 per acre) has a large influence on the total construction costs that are reflected in Table 21.

Table 21 – Quantities and Costs for Measure 12

Failure Reach	Number of Groins	Volume of Rock (cubic yards)	Construction Cost	Expected Annual Damages
1	7	7,586	$4,289,546	$1,633,000
2	3	2,567	$1,457,509	$0
3	9	7,700	$4,372,528	$0

Table 21 also shows the expected annual damages, as shown in Table 2. Because there are no FDA economic damages associated with Failures 2 and 3, this measure would only include improvements along Failure Reach 1. This is a high-cost measure that would be difficult to design, permit, and construct on the expedited schedule associated with an interim plan.

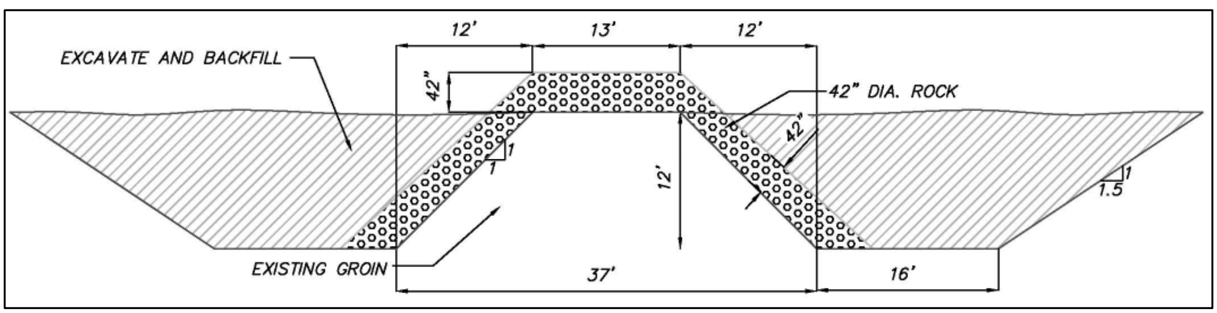


Figure 18 – Typical Detailed View of Measure 12 through Cut Portion of Groin

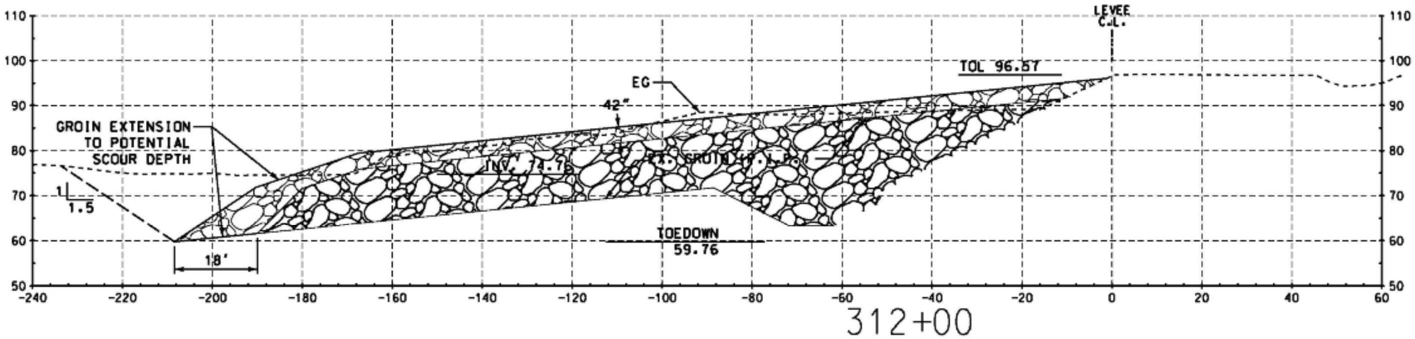


Figure 19 – Typical Cross Section of Measure 12, Failure Reach 1

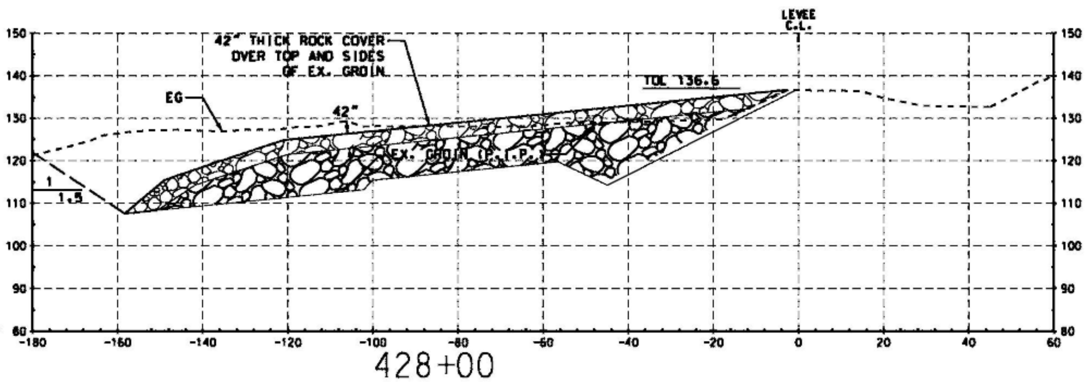


Figure 20 – Typical Cross Section of Measure 12, Failure Reach 2

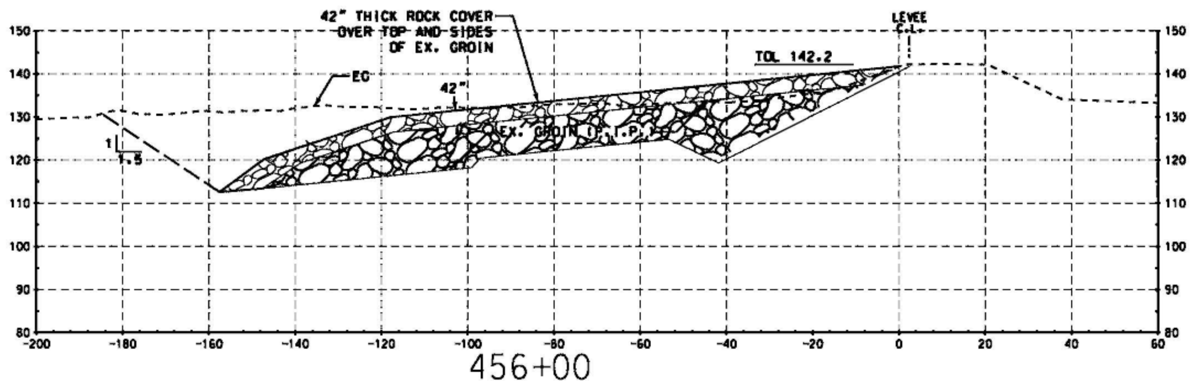


Figure 21 – Typical Cross Section of Measure 12, Failure Reach 3

Measure 13: Full Bank Protection. This measure involves inserting soil cement with a top width of 8 feet along the height of the riverside bank. The soil cement would extend from the top of the levee to the toedown elevation (15 feet below the invert elevation) and have a slope of 1H:1V. The benefit of this measure is that it provides full bank protection while minimizing the mitigation costs because of the small affected surface area. A typical cross section is shown in Figure 22. Table 22 provides the quantities and costs for each failure reach.

Table 22 – Quantities and Costs for Measure 13

Failure Reach	Length (feet)	Volume of Soil Cement (cubic yards)	Total Construction Cost	Expected Annual Damages
1	2,100	22,300	$5,034,704	$1,633,000
2	2,000	29,239	$7,233,882	$0
3	3,400	49,706	$12,297,599	$0

Table 22 also shows the expected annual damages, as shown in Table 2. Because there are no damages associated with Failure Reaches 2 and 3, this measure would only include improvements along Failure Reach 1. This is a high-cost measure that would be difficult to design, permit, and construct on the expedited schedule associated with an interim plan.

