APPENDIX E

DRAFT

Avenue 54 to Thermal Drop Structure

Channel Improvement Project

Hydraulic Basis of Design, 50% Plan Set

Submitted to:

Coachella Valley Water District
75515 Hovley Lane East
Palm Desert, CA 92211
Attention: Tesfaye Demissie

Submitted by:

northwest hydraulic consultants
2600 Capitol Avenue, Suite 140
Sacramento, CA 95816

Contact: Brady McDaniel, P.E.
Phone: (916) 371-7400
bmcdaniel@nhcweb.com

August 2017

DRAFT
Avenue 54 to Thermal Drop Structure
Channel Improvement Project
Hydraulic Basis of Design, 50% Plan Set

Submitted to:

Coachella Valley Water District
75515 Hovley Lane East
Palm Desert, CA 92211
Attention: Tesfaye Demissie

Submitted by:

northwest hydraulic consultants
2600 Capitol Avenue, Suite 140
Sacramento, CA 95816

Contact: Brady McDaniel, P.E.
Phone: (916) 371-7400
bmcdaniel@nhcweb.com

September 2017

File 5000876

Report Prepared by:

____
Hank Fehlman, P.E.

Reviewed by:

____
Brady McDaniel, P.E.

DISCLAIMER

This document has been prepared by **northwest hydraulic consultants** in accordance with generally accepted engineering practices and is intended for the exclusive use and benefit of WSP, Inc., and the Coachella Valley Water District and their authorized representatives for specific application to the Avenue 54 to Thermal Drop Structure Project. The contents of this document are not to be relied upon or used, in whole or in part, by or for the benefit of others without specific written authorization from **northwest hydraulic consultants**. No other warranty, expressed or implied, is made.

northwest hydraulic consultants and its officers, directors, employees, and agents assume no responsibility for the reliance upon this document or any of its contents by any parties other than the client for whom the document was prepared.

Table of Contents

List of Figures

List of Tables

List of Appendices

1. Introduction and Background

1.1 Purpose

The Coachella Valley Water District (CVWD) has directed WSP, Inc. (WSP) to design flood protection improvements within the Coachella Valley Stormwater Channel (CVSC) between Avenue 54 and the Thermal Drop Structure. Northwest Hydraulic Consultants (NHC), a subconsultant to WSP, has been tasked with developing the hydraulic basis of design for the project. This report summarizes the hydraulic analyses completed to support the engineering design of the proposed improvement project. The current plan set has been developed to the 50% design level. This report does not identify or address pipeline or utility crossings. These will be addressed by WSP in future reports.

This study follows a previous alternatives evaluation study of the CVSC between 52nd Avenue and Lincoln Street completed by Tetra Tech (2014), and a focused evaluation of alternatives for the project reach prepared by the WSP/NHC project team for inclusion in the EIS for this project. The Tetra Tech (2014) study had recommended a fully-lined channel for the portion of the project reach between Airport Boulevard and the Thermal drop structure. Initial environmental review conducted for WSP's study indicated that environmental permitting for that alternative would prove difficult. The subsequent focused alternatives analysis conducted by the WSP/NHC project team resulted in the channel improvement plan evaluated herein, which includes reduced extents of full concrete lining, as described in Section 3 below.

1.2 Report Organization and Conventions Used

Section 1 describes the project background and purpose. Section 2 summarizes the design criteria for the project. Section 3 presents a description of the proposed project components. Section 4 summarizes the hydraulic basis of the design.

Existing condition topographic information presented in this report is based off site surveys performed by WSP in 2015 and 2016. Project plan geometries used for the hydraulic modeling summarized in this report are based on digital surfaces provided by WSP in June 2017. Horizontal locations referenced in this report are based on the NAD83 California State Plane Zone 6 coordinate system. Elevations referenced in this report are in the North American Vertical Datum of 1988 (NAVD88).

All references in this report to the CVSC are oriented looking downstream. Therefore, the west bank of the CVSC is referred to as the right bank. Conversely, the east bank of the CVSC is referred to as the left bank.

Cross-section and bridge stationing presented in this report is measured along the channel baseline developed for this project by WSP, shown in the 50% design plans presented in Appendix C. The current baseline is almost identical to the original baseline used to design and construct the channel in 1972.

2. Design Criteria

Relevant design criteria for the project are summarized below. CVWD has directed the project team to design the improvements to meet CVWD's 100-year Plus flood protection standard, which as described below meets or exceeds FEMA minimum standards for passage of the 100-year flood.

2.1 Water Surface Elevations

Design water levels are calculated for peak flows contained within the CVSC channel, assuming no upstream breakout or overtopping of flows to the floodplain. The CVWD design criteria used for this project for setting top of slope protection is the 100-year water level plus 4 feet of freeboard throughout for leveed and incised reaches. The design will also comply with the FEMA standard, which is 100-year flood plus three feet of freeboard for leveed reaches, with an extra foot of freeboard in the vicinity of bridges.

2.2 Hydrology

The upstream inflows for the hydraulic model will be the 100-year peak flow of 39,000 cfs as developed in USACE (1980). This is consistent with the flows used for the ongoing FEMA CVSC Physical Map Revision (PMR) that will remap the project area flood hazards upon completion.

2.3 Scour Analysis

Scour refers to the channel bed lowering below its normal level that can occur over sizeable areas during the passage of a large flood (termed *general scour* in this report) and the bed lowering over small areas that results from the interaction of the flood with structures or features within the channel (termed *local scour* in this report). The following summarizes the design standards used to calculate each type of scour for the project:

- General Scour – Calculated using the Blench regime depth equation with a zero-bed factor and an adjustment factor for channel conditions as specified in the Development Design Manual (DDM)
- Local Scour – The proposed channel improvements include full channel lining at bridge crossings, so local scour associated with these support structures will not be a factor. Potential scour at the downstream end of the two fully lined reaches within the project limits was assessed using the Blench regime depth equation, contrasted with the results of sediment routing and potential drop scour calculations.

2.4 Bank Protection Configuration

Concrete slope protection will be used for slope protection along the length of the project. Bank protection will slope at 1.5:1 (horizontal to vertical) to the approximate thalweg of the channel, and extend to the computed scour depth at a 1:1 slope. For hydraulic analysis, a graffiti barrier of soil placed at a 3:1 slope is assumed to cover the otherwise exposed concrete slope protection in the reaches of the project with unlined inverts. No graffiti barrier will be constructed at fully-lined locations (bridges) so that maximum channel cross-section is preserved.

3. Description of the Preferred Alternative

The proposed improvements associated with the channel improvement plan extend from 300 feet downstream of the existing Thermal Drop Structure (located at Station 569+50) to Avenue 54 (Station 675+00), a distance of about 2 miles. Plan and profile views of the preferred alternative are illustrated in Figures 1 and 2. Detailed 50% design plans are presented in Appendix C.

Airport Boulevard crosses the project reach at Station 617+50, Union Pacific Railroad (UPRR) bridges cross at Stations 584+20 and 583+60, and Highway 111 crosses at Station 582+00. Major items of the preferred alternative include: (1) lowering the Thermal Drop Structure; (2) lowering the channel invert profile between the Thermal Drop Structure to just upstream of the Airport Boulevard bridge; (3) concrete lining of the channel banks along the entire length of the project reach; (4) lining of the channel invert in the vicinity of the Airport Boulevard crossing; and, (5) lining of the channel invert in the vicinity of the Railroad bridges and Highway 111 crossings.

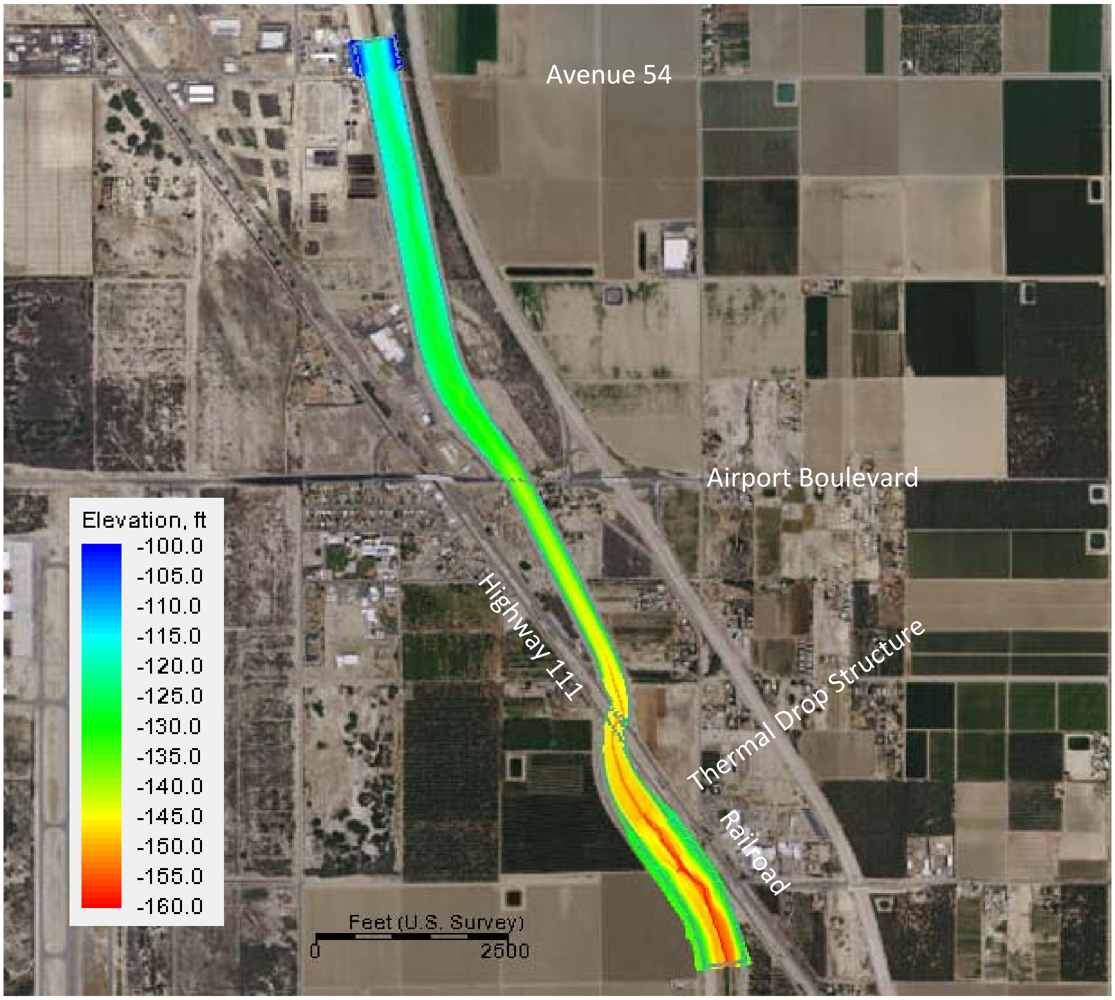


Figure 1. Aerial view of the project reach with existing ground elevations shown along the channel

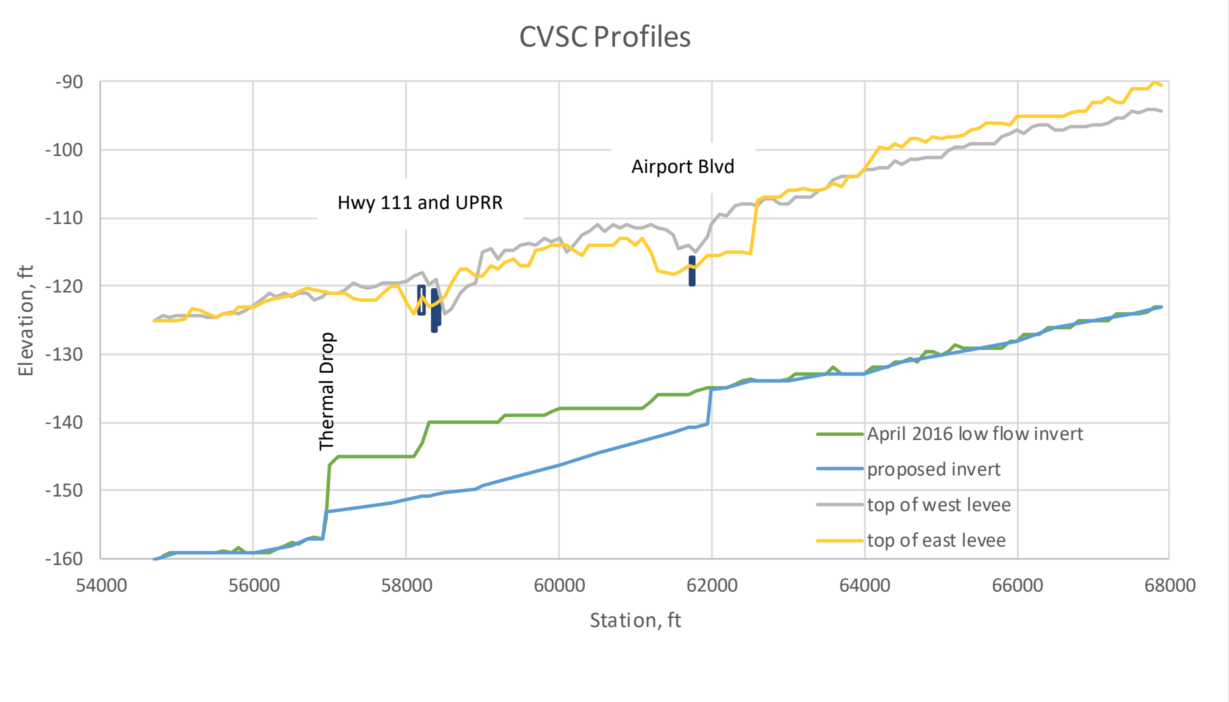


Figure 2. Profile view of the project reach

A detailed view of the proposed improvement in a local reach of the project between Airport Boulevard and the upstream end of the study reach is shown in Figure 3. The typical section in this reach, shown in Figure 4, has a topwidth of 450 feet. The invert slope through this reach is approximately 0.002. Much of the channel bottom through this reach is to remain unmodified.

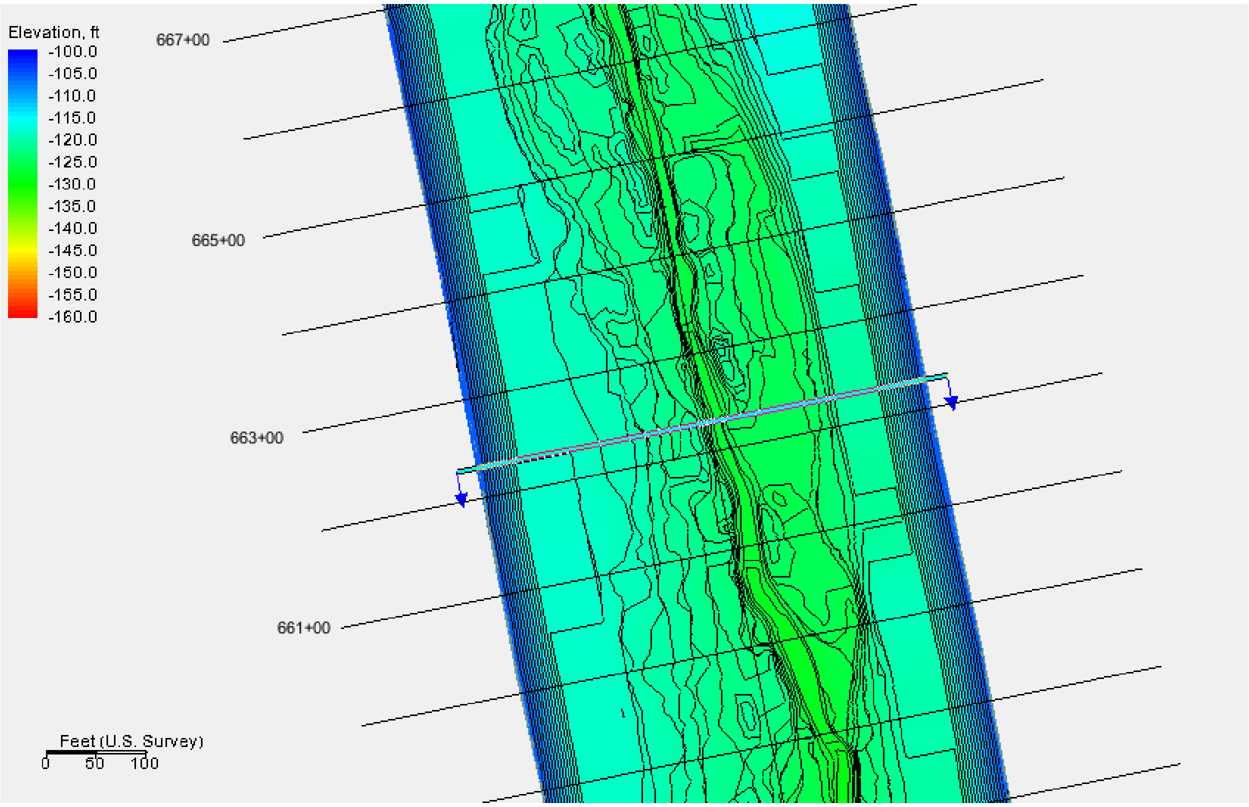


Figure 3. Plan view of the proposed channel in the reach upstream of the Airport Boulevard bridge

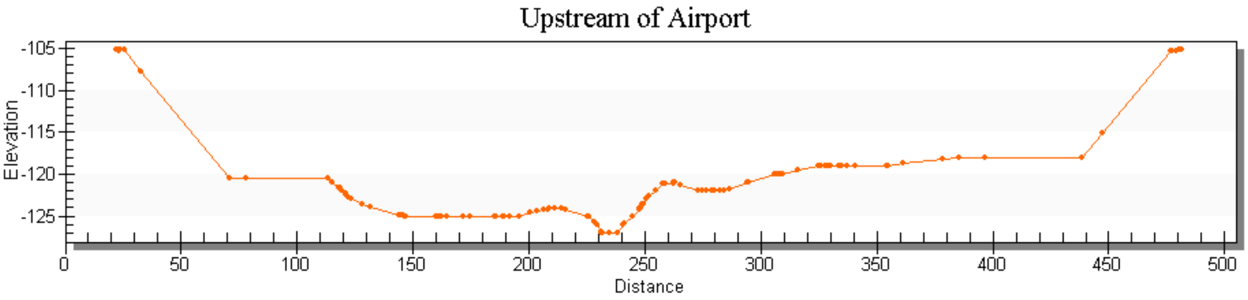


Figure 4. Typical section in the reach upstream of the Airport Boulevard Bridge

Proposed improvements near the Airport Boulevard crossing are shown in Figure 5, with a section view in Figure 6. The preferred alternative will enlarge and fully line the existing cross-section through this crossing, to increase flood conveyance capacity while protecting the existing bridge supports. The proposed section will be of compound type, with a wide low flow section and steeper banks. The section will have a top width of approximately 300 ft. The upstream end of this fully-lined reach includes a steep segment that will connect the existing grade upstream with the lowered reach through the bridge crossing (see Figure 2, near Station 620+00). The upstream end of this lined reach (and the required cutoff wall, discussed in Section 4) will serve as a grade control for the reach immediately upstream.

