# Appendix A
# Independent Technical Review

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<table>
<tr><td colspan="2">Technical Review Comments</td><td colspan="2">Project: Ventura River (VR-1) LEVEE</td><td colspan="2">Location: Ventura CA</td></tr>
<tr><td colspan="2">Date: Sept. 9, 2011</td><td>Reviewer: Mike Zeller</td><td>Tel: 520-623-7980</td><td rowspan="2">Action Taken by: Chung Cheng Yen</td><td rowspan="2">Back Check By: (initials)</td></tr>
<tr><td colspan="2"><u>Office</u><br>Irvine, CA</td><td><u>Type of Document</u><br>Report</td><td><u>Discipline</u><br>Hydrology</td></tr>
<tr><td>Item No.</td><td>Section/Page</td><td colspan="2">COMMENTS</td><td>Action Taken:</td><td>By:</td></tr>
<tr><td colspan="6">GENERAL</td></tr>
<tr><td>1</td><td></td><td colspan="2">Detail editorial comments are provided in the document in addition to ones listed here.</td><td>Revised report accordingly.</td><td>MEZ</td></tr>
<tr><td colspan="6">SPECIFIC</td></tr>
<tr><td>1</td><td>Table 5</td><td colspan="2">I do not know why the “blue shading” remains, but I recommend removing it from the table.</td><td>Revised table format</td><td>MEZ</td></tr>
<tr><td>2</td><td>Table 8</td><td colspan="2">I am not sure what value the “Ratios of Peak Flows” in Table 8 provides, relative to the results of this investigation.</td><td>The ratios were used to estimate the flows at locations downstream of the stream gage</td><td>MEZ</td></tr>
</table>

| Technical Review Comments | | Project: Ventura River (VR-1) LEVEE | | Location: Ventura CA |
|---|---|---|---|---|
| Date: Sept. 9, 2011 | | Reviewer: Mike Zeller | Tel: 520-623-7980 | |
| Office<br>Irvine, CA | | Type of Document<br>Report | Discipline<br>Hydraulics | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |

| Item No. | Section/Page | COMMENTS | Action Taken: | By: |
|---|---|---|---|---|
| **GENERAL** | | | | |
| 1 | | Detail editorial comments are provided in the document in addition to ones listed here. | Revised report accordingly. | MEZ |
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| **SPECIFIC** | | | | |
| 1 | 3.2/4 | Why would more obstruction be added to a 250 square-foot, or larger, opening than to a 100 to 250 square-foot opening? This seems backwards to me. If anything, debris potential would be worse at a smaller opening on the same fluvial system. | This is a direct quotation from FEMA FIS report. | MEZ |
| 2/3 | 3.5/5 | Is this meant to be here? Is it correct? If so, why is it yet to be determined? That is, why has it not already been done—has the modeling not yet been completed? | This sentence had been removed and incorporated in the scour analysis section | MEZ |
| 4 | 4/5 | Is this blank because the modeling described in Section 3.5 of this report has yet to be completed? | Alternatives will be addressed in the Basis of Design Report | MEZ |
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| Technical Review Comments | | Project: Ventura River (VR-1) LEVEE | | Location: Ventura CA |
|---|---|---|---|---|
| Date: Oct. 5, 2011 | Reviewer: Mike Zeller | Tel: 520-623-7980 | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |
| **Office** Irvine, CA | **Type of Document** Report | **Discipline** Hydraulics/Scour Analysis | | |

| Item No. | Section/ Page | COMMENTS | Action Taken: | By: |
|---|---|---|---|---|
| **GENERAL** | | | | |
| 1 | | Further comments are provided in the document in addition to ones listed here. | Revised report accordingly. | MEZ |
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| **SPECIFIC** | | | | |
| 1 | Scour Calculations /5 | Using $S_e$ = 0.0055 and $R_h$ = 11.11, I get n = 0.029. This n-value is somewhat consistent with my findings in **Comment MEZ2**. | Revised calculations by using smallest $D_{50}$. | MEZ |
| 2 | Scour Calculations /5 | Why did you decide to adopt a $D_{50}$ = 2 mm? The data indicate that the smallest $D_{50}$ =46.2 mm and that the largest $D_{50}$ = 121.2 mm. These data yield a range of $n_g$ from 0.029 to 0.034. | Revised calculations by using smallest $D_{50}$. | MEZ |
| 3 | Scour Calculations /6 | Why do you use 1.65 here for relative submerged density and 1.92 below? Should ne the same if sediments are the same. | Revised calculations by using 1.65. | MEZ |
| 4 | Scour Calculations /6 | Make sure this calculation is correct by using a relative submerged density of 1.65. | Revised calculations by using 1.65. | MEZ |
| 5 | Scour Calculations /6 | Do you mean hydraulic <u>depth</u> or hydraulic <u>radius</u> here? Maybe the two are nearly the same along the study reach? | Hydraulic depth was used. | MEZ |
| 6 | Table 5/9 | I used the values for a straight reach in Table 5 and Table 6 because bend scour (curvature) was included as a separate component later on in the document. | Accepted changes | MEZ |
| 7 | Scour Calculations /9 | Later, in Table 7A and Table 7B, you state that average flow velocity is 13.83 ft/sec and mean flow depth is 14.38 ft, which is a unit discharge of 198.88 cfs/ft, not 70.57 cfs/ft as was indicated in Table 6. Accordingly, I changed the value in Table 6 to 198.88 cfs/ft to be consistent with Table 7A and Table 7B data. | Accepted changes | MEZ |

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| **Office** Irvine, CA | **Type of Document** Report | **Discipline** Hydraulics/Scour Analysis | | |

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| 8 | Table 7/10 | Previously, you state that the velocity and hydraulic depth are 13.83 ft/sec and 11.11 feet. Now, in Table 7, you state that the velocity is 13.83 ft/sec and the mean depth is 14.38 feet. I assume that the depths are being used for different purposes in these two instances, correct? | Revised calculations to be consistent throughout the analysis | MEZ |

| Technical Review Comments | | Project: Ventura River (VR-1) LEVEE | | Location: Ventura CA |
|---|---|---|---|---|
| Date: Oct. 26, 2011 | Reviewer: Mike E. Zeller | Tel: 520-618-2444 | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |
| **Office:** Irvine, CA | **Type of Document** Report | **Discipline** Hydraulics/Scour Analysis | | |

| Item No. | Section/Page | COMMENTS | Action Taken: | By: |
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| **GENERAL** | | | | |
| 1 | | Further comments are provided in the document in addition to ones listed here. | Revised report accordingly. | MEZ |
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| **SPECIFIC** | | | | |
| 1 | Scour Calculations /10 | Use the max unit discharge (max velocity times max depth). | Revised calculations accordingly. | MEZ |
| 2 | Scour Calculations /14 | This process, since it is long term, would be classified as degradation, rather than scour (which is typically associated with a single flow event). | Acknowledged. | MEZ |
| 3 | Table 12/17 | Use the mean value here, not the maximum value. | Accepted changes | MEZ |
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| Technical Review Comments | | Project: Ventura River (VR-1) LEVEE | | Location Ventura CA |
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| Date: Nov. 1, 2011 | Reviewer: Mike Zeller | Tel: 520-618-2444 | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |
| **Office**<br>Irvine, CA | **Type of Document**<br>Report | **Discipline**<br>Hydraulics/Scour Analysis | | |

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| **GENERAL** | | | | |
| 1 | | Further comments are provided in the document in addition to ones listed here. | Revised report accordingly. | MEZ |
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| Date: Oct. 19, 2011 | Reviewer: Mike E. Zeller | Tel: 520-623-7980 | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |
| **Office:** Irvine, CA | **Type of Document** Report | **Discipline** Hydraulics/Risk and Uncertainty | | |
| Item No. | Section/Page | **COMMENTS** | Action Taken: | By: |
| **GENERAL** | | | | |
| 1 | | Detail editorial comments are provided in the document. | Revised report accordingly. | MEZ |
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| Technical Review Comments | | Project: Ventura River (VR-1) LEVEE | | Location: Ventura CA |
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| Date: Nov. 2, 2011 | Reviewer: Mike Zeller | Tel: 520-623-7980 | Action Taken by: Chung Cheng Yen | Back Check By: (initials) |
| **Office**<br>Irvine, CA | **Type of Document**<br>Report | **Discipline**<br>Hydraulics/Risk and Uncertainty | | |

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| 1 | | Further comments are provided in the document in addition to ones listed here. | Revised report accordingly. | MEZ |
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| **SPECIFIC** | | | | |
| 1 | Methodology /2 | Do you mean Conditional Non-Exceedance Probability? Please clarify and correct, if necessary. | Yes, revised accordingly. | MEZ |
| 2 | HEC-FDA Model Setup/2 | Why would a record length of only 10 years, which is the minimum record length recommended by the Corps, be called "conservative"? It seems to me it that the case would be just the opposite. | Because the short record length will increase the uncertainty in the HEC-FDA model | MEZ |
| 3 | HEC-FDA Model Results/4 | Is this what you mean to say here? 'AEP' | Acknowledged the changes. | MEZ |
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| Technical Review Comments | Project: Ventura River (VR-1) LEVEE | | Location Ventura CA | |
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| Date: October 31, 2011 | Reviewer: Bill Fullerton | Tel: 206-838-6250 | Action taken by: Mike E. Zeller (MEZ), Chung Cheng Yen (CC), Ike Pace (Ike) | Back Check By: (initials) 3/3/2012 |
| **Office:** Irvine, CA | **Type of Document** Report | **Discipline** Hydraulics/Scour | | |

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| **GENERAL** | | | | |
| 1 | | The document needs to have someone technical edit it. I did not thoroughly edit it. | (Ike : Send to technical editor on 11-3-2011) | WTF |
| 2 | | We present quite a few different methods to calculate various components of scour. At the end of the report, we add up three components to develop our long term estimate (Table 13). However, we don't provide our reasoning for including the ones we have or excluding the ones we did. | (MEZ: The different methods basically were used because they follow the procedures that the BOR employed when they performed their scour analyses along the Ventura River in 2008. The scour components that were eventually incorporated either implicitly include the scour methods that were ones excluded; or, they are independent from and additive to the incorporated components that yield total scour results.) | WTF |
| 3 | | Further comments are provided in the document in addition to ones listed here. | (MEZ: See my response comments, therein.) | WTF |
| 4 | | There is a great deal of uncertainty associated with these estimates. This needs to be kept in mind when they are applied and appropriate caveats included. | (MEZ: Are such caveats not something that should come at a later date, when final design parameters are selected? To me, that is the most appropriate time for their inclusion.)<br><br>(CCY: Uncertainties associated with scour estimates and the removal of Matilija Dam were discussed prior to the presentation of Table 16 which recommends the scour depths for VR-1 reach.) | WTF – Yes, that is what the comment states – Keep that in mind "when they are applied". OK WTF |
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| **SPECIFIC** | | | | | |
| 1 | | There seems to be some confusion in the sediment sample numbering. For Samples 2 and 8, the D50s in Table 2 do not agree with Figure 3. In Figure 3, it is sample 2 with the 46mm D50. Sample 8 has a D50 of about 60mm; the values are reversed in the table and the text. Are they wrong on the graph or in the tables and text? It appears these two samples have been swapped at some point. Also, the text calls out sample three being small, but this is not the case based on Table 2. The confusion over the bed material sizes needs to be straightened out since the text reference these samples and many of the calculations used the sediment size data. (Note: I would recommend that if we do further analysis to support design, we should do our own bed material sample collection effort. This would include surface and subsurface samples. The latter should be to a depth on the order of expected scour.) | | **(MEZ: Yes, we are aware of this confusion; and we are working to correct the mislabeling.)** | WTF |
| 2 | | Check the units/equations for the USBR method. In particular, the competent velocities seem way too low for the sizes of sediments involved. 2 fps does not seem like it would move 2" gravel nor 4 fps move 10" cobble, but that is what Tables 9a and 9b indicate. | | **(MEZ: I agree that the values seem low, although they have been checked and verified as consistent with the results using a BOR equation for computing competent velocity as contained in the report titled *Hydrology, Hydraulics, and Sediment Studies for the Meiners Oaks and Live Oak Levees - DRAFT Report* [BOR 2008]. These values for competent velocity are intended to represent the minimum velocity necessary to initiate movement of individual sediment particles, although I agree with Bill That they seem to yield too low of values. We will revisit this matter, using a different equation for competent velocity and calculate new results for inclusion in Table 9 of the scour report.)** | WTF |
| 3 | | Need to check if antidunes will form with the sediment sizes involved. Also, should the entire antidunes height be added in for scour or 50 percent? | | **(MEZ: Given the potential for worst-case hydraulic parameters, the resultant Froude number would be high (close to 1.0) and thus the stream power would be well into upper flow regime. It is believed that, under such conditions, anti-dunes would likely form. Bedform scour was set at 50 percent of the corresponding anti-dune heights.)** | WTF |

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<tr><td>4</td><td></td><td colspan="2">The largest component of scour in the final analysis is bend scour, varying form ~ 60% to 80% of the total scour estimates in Table 13. Based on the description in the text, it appears the geometry for the bend scour was derived from evaluation of remnant low flow channel braids in the aerials. However, the low flow geometry may not be appropriate for conditions at high flow. I suggest discussing this with Mike Harvey and see what he thinks about the use of this information to predict bend scour at high flows. Also, an aerial photo illustrating the determination of the bend scour geometry would be helpful.<br><br>Response to comment is not consistent with our discussion to decrease bend scour in the lower reaches of the study area due to the lower level of braiding. Please revise response and refer to Table 16. (WTF)</td><td>(MEZ: I have personal knowledge of past issues with lateral migration along other watercourses where severe impingement angles at channel banks created by interior “low-flow” channels have developed over time. These severe impingement angles at channel banks have increased scour potential (sometimes) far beyond what was originally contemplated. In my judgment, dependent upon the magnitude of conveyance in the “low-flow” channels during major floods, high-flow conditions may not necessarily negate the severity of the impingement angles to any significant degree. Unfortunately, without knowing what will actually occur in the future during the passage of a major flood, it is my opinion that it is better to be “safe than sorry” when estimating bend scour.)<br><br>(CCY: The historical fluvial geomorphology and the historical streamed profiles were taking into account in recommending the final scour depths for the VR-1 reach. Detailed discussions were presented in the final report prior to the presentation of Table 16.)</td><td>See my Insert in bold under the comment<br>OK WTF</td></tr>
<tr><td>5</td><td></td><td colspan="2">I don’t understand the profile at the bridge, SR33 in Figure 6. D/S of the bridge, the channel has degraded about 14 feet, but U/S it appears to have aggraded a similar amount. Is there a grade control here? Or is there a plotting error? Or am I just misinterpreting the figure (It is difficult to distinguish the 3 broken blue line types.). If there is a big discontinuity in the profile at this location, it should be discussed</td><td>(MEZ: I am in agreement with this…discussion at the bridge profile should be included.)<br><br>(CCY: There exists a culvert that conveys flow from Canada de San Joaquin to Ventura River, added to text and figures)</td><td>WTF</td></tr>
</table>

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| **Date:** October 31, 2011 | | **Reviewer:** Bill Fullerton | **Tel:** 206-838-6250 | **Action taken by: Mike E. Zeller (MEZ), Chung Cheng Yen (CC), Ike Pace (Ike)** | Back Check By: (initials) **3/3/2012** |
| **Office:** Irvine, CA | | **Type of Document** Report | **Discipline** Hydraulics/Scour | | |
| Item No. | Section/Page | **COMMENTS** | | Action Taken: | By: |
| 6 | | I don't follow the discussion or assignment of the equilibrium slope/existing slope for the downstream reach. In some locations in the report it is given a value of 0.003138 and others 0.004. Also, we base a lot of our results on the existing slopes we take from the thalweg profile. However, I look at the profile and there is quite a bit of variation in the bed. It seems the slope in the lower 2 or 3 reaches could vary quite a bit by just using a point one station U/S or D/S for the calculation. It would be good to show the profile delineated so the reader could actually see what locations were used for the slope calculations. | | **(MEZ: I believe that CC has delineated such a profile…it is included as Figure 6 in the scour report. Does Bill want more information included than what is depicted in this figure?)**<br><br>**(CCY: Figure has been included in the text.)** | WTF |
| 7 | | I edited the discussion on the USBR assumptions for sediment supply post Matilija Dam removal. Some of the wording was unnecessarily provocative. | | **(CCY: Revised the text.)** | WTF |

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<tr><td>8</td><td></td><td colspan="2">We indicate that there are more factors contributing to sediment reduction in the watershed than Matilija Dam. We give a rough estimate that 55% of the degradation that would be predicted w/out Matilija Dam removal is appropriate for the condition with the dam removed. We say the 55% is based on there still being other factors (dams, diversion, debris basins, urbanization) that reduce the sediment load. However, there is no further discussion of what % reduction the other factors may account for. How did we come up with the 55% value? Did we look at the areas controlled by other man made features? If this is the case, a table that identifies watershed areas above features and the area that has been urbanized would be useful. Also, I would be cautious attributing much sediment reduction to Robles Dam. This is a diversion and I imagine it has minimal storage volume. We should also check if it has features to pass/flush sediments such as sluice gates. Similarly, is most of the urbanized area in the low lying and flatter area of the watershed? If so, I would imagine this was not the high sediment production zone compared to the steep mountainous areas? So again, this influence on sediment supply may be overstated if we are just looking at the % of the watershed that has been urbanized.<br><br>Setting a value of 55% implies a level of accuracy not consistent with “engineering judgment. Suggest a 50% value (WTF)</td><td>(MEZ: The report states that a remaining issue is the timing of the sediment replenishment in the lower portions of the Ventura River after removal of the Matilija Dam. If a large flood were to occur immediately after dam removal, the sediment wave would not progress downstream as fast as the flood, and downstream degradation (within the VR-1 Levee reach) potentially would still be large---at least temporarily (and how long is temporarily?) until the sediment wave makes its way down to the VR-1 reach. Thus, engineering judgment was used to select an “in-between” value (55%) for long-term degradation—a value that includes consideration for the timing of the progression of a sediment wave in a downstream direction, as well as other manmade factors that continue to reduce watershed sediment supply (notwithstanding the complete removal of Matilija Dam). By the way, sediment reduction due to Robles Dam was given no special weight; it is just considered to be one of the many manmade features (other than Matilija Dam) that continue to have the potential to disrupt system sediment continuity, even if such disruption is limited from this specific feature.)<br><br>(CCY: After the initial response discussions with WTF, the 50% was used in the final report.)</td><td>See suggestion in bold below the comment<br>OK WTF</td></tr>
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| 9 | | One factor that could cause degradation that we don't discuss is the levees themselves. The levees can concentrate the flows and energy, resulting in higher sediment transport capcity and long term degradation. This is dependent on what flows the levees start providing confinement and how much the sediment transport capacity is increased. This would be something to consider in the next level of analysis. | **(MEZ: Currently, throughout the VR-1 Levee reach the location of the levee is outside of the primary areas of flow conveyance and the top of the levee is well above most WSELs that would occur during flows on the Ventura River. At one time, when first constructed, the confinement of flow by the levee system may have had the potential to contribute to channel degradation during major floods; but at the present time, I believe not so much because the severity of past streambed degradation has lowered the channel thalweg to a significant extent and has correspondingly increased the hydraulic conveyance capacity of the principal channel system.)** | WTF |
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| Technical Review Comments | Project: Ventura River (VR-1) LEVEE | | | Location: Ventura, CA | |
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| Date: March 5, 2012 | Reviewer: Yenhsu Chen | Tel: 949-809-5000 | | Action taken by: Jung Suh | Back Check By: (initials) 3/6/2012 |
| Office: Irvine, CA | Type of Document Alternative Plans (Drawings) | Discipline Design | | | |

| Item No. | Section/Page | COMMENTS | Action Taken: | By: |
|---|---|---|---|---|
| **GENERAL** | | | | |
| | General | See redlines on the Plans set. | Redlines on the plans were addressed. | YHC |
| | Fig 4.5 | Does the toedown depth include the computed scour? Provide a column to show the toedown depth. | Toedown depth is based on the computed scour at the structure. A column for doedown depth is added to Table 4.2. | YHC |
| **SPECIFIC** | | | | |
| | Sht 1 | Change VR-1 to "River Mouth to Canada San Joaquin" | Concur. Change is made. | YHC |
| | Sht 2 | Change "Sta 21+22.5" to "Sta 21+28.5" | Concur. Change is made. | YHC |
| | Sht 3 | The plans called out the removal of 310' for a private retaining wall and repair the correspondent embankment. Are we going outside of RW? | The retaining wall threatens the integrity of levee and needs to be removed. In the construction level design, necessary easement would be acquired. | YHC |
| | Sht 3 | Sta 45 to Sta 47 – It seems two unpermitted structures is the area. What to do? | The unpermitted structures would be removed (Construction Note 5). | YHC |
| | Sht 7 | Sta 127 to Sta 128 – It appears something is on the landside toe. Verify? | It is an existing SD inlet structure and needs to be protected in place. | YHC |
| | Sht 8 | Show levee raising from Sta 139 to Sta 142 on plan. | Concur. It is levee raising by addition of concrete floodwall. Change is made. | YHC |
| | Sht 8 | See redlines for "Vegetation Removal" | Concur. Changes are made per the redlines. | YHC |
| | Sht 8 | It seems unpermitted structures located in Sta 144 to Sta 145. Verify. | Construction Note 5 is added for removal. | YHC |
| | Sht 8 | Complete the RW closure line at the upstream end. | Concur. Change is made. | YHC |
| | Sht 9 | Revise "rock" to "riprap" in this sheet. | Concur. Change is made. | YHC |
| | Sht 9 | Show the computed scour limit downstream of each grade control structure | A note stating the local scour limit by the GCS for the GCS alternative is not shown for clarity. | YHC |
| | Sht 10 | Revise "rock" to "riprap" | Concur. Change is made. | YHC |
| | Sht 10 | Show the computed scour limit downstream of each grade control structure | A note stating the local scour limit by the GCS for the GCS alternative is not shown for clarity. | YHC |
| | Sht 10 | Change from "Sta 88+00 to 150+00" to "Sta 88+00 to 149+00" | Concur. Change is made. | YHC |
| | Sht 11 | Change "rock" to "riprap" | Concur. Change is made. | YHC |

# Appendix B
# Hydrology

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VENTURA RIVER LEVEE EVALUATION AND REHABILITATION
Feasibility Study

# 1. INTRODUCTION

The Ventura River Levee (VCWPD ID No. VR-1) is located in the city of San Buenaventura in Ventura County, California. The levee system extends from the Pacific Ocean to the Cañada de San Joaquin (Figure 1). The VR-1 system is located along the left side of the Ventura River. The levee system consists of embankment levees, side drainage penetrations, and a stop-log structure in the levee at a bike trail crossing. The levee system is intended to protect existing residential, commercial, industrial, and potentially developable property in low-lying areas within the base flood floodplain of the Ventura River Watershed.

## 1.1 Purpose of Report

This report was prepared in support of the VR-1 reach evaluation and rehabilitation documents that meet the requirements of the U.S. Army Corps of Engineers (the Corps). If the Corps takes the lead on this study, these documents can be used in the future to complete the rehabilitation and certification of the VR-1 system.

## 1.2 History of VR-1 System

The Ventura River Levee was authorized by the Flood Control Act approved December 22, 1944, Public Law 534, 78th Congress, Chapter 665, 2nd Session (H.R. 4485), substantially in accordance with the recommendations of the Chief of Engineers in House Document No. 323, 77th Congress, 1st Session.

The levee system was designed by the Corps and completed on December 31, 1948. The levee protects the western part of the city of Ventura and the suburban area adjacent to the north from a flood of a magnitude of 150,000 cubic feet per second (cfs). This is the maximum peak design flow that is estimated to occur as the result of the more severe regional storm coupled with conditions fairly conductive to runoff,

The levee system begins at the Pacific Ocean in Ventura County and continues upstream to the confluence of the Ventura River with the Cañada de San Joaquin. The length of the levee along the Ventura River is approximately 2.65 miles, with an embankment height up to 10 feet above natural ground on the landward side. The levee's earthen berm is protected by loose riprap and grouted riprap, with an access road along the top that is approximately 18 to 26 feet wide.

## 2. REVIEW OF PREVIOUS HYDROLOGY STUDIES

Reports pertinent to the hydrologic analysis were reviewed and are summarized in the following subsections.

### 2.1 Corps 1941 Report – Preliminary Examination and Survey of Ventura River

Design flood data along the Ventura River from Corps 1941 report (USACE 1941) are summarized in Table 1.

**Table 1 – Summary of Ventura River Design Flood Data**

| Dam Site or Other Concentration Point | Drainage Area (square miles) | Peak Flow (cfs) |
|---|---|---|
| At mouth | 228.0 | 150,000 |
| Foster Park Dam site | 190.0 | 145,000 |
| Above Coyote Creek | 149.0 | 121,000 |
| Above San Antonio Creek | 95.0 | 89,000 |
| Matilija Dam site | 72.0 | 90,000 |

Source: Excerpted from USACE 1941.

The report recommended that a levee on the left bank (looking downstream), extending from high ground below the Ventura Avenue oil field to the mouth of the river, would be adequate to protect the areas in the vicinity of lower Ventura River. It also recommended that a debris basin at the mouth of Stewart Canyon, with a concert channel, would be adequate to protect the city of Ojai (Figure 2).

### 2.2 Corps 1947 Report – Definitive Project Report

This report recommended a discharge of 150,000 cfs for the design of VR-1 from the Pacific Ocean to the Cañada de San Joaquin.

### 2.3 FEMA Flood Insurance Study Report

The current Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) (FEMA 2010) provides the peak discharges at various locations along the Ventura River (Table 2) and includes general discussions of the watershed hydrology. At this time, the exact methodologies of hydrologic and flood-frequency analyses that were used to generate the peak discharges along the Ventura River are unclear.

**Table 2 – Summary of FEMA FIS Ventura River Peak Discharge**

| Flooding Source and Location | Drainage Area (square miles) | Annual Exceedance Probability Discharge[1] (cfs) | | | |
|---|---|---|---|---|---|
| | | 10 Percent | 2 Percent | 1 Percent | 0.2 Percent |
| At mouth at Pacific Ocean | 226.0 | 34,000 | 67,000 | 78,000 | 103,000 |
| At Shell Chemical Plant | 222.0 | 34,000 | 66,000 | 77,000 | 102,000 |
| At Casitas Vista Road | 184.0 | 30,000 | 58,000 | 68,000 | 90,000 |
| At Casitas Springs | 143.0 | 29,000 | 55,000 | 65,000 | 86,000 |
| At Baldwin Road | 81.0 | 16,000 | 31,000 | 36,000 | 48,000 |
| Downstream of confluence of North Fork Matilija Creek | 70.4 | 15,000 | 30,000 | 34,500 | 46,000 |
| Upstream of confluence of North Fork Matilija Creek | 54.3 | 12,000 | 23,500 | 27,500 | 36,500 |

Source: Excerpted from FEMA 2010.
1. Peak discharges are the same as those in the 1997 FIS report (FEMA 1997).

### 2.4 USBR/Corps Feasibility Study

The most recent flood-frequency analysis of the Ventura River was conducted by Bullard (2002a,b) of the U.S. Bureau of Reclamation and presented in Appendix D (USBR 2004b) of the *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report* (USBR 2004a). Specifically, the Bullard's reports (2002a,b) provide detailed documentation of the hydrologic analyses, which meet FEMA requirements.

The following excerpt describes the USBR flood-frequency analyses (Bullard 2002a and 2002b):

*A flood-frequency analysis was performed for the entire length of the Ventura River. Frequency discharges for the 2-, 5-, 10-, 20-, 50-, 100-, and 500-year events were developed. The analysis is detailed in a separate report (Bullard, 2002a). Three stream gage records were used in the initial analysis: Matilija Creek above the Matilija Reservoir (USGS gage 11114500), Matilija Creek at Matilija Hot Springs (USGS gage 11115500) and Ventura River near Ventura (USGS gage 11118500). To determine the selected return period flows, various methodologies were investigated and it was determined that a top-fitting method was most appropriate for the Ventura River. The standard method recommended in Bulletin 17B that uses the Log-Pierson Type III Probability distribution did not fit the data. It is expected that the distribution does not work well in this region of the county because of the peculiarities of the weather patterns. The top fitting method used the 7 largest floods and the frequency of those floods were fit with a regression equation and this regression equation was used to determine the flood magnitudes with a 10-, 20-, 50-, 100- and 500-year return period. To obtain the flood magnitudes with 2- and 5-year return periods, a separate analysis of partial duration series was performed (Bullard, 2002b).*

*Regression equations were fit to the peak flow and return period data using the top 20, the top 10 and the top 7 peaks for each gauge record. ... The decision as to what level of "censoring" to use does not appear to be very sensitive for these data if 10 or fewer events were selected. The difference between fitting the top 10 or the top 7 points will produce only minor differences in the estimated 100-year flood peaks. It was decided to use the top fitting technique with the top 7 peak events or about the top 10 percent of the data. It is noted that the top 7 events extend from 10- to about 70-years based on the Weibull plotting positions. ... The resulting 10-, 50-, 100-, and 500-year peak flow estimates for each site are summarized in following table:*

| *Recommended Peak Flows for the Ventura River at Existing Stream Gage Site (by top-end fitting of peak flow data)* | |
|---|---|
| *Location* | *Ventura River at Ventura, CA (at Casitas Road Bridge – Location of USGS Gage No, 11118500)* |
| *Return Period (Years)* | *Peak Flows (cfs)* |
| *10* | *36,400* |
| *20* | *46,400* |
| *50* | *59,700* |
| *100* | *69,700* |
| *500* | *93,100* |

The USBR report (2004b) compared the recommended peak discharges to the peak flows at the same location provided in the 1997 FEMA FIS. This comparison is presented in Table 3.

**Table 3 – Comparison of Peak Discharges of USBR Analysis and FEMA FIS**

| **Ventura River at Ventura, California (at Casitas Vista Road Bridge – Location of USGS Gage No, 11118500)** | | |
|---|---|---|
| **Return Period (Years)** | **Peak Flows (cfs)** | |
| | **2004 USBR Analysis[1]** | **1997 FEMA FIS[2]** |
| 10 | 36,400 | 30,000 |
| 20 | 46,400 | - |
| 50 | 59,700 | 58,000 |
| 100 | 69,700 | 68,000 |
| 500 | 93,100 | 90,000 |

1. USBR flood-frequency analyses were performed in 2002 and used 68 peak flows from 1933 to 2000 (Bullard, 2002a).
2. Peak discharges remained the same in the 2010 FEMA FIS.

### 2.5 Ventura River Watershed Hydrology Model

Tetra Tech (2008a) developed a hydrologic model of the Ventura River Watershed using the U. S. Environmental Protection Agency's (EPA's) Hydrologic Simulation Program—FORTRAN (HSPF) model (USEPA 2001) for the Ventura County Watershed Protection District. The work resulted in the completion of a baseline (existing-conditions) hydrologic model, as well as a natural-conditions scenario.

The HSPF model of the Ventura River Watershed has been calibrated and validated to continuous-flow gage data from October 1986 through September 2007 (Tetra Tech 2008b). The ability of the model to estimate flood peaks is one area of interest. In order to evaluate this issue further, the model was run for the period of October 1967 through September 2007, providing 40 water years of data for analysis. Long records of observed peaks are available for eight gages and are also limited to the period encompassing water years 1968 through 2007 in order to provide a common basis for comparison.

The U.S. Geological Survey (USGS) PeakFQ program (2007) provides flood-frequency analyses according to Bulletin 17B methodology (U.S Water Resources Council 1981). This analysis was applied to both observed and simulated annual peak series. The results presented in this appendix focus on the flow peak comparison for the 100-year flood event, which is the design flow of interest for the intended floodplain applications in the Ventura River Watershed. The results of the predictive analyses for a 100-year peak flow at USGS Gage No. 1118500 (Figure 3) are presented in Table 4.

**Table 4 – Comparisons of 100-Year Discharge of Observed and HSPF Model Data**

| Location | USGS Gage No. | Number of Observed Peaks | 100-Year Discharge from Observed Peaks (cfs) | Station Skew for Observed Data | 100-Year Discharge from HSPF Peaks (cfs) | Station Skew for HSPF | Assumed Regional Skew |
|---|---|---|---|---|---|---|---|
| At Foster Park | 11118500 | 39 | 183,300 | –0.21 | 125,700 | –0.131 | –0.25 |

## 3. ANALYSIS OF USGS 2011 DATA SET

### 3.1 USGS Stream Gage

The closest USGS stream gage is located at the Casitas Vista Road Bridge, approximately 3.44 miles upstream of the confluence of the Ventura River and the Cañada de San Joaquin, where the VR-1 terminates (Figure 1). The gage for the Ventura River near Ventura (USGS Gage No. 11118500) has been in operation since 1929. The peak-flow data are available for water years 1933 to 2009 (USGS 2011).

General information for the gage site is provided in Table 5.

**Table 5 – USGS Gage No. 11118500 on Ventura River near Ventura**

<table>
<tr><td colspan="4">DESCRIPTION:<br>Latitude 34°21'08",  Longitude 119°18'27" North American Datum of 1927<br>Ventura County, California, Hydrologic Unit 18070101<br>Drainage area:  188 square miles<br>Datum of gage:  200.00 feet above National Geodetic Vertical Datum of 1929<br>AVAILABLE DATA[1]:</td></tr>
<tr><th>Data Type</th><th>Begin Date</th><th>End Date</th><th>Count</th></tr>
<tr><td><u>Real-Time</u></td><td colspan="3">-- Previous 120 days --</td></tr>
<tr><td colspan="4"><u>Daily Data</u></td></tr>
<tr><td>Discharge,<br>cubic feet per second</td><td>1929-10-01</td><td>2011-05-23</td><td>29810</td></tr>
<tr><td colspan="4"><u>Daily Statistics</u></td></tr>
<tr><td>Discharge,<br>cubic feet per second</td><td>1929-10-01</td><td>2009-09-30</td><td>29220</td></tr>
<tr><td colspan="4"><u>Monthly Statistics</u></td></tr>
<tr><td>Discharge,<br>cubic feet per second</td><td>1929-10</td><td>2009-09</td><td></td></tr>
<tr><td colspan="4"><u>Annual Statistics</u></td></tr>
<tr><td>Discharge,<br>cubic feet per second</td><td>1930</td><td>2009</td><td></td></tr>
<tr><td><u>Peak streamflow</u></td><td>1933-01-19</td><td>2009-02-16</td><td>77</td></tr>
</table>

| | | | |
|---|---|---|---|
| **Field measurements** | 1938-02-18 | 2011-05-05 | 471 |
| **Field/Lab water-quality samples** | 1908-11-05 | 2011-04-20 | 417 |
| **Additional Data Sources** | **Begin Date** | **End Date** | **Count** |
| **Instantaneous-Data Archive** (data not available on the website) | 1988-10-01 | 2008-09-30 | 630873 |
| **Annual Water-Data Report (pdf)** (data not available on the website) | 2005 | 2009 | 5 |
| 1. Record for this site is maintained by the USGS California Water Science Center. | | | |

### 3.2 10-Year, 20-Year, 50-Year, 100-Year, and 500-Year Peak Flows

The first effort to create peak flows for various return periods consisted of running the updated peak-flow data sets, available for water years 1933 to 2009, using the Hydrologic Engineering Center's Statistical Software Package (HEC-SSP) (USACE 2009), which incorporated the Water Resources Council Bulletin 17B procedure. The prescribed regional skew value -0.3 was applied. This first effort produced nearly identical return-period peak-discharge results as those presented in the USBR report (2004a). The fitted flood-frequency curve significantly overestimated the largest of the recorded peak flows of events in the last 77 years of data (i.e., 63,600 cfs recorded on February 10, 1978). Therefore, the same top-end-fitting approach described in the USBR 2004 report was applied to the 77-year peak-flow data set. Regression equations were fit to the peak-flow and return-period data using the top 20, top 10, and top 7 peaks, as shown on Figure 4. Within the additional 9 years of data, a peak flow of 41,000 cfs, recorded on January 25, 2005, made the top 7 peaks of the historical peak discharges; and peak flows of 19,100 cfs, recorded on Mach 6, 2001, and 14,400 cfs, recorded on January 27, 2008, made the top 20 peaks of the historical peak discharges. Differences in the estimated peak discharges based on regression equations developed using the top 10 peaks versus the top 7 peaks are minor, as shown in Table 6. Therefore, the regression equation developed using the top 7 peaks was selected, and it agrees with the USBR analyses.

**Table 6 – Peak Discharges of 2011 Peak-Flow Data Set**

| Ventura River at Ventura, California (at Casitas Vista Road Bridge – Location of USGS Gage No, 11118500) | | |
|---|---|---|
| **Return Period (Years)** | **Peak Flows[1] (cfs)** | |
| | **Using Top 7 Peaks** | **Using Top 10 Peaks** |
| 10 | 36,500 | 36,900 |
| 20 | 45,800 | 46,000 |
| 50 | 58,000 | 58,000 |
| 100 | 67,300 | 67,100 |
| 500 | 88,800 | 88,200 |
| 1. Flood-frequency analyses were based on 77 peak flows, from 1933 to 2009. | | |

Estimated peak flows from the 2004 USBR report, the 2010 FEMA FIS, and a 2011 peak-flow data set are summarized in Table 7 and graphically presented in Figure 5. Based on the 2011 peak-flow data set, the estimated peak flows are lower than the 2004 USBR values but closer to the 2010 FIS peak flows for the 50-year, 100-year, and 500-year frequencies. Deviation of the 10-year peak flow remains the same compared to the 2010 FIS value and the top-end-fitting analysis.

**Table 7 – Comparison of Peak Discharges of USGS Gage No. 11118500**

| **Ventura River at Ventura, California (at Casitas Vista Road Bridge – Location of USGS Gage No, 11118500)** | | | |
|---|---|---|---|
| **Return Period (Years)** | **Peak Flows (cfs)** | | |
| | **2004 USBR[1]** | **2010 FEMA FIS** | **2011 Data Set[2]** |
| 10 | 36,400 | 30,000 | 36,500 |
| 20 | 46,400 | Not Available | 45,800 |
| 50 | 59,700 | 58,000 | 58,000 |
| 100 | 69,700 | 68,000 | 67,300 |
| 500 | 93,100 | 90,000 | 88,800 |

1. USBR flood-frequency analyses were performed in 2002, using 68 peak flows from 1933 to 2000 (Bullard 2004a).
2. Flood-frequency analyses were based on 77 peak flows, from 1933 to 2009.

### 3.3 2-Year and 5-Year Peak Flows

Flood magnitudes with 2-year and 5-year return periods were based on a partial-duration analysis (Bullard 2002b). The data used in the partial-duration analysis is for the water years between 1933 and 1959, due to the completion of Casitas Dam in 1959. Therefore, no additional analyses were conducted.

### 3.4 Distribution of Peak Flows along Lower Ventura River

In the USBR report (2004a,b), peak-flow estimates were distributed to other locations along the Ventura River based on ratios of flows provided in the 1997 FEMA FIS. In the 1997 FIS, peak flows are provided for the two gaged sites: one located below Matilija Reservoir (USGS Gage No. 11115500) and the other located on the Ventura River at Ventura, at the Casitas Vista Road Bridge (USGS Gage No. 11118500). Flows for four other sites on the main stem of the Ventura River are also provided. Ratios of the peak flows for the ungaged sites to those for the gage sites were calculated and used to distribute the new peak flows to the ungaged sites listed in the 1997 FIS. For the 20-year event, the 1997 FIS does not provide peak-flow values. The 20-year peak flows in the Ventura River were estimated by taking the average of the ratios for the various locations at the 10-year and the 50-year peak-discharge levels. These assumed ratios were then applied to the calculated 20-year peak flows at the gaged sites. The 2-year and 5-year peak flows were distributed along the river based on the drainage area versus discharge relationship developed in the partial-duration analyses.

Since the 2010 FIS peak flows are essentially identical to the 1997 FIS peak flows, the same ratios were used to calculate the peak flows at other locations (Table 8). Table 9 summarizes the predicted peak flows along the lower Ventura River.

**Table 8 – Ratio of Peak Flows to USGS Gage No. 11118500 (at Casitas Vista Road Bridge)**

| Location | Drainage Area (sq. miles) | Return Period (years) 2010 FIS-Recommended Peak Flows (cfs) | | | | |
|---|---|---|---|---|---|---|
| | | 10 | 20[1] | 50 | 100 | 500 |
| At Casitas Vista Road Bridge | 188 | 30,000 | NA | 58,000 | 68,000 | 90,000 |
| At Shell Chemical Plant | 222 | 34,000 | NA | 66,000 | 77,000 | 102,000 |
| At mouth at Pacific Ocean | 226 | 34,000 | NA | 67,000 | 78,000 | 103,000 |
| | | **Ratios of Peak Flows** | | | | |
| | | 10 | 20[2] | 50 | 100 | 500 |
| At Casitas Vista Road Bridge | 188 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 |
| At Shell Chemical Plant | 222 | 1.1333 | 1.1356 | 1.1379 | 1.1324 | 1.1333 |
| At mouth at Pacific Ocean | 226 | 1.1333 | 1.1443 | 1.1552 | 1.1471 | 1.1444 |

1. Not available from 2010 FIS.
2. Average of 10-year and 50-year ratios

**Table 9 – Summary of Peak Flows along Lower Ventura River**

| Location | Return Period (years) 2010 FIS-Recommended Peak Flows (cfs) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 2[1] | 5[1] | 10 | 20[1] | 50 | 100 | 500 |
| At Casitas Vista Road Bridge | NA | NA | 30,000 | NA | 58,000 | 68,000 | 90,000 |
| At Shell Chemical Plant[2] | NA | NA | 34,000 | NA | 66,000 | 77,000 | 102,000 |
| At mouth at Pacific Ocean[2] | NA | NA | 34,000 | NA | 67,000 | 78,000 | 103,000 |
| | **Return Period (years) 2004 USBR-Recommended Peak Flows (cfs)** | | | | | | |
| | 2[3] | 5[4] | 10 | 20 | 50 | 100 | 500 |
| At Casitas Vista Road Bridge | 4,520 | 11,060 | 36,400 | 46,400 | 59,700 | 69,700 | 93,100 |
| At Shell Chemical Plant[2] | 5,080 | 12,250 | 41,300 | 52,700 | 67,900 | 78,900 | 105,500 |
| At mouth at Pacific Ocean[2] | 5,130 | 12,370 | 41,300 | 53,100 | 69,400 | 81,700 | 110,900 |
| | **Return Period (years) Peak Flows Recommended in 2011 Peak-Flow Data Set (cfs)** | | | | | | |
| | 2[3] | 5[4] | 10 | 20 | 50 | 100 | 500 |
| At Casitas Vista Road Bridge | 4,520 | 11,060 | 36,500 | 45,800 | 58,000 | 67,300 | 88,800 |
| At Shell Chemical Plant[2] | 5,080 | 12,250 | 41,400 | 52,000 | 66,000 | 76,200 | 100,600 |
| At mouth at Pacific Ocean[2] | 5,130 | 12,370 | 41,400 | 52,400 | 67,000 | 77,200 | 101,600 |

1. Not available from 2010 FIS.
2. 10-, 20-, 50-, 100-, and 500-year values were obtained by multiplying the associated factors in Table 8.
3. 2-year regression equation: Peak = (12.074) · Drainage Area + 2402.4.
4. 5-year regression equation: Peak = (30.787) · Drainage Area + 5413.4.

## 4. FEMA FIS HEC-RAS MODEL

The current FEMA FIS Hydrologic Engineering Center's River Analysis System (HEC-RAS) model for the main stem of the Ventura River was prepared by HDR, the FEMA study contractor (HDR 2010). Three cross sections, where flow changed as specified in the HEC-RAS model, are located downstream of the USGS Gage No. 111185000 at the Casitas Vista Road Bridge (Figure 3). Table 10 compares the peaks flows used in the HEC-RAS model to the recommended peak flows in the USBR report (2004a) and to the peak flows generated using the 2011 peak-flow data set.

**Table 10 – Comparison of Peak Flows below USGS Gage No. 11118500**

| HEC-RAS Station | Location | | | |
|---|---|---|---|---|
| **315+73** | **Peak Flows at Casitas Vista Road Bridge (cfs)** | | | |
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| FIS model | 36,400 | 59,700 | 69,700 | 93,100 |
| 2010 FIS report | 30,000 | 58,000 | 68,000 | 90,000 |
| 2004 USBR report | 36,400 | 59,700 | 69,700 | 93,100 |
| 2011 peak-flow data set | 36,500 | 58,000 | 67,300 | 88,800 |
| | | | | |
| **HEC-RAS Station** | **Location** | | | |
| **310+06.64** | **Peak Flows Downstream of Casitas Vista Road Bridge (cfs)** | | | |
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| FIS model | 36,583 | 70,055 | 93,593 | 36,583 |
| 2010 FIS report | NA | NA | NA | NA |
| 2004 USBR report | NA | NA | NA | NA |
| 2011 peak-flow data set | NA | NA | NA | NA |
| | | | | |
| **HEC-RAS Station** | **Location** | | | |
| **240+04.01** | **Peak Flows at Shell Chemical Plant (cfs)** | | | |
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| FIS model | 41,300 | 67,900 | 78,900 | 105,500 |
| 2010 FIS report | 34,000 | 66,000 | 77,000 | 102,000 |
| 2004 USBR report | 41,300 | 67,900 | 78,900 | 105,500 |
| 2011 peak-flow data set | 41,400 | 66,000 | 76,200 | 100,600 |
| | | | | |
| **HEC-RAS Station** | **Location** | | | |
| **158+65.15** | **Peak Flows Downstream of Shell Road (cfs)** | | | |
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| FIS model | 41,438 | 68,126 | 79,166 | 105,500 |
| 2010 FIS report | NA | NA | NA | NA |
| 2004 USBR report | NA | NA | NA | NA |
| 2011 Peak-flow data set | NA | NA | NA | NA |
| | | | | |
| **HEC-RAS Station** | **Location** | | | |
| **0+0** | **Peak Flows at Mouth at Pacific Ocean (cfs)** | | | |
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| FIS model | 41,438 | 68,126 | 79,166 | 105,500 |
| 2010 FIS report | 34,000 | 67,000 | 78,000 | 103,000 |
| 2004 USBR report | 41,300 | 69,400 | 81,700 | 110,900 |
| 2011 peak-flow data set | 41,400 | 67,000 | 77,200 | 101,600 |
| NA = not available | | | | |

The peak flows used in the current FIS HEC-RAS model are higher or compatible with the estimates obtained using the 2011 peak-flow data set. Therefore, the discharges of the current FIS model were retained and used in the hydraulic and risk and uncertainty analyses.

## 5. CAÑADA DE SAN JOAQUIN

The upstream portion of the VR-1 system is located in the Cañada de San Joaquin tributary. The discharge of 2,300 cfs was used in designing this portion of the levee according to the Corps 1947 Definitive Project Report. Peak flows for the Cañada de San Joaquin were also provided by Ventura County (2010) using the EPA's Hydrologic Simulation Program—FORTRAN (HSPF) model (USEPA 2001).

Table 11 provides the recommended peak discharges, which coincide with the FIS HEC-RAS model, to be used in the hydraulic and risk and uncertainty analyses.

**Table 11 – Canada de San Joaquin Peak Flows**

| Location | Return Period | | | |
|---|---|---|---|---|
| | **10-Year** | **50-Year** | **100-Year** | **500-Year** |
| Confluence with Ventura River | 630 cfs | 1,720 cfs | 2,420 cfs | 4,720 cfs |

## 6. REFERENCES


Bullard, K.L. 2002a. *Ventura River Peak Flow Flood Frequency Study for Use with Matilija Dam Ecosystem Restoration Feasibility Study, Ventura County, California*. U.S. Bureau of Reclamation, Technical Service Center, Denver, Colorado.

Bullard, K.L. 2002b. *Ventura River 2- and 5*-year *Flood Peaks and Flow Duration Curves for Use with Matilija Dam Ecosystem Restoration Feasibility Study, Ventura County, California*. U.S. Bureau of Reclamation, Technical Service Center, Denver, Colorado.

FEMA. 1997. *Flood Insurance Study, Ventura County, California*. Federal Emergency Management Agency. Revised September 3, 1997.

FEMA 2010. *Flood Insurance Study, Ventura County, California and Incorporated Areas*. Federal Emergency Management Agency. January 20, 2010.

HDR. 2010. *Hydraulic Modeling for Ventura River, Ventura County, California, FIS*. March 2010.

Tetra Tech. 2008a. *Data Summary Report, Ventura River Watershed Hydrology Model*. Prepared for Ventura County Watershed Protection District by Tetra Tech Inc., San Diego, California.

Tetra Tech. 2008b. *Model Calibration and Validation Report (Draft), Ventura River Watershed Hydrology Model*. Prepared for Ventura County Watershed Protection District by Tetra Tech Inc., San Diego, California.

USACE. 1941. *Preliminary Examination and Survey of Ventura River, Ventura County, California*. U.S. Army Corps of Engineers

USACE. 1947. *Definitive Project Report, Ventura River Improvement, Ventura River Levee, Ocean to Canada de San Joaquin*. U.S. Army Corps of Engineers, Los Angeles District, California.

USACE. 2009. Hydrological Engineering Center, HEC-SSP, Statistical Software Package, Version 1.1. U.S. Army Corps of Engineers. May 5, 2009.

USACE. 2010. Hydrological Engineering Center, HEC-RAS, River Analysis System, Version 4.1.0. U.S. Army Corps of Engineers. January 2010.

USBR. 2004a. *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report.* Main Report. Prepared for the U.S. Army Corps of Engineers, Los Angeles District, California, by the U.S. Bureau of Reclamation.

USBR. 2004b. *Matilija Dam Ecosystem Restoration Feasibility, Ventura County, California, Final Report.* Appendix D – Hydrologic, Hydraulic and Sediment Studies, Ventura River, Ventura County, California. Prepared for the U.S. Army Corps of Engineers, Los Angeles District, California, by the U.S. Bureau of Reclamation.

USEPA. 2001. *Hydrologic Simulation Program—FORTRAN (HSPF) User's Manual.* National Exposure Research Laboratory Office of Research and Development, U.S. Environmental Protection Agency, Athens, Georgia.

USGS. 2011. National Water Information System: Web Interface at http://nwis.waterdata.usgs.gov/ca/nwis/peak?site_no=11118500&agency_cd=USGS&format=html. U.S. Geological Survey, accessed on May 23, 2011.

