PRELIMINARY HYDRAULIC DESIGN OF THE DISCHARGE CANALS FOR PUMP STATIONS S-332B & S-332C

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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) and the North Detention Area (NDA) of the eastern Everglades National Park in Miami-Dade County. It is proposed that each of these pump stations be relocated about 300 feet to the south and west from their current location. In order to convey each pump station's discharge of 575 cfs from its proposed location to the SDA, an above-ground channel connecting each pump station to the SDA is proposed. These proposed channels extend approximately 3600 feet and 3400 feet from pump stations S-332B and S-332C, respectively, to the SDA in the east-west direction. The design head water stage for each channel at its eastern end is 10.0 feet NGVD while 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 to the North Detention Area (NDA) when S-332B is discharging at capacity. Therefore, it will be necessary to maintain a certain range of stages 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 a channel headwater stage of 10.0 feet NGVD at its eastern end. A weir is proposed at the downstream end of the channel to achieve this objective.

For both S-332B and S-332C, the proposed design is comprised of a trapezoidal channel with a bottom width of 36 feet and 4H:1V side slopes. Additionally, a trapezoidal broad crested weir with 2H:1V side slopes and a fixed crest elevation of 8.2 feet NGVD is proposed near the downstream end of the channel, about 100 feet upstream of the SDA. The weir length required to pass the design discharge of 575 cfs was determined to be 265 feet. Since the required weir length was larger than the channel width at the weir crest elevation, a channel transition about 820 feet long is also included in the design. The associated headwater stage at the eastern end of the channel was determined to be 9.7 feet NGVD.

The S-332C discharge channel was designed with and without a weir near its downstream end. The proposed design is comprised of a channel bottom width of 36 feet and 46 feet, respectively, for the scenarios with and without the weir. The associated headwater stages at the eastern end of the channel were determined to be 9.7 feet NGVD and 9.6 feet NGVD, respectively, for the scenarios with and without the weir.

For the S-332B discharge channel, lateral diversion culverts were sized to divert 250 cfs by gravity into the NDA. These culverts connect to the channel about 320 feet west of S-332B, with inverts set at the same elevation as the channel invert elevation. It was determined that a total of five reinforced concrete culvert barrels with a span of 6 feet and a rise of 3 feet are needed. Alternatively, either seven HDPE barrels 5 feet in diameter or nine barrels 4 feet in diameter can also be used. Since HDPE pipe is generally easier to transport and install, HDPE conduits may be preferable over cast-in-place RCP culverts.

Cost savings in the current design can be achieved if the design head water stage for each channel is increased above 10 feet NGVD. Consequently, for cost comparison purposes alternative design criteria (Option 1) were considered in addition to the original design criteria mentioned above. Option 1 design criteria are essentially the same as the original design criteria except that the maximum allowable headwater stage for the discharge channel is 12.0 feet. An analysis of this alternative revealed that a channel bottom width of only 15 feet would be needed to pass the design discharge of 575 cfs while the weir length was reduced to 78 feet if its crest was set at 9.2 feet NGVD. Under these conditions the length of the required transition was reduced to 226 feet for both S-332B and S-332C discharge channels.

The hydraulic performances of the S-332B discharge channel and diversion conduit designs discussed above were examined for a pump station discharge rate of 650 cfs. Such a scenario is possible when all the available pumps are running. At this higher flow rate, the stage at the eastern end of the channel was

estimated to be 9.9 ft NGVD for the original channel design. Similarly, the headwater stage associated with the channel design for Option 1 would be 11.6 ft NGVD at this higher flow rate.

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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) and the North Detention Area (NDA) located in the eastern 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, where in each case the permanent pump station will be situated several hundred feet to the south and west of the current pump station location. An above-ground channel connecting the tail water pool of each pump station to the SDA is proposed to convey its design outflow of 575 cfs. 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). To achieve this design objective, a weir is proposed at the downstream end of the S-332B outlet channel, about 100 feet upstream of the SDA. Lateral gated culverts are proposed to be tied-in to the S-332B discharge channel to divert the 250 cfs to the NDA on an as-needed basis.

The objective of this task was to develop conceptual designs of the required components of the pump station discharge channels needed to pass the design discharge of 575 cfs to the SDA while satisfying the design constraints. These constraints are:

1. A maximum headwater stage of 10.0 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 flow velocity in each channel is 4.5 feet/sec.

3. Additionally, for the S-332B discharge canal, a minimum headwater stage of 9.0 feet NGVD must be maintained at the weir to provide sufficient head for diverting 250 cfs to the NDA through the proposed gated culverts. For the proposed diversion culverts, the tailwater stage at the intake channel of the NDA is taken to be 8.5 feet NGVD.

4. The invert elevation of the discharge channel and all diversion conduits 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.

An alternative design option was also considered for cost comparison purposes. In this option (Option 1), the maximum allowable stage at the upstream end of the channel is 12.0 feet NGVD as opposed to the stage of 10.0 feet NGVD specified for the original design.

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.

