A.A.0 Engineering Design Addendum

A.A.1 Purpose

The purpose of this Addendum is to describe design refinements to the Site 1 Impoundment project that have been made since submission of the preliminary Final PIR (pFPIR). Selected features with new designs that differ from those presented in the Engineering Appendix are included in this submittal with a description of the design change, rationale for the change, and cost associated with that change. Additional analyses were necessary to address embankment design (pFPIR, Section A.2.2) that subsequently formed the basis of this Addendum attached to the pFPIR, while other features were optimized in the time span between the pFPIR and this submittal. These inclusive design changes are expected to have an insignificant impact on benefits established by the Selected Plan presented in the pFPIR. However, additional analyses to be conducted during detailed design efforts may lead to refinements in design and operational assumptions to ensure attainment of expected PIR benefits or increasing those project benefits.

A.A.2 Recent Efforts

Descriptions of efforts that have been completed or in progress are provided in the following subsections in approximate chronological order.

A.A.2.1 Reservoir Design

Reservoirs constructed for storage pools above natural grade in areas of low topographic relief have perimeter embankments typically all the way around (360°). These perimeter embankments with extensive reaches—especially those having an armoring requirement—have a significant impact on construction costs. The embankment height design and analyses included in the preliminary Final PIR was based on Jacksonville District interpretation of engineering policies and regulations at the time of conducting the Water Preserve Area Draft Feasibility Study (2001). The earlier methodology proved to be an outdated approach that has been updated by USACE Engineering Regulations and Manuals, and Federal Guidelines for Dam Safety (FEMA). The current methodology is based on the updated USACE references, new information obtained through extensive literature research, as well as with joint coordination with the non-Federal sponsor (NFS) to meet the State of Florida’s needs.

A.A.2.2 Design Criteria Memorandum

Joint coordination with the NFS included discussions of design parameters, interpretations of USACE Regulations, industry practices, and required agreements as stated in the preliminary Final PIR, Section A.2.2. These coordinated efforts resulted in the production of the Design Criteria Memorandum(s) (DCM) that provided the Jacksonville District, as well as the NFS’s A/E consultants, guidance to perform proper embankment height design (and other feature designs too). The DCMs pertinent to this project include DCM-1 to determine Hazard Potential Classification of the reservoir and embankment reaches, DCM-2 to determine minimum embankment height with wind and wave analyses, and DCM-3 to determine ungated spillway configurations for uncontrolled overflow to preserve embankment integrity. Other DCMs were utilized too, but none more than DCM-2. DCM-2 is attached to this addendum for reference.

A.A.2.3 South Florida Water Management District Acceler8 Program

The South Florida Water Management District (SFWMD) as the NFS created an engineering design and construction program to accelerate implementation of specific components of CERP. Their A/E consultants (Acceler8) have performed and are currently performing analyses that are typically conducted during the follow-on design phase of USACE projects. Results of Acceler8 analyses were used in conjunction with Jacksonville District (SAJ) efforts to identify and refine/correct project feature designs for the betterment of the project. Discussions with the NFS on these issues resulted in the joint performance of the Value Engineering (VE) study.

A.A.2.4 Value Engineering

A formal VE Study was conducted from 15 May through 19 May 2006. The participants included the NFS, their A/E consultants, and SAJ design team members.

Value Engineering (VE) is a process used to study the functions a project is to accomplish. As a result, the VE team examines how these functions are being met and identifies alternative ways to achieve the equivalent function while increasing the value and cost ratio of the project. The project was studied using the USACE standard VE methodology. The Final VE report is attached to this addendum and includes comprehensive documentation of the five phase process of the study as well as the study results.

Although substantial cost savings for several project features were identified in the VE study, the total cost of the project increased. This is largely due to the new embankment design that includes armoring in response and acknowledgment of current and appropriate reservoir design criteria.

A.A.3 Design Changes from PIR Design

The following provides a summary of the project features that have undergone design changes from that presented in the preliminary Final PIR. An abbreviated description of the recommended change is included here for convenience. Descriptions of changes in greater detail are included in later sections within this addendum; features will follow same order as listed here.

- L-508I Internal Levee: Deleted from the preliminary Final PIR plan.

- L-508N Perimeter Embankment: Embankment changed by a height increase of 1-foot, from 16 feet to 17 feet above average natural grade. Extent of embankment armoring changed too, from 4,000 feet of linear placement to all around the pool-side perimeter. Also, armoring material changed from riprap to composite of soil cement plate and stair-step. The design and cost estimate outlined in this addendum provides for an overwash rate of 0.05 cfs per foot, no erosion protection on the outer face of the embankment, and an erosion protection plate thickness of 8 inches in the non-wave impact zone and 16 inches (or 12 inch stepped) in wave impact zone. These design assumptions may be refined based on further analyses to be conducted during detailed design efforts.

