PRELIMINARY HYDRAULIC DESIGN OF THE DISCHARGE CHANNELS FOR PUMP STATIONS S-332B & S-332C - PARALLEL WEIR OPTION

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EXECUTIVE SUMMARY

Pump stations S-332B and S-332C convey water from the L-31N borrow canal to the South Detention Area (SDA) located adjacent to the eastern boundary of Everglades National Park in Miami-Dade County. In addition to supplying water to the SDA, pump station S-332B also conveys water to the North Detention Area (NDA) occasionally. It is proposed that each of these pump stations be relocated about 300 feet to the south and west from their current locations.

A preliminary design (hereafter called the Original design) of both the above-ground S-332B & C discharge channels was completed in 2018 and was submitted to the U.S Army Corps of Engineers (USACE). The original design connects both of these pump stations directly to the SDA closer to the ENP. This requires discharge channel lengths of about 3,600 feet and 3,400 feet for the S-332B & the S-332C channels, respectively, from their respective pump stations. Upon request from the USACE, the current analysis, hereafter referred to as the Parallel Weir Option, investigates shortening the length of the above-ground channels by connecting them to the newly constructed flow-ways of the SDA where the existing culverts discharge, instead of directly connecting them to the SDA itself.

In order to convey each pump station's design discharge of 575 cfs from its proposed location to the existing flow-way, an above-ground channel connecting each pump station to its respective downstream flow-way is proposed. These proposed channels extend approximately 1900 feet from the pump stations to the existing flow-ways in an east-west direction. The design head water stage for each channel at its eastern end is 12.0 feet NGVD. The design tail water stage at the SDA is 8.5 feet NGVD.

An additional design objective of the S-332B discharge channel is that it must be able to supply a diverted discharge of 250 cfs by gravity to the NDA when S-332B is discharging at its capacity of 575 cfs. Therefore, a certain stage range needs to be maintained in the channel along its reach where the diversions occur in order to enable 250 cfs to flow by gravity to the NDA without exceeding the stage constraint of 12.0 feet NGVD at its eastern end. A weir is proposed at the downstream end of the channel to achieve this objective. Furthermore, two alternatives are proposed for diverting the discharge of 250 cfs to the NDA. The first option placed the diversion facility in the proposed above-ground concrete channel, about 1275 feet downstream of the pump station. The second option diverts the water from the existing flow-way. In both options gated concrete box culverts are used to convey the diverted flows.

For both S-332B and S-332C, the proposed design consists of an above-ground trapezoidal channel with a bottom width of 15 feet and 4H:1V side slopes. The discharge area also includes a trapezoidal broad crested weir with 2H:1V side slopes and a fixed crest elevation of 10.0 feet NGVD at the downstream end of the channel. The weir length required to pass the design discharge of 575 cfs was determined to be approximately 192 feet. In order to connect the proposed above-ground channels to the existing flow-ways, both the S-332B & C channels bend 90 degrees northerly while expanding to a larger pool area. In both facilities the broad crested weir is situated at the downstream end of the pool.

For the S-332B discharge channel, lateral gated diversion culverts were sized to divert 250 cfs by gravity into the NDA. These culverts connect to the north detention area, with inverts set at the same elevation as the channel invert (6.0 feet NGVD). For diversion Option 1, it was determined that a total of two reinforced concrete box culverts with a span of 5.8 feet and a rise of 6 feet can convey the required flow of 250 cfs to the NDA. Similarly, for diversion Option 2, six reinforced concrete box culverts with a span and a rise of 8 feet by 6 feet, respectively, are required.

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INTRODUCTION

Background and Objectives

Temporary Pump stations S-332B and C convey water from the L-31N canal to the South Detention Area (SDA) located adjacent to the eastern boundary of Everglades National Park (ENP), Miami-Dade County. Pumped discharges are currently conveyed to the SDA through buried conduits. A permanent structure is proposed to replace each of these pump stations. The proposed permanent pump station locations will be situated several hundred feet to the south- west of the current pump stations location. An above-ground concrete lined channel connecting the tail water pool of each pump station to the existing flow-way is proposed. Additionally, the design of the S-332B discharge channel should ensure that there is sufficient head available within the channel to enable a diversion of 250 cfs to flow by gravity to the North Detention Area (NDA) when the stage at the NDA is at 8.5 feet NGVD. Weirs are proposed at the downstream ends of the discharge channels, each located about 100 feet south of the southern levee for the existing flow-way to the SDA. Additionally, lateral gated culverts are proposed to be tied-in to either the S-332B discharge channel or to the existing flow-way to divert 250 cfs to the NDA on an as-needed basis.

For both S-332B and S-332C, the objective was to develop a conceptual design of a discharge channel that can pass the design discharge of 575 cfs to the SDA while satisfying design constraints. These constraints are:

1. A maximum headwater stage of 12.0 feet NGVD is allowed at the upstream end of both channels. The design tailwater stage in the SDA is 8.5 feet NGVD.

2. The maximum allowable flow velocity in each channel is 4.5 feet/sec.

3. For the proposed diversion culverts, the design tailwater stage in the NDA is 8.5 feet NGVD.

4. The invert elevation of each discharge channel and diversion culvert is constant and equal to 6 feet NGVD (near land surface).

Figure 1 shows both the current and proposed future locations of S-332B and S-332C. Shown also are the proposed corridors of the discharge channels.

It should be emphasized that all designs developed as part of this effort should be considered conceptual and preliminary. They are intended to be used for cost estimating purposes only. Additional hydraulic design analyses of the proposed facilities will be needed prior to the next design phase. Additional design phases must be completed prior to construction.

DESIGN OF SYSTEM COMPONENTS

Detailed description of the discharge area conceptual design is provided in the following sections. The components of the discharge area include 1) the above-ground channel, 2) the weir headwater pool, and 3) the weir (**Figure 2**). Additionally, the characteristics of the diversion conduits are discussed for the S-332B discharge channel. The weir is located at the end of the pool which ultimately connects to the existing flow-way. In between the downstream end of the weir and the existing flow-way, an extension of the channel is included, where the channel width is equal to the length of the weir.

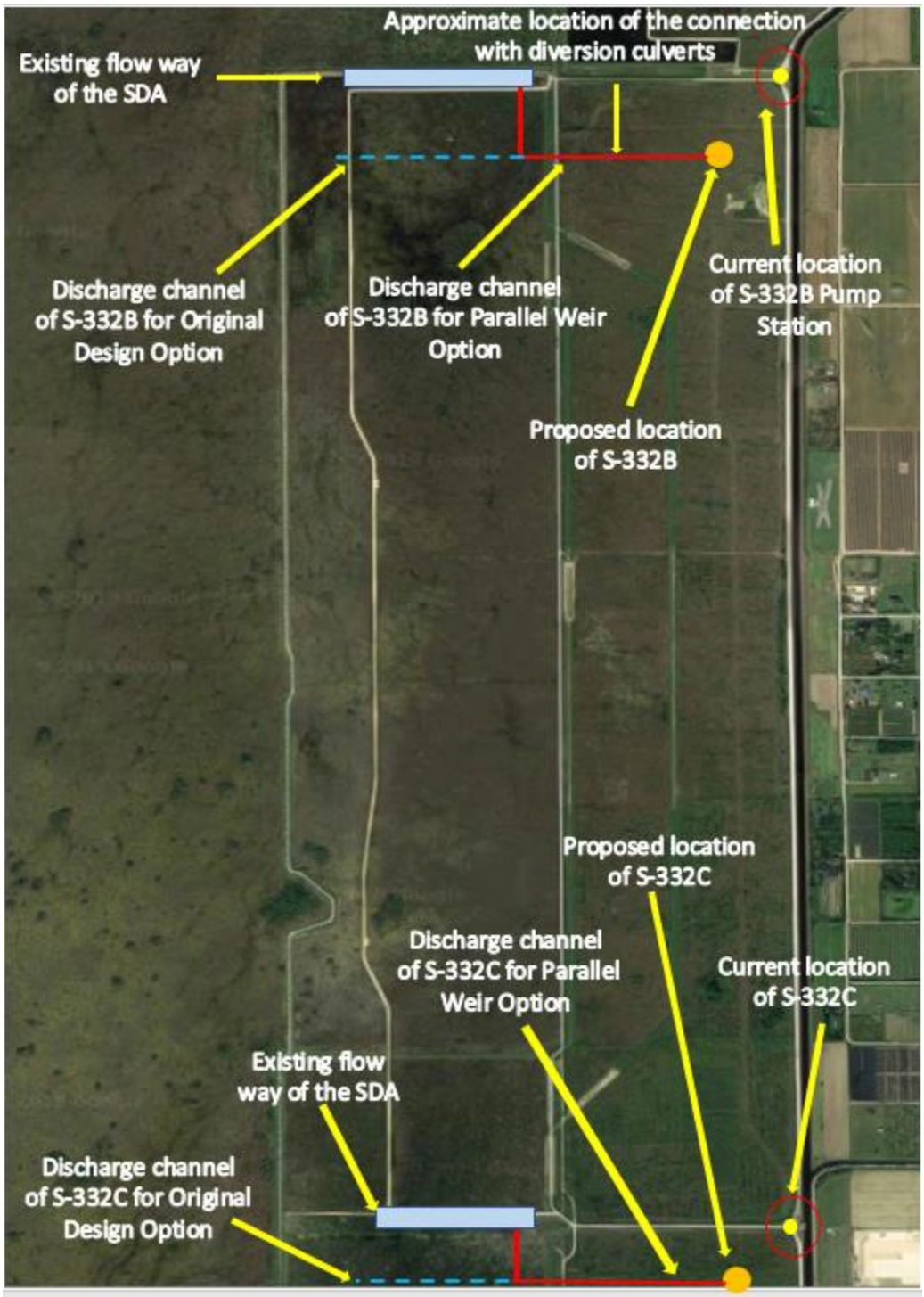


Figure 1. Configuration of the proposed facilities for relocated pump stations S-332B and S-332C

Design of the Proposed Concrete-Lined Channel

The primary channel design features are provided in **Table 1** and are based on lining them with concrete. The bottom elevation and side slope of each channel were established prior to the hydraulic design process. The bottom elevation of 6.0 feet NGVD is approximately equal to the average land surface elevation in the project area and was selected to minimize excavation and land disturbance. The 4H:1V side slope was based on access and safety considerations.

Figure 2. Schematic displaying the proposed pump station location, the weir and the concrete lined discharge channel (NTS)

The channel bottom width was sized so as to maintain a stage equal to or less than 12 feet NGVD at the upstream end while limiting flow velocities to 4.5 ft/sec. This limiting flow velocity of 4.5 ft/sec is based on the successful performance of the concrete lined discharge channels for pump stations S-199 and S-200.

Table 1. Concrete lined channel specifications for S-332B & C

Design Feature	Description
Bottom Width (ft)	15
Bottom Elevation (ft NGVD)	6.0
Side Slope (H:V)	4:1
Top Elevation (ft NGVD)	13
Length (ft)	Approx. 1900 ft
Maximum allowable velocity in the channel (ft/s)	4.5

Weir Design

The crest elevation of the trapezoidal weir separating the proposed discharge channel from the existing flow-way (**Figure 2**) was set to 10.0 feet NGVD. The resultant weir length required to pass the design flow of 575 cfs was calculated using **Equation 1** (Fritz and Hager, 1998):

$$\boldsymbol{Q = C_d L\sqrt{2gH_T^3}} \qquad \text{Equation 1}$$

In **Equation 1**, C_d = the discharge coefficient (determined using **Equation 2**), L = the weir length, H_T = the total upstream hydraulic head measured from the weir crest, and g = the acceleration due to gravity (32.17 ft/s^3). Fritz and Hager (1998) determined through experimentation that the discharge coefficient can be estimated from

$$C_d = 0.43 + 0.06\sin[\pi(\xi - 0.55)] \qquad \text{Equation 2}$$

where ξ = the relative crest length, given by

$$\xi = \frac{H_T}{H_T + L_w} \qquad \text{Equation 3}$$

In **Equation 3**, L_w = the weir top width in the direction of flow. L_w was set to 15 feet to enable passage over the weir by vehicular traffic. The total upstream hydraulic head H_T was based on a weir headwater stage of 11.0 feet NGVD. This head water stage was chosen to provide an allowance of one foot of head loss between the weir and upstream end of the channel. According to Fritz and Hager (1998), the approach velocity head needs to be adjusted by a factor of 5/3 when the ratio of H_T to the weir height exceeds 1/6. This adjustment is needed to account for the effects of both a non-uniform approach velocity distribution and the larger value of the ratio described above. Further details are provided by Wilsnack (2013).

In order to determine the length of the weir L, subsequent iterations of Equations 1 – 3 are needed since the approach velocity is dependent on L and is not known apriori. Iterations are continued until the bottom width of the approach channel and the required weir length at the selected crest elevation are approximately equal. After several iterations, the weir length was determined to be 192 feet. This length is substantially larger than the width of the approach channel. Therefore, the channel was expanded to create a head water pool for the weir, where the bottom width at the downstream end was set equal to the length of the weir.

Table 2 summarizes the resultant weir specifications for both the S-332B & C channels:

Table 2. Weir specifications

Design Feature	Description
Geometry	Broad crested
Side slope (H:V)	2
Top width (ft)	15
Length (ft)	192
Crest Elevation (ft NGVD)	10.0
Headwater stage at weir (ft NGVD)	11.0

The flow regime associated with the weir design discharge was determined to be a plunging jet. **Equation 4** (Wilsnack, 2013 or Fritz and Hager, 1998) was used to calculate the length of surface roller (L_r) downstream of the weir toe for the plunging jet.

$$L_r = 4.3d \qquad \text{Equation 4}$$

where d = the downstream tailwater depth. **Equations 5** and **6** were used to calculate the approach (absolute maximum) velocity u_A within the plunging jet and the maximum streamwise velocity u_M at any distance between the weir and the downstream edge of the surface roller for the plunging jet flow regime.

$$u_A = \sqrt{gd(1 - \frac{h}{H})} \qquad \text{Equation 5}$$

$$u_M = (u_A - u_d)e^{-2.3\chi^2} + u_d \qquad \text{Equation 6}$$

In **Equation 6**, u_d = the mean velocity in the downstream channel and χ = the dimensionless downstream distance from the edge of the weir crest to any point in the channel up to the downstream edge of the surface roller or recirculation zone, where $0 \leq \chi \leq 1$.

The maximum flow velocity within the tail water pool over the length of the surface roller was determined in order to identify the potential need for any additional erosion protection measures. The results are shown in **Table 3**. Since the maximum velocity within the plunging jet exceeds 4.5 ft/s, additional erosion protection measures may be needed on the downstream side of the weir and in the channel over the distance occupied by the surface roller. It should be emphasized that the recommendation of additional erosion protection measures only applies to the downstream side of the weir over the distance occupied by the surface roller. The maximum velocity computed here is not indicative of any potential erosion problems further downstream in the existing flow-way.

Table 3. Plunging jet properties

Design Feature	Description
Length of surface roller, L_r	13.7 feet
Max. velocity within plunging jet, u_A	14.0 ft/s

Weir Head Water Pool Design

As shown in **Figure 2**, both the S-332B & C discharge channels curve 90 degrees northward to connect with the existing flow-way at the weir. The bottom width of the channel expands from 15 feet at the upstream end of the pool to 192 feet at its downstream end. The weir head water pool serves two primary objectives. First, it transitions the discharge channel to the 192-feet long weir. Second, it serves to reduce the approach velocities of the weir, resulting in a more uniform flow distribution along the weir crest. Additionally, the reduction in approach velocities is conducive to the settling of the sediment particles prior to discharge over the weir, thereby enhancing the quality of water being discharged to the SDA.