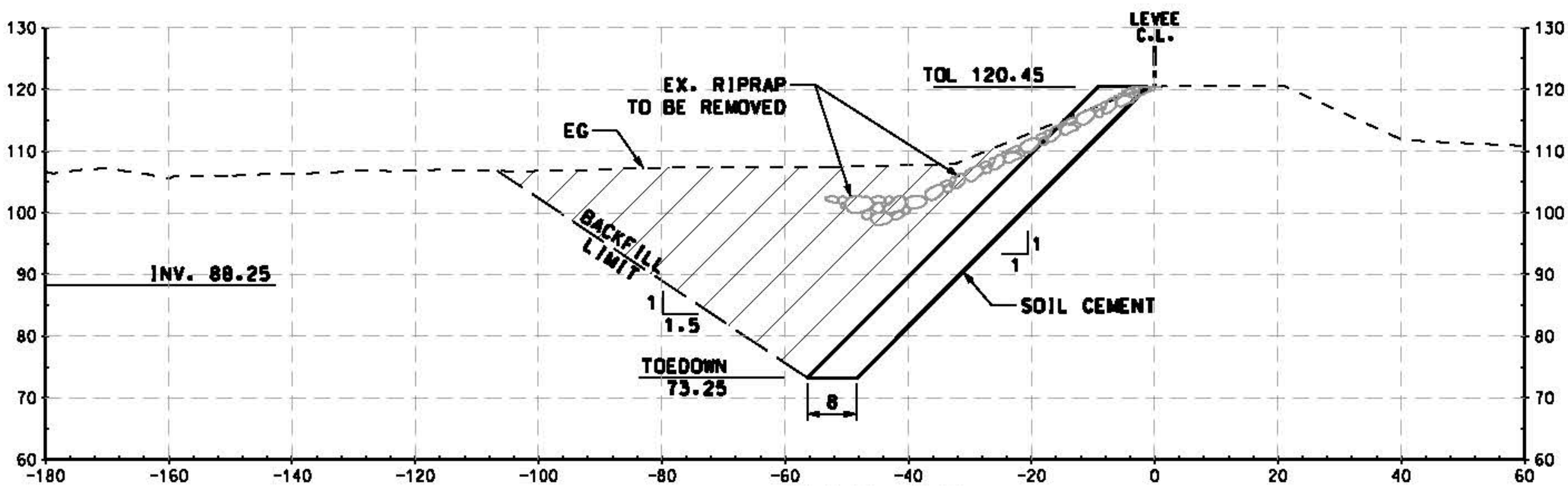


Figure 22 – Typical Cross Section of Measure 13

5.2 Overtopping with Breach

The levee "overtopping with breach" failure scenario (Failure 4) is associated with significant damages and at-risk population. Two of the potential measures applicable to the "overtopping with breach failure" are identified (Table 23). Many of the measures associated with the levee "breach prior to overtopping" failure mode (Failure 1) are also appropriate for risk reduction for the "overtopping with breach" failure mode as well; however, they are described in Section 5.1 and are not repeated here.

Table 23 – Costs for Measures 14 and 15

Measure No.	Description of Measure	Total Construction Cost	Expected Annual Damages
14	Landside Protection	$3,281,000	$971,000
15	Levee Raising	$1,181,000	$971,000

Measure 14: Landside Protection. In recognition that the Failure 4 reach is the most likely location for the levee to be overtopped, a superior levee section would be built. This section would include hardening of the landside slope in order to safely convey overtopping flows. This would not prevent flooding from the adjacent landside area but it would limit the flooding to the amount caused by the overtopping flows rather than the much larger flows associated with a breach of the levee due to overtopping. Table 23 provides the cost associated with this measure, as well as the expected annual damages. Figure 23 shows the conceptual cross section of this measure.