- S-525A Inflow Pump Station: Changed by the reduction of total pump capacity from 1,360 cfs to 650 cfs pumping capacity. Reduction of Pump Station capacity eliminates need for Hillsboro Canal Improvement feature that is now deleted from the preliminary Final PIR plan.

- S-526B Service and Auxiliary Spillway (unnumbered emergency overflow spillway in preliminary Final PIR): Changed from a single elevation crest with a 50-foot length to a dual crested spillway with a total length of 69-foot. Detailed engineering information provided in L-508N Perimeter Embankment (D-1 Embankment) section.

- S-526C Auxiliary Spillway (new in Plan): Added spillway in L-40 Levee with a single elevation crest with a 500-foot length. Detail engineering information provided in L-508N Perimeter Embankment (D-1 Embankment) section.

- SG-527A Fixed Weir: Changed from a notched concrete weir with stilling basin to a sheetpile-concrete cap weir with riprap downstream protection.

- SG-527B Gated Culverts: Changed in construction dewater technique from wellpoint to use of a constructed cofferdam.

- S-528A Gated Culverts: Deleted from preliminary Final PIR plan with L-508I Levee. Detail engineering information provided in L-508I Internal Levee section.

- Extent of required earthwork such as clearing and grubbing have changed, an increase associated with the increase in armoring primarily, need of space for construction thereof.

New Structure Designation System

Historically water control features designed by USACE have been designated by a single letter, e.g. "S," then followed by a number. Water control structures such as pump stations, spillways and culverts were designated by the letter "S" followed by a number (e.g. S-80, S-96A, etc). Levees have been designated by the letter "L" followed by a number (e.g. L-14, L-67A, etc). Canals have been designated by the letter "C" followed by a number (e.g. C-11, C-1W, etc). Levee and embankment features occasionally involved using material immediately adjacent to the levee. The excavation is called a "borrow" ditch or canal. The borrow ditch also served as a seepage collection canal or for recreation purposes and was designated as a "borrow canal" (e.g. L-40 Borrow Canal). Water control structures owned and operated by the South Florida Water Management District SFWMD that did not have Federal involvement have historically been designated by the letter "G" followed by a number (e.g. G-103, G-252H, etc.)."

Prior to the Comprehensive Everglades Restoration Project (CERP), agreements between the sponsor and the Federal government called for the USACE to design and build the structures that were handed over to the local sponsor for operation and maintenance. All operation and maintenance costs were borne by the sponsor. CERP is also a cooperative effort that is cost-shared project between the South Florida Water Management District and the USACE. A unique feature of the CERP agreements is that the commitment of the USACE has been expanded to share the cost of operations and maintenance. Subsequently, water control features shall be designated to confirm the origin of the project feature and O&M responsibilities.

Structures for the CERP project will be designated to reflect the dual O&M responsibilities of the Federal Government and the local sponsor. Water control structures such as pump stations, spillways and culverts shall be designated by the letters "SG" followed by a number (e.g. SG-520, SG-620A, etc). Levees shall be designated by the letters "LG" followed by a number (e.g. LG-14, LG-67A, etc). Canals shall be designated by the letters "CG" followed by a number (e.g. CG-11, CG-1W, etc). Borrow canals shall be designated as a "borrow canal" (e.g. LG-40 Borrow Canal). Embankment construction that meet the definition of a Dam shall be designated by the letter "D" followed by a number (e.g. D-1, D-2W, etc.). Seepage canals for dams shall be designated as a "borrow canal" (e.g. D-1 Borrow Canal).

New Structure numbers for Site 1 Impoundment are provided in the following. Note that Plates and other documentations will be updated before final submittal for record.

Old		**New**
C-508N	**Seepage Collection Canal**	**D-1 Borrow Canal**
L-508I	**Internal Levee**	**To be Deleted**
L-508N	**Perimeter Embankment**	**D-1 Embankment**
S-525A	**Inflow Pump Station**	**SG-525A Inflow Pump Station**
S-525B	**Seepage Pump Station**	**SG-525B Seepage Pump Station**
S-526A	**Gated Discharge Structure**	**SG-526A Gated Discharge Structure**
S-526B	**Service-Auxiliary Spillway**	**SG-526B Service-Auxiliary Spillway**
S-526C	**Auxiliary Spillway**	**SG-526C Auxiliary Spillway**
S-527A	**Fixed Weir**	**SG-527A Fixed Weir**
S-527B	**Gated Culvert Structure**	**SG-527B Gated Culvert Structure**
S-528A	**Gated Culvert Structure**	**To be Deleted**

A.A.3.1 L-508I Internal Levee

Originally, the internal levee was multi-purpose with the following objectives: (1) break up wind fetch to reduce wind setup and wind-wave generation; (2) improve long-term storage efficiency by reducing evapotranspiration (ET) losses with primary use of western cell only--reduced exposed surface area; (3) ease seepage management O&M with additional buffer of lower pool on eastern side, and (4) provide additional embankment area for future ASR well implementation.

