Ventura County
Watershed Protection District

Santa Clara River Levee (SCR-1) Evaluation and Rehabilitation Study

Ventura County, California

Basis of Design Report

January 2015

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Prepared for:

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Prepared by:

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EXECUTIVE SUMMARY

In 1969, the Santa Clara River Levee (SCR-1) was breached during a severe flood event with an estimated peak discharge of 165,000 cubic feet per second (cfs). In an attempt to mitigate the damage during high flows, a feasibility-level design has been prepared on the basis of a preferred design alternative (soil cement bank protection) to strengthen the levee embankment and minimize the damage due to scour and flooding.

The SCR-1 system, which was designed by the U.S. Army Corps of Engineers (Corps) in 1958 to control the standard project flood discharge of 225,000 cfs, is located in the city of Oxnard and unincorporated areas of Ventura County, California. SCR-1, owned and operated by the Ventura County Watershed Protection District (VCWPD), is 4.72 miles long and located along the southeast bank of the Santa Clara River between U.S. Highway (Hwy) 101 and Saticoy.

Based on previous work for the FEMA Levee Certification Program, SCR-1 does not currently meet the requirements set forth in Code of Federal Regulations Title 44, Section 65.10 (44 CFR 65.10) of the National Insurance Program regulations. Additionally, the most recent Corps periodic inspection report published in 2011 also rated SCR-1 as “unacceptable,” making the levee system ineligible for federal funding of repairs if damaged during a flood event.

The purpose of this project is to develop a plan for addressing the deficiencies to reduce the flood risks to the community and to meet the levee criteria of both FEMA and the Corps. This report documents the technical studies and development of the feasibility-level design plans and cost estimates.

The hydrology and hydraulic analysis of SCR-1 was performed to verify the adequacy of existing levee freeboard and determine the minimum top of levee elevations required to meet FEMA/Corps freeboard requirements. At various stages of the design development, the minimum top of levee elevations were defined as the highest of the following elevations:

- The 100-year flow (226,000 cfs) water surface elevations, with a conditional non-exceedance probability of at least 90 percent (with a minimum of 3 feet of freeboard).
- The design flow (250,000 cfs) water surface elevations with FEMA freeboard requirements.
- The 200-year flow (303,970 cfs) water surface elevations with FEMA freeboard requirements.

The levee toedown elevations were determined on the basis of the scour analysis and defined as the river invert elevation minus the potential scour depth, which varies from 5 to 15 feet along the levee alignment.

The selection of a preferred design alternative for the levee improvements was achieved by means of a screening process that involved various levels of the design analyses: preliminary design and conceptual-level design. The study of preliminary design alternatives included a total of six levee revetment types that varied from soil cement (Alternative A) to grouted riprap

(Alternative F). Using variation in design geometry and combining them with other types of revetments, a total of 12 individual design alternatives were analyzed and compared on the basis of typical sections, quantities, and cost estimates. Based on the preliminary designs, VCWPD selected three alternatives for further evaluation by preparing the conceptual-level design documents:

- Alternative 1 – an 8-foot-wide section of soil cement with a slope of 1H:1V
- Alternative 2 – a 42-inch-thick layer of loose ½-ton rock riprap with a slope of 2H:1V
- Alternative 3 – a 30-inch-thick layer of grouted ¼-ton rock riprap with a slope of 2H:1V above ground and 1.5H:1V below ground

The conceptual-level design documents including design drawings, engineering calculations, and cost estimates were prepared for the three selected alternatives using the design flow and the 200-year flow for comparison.

Based on the engineering feasibility, environmental impact, and overall construction costs of the conceptual-level designs, VCWPD selected a preferred design alternative of soil cement to be studied further in the feasibility-level design. Based on hydraulic analyses, the existing upstream part of the project is entrenched and is not considered a levee condition. Therefore the feasibility-level design alignment was modified near the Central Avenue Drain levee penetration. In the revised levee alignment, SCR-1 was realigned to turn toward the east immediately upstream of the Central Avenue Drain levee penetration and continue east along the northerly edge of the Central Avenue Drain until it ties into high ground near Vineyard Avenue. It should be noted that the realignment of SCR-1 will need to undergo the FEMA review process for official approval.

The feasibility-level design documents including design drawings, engineering calculations, and cost estimates were prepared using the governing parameters from both the design flow and the 100-year flow.

The feasibility-level design includes an 8-foot-wide soil cement section along the levee with the exception of a portion underneath the Hwy 101 Bridge. Under the bridge, there is limited access for heavy construction equipment, requiring grouted riprap placement on the lower side slope and toedown and a reinforced concrete floodwall on top to achieve the levee height requirement. The design involves the construction of access ramps on the riverward side of the levee, a bicycle ramp on the landward side of the levee, and a launchable stone near the point of realignment. The design also involves modification of the existing storm drain penetrations to accommodate the new levee design.

A feasibility-level cost estimate was prepared for planning purposes and includes mitigation costs for all areas affected or disturbed by the construction of the SCR-1 improvements.

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1.0 INTRODUCTION

In 1969, the Santa Clara River 1 Levee (SCR-1) was breached during a severe flood event with an estimated peak discharge of 165,000 cubic feet per second (cfs). In an attempt to mitigate the damage during, high flows, a feasibility-level design based on a preferred design alternative (soil cement bank protection) has been prepared to strengthen the levee embankment and minimize the damage due to scour and flooding. This basis of design report includes the analysis of the feasibility-level design, design assumptions and criteria, and planning-level cost estimates for planning purposes.

1.1 Description and Purpose

The SCR-1 system is located in the city of Oxnard and unincorporated areas of Ventura County, California. SCR-1 is approximately 4.72 miles long, extending along the southeast bank of the Santa Clara River from U.S. Highway 101 at its downstream terminus to the west end of South Mountain in Saticoy (Figures 1.1 and 1.2). SCR-1 is owned and operated by the Ventura County Watershed Protection District (VCWPD). The levee protects property to the south and east from flood inundation that would affect a variety of local land uses, including residential, agricultural, commercial, and industrial. The current Federal Emergency Management Agency (FEMA) floodplain associated with SCR-1 (FEMA 2010) is shown in Figure 1.3.

SCR-1 was originally designed in 1958 with the intent of controlling the U.S. Army Corp of Engineers' (Corps') calculated standard project flood discharge of 225,000 cfs emanating from the Santa Clara River watershed. The existing levee height varies from approximately 4 to 13 feet. The compacted fill embankment has a top width of 18 feet, and the embankment slopes are 2 feet horizontal to 1 foot vertical (2H:1V) on both the landward side and the riverward side of the levee. The riverward side of the embankment has a 1.5- to 2-foot-thick loose rock revetment, which was grouted with concrete in the vicinity of the highway bridges. The rock revetment extends from the top of the embankment to varying depths. The levee system also includes 75 rock groins along the levee toe.

1.2 History of Previous Inspections

During previous work, conducted as part of the FEMA Levee Certification Program, it was determined that SCR-1 does not currently meet the requirements set forth in Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10) of the National Flood Insurance Program regulations. As part of the FEMA levee certification work, a field investigation was performed, and it identified deficiencies in SCR-1 that require rehabilitation. The most recent Corps periodic inspection report, rated SCR-1 as "unacceptable" (USACE 2011). This unacceptable rating resulted in the placement of the levee system on "inactive" status in the Corps' Public Law 84-99 (PL 84-99) Program. Consequently, SCR-1 is currently ineligible for federal funding of repairs if damaged during a flood event.

1.3 Project Purpose and Scope of Work

This report was prepared in support of the SCR-1 reach evaluation and rehabilitation study to meet the feasibility requirements of the Corps. If the Corps takes the lead on this study, this document can be used in the future to complete the rehabilitation of the SCR-1 system.
Concurrent with this study, RBF Consulting (RBF) was contracted to develop technical studies and final design for the levee system just downstream of the SCR-1 reach, identified as Santa Clara River Levee (SCR-3). SCR-3 is also owned and operated by VCWPD and provides flood protection to residents and businesses in Oxnard It extends along the southeast bank of the Santa Clara River from U.S. Highway 101 to the to the eastern edge of the closed Ballard Landfill at its downstream terminus.

The purpose of this project is to develop a plan for addressing the deficiencies to reduce the flood risks to the community and to meet the levee criteria of both FEMA and the Corps. This report documents the technical studies and development of the feasibility-level design plans and cost estimates associated with the preferred design alternative (soil cement bank protection), which was selected through the previous selection/screening process, summarized in Sections 3.0 and 4.0.

1.4 Survey Mapping

The existing topographic mappings of the project area were previously provided by VCWPD as part of its effort to develop the FEMA Provisionally Accredited Levee (PAL) Response Report with Tetra Tech in 2009 (Tetra Tech 2009). The project topographic mapping used for the design was created by merging two separate mappings from the 2009 report: a 2005 light detection and ranging (LiDAR) survey of the Ventura County area and a 2009 ground survey of the SCR-1 system. The 2009 ground survey was based on a cross-sectional survey at approximately 50-foot intervals, which extended 20 feet from the existing levee toes on both the landward and the riverward side of the levee. The two mappings were merged using the Bentley MicroStation InRoads V8i software.

The horizontal control of the project topographic mapping is based on the California Coordinate System, Zone V Epoch 200.35, North American Datum of 1983 (NAD 83). The vertical control is based on the North American Vertical Datum of 1988 (NAVD 88). All units are in U.S. survey feet.

Figure 1.1 – Vicinity Map of Santa Clara River Levee (SCR-1)

Figure 1.2 – Location Map of Santa Clara River Levee (SCR-1)

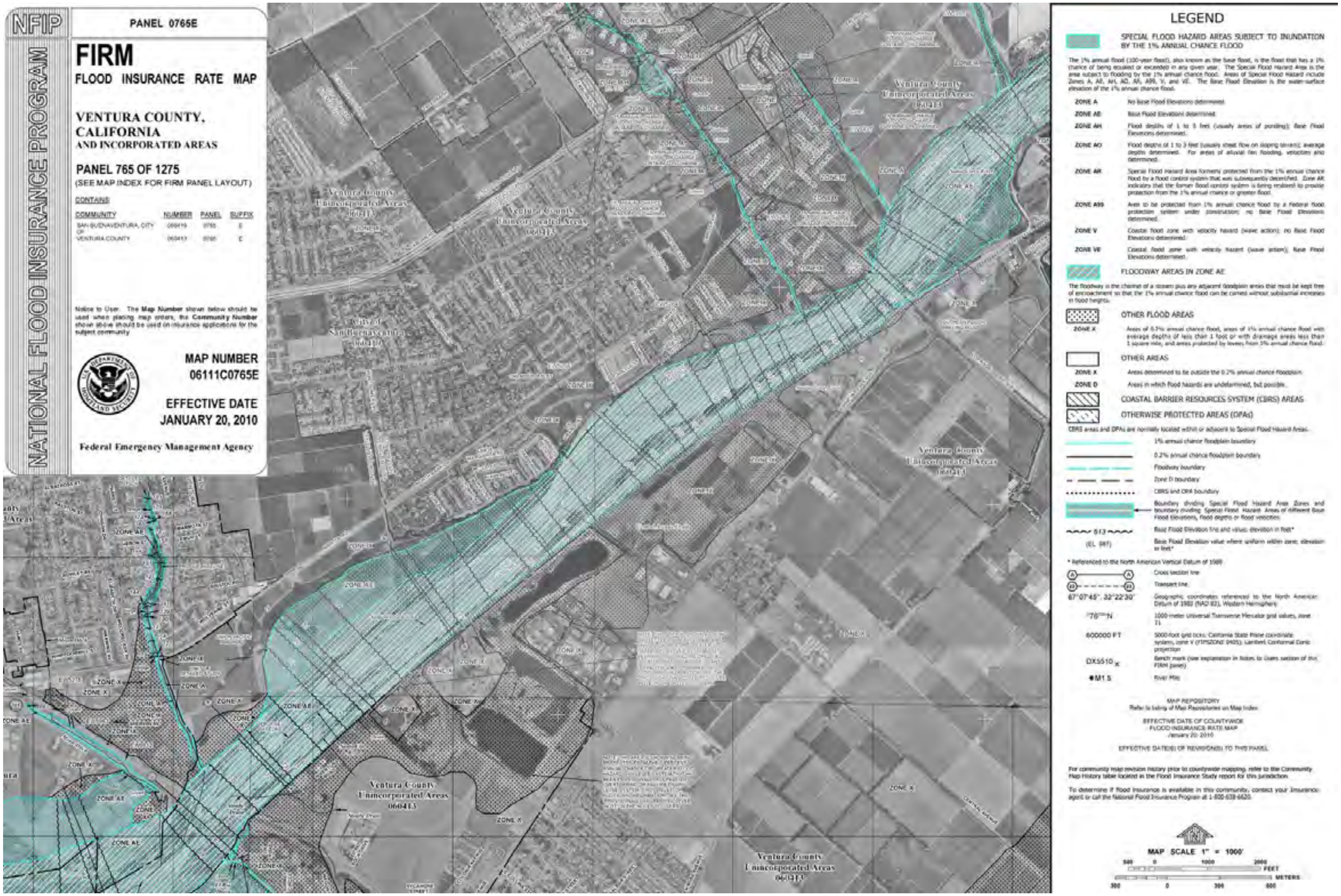

Figure 1.3 – Current FEMA Digital Flood Insurance Rate Map

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2.0 HYDROLOGY AND HYDRAULICS

This section explains the hydrology and hydraulic design criteria and assumptions. The detailed hydrology and hydraulic (H&H) analysis of SCR-1 is provided in Appendix I.

2.1 Hydrology

The U.S. Geological Survey (USGS) operated a gage on the Santa Clara River from October 1927 through September 2004. The gage was located at the Ventura Boulevard/Hwy 101 Bridge at Montalvo (Gage 11114000).

The most recent hydrologic study was conducted by AQUA TERRA Consultants in December 2009 using the Hydrologic Simulation Program—FORTRAN (HSPF) model. The results of a flood flow frequency analysis of four USGS) stream gages in Ventura County along the Santa Clara River were used to calibrate the 100-year HSPF design storm model (AQUA TERRA 2009). In June 2011, VCWPD, LACDPW, and the Corps prepared a report addendum to supplement the 2009 AQUA TERRA report.

The adopted discharge frequency values for the Santa Clara River at SCR-1 are shown in **Error! eference source not found.**2.1. The discharge frequency values were taken directly from Table 2.3 of the Santa Clara River Feasibility Study Report Addendum (VCWPD et al. 2011).

Table 2.1 – Adopted Discharge Frequency Values at Saticoy for Santa Clara River Levee

2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	25-Year (cfs)	50-Year (cfs)	100-Year (cfs)	200-Year (cfs)	500-Year (cfs)
9,784	32,544	59,212	109,384	160,686	226,000	303,970	441,152

cfs = cubic feet per second

For design purposes, VCWPD also requires the evaluation of a "design flow" considered to be 10 percent greater than the base level flow (100-year). The design flow to be used is 250,000 cfs.

2.2 Hydraulic Model

Channel hydraulics were computed using Hydrologic Engineering Center—River Analysis System (HEC-RAS), Version 4.1.0 (USACE 2010a) program, developed by the Corps for open-channel reaches. For the rehabilitation study of the SCR-1 system, the baseline hydraulic model was constructed by revising the most recent 2012 Corps hydraulic model, which includes the currently proposed levee improvements for SCR-1 and SCR-3 and the proposed improvements to the Olivas Park Drive extension downstream of Hwy 101.