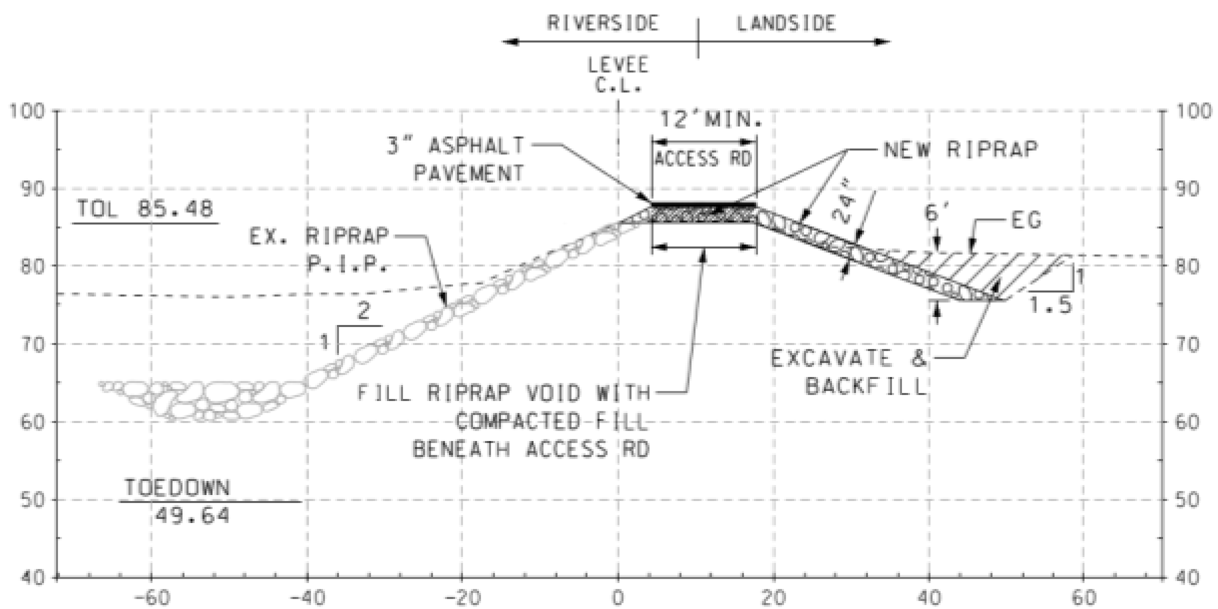


Figure 23 – Typical Cross Section of Measure 14

This measure would not be cost-effective considering the expected annual damages. In addition, the level of protection currently provided in this reach (protection for greater than the 100-year event) is commensurate with the level of protection provided regionally by other flood control facilities.

Measure 15: Levee Raising. This measure would raise the levee along this failure reach to provide protection for the 200-year event. The cost associated with this measure is provided in Table 23, and Figure 24 shows the conceptual cross section of this measure.