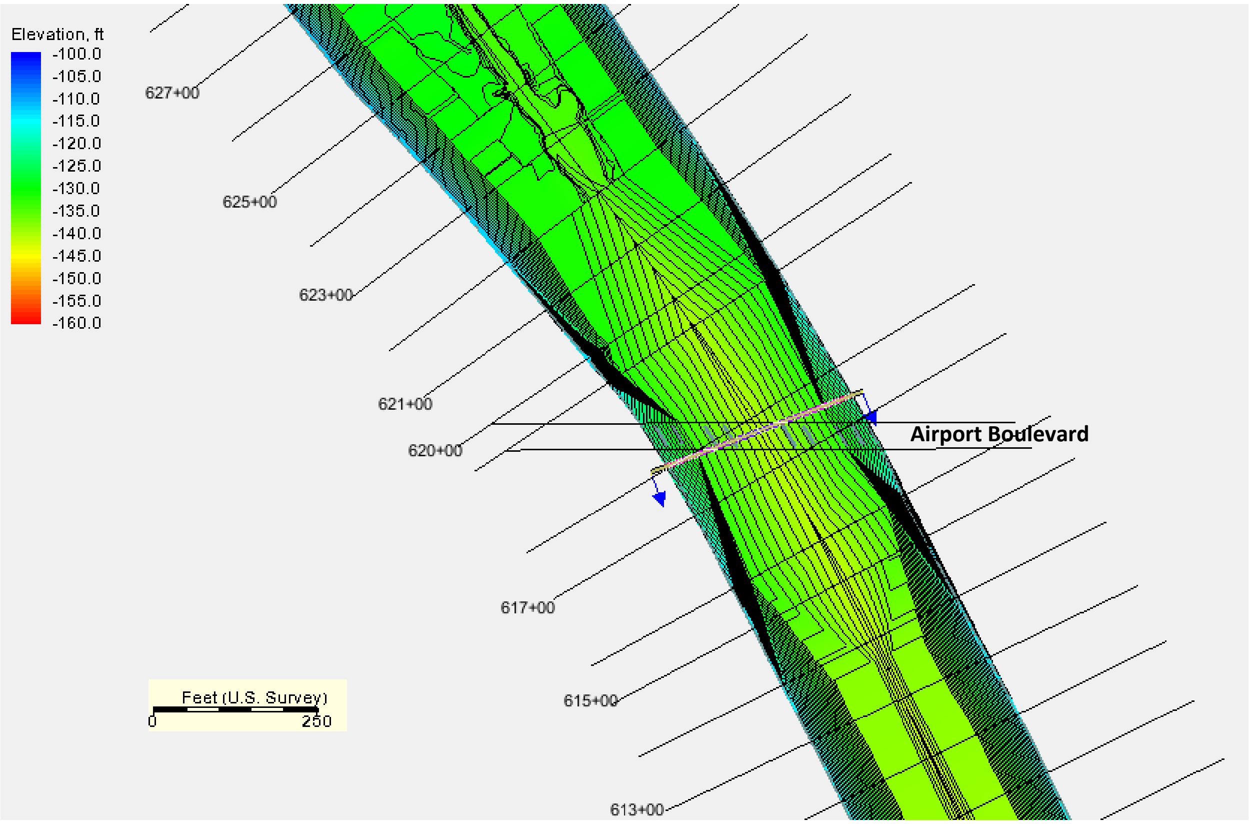


Figure 5. Plan view of the proposed channel through the Airport Boulevard crossing

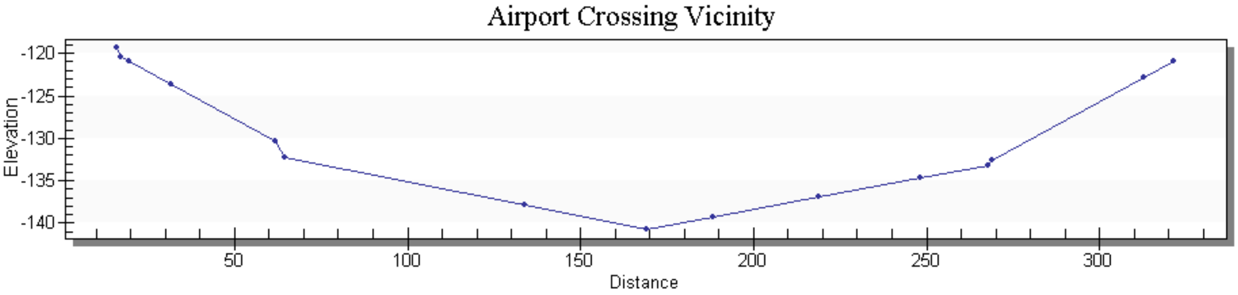


Figure 6. Section view of the proposed channel beneath the Airport Boulevard Bridge

Plan and section views of the preferred alternative in the reach between the Airport Boulevard and UPRR bridge crossings are shown in Figures 7 and 8, respectively. Along this reach, the existing channel bottom will be lowered approximately 5 feet. The proposed cross-section includes a 5-ft deep low flow channel centered between the channel banks. The top width of the channel through this reach is approximately 300 ft.

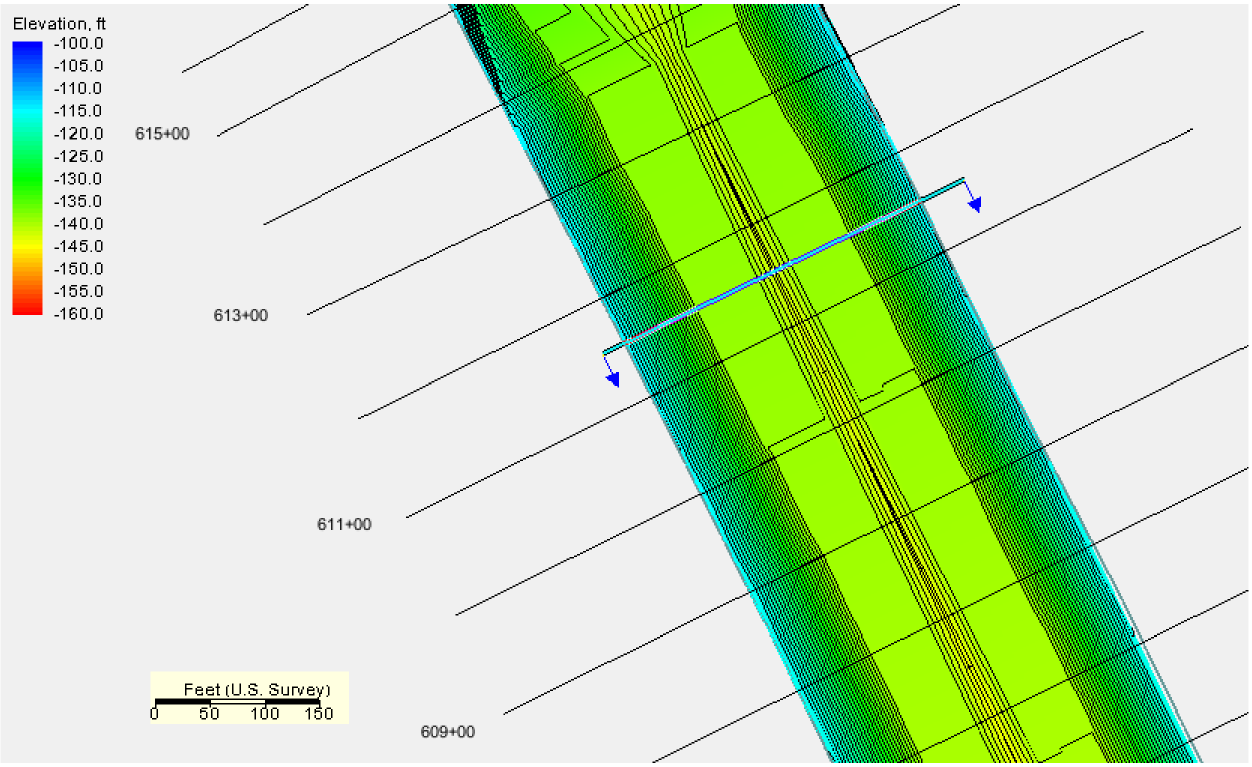


Figure 7. Plan view of the proposed channel between the Airport and UPRR crossings

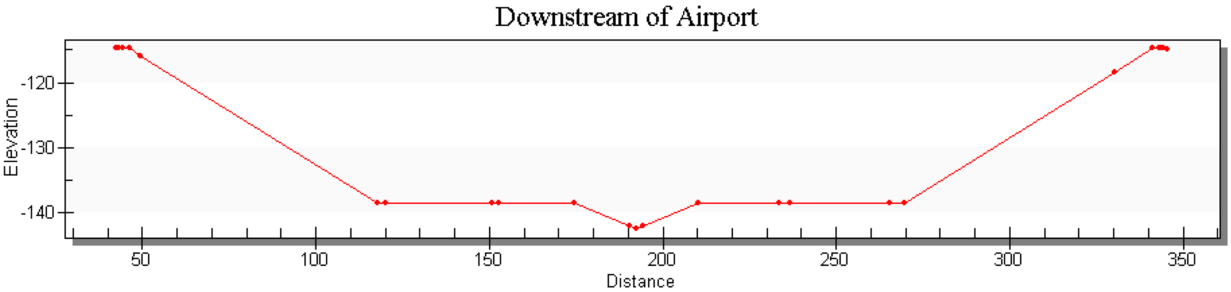


Figure 8. Section view of the improved channel between the Airport and UPRR crossings

A fully-lined cross-section of varying dimensions is proposed for the reach through the existing UPRR crossings and extending downstream of the Highway 111 bridge. A rectangular low flow channel is proposed through this segment of the study reach to maximize flow conveyance capacity through the existing cluster of bridge supports. The top width of the proposed section varies from about 230 to 350 ft. The slope of the invert is 0.002. Plan and section views of this portion of the project reach are shown in Figures 9 and 10, respectively.

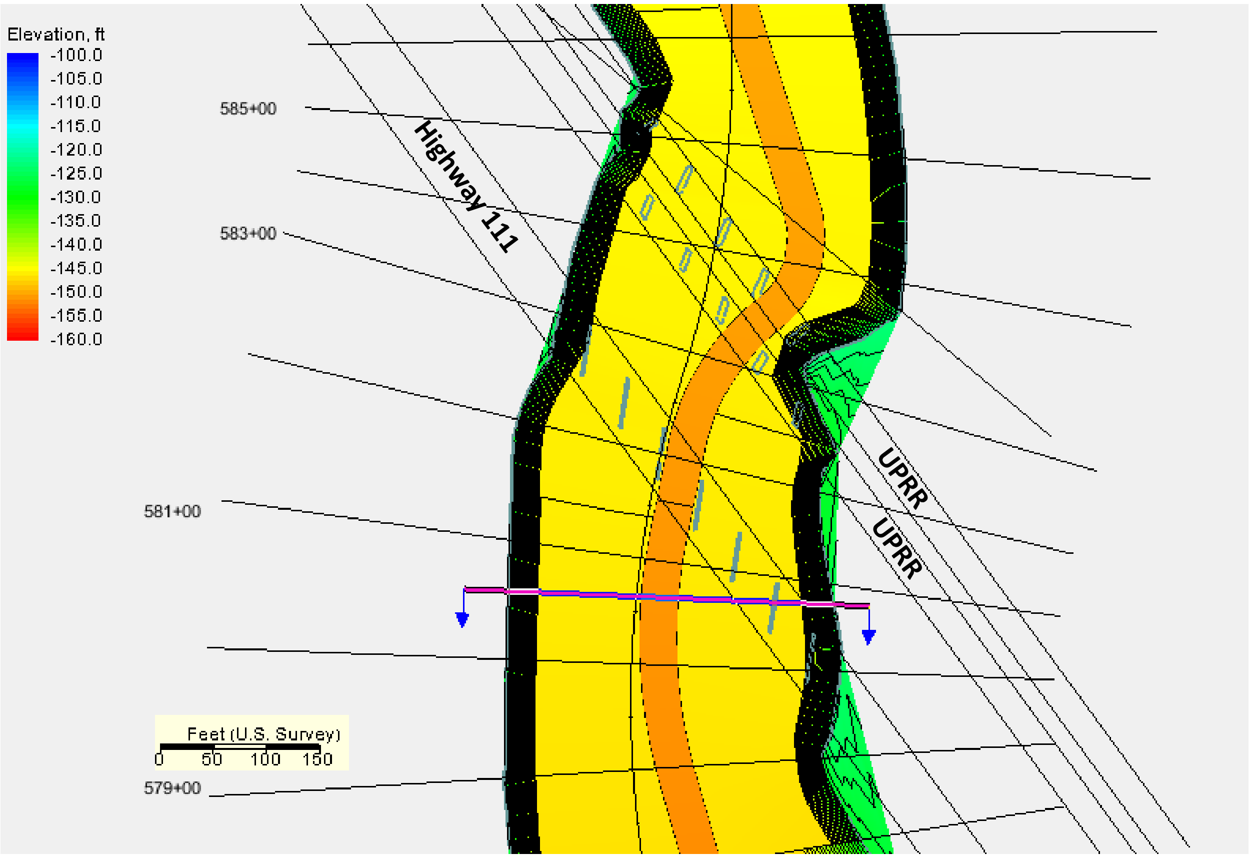


Figure 9. Plan view of the proposed project in the UPRR/Hwy 111 crossings reach

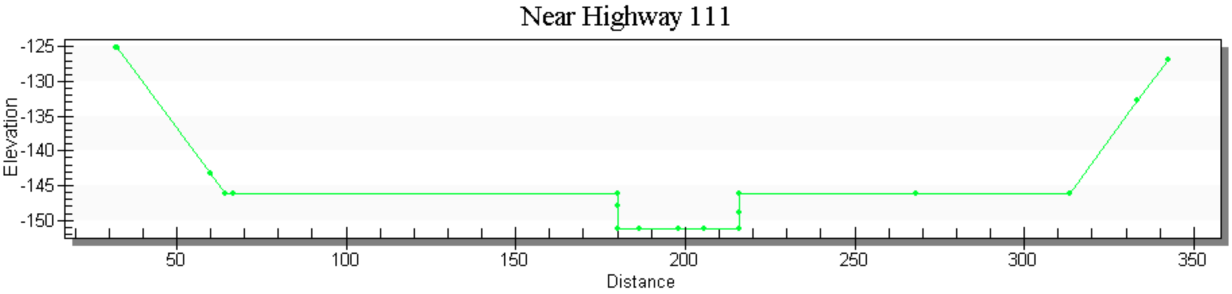


Figure 10. Typical section of the proposed channel in the UPRR/Hwy 111 reach

Downstream of the RR-Highway 111 reach, the proposed channel invert will be lower compared to existing conditions and the channel section will be similar to that of the upstream reach, but the invert will remain unlined. A trapezoidal low flow channel will be constructed midway between the lined channel banks. The existing drop structure at the downstream end of this reach will be modified (lowered) to conform to the projected improved invert slope. Project improvements will extend downstream of the existing Thermal Drop Structure to Station 567+00. Figures 11 and 12 present plan and typical section views of this portion of the project.