USGS. 2007. PeakFQ computer program, Annual Flood Frequency Analysis using Bulletin 17B Guidelines, U.S. Geological Survey.

U.S. Water Resources Council. 1981. *Guidelines for Determining Flood Flow Frequency.* Bulletin 17B of the Hydrology Committee.

Ventura County. 2010. *Ventura River Watershed Design Storm Modeling,* Addendum 1, Hydrology Section. Water Resources Technology Division, Ventura County Watershed Protection District, Ventura, California. August 2010.


**Figure 1 --Location Map of VR-1 System**

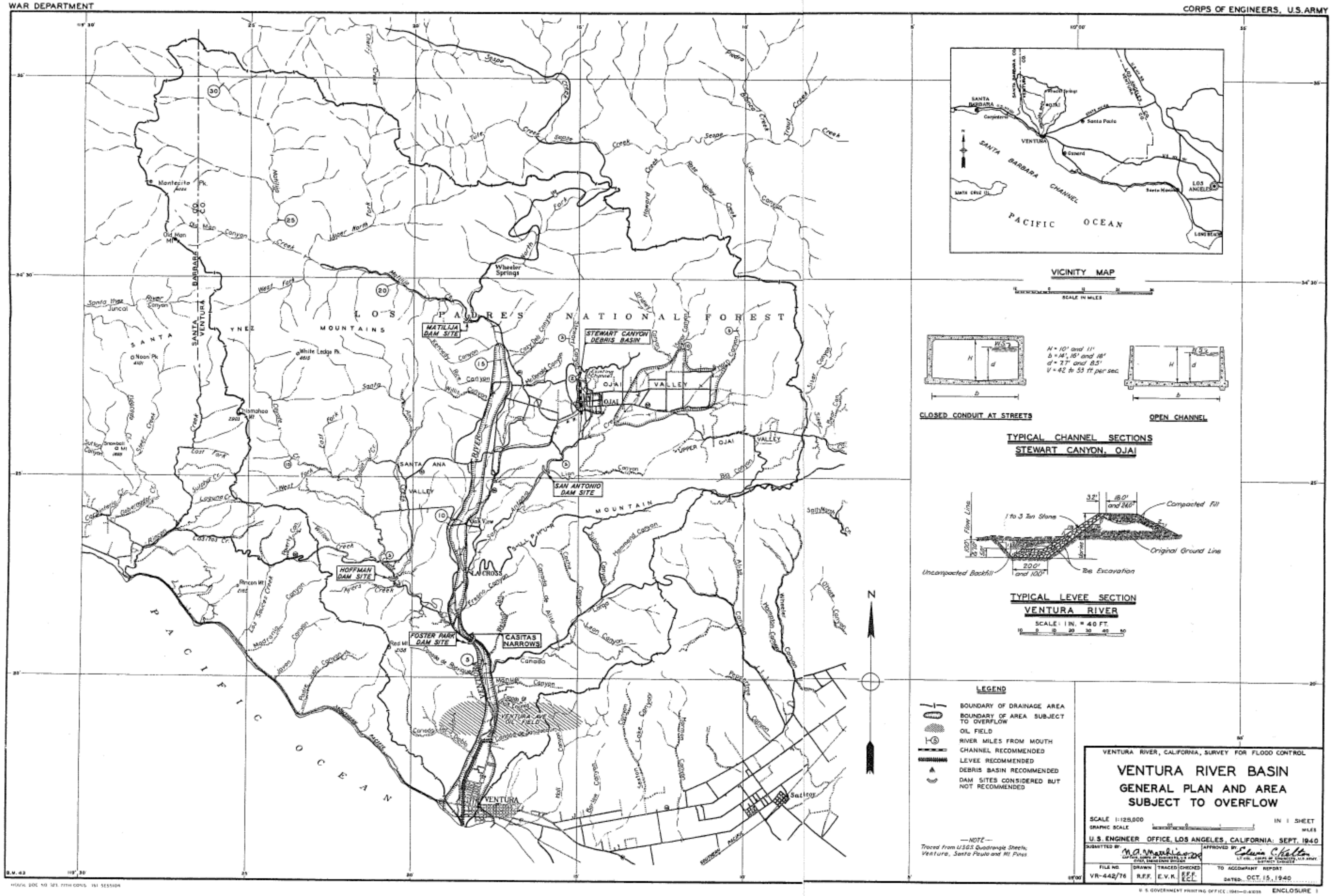


**Figure 2 – Improvements Recommended in U.S. Army Corps of Engineers 1941 Report**

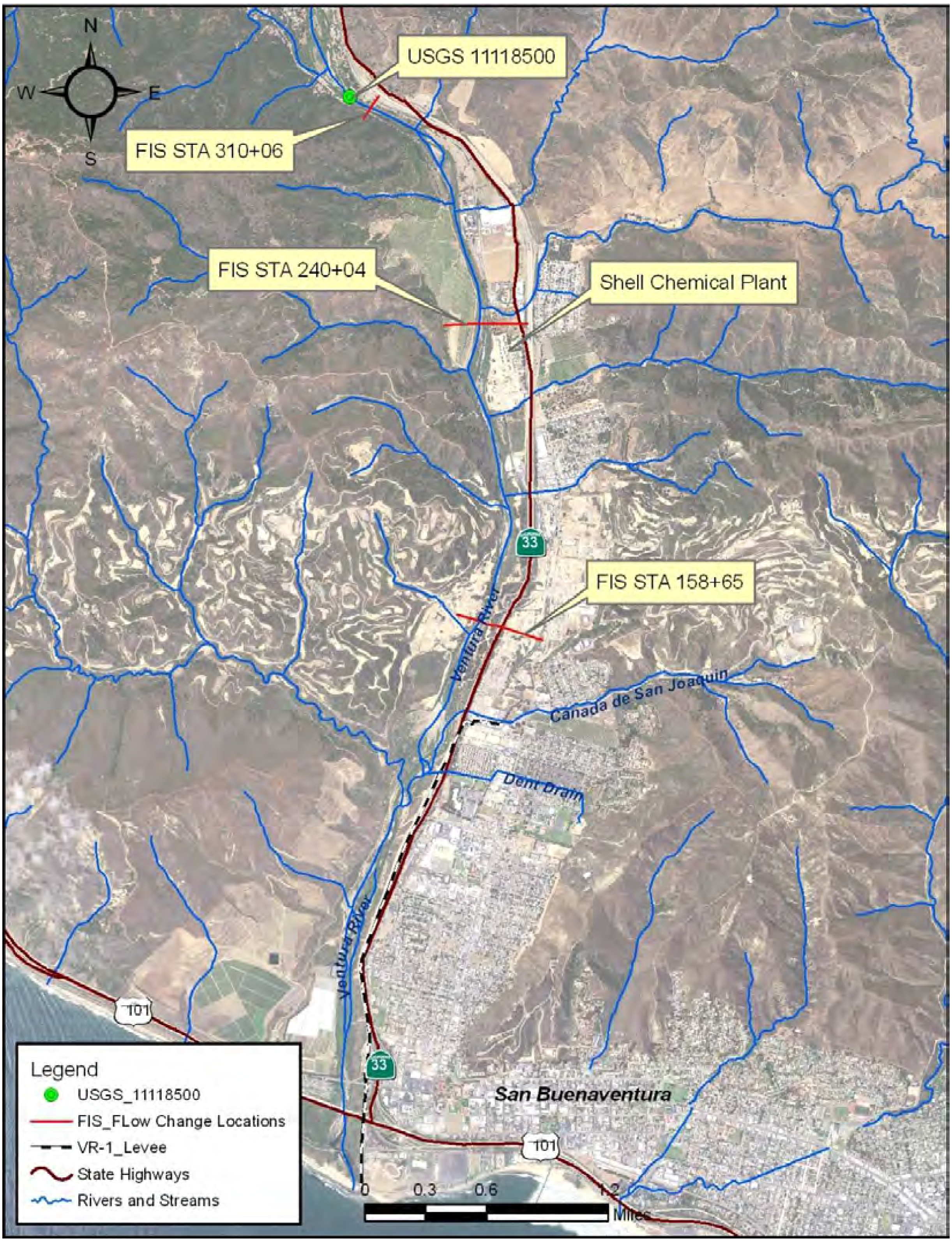


**Figure 3 – Location Map of VR-1 System and USGS Gage No. 11118500**

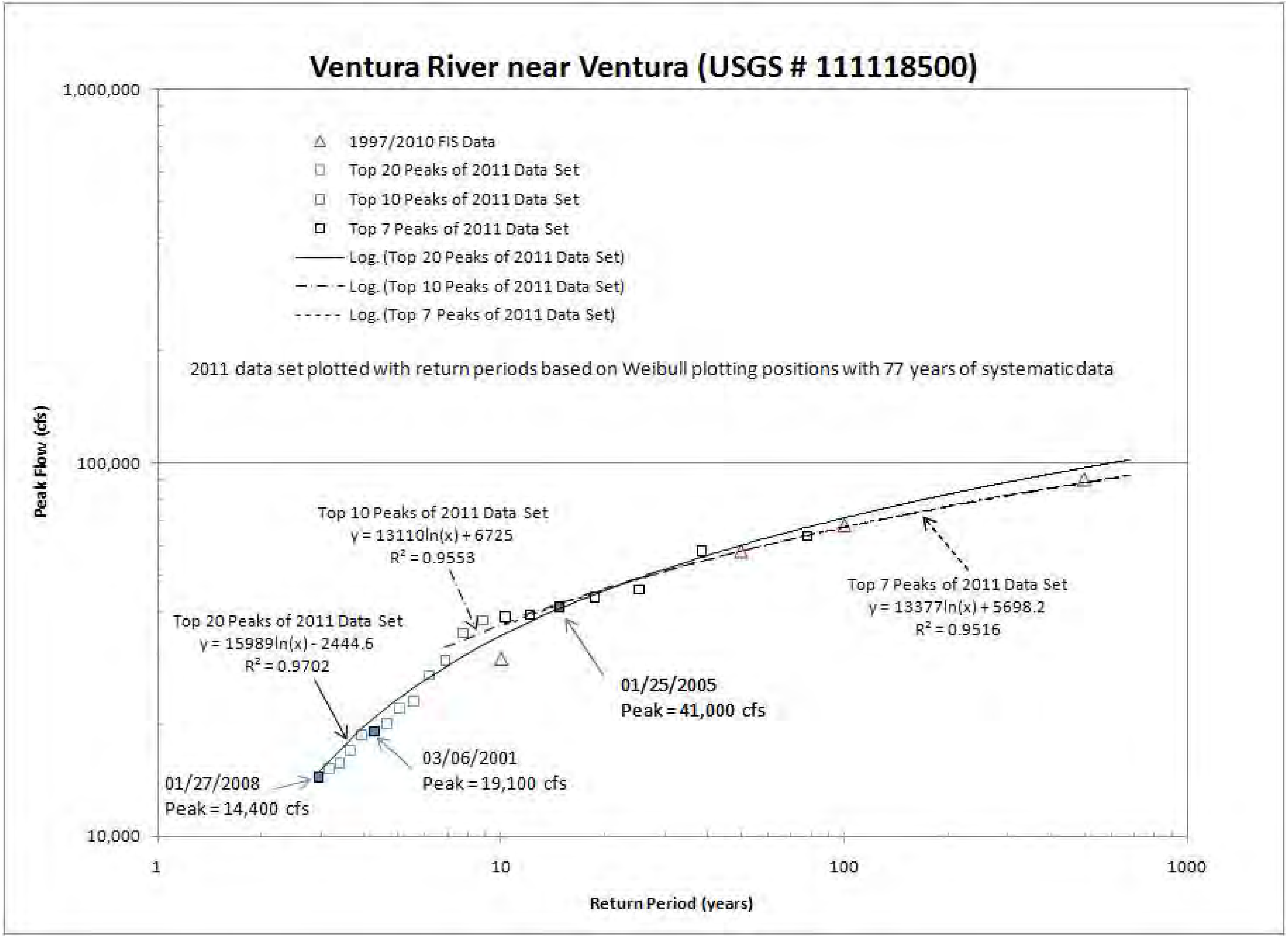


**Figure 4 – Top-End-Fitting Analyses at USGS Gage No. 11118500**

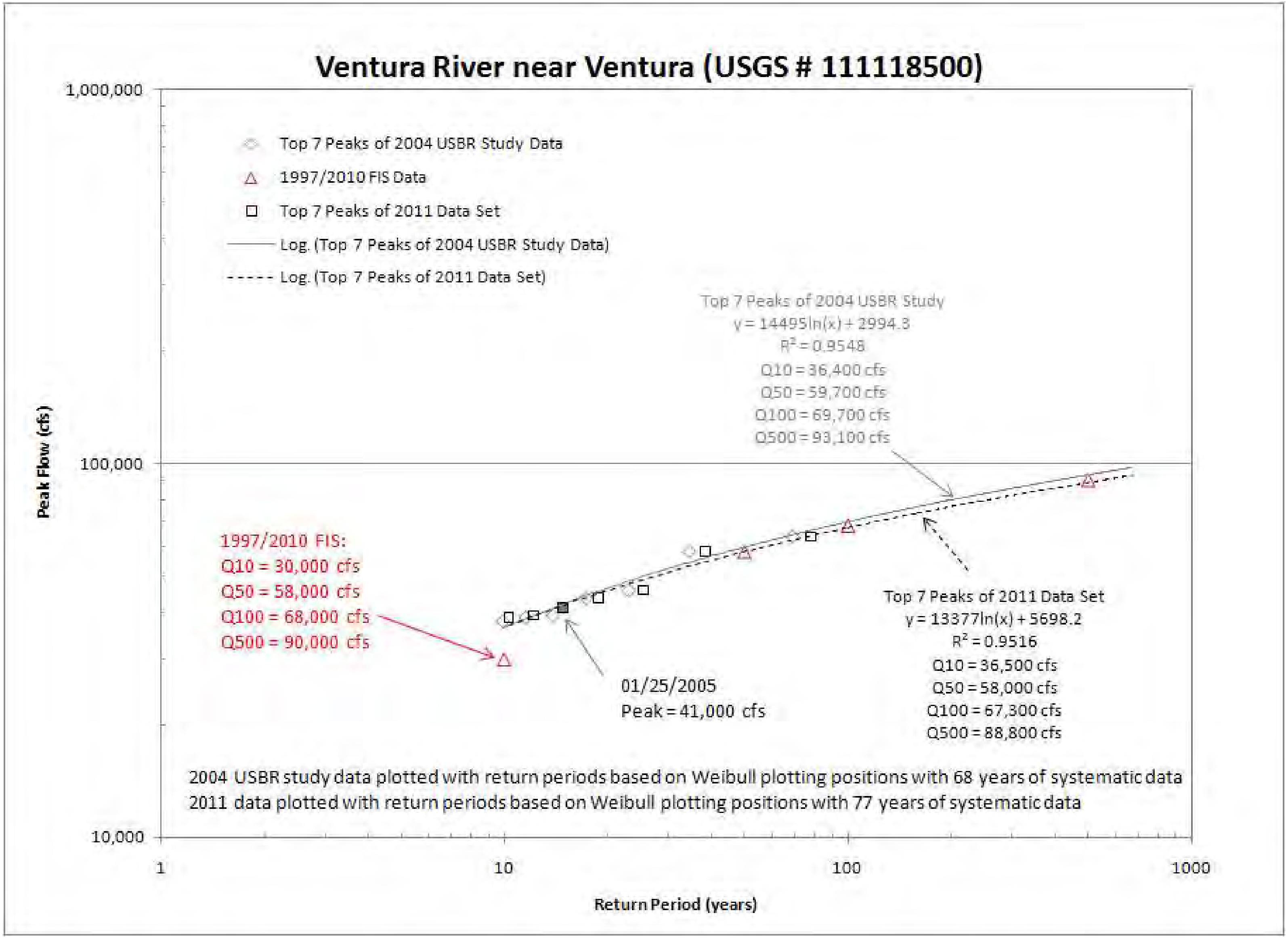


**Figure 5 – Summary of Estimated Peak Flows at USGS Gage No. 11118500**

# Appendix C
# Hydraulics

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Hydraulics Appendix
VENTURA RIVER LEVEE EVALUATION AND REHABILITATION
Feasibility Study

# 1. INTRODUCTION

The Ventura River Levee (VCWPD ID No. VR-1) is located in the city of San Buenaventura, in Ventura County, California. The levee system extends from the Pacific Ocean, at its downstream terminus, to the Cañada de San Joaquin, at its upstream terminus (Figure 1). The VR-1 system is located along the left side of the Ventura River. The levee system consists of embankment levees, side drainage penetrations, and a stop-log structure in the levee at a bike trail crossing. The levee system is intended to protect existing residential, commercial, industrial, and potentially developable property located in low-lying areas within the base flood (100-year) floodplain of the Ventura River Watershed.

## 1.1. Purpose of Report

This report was prepared in support of the VR-1 reach evaluation and rehabilitation documents, which meet the requirements of the U.S. Army Corps of Engineers (the Corps). If the Corps takes the lead on this study, these documents can be used in the future to complete the rehabilitation and certification of the VR-1 system.

## 1.2. History of VR-1 System

The Ventura River Levee was authorized by the Flood Control Act approved on December 22, 1944, Public Law 534, 78th Congress, Chapter 665, 2nd Session (H.R. 4485), substantially in accordance with the recommendations of the Chief of Engineers in House Document No., 323, 77th Congress, 1st Session.

The levee system was designed by the Corps and completed on December 31, 1948. The levee protects the western part of the city of Ventura, as well as the suburban area immediately adjacent and to the north, from a flood having a peak-discharge magnitude of 150,000 cubic feet per second (cfs). This is the maximum peak design flow that is estimated to occur as the result of a more severe regional storm coupled with conditions fairly conductive to runoff.

The levee system begins at the Pacific Ocean in Ventura County and continues upstream to the confluence of the Ventura River with the Cañada de San Joaquin. Along the Ventura River, the length of the levee is approximately 2.65 miles, with an embankment height on the landward side that is up to 10 feet above natural ground. The earthen berm that makes up the levee is protected both by loose riprap and grouted riprap, depending on the project stationing; and there is an access road along the top that is approximately 18 feet to 26 feet wide.

The history of VR-1 improvements and modifications are described in the periodic inspection report prepared by Fugro West (2011).

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## 2. REVIEW OF PREVIOUS HYDRAULICS STUDIES

Reports pertinent to the hydraulics analysis were reviewed and are summarized in the following sections.

### 2.1. Corps 1941 Report – Preliminary Examination and Survey of Ventura River

The plan recommended for protection of the city of Ventura consisted of a levee on the left bank, extending from the Pacific Ocean, at river mile (RM) 0.05, to a point located on the southern bank of the Cañada de San Joaquin, at RM 2.6, where the levee would cross the railroad (USACE 1941). From this point, the levee would follow the left bank of Cañada de San Joaquin until the crown of the levee meets the existing ground level. The levee would be constructed of compacted earth fill, with a landward slope of 2 horizontal to 1 vertical (2H:1V) and a 1.25H: 1V at riverward slope. The width of crown would vary from 18 to 24 feet with an additional 3.2 feet of stone revetment and placed on the riverward slope with 1.5H: 1V slope to a depth of 7 to 10 feet below existing flowline. The levee toe apron would be 5 feet deep and 10 to 20 feet long consisting of 1 to 3 ton stone. The levee would be designed to protect the city of Ventura from a flood having a peak-discharge magnitude of 150,000 cfs, with computed channel velocities varying between 8 and 17 feet per second, depending on the stationing location. In the upper reaches of the levee, a freeboard of 3 feet would be provided; and in the lower reaches, the freeboard would be 5 feet. A blanket of dumped derrick stone would protect the levee from scour.

### 2.2. Corps 1947 Report – Definitive Project Report

According to the Definitive Project Report (USACE 1947a, b) and the as-built plans (USACE 1947c), typical dimensions of VR-1 range from about 6 to 22 feet above the existing ground surface on the riverward side, a top (crown) width ranging from 18 to 26 feet, and side slopes of 2H: 1V and 2.25H:1V for the dumped (ungrouted) stone section and the grouted stone section, respectively, on the riverward side and a 2H: 1V side slope on the landward side. The levee height on the landward side is not indicated in the system documents. From Station 10+95 to Station 11+95, revetment on both the riverward and landward sides of the levee consists of 24 inches of filter material covered by a layer of ungrouted 6-ton-maximum quarry stone with a thickness of 6 feet. From Station 11+95 to Station 21+30, revetment on the riverward side of the levee consists of 24 inches of filter material covered by a layer of ungrouted 6-ton-maximum quarry stone with a thickness of 6 feet. From Station 21+30 to Station 48+66, revetment on the riverward side of the levee consists of 24 inches of filter material covered by a layer of ungrouted 3-ton-maximum quarry stone varying in thickness from 3 feet at the crown to 5 feet at the toe. From Station 48+66 to Station 150+57, revetment on the riverward side of the levee consists of grouted quarry stone varying in thickness from 1.5 feet at the crown to 2 feet at the toe. It should be noted that ~~a~~ 2 feet of filter blanket and a dumped stone toe apron extending to 8 feet above the low point of the toe, as described in the October 1947 Definite Project Repot (USACE 1947a) were eliminated prior to the construction (USACE 1947b) for this reach of levee due to lack of exact analyses justification. The levee would be designed to protect the city of Ventura from a flood having a peak discharge magnitude of 150,000 cfs with a freeboard of 2 feet from Station 10+00 to Station 42+00 and a freeboard of 3 feet from Station 42+00 to Station 150+70 based on a roughness coefficient of 0.040.

### 2.3. FEMA FIS Report

The current Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) (FEMA 2010) provides the base flood (100-year) water-surface elevations at various locations along the Ventura River (Figure 2) in the vicinity of the VR-1 system (Table 1).

**Table 1 – Summary of FEMA FIS Ventura River Floodway Data**

| Flooding Source | | Base Flood Water-Surface Elevation (feet NAVD 88) | | Floodway Mean Velocity (feet/second) | Flow Depth[2] above Streambed Without Floodway (feet) |
|---|---|---|---|---|---|
| **Cross Section** | **Distance[1]** | **Without Floodway** | **With Floodway** | | |
| A | 1,540 | 17.2 | 17.2 | 6.9 | 10.0 |
| B | 2,570 | 20.7 | 20.7 | 11.9 | 10.4 |
| C | 3,700 | 29.0 | 29.1 | 9.7 | 15.4 |
| D | 5,500 | 40.7 | 41.1 | 13.0 | 15.9 |
| E | 7,500 | 49.8 | 49.8 | 8.5 | 13.0 |
| F | 9,500 | 65.5 | 65.5 | 15.7 | 14.7 |
| G | 11,500 | 78.9 | 78.9 | 13.8 | 9.1 |
| H | 13,500 | 97.8 | 97.8 | 17.6 | 11.2 |

Source: Excerpted from FEMA 2010.
1. Feet upstream of Pacific Ocean.
2. Measured from FIS Ventura River Flood Profiles (Exhibits Planes: 177P to 179P).

NAVD = North American Vertical Datum of 1988

The base flood elevations were computed with base flood discharges varying from 77,000 cfs to 78,000 cfs for the lower Ventura River using Manning's coefficients varying from 0.025 to 0.040 for the main channel and 0.040 to 0.050 for the overbank areas. The starting water-surface elevation was based on the mean higher high water (MHHW)-surface elevation at the Pacific Ocean. The Ventura River was considered to have medium debris potential; therefore, bridge openings and culvert sizes were adjusted in accordance with the following criteria:

- At all reinforced-concrete box culverts and bridge crossings where the cross-sectional end area was 100 square feet or less, the pier widths were doubled. Where the crossing consisted of two or more circular pipes, the cross-sectional end area was reduced by 20 percent.
- At all bridge crossings with cross-sectional end areas between 100 and 250 square feet, 1 foot of width was added to each side of each pier.
- At all bridges with cross-sectional end areas greater than 250 square feet, 2 feet of width were added to each side of each pier.

### 2.4. USBR/Corps Feasibility Study

Appendix D (USBR 2004b) of the *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report* (USBR 2004a) detailed the hydraulic analysis of the Ventura River and the development of the hydraulic model was summarized herein. Based on an aerial survey conducted on October 10, 2001, the U.S. Bureau of Reclamation (USBR) developed digital terrain models and orthorectified photographs for the project study reaches. Microstation, CADD, and InRoads software programs were used to develop design surfaces from these data and to create the geometry for the hydraulic model. Cross sections were constructed at intervals of approximately 500 feet along the study reaches, starting just upstream of the sediment deposition behind Matilija Dam and extending downstream to the mouth of the Ventura River, at the Pacific Ocean. In addition, eight bridges were surveyed in the field to more accurately model bridge geometry throughout the study reaches.

The Corps computer program Hydrologic Engineering Center's River Analysis System (HEC-RAS), Version 3.0, was initially used to simulate the hydraulics for each flood (the final HEC-RAS version used was 4.1.0 [USACE 2010a]). Results were generated for each cross section along the study reach. The hydraulic model was calibrated based on observed data at the Foster Park gage. This study assumes that discharges under existing conditions and future conditions would remain the same, based on current and future land-use comparisons. VR-1 extends from the Pacific Ocean at RM 0.05 to RM 2.5 approximately. The hydraulic model indicated that all peak discharges, from the 2-year flood peak to the 500-year flood peak, are confined to the main channel of the river by VR-1.

## 3. FEMA FIS HEC-RAS Model

The current FEMA FIS HEC-RAS model for the main stem of the Ventura River was prepared by HDR, the FEMA study contractor (HDR 2010b). HEC-RAS cross sections, extending from the Shell Road bridge to the Pacific Ocean, are shown in Figure 3. The peak discharge of the base flood used in the HEC-RAS model for the reach below the Shell Road Bridge is 79,166 cfs (see Hydrology Appendix). *Guide for Selecting Manning's Roughness Coefficients for the Natural Channels and Floodplains* (USGS 1989), was used, in conjunction with a field visit, to estimate the Manning's roughness coefficients for the channel and overbanks. The estimated Manning's coefficients are 0.033 and 0.068 for the main channel and overbank areas, respectively. The starting water-surface elevation for the HEC-RAS model was taken from a previously prepared USBR HEC-RAS model (USBR 2004b). The starting water-surface elevation is 2.53 feet relative to the North American Vertical Datum of 1988 (NAVD 88). It should be noted that the USBR HEC-RAS model (Figure 4) extended the cross sections approximately 0.22 mile from the river mouth into the Pacific Ocean, and the current FEMA FIS HEC-RAS model started its first cross section approximately 150 feet upstream of the river mouth (Figure 3).

A separate FEMA FIS HEC-RAS model (HDR 2010a) was set up for the Cañada de San Joaquin tributary. The peak flows for the Cañada de San Joaquin were provided by Ventura County using the U.S. Environmental Protection Agency's (EPA's) Hydrologic Simulation Program—FORTRAN (HSPF) model (USEPA 2001) (see Hydrology Appendix). The Manning's coefficients were estimated according to the same procedures as those used for the Ventura River. The Manning's coefficients are 0.030 and 0.032 for the main channel and 0.034 and 0.033 for the overbank areas along the VR-1 reach. The starting water-surface elevation for the HEC-RAS model was assumed to have a normal depth slope of 0.05, computed using stations and invert elevations of the first two cross sections in the model.

Figure 5 shows the FIS HEC-RAS models cross sections at the confluence of the Ventura River and the Cañada de San Joaquin. The Cañada de San Joaquin could potentially be influenced by the water-surface elevations from Station 131+91.51 to Station 134+89.16 located in the Ventura River. Table 2 summarizes the computed water-surface elevations in the vicinity of the confluence.

**Table 2 – Computed Water-Surface Elevations (feet, NAVD 88) in Vicinity of Ventura River and Cañada de San Joaquin Confluence**

| | FIS HEC-RAS Model Station | | | | |
|---|---|---|---|---|---|
| | Ventura River | | | Cañada de San Joaquin | |
| **Storm Event** | **131+91.51** | **133+63.41** | **134+89.16** | **4+08.95** | **5+92.30[1]** |
| 2-year | 83.03 | 83.75 | 84.46 | 85.72 | 95.68 |
| 5-year | 84.52 | 85.05 | 86.20 | 86.47 | 96.65 |
| 10-year | 87.34 | 89.08 | 89.91 | 87.16 | 97.51 |
| 50-year | 89.21 | 91.58 | 92.50 | 88.46 | 99.07 |
| 100-year | 90.16 | 92.48 | 93.46 | 89.23 | 100.21 |
| 500-year | 91.93 | 94.64 | 95.64 | 91.02 | 101.72 |

1. Downstream of State Route 33 culverts and at critical flow conditions.

Table 2 indicates that the water-surface elevations at the Ventura River have no influence on the channel hydraulics of the Cañada de San Joaquin because at Station 5+92.30, downstream of the State Route (SR) 33 culverts, the computed water-surface elevations are higher than the computed water-surface elevations in the Ventura River for all flood events. Furthermore, the critical flow conditions are computed at this cross section and acting as the downstream control water-surface elevations for the Cañada de San Joaquin channel. Therefore, the Ventura River and the Cañada de San Joaquin can be treated as two independent streams.

### 3.1. Baseline HEC-RAS Model Setup

For the rehabilitation study of the VR-1 system, the baseline hydraulic model was constructed by revising the FEMA FIS model as follows:

- The VR-1 cross sections, spaced at 100-foot intervals, were surveyed in November/December 2010. The surveyed cross section is 250 feet wide, with 150 feet and 100 feet from levee control line toward the riverside and the landside, respectively. The interpolated survey sections that coincided with the HEC-RAS stations were developed and used to modify the levee portion for each of the FEMA FIS HEC-RAS cross sections.
- The MHHW-surface elevation at the Pacific Ocean was used as the starting water-surface elevation in the baseline model. Two National Oceanic and Atmospheric Administration (NOAA) tidal stations are located nearby. The Santa Barbara station (9411340) is approximately 22.6 miles to the northwest, and the Santa Monica station (9410840) is approximately 47.9 miles southeast of the Ventura River mouth. Table 3 summarizes the tidal elevations based on the station datum. The MHHW-surface elevations are 5.30 feet (NAVD 88) and 5.24 feet (NAVD 88), the maximum observed water levels are 7.26 feet (NAVD 88) and 8.31 feet (NAVD 88), and the highest astronomical tides are 7.10 feet (NAVD 88) and 7.08 feet (NAVD 88) for Santa Barbara station and Santa Monica station, respectively. The MHHW-surface elevation of 5.30 feet (NAVD 88) was used as the HEC-RAS starting water-surface elevation because the Santa Barbara station is closer to the Ventura River mouth.

**Table 3 – Summary of Tidal Elevations on Station Datum**

| Datum | Santa Barbara[1] | Santa Monica[2] | Description |
|---|---|---|---|
| MHHW | 8.59[3]/5.30[4] | 7.87[3]/5.24[4] | Mean higher high water |
| MHW | 7.83[3]/4.54[4] | 7.13[3]/4.50[4] | Mean high water |
| MTL | 6.00[3]/2.71[4] | 5.25[3]/2.62[4] | Mean tide level |
| MSL | 5.98[3]/2.69[4] | 5.23[3]/2.60[4] | Mean sea level |
| DTL | 5.89[3]/2.60[4] | 5.16[3]/2.53[4] | Mean diurnal tide level |
| MLW | 4.18[3]/0.89[4] | 3.37[3]/0.74[4] | Mean low water |
| NAVD 88 | 3.29[3]/0.00[4] | 2.63[3]/0.00[4] | North American Vertical Datum of 1988 |
| MLLW | 3.20[3]/-0.09[4] | 2.44[3]/-0.19[4] | Mean lower low water |
| STND | 0.00[3]/-3.29[4] | 0.00[3]/-2.63[4] | Station datum |

| Datum | Santa Barbara[1] | Santa Monica[2] | Description |
|---|---|---|---|
| Maximum | 10.55[3]/7.26[4] | 10.94[3]/8.31[4] | Maximum observed water level |
| HAT | 10.39[3]/7.10[4] | 9.71[3]/7.08[4] | Highest astronomical tide |
| 1. Tidal datum analysis period: 01/01/1991 to 12/31/1997. 2. Tidal datum analysis period: 01/01/1984 to 12/31/2001. 3. Elevations based on Station datum. 4. Elevations based on NAVD 88 datum. | | | |

- Two unpermitted reinforced-concrete floodwalls were depicted on the 2010 survey topographic map of VR-1. Information regarding the design and construction of the floodwalls was unavailable. In the absence of such documentation, the floodwalls are located from approximately levee Station 21+48 to Station 26+98 and from levee Station 27+38 to Station 37+65 for Floodwall 1 and Floodwall 2, respectively. Surveyed top-of-floodwall elevations were used in the freeboard analysis. Top-of-floodwall elevations vary from 24.67 to 25.86 feet and from 25.89 to 29.92 feet for Floodwall 1 and Floodwall 2, respectively.

- Bridge openings and culvert sizes were adjusted in accordance with Corps *Hydrology and Hydraulic Policy Memorandum No.4 – Debris Loading on Bridges and Culverts* (USACE 2004).

This hydraulic model will be the baseline model from which the new hydraulic models could be developed as needed to reflect the channel and/or levee-improvement alternatives for the VR-1 system.

### 3.2. Comparison of Baseline and FEMA FIS HEC-RAS Models

The computed 100-year water-surface elevations along the VR-1 reach are listed in Table 4 for the baseline and FEMA FIS HEC-RAS models. The assumed starting water-surface elevation of 5.30 feet in the baseline model was lower than the computed critical flow depth of 9.31 feet and was replaced by the critical flow depth as the starting water-surface elevation by the HEC-RAS computer program. Even the highest observed water elevation of 7.26 feet (NAVD 88), which included the actual storm wind-wave setup is less than the computed critical flow depth at the first HEC-RAS cross section. Therefore, the tide at the Ventura River mouth has no influence on the Ventura River water-surface profile.

The surveyed sections located downstream of the Southern Pacific Railroad (SPRR) bridge indicated 2 to 4 feet more deposition near the levee toe compared to the deposition in the FIS sections. Therefore, the computed water-surface elevations were higher for the baseline model in this area. Including the bridge pier debris option in the baseline model, the computed water-surface elevations are higher at the SPRR, State Highway 101, and Main Street bridges for a maximum value of 0.5 feet. There were no significant differences between these two models in terms of the water-surface elevations from upstream of the Main Street bridge to the end of the VR-1 system.

Table 4 indicates the freeboard deficiencies upstream of the SPRR bridge, downstream and upstream of the Highway 101 Bridge, and upstream of the SR 33 culverts.

**Table 4 – Comparison of Baseline and FEMA FIS HEC-RAS Models**

| Approximate Levee Station | HEC-RAS Station | Channel Thalweg[1] (feet) | Computed Water-Surface Elevation[1] (feet) | | Top-of-Levee Elevation[1] (feet) | | Computed Freeboard (feet) | | FEMA-Required Freeboard |
|---|---|---|---|---|---|---|---|---|---|
| | | | FIS | Baseline | FIS | Baseline | FIS | Baseline | |
| End of Levee at Approximately Levee Station 149+22.88 | | | | | | | | | |
| 149+15.14 | 16+95.32 | 106.94 | 109.20 | 109.19 | 119.87 | 120.40 | 10.67 | 11.21 | 3.5[2] |
| Crossing in OST Yard | | | | | | | | | |
| 148+82.01 | 16+50.47 | 106.28 | 108.83 | 108.89 | 119.80 | 119.15 | 10.97 | 10.26 | 3[2] |
| 148+34.80 | 16+05.52 | 105.64 | 108.74 | 108.80 | 118.60 | 117.88 | 9.86 | 9.08 | 3[2] |
| Building Over Channel | | | | | | | | | |
| 145+74.67 | 13+40.21 | 101.05 | 106.99 | 107.00 | 114.95 | 114.35 | 7.96 | 7.35 | 3[2] |
| 143+95.45 | 11+73.49 | 99.04 | 106.99 | 107.00 | 112.49 | 111.65 | 5.50 | 4.65 | 3[2] |
| Crossing in OST Yard | | | | | | | | | |
| 142+98.93 | 10+82.10 | 98.44 | 106.99 | 107.00 | 111.09 | 111.03 | 4.10 | 4.03 | 3[2] |
| 142+04.16 | 9+99.46 | 97.81 | 106.99 | 107.00 | 109.88 | 108.49 | 2.89 | 1.49 | 3 |
| 140+17.38 | 8+49.36 | 97.58 | 106.99 | 107.00 | 108.10 | 108.12 | 1.11 | 1.12 | 3[2] |
| Crossing in OST Yard | | | | | | | | | |
| 139+74.04 | 8+14.85 | 96.96 | 106.99 | 107.00 | 108.12 | 107.91 | 1.13 | 0.91 | 3[2] |
| Ojai Trail Bike Path | | | | | | | | | |
| 139+12.19 | 7+75.05 | 97.68 | 106.58 | 106.59 | 107.96 | 107.98 | 1.38 | 1.39 | 3[2] |
| Confluence of Cañada de San Joaquin and Ventura River | | | | | | | | | |
| 137+93.73 | 130+21.47 | 74.05 | 88.77 | 88.77 | 108.51 | 107.52 | 19.74 | 18.75 | 3 |
| 136+70.09 | 128+77.79 | 73.25 | 86.07 | 86.07 | 108.11 | 107.50 | 22.04 | 21.43 | 3 |
| 133+76.91 | 125+97.31 | 71.15 | 83.49 | 83.54 | 106.58 | 104.81 | 23.09 | 21.27 | 3 |
| 131+26.07 | 123+40.06 | 70.09 | 81.39 | 81.30 | 102.00 | 100.52 | 20.61 | 19.22 | 3 |
| 125+39.64 | 117+27.16 | 62.75 | 76.70 | 76.71 | 97.30 | 96.48 | 20.60 | 19.77 | 3 |
| 120+30.35 | 112+51.45 | 59.23 | 73.05 | 73.03 | 92.83 | 92.13 | 19.78 | 19.10 | 3 |
| 115+74.55 | 107+31.75 | 58.28 | 69.82 | 69.83 | 88.96 | 88.60 | 19.14 | 18.77 | 3 |
| 111+68.84 | 101+56.63 | 51.14 | 68.42 | 68.42 | 86.67 | 86.15 | 18.25 | 17.73 | 3 |
| 106+21.28 | 96+36.13 | 46.46 | 67.63 | 67.64 | 82.87 | 82.55 | 15.24 | 14.91 | 3 |
| 101+51.86 | 91+88.07 | 43.60 | 62.15 | 62.19 | 79.48 | 79.40 | 17.33 | 17.21 | 3 |
| 96+73.53 | 86+86.77 | 41.59 | 59.32 | 59.31 | 76.69 | 76.08 | 17.37 | 16.77 | 3 |
| 91+69.01 | 81+75.15 | 38.25 | 54.78 | 54.76 | 72.53 | 71.99 | 17.75 | 17.23 | 3 |
| 86+66.14 | 76+71.02 | 38.02 | 53.68 | 53.62 | 67.86 | 67.76 | 14.18 | 14.14 | 3 |
| 81+92.17 | 71+78.09 | 31.85 | 52.60 | 52.49 | 63.94 | 63.85 | 11.34 | 11.36 | 3 |
| 76+97.11 | 66+72.70 | 29.76 | 47.49 | 47.42 | 60.32 | 60.10 | 12.83 | 12.68 | 3 |

| Approximate Levee Station | HEC-RAS Station | Channel Thalweg[1] (feet) | Computed Water-Surface Elevation[1] (feet) | | Top-of-Levee Elevation[1] (feet) | | Computed Freeboard (feet) | | FEMA-Required Freeboard |
|---|---|---|---|---|---|---|---|---|---|
| | | | FIS | Baseline | FIS | Baseline | FIS | Baseline | |
| 72+41.97 | 61+69.65 | 24.75 | 46.33 | 46.26 | 57.02 | 56.93 | 10.69 | 10.67 | 3 |
| 67+16.18 | 56+54.25 | 22.57 | 44.54 | 44.63 | 52.94 | 51.90 | 8.40 | 7.27 | 3 |
| 62+02.33 | 51+44.70 | 19.79 | 42.03 | 42.02 | 48.60 | 47.93 | 6.57 | 5.91 | 3 |
| 56+70.27 | 46+36.19 | 19.19 | 38.26 | 38.29 | 44.49 | 43.91 | 6.23 | 5.62 | 3 |
| 52+03.73 | 41+35.26 | 18.68 | 34.84 | 34.76 | 40.85 | 39.42 | 6.01 | 4.66 | 3 |
| 47+57.50 | 36+21.49 | 17.30 | 30.99 | 31.00 | 36.61 | 36.51 | 5.62 | 5.51 | 3 |
| 43+09.86 | 31+12.68 | 12.87 | 27.57 | 27.96 | 33.44 | 33.63 | 5.87 | 5.67 | 3 |
| 39+12.75 | 28+69.57 | 11.51 | 27.35 | 27.80 | 32.73 | 34.12 | 5.38 | 6.32 | 4 |
| Main Street Bridge | | | | | | | | | |
| 38+31.16 | 27+33.19 | 10.77 | 26.22 | 26.38 | 32.84 | 30.63 | 6.62 | 4.25 | 4 |
| 35+75.06 | 24+96.20 | 9.62 | 25.63 | 25.92 | 30.40 | 29.89[3] | 4.77 | 3.97 | 3 |
| 31+04.66 | 20+56.05 | 5.17 | 25.52 | 25.81 | 28.40 | 29.72[3] | 2.88 | 3.91 | 4 |
| Highway 101 Bridge | | | | | | | | | |
| 26+77.53 | 16+51.52 | 5.34 | 21.71 | 22.20 | 25.27 | 25.87[3] | 3.56 | 3.67 | 4 |
| 22+06.03 | 10+71.01 | 3.02 | 21.74 | 22.23 | 23.11 | 25.00[3] | 1.37 | 2.77 | 4 |
| South Pacific Railroad Bridge | | | | | | | | | |
| 20+54.54 | 6+94.09 | 2.92 | 13.24 | 13.24 | 19.90 | 19.94 | 6.66 | 6.70 | 4 |
| 16+14.47 | 3+56.51 | 2.74 | 10.91 | 11.35 | 17.94 | 18.26 | 7.03 | 6.91 | 3 |
| 13+75.34 | 1+62.99 | 2.22 | 9.40 | 10.26 | 16.97 | 17.31 | 7.57 | 7.05 | 3 |
| 11+97.27 | 0+43.85 | 2.33 | 8.04[4] | 9.31[5] | 17.02 | 16.86 | 8.98 | 7.55 | 3.5 |

1. NAVD 88.
2. Crossings (culverts) in the Cañada de San Joaquin have limited capacity and were inundated by the 100-year flood. Therefore, no additional freeboard is required within 100 feet of the structures on either side.
3. Top of floodwall elevations.
4. Starting water-surface elevation was set to 2.53 feet but defaulted to critical flow depth.
5. Starting water-surface elevation was set to 5.30 feet but defaulted to critical flow depth.

# 4. SCOUR ANALYSIS

## 4.1. Review of Previous Studies

The *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report,* Appendix D—Hydrologic, Hydraulic, and Sediment Studies (USBR 2004b) are summarized herein.

The Ventura River Watershed was divided into nine study reaches (Figure 6), and VR-1 is located in Reach 1 and Reach 2. Reach 1 is the estuary reach, which starts at the river mouth, at RM 0, and extends to RM 0.6, located near the Main Street Bridge. Reach 2 lies between RM 0.6 and RM 5.95 and extends from the Main Street Bridge to the Foster Park Bridge. The VR-1 reach extends from the Pacific Ocean upstream along the Ventura River between RM 0.05 and RM 2.5 approximately.

In October 2001, a combined total of 18 bed material samples were collected in the Ventura River and Matilija Creek, a tributary of the Ventura River. The samples were spaced approximately every mile, starting at the mouth of the Ventura River and ending 1 mile upstream of Matilija Dam (RM 17.9). Two additional samples of beach sand were collected by USBR along the shoreline near the mouth of the Ventura River. Samples 4, 3, 2, 1, and 8, sequentially from downstream to upstream, are located within the VR-1 reach (Figure 7). The bed material generally becomes coarser with increasing upstream distance from the ocean). Near the ocean, the median particle diameter, $d_{50}$, is approximately 70 to 80 millimeters (mm). However, there is a notable exception to the general trend of increasing particle size with increasing upstream distance in the VR-1 reach. Sample 3 (RM 0.6) had a significant amount of sands on the surface. Therefore, the $d_{16}$ of Sample 3 was much smaller than the $d_{16}$ of the other samples. The large amount of fine material could be because the sample was collected closer to the ocean, and the sampling location is immediately upstream of the Main Street Bridge. The sediment loads in the Ventura River are dominated by infrequent flood events, as evidenced by the sediment loads indicated in Figure 8. It is estimated that between 1969 and 1981, more than 96 percent of the sediment load was transported during large individual flood events that occurred in 1969, 1978, and 1980.

The 1970 topographic data represent the river channel as it existed 23 years after the completion of Matilija Dam and about 1 year after the large flood events that occurred in January and February 1969. The 1970 contour data are noted as having a maximum potential error of +/–2.5 feet (USACE 1971). The original coordinates of the 1970 data were not found, so their locations had to be determined from plan-view drawings included in the 1971 flood report (USACE 1971). Based on these drawings, the 1970 sections are generally within a few tens of feet in longitudinal distance from the 2001 cross section locations to which they are compared in this report. Therefore, any changes between the 1970 and 2001 thalweg elevations within a range of +/–2.5 feet may only be a reflection of short-duration channel dynamics and error within the data, particularly if it is only at one location. Changes greater than 2.5 feet over a group of cross sections would more likely indicate long-term changes in the channel bed. A 3-point moving average of the change in channel bed elevation between 1970 and 2001 was computed. A thalweg value was used rather than an average channel bed elevation, because the Ventura River is wide and often has multiple bars between channels that would make it difficult to compute average channel bed elevations. Based on the comparison, the Ventura River has experienced significant erosion since 1970 at three locations (Figure 9). Reach 2, where VR-1 is located, has had the largest channel changes and widespread degradation since 1970. A maximum degradation of approximately 8 feet was observed at the upstream end of VR-1. At RM 0.4 and 1.8, the degradation depths were approximately 7 feet.

## 4.2. Scour Calculations

An alluvial channel system is very dynamic due to changes in sediment transport, river geomorphology, human-induced effects, and other factors. Empirical equations were developed to co-relate the channel hydraulics and channel materials in estimating the potential scour depth. It is presumed that an alluvial channel will not experience supercritical flow conditions for any significant length along the channel (Grant 1997). Therefore, the critical flow condition should be the worst-case condition.

### 4.2.1. 4.2.1 Estimation of Critical Flow Velocity

The following iterative process (Zeller 2011) is used to yield a Manning's *n* value that produces critical flow conditions, more or less, along the system. Using the existing (standard) HEC-RAS model, determine the following hydraulic conditions in the channel: hydraulic radius and energy slope. Then, use the following formula to estimate the smallest Manning's channel *n* value to be used for assessing erosion/scour (which presumes the absence of significant amounts of vegetation in the channel):

$$n = (0.2619)(S_e^{1/2})(R_h^{1/6})$$

The substitution of the energy slope, $S_e$, and the hydraulic radius, $R_h$, from the existing HEC-RAS model will provide a first approximation of the lowest Manning's *n* value to use for erosion/scour. Once the Manning's *n* value is calculated using the preceding equation, insert this value into HEC-RAS and rerun the model. Then take the computed $S_e$ and $R_h$ values using the lower Manning's *n* value, and determine a new Manning's *n* value. Substitute this new Manning's *n* value into HEC-RAS, and rerun again. After two or three iterations, one should be able to "zero-in" on a final value for the Manning's *n* value (i.e., the value will not change significantly).

There is, obviously, a lower limit to the Manning's *n* value, which would be the *n* value created only by grain roughness (excluding form roughness). A widely used equation (Anderson et al. 1970) for computing this value is the following:

$$n_g = 0.0152(D_{50}^{1/6})$$

Where:
$D_{50}$ = median particle diameter, in mm.

For $D_{50}$ = 48 millimeters (smallest of five samples), using the preceding equation yields $n_g \approx$ 0.029, which was assumed to be the lower limit in this iterative process.

The final HEC-RAS channel hydraulic results were used in the following scour calculations.

#### 4.2.2. Scour Calculations

Due to the variations of the computed hydraulic parameters, the VR-1 reach was divided into four hydraulically similar segments for scour calculations. HEC-RAS cross sections associated with each segment and a soil sample associated within each segment are indicated in Table 5 and shown on Figure 10.

**Table 5 – VR-1 Reach Scour Calculation Segments**

| Segment No. | From Station | To Station | Soil Sample No. |
|---|---|---|---|
| 1 | 0+43.85 | 27+33.19 | 4 |
| 2 | 28+69.57 | 41+35.26 | 3 |
| 3 | 46+36.19 | 96+36.13 | 2 |
| 4 | 101+56.63 | 130+21.47 | 1 and 8 |

Table 6 lists the locations of the USBR bed material samples with the associated median particle diameter, $D_{50}$, for each soil sample.

**Table 6 –USBR Ventura River Bed Material Sampling Locations within VR-1 Reach**

| Sample No. | USBR River Mile | Latitude | Longitude | Approximate HDR RAS Station | $D_{50}$ (mm) |
|---|---|---|---|---|---|
| 4 | 0.5 | $34^0$ 16' 50.60" | $119^0$ 18' 29.90" | 22+88.56 | 74.0[1]/78.0[2] |
| 3 | 0.6 | $34^0$ 16' 58.60" | $119^0$ 18' 30.80" | 31+12.68 | 79.6[1]/72.0[2] |
| 2 | 1.2 | $34^0$ 17' 30.18" | $119^0$ 18' 28.63" | 64+21.02 | 60.2[1]/48.0[2] |
| 1 | 2.2 | $34^0$ 18' 14.53" | $119^0$ 18' 7.80" | 113+17.85 | 121.2[1]/95.0[2] |
| 8 | 2.5 | $34^0$ 18' 27.22" | $119^0$ 17' 59.97" | 129+70.76 | 46.2[1]/62.0[2] |

1. Value obtained from USBR report (2004b).
2. Value obtained by interpolation between adjacent data pairs and used in this study.