HEC-RAS MODEL

As stated earlier, HEC-RAS was used to simulate flow through each of the proposed channels and determine the required bottom widths. Flow through the S-332B flow-way to the SDA was also simulated. The model features are discussed below.

Terrain Data

A Digital Elevation Model (DEM) with a resolution of five feet within the project area was created in 2016 and archived in the SFWMD GIS database. This DEM was converted from NAVD 1988 to NGVD29 in ArcGIS using a raster calculator. The conversion factor from NAVD88 to NGVD29 for the project area is approximately +1.567 feet. The resultant DEM was used as the base terrain for each HEC-RAS model.

For the models of the S-332B&C discharge channels, RAS Mapper in HECRAS was used to create the modified terrain with the entire channel burned into it. 1D river reach cross sections created in HECRAS for both the channel and the weir head water pool were subsequently used to modify the terrain. Various iterations of model development and terrain creation were carried out in HECRAS to configure and size the pool. Additionally, for diversion Option 1, the S-332B channel was gradually enlarged at a ratio of 1:4 near the diversion location from a bottom width of 15 feet to 40 feet over a distance of about 100 feet. RAS Mapper was then used to "burn in" the enhanced channel bathymetry to the base terrain. **Figure 3** shows the modified terrain created for the proposed S-332B channel.

Similarly, the same base DEM and procedure was used to construct the model terrain for the existing USACE flow-way. The bottom elevation of both the existing flow-way and the proposed approach channel downstream of the weir was set to 5.5 feet NGVD based on information contained in the record drawing (Harry Pepper & Associates, 2000) for the concrete slab located at the outlet of the existing S-332B discharge pipes.

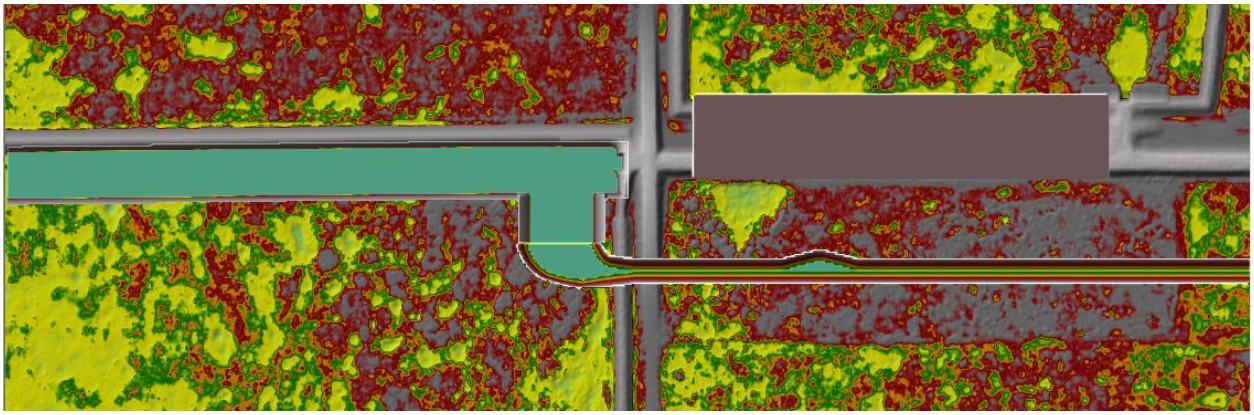


Figure 3. New terrain for the proposed S-332B channel superimposed on the existing terrain

Primary Features

Discharge Channel

HECRAS 1D and 2D capabilities were used to model the proposed design. The channel reaches with a constant bottom width of 15 feet were modelled as 1D river reaches. The downstream pool at the bend was

modeled as a 2D flow area. The upstream end of this 2D flow area was located at a distance of five channel hydraulic diameters upstream from the beginning of the 90-degree bend. Additionally, the enlarged area of the channel at the diversion location was modelled as a 2D flow area. Both the upstream and downstream ends of this 2D flow area were located at a distance of five hydraulic diameters from where the channel transition starts and ends, respectively. Both 2D flow areas were directly connected to the adjacent 1D river reaches (**Figure 4**).

A grid size of 1.5 feet by 1.5 feet was specified in both 2D flow areas. 1D river reach cross sections were interpolated every 15 feet. A Manning's n value of 0.013 was specified for the entire concrete-lined system.

Existing Flow-Way

Since the existing S-332B flow-way is narrower than the existing S-332C flow-way, discharges through

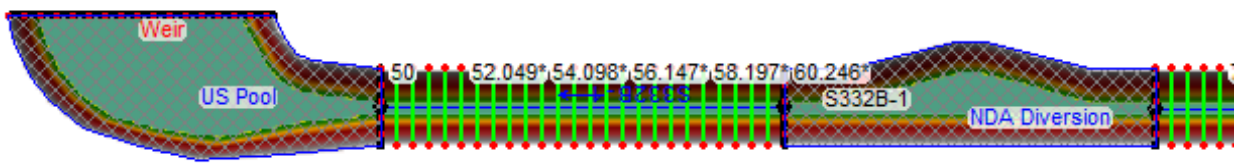


Figure 4. 2D Flow areas directly connected to the 1D river reaches

only the former were modelled. The existing flow-way that connects the S-332B discharge channel to the SDA was modelled as a combination of a 1D channel and 2D flow area (**Figure 5**). The model changed from 2D to 1D at a location downstream of the bend where flow was nearly one-dimensional. The primary purpose of modeling the design discharge through this area was to determine the resultant stages near the east end of the flow-way. These stages were needed to determine the tail water stage for the weir along with the available head water stage for the proposed diversion structure. A Manning's n value of 0.045 was applied to the entire flow-way.

Figure 5. HECRAS model for the exisiting USACE flow-way

Boundary and Initial Conditions

In each channel a flow hydrograph was used for the upstream boundary condition while a discharge rating curve was applied to the downstream boundary. The flow hydrograph at the upstream boundary was ramped up in 10 minutes intervals from 10 cfs to 575 cfs over a period of 0.15 days.

The downstream rating curve represents the stage-discharge relationship for the broad crested weir given by Equations 1 - 3. Calculations performed for deriving the rating curve for the weir are provided in **Appendix A**.

Since the 2D flow areas are connected to 1D river reaches, the HECRAS model cannot start dry. Therefore, 0.1 feet of water was used for an initial stage. For a sensitivity analysis, another simulation was performed with the model starting at an initial water level of 10.0 feet NGVD, corresponding to the crest elevation of the weir. The initial condition ramp-up time option was selected to establish flow consistent with the initial conditions throughout the 2D areas before the start of the simulation. Initial conditions were simulated for 2 hours with a ramp up fraction of 50%.

The total simulation times for the model runs was set long enough to achieve steady state conditions with the design discharge.

Computation Options and Tolerances

Eddy Viscosity Transverse Mixing Coefficients of 1 and 3 were used as indicated in the model scenario descriptions given below for the scenarios involving the concrete channel, while for the existing flow-way eddy coefficient of 2 was used. Limiting values of this parameter were incorporated into the simulations in order to investigate the effect of this parameter on flow velocity and eddy zone formation. Whenever possible, it is recommended that this parameter be determined through model calibration. Since model calibration is not possible in this design, best engineering judgment was applied while selecting lower and upper limits to a range of the eddy coefficient.

The computational time step is a function of the spatial discretization and the flow velocities. For this analysis, the computation time step was adjusted based on the maximum Courant Number. Adaptive time stepping was used so that the Courant number did not exceed 1.

Diversion Conduit Design for the S-332B Discharge Channel

In order to divert a flow of 250 cfs from the S-332B channel to the NDA, the diversion conduits were connected to the channel approximately 1275 feet downstream of the S-332B pump station in diversion Option 1. In Option 2, the diversion culverts are connected to the east end of existing flow-way. The stage in the NDA was set at the design value of 8.5 ft NGVD.

The diversion conduits were modelled as a SA/2D connection which connects the upstream 2D flow area at the diversion point in the channel to the NDA on the downstream end. The NDA was modelled as a large storage area with a constant stage of 8.5 NGVD. Fictitiously high values of storage were specified in order to minimize stage fluctuations in the storage area during the simulation.

The SA/2D connection was modelled as a combination of a weir embankment and a culvert. The top of the embankment was placed at 13 feet NGVD. In option 1, two concrete box culverts with a span of 5.8 feet and a rise of 6 feet were used to convey 250 cfs to the NDA. The barrel rise was set to six feet in order to enhance access needed for maintenance purposes. An entrance loss coefficient of 0.7 was entered in HECRAS to represent the entrance condition of the culvert which is mitered to the slope of the embankment (HEC, 2016). Flow through the culverts is controlled by the outlet tailwater conditions along with the conveyance of the barrels. The culverts flow partially full over their entire lengths. HECRAS uses the Direct Step Method to calculate the headwater stages of the culvert barrels based on either the tailwater depth of 2.5 feet or the critical depth of the culvert, whichever is higher.

For Option 2, six concrete box culverts with a span of 8 feet and a rise of 6 feet were incorporated into the model to convey the diverted flow of 250 cfs to the NDA. An entrance loss coefficient of 0.5 was used for this option (HEC, 2016) since a vertical head wall was assumed.

Table 4 summarizes the diversion conduit designs for each option.

MODEL SCENARIOS

For the S-332B discharge channel, two separate models were created in order to represent two different design scenarios. In the first scenario, all of the design discharge is conveyed to the SDA while in the second scenario, part of the flow is diverted to the NDA. For S-332C, discharges are conveyed to the SDA only – no diversions are considered. **Table 5** summarizes the model scenarios. As mentioned previously, each design scenario was simulated with eddy coefficients of 1 and 3 for the scenarios involving the concrete channel, since flow velocities within the 2D areas were found to be sensitive to the value of this parameter. Scenarios involving the existing flow-way were simulated with eddy coefficient of 2.

Table 4. Diversion conduit specifications

Design Feature	Box Culvert Specification
Manning's roughness*	0.012
Invert elevation (ft NGVD)	6
Option 1	
Size	5.8 ft span X 6 ft rise
No. of Barrels	2
Length (ft)	100
Outlet velocity (ft/s)	8.5
Headwater stage in the channel @ the diversion location (ft NGVD)	10.3
Option 2	
Size	8 ft span X 6 ft rise
No. of Barrels	6
Outlet velocity (ft/s)	2.5
Length (ft)	275
Headwater stage in the existing flow-way @ the diversion location (ft NGVD)	8.62
Notes	Erosion protection measures may be required at the outlet due to the higher outlet velocity in Option 1

*obtained from HDS5 of FHWA, 2012

Table 5. Scenarios considered in the model

Discharge Channel	Description	Scenario	Eddy Coefficient
S332-B	Flow towards the SDA only	1A	1
	Flow towards the SDA only	1B	3
	Flow towards the SDA & the NDA	2A	1
	Flow towards the SDA & the NDA	2B	3
S-332C	Flow towards the SDA only	3A	1
	Flow towards the SDA only	3B	3
Existing S-332B Flow-way	Flow towards the SDA only	4	2
	Flow towards the SDA and the NDA	5	2

MODEL RESULTS

1. Scenario 1A

Figures 6 & **7** display the velocity distribution and the maximum velocity within the head water pool of the weir. The maximum velocity within the 2D flow area occurs right at the boundary of the 1D river reach and the 2D flow area as marked in **Figure 6** by a vertical red line. The velocity distribution along this boundary is shown in **Figure 7**. **Figure 8** provides the outflow from the weir. **Figure 9** displays the headwater stage of the weir with respect to the simulation time while **Figure 10** provides the headwater stage along the length of the weir.

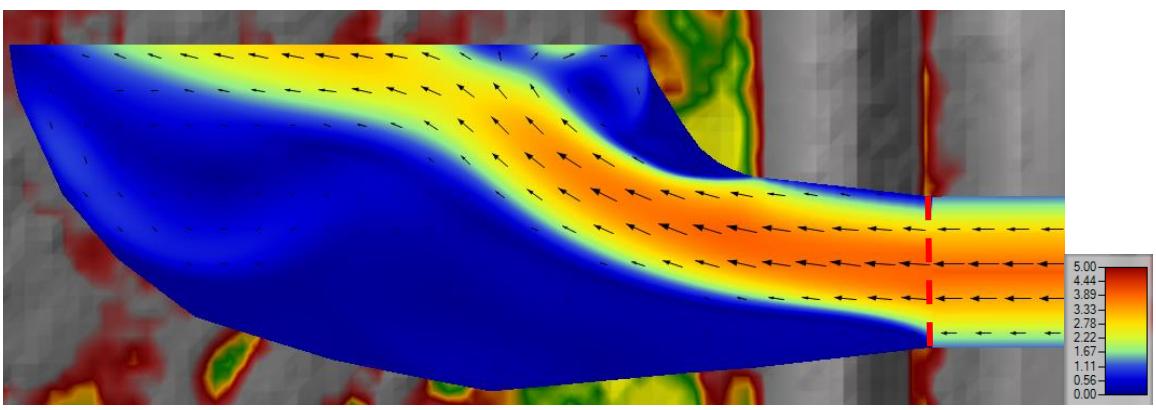


Figure 6. Velocity distribution in the 2D flow area (red line denotes the location where maximum velocity occurred within the 2D flow area)

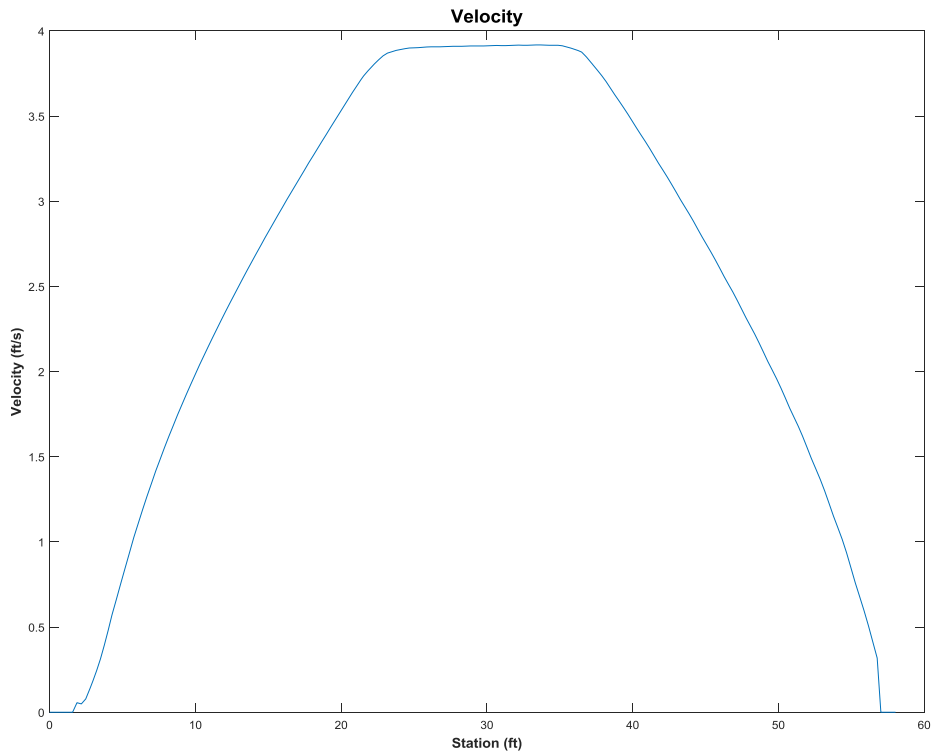


Figure 7. Velocity in the 2D flow area (location is marked in Figure 6)