2.3 Water Surface Elevation and Freeboard

To verify the adequacy of the existing FEMA/Corps freeboard along SCR-1 and determine the minimum design top of levee elevations necessary to meet the freeboard requirement, the 100-year flow (226,000 cfs), the design flow (250,000 cfs), and the 200-year flow (303,970 cfs) were used at various stages of design development in the hydraulic analyses to predict the water surface elevations. Per FEMA, 3 feet of freeboard would typically be required along SCR-1, whereas 4 feet are required 100 feet upstream and downstream of Hwy 118 and Hwy 101 bridges. An additional 0.5 foot of freeboard is required at the upstream end of the levee.

For new and existing levees being evaluated for the National Flood Insurance Program (NFIP) under Corps procedures, a risk and uncertainty (R&U) analysis is required, as described in Engineer Circular (EC) 1110-2-6067 (USACE 2010c). The R&U analysis was performed to verify that the existing top of levee elevations would provide a conditional non-exceedance probability (CNP) of at least 90 percent with a minimum of 3 feet of freeboard above the 100-year water surface profile.

The results of the hydraulic analysis indicated that if the existing top of levee elevation is higher than the calculated minimum top of levee elevation, the levee at that particular location would not need to be raised and would be left at its current elevation, even if it is higher than required. Where the existing top of levee elevation is lower than the required minimum top of levee elevation, based on the freeboard and CNP requirements, the levee would be raised to the minimum elevation. The calculations for the minimum top of levee elevations are included in the H&H report (Appendix I).

2.4 Sediment Transport

A three-level approach was applied along the SCR-1 reach to provide insight into potential future fluvial changes affecting SCR-1 over the long term and during single flood events. It included a Level 1 qualitative geomorphic assessment, a Level 2 general quantitative trend analyses, and a Level 3 numerical sediment-transport routing analysis.

The trends and potential magnitudes predicted by the model need to be considered during the design process and accounted for in the structure's maintenance plans. In general, the predicted aggradation/degradation results reasonably correspond with one another with the exception of historical measurements of the two downstream reaches, where aggradation occurred between 1993 and 2005. The calculations for the aggradation/degradation results are included in the H&H report (Appendix I).

2.5 Scour

Levee erosion protection must extend deep enough to cover the potential scour depth below the river invert. Generally, this additional depth of protection below the invert elevation is referred to as a "toedown" depth. The calculations for predicted scour are included in the H&H report (Appendix I).

3.0 SCREENING PROCESS FOR DESIGN ALTERNATIVE

This section describes the design alternatives resulting from the preliminary screening. Cross-sectional drawings of the preliminary screening design alternatives are provided in Appendix II.

3.1 Design Assumptions

For the preliminary design phase, the existing levee alignment was used, from Hwy 101 to Saticoy. The water surface elevations from the design flow along with the FEMA freeboard requirements were used in determining the required height of the levee. A scour depth of 15 feet below the river invert was assumed. Four cross sections along the length of the levee showing the respective alternative designs were used to represent typical levee conditions. For each design alternative, an excavation/backfill slope of 1.5H:1V was assumed for placement of the toedown protection. It was assumed that any removed material, including existing riprap revetment and launchable stone, would be reused as backfill material if it meets the gradation requirement.

3.2 Design Calculations

Engineer Manual (EM) 1110-2-1601 (USACE 1994) was consulted in calculating the required length and height of launched toe sections and the riprap size.

3.3 Preliminary Design Alternatives

Twelve preliminary design alternatives resulted from the preliminary screening.

3.3.1 Alternative A1

The design of Alternative A1 consists of a soil cement revetment with a slope of 2H:1V from the top of levee down to the toedown depth (Figure 3.1). The 2H:1V soil cement slope would allow the majority of the existing riprap to remain in place.

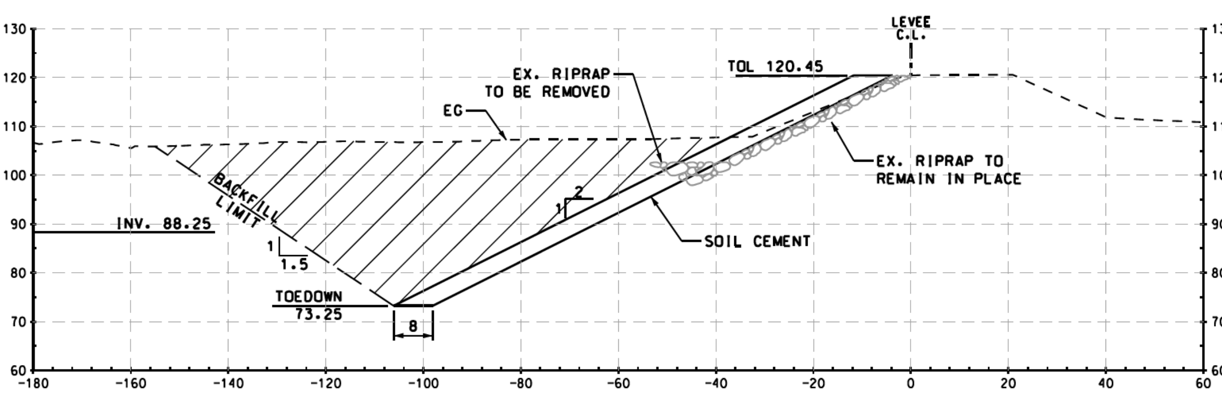

Figure 3.1 – Typical Cross Section of Alternative A1 (Sta. 368+00)

3.3.2 Alternative A2

The design of Alternative A2 consists a soil cement revetment with a slope of 2H:1V from the top of levee to the invert elevation and a slope of 1H:1V from the invert elevation to the toedown depth (Figure 3.2). The 2H:1V slope to the invert elevation would allow the existing riprap with a slope of 2H:1V to remain in place. The 1H:1V slope below the invert would conserve the footprint of the affected area and save on the cost for mitigation.

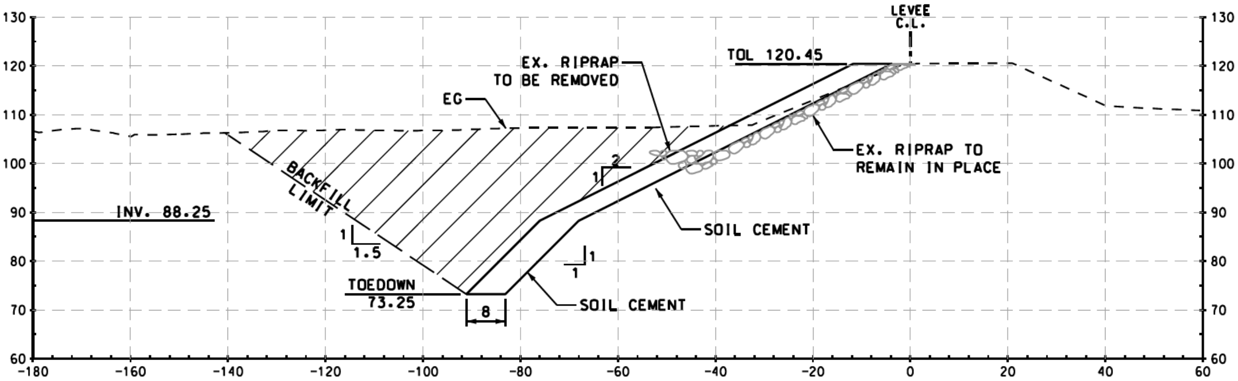

Figure 3.2 – Typical Cross Section of Alternative A2 (Sta. 368+00)

3.3.3 Alternative A3

The design of Alternative A3 consists of a soil cement revetment with a slope of 1H:1V from the top of levee to the toedown depth (Table 3.3). All the existing riprap including the launch toe would be removed prior to the placement of soil cement. The overall affected area would be smaller in comparison to that of Alternatives A1 and A2 due to the continuous slope of 1H:1V along the height of the protected levee.

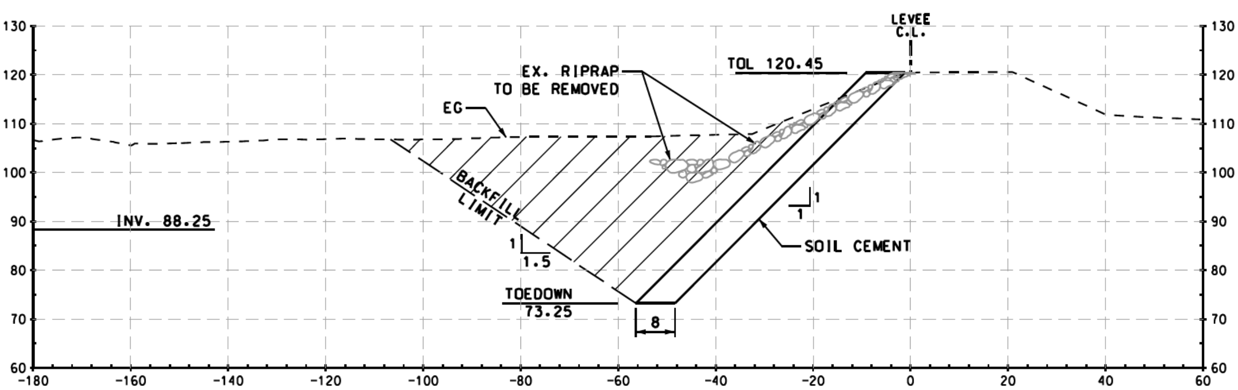

Figure 3.3 – Typical Cross Section of Alternative A3 (Sta. 368+00)

3.3.4 Alternative A4

The design of Alternative A4 consists of a soil cement revetment with a 1H:1V slope from the top of levee to approximately 5 feet above the invert elevation (Figure 3.4). A sheet pile wall would be installed to protect the levee from the remaining potential scour depth. The sheet pile length was assumed to be three times the potential length of exposed sheet pile. For the majority of the levee, this depth would equal an overall sheet pile length of 60 feet. Due to the high cost for sheet piles and mitigation, the sheet pile would be placed 5 feet above the invert to effectively manage the cost of both items.

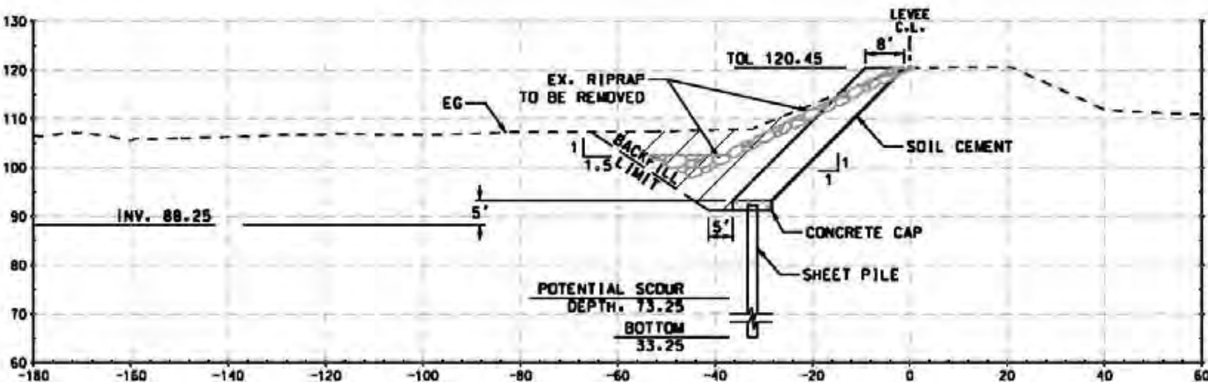

Figure 3.4 – Typical Cross Section of Alternative A4 (Sta. 368+00)

3.3.5 Alternative B1

The design of Alternative B1 consists of sheet pile at the top of levee that extends down to the required sheet pile depth (Figure 3.5). As in Alternative A4, the sheet pile length was assumed to be three times the potential length of exposed sheet pile. This length of the sheet pile would require tie-backs and likely king-piles to withstand the lateral loading. Although this alternative has minimal cost for mitigation, the overall cost for construction would escalate quickly because of the extensive length of sheet pile.

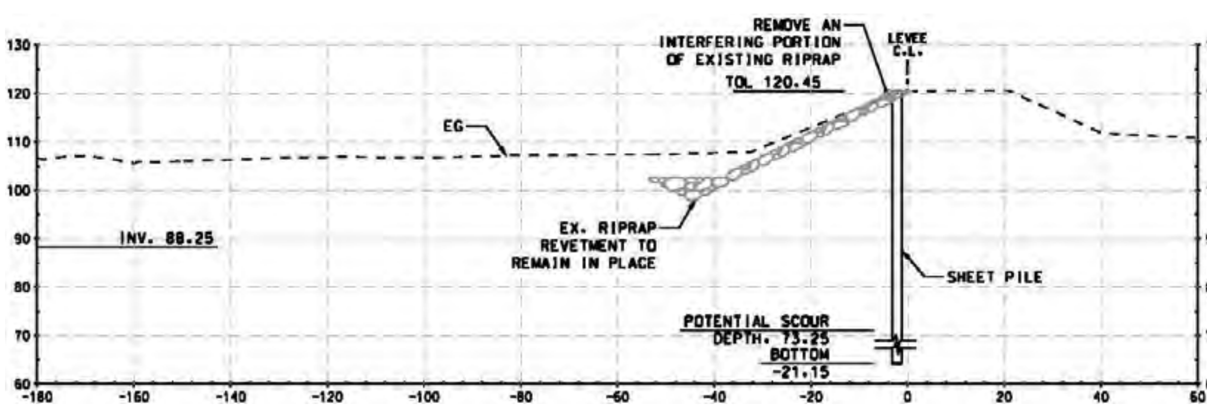

Figure 3.5 – Typical Cross Section of Alternative B1 (Sta. 368+00)

3.3.6 Alternative B2

The design of Alternative B2 is similar to that of Alternative A4, with the sheet pile installed approximately 5 feet above the invert (Figure 3.6). The total length of the sheet pile is three times the potential length of exposed sheet pile. A 42-inch-thick riprap revetment at a 2H:1V slope would provide protection from the top of levee to the sheet pile. Alternative B2 attempts to minimize the affected area and the length of sheet pile.

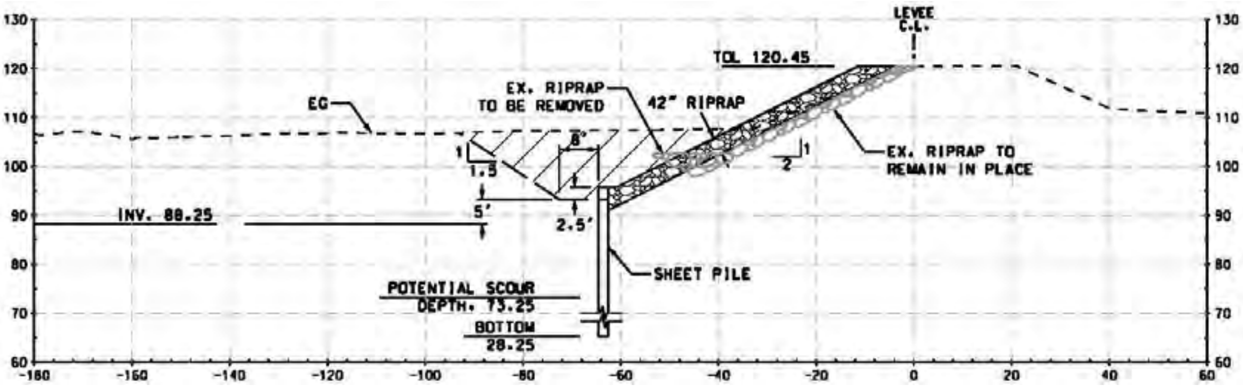

Figure 3.6 – Typical Cross Section of Alternative B2 (Sta. 368+00)

3.3.7 Alternative C

The design of Alternative C consists of a 42-inch-thick layer of ½-ton rock riprap from the top of levee to the toedown depth at a slope of 2H:1V (Figure 3.7). A small portion of the existing launch toe would be removed.