However, current wind-wave analyses demonstrate that the internal levee is not effective in reducing wind-wave growth; although it does effectively break up wind fetch for wind setup. The savings of perimeter embankment height is on the order of a half-foot, a result that does not support the need for an internal levee based on cost effectiveness. Also, hydrologic modeling demonstrated that the reservoir is full 30% to 40% of the time (SFWMM and Acceler8 analyses). Thus, reduced surface area--reduced ET losses--with primary use of the western cell is not realized through simulations. Lastly, it was shown that an efficient configuration of Aquifer Storage and Recovery (ASR) wells would not require an internal levee as originally expected early in the WPA Feasibility Study efforts (SFWMD analysis).

Current analyses resulted in the decision that armoring would be required for all embankment surfaces that are exposed to breaking wave action within the reservoir because of the non-cohesive silty sand properties of the embankment material. Therefore, in order for the internal levee to be cost-effective without continuous O&M repair, the internal levee would also require armoring. As suitable riprap is not readily available onsite, soil cement would be the armoring material of choice. In summary, because of (1) cost of armoring the levee surfaces on both sides, (2) small worth of benefits that would be obtained, and (3) elimination of need for control structure S-528A Gated Culverts, the PDT removed L-508I Internal Levee from the preliminary Final PIR plan.

A.A.3.2 D-1 Embankment

The perimeter embankment (L-508N) was originally designed based on Jacksonville interpretation of ERs and guidance EMs known at the time. The resulting design embankment heights were ultimately based on minimum required superiority as provided in ER 1110-8-2(FR). Thus, the design embankment heights were 16.0 feet above the average natural grade.

Additional research by the Jacksonville District and later through coordination efforts with the NFS (and their A/E consultants) led the design team to a methodology that provides for more appropriate embankment heights. This effort culminated with the approval of DCM-2 that includes detailed information with references and is provided as an attachment to this addendum. With an a implemented seasonal operational schedule (Hurricane season having a 3-foot lower pool), the resulting design embankment heights increased by 1.0 foot to 17.0 feet above average natural grade, exceeding the minimum required superiority by 0.5 feet. Over-wash rates with a supporting risk and uncertainty analysis formed the basis for final design acceptance.

As stated above, the design D-1 Embankment height (eastern and southern boundaries) is set at 17.00 feet above average natural grade, or 27.00 ft-NGVD. The design height may be modified during detail design efforts (or by SFWMD A/E consultants) based upon new and rigorous analyses demonstrating a need for a different design height, either added to for safety or subtracted from for cost-effectiveness, with consideration of acceptable risk and uncertainty.

A.A.3.2.1 Procedure to Determine Embankment Height

For convenience of the reviewer, the determination methodology is presented here in abbreviated form to illustrate the approach used for the design of this feature. Footprint of reservoir and required pool depth is assumed to have been fixed based on sub-regional hydrologic modeling and evaluated benefits. However, for Site 1 the operating normal full pool depth is based on an operational schedule that is discussed in a following sub-section.

- Determine or reaffirm reservoir's hazard potential classification (HPC).
- Determine appropriate design criteria for selected HPC (high and low criteria presented in DCM-2, significant was left open dependent on risk-economic damage assessment analyses).
- Determine appropriate rainfall or storm event and routing sequence/procedure.
- Determine appropriate configuration of service spillway.
- Determine appropriate configuration of auxiliary spillway(s). This step may be approached iteratively with other components of analysis.
- Perform hydrologic routing simulations to obtain surcharge pool stages.
- Determine appropriate fetch length for calculating wind setup.
- Determine appropriate design wind speed.
- Perform wind setup analysis.
- Perform wind-wave height analysis.
- Perform wave run up and overwash analyses.
- Select embankment height with overtopping (wave over-wash) as a constraint based on embankment material, downstream conditions, and risk and uncertainty level of acceptability.