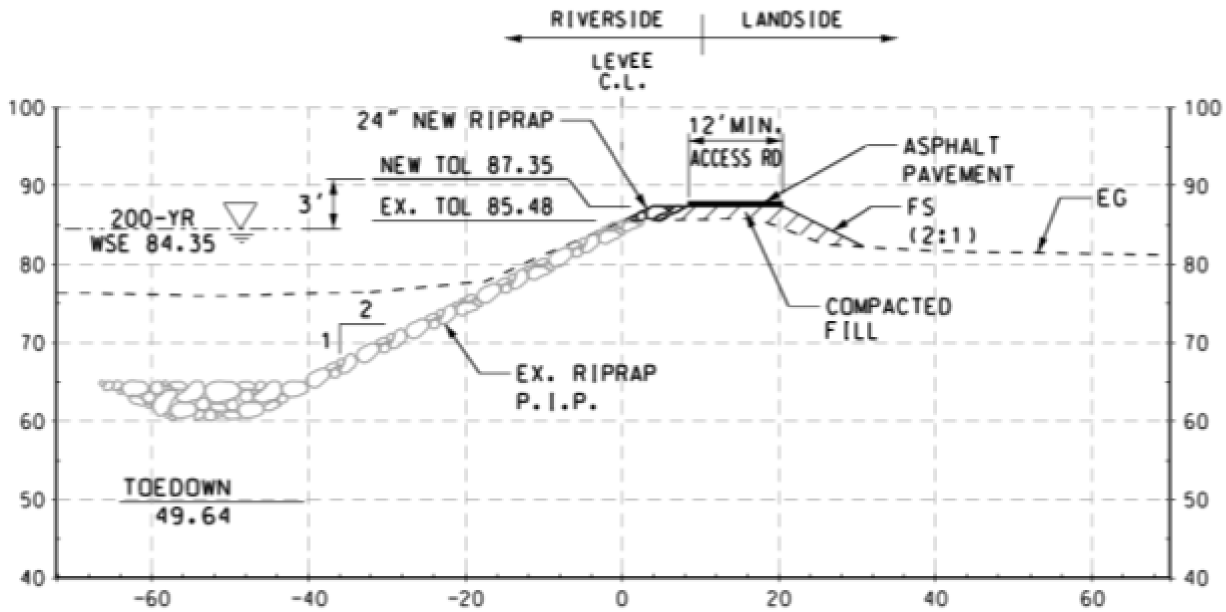


Figure 24 – Typical Cross Section of Measure 15

This measure would not be cost-effective considering the expected annual damages. In addition, the level of protection currently provided in this reach (protection for greater than the 100-year event) is commensurate with the level of protection provided regionally by other flood control facilities.

5.3 Malfunction of Levee System Components

The "malfunction of levee system components" failure scenarios (Failures 5 and 6) consist of failures of the storm drains and the sluice gate structure. The expected annual damages would be low, as described in Section 4. Because of the minor consequences, no additional structural measures were identified. Many of the measures identified for Failure 1 would also provide risk reduction for Failures 5 and 6.

6.0 PRIORITIZATION OF IRRMS

Engineering and Construction Bulletin No. 2012-1 (USACE 2012) provides a series of factors to be considered when determining appropriate IRRMs:

- **Factor a (Reduce Inundation).** How will the probability of inundation of the leveed area and the associated consequences be reduced using structural and nonstructural measures?

- **Factor b (No Life Safety Risk Increase).** Is it ensured that implementation of the IRRM will not increase the life safety risks at any time?
- **Factor c (No Adverse Impacts).** Have the IRRMs been assessed for potential adverse impacts on the levee system, including the areas upstream and downstream of the levee system?
- **Factor d (Detection Confidence).** Does the IRRM increase the level of confidence that any changes associated with the levee safety issue (failure mode) will be promptly detected?
- **Factor e (Notification Confidence).** Does the IRRM increase the confidence that emergency management agencies will be notified promptly when a levee safety issue is detected?
- **Factor f (Warning Time and Evacuation).** Does the IRRM increase the warning time and effectiveness of evacuation of the population at risk?
- **Factor g (Reduce Loading).** Does the IRRM reduce the probability of the initiating load? For example, if adjusting an upstream reservoir water control plan, the consequences of changing the reservoir water control plan should be discussed.
- **Factor h (Public Trust).** Does the IRRM preserve the public trust and address stakeholder issues? (For this evaluation, it is assumed that public trust is focused on the long-term goal of reduced risk to property and life safety).
- **Factor i (Problem Understanding)**. Is the degree of confidence or uncertainty in understanding the scope of the problem and effectiveness of the interim solution improved?
- **Factor j (Permanent Solution).** Can the IRRM be incorporated into permanent solutions?
- **Factor k (Other Impacts).** Are there impacts on the authorized project purposes or other project benefits?
- **Factor l (Cost Effectiveness).** Do the recommended IRRMs maximize cost-effectiveness?
- **Factor m (Social/Environmental Impacts).** Have the recommended IRRMs considered any social disruption and environmental impacts?