Large sediment sizes were observed in the bed material sample gradation curves (Figure 11). Accordingly, a quick computation of the armoring size particle for the average reach hydraulic conditions was developed to determine whether the VR-1 reach would develop an armor layer. Equation TS14B-4 of Technical Supplement 14B in Part 654 of the *National Engineering Handbook* (NEH) (NRCS 2007) was used for this purpose:

$$D_x = K\left(\frac{yS_e}{\Delta S_g}\right)^a\left(\frac{U_*}{\nu}\right)^b$$

Where:

$D_x$ = armoring size particle, in feet.
$y$ = flow depth, in feet.
$S_e$ = energy slope, in feet/foot.

$\Delta S_g$ = relative submerged density of bed material sediments ≈ 1.65.
$U_*$ = shear velocity = $(gyS_e)^{0.5}$
(where: g = acceleration of gravity, 32.2 feet/second$^2$)
ν = kinematic viscosity of water, ft$^2$/second, and
K, a, and b are constants based on the particle Reynolds number, as shown in Table 7.

**Table 7 - Constants for Computation of Minimum Armoring Particle Size**

| Particle Reynolds Number, $(U_* \cdot D_{50}/\nu)$ | K | a | b |
|---|---|---|---|
| ≤10 | 68 | 1.67 | 0.67 |
| Between 10 and 500 | 27 | 0.86 | –0.14 |
| ≥500 | 17 | 1.00 | 0.00 |

Using the preceding equation in conjunction with an energy slope assumed to be equal to the thalweg slope (Figure 12), the average flow depth, and a relative submerged density of 1.65, the armoring particle sizes were computed for each segment. The computational results are presented in Table 8.

In addition, an equation developed by (Zeller 1999), using the incipient motion criterion, is expressed as follows:

$$D_x = \frac{0.262 * V^3}{Y_h^{1/2}}$$

Where:

$D_x$ = armoring size particle, in millimeters

Using the Zeller equation in conjunction with a maximum velocity and a maximum flow depth, the armoring particle sizes are computed for each segment and also listed in Table 8 in order to compare the computational results with the estimates from the NEH equation. Note the influence of maximum flow velocity in the Zeller equation, which is not a factor in the USBR method for calculating armoring sizes.

**Table 8 – Comparison of Minimum Armoring Particle Sizes**

| Segment No. | Bed Material Sample No. | Thalweg Slope | Max. Flow Depth (feet) | Max. Flow Velocity (feet/second) | Min. Armoring Particle Size ($D_x$) (mm) | |
|---|---|---|---|---|---|---|
| | | | | | NEH Equation | Zeller Equation |
| 1 | 4 | 0.003138 | 12.77 | 11.33 | 126 | 107 |
| 2 | 3 | 0.005665 | 14.46 | 13.21 | 257 | 159 |
| 3 | 2 | 0.005454 | 17.91 | 18.85 | 307 | 415 |
| 4 | 1 and 8 | 0.007997 | 12.42 | 19.18 | 312 | 525 |

The bed material gradation distribution curves (Figure 11) indicate that the percentage of the soil sample in weight larger than the minimum armoring particle sizes (based on the NEH equation) are 23 percent, 11 percent, 1 percent, 4 percent, and 1 percent for Soil Samples 4, 3, 2, 1, and 8, respectively, located sequentially from downstream to upstream. The results in Table 8 suggests that riverbed armoring likely does not occur (particularly in the two upstream segments—Segment 3 and Segment 4), or it would only occur locally in specific areas where large sizes of stones in sufficient quantities are available to form a "spot" armor layer.

Based on Technical Supplement 14B of NEH Part 654 (NRCS 2007), the total armoring scour depth, $z_t$, can be calculated as follows:

$$z_t = T - D_x$$

Where:

T = thickness of the active layer of the bed, in feet.
$D_x$ = smallest armor size or size of the smallest non-transportable particle present in the bed material, in feet.

T is related to $D_x$ as follows:

$$T = \frac{D_x}{(1-e)P_x}$$

Where:
e = porosity of the bed material, which can be estimated as follows:

$$e = 0.245 + \frac{0.0864}{(0.1D_{50})^{0.21}}$$

Where:
$D_{50}$ = median grain size of sediment mixture, in feet.
$P_x$ = fraction of bed material (expressed as a decimal) of a size equal to or coarser than $D_x$.

Typically, in order to ensure that an armor layer will form across the entire channel cross section, the thickness of T, the active layer of the bed, should not be based on a $D_x$ particle size that represents a fraction of the bed material, which is smaller than 10 percent (i.e., a particle size larger than $D_{90}$).

Accordingly, using the previous calculated $D_x$ (based on the NEH equation) and the percentage of weight larger than $D_x$ with $D_{50}$ of each bed material sample, the computed total scour depths are presented in Table 9. Table 9 illustrates that as the percentage of large stones increases in the streambed, the estimated total armoring scour depth becomes smaller.

**Table 9 – Estimated Armoring Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $D_{50}$ (feet) | e | $P_x$ | T (feet) | $z_t$ (feet) |
|---|---|---|---|---|---|---|
| 1 | 4 | 0.2554 | 0.301 | 0.23 | 2.62 | 2.21 |
| 2 | 3 | 0.2375 | 0.302 | 0.11 | 10.81 | 9.96 |
| 3 | 2 | 0.1569 | 0.307 | 0.01 | 135.01 | 134.00 |
| 4 | 1 | 0.3120 | 0.299 | 0.04 | 34.27 | 33.25 |
| | 8 | 0.2045 | 0.304 | 0.01 | 145.92 | 144.89 |

The results presented in Table 5 suggest that the potential for armoring to occur within the VR-1 reach is sporadic, at best, and that, in general, armoring of the streambed would not occur in a continuous and consistent enough manner to control overall changes in the vertical profile of the VR-1 reach.

Consequently, additional assessment of the erosion mechanisms that can affect changes in the vertical profile of the VR-1 reach, both on a short-term (scour) and long-term (degradation) basis, are examined in the following paragraphs.

#### 4.2.3. General Scour

General scour refers to all types of scour that are not local. General scour commonly, but not necessarily, occurs over the entire cross section and may involve reaches of varying length, depending on the type of scour and site-specific conditions. General scour includes contraction scour and bend scour. An empirical approach, presented by Blodgett (1986), was derived from the measurements of 21 sites and yielded the following relationship for mean and maximum scour conditions (it is not indicated whether curvature effects are considered):

$$z_t(\mathrm{mean}) = 1.42 \cdot D_{50}^{-0.115}$$

$$z_t(\mathrm{max}) = 6.5 \cdot D_{50}^{-0.115}$$

When using the preceding equations for the VR-1 reach, the computed total scour depths vary from 1.62 to 8.04 feet, with the average mean and maximum values of 1.68 and 7.71 feet, respectively. The results are presented in Table 10.

**Table 10 – Computed Blodgett Total Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $D_{50}$ (feet) | Mean $z_t$ (feet) | Maximum $z_t$ (feet) |
|---|---|---|---|---|
| 1 | 4 | 0.2554 | 1.66 | 7.60 |
| 2 | 3 | 0.2375 | 1.68 | 7.67 |
| 3 | 2 | 0.1569 | 1.76 | 8.04 |
| 4 | 1 | 0.3120 | 1.62 | 7.43 |
| | 8 | 0.2045 | 1.70 | 7.80 |
| | | Average | 1.68 | 7.71 |

Note: Calculations are based on the approach of Blodgett (1986).

The methods for scour estimation presented in the *Hydrology, Hydraulics, and Sediment Studies for the Meiners Oaks and Live Oak Levees Draft Report* (USBR 2008), were also used herein and presented in the following calculations.

The scour equation of Lacey (1930) is as follows:

$$d_s = 0.47Z\left(\frac{Q}{f}\right)^{1/3}$$

Where:

$d_s$ = depth of scour below thalweg, in feet.
Z = 0.25 for a straight reach, 0.5 for a moderate bend, and 1.25 for a vertical rock bank.
Q = flow rate in channel at design discharge, in cfs.
f = $1.76(D_{50})^{1/2}$.

Using the preceding Lacey equation for the VR-1 reach, the computed scour depths for a straight reach vary from 1.96 to 2.19 feet, with an average depth of 2.06 feet. The results are presented in Table 11.

**Table 11 – Computed Lacey Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $D_{50}$ (mm) | $Z^1$ | $Q^2$ (cfs) | f | $d_s$ (feet) |
|---|---|---|---|---|---|---|
| 1 | 4 | 78.0 | 0.25 | 79,166 | 15.54 | 2.02 |
| 2 | 3 | 72.0 | 0.25 | 79,166 | 14.93 | 2.05 |
| 3 | 2 | 48.0 | 0.25 | 79,166 | 12.19 | 2.19 |
| 4 | 1 | 95.0 | 0.25 | 79,166 | 17.15 | 1.96 |
| | 8 | 62.0 | 0.25 | 79,166 | 13.86 | 2.10 |
| | | | | | Average | 2.06 |

Note: Calculations are based on the equation of Lacey (1930).
1. The *Hydrology, Hydraulics, and Sediment Studies for the Meiners Oaks and Live Oak Levees Draft Report* conservatively used 1.25 for Z (USBR 2008). This study used a Z of 0.25 (for a straight reach).
2. 100-year discharge within the VR-1 reach

The scour equation of Blench (1969) is as follows:

$$d_s = Z \frac{q_f^{2/3}}{F_{bo}^{1/3}}$$

Where:

Z = 0.6 for a straight reach, 1.0 for a moderate bend, and 1.25 for a vertical rock bank or wall.
$q_f$ = design discharge per unit width, in cfs/foot.
$F_{bo}$ = $1.75(d_{50})^{0.25}$.

Using the preceding Blench equation for the VR-1 reach, the computed scour depths for a straight reach vary from 9.54 to 13.57 feet. The results are presented in Table 12.

**Table 12 – Computed Blench Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $D_{50}$ (mm) | $Z^1$ | $q_f^2$ (cfs/foot) | $F_{bo}$ | $d_s$ (feet) |
|---|---|---|---|---|---|---|
| 1 | 4 | 78.0 | 0.6 | 144.68 | 5.20 | 9.54 |
| 2 | 3 | 72.0 | 0.6 | 191.02 | 5.10 | 11.56 |
| 3 | 2 | 48.0 | 0.6 | 337.57 | 4.61 | 17.48 |
| 4 | 1 | 95.0 | 0.6 | 238.29 | 5.46 | 13.10 |
| | 8 | 62.0 | 0.6 | 238.29 | 4.91 | 13.57 |
| | | | | | Average | 13.05 |

Note: Calculations are based on the equation of Blench (1969).

| | |
|---|---|
| 1. | The *Hydrology, Hydraulics, and Sediment Studies for the Meiners Oaks and Live Oak Levees Draft Report* conservatively used 1.0 for Z (USBR 2008). This study used a Z of 0.6 (for a straight reach). |
| 2. | Average 100-year discharge per unit width for each segment. |

The depth of scour, $d_s$, can also be computed by the "limiting velocity method," as described by the USBR (1984). The equation to be used with this method is as follows:

$$d_s = d_m \left(\frac{V_m}{V_c} - 1\right)$$

Where:

$d_m$ = maximum depth, in feet.
$V_m$ = maximum channel velocity, in feet/second.
$V_c$ = minimum competent velocity, in feet/second.

The competent velocity, $V_c$, can be estimated as follows:

$$V_c = 11.17\, Y^{1/6} D_c^{1/3}$$

Where:

Y = maximum depth of flow, in feet.
$D_c$ = $D_{50}$ of surface bed material, in feet.

Using the preceding equation for the limiting velocity method along the VR-1 reach, the computed scour depths vary as presented in Table 13.

**Table 13 – Computed USBR Limiting Velocity Method Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $D_{50}$ (feet) | $V_c$ | $V_m$[1] | $d_m$[1] | $d_s$ (feet) |
|---|---|---|---|---|---|---|
| 1 | 4 | 0.2554 | 10.83 | 11.33 | 12.77 | 0.59 |
| 2 | 3 | 0.2375 | 10.80 | 13.21 | 14.46 | 3.23 |
| 3 | 2 | 0.1569 | 9.74 | 18.85 | 17.91 | 16.75 |
| 4 | 1 | 0.3120 | 11.53 | 19.18 | 12.42 | 8.24 |
| | 8 | 0.2045 | 10.01 | 19.18 | 12.42 | 11.38 |
| | | | | | Average | 8.04 |

Note: Calculations are based on the limiting velocity method (USBR 1984).
1. Maximum 100-year channel velocity and flow depth within each segment.

### 4.2.4. Bedform Scour

Design flow along the VR-1 reach will be at, or near, critical-flow conditions. Ordinarily under such conditions in sandbed streams, upper regime flow would exist, and the types of bedforms expected would be "antidunes." However, due to the large sizes of the $D_{50}$ particles along the VR-1 reach, flow will actually be in the lower flow regime, but dunes are not likely to form, primarily due to the large $D_{50}$ sediment sizes present.

Accordingly, bedform scour depths are assumed to be 0.00 along the VR-1 reach.

### 4.2.5. Bend Scour

The multiple threads of low-flow thalwegs that exist along the VR-1 reach provide the potential for high-velocity flow filaments to strike adjacent channel banks at a large enough angle of curvature to warrant the consideration of a bend-scour component when estimating total streambed scour. Furthermore, over the next 50+ years, there is a potentail for flows within low-flow thalwegs of the Ventura River to strike the adjacent channel banks at any point along the VR-1 reach.

Bend scour can be calculated using a formula developed by the Corps (Maynord 1993):

$$\frac{y_{max}}{y_c} = 1.10\left[1.8 - 0.051\left(\frac{R_c}{W_i}\right) + 0.0084\left(\frac{W_i}{y_c}\right)\right]$$

Where:

$y_{max}$ = maximum water depth at the bend, in feet.
$y_c$ = mean water depth in the crossing upstream of the bend, in feet.
$R_c$ = bend radius of curvature, in feet.
$W_i$ = channel width at the bend inflection point, in feet.

(Note: The coefficient 1.10 represents a Corps-recommended factor of safety.)

On the basis of a review of topograhic and aerial maps of the VR-1 reach, it was determined that reasonable curvature conditions along the study reach are such that $R_c/W_i$ = 3.54 (strike angle of ≈ 28.8 degrees) and $W_i/y_c$ = 20 (minimum value to use when $W_i/y_c$ <20 for thalweg). For these conditions, $y_{max} \approx 2y_c$; therefore, the computed bend scour ($d_b$) = $y_{max}$ - $y_c$ ≈ $y_c$. Thus, the computed bend scour depths vary from 12.42 to 17.91 feet along the four segments of the VR-1 reach. The results are presented in Table 14 for each segment.

**Table 14 – Computed Bend Scour Depths within VR-1 Reach**

| Segment No. | Sample No. | $y_c$ (feet) | $d_b$ (feet) |
|---|---|---|---|
| 1 | 4 | 12.77 | 12.77 |
| 2 | 3 | 14.66 | 14.66 |
| 3 | 2 | 17.91 | 17.91 |
| 4 | 1 and 8 | 12.42 | 12.42 |
| | | Average | 14.44 |

### 4.2.6. FIS and USBR HEC-RAS Models—Thalweg Comparison

The USBR HEC-RAS model (USBR 2004b) was based on the 2001 topographic data, and the current FIS HEC-RAS model (HDR 2010b) was based on the 2005 light detection and ranging (lidar) data. The channel thalweg elevations of both models along the VR-1 system are plotted in Figure 13. In addition, the elevations of the as-built levee toe, the approximate 1974 channel thalweg elevations (assuming 8 feet above the as-built toe elevations), and the approximate 1970 channel thalweg elevations (estimated from Figure 4), are shown in Figure 13.

The approximate thalweg elevations of 1970, 2001, and 2005 are lower than the approximate 1947 channel thalweg profile along most of the VR-1 reach. The approximate thalweg elevations of 1970 are higher than the as-built levee toe elevations, but the 2001 and 2005 thalweg profiles are lower than the as-built levee toe elevations from upstream of Segment 3 to the end of Segment 4. It is noted that the thalweg profile of 2005 shows streambed aggradation in comparison to the thalweg profile of 2001, except from Levee Station 55+00 to Levee Station 80+00 and from Levee Station 95+00 to Levee Station 100+00. The thalweg profile of 2005 also indicates that from approximately the Main Street Bridge to the river mouth, the thalweg profile is close to the approximate thalweg profile of 1947 (i.e., a streambed slope of approximately 0.004 foot/foot). The thalweg profiles of 2001 and 2005 are slightly above or below the as-built toe elevations from approximately Levee Station 70+00 to the confluence of the Ventura River with the Cañada de San Joaquin. By comparing the approximate thalweg profile of 1947 to the lowest thalweg profiles of 2001 and 2005, the maximum depths of degradation, which are located at the upstream portion of the VR-1 system, vary from 11.5 to 14.0 feet, as shown in Figure 13. It should be noted that the original levee in the vicinity of the confluence of the Ventura River and the Cañada de San Juan was buried underneath SR 33 after the completion of the freeway. The flow from the Cañada de San Joaquin is conveyed to the Ventura River by a double 10-foot-wide by 8-foot-high reinforced-concrete box. Therefore, Figure 13 shows a gap in the thalweg profile between the Ventura River and the Cañada San Joaquin, where it is labeled as SR 33 culverts.

#### 4.2.7. Long-Term Streambed Degradation

Long-term streambed degradation occurs as a result of disruption in system sediment continuity. In the Ventura River, such disruption is almost entirely the result of human-made changes to the fluvial characteristics of the contributing watershed, such as the construction of dams and other impoundments, watershed urbanization, and the confinement of flood flows as a consequence of levee construction. Because this disruptive process occurs on a watershed-wide basis, it generally acts independently of scour processes that occur over the short term (e.g., during individual flood events), although short-term scour is often an inherent component of long-term degradation processes.

As noted previously herein, over a 32-year period (1970 to 2001), the VR-1 reach experienced significant long-term streambed degradation, ranging between 7.0 and 8.0 feet (i.e., 0.22 and 0.25 foot/year, respectively—an average of 0.24 foot/year). As noted above, streambed degradation has been caused primarily by (1) the construction of Matilija Dam; (2) the construction of other significant water-impoundment facilities within the Ventura River Watershed that are located upstream of the VR-1 reach; (3) the confinement of flood flows due to the construction of levees; and (4) to a lesser degree, past and ongoing urbanization of the adjacent and upstream portions of the Ventura River Watershed.

Long-term degradation can be projected using the equilibrium slope method, which is described as follows. Over the past 60+ years, thalweg elevations have not changed much along the VR-1 reach located between the Main Street Bridge and the mouth of the Ventura River. Along this reach, a portion of the streambed slope has remained stable, at approximately 0.004 foot/foot. The streambed slopes along the four designated segments of VR-1 are as follows:

- Segment 1: 0.003138 (encompasses the reach from the Main Street Bridge to the river mouth)
- Segment 2: 0.005665
- Segment 3: 0.005454
- Segment 4: 0.007997

If it is assumed that a slope of 0.004 foot/foot represents the "dynamic equilibrium slope" under current conditions along the VR-1 reach, then projected streambed degradation along each segment (1 through 4) of VR-1 can be computed by assuming that the farthest segment downstream is the pivot point for control of upstream streambed degradation. That is,

Degradation = $\frac{8}{13}\left(\mathrm{S_n} - \mathrm{S_{eq}}\right)\mathrm{L_{seg}}$ (equation from USBR 1987)

Therefore,

For Segment 2, degradation = $0.6154(0.005665 - 0.004)L_2 = Deg_{S2.}$
For Segment 3, degradation = $Deg_{S2} + 0.6154(0.005454 - 0.004)L_3 = Deg_{S3.}$
For Segment 4, degradation = $Deg_{S3} + 0.6154(0.007997 - 0.004)L_4 = Deg_{S4.}$

Accordingly,

$L_2$ = 1,265.70 feet
$L_3$ = 4,999.94 feet
$L_4$ = 2,864.84 feet

Therefore, long-term degradaton based on the equilibrium slope concept becomes the following:

Segment 2 = 0.6154(0.001665)1265.70 = 1.30 feet = $Deg_{S2}$
Segmnet 3 = 1.30 feet +0.6154(0.001454)4999.94 = 1.30 feet +4.47 feet = 5.77 feet = $Deg_{S3}$
Segment 4 = 5.77 feet +0.6154(0.003997)2864.84 = 5.77 feet +7.05 feet = 12.82 feet = $Deg_{S4}$

Consequently, using the equilibrium slope method, streambed degradation varies from 1.30 feet at the upstream end of Segment 2 to 12.82 feet at the upstream end of Segment 4.

The preceding results would be valid in the absence of the removal of Matilija Dam.

The preceding approach indicates that long-term degradation diminishes in the downstream direction, which is consistent with observations over the past 40+ years (since 1970). In fact, in Segment 1, the equilibrium slope method predicts that aggradation should occur, which makes sense given the influence of the Pacific Ocean and its backwater effects.

### 4.3. Conclusions and Recommendations

The results of various scour estimations are summarized in Table 15, and considering all of the scour methods and their applications and limitations, the largest scour depth resulting from a single event is estimated to be equal to the bend scour along Segment 3, which is 17.91 feet.

Single-event scour depths along the other segments are 12.77 feet for Segment 1; 14.46 feet for Segment 2; and 12.42 feet for Segment 4.

**Table 15 – Summary of Computed Average Scour Depths within VR-1 Reach**

| Analysis Methodology | Scour Depth (feet) | | | |
|---|---|---|---|---|
| | Segment 1 | Segment 2 | Segment 3 | Segment 4 |
| Armoring scour depth, NEH Technical Supplement 14B (NRCS 2007) | 2.21 | 9.96 | NA[1] | NA[1,2] |
| Single-event scour, Blodgett (1986) | 1.66 | 1.68 | 1.76 | 1.62/1.70[2] |
| Single-event scour, Lacey (1930) | 2.02 | 2.05 | 2.19 | 1.96/2.10[2] |
| Single-event scour, Blench (1969) | 9.54 | 11.56 | 17.48 | 13.10/13.57[2] |
| Single-event scour, Limiting Velocity Method | 0.59 | 3.23 | 16.79 | 8.24/11.38[2] |
| Bend scour, Maynord (1993) | 12.77 | 14.46 | 17.91 | 12.42 |
| Long-term streambed degradation | NA | 1.30 | 5.77 | 12.82 |

1. Armoring depth would not occur.
2. Values computed based on Sample 1/Sample 8.

NA = not applicable

The USBR report (2004b) concludes that under the hypothetical assumption of an absence of any other human-made improvements in the Ventura River Watershed, removal of Matilija Dam would cause erosional trends in the Ventura River to reverse and, to an extent, become depositional trends, finally trending toward the occurrence of a balanced condition (dynamic equilibrium). The deposition would re-create a riverine morphology, in terms of the characteristics of the channel and riverbed materials, more similar to pre-dam conditions. The time to reach equilibrium for the recommended dam removal alternative would be approximately 20 years. According to the latest schedule (Figure 14), the Matilija Dam Ecosystem Restoration Project is scheduled to be completed in 2018, and the equilibrium conditions could be achieved in 2038 (i.e., the thalweg could return to 1947 pre-dam conditions) (Ventura County 2009).

The USBR report (2004a) does not consider other factors that have also altered the sediment balance within the Ventura River Watershed. For example, the presence of Casitas Dam, and, to a lesser extent, the Robles Diversion Dam and the McDonald Canyon Detention Basin; the Stewart Canyon Debris Basin; and the Dent Debris Basin. Some reduction in sediment supply is also likely due to extensive urbanization in the lower portions of the watershed over the past 60+ years. All of these human-induced influences may still result in some level of system-wide sediment imbalance remaining in the Ventura River system, despite the planned removal of Matilija Dam.

One can estimate the sediment reduction that will remain in the watershed after the removal of Matilija Dam (i.e., from the presence of Casitas Dam, the Robles Diversion Dam, the McDonald Canyon Detention Basin; the Stewart Canyon Debris Basin; the Dent Debris Basin; as well as the extensive urbanization that has occurred in the lower portions of the watershed). This approach will provide an "in-between" estimate of long-term streambed degradation.

It is known that essentially 100 percent of the sediment was removed from sediment supply areas located upstream of Matilija Dam, but not that much was removed from the downstream areas due to the presence of the other human-made features noted above. A determination of the percentage reduction in sediment supply that would still remain after the Matilija Dam removal (assuming that it would be removed) provides a means of estimating the "in-between" long-term degradation value. In this case, the percentage reduction in sediment supply that would remain after the removal of Matilija Dam is based on engineering judgment.

A remaining issue, though, is the timing of the sediment replenishment in the lower portions of the Ventura River after removal of Matilija Dam. If a large flood were to occur immediately after the dam is removed, the sediment wave would not progress downstream as fast as the flood, and downstream degradation (within the VR-1 reach) potentially would still be large—at least temporarily until the sediment wave makes its way down to the VR-1 reach. Accordingly, the "in-between" value should be estimated on the conservative (larger) side, in order to account for this possibility.

The maximum long-term degradation estimate (from the equilibrium slope method) is 12.82 feet at the upstream end of the VR-1 reach. Using engineering judgment and assuming the removal of Matilija Dam, a reasonable "in between" value would be about 50 percent, or approximately 6.4 feet, of long-term degradation at this location. Estimates for long-term degradation along the other segments of the VR-1 Levee should also be adjusted downward in the same way.

Even though long-term streambed degradation will be diminished to some extent after the removal of Matilija Dam, single-event scour also poses a potential risk to the VR-1 system. However, as was the case with the maximum estimate for long-term streambed degradation on a segment-by-segment basis, the magnitude of the maximum single-event scour computed for each segment along the VR-1 Levee has to be tempered based upon the historical fluvial geomorphology of the Ventura River System since construction of the Matilija Dam.

Historical streambed profiles indicate that the Ventura River channel has remained relatively stable, both horizontally and vertically, in downstream reaches where it approaches its outfall into the Pacific Ocean. In addition, due to the presence of several bridge crossings in these downstream reaches, the thalweg of the Ventura River has remained relatively fixed in location, not wandering from side to side, basically remaining along the west side of the primary channel. Consequently, the potential for significant bend scour to occur along these downstream reaches is reduced as a consequence of fixed points (i.e., the bridge crossings) and inherent geomorphologic processes. The downstream reaches affected by these processes, which reduce predicted bend scour, are Segment 1 and Segment 2. It is noted that regular monitoring of these two segments of the VR-1 Levee system should be conducted to assure that the historical geomorphic processes persist into the future that currently control the potential for bend scour.

As a result of the preceding, single-event general scour is the controlling scour mechanism in Segment 1 and Segment 2.

Accordingly, it is recommended that the VR-1 system be protected to the sum of the maximum predicted single-event scour depths along Segments 1 through Segment 4, with scour values appropriately reduced as described in the preceding paragraphs of this document, due either to bend scour or single-event general scour, plus the predicted “in-between” long-term degradation at the upstream end of each of the four segments, as identified, and extended below the streambed thalweg profiles that existed in 1947 (i.e., under pre-Matilija Dam conditions). Computational results are presented in Table 16. All previously computed values have been rounded to the nearest whole foot.

**Table 16 – Computed Total Scour Depths within VR-1 Reach**

| | Scour-Depth Components (feet) | | | |
|---|---|---|---|---|
| **Analysis Methodology** | **Segment 1**[1] | **Segment 2**[1] | **Segment 3** | **Segment 4** |
| Single Event or Bend scour | 5 | 6 | 18 | 12 |
| Long-term streambed degradation[2] | NA | 1 | 3 | 7 |
| Total scour[3] | 5 | 7 | 21 | 19 |

1. Single-event scour is based upon the average value of all methods listed in Table 15, except that a minimum single-event scour of 5 feet applied as a factor of safety to account for nonuniform flow distribution
2. Based on 50 percent of the long-term degradation that would occur without the removal of Matilija Dam
3. Sum of bend scour + long-term streambed degradation

NA = not applicable

## 5. SEDIMENT TRANSPORT ANALYSIS

The USBR study (2004b) sediment analysis was reviewed and is summarized herein. Additional sediment transport analysis is recommended during the final levee design if the future conditions are drastically different from those used in the USBR study.

### 5.1. Sediment Deposition at Matilija Dam

The USBR study (2004b) estimated that an additional 3.5 million cubic yards of sediment will be deposited behind the dam in the next 35 to 40 years, and more material will pass over the dam as the structure becomes less efficient in trapping material during storm events. As Matilija Reservoir continues to fill with sediment, the sediment trapping efficiency will decrease as time progresses, allowing more sediment to pass over the dam (Table 17).

**Table 17 – Projected Deposition with Matilija Dam in Place**

| Year | Dam Crest Elevation (feet) | Estimated Reservoir Storage (acre-feet) | Estimated Trap Efficiency (%) | Estimated Deposited Volume (cubic yards) |
|---|---|---|---|---|
| 2003 | 1,095 | 500 | 45 | 5,800,000 |
| 2010 | 1,095 | 150 | 27 | 6,900,000 |
| 2020 | 1,095 | 45 | 5 | 7,800,000 |
| 2030 | 1,095 | 14 | 0 | 8,600,000 |
| 2040 | 1,095 | 4 | 0 | 9,300,000 |
| 2050 | 1,095 | 1 | 0 | 9,300,000 |
| 2060 | 1,095 | 0 | 0 | 9,300,000 |

The reservoir is predicted to have less than 50 acre-feet of storage by 2020. Aerial photography from 2001 shows the delta to be within 1,200 feet of the dam face. The average rate of delta progression, estimated by comparing aerial photographs taken in 1973, 1985, and 2001, is 46 feet/year. This indicates that the delta would reach the dam face in approximately 25 years. However, it is expected that the delta progression rate will slow and the delta will reach the dam face at the same time the equilibrium condition of the reservoir is achieved, which is projected to be within 35 to 40 years.

### 5.2. Ventura River Morphology (Deposition and Erosion Patterns)

The river and creek beds are mostly dominated by cobbles, although there is a large range of sediment sizes present in the streambeds. Throughout the entire area that was sampled, there were sands interspersed between the larger rocks. The results of the streambed sediment sampling indicate that the particle size generally increases as the upstream distance increases. Near the ocean (RM 0), the average material diameter is approximately 3 inches; just downstream of Matilija Dam (RM 16.5), it increases to over 12 inches. This is consistent with typical sediment distribution in natural river channels, in which materials tend to be coarser in the upstream reaches where the slopes are steeper, and flows have more velocity and, therefore, more energy to transport larger sediment particles. Sampling upstream of the dam was limited to one location. Correlation with trends in downstream material diameter is not possible, due to the presence of the dam.

The Ventura River has experienced significant erosion in the past 30 years, based on comparisons between the 1971 and 2001 surveys. The water leaving Matilija Dam is sediment starved and picks up sediment from the downstream river channel to replenish itself. Erosion, mostly in the form of degradation, has occurred throughout most of the Ventura River, with the exception of a few locations.

It is expected that the Ventura River erosional rate will slow significantly throughout the Ventura River. This conclusion is based on the comparison of selected historical and current cross section measurements, sediment routing, and computation of the required depth for full armoring. In the future, storm flows downstream of Matilija Dam will not be as sediment starved as sediment trapping behind the dam gradually decreases. Therefore, sediment loads will gradually increase in the Ventura River as less sediment is trapped behind Matilija Dam, and less sediment will erode from the Ventura River streambed. The general geomorphology of the Ventura River is summarized in Table 18 for Reach 2 and Reach 1 (Figure 6) where the VR-1 reach is located (approximately from RM 0.05 to RM 2.5).

**Table 18 – Geomorphic Descriptions of Reaches of Lower Ventura River**

| Reach No. | Landmarks | River Mile/ Physical Structures | General Geomorphic Characteristics |
|---|---|---|---|
| 2b | Foster Park—Shell Road | RM 6.1 to RM 3.0. Two bridge crossings: (1) at Casitas Vista Road (RM 5.95) and (2) at Shell Road (RM 3.2). | Narrow canyon reach opens into wide valley flanked by broad flat alluvial terraces. River channel width remains narrow and becomes deeply incised in alluvium in the lower portion of the reach. Bedrock is exposed in the channel bank at several locations in the upper part of the reach (northern flank of the Ventura Avenue Anticline). |
| 2a | Shell Road—estuary | RM 3.0 to RM 0.6. Upper terminus of Ventura River Levee (RM 2.5) | Similar characteristics as those in Reach 2b, with the exception that the valley and active channel continue to widen in a downstream direction and that no bedrock was observed in the reach. |
| 1 | Mouth of the Ventura River/estuary | RM 0.6 to RM 0.0. Three bridge crossings: (1) at Main Street (RM 0.6), (2) at Highway 101 (RM 0.45), and (3) at Southern Pacific Railroad (RM 0.2). Beginning of the VR-1 (RM 0.05). | The morphology of the reach formed primarily in response to large floods, tidal influence, and coastal processes.<br>Affected by channelization and three bridge crossings. |

The 1970 topographic data represent the river channel as it existed 23 years after completion of Matilija Dam and about 1 year after the large flood events that occurred in January and February of 1969. The 1970 contour data are noted as having a maximum potential error of +/–2.5 feet (USACE 1971). The original coordinates of the 1970 data were not found, so their locations had to be determined from plan-view drawings contained in a 1971 flood report (USACE 19171). Based on these drawings, the 1970 sections are generally within a few tens of feet in longitudinal distance from the 2001 cross section locations to which they are compared in the 2004 USBR report. Therefore, any changes between the 1970 and 2001 thalweg elevations within a range of +/–2.5 feet may only be a reflection of short-duration channel dynamics and error within the data, particularly if it is only at one location. Changes beyond 2.5 feet over a group of cross sections would more likely indicate long-term changes in the channel bed. A 3-point moving average of the in-channel change in bed elevation between 1970 and 2001 was computed. A thalweg value was used, rather than an average channel bed elevation, because the Ventura River is wide and often has multiple bars between channels, which makes it difficult to compute average channel bed elevations. Based on a thalweg comparison, the Ventura River has experienced significant erosion at three locations since 1970 (Figure 9).

Reach 2, where VR-1 is located, has had the largest channel changes and widespread degradation since 1970. A maximum degradation of approximately 8.2 feet was observed at the upstream end of VR-1. At RM 0.4 and RM 1.8, the degradation depths were approximately 7 feet. Between 1970 and 2001, floods have occurred relatively frequently, and several have been very large, with the largest (the flood of record) occurring in 1978.

The geomorphology of the Ventura River was also analyzed using available aerial photographs. The primary sources of historical aerial photographs for the Ventura River and for Matilija Creek were the Ventura County Flood Control District, the Ventura County Mapping Department, the U.S. Forest Service, and the U.S. Geological Survey. Photograph sets taken on three dates were selected for use in the USBR study: September 13, 1947; January 30, 1970; and September 9, 2001. In preparation for inclusion in a geographic information system (GIS) and subsequent analysis, the USBR scanned, orthorectified, and combined these photograph sets into a mosaic. Each set of combined photographs was brought into the GIS as a layer and was projected using a single coordinate system (State Plane, Zone 5, North American Datum of 1983 [NAD 83]).

A plot of the active channel widths as they existed in 1947, 1970, and 2001 is depicted in Figure 15. The most striking conclusion from the graph is the similarity of channel section widths in 1947 and 2001, especially when compared to the large channel section widths that existed in 1970. The major cause of the large section widths in 1970 was the extreme nature of the 1969 flood. The 1969 flood peak was high enough and the flow duration was long enough to remove large amounts of vegetation from the floodplains and rework the channel significantly. After the 1969 flood, the channel gradually returned to a narrow width as vegetation repopulated the floodplain.

Based on the interpretation of aerial photographs, it appears that the coarse sediment along the Ventura River is almost unlimited in supply. In addition to the sediment yield from the basin, a tremendous amount of sediment is currently stored in the floodplain and fluvial terraces along the river. Despite the general belief that the largest proportion of the total sediment load in a river is transported by flows that are in the range of the mean annual flood (Wolman and Miller 1960) a variety of data from the western United States seems to indicate that the largest proportions of fluvial sediments are actually transported by infrequent, large-magnitude floods. For example, during the 1969 flood season, the suspended sediment flux on the Ventura River was greater than that during the preceding 25 years (Inman and Jenkins 1999). The record of sedimentation at

Matilija Dam supports this conclusion. During the 1969 flood year, the total storage capacity of the dam's reservoir was reduced by about 1,000 acre-feet, or by about 14 percent of its total design storage capacity. This volume of storage reduction is about three times the volume of storage reduction that occurred during the preceding 22 years. Therefore, it appears that the most effective mode of sediment transport within the Ventura River basin occurs during the passage of larger magnitude floods. This idea is also supported by comparisons of historical aerial photography, which indicate that dramatic changes in river channel morphology occur immediately after large magnitude floods.

Whereas the geologic setting primarily controls the current morphology of the river, the current climate conditions (during the last 35 years) and the associated hydrology strongly influence the movement of sediment within the river system and, consequently, the channel form. Based on the climate regime and the geomorphology, it is apparent that transport of the sediment that is coarser than approximately 10 mm in diameter is limited. That is, there is more coarse sediment available within the contributing drainage basin than the amount that can be transported by the Ventura River. This is largely a reflection of the physiography, in particular the semi-arid climate, the nature of the bedrock, and active tectonics responsible for high uplift rates and steep watershed slopes.

As mentioned previously, since 1971, degradation has been documented along three reaches of the Ventura River. The degradation could occur for any one of, or any combination of, the following five reasons (R1—R5):

- R1: A shift from a relatively dry period to a wet period.
- R2: Trapping of sediment behind Matilija Dam and associated downstream degradation.
- R3: Trapping and removal of sediment at the Robles Diversion Dam.
- R4: Trapping of sediment and water behind Casitas Dam.
- R5: Urbanization of watershed upland areas.

The hydrological record and the large sediment supply in the Ventura River floodplain certainly support R1. Because the coarse sediment sizes are transport-limited, increasing the volume of water will initiate sediment imbalance in the system, which will, in turn, cause degradation. The degradation of the Ventura River may be the result of an increased ability of the river to move sediment, and in this particular case, the movement of sediment stored in the channel and adjacent floodplain. An analysis of the stream gaging records in the Ventura River basin suggests that the 40-year period beginning with the 1969 floods has been a relatively wet period, particularly when compared to the previous 40-year period of record for the same basin.

The impact of Matilija Dam and the Robles Diversion Dam will have the most profound influence on degradation in the reaches of the Ventura River located immediately below Matilija Dam. Because Matilija Creek provides most of the sediment in the reaches above San Antonio Creek, the entrapment of most of its bed-material sediment load at Matilija Dam and Robles Diversion Dam has had a significant effect on degradation in the upper reaches of the Ventura River. Therefore, although R1 is probably the single largest factor leading to the degradation of the Ventura River system, as a whole, R2 through R4 together are likely significant contributing causes of degradation, particularly in those reaches located immediately below Matilija Dam and immediately below the Robles Diversion Dam.

### 5.3. Matilija Dam Removal Alternatives

A full array of structural and nonstructural measures was formulated to address the identified problems and opportunities, including measures related to dam removal, dam retention, mechanical and natural sediment transport, stabilization of deposited sediments, levee and bridge modifications, protection of existing water supply facilities, recreation, and management of exotic and invasive species. These measures were combined to formulate, evaluate, and compare alternative plans to accommodate future conditions at and downstream of Matilija Dam.

The items relating to sediment management for each alternative are briefly described below.

#### 5.3.1. No Action Alternative

The No Action alternative assumes that the dam would remain in-place for a future 50-year period of analysis. The dam would be monitored for safety purposes, but no modifications to the structure are assumed to be necessary.

#### 5.3.2. Alternative 1

Alternative 1 consists of complete removal of the dam, in one phase, and mechanical removal of the trapped sediment, including disposal of the fines and selling of the aggregates. The components are the following:

- Removal of reservoir fines by hydraulic slurry line. There are approximately 2.1 million cubic yards of sediment in the reservoir area, consisting of ±30 percent clay, ±53 percent silt, and ±17 percent sand, which would be removed and deposited on the upland terraces located in the downstream river valley.
- Complete removal of the dam in one stage.
- Construction of a temporary revetment to stabilize the remaining sediment.
- Removal of remaining sediment, by truck, over a specified period of time.

#### 5.3.3. Alternative 2A

Alternative 2A consists of complete removal of the dam, in one phase, and natural (fluvial) transport of a portion of the trapped sediments. Fines in the reservoir area would be hydraulically conveyed (via a slurry pipeline) to a nearby off-site location. The components are the following:

- Removal of reservoir fines by hydraulic slurry line as in Alternative 1.
- Complete removal of the dam in one stage.
- Construction of a pilot channel through sediments.
- Natural erosion of remaining sediment.

#### 5.3.4. Alternative 2B

Alternative 2B consists of complete removal of the dam, in one phase, and natural (fluvial) transport of all of the trapped sediments behind the dam, including the natural transport of "reservoir area" fines.

- Complete removal of the dam, in one stage. Some reservoir sediment would be removed from behind the dam to facilitate dam removal. This sediment would be placed on top of the delta sediment and would be allowed to erode along with the delta sediments.
- Construction of a pilot channel through sediments.
- Natural erosion of all remaining sediments.

#### 5.3.5. Alternative 3A

Alternative 3A consists of incremental removal of the dam and natural (fluvial) transport of a portion of the sediments currently trapped behind the dam, including hydraulic conveyance (via a slurry pipeline) of "reservoir area" fines to a nearby off-site location.

- Removal of reservoir fines by hydraulic slurry line, as in Alternative 1.
- Removal of the dam to a low-brink elevation of 1,020 feet.
- A waiting period until the "first flood" passes through reservoir.
- Removal of the remaining portions of the dam.

#### 5.3.6. Alternative 3B

Alternative 3B consists of incremental removal of the dam and natural (fluvial) transport of all of the trapped sediment, including the natural transport of "reservoir area" fines.

- Removal of the dam to a low-brink elevation of 1,020 feet. Some reservoir sediment would be removed from behind the dam to facilitate dam removal. This sediment would be placed on top of the delta sediment and would be allowed to erode along with the delta sediments.
- A waiting period until the "first flood" passes through reservoir.
- Removal of the remaining portions of the dam.

#### 5.3.7. Alternative 4A

Alternative 4A consists of complete removal of the dam, in one phase, and permanent stabilization and storage of the trapped sediments within the reservoir basin.

- Removal of reservoir fines by hydraulic slurry line, as in Alternative 1.
- Complete removal of the dam, in one stage.
- Construction of pilot channel through sediments, and stabilization of all remaining sediments.

### 5.3.8. Alternative 4B

Alternative 4B consists of complete removal of the dam, in one phase, and temporary stabilization and storage of the trapped sediments within the reservoir basin.

- Removal of reservoir fines by hydraulic slurry line, as in Alternative 1.
- Complete removal of the dam, in one stage.
- Construction of a pilot channel through sediments and temporary stabilization of all remaining sediments.
- Staged removal of temporary stabilization structures, until all the structures are removed.
- Allowance of the passage of one flood through the reservoir area before any revetment is removed.
- At least three stages of revetment removal and most likely four separate removals of revetment.

## 5.4. Sediment Transport Model

In the USBR report (2004b), the GSTARS-1D (Generalized Sediment Transport Model for Alluvial River Simulation—One Dimensional) model (Yang et al. 2004) was used to model the sediment transport resulting from the removal of Matilija Dam. The model, which was developed by USBR with support from the EPA, requires multiple inputs that can be divided into three main types: hydrologic input, hydraulic input, and sediment input.

### 5.4.1. Hydrologic Input

Several different hydrological inputs were used in the evaluations of the alternatives. The 1991 to 2001 Ventura River hydrograph was simulated five times in succession to generate a 50-year flood hydrograph. The period 1954 to 1960 was used to represent a “dry hydrograph,” and it was used to analyze the turbidity impacts associated with a drought period. In addition to the long-term simulations, several single floods were simulated. The floods of 1998 and 1991 were chosen as representative floods corresponding to the approximate 15-year and 3- to 4-year floods, respectively. The 100-year flood was simulated by increasing the flows of the 1991 flood by an appropriate multiplication factor to generate a predicted 100-year flood peak.

### 5.4.2. Hydraulic Input

The channel geometry used in the sediment calculations was the same as that used in the floodplain analysis. Cross sections were usually spaced approximately 500 feet apart. The hydraulic roughness coefficients used in the model are listed in Table 19. The GSTARS-1D model does not allow the roughness to change with flow rate or with water depth; therefore, it is necessary to use a constant roughness. The roughness coefficients were increased slightly in the canyon immediately downstream of Matilija Dam because of the presence of large boulders in this channel segment.

Bridges were not included in the sediment model. The bridges that could potentially affect the simulation are the Camino Cielo Bridge and the Santa Ana Bridge. Camino Cielo is a low-flow crossing, and has the potential effect of increasing the sediment deposition immediately upstream of this structure. The impact associated with Camino Cielo is expected to be confined within only the area located at, and not more than approximately 250 feet upstream of, the structure. For the Santa Ana Bridge, the USBR report (2004a) suggested that the bridge be redesigned to accommodate the increased sediment loads. The sediment modeling performed at this location assumed the removal of the Santa Ana Bridge and replacement with a bridge that allows the passage of the 100-year flood.

**Table 19 – Channel Roughness Coefficients Used in the Sediment Model**

| River Mile | Channel Roughness Coefficient |
|---|---|
| >16.47 | 0.050 |
| 16.47 to 15 | 0.065 |
| 15 to 14.5 | 0.055 |
| 14.5 to 0 | 0.045 |

### 5.4.3. Sediment Transport Input

The data and relationships required for calculations of sediment transport are (1) the incoming sediment load, (2) the sediment gradations in the bed and reservoir, (3) transport relationships for noncohesive sediment, (4) transport relationships for cohesive sediment, and (5) initial cohesive sediment bulk density. Detailed descriptions of each sediment-transport component are included in Appendix D of the *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report* (USBR 2004b).

### 5.4.4. Summary of Historical Comparison

The GSTARS-1D model reproduced the general trends of the Ventura River during the period of 1971 to 2001. However, the erosion predicted by the model was, in general, more localized than the erosion observed in the river. In reality, the measured erosion occurred over longer reaches than the reach lengths predicted by the model. A similar behavior is expected when the model is applied to the prediction of sediment transport under the various alternatives.

Although the general behavior of the river will be captured by the model, the model likely will predict deposition that is more localized than that which will actually occur. For example, the model may predict that deposition is severe over a channel distance of a few thousand feet, whereas, in reality, the actual deposition may be spread out over a longer distance.

The model is somewhat sensitive to the roughness coefficient, the critical shear stress, and the active layer thickness associated with corresponding flows. However, the differences are generally small and do not qualitatively change the model predictions. The base parameter set is considered sufficient for the analysis of alternatives. The choice of the optimal set of parameters is difficult because the 1970 data set is incomplete. Most importantly, bed material samples for 1970 are unavailable, and the exact locations of the 1970 cross sections are unknown.

## 5.5. General Description of Impacts of Reach 2 and Reach 1 for Each Alternative Based on Sediment Transport Model

### 5.5.1. Deposition in Reach 2 (Foster Park to Main Street)

**Action Alternatives.** The reach from RM 5.5 to RM 2 has experienced the most erosion of any reach along the Ventura River. The erosion is likely a result of constriction of the channel by bridges, as well as the trapping of upstream watershed sediments by Casitas Dam. For all of the alternatives, the erosion is expected to continue from RM 5 to RM 3 but at different rates over time. Downstream of RM 3, deposition is predicted to occur under all of the alternatives.

**No Action Alternative**. The USBR report (2004a) indicates that there would be up to 4 feet of additional degradation in these reaches over the next 50 years. The erosion would primarily occur between RM 5 and RM 3. At the Shell Road Bridge, at RM 3.2, survey data collected between 1975 and 1994 indicate that approximately 10 feet of bed degradation occurred, along with a narrowing of the channel. Since 1971, degradation has lowered the active channel by almost 16 feet downstream of the Shell Road Bridge near RM 3. The USBR sediment model indicates that there would be an additional 4 feet of erosion in this reach over the next 50 years (Table 20).

**Table 20 – Average Sediment Deposition by Reach under No Action Alternative**

| Reach No. | Average Deposition/Erosion by Reach (feet) | | | | 50-Year Min./Max. Deposition Range per Reach (feet) |
|---|---|---|---|---|---|
| | Year 1 | Year 3 | Year 10 | Year 50 | |
| 2 | –0.1 | –0.1 | –0.2 | –0.1 | –4.0 to 2.0 |
| 1 | 0.3 | 0.7 | 0.8 | 1.4 | 0.0 to 2.0 |

Use of mechanical removal/permanent stabilization (Alternatives 1 and 4a) would yield up to 3 feet of additional degradation in Reach 2 from RM 5 to RM 3 over the next 50 years. Downstream of RM 3, the channel may aggrade 4 to 6 feet. The deposition in the lower portion of Reach 2 is affected by the overprediction of deposition in the estuary reach, which is discussed below.

Use of combined natural transport/temporary stabilization (Alternatives 2a, 2b, 3a, 3b, and 4b) would yield up to 2 feet of additional degradation in the reach from RM 5 to RM 3 over the next 50 years. Downstream of RM 3, the model is showing aggradation of 4 to 9 feet; however, this is likely an overestimate. The deposition in the lower portion of Reach 2 is affected by the overprediction of deposition in the estuary reach, which is discussed below.

### 5.5.2. Deposition in Reach 1 (Estuary Reach)

During the past 80 years, sand supplies from the Ventura River Watershed have been markedly reduced as a result of dam construction, watershed improvements, and riverbed sand and gravel mining. A 1989 study estimated that the Ventura River now delivers roughly 70 percent of its former natural yield of sands to the ocean (BEACON 1989). The sediment-transport modeling for the 1989 study resulted in a delivery of about 83 percent of the former equilibrium condition of sand transported to the ocean. Overall, watershed changes have resulted in beach erosion.

The model predicted between 8 feet of deposition for the No Action Alternative and 10 feet of deposition for Alternative 2B. The relative differences between the two alternatives are likely correct, but the magnitudes of depositions are overpredicted. In the comparison with historical data, the actual data from 1970 to 2001 indicate that the reach from RM 1 to RM 0 has remained relatively stable, but the model showed up to 7 feet of deposition. The likely reasons for the discrepancies in the model are the model's inability to correctly represent the flow hydraulics in this reach. In addition, the model does not account for the fact that the ocean currents carry away sediment from the beach. Wave action at the beach erodes the delta of sediment deposited by the Ventura River, and this is not represented in the numerical sediment-transport model. In assessing impacts, it is recommended that the No Action Alternative be assumed to result in no deposition in the estuary reach. For the other alternatives, the differences in deposition between the No Action Alternative and the other alternatives should be used to assess impacts. The No Action Alternative is assumed to result in no deposition in Reach 1 (Table 21). The amount of sediment deposition for each of the other alternatives is the computed difference in deposition between that of the particular alternative and that of the No Action Alternative.