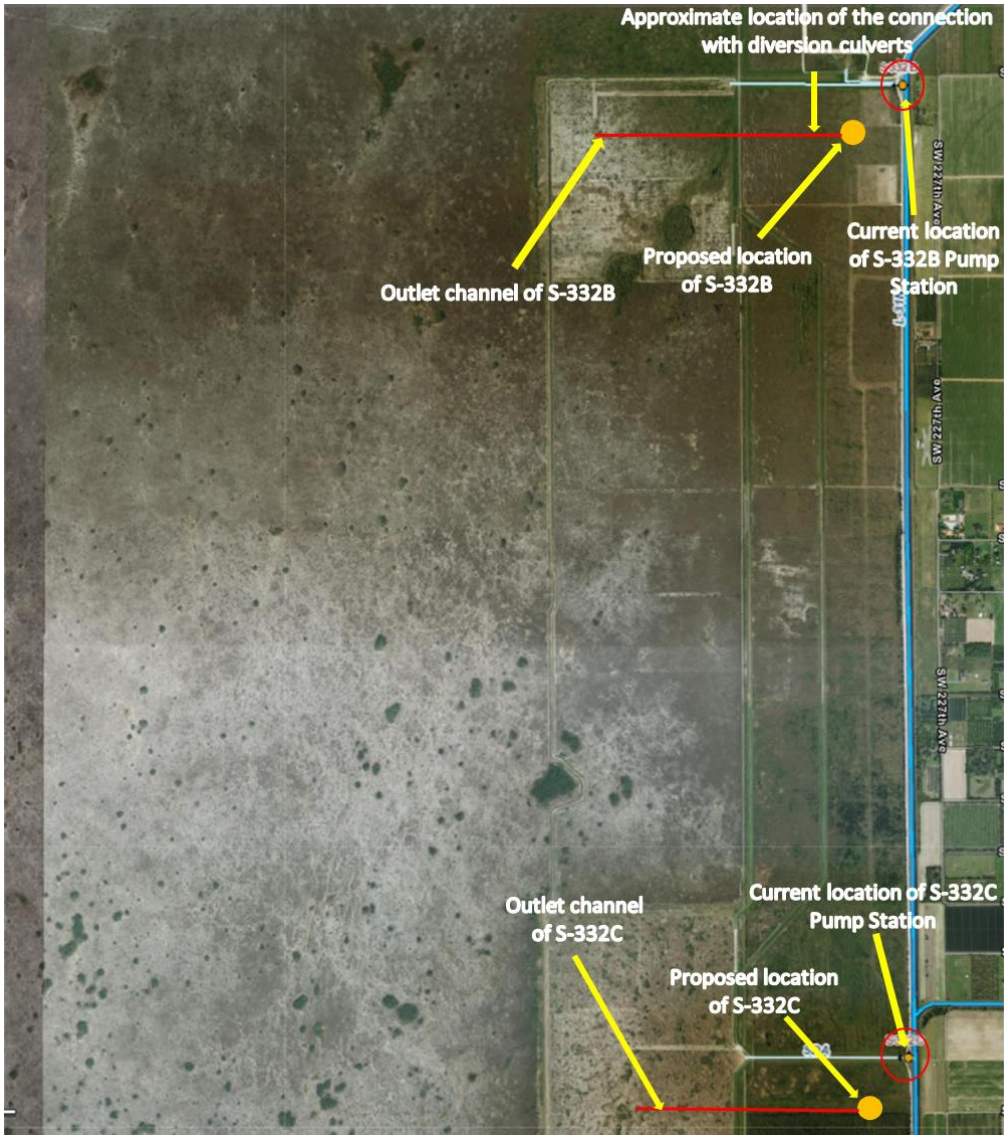

Figure 1. Configuration of the proposed facilities for relocated pump stations S-332B and S-332C

DESIGN OF SYSTEM COMPONENTS

Detailed descriptions of the various components of the discharge facility design are provided in the following sections. These components include the channel, the weir and the transitional channel reach. Additionally, for the S-332B discharge canal, the characteristics of the diversion conduits are also discussed.

Design of the Proposed Concrete-Lined Channel

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. A side slope of 4H:1V was

based on maintenance and safety considerations. The direct step method was used to perform a backwater analysis of each channel upstream of the weir in order to determine the required bottom width. The channel was sized so as to maintain a stage equal to or less than 10 feet NGVD at its upstream end while maintaining maximum velocities within the channel at or below the limit of 4.5 ft/sec. Details of the backwater computations for the original design are provided in Appendix A1. Similarly, for Option 1 each channel was sized to obtain a stage equal to or less than 12.0 feet NGVD at its upstream end (Appendix A2). **Table 1** summarizes the resultant channel specifications.

Table 1. Channel Specifications for S-332B

Design Feature	Design Options Based on Stage Limits	
	Original (10 feet NGVD)	***Option 1 (12 feet NGVD)***
Bottom Width (ft)	36	15
Bottom Elevation (ft NGVD)	6	6
Side Slope (H:V)	4	4
Top Elevation (ft NGVD)	11	13

Weir Design

A weir crest elevation of 8.2 feet NGVD was assumed initially. The weir length required to pass a design flow of 575 cfs was calculated using Equation 1 (Fritz and Hager, 1998):

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

In Equation 1, C_d = the discharge coefficient (determined using Equation 2), L = the weir length, H_T = the total upstream hydraulic head, 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.

In all design calculations for the weir, L_w was set at 15 feet to enable passage over the weir by vehicular traffic. The total upstream hydraulic head H_T was calculated assuming a headwater stage of 9.0 feet NGVD for the original design. 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 (Wilsnack, 2013).

The length of the weir determined using Equation 1 is substantially larger than the width of the channel at the weir crest. Therefore, a channel expansion transition was needed to accommodate the weir length. Subsequent iterations of weir design and channel transition design (discussed in the next section) were performed until the bottom width of the approach channel and the required weir length at the selected crest elevation were approximately equal. After several iterations, the weir length was determined to be 265 feet.

The design procedure described above was likewise applied to design Option 1. In this case the weir headwater stage was set at 11.0 feet NGVD. **Table 2** summarizes the resultant weir specifications for both design options.