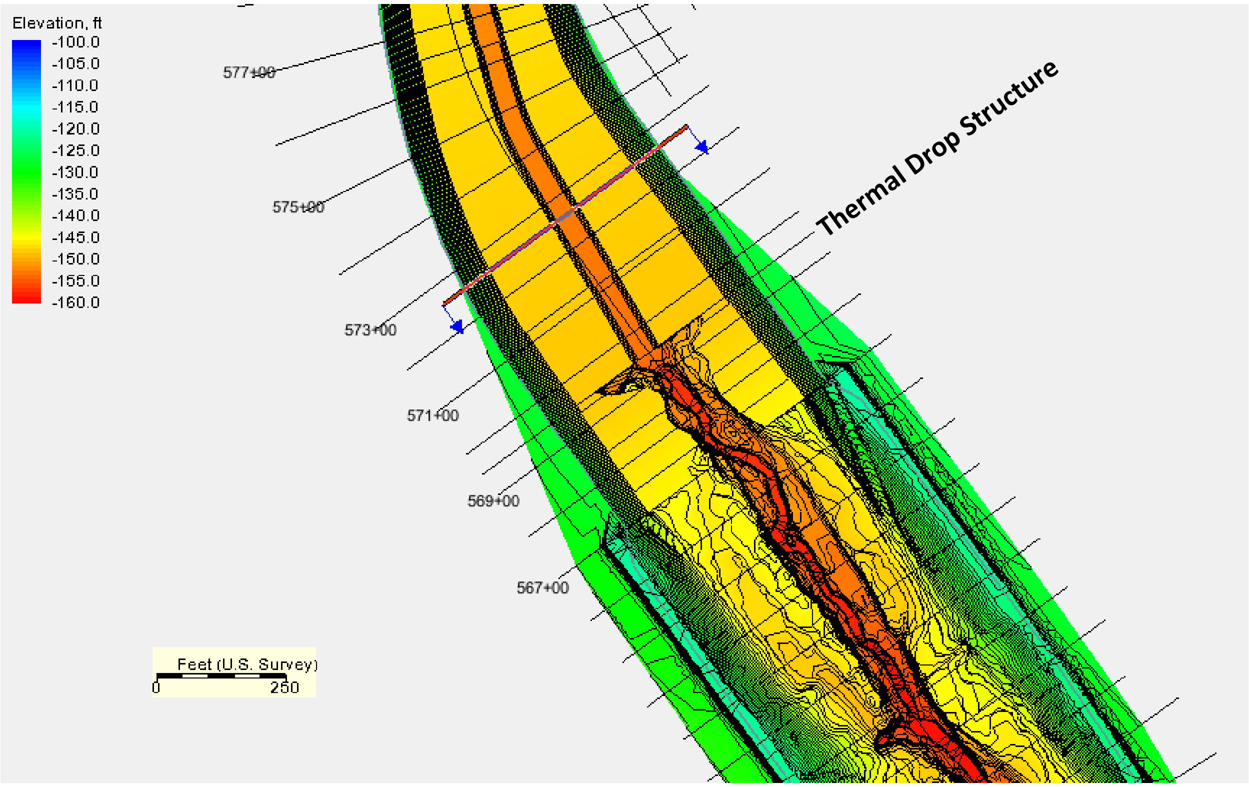


Figure 11. Plan view of the proposed channel near the downstream end of the project

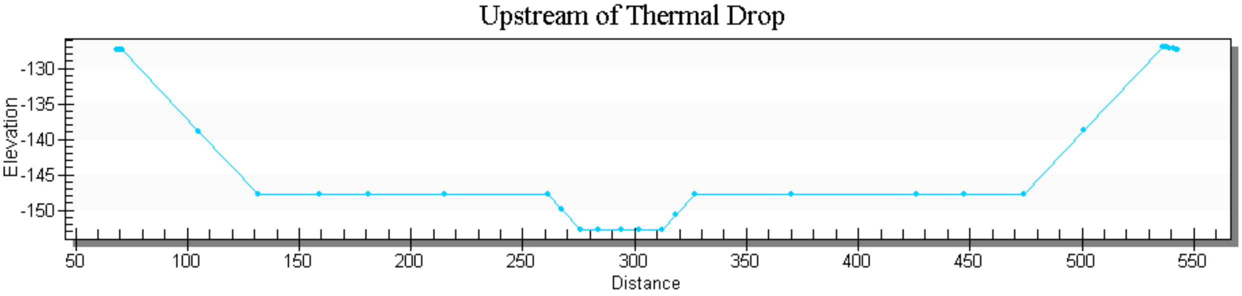


Figure 12 Section view of the proposed project upstream of the existing Thermal Drop Structure.

4. Hydraulic Analysis

Hydraulic analysis of the preferred alternative was conducted using the SRH-2D model developed by the US Department of Interior, Bureau of Reclamation. The model domain extends from approximately 400 feet upstream of Avenue 52 to 2,300 feet downstream of the Thermal Drop Structure. The model is geo-referenced to the NAD83 California State Plane Zone 6 coordinate system, and all elevations are in the North American Vertical Datum of 1988 (NAVD88).

Two-dimensional (2D) analyses were deemed necessary for the project reach due to the complexities of the channel alignment and bridge crossing structures, particularly in the vicinity of the existing UPRR crossings. Support structures in this vicinity are not streamlined to the flow direction and disrupt the flow pattern. Bridge losses through this reach of the channel were more reliably assessed through application of the 2D model than would be with a one-dimensional (1D) model.

1D models (HEC-RAS) of the project reach were also developed as a check on the 2D modeling, and for additional sensitivity analyses. The model input parameters and key assumptions are discussed below.

4.1 Model Parameters

4.1.1 2D Model Mesh

The geometry for the 2D hydraulic model was developed from Computer-Aided Design (CAD) surfaces representing the proposed geometry. Graffiti barriers (3:1 fill slopes that cover the concrete slope protection) were assumed to be in place along the reaches of the proposed project with unlined inverts. A computational mesh with 370,562 elements was developed using Aquaveo's SMS software. Bridge structure supports were modeled as voids in the model mesh. The locations and dimensions of bridge structure supports were developed from aerial photos, bridge plans, and survey information developed by WSP. Node spacing of the computational mesh varied from 2 feet to 20 feet to capture the topographic variation of the domain. Larger node spacing had initially been applied but was found to create instabilities, particularly along the wet-dry boundary of the simulation. The node spacing was densified along the domain boundaries and at areas of abrupt topographic change in a trial-and-error process to enable a stable solution set. The final computational mesh is shown in the vicinity of the UPRR and Highway 111 crossings in Figure 13.

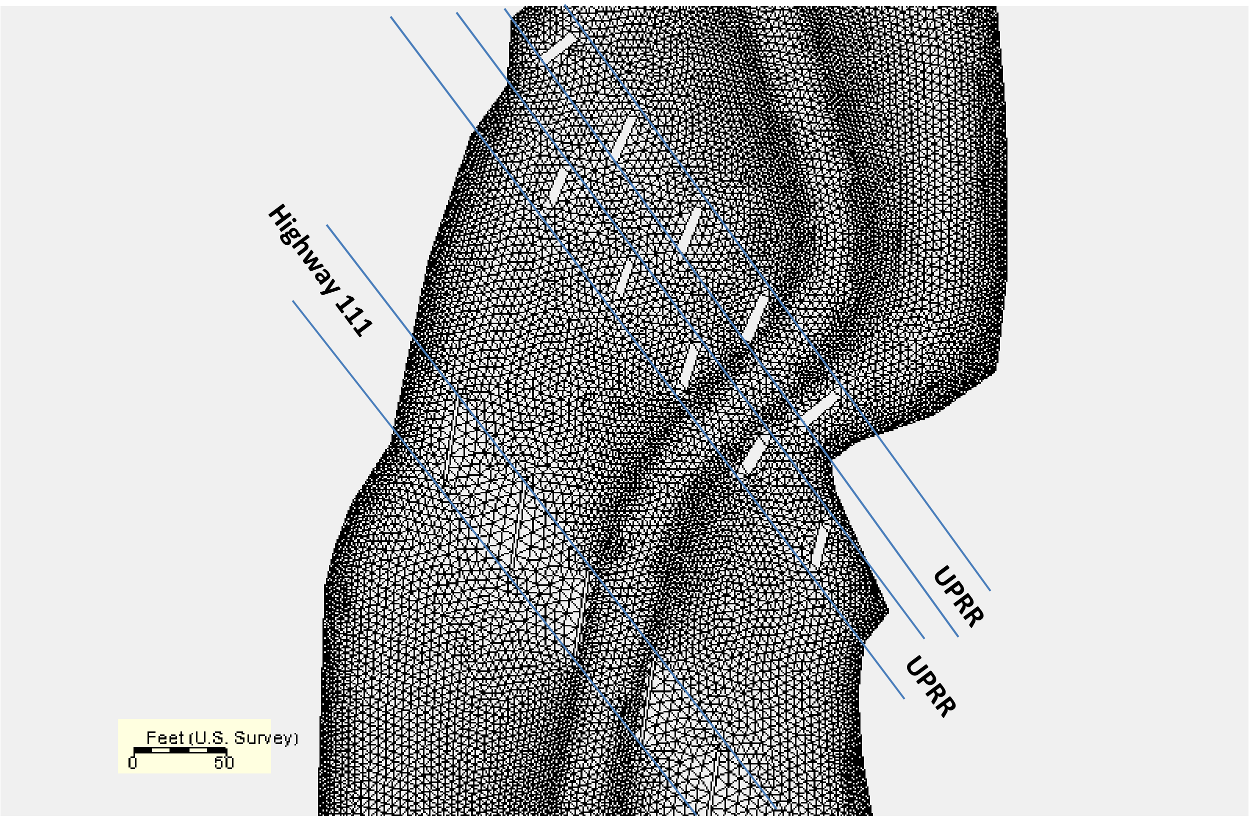


Figure 13. 2D model mesh in the vicinity of the UPRR and Hwy 111 crossings

4.1.1 1D Model Configuration

The HEC-RAS models of the project reach were developed from the same geometry used for development of the 2D model mesh. Cross-sections were placed at a nominal spacing of 500 feet, with additional detail in the vicinity of the bridge crossings. Bridge geometric information was obtained from the existing condition hydraulics model (NHC 2012a) and survey information developed by WSP. The cross-sectional layout of the 1D model is shown in Figure 14.

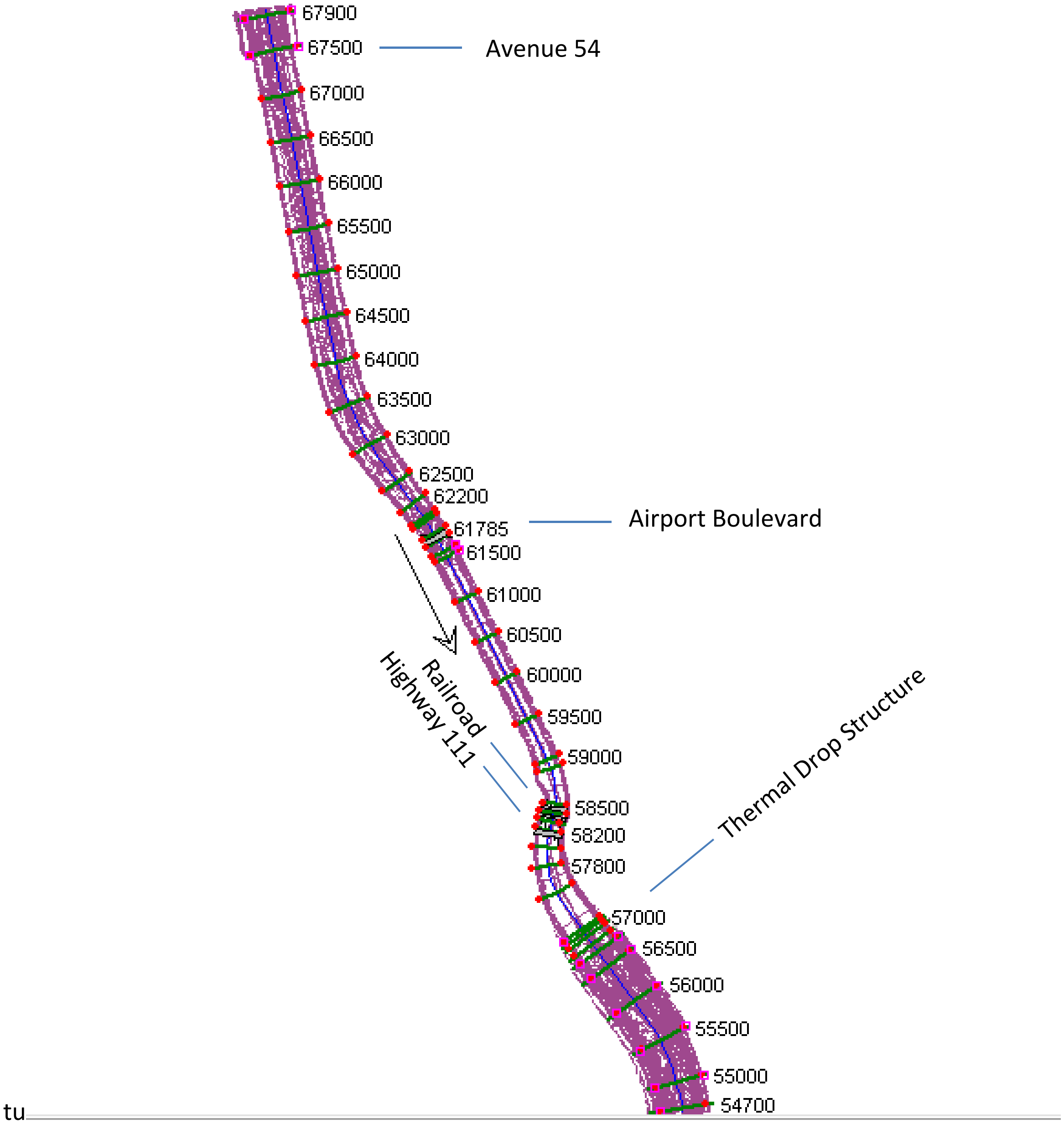


Figure 14. HEC-RAS workmap

4.1.2 Channel Roughness

Channel roughness values (Manning's roughness coefficients) depend on several factors including character and size of bed material, bed topography, obstructions, channel shape and planform, character and density of vegetation, and water stage. There are a number of methods allowing approximate estimation of roughness of streams based on their visual appearance. These methods are subjective, and typically give a wide range of possible roughness values.

The roughness values applied in the 1D and 2D models of the project reach are summarized in Table 1. The selected roughness values applied in the models are based on observed channel conditions discussed in NHC (2015), and NHC's understanding of CVWD's anticipated channel maintenance practices. These selected channel roughness values assume the following maintenance practices:

- Occasional maintenance of the low flow channel to remove large diameter vegetation will be allowed
- The channel terraces and levee banks are maintained on a regular basis
- Sediment accumulated in the lined reaches will be cleared annually prior to the flood season.

Support for the values listed in Table 1 for the unlined portions of the project (using a resistance components-based approach) is included in Appendix A. The value of 0.019 selected for the fully-lined reaches of the project was based on an assessment summarized in Tetra Tech's analysis of the project reach (Tetra Tech, 2014), which assumed a moving layer of bed material over concrete lining. The sensitivity of the design to these factors is addressed in Section 4.4.1 of this report.

Table 1. Manning's roughness coefficients.

Area	Manning n Value
Low flow areas between maintained terraces	0.060
Graffiti barriers	0.028
Maintained unlined terraces	0.028
Fully-lined reaches	0.019

4.1.3 Hydraulic Boundary Conditions

The hydraulic boundary conditions for both the 1D and 2D models consist of upstream inflows and downstream water surface elevations. The upstream inflows in the model were the 100-year peak flow of 39,000 cfs developed in USACE (1980). This inflow is consistent with the previous NHC analysis (2012a) of the project reach. The downstream model boundary was set at a known water surface elevation (-137 feet), obtained from the same NHC analysis (2012a).

4.2 Computational Parameters

The 2D simulation was run in unsteady mode with a constant discharge (39,000 cfs), with the kinetic energy turbulence model applied. The simulation was run until steady results were obtained throughout the domain. A 0.2 second time step was required in the simulation to enable the simulation to converge.

The HEC-RAS models were run in steady state mode. The mixed flow regime was applied to check the potential for supercritical flow, though the results indicated that subcritical conditions were maintained throughout. Default calculation tolerances were used in the model.

4.3 Model Calibration

Model calibration typically involves adjusting roughness coefficients and other model parameters to obtain reasonable agreement with measured high-water data. Due to the absence of measured high flows and corresponding high-water marks for this reach of the CVSC, the present model could not be calibrated. The bridge losses in the 1D model were calibrated using the results of the 2D analysis, and sensitivity analyses were conducted to determine the effect of variable roughness and sediment transport on the computed water surface elevations (described in Section 4.4.1 of this report).

4.4 Water Surface Profile

The SRH-2D model was used to compute the hydraulic performance of the project reach under peak 100-year flow conditions. The model results were used to establish the top of lining elevations along the project reach. It should be noted that the configuration used for the final proposed with-project configuration was the culmination of numerous trials to develop a workable solution that met the design objective, which involved maintaining the existing bridge structures and limiting the extent of channel bottom lining to minimize environmental impacts

Computed water surface elevations along the west and east banks and the centerline of the project reach are summarized in Appendix B. Average hydraulics from the 1D model are also presented. The 2D (centerline) and 1D (average) 100-year water surface profiles are compared in Figure 15. The profiles are very similar, with significant deviations only in the vicinity of the bridge crossings, where the 2D simulation results indicate that the hydraulic characteristics vary significantly across the width of the cross-section, as shown, for example, in Figure 16.

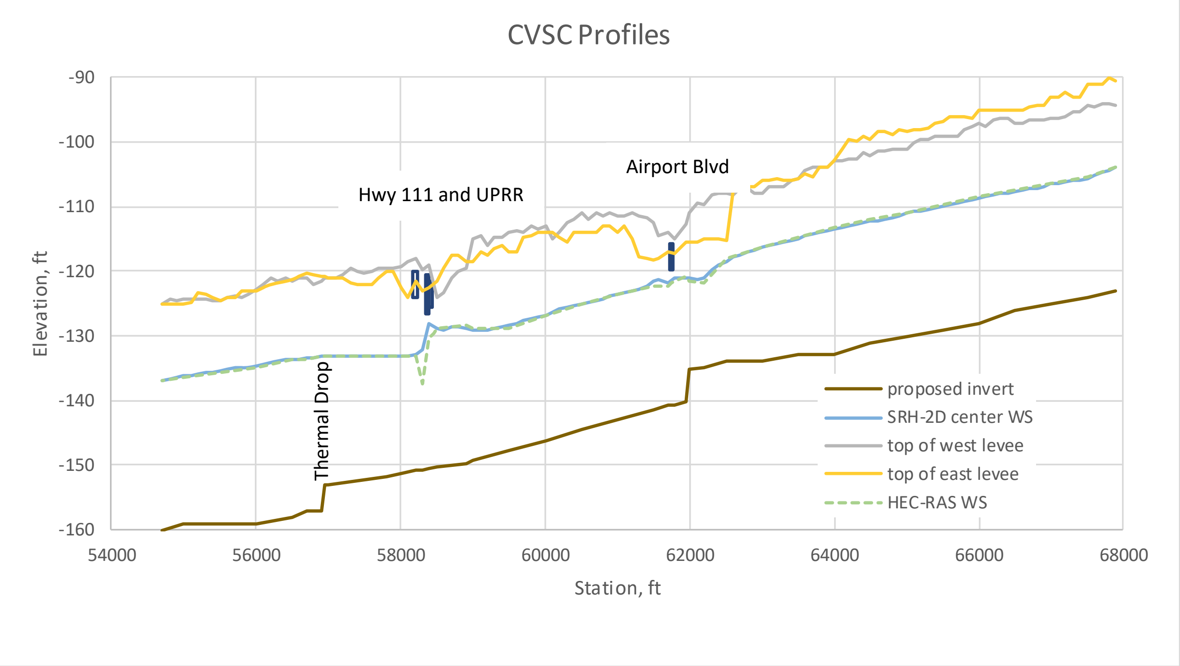


Figure 15. Computed 100-year water surface profiles.