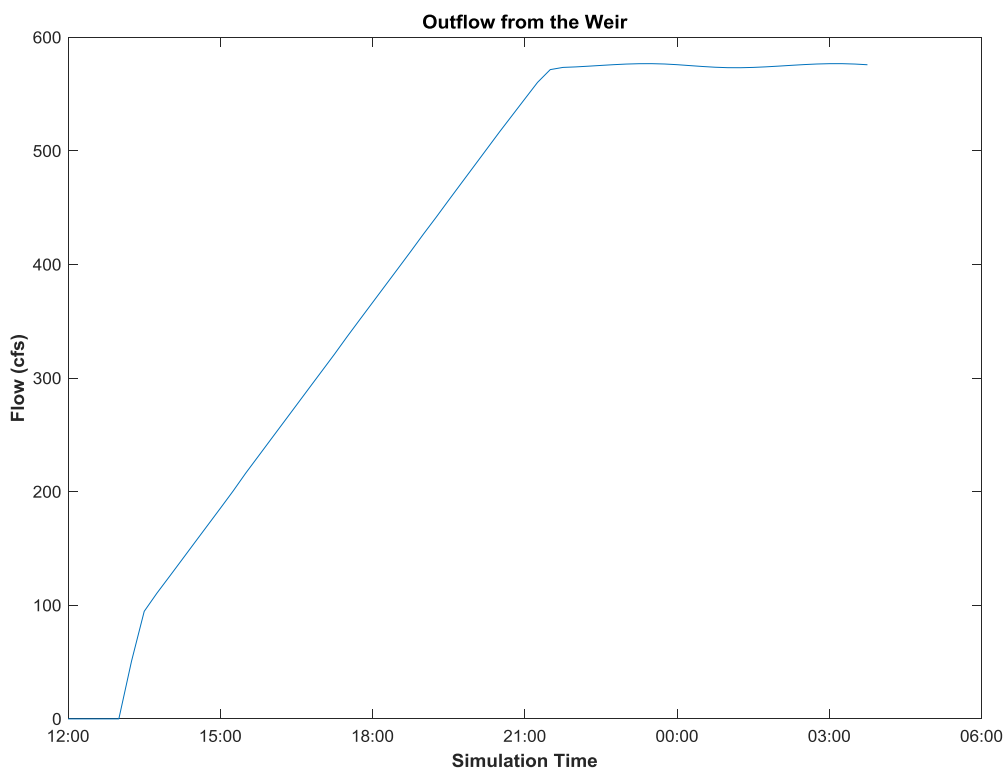


Figure 8. Weir outflow for Scenario 1A

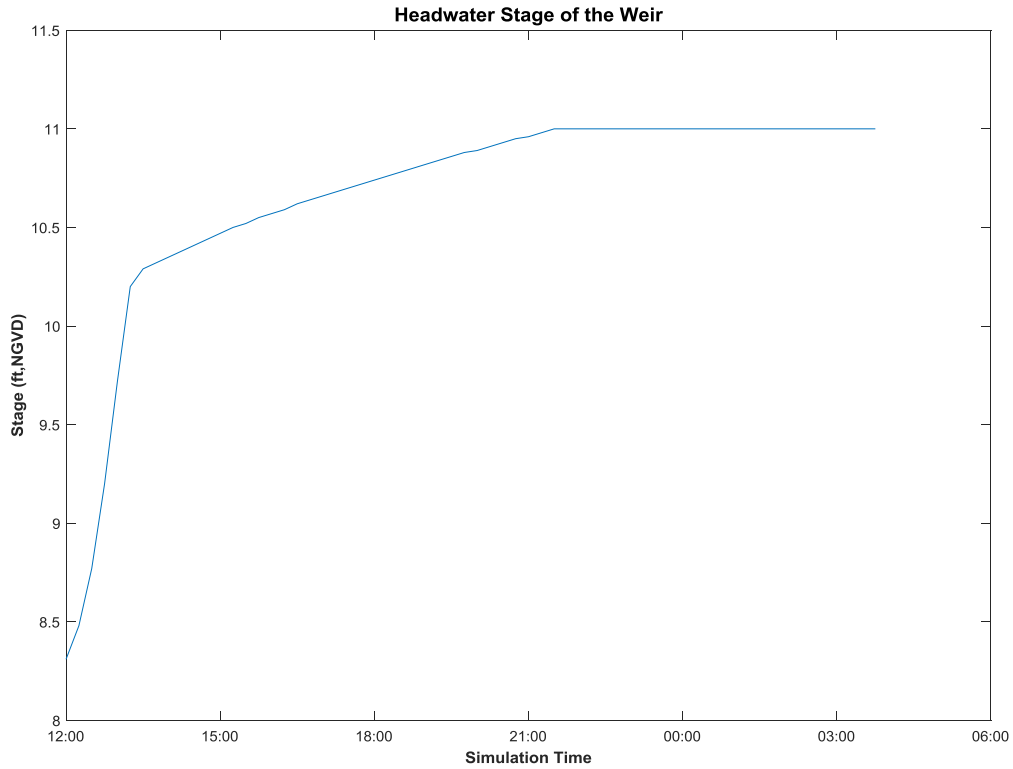


Figure 9. Weir headwater stage for Scenario 1A

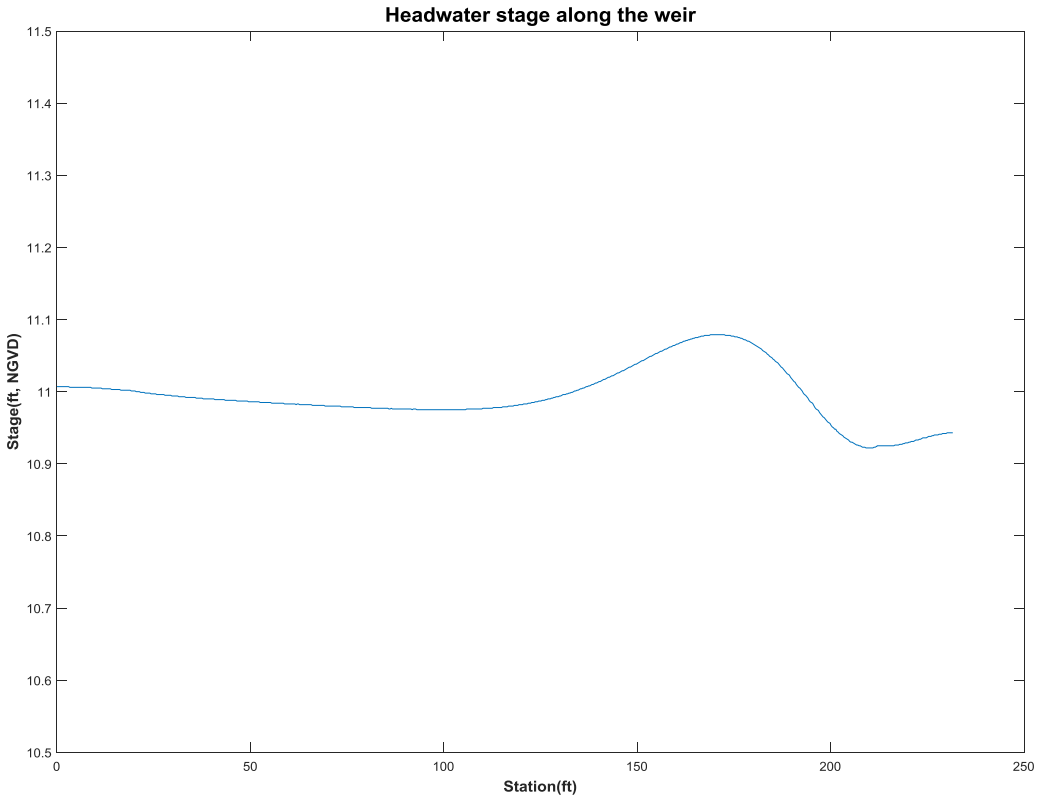


Figure 10. Scenario 1A headwater stage along the weir (station 0 represents the left side of the weir)

Table 6 shows the model results within the portion of the S-332B channel modeled with the 1D river reach. The computed headwater stage at the eastern end of the S-332B channel is 11.33 ft NGVD which is below the allowable maximum stage of 12 ft NGVD.

Table 6. S-332B discharge channel specifications

Design Feature	Description
Headwater stage at the u/s end of the channel (ft, NGVD)	11.33
Mean velocity in the channel (ft/sec)	3.13
Froude No. in the channel	0.31

2. Scenario 1B

Scenario 1B is the same as 1A except that an eddy coefficient of 3 is used. **Figures 11** & **12** display the velocity distribution and the maximum velocity, respectively, in the 2D flow area. The maximum velocity within the 2D area occurred right at the boundary of the 1D and the 2D flow areas and is very similar to the value obtained for Scenario 1A. **Figures 13** & **14** provide the flow and the headwater stage, respectively, at the weir with respect to the simulation time. **Figure 15** provides the headwater stage along the weir. It is observed that the headwater stage is constant along the weir with slight variation at the right side of the weir.

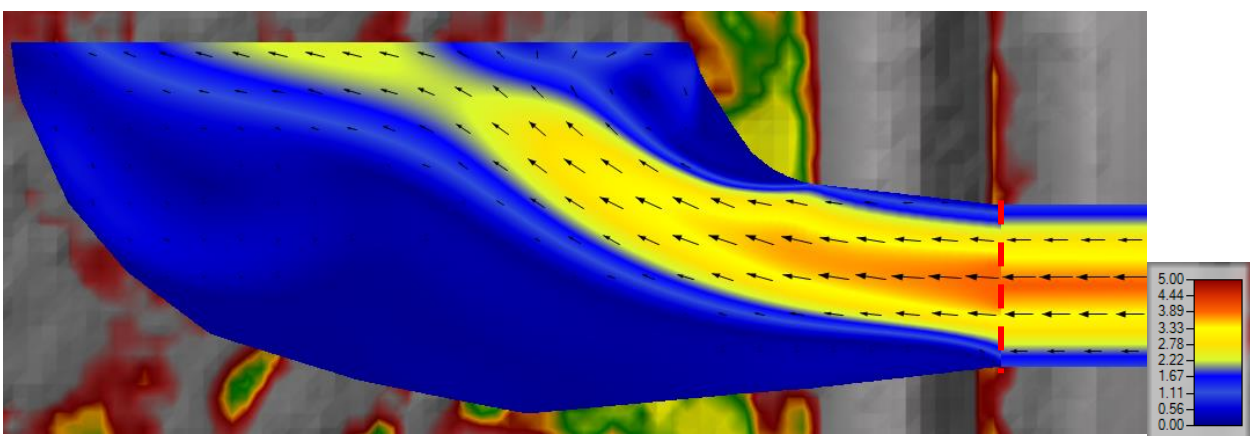


Figure 11. Velocity distribution in the 2D flow area for Scenario 1B (vertical red line denotes the location where the maximum velocity occurred within the 2D area)

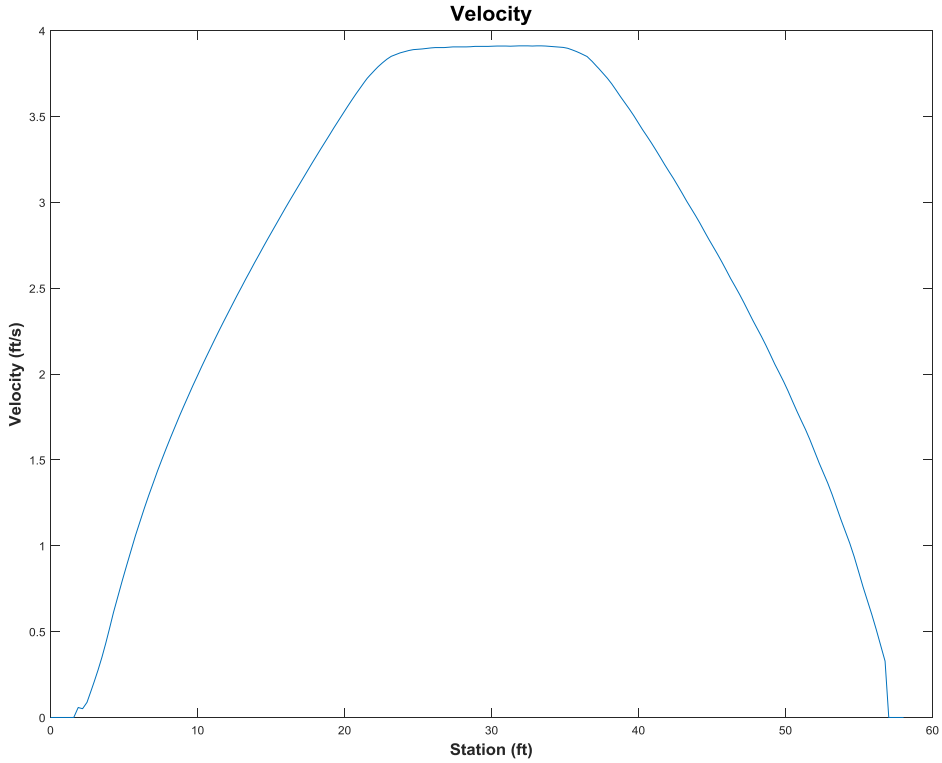


Figure 12. Velocity in the 2D flow area for Scenario 1B (location of maximum velocity is marked by a red line in Figure 11)