**Table 21 – Summary of Sediment Deposition for All Alternatives after 50 Years of Simulation**

| Reach No. | Average Sediment Deposition/Erosion (feet) | | | | | | |
|---|---|---|---|---|---|---|---|
| | **No Action Alt.** | **Alt. 1 and Alt. 4a** | **Alt. 2a** | **Alt. 2b** | **Alt. 3a** | **Alt. 3b** | **Alt. 4b** |
| 1 | 0.0 | 0.1 | 0.0 | 1.2 | 0.0 | 1.3 | 0.0 |
| 2 | 1.5 | 2.2 | 3.4 | 3.6 | 3.5 | 3.6 | 3.6 |

### 5.6. Comparison and Evaluation of Alternative Plans

Under the hypothetical assumption of an absence of any other human-made improvements in the Ventura River Watershed, removal of Matilija Dam would cause erosional trends in the Ventura River to reverse and become depositional trends, and finally a balanced condition (equilibrium) would result.

The deposition would re-create a riverine morphology, in terms of the characteristics of the channel and riverbed materials more similar to pre-dam conditions. The time required to reach equilibrium would be different for each of the alternatives. Under Alternative 1 and Alternative 4A, equilibrium would be reached in 50 years; under Alternative 2A, Alternative 2B, Alternative 3A, and Alternative 3B, equilibrium would be reached within 10 years; and under Alternative 4B, equilibrium would be reached within approximately 20 years. For the future under the No Action Alternative, equilibrium would occur within approximately 100 years.

### 5.7. Summary of Flood Mitigation

The USBR hydraulic model indicated that all discharges from the 2-year flood peak to the 500-year flood peak would be confined to the main channel by VR-1; therefore, no mitigation would be required for the VR-1 reach.

### 5.8. Recommended Plan

Alternative 4B with the addition of a desilting basin as an associated feature has been chosen as the recommended plan. Removal of Matilija Dam would cause erosion trends downstream to reverse and become depositional trends, eventually restoring more stable (equilibrium) conditions in the Ventura River reaches. The deposition would recreate a riverine morphology, in terms of the characteristics of the channel and riverbed materials, similar to pre-dam conditions. The estimated timeframe required to reach equilibrium is approximately 20 years after the completion of the recommended plan.

The process of returning the river to pre-dam conditions would increase the flooding risk to infrastructure that has developed along the river corridor since the construction of the dam. The recommended plan includes features to mitigate the induced flood risk, including removal of structures, replacement of a bridge, and the raising and extension of downstream levees and floodwalls but none within the VR-1 reach.

### 5.9. VR-1 Channel Hydraulics under USBR No Action and Recommended Plans

In general, the VR-1 reach is subjected to sediment deposition for the no action plan and the recommended plan (Alternative 4B) based on the USBR sediment transport model results as shown in Table 21. USBR sediment transport model estimated no changes in stream bed for Reach 1 (from river mouth to Main Street Bridge) which is coincided with the sediment segment no.1 discussed in Section 4 and 1.5 and 3.6 feet of deposition along the rest of the VR-1 levee reach for the USBR no action and recommended plans, respectively.

Additional HEC-RAS models were performed to investigate the channel hydraulics due to the potential sediment deposition in the future under both scenarios of VR-1 levee reach along the Ventura River. No additional HEC-RAS models were performed for the Cañada de San Joaquin because sediment transport analyses were not available at the time of this study commenced. Two modeling approaches were used to conduct the analyses. One approach was to utilize the fixed sediment elevation option in the HEC-RAS and the other approach was to raise the channel bed uniformly within the areas inundated by the 2-year flood event. The first approach assumed deposition occurred locally in the vicinity of the channel thalweg and the second approach assumed uniformly distribution of the deposition. Tables 22 and 23 compare the computed 100-year water surface elevations of VR-1 levee reach between the baseline conditions and the USBR no action and recommended plans conditions. The results indicate that the VR-1 levee potential deposition reach satisfied the FEMA freeboard criteria for the UBRS no action and recommended plans conditions.

**Table 22 – Comparison of Baseline and USBR No Action Plan**

| HEC-RAS Station | Channel Thalweg[1] (feet) | Computed Water-Surface Elevation (feet) | | | Top-of-Levee Elevation (feet) | Computed Freeboard (feet) | | | FEMA-Required Freeboard |
|---|---|---|---|---|---|---|---|---|---|
| | | Baseline | Approach #1 | Approach #2 | | Baseline | Approach #1 | Approach #2 | |
| Confluence of Cañada de San Joaquin and Ventura River | | | | | | | | | |
| 130+21.47[1] | 74.05 | 88.77 | 88.80 | 89.60 | 107.52 | 18.75 | 18.72 | 17.92 | 3 |
| 128+77.79[1] | 73.25 | 86.07 | 86.12 | 87.22 | 107.50 | 21.43 | 21.38 | 20.28 | 3 |
| 125+97.31[1] | 71.15 | 83.54 | 83.58 | 83.98 | 104.81 | 21.27 | 21.23 | 20.83 | 3 |
| 123+40.06[1] | 70.09 | 81.30 | 81.41 | 81.82 | 100.52 | 19.22 | 19.11 | 18.70 | 3 |
| 117+27.16[1] | 62.75 | 76.71 | 76.84 | 77.17 | 96.48 | 19.77 | 19.64 | 19.31 | 3 |
| 112+51.45[1] | 59.23 | 73.03 | 73.20 | 74.04 | 92.13 | 19.10 | 18.93 | 18.09 | 3 |
| 107+31.75[1] | 58.28 | 69.83 | 70.01 | 70.94 | 88.60 | 18.77 | 18.59 | 17.66 | 3 |
| 101+56.63[1] | 51.14 | 68.42 | 68.68 | 69.16 | 86.15 | 17.73 | 17.47 | 16.99 | 3 |
| 96+36.13[1] | 46.46 | 67.64 | 67.94 | 68.36 | 82.55 | 14.91 | 14.61 | 14.19 | 3 |
| 91+88.07[1] | 43.60 | 62.19 | 62.67 | 63.56 | 79.40 | 17.21 | 16.73 | 15.84 | 3 |
| 86+86.77[1] | 41.59 | 59.31 | 59.42 | 59.64 | 76.08 | 16.77 | 16.66 | 16.44 | 3 |
| 81+75.15[1] | 38.25 | 54.76 | 55.02 | 55.62 | 71.99 | 17.23 | 16.97 | 16.37 | 3 |
| 76+71.02[1] | 38.02 | 53.62 | 53.71 | 54.46 | 67.76 | 14.14 | 14.05 | 13.30 | 3 |
| 71+78.09[1] | 31.85 | 52.49 | 52.67 | 53.35 | 63.85 | 11.36 | 11.18 | 10.50 | 3 |
| 66+72.70[1] | 29.76 | 47.42 | 47.58 | 48.26 | 60.10 | 12.68 | 12.52 | 11.84 | 3 |
| 61+69.65[1] | 24.75 | 46.26 | 46.36 | 46.47 | 56.93 | 10.67 | 10.57 | 10.46 | 3 |
| 56+54.25[1] | 22.57 | 44.63 | 44.82 | 44.77 | 51.90 | 7.27 | 7.08 | 7.13 | 3 |
| 51+44.70[1] | 19.79 | 42.02 | 42.25 | 42.98 | 47.93 | 5.91 | 5.68 | 4.95 | 3 |
| 46+36.19[1] | 19.19 | 38.29 | 38.42 | 38.10 | 43.91 | 5.62 | 5.49 | 5.81 | 3 |
| 41+35.26[1] | 18.68 | 34.76 | 34.84 | 34.90 | 39.42 | 4.66 | 4.58 | 4.52 | 3 |
| 36+21.49[1] | 17.30 | 31.00 | 31.04 | 31.12 | 36.51 | 5.51 | 5.47 | 5.39 | 3 |
| 31+12.68[1] | 12.87 | 27.96 | 27.98 | 28.53 | 33.63 | 5.67 | 5.65 | 5.10 | 3 |
| 28+69.57[1] | 11.51 | 27.80 | 27.82 | 28.23 | 34.12 | 6.32 | 6.30 | 5.89 | 4 |
| Main Street Bridge | | | | | | | | | |
| 27+33.19 | 10.77 | 26.38 | 26.38 | 26.38 | 30.63 | 4.25 | 4.25 | 4.25 | 4 |
| 24+96.20 | 9.62 | 25.92 | 25.92 | 25.92 | 29.89 | 3.97 | 3.97 | 3.97 | 3 |
| 20+56.05 | 5.17 | 25.81 | 25.81 | 25.81 | 29.72 | 3.91 | 3.91 | 3.91 | 4 |
| Highway 101 Bridge | | | | | | | | | |
| 16+51.52 | 5.34 | 22.20 | 22.20 | 22.20 | 25.87 | 3.67 | 3.67 | 3.67 | 4 |
| 10+71.01 | 3.02 | 22.23 | 22.23 | 22.23 | 25.00 | 2.77 | 2.77 | 2.77 | 4 |
| South Pacific Railroad Bridge | | | | | | | | | |
| 6+94.09 | 2.92 | 13.24 | 13.24 | 13.24 | 19.94 | 6.70 | 6.70 | 6.70 | 4 |
| 3+56.51 | 2.74 | 11.35 | 11.35 | 11.35 | 18.26 | 6.91 | 6.91 | 6.91 | 3 |
| 1+62.99 | 2.22 | 10.26 | 10.26 | 10.26 | 17.31 | 7.05 | 7.05 | 7.05 | 3 |
| 0+43.85 | 2.33 | 9.31 | 9.31 | 9.31 | 16.86 | 7.55 | 7.55 | 7.55 | 3.5 |

1. Average deposition is 1.5 feet.

**Table 23 – Comparison of Baseline and USBR Recommended Plan**

| HEC-RAS Station | Channel Thalweg[1] (feet) | Computed Water-Surface Elevation (feet) | | | Top-of-Levee Elevation (feet) | Computed Freeboard (feet) | | | FEMA-Required Freeboard |
|---|---|---|---|---|---|---|---|---|---|
| | | Baseline | Approach #1 | Approach #2 | | Baseline | Approach #1 | Approach #2 | |
| Confluence of Cañada de San Joaquin and Ventura River | | | | | | | | | |
| 130+21.47[1] | 74.05 | 88.77 | 88.92 | 91.40 | 107.52 | 18.75 | 18.60 | 16.12 | 3 |
| 128+77.79[1] | 73.25 | 86.07 | 86.19 | 89.22 | 107.50 | 21.43 | 21.31 | 18.28 | 3 |
| 125+97.31[1] | 71.15 | 83.54 | 83.77 | 85.58 | 104.81 | 21.27 | 21.04 | 19.23 | 3 |
| 123+40.06[1] | 70.09 | 81.30 | 81.86 | 82.68 | 100.52 | 19.22 | 18.66 | 17.84 | 3 |
| 117+27.16[1] | 62.75 | 76.71 | 77.21 | 78.19 | 96.48 | 19.77 | 19.27 | 18.29 | 3 |
| 112+51.45[1] | 59.23 | 73.03 | 74.70 | 75.51 | 92.13 | 19.10 | 17.43 | 16.62 | 3 |
| 107+31.75[1] | 58.28 | 69.83 | 70.83 | 72.84 | 88.60 | 18.77 | 17.77 | 15.76 | 3 |
| 101+56.63[1] | 51.14 | 68.42 | 69.46 | 70.06 | 86.15 | 17.73 | 16.69 | 16.09 | 3 |
| 96+36.13[1] | 46.46 | 67.64 | 68.70 | 69.14 | 82.55 | 14.91 | 13.85 | 13.41 | 3 |
| 91+88.07[1] | 43.60 | 62.19 | 63.94 | 64.80 | 79.40 | 17.21 | 15.46 | 14.60 | 3 |
| 86+86.77[1] | 41.59 | 59.31 | 59.78 | 60.15 | 76.08 | 16.77 | 16.30 | 15.93 | 3 |
| 81+75.15[1] | 38.25 | 54.76 | 55.70 | 56.88 | 71.99 | 17.23 | 16.29 | 15.11 | 3 |
| 76+71.02[1] | 38.02 | 53.62 | 53.98 | 55.72 | 67.76 | 14.14 | 13.78 | 12.04 | 3 |
| 71+78.09[1] | 31.85 | 52.49 | 53.10 | 54.65 | 63.85 | 11.36 | 10.75 | 9.20 | 3 |
| 66+72.70[1] | 29.76 | 47.42 | 48.01 | 49.66 | 60.10 | 12.68 | 12.09 | 10.44 | 3 |
| 61+69.65[1] | 24.75 | 46.26 | 46.84 | 47.08 | 56.93 | 10.67 | 10.09 | 9.85 | 3 |
| 56+54.25[1] | 22.57 | 44.63 | 45.40 | 45.11 | 51.90 | 7.27 | 6.50 | 6.79 | 3 |
| 51+44.70[1] | 19.79 | 42.02 | 42.01 | 43.31 | 47.93 | 5.91 | 5.92 | 4.62 | 3 |
| 46+36.19[1] | 19.19 | 38.29 | 38.65 | 38.78 | 43.91 | 5.62 | 5.26 | 5.13 | 3 |
| 41+35.26[1] | 18.68 | 34.76 | 34.94 | 35.15 | 39.42 | 4.66 | 4.48 | 4.27 | 3 |
| 36+21.49[1] | 17.30 | 31.00 | 31.09 | 31.31 | 36.51 | 5.51 | 5.42 | 5.20 | 3 |
| 31+12.68[1] | 12.87 | 27.96 | 28.15 | 29.54 | 33.63 | 5.67 | 5.48 | 4.09 | 3 |
| 28+69.57[1] | 11.51 | 27.80 | 27.93 | 29.15 | 34.12 | 6.32 | 6.19 | 4.97 | 4 |
| Main Street Bridge | | | | | | | | | |
| 27+33.19 | 10.77 | 26.38 | 26.38 | 26.38 | 30.63 | 4.25 | 4.25 | 4.25 | 4 |
| 24+96.20 | 9.62 | 25.92 | 25.92 | 25.92 | 29.89 | 3.97 | 3.97 | 3.97 | 3 |
| 20+56.05 | 5.17 | 25.81 | 25.81 | 25.81 | 29.72 | 3.91 | 3.91 | 3.91 | 4 |
| Highway 101 Bridge | | | | | | | | | |
| 16+51.52 | 5.34 | 22.20 | 22.20 | 22.20 | 25.87 | 3.67 | 3.67 | 3.67 | 4 |
| 10+71.01 | 3.02 | 22.23 | 22.23 | 22.23 | 25.00 | 2.77 | 2.77 | 2.77 | 4 |
| South Pacific Railroad Bridge | | | | | | | | | |
| 6+94.09 | 2.92 | 13.24 | 13.24 | 13.24 | 19.94 | 6.70 | 6.70 | 6.70 | 4 |
| 3+56.51 | 2.74 | 11.35 | 11.35 | 11.35 | 18.26 | 6.91 | 6.91 | 6.91 | 3 |
| 1+62.99 | 2.22 | 10.26 | 10.26 | 10.26 | 17.31 | 7.05 | 7.05 | 7.05 | 3 |
| 0+43.85 | 2.33 | 9.31 | 9.31 | 9.31 | 16.86 | 7.55 | 7.55 | 7.55 | 3.5 |

1. Average deposition is 3.6 feet.

# 6. RISK AND UNCERTAINTY ANALYSIS

As described in Engineer Circular (EC) 1110-2-6067 (USACE 2010b), an assessment of the probability of capacity exceedance and the preparation of an uncertainty analysis of levee containment are both required by the Corps for all new and existing levees. The analysis procedures are described in Engineer Manual (EM) 1100-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies (USACE 1996) and Engineer Regulation (ER) 1105-2-101, Planning Risk Analysis for Flood Damage Reduction Studies (USACE 2006). The manual and regulation discuss uncertainties associated with various elements, such as discharge-probability, stage-discharge, structural and geotechnical performances, and so on, as well as methodologies to determine them. However, the current Corps methodology for the risk and uncertainty (R&U) analysis is not capable of integrating structural and geotechnical uncertainties into the analysis. Therefore, the analyses for only discharge-probability and stage-discharge functions are included in this study.

## 6.1. Methodology

Per EC 1110-2-6067, the R&U analysis explicitly and analytically estimates the uncertainties of key elements in terms of probability distributions, rather than single-point estimates.

The sources of uncertainty for the analysis used in this study are the following:

- **Discharge-probability.** For a flood or storm event with a given probability of occurrence, there is uncertainty regarding the resulting discharge at a specific location along the stream or river. The reliability of discharge-probability estimates is directly linked to the historical record of the available stream gage data. In those instances where records are short or incomplete, the associated uncertainty increases. In order to address this uncertainty, an analytical or graphical method is typically used to determine statistical distributions of discharge for a range of probabilities at locations throughout the channel reach.

- **Stage-discharge.** For a given discharge, there is uncertainty regarding the resulting water-surface elevation at a given location. Factors contributing to this uncertainty include bedforms, water temperatures, debris or other obstructions, unsteady flow effects, variation in hydraulic roughness with season, sediment transport, channel scour or deposition, changes in channel shape during or as a result of flood events, as well as other factors. In order to address this uncertainty, standard deviation estimates are developed for stages associated with a range of discharges at locations throughout the channel reach.

The Corps has developed software specifically designed for conducting the R&U analysis. This software is referred to as the HEC-Flood Damage Analysis (HEC-FDA) program (Version 1.2.5a [USACE 2010c] was used for this analysis). This program applies a Monte Carlo simulation process, whereby the expected value of damages is determined explicitly through a numerical integration technique accounting for uncertainty in the basic parameters described above. Data requirements for the program include hydrologic and hydraulic data, including water-surface profiles, frequency-discharge relationships, and stage-discharge relationships. For this study, water-surface profiles were developed using the Corps HEC-RAS program. The resultant relationships were then entered into the HEC-FDA program. R&U parameters, as previously described in detail, were also entered into the HEC-FDA program.

The HEC-FDA program reports flood risk in terms of project performance, considering hydrologic and hydraulic factors. In order to describe performance risk in probabilistic terms, the HEC-FDA program produces three statistical measures: (1) annual exceedance probability (AEP), (2) long-term risk, and (3) conditional non-exceedance probability (CNP) by events. AEP represents the chance of having a damaging flood equaled or exceeded in any given year. Long-term risk represents the probability of having one or more damaging floods over a specified period of time. CNP represents the chance of not having a damaging flood, given an event with a specific magnitude.

## 6.2. HEC-FDA Model Setup

### 6.2.1. Discharge-Probability

For this study, the flood flow frequencies for the VR-1 reach along the Ventura River are based on stream gage data acquired during the period of 1933 to 2000 and were developed using an analytical method (see Hydrology Appendix). Systematic record length is recommended by EM 1110-2-1619 for an analytical distribution fitted with a long-period gaged record. Therefore, the total record length of 68 years was adopted as the equivalent record length.

The upstream portion of the VR-1 system is located in the Cañada de San Joaquin tributary. Peak flows for the Cañada de San Joaquin were provided by Ventura County (2010), using the EPA's Hydrologic Simulation Program—FORTRAN (HSPF) model (USEPA 2001). These values were then entered into the HEC-FDA model, and a graphical method was used to determine the statistical distributions for the Cañada de San Joaquin. An equivalent record length of 10 to 15 years is recommended by EM 1110-2-1619 for a rainfall-runoff-routing model without model calibration. An equivalent record length of 10 years was adopted as a conservative measure.

### 6.2.2. Stage-Discharge

For this study, the standard deviations of error for stages associated with a range of discharges were estimated according to the procedures outlined in Section 5.5 of EM 1110-2-1619.

The standard deviation of the total stage uncertainty, St, is expressed by the following equation:

$$\mathrm{St} = \sqrt{\mathrm{Snatural}^2 + \mathrm{Smodel}^2}$$

where:

Snatural = the uncertainty in stage for ungaged stream reaches
Smodel = the uncertainty in the computed water-surface profiles.

Equation 5-5 and Equation 5-7 in EM-1110-2-1619 were used to estimate Snatural and Smodel, respectively.

### 6.2.3. Critical Levee Sections

The critical section along a levee is the cross section with the least amount of freeboard. Accordingly, each cross section along the levee was evaluated, and the one with the least amount of freeboard was selected as the critical levee section.

Three critical levee sections were selected along the VR-1 system downstream of the Main Street Bridge: at HEC-RAS Station 10+71.01 (upstream of the Highway 101 bridge), at Station 16+51.52 (downstream of the Highway 101 bridge), and at Station 20+56.05 (upstream of the SPRR bridge). The computed values of freeboard at these sections are 1.55 feet, 3.12 feet, and 0.38 feet, respectively. The FEMA freeboard requirement at these three critical locations is a minimum of 3 feet, plus an additional 1 foot (for a total of 4 feet) within 100 feet of either side of the bridge. One additional critical levee section was selected at HEC-RAS Station 31+12.68 as the minimum computed freeboard (see Tables 22 and 23) along the VR-1 system upstream of the Main Street Bridge.

Four critical levee sections were selected along the Cañada de San Joaquin, immediately upstream of SR 33: at HEC-RAS Station 7+75.05, at Station 8+14.85, at Station 8+49.36, and at Station 9+99.46. The computed values of freeboard at these sections are 1.39 feet, 0.91 feet, 1.12 feet, and 1.49 feet, respectively. The FEMA freeboard requirement at these four critical locations is a minimum of 3 feet.

Table 24 summarizes the existing top-of-levee elevations, the computed freeboard, and the FEMA-required freeboard for a 1 percent chance exceedance flood (i.e., a 100-year flood event).

**Table 24 –Existing Top-of-Levee Elevations for Critical Levee Sections**

| Critical Section | Reach | Top of Levee (feet) | Computed Freeboard (feet) | FEMA-Required Freeboard (feet) |
|---|---|---|---|---|
| Station 10+71.01 | Between SPRR and Main Street | 25.00[1] | 2.77 | 4[2] |
| Station 16+51.52 | | 25.87[1] | 3.67 | 4[3] |
| Station 20+56.05 | | 29.72[1] | 3.91 | 4[4] |
| Station 31+12.68 | Upstream of Main Street | 33.63 | 4.09 | 3 |
| Station 7+75.05 | Upstream of SR 33 in Cañada de San Joaquin | 107.98 | 1.39 | 3 |
| Station 8+14.85 | | 107.91 | 0.91 | 3 |
| Station 8+49.36 | | 108.12 | 1.12 | 3 |
| Station 9+99.46 | | 108.49 | 1.49 | 3 |

1. Top-of-floodwall elevations.
2. Upstream of SPRR bridge.
3. Downstream of Highway 101 bridge.
4. Upstream of Highway 101 bridge.

The total stage uncertainties that were computed for the VR-1 system are based on Equation 5-5 and Equation 5-7 in EM-1110-2-1619. The results for each critical section are summarized in Table 25. Table 25 shows that the uncertainty associated with the stream reach, Snatural, is higher than the uncertainty associated with the model, Smodel, because all of the critical levee sections are more influenced by the structures (bridges and culverts) that cause flow constriction than by the roughness of the stream channel.

**Table 25 – Total Stage Uncertainty for Critical Levee Sections**

| Critical Section | Reach | $S_t$ | $S_{natural}$ | $S_{model}$ |
|---|---|---|---|---|
| Station 10+71.01 | Downstream of Main Street | 0.88 | 0.80 | 0.38[1] |
| Station 16+51.52 | | 0.90 | 0.76 | 0.47[1] |
| Station 20+56.05 | | 1.31 | 1.25 | 0.40[2] |
| Station 31+12.68 | Upstream of Main Street | 1.13 | 1.09 | 0.33 |
| Station 7+75.05 | Upstream of SR 33 in Cañada de San Joaquin | 1.11 | 1.11 | 0.00[3] |
| Station 8+14.85 | | 1.14 | 1.14 | 0.00[3] |
| Station 8+49.36 | | 1.14 | 1.14 | 0.00[3] |
| Station 9+99.46 | | 1.14 | 1.14 | 0.00[3] |

1. Influenced by SPRR bridge.
2. Influenced by Highway 101 bridge.
3. Influenced by SR 33 culverts.

EC 1110-2-6067 requires that a levee or an incised channel have at least a 90 percent assurance of excluding the 100-year flood for all reaches of the system or have at least a 95 percent assurance of at least 2 feet of freeboard above the 100-year flood.

Table 26 shows a CNP of 94.04 percent for Station 10+71.01, which has 2.77 feet of freeboard above the 100-year flood; however, FEMA requires 3.00 feet of freeboard above the 100-year flood, with an additional 1 foot of required freeboard within 100 feet of a bridge. All other critical sections in the Cañada de San Joaquin have a CNP of less than 90 percent; therefore, the VR-1 system does not satisfy the EC 1110-2-6067 freeboard requirements regardless the reach upstream of Main Street has a CNP above 90 percent and satisfied the FEMA freeboard criteria, given the existing top-of-levee and/or top-of-floodwall elevations.

**Table 26 – Conditional Non-Exceedance Probability for Existing Top-of-Levee Elevations**

| | Conditional Non-Exceedance Probability by Event | | | | | |
|---|---|---|---|---|---|---|
| **Location** | **10%** | **4%** | **2%** | **1%** | **0.4%** | **0.2%** |
| Station 10+71.01 | 0.9897 | 0.9702 | 0.9536 | 0.9404 | 0.9267 | 0.9179 |
| Station 16+51.52 | 0.9981 | 0.9943 | 0.9911 | 0.9886 | 0.9860 | 0.9844 |
| Station 20+56.05 | 0.9977 | 0.9935 | 0.9906 | 0.9882 | 0.9857 | 0.9842 |
| Station 31+12.68 | 0.9997 | 0.9990 | 0.9985 | 0.9981 | 0.9978 | 0.9975 |
| Station 7+75.05 | 0.9999 | 0.9460 | 0.9046 | 0.7614 | 0.6252 | 0.5523 |
| Station 8+14.85 | 0.9997 | 0.9185 | 0.8598 | 0.6761 | 0.5094 | 0.4225 |
| Station 8+49.36 | 0.9998 | 0.9311 | 0.8803 | 0.7146 | 0.5608 | 0.4793 |
| Station 9+99.46 | 0.9999 | 0.9504 | 0.9117 | 0.7798 | 0.6541 | 0.5846 |

### 6.2.4. HEC-FDA Model Results for Proposed Top-of-Levee Elevations

The proposed top-of-levee elevations were set at 3 feet above the 100-year flood elevations for all the critical sections that failed to meet the EC 1110-2-6067 requirements. Comparisons are presented in Table 27.

**Table 27 – Proposed Top-of-Levee Elevations for Critical Levee Sections**

| Critical Section | Reach | Proposed Top-of-Levee Elevation (feet) | Computed Freeboard (feet) | FEMA-Required Freeboard (feet) |
|---|---|---|---|---|
| Station 10+71.01 | Downstream of Main Street | 25.23 | 3.00 | 4[1] |
| Station 16+51.52 | | 25.87[2] | 3.67 | 4[3] |
| Station 20+56.05 | | 29.72[2] | 3.91 | 4[4] |
| Station 7+75.05 | Upstream of SR 33 in Cañada de San Joaquin | 109.59 | 3.00 | 3 |
| Station 8+14.85 | | 110.00 | 3.00 | 3 |
| Station 8+49.36 | | 110.00 | 3.00 | 3 |
| Station 9+99.46 | | 110.00 | 3.00 | 3 |

1. Upstream of SPRR bridge.
2. Unchanged.
3. Downstream of Highway 101 bridge; no change in top-of-levee elevation.
4. Upstream of Highway 101 bridge.

The results of the HEC-FDA model for the proposed top-of-levee elevations are presented in Table 28 and Table 29. Table 28 shows that there is a 0.2 percent chance of flooding (the worst case among the critical sections) in any given year due to overtopping of the levee analyzed along the VR-1 system.

**Table 28 – Performance Described by Annual Exceedance Probability and Long-Term Risk for Proposed Top-of-Levee Elevations**

| Location | Annual Exceedance Probability | | Long-Term Risk (Probability of Exceedance over Indicated Period) | | |
|---|---|---|---|---|---|
| | Median | Expected | 10 Years | 30 Years | 50 Years |
| Station 10+71.01 | 0.0001 | 0.0020 | 0.0201 | 0.0591 | 0.0965 |
| Station 16+51.52 | 0.0001 | 0.0006 | 0.0061 | 0.0181 | 0.0300 |
| Station 20+56.05 | 0.0001 | 0.0007 | 0.0066 | 0.0197 | 0.0326 |
| Station 7+75.05 | 0.0001 | 0.0011 | 0.0109 | 0.0324 | 0.0534 |
| Station 8+14.85 | 0.0001 | 0.0012 | 0.0115 | 0.0342 | 0.0564 |
| Station 8+49.36 | 0.0001 | 0.0012 | 0.0115 | 0.0342 | 0.0563 |
| Station 9+99.46 | 0.0001 | 0.0012 | 0.0115 | 0.0342 | 0.0563 |

Table 29 shows that the CNP for the proposed top-of-levee elevations (i.e., a minimum of 3 feet above the 100-year flood) is greater than the 95 percent assurance and that the VR-1 system satisfies the EC 1110-2-6067 freeboard requirements.

**Table 29 – Conditional Non-Exceedance Probability for Proposed Top-of-Levee Elevations**

| | **Conditional Non-Exceedance Probability by Event** | | | | | |
|---|---|---|---|---|---|---|
| **Location** | **10%** | **4%** | **2%** | **1%** | **0.4%** | **0.2%** |
| Station 10+71.01 | 0.9934 | 0.9808 | 0.9701 | 0.9618 | 0.9532 | 0.9474 |
| Station 16+51.52 | 0.9981 | 0.9943 | 0.9911 | 0.9886 | 0.9860 | 0.9844 |
| Station 20+56.05 | 0.9977 | 0.9935 | 0.9906 | 0.9882 | 0.9857 | 0.9842 |
| Station 7+75.05 | 1.0000 | 0.9935 | 0.9873 | 0.9626 | 0.9374 | 0.9228 |
| Station 8+14.85 | 1.0000 | 0.9931 | 0.9864 | 0.9609 | 0.9347 | 0.9194 |
| Station 8+49.36 | 1.0000 | 0.9929 | 0.9862 | 0.9604 | 0.9348 | 0.9197 |
| Station 9+99.46 | 1.0000 | 0.9931 | 0.9863 | 0.9605 | 0.9351 | 0.9199 |

## REFERENCES


Anderson, A.G., A.S. Paintal, and J. T Davenport. 1970. *Tentative Design Procedure for Riprap Lined Channels*. NCHRP Report Number 108. Highway Research Board. 1970.

BEACON. 1989. *Comprehensive Sand Management Main Report*. Prepared for Beach Erosion Authority for Control Operations and Nourishment by Noble Consultants Inc., Los Angeles, California.

Blench, T., 1969. A Regime Theory Treatment of Canals and Rivers for Engineers and Hydrologists, University of Alberta Press, Edmonton, Canada.

Blodgett, J.C. 1986. *Rock Riprap Design for Protection of Stream Channels near Highway Structures, Volume 1 -- Hydraulic Characteristics of Open Channels*, U.S. Geological Survey Water Resources Investigations Report 86-4127.

FEMA. 2010. *Flood Insurance Study, Ventura County, California and Incorporated Areas*, Federal Emergency Management Agency. January 20, 2010.

Fugro West. 2011. *Periodic Levee Inspections Ventura River 1 Levee (VR-1), Ventura County, California.* Prepared for the U.S. Army Corps of Engineers, Los Angeles District, by Fugro West Inc., Ventura, California.

Grant, G.E. 1977. Critical Flow Constraints Flow Hydraulics in Mobile-Bed Streams: A New Hypothesis. *Water Resource Research* 33(2): 349–358. February 1977.

HDR. 2010a. *Hydraulic Modeling for Cañada de San Joaquin, Ventura River and Tributaries FIS, Ventura County, California, FIS*. February 2010.

HDR. 2010b. *Hydraulic Modeling for Ventura River, Ventura County, California, FIS*. March 2010.

Inman, D.L., and Jenkins, S.A.. 1999. Climate Change and the Episodicity of Sediment Flux of Small California Rivers. *Journal of Geology,* Vol. 107, P 251–270.

Lacey, G. 1930. *Stable Channels in Alluvium,* Paper 4736, Proc. of The Institution of Civil Engineers, Vol. 229, William Clowes & Sons Ltd., London, U.K.

Maynord. S. T. 1993. Flow Impingement. Snake River. Wyoming. Technical Report HL-93-9. US Army Engineer Waterways Experiment Station. Vicksburg, Mississippi.

NRCS (2007). *National Engineering Handbook*, Part 654. Technical Supplement 14B – Scour Calculations. Natural Resources Conservation Service. August 2007.

USACE. 1941. *Preliminary Examination and Survey of Ventura River, Ventura County, California.* U.S. Army Corps of Engineers, Los Angeles District, California.

USACE. 1947a. *Definitive Project Report, Ventura River Improvement, Ventura River Levee, Ocean to Canada de San Joaquin.* U.S. Army Corps of Engineers, Los Angeles District, California. October 1947.

USACE. 1947b. *Definitive Project Report, Ventura River Improvement, Ventura River Levee, Ocean to Canada de San Joaquin.* U.S. Army Corps of Engineers, Los Angeles District, California. November1947.

USACE. 1947c. *As-Built Plans for Ventura River Improvement, Ventura River Levee, Ocean to Canada de San Joaquin.* U.S. Army Corps of Engineers, Los Angeles District, California.

USACE. 1971. Flood Plain Information, Ventura River (Including Coyote Creek). Prepared for Ventura County, California, by U.S. Army Corps of Engineers, Los Angeles District, California.

USACE. 1996. Engineer Manual 1110-2-1619. *Risk-Based Analysis for Flood Damage Reduction Studies. U.S. Army Corps of Engineers*. August 1, 1996.

USACE. 2004. Memorandum for Record: *Hydrology and Hydraulics Policy Memorandum No. 4, Debris Loading on Bridges and Culverts*. U.S. Army Corps of Engineers. August 4, 2004.

USACE. 2006. Engineer Regulation 1105-2-101. *Planning Risk Analysis for Flood Damage Reduction Studies*. U.S. Army Corps of Engineers. January 3, 2006.

USACE. 2010a. Hydrological Engineering Center, HEC-RAS, River Analysis System, Version 4.1.0. U.S. Army Corps of Engineers. January 2010.

USACE. 2010b. Engineer Circular 1110-2-6067. *USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation*. U.S. Army Corps of Engineers. August 31, 2010.

USACE. 2010c. HEC Flood Damage Analysis (HEC-FDA), Version 1.2.5a. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California. October 2010.

USBR 1984. *Computing Degradation and Local Scour*, Technical Guideline for Bureau of Reclamation, Sedimentation and River Hydraulics Section, Engineering and Research Center, Denver, Colorado.

USBR. 1987. *Design of Small Dams*. Third Edition. Bureau of Reclamation. 1987.

USBR. 2004a. *Matilija Dam Ecosystem Restoration Feasibility Study, Final Report.* Main Report. Prepared for the U.S. Army Corps of Engineers, Los Angeles District, California, by the U.S. Bureau of Reclamation.

USBR. 2004b. *Matilija Dam Ecosystem Restoration Feasibility, Ventura County, California, Final Report.* Appendix D – Hydrologic, Hydraulic and Sediment Studies, Ventura River, Ventura County, California. Prepared for the U.S. Army Corps of Engineers, Los Angeles District, California, by the U.S. Bureau of Reclamation.

USBR. 2008. *Hydrology, Hydraulics, and Sediment Studies for the Meiners Oaks and Live Oak Levees Draft Report.* Denver, Colorado.

USEPA. 2001. *Hydrologic Simulation Program—FORTRAN (HSPF) User's Manual.* National Exposure Research Laboratory Office of Research and Development, U.S. Environmental Protection Agency, Athens, Georgia.

USGS. 1989. *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, Water-Supply Paper 2339, U.S. Geological Survey, Denver, Colorado.

U.S. Water Resources Council. 1981. *Guidelines for Determining Flood Flow Frequency.* Bulletin 17B of the Hydrology Committee.

Ventura County. 2009. Matilija Dam Ecosystem Restoration Project website at http://www.matilijadam.org. Ventura County Watershed Protection District, Ventura County, California. Last updated September 16, 2009. Accessed on October 24, 2011.

Ventura County. 2010. *Ventura River Watershed Design Storm Modeling,* Addendum 1, Hydrology Section. Water Resources Technology Division, Ventura County Watershed Protection District, Ventura, California. August 2010.

Wolman, M.G. and J.P. Miller. 1960. *Magnitude and Frequency of Forces in Geomorphic Processes,* Journal of Geology, Vol. 68, No. 1, P 54-74.

Yang, C.T., J. Huang, and B.P. Greimann. 2004. *User's Manual for GSTARS-1D 1.0 (A Generalized Sediment Transport Model for Alluvial River Simulation – One Dimensional version 1.0*, Draft publication of Bureau of Reclamation, Denver, Colorado.

Zeller, M. E. 1981. *Scour Depth Formula for Estimation of Toe Protection Against General Scour*, Pima County Department of Transportation and Flood Control District, Tucson, Arizona.

Zeller, M.E. 1999. Armoring Criteria (unpublished), (email to Chung-Cheng Yen, Tetra Tech, Irvine, California). [Michael Zeller, Principal Water Resources Engineer, Tetra Tech, Tucson, Arizona].

Zeller, M. E. 2011. Personal communication (email to Chung-Cheng Yen, Tetra Tech, Irvine, California). [Michael Zeller, Principal Water Resources Engineer, Tetra Tech, Tucson, Arizona]. September 12, 2011


**Figure 1 – Location Map of VR-1 System**


**Figure 2 – Location Map of VR-1 System and 2010 FIS Cross Sections**

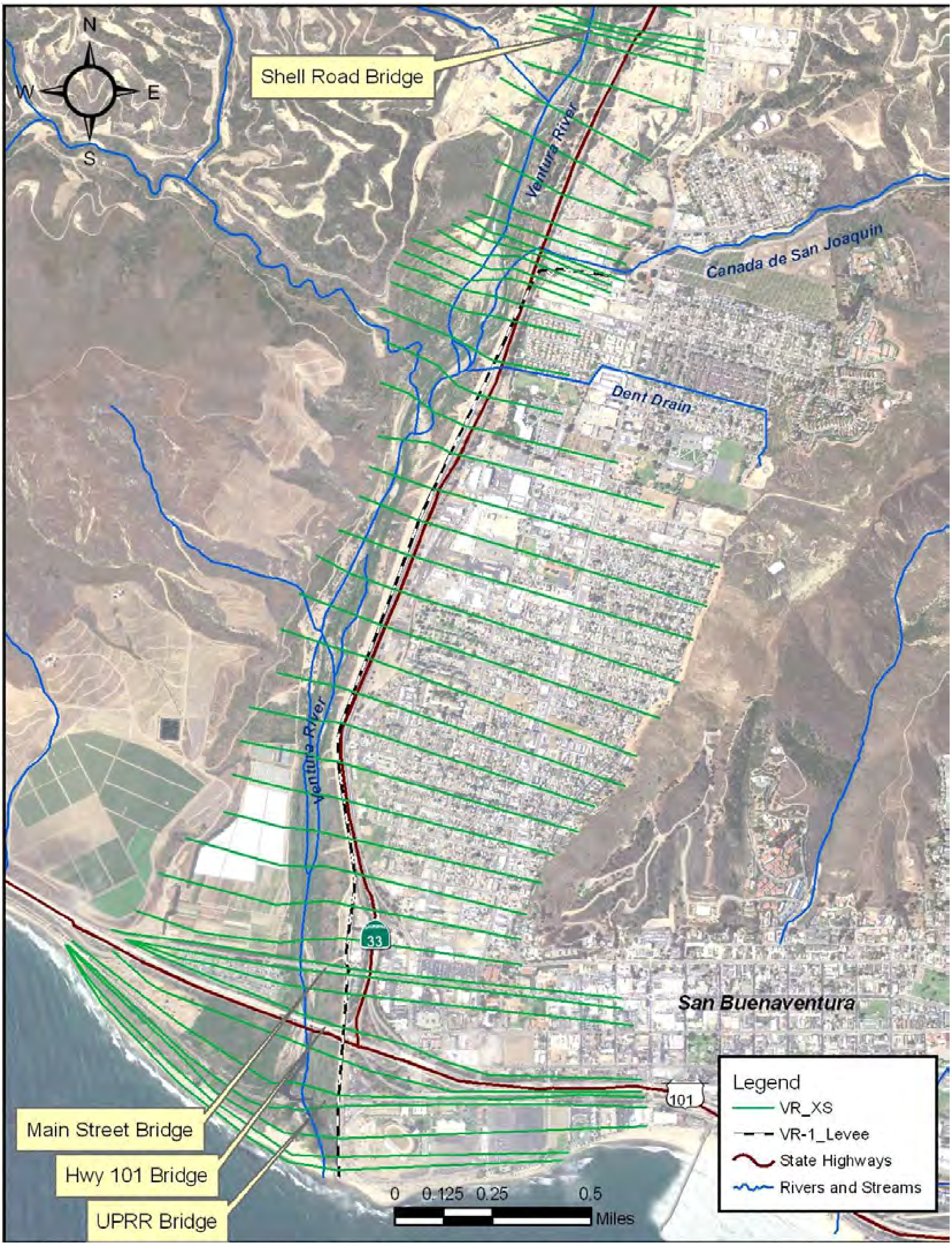


**Figure 3 – Location Map of VR-1 System and Current FIS HEC-RAS Cross Sections**

**Exhibit B6**
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
100-year event

| | River Sta | Q Total (cfs) | Min Ch El (ft) | W.S. Elev (ft) | Crit W.S. (ft) | E.G. Elev (ft) | E.G. Slope (ft/ft) | Vel Chnl (ft/s) | Flow Area (sq ft) | Top Width (ft) | Froude # Chl |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | -0.2258 | 78900 | -18.0 | 2.53 | -8.36 | 2.72 | 0.00018 | 3.5 | 22673.4 | 1479.0 | 0.16 |
| | -0.1784 | 78900 | -14.3 | 2.43 | | 2.88 | 0.00068 | 5.4 | 14665.1 | 1342.0 | 0.29 |
| | -0.1311 | 78900 | -10.5 | 2.30 | | 3.34 | 0.00238 | 8.2 | 9622.0 | 1204.0 | 0.51 |
| | -0.0837 | 78900 | -4.7 | 2.54 | 2.54 | 5.34 | 0.01036 | 13.4 | 5871.0 | 1059.0 | 1.01 |
| | -0.0364 | 78900 | -1.8 | 5.11 | 5.11 | 8.15 | 0.00997 | 14.0 | 5641.6 | 929.0 | 1.00 |
| River Mouth (RM 0) | 0.0477 | 78900 | 2.5 | 10.88 | 10.88 | 13.91 | 0.00821 | 14.1 | 6120.4 | 1359.3 | 0.94 |
| | 0.1052 | 78900 | -0.3 | 14.26 | 10.74 | 14.90 | 0.00138 | 7.6 | 17914.1 | 2483.1 | 0.41 |
| | 0.1383 | 78900 | 1.0 | 14.45 | 10.47 | 15.13 | 0.00145 | 7.9 | 17548.2 | 2386.4 | 0.42 |
| | 0.1909 | 78900 | -1.3 | 14.42 | 10.86 | 15.68 | 0.00076 | 9.7 | 15598.4 | 2858.4 | 0.53 |
| SPRR Bridge | 0.191 | Mult Open | | | | | | | | | |
| | 0.1945 | 78900 | -1.3 | 15.05 | 10.81 | 16.13 | 0.00061 | 9.0 | 17472.6 | 3057.5 | 0.48 |
| | 0.3579 | 78900 | 3.6 | 16.16 | 16.16 | 19.11 | 0.00924 | 13.9 | 5947.0 | 3256.1 | 0.97 |
| | 0.4394 | 78900 | 0.0 | 18.61 | 18.61 | 21.70 | 0.00321 | 14.1 | 5594.4 | 2416.1 | 1.00 |
| HWY 101 Bridge | 0.4395 | Mult Open | | | | | | | | | |
| | 0.464 | 78900 | 0.0 | 22.02 | 18.59 | 23.30 | 0.00075 | 9.1 | 8692.5 | 2907.2 | 0.52 |
| | 0.5204 | 78900 | 5.6 | 22.19 | 21.60 | 23.92 | 0.00381 | 12.2 | 12503.3 | 2756.0 | 0.67 |
| | 0.5922 | 78900 | 6.2 | 24.39 | 24.39 | 26.95 | 0.00340 | 12.8 | 6148.6 | 2637.5 | 1.00 |
| Main Street Bridge | 0.5923 | Mult Open | | | | | | | | | |
| | 0.6028 | 78900 | 6.2 | 26.26 | 24.36 | 27.63 | 0.00122 | 9.4 | 8403.6 | 2805.5 | 0.63 |
| | 0.6629 | 78900 | 11.8 | 26.69 | 26.40 | 28.46 | 0.00455 | 13.0 | 12926.2 | 2826.1 | 0.73 |
| | 0.7577 | 78900 | 15.5 | 29.39 | 29.39 | 31.50 | 0.00556 | 15.1 | 12351.0 | 2531.5 | 0.82 |
| | 0.8523 | 78900 | 17.6 | 33.44 | 33.44 | 36.10 | 0.00398 | 14.8 | 10585.5 | 2253.7 | 0.72 |
| | 0.947 | 78900 | 19.3 | 36.09 | 36.09 | 39.56 | 0.00522 | 16.8 | 8289.6 | 1380.4 | 0.82 |
| | 1.0417 | 78900 | 20.7 | 38.09 | 36.13 | 42.01 | 0.00420 | 16.6 | 6252.8 | 807.4 | 0.76 |
| | 1.1364 | 78900 | 23.0 | 41.69 | 41.69 | 47.34 | 0.00603 | 20.6 | 5692.0 | 665.1 | 0.91 |
| | 1.2311 | 78900 | 25.3 | 46.95 | 41.78 | 49.35 | 0.00215 | 12.8 | 7589.0 | 948.8 | 0.55 |
| | 1.3258 | 78900 | 30.2 | 47.67 | 44.85 | 50.89 | 0.00318 | 15.0 | 6741.5 | 656.1 | 0.67 |
| | 1.4205 | 78900 | 30.6 | 50.61 | 44.51 | 52.10 | 0.00145 | 10.1 | 9857.5 | 999.8 | 0.45 |
| | 1.5152 | 78900 | 36.1 | 51.67 | 47.09 | 52.93 | 0.00178 | 9.0 | 8852.7 | 901.8 | 0.47 |
| | 1.6098 | 78900 | 36.9 | 51.29 | 51.29 | 55.47 | 0.00803 | 16.6 | 5188.8 | 693.8 | 0.97 |
| | 1.7045 | 78900 | 41.8 | 55.96 | 55.96 | 60.44 | 0.00766 | 19.0 | 6125.1 | 692.1 | 0.98 |
| | 1.7992 | 78900 | 43.2 | 61.75 | 61.75 | 67.37 | 0.00663 | 19.9 | 4980.8 | 643.3 | 0.93 |
| | 1.8939 | 78900 | 43.8 | 67.22 | 61.80 | 69.43 | 0.00210 | 12.5 | 7810.4 | 677.3 | 0.54 |
| | 1.9886 | 78900 | 50.2 | 68.67 | 64.56 | 70.57 | 0.00240 | 11.5 | 8147.1 | 731.0 | 0.56 |
| | 2.0827 | 78900 | 53.4 | 69.88 | 67.71 | 72.08 | 0.00342 | 12.7 | 8566.5 | 1052.5 | 0.66 |
| | 2.178 | 78900 | 59.1 | 71.78 | 71.22 | 74.59 | 0.00624 | 14.9 | 7781.0 | 1131.5 | 0.86 |
| | 2.2727 | 78900 | 62.2 | 75.48 | 72.79 | 76.67 | 0.00268 | 8.8 | 9153.4 | 1213.4 | 0.55 |
| | 2.3674 | 78900 | 68.0 | 77.10 | 77.10 | 79.54 | 0.01052 | 12.5 | 6294.4 | 1286.5 | 1.00 |
| | 2.4621 | 78900 | 70.6 | 82.13 | 81.71 | 84.19 | 0.00815 | 11.5 | 6851.5 | 1312.9 | 0.89 |
| Confluence with Canada de San Jaoquin | 2.5568 | 78900 | 72.7 | 89.61 | 89.61 | 94.52 | 0.00736 | 18.7 | 5314.3 | 562.6 | 0.96 |
| | 2.6515 | 78900 | 79.3 | 96.07 | 96.07 | 101.70 | 0.00710 | 20.1 | 4973.1 | 457.6 | 0.96 |
| | 2.7462 | 78900 | 81.7 | 99.46 | 99.46 | 106.11 | 0.00685 | 20.9 | 4132.8 | 346.4 | 0.96 |
| | 2.8409 | 78900 | 82.5 | 105.28 | 101.69 | 108.80 | 0.00350 | 15.1 | 5239.1 | 346.6 | 0.68 |
| | 2.9356 | 78900 | 84.4 | 106.43 | | 111.30 | 0.00509 | 17.8 | 4703.2 | 397.8 | 0.82 |
| | 3.0303 | 78900 | 91.7 | 111.39 | 111.39 | 118.37 | 0.00773 | 21.2 | 3720.4 | 267.3 | 1.00 |
| | 3.125 | 78900 | 95.5 | 116.87 | 116.87 | 124.70 | 0.00758 | 22.5 | 3513.5 | 225.2 | 1.00 |
| | 3.1546 | 78900 | 99.3 | 118.18 | 118.18 | 126.20 | 0.00254 | 22.7 | 3471.8 | 216.8 | 1.00 |