Table 2. Weir Specifications

Design Feature	Design Option	
	Original	***Option 1***
Geometry	Broad crested	Broad crested
Side slope (H:V)	2	2
Top width (ft)	15	15
Length (ft)	265	78
Crest Elevation (ft NGVD)	8.2	9.2
Headwater stage at weir (ft NGVD)	9.0	11.0

The flow regime associated with each weir discharge was determined using Figure F4 of Wilsnack (2013). For the weir specifications pertaining to the original design, a relative crest length ξ of 0.05 was calculated and the submergence ratio y_T was determined to be 0.38. For these values of ξ and y_T, the flow regime over weir embankment was determined to be in a transition region between a plunging jet and a surface jet. Thus, Equations 4 and 5 (Wilsnack, 2013 or Fritz and Hager, 1998) were used to calculate both the length of surface roller (L_r) downstream of the weir toe for a plunging jet and the length of the surface jet recirculation zone (L_R) downstream of the weir crest, respectively.

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

$$L_R = 6.8d(1 - \frac{h}{H})^{\frac{1}{6}} \qquad \text{Equation 5}$$

where d = the downstream tailwater depth and h = the tailwater depth above weir crest.

Equations 6 and 7 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.

$$u_A = \sqrt{gd(1 - \frac{h}{H})} \qquad \text{Equation 6}$$

$$u_M = (u_A - u_d)e^{-2.3\chi^2} + u_d \qquad \text{Equation 7}$$

In Equation 7, 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 $0 \leq \chi \leq 1$.

Similarly, Equations 8 and 9 were used to calculate the maximum backward velocity u_A within the recirculation zone for the surface jet regime, and the maximum backward streamwise velocity u_m at any location χ between the weir and the end of the surface jet recirculation zone.

$$u_a = -0.25\sqrt{gd(1 - \frac{h}{H})} \qquad \text{Equation 8}$$

$$u_m = 2u_a\sqrt{\chi(1 - \chi)} \qquad \text{Equation 9}$$

In design Option 1, the flow over the weir occurs as a plunging jet. **Table 3** summarizes the conditions of flow over the weir for both design options. Maximum flow velocities within the tail water pool over the length of a surface roller or recirculation zone were determined to identify the potential need for any additional erosion protection measures. Since the maximum velocity within a 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 and the downstream channel perimeter 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 SDA since the approach velocity in the SDA is low and historical discharges onto the surficial limestone have not caused any substantial erosion.

Table 3. Weir flow conditions

Original Design	
Plunging jet	
Length of surface roller, L_r	11.8 feet
Max. velocity at top of weir, within plunging jet, u_A	7.1 ft/s
Surface jet	
Length of recirculation zone, L_R	15.7 feet
Max. backward velocity with the recirculation zone, u_a	-1.8 ft/s
Option 1 Design	
Plunging jet	
Length of surface roller, L_r	11.8 feet
Max. velocity at top of weir, within plunging jet, u_A	9.0 ft/s

Channel Transition Design

In order to accommodate the required weir length for the S-332B discharge channel, the channel bottom has to transition from 36 feet wide to 265 feet at some location upstream of the weir for the original design option. Therefore, a transitional reach was designed to minimize energy losses and flow separation. Appendix I of Wilsnack (2013) provides guidelines on designing a transitional reach. The length of an expansion transition should be designed so that a straight line connecting the water surface edges at the two ends of the transition makes an optimum angle of about 8˚ with the centerline of the transition (Wilsnack, 2013). The calculations for the channel transition design are provided in Appendix A1, where channel bottom widths were determined at various locations along the transition.

The same methodology was applied to the Option 1 design, where the channel bottom transitions from 15 feet to 78 feet over a distance of about 226 feet. **Table 4** provides the dimensions of the resultant transitional reach designs.

Table 4. Channel transition dimensions

	Original	**Option 1**
Length of transition (ft)	820	226
Bottom width at u/s end of transition (ft)	36	15
Bottom width at d/s end of transition (ft)	265	78

In order to get a uniform flow condition approaching the weir, the width of the downstream end of the transitional reach was extended between the end of the transitional reach and the weir a distance of five times the hydraulic diameter of the channel cross section at the downstream end of the transition. Additionally, the same channel width was maintained downstream of the weir over a distance of 100 feet

to the downstream end of the channel at the eastern boundary of the SDA. This feature was added to each design to minimize the velocity and turbulence of the flow approaching the SDA.

Figures 2 and 3 show the schematic representation of the proposed S-332B discharge canal for the original and Option 1 designs. The dimensions shown are similar for the S-332C discharge channel except for the length of the main channel.

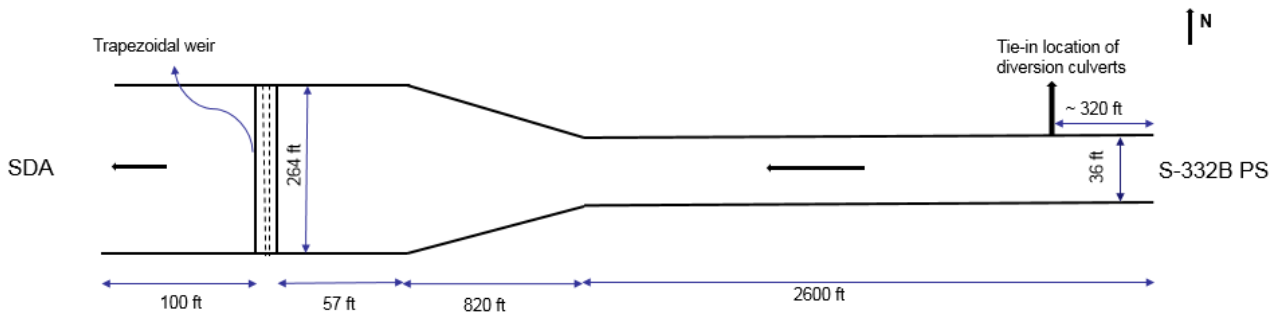

Figure 2. Schematic of the proposed discharge canal for S-332B, original design option (NTS)