A.A.3.2.2 New Operating Schedule

A new operation schedule is proposed with this addendum that is a change from the constant normal full operating pool level of 18.00ft-NGVD (8.0 feet deep measured from assumed average interior grade of 10ft-NVD) in the preliminary Final PIR. For the Hurricane season that approximates the wet season, the design normal full pool is 5.0 feet deep, relative to average interior grade. The reason for the lower pool is to reduce unnecessary embankment height that would otherwise be required to ensure integrity of the structure under design parameters that includes the 100-yr wind speed. For the off-season that approximates the dry season, the design normal full pool is 8.0 feet deep, relative to average interior grade. This greater depth makes use of maximum storage going into the dry-season and allows majority capture of winter off-season storms for maximization of benefits achieved mostly in and throughout the dry-season. The design wind speed for the off-season pool retains an equivalency to a Category 1 Hurricane.

Water release operations and benefit analyses in the PIR assumed that water supply discharges from the impoundment would be terminated when the water depth reached approximately one foot. The purpose of this operational criterion was to minimize the period of dry-out in the impoundment for water quality and wildlife habitat/foraging purposes. However, model simulations showed that even with this operational limit, the impoundment still dried out at times during the 36-year period of record.

During detailed design efforts, further analyses (e.g. water budget, water quality, etc.) will be conducted in an attempt to maximize use of the impoundment storage capacity. It is believed that these analyses will justify a refinement to the Draft Project Operating Manual to continue water supply discharges as long as water is available in the impoundment (below depth of 1 foot), thereby resulting in a potentially significant increase in project benefits compared to what was evaluated in the PIR. With this likely refinement to the proposed operations, the Draft Project Operating Manual will be revised in accordance with Section 385.28 of the CERP Programmatic Regulations (33 CFR Part 385). The Final Project Operating Manual will be prepared in compliance with USACE policy and regulations.

Background, Function and Operations

The preliminary Final PIR plan provided for an 8-foot deep storage reservoir on a year-round basis for desired benefits. Normal operations of the reservoir included capture of excess water in Hillsboro Canal when storage was available, i.e. reservoir not at normal full pool level. Water sources for producing "excess" primarily included storm runoff from the Hillsboro Basin and secondarily from Loxahatchee National Wildlife Refuge (LNWR) and North Springs Improvement District (NSID). When dry times occur, water is released from the reservoir to meet water supply demands, thus, retaining natural system water within the natural system (the Everglades).

The reservoir creates a hydraulic head by holding water above the natural water table that reduces seepage from LNWR, thus, retaining more natural system water that would normally seep out into expanding urban development as groundwater flow. Stored water in the reservoir effectively replaces LNWR seepage with leakage through the reservoir bottom that provides groundwater recharge essential to public wellfields, prevention of saltwater intrusion, and maintenance of the coastal estuary salinity envelope in a control manner. The use of "control manner" is appropriate as excess seepage is return pumped into storage primarily with the SG-525B Seepage Pump Station or the SG-525A Inflow Pump Station, if required.

Also, the capturing of excess water from Hillsboro Canal provides some benefits in attenuating freshwater slugs into the coastal estuary that otherwise rapidly alters the desired salinity envelope. Historically, runoff from undeveloped and natural lands took place at a much lower maximum rate over an extended period of time. Whereas with developed lands, runoff occurs at what may be described as a peak flow over a short duration. This slug stresses the natural estuarine system with an overall negative impact.

Reservoir Fullness

The reservoir includes ±13,500 acre-feet of storage over a surface area of ±1,660 acres. The Hillsboro Canal Basin, in which the reservoir is located, covers a contributing area of ±102 square miles. Simple calculations show that the reservoir is capable of storing a maximum of 0.20 feet--2.4" of rainfall out of an annual average of 53"--of water over the Hillsboro Basin alone. The C-15 Basin that includes ±74 square miles, at times also contributes runoff to the Hillsboro Canal (An Atlas of Eastern Palm Beach County Surface Water Management Basins, SFWMD, 1988). Because of the contributing basin(s) size, it is easy to understand how modeling results demonstrate that the reservoir is full 30% to 40% of the time on average over the period of record. The "full" reservoir occurs and extends primarily through the major part of the wet season. However, there are winter off-season storms that occur periodically throughout the dry season that allow the reservoir to be filled and refilled again with extended dry times between. It is important to note here that the reservoir, even with an all-year 8-foot allowable pool depth (preliminary Final PIR), does experience dry-outs occasionally as demonstrated through model simulations.