**Figure 4 – USBR HEC-RAS Model Summary Table**

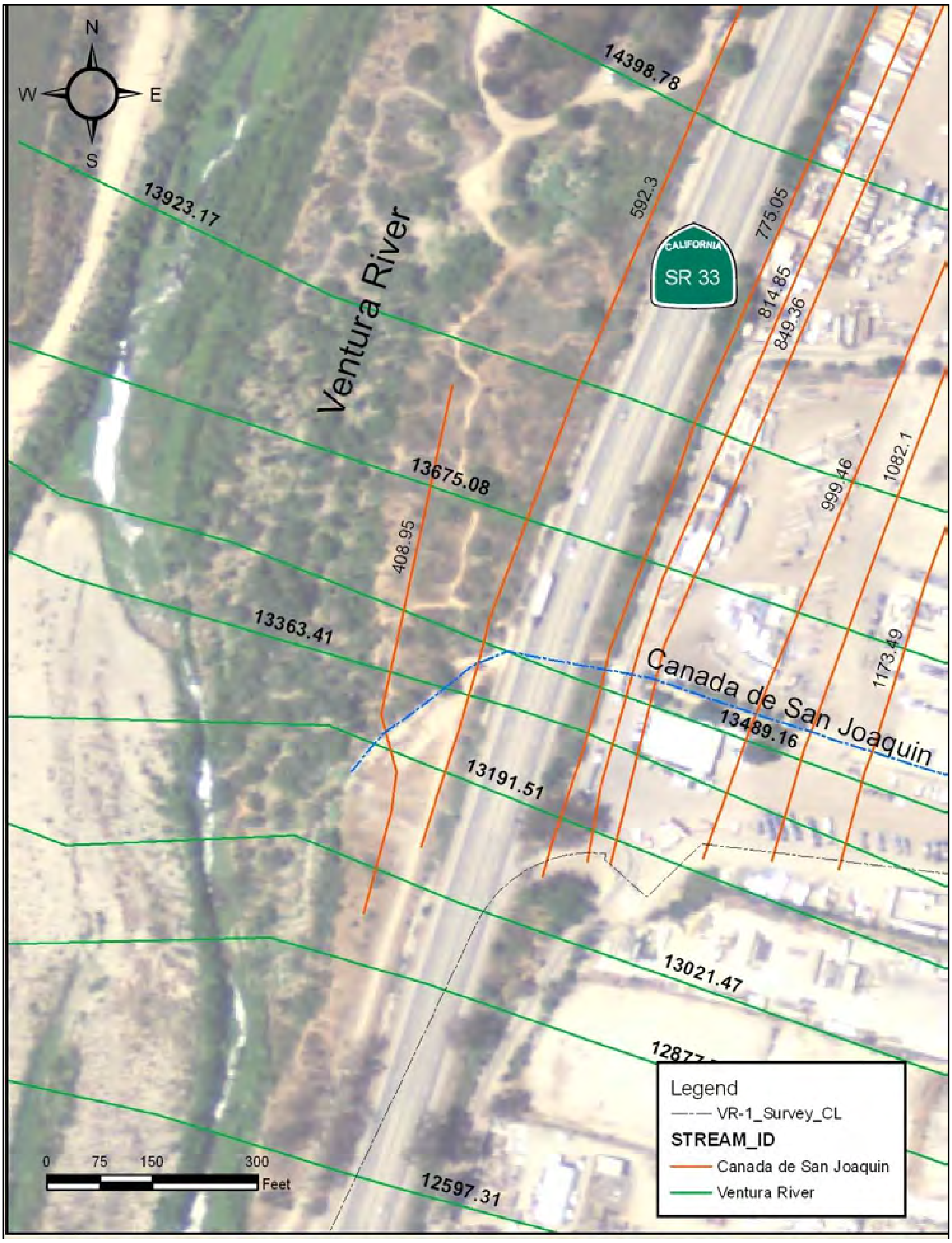


**Figure 5 – Cross Sections at Confluence of Ventura River and Canada de San Joaquin**

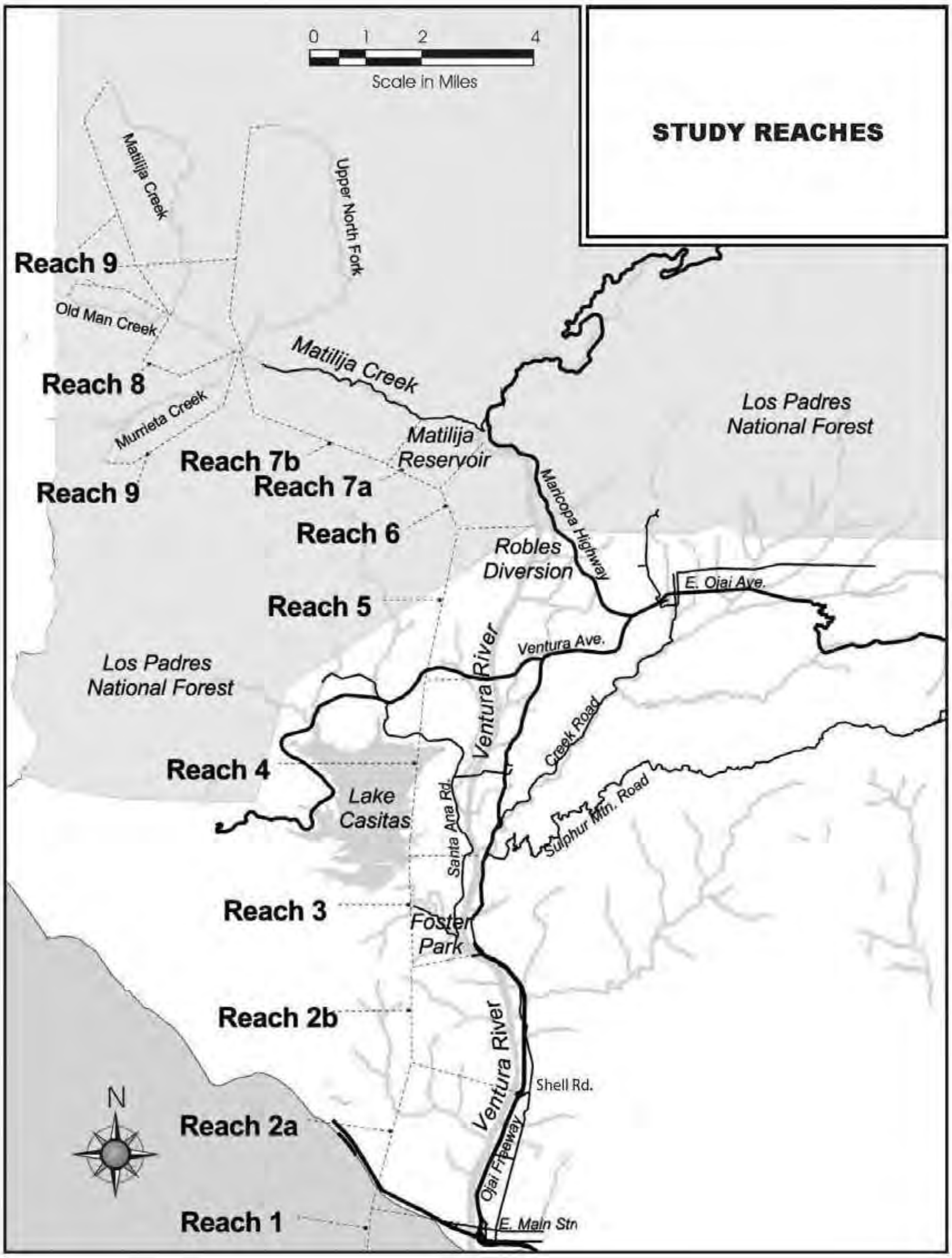


**Figure 6 –USBR Ventura River Study Reaches**

**Figure 7 - Locations of Bed Material Samples**

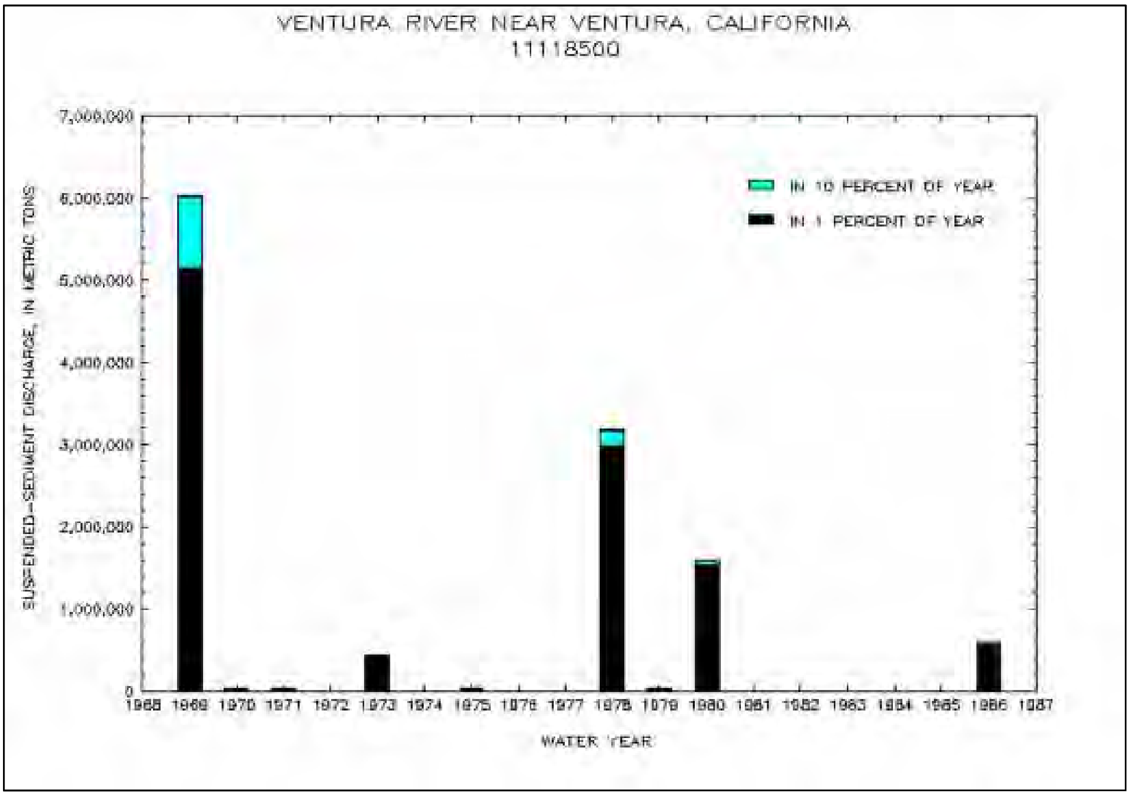


**Figure 8 – Sediment Transport at USGS Gage 11118500**

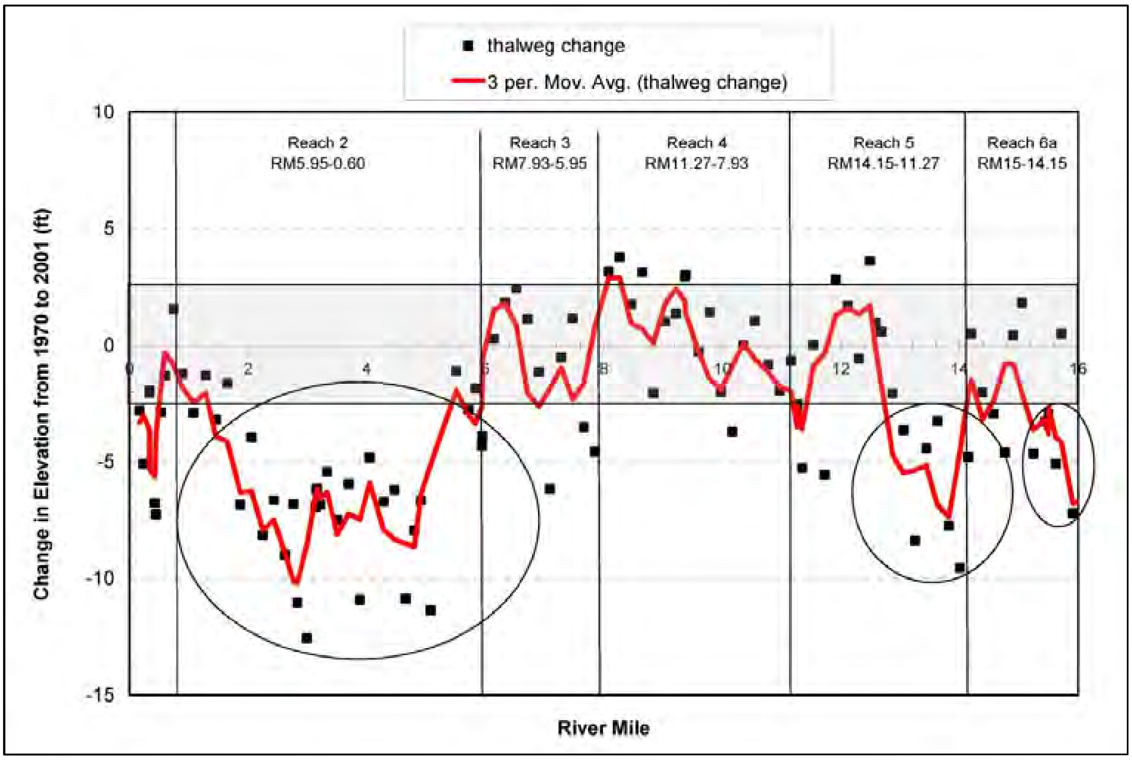


**Figure 9 – Elevation Changes of Ventura River Thalweg between 1970 and 2001**

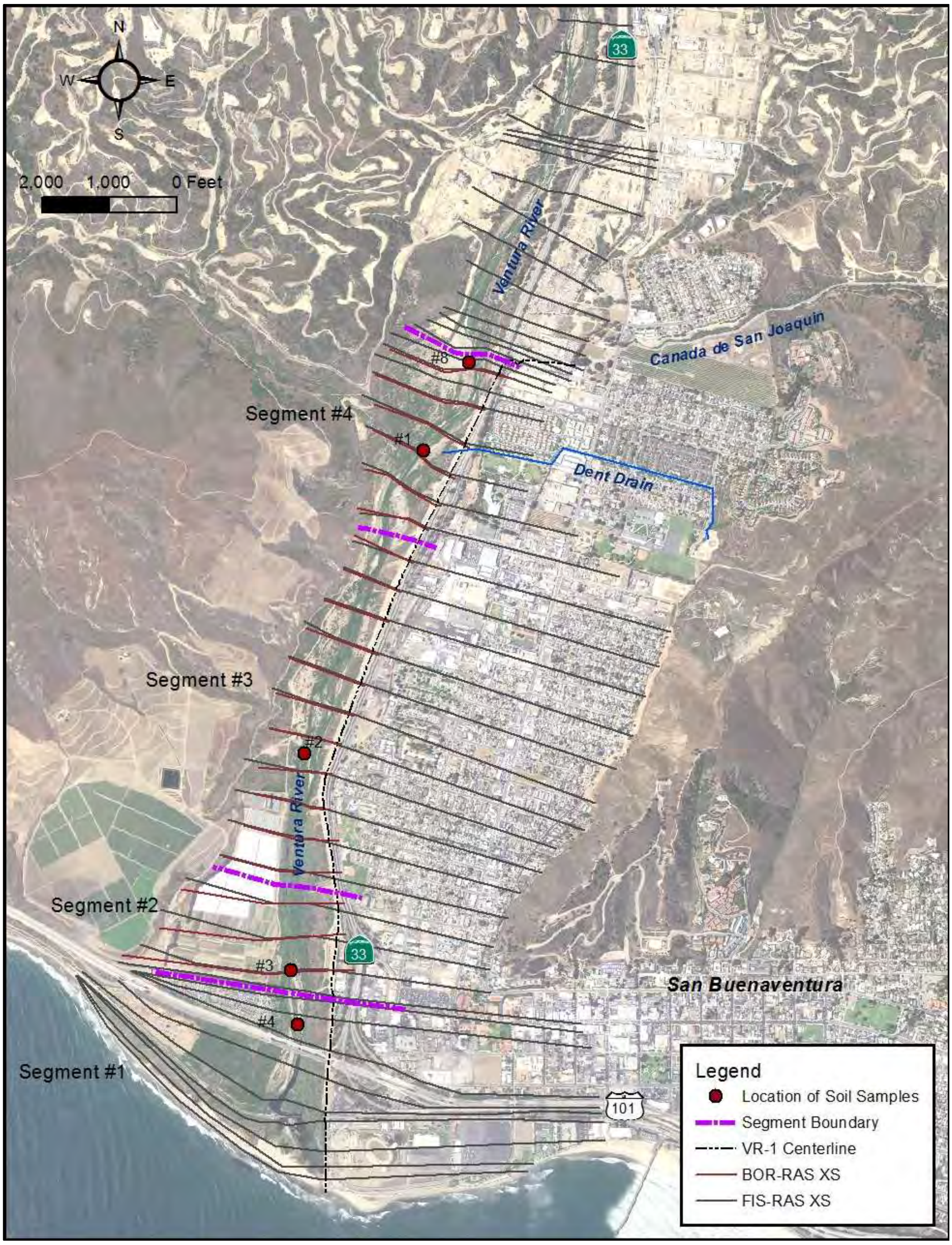


**Figure 10 – Ventura River Scour Calculation Segments**

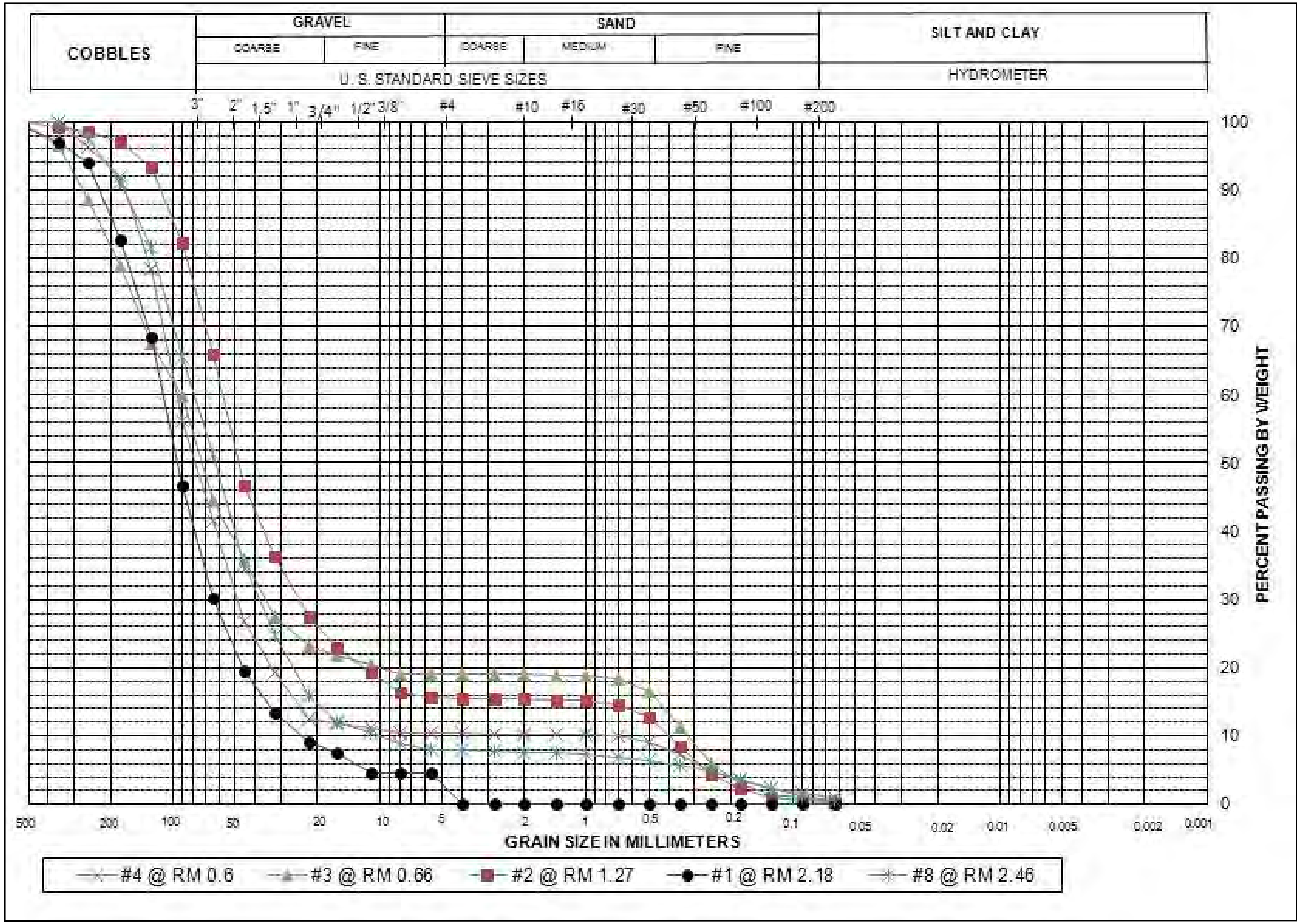


**Figure 11 – Bed Material Gradation Curves**

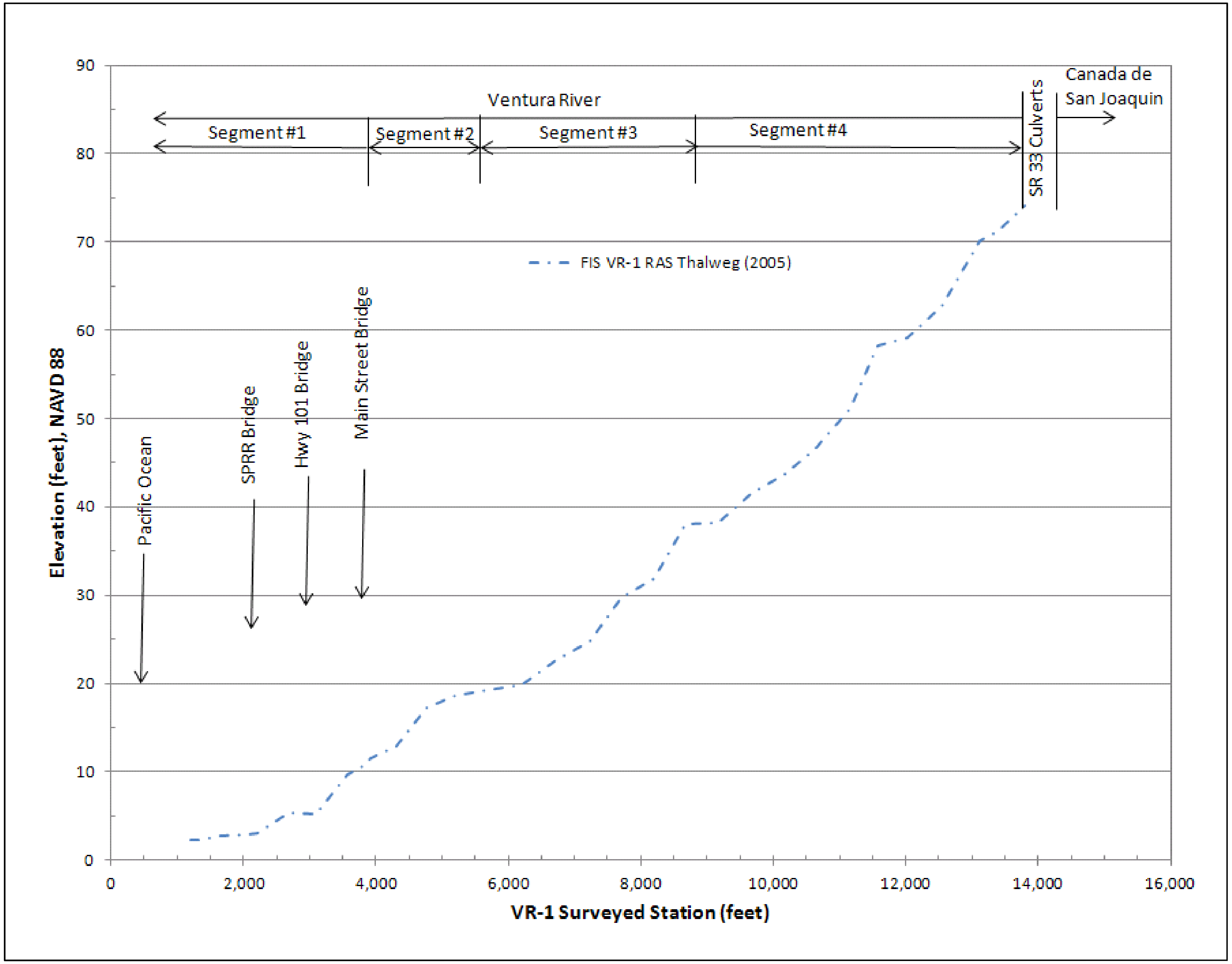


**Figure 12 – Thalweg Profile along VR-1 Reach**

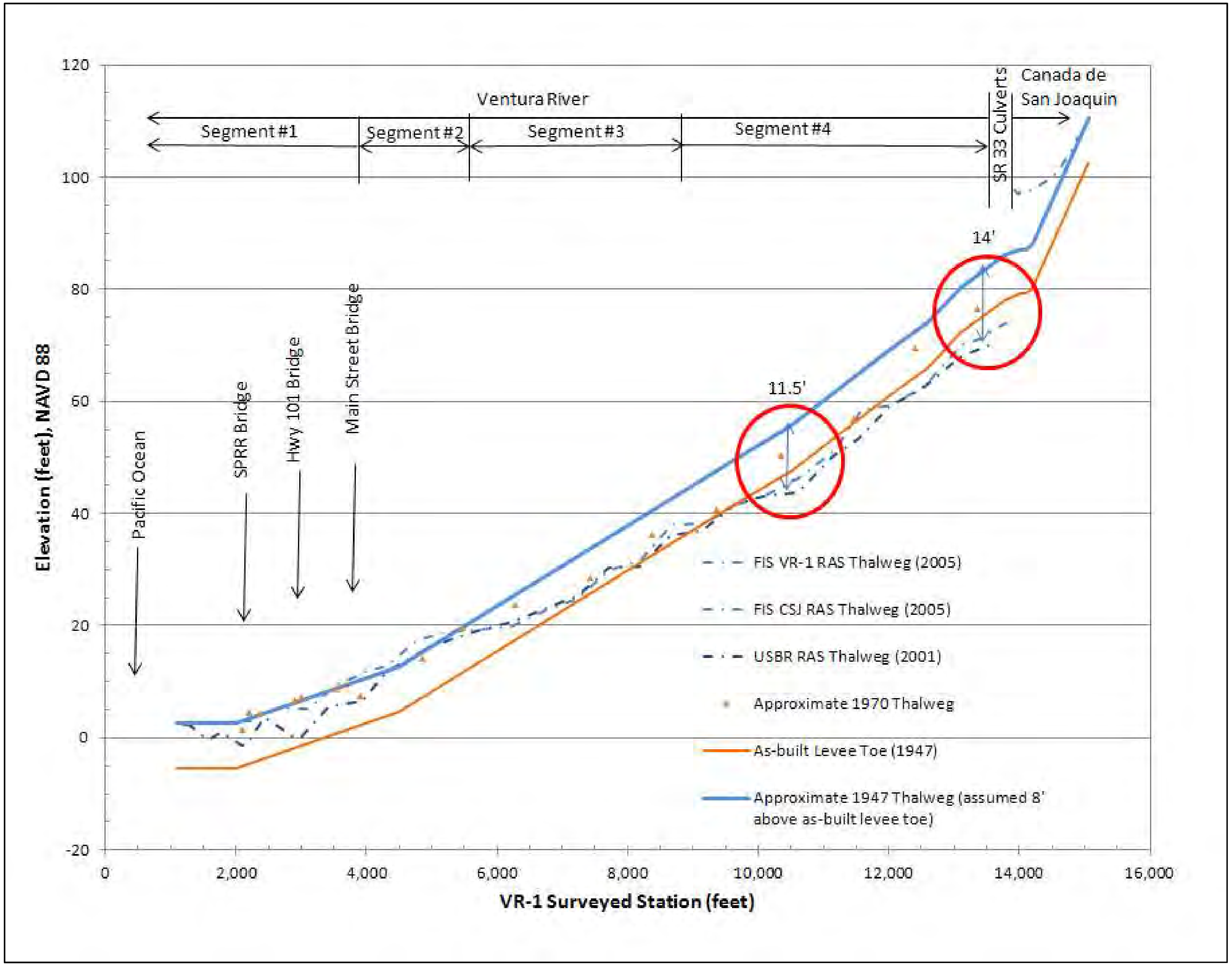


**Figure 13 – Comparison of Thalweg Profiles**

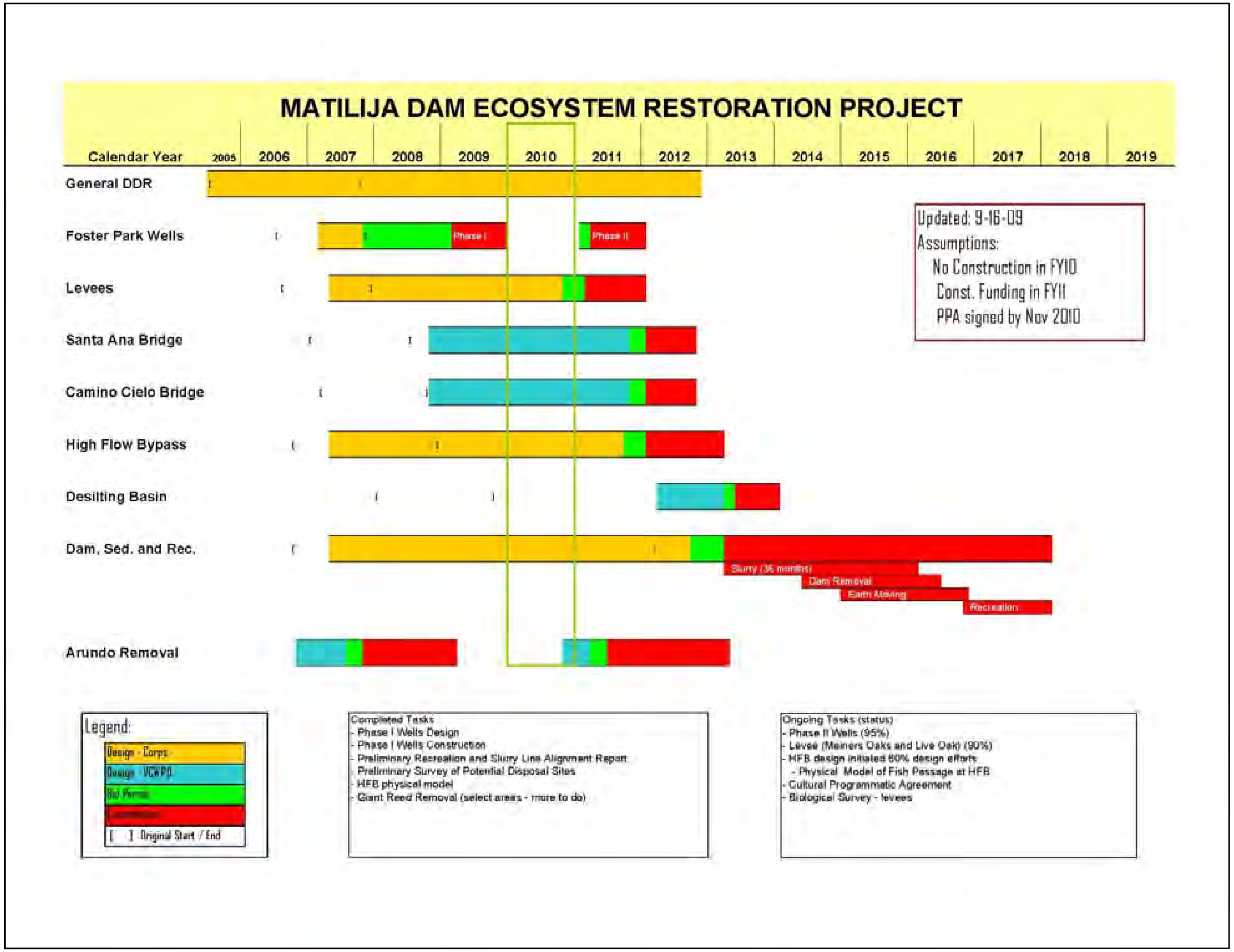


**Figure 14 – Matilija Dam Ecosystem Restoration Project Schedule**

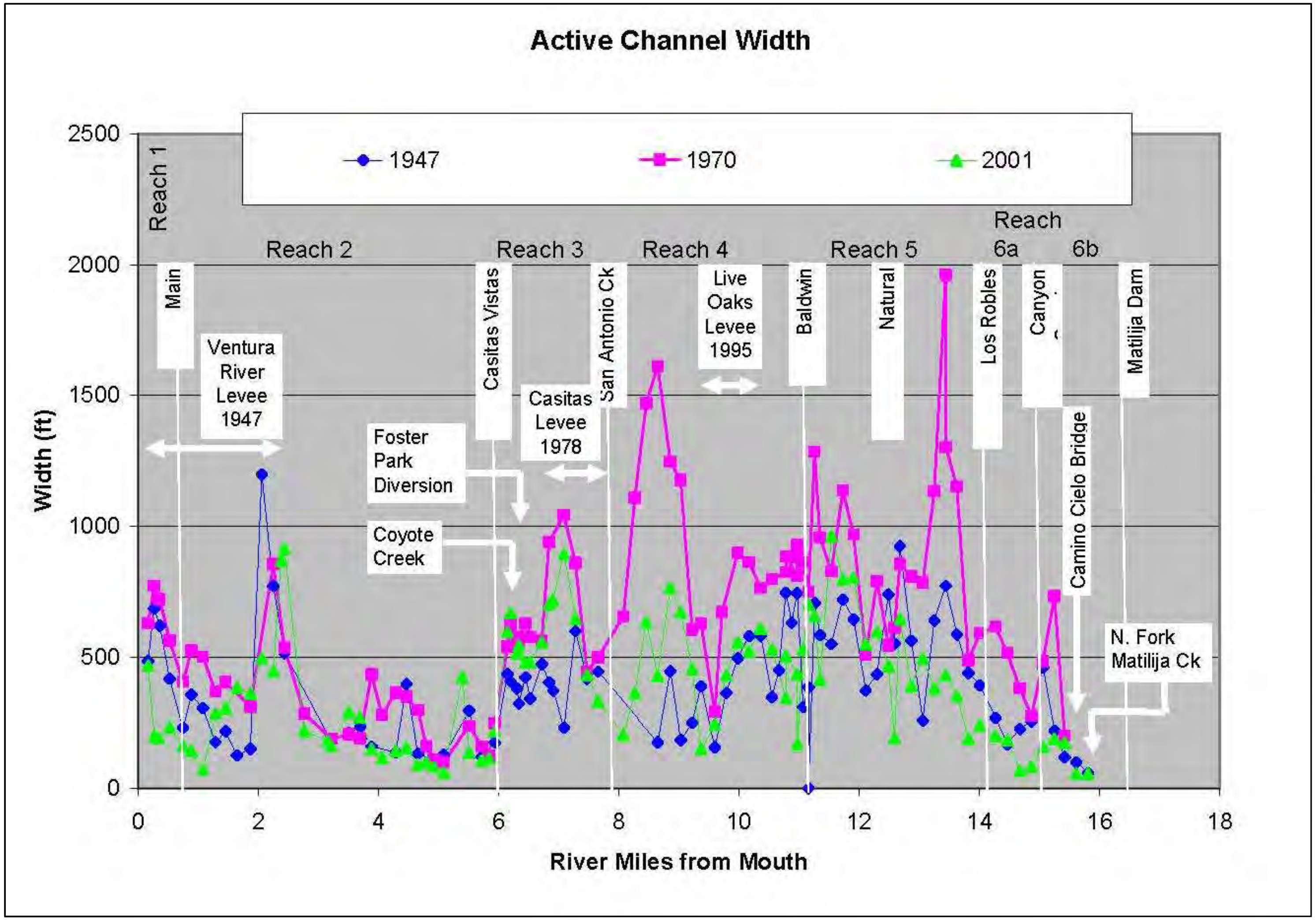


**Figure 15 – Historical Active Channel Widths of the Ventura River in 1947, 1970, and 2001**

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# Appendix D
# Alternatives Plans

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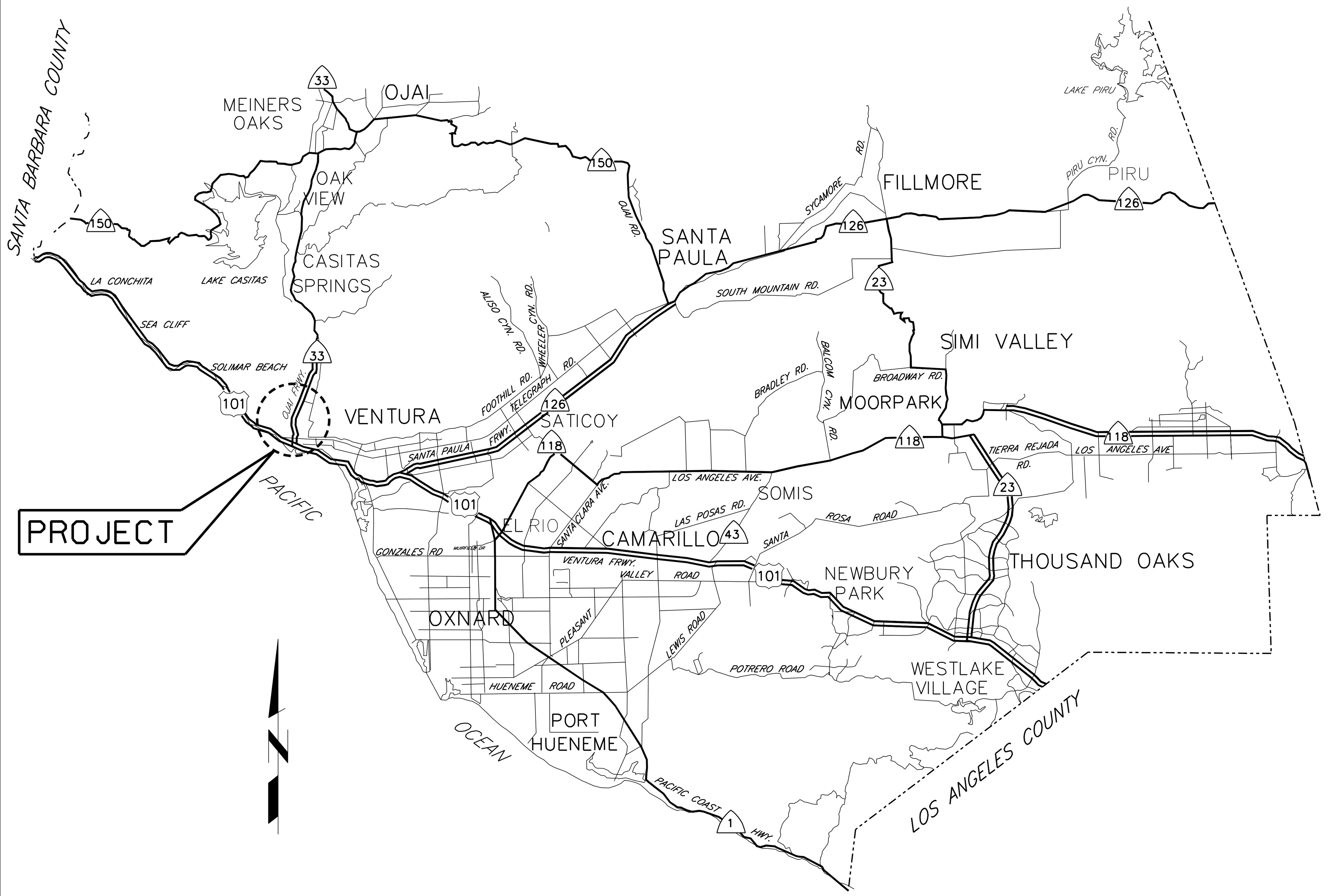
# COUNTY OF VENTURA, CALIFORNIA

# VENTURA RIVER LEVEE EVALUATION AND REHABILITATION

## RIVER MOUTH TO CANADA DE SAN JOAQUIN




VICINITY MAP
N.T.S.

## INDEX TO DRAWINGS

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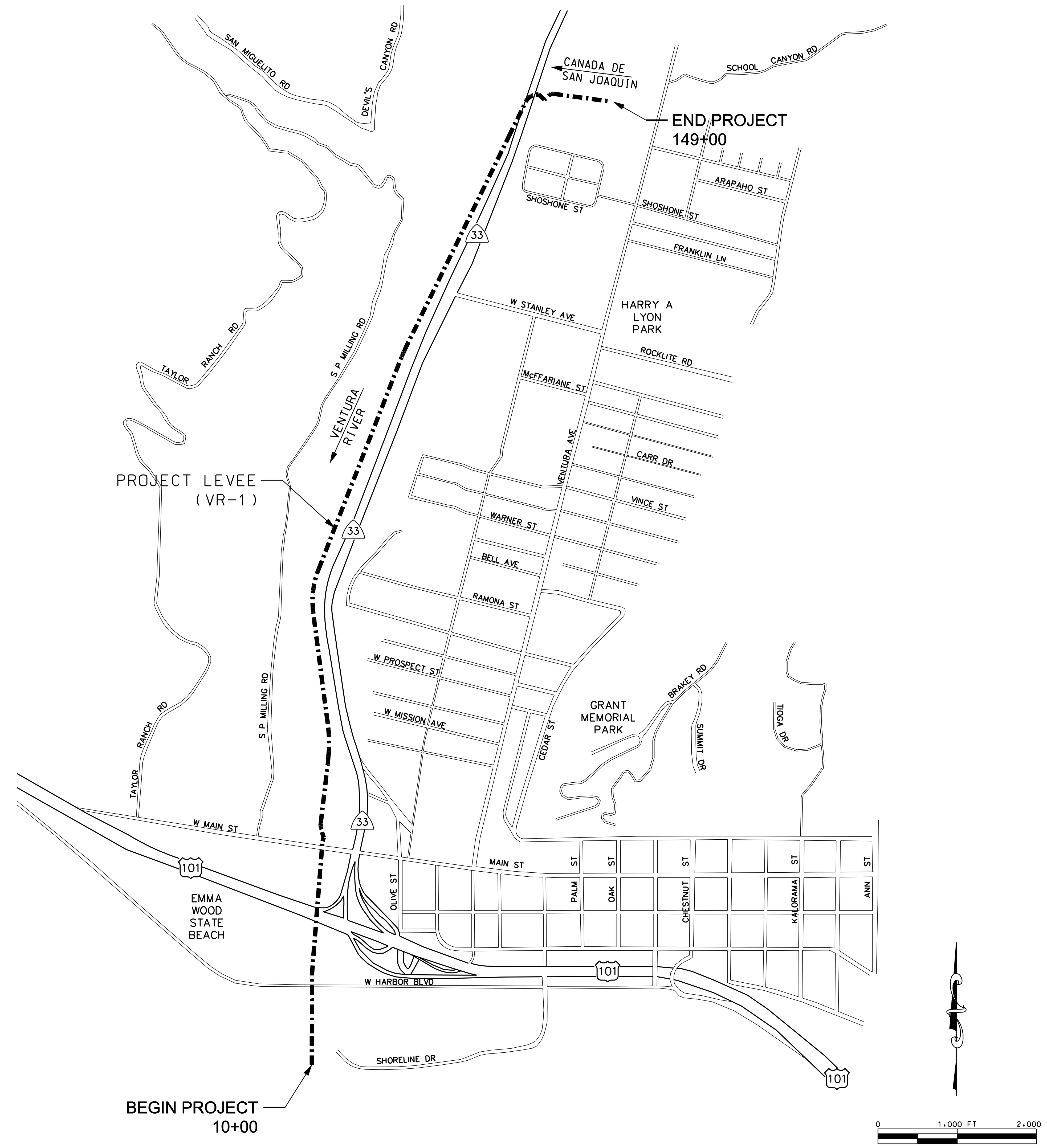
| 2 | ALTERNATIVES PLAN (1) |
| 3 | ALTERNATIVES PLAN (2) |
| 4 | ALTERNATIVES PLAN (3) |
| 5 | ALTERNATIVES PLAN (4) |
| 6 | ALTERNATIVES PLAN (5) |
| 7 | ALTERNATIVES PLAN (6) |
| 8 | ALTERNATIVES PLAN (7) - CANADA DE SAN JOAQUIN |
| 9 | PROFILES (1) LEVEE STATION 10+00 TO 88+00 |
| 10 | PROFILES (2) LEVEE STATION 88+00 TO 149+00 |
| 11 | TYPICAL SECTIONS |

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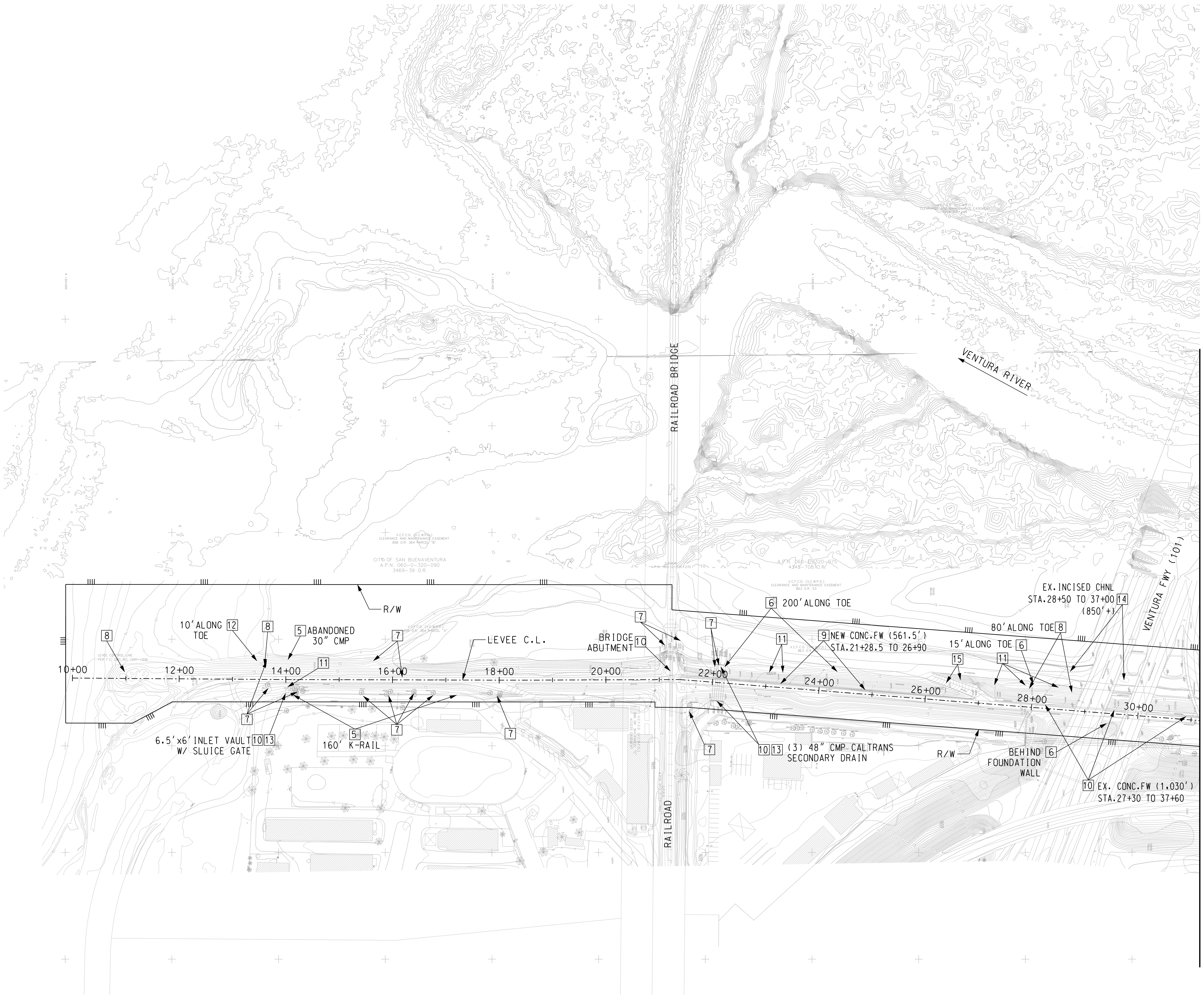
COUNTY OF VENTURA, CALIFORNIA
VENTURA RIVER LEVEE EVALUATION, AND REHABILITATION (VR-1)

GENERAL PLAN

SHEET REFERENCE NUMBER:

SHEET 1 OF 11




MATCHLINE STATION 31+00.00 SEE SHT.3

## CONSTRUCTION NOTES

5 REMOVE UNPERMITTED ENCROACHMENT PER PLAN HEREON.

6 REMOVE SEDIMENT DEPOSITION PER PLAN HEREON.

7 REMOVE VEGETATION WITHIN 15' FROM BOTH RIVERSIDE AND LANDSIDE TOES OF THE LEVEE PER PLAN HEREON.

8 RESTORE DISPLACED RIPRAP PLACEMENT PER PLAN HEREON.

9 REMOVE EX. CONCRETE CURB AND CONSTRUCT CONC. FLOODWALL PER PLAN HEREON AND PROFILE ON SHT.9.

10 REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

11 REMOVE ANIMAL BURROWS BY APPLYING COMPACTED FILL PER PLAN HEREON.

12 REPAIR LEVEE EMBANKMENT EROSION PER PLAN HEREON.

13 RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

14 FILL IN INCISED CHANNEL NEAR RIVERSIDE TOE OF LEVEE PER PLAN HEREON.

15 REGRADE SIDESLOPE OF RAMP TO BE FLATTER THAN 2:1 AND PLACE RIPRAP PER HEREON.

0 200 400

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ALTERNATIVES PLAN (1)

SHEET REFERENCE NUMBER:


## CONSTRUCTION NOTES

5 REMOVE UNPERMITTED ENCROACHMENT PER PLAN HEREON.

6 REMOVE SEDIMENT DEPOSITION PER PLAN HEREON.

7 REMOVE VEGETATION WITHIN 15' FROM BOTH RIVERSIDE AND LANDSIDE TOES OF THE LEVEE PER PLAN HEREON.

8 RESTORE DISPLACED RIPRAP PLACEMENT PER PLAN HEREON.

10 REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

11 REMOVE ANIMAL BURROWS BY APPLYING COMPACTED FILL PER PLAN HEREON.

13 RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

14 FILL IN INCISED CHANNEL NEAR RIVERSIDE TOE OF LEVEE PER PLAN HEREON.

15 REGRADE SIDESLOPE OF RAMP TO BE FLATTER THAN 2:1 AND PLACE RIPRAP PER HEREON.

18 REPAIR LEVEE EMBANKMENT TO THE AS-BUILT CONDITIONS PER PLAN HEREON.

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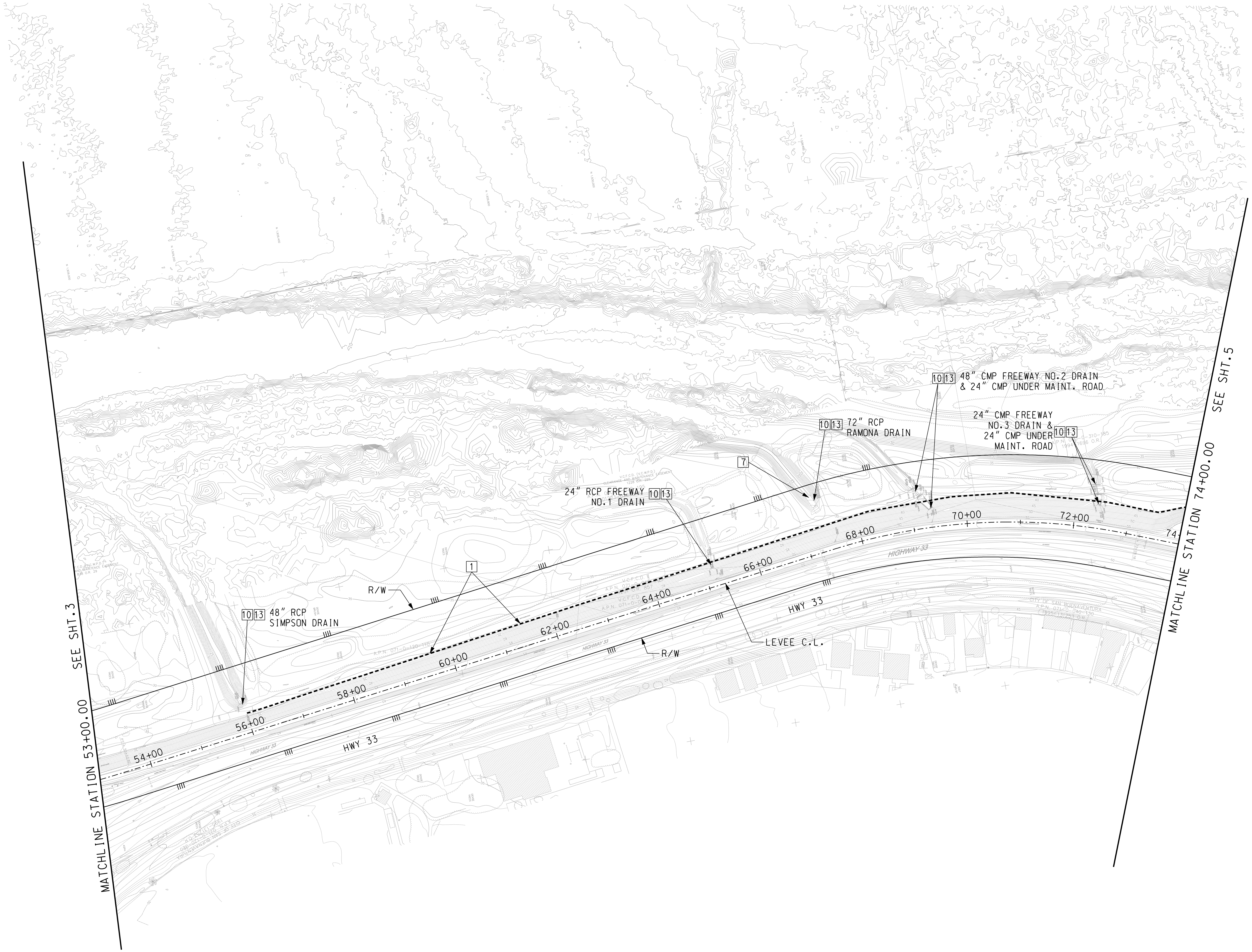
ALTERNATIVES PLAN (2)

SHEET REFERENCE NUMBER:

SHEET 3 OF 11



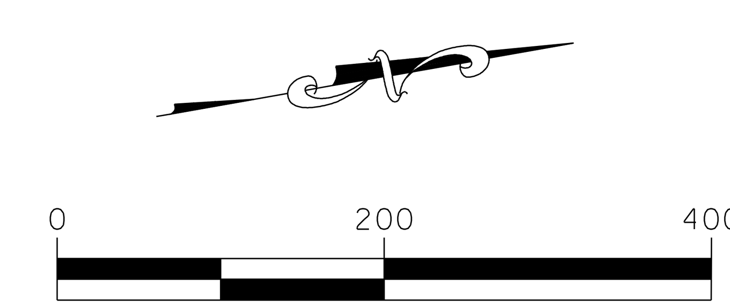

## CONSTRUCTION NOTES

1. CONSTRUCT SCOUR PROTECTION WITH TOEDOWN PER PLAN HEREON. PROFILES ON SHTS. 9 AND 10, AND TYP. SECTIONS ON SHT.11.

7. REMOVE VEGETATION WITHIN 15' FROM BOTH RIVERSIDE AND LANDSIDE TOES OF THE LEVEE PER PLAN HEREON.

10. REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

13. RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

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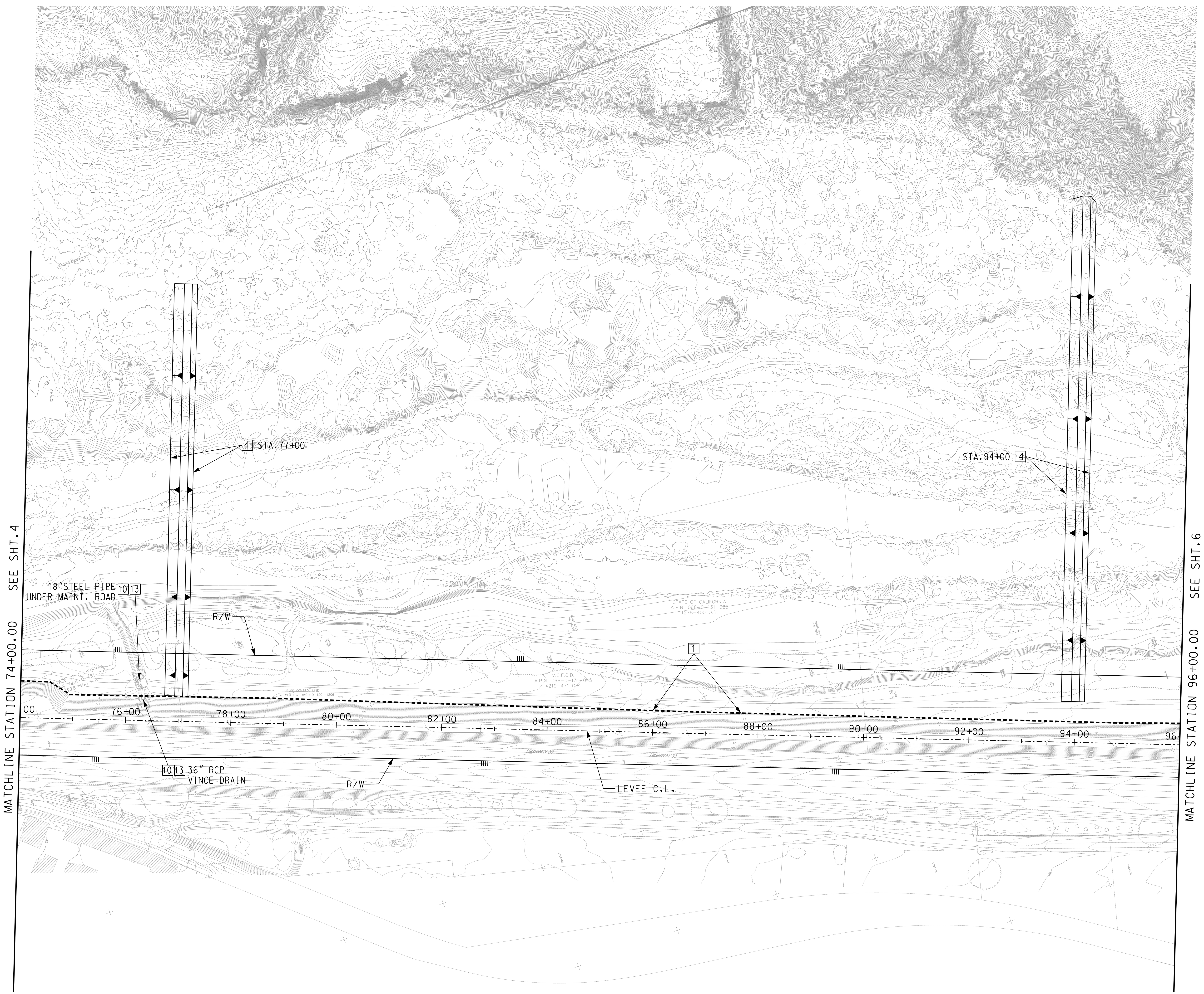
ALTERNATIVES PLAN (3)

SHEET REFERENCE NUMBER:

SHEET 4 OF 11




## CONSTRUCTION NOTES

1. CONSTRUCT SCOUR PROTECTION WITH TOEDOWN PER PLAN HEREON, PROFILES ON SHTS. 9 AND 10, AND TYP. SECTIONS ON SHT.11.

4. CONSTRUCT GRADE CONTROL STRUCTURE PER PLAN HEREON AND TYP. SECTIONS ON SHT.11.

10. REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

13. RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

0 200 400

US Army Corps of Engineers

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COUNTY OF VENTURA, CALIFORNIA
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ALTERNATIVES PLAN (4)

SHEET REFERENCE NUMBER:

## CONSTRUCTION NOTES

1 CONSTRUCT SCOUR PROTECTION WITH TOEDOWN PER PLAN HEREON, PROFILES ON SHTS. 9 AND 10, AND TYP. SECTIONS ON SHT.11.

4 CONSTRUCT GRADE CONTROL STRUCTURE PER PLAN HEREON AND TYP. SECTIONS ON SHT.11.

10 REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

12 REPAIR LEVEE EMBANKMENT EROSION PER PLAN HEREON.

13 RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

15 REGRADE SIDESLOPE OF RAMP TO BE FLATTER THAN 2:1 AND PLACE RIPRAP PER HEREON.

19 PERFORM MAINTENANCE OF STRUCTURE TO ENSURE OPERABLE CONDITION PER PLAN HEREON.


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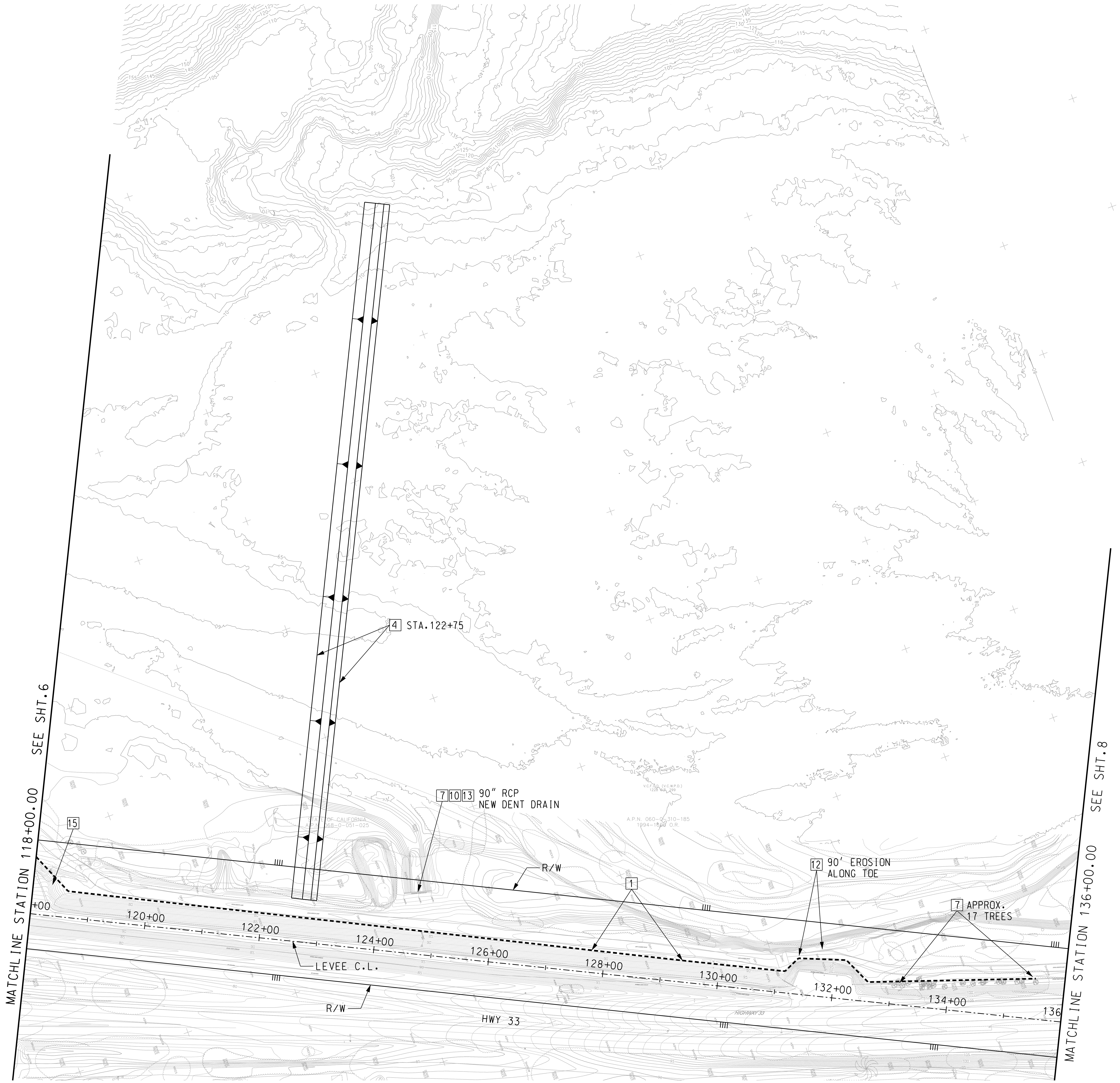
COUNTY OF VENTURA, CALIFORNIA
VENTURA RIVER LEVEE EVALUATION, AND REHABILITATION (VR-1)

ALTERNATIVES PLAN (5)

SHEET REFERENCE NUMBER:

SHEET 6 OF 11




## CONSTRUCTION NOTES

1 CONSTRUCT SCOUR PROTECTION WITH TOEDOWN PER PLAN HEREON. PROFILES ON SHTS. 9 AND 10, AND TYP. SECTIONS ON SHT.11.

4 CONSTRUCT GRADE CONTROL STRUCTURE PER PLAN HEREON AND TYP. SECTIONS ON SHT.11.

7 REMOVE VEGETATION WITHIN 15' FROM BOTH RIVERSIDE AND LANDSIDE TOES OF THE LEVEE PER PLAN HEREON.

10 REPAIR DETERIORATING SURFACES OR JOINTS OF CONCRETE STRUCTURES PER PLAN.

12 REPAIR LEVEE EMBANKMENT EROSION PER PLAN HEREON.

13 RESTORE EX. STORM DRAIN BY REMOVING SEDIMENT, REPAIRING GATE ASSEMBLY, AND GRADING ADJACENT AREAS PER PLAN HEREON.

15 REGRADE SIDESLOPE OF RAMP TO BE FLATTER THAN 2:1 AND PLACE RIPRAP PER HEREON.

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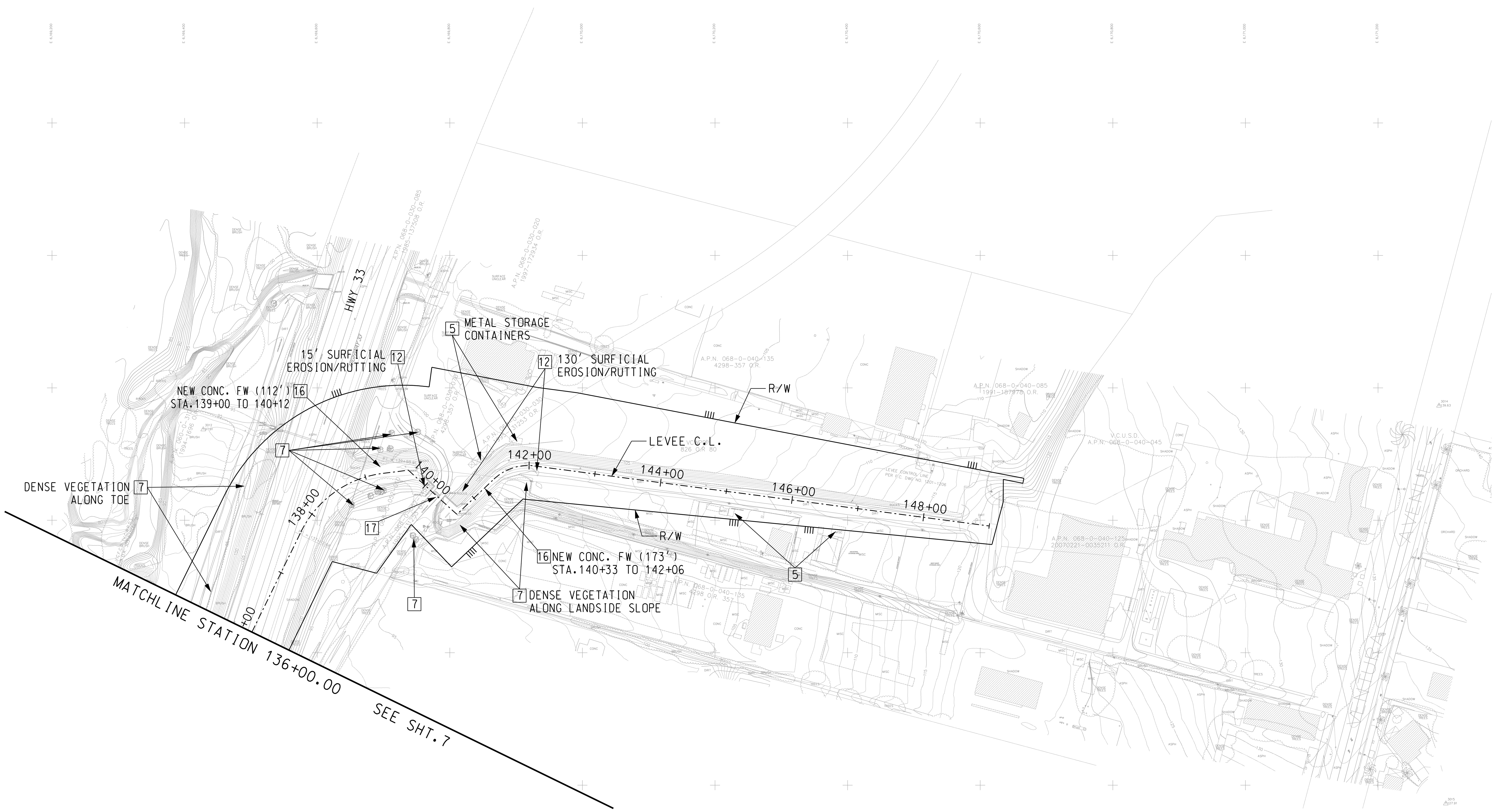
ALTERNATIVES PLAN (6)

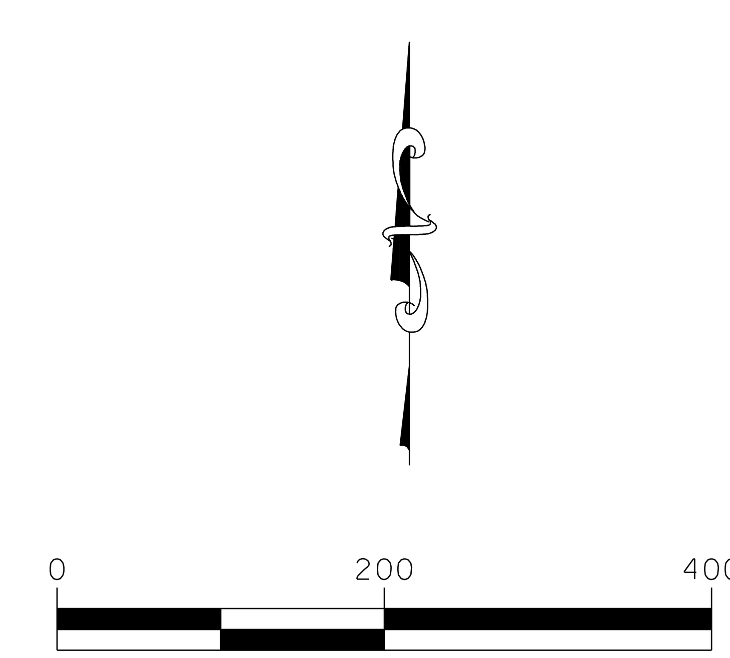
SHEET REFERENCE NUMBER:

SHEET 7 OF 11

## CONSTRUCTION NOTES

- [5] REMOVE UNPERMITTED ENCROACHMENT PER PLAN HEREON.
- [7] REMOVE VEGETATION WITHIN 15' FROM BOTH RIVERSIDE AND LANDSIDE TOES OF THE LEVEE PER PLAN HEREON.
- [12] REPAIR LEVEE EMBANKMENT EROSION PER PLAN HEREON.
- [16] CONSTRUCT CONC. FLOODWALL PER PLAN HEREON AND PROFILE ON SHT.9.
- [17] REPLACE AN EXISTING 20' WIDE STOPLOG STRUCTURE TO MEET NEW TOL PER PLAN HEREON AND PROFILE ON SHT.10.

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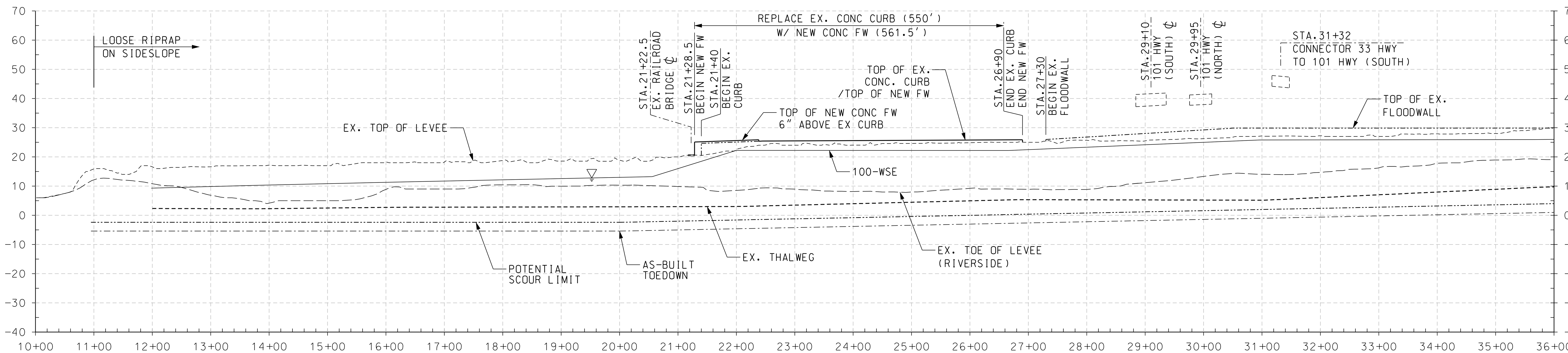
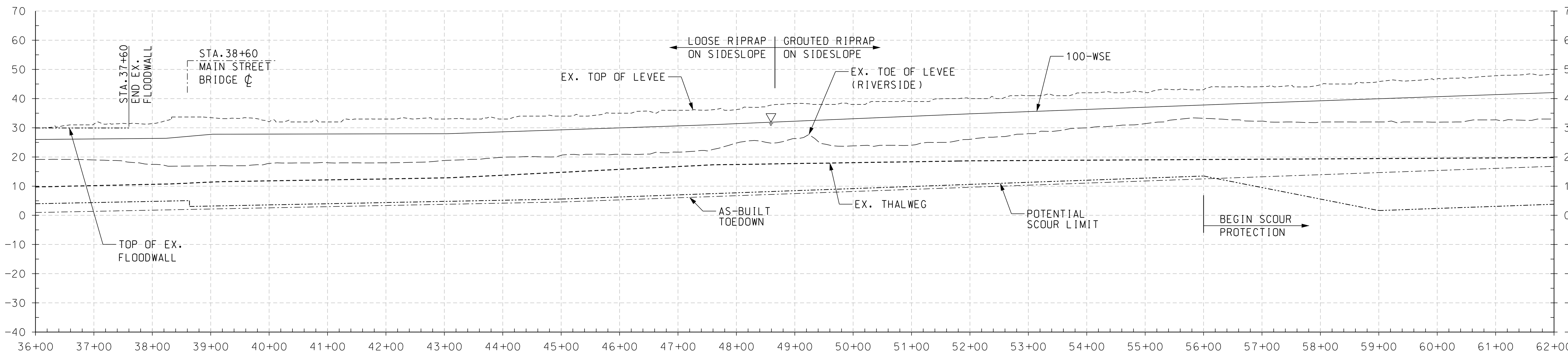
COUNTY OF VENTURA, CALIFORNIA
VENTURA RIVER LEVEE EVALUATION, AND REHABILITATION (VR-1)

ALTERNATIVE (7) –
CANADA DE SAN JOAQUIN

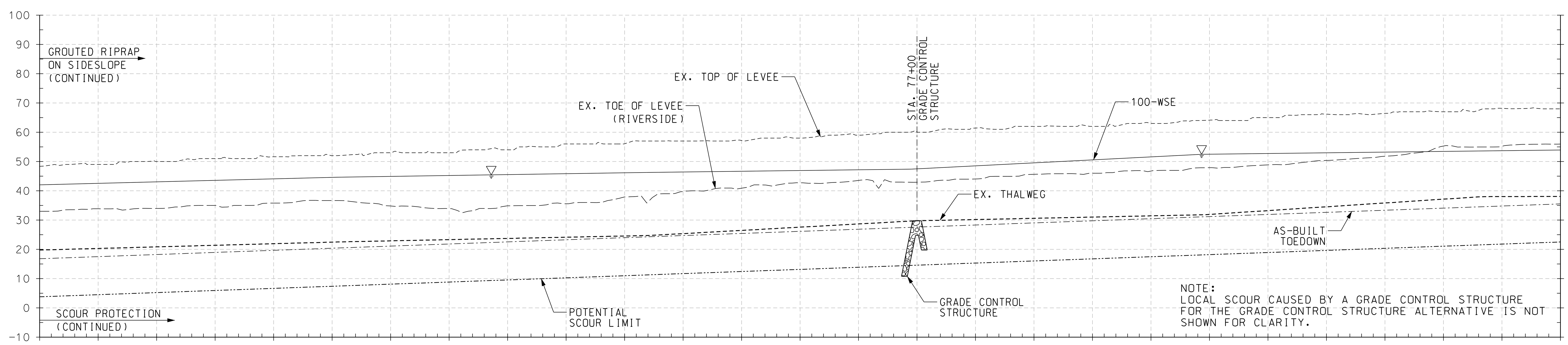
SHEET REFERENCE NUMBER:




PROFILE ALONG VENTURA RIVER (LEVEE STA.10+00 TO STA.36+00)




PROFILE ALONG VENTURA RIVER (LEVEE STA.36+00 TO STA.62+00)




PROFILE ALONG VENTURA RIVER (LEVEE STA.62+00 TO STA.88+00)

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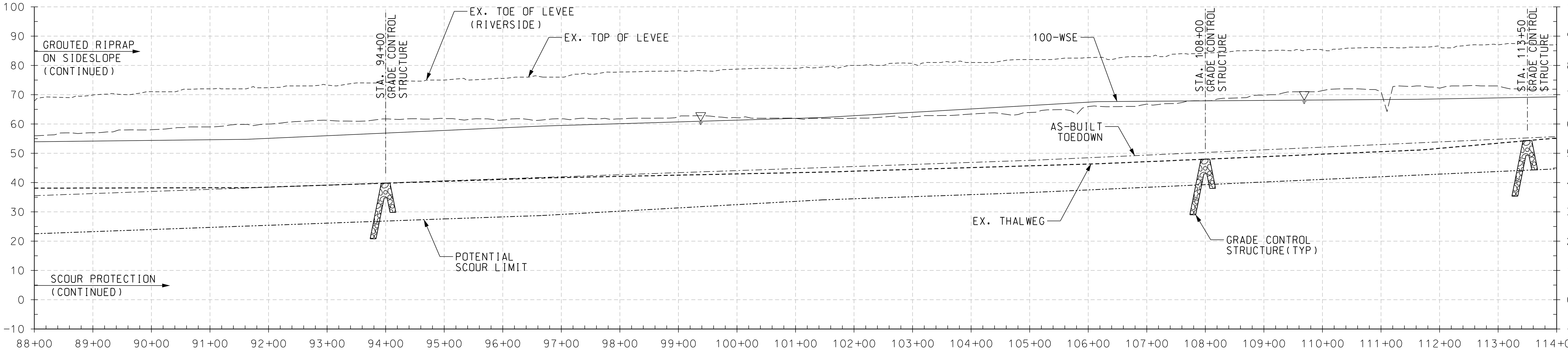
COUNTY OF VENTURA, CALIFORNIA
VENTURA RIVER LEVEE EVALUATION, AND REHABILITATION (VR-1)

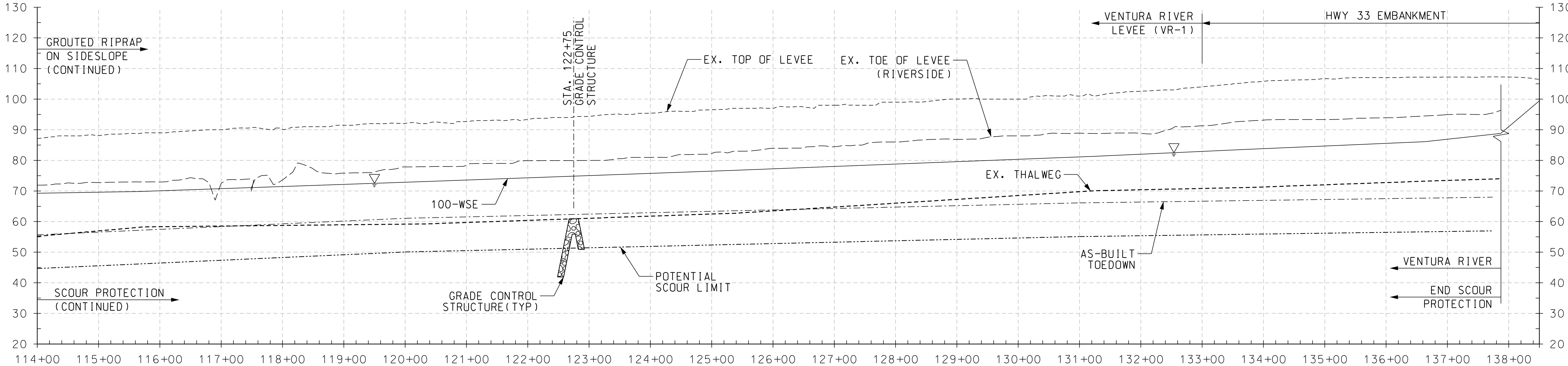
PROFILES (1)
LEVEE STATION 10+00 TO 88+00

SHEET REFERENCE NUMBER:

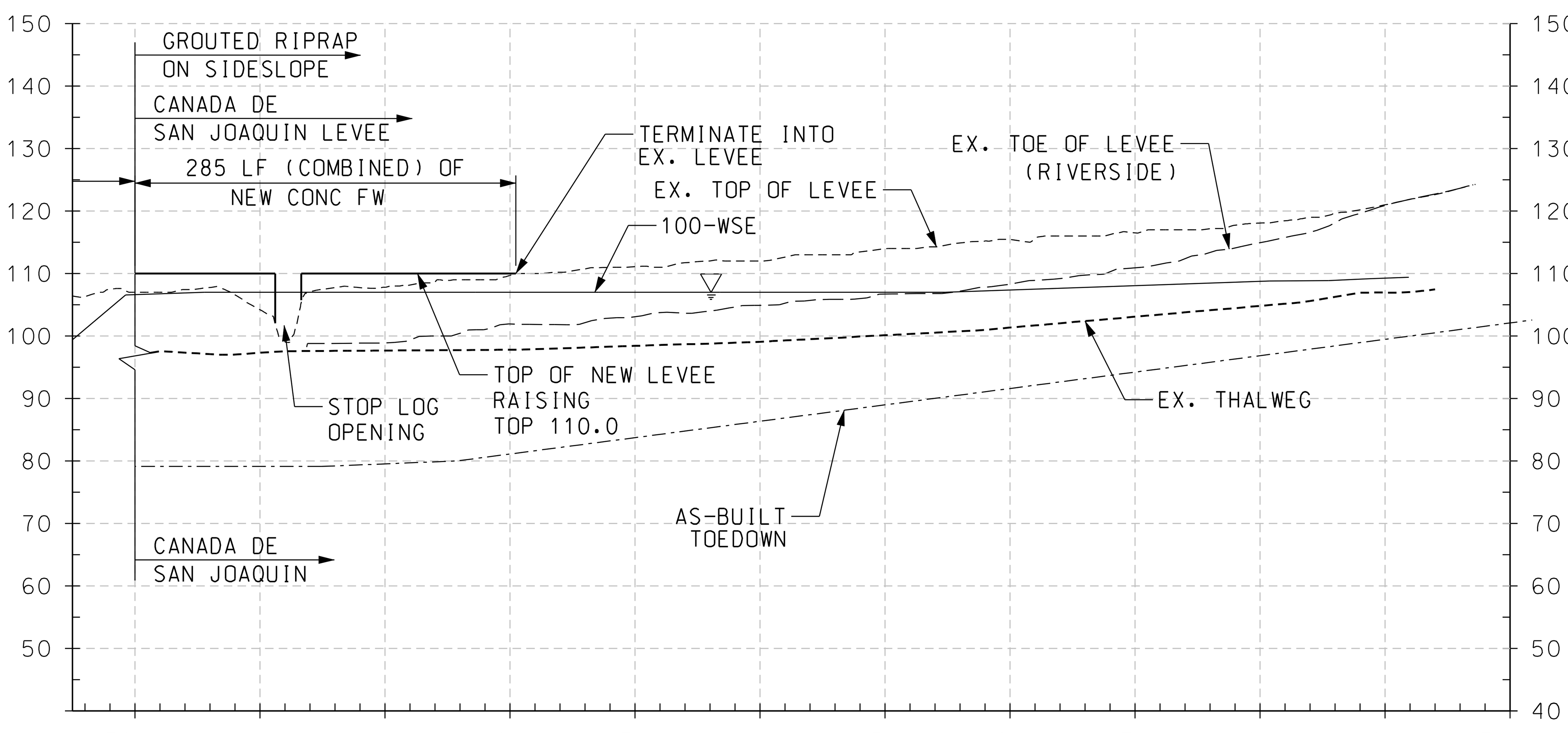
SHEET 9 OF 11




PROFILE ALONG VENTURA RIVER (LEVEE STA.88+00 TO STA.114+00)




PROFILE ALONG VENTURA RIVER (LEVEE STA.114+00 TO STA.138+50)




PROFILE ALONG VENTURA RIVER (LEVEE STA.138+50 TO STA.149+00)

NOTE:
LOCAL SCOUR CAUSED BY A GRADE CONTROL STRUCTURE FOR THE GRADE CONTROL STRUCTURE ALTERNATIVE IS NOT SHOWN FOR CLARITY.

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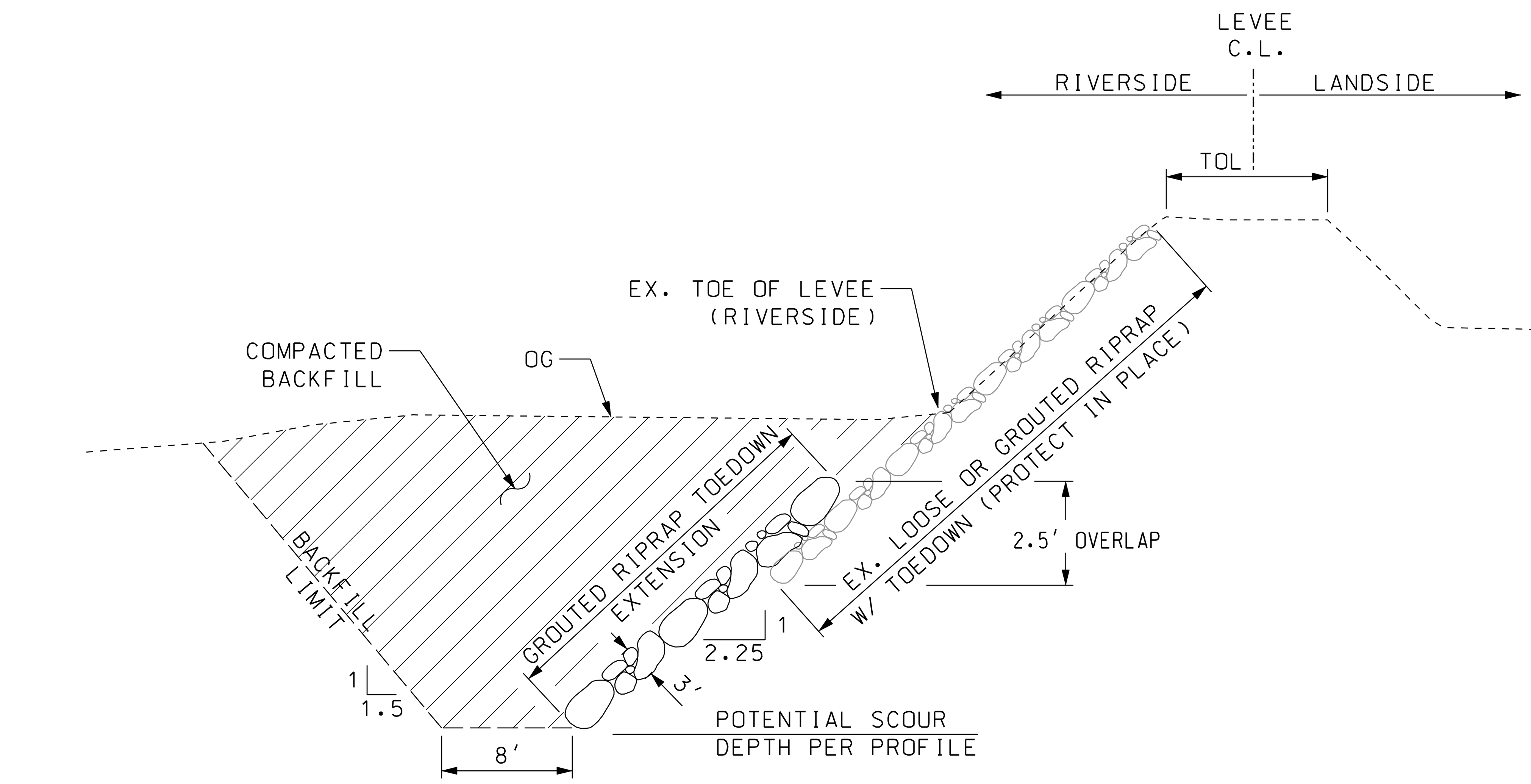
VENTURA RIVER LEVEE EVALUATION, AND REHABILITATION (VR-1)

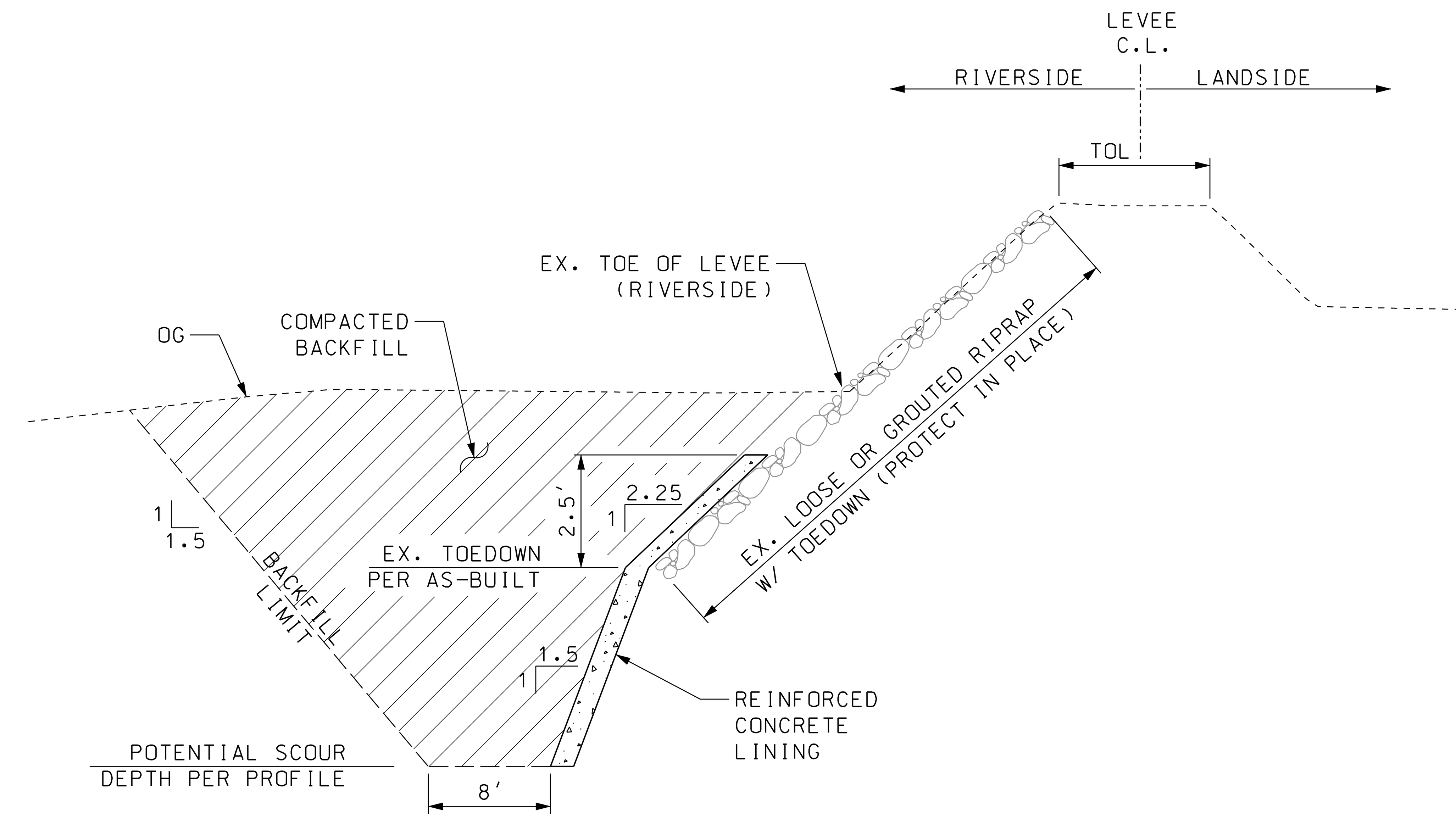
PROFILES (2)
LEVEE STATION 88+00 TO 149+00

SHEET REFERENCE NUMBER:

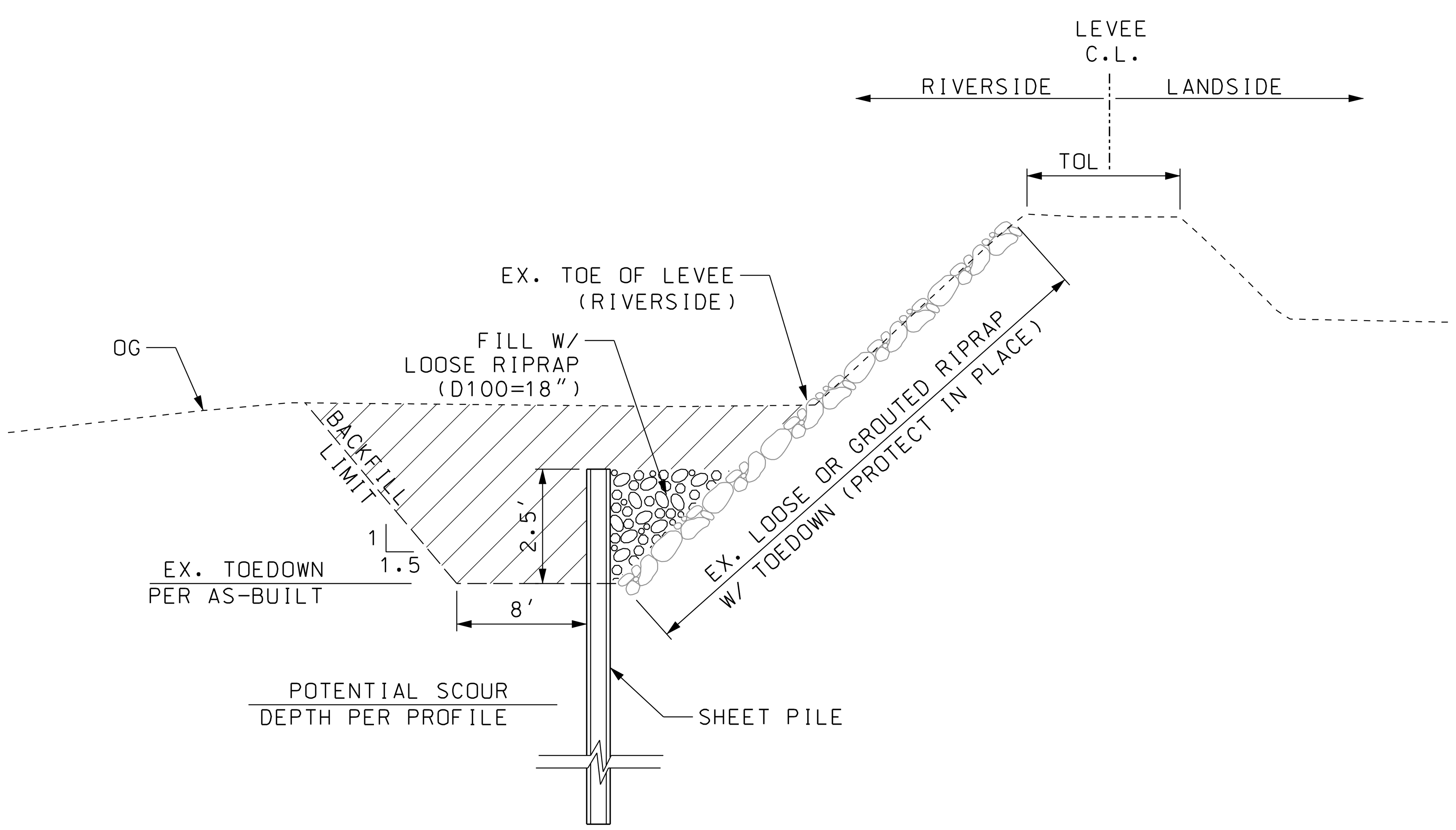
SHEET 10 OF 11



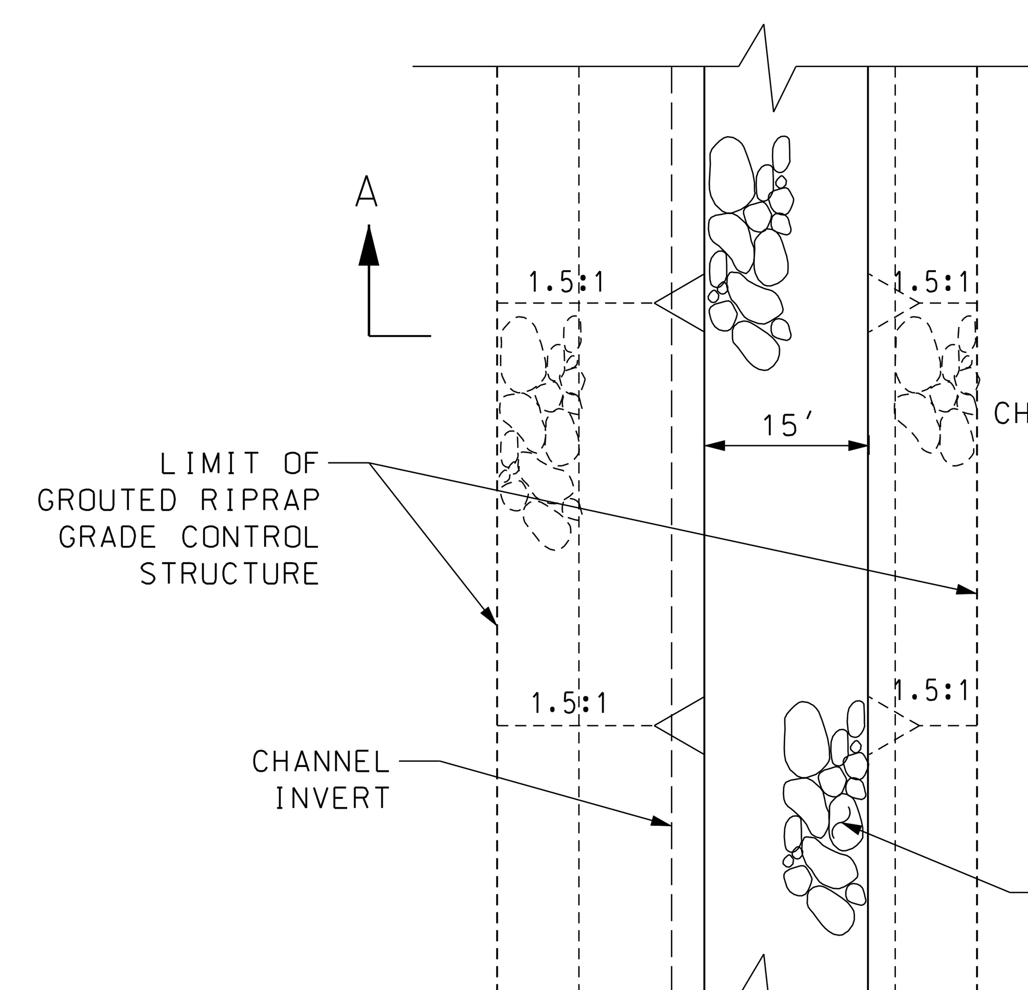

TYPICAL SECTION - GROUTED RIPRAP TOEDOWN EXTENSION
N.T.S.