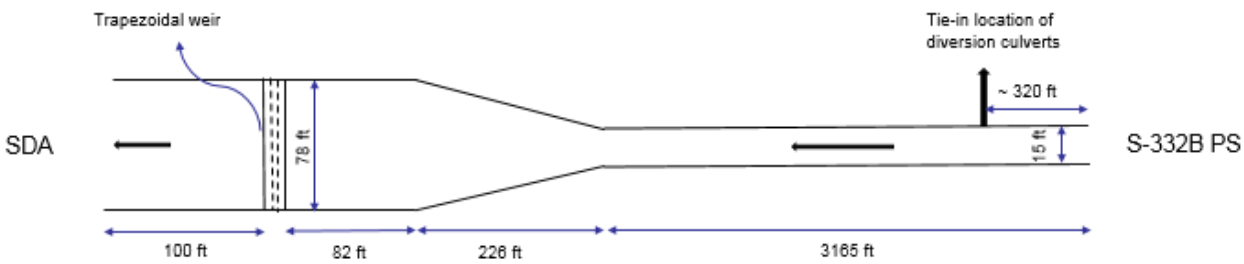

Figure 3. Schematic of the proposed discharge canal for S-332B, Option 1 design (NTS)

Final Water Surface Profile for S-332B Discharge Channel

Since the preliminary backwater calculations performed as part of the previous design steps did not consider the channel transition, a HECRAS model of the entire channel upstream of the weir was constructed in order to verify that design constraints for the water surface profile were satisfied. In the model, the transitional reach was discretized using the same incremental lengths used in the design process. The cross section spacing in the main channel was interpolated at an incremental length of 50 feet. **Table 5** summarizes the simulated channel stages at the eastern end.

Table 5. Simulated design head water stages without any flow diversion to the NDA

Discharge canal	Headwater stages (ft NGVD)	
	Original Design	*Design Option 1*
S-332B	9.7	11.5

Diversion Conduit Design for the S-332B Discharge Canal

In order to divert a flow of 250 cfs from the S-332B channel to the NDA, diversion conduits were assumed to be connected to the channel approximately 320 feet downstream of the S-332B pump station. This represents the tentative location of the current diversion structure that minimizes the lengths of the diversion conduits. As indicated previously, the downstream NDA stage was taken to be 8.5 ft NGVD. In order to determine the stage in the S-332B channel where the diversion conduits will be connected, backwater calculations were performed using HECRAS.

The head water elevation of the weir associated with the non-diverted flow of 325 cfs was used as the downstream boundary condition in the model. Since the weir crest elevation and weir length were different for the different design options, separate HECRAS simulations were performed for each option to determine the stage in the main channel where diversion conduits tie into (i.e. the design head water stage for the diversion conduits).

The simulated headwater stages for the diversion conduits are summarized in **Table 6** for the different design options. These stages represent average stages over the main channel reach where the diversion conduits will be connected.

Table 6. Simulated canal stages at the diversion location

Design Option	Stage (ft NGVD) at diversion location
Original	9.06
Option 1	10.64

Once the design headwater and tailwater stages were established for the diversion conduits, the direct step method was used to determine the water surface profile within each conduit over the length where open channel conditions exist. For the remaining length of pipe where full pipe flow occurs, Manning's formula was used to calculate the full barrel flow head loss. The entrance head loss and the velocity head were added to the computed upstream stage in the conduit to determine the required diversion head water elevation. The conduit sizes and numbers were varied until the average stage in the S-332B channel simulated using HECRAS agreed closely with the diversion head water stage calculated using the method described above.

Table 7 summarizes the resultant diversion conduit designs for each design option. In each case both RCP box culverts and HDPE pipe were considered.

Table 7. Diversion conduit specifications

Design Feature	Box Culvert Specification	HDPE Pipe Specification	
Tailwater stage	8.5 ft NGVD		
Length	Approximately 365 ft		
Entrance loss coefficient	0.5		
Manning's roughness*	0.012	0.010	0.010
Original Design Option			
Size	6 ft span X 3 ft rise	5 ft	4 ft
No. of Barrels	5	7	9
Outlet velocity (ft/s)	3.3	3.8	3.8
Calculated headwater stage in the channel @ diversion location (ft NGVD)	8.9	9.0	9.0
Invert elevation (ft NGVD)	6		
Option 1 Design			
Size	3 ft span X 3 ft rise	3 ft	-
No. of Barrels	5	7	-
Outlet velocity (ft/s)	6.7	5.9	-
Calculated headwater stage in the channel @ diversion location (ft NGVD)	10.5	10.5	-
Notes	Erosion protection measures may be required at the outlet due to the higher outlet velocity		
Invert elevation (ft NGVD)	6		

*derived from HDS5 of FHWA, 2012

A caveat associated with the conduit design is that an average headwater stage over the S-332B channel reach where the diversion conduits will be connected is used for designing all conduits. This is considered sufficient for the purposes of preliminary design. Strictly speaking, since flow will be sequentially decreased in the main channel by each diversion conduit, the channel stages along the affected reach will vary. The effects of this will be addressed in the later design phases.

For the Option 1 design, the outlet velocity of the culverts was estimated to be about 6.7 feet/second, which may require localized erosion protection at the outlet end. An additional scenario in which 500 cfs flow is being diverted to the NDA while the tailwater stage at the NDA is at 8.0 ft NGVD was also examined. The detailed calculations of this scenario are provided in Appendix B.