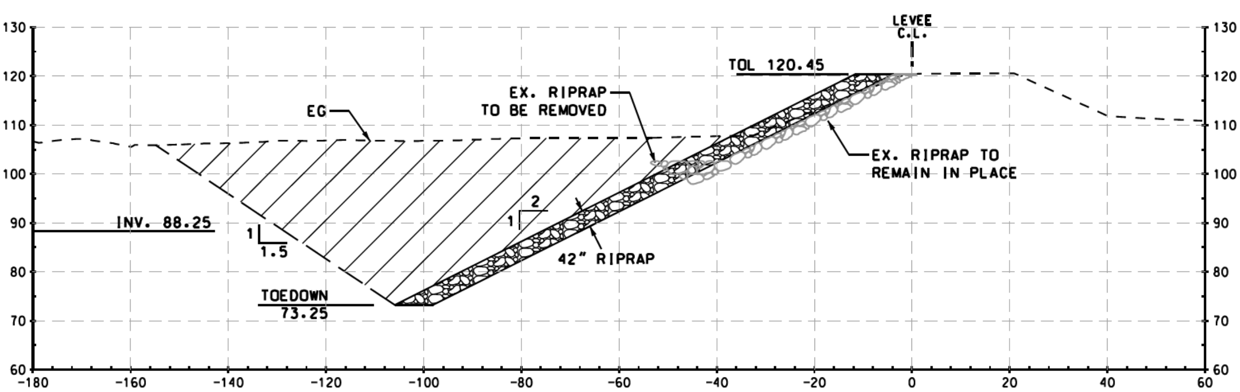

Figure 3.7 – Typical Cross Section of Alternative C (Sta. 368+00)

3.3.8 Alternative D1

The design of Alternative D1 would involve placing a 42-inch-thick layer of ½-ton rock riprap at a slope of 2H:1V from the top of levee to the toedown depth and constructing a new weighted toe (launchable stone) at the channel invert (Figure 3.8). The required dimensions to protect against 15 feet of scour are 9 feet in height and 20 feet in length (USACE 1994). The height of the launchable stone above the potential scour depth may be too much for adequate coverage.

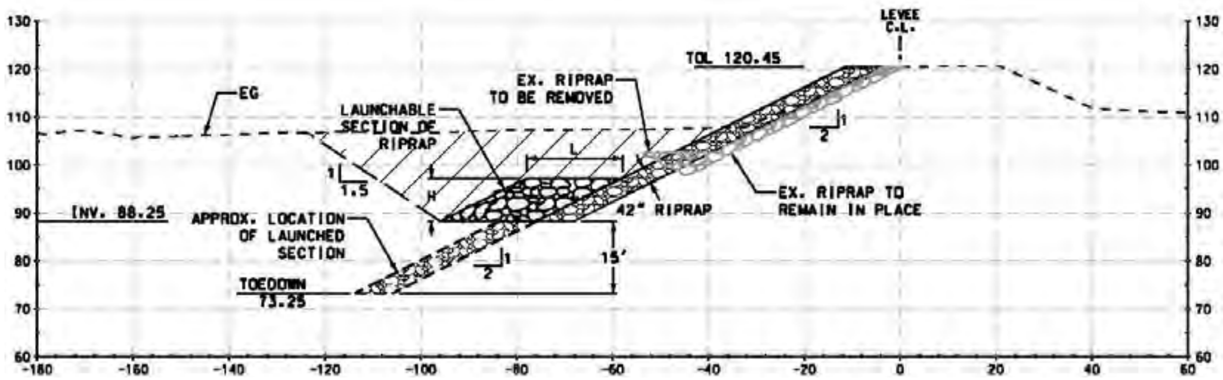

Figure 3.8 – Typical Cross Section of Alternative D1 (Sta. 368+00)

3.3.9 Alternative D2

The design of Alternative D2 would involve placing a 42-inch-thick layer of ½-ton rock riprap at a slope of 2H:1V from the top of levee to the toedown depth and constructing a new weighted toe (launchable stone) no more than 10 feet from the existing ground surface (Figure 3.9). The resulting required dimensions to protect against 15 feet of scour are 9 feet in height and 20 to 43 feet in length (USACE 1994). The height of the launchable stone above the potential scour depth is excessive.

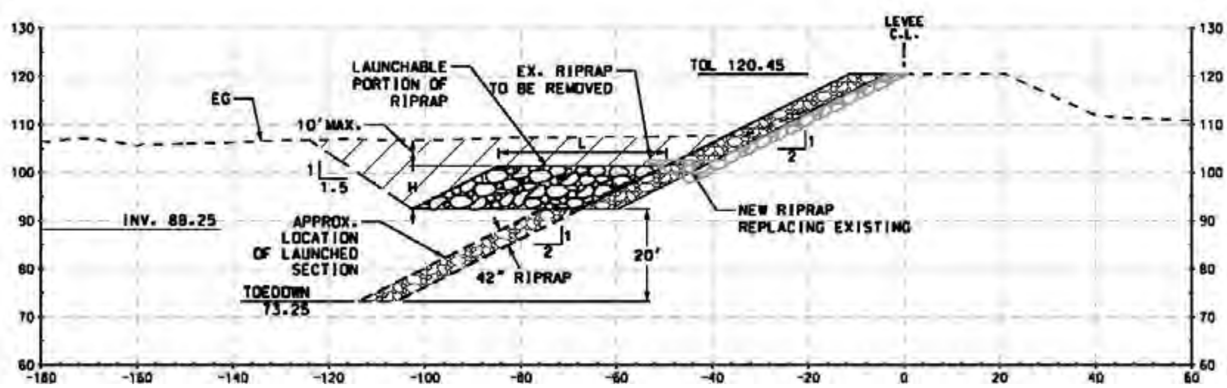

Figure 3.9 – Typical Cross Section of Alternative D2 (Sta. 368+00)

3.3.10 **Alternative E**

The design of Alternative E would involve overlaying the existing groins with a 42-inch-thick layer of ½-ton rock riprap and extending the groin tips to the potential scour depth (Figures 3.10 and 3.11). The groin extensions would be approximately 10 feet thick and have varying lengths depending on their location along the levee. The amount of excavation required to construct the groin extensions would be significant, and the excavation would likely consist of one continuous area along the levee because the existing groins are so close together. The excavation limit would result in significant impacts and mitigation costs.

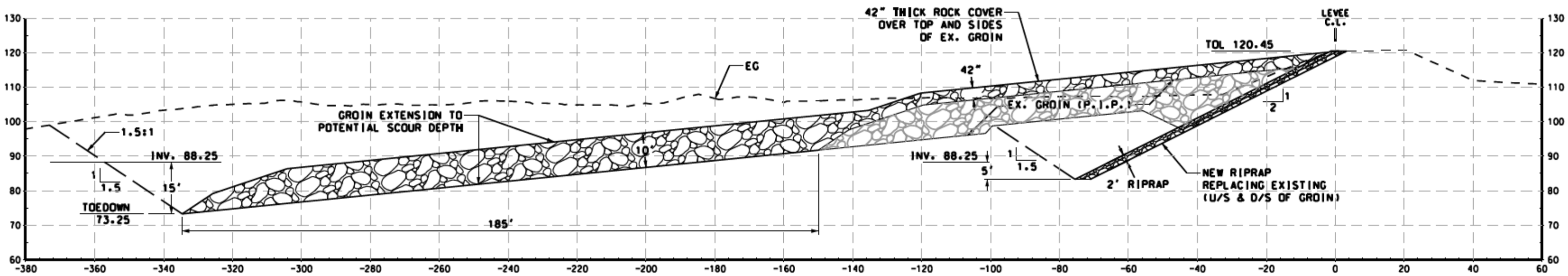

Figure 3.10 – Typical Cross Section of Alternative E (Sta. 368+00)

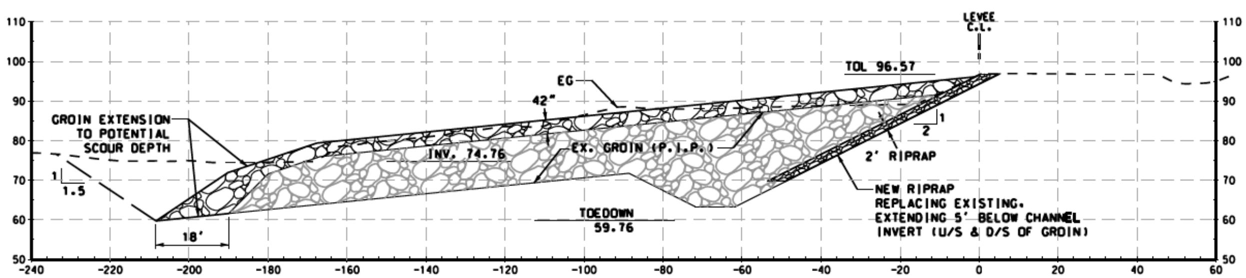

Figure 3.11 – Typical Cross Section of Alternative E (Sta. 312+00)

3.3.11 Alternative F1

The design of Alternative F1 would involve placing a 30-inch-thick layer of grouted ¼-ton rock riprap with a slope of 2H:1V from the top of levee to the toedown depth (Figure 3.12). The 2H:1V slope would allow the existing riprap with a slope of 2H:1V to remain in place.

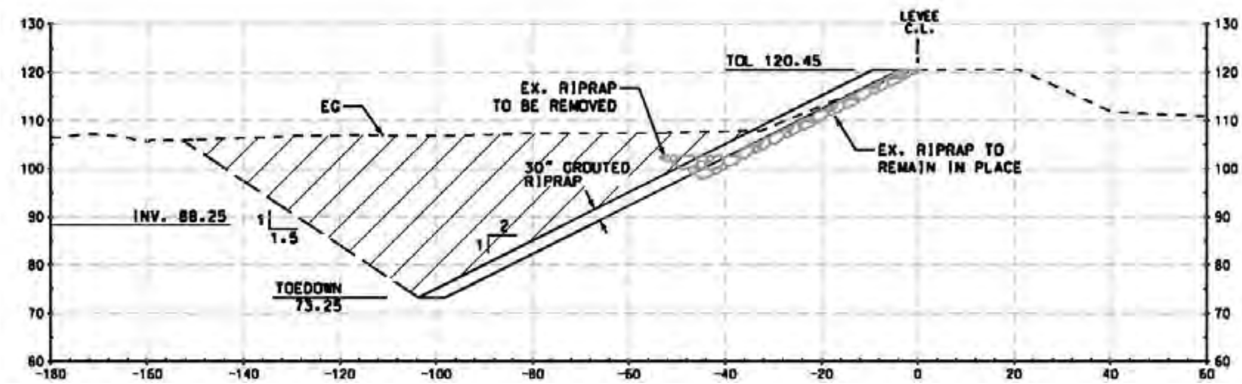

Figure 3.12 – Typical Cross Section of Alternative F1 (Sta. 368+00)

3.3.12 Alternative F2

The design of Alternative F2 would involve placing a 30-inch-thick layer of grouted ¼-ton rock riprap with a slope of 2H:1V from the top of levee to the riverside toe and a slope of 1.5H:1V from the riverside toe elevation to the toedown depth (Figure 3.13). The 2H:1V slope would allow the existing riprap with a slope of 2H:1V to remain in place. The 1.5H:1V slope below the invert would conserve the footprint of the affected area and save on the cost for mitigation.

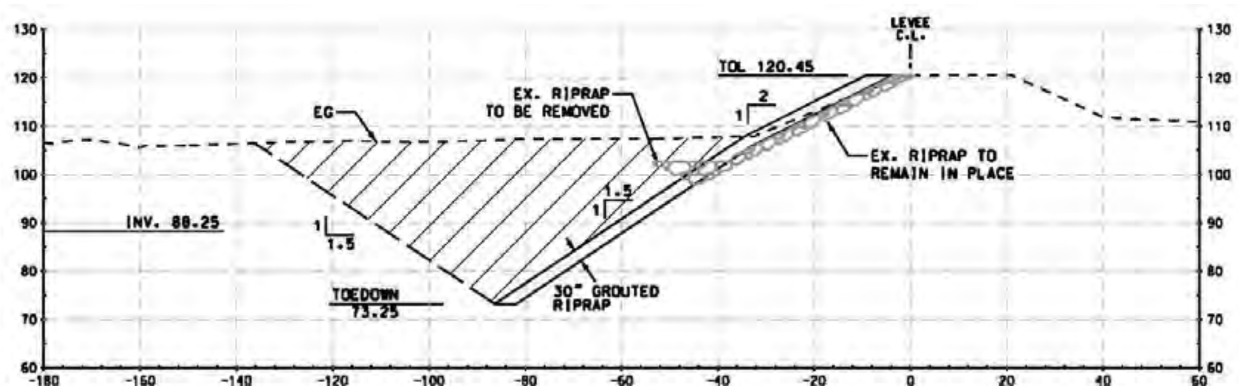

Figure 3.13 – Typical Cross Section of Alternative F2 (Sta. 368+00)

3.4 Summary of Costs

Cost estimates were prepared for the 12 preliminary design alternatives for comparison purposes only (Table 3.1). The cost estimates are based on quantities calculated from the typical cross sections and unit costs from recent and similar local projects. The affected area was measured from the control line along the levee to the limit of excavation. The mitigation area requirements for affected area are approximately 5:1 and 3:1 for permanent and temporary impacts, respectively. The unit cost for mitigation was assumed to be $150,000 per acre.

The unit costs, except for mitigation, were marked up by 30 percent to account for uncertainties; however, for comparison purposes other costs such as those for mobilization; planning, engineering, and design; and construction management were not included because they would be similar for each alternative. The costs presented in this section are not intended for budgetary purposes. Estimates of preliminary screening design costs for all the alternatives are provided in Appendix III.

Table 3.1 – Summary of Costs for the Preliminary Screening Design Alternatives

Alternative	Cost Excluding Mitigation	Total Cost (Including Mitigation)
Alternative A1	$48,254,300	$99,095,200
Alternative A2	$43,251,700	$89,272,500
Alternative A3	$42,026,900	$77,607,100
Alternative A4	$131,653,200	$151,760,500
Alternative B1	$362,063,100	$362,063,100
Alternative B2	$149,632,100	$175,300,300
Alternative C	$70,795,700	$118,887,000
Alternative D1	$78,457,600	$115,523,600
Alternative D2	$89,744,500	$122,909,300
Alternative E	$104,101,700	$212,596,300
Alternative F1	$79,118,500	$129,320,300
Alternative F2	$64,730,300	$108,621,100

4.0 CONCEPTUAL-LEVEL DESIGN ALTERNATIVES

From the 12 preliminary screening design alternatives, 3 alternatives were selected on the basis of engineering feasibility, environmental impact, and overall construction cost:

- Alternative 1 – soil cement revetment with a slope of 1H:1V from the top of levee to the toedown depth (formerly Alternative A3);
- Alternative 2 – loose riprap 42 inches thick with a slope of 2H:1V from the top of levee to the toedown depth (formerly Alternative C);
- Alternative 3 – grouted riprap 30 inches thick with a slope of 2H:1V from the top of levee to the riverside toe and a slope of 1.5H:1V from the riverside toe elevation to the toedown depth (formerly Alternative F2).