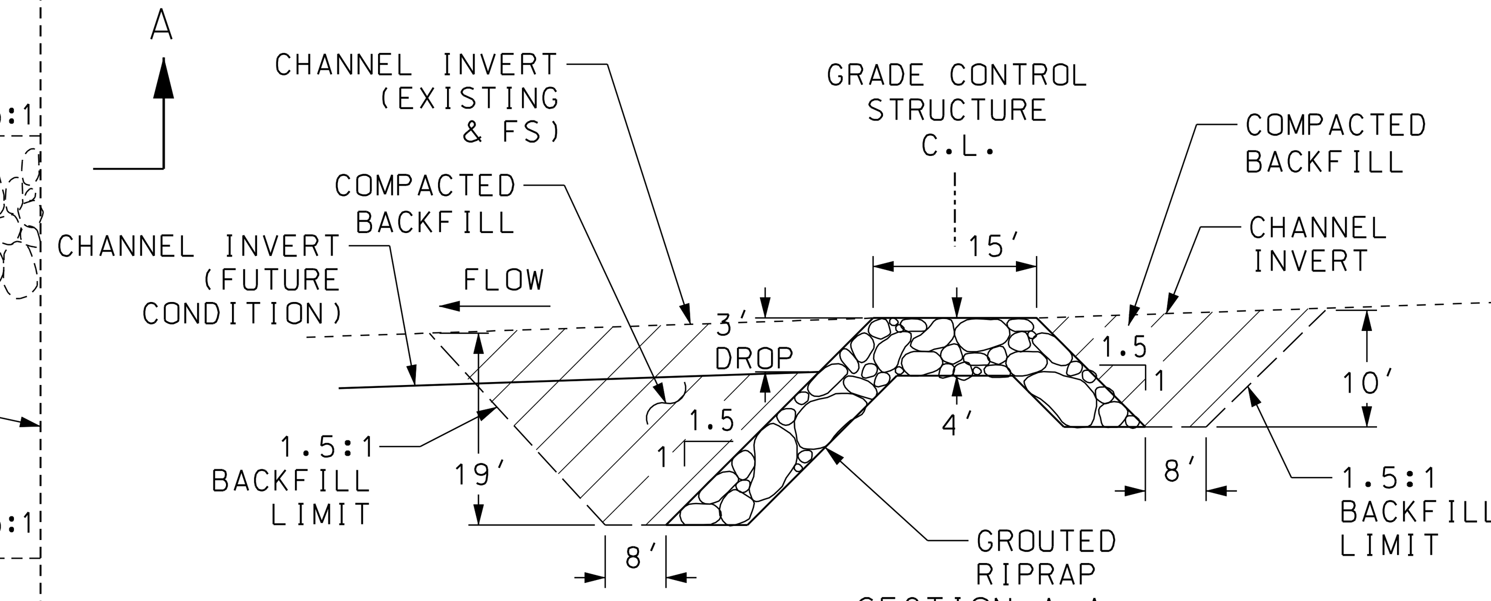

TYPICAL SECTION - CONCRETE LINING TOEDOWN EXTENSION
N.T.S.




TYPICAL SECTION - SHEET PILES TOEDOWN EXTENSION
N.T.S.

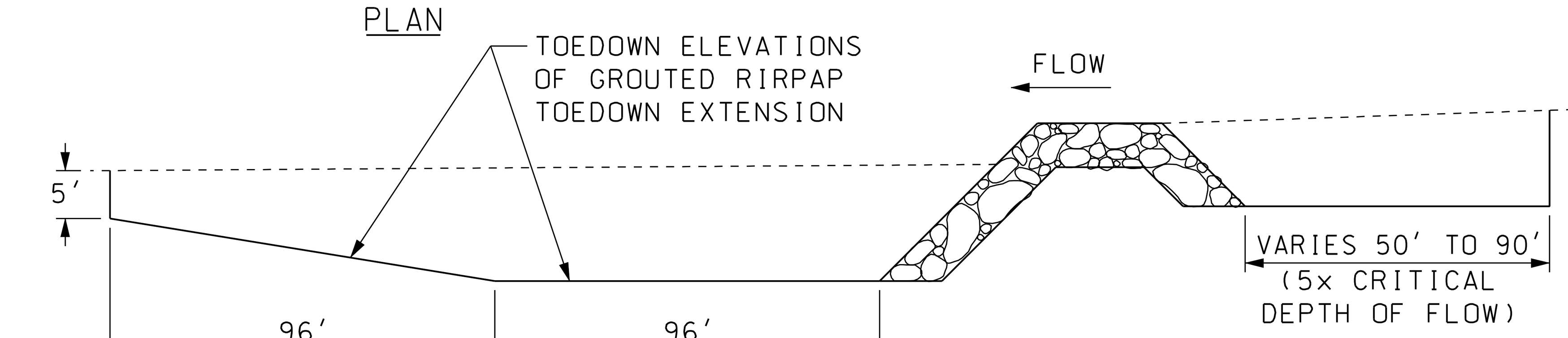



PLAN




SECTION A-A

NOTE:
THE CONTRACTOR SHALL CONSTUCT A LOW FLOW NOTCH AT THE TOP OF GRADE CONTROL STRUCTURE APPROXIMATELY AT THE SAME LOCATION WHERE THE EXISTING LOW FLOW CHANNEL IS LOCATED AT, BUT AWAY FROM TOES OF EITHER BANKS OF THE RIVER.




NOTE:
THE CONTRACTOR SHALL CONSTUCT GROUTED RIPRAP TOEDOWN EXTENSION DOWN TO THE ELEVATIONS AND DISTANCES SHOWN THIS PROFILE HEREON ON THE EXISTING LEVEE.

TYPICAL SECTION - GRADE CONTROL STRUCTURE
N.T.S.

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COUNTY OF VENTURA, CALIFORNIA
VENTURA RIVER LEVEE EVALUATION, AND
REHABILITATION (VR-1)

TYPICAL SECTIONS

SHEET REFERENCE NUMBER:

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# Appendix E
# Civil Design

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# Appendix E.1
# CHANLPRO Output Printouts

Segment 1 - Straight

PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH

INPUT PARAMETERS

| | |
|---|---|
| SPECIFIC WEIGHT OF STONE,PCF | 156.0 |
| LOCAL FLOW DEPTH,FT | 10.2 |
| CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ | |
| AVERAGE CHANNEL VELOCITY,FPS | 11.33 |
| COMPUTED LOCAL DEPTH AVG VEL,FPS | 11.33 |
| (LOCAL VELOCITY)/(AVG CHANNEL VEL) | 1.00 |
| SIDE SLOPE CORRECTION FACTOR K1 | .88 |
| CORRECTION FOR VELOCITY PROFILE IN BEND | 1.00 |
| RIPRAP DESIGN SAFETY FACTOR | 1.10 |

SELECTED STABLE GRADATIONS
ETL GRADATION

| NAME | COMPUTED D30 FT | D30(MIN) FT | D100(MAX) IN | D85/D15 | N=THICKNESS/ D100(MAX) | CT | THICKNESS IN |
|---|---|---|---|---|---|---|---|
| 3 | | .61 | 15.00 | 1.70 | NOT STABLE | | |
| 4 | .73 | .73 | 18.00 | 1.70 | 1.05 | .99 | 19.0 |
| 5 | .74 | .85 | 21.00 | 1.70 | 1.00 | 1.00 | 21.0 |

LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT

| D100(MAX) IN | 100 | | 50 | | 15 | | D30(MIN) FT | D90(MIN) FT |
|---|---|---|---|---|---|---|---|---|
| 18.00 | 276 | 110 | 82 | 55 | 41 | 17 | .73 | 1.06 |
| 21.00 | 438 | 175 | 130 | 88 | 65 | 27 | .85 | 1.23 |

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

| D100(MAX) | D100(MIN) | D50(MAX) | D50(MIN) | D15(MAX) | D15(MIN) |
|---|---|---|---|---|---|
| 18.0 | 13.3 | 12.0 | 10.5 | 9.5 | 7.1 |
| 21.0 | 15.5 | 14.0 | 12.3 | 11.1 | 8.3 |

Segment 1 - Bend

```
    PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, BENDWAY
                   INPUT PARAMETERS
    SPECIFIC WEIGHT OF STONE,PCF              156.0
    MINIMUM CENTER LINE BEND RADIUS,FT       1200.0
    WATER SURFACE WIDTH,FT                   2970.0
    LOCAL FLOW DEPTH,FT                        10.2
    CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ
    AVERAGE CHANNEL VELOCITY,FPS              11.33
    COMPUTED LOCAL DEPTH AVG VEL,FPS          17.94
    (LOCAL VELOCITY)/(AVG CHANNEL VEL)         1.58
    SIDE SLOPE CORRECTION FACTOR K1             .88
    CORRECTION FOR VELOCITY PROFILE IN BEND    1.22
    RIPRAP DESIGN SAFETY FACTOR                1.10

 ***NO STABLE GRADATIONS FOUND***
```

Segment 2 - Straight

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, STRAIGHT REACH

INPUT PARAMETERS

| | |
|---|---|
| SPECIFIC WEIGHT OF STONE,PCF | 156.0 |
| LOCAL FLOW DEPTH,FT | 11.6 |
| CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ | |
| LOCAL DEPTH AVG VELOCITY,FPS | 13.21 |
| SIDE SLOPE CORRECTION FACTOR K1 | .88 |
| CORRECTION FOR VELOCITY PROFILE IN BEND | 1.00 |
| RIPRAP DESIGN SAFETY FACTOR | 1.10 |

SELECTED STABLE GRADATIONS
ETL GRADATION

| NAME | COMPUTED D30 FT | D30(MIN) FT | D100(MAX) IN | D85/D15 | N=THICKNESS/ D100(MAX) | CT | THICKNESS IN |
|---|---|---|---|---|---|---|---|
| 5 | | .85 | 21.00 | 1.70 | NOT STABLE | | |
| 6 | .97 | .97 | 24.00 | 1.70 | 1.35 | .92 | 32.3 |
| 7 | 1.05 | 1.10 | 27.00 | 1.70 | 1.00 | 1.00 | 27.0 |

| D100(MAX) IN | LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100 | | 50 | | 15 | | D30(MIN) FT | D90(MIN) FT |
|---|---|---|---|---|---|---|---|---|
| 24.00 | 653 | 261 | 193 | 131 | 97 | 41 | .97 | 1.40 |
| 27.00 | 930 | 372 | 275 | 186 | 138 | 58 | 1.10 | 1.59 |

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

| D100(MAX) | D100(MIN) | D50(MAX) | D50(MIN) | D15(MAX) | D15(MIN) |
|---|---|---|---|---|---|
| 24.0 | 17.7 | 16.0 | 14.0 | 12.7 | 9.5 |
| 27.0 | 19.9 | 18.0 | 15.8 | 14.3 | 10.7 |

Segment 2 - Bend

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

| | |
|---|---|
| SPECIFIC WEIGHT OF STONE,PCF | 156.0 |
| MINIMUM CENTER LINE BEND RADIUS,FT | 1200.0 |
| WATER SURFACE WIDTH,FT | 2515.0 |
| LOCAL FLOW DEPTH,FT | 11.6 |
| CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ | |
| LOCAL DEPTH AVG VELOCITY,FPS | 13.21 |
| SIDE SLOPE CORRECTION FACTOR K1 | .88 |
| CORRECTION FOR VELOCITY PROFILE IN BEND | 1.22 |
| RIPRAP DESIGN SAFETY FACTOR | 1.10 |

SELECTED STABLE GRADATIONS
ETL GRADATION

| NAME | COMPUTED D30 FT | D30(MIN) FT | D100(MAX) IN | D85/D15 | N=THICKNESS/ D100(MAX) | CT | THICKNESS IN |
|---|---|---|---|---|---|---|---|
| 6 | | .97 | 24.00 | 1.70 | NOT STABLE | | |
| 7 | 1.10 | 1.10 | 27.00 | 1.70 | 1.79 | .85 | 48.4 |
| 8 | 1.22 | 1.22 | 30.00 | 1.70 | 1.22 | .95 | 36.6 |
| 9 | 1.29 | 1.34 | 33.00 | 1.70 | 1.00 | 1.00 | 33.0 |

| D100(MAX) IN | LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100 | | 50 | | 15 | | D30(MIN) FT | D90(MIN) FT |
|---|---|---|---|---|---|---|---|---|
| 27.00 | 930 | 372 | 275 | 186 | 138 | 58 | 1.10 | 1.59 |
| 30.00 | 1276 | 511 | 378 | 255 | 189 | 80 | 1.22 | 1.77 |
| 33.00 | 1699 | 679 | 503 | 340 | 251 | 106 | 1.34 | 1.94 |

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

| D100(MAX) | D100(MIN) | D50(MAX) | D50(MIN) | D15(MAX) | D15(MIN) |
|---|---|---|---|---|---|
| 27.0 | 19.9 | 18.0 | 15.8 | 14.3 | 10.7 |
| 30.0 | 22.1 | 20.0 | 17.5 | 15.9 | 11.9 |
| 33.0 | 24.3 | 22.0 | 19.3 | 17.5 | 13.1 |

Segment 3 - Straight

```
    PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH
                   INPUT PARAMETERS
    SPECIFIC WEIGHT OF STONE,PCF               156.0
    LOCAL FLOW DEPTH,FT                         14.3
    CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ
    AVERAGE CHANNEL VELOCITY,FPS               18.85
    COMPUTED LOCAL DEPTH AVG VEL,FPS           18.85
    (LOCAL VELOCITY)/(AVG CHANNEL VEL)          1.00
    SIDE SLOPE CORRECTION FACTOR K1              .88
    CORRECTION FOR VELOCITY PROFILE IN BEND     1.00
    RIPRAP DESIGN SAFETY FACTOR                 1.10


                        SELECTED STABLE GRADATIONS
                               ETL GRADATION

    NAME    COMPUTED D30(MIN)  D100(MAX)  D85/D15  N=THICKNESS/   CT
    THICKNESS
             D30 FT     FT         IN                 D100(MAX)          IN
     12                1.95      48.00       1.70    NOT STABLE
     13       2.19     2.19      54.00       1.70       1.45        .90    78.3

  D100(MAX)        LIMITS OF STONE WEIGHT,LB          D30(MIN)  D90(MIN)
     IN          FOR PERCENT LIGHTER BY WEIGHT           FT        FT
                   100           50           15
   54.00     7443   2977   2203   1489   1102    465    2.19      3.17

           EQUIVALENT SPHERICAL DIAMETERS IN INCHES
  D100(MAX)  D100(MIN)  D50(MAX)  D50(MIN)  D15(MAX)  D15(MIN)
   54.0       39.8       36.0      31.6      28.6      21.4
```

Segment 3 - Bend

```
    PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, BENDWAY
                   INPUT PARAMETERS
    SPECIFIC WEIGHT OF STONE,PCF              156.0
    MINIMUM CENTER LINE BEND RADIUS,FT       1200.0
    WATER SURFACE WIDTH,FT                    777.0
    LOCAL FLOW DEPTH,FT                        14.3
    CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ
    AVERAGE CHANNEL VELOCITY,FPS              18.85
    COMPUTED LOCAL DEPTH AVG VEL,FPS          29.85
    (LOCAL VELOCITY)/(AVG CHANNEL VEL)         1.58
    SIDE SLOPE CORRECTION FACTOR K1             .88
    CORRECTION FOR VELOCITY PROFILE IN BEND    1.22
    RIPRAP DESIGN SAFETY FACTOR                1.10

 ***NO STABLE GRADATIONS FOUND***
```

Segment 4 - Straight

```
PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH
               INPUT PARAMETERS
SPECIFIC WEIGHT OF STONE,PCF              156.0
LOCAL FLOW DEPTH,FT                         9.9
CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ
AVERAGE CHANNEL VELOCITY,FPS              19.18
COMPUTED LOCAL DEPTH AVG VEL,FPS          19.18
(LOCAL VELOCITY)/(AVG CHANNEL VEL)         1.00
SIDE SLOPE CORRECTION FACTOR K1             .88
CORRECTION FOR VELOCITY PROFILE IN BEND    1.00
RIPRAP DESIGN SAFETY FACTOR                1.10

***NO STABLE GRADATIONS FOUND***
```

# Appendix E.2
# Grouted Riprap Thickness Determination

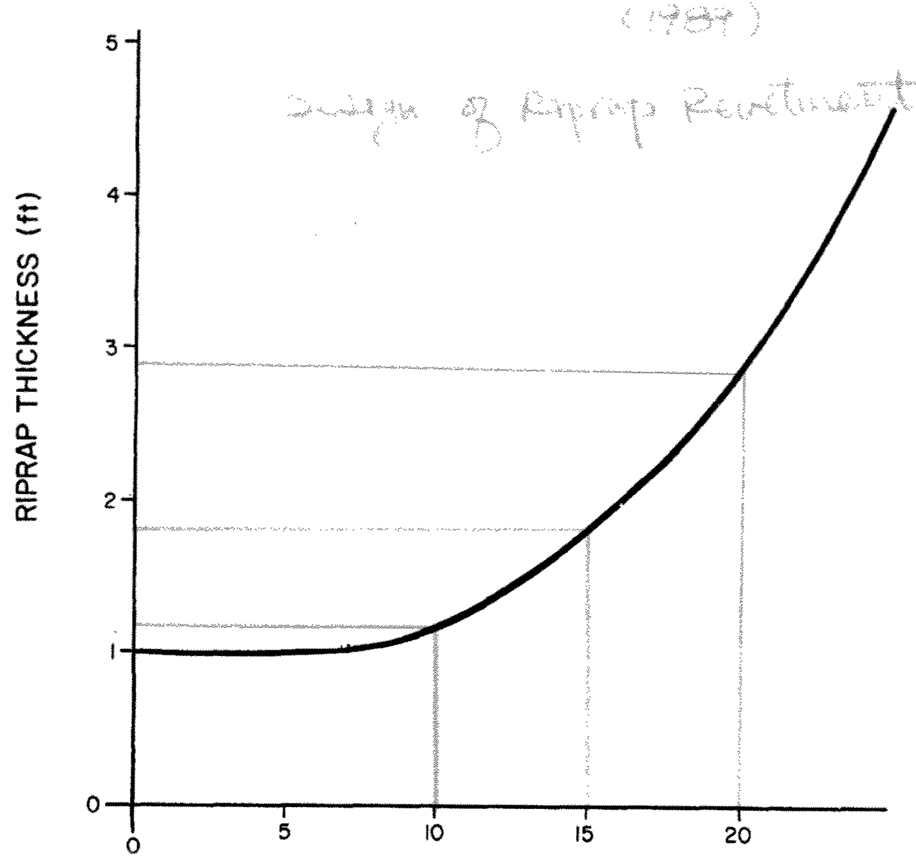


Figure 57. Required blanket thickness as a function of flow velocity.

<u>Rock Grading</u>: Table 6 provides guidelines for rock gradation in grouted riprap installations. Six size classes are listed.

<u>Rock Quality</u>: Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. However, the specifications for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout.

In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

<u>Grout Quality and Characteristics</u>: Grout should consist of good strength concrete using a maximum aggregate size of 3/4 in and a slump of 3 to 4 in (7.6 to 10.2 cm). Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability.

The volume of grout required will be that necessary to provide penetration to the depths shown in table 6.

The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate.

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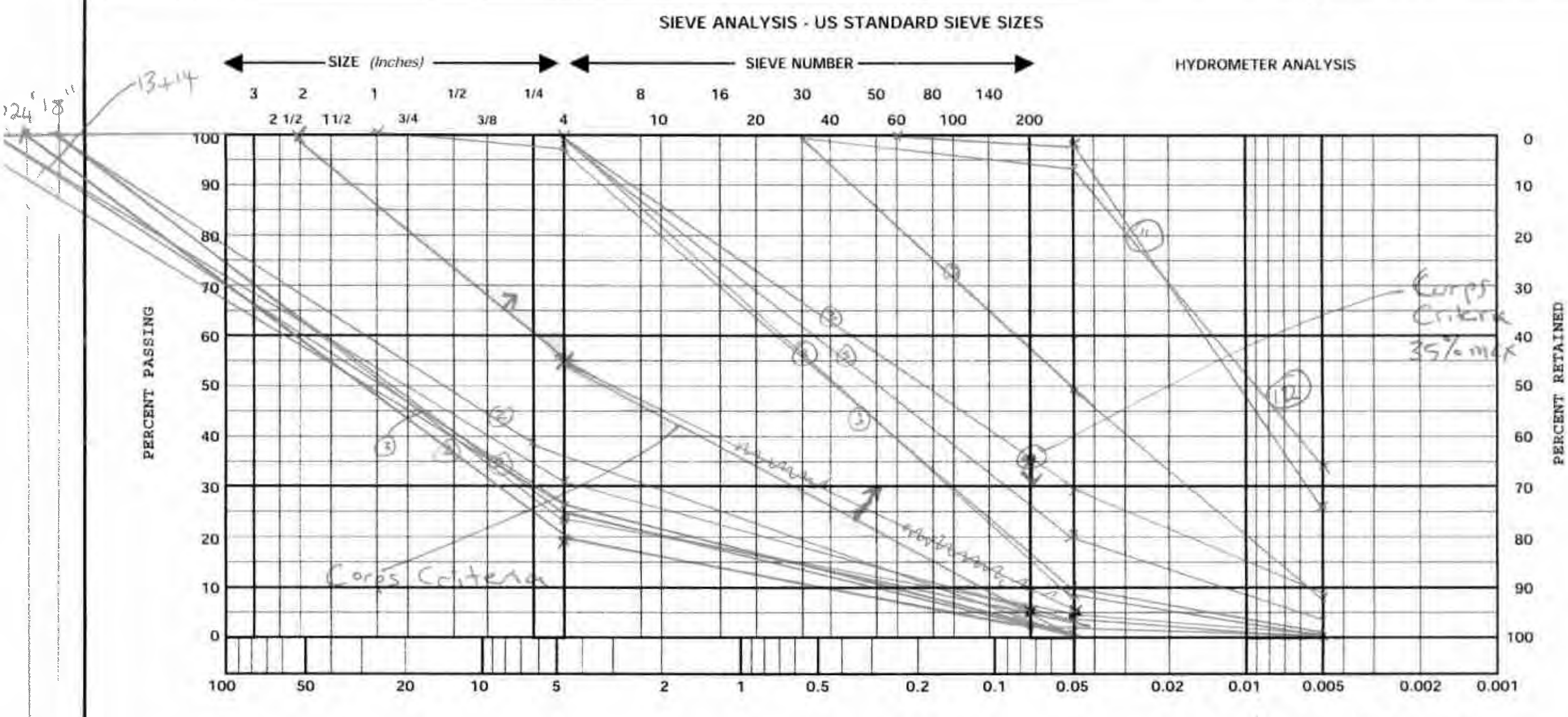
# Appendix E.3
# Gradation Curves based on
# As-built Test Pit Logs

# GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

| 1. PROJECT | 2. DATE |
|---|---|
| | |

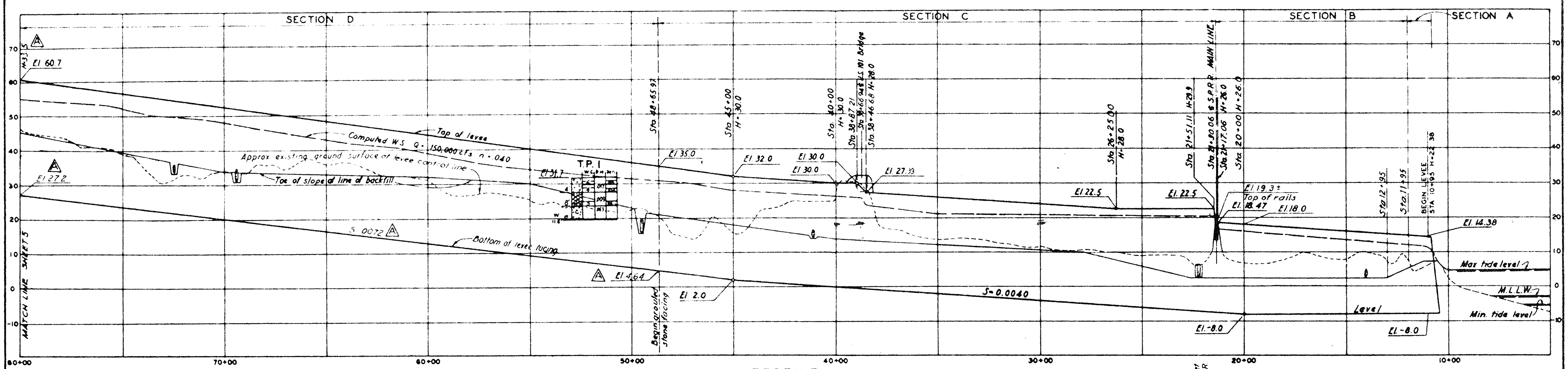



4 / 14 Gradations
ALL FROM TP-3

| EXCAVATION NUMBER | SAMPLE NUMBER | LL | PL | PI | Cu $(D_{60}/D_{10})$ | Cc $(D_{30})^2/(D_{60} \times D_{10})$ | SOIL DESCRIPTION/REMARKS | CLASSIFICATION (USCS) |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | |

| 3. TECHNICIAN (Signature) | 4. PLOTTED BY (Signature) | 5. CHECKED BY (Signature) |
|---|---|---|
| | | |

DD FORM 1207, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Adobe Professional 7.0

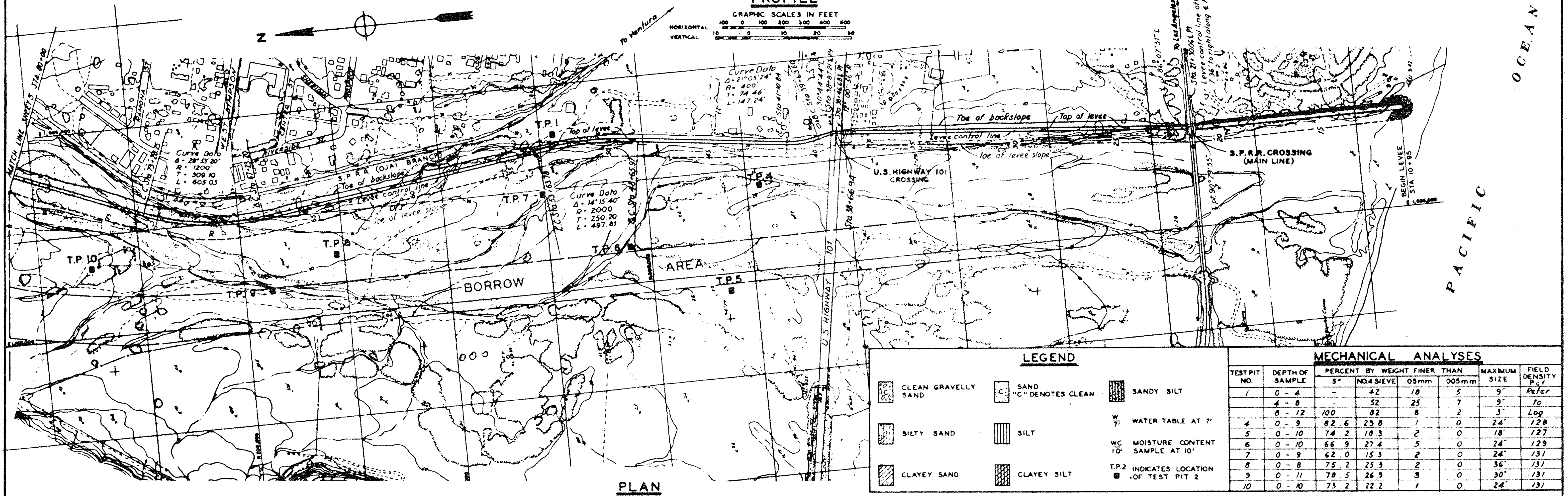



## PROFILE

GRAPHIC SCALES IN FEET

HORIZONTAL 100 0 100 200 300 400 500

VERTICAL 10 0 10 20 30




## LEGEND

| | |
|---|---|
| CLEAN GRAVELLY SAND | |
| SAND "C" DENOTES CLEAN | |
| SANDY SILT | |
| SILTY SAND | |
| SILT | |
| $\frac{W}{7'}$ | WATER TABLE AT 7' |
| $\frac{WC}{10'}$ | MOISTURE CONTENT SAMPLE AT 10' |
| CLAYEY SAND | |
| CLAYEY SILT | |
| T.P.2 ■ | INDICATES LOCATION OF TEST PIT 2 |

## MECHANICAL ANALYSES

| TEST PIT NO. | DEPTH OF SAMPLE | PERCENT BY WEIGHT FINER THAN 5" | NO.4 SIEVE | .05mm | .005mm | MAXIMUM SIZE | FIELD DENSITY Pcf |
|---|---|---|---|---|---|---|---|
| 1 | 0 - 4 | – | 42 | 18 | 5 | 9" | Refer |
| | 4 - 8 | – | 52 | 25 | 7 | 9" | to |
| | 8 - 12 | 100 | 82 | 8 | 2 | 3" | Log |
| 4 | 0 - 9 | 82.6 | 23.8 | 1 | 0 | 24" | 128 |
| 5 | 0 - 10 | 74.2 | 18.3 | 2 | 0 | 18" | 127 |
| 6 | 0 - 10 | 66.9 | 27.4 | 5 | 0 | 24" | 129 |
| 7 | 0 - 9 | 62.0 | 15.3 | 2 | 0 | 24" | 131 |
| 8 | 0 - 8 | 75.2 | 25.3 | 2 | 0 | 36" | 131 |
| 9 | 0 - 11 | 78.5 | 26.9 | 3 | 0 | 30" | 131 |
| 10 | 0 - 10 | 73.2 | 22.2 | 1 | 0 | 24" | 131 |

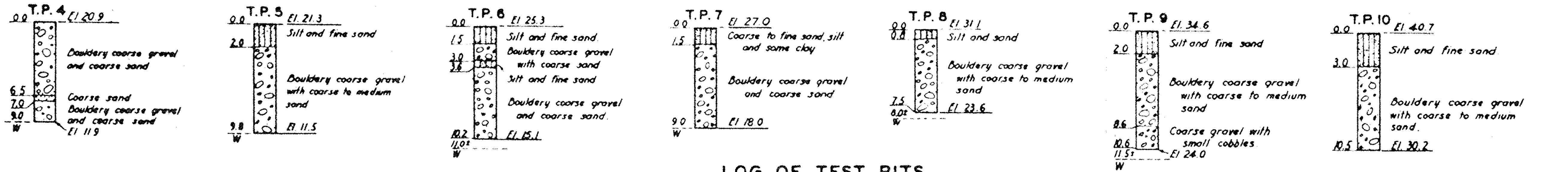
NOTE:
CONTOUR INTERVAL 5 FEET
TOPOGRAPHY BY MULTIPLEX PROJECTION FROM AERIAL PHOTOS FLOWN OCT. 1945
GRID: ZONE 5 CALIFORNIA, LAMBERT PLANE COORDINATES, N.A. DATUM (1927)

DATUM IS MEAN SEA LEVEL

## PLAN

GRAPHIC SCALE IN FEET

100 0 100 200 300 400 500




## LOG OF TEST PITS

### REFERENCE DRAWINGS

For foundation and borrow exploration
Sta 80+00 to Sta 150+85 ............ See sheet 5

### GENERAL NOTES

The soil classifications and test data shown hereon are the best information available and are not certified to be exact or intended to apply at any place other than where test pits were excavated.

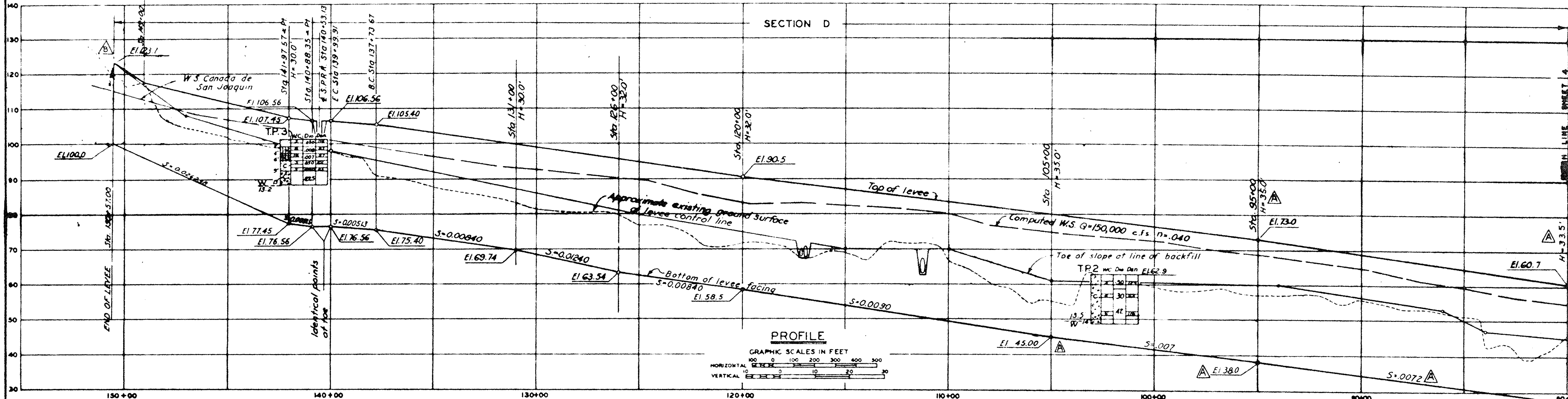
| REVISION | DATE | DESCRIPTION | BY |
|---|---|---|---|
| A | 1-28-48 | Revised bottom of levee facing from Sta 45+00 to Sta 105+00 | [signature] |

DEPARTMENT OF THE ARMY--CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES CALIFORNIA

| | |
|---|---|
| DRAWN BY: W.W.C | VENTURA RIVER BASIN, CALIFORNIA |
| TRACED BY: W.W.C. | **VENTURA RIVER IMPROVEMENT** |
| CHECKED BY: V.P.P. J.M.B. | VENTURA RIVER LEVEE |
| SUBMITTED BY: [signature] CHIEF, DESIGN BRANCH | OCEAN TO CANADA DE SAN JOAQUIN FOUNDATION & BORROW EXPLORATION STA 10+95 TO STA 80+00 |
| APPROVAL RECOMMENDED: [signature] CHIEF, ENGINEERING DIVISION | APPROVED [signature] COLONEL, C.E., DISTRICT ENGINEER. |
| SCALE: AS SHOWN | DATE: NOVEMBER, 1947 |
| TO ACCOMPANY SPECIFICATIONS, 4-04-353 ENG-48-22 DATED 10 DECEMBER 1947 | SHEET 4 OF 18 — FILE NO. (O.D. SERIES) 459/4 REVISION A |

EXHIBIT "B"

SHEET 4




## LOG OF TEST PITS

**T.P.16**
- 0.0–1.0 Silt and fine sand
- 1.0–3.5 Sand and bouldery gravel (6" max. size)
- 3.5–6.0 Sand and coarse gravel
- 6.0 Bouldery coarse gravel at 6.0' (30" max. size)

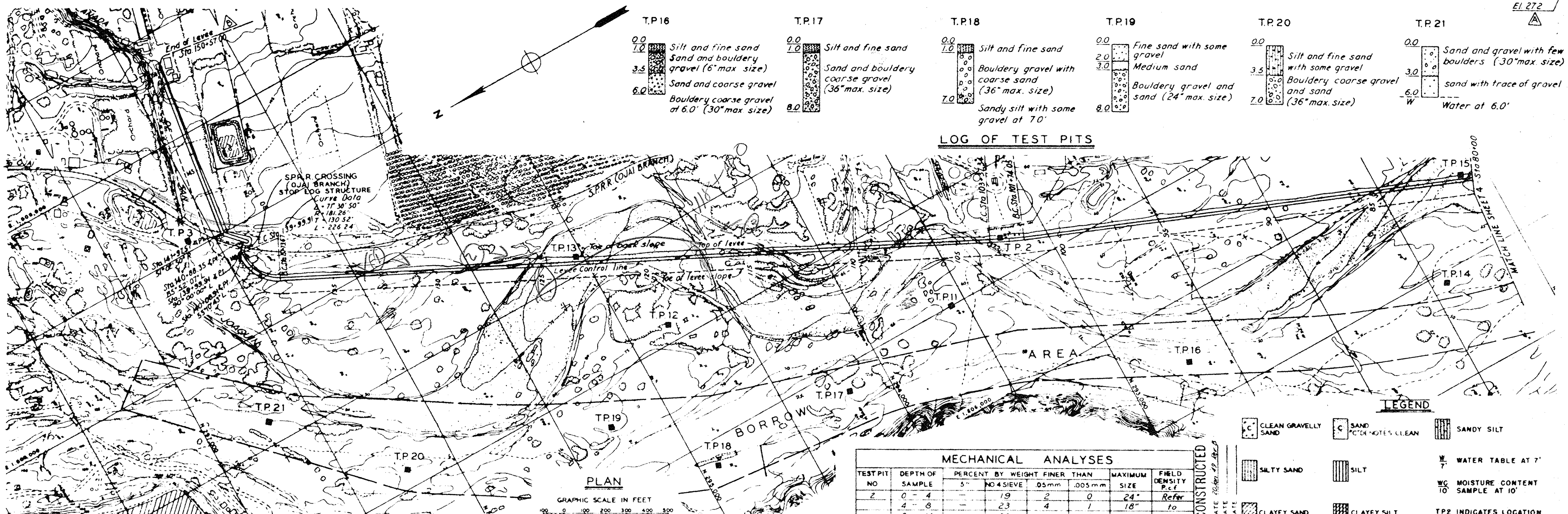
**T.P.17**
- 0.0–1.0 Silt and fine sand
- 1.0–8.0 Sand and bouldery coarse gravel (36" max. size)

**T.P.18**
- 0.0–1.0 Silt and fine sand
- 1.0–7.0 Bouldery gravel with coarse sand (36" max. size)
- 7.0 Sandy silt with some gravel at 7.0'

**T.P.19**
- 0.0–2.0 Fine sand with some gravel
- 2.0–3.0 Medium sand
- 3.0–8.0 Bouldery gravel and sand (24" max. size)

**T.P.20**
- 0.0–3.5 Silt and fine sand with some gravel
- 3.5–7.0 Bouldery coarse gravel and sand (36" max. size)

**T.P.21**
- 0.0–3.0 Sand and gravel with few boulders (30" max. size)
- 3.0–6.0 Sand with trace of gravel
- Water at 6.0'




## LEGEND

- CLEAN GRAVELLY SAND (C)
- SAND ("C" DENOTES CLEAN)
- SANDY SILT
- SILTY SAND
- SILT
- CLAYEY SAND
- CLAYEY SILT
- W/7' WATER TABLE AT 7'
- WC/10' MOISTURE CONTENT SAMPLE AT 10'
- TP2 ■ INDICATES LOCATION OF TEST PIT 2

DATUM IS MEAN SEA LEVEL

## MECHANICAL ANALYSES

| TEST PIT NO | DEPTH OF SAMPLE | PERCENT BY WEIGHT FINER THAN 5" | NO 4 SIEVE | .05 mm | .005 mm | MAXIMUM SIZE | FIELD DENSITY P.c.f |
|---|---|---|---|---|---|---|---|
| 2 | 0 - 4 | — | 19 | 2 | 0 | 24" | Refer |
| | 4 - 8 | — | 23 | 4 | 1 | 18" | to |
| | 8 - 14 | — | 38 | 1 | 0 | 18" | Log |
| 3 | 0 - 2 | 100 | 100 | 20 | 4 | No. 4 Sieve | |
| | 2 - 4 | 100 | 100 | 29 | 8 | No 4 Sieve | Refer |
| | 4 - 6 | 100 | 100 | 49 | 7 | No.30 Sieve | to |
| | 6 - 8 | 100 | 97 | 10 | 2 | 1" | Log |
| | 8 - 9 | 100 | 100 | 8 | 1 | No. 4 Sieve | |
| | 9 - 13 | — | 24 | 4 | 0 | 18" | |
| 11 | 0 - 13 | 100 | 100 | 98 | 26 | No. 60 Sieve | — |
| 12 | 0 - 12 | 100 | 100 | 94 | 34 | No. 30 Sieve | — |
| 13 | 0 - 10 | 75.5 | 26.2 | 2.0 | 1.0 | 30" | — |
| 14 | 0 - 12 | 66.1 | 30.7 | 4 | 1 | 30" | 131 |
| 15 | 0 - 10 | 709 | 25.7 | 2 | 0 | 48" | 131 |

RECORD DRAWING - AS CONSTRUCTED
Contract NOA 353 Eng 2200
DATE 20 Jan 49 Rev. B

## LOG OF TEST PITS

**T.P.11** (El. 65.0)
- 0.0–3.0 Gravelly coarse to fine sand with some boulders
- 3.0–5.0
- 5.0–7.0 Loose coarse sand
- 7.0–8.7 Hard greenish clayey silt
- 8.7–11.1 Loose grayish-brown sand
- 11.1–12.5 Hard greenish clayey silt
- El. 52.5

**T.P.12** (El. 71.5)
- 0.0–3.0 Loose sand and gravel with small cobbles
- 3.0–8.1 Compacted medium to fine sand and silt
- 8.1–12.5 Hard greenish clayey silt
- 12.5–13.0 Plastic black silty clay
- El. 59.8

**T.P.13** (El. 76.2)
- 0.0–10.0 Coarse gravel and coarse sand with large boulders
- 10.0 El. 66.2
- 11.0 W

**T.P.14** (El. 51.3)
- 0.0–1.0 Silty fine sand
- 1.0–4.0 Bouldery coarse gravel interbedded with silt
- 4.0–11.8 Bouldery coarse gravel and sand
- 11.8 El. 39.5
- 13.1 W

**T.P.15** (El. 40.6')
- 0.0–9.5 Bouldery coarse gravel in slightly cemented coarse to fine sand
- 9.5 El. 31.1'

## REFERENCE DRAWINGS

For foundation & borrow exploration Sta. 10+95 to Sta. 80+00 ........ See sheet 4

## GENERAL NOTES

The soil classifications and test data shown hereon are the best information available and are not certified to be exact or intended to apply at any place other than where test pits were excavated.

| REVISION | DATE | DESCRIPTION | BY |
|---|---|---|---|
| B | 24 Jan 49 | Changed Station at end of Levee - As Constructed | B.H.B. |
| A | 1 28 48 | Revised bottom of levee facing from Sta. 45+00 to Sta. 105+00 | |

DEPARTMENT OF THE ARMY -- CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES, CALIFORNIA

VENTURA RIVER BASIN, CALIFORNIA
# VENTURA RIVER IMPROVEMENT
## VENTURA RIVER LEVEE
OCEAN TO CANADA DE SAN JOAQUIN
FOUNDATION & BORROW EXPLORATION
STA. 80+00 TO STA. 150+85

DRAWN BY M.G.
TRACED BY M.G.
CHECKED BY V.P.P. J.M.B.
SUBMITTED BY Lloyd H. Fitts, CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED: H. W. Thompson, CHIEF, ENGINEERING DIVISION
APPROVED: A. W. Moore, COLONEL, C.E., DISTRICT ENGINEER
SCALE AS SHOWN
DATE NOVEMBER 1947
TO ACCOMPANY SPECIFICATIONS 4-04-353 ENG-48-22 DATED 10 DECEMBER 1947
SHEET 5 OF 18
FILE NO (O D SERIES) 459/5 REVISION B

EXHIBIT "B" SHEET 5

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# Appendix F
# Cost Estimates

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COST ESTIMATE

Project ID: **T 27654**
Project Title: VR-1 Eval and Rehab
Date: **3/6/12**

| Alternative | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| | Grt Riprap | Conc. Lining | Sht Pile | GCS |
| Grand Total: | $25,593,400 | $25,131,600 | $34,942,500 | $26,599,800 |

## Alternative 1 - Grouted Riprap Protection with Toedown

| | Contract Items | Unit | Quantity | Unit Cost | Total Cost | |
|---|---|---|---|---|---|---|
| 1 | Mobilization | LS | 1 | $ 760,000 | $760,000 | 5% of total construction costs |
| 2 | Clearing and Grubbing | Acre | 22.86 | $ 4,000 | $91,440 | |
| | | | | | | |
| 3 | Riprap Protection w/ Toedown - (Sta.56+00 to 133+00) | LF | 7,700 | $ 1,808.73 | $13,927,240 | |
| 3.1 | *Grouted Riprap* | *CY* | *30,216* | *$ 135* | *$4,079,160* | |
| 3.2 | *Excavation* | *CY* | *630,613* | *$ 8* | *$5,044,904* | |
| 3.3 | *Compacted Backfill* | *CY* | *600,397* | *$ 8* | *$4,803,176* | |
| | | | | | | |
| 4 | Removal of Unpermitted Encroachment | LS | 1 | $ 20,000 | $ 20,000 | 160' K-Rail (DS of RR BG) & Metal Storages (Canada de SJ) & metal Building near Main st |
| 5 | Sediment Removal | SY | 4,600 | $ 3 | $13,800 | assumed 1' average thickness of sediment deposition |
| 6 | Vegetation Removal | LS | 1 | $ 200,000 | $ 200,000 | 120 trees (12" dia.) based on Google Earth; $1000 per tree removal + bulking 50% |
| 7 | Restoration of Displaced Existing Riprap | CY | 1,570 | $ 80 | $125,600 | restore per As-built riprap thickness (4 to 6' thick); |
| 8 | Floodwall - Ventura River (including removal of ex. Conc. Curb) | LF | 561.5 | $ 520 | $291,980 | |
| 9 | Floodwall - Canada de San Joaquin | LF | 285.0 | $ 480 | $136,800 | |
| 10 | Restoration of Existing Storm Drain System | LS | 1 | $ 39,000 | $39,000 | 13 STORM DRAIN SYSTEMS |
| 11 | Animal Burrow Removal | LS | 1 | $ 10,000 | $10,000 | |
| 12 | Levee Embankment Surface Erosion Repair | SY | 270 | $ 7.50 | $2,025 | Assumed fine grading with addition of import material (270 SYx6" deep) |
| 13 | Removal of Incised Channel near Riverside Toe | LF | 850 | $ 30 | $25,500 | 850' long; 1007.4CY of fill; $25/CY of fill |
| 14 | Regrading of Existing Access Ramps | EA | 3 | $ 5,000 | $15,000 | Grading to 2:1 sideslope |
| 15 | Stop-log Structure Replacement | EA | 1 | $ 75,000 | $75,000 | 11.4'H (110-98.5) x 20'W structure |
| 16 | Repair of Levee Embankment to As-built Conditiom (including removal of ex. Retaining wall) | LF | 310 | $ 140 | $43,400 | |
| 17 | Permit Application for Unpermitted Ex. Features | LS | 1 | $ 100,000 | $100,000.00 | 151 individual items (light poles, signs, etc.); $600 per each item |
| | | | | | | |
| | | | | | | |

| | | |
|---|---|---|
| | Subtotal: | $15,876,785 |
| Planning, Engineering, and Design (@ 12%) | | $1,905,214 |
| Construction Management (@ 12%) | | $1,905,214 |
| | Subtotal: | $19,687,213 |
| Contingencies (@ 30%) | | $5,906,164 |
| | Subtotal: | $25,593,377 |
| | Grand Total: | $25,593,400 |

## Alternative 2 - Concrete Lining

| | Contract Items | Unit | Quantity | Unit Cost | Total Cost | |
|---|---|---|---|---|---|---|
| 1 | Mobilization | LS | 1 | $ 740,000 | $740,000 | 5% of total construction costs |
| 2 | Clearing and Grubbing | Acre | 20.25 | $ 4,000 | $81,000 | |
| | | | | | | |
| 3.1 | Concrete Lining - (Sta.56+00 to 133+00) | LF | 7,700 | $ 1,775.48 | $13,671,224 | |
| 3.1.1 | *Reinforced Concrete Lining* | *CY* | *8,031* | *$ 800* | *$6,424,800* | |
| 3.1.2 | *Excavation* | *CY* | *456,917* | *$ 8* | *$3,655,336* | |
| 3.1.3 | *Compacted Backfill* | *CY* | *448,886* | *$ 8* | *$3,591,088* | |
| | | | | | | |
| 4 | Removal of Unpermitted Encroachment | LS | 1 | $ 20,000.00 | $ 20,000 | 160' K-Rail (DS of RR BG) & Metal Storages (Canada de SJ) & metal Building near Main st |
| 5 | Sediment Removal | SY | 4,600 | $ 3.00 | $13,800 | assumed 1' average thickness of sediment deposition |
| 6 | Vegetation Removal | LS | 1 | $ 200,000.00 | $ 200,000 | 120 trees (12" dia.) based on Google Earth; $1000 per tree removal + bulking 50% |
| 7 | Restoration of Displaced Existing Riprap | CY | 1,570 | $ 80.00 | $125,600 | restore per As-built riprap thickness (4 to 6' thick) |
| 8 | Floodwall - Ventura River (including removal of ex. Conc. Curb) | LF | 561.5 | $ 520 | $291,980 | |
| 9 | Floodwall - Canada de San Joaquin | LF | 285.0 | $ 480 | $136,800 | 305' feet - 20' openging for stoplog structure |
| 10 | Restoration of Existing Storm Drain System | LS | 1 | $ 39,000.00 | $39,000 | 13 STORM DRAIN SYSTEMS |
| 11 | Animal Burrow Removal | LS | 1 | $ 10,000.00 | $10,000 | |
| 12 | Levee Embankment Surface Erosion Repair | SY | 270 | $ 7.50 | $2,025 | Assumed fine grading with addition of import material (270 SYx6" deep) |
| 13 | Removal of Incised Channel near Riverside Toe | LF | 850 | $ 30.00 | $25,500 | 850' long; 1007.4CY of fill; $25/CY of fill |
| 14 | Regrading of Existing Access Ramps | EA | 3 | $ 5,000.00 | $15,000 | Grading to 2:1 sideslope |
| 15 | Stop-log Structure Replacement | EA | 1 | $ 75,000.00 | $75,000 | 11.4'H (110-98.5) x 20'W structure |
| 16 | Repair of Levee Embankment to As-built Conditiom (including removal of ex. Retaining wall) | LF | 310 | $ 140.00 | $43,400 | |
| 17 | Permit Application for Unpermitted Ex. Features | LS | 1 | $ 100,000.00 | $100,000.00 | 151 individual items (light poles, signs, etc.); $600 per each item |
| | | | | | | |
| | | | | | | |

| | |
|---|---|
| Subtotal: | $15,590,329 |
| | |
| Planning, Engineering, and Design (@ 12%) | $1,870,840 |
| Construction Management (@ 12%) | $1,870,840 |
| Subtotal: | $19,332,008 |
| | |
| Contingencies (@ 30%) | $5,799,602 |
| Subtotal: | $25,131,611 |
| | |
| Grand Total: | $25,131,600 |