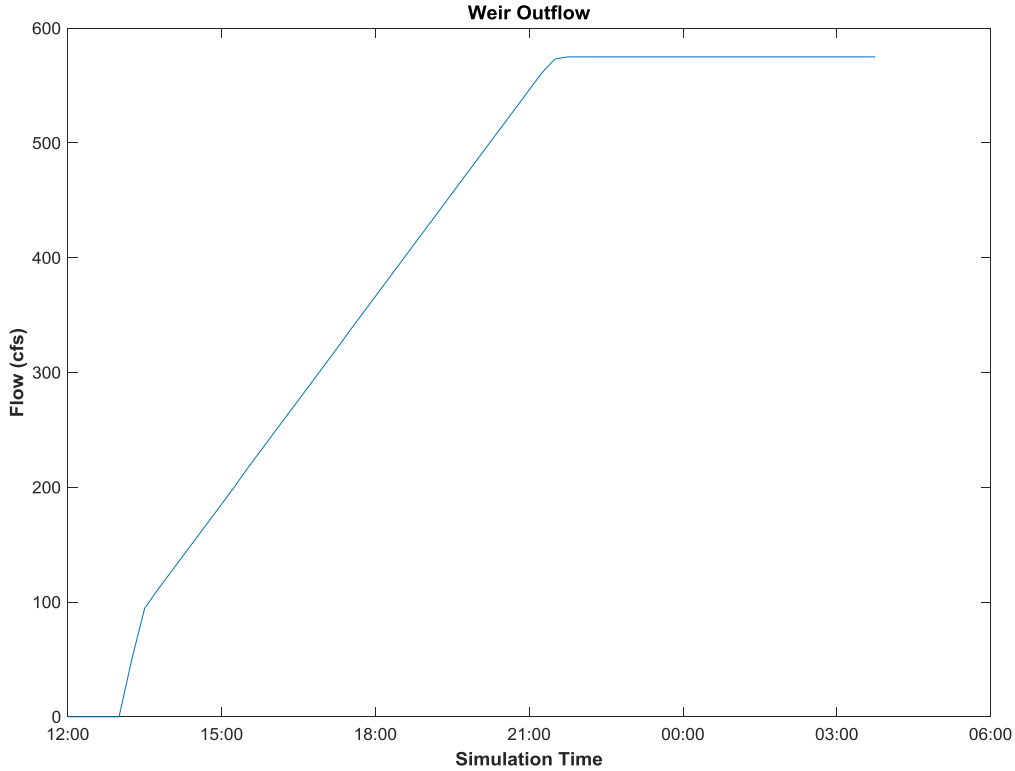


Figure 13. Weir outflow for Scenario 1B

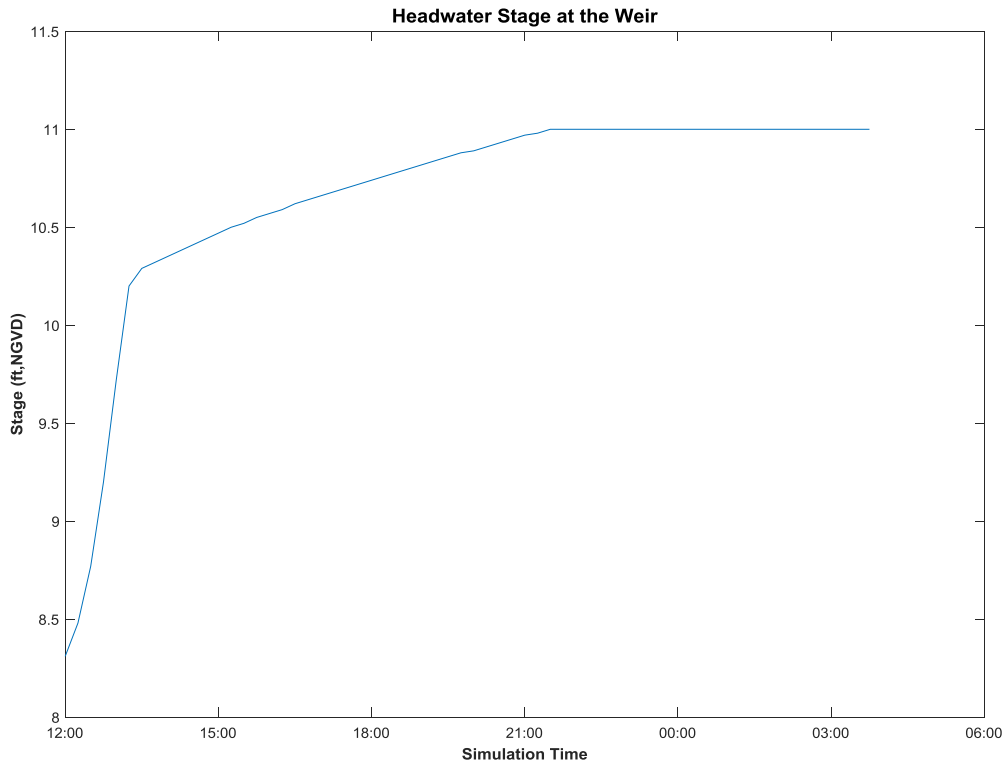


Figure 14. Scenario 1B headwater stage

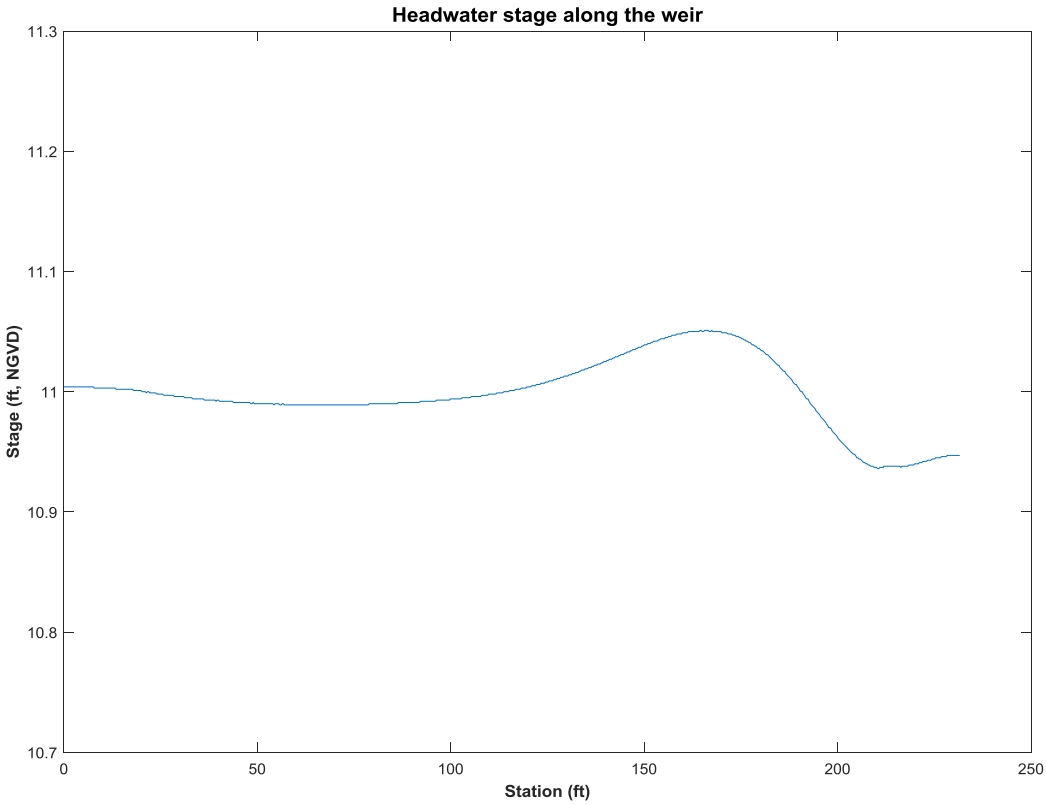


Figure 15. Scenario 1B headwater stage along the weir (station 0 represents the left end of the weir)

3. Scenario 2A

In scenario 2A, a nominal flow of 250 cfs is diverted from the S-332B discharge channel to the NDA while the remainder of the design discharge flows to the downstream weir. It was found that two 5.8 feet by 6 feet reinforced concrete box culverts are needed to convey the design flow rate of 250 cfs to the NDA while adhering to the design constraints mentioned earlier. A culvert headwater stage of about 10.3 feet NGVD was observed at the diversion location.

Figures 16 & **17** display the velocity distribution within the weir head water pool and near the diversion location, respectively. The maximum velocity in the pool is about 2.5 ft/sec. **Figure 18** provides the maximum velocity in the 2D flow area at the diversion location. When the lateral culverts operate to divert the flow towards the NDA, an eddy is formed in the channel. Immediately downstream of the eddy, a maximum velocity of about 4.9 ft/s was observed. Since this velocity exceeds 4.5 ft/s, additional channel reinforcements may be needed in this area.

Figure 19 presents the headwater elevation at the weir and the diversion culverts with respect to the simulation time. **Figure 20** shows the headwater stage along the weir length and along the channel longitudinally at the inlet of the diversion culvert openings. The headwater stage along the length of the weir is observed to be constant with very small variations towards the right end of the weir. A sharp dip is observed in the channel stage where the culverts are located.

Figure 21 shows the outflow from the weir as well as the total flow diverted by the culvert towards the NDA. The diverted discharge rate was about 252 cfs while about 323 cfs continued downstream to the weir.

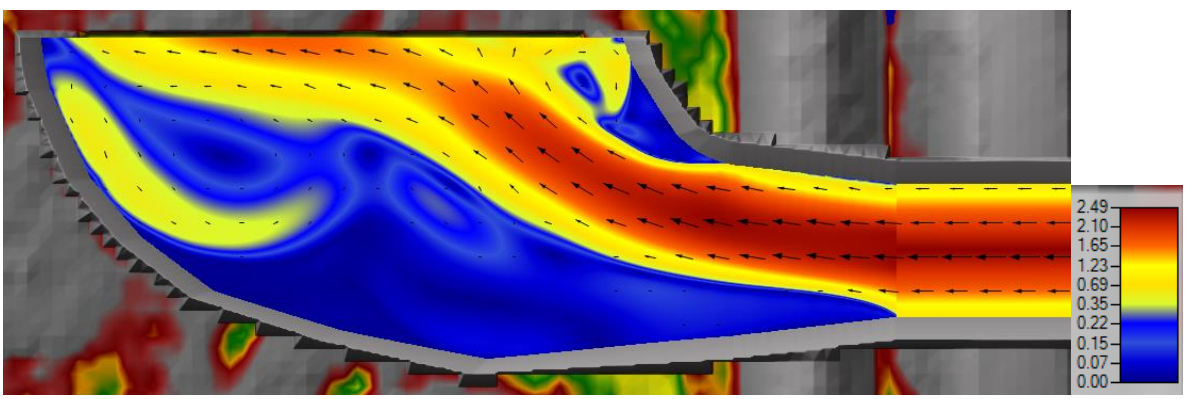


Figure 16. Velocity distribution in the 2D flow area at the pool (Scenario 2A)

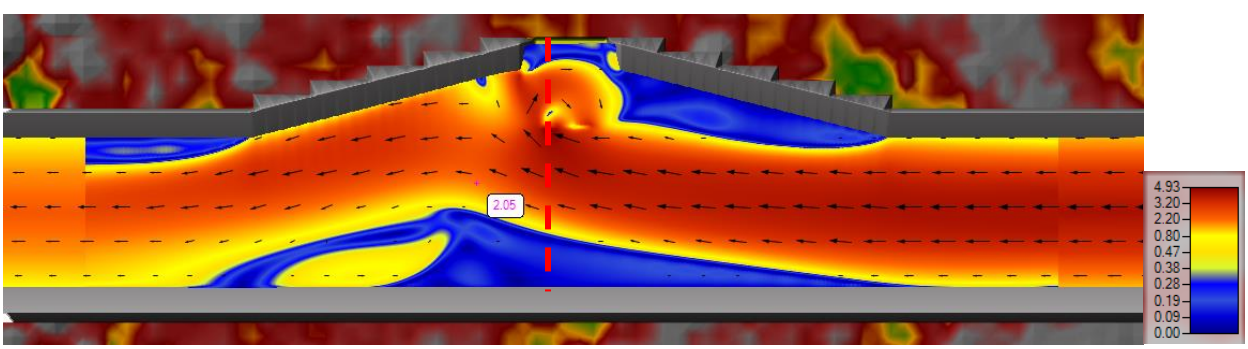


Figure 17. Velocity distribution in the 2D flow area at the diversion location (Scenario 2A)

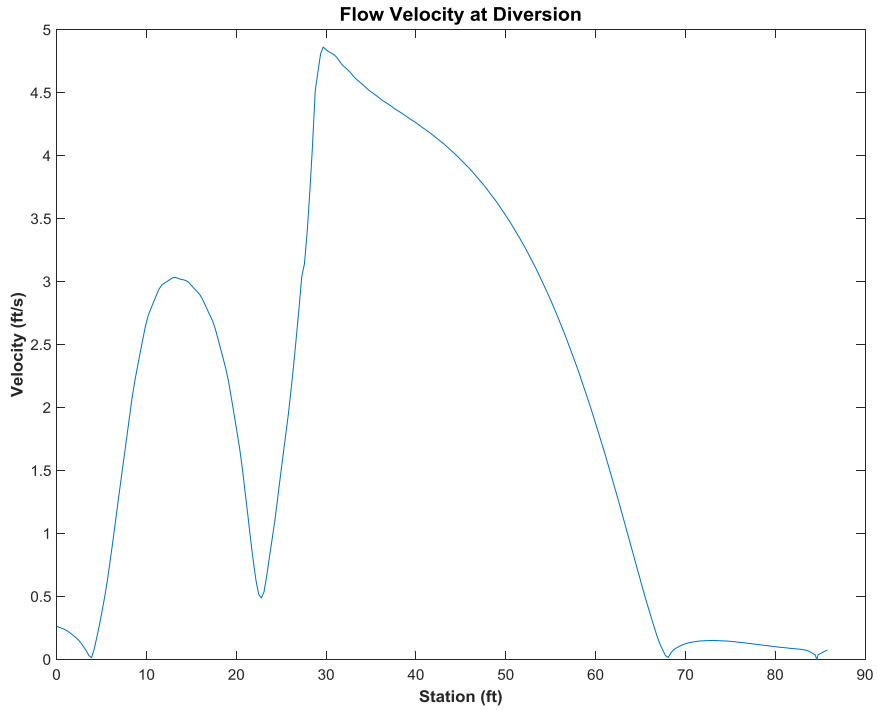


Figure 18. Velocity in the 2D flow area at the diversion location for Scenario 2A (marked by a red line in Figure 17)

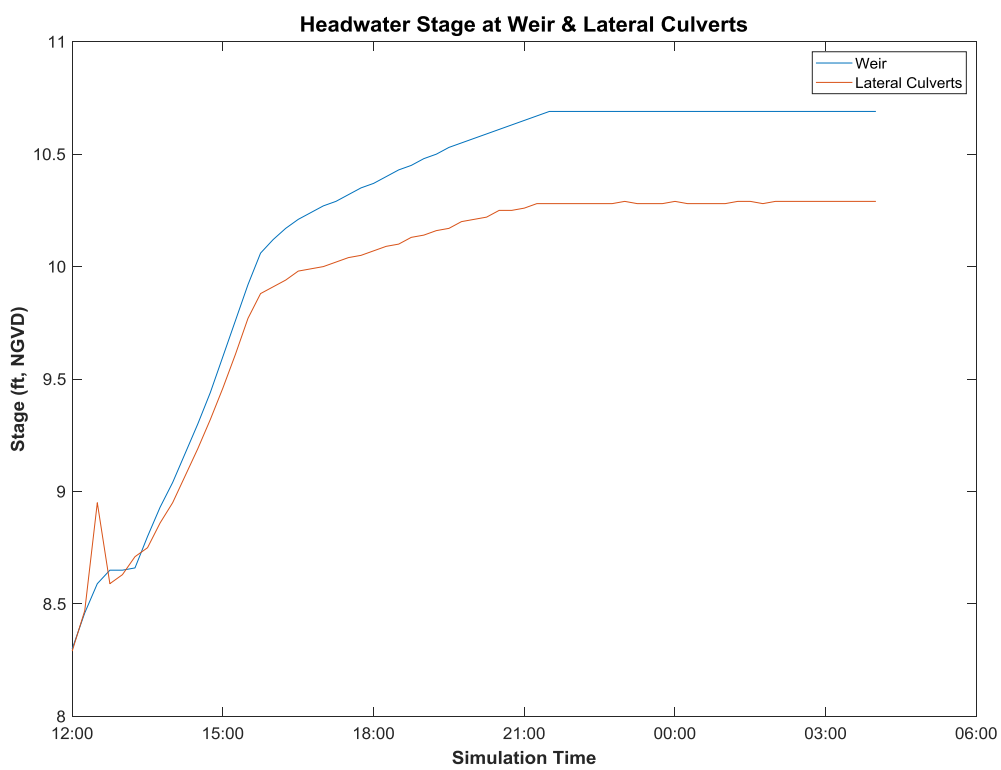


Figure 19. Scenario 2A headwater stage for the weir and the lateral culverts

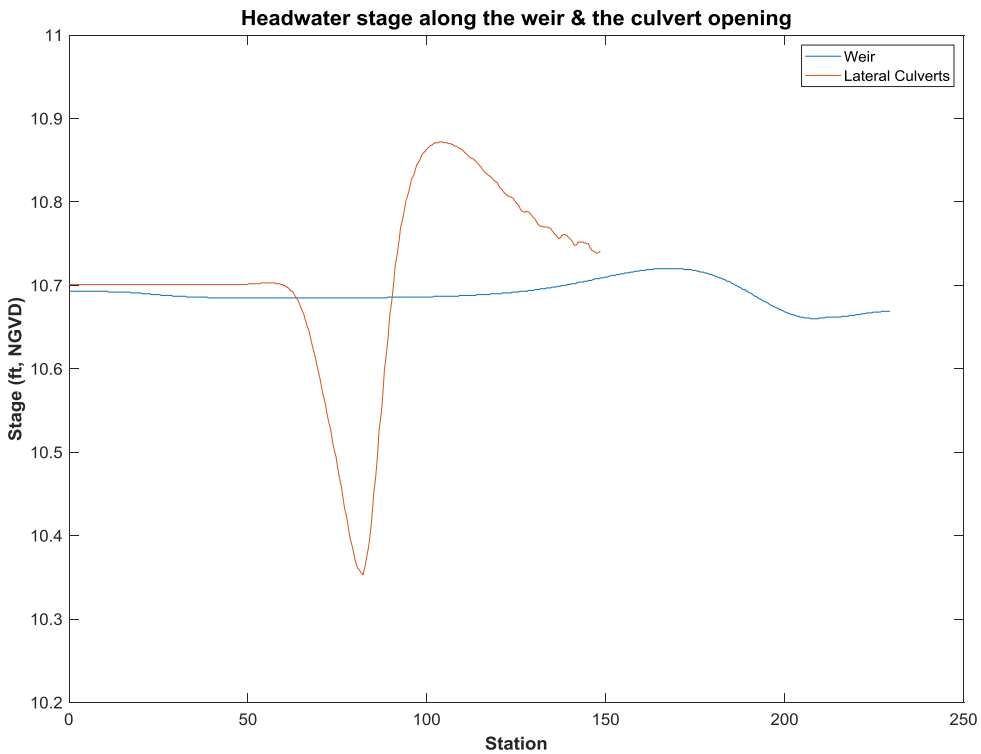


Figure 20. Scenario 2A headwater stage along the weir and the diversion culvert entrances