Conceptual-level design drawings of the three alternatives are provided in Appendix IV.

4.1 Design Assumptions

For the conceptual-level design phase, the existing levee alignment was used, from Hwy 101 to Saticoy. The water surface elevations from the design flow and the 200-year flow along with the FEMA freeboard requirements were used in determining the required height of the levee. A scour depth of 15 feet below the river invert was assumed. Five cross sections along the length of the levee showing the respective alternative designs were used to represent typical levee conditions. An excavation/backfill slope of 1.5H:1V was assumed for placement of the toedown protection for each design alternative. Where the levee needs to be raised, the landward side toe of slope was held in its current location and extended up at a slope of 2H:1V.

Additionally, the following components were incorporated into the conceptual-level design alternatives:

- A cable guard fence along 1H:1V slopes
- Access ramps and turnouts per the minimum requirements of the Corps or the County of Ventura
- Hard surface on top of levee:
    - Asphalt over extended soil cement for Alternative 1
    - Asphalt pavement over crushed miscellaneous base (CMB) pavement for Alternatives 2 and 3

4.2 Design Calculations

EM 1110-2-1601 was consulted in calculating the required riprap size.

4.3 Alternative 1 – Soil Cement

Alternative 1 consists of soil cement bank protection with a side slope of 1(H):1(V). Typical cross sections at Levee Station 270+00 for the design flow and the 200-year flow are provided in Figures 4.1 and 4.2, respectively.

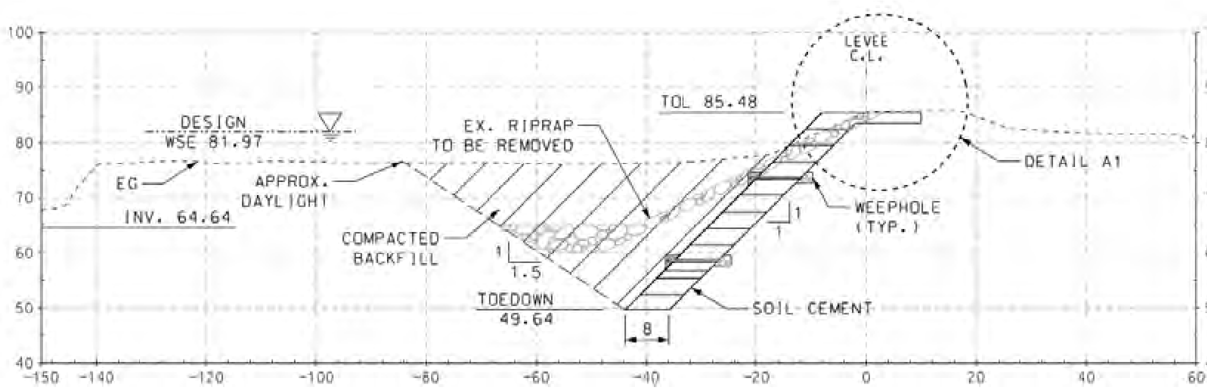

Figure 4.1 – Typical Cross Section of Alternative 1, Soil Cement for Design Flow (Sta. 270+00)

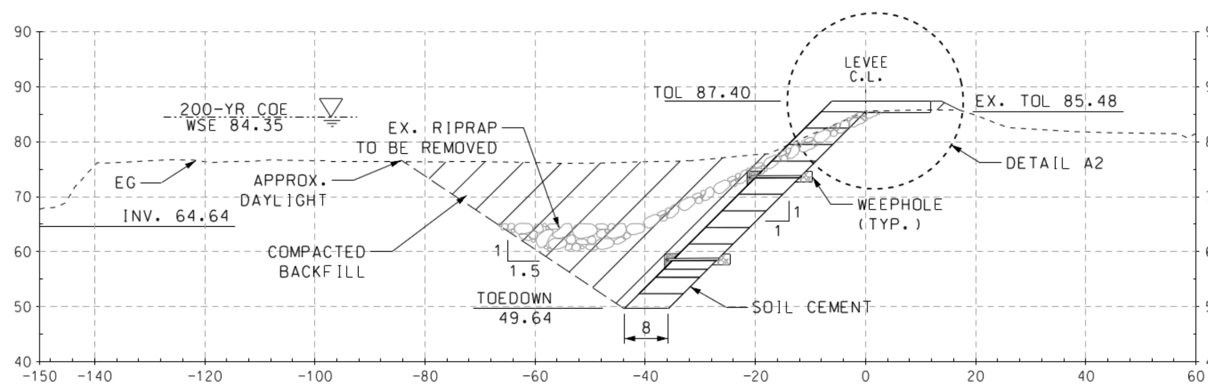

Figure 4.2 – Typical Cross Section of Alternative 1, Soil Cement for 200-Year Flow (Sta. 270+00)

4.4 Alternative 2 – Loose Riprap

Alternative 2 consists of a 42-inch-thick layer of loose ½-ton rock riprap with a slope of 2H:1V from the top of levee to the toedown depth. Typical cross sections at Levee Station 270+00 for the design flow and the 200-year flow are provided in Figures 4.3 and 4.4, respectively.

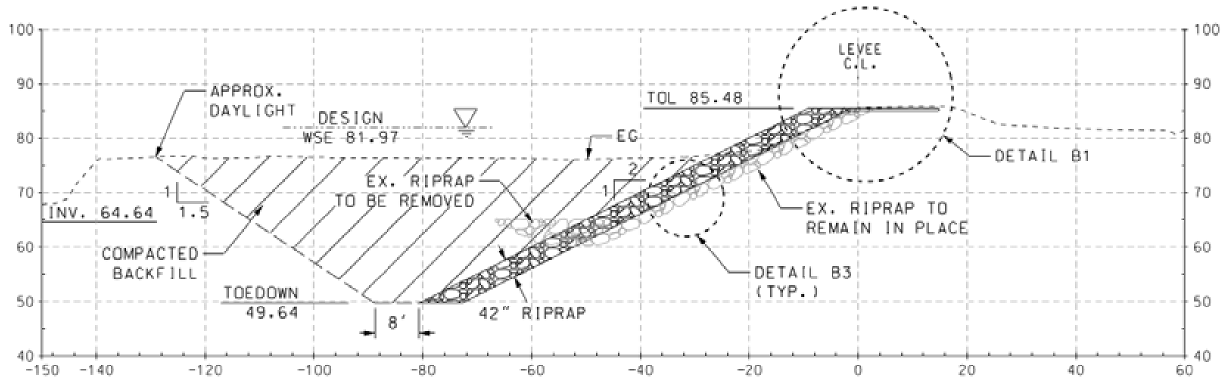

Figure 4.3 – Typical Cross Section of Alternative 2, Loose Riprap for Design Flow (Sta. 270+00)

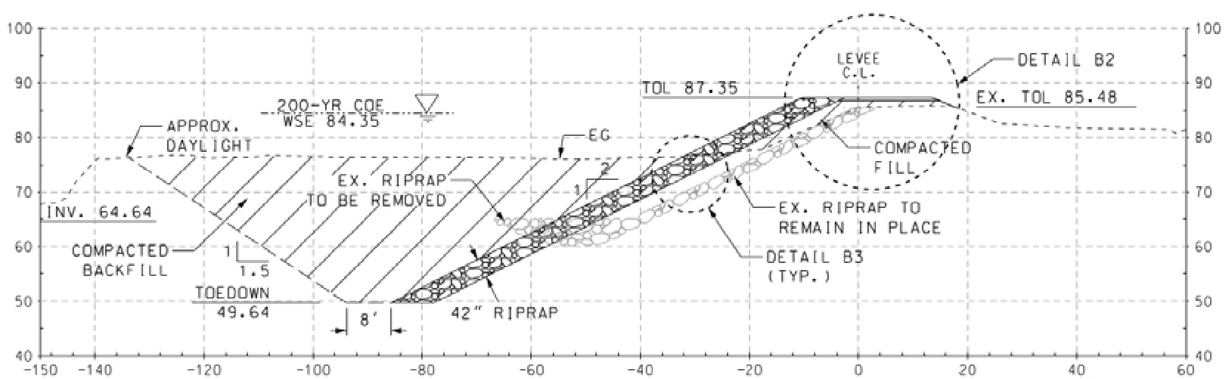

Figure 4.4 – Typical Cross Section of Alternative 2, Loose Riprap for 200-Year Flow (Sta. 270+00)

4.5 Alternative 3 – Grouted Riprap

Alternative 3 consists of a 30-inch-thick layer of grouted ¼-ton rock riprap with a slope of 2H:1V from the top of levee to the riverside toe and a slope of 1.5H:1V from the riverside toe elevation to the toedown depth. Typical cross sections at Levee Station 270+00 for the design flow and the 200-year flow are provided in Figures 4.5 and 4.6, respectively.

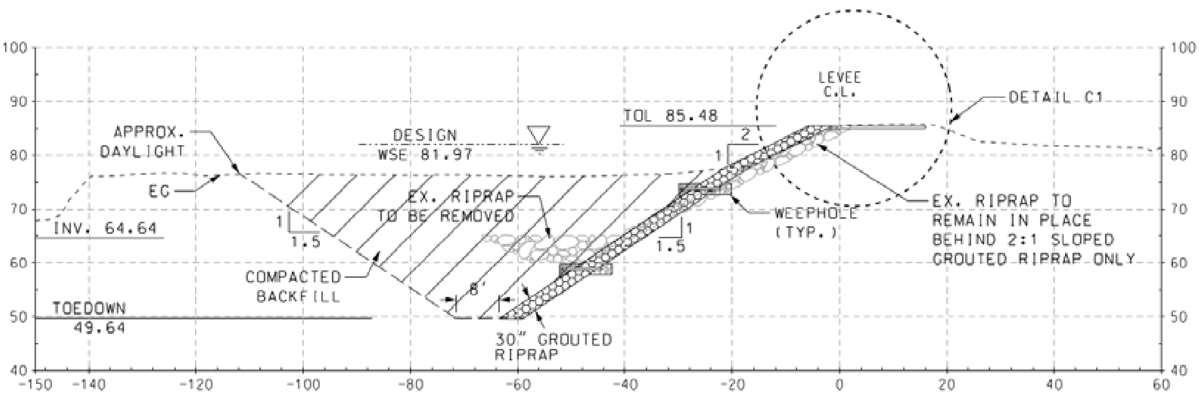

Figure 4.5 – Typical Cross Section of Alternative 3, Grouted Riprap for Design Flow (Sta. 270+00)

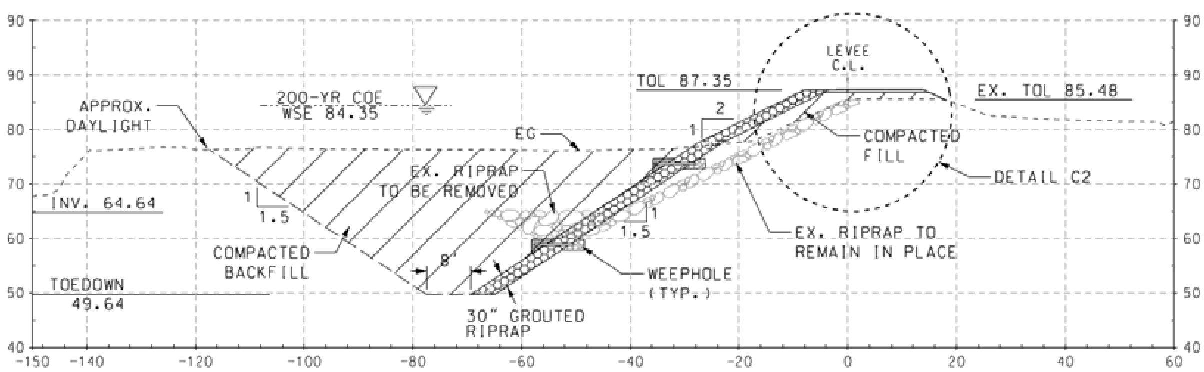

Figure 4.6 – Typical Cross Section of Alternative 3, Grouted Riprap for 200-Year Flow (Sta. 270+00)

4.6 Common Design Features

Design features that are common to the three conceptual-level alternatives are a levee/floodwall under Hwy 101, access ramps along the levee, and storm drain improvements.

4.6.1 Levee/Floodwall under Highway 101

The extension of the levee under Hwy 101 would be the best solution to allow continuity of the SCR-1 and SCR-3 levees. Finalization of the hydraulic analysis and field survey data to calculate the actual bridge soffit will determine whether a floodwall would be able to provide this continuity. The information shown on Figure 4.7 is subject to change based on the final analysis and field data.

In addition, the levee embankment would require erosion protection against abutment and pier scour. Construction access under the bridge would be difficult and require close coordination with the California Department of Transportation (Caltrans). The erosion protection is assumed to consist of grouted riprap.

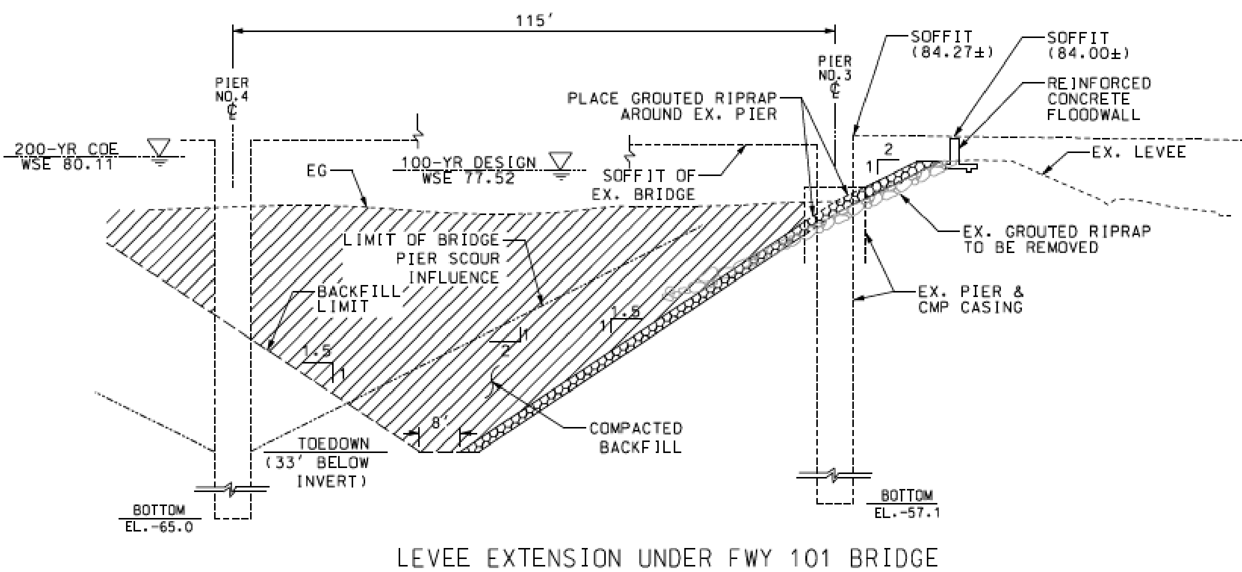

Figure 4.7 – Typical Cross Section of Levee/Floodwall under Hwy 101 Bridge

4.6.2 Access Ramps

Access ramps would be constructed along the levee at the current locations of major ramps to meet the minimum requirements of EM 111-2-1913 (USACE 1996). The access ramps on the riverside of the levee would have a minimum of 16-foot-wide reinforced concrete pavement and a maximum slope of 10 percent (Figure 4.8). The access ramps on the landward side of the levee would have a minimum of 16-foot-wide CMB pavement and a maximum slope of 8 percent.