Wind Speed Parameter

For reservoir design, specifically embankment heights, it is important to understand in no uncertain terms that the most sensitive design parameter is the wind speed. USACE Regulations stipulates for High Hazard Potential Classification reservoirs (Standard 1 as described in ER 1110-8-2(FR)), that a reasonable design wind speed is to be applied to the appropriate surcharge pool for analyzing wind and wave impacts on embankments--primarily by wave overtopping or over-wash quantification. Additionally, General Design Memorandum (GDM) provides the guidance that levee design in the south Florida region should provide protection against storms of high wind speeds for durations of 3 to 4 hours (Central and Southern Florida Project, Partial Definite Project Report for Flood Control and Other Purposes, Part 1, 1951). High intensity hurricanes require ideal to very good conditions to sustain strength, thus, movement overland typically causes hurricanes to degrade relatively quickly unless directional movement is rapid. With either degrading or rapid directional movement conditions, sustained winds from a particular direction are not expected to occur longer than a few hours in duration. The important parameter noted here is "sustained" winds. These are the winds that develop wind setup and generate waves that produce over-wash on embankments with a volume rate of concern.

Southern Florida is geographically a lower sub-tropical region and is therefore prone to cyclonic storm events (tropical storms and hurricanes) that originate in the tropics and typically provide either very heavy rainfalls or very high wind speeds. There is a remote probability that a combination of both very heavy rain and very high wind speeds can occur. Recognizing that larger or high intensity cyclonic events occur almost exclusively during the Hurricane season (1 June through 30 November), design criteria in DCM-2 established a higher wind speed referenced as the 1 in 100-year return frequency that can be expected as the “reasonable” design wind speed.

Very high wind speeds with long durations associated with large cyclonic events are not expected during the off-season (non-Hurricane season). Due to absence of mountains or hills that may channel sustained high winds due to changing weather conditions (as is experienced in other parts of the country), south Florida high winds that accompany off-season storms may be high with short duration (gusts) or sustained at a much lower velocity (e.g. near gale of 35 mph or Force 7 on Beaufort scale over water for a duration of nearly 36 hours has been experienced by this writer in the Jacksonville area). Therefore, a somewhat lower wind speed than that experienced during the dry season (though still a high wind speed), is thought “reasonable” during the off-season. The design wind speed for the off-season was selected to be that equivalent to a middle Category 1 Hurricane or 85 mph wind speed (one-minute average), or converted to one of one-hour average for wind-wave analyses, i.e. 68mph wind speed.

New Operating Schedule

With all information at hand and with an interest in constructing a cost-effective reservoir that gains maximum benefits expected with implementation of the project, a seasonal operating schedule became the choice condition of operations. Application of the higher 100-yr wind speed on the Hurricane season pool depth of 5 feet and the application of the equivalent to a Category 1 Hurricane on the off-season pool of 8 feet provided the most cost effective solution while maintaining essentially all benefits derived with the all-season 8-foot normal pool depth (preliminary Final PIR). Recognizing that Hurricane season extends from 1 June through 30 November, herein are the proposed guidelines for the operating schedule of normal full pool levels. As high intensity hurricanes don’t develop until July (at earliest) and most late season hurricanes don’t impact the mainland, the proposed schedule is as follows:

- Five-foot normal full pool (15.00ft-ngvd) is operationally and strictly adhered to from 15 June to 15 November.
- Eight-foot normal full pool (18.00ft-ngvd) is operational adhered to, as conditions allow, outside the five-foot pool schedule. Essentially, this means that after 15 November, the reservoir can exceed the five-foot pool with season-ending storms up the eight foot pool. Also, the reducing of the eight-foot pool to the five-foot pool through operations must be completed by 15 June.

Impact to PIR Benefits

Analyses by Acceler8 A/E consultants demonstrated a minor loss of 4% of the water supply benefits with the new proposed operation schedule based on terminating water supply discharges when the depth drops to approximately one foot. As a natural system benefit, this means that only 4% of water supply demands are required from other sources than captured runoff from the Hillsboro Basin. In other words, if the project provided 71% of all water supply needs through capture of excess water, the new operation schedule reduces that benefit to 67%. This difference measured through modeling is considered to be insignificant and well within the expected margin of error tolerance of the methodology. Using the same analyses without terminating water delivery discharges at one foot depths resulted in the possibility of meeting up to 81% of the water supply demands. Increasing the amount of water available to meet water supply demands will likely increase project benefits in the natural system. As previously noted, these analyses will be updated and further refined during detail design efforts.

By maintaining a storage pool all-year, when excess water is available, the seepage reduction from LNWR benefit is still realized and is not expected to diminish. Similarly, groundwater recharge will still effectively take place and this benefit is not expected to be reduced as well.