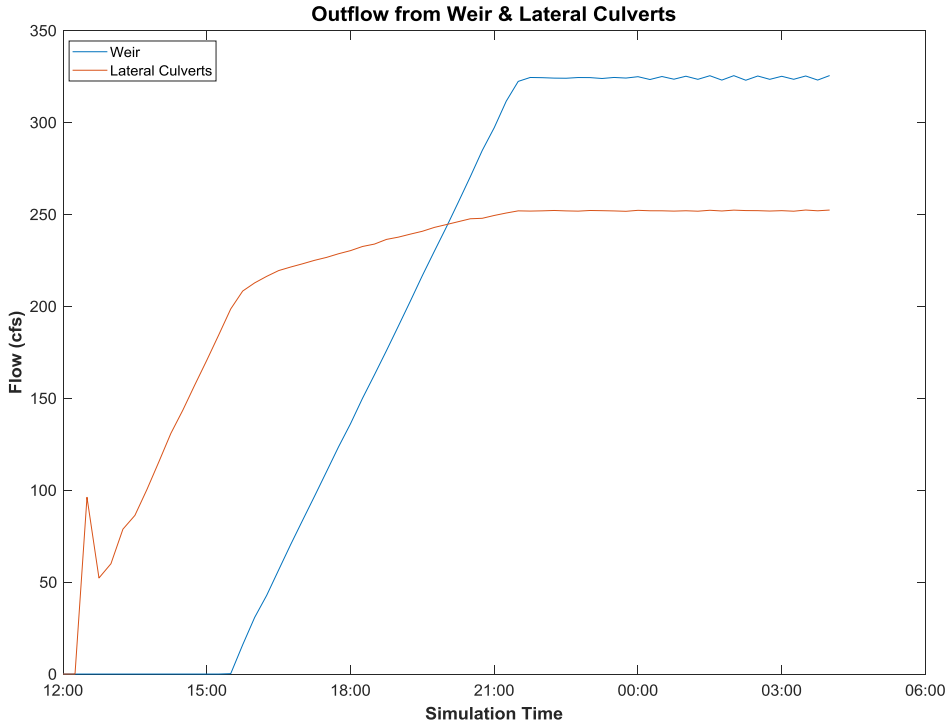


Figure 21. Weir and diversion discharge rates for Scenario 2A

4. Scenario 2B

Scenario 2B is the same as 2A except that an eddy coefficient of 3 is used. **Figures 22** & **23** show the velocity distribution and the maximum velocity in the weir head water pool and within the channel enlargement at the diversion location, respectively. **Figure 24** reveals the maximum velocity at the diversion location which is about 3.9 ft/s in this scenario compared to about 4.9 ft/s obtained in Scenario 2A. **Figure 25** shows the weir outflow along with the total flow discharged by the diversion culverts. The total flow diverted by the culvert barrels is about 257 cfs, compared to the 252 cfs obtained in Scenario 2A. This slight increase in the total flow diverted by the culverts may be due to a reduction in turbulence at the entrance of the culvert barrels when the eddy coefficient is increased to 3. **Figure 26** provides the headwater stage along the weir in addition to the headwater stage in the channel at the inlet of the diversion culvert openings.

5. Scenario 3A

Scenario 3A represents the S-332C discharge channel when the total flow of 575 cfs is conveyed to the SDA. An eddy coefficient of 1 is used in this scenario.

Figures 27 and **28** display the velocity distribution and the maximum velocity, respectively, within the weir head water pool. A maximum velocity of 4 ft/s is observed right at the boundary of the 1D and 2D areas. **Figures 29** and **30** show the weir outflows and headwater stages, respectively, with respect to the model simulation time. **Figure 31** represents the headwater stage along the weir.

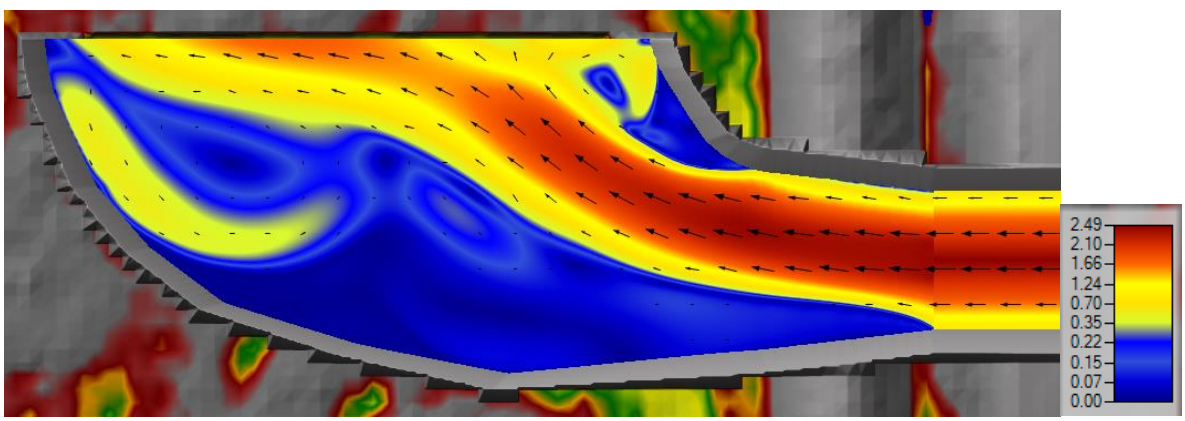


Figure 22. Velocity distribution in the 2D flow area at the pool (Scenario 2B)

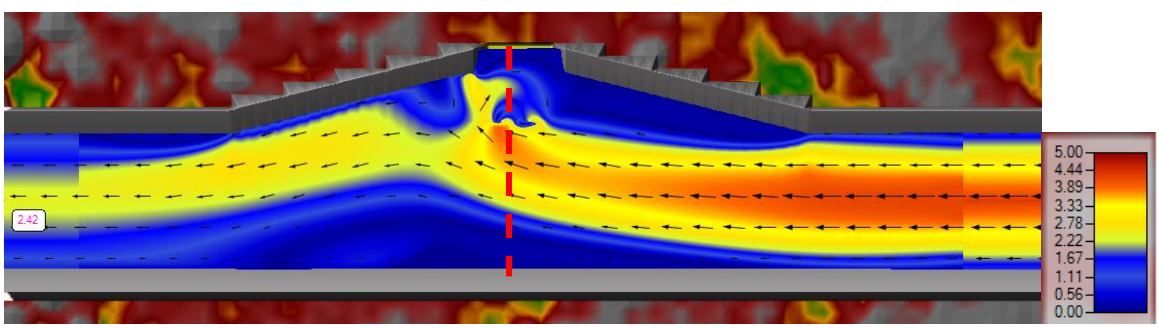


Figure 23. Velocity distribution in the 2D flow area at the diversion location (Scenario 2B)

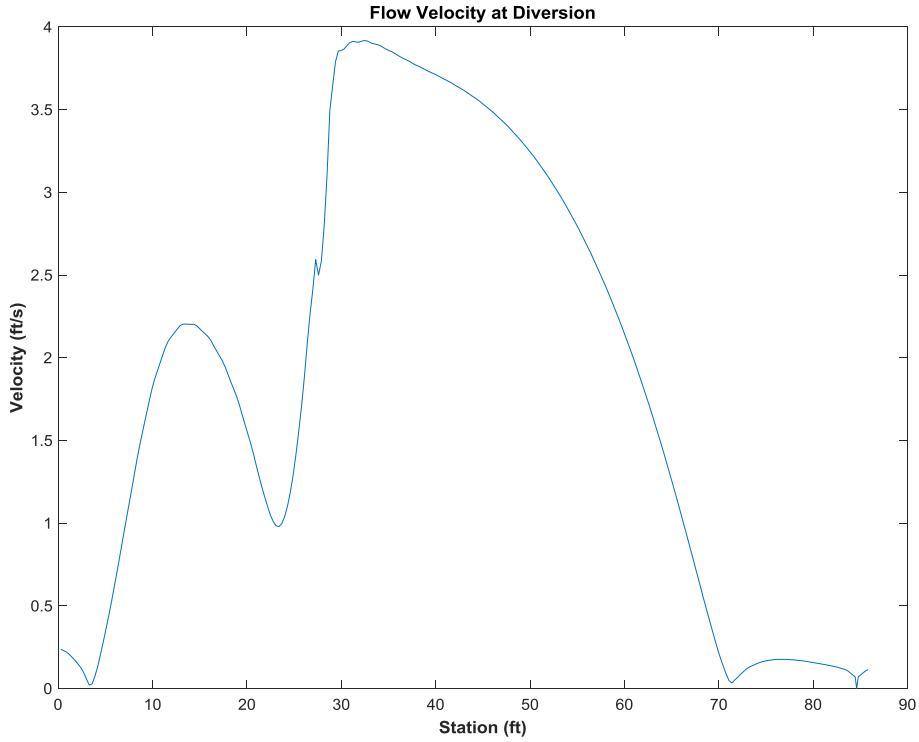


Figure 24. Velocity in the 2D flow area at the diversion location for Scenario 2B (specified in Figure 23 by a red vertical line)

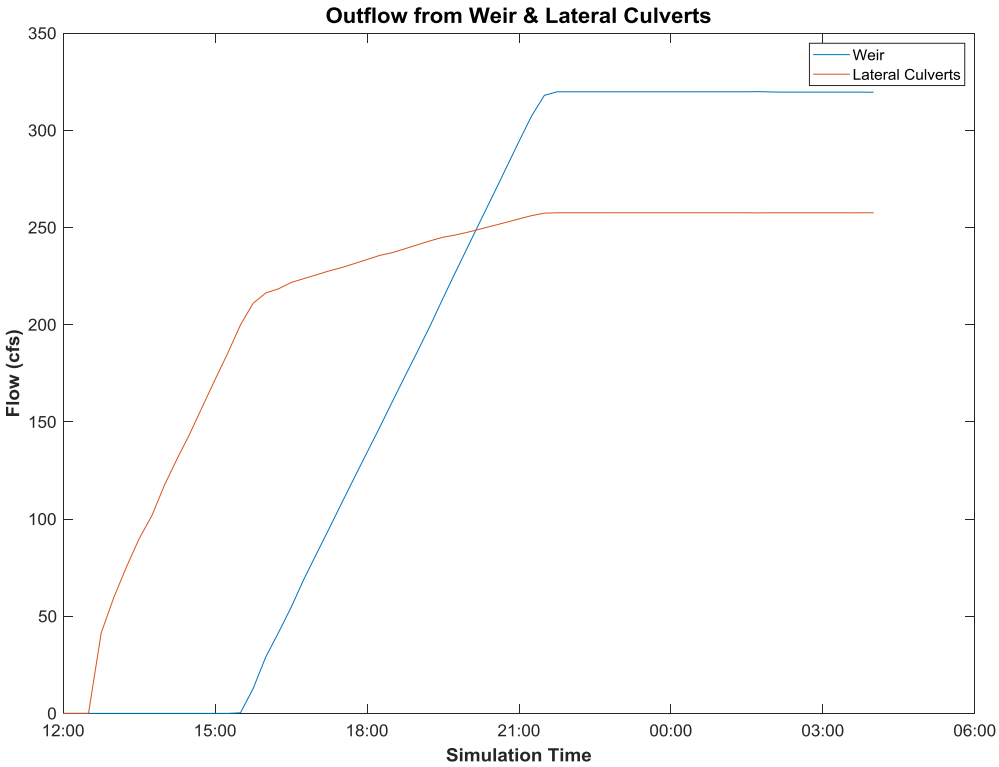


Figure 25. Weir and diversion outflows for Scenario 2B

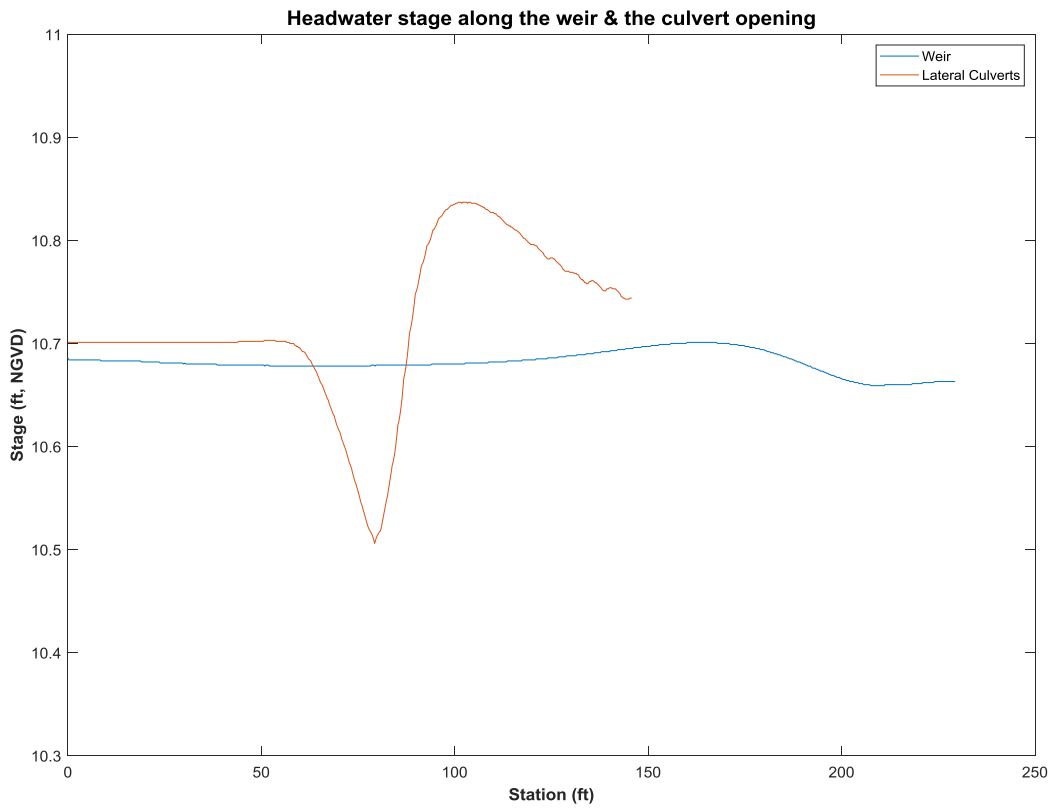


Figure 26. Scenario 2B headwater stage along the weir and the diversion culvert entrances