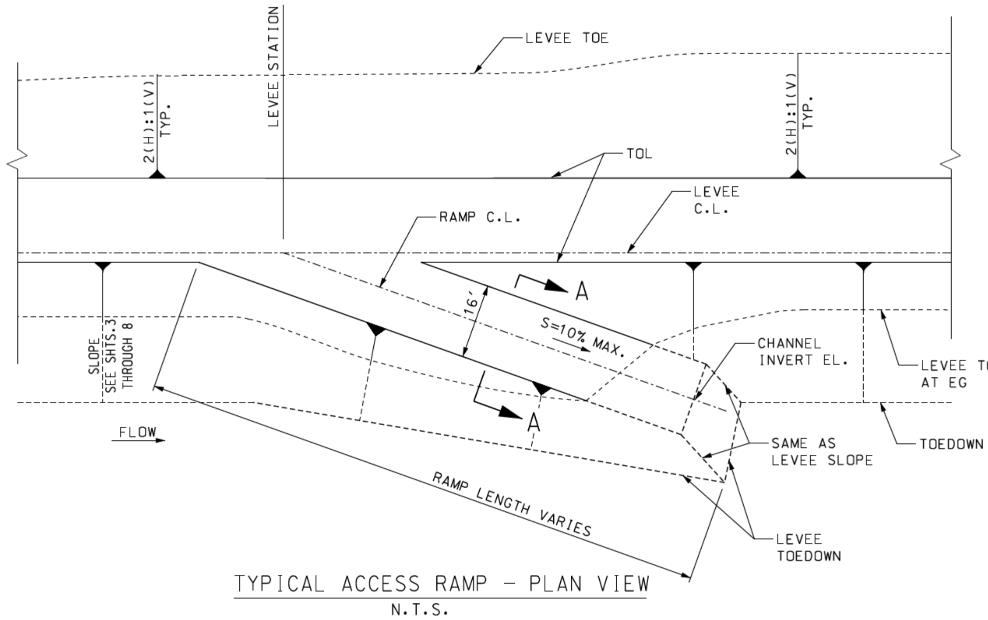

Figure 4.8 – Plan View of Access Ramp

4.6.3 Storm Drain Improvements

The storm drain penetrations through the levee would require improvements. Most of the penetrations have a gatewell with flap gates on the landward side of the levee. It is assumed that these facilities would be removed and replaced with a headwall structure. Because the levee improvements would move the face of the levee, most of the storm drain penetrations would need to be shortened or extended. New outlet headwall structures and flap gates would be constructed on the riverside of the levee. The known penetrations are indicated in Table 4.1.

Table 4.1 – Summary of Existing Levee Penetrations

Levee Station	Description
480+00	Side Drain No. 1, 42-inch-diameter reinforced concrete pipe and flap gate (on landward side)
442+00	Side Drain No. 2, 48-inch-diameter reinforced concrete pipe and flap gate (on landward side) just upstream of Los Angeles Ave.
422+25	Commercial drain from asphalt plant (not found during the December 9, 2008, field inspection)
410+60	Side Drain No. 3, 48-inch-dimater reinforced concrete pipe and flap gate (on landward side)
385+77	12-inch-diameter metal pipe commercial drain from process plant
351+50	Central Avenue Drain, two 72-inch-diameter reinforced concrete pipes with flap gates (on riverward side)
316+60	Side Drain No. 4, 48-inch-diameter reinforced concrete pipe and flap gate (on landward side)
282+00	Side Drain No. 6, 48-inch-diameter reinforced concrete pipe and flap gate (on landward side)
246+20	Stroube Drain – Unit I, 10-foot-wide by 9-foot-high reinforced concrete box with sluice gate

4.7 Summary of Costs

Cost estimates were prepared for the three conceptual-level design alternatives for comparison purposes only (Table 4.2). The cost estimates are based on quantities calculated from computer-aided design and drafting (CADD) and engineering calculations, and unit costs from recent and similar local projects. The affected area was measured from the control line along the levee to the limit of excavation. The mitigation area requirements for the affected area are approximately 5:1 and 3:1 for permanent and temporary impacts, respectively. The unit cost for mitigation was assumed to be $150,000 per acre.

The costs for the conceptual-level design alternatives include mobilization; clearing and grubbing; levee slope protection; improvements under Hwy 101; access ramps; storm drain penetration improvements; planning, engineering, and design; construction management; and contingencies. Cost estimates for the three conceptual-level design alternatives are provided in Appendix V.

Table 4.2 – Summary of Costs for the Conceptual-Level Design Alternatives

Alternative	Design Option	Cost Excluding Mitigation	Total Cost
Alternative 1	Design flow	$66,741,400	$98,841,400
Alternative 2	Design flow	$104,935,600	$151,735,600
Alternative 3	Design flow	$97,319,900	$138,269,900
Alternative 1	Corps 200-year flow	$68,794,200	$101,644,200
Alternative 2	Corps 200-year flow	$109,060,200	$156,910,200
Alternative 3	Corps 200-year flow	$105,431,300	$147,285,000

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5.0 DEVELOPMENT OF FEASIBILITY-LEVEL DESIGN

From the three conceptual-level design alternatives presented in Section 4.0, VCWPD has selected Alternative 1, Soil Cement, as its preferred design alternative for improving SCR-1. The feasibility-level levee improvement measures for remediating deficiencies and meeting the levee criteria for both FEMA and the Corps are discussed in the following subsections. The feasibility-level design drawings are included in Appendix VI.

5.1 Realignment of SCR-1

The existing alignment of the SCR-1 system follows the current earthen embankment along the easterly edge of the river from Hwy 101 to Saticoy, as shown in Figures 1.2 and 1.3. For the feasibility-level design, a portion of SCR-1 upstream of approximately Station 350+00 (near the levee penetration for the Central Avenue Drain) was evaluated for realignment. As shown in the construction cost estimates in the preliminary screening design analysis (Table 3.1) and the conceptual-level design analysis (Table 4.2), improving and rehabilitating the current SCR-1 system in its entirety would likely result in significant construction and mitigation costs. However, the existing properties that are likely to be protected by the upper portion of SCR-1 (upstream of Station 350+00) consist of mostly farm land, spreading grounds, and quarries, and the benefit of protecting these properties may be outweighed by the improvement costs.

Additionally, the upper portion of SCR-1 is entrenched, and the 100-year flow water surface elevations along this portion of the levee are lower than those at the toe of the levee on the landward side. Therefore in this upper reach, the existing levee embankment is no longer required to provide flow containment and freeboard during the 100-year flow condition.

Based on coordination with VCWPD, it was decided that the upstream limit of the SCR-1 system would be revised to terminate near the Central Avenue Drain penetration (Levee Station 350+49). The levee would then extend farther east along the northerly edge of the existing Central Avenue Drain system to tie into high ground near Vineyard Avenue (Figure 5.1). In the figure, the realigned levee follows the “Levee Rehabilitation along Existing Alignment” from Hwy 101 to Levee Station 350+70 and then turns eastward along the “New Tie-in Levee Alignment” from Levee Station 500+00 to Levee Station 540+00. The alignment of the “Existing Levee Maintained in Place” would no longer be considered part of the SCR-1 system but would be left in place.

It should be noted that the potential revision and realignment of the SCR-1 system still needs to undergo the FEMA review process for an official approval.

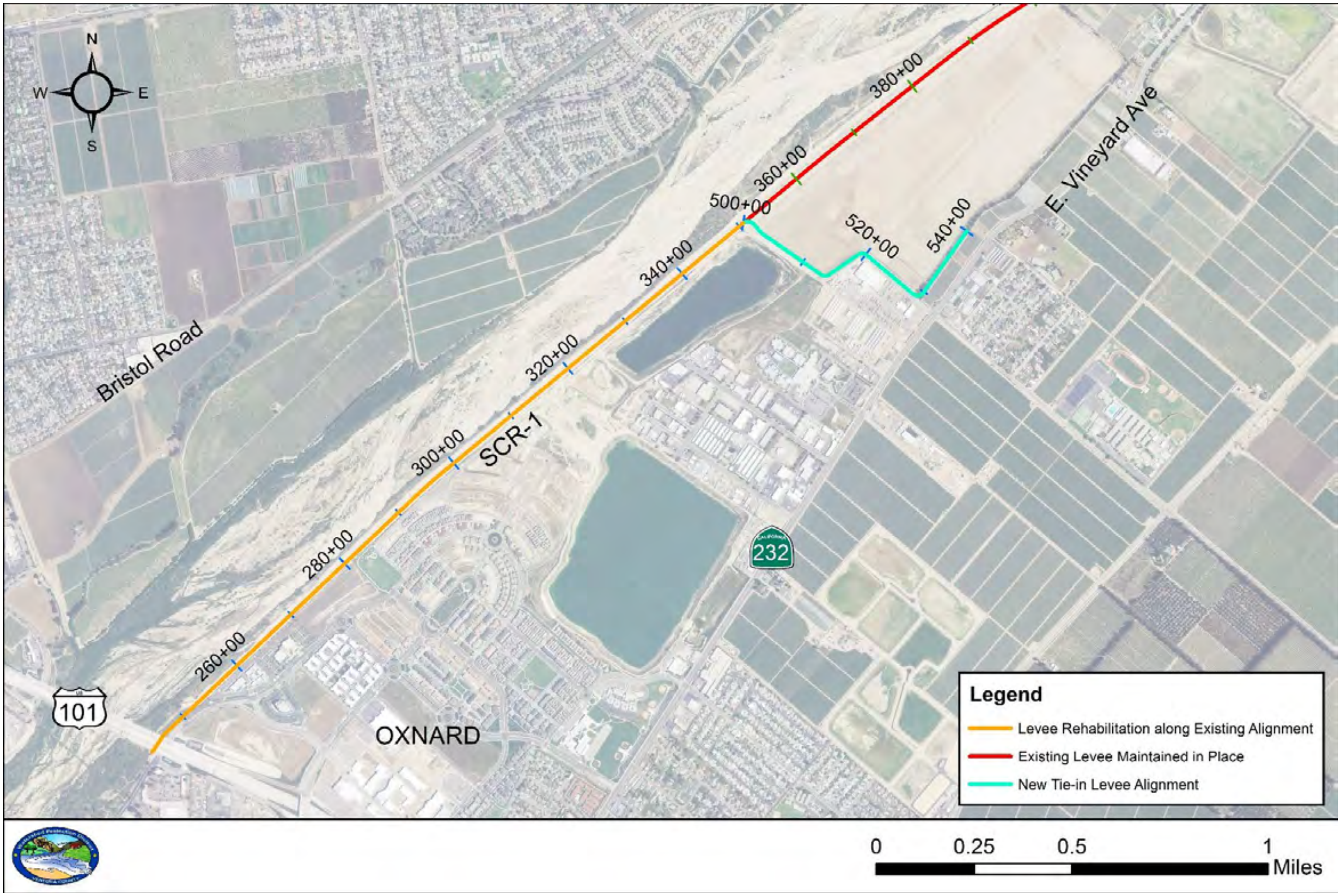

Figure 5.1 – Comparison of Existing Levee Alignment and Realigned Levee Alignment along SCR-1

5.2 Hydrology and Hydraulics

This section explains the hydraulic design criteria and assumptions for the preferred alternative, which has been developed to a feasibility-level design. A detailed hydrology and hydraulic (H&H) analysis of SCR-1 is provided in Appendix I.

5.2.1 Water Surface Elevation and Freeboard

To verify the adequacy of the existing FEMA freeboard along SCR-1 and determine the minimum design top of levee elevations to meet the freeboard requirement, the 100-year flow (226,000 cfs) and design flow (250,000 cfs) were used in the hydraulic analysis to predict the water surface elevations. Per FEMA, 3 feet of freeboard would typically be required along SCR-1, whereas 4 feet are required 100 feet upstream and downstream of Hwy 118 and Hwy 101 bridges. An additional 0.5 foot of freeboard is required at the upstream end of the levee.

In using the 100-year flow to determine the adequacy of the existing freeboard per the Corps requirements, an R&U analysis was performed to verify that the existing top of levee elevations would provide a CNP of at least 90 percent in addition to meeting the requirement of 3-foot minimum freeboard height above the 100-year water surface profile. The R&U analysis is required by the Corps for all new and existing levees, as described in EC 1110-2-6067 (USACE 2010). For consistency with the design requirements of SCR-3 immediately downstream of SCR-1, which are being prepared by others, the freeboard condition for the design flow water surface elevation profile was also analyzed. In the design flow analysis, FEMA freeboard requirements were used. The water surface elevations and freeboard conditions for various flood events are summarized in Table 5.1.

Table 5.1 – Summary of Existing Freeboard for Various Flow Conditions

HEC-RAS River Station	Approx. Levee Station	Existing Top of Levee Elevation (feet)	100-Year WSE (feet)	Existing Freeboard (feet)	Design Flow WSE (feet)	Existing Freeboard (feet)	FEMA-Required Freeboard (feet)
250+62	246+49	79.40	75.70	3.70[1]	76.98	4.28	4
257+50	252+54	82.39	76.80	5.59	78.00	4.39	3
263+56	257+94	82.19	77.70	4.49[1]	78.87	3.32	3
269+30	263+72	84.14	79.11	5.03	80.18	3.96	3
275+00	269+50	85.50	80.05	5.45	81.06	4.44	3
282+20	276+41	87.00	82.04	4.96	82.99	4.01	3
289+32	283+36	87.90	83.91	3.99[1]	84.77	3.13	3
296+50	290+27	89.95	85.77	4.18[1]	86.59	3.36	3
303+52	296+08	91.75	87.60	4.15[1]	88.41	3.34	3
309+00	301+43	93.55	88.51	5.04	89.29	4.26	3
324+80	316+11	98.18	92.71	5.47	93.50	4.68	3
342+20	334+19	106.12	97.55	8.57	98.39	7.73	3
359+30	349+98	111.10	102.76	8.34	103.65	7.45	3

HEC-RAS River Station	Approx. Levee Station	Existing Top of Levee Elevation (feet)	100-Year WSE (feet)	Existing Freeboard (feet)	Design Flow WSE (feet)	Existing Freeboard (feet)	FEMA-Required Freeboard (feet)
364+41	516+03 [2]	106.40	104.48	1.92[1]	105.45	0.95	3
369+50	530+30 [2]	109.30	105.28	4.02[1]	106.25	3.05	3
379+60	539+39 [2]	111.50	107.25	4.25[1]	108.31	3.19	3.5

1. This cross section for 100-year water surface profile does not meet the minimum 90 percent conditional non-exceedance probability requirement regardless of the freeboard condition.
2. This levee station follows the realigned SCR-1 as described in Section 5.1.