The ability of each considered IRRM to address Factors a through m was evaluated in terms of three qualitative ratings (Table 24):

● Factor is well addressed by this IRRM.
◒ Factor is marginally addressed by this IRRM.
○ Factor is not addressed by this IRRM.

The costs from Section 5 are also included in the table for reference. Blue highlighting indicates a recommended measure based on the number of factors addressed. Light yellow highlighting indicates a measure recommended due to one or more major factors. These major factors are individually highlighted in dark yellow.

Table 24 – Assessment Matrix for Recommended IRRMs

Measure	a. Reduce Inundation	b. No Life Safety Risk Increase	c. No adverse Impacts to Levee	d. Detection Confidence	e. Notification Confidence	f. Warning Time & Evacuation	g. Reduce Loading	h. Public Trust	i. Problem Understanding	j. Permanent Solutions	k. Other Project Impacts	l. Cost Effectiveness	m. Social / Environmental Impacts	CONCEPTUAL COST OF MEASURE
1. FWEEP	○	●	●	●	●	●	○	◒	○	●	●	●	●	$50,000
2. Update Hydrologic/Hydraulic Study	○	●	●	○	○	○	○	◒	●	●	●	●	●	$15,000
3. Update Scour Study	◒	●	●	○	○	○	○	◒	●	●	●	●	●	$80,000
4. Develop Local NLD	◒	●	●	◒	◒	○	○	◒	◒	●	●	●	●	$20,000
5. GIS Performance Database	◒	●	●	◒	◒	○	○	◒	◒	●	●	●	●	$10,000
6. Develop Supplemental O&M Manual	○	●	●	●	●	●	○	◒	○	●	●	●	●	$25,000
7. Visual Markers	◒	●	●	●	●	●	○	◒	◒	○	●	●	●	$50,000
8. Stockpile Materials	●	●	●	○	○	●	○	◒	○	◒	●	◒	◒	$600,000
9. Diversion to Gravel Pit	●	◒	●	○	○	●	●	○	○	○	◒	○	◒	**$1,500,000**
10. Diversion to Upstream Reservoir	●	◒	●	○	○	●	●	○	○	○	◒	○	◒	**NA**
11. Launchable Rock	●	●	●	◒	○	●	○	●	○	◒	●	○	○	**$3,300,000**
12. Improve Groins	●	●	●	○	○	●	◒	●	○	◒	●	○	○	**$4,300,000**
13. Full Bank Protection	●	●	●	○	○	○	○	●	○	●	●	○	○	**$5,000,000**
14. Landside Protection	●	●	●	○	○	○	○	●	○	○	●	○	●	**$3,300,000**
15. Levee Raising	●	●	●	○	○	○	○	●	◒	●	●	○	○	**$1,200,000**

LEGEND:
- ● **Factor is well addressed by this measure**
- ◒ **Factor is marginally addressed by this measure**
- ○ **Factor is not addressed by this measure**

Initial measures selected based on number of factors addressed

Additional measures selected based on major factors needing additional support.
Major factors shown as: ●

7.0 RECOMMENDED IRRMS

As shown in Table 24, the following measures were selected for implementation:

1. **Develop a FWEEP.** This measure, along with the Supplemental O&M Manual and Visual Markers, addresses the greatest number of factors considered in the prioritization. It is a cost-effective measure relative to the costs of other measures.

2. **Update Hydrologic/Hydraulic Studies.** This measure was selected primarily because it supports the development of the permanent solution.

3. **Update Scour Study.** This measure was selected primarily because it supports the development of the permanent solution.

4. **Develop a Local NLD.** This measure was selected primarily because of its support of the permanent solution. It will provide a sustainable process for documenting and storing relevant information for the SCR-1 system.

5. **Develop a GIS Performance Database.** This measure will complement the Local NLD measure and is intended to be a GIS layer in the Local NLD. It will provide a sustainable method for evaluating and retaining information related to the system performance over time.