Benefits derived through the capture of excess water in Hillsboro Canal to reduce freshwater slugs to the estuaries may be diminished, though this is expected to be an insignificant loss. With the reservoir full 30% to 40% of the time—mostly through the wet season that approximately coincides with the Hurricane season—available storage of excess flows is minimized after the first “fill-up” at beginning of the season. The reason supporting this minimization is that generally there is little water supply demand in between frequently occurring storm events that trigger releases to be made. The slug reduction benefit really takes when there has been a release from the reservoir that instrumentally provides more storage availability. Then, when a storm of consequence occurs, the reservoir is enabled to significantly reduce total runoff (read attenuate) that gives the true benefit.

A.A.3.2.3 Determine Hazard Potential Classification (HPC)

Reservoir and D-1 Embankment

The reservoir’s HPC has not changed from the preliminary Final PIR, i.e. remains with a High HPC. This was based on preliminary dam-break analysis performed during the WPA Feasibility Study efforts. Existing developed areas immediately to the east of the reservoir (and expected to the south in the near future) provides the reason why this HPC was selected without further refinement of the existing dam-break analysis. Therefore, D-1 Embankment will be designed appropriately for one where significant risk to life is of importance. However, the selected reservoir HPC does not necessarily mean that all embankment reaches have to be designed with the same criteria required for one with a High HPC. If an embankment reach can realistically be viewed as having a lower HPC—through numerical or logical analysis—it may be designed with other, but still appropriate design criteria (see DCM-2, Section 2.2). For this project, the north and western embankment reaches consist of the existing C&SF L-40 Levee. The downstream side of this levee (from the reservoir perspective) lies along the LNWR that is contained within its own perimeter levee system.

LNWR and L-40 Levee

LNWR is a natural everglades environment of ±221 square miles with permanent pool water levels (operating schedule 14.0ft-ngvd to 17.0ft-ngvd), emergent vegetation, and tree islands. Should a breach in L-40 Levee occur there would be: (1) no increase in risk to the public, (2) no loss of public property would be realized, and (3) no significant harm to the natural environment would occur. The first two reasons are obvious through examination of aerial photographs and local area knowledge, thus, require no further explanation.

The question now becomes, should a breach occur in L-40 Levee, would the uncontrolled discharge from Site 1 during the “dam-break” cause significant harm to LNWR. Stated in the preceding, LNWR covers an area of ±221 square miles with water levels governed by an operations schedule. The operations schedule reflects a desire, with respect to environmental consideration, to maintain the pool between 14.0ft-ngvd and 17.0ft-ngvd. The schedule fluctuates with seasonal timing (read hydroperiods) and is illustrated in the **Figure A.A-1.** Since the Site 1 reservoir covers a surface area of ±1,660 acres compared to LNWR ±141,440 acres—or 1.17% of LNWR—the reservoir is of insignificant size compared to the LNWR. Further, using the idea that a breach caused by storm conditions would most likely occur in the month of September, consider a surcharge reservoir stage at 18.00ft-ngvd and a conservative low pool in LNWR of 16ft-ngvd (storm that added 3 feet of water to reservoir added zero rainfall in LNWR). Simple calculations demonstrate that the breach would create an additional 0.0235 feet (0.28”) of water level in LNWR. Water quality impact is also taken into consideration because it is an important factor with respect to public law(s) and statue(s) intent in reducing nutrient loads into the everglades. With a reservoir Hurricane season operating pool of 5 feet, the 3 feet of rainfall is apportioned 37% of the stored volume. Remembering that a breach would occur during this time at the top, and not the bottom (LNWR permanent pool on downstream side), majority of the “breached” water would be rain water with low level nutrient loading. With this overall basis of understanding, the L-40 Levee was viewed as having an independent low hazard potential associated with dam-break scenarios and consequences thereof.