Red value represents a value that does not meet the particular criterion.
FEMA = Federal Emergency Management Agency
HEC-RAS = Hydrologic Engineering Center—River Analysis System
WSE = water surface elevation

As shown in Table 5.1, the top of levee elevations at River Station (RS) 250+62 immediately upstream of the Hwy 101 Bridge and RS 364+41 do not meet the FEMA freeboard and CNP requirements for the 100-year water surface elevation. Also, six cross sections (RS 263+56, RS 289+32, RS 296+50, RS 303+52, RS 369+50, and RS 379+60) do not meet the CNP requirement for the 100-year flood event while providing the required FEMA freeboard. For the design flow condition, two cross sections (RS 364+41 and RS 379+60) do not provide the required FEMA freeboard.

The required minimum top of levee elevations along SCR-1 are presented in Table 5.2. The calculations for the minimum top of levee elevations are included in the H&H report (Appendix I).

Table 5.2 – Minimum Top of Levee Elevations Required along SCR-1

HEC-RAS River Station	Approx. Levee Station	Existing Top of Levee Elevation (feet)	Required Minimum Top of Levee Elevation (feet)	Required Levee Raising (feet)
250+62	246+49	79.40	81.26	**1.86**
257+50	252+63	82.39	82.39	0.00
263+56	257+94	82.19	82.99	**0.80**
269+30	263+83	84.14	84.14	0.00
275+00	269+50	85.50	85.50	0.00
282+20	276+52	87.00	87.00	0.00
289+32	283+36	87.90	88.50	**0.60**
296+50	290+40	89.95	90.20	**0.25**
303+52	296+08	91.75	92.00	**0.25**
309+00	301+55	93.55	93.55	0.00
324+80	316+23	98.18	98.18	0.00
342+20	333+85	106.12	106.12	0.00
359+30	350+11	111.10	111.10	0.00
364+41	516+03[1]	106.40	109.20	**2.80**

HEC-RAS River Station	Approx. Levee Station	Existing Top of Levee Elevation (feet)	Required Minimum Top of Levee Elevation (feet)	Required Levee Raising (feet)
369+50	530+30[1]	109.30	110.00	**0.70**
379+60	539+39[1]	111.50	112.00	**0.50**
1. This levee station follows the realigned SCR-1 as described in Section 5.1				

5.2.2 Scour

The bottom of the levee protection would need to extend deep enough to cover the potential scour depth below the river invert. Generally, this additional depth of protection below the river invert elevation is referred to as the "toedown" depth. For the feasibility-level design, the scour depth varies from 5 to 15 feet. The scour depths used to determine the levee toedown design were calculated in the H&H analysis (Appendix I) and are summarized in Table 5.3. It should be noted that the levee stationing shown in the table is based on the realigned levee control line, which turns east near the Central Avenue Drain.

Table 5.3 – Scour Depth along SCR-1

Approximate Levee Station[1]	Scour Depth (feet)
Under Highway 101 Bridge	15
Upstream of Highway 101 Bridge to 252+54	15 to 12
252+54 to 326+71	12
326+71 to 327+00	12 to 15
327+00 to 365+25	15
365+25 to 510+00	15
510+00 to 511+00	15 to 5
511+00 to upstream end of SCR-1	5
1. Levee station is based on the realigned levee control line.	

5.3 Top of Levee Elevation and Toedown Elevation

Top of levee elevations and toedown elevations for the levee improvements were determined on the basis of the minimum top of levee elevations (Table 5.2) and design scour depth (Table 5.3), respectively. The design top of levee elevations (Table 5.4) and toedown elevations (Table 5.5) were adjusted to avoid unnecessary grade breaks in the profile and to replace abrupt changes in elevation with smoother transitions. These design elevations are also provided in the profiles of the feasibility-level design drawings (Appendix VI).

Table 5.4 – Summary of Top of Levee Elevations

HEC-RAS River Station	Levee Station[1] (along Design CL)	Existing TOL (along Design CL) (feet)	Minimum Proposed TOL[2] (feet)	TOL GB	Design TOL (feet)	Design TOL Slope (feet/foot)
246+53.2	242+40	78.60	80.30			
	243+87	78.00	80.65		81.26	
Highway 101						
	246+32	78.80	81.22	GB	81.26	0.0000
250+62	246+49	79.40	81.26		81.29	0.0018
257+50	252+54	82.39	82.39	GB	82.39	0.0018
263+56	257+94	82.19	82.99		83.24	0.0016
269+30	263+72	84.14	84.14	GB	84.14	0.0016
275+00	269+50	85.50	85.50	GB	85.50	0.0024
282+20	276+41	87.00	87.00	GB	87.00	0.0022
289+32	283+36	87.90	88.50		88.60	0.0023
296+50	290+27	89.95	90.20	GB	90.20	0.0023
303+52	296+08	91.75	92.00	GB	92.00	0.0031
309+00	301+43	93.55	93.55	GB	93.55	0.0029
324+80	316+11	98.18	98.18	GB	98.18	0.0032
342+20	334+19	106.12	106.12	GB	106.12	0.0044
359+30	349+98	111.10	111.10	GB	111.10	0.0032
	501+57	104.50	109.20	GB	111.10	0.0000
	503+82	105.05	109.20	GB	109.20	-0.0084
364+41	516+03	106.40	109.20	GB	109.20	0.0000
369+50	530+30	109.30	110.00	GB	110.00	0.0006
379+60 [3]	539+39	111.50	112.00		112.00	0.0022

1. Station equation: 350+69.36 (downstream) = 500+00.00 (upstream).

2. Minimum proposed top of levee elevations at Levee Stations 243+87 and 246+32 are interpolations between HEC-RAS River Stations 246+53.2 and 250+62.

3. Although the upstream limit of the levee structure is at Levee Station 540+20.39, the upstream limit of the levee segment that is parallel to the river is Levee Station 539+39 (HEC-RAS River Station 379+60). Therefore, the minimum proposed top of levee at HEC-RAS River Station 379+60 is used as a design parameter for the levee segment between Levee Stations 539+39 and 540+20.39.

CL = control line
GB = grade break
HEC-RAS = Hydrologic Engineering Center—River Analysis System
TOL = top of levee

Table 5.5 – Summary of Toedown Elevations

Levee Station (along Design Control Line)	Existing River Invert/Bottom of Basin Elevation[1] (feet)	Design Scour Depth feet)	Design Toedown Elevation (feet)	Design Toedown Slope (feet/foot)
243+87	52.65	15	37.60	
Highway 101				
246+32	54.04	15	39.00	0.0057
252+54	55.48	12	43.48	0.0072
276+41	66.17	12	54.17	0.0045
290+27	68.97	12	56.95	0.0020
311+27	73.64	12	61.64	0.0022
321+44	76.89	12	64.89	0.0032
326+71	79.08	12	67.00	0.0040
327+00	79.12	15	64.12	-0.1000
339+89	81.04	15	66.04	0.0015
350+69/500+00	91.75	15	68.55	0.0023
510+00	84.00	15	69.00	0.0005
511+00	84.00	5	79.00	0.1000
513+00	84.00	5	79.00	0.0000
523+00	84.12	5	79.00	0.0000
523+50	78.50	5	73.50	-0.1100
530+80	79.33	5	73.50	0.0000
532+80	86.00	5	81.00	0.0375
540+00	88.36	5	83.36	0.0033
539+39	88.36	5	83.36	0.0000

1. Upstream of Station 500+00, the design scour depth is measured relative to either the existing river invert or the bottom of the basin, whichever is lower.

5.4 Soil Cement Bank Protection

Based on the preferred design alternative selected through the screening process described in Sections 3.0 and 4.0, soil cement bank protection would be constructed along realigned SCR-1. This bank protection would consist of a soil cement revetment with a slope of 1H:1V from the top of levee (Table 5.4) to the toedown depth (Table 5.5) as shown in Figure 5.2. An interfering portion of existing riprap would be removed before placement of the soil cement. The soil cement would cover an 8-foot-wide area, with a 1-foot minimum overbuild. Only the portion of the overbuild between the top of levee and 5 feet below the adjacent ground would be trimmed for neatness along the existing levee alignment (downstream of Station 350+70), as shown in Figure 5.2.

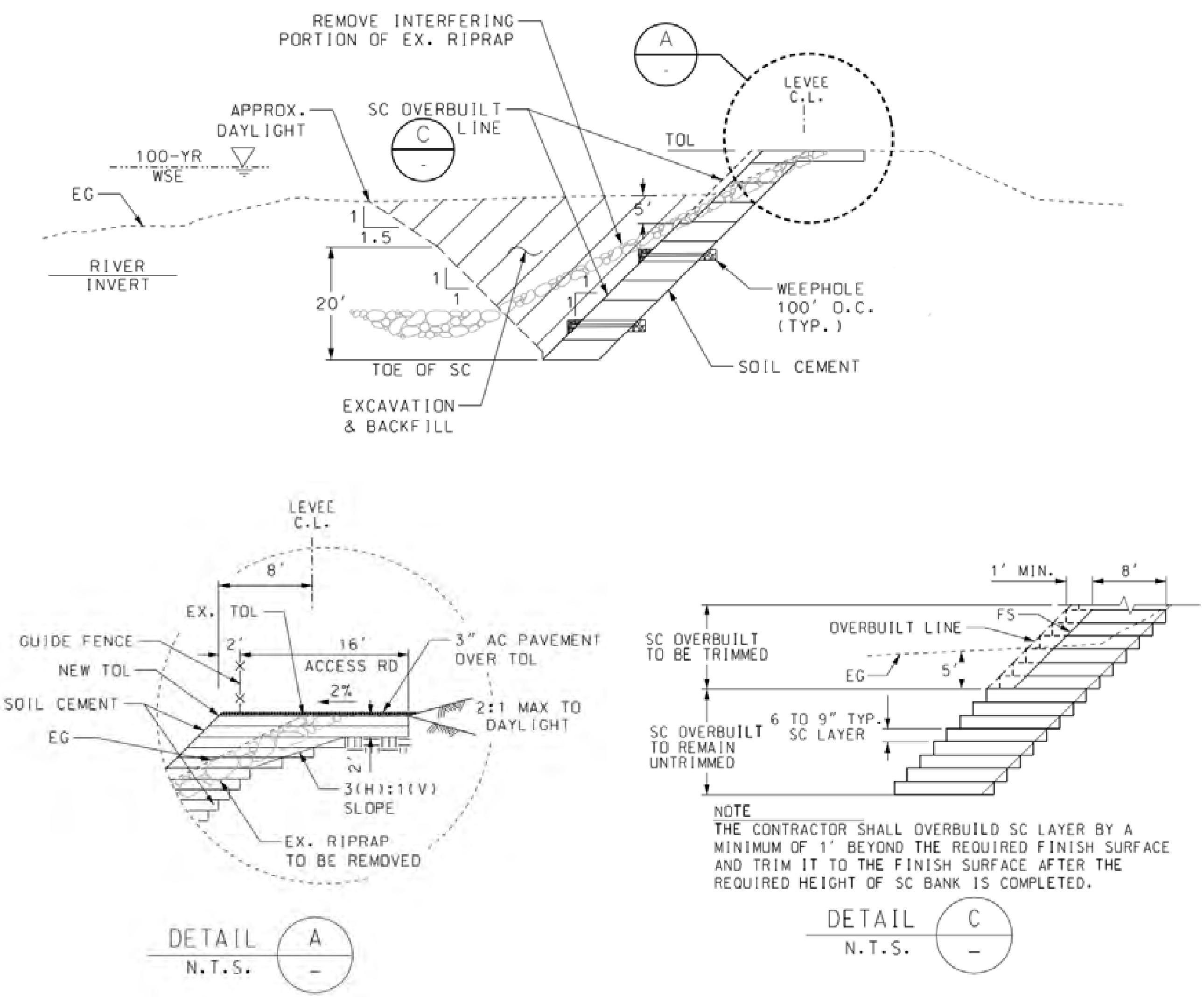

Figure 5.2 – Typical Soil Cement Bank Protection along Existing Levee Alignment

Along the new tie-in levee alignment (upstream of Station 350+70/500+00), the riverside face of untrimmed soil cement is buried under 5-foot-wide compacted fill with a slope of 2H:1V. In this reach, the new levee is subject to raising (3 to 4 feet). In order to prevent the 2H:1V-sloped fill limit of the raised levee on the landward side from covering the existing Central Avenue Drain, the new levee would be pushed away from the existing top of levee/top of bank along the storm drain alignment, as shown in Figure 5.3.

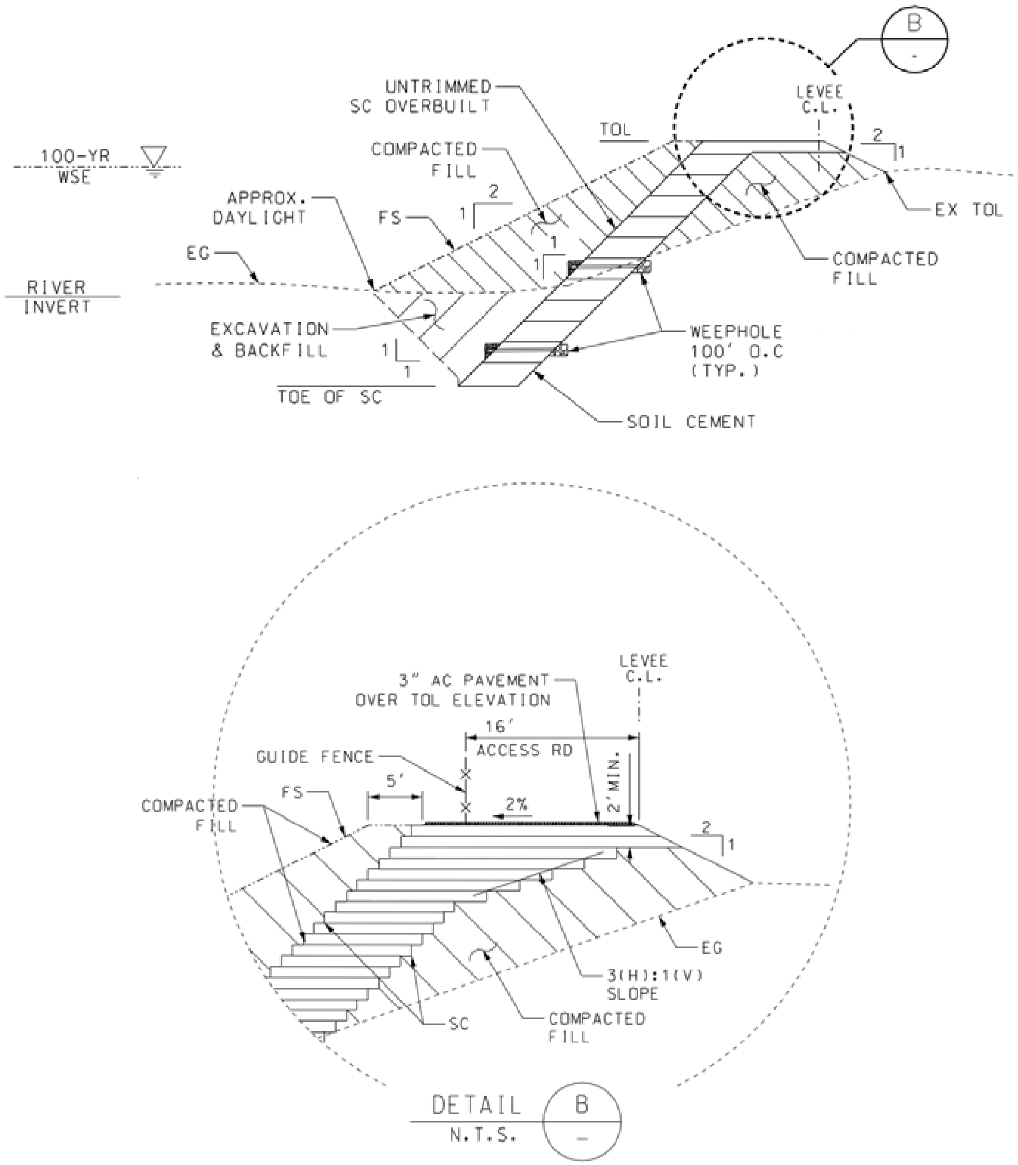

Figure 5.3 – Typical Soil Cement Bank Protection along Tie-In Levee Alignment

Generally, on the backside of the soil cement (between soil cement and existing levee), overexcavation of the existing levee embankment would be required to ensure that the soil cement is constructed against competent material. This overexcavation would consist of excavating the existing levee material within the 1.5H:1V excavation limit projected landward from the bottom of the soil cement and replacing it with competent material. Also, due to the overall height of the soil cement bank, stability factors of safety may require deepening of the soil cement toedown elevation or widening of the lower section of the soil cement. These requirements will need to be based on future site-specific geotechnical recommendations.