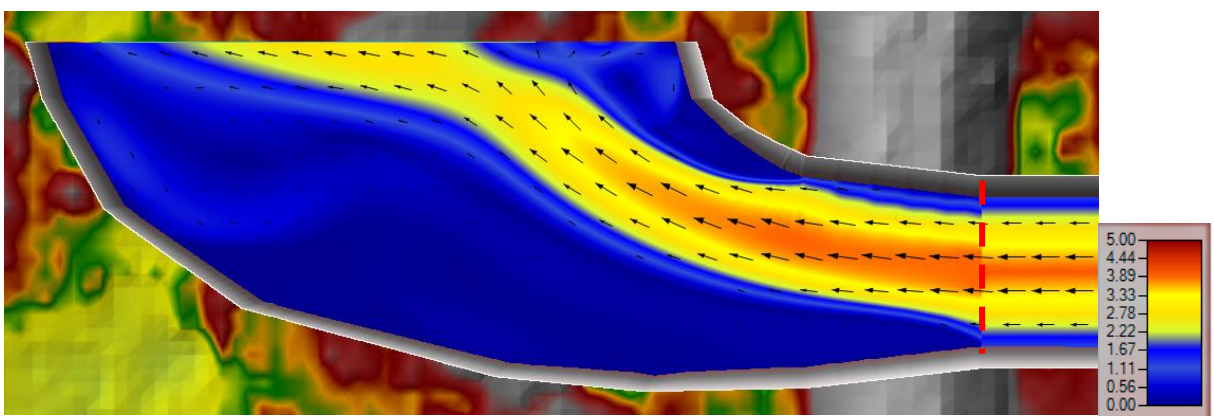


Figure 27. Velocity distribution within the S-332C weir head water pool

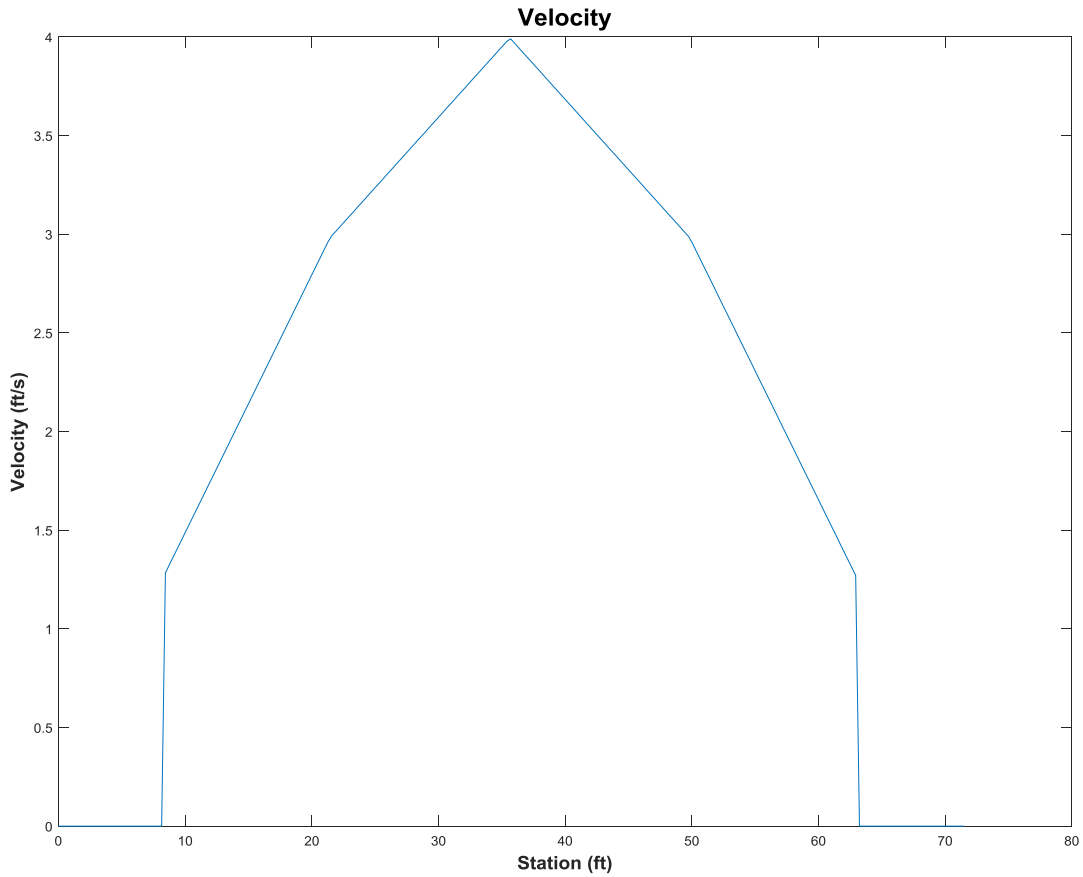


Figure 28. Velocity in the 2D flow area for Scenario 3A (location is marked by a red vertical line in Figure 27)

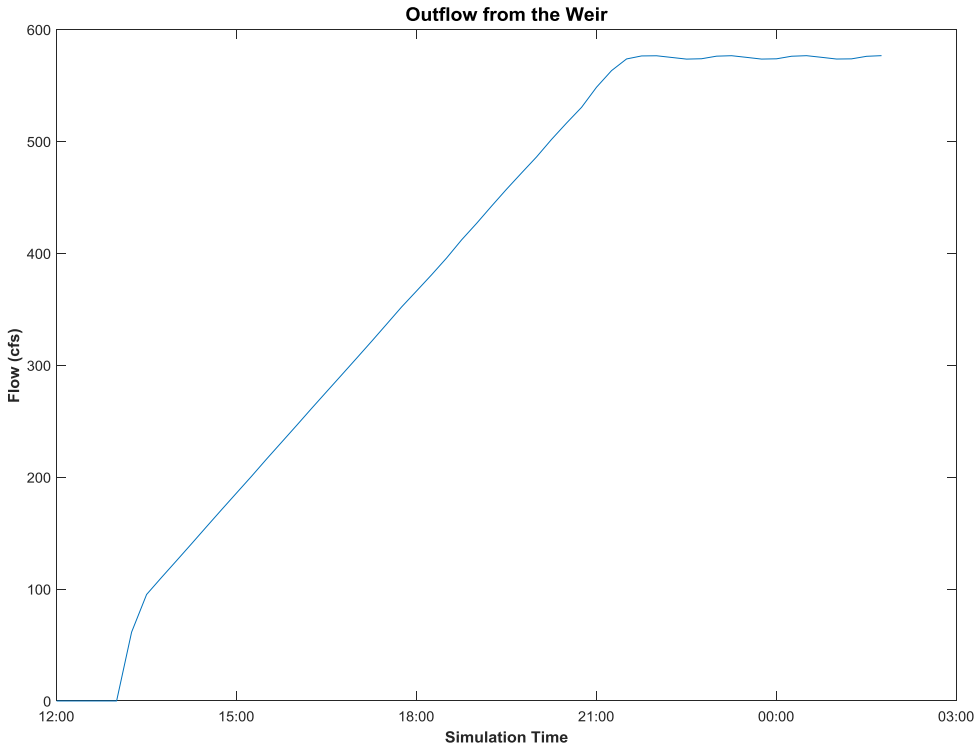


Figure 29. Weir outflow for Scenario 3A

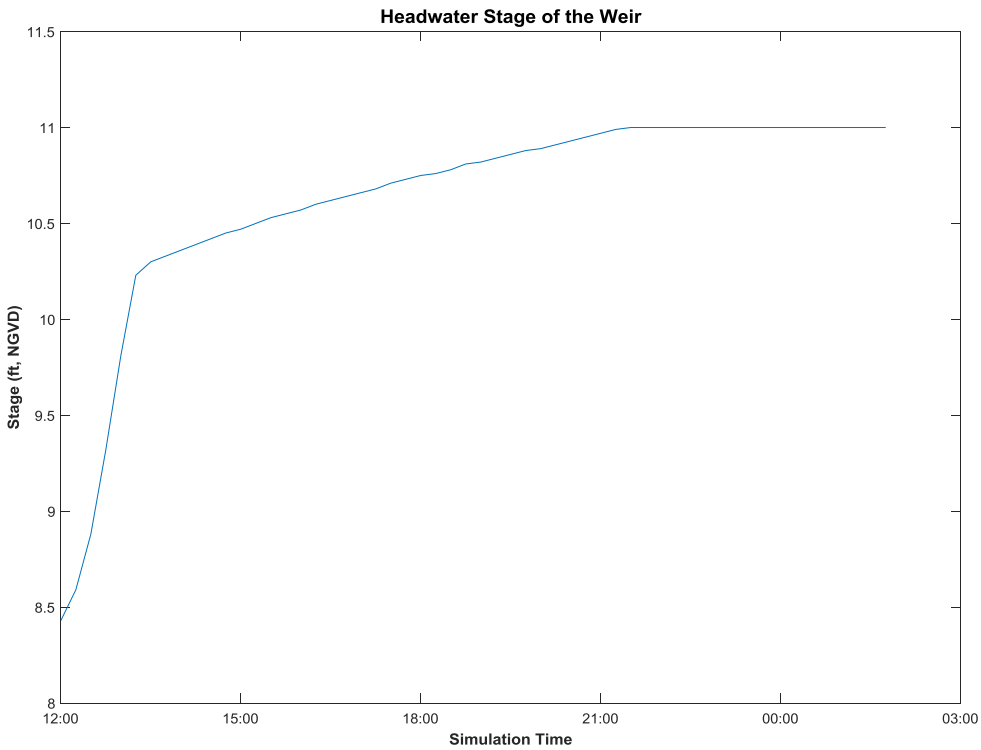


Figure 30. Headwater stage at the S-332C weir

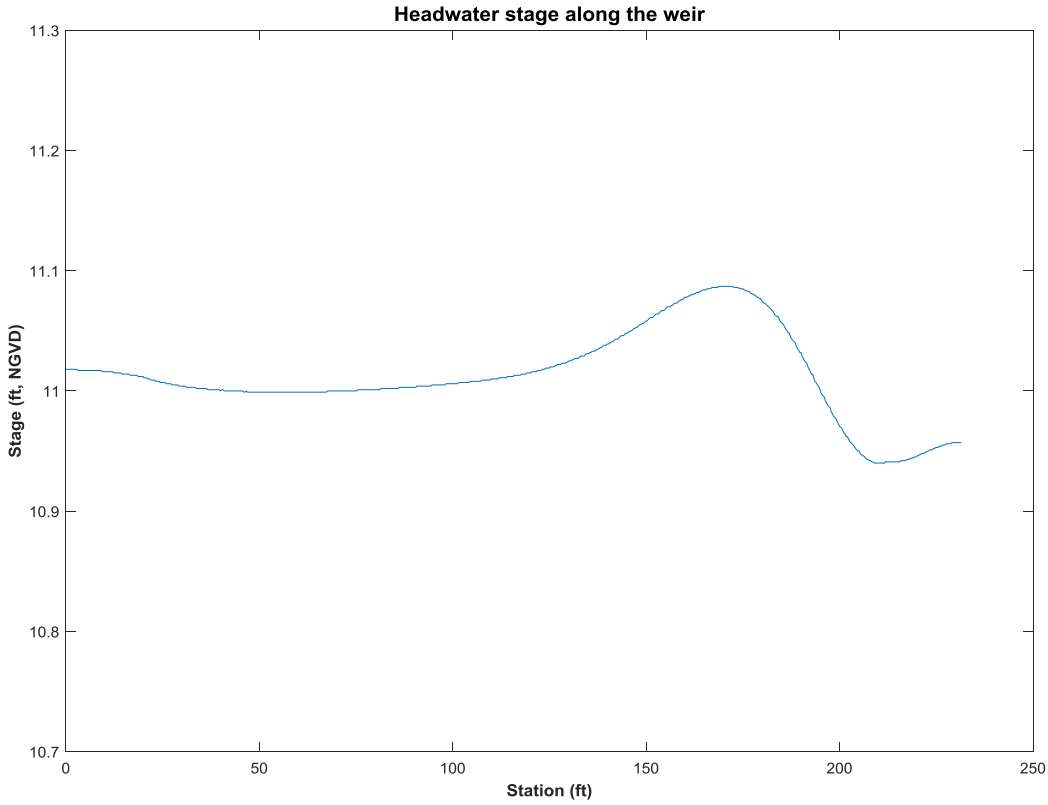


Figure 31. Headwater stage along the S-332C weir (station 0 indicates the left side of the weir)

Table 7 summarizes the results obtained for the 1D river reaches of the S-332C channel.

Table 7. S-332C discharge channel specifications

Design Feature	Description
Headwater stage at the u/s end of the channel (ft, NGVD)	11.33
Mean velocity in the channel (ft/sec)	3.28
Froude No. in the channel	0.32

6. Scenario 3B

Scenario 3B is the same as scenario 3A except that an eddy coefficient of 3 is used. **Figure 32** represents the velocity distribution in the weir head water pool. A maximum velocity of about 4 ft/s was observed at the upstream-most boundary of the 2D flow area. This maximum velocity is close to the one obtained in Scenario 3A. **Figure 33** represents the headwater stage along the weir length which demonstrates that the headwater stage is constant along most of the weir length with some slight fluctuations towards the right side of the weir.