## Alternative 3 - Sheet Piles

| | Contract Items | Unit | Quantity | Unit Cost | Total Cost | |
|---|---|---|---|---|---|---|
| 1 | Mobilization | LS | 1 | $ 1,030,000 | $1,030,000 | 5% of total construction costs |
| 2 | Clearing and Grubbing | Acre | 12.55 | $ 4,000 | $50,200 | |
| | | | | | | |
| 3 | Sheet Pile - (Sta.11+00 to 133+00) | LF | 7,700 | $ 2,532.23 | $19,498,206 | |
| 3.1 | *Sheet Pile* | *SF* | *350,250* | *$ 45* | *$15,761,250* | Unit cost from SAR |
| 3.2 | *Loose Rock* | *CY* | *1,996* | *$ 80* | *$159,704* | |
| 3.3 | *Excavation* | *CY* | *224,576* | *$ 8* | *$1,796,611* | |
| 3.4 | *Compacted Backfill* | *CY* | *222,580* | *$ 8* | *$1,780,641* | |
| | | | | | | |
| 4 | Removal of Unpermitted Encroachment | LS | 1 | $ 20,000.00 | $ 20,000 | 160' K-Rail (DS of RR BG) & Metal Storages (Canada de SJ) & metal Building near Main st |
| 5 | Sediment Removal | SY | 4,600 | $ 3.00 | $13,800 | assumed 1' average thickness of sediment deposition |
| 6 | Vegetation Removal | LS | 1 | $ 200,000.00 | $ 200,000 | 120 trees (12" dia.) based on Google Earth; $1000 per tree removal + bulking 50% |
| 7 | Restoration of Displaced Existing Riprap | CY | 1,570 | $ 80.00 | $125,600 | restore per As-built riprap thickness (4 to 6' thick) |
| 8 | Floodwall - Ventura River (including removal of ex. Conc. Curb) | LF | 561.5 | $ 520 | $291,980 | |
| 9 | Floodwall - Canada de San Joaquin | LF | 285.0 | $ 480 | $136,800 | |
| 10 | Restoration of Existing Storm Drain System | LS | 1 | $ 39,000.00 | $39,000 | 13 STORM DRAIN SYSTEMS |
| 11 | Animal Burrow Removal | LS | 1 | $ 10,000.00 | $10,000 | |
| 12 | Levee Embankment Surface Erosion Repair | SY | 270 | $ 7.50 | $2,025 | Assumed fine grading with addition of import material (270 SYx6" deep) |
| 13 | Removal of Incised Channel near Riverside Toe | LF | 850 | $ 30.00 | $25,500 | 850' long; 1007.4CY of fill; $25/CY of fill |
| 14 | Regrading of Existing Access Ramps | EA | 3 | $ 5,000.00 | $15,000 | Grading to 2:1 sideslope |
| 15 | Stop-log Structure Replacement | EA | 1 | $ 75,000.00 | $75,000 | 11.4'H (110-98.5) x 20'W structure |
| 16 | Repair of Levee Embankment to As-built Conditiom (including removal of ex. Retaining wall) | LF | 310 | $ 140.00 | $43,400 | |
| 17 | Permit Application for Unpermitted Ex. Features | LS | 1 | $ 100,000.00 | $100,000.00 | 151 individual items (light poles, signs, etc.); $600 per each item |
| | | | | | | |
| | | | | | | |
| | | | | Subtotal: | $21,676,511 | |
| | | Planning, Engineering, and Design (@ 12%) | | | $2,601,181 | |
| | | Construction Management (@ 12%) | | | $2,601,181 | |
| | | | | Subtotal: | $26,878,873 | |
| | | Contingencies | | (@ 30%) | $8,063,662 | |
| | | | | Subtotal: | $34,942,535 | |
| | | | | Grand Total: | $34,942,500 | |

| | Alternative 4 - Grade Control Structures | | | | | |
|---|---|---|---|---|---|---|
| | **Contract Items** | **Unit** | **Quantity** | **Unit Cost** | **Total Cost** | |
| 1 | Mobilization | LS | 1 | $ 790,000 | $790,000 | 5% of total construction costs |
| 2 | Clearing and Grubbing | Acre | 14.69 | $ 4,000 | $58,760 | |
| | | | | | | |
| 3.1 | Grade Control Structures - Structure | EA | 5 | $ 1,668,480.20 | $8,342,401 | |
| 3.1.1 | *Grouted Riprap* | *CY* | *40,543* | *$ 135* | *$5,473,305* | |
| 3.1.2 | *Excavation* | *CY* | *199,590* | *$ 8* | *$1,596,720* | |
| 3.1.3 | *Compacted Backfill* | *CY* | *159,047* | *$ 8* | *$1,272,376* | |
| | | | | | | |
| 3.2 | Grade Control Structures - Embankment Slope Protection | EA | 5 | $ 1,242,376.00 | $6,211,880 | |
| 3.2.1 | *Grouted Riprap* | *CY* | *6,097* | *$ 135* | *$823,095* | |
| 3.2.2 | *Excavation* | *CY* | *123,138* | *$ 20* | *$2,462,760* | |
| 3.2.3 | *Compacted Backfill* | *CY* | *117,041* | *$ 25* | *$2,926,025* | |
| | | | | | | |
| 4 | Removal of Unpermitted Encroachment | LS | 1 | $ 20,000.00 | $ 20,000 | 160' K-Rail (DS of RR BG) & Metal Storages (Canada de SJ) & metal Building near Main st |
| 5 | Sediment Removal | SY | 4,600 | $ 3.00 | $13,800 | assumed 1' average thickness of sediment deposition |
| 6 | Vegetation Removal | LS | 1 | $ 200,000.00 | $ 200,000 | 120 trees (12" dia.) based on Google Earth; $1000 per tree removal + bulking 50% |
| 7 | Restoration of Displaced Existing Riprap | CY | 1,570 | $ 80.00 | $125,600 | restore per As-built riprap thickness (4 to 6' thick) |
| 8 | Floodwall - Ventura River (including removal of ex. Conc. Curb) | LF | 561.5 | $ 520 | $291,980 | |
| 9 | Floodwall - Canada de San Joaquin | LF | 285.0 | $ 480 | $136,800 | |
| 10 | Restoration of Existing Storm Drain System | LS | 1 | $ 39,000.00 | $39,000 | 13 STORM DRAIN SYSTEMS |
| 11 | Animal Burrow Removal | LS | 1 | $ 10,000.00 | $10,000 | |
| 12 | Levee Embankment Surface Erosion Repair | SY | 270 | $ 7.50 | $2,025 | Assumed fine grading with addition of import material (270 SYx6" deep) |
| 13 | Removal of Incised Channel near Riverside Toe | LF | 850 | $ 30.00 | $25,500 | 850' long; 1007.4CY of fill; $25/CY of fill |
| 14 | Regrading of Existing Access Ramps | EA | 3 | $ 5,000.00 | $15,000 | Grading to 2:1 sideslope |
| 15 | Stop-log Structure Replacement | EA | 1 | $ 75,000.00 | $75,000 | 11.4'H (110-98.5) x 20'W structure |
| 16 | Repair of Levee Embankment to As-built Conditiom (including removal of ex. Retaining wall) | LF | 310 | $ 140.00 | $43,400 | |
| 17 | Permit Application for Unpermitted Ex. Features | LS | 1 | $ 100,000.00 | $100,000.00 | 151 individual items (light poles, signs, etc.); $600 per each item |
| | | | | | | |
| | | | | | | |
| | | | | Subtotal: | $16,501,146 | |
| | | | | | | |
| | | Planning, Engineering, and Design (@ 12%) | | | $1,980,138 | |
| | | Construction Management (@ 12%) | | | $1,980,138 | |
| | | | | Subtotal: | $20,461,421 | |
| | | | | | | |
| | | Contingencies | | (@ 30%) | $6,138,426 | |
| | | | | Subtotal: | $26,599,847 | |
| | | | | | | |
| | | | | Grand Total: | $26,599,800 | |

**Toe Protection**      **updated:** 2/8/2012

***(Grouted) Riprap Protection w/ Toedown***

| Statioin | Length [LF] | Avg. Scour Depth [LF] | h (depth of RR) [LF] | T [FT] | Riprap [SF] | Volume: Riprap [CY] | Volume: Excav. [CY] | Volume: Backfill [CY] | Unit Price: Grt.Riprap [$/CY] | Unit Price: Excav. [$/CY] | Unit Price: Backfill [$/CY] | Cost: Riprap [$] | Cost: Excav. [$] | Cost: Backfill [$] | Clr & Grb [SF] | Clr & Grb [Acre] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1100.0 | | | | | | | | | | | | | | | | |
| 2730.0 | | | | | | | | | | | | $ - | $ - | $ - | | |
| 5600.0 | | | | | | | | | | | | $ - | $ - | $ - | | |
| 6000.0 | 400.0 | 28.0 | 11.5 | 3.0 | 84.9 | 1257.0 | 28158.0 | 26901.0 | 135.00 | 8.00 | 8.00 | $ 169,695 | $ 225,264 | $ 215,208 | 48152.0 | 1.11 |
| 6200.0 | 200.0 | 28.0 | 15.5 | 3.0 | 114.4 | 847.0 | 14079.0 | 13232.0 | 135.00 | 8.00 | 8.00 | $ 114,345 | $ 112,632 | $ 105,856 | 24076.0 | 0.55 |
| 8800.0 | 2600.0 | 29.0 | 15.5 | 3.0 | 114.4 | 11015.0 | 194797.0 | 183782.0 | 135.00 | 8.00 | 8.00 | $ 1,487,025 | $ 1,558,376 | $ 1,470,256 | 322738.0 | 7.41 |
| 9700.0 | 900.0 | 35.0 | 15.5 | 3.0 | 114.4 | 3813.0 | 94506.0 | 90693.0 | 135.00 | 8.00 | 8.00 | $ 514,755 | $ 756,048 | $ 725,544 | 131967.0 | 3.03 |
| 11400.0 | 1700.0 | 28.0 | 13.5 | 3.0 | 99.6 | 6273.0 | 119670.0 | 113397.0 | 135.00 | 8.00 | 8.00 | $ 846,855 | $ 957,360 | $ 907,176 | 204646.0 | 4.70 |
| 13300.0 | 1900.0 | 33.0 | 13.5 | 3.0 | 99.6 | 7011.0 | 179403.0 | 172392.0 | 135.00 | 8.00 | 8.00 | $ 946,485 | $ 1,435,224 | $ 1,379,136 | 264347.0 | 6.07 |
| total: | 7700.0 [LF] | | | | | 30216.0 | 630613.0 | 600397.0 [CY] | | | | $ 4,079,160 | $ 5,044,904 | $ 4,803,176 | | 22.86 [Acre] |

| | |
|---|---|
| **Total:** | $ 13,927,240 |
| **Total (/ LF):** | $ 1,808.7 |

'Avg. Scour Depth' = depth between ex. Toe and potential scour limit, per profile
'h' = height of new riprap extension
'T' = Riprap protection tickness

***Concrete Lining***

| Statioin | Length [LF] | Avg. Scour Depth [LF] | h (depth of conc) [LF] | t [FT] | Conc [SF] | Volume: Conc [CY] | Volume: Excav. [CY] | Volume: Backfill [CY] | Unit Price: Conc [$/CY] | Unit Price: Excav. [$/CY] | Unit Price: Backfill [$/CY] | Cost: Riprap [$] | Cost: Excav. [$] | Cost: Backfill [$] | Clr & Grb [SF] | Clr & Grb [Acre] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1100.0 | | | | | | | | | | | | | | | | |
| 2730.0 | | | | | | | | | | | | $ - | $ - | $ - | | |
| 5600.0 | | | | | | | | | | | | $ - | $ - | $ - | | |
| 6000.0 | 400.0 | 28.0 | 15.0 | 0.83 | 23.88 | 354.00 | 20114.35 | 19760.4 | 350.00 | 8.00 | 8.00 | $ 123,900 | $ 160,915 | $ 158,083 | 43100.0 | 0.99 |
| 6200.0 | 200.0 | 28.0 | 19.0 | 0.83 | 29.88 | 221.00 | 10090.51 | 9869.5 | 350.00 | 8.00 | 8.00 | $ 77,350 | $ 80,724 | $ 78,956 | 20950.0 | 0.48 |
| 8800.0 | 2600.0 | 29.0 | 19.0 | 0.83 | 29.88 | 2877.00 | 140108.10 | 137231.1 | 350.00 | 8.00 | 8.00 | $ 1,006,950 | $ 1,120,865 | $ 1,097,849 | 282100.0 | 6.48 |
| 9700.0 | 900.0 | 35.0 | 19.0 | 0.83 | 29.88 | 996.00 | 69673.96 | 68678.0 | 350.00 | 8.00 | 8.00 | $ 348,600 | $ 557,392 | $ 549,424 | 117900.0 | 2.71 |
| 11400.0 | 1700.0 | 28.0 | 17.0 | 0.83 | 26.88 | 1692.00 | 85533.22 | 83841.2 | 350.00 | 8.00 | 8.00 | $ 592,200 | $ 684,266 | $ 670,730 | 180625.0 | 4.15 |
| 13300.0 | 1900.0 | 33.0 | 17.0 | 0.83 | 26.88 | 1891.00 | 131396.88 | 129505.9 | 350.00 | 8.00 | 8.00 | $ 661,850 | $ 1,051,175 | $ 1,036,047 | 237500.0 | 5.45 |
| total: | 7700.0 [LF] | | | | | 8031.0 | 456917.0 | 448886.0 [CY] | | | | $ 2,810,850 | $ 3,655,336 | $ 3,591,088 | | 20.25 [Acre] |

| | |
|---|---|
| **Total:** | $ 10,057,274 |
| **Total (/ LF):** | $ 1,306.1 |

'Avg. Scour Depth' = depth between ex. Toe and potential scour limit, per profile
'h' = height of new concrete lining
't' = thickness of concrete lining

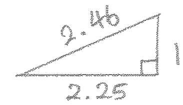
**TETRA TECH, INC.**

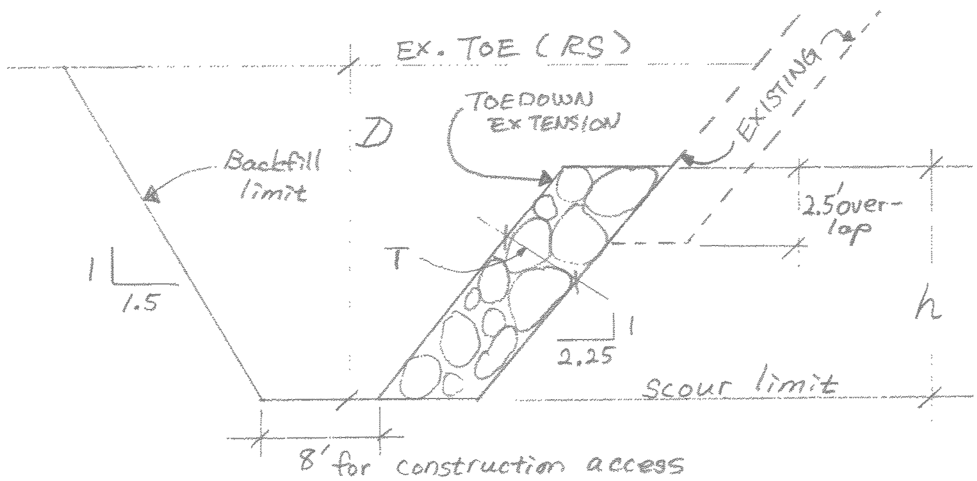
CLIENT ______ JOB NO. ______ PAGE ______

PROJECT T27654 VR-1 COMPUTED BY JWS DATE 2/8/12

DETAIL COST-ESTIMATE: Scour Protection CHECKED BY ______ DATE ______

## (Grouted) Riprap Protection




$$A_{RIPRAP} = 2.46 * T * h$$

$$A_{EXCAVATION} = \left(\frac{[(2.46*T)+8'] + [(2.46*T)+8'+1.5D+2.25D]}{2}\right)*D$$

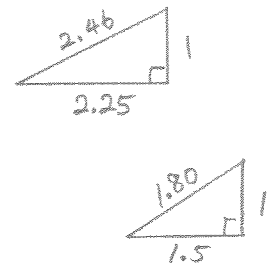
$$= \frac{(4.92T + 16 + 3.75D)D}{2}$$

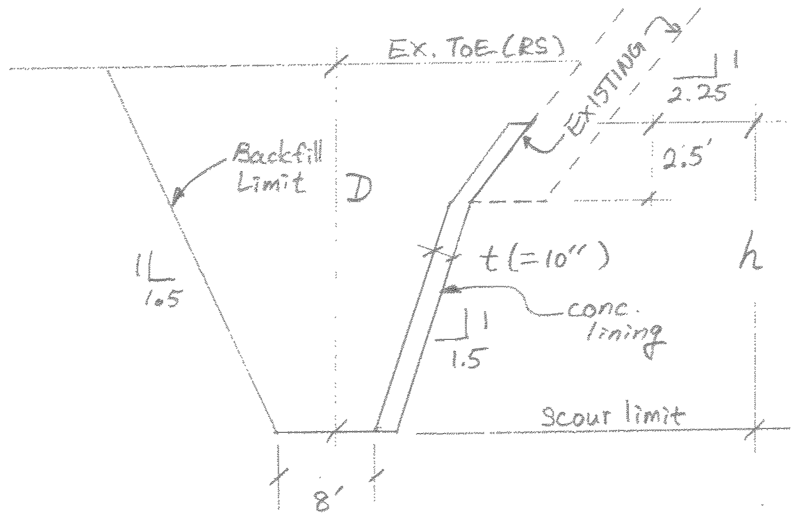
$$A_{BACKFILL} = A_{EXCAVATION} - A_{RIPRAP}$$

TETRA TECH, INC.

| | | | | | |
|---|---|---|---|---|---|
| CLIENT | | JOB NO. | | PAGE | |
| PROJECT | T27654 VR-1 | COMPUTED BY | JWS | DATE | 2/8/12 |
| DETAIL | COST-ESTIMATE: Scour Protection | CHECKED BY | | DATE | |

Concrete Lining




$$A_{conc.} = 1.80t\,(h-2.5) + 2.46t * 2.5$$

$$= 1.80 * t * h - 4.5t + 6.15t$$

$$= 1.80t*h + 1.65t$$

$$A_{EXCAV} = [(8' + 1.80t) + (8' + 1.80t + ((h-2.5)*1.5)*2)] * (h-2.5)*0.5$$

$$+ \frac{1}{2}\{[(8 + 1.80t) + ((h-2.5)*1.5*2)]*2 + (D-h+2.5)*(1.5+2.25)\} * (D-h+2.5)$$

$$= 0.5(3.6t + 3h + 8.5)(h-2.5) + 0.5(3.6t + 2.25h + 3.75D - 12.125)(D-h+2.5$$

$$A_{BACKFILL} = A_{EXCAV} - A_{conc.}$$

***Sheet Pile***

| Statioin | Length [LF] | Avg. Scour Depth [LF] | h (ex. toedown) [LF] | avg. H [FT] | Length & Volumes: Sht Pile [SF] | Loose rock [CY] | Excav. [CY] | Backfill [CY] | Unit Price: Sht pile [$/SF] | Loose R [$/CY] | Excav. [$/CY] | Backfill [$/CY] | Cost: Sht Pile [$] | Loose R [$] | Excav. [$] | Backfill [$] | Clr & Grb [SF] | Clr & Grb [Acre] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1100.0 | | | | | | | | | | | | | | | | | | |
| 2730.0 | | | | | | | | | | | | | | | | | 0.0 | 0.00 |
| 5600.0 | | | | | | | | | | | | | | | | | 0.0 | 0.00 |
| 6000.0 | 400.0 | 28.0 | 12.0 | 55.5 | 22200.0 | 103.7 | 5777.8 | 5674.1 | 45.00 | 80.00 | 8.00 | 8.00 | $ 999,000 | $ 8,296 | $ 46,222 | $ 45,393 | 28400.0 | 0.65 |
| 6200.0 | 200.0 | 28.0 | 15.0 | 46.5 | 9300.0 | 51.9 | 4236.1 | 4184.3 | 45.00 | 80.00 | 8.00 | 8.00 | $ 418,500 | $ 4,148 | $ 33,889 | $ 33,474 | 14200.0 | 0.33 |
| 8800.0 | 2600.0 | 29.0 | 16.0 | 46.5 | 120900.0 | 674.1 | 61629.6 | 60955.6 | 45.00 | 80.00 | 8.00 | 8.00 | $ 5,440,500 | $ 53,926 | $ 493,037 | $ 487,644 | 184600.0 | 4.24 |
| 9700.0 | 900.0 | 35.0 | 22.0 | 46.5 | 41850.0 | 233.3 | 37583.3 | 37350.0 | 45.00 | 80.00 | 8.00 | 8.00 | $ 1,883,250 | $ 18,667 | $ 300,667 | $ 298,800 | 63900.0 | 1.47 |
| 11400.0 | 1700.0 | 28.0 | 15.0 | 46.5 | 79050.0 | 440.7 | 36006.9 | 35566.2 | 45.00 | 80.00 | 8.00 | 8.00 | $ 3,557,250 | $ 35,259 | $ 288,056 | $ 284,530 | 120700.0 | 2.77 |
| 13300.0 | 1900.0 | 33.0 | 22.0 | 40.5 | 76950.0 | 492.6 | 79342.6 | 78850.0 | 45.00 | 80.00 | 8.00 | 8.00 | $ 3,462,750 | $ 39,407 | $ 634,741 | $ 630,800 | 134900.0 | 3.10 |
| total: | 7700.0 [LF] | | | | 350250.0 | 1996.3 | 224576.4 | 222580.1 [CY] | | | | | $ 15,761,250 | $ 159,704 | $ 1,796,611 | $ 1,780,641 | | 12.55 [Acre] |

| | |
|---|---|
| **Total:** | $ 19,498,206 |
| **Total (/ LF):** | $ 2,532.2 |

'Avg. Scour Depth' = depth between ex. Toe and potential scour limit, per profile
'avg. H' = Average height of sheet piles; 3*"average scour depth"
'h (ex. Toedown)' = depth between ex. Toe and existing toedown
'L' = length of launchable rock
Assume sheet pile is PZC 12 w/ width of 27.88 in.(2.32') and 50.4 lb/VLF
'Sht Pile' is the total vertical linear feet of sheet pile, computed by dividing a total square feet of sheet pile area ('Length' x 'avg. H') by a width of each sheet pile (2.33').

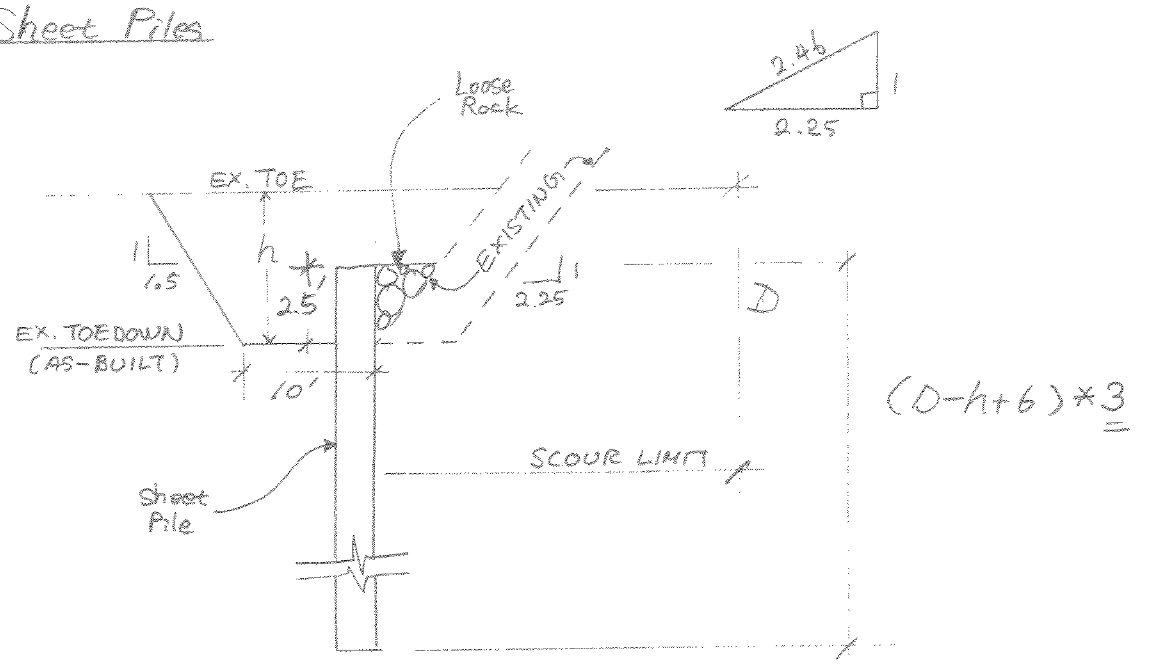
TETRA TECH, INC.

CLIENT ______ JOB NO. ______ PAGE ______

PROJECT T27654 VR-1 COMPUTED BY JWS DATE 2/8/12

DETAIL COST ESTIMATE: Scour Protection CHECKED BY ______ DATE ______

## Sheet Piles




$$H_{SHT\ PILE} = (D - h + 2.5) * 3$$

$$A_{EXCAV.} = (\overset{20}{10' + 10'} + \overset{3.25h}{1.5h + 2.25h}) * h * 0.5$$

$$= (3.75h + 20)\,0.5h$$

$$= 1.875h^2 + 10h$$

$$A_{Loose\ Rock} = (2.5' \times 2.5' \times 2.25) * 0.5$$

$$= 7.0$$

$A_{BACKFILL} = A_{EXCAV} - A_{LOOSE\ Rock}$ (sheet pile not accounted)

**Grade Control Structure** **updated:** **2/6/2012**

***Structure only***

| | | | Cross Sectional Areas | | | Volumes | | | Unit Price | | | Cost | | | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| GCS No. | Location Statioin | Length [LF] | GCS [SF] | Excav. [SF] | Backfill [SF] | GCS [CY] | Excav. [CY] | Backfill [CY] | GCS [$/CY] | Excav. [$/CY] | Backfill [$/CY] | GCS [$] | Excav. [$] | Backfill [$] | Cost [$] |
| 1 | 7700.0 | 780.0 | 235.4 | 1158.9 | 923.5 | 6801.0 | 33480.0 | 26679.0 | 135 | 8 | 8 | $ 918,135 | $ 267,840 | $ 213,432 | $ 1,399,407 |
| 2 | 9400.0 | 960.0 | 235.4 | 1158.9 | 923.5 | 8370.0 | 41206.0 | 32836.0 | 135 | 8 | 8 | $ 1,129,950 | $ 329,648 | $ 262,688 | $ 1,722,286 |
| 3 | 10800.0 | 760.0 | 235.4 | 1158.9 | 923.5 | 6626.0 | 32621.0 | 25995.0 | 135 | 8 | 8 | $ 894,510 | $ 260,968 | $ 207,960 | $ 1,363,438 |
| 4 | 11350.0 | 940.0 | 235.4 | 1158.9 | 923.5 | 8196.0 | 40347.0 | 32151.0 | 135 | 8 | 8 | $ 1,106,460 | $ 322,776 | $ 257,208 | $ 1,686,444 |
| 5 | 12275.0 | 1210.0 | 235.4 | 1158.9 | 923.5 | 10550.0 | 51936.0 | 41386.0 | 135 | 8 | 8 | $ 1,424,250 | $ 415,488 | $ 331,088 | $ 2,170,826 |
| | | | | | | 40543.0 | 199590.0 | 159047.0 [CY] | | | | | | | |

| Clr & Grb [SF] | Clr & Grb [Acre] |
|---|---|
| 79560.0 | 1.83 |
| 97920.0 | 2.25 |
| 77520.0 | 1.78 |
| 95880.0 | 2.20 |
| 123420.0 | 2.83 |
| | 10.89 [Acre] |

| | |
|---|---|
| **Total:** | $ 8,342,401 |
| **Total (EA):** | $ 1,668,480 |

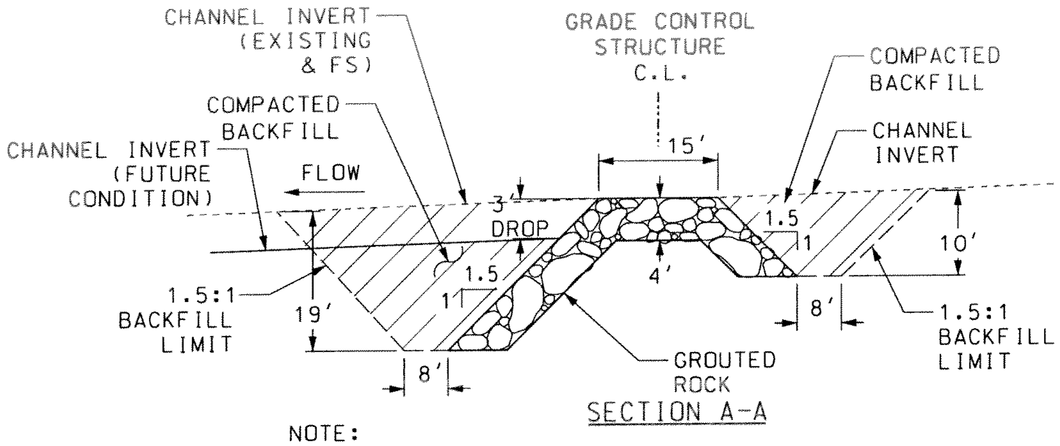
**TETRA TECH, INC.**

CLIENT ______ JOB NO. ______ PAGE ______

PROJECT ______ COMPUTED BY JWS DATE 2/9/12

DETAIL COST ESTIMATE – GCS (Structure) CHECKED BY ______ DATE ______

## Grade Control Structure (Structure)




$$A_{GCS} = \frac{(15+15+1.5\times4\times2)\times4}{2} + (7.21\times6) + (7.21\times15)$$

$$= 84 + 43.26 + 108.15$$

$$= \underline{235.41\ SF}$$

$$A_{BACKFILL} = \frac{(19\times(19\times1.5\times2))}{2} + \frac{(10\times(10\times1.5\times2))}{2}$$

$$= 541.5\ SF + 150\ SF$$

$$= \underline{691.5\ SF}$$

$$A_{EXCAV} = A_{GCS} + A_{BACKFILL}$$

$$= \underline{926.91\ SF}$$

**Grade Control Structure**

updated: 2/7/2012

***Embankment Slope Protection***

*(variables are based on attached figure)*

*(values are based on the method described in the Hydraulic Appendix)*

| GCS No. | Location Statioin | Dd [LF] | Du [LF] | Dc [LF] | Length of Slope Protection [LF] | D [LF] | h [LF] | T [LF] | Riprap [SF] | Volume: Riprap [CY] | Volume: Excav. [CY] | Volume: Backfill [CY] | Unit Price: Grt.Riprap [$/CY] | Unit Price: Excav. [$/CY] | Unit Price: Backfill [$/CY] | Cost: Riprap [$] | Cost: Excav. [$] | Cost: Backfill [$] | Clr & Grb [SF] | Clr & Grb [Acre] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 7700.0 | 16.0 | 10.0 | 17.0 | | | | | | | | | | | | | | | | |
| | **DS** | | | | 96.0 | 27.0 | 11.0 | 3.0 | 99.6 | 354.0 | 6336.0 | 5982.0 | $ 135 | $ 20 | $ 25 | $ 47,790 | $ 126,720 | $ 149,550 | 10344.0 | 0.24 |
| | **DS** | | | | 96.0 | 32.0 | 16.0 | 3.0 | 136.5 | 485.0 | 8577.0 | 8092.0 | $ 135 | $ 20 | $ 25 | $ 65,475 | $ 171,540 | $ 202,300 | 11904.0 | 0.27 |
| | **US** | | | | 85.0 | 23.0 | 8.0 | 3.0 | 77.5 | 244.0 | 4236.0 | 3992.0 | $ 135 | $ 20 | $ 25 | $ 32,940 | $ 84,720 | $ 99,800 | 8053.8 | 0.18 |
| | | | | | | | | | | | | | | | *subtotal:* | $ 146,205 | $ 382,980 | $ 451,650 | | |
| 2 | 9400.0 | 16.0 | 10.0 | 18.0 | | | | | | | | | | | | | | | | |
| | **DS** | | | | 96.0 | 35.0 | 13.0 | 3.0 | 114.4 | 407.0 | 10081.0 | 9674.0 | $ 135 | $ 20 | $ 25 | $ 54,945 | $ 201,620 | $ 241,850 | 12840.0 | 0.29 |
| | **DS** | | | | 96.0 | 40.0 | 18.0 | 3.0 | 151.3 | 538.0 | 12854.0 | 12316.0 | $ 135 | $ 20 | $ 25 | $ 72,630 | $ 257,080 | $ 307,900 | 14400.0 | 0.33 |
| | **US** | | | | 90.0 | 32.0 | 8.0 | 3.0 | 77.5 | 258.0 | 8041.0 | 7783.0 | $ 135 | $ 20 | $ 25 | $ 34,830 | $ 160,820 | $ 194,575 | 11160.0 | 0.26 |
| | | | | | | | | | | | | | | | *subtotal:* | $ 162,405 | $ 619,520 | $ 744,325 | | |
| 3 | 10800.0 | 16.0 | 10.0 | 14.0 | | | | | | | | | | | | | | | | |
| | **DS** | | | | 96.0 | 32.0 | 15.0 | 3.0 | 129.2 | 459.0 | 8577.0 | 8118.0 | $ 135 | $ 20 | $ 25 | $ 61,965 | $ 171,540 | $ 202,950 | 11904.0 | 0.27 |
| | **DS** | | | | 96.0 | 37.0 | 20.0 | 3.0 | 166.1 | 590.0 | 11150.0 | 10560.0 | $ 135 | $ 20 | $ 25 | $ 79,650 | $ 223,000 | $ 264,000 | 13464.0 | 0.31 |
| | **US** | | | | 70.0 | 30.0 | 13.0 | 3.0 | 114.4 | 297.0 | 5571.0 | 5274.0 | $ 135 | $ 20 | $ 25 | $ 40,095 | $ 111,420 | $ 131,850 | 8225.0 | 0.19 |
| | | | | | | | | | | | | | | | *subtotal:* | $ 181,710 | $ 505,960 | $ 598,800 | | |
| 4 | 11350.0 | 16.0 | 10.0 | 12.0 | | | | | | | | | | | | | | | | |
| | **DS** | | | | 96.0 | 32.0 | 14.0 | 3.0 | 121.8 | 433.0 | 8577.0 | 8144.0 | $ 135 | $ 20 | $ 25 | $ 58,455 | $ 171,540 | $ 203,600 | 11904.0 | 0.27 |
| | **DS** | | | | 96.0 | 38.0 | 19.0 | 3.0 | 158.7 | 564.0 | 11705.0 | 11141.0 | $ 135 | $ 20 | $ 25 | $ 76,140 | $ 234,100 | $ 278,525 | 13776.0 | 0.32 |
| | **US** | | | | 60.0 | 27.0 | 11.0 | 3.0 | 99.6 | 221.0 | 3960.0 | 3739.0 | $ 135 | $ 20 | $ 25 | $ 29,835 | $ 79,200 | $ 93,475 | 6465.0 | 0.15 |
| | | | | | | | | | | | | | | | *subtotal:* | $ 164,430 | $ 484,840 | $ 575,600 | | |
| 5 | 12275.0 | 16.0 | 10.0 | 10.0 | | | | | | | | | | | | | | | | |
| | **DS** | | | | 96.0 | 32.0 | 15.0 | 3.0 | 129.2 | 459.0 | 8577.0 | 8118.0 | $ 135 | $ 20 | $ 25 | $ 61,965 | $ 171,540 | $ 202,950 | 11904.0 | 0.27 |
| | **DS** | | | | 96.0 | 37.0 | 20.0 | 3.0 | 166.1 | 590.0 | 11150.0 | 10560.0 | $ 135 | $ 20 | $ 25 | $ 79,650 | $ 223,000 | $ 264,000 | 13464.0 | 0.31 |
| | **US** | | | | 50.0 | 29.0 | 12.0 | 3.0 | 107.0 | 198.0 | 3746.0 | 3548.0 | $ 135 | $ 20 | $ 25 | $ 26,730 | $ 74,920 | $ 88,700 | 5712.5 | 0.13 |
| | | | | | | | | | | | | | | | *subtotal:* | $ 168,345 | $ 469,460 | $ 555,650 | | |
| | | | | | | | | | | 6097.0 | 123138.0 | 117041.0 [CY] | | | Total: | $ 823,095 | $ 2,462,760 | $ 2,926,025 | | 3.80 [Acre] |

| | |
|---|---|
| **Total:** | $ 6,211,880 |
| **Total (EA):** | $ 1,242,376 |

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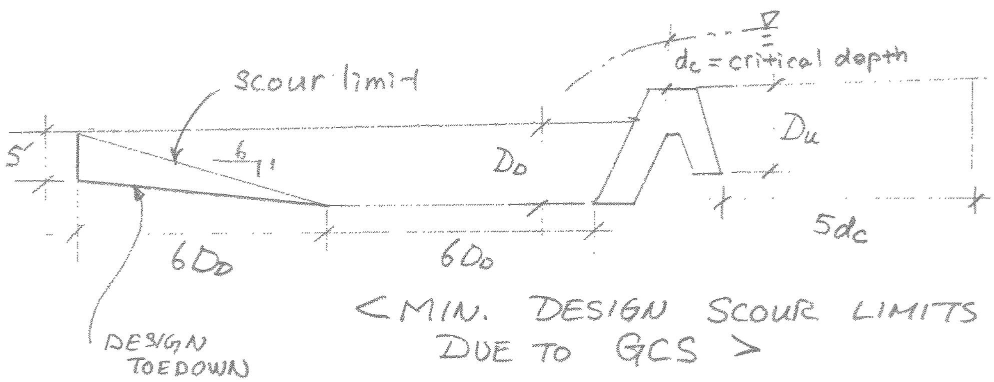
CLIENT ______ JOB NO. ______ PAGE ______

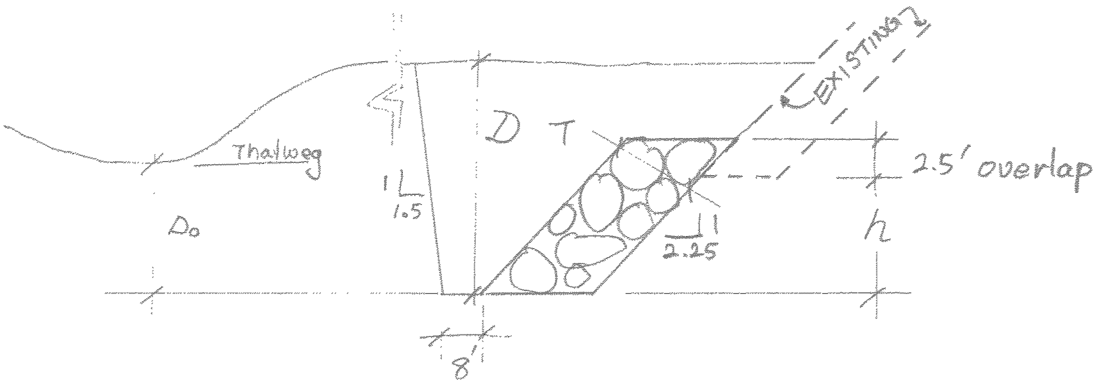
PROJECT ______ COMPUTED BY ______ DATE ______

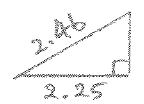
DETAIL COST ESTIMATE – GCS (bank Scour) CHECKED BY ______ DATE 2/6/12

## Bank / Levee Protection near GCS
(Based on Hydraulics Appendix)




<MIN. DESIGN SCOUR LIMITS DUE TO GCS>




$$A_{RIPRAP} = 2.46 * T * (h+25)$$

$$A_{EXCAV} = \frac{\left([(2.46*T)+8'] + [(2.46*T)+8'+1.5D+2.25D]\right)*D}{2}$$

$$= \frac{(4.92T + 16 + 3.75D)\,D}{2}$$

$$A_{BACKFILL} = A_{EXCAV} - A_{riprap}$$

**Floodwall Calculation** **updated: 1/4/2012**

*(Cross sectional areas of Vertical portion and foundation of floodwalls are based on the attached scketch.*
*Dimensions of FW are not based on actual structural analysis but based on ex. Floodwall design in Ventura County (ASR-2).)*

**Ex. Conc. Curb Removal** *(unit price (prorated): 02 41 13.90 0400 Selective Demo, Ret. Wall)*

| Statioin | Length [LF] | Avg. FW Ht [LF] | Xsec Area [SF] | Xsec Area [SY] | Volume [CY] | Unit Price [$/LF] | Cost [$] | |
|---|---|---|---|---|---|---|---|---|
| 2140.0 | | | | | | | | |
| 2200.0 | 60.0 | 3.5 | 6.7 | 0.74 | 14.81 | $ 111.4 | $ 6,685.0 | $191/LF of 6' high FW |
| 2238.5 | 38.5 | 2.5 | 5.0 | 0.56 | 7.13 | $ 79.6 | $ 3,064.0 | |
| 2435.0 | 196.5 | 1.5 | 3.3 | 0.37 | 24.26 | $ 47.8 | $ 9,382.9 | |
| 2690.0 | 255.0 | 1.0 | 2.5 | 0.28 | 23.61 | $ 31.8 | $ 8,117.5 | |
| | 550 | | | | | | $ 27,249 | |
| | | | | | | Total (Removal Only): | $ 27,000 | |

**New Conc. Floodwall - Ventura River**

| Statioin | Length [LF] | Avg. FW Ht [LF] | Xsec Area [SF] | | Xsec Area [SY] | | Volume [CY] | | Unit Price | | Cost [$] | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Vert. | Found. | Vert. | Found. | Vert. | Found. | Vert [$/CY] | Horiz [$/CY] | Vert | Horiz |
| 2128.5 | | | | | | | | | | | | |
| 2200.0 | 71.5 | 4.0 | 5.0 | 14.0 | 0.56 | 1.56 | 13.24 | 37.07 | $ 1,000 | $ 1,000 | $ 13,241 | $ 37,074 |
| 2238.5 | 38.5 | 2.5 | 3.5 | 9.5 | 0.39 | 1.06 | 4.99 | 13.55 | $ 1,000 | $ 1,000 | $ 4,991 | $ 13,546 |
| 2435.0 | 196.5 | 1.5 | 2.5 | 9.5 | 0.28 | 1.06 | 18.19 | 69.14 | $ 1,000 | $ 1,000 | $ 18,194 | $ 69,139 |
| 2690.0 | 255.0 | 1.0 | 2.0 | 9.5 | 0.22 | 1.06 | 18.89 | 89.72 | $ 1,000 | $ 1,000 | $ 18,889 | $ 89,722 |
| | 561.5 | | | | | | | | | | $ 55,315 | $ 209,481 |
| | | | | | | | | | | | Total (New FW Only): | $ 265,000 |
| | | | | | | | | | | **Total (Removal and new):** | | $ 292,000 |
| | | | | | | | | | | **Total (Removal and new / LF):** | | $ 520.00 |

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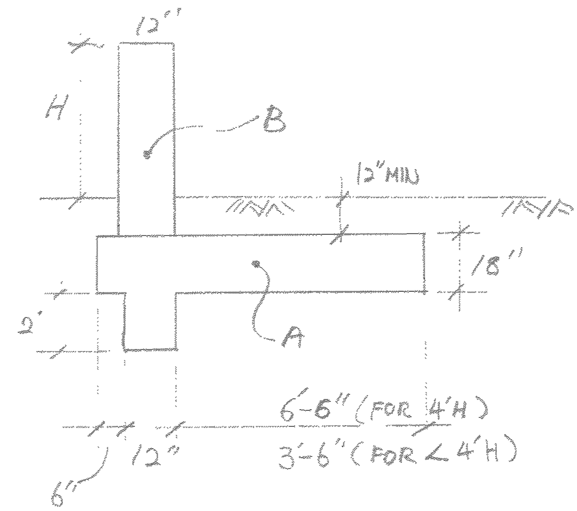
PROJECT T27654 VR-1 COMPUTED BY ______ DATE ______

DETAIL COST EST. – FW CHECKED BY ______ DATE ______

NEW FW




FOUNDATION:

$$A = \left[\left(6'' + 12'' + (6'\text{-}6'')\right) \times (18'')\right] + \left[2' \times 12''\right]$$

$$= 12\,SF + 2\,SF$$

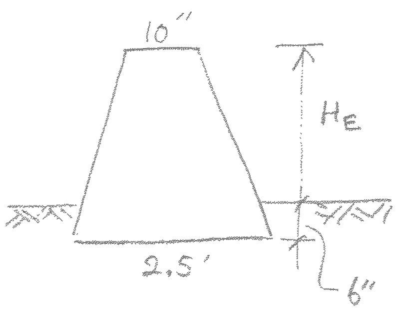
$$= 14\,SF\ (\text{FOR } 4'H) \text{ OR } 9.5\,SF\ (H < 4')$$

VERTICAL WALL:

$$B = (H + 12'') \times 12''$$

$$= (H + 1')\ SF$$

Ex. Conc. Curb




Sheet Pile

Assume PZC13 and the total height of pile, H, = 3 * "Scour depth"

Weight of PZC13 = 50.4 lb/LF

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Sediment Removal

| NEAR | L | W | |
|---|---|---|---|
| 11+00 | 130' | 10' | = 1300 SF |
| 22+00 → 24+00 | 200' | 17' | = 3400 |
| 28+00 | 15' | 20' | = 3000 |
| 40+00 | 65' | 15' | = 975 |
| 42+00 → U/S | 530' | 15' | = 7950 |
| | | | 16,625 SF |

16,625 SF → 40898 SF on 2.25:1 slope

4544 SY

Assume AVG thickness of 12", # of sed to be removed

≅ 1,515 CY

$ 5.80 / CY Excavation 1' to 4' deep, 3/4 CY excavator

$ 2.35 / CY hauling, 8 CY TRUCK, cycle 0.5 mile, 15 MPH

≈ $ 8.15/CY

∴ $8.15 CY × 1,515 CY = $ 12,347 TOTAL JOB

$12,347 / (4,544 SY) = $ 2.72 / SY ≈ $ 2.80 / SY

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CLIENT ______ JOB NO. ______ PAGE ______

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DETAIL ______ CHECKED BY ______ DATE ______

Levee Embankment Surface Erosion

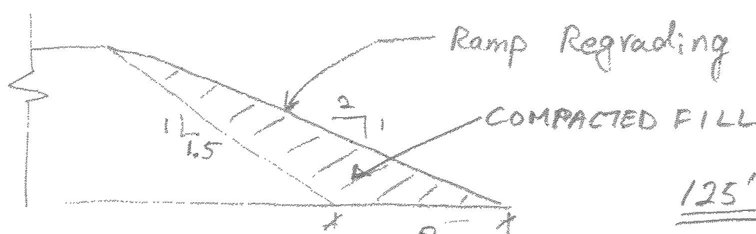
| NEAR | L × W | | |
|---|---|---|---|
| 14+00 | 10′ × 10′ | = 100 SF | TOE |
| 131+00 | 90′ × 10′ | = 900 SF | TOE |
| 140+00 | 15′ × 35′ | = 525 SF | SLOPE |
| 142+00 | 130′ × 7′ | = 910 SF | TOP |
| | | 2,435 SF | |
| | | = 270 SY | |

① Assume 6″ deep erosion, $270\ SY \times \frac{0.5'\ deep}{3'/YD} = 45\ CY$ of MATERIAL

$\$20/_{CY\ of\ Import} \times 45\ CY = \$\underline{\underline{900}} \rightarrow \$3.33/_{SY}$

② Fine grading $\$3.73/_{SY}$ [RSM 31.22 16.10-0012]

$\Rightarrow \$3.33 + \$3.73 = \$7.^{10} \rightarrow \$7.^{50}/SY$




$\underline{125'}$ Long & Avg ht of $\underline{\underline{9'}}$ (0′ to 18′)

AT 18′H, B = 9′, ∀fill = 81 SF

AT 0′H, B = 0′, ∀fill = 0 SF

Avg. ∀fill = 40.5 SF

∀ of fill required = 40.5 SF × 125′

$= \underline{\underline{187.5}}\ CY \times \$12/_{CY}$

$= \$2,250\ /_{EA}$

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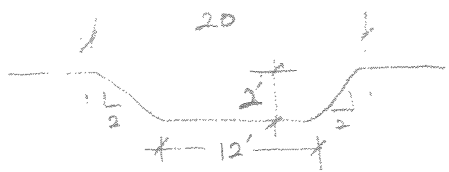
INCISED CHANNEL RESTORATION

STA. 28+50 to 37+00 → 850' long

Avg. width = 20'

Avg. depth = 3'




$\forall_{Fill} = (20' + 12') \times 2' \times 0.5 \times 850'$

$= 1007.4$ CY

$\therefore$ 1007.4 CY × $\frac{\$25}{\text{CY of FILL}}$ = \$25,185 = \$30/LF

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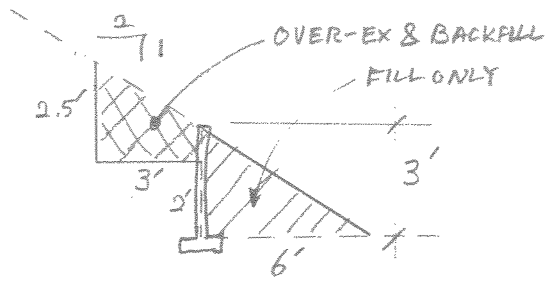
## Repair Levee Embankment to As-built Conditions

L = 310'




$A_{OVER\text{-}EX} = (1+2.5)\times 3 \times \frac{1}{2}$
$= 5.25\,SF$

$A_{FILL} = A_{over\text{-}ex} + 6\times 3\times\frac{1}{2}$
$= 5.25 + 9$
$= 14.25\,SF$

→. Removal of 3'H retaining wall,

$ 191/LF for 6'H RW removal ⇒ $ 100/LF for 3'H

→. $V_{OVER\text{-}EX}$ = 60 CY

60 CY × $ 10/CY of excav. = $ 600

$V_{FILL}$ = 164 CY

164 CY × $20/CY of fill = $ 3272

$ 3872 → $ 12.5/LF

∴ $100 + $ 12.5 = $ 112.5/LF × 25% bulking

≅ $ 140/LF

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