5.5 Levee Extension under the Hwy 101 Bridge

A reinforced concrete floodwall would be constructed along the existing levee underneath the Hwy 101 Bridge to connect with the SCR-3 system, which is being analyzed for levee improvements by others (Figure 5.4). Limited space underneath the bridge is likely to prevent access by the heavy construction equipment and vehicles needed for the soil cement construction. Use of a floodwall would allow SCR-1 to be raised to meet the top of levee elevation requirements under the bridge soffit. Along the riverward side slope of the levee, the existing grouted riprap would be replaced with new grouted riprap that has a deeper toedown. As shown in Figure 5.4, because the toe of the new grouted riprap protection would be outside the limit of influence on the adjacent bridge pier in terms of scour, the toedown depth would not be subjected to the bridge pier scour.

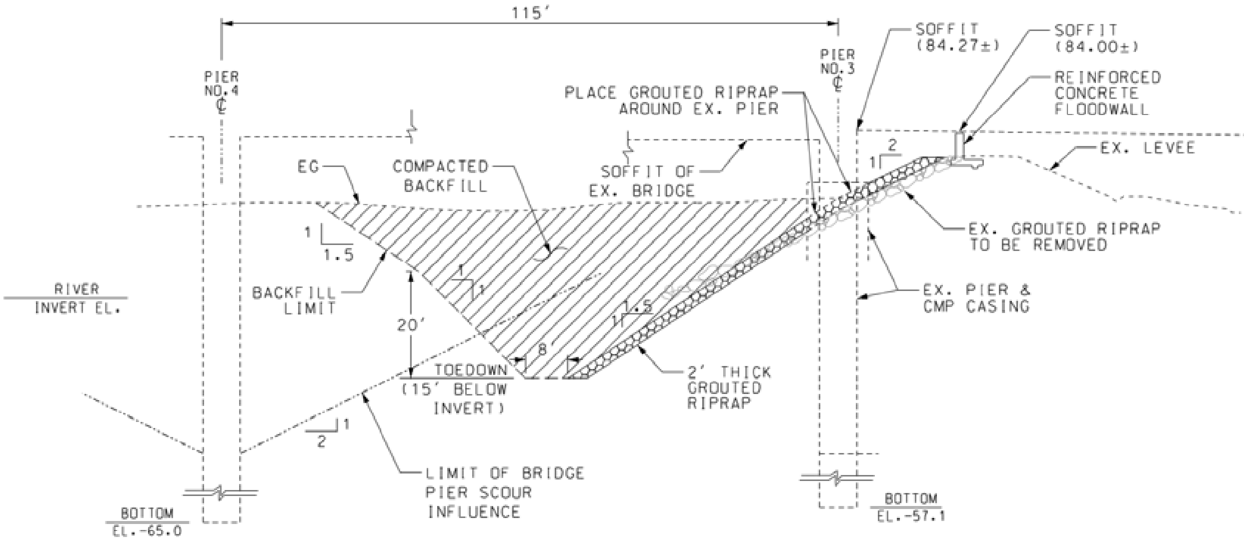

Figure 5.4 – Typical Levee Protection underneath Hwy 101 Bridge

Construction access under the bridge would likely be limited and, therefore, require close coordination with Caltrans.

5.6 Access Ramps

Sixteen-foot-wide access ramps would be constructed along the riverward side of the levee at the current locations of major ramps to meet the minimum requirements of EM 111-2-1913 (USACE 1996) (Figure 5.5). The access ramps would have a 10 percent slope profile and be paved with reinforced concrete. The 10 percent slope would allow river access down to the channel invert elevation. Below the invert elevation, the access ramp would continue down at a 1H:1V slope to provide toedown protection similar to that of the adjacent levee sections.

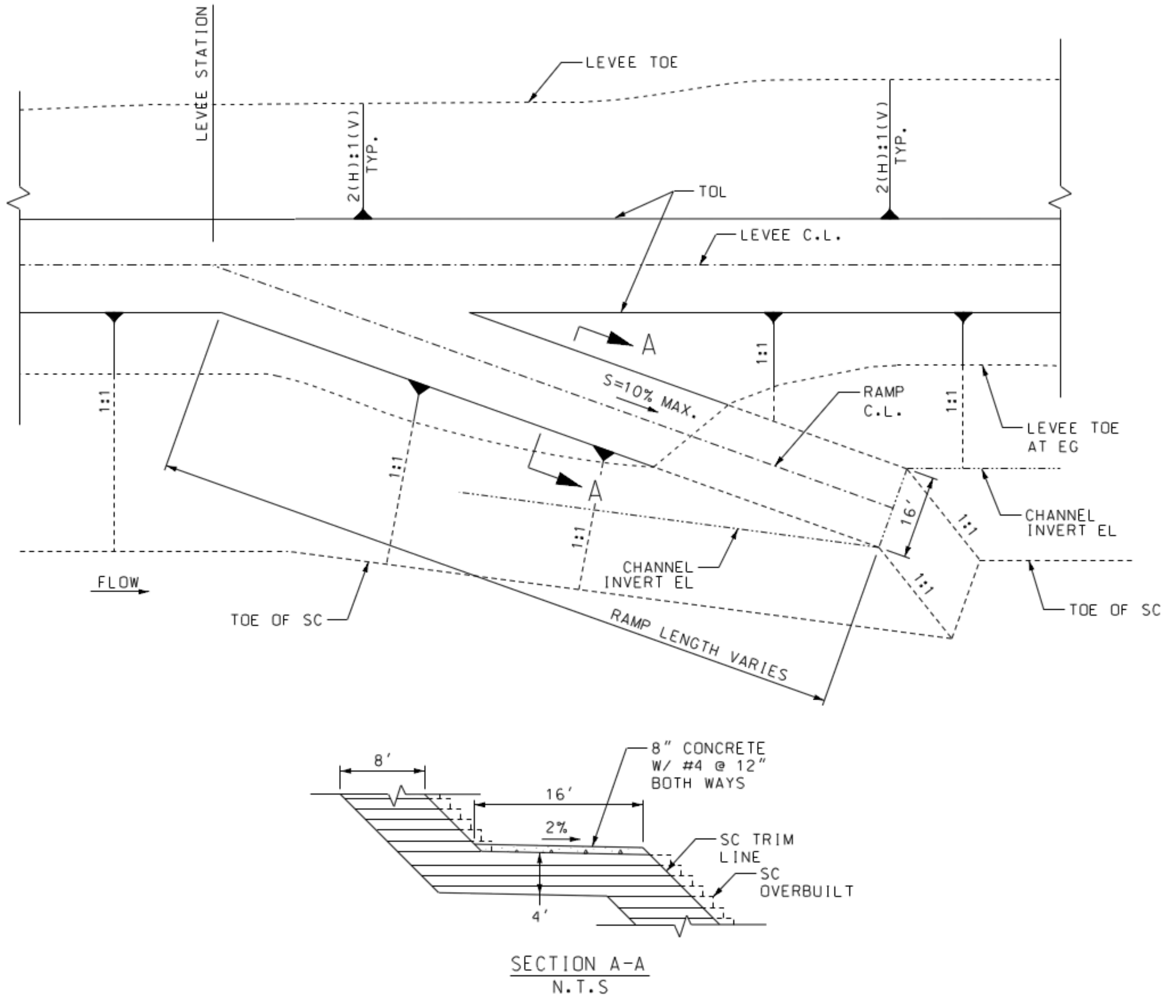

Figure 5.5 – Typical Access Ramp

Additionally, an 8 percent sloped access ramp of asphalt concrete pavement would be constructed in the vicinity of the Hwy 101 Bridge on the landward side of the levee. This ramp would be used to connect with the future bicycle path downstream of SCR-1.

5.7 Storm Drain Penetration

The storm drain penetrations through the levee would require modification or replacement of the existing structures to accommodate the new soil cement bank protection. Soil cement with a slope of 1H:1V would require shortening of the existing pipe or box culvert and replacement of existing outlet structures with new structures. Currently, a total of four storm drains are identified within the project limits (Table 5.6).

Table 5.6 – Summary of Existing Storm Drain Penetrations

Levee Station	Description
350+49	Central Avenue Drain, two 72-inch-diameter reinforced concrete pipes (flap gates on riverward side)
316+80	Side Drain No. 6, 48-inch-diameter reinforced concrete pipe (flap gate on landward side)
282+00	Side Drain No. 4, 48-inch-diameter reinforced concrete pipe (flap gate on landward side)
253+13	Stroube Drain – Unit I, 10-foot-wide by 9-foot-high reinforced concrete box with sluice gate

The Stroube Drain has a recently installed sluice gate in the gatewell structure on the landward side of the levee and would not require a replacement of the closure device, whereas the Central Avenue Drain would require removal, salvage, and reinstallation of the existing flap gates to the new outlet structure. Side Drain Nos. 4 and 6 would require removal and disposal of the existing flap gates and installation of new flap gates to the new outlet structure.

A typical storm drain modification for Side Drain Nos. 4 and 6 is shown in Figure 5.6. Detailed drawings of the modifications of the Stroube and Central Avenue Drains are included in Appendix IV.

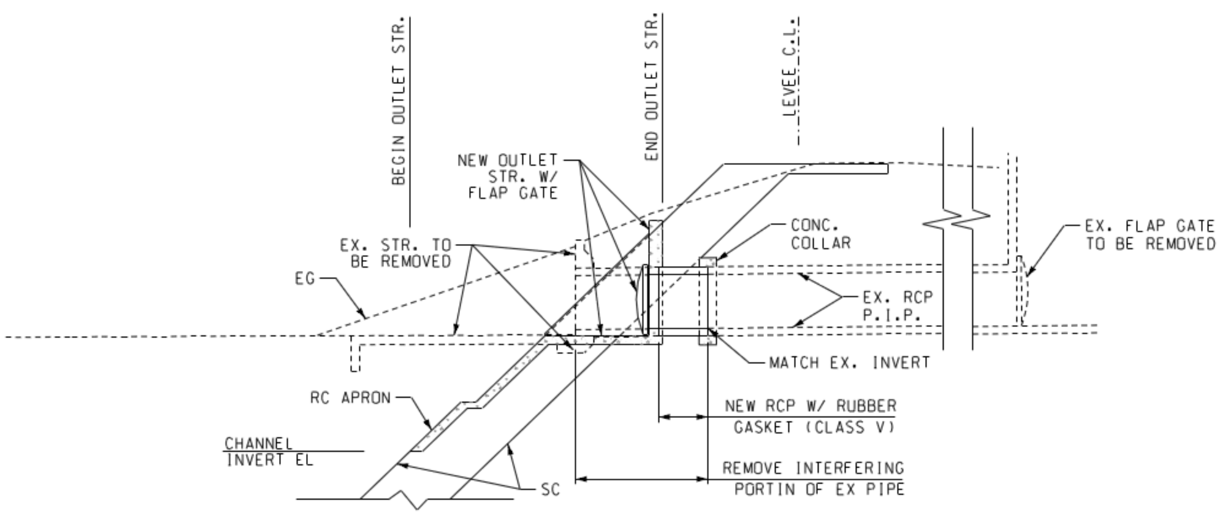

Figure 5.6 – Typical Storm Drain Modification (Side Drain Nos. 4 and 6)

5.8 Launchable Stone

Launchable stone would be constructed along the levee toe between Stations 500+00 and 502+00 where the existing levee alignment turns to the east (Figure 5.7). The launchable stone would provide additional protection against the impinging flow of the river, without deepening the soil cement design in that area.

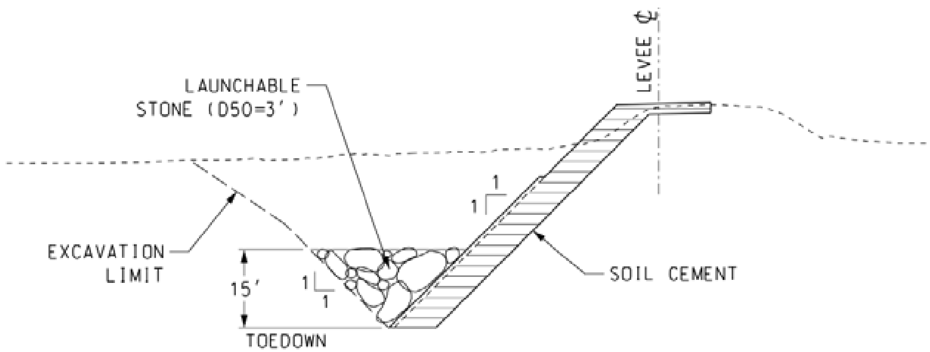

Figure 5.7 – Typical Launchable Stone

5.9 Environmental Considerations

VCWPD has conducted bird surveys along the project limits and has identified nesting habitat for the Least Bell's Vireo and the Southwestern Willow Flycatcher, both endangered bird species. Additional biological surveys and assessments will be required to support the environmental documentation and permits for project construction.

For the feasibility-level design, impacts on the disturbed existing vegetation and/or natural habitat have been evaluated in terms of mitigation cost. The mitigation cost was determined according to the guidelines provided by VCWPD and presented in Section 6.0.

5.10 Geotechnical Design Considerations

No subgrade exploration or laboratory analysis has been performed for this study. The design and subsequent cost estimates have been determined on the basis of the best engineering judgment from similar projects in the area.

The soil cement slope protection is expected to adequately impede seepage such that the landward side slope of the levee would not be adversely affected by steady-state seepage conditions resulting from the design-level flood. Where large descending slopes are present on the landward side of the levee at existing gravel quarry locations, it is anticipated that if steady-state seepage conditions develop, the water level in the quarry would be elevated and would act as a counterbalance to seepage forces and exit gradients.

The current minimum design width for the soil cement prism is 8 feet. This width would need to be confirmed during the construction-level design with regard to stability under conditions of toe scour. The width may need to be adjusted on the basis of the strength of the foundation soil below the scour depth. This evaluation would require the collection of site-specific strength data by means of subsurface exploration.