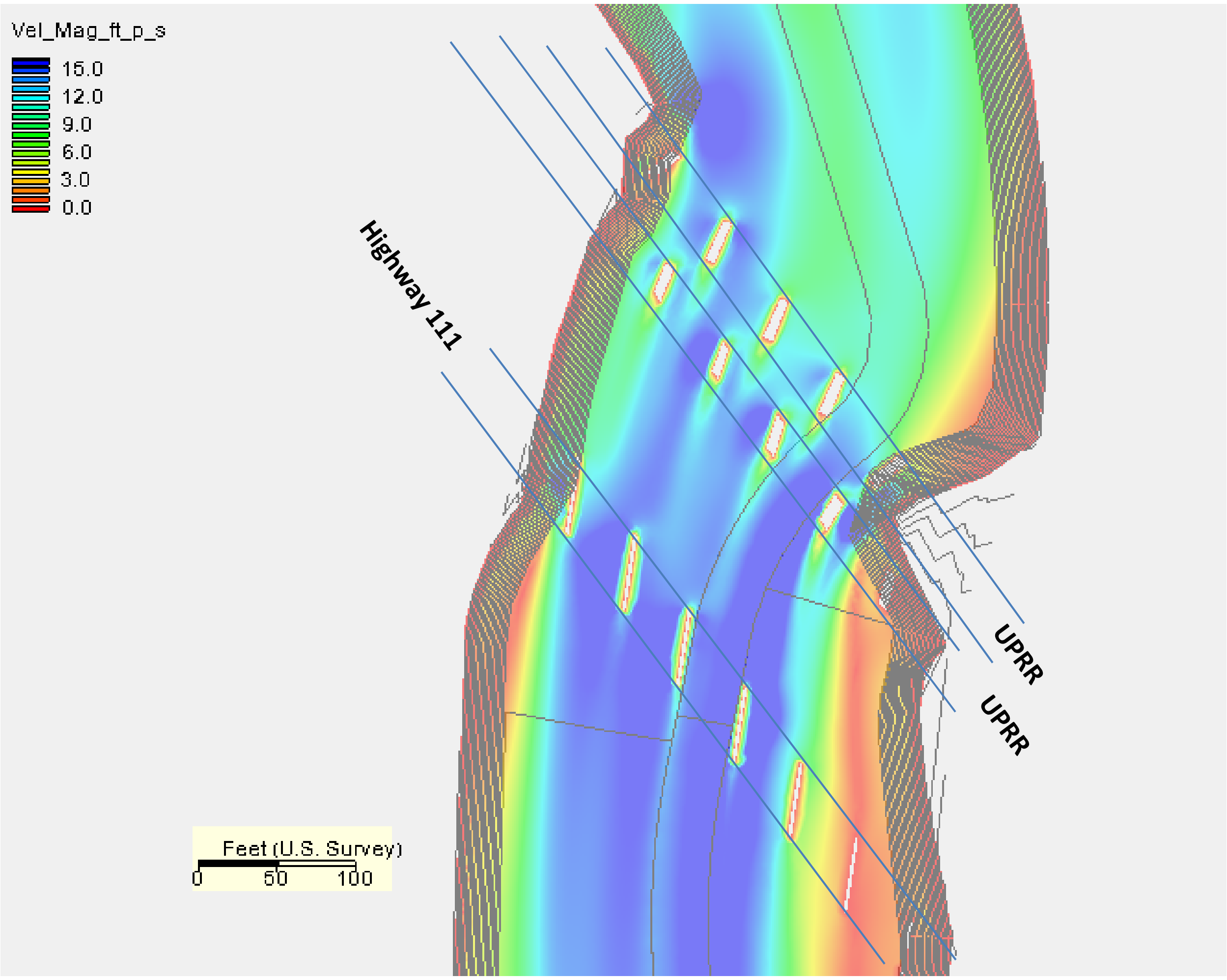


Figure 16. Computed velocity variation in the vicinity of the UPRR and Hwy 111 crossings (results from 2D simulation)

Locations of leveed conditions along the project reach (defined as areas where the design water surface elevation is higher than the ground elevation outside of the channel and adjacent to the levees) were determined through comparison of the computed centerline profiles to available topography. The results are presented in Table 2. As shown in Figure 17, at some locations the ground surface immediately adjacent to the existing levee includes a local depression that extends below the computed water surface level. These locations are noted in Table 2, but may not be considered significant for leveed condition determination. If these locations are excluded, then the only locations remaining in leveed condition with project will be located along both banks between Stations 648+00 and 650+00, and along the right bank between Stations 634+00 and 636+00. The plan set included in Appendix C shows profile views of the design water surface, with indications of where leveed conditions will remain along the project.

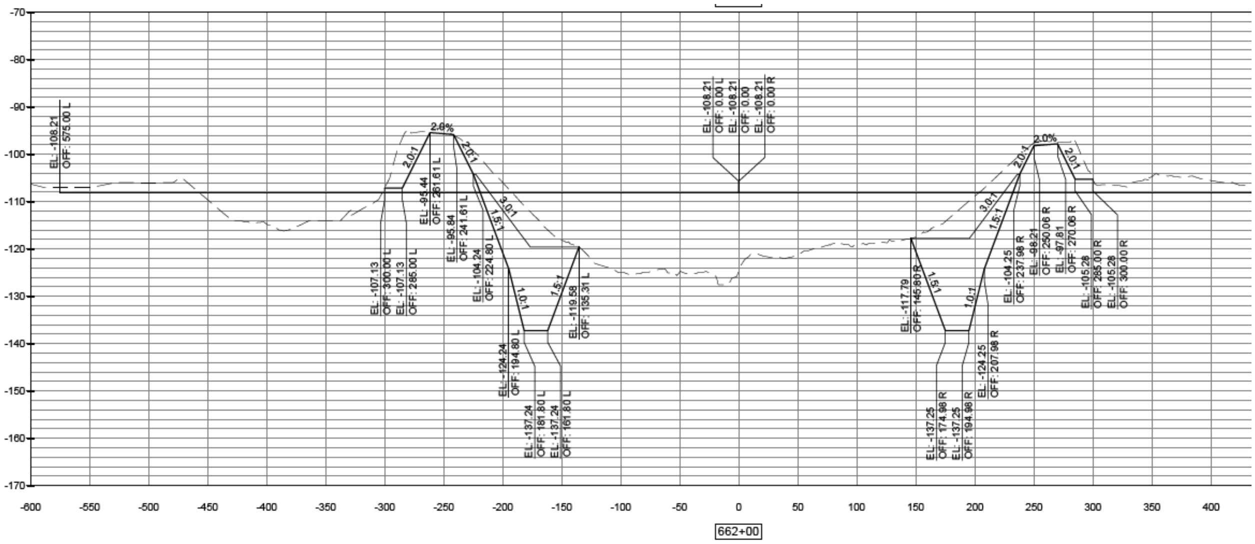


Figure 17. Cross- section showing local depression on the left side

Table 2. With project reaches with leveed condition

Vicinity (Stations)	Left side	Right side
674+00 to 675+00	no	no
652+00 to 672+00	local	no
648+00 to 650+00	yes	yes
642+00 to 646+00	no	no
640+00	no	local
638+00	no	no
634+00 to 636+00	no	yes
628+00 to 632+00	no	local
527+00 to 626+00	no	no

4.4.1 Sensitivity Analysis

As a first step in sensitivity analysis, an evaluation of the capacity of the proposed project to manage sediment transport through the project reach was made using the quasi-dynamic sediment transport routines available in HEC-RAS. The analysis was similar to that conducted by Tetra Tech (2014) in their alternatives evaluation study of the project reach. Tetra Tech had applied both the Toffaleti and Yang sediment transport equations to the with-project condition they evaluated, and indicated some likelihood for sediment accumulation in the lined reaches they were considering at the time. The Yang sediment transport equation provided the more conservative results, and was selected for use in the analysis conducted for this study.

Historical invert profiles along the project reach are compared in Figure 18. The 1995 profile (Bechtel 1995) seems to indicate that some form of crossing once was present at the upstream end of the project reach, possibly an extension of Avenue 54. This previous crossing had disrupted the channel profile, with accumulation upstream and scour downstream, compared to the 1971 as-built profile (CVWD, 1971). Accumulation was also evident in the vicinity of the Airport Boulevard and UPRR crossings between 1971 and 1995. The 2016 profile indicates a slight accumulation of material along most of the project reach in the period since the 1995 Bechtel study, with some scour evident downstream of the Thermal drop structure.

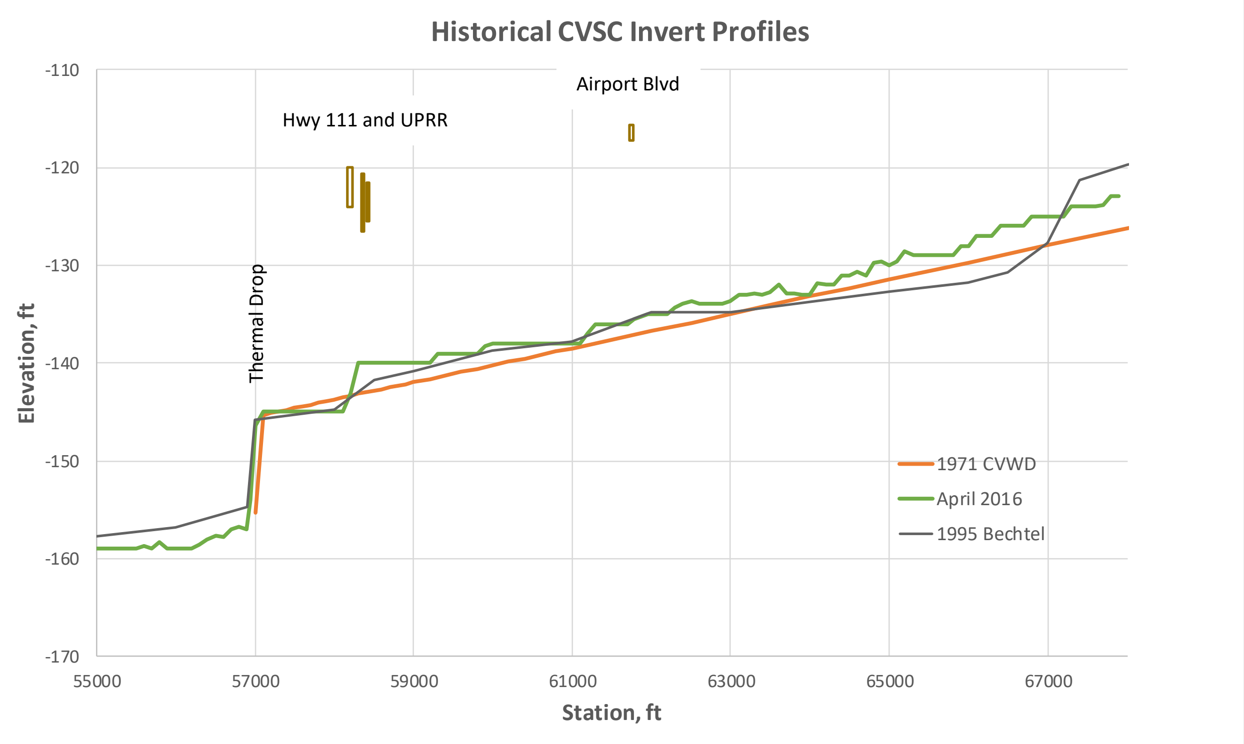


Figure 18. Historical invert profiles though the project reach

Sediment gradations used in the sediment transport analysis were obtained from the in-channel borings conducted by GENTERRA (2016). The size distributions from the GENTERRA samples are contrasted with the relevant samples in the Tetra Tech (2012) study in Figure 19. From these samples, a composite size distribution was developed for use in the sediment transport analysis.

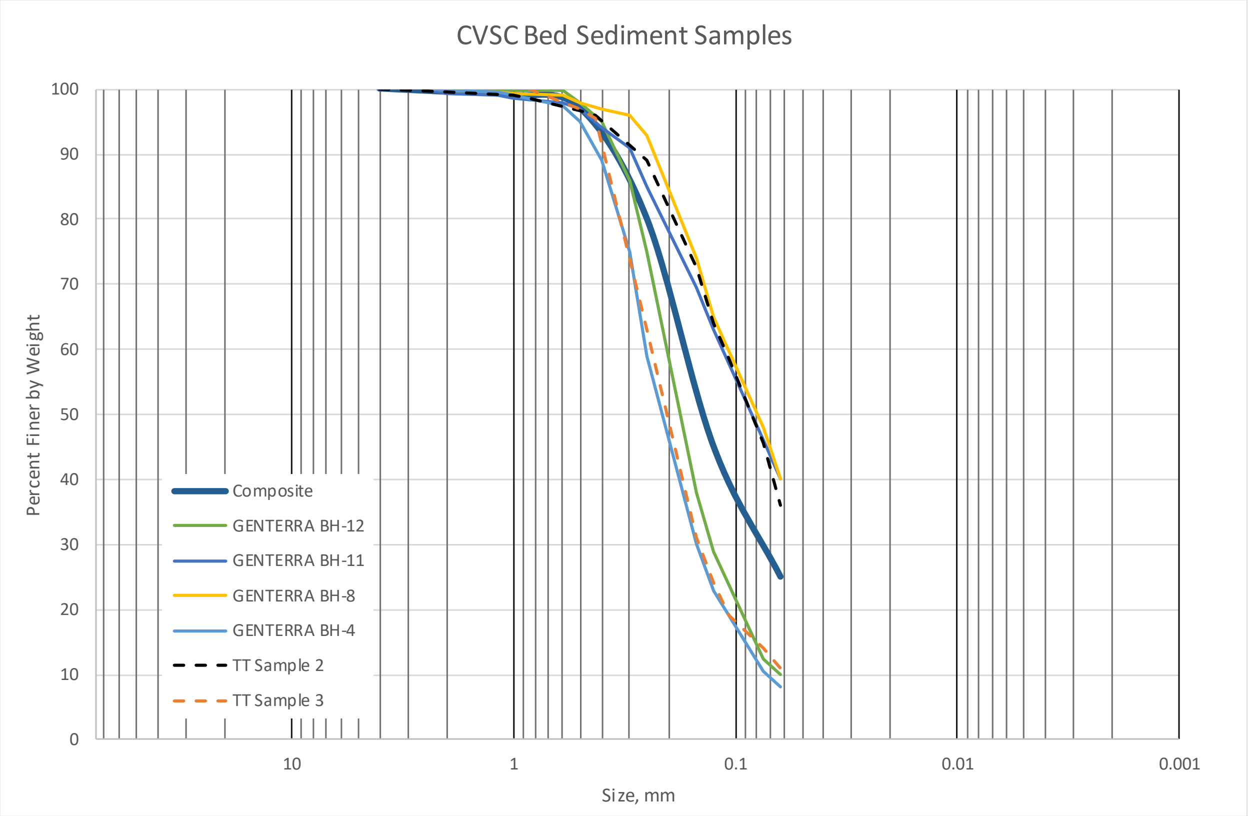


Figure 19. Sediment samples along the study reach

The 100-year hydrograph used for the sediment routing analysis was obtained from the Tetra Tech (2014) study (see Figure 20). This hydrograph was scaled to represent 10-year and 2-year flood conditions (for an analysis of more frequent flooding events as well) using information obtained from COE (1980) and AEI-CASC (2010).

The post-flood invert profiles computed for the project reach under the 2-year, 10-year and 100-year flood hydrographs are shown in Figure 21. The simulations indicate that accumulation could occur within the proposed lined reaches in the vicinity of the Airport Boulevard and UPRR crossings. Deposition is also expected in the reach downstream of the lowered Thermal Drop Structure. Maximum accumulation depths range from over 5 feet under 100-year flood conditions, to about 1 foot under 2-year flood conditions. Bed lowering (incision) of up to 2 feet of the channel invert profile was computed in the reach upstream of Airport Boulevard, and in the reach between the bridge crossings.