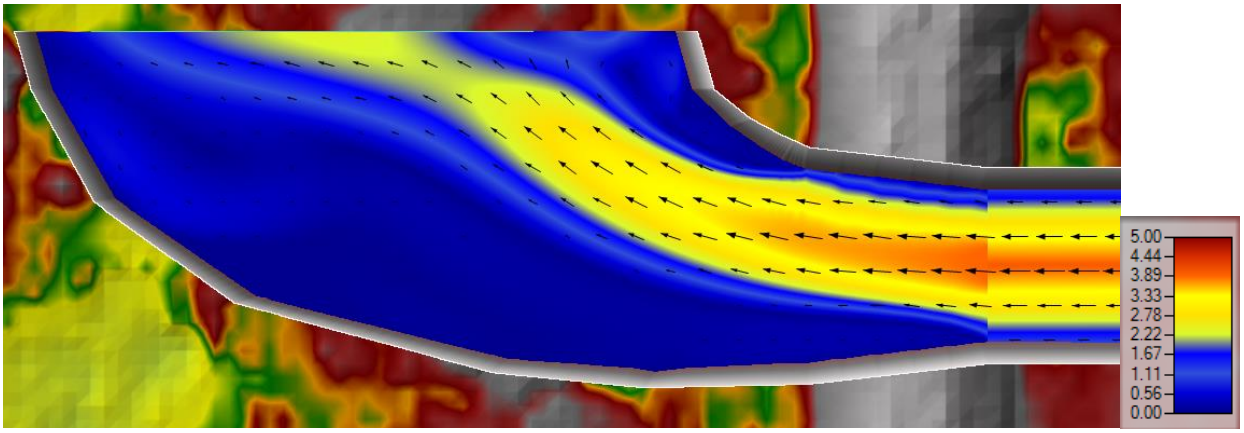


Figure 32. Velocity distribution within the 2D area of the S-332C channel

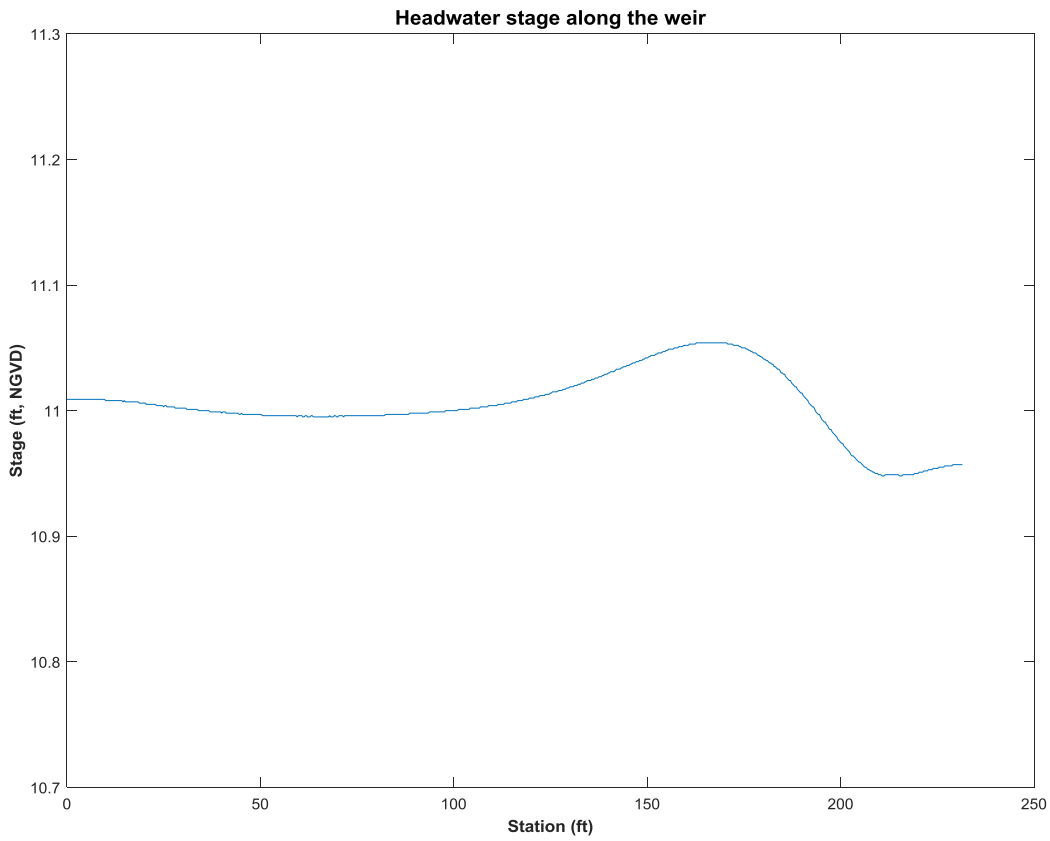


Figure 33. Headwater stage along the S-332C weir (station 0 indicates the left side of the weir)

7. Scenario 4

Scenario 4 addresses conditions within the existing S-332B flow-way when all of the flow is conveyed to the SDA only. An eddy coefficient of 2 was used in this scenario since, at the selected grid resolution, it was the maximum value of this parameter for which a stable solution could be achieved. Additionally, an estimated seepage loss of 1 cfs per acre per foot of head difference (P. F. Linton, personal communication) was applied to the flow-way to account for seepage losses to L-31N. The head difference is taken between

the water stage in the flow-way and the L-31N canal stage. Given an average flow-way stage of 8.75 feet (see below) along with a L-31N stage of 4.5 feet (Wilsnack, 2018), this unit seepage rate results in a total flow-way seepage loss of about 22 cfs. From this total seepage rate, 16 cfs was removed from the 1D river reach while the remaining 6 cfs was removed from the inflow rate to the flow-way.

Figure 34 represents the velocity distribution and vectors between the weir and the location downstream of the bend where flow becomes essentially one-dimensional.

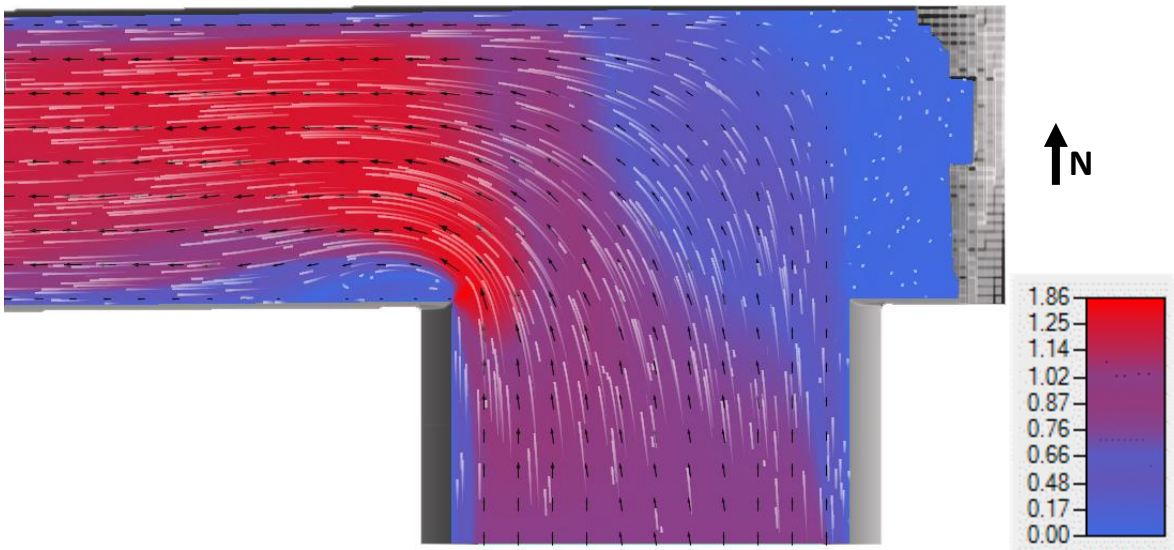


Figure 34. Velocity distribution within the existing flow-way at 575 cfs (less estimated seepage)

In **Figure 34**, a maximum velocity of about 1.9 ft/s occurs just to the right of the flow separation zone that occurs along the southern edge of the flow-way, just after the 90-degree turn at the end of the proposed entrance channel. While the velocities determined in this simulation should not pose a problem for the flow-way perimeter, the results shown indicate that a more favorable flow pattern can be obtained by increasing the radius of curvature of the 90-degree intersection of the flow-way banks with the proposed inflow channel banks.

Figure 35 provides the centerline water surface profile between the SDA and the east end of the flow-way. At the design discharge rate, the model results indicate that the total head loss through the flow-way will be about 0.5 foot, with stages near the upstream end of the flow-way being close to 9 feet NGVD. Consequently, discharges over the proposed weir under design conditions will be unsubmerged.

8. Scenario 5

In Scenario 5, discharges into the flow-way are conveyed to both the SDA and the NDA. **Figure 36** provides the flow through the diversion culverts while **Figure 37** shows the computed headwater stage at these culverts. It is evident that with a diversion rate of about 250 cfs, stages near the east end of the flow-way will be approximately 8.6 feet NGVD. Given a design NDA stage of 8.5 feet, the resultant design head differential for the diversion culverts is only about 0.1 foot. This implies that diverting 250 cfs directly from the S-332B flow-way to the NDA is not likely to be feasible and therefore is not recommended.

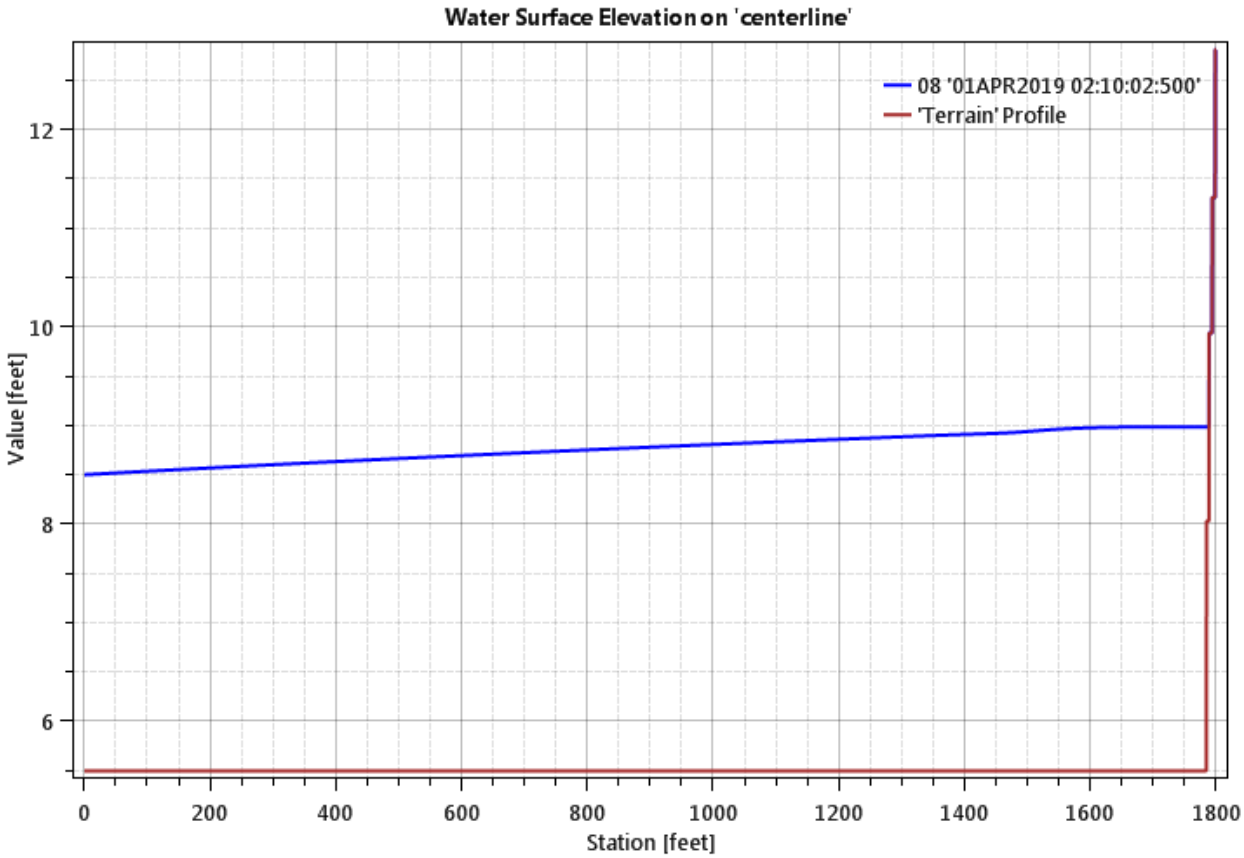


Figure 35. Water surface profile along the centerline of the S-332B flow-way at design discharge minus seepage (Station 0 denotes the downstream end).

9. Eddy Formation Zones

Figure 38 illustrates the eddy formation zones within the 2D flow areas in all the scenarios. The channel expansion from 15 feet to 192 feet, is substantial and occurs over a relatively short distance. This leads to the eddy formations shown. In the areas where low velocities are observed within these eddy zones, sediment deposition may occur. Therefore, periodic maintenance may be required. Further examination of **Figure 38** reveals that the length of eddy zone (denoted by red arrows) is slightly longer when an eddy coefficient of 1 was used instead of 3. Similarly, flow velocities are slightly higher in the scenarios based on an eddy coefficient of 1 compared to those that utilized a value of 3. Nonetheless, the maximum velocities and weir headwater distributions discussed earlier indicate that the adverse flow features identified in this study should not inhibit the functionality of the proposed hydraulic design for the pump station discharge channels.