6. **Develop a Supplemental O&M Manual.** This measure, along with the FWEEP and Visual Markers, addresses the greatest number of factors considered in the prioritization. It is a cost-effective measure that will allow the documentation of experience that has been gained with the system (both in the maintenance and the performance).

7. **Visual Markers.** This measure, along with the FWEEP and Supplemental O&M Manual, addresses the greatest number of factors considered in the prioritization. This measure supports a reduction in the risk of inundation by providing a process for assessing any scour during a storm event that could lead to a levee breach. This is a critical factor, and this is the most cost-effective measure identified to support this factor.

8. **Stockpile Materials.** After Visual Markers, this is one of the more cost-effective measures identified to support a reduction in inundation risk. This measure supports this factor by providing a process in which any potential bank failure can be effectively addressed during a storm event by providing flood fighting materials near the site.

The measures that were not recommended for implementation were eliminated on the basis of the results of the evaluation of the risk factors and costs/complexity to implement (Table 24). Section 5 discusses these measures and provides more details about their drawbacks.

8.0 SCHEDULE AND COST OF IRRM RECOMMENDATIONS

The estimated costs for each IRRM is shown in Table 24 and the anticipated schedule for implementation of the selected measures is presented in Attachment 2.

8.1 Develop a FWEEP

The FWEEP has been developed and provided to VCWPD for implementation (Tetra Tech 2015b). No risk assessment or environmental assessment is appropriate for this measure.

8.2 Update Hydrologic/Hydraulic Studies

The hydrologic/hydraulic study update has been prepared and provided to VCWPD (Tetra Tech 2015a). No risk assessment or environmental assessment is appropriate for this measure.

8.3 Update Scour Study

The scour study update has been prepared and provided to VCWPD (Tetra Tech 2015a). No risk assessment or environmental assessment is appropriate for this measure.

8.4 Develop a Local NLD

This measure is currently being developed by VCWPD. Tetra Tech staff met with VCWPD staff to go over the measure and provided recommendations (Attachment 4). This measure will be implemented concurrent with the GIS Performance Database and is discussed as part of that measure. No risk assessment or environmental assessment is appropriate for this measure.

8.5 Develop a GIS Performance Database

This measure is currently being developed by VCWPD. Tetra Tech staff met with VCWPD staff to go over the measure and provided recommendations (Attachment 4). The database including the Local NLD information will be developed in 2015 using X funds. It is anticipated that this database will be active and in use for the 2016 storm season. This is considered a pilot study and will be limited to the SCR-1 levee only. Depending on the value added by this tool, VCWPD will consider future expansion to include other flood facilities. No risk assessment or environmental assessment is appropriate for this measure.

8.6 Develop a Supplemental O&M Manual

The supplemental O&M manual has been prepared and provided to VCWPD (Attachment 5). Along with the GIS Performance Database, this measure will be implemented for the 2016 storm season. No risk assessment or environmental assessment is appropriate for this measure.

8.7 Visual Markers

This measure is currently being developed by VCWPD. Tetra Tech staff met with VCWPD staff to go over the measure and provided recommendations (Attachment 6). Visual markers will be installed using X funds prior to the start of the 2016 storm season. The potential environmental issue anticipated is interference with established least Bell's vireo breeding territories – an endangered bird that has been identified as a resident of this area of the river. To avoid any bird conflicts, the markers will be installed between September 15th and March 1st. This is outside of the bird breeding season. No risk assessments are appropriate for this measure.

8.8 Stockpile Materials

This measure is currently being developed by VCWPD. Tetra Tech provided recommendations on stockpile materials (Attachment 7). Materials to address the potential failure scenarios will be stockpiled in Site 3 and Site 5. The preparation of these sites is expected to be completed prior to the start of the 2016 storm season. In addition, materials will be purchased and stockpiled prior to the start of the 2016 storm season and will use X funds. Because the areas are owned by the County, are outside of the riparian corridor of the river, and (for Site 5) currently being used for this purpose there is no environmental or risk assessment needed.