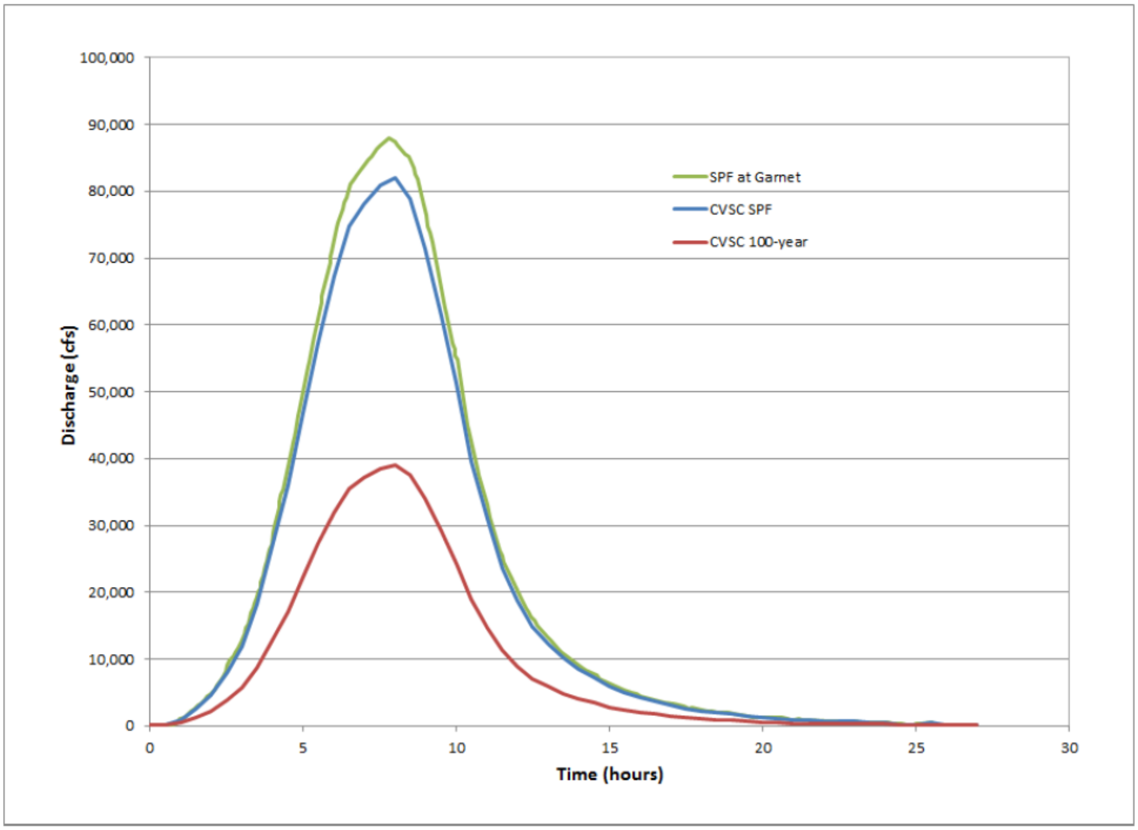


Figure 20. Flood hydrographs, from Tetra Tech (2014)

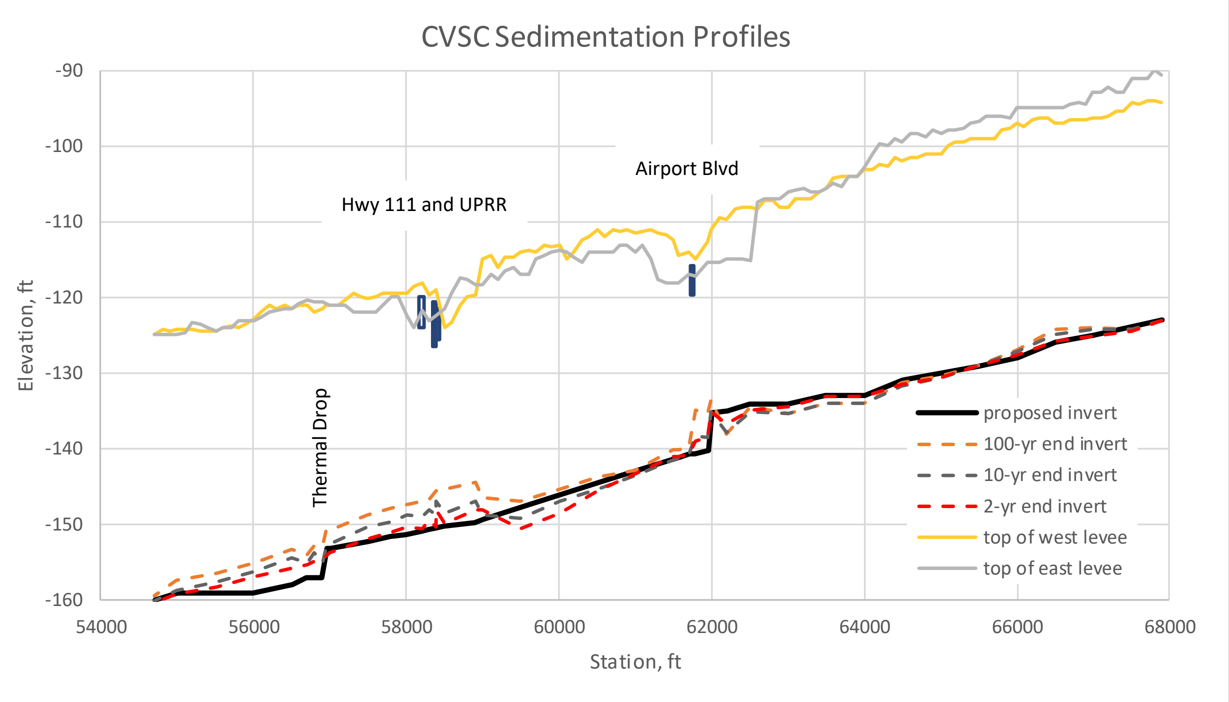


Figure 21. Computed post-flood sedimentation profiles, with-project conditions

The computed erosion and sedimentation along the proposed channel also has an effect on peak water surface profiles, as indicated in Figure 22. Computed with-sedimentation 100-year water surface profile are significantly higher than the clear water results, though flood levels are still contained by the existing levee heights and proposed lining elevations. It should be noted that the results of the sediment transport analysis are approximate, and very sensitive to the transport equation applied. They do indicate, however, that maintenance of the lined reaches of the project will be required to maintain conveyance capacity, and that more than 3 feet of freeboard could be required along the project as an allowance for potential intra-flood sedimentation.

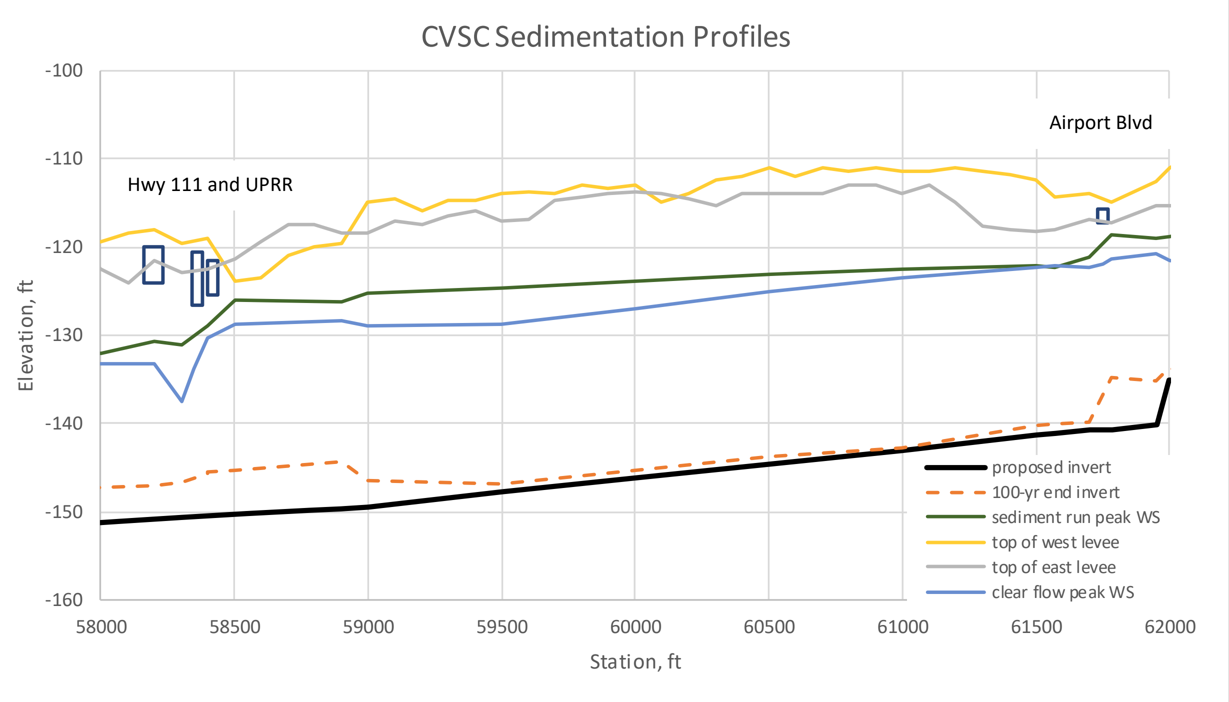


Figure 22. Computed 100-year flood and sediment profiles

The sediment transport results indicate that the Manning n value for the fully lined reaches of the project is an additional factor of uncertainty along the project. With accumulation of sediment, n values higher than 0.019 may be applicable in these reaches. Figure 23 illustrates the sensitivity of the computed 100-year water surface to roughness value adjustment, with the original computation results compared to those computed assuming n = 0.028 in the lined reaches. The higher n increases water surface elevations, most notably upstream of Airport Boulevard. Note that higher water surface elevations upstream of the Airport crossing would tend to raise the minimum scour elevations in this reach.

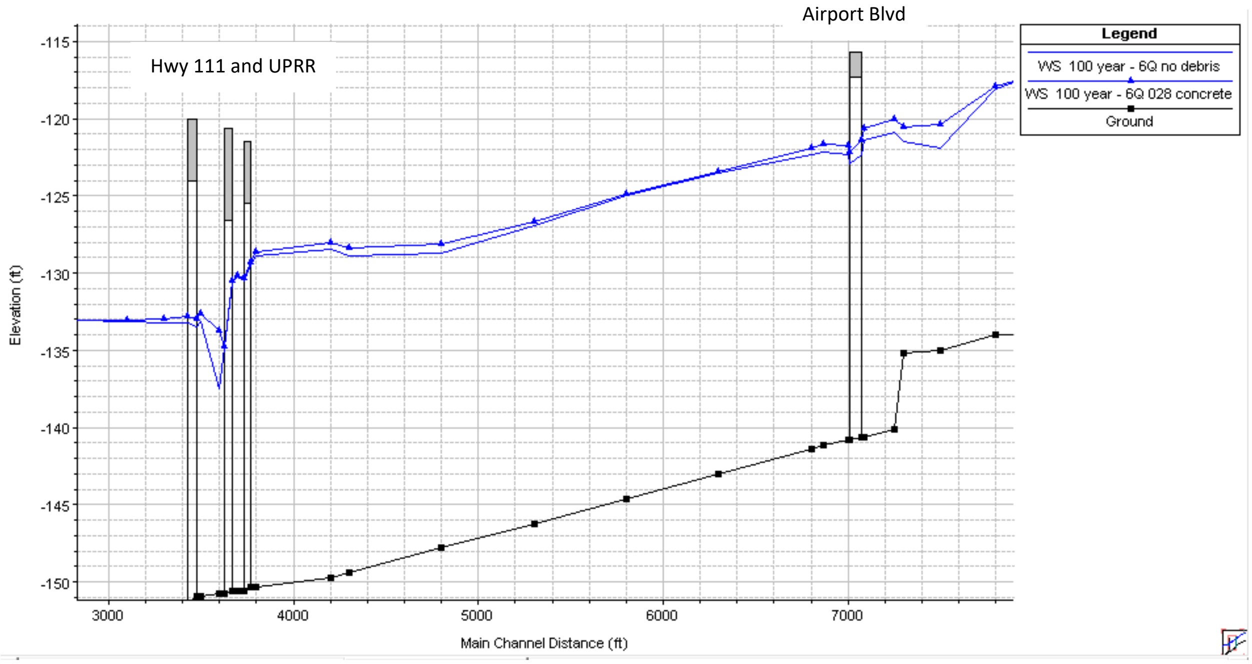


Figure 23. Effect of higher roughness values applied in the lined reaches of the project on computed water surface elevation

An additional factor of uncertainty is the potential for debris accumulation on the existing bridge support structures. Widening the support piers by 2 feet on each side to account for debris raises water surface elevations by approximately 2 feet upstream of the crossings, as shown in Figure 24.

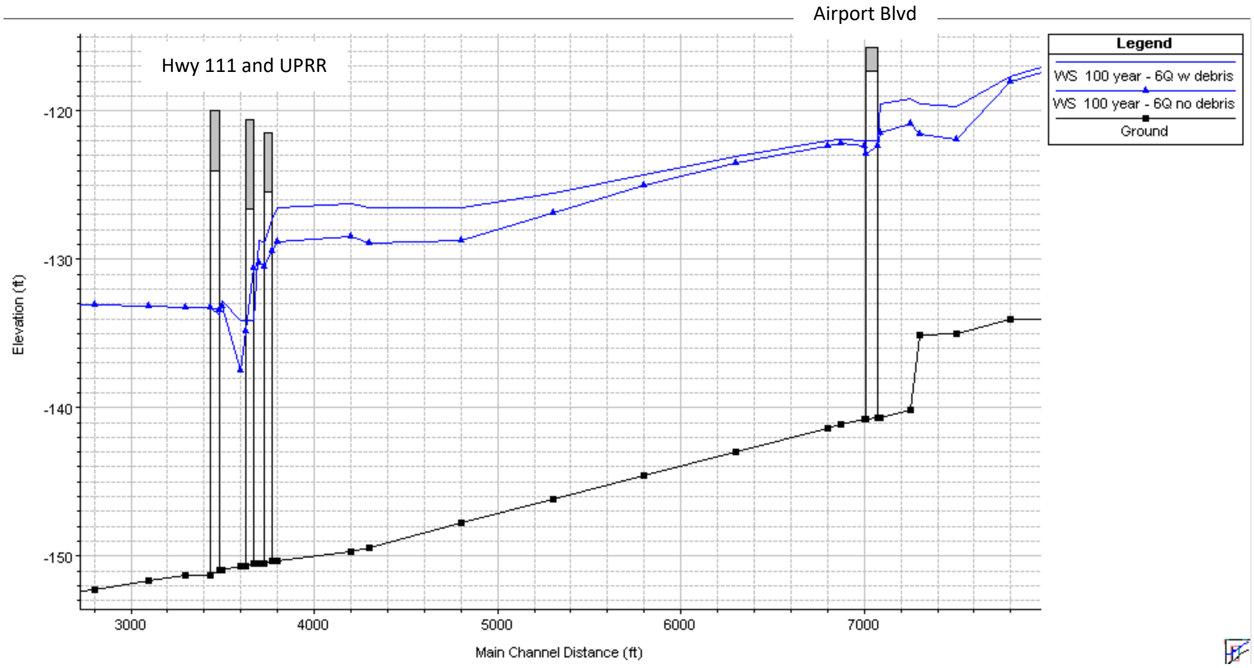


Figure 24. Effect of pier debris accumulation on computed 100-year water surface profiles

Finally, the degree of vegetation growth in the low flow channel and its permanence during a design event is an additional item of uncertainty. Figure 25 compares the computed 100-year water surface profile along the project under design conditions to a reduced roughness condition of n = 0.028 in the low flow portions of the unlined reaches. This change in resistance would result in significantly lower water levels along the project reach, particularly upstream of Airport Boulevard and downstream of Highway 111.

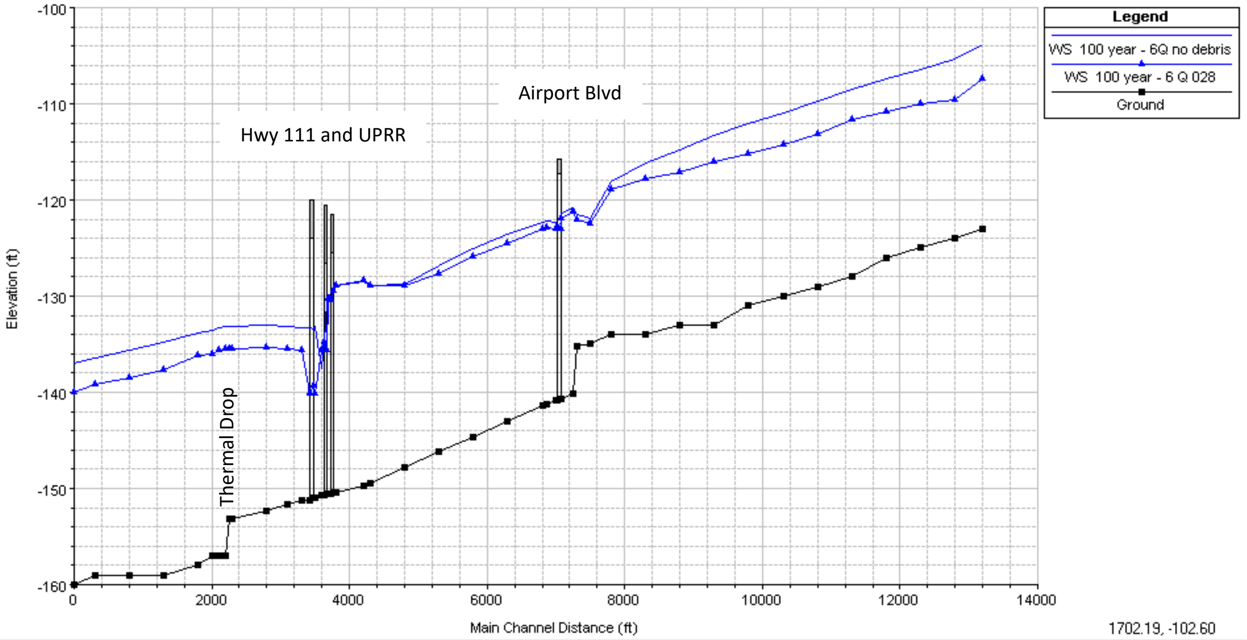


Figure 25. Effect of lower roughness in unlined sections on computed 100-year water surface profiles

Figure 26 provides a comparison of the computed 100-year water surface profile along the project under design conditions to an increased roughness condition of n=0.100 in the low flow portions of the unlined reaches. Up to 3 ft of freeboard loss along the project reach would be expected with this increased resistance condition.

The sensitivity tests show a high variability in potential water surface elevations for varying sedimentation, debris accumulation and roughness conditions along the project reach. The results of the sensitivity tests demonstrate the importance of regular maintenance of the flood channel to maintain its flood conveyance capacity, and indicate the importance of the freeboard allowance to account for these performance uncertainties.