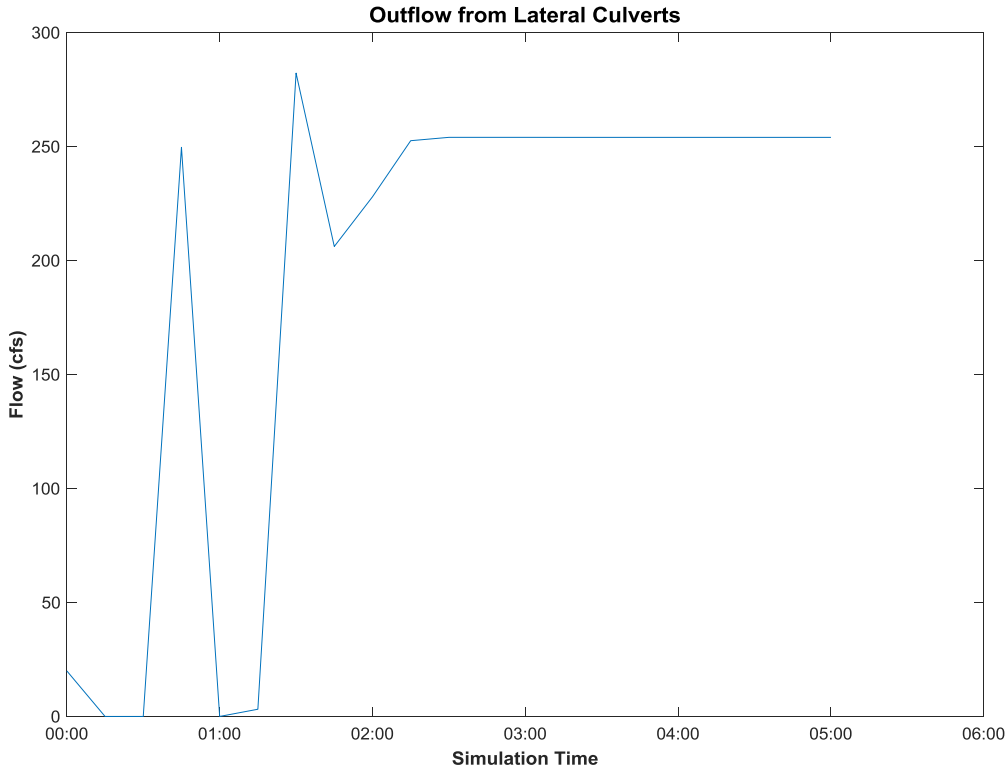


Figure 36. Diversion outflow for Scenario 5

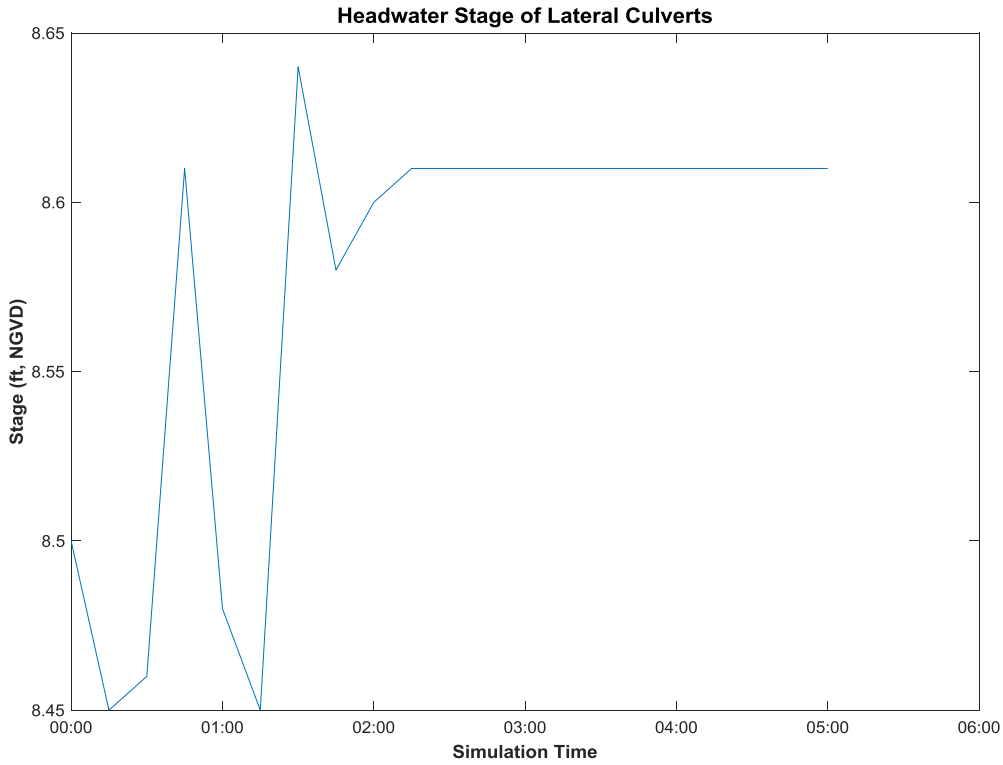
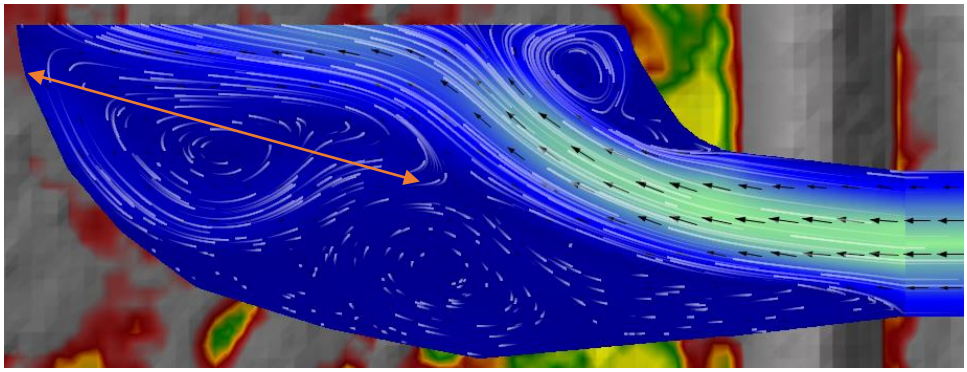
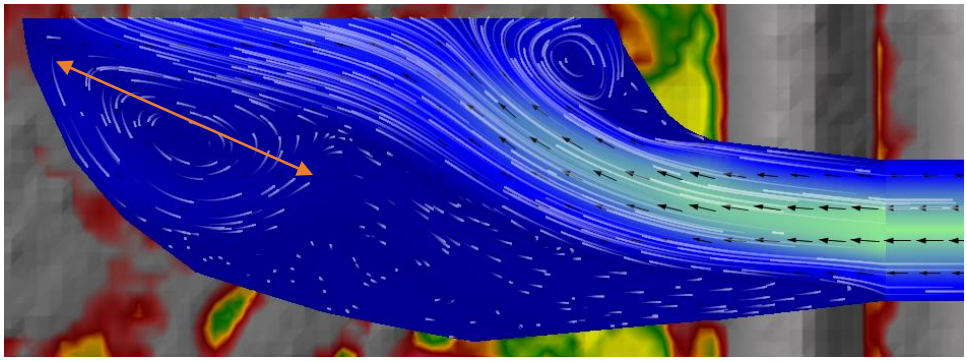


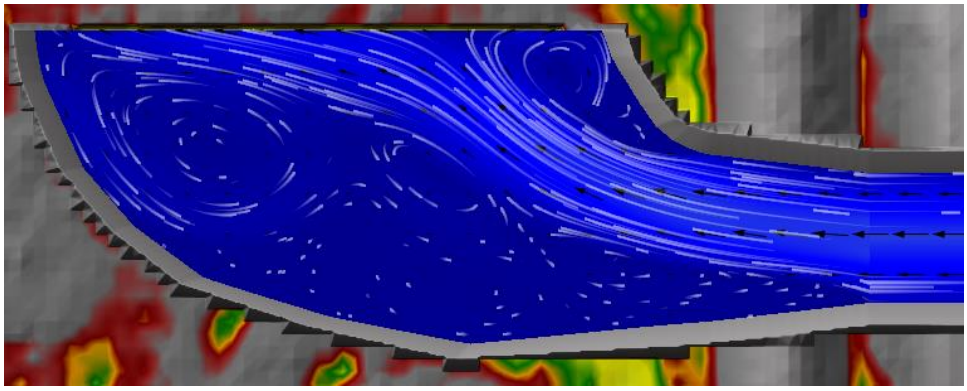
Figure 37. Headwater stage at the diversion culvert entrances for Scenario 5



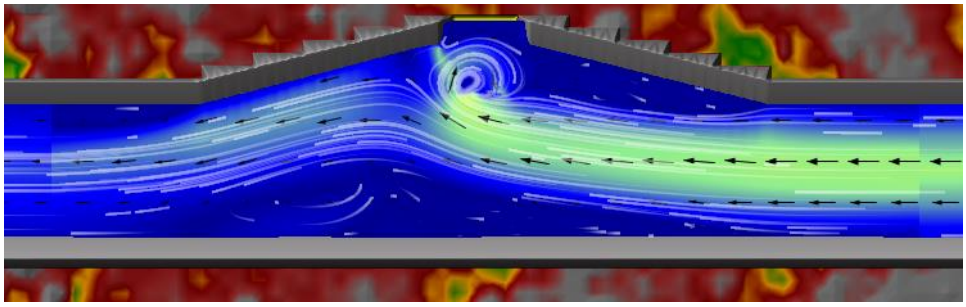
Scenario 1A



Scenario 1B

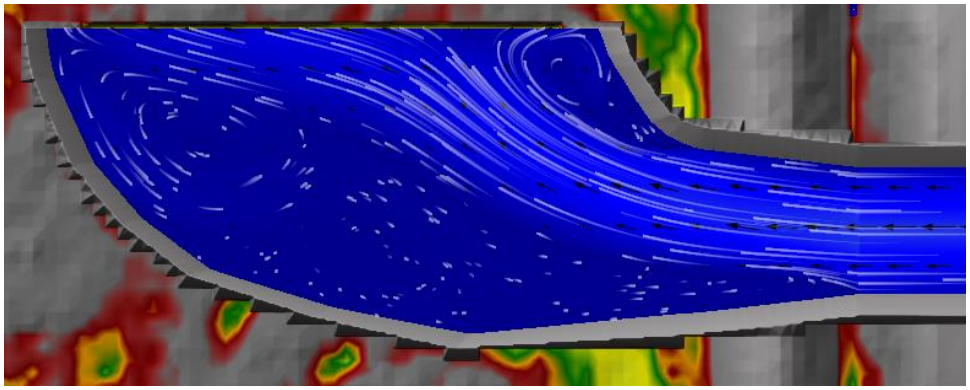


Scenario 2A

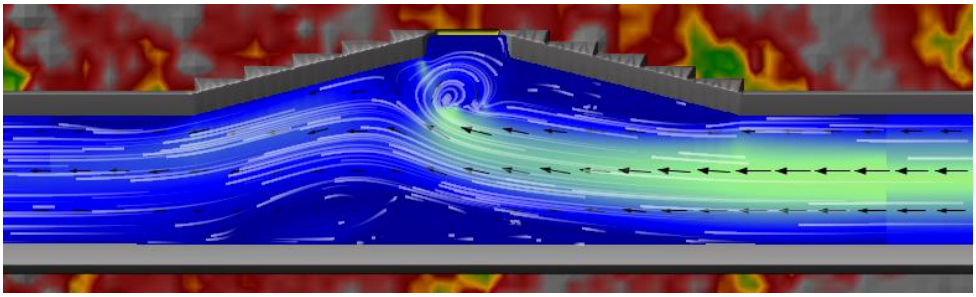


Scenario 2A

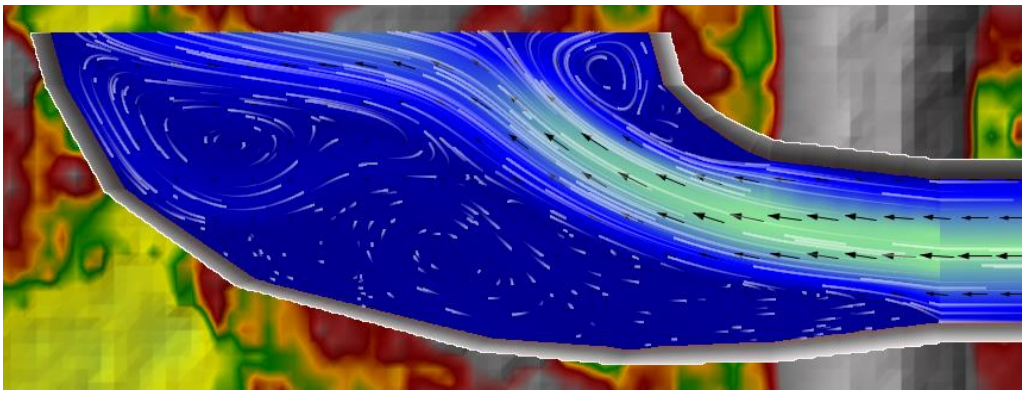
Figure 38. Eddy zone formations



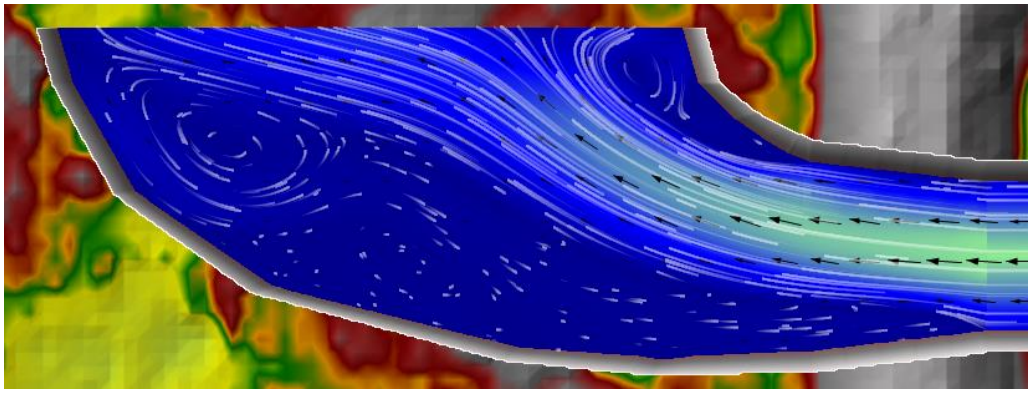
Scenario 2B



Scenario 2B



Scenario 3A



Scenario 3B

Figure 38. Eddy zone formations (Con't)

SUMMARY AND CONCLUSIONS

This report summarizes the preliminary designs of the S-332B and S-332C discharge channels associated with an alternative design referred to as the Parallel Weir Option. These designs were prepared in enough detail to support a preliminary cost estimate of the project.

The above-ground discharge channel for the S-332B pump station extends approximately 1,900 feet west of the proposed pump station location towards the existing flow-way. In model Scenario 1A where all pumped discharges are conveyed to the SDA, the associated upstream headwater stage of the channel was computed to be about 11.33 feet NGVD. Under these conditions the mean velocity in the main channel was about 3.13 ft/s, while a maximum velocity of about 3.9 ft/s was observed within the weir head water pool. The required width of the main channel bottom was determined to be 15 feet, which expands near its downstream end to width of 192 feet at the terminal weir. The weir crest was set at an elevation of 10.0 feet NGVD. To divert a design flow of 250 cfs to the NDA, two box culverts with a span of 5.8 feet, a rise of 6 feet and a length of 100 feet are proposed in diversion Option 1. In diversion Option 2, six concrete box culverts 8 feet wide by 6 feet high and 275 feet long are required. All diversion culverts have both inlet and outlet inverts set at 6.0 feet NGVD. The headwater elevation of the culverts was computed to be about 10.3 feet NGVD for Option 1 and 8.62 feet NGVD for Option 2. A design tailwater elevation of 8.5 feet NGVD was applied to the North Detention Area in both options. Due to the low head differential in Option 2, diverting 250 cfs directly from the S-332B flow-way is not recommended.

When the design diversion rate is removed from the channel through lateral culverts, an eddy was observed near the diversion location. The maximum velocity at this location can range from 3.9 ft/s to 4.9 ft/s when the diversion culverts are operated. It therefore may be necessary to provide additional reinforcements to the channel perimeter over a length of about 200 feet at the diversion location.

The maximum velocity in the weir head water pool can vary, depending on how the effects of turbulence are accounted for and can be as high as 4 ft/s at the upstream boundary of the 2D area. Similarly, the approach velocity to the weir can range from approximately 2.3 ft/s to about 3 ft/s depending on the effect of turbulence. This can be investigated further during later design stages.

The design of the S-332C discharge channel is essentially the same as the S-332B channel design without the diversion facility. This design utilizes the same weir and the same discharge channel dimensions without any channel bottom width extension to accommodate the NDA diversion. The stage at the eastern end of the S-332C discharge channel was determined to be 11.33 feet NGVD.

Downstream of each weir, a channel with a bottom width of 192 feet should be constructed to connect the weir to the existing flow-way. Under design conditions a plunging jet occurs as the flow passes over the weir while a surface roller about 13.7 feet long is formed downstream of the weir. A maximum velocity of about 14 ft/s can occur along the downstream side of the weir within the plunging jet. The channel connecting the weir with the flow-way should be lined with concrete to at least the end of the surface roller.

In both designs, the channel expands somewhat rapidly near its downstream end over a relatively short distance. Due to the sudden expansion, some large eddies were observed in the weir head water pool. Due to the low velocities within certain regions of these eddies, there is a higher possibility of sediment deposition, possibly necessitating more frequent maintenance.

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Appendix A - Detailed Calculations

Weir Design and Rating Curve Development:

1. Hydraulic Design of S332B&C-Parallel Weir Option.xlsx

HECRAS files for S-332B:

1. S332B.p25.zip (Scenario 1A)
2. S332B.p28.zip (Scenario 1B)
3. S332B.p26.zip (Scenario 2A)
4. S332B.p27.zip (Scenario 2B)

HECRAS files for S-332C:

1. S332C.p03.zip (Scenario 3A)
2. S332C.p04.zip (Scenario 3B)

HECRAS files for the existing flow-way:

1. Existing_Flowway.p08.zip (Scenario 4)
2. Existing_Flowway.p14.zip (Scenario 5)