Water surface profiles in the S-332B discharge channel are presented in **Figures 4 and 5** for both design options. Shown also is the top elevation of the proposed levee. Water surface profiles are presented for both the scenarios with and without flow diversion to the NDA.

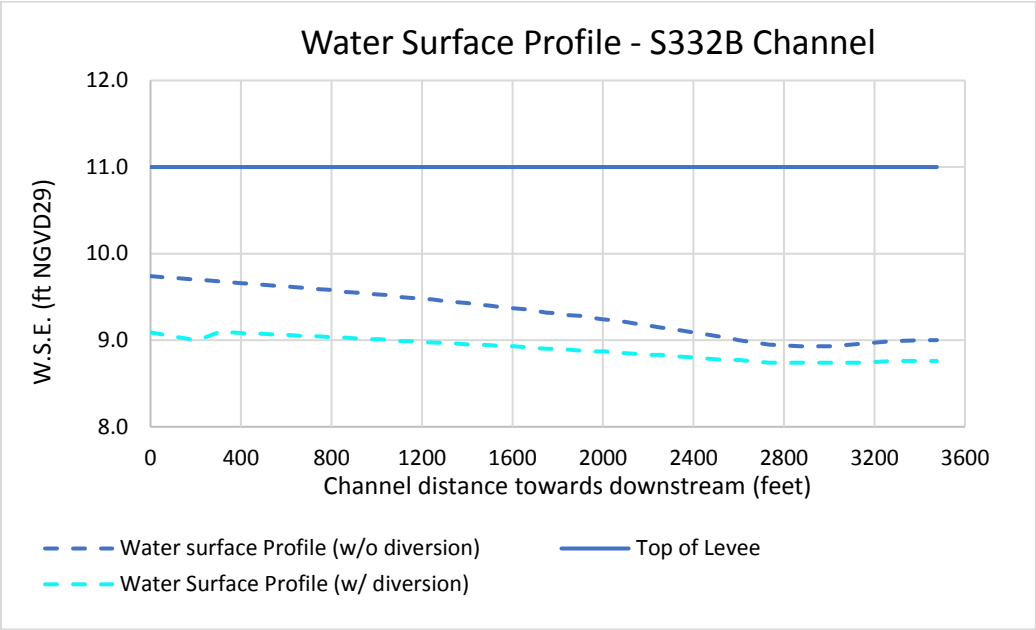

Figure 4. S-332B channel water surface profile, with and without flow diversion to the NDA for the original design

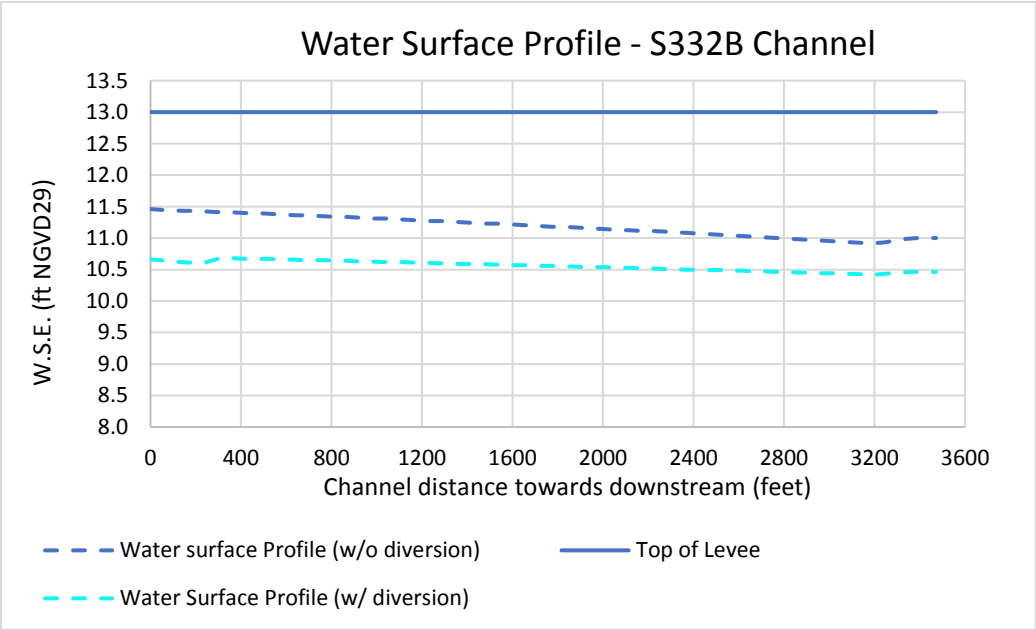

Figure 5. S-332B channel water surface profile, with and without flow diversion to the NDA for the Option 1 design

S-332C Discharge Channel Design

The discharge channel for S-332C was designed both with and without the weir near the downstream end. With the weir, the same two design options were investigated where the headwater stage of the channel was limited to either 10 feet (original design option) and 12 feet (Option 1 design). The specifications of the channel, the weir and the transitional channel reach were similar to the S-332B design in this case. Without the weir, maintaining a flow velocity at or below 4.5 feet/sec was identified as the limiting factor in the design, irrespective of whether the maximum headwater stage is 10 feet or 12 feet NGVD. **Table 8** shows the resultant specifications of S-332C discharge channel.

Table 8. S-332C discharge channel specifications

Specification	Without weir	With weir	
		Original Design	Option 1 Design
Channel bottom width (ft)	46	36	15
HW stage at the eastern end (ft NGVD)	9.6	9.7	11.43
Maximum velocity (ft/sec)	4.4	4.0	3.4
Weir crest elevation (ft NGVD)	-	8.2	9.2
Weir length (ft)	-	265	78
HW stage at weir (ft NGVD)	-	9.0	11.0
Channel transition length (ft)	-	820	226

Water surface profiles in the S-332C discharge channel are presented in **Figures 6 and 7** for the original design option and the Option 1 design. Shown also is the top elevation of the proposed levee. Water surface profiles are presented for both the scenarios with and without the downstream weir.