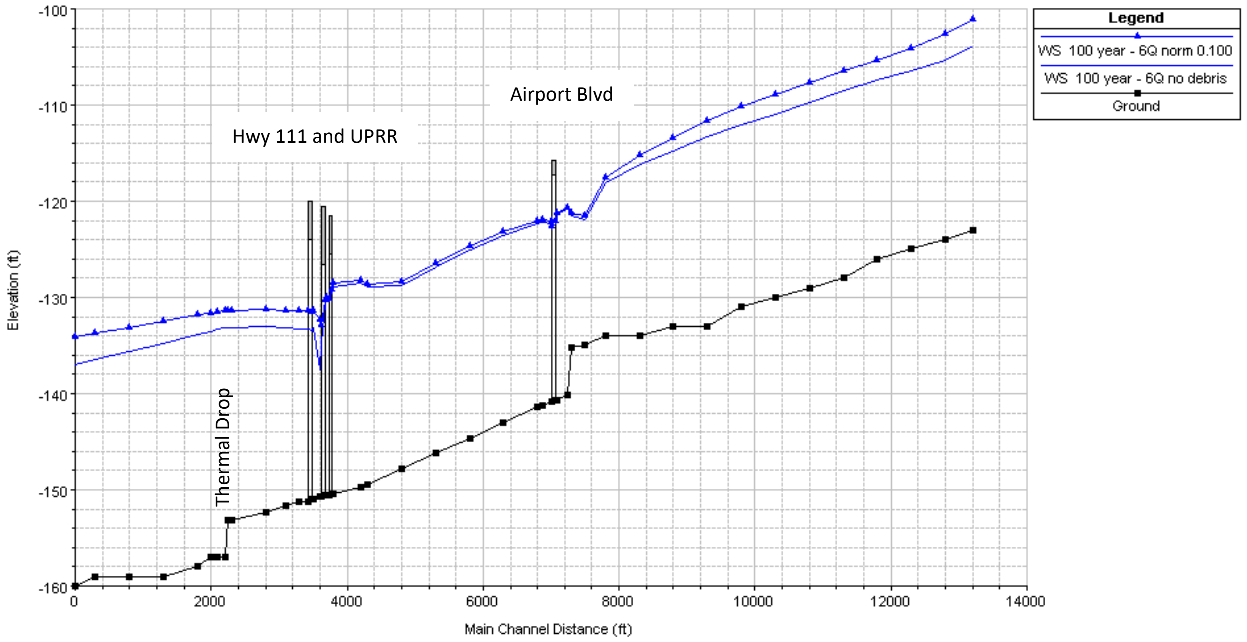


Figure 26. Effect of higher roughness in unlined sections on computed 100-year water surface profiles

4.4.2 Superelevation

Superelevation of the water surface occurs at channel bends. Superelevation was observed in the two-dimensional computational results, and contrasted with the results expected using an equation developed by the Corps of Engineers, as summarized in Ref. 1:

$$\Delta Y_b = C\,\frac{V^2 w}{gr}$$

with:

ΔY_b = superelevation allowance
C = a coefficient that varies with flow regime and geometry
g = acceleration of gravity
V = velocity of upstream flow (ft/s)
w = channel topwidth
r = radius of curvature at channel centerline

The value of the coefficient C is 0.5 for subcritical flow.

Spot checking of the SRH-2D analysis results indicates that the simulated superelevation from the 2D results are comparable to those computed using the HEC-RAS output and the above equation. Therefore, superelevation was shown to be accounted for the in the SRH-2D model.

4.5 Toe-down Analysis

The calculation of scour is required for the design of the minimum toe elevation for the proposed bank protection.

The depths of general scour along the project reach were determined in accordance with the CVWD's DDM for scour calculations for bank protection. The 100-year event was used to calculate potential general scour magnitude along the project reach. As discussed in more detail below, scour magnitude under peak SPF flow conditions was also computed for comparison, considering the current CVWD policy for scour allowance near structures. A discussion with CVWD upon review of this document is recommended to define the standards for this project.

4.5.1 General Scour

The general scour depth was calculated using the Blench regime equation:

$$D_{fo} = (q_f^2/F_{bo})^{1/3} \qquad (1)$$

In this equation, D_{fo} is the regime depth (feet) below the design water surface, q_f is the unit design discharge (ft^2/s) calculated from the design discharge and water surface width, and F_{bo} is the zero-bed factor (ft/s^2), which is a function of the median grain size of the bed material. The 100-year flow is 39,000 cfs. The design surface widths were determined from the modeling results.

The median grain size of the composite sample developed for the project reach is 0.15 mm (see Figure 19, above). Based on this median size, F_{bo} is about 0.7 ft/s^2.

The maximum scour depth, d_s, was calculated by applying a Z-factor to the regime depth:

$$d_s = Z*D_{fo} \qquad (2)$$

In the above equation, Z varies depending on the general nature of the channel. For reasonably straight channels such as the unlined invert portions of the CVSC project reach, Z is commonly assumed to be about 1.25 and this factor was applied to calculate minimum scour elevations.

The unit discharge along the project reach varies significantly, due to the changes in the channel top width. The computed variation in channel top width under 100-year flood conditions is shown in Figure 27.

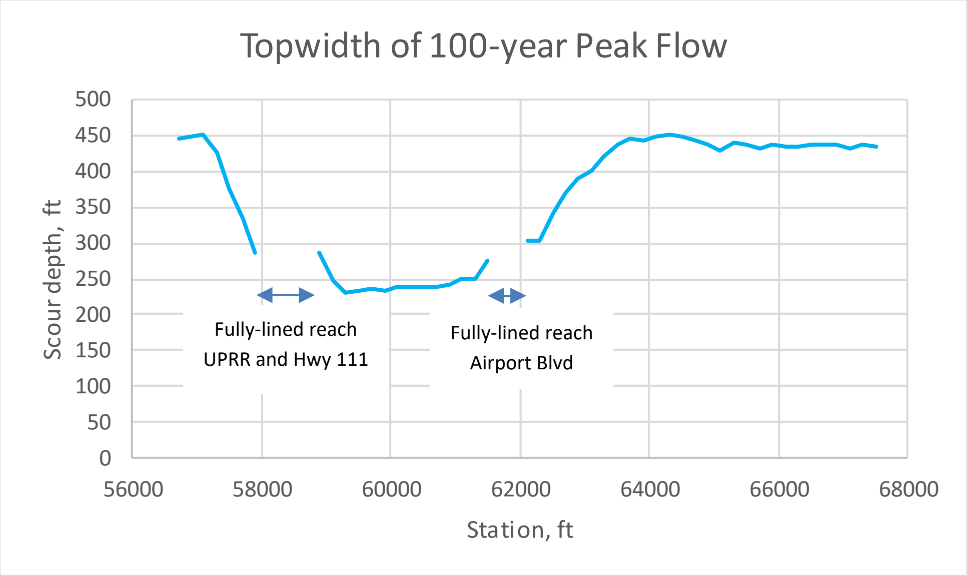


Figure 27. Top width variation, unlined invert reaches of the project channel

The computed scour depths (below the 100-year water surface) along the project reach are shown in Figure 28.

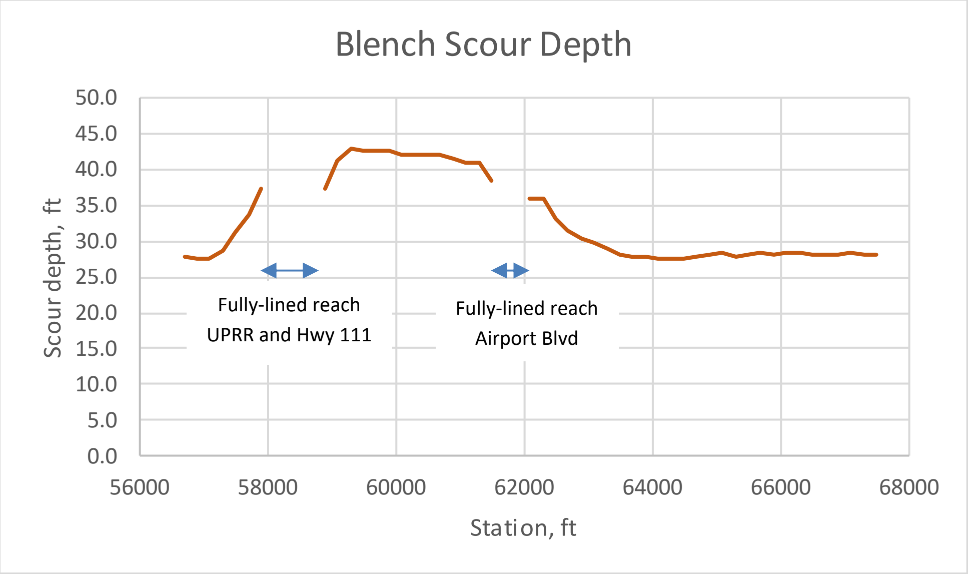


Figure 28. Computed 100-year Blench scour depth (below water surface), unlined invert reaches of the project channel

A profile view of the recommended toe-down allowances (minimum scour elevations) along the project reach is shown in Figure 29. The allowances are the same for the right and left banks.

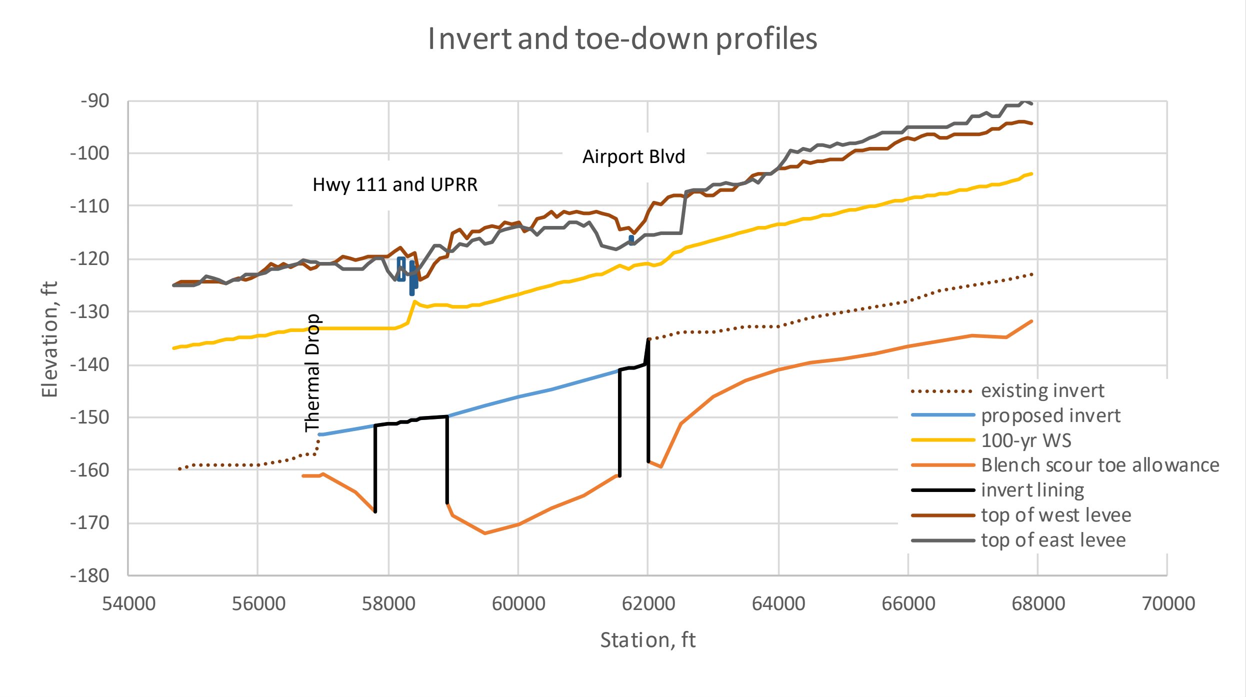


Figure 29. Computed general scour toe allowance profile for 100-year Event

A table of recommended toe elevations for the proposed bank protection is presented in Appendix B.

4.5.2 Local Scour

The project reach will include two fully-lined reaches – near Airport Boulevard and near the UPRR crossings – that will both require cut-off walls at their upstream and downstream ends. At both ends of both locations, it is recommended that the cut off walls extend at least to the elevation of the proposed toe down elevation for the adjacent bank protection, computed using the Blench scour equation (as shown in Figure 29). The upstream walls could experience less invert variability, since the upstream cut-off wall will act as a sediment ‘dam’, which will tend to stabilize the upstream reach. A downstream cutoff wall, however, can potentially turn into a drop structure, if the downstream channel degrades, exposing the wall as a discontinuity in the channel profile. An illustration of potential drop exposure downstream of the Airport Boulevard lining is shown in Figure 30. Note that at this location the potential drop in the downstream invert profile will be controlled by the lining proposed below the UPRR crossings. A horizontal slope projected upstream from the invert lining at the UPRR crossing would result in an 8.6-ft exposure of the cutoff wall at the downstream end of the Airport Boulevard lining;

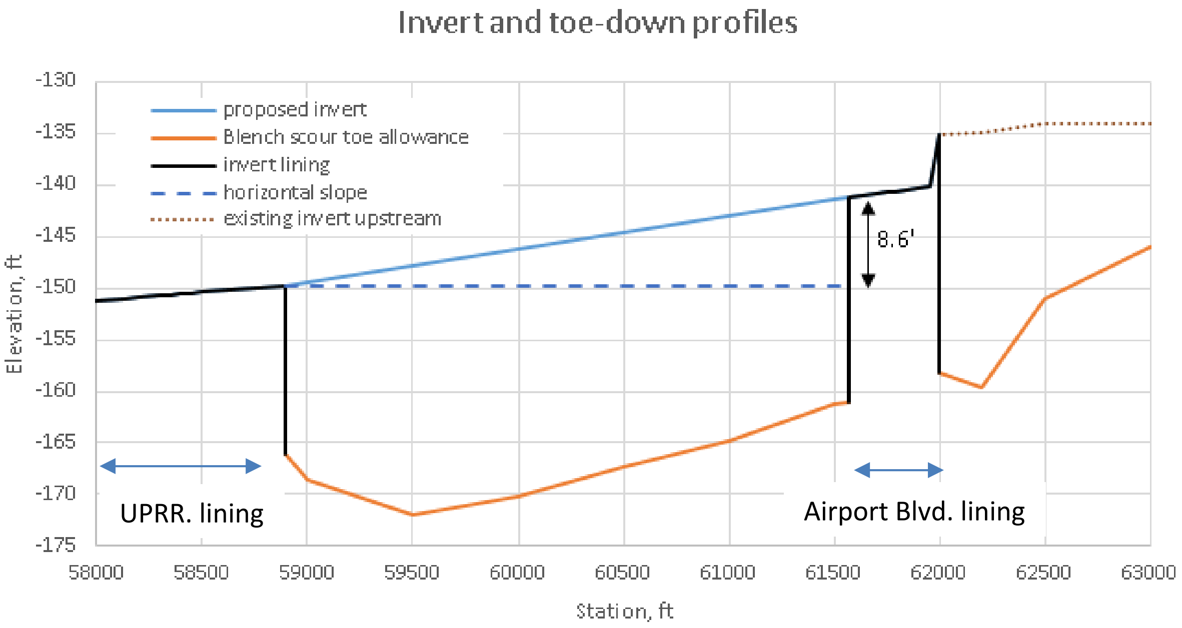


Figure 30. Potential cutoff wall exposure downstream of the Airport Boulevard lining

Downstream of the lined reach at the UPRR and Highway 111 crossings, the maximum potential change in the downstream invert level is less definitive. For comparison purposes, an illustration of an equivalent 8.6-ft drop at this location is shown in Figure 31. Note that the proposed channel excavation in the lower reach of the project will result in a drop in the low flow portion of the channel only, as illustrated in Figure 32. The average channel bottom will not see an abrupt drop in elevation at the downstream end of the project reach. For cutoff wall sizing purposes, an 8.6-ft allowance for drop exposure at this location, though somewhat arbitrary, appears adequate at this location, as well, particularly since the expected trend in the reach downstream of this location is aggradation, according to the sediment routing results presented in Section 4.4.1. However, the stability of the reach downstream of this cutoff wall would be better defined if the existing Thermal Drop Structure were to be modified rather than removed, with its grade control function maintained (note that removal of the Thermal Drop Structure is the current plan, according to WSP).