9.0 REFERENCES

Aqua Terra. 2009. *Hydrologic Modeling of the Santa Clara River Watershed with the U.S. EPA Hydrologic Simulation Program – FORTRAN (HSPF)*. Mountain View, California.

California Department of Water Resources, Division of Safety of Dams. 2010. Dams within the Jurisdiction of the State of California. <http://www.water.ca.gov/damsafety/docs/Juris(N-S)2010.pdf>

FEMA (Federal Emergency Management Agency). 2010. Flood Insurance Study, Ventura County, California, and Incorporated Areas. Alexandria, Virginia. January 20.

FEMA (Federal Emergency Management Agency). 2013. *Analysis and Mapping Procedures for Non-Accredited Levee Systems.* July 2013.

Tetra Tech (Tetra Tech, Inc.). 2015a. Hydrology and Hydraulic Report, Santa Clara River Levee (SCR-1), Evaluation and Rehabilitation Study, Ventura County, California. Prepared for Ventura County Watershed Protection District, by Tetra Tech, Inc., Irvine, California. January.

Tetra Tech (Tetra Tech, Inc.). 2015b. Flood Warning and Emergency Evacuation Plan, Santa Clara River Levee (SCR-1), Ventura County, California. Prepared for Ventura County Watershed Protection District, by Tetra Tech, Inc., Irvine, California. February.

USACE (U.S. Army Corps of Engineers). 1958a. *Santa Clara River Levee, Design Memorandum No. 1 (Hydrology)*. Los Angeles District, California.

USACE. 1958b. *Santa Clara River Basin, California: Santa Clara River Levee, Design Memorandum No. 2 (General Design)*. Los Angeles District, California.

USACE. 1961. Santa Clara River Levee Plans for Construction of Levee and Appurtenances, Santa Clara River Basin, California. Los Angeles District, California.

USACE. 1963. Operation and Maintenance Manual for Santa Clara River Levee. Los Angeles District, California.

USACE. 1969. *Floods in Southern California during January and February, 1969*. Los Angeles District, California.

USACE. 1971. *Santa Clara River Levee, Levee and Channel Restoration Project*. Los Angeles District, California.

USACE. 1994. *Hydraulic* Design *of Flood Control Channels*. Engineer Manual 1110-2-1601. Original publication July 1, 1991; revised June 30, 1994.

USACE. 2011. *Santa Clara River 1 Levee System, Periodic Inspection Report No. 1, August 2011*. Los Angeles District, California.

USACE. 2012. *Engineering and* Construction *Bulletin No. 2012-1, CECW-CE. Expires 20 January 2014.* USACE. 2012. Santa Clara River Watershed Feasibility Study. Prepared for U.S. Army Corps of Engineers, Los Angeles District, California, by CDM-Smith. Irvine, California. October.

VCWPD (Ventura County Watershed Protection District). 2009. *FEMA PAL Response Report, Santa Clara River Levee (SCR-1), Highway 101 to Saticoy*. California.

VCWPD, Los Angeles County Department of Public Works (LCDPW), and U.S. Army Corps of Engineers (USACE). 2011. *Report Addendum Final: Hydrologic Modeling of the Santa Clara River with U.S. EPA Hydrologic simulation Program – FORTRAN (HSPF)*. California.

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ATTACHMENT 1

HEC-RAS Modeled Floodplain Boundary, and Potential Failure Features

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ATTACHMENT 2

Implementation of Measures
Tentative Project Schedule

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ATTACHMENT 3

FWEEP
(under separate cover)

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ATTACHMENT 4

Memorandum – Measures 4 & 5
Local Levee Database

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ATTACHMENT 5

Memorandum – Measure 6 O&M Supplement

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ATTACHMENT 6

Memorandum – Measure 7 Visual Markers

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ATTACHMENT 7

Memorandum – Measure 8 Stockpile Materials

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