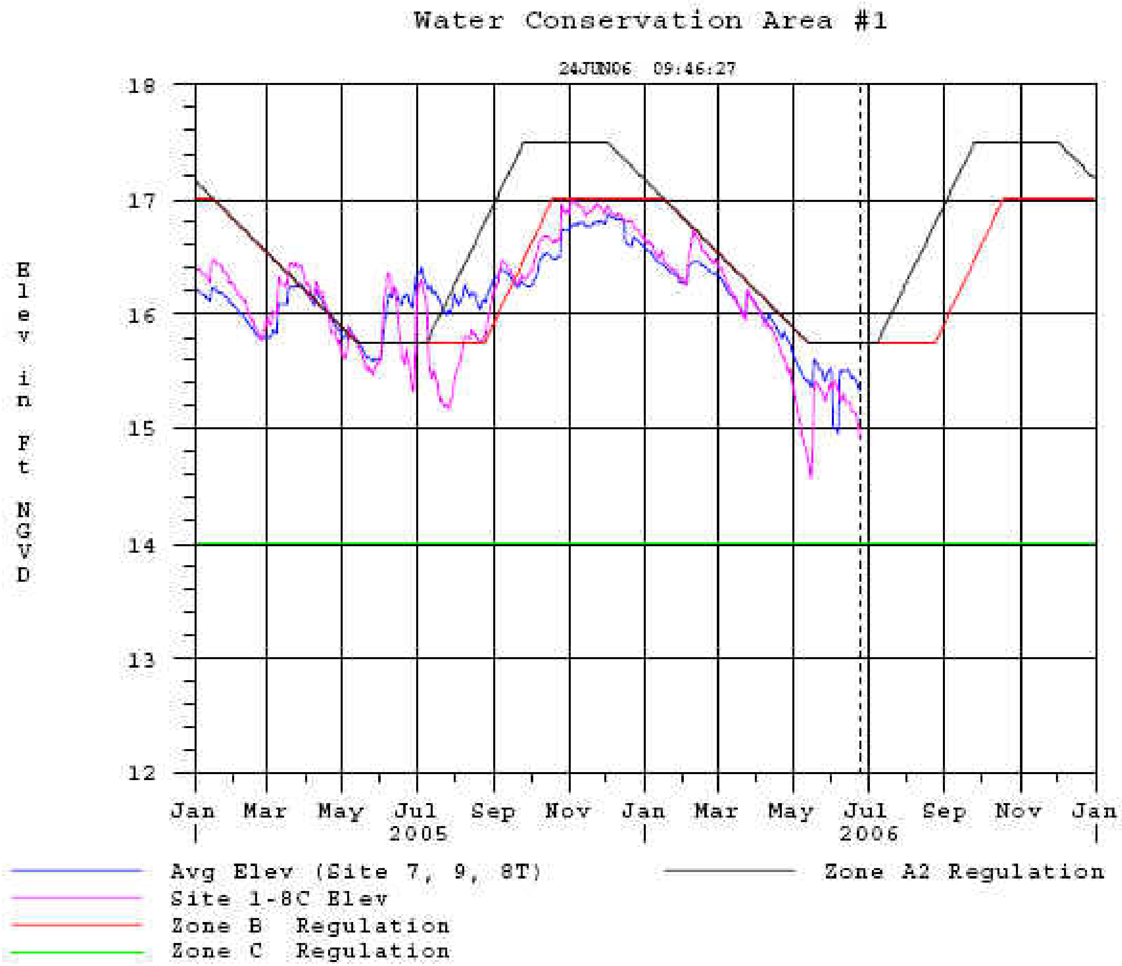

Figure A.A-1 – LNWR Operating Schedule

A series of **Progression Tables** are inserted in certain sub-sections to allow the reviewer a means to understand how the embankment design procedure moves sequentially forward relying on previous data/information to calculate the next step, thus resulting in the final design height of the embankment. Columns to the right are intentionally left blank until that sub-section's steps are performed as described, also sequentially.

Progression Table 1

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)						
8-foot (HHPC)						
8-foot (LHPC)						

A.A.3.2.4 Determine Appropriate Level of Design Criteria

From the Hazard Potential Classification sub-section, the D-1 Embankment was designed as a dam with a High HPC. The C&SF L-40 Levee was designed as a dam with a Low HPC. Although L-40 Levee will be designed with a lower HPC, the reservoir remains one with a High HPC. This is important because as a High HPC, the reservoir will be required to safely pass the Inflow Design Flood (IDF) using the Probable Maximum Precipitation (PMP) event regardless of the operating normal pool stage. The difference in design criteria is found mostly in the assigned wind speed for application and other combination of rain event with wind for the L-40 Levee, i.e. Low HPC. Design criteria and steps identified in DCM-2 (attached to addendum) were followed as the document abides by Federal and State Regulations and requirements.

Progression Table 2

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP					
8-foot (HHPC)	PMP					
8-foot (LHPC)	100-yr					

A.A.3.2.5 Appropriate Rainfall Event

As stated in preceding sub-section, the reservoir with a High HPC is required to safely pass the IDF using the PMP event. The following provides the steps in determining the PMP event with calculation results.

The PMP was developed according to guidelines from the Hydrometeorological Report (HMR) No. 51 (Probable Maximum Precipitation Estimates, Eastern U.S. and HMR No. 52 (Probable Maximum Storm Computation). A value of 55.7 inches was selected from the all season 72-hr, 10 mi^2 PMP chart.