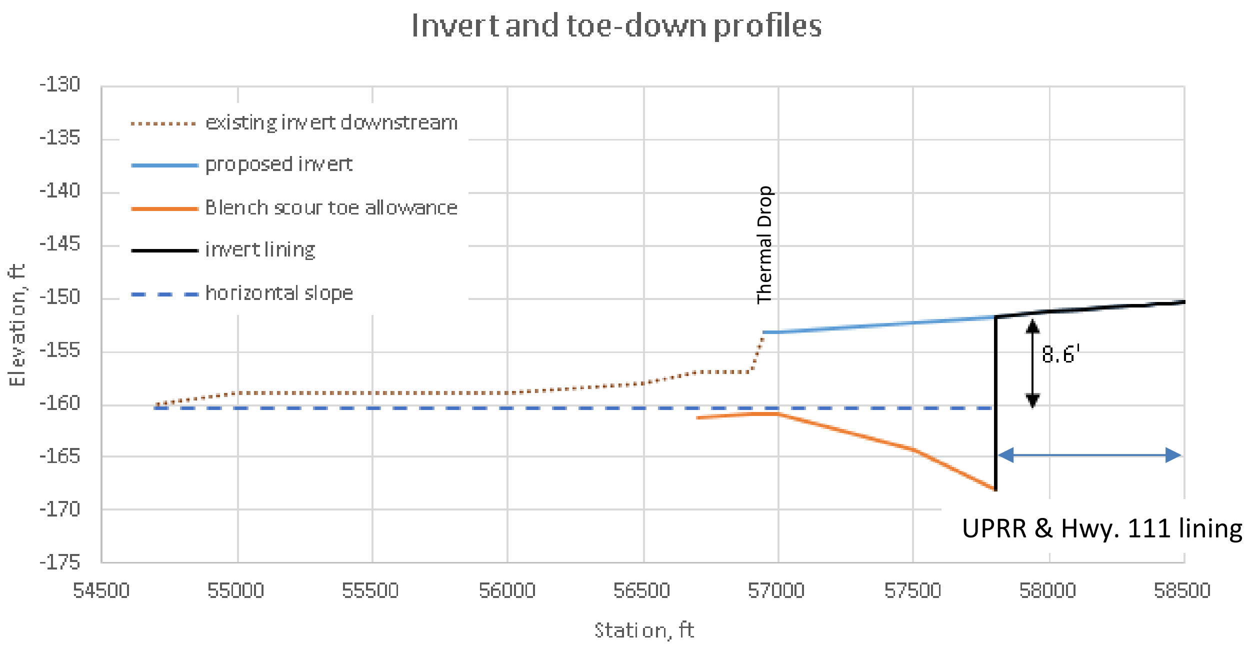


Figure 31. Potential cutoff wall exposure downstream of the UPRR - Highway 111 lining

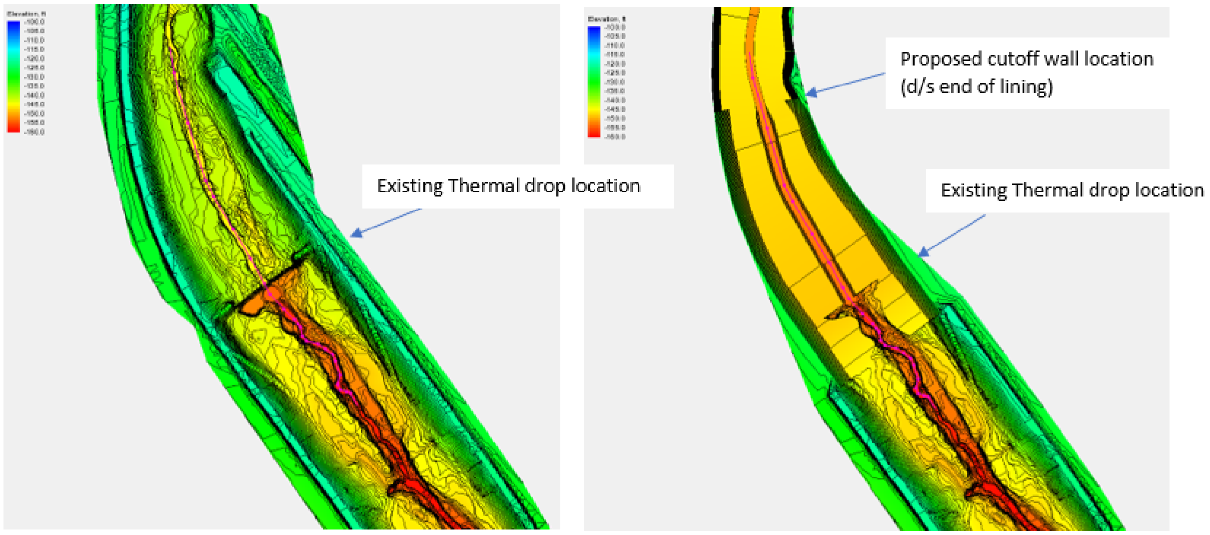


Figure 32. Channel topography near the Thermal Drop Structure, existing (left) and with-project (right)

An additional CVWD criteria may require modification of the toe burial depths for each of these cutoff walls: under current CVWD policy, structures within the channel and in the vicinity of bridge crossings are to be designed for scour considering SPF conditions. To address this additional criteria, water surface profiles for the peak SPF flows were computed for the with-project condition. The peak flow rate for the SPF is 82,000 cfs in the project reach, more than double that of the peak 100-year flow rate. While overflow of the project banks is possible along the project reach under the SPF, for scour estimation purposes, the hydraulic performance of the peak SPF flow through the project reach was computed assuming all flows were contained within the channel banks. The calibrated with-project HEC-RAS model was used for the SPF computations. A comparison of water surface profiles through the project reach under 100-year and SPF conditions is shown in Figure 33.

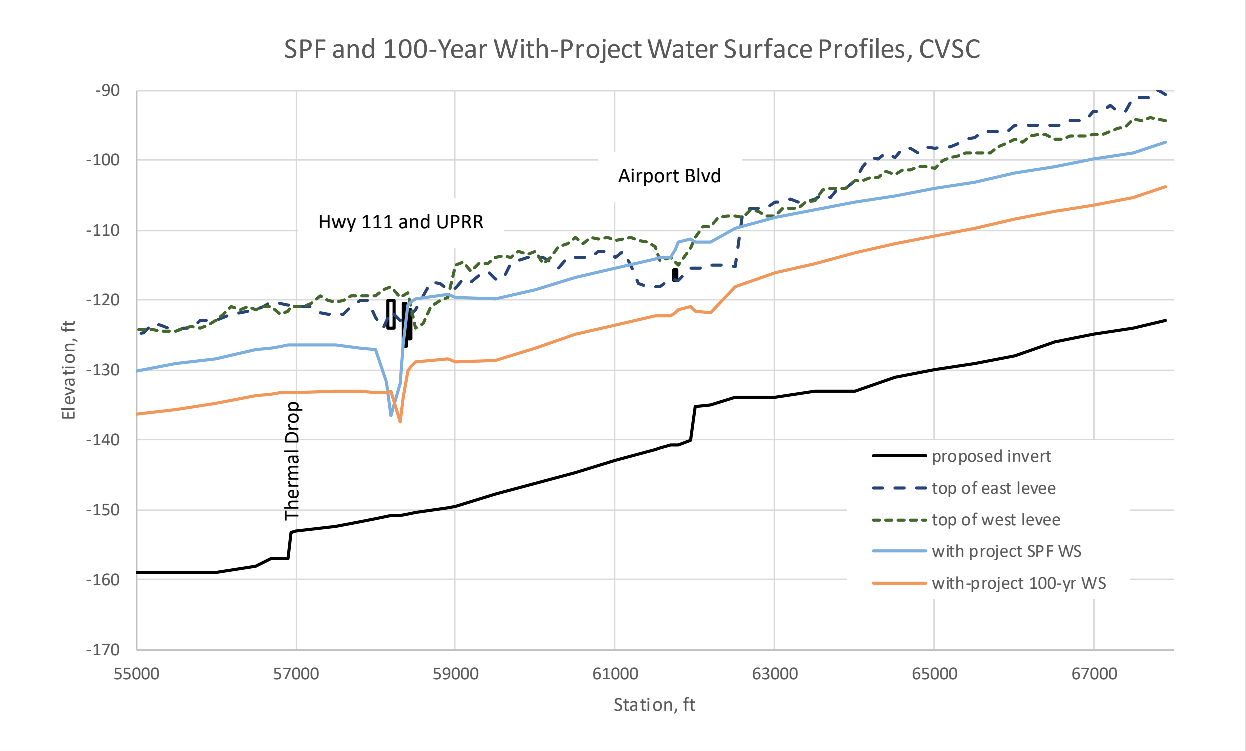


Figure 33. SPF and 100-year water surface profiles, with-project conditions

The potential need for additional scour allowance at the proposed cutoff walls due to the SPF criteria and exposure potential is evaluated in the following paragraphs.

As discussed above, with exposure due to downstream degradation, the proposed cutoff walls at the downstream end of each lined reach could act as drop structures. The Veronese equation (USBR, 1972) provides an estimate of the drop scour that results due to weir flow over a vertical drop into a tail water. The Veronese equation is:

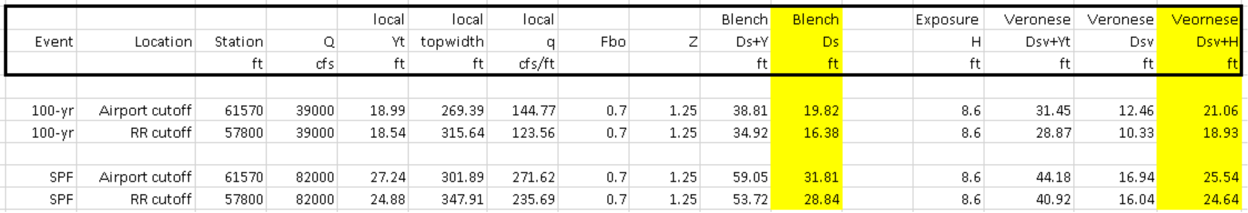
$D_s + Y_t = 1.32 * H^{0.225} * q^{0.54}$ (foot-pound-sec units)

with D_s = the scour depth below the downstream channel invert
Y_t = the tailwater depth
H = the difference in the energy grade line upstream and downstream of the drop, and
q = the unit discharge over the drop crest.

In the above equation, the drop height is commonly used as an estimate for H.

Blench scour and Veronese scour estimates under 100-year and SPF conditions at each downstream cutoff wall location are summarized in Table 3. The Veronese scour estimates were computed assuming 8.6 feet of exposure (and energy drop) at each cutoff wall. According to these calculations, a cutoff wall extending approximately 20 below the channel invert will be adequate at each location considering 100-year design flood criteria. If the SPF criteria were to be followed the toe allowance would increase to approximately 30 feet below the channel invert. Specific guidance and transition configuration will be developed following review and discussion of these preliminary computations with CVWD.

Table 3. Blench and Veronese scour calculations at cutoff walls, 100-year versus SPF conditions



Event	Location	Station ft	Q cfs	local Yt ft	local topwidth ft	local q cfs/ft	Fbo	Z	Blench Ds+Y ft	Blench Ds ft		Exposure H ft	Veronese Dsv+Yt ft	Veronese Dsv ft	Veornese Dsv+H ft
100-yr	Airport cutoff	61570	39000	18.99	269.39	144.77	0.7	1.25	38.81	19.82		8.6	31.45	12.46	21.06
100-yr	RR cutoff	57800	39000	18.54	315.64	123.56	0.7	1.25	34.92	16.38		8.6	28.87	10.33	18.93
SPF	Airport cutoff	61570	82000	27.24	301.89	271.62	0.7	1.25	59.05	31.81		8.6	44.18	16.94	25.54
SPF	RR cutoff	57800	82000	24.88	347.91	235.69	0.7	1.25	53.72	28.84		8.6	40.92	16.04	24.64

4.6 Hydraulic and Channel Stability Impacts

A plot comparing existing and proposed 100-year water surface profiles through the project reach is shown in Figure 34. The with-project water surface profiles converge with the previous existing conditions model results (NHC 2012a) at the upstream and downstream ends of the model, assuming resistance conditions assumed are consistent between the two models. The proposed condition model assumes higher resistance conditions in the reach upstream of the Airport Boulevard crossing for design purposes. Proposed improvements are expected to significantly reduce backwater conditions upstream of the UPRR and Airport Boulevard bridge crossings. Water surface profiles are expected to transition to pre-project levels at the project boundaries.

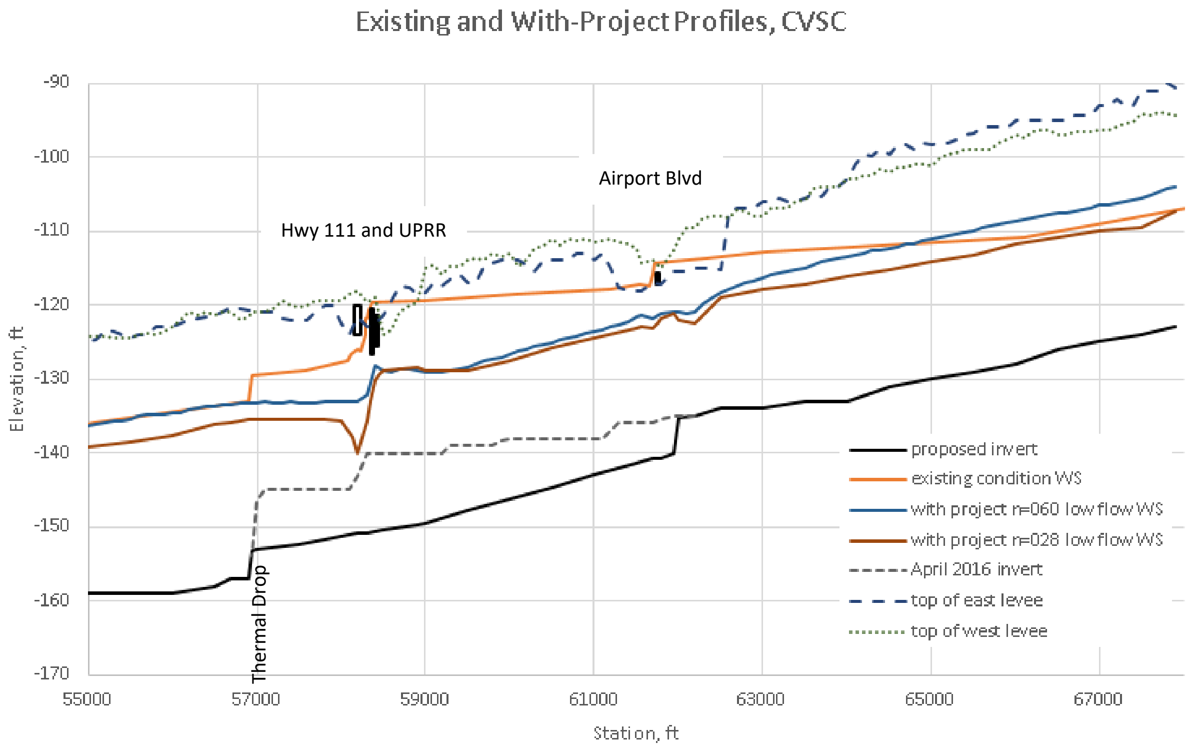


Figure 34. A comparison of existing and with-project invert and 100-year water surface profiles through the project reach

Sediment transport analyses were used to assess the potential channel stability impacts that may be associate with the proposed project. Local transport characteristics under with-project conditions are presented in Section 4.1.1. These local transport characteristics were contrasted with those computed under existing conditions for assessment of potential off-site channel stability impacts.

NHC's existing condition HEC-RAS model (NHC, 2102a) was used to estimate sediment transport through the project reach under 100-year flood conditions, using the same transport equations and sediment size distributions used for the with-project analysis. Pre-and post-100-year flood channel profiles for the existing conditions simulation are shown in Figure 35. The computations indicate that accumulation would be expected upstream of the UPRR crossings, with some scour immediately downstream (checked by the Thermal Drop Structure), and with some accumulation downstream of this drop. The with-project model indicates similar trends, though rearranged somewhat as shown in Figure 36. Less accumulation (and some scour) is expected within the project limits upstream of Airport Boulevard, but no significant differences in trends are indicated at the upstream or downstream ends of the project reach.

The computation results indicate that sediment transport trends through the project reach are similar under both existing and with-project conditions. The with-project condition has more conveyance capacity through the bridges, particularly Airport Boulevard, but the widened and lowered reach through the downstream end of the project tends to have similar transport inefficiencies as the existing bridges + drop structure combination.

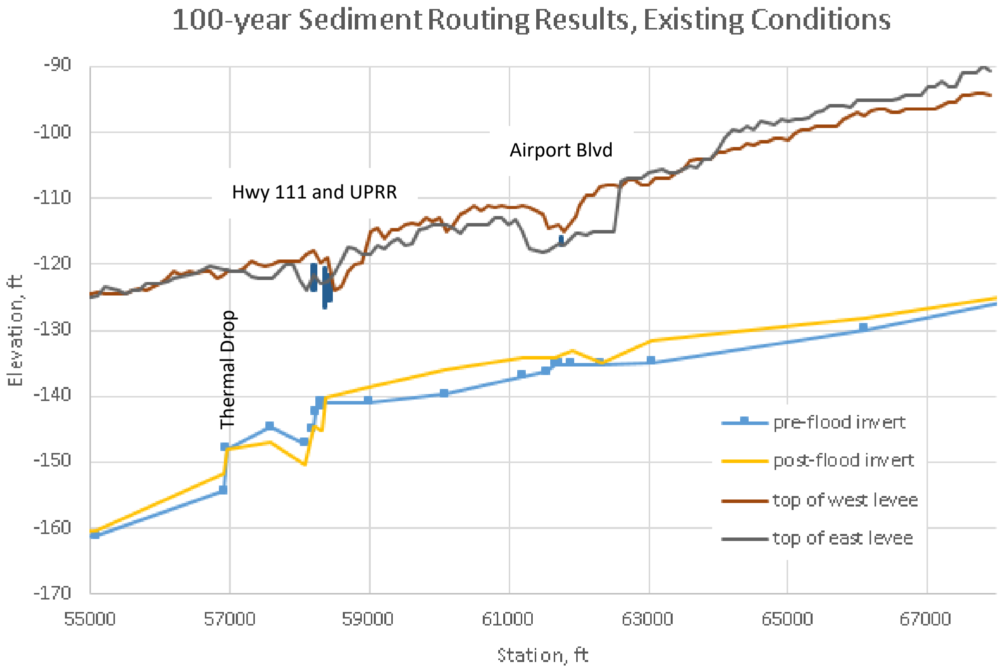


Figure 35. Computed post 100-year flood sedimentation profiles, existing conditions

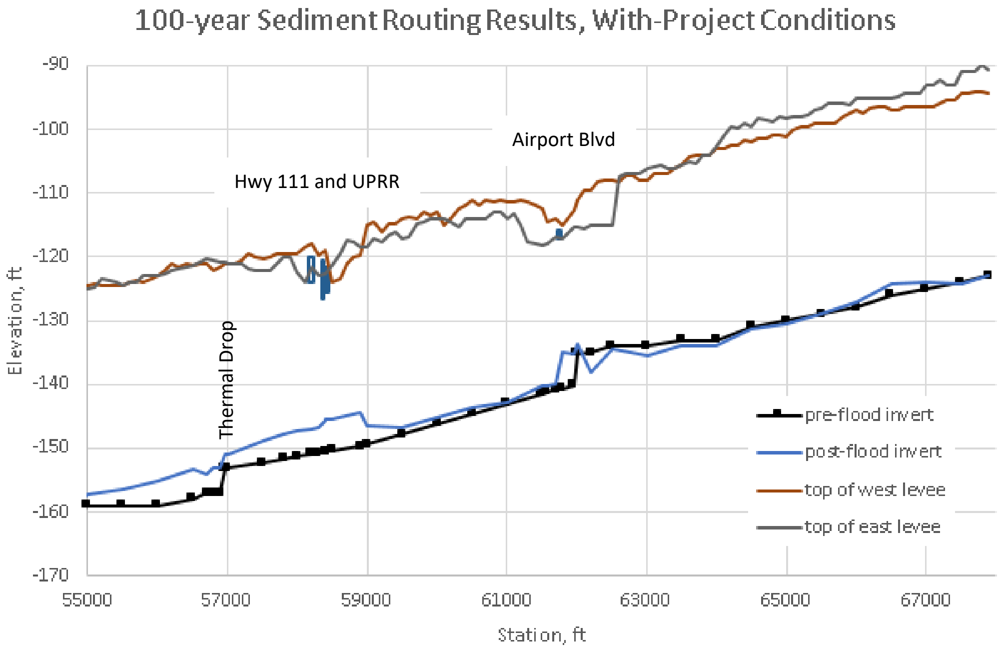


Figure 36. Computed post 100-year flood sedimentation profiles, with-project conditions

5. Summary

The proposed project is a revised version of the preferred plan from the alternatives study recently completed by Tetra Tech (2014). The extent of the full concrete lining has been reduced, and the existing Airport Boulevard Bridge has been incorporated into the plan.