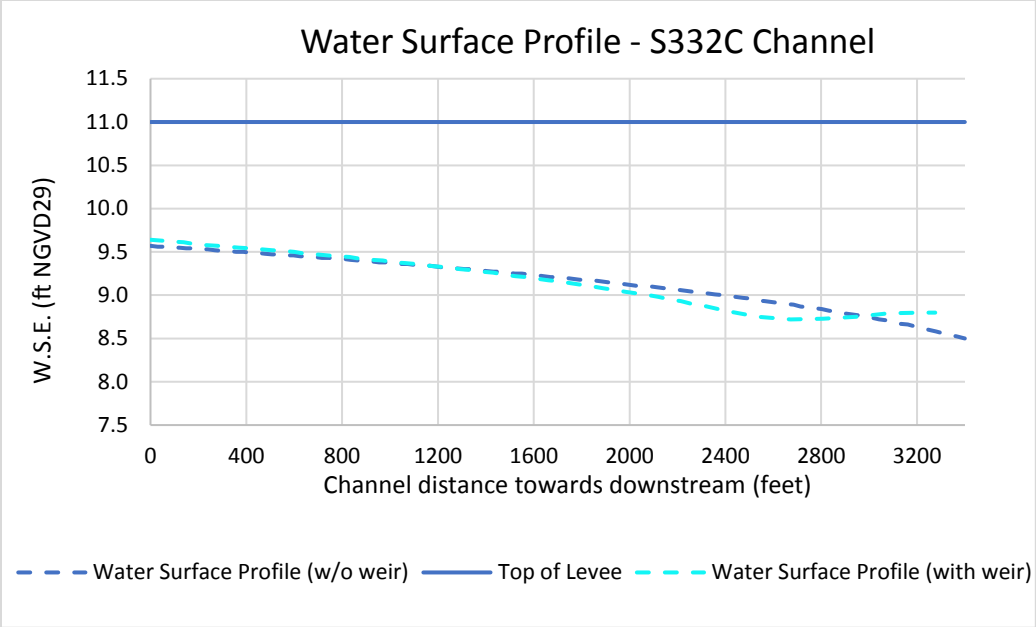

Figure 6. S-332C channel water surface profile for the original design

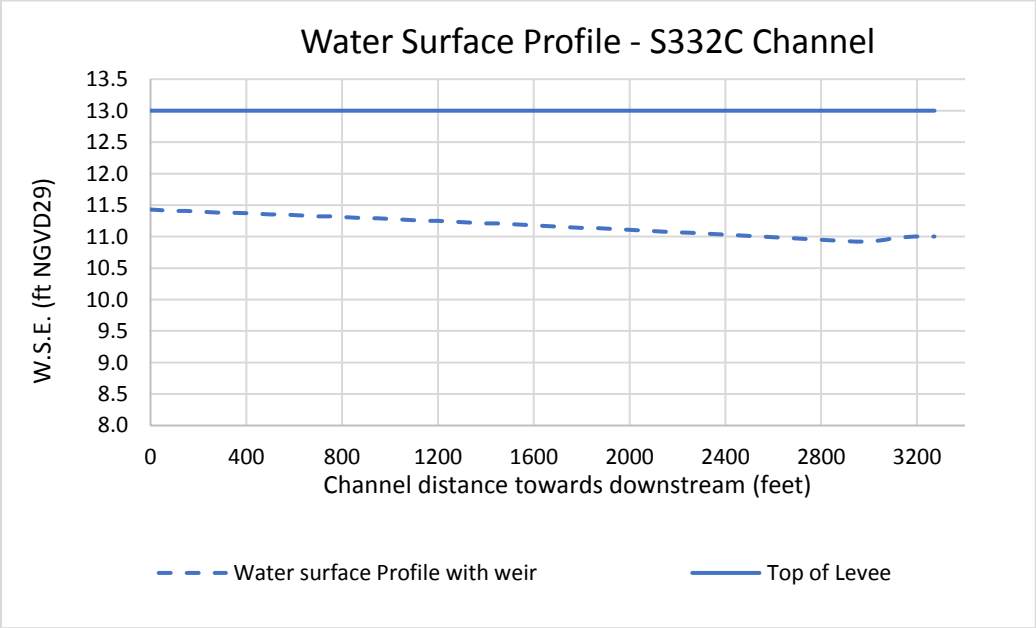

Figure 7. S-332C channel water surface profile for the Option 1 design

SUMMARY AND CONCLUSIONS

This report summarizes the preliminary designs of the discharge canals for pump stations S-332B and S-332C. These designs were prepared in enough detail to support the cost estimation of the project. Different design options suggested by P. F. Linton (via email communication dated 10/07/2018) were evaluated as the part of this phase of design.

For the original design, backwater calculations were used to determine the required channel bottom width of 36 feet, which expands near its downstream end to 265 feet over a transition length of 820 feet. At the end of the transitional reach, over a distance equal to five times the hydraulic diameter of the channel, the bottom width of 265 feet is maintained to create a more uniform flow condition upstream of the broad-crested weir. The weir crest was set at an elevation of 8.2 feet NGVD while a weir length of 265 feet is required to pass the design flow of 575 cfs. The associated upstream headwater stage of the channel was computed to be about 9.74 feet for the S-332B channel. The discharge canal for S-332B extends about 3600 feet west of the structure.