The backslope of the currently proposed soil cement protection is 1H:1V with maximum backcut slope heights in excess of 40 feet. A preliminary evaluation indicates that the backcut may need to be flattened to achieve an adequate factor of safety for construction. A flatter backcut slope would require the placement of general backfill behind the soil cement prism. Further evaluation based on site-specific testing would need to be performed during the final design phase.

Hydrogeologic evaluations performed by others for gravel quarries adjacent to the levee were reviewed with respect to potential groundwater levels in the area of the proposed improvements. The interpretation of groundwater levels in existing monitoring wells and surface water levels in quarry pits in the area suggest that groundwater can fluctuate significantly over time. Historical high groundwater along the levee alignment could range from 5 to 10 feet above the proposed toe of the soil cement protection. Under these conditions, temporary dewatering of the excavation for the soil cement would likely be necessary.

The future construction-level design would require geotechnical analysis based on the subgrade exploration of the project area to finalize the design details. The geotechnical study should evaluate the key aspects of the project, including the following:

- Required depth and limits of overexcavation of the existing levee to provide competent support of the soil cement construction
- Suitability of excavated material for use in soil cement and/or general backfill
- Preliminary mix design for the soil cement
- Stability of the soil cement slope under conditions of flooding, seismic activity, and scour
- Foundation conditions for reinforced concrete floodwalls under the Hwy 101 Bridge
- Construction considerations, including temporary stability and dewatering requirements

6.0 COST

Cost estimates were prepared for the feasibility-level design for planning purposes. The cost estimates are based on quantities calculated from the three-dimensional surfaces using MicroStation InRoads software and engineering calculations, and unit costs from recent and similar local projects. A discussion of the cost to mitigate the impacts in areas affected by the construction activities is provided in Section 6.3.

The unit prices for soil cement and earthwork were selected on the basis of the average values in the bid abstracts for the recently completed flood control projects in Santa Barbara and Ventura Counties. The recent projects were selected on the basis of similar design features and construction sizes. The unit prices for other cost items were based on the RSMeans cost database and engineering judgment.

No subsurface analysis was performed for this study, and updated geotechnical exploration may alter the quantities shown in the cost estimates. The cost estimates exclude any fees or permits required for construction or maintenance activities, as well as costs associated with real estate requirements.

Detailed backup for the feasibility-level design cost estimates is provided in Appendix VII.

The feasibility-level construction cost for the proposed improvements is approximately **$32,780,000** without mitigation cost and **$39,740,000** with mitigation cost.

6.1 Key Cost Estimate Assumptions

The unit prices used in the cost estimate for the feasibility-level design are based on several key assumptions. The following are the assumptions for the primary construction items:

- **Excavation.** All excavated material would be stockpiled on-site and made available for reuse as backfill and for use in the soil cement.
- **Backfill.** All material for use as backfill would come from the excavated materials. No borrow fill, except for launchable stone, is assumed to be delivered to the project site.
- **Soil cement.** The soil cement would be mixed on-site, and all of the soil would come from the previously excavated material.
- **Riprap demolition.** The demolished material would be stockpiled on-site and reused if the material meets the specifications.

6.2 Contingency Development

Contingencies represent allowances to cover unknowns, uncertainties, and/or unanticipated conditions that are not possible to adequately evaluate from the data on hand at the time the cost estimate is prepared but must be represented by a sufficient cost to cover the identified risks. Per Corps guidance, a risk-based contingency must be developed and used to calculate a project's contingency at the feasibility-level of design. For this cost estimate, an abbreviated risk analysis (ARA) spreadsheet has been used to calculate the project contingency of 29.20 percent. This

ARA documents the risks for the different construction elements unique to this project and develops a weighted average contingency for the whole project based on the individual construction element contingencies. The ARA is provided in Appendix VIII.

6.3 Mitigation of Affected Areas

The floodplain of the Santa Clara River that is affected or disturbed by the construction of the SCR-1 improvements will likely require mitigation. Mitigation for this project was developed in accordance with the mitigation guidelines established through close coordination with VCWPD.

The mitigation areas can be categorized into areas with permanent impact and areas with temporary impact. The permanent impact area includes the areas within the footprints of both the aboveground and below ground soil cement structure. The temporary impact area includes any disturbed areas, mostly within the excavation limits but outside the permanent impact areas. The total acreage of required mitigation is the total acreage of the affected area, multiplied by a mitigation factor based on the guidelines in Table 6.1. Typical sections showing how these mitigation factors are applied along the levee alignment are also included in Figures 6.1 through 6.4.

Table 6.1 – Summary of Mitigation Factors

Mitigation Type	Type of Impact	Description	Mitigation Factor
A	Permanent	Conversion of existing unvegetated riprap to soil cement	2:1
B	Permanent	Conversion of existing vegetated riprap to soil cement	4:1
C	Temporary	Existing vegetated ground	2:1
D	Temporary	Existing unvegetated ground	1:1

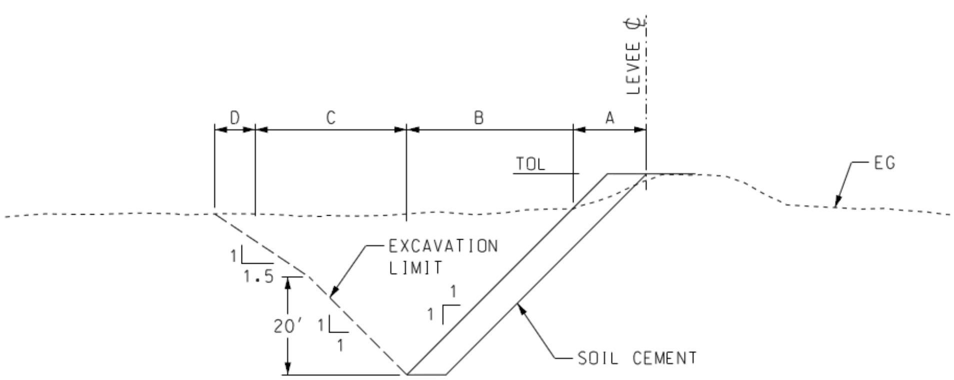

Figure 6.1 – Typical Mitigation Section (Sta. 246+00, Hwy 101 Bridge, to Sta. 252+00, Stroube Drain)

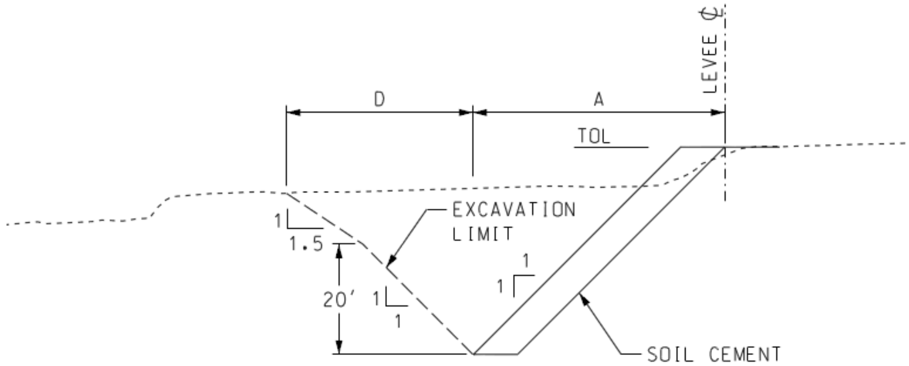

Figure 6.2 – Typical Mitigation Section (Sta. 252+00, Stroube Drain, to Sta. 501+00, Central Avenue Drain)

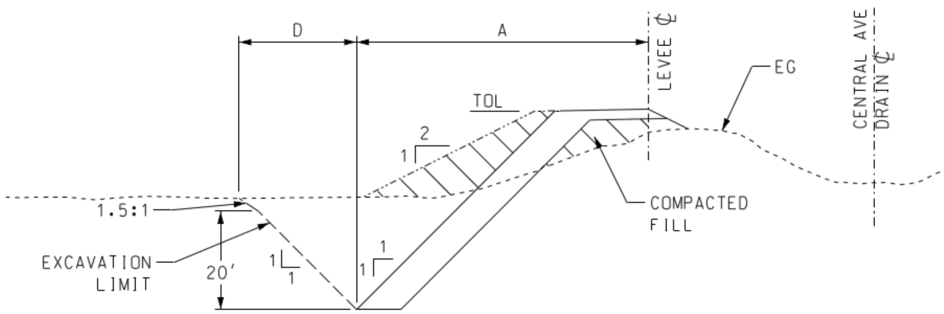

Figure 6.3 – Typical Mitigation Section (Sta. 501+00, Central Avenue Drain, to Sta. 528+00)

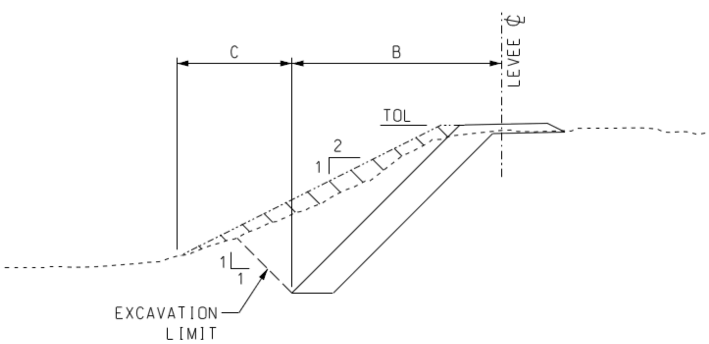

Figure 6.4 – Typical Mitigation Section (Sta. 528+00 to Sta. 540+00)

6.4 Total Project Cost Estimate

A breakdown of the feasibility-level cost estimate is provided in Table 6.2.

Table 6.2 – Summary of Costs for the Feasibility-Level Design

	Feasibility-Level Design				
	Contract Items	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
1	Mobilization (5% of Total Construction Cost)	LS	1	$ 922,000	$ 922,000
2	Clearing and Grubbing	AC	28.3	$ 4,000	$ 113,200
3	Diversion and Control of Water	LS	1	$ 500,000	$ 500,000
4	Soil Cement Levee Slope Protection	LF	14,458	$ 1,108.59	$ 16,028,022
4.1	Soil Cement	CY	181,361	$ 45	$ 8,161,245
4.2	Existing Riprap Removal[1]	CY	88,500	$ 7.75	$ 685,875
4.3	Excavation	CY	512,574	$ 5.75	$ 2,947,301
4.4	Backfill (Toedown Construction)	CY	377,226	$ 4.50	$ 1,697,517
4.5	Compacted Fill (Levee Raising)	CY	12,493	$ 6.00	$ 74,958
4.6	Compacted Fill (5-Foot Earthen Cover)	CY	37,198	$ 4.50	$ 167,391
4.7	Launchable Stone (Sta. 500+00 to Sta. 502+00)	CY	1,700	$ 150.00	$ 255,000
4.8	Weephole	LS	1	$ 1,107,000	$ 1,107,000
4.9	Cable Guard Fence	LF	14,458	$ 20	$ 289,160
4.10	Asphalt Concrete Pavement (along Toe of Levee)	SY	25,703	$ 25	$ 642,575
5	Levee Protection under Hwy 101 Bridge	LF	245	$ 2,878.38	$ 705,203
5.1	Grouted Riprap Toedown Extension	CY	1,656	$ 228	$ 377,568
5.2	Floodwall	LF	245	$ 670	$ 164,150
5.3	Excavation	CY	13,143	$ 7.00	$ 92,001
5.4	Backfill (Toedown Construction)	CY	11,914	$ 6.00	$ 71,484

Feasibility-Level Design

	Contract Items	Unit	Quantity	Unit Cost	Total Cost
6	Construction of Access Ramp	EA	6	$ 134,375	$ 806,250
6.1	Soil Cement	CY	11830	$ 45	$ 532,350
6.2	Compacted Fill	CY	8150	$ 6.00	$ 48,900
6.3	Reinforced Concrete Pavement	SY	3000	$ 75	$ 225,000
7	Construction of Access Ramp for Bicycle Path Connection	EA	1	$ 24,150	$ 24,150
7.1	Compacted Fill	CY	900	$ 6.00	$ 5,400
7.2	Asphalt Concrete Pavement	SY	350	$ 25	$ 8,750
7.3	Removal of Existing Retaining Wall	LS	1	$ 10,000.00	$ 10,000
8	Storm Drain Outlet Replacement (Storm Drain Nos.4 and 6)	EA	2	$ 35,000	$ 70,000
8.1	Removal of Existing Storm Drain System	EA	2	$ 6,000	$ 12,000
8.2	New Storm Drain System	EA	2	$ 14,000	$ 28,000
8.3	Flap Gate	EA	2	$ 15,000	$ 30,000
9	Storm Drain Outlet Replacement (Stroube Drain)	EA	1	$ 133,000	$ 133,000
9.1	Removal of Existing Storm Drain System	EA	1	$ 53,000	$ 53,000
9.2	New Storm Drain System	EA	1	$ 80,000	$ 80,000
10	Storm Drain Outlet Replacement (Central Ave. Drain)	EA	1	$ 70,000	$ 70,000
10.1	Removal of Existing Storm Drain System	EA	1	$ 16,000	$ 16,000
10.2	New Storm Drain System	EA	1	$ 40,000	$ 40,000
10.3	Removal and Reinstallation of Existing Flap Gates	EA	2	$ 7,000	$ 14,000
				Subtotal:	$ 19,371,825
		Planning, Engineering, and Design			$ 4,000,000

	Feasibility-Level Design				
	Contract Items	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
		Construction Management			$ 2,000,000
				Subtotal	$ 25,371,825
		Contingencies		(@ 29.2%)	$ 7,408,573
				Subtotal	**$32,780,397**
11	Along the Existing Levee Alignment (Sta.246+00 to Sta. 501+00)	AC	32.3		$ 4,845,000
11.1	Permanent Mitigation (2:1 Ratio)	AC	22.0	$ 150,000	$ 3,300,000
11.2	Permanent Mitigation (4:1 Ratio)	AC	2.0	$ 150,000	$ 300,000
11.3	Temporary Mitigation (2:1 Ratio)	AC	2.0	$ 150,000	$ 300,000
11.4	Temporary Mitigation (1:1 Ratio)	AC	6.3	$ 150,000	$ 945,000
12	Along the Tie-In Levee Alignment (Sta.501+00 to 504+00)	AC	14.1		$ 2,115,000
12.1	Permanent Mitigation (2:1 Ratio)	AC	6.6	$ 150,000	$ 990,000
12.2	Permanent Mitigation (4:1 Ratio)	AC	5.6	$ 150,000	$ 840,000
12.3	Temporary Mitigation (2:1 Ratio)	AC	1.0	$ 150,000	$ 150,000
12.4	Temporary Mitigation (1:1 Ratio)	AC	0.9	$ 150,000	$ 135,000
				Grand Total:	$ 39,740,397

1. Cost of existing riprap removal assumes that the removed material will be stockpiled and reused if the material meets the specifications.

7.0 REFERENCES

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APPENDIX I

Hydrology and Hydraulics Report

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APPENDIX II

Preliminary Screening Design Drawings

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APPENDIX III

Preliminary Screening Design Cost Estimates

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APPENDIX IV

Conceptual-Level Design Drawings

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APPENDIX V

Conceptual-Level Design Cost Estimates

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APPENDIX VI

Feasibility-Level Design Drawings

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APPENDIX VII

Feasibility-Level Cost Estimates

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APPENDIX VIII

Abbreviated Risk Analysis

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