Major items of the proposed project include: (1) lowering the Thermal Drop Structure; (2) lowering the channel invert profile between the Thermal Drop Structure to just upstream of the Airport Boulevard bridge; (3) concrete lining of the channel banks along the entire length of the project reach; (4) lining of the channel invert in the vicinity of the Airport Boulevard crossing; and, (5) lining of the channel invert in the vicinity of the UPRR and Highway 111 crossings.

Key constraints and obstacles along the project reach include the low bank and soffit elevations at the Airport Boulevard Bridge, the sharp bend upstream of the UPRR Bridge, and the relatively wide, miss-aligned piers and low soffit elevations at the UPRR crossings. The channel along the project reach has been deepened (the project invert has been lowered) to convey the design flow through these system constraints. Pipelines and utilities that cross under the existing channel will need to be modified to accommodate the lower channel profile. These crossings will be identified in the WSP design report.

The design capacity for the proposed project is the peak 100-year flow rate of 39,000 cfs. The 50% plans provide 4 feet of freeboard to the top of the proposed concrete lining on the banks (see profile comparisons on the 50% plan set, Appendix C). All existing bridges are incorporated into the plan with existing spans and soffit levels. Toe allowances have been computed considering scour levels at the peak 100-year flow rate, with peak SPF (Q = 82,000 cfs) scour magnitudes provided for consideration at two cutoff wall locations, in accordance with current CVWD design policy. Further consultation with CVWD will be required to establish the recommended scour allowance and bank toe transitions in the vicinity of the two cutoff wall locations.

The existing project reach tends to accumulate sediment, and vegetation density within the channel has increased in recent years in the periods between maintenance activities. These same tendencies are expected to continue under with-project conditions. Sediment, debris and vegetation management will be required to maintain the design capacity of the project reach.

Upstream and downstream hydraulic and channel stability impacts associated with the proposed plan are expected to be minimal.

Current plans include the complete removal of the Thermal Drop Structure. Given the uncertainties associated with channel stability downstream of the project reach, it may be advisable to modify, rather than remove this structure. The modification would include changes to reflect the proposed channel grading, with existing toe and energy dissipation features protected in place to maintain the grade control function this structure has historically provided.

6. References

AEI-CASC (2010). Floodplain Study and Sediment Transport Analyses Report for the Whitewater River at the Cathedral Canyon Drive Low-Water Crossing Replacement with a New Bridge Project. Submitted to Coachella Valley Water District by AEI-CASC Consulting, September 2010.

Bechtel (1995). Flood Insurance Study: Whitewater River and Morongo Wash Areas. Cities of Cathedral City, Rancho Mirage and Palm Desert. Prepared by Bechtel Corporation, submitted to CVWD, March 20, 1995.

CVWD (1971), Coachella Valley Stormwater Channel, Plan and Profile Drawings. June 25, 1971.

CVWD (2013). Development Design Manual, Appendix K. Coachella Valley Water District. August 2013.

GENTERRA (2016), Geotechnical Investigation Report, Proposed Coachella Valley Stormwater Channel Improvement Project from Avenue 54 to Thermal Drop Structure. Prepared for Coachella Valley Water District by GENTERRA Consultants, Inc., April 2016.

NHC (2012a). Coachella Valley Stormwater Channel Hydraulic Analysis: Existing Conditions. Report prepared for Coachella Valley Water District by Northwest Hydraulic Consultants. October 9, 2012.

NHC (2012b). Jefferson Street Grade Control Structure and Sewer Line Replacement. Final Basis of Design Report prepared for Coachella Valley Water District by Northwest Hydraulic Consultants, October 24, 2012.

NHC (2015). CVSC Existing Conditions Channel Capacity Assessment (draft document). Prepared for Coachella Valley Water District by Northwest Hydraulic Consultants, 2015.

Tetra Tech (2014). Eastern Coachella Valley Stormwater Master Drainage Plan, Coachella Valley Stormwater Channel Alternatives Study, Final Report. Prepared for Coachella Valley Water District by Northwest Hydraulic Consultants, May 2014.

USACE (1980). Whitewater River Basin Feasibility Report for Flood Control and Allied Purposes San Bernardino and Riverside Counties, California, Appendix 1: Hydrology. U.S. Army Engineer District.

USACE (1970), Hydraulic Design of Flood Control Channels, Engineering Manual EM 1110-2-1601, U.S. Army Engineer District.

USBR (1972), Design of Small Dams.

Appendix A – Roughness Components

Components-Based Composite Manning n Estimation

Manning n Components			Low Flow Channel						Terrace			
			As-Built	Maintained	Small Veg	Med Veg	Large Veg	Very Lg Veg	As-Built	Maintained	Small Veg	Med Veg
Base n												
	clay	0.02										
	sand-fine gravel	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
	coarse	0.026										
Bank Irregularity												
	smooth	0	0						0			
	minor	.001-.005		0.005	0.005	0.005	0.005	0.005		0.001	0.001	0.001
	moderate	.006-.01										
	severe	.011-.020										
Variation in Section												
	gradual	0	0						0			
	occasional	.001-.005		0.001	0.001	0.001	0.001	0.001		0.001	0.001	0.001
	frequent	.010-.015										
Obstructions												
	negligible	0-.004	0						0	0	0	0
	minor	.005-.015		0.005	0.005	0.005	0.005	0.005				
	appreciable	.020-.030										
	severe	.040-.060										
Vegetation												
	small	.002-.010	0		0.006				0	0.002	0.006	
	medium	.010-.025		0.025		0.0175						0.0175
	large	.025-.05					0.0375					
	very large	.05-.10						0.075				
Meandering (multiplier)												
	minor	1	1	1	1	1	1	1	1	1	1	1
	appreciable	1.5										
	severe	1.3										
Computed n Value			**0.024**	**0.060**	**0.041**	**0.053**	**0.073**	**0.110**	**0.024**	**0.028**	**0.032**	**0.044**

Appendix B – Hydraulic Output Data

100-year Water Surface Profiles
2D Simulation Results

Station ft	East WS ft	Center WS ft	West WS ft
67900	-103.385	-103.998	-103.954
67800	-104.206	-104.263	-104.565
67700	-105.127	-104.672	-104.689
67600	-104.64	-105.045	-105.094
67500	-106.438	-105.438	-105.691
67400	-106.036	-105.696	-105.434
67300	-105.663	-105.95	-106.059
67200	-105.958	-106.112	-106.164
67100	-106.282	-106.312	-106.379
67000	-106.545	-106.451	-106.272
66900	-106.784	-106.727	-106.624
66800	-107.024	-106.987	-107.037
66700	-107.244	-107.187	-107.085
66600	-107.497	-107.389	-107.325
66500	-107.614	-107.541	-107.524
66400	-107.688	-107.763	-107.77
66300	-107.919	-107.952	-107.992
66200	-108.196	-108.17	-108.151
66100	-108.348	-108.437	-108.444
66000	-108.638	-108.592	-108.609
65900	-108.8	-108.954	-108.923
65800	-109.009	-109.099	-109.136
65700	-109.263	-109.323	-109.306
65600	-109.826	-109.636	-109.554
65500	-110.017	-109.924	-109.896
65400	-110.215	-110.168	-110.133
65300	-110.494	-110.362	-110.396
65200	-110.635	-110.678	-110.734
65100	-110.807	-110.808	-110.803
65000	-111.08	-111.078	-111.175
64900	-111.42	-111.37	-111.29
64800	-111.688	-111.69	-111.816
64700	-111.906	-111.874	-111.944
64600	-112.261	-112.14	-112.116
64500	-112.319	-112.365	-112.424
64400	-112.569	-112.602	-112.601
64300	-112.846	-112.774	-112.81
64200	-113.265	-113.105	-112.947
64100	-113.535	-113.414	-113.38
64000	-113.714	-113.676	-113.549

HEC-RAS 100-year Simulation Results

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
67900	39000	-123	-103.95	-102.83	0.003414	8.49	4595.31	440.48	0.46
67500	39000	-124	-105.48	-104.25	0.003677	8.9	4384.4	406.92	0.48
67000	39000	-125	-106.47	-105.65	0.001875	7.28	5359.12	441.1	0.37
66500	39000	-126	-107.39	-106.58	0.001827	7.18	5431.81	436.9	0.36
66000	39000	-128	-108.39	-107.55	0.002029	7.33	5317.24	437.96	0.37
65500	39000	-129	-109.72	-108.72	0.002657	8.02	4862.33	436.27	0.42
65000	39000	-130	-110.97	-110.02	0.002489	7.82	4987.79	436.81	0.41
64500	39000	-131	-112.17	-111.26	0.002423	7.63	5112.02	448.93	0.4
64000	39000	-133	-113.43	-112.5	0.002509	7.73	5044.86	447.61	0.41
63500	39000	-133	-114.99	-113.9	0.003084	8.36	4667.64	434.2	0.45
63000	39000	-134	-116.47	-115.34	0.002656	8.54	4566.92	398.87	0.44
62500	39000	-134	-118.33	-116.77	0.002894	10.05	3879.2	341.31	0.53
62200	39000	-135.13	-122.42	-118.4	0.008597	16.08	2425.48	278.74	0.96
62000	39000	-139.15	-121.12	-119.41	0.000642	10.5	3714.03	298.99	0.53
61950	39000	-140.13	-121.01	-119.5	0.000512	9.89	3943.3	291.91	0.47
61785	39000	-140.66	-121.31	-119.61	0.000624	10.47	3725.3	296.07	0.52
61700	39000	-140.77	-122.18	-120.32	0.00072	10.95	3561.16	294.85	0.56
61570	39000	-141.17	-122.16	-120.45	0.000564	10.5	3715.47	269.44	0.5
61500	39000	-141.39	-122.43	-120.51	0.000678	11.12	3506.8	269.03	0.54
61000	39000	-143	-123.54	-121.14	0.002425	12.43	3137.95	244.43	0.61
60500	39000	-144.59	-125.06	-122.44	0.002696	12.99	3002.74	237.01	0.64
60000	39000	-146.2	-127.05	-124	0.003404	14.03	2778.95	227.76	0.71
59500	39000	-147.8	-129.27	-125.87	0.004013	14.78	2638.53	226.11	0.76
59000	39000	-149.45	-128.58	-127.27	0.000806	9.2	4237.59	279.44	0.42
58900	39000	-149.73	-128.6	-127.34	0.000379	9.01	4329.78	281.44	0.4
58500	39000	-150.35	-129.18	-127.55	0.000513	10.25	3803.95	255.34	0.47
58401	39000	-150.53	-130.64	-129.14	0.000496	9.8	3977.91	279.59	0.46
58400	39000	-150.53	-130.64	-129.14	0.000496	9.8	3977.71	279.59	0.46
58300	39000	-150.73	-137.53	-129.98	0.005019	22.06	1768.26	208.57	1.33
58200	39000	-150.92	-133.22	-131.26	0.000767	11.22	3476.28	277.02	0.56
58000	39000	-151.3	-133.34	-131.64	0.000651	10.45	3731.08	293.5	0.52
57800	39000	-151.68	-133.2	-131.86	0.000491	9.32	4184.85	317.37	0.45
57500	39000	-152.26	-133.14	-132.17	0.00098	7.88	4947.13	373.91	0.38
57000	39000	-153.12	-133.24	-132.64	0.000541	6.25	6237.45	446.74	0.29
56950	39000	-153.17	-133.28	-132.66	0.000545	6.27	6215.44	444.93	0.3
56900	39000	-157	-133.3	-132.7	0.000668	6.21	6278.31	445.96	0.29
56800	39000	-157	-133.42	-132.77	0.000751	6.47	6029.36	447.23	0.31
56700	39000	-157	-133.62	-132.87	0.001033	6.92	5632.46	445.58	0.34
56500	39000	-158	-133.86	-133.09	0.001112	7.06	5522.94	427.96	0.35
56000	39000	-159	-134.94	-133.84	0.001952	8.4	4641.31	416.27	0.44
55500	39000	-159	-135.83	-134.79	0.001768	8.19	4762.48	420.9	0.43
55000	39000	-159	-136.64	-135.62	0.001535	8.11	4811.48	389.83	0.41
54700	39000	-160	-137.26	-136.16	0.002081	8.41	4635.57	390.01	0.43

100-year Water Surface Profiles
2D Simulation Results

Station ft	East WS ft	Center WS ft	West WS ft
63900	-114.07	-113.89	-113.762
63800	-114.227	-114.185	-114.056
63700	-114.398	-114.378	-114.306
63600	-114.824	-114.752	-114.404
63500	-115.104	-115.137	-115.226
63400	-115.707	-115.523	-115.256
63300	-116.048	-115.809	-115.627
63200	-116.185	-115.988	-115.545
63100	-116.089	-116.266	-116.5
63000	-116.726	-116.616	-116.472
62900	-117.147	-117.026	-116.821
62800	-117.591	-117.41	-117.08
62700	-117.886	-117.833	-117.932
62600	-118.248	-118.29	-118.533
62500	-118.92	-118.914	-118.691
62400	-119.72	-119.565	-119.908
62300	-119.884	-120.376	-120.169
62200	-121.956	-122.813	-122.428
62100	-122.82	-122.38	-122.786
62000	-121.332	-121.347	-121.473
61950	-120.972	-121.085	-121.119
61785	-120.807	-121.294	-122.245
61700	-121.098	-121.557	-121.629
61570	-121.333	-121.394	-121.445
61500	-121.357	-121.678	-121.689
61400	-122.26	-122.369	-122.172
61300	-123.083	-122.942	-122.788
61200	-122.882	-123.123	-123.107
61100	-123.32	-123.376	-123.4
61000	-123.585	-123.658	-123.724
60900	-123.814	-123.928	-124.053
60800	-124.318	-124.231	-124.22
60700	-124.518	-124.491	-124.55
60600	-124.831	-124.812	-124.799
60500	-125.182	-125.102	-125.182
60400	-125.45	-125.359	-125.549
60300	-125.822	-125.56	-125.619
60200	-125.611	-125.89	-126.575
60100	-126.316	-126.309	-126.342
60000	-126.841	-126.665	-126.916

100-year Water Surface Profiles
2D Simulation Results

Station ft	East WS ft	Center WS ft	West WS ft
59900	-126.977	-126.912	-127.014
59800	-127.318	-127.163	-126.932
59700	-127.456	-127.579	-127.571
59600	-127.822	-127.961	-128.253
59500	-128.051	-128.216	-128.36
59400	-128.399	-128.44	-128.596
59300	-128.886	-128.692	-129.039
59200	-128.911	-128.9	-129.462
59100	-128.78	-129.004	-129.145
59000	-128.825	-128.715	-128.929
58900	-128.228	-128.417	-128.468
58800	-128.149	-128.336	-128.257
58700	-128.318	-128.441	-128.117
58600	-128.363	-128.897	-129.148
58500	-127.949	-129.351	-132.256
58400	-127.669	-130.445	-130.779
58300	-135.093	-131.492	-131.149
58200	-134.3	-133.292	-133.567
58100	-133.937	-133.236	-132.88
58000	-133.899	-133.179	-132.728
57900	-133.81	-133.07	-132.569
57800	-133.732	-133.069	-132.484
57700	-133.696	-133.437	-133.448
57600	-133.651	-133.421	-133.168
57500	-133.603	-133.345	-133.046
57400	-133.57	-133.397	-133.003
57300	-133.548	-133.44	-132.982
57200	-133.527	-133.437	-132.999
57100	-133.509	-133.434	-133.047
57000	-133.49	-133.427	-133.104
56950	-133.475	-133.42	-133.115
56900	-133.45	-133.384	-133.192
56800	-133.417	-133.411	-133.274
56700	-133.447	-133.542	-133.209

Bank Protection Toe-Down Allowances

Station ft	Topwidth of Flow ft	Blench Scour (below WS) ft
67500	429	28.5
67300	440	28.0
67100	434	28.2
66900	438	28.1
66700	434	28.2
66500	437	28.1
66300	433	28.3
66100	434	28.2
65900	434	28.2
65700	427	28.6
65500	433	28.3
65300	437	28.1
65100	436	28.2
64900	432	28.3
64700	436	28.2
64500	447	27.7
64300	440	28.0
64100	441	27.9
63900	439	28.0
63700	437	28.1
63500	432	28.3
63300	417	29.0
63100	396	30.0
62900	390	30.3
62700	364	31.8
62500	340	33.2
62300	301	36.0
61500	271	38.7
61300	247	41.1
61100	247	41.1
60900	241	41.8
60700	240	41.9
60500	237	42.3
60300	238	42.2
60100	235	42.5
59900	233	42.8
59700	234	42.6
59500	234	42.6

Bank Protection Toe-Down Allowances

Station ft	Topwidth of Flow ft	Blench Scour (below WS) ft
59300	231	43.0
59100	246	41.2
58900	283	37.6
57900	282	37.6
57700	332	33.8
57500	373	31.2
57300	422	28.8
57100	450	27.6
56900	445	27.8
56700	446	27.7

Appendix C – Design Plans

Plans provided as PDF attachment

Titled: Avenue 54 to Thermal Drop Structure CVSC Improvement Project