Similarly, for the S-332C discharge canal, options with and without a weir were considered for the original design. The channel bottom width of 36 feet and 46 feet would be required to keep the flow velocity below the target value of 4.5 feet/sec with and without the weir, respectively. Similarly, the stage at the eastern end of the S-332C discharge channel was computed to be 9.7 feet NGVD and 9.6 feet NGVD for the options with and without the weir, respectively. If the weir is included, a similar design configuration described above for S-332B would be required; i.e. a weir crest elevation of 8.2 feet NGVD and a length of 265 feet. Similarly, a channel transition reach 820 feet long is required to accommodate the weir. The S-332C discharge canal extends about 3400 feet west from the structure.

Different types of diversion conduits were examined to divert 250 cfs to the NDA from the S-332B discharge canal. For the original design option, the hydraulic analysis showed that five barrels with spans of 6 feet span and rises of 3 feet are able to convey the 250 cfs flow under the design constraints. Alternatively, either seven barrels of 5-feet diameter HDPE pipes or nine barrels of 4-feet diameter HDPE pipe culverts can convey the design discharge under the same constraints.

For design Option 1, a channel bottom width of 15 feet was determined using the same methodology. This bottom width was also specified to enable passage of maintenance vehicles. This dimension expands near its downstream end to 78 feet over a transitional reach length of about 226 feet. The weir crest was set at an elevation of 9.2 feet NGVD while the associated weir headwater stage is 11.0 feet NGVD. The required weir length was determined to be 78 feet. For this weir design maximum flow velocities within potential recirculation zones and surface rollers were found to exceed 4.5 ft/sec. Consequently, additional erosion control measures may be needed downstream of the proposed weir. The resultant upstream headwater stage in the S-332B channel for this design was estimated to be 11.5 feet NGVD.

The headwater stage at the eastern end of the S-332C discharge channel for the design Option 1 was estimated to be 11.43 feet NGVD. The configurations for the channel, the weir and the channel transitional reach are similar to those of the S-332B design.

Under the same design conditions, it was found that either five 3-feet by 3-feet box culverts or seven 3-feet diameter HDPE pipe culverts were sufficient to divert 250 cfs to the NDA when its stage is at 8.5 feet NGVD. The outlet velocity at the design flow through each diversion conduit is close to 6.7 feet/second. Consequently, some erosion control measures such as riprap or stone pitching may be needed at the outlets of the culverts. This should be accounted for in the project cost estimates.

The Option 1 design developed in this effort indicates that additional cost savings (in terms of channel width, weir length and diversion conduits) and operational flexibility could be achieved if a flow depth greater than 4 feet is allowed at the upstream end of each discharge canal; i.e. if the maximum allowable

tail water stage for each pump station is increased from 10 feet NGVD to 12 feet NGVD. Since higher cost savings and a reduced design footprint can be achieved in the Option 1 design, it is the preferred alternative and should be investigated in further detail in the later stages of the design process.

REFERENCES

Chow V.T., 1959. *Open Channel Hydraulics*. McGraw Hill Book Company Inc.

Federal Highway Administration, 2012. *Hydraulic Design of Highway Culverts*, HDS No. 5 Publication No. FHWA-HIF-12-026, U.S. Department of Transportation, Washington, D.C.

Fritz and Hager, 1998. Hydraulics of Embankment Weirs, *Journal of Hydraulic Engineering*. Vol. 124, No. 9, pp. 963-971.

Wilsnack M. M, 2013. Guidelines for the Hydraulic Design of Temporary Water Control Structure. South Florida Water Management District, West Palm Beach.

Appendix A1 - Detailed calculations - original design option

Spreadsheet calculations:

1. Hydraulic Design of S332B-Original Design.xlsx
2. Hydraulic Design of S332C-Original Design-with weir.xlsx
3. Hydraulic Design of S332C-Original Design-without weir.xlsx

HECRAS files for S-332B:

1. S332B-Original Design-Without Flow Diversion.zip
2. S332B-Original Design-With Flow Diversion.zip

HECRAS files for S-332C:

1. S332C-Original Design-with weir.p01.zip
2. S332C-Original Design-without weir.p02.zip

Appendix A2 - Detailed calculations - Option 1 design

Spreadsheet calculations:

1. Hydraulic Design of S332B-Option 1 Design.xlsx
2. Hydraulic Design of S332C-Option 1 Design.xlsx

HECRAS files for S-332B:

1. S332B-Option 1-Without Flow Diversion.p05.zip
2. S332B-Option 1-With Flow Diversion.p06.zip

HECRAS file for S-332C:

1. S332C-Option1 Design.p01.zip

Appendix B - Diversion conduit design for diverting 500 cfs from the S-332B discharge channel to the NDA

In order to divert a flow of 500 cfs from the S-332B channel to the NDA, diversion conduits were assumed to be connected to the channel approximately 320 feet downstream of the S-332B pump station. The NDA stage was assumed to be 8.0 ft NGVD. In order to determine the stage in the S-332B channel where the diversion conduits will be connected, backwater calculations were performed using HECRAS.

The head water elevation of the weir associated with the non-diverted flow of 75 cfs was used as the downstream boundary condition in the model. Since the weir crest elevation and weir length were different for the different design options, separate HECRAS simulations were performed for each option to determine the stage in the main channel where the diversion conduits are connected (i.e. the design head water stage for the diversion conduits).

The simulated headwater stages for the diversion conduits are summarized **Table B-1** for the different design options:

Table B-1. Simulated canal stages at the diversion location for the design options.

Design Option	Stage (ft NGVD) at diversion location
Original	8.30
Option 1	9.60

Once the design headwater and tailwater stages were established for the diversion conduits, the direct step method was used to determine the water surface profile through each conduit barrel over the length where open channel flow conditions occurred. Over the remaining portion of the culvert where full pipe flow occurs, Manning's formula was used to calculate the full barrel flow head loss. The entrance head loss and the velocity head were added to the computed upstream stage in the conduit to determine the required stage in the S-332B channel at the diversion location. **Table B-2** summarizes the resultant diversion conduit designs for each design option.

Table B-2. Diversion conduit specifications for a 500 cfs diversion to the NDA (stage = 8.0 ft NGVD).

Design Feature	Box Culvert Specification	HDPE Pipe Specification	
Tailwater stage	8.0 ft NGVD		
Length	Approximately 365 ft		
Entrance loss coefficient	0.5		
Manning's roughness*	0.012	0.010	0.010
Original Design Option			
Size	6 ft span X 3 ft rise	5 ft	4 ft
No. of Barrels	14	23	29
Discharge velocity (ft/s)	3.0	3.1	3.1
Calculated headwater stage in the channel @ diversion location (ft NGVD)	8.3	8.3	8.3
Invert elevation (ft NGVD)	6		
Option 1 Design			
Size	3 ft span X 3 ft rise	3 ft	-
No. of Barrels	13	18	-
Calculated headwater stage in the channel @ diversion location (ft NGVD)	9.6	9.6	-
Discharge velocity (ft/s)	6.5	6.3	-
Notes	Erosion protection measures required at the outlet due to the higher discharge velocity		
Invert elevation (ft NGVD)	6		

*derived from HDS5 of FHWA, 2012

As observed from **Table B-2** for the original design option, the headwater stage in the channel at the diversion location is at 8.3 feet NGVD when 500 cfs is being diverted to the NDA, therefore if the NDA stage is at 8.5 feet NGVD, no flow can be diverted to the NDA. For Option 1, 500 cfs can still be diverted to the NDA even if the NDA stage is at 8.5 ft NGVD. For this to occur either six RCP barrels with a 6-feet span by 3-feet rise, or 20 barrels of 3-feet HDPE pipe would be required. **Table B-3** summarizes the resultant diversion conduit designs for Option 1 with the NDA stage at 8.5 feet NGVD.

Table B-3. Diversion conduit specifications for a 500 cfs diversion to the NDA when the NDA stage is 8.5 ft NGVD.

Design Feature	Box Culvert Specification	HDPE Pipe Specification	
Tailwater stage	8.5 ft NGVD		
Option 1 Design			
Size	6 ft span X 3 ft rise	3 ft	-
No. of Barrels	6	20	-
Calculated headwater stage in the channel @ diversion location (ft NGVD)	9.52	9.61	-
Discharge velocity (ft/s)	5.6	7.1	-
Notes	Erosion protection measures required at the outlet due to the higher discharge velocity		
Invert elevation (ft NGVD)	6		

Appendix C – System performance under a flow rate of 650 cfs

In response to a request from the Office of Water Control, the hydraulic performances of the pump station discharge facilities for S-332B, designed for a flow rate of 575 cfs, were examined under a discharge rate of 650 cfs. At this higher discharge rate, it was found that the headwater stage at the eastern end of the discharge channel would be 9.9 ft NGVD for the original channel design, without any diversion to the NDA. Similarly, the headwater stage associated with the channel design for Option 1 would be 11.6 ft NGVD at 650 cfs.

HECRAS was used to compute the headwater stage at the diversion location when flow is being diverted to the NDA. The head water elevation of the weir associated with the non-diverted flow was used as the downstream boundary condition in the model. The simulated headwater stages for the diversion conduits are summarized in **Table C-1** for the different design options of the discharge channel and diversion conduits. The stages presented in **Table C-1** are averaged over the estimated reach where the diversion conduits connect to the main channel.

Table C-1. Simulated canal stages at the diversion location

Design Feature	Box Culvert	HDPE Pipe	
Tailwater stage at NDA	8.5 ft NGVD		
Original Design Option			
Size	6 ft span X 3 ft rise	5 ft	4 ft
Simulated HW stage @ diversion location	9.09	9.12	9.17
Option 1 Design			
Size	3 ft span X 3 ft rise	3 ft	-
Simulated HW stage @ diversion location	10.82	10.84	-

As evident from **Table C-1** the headwater stages in both the options are higher than the design criteria for which the diversion culverts were originally designed, implying that a flow higher than 250 cfs would flow to the NDA due to the higher head differential. **Table C-2** demonstrates the amount of flow diverted to the NDA in each diversion conduit design:

Table C-2. Summary of NDA diversions

Design Feature	Box Culvert	HDPE Pipe	
Original Design Option			
Size	6 ft span X 3 ft rise	5 ft	4 ft
No. of Barrels	5	7	9
HW stage @ diversion location (ft NGVD)	9.09	9.12	9.13
Total diversion flow (cfs)	305	290	275
Discharge velocity (ft/s)	4.0	4.4	4.2
Option 1 Design			
Size	3 ft span X 3 ft rise	3 ft	-
No. of Barrels	5	7	-
HW stage @ diversion location (ft NGVD)	10.8	10.8	-
Flow diverted (cfs)	270	265	-
Discharge velocity (ft/s)	7.2	6.9	-

At 650 cfs, the headwater stage at the eastern end of the S-332C channel would be 9.85 ft NGVD and 9.74 ft NGVD for the original channel design, with and without the terminal weir, respectively. A maximum velocity of 5.0 ft/sec was observed in the channel without the weir. Similarly, the headwater stage associated

with this channel under the Option 1 design conditions would be 11.53 ft NGVD at this higher flow rate with the weir crest elevation fixed at 9.2 feet NGVD.