A synthetic 72-hr PMP distributed rainfall event was developed according to HMR-52 to contain every PMP storm duration from 6 to 72 hours (**Table A.A-1**). The peak hour rainfall rate was calculated and smoothing was applied to yield the 1-hour distribution of the probable maximum storm (**Figure A.A-2**).

Table A.A-1 – PMP for Storm Durations 6 to 72 hours

Storm Duration (hr)	6	12	24	48	72
PMP (in)	32.0	38.7	47.1	51.8	55.7

Table A.A-2 - Precipitation for each 6-hour increment of the 72 hr PMP

6-Hour Increment	1	2	3	4	5	6	7	8	9	10	11	12
Precipitation (in)	1.0	1.0	1.0	1.0	1.0	1.2	1.8	5.4	32.0	6.7	3.0	0.7

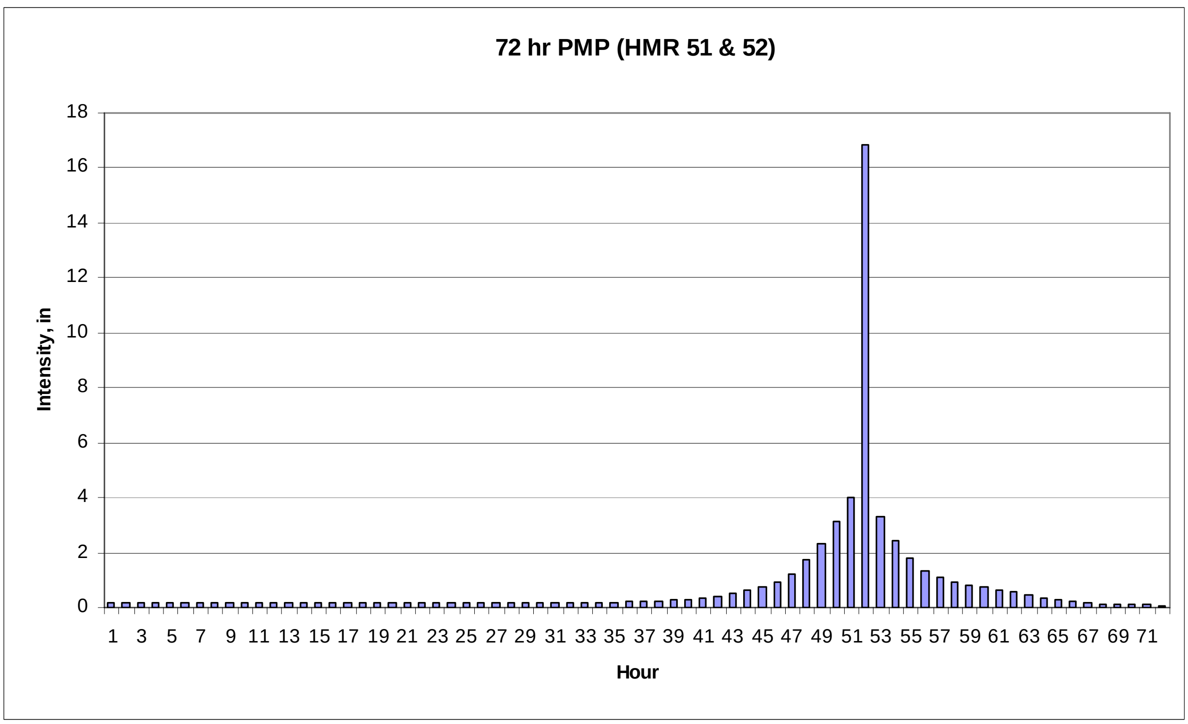

Figure A.A-2 Hourly Distribution of the Synthetic 72 hr PMP

A.A.3.2.6 Routing PMP with Determination of Appropriate Spillway(s)

Procedure for Analysis

The Corp's model HEC-RAS (River Analysis System) was employed to determine the maximum pool (or surcharge pool) elevation and peak discharge. The model configuration consisted of a storage area equal to 2.6 square miles with two weir outlets. Two overflow spillways are designed as lower sections of the levees to maintain levee integrity. One dual spillway, service and auxiliary, discharges into the Hillsboro Canal and the other auxiliary discharges over the L-40 Levee to the Loxahatchee National Wildlife Refuge. Since the impoundment itself is the tributary basin and its surface is considered impermeable when full, all storm precipitation was readily available for routing without any loss. The initial pool stage used in the routing model of Site 1 Impoundment was the off-season higher normal full pool stage of 18.00ft-ngvd. It is obvious that design that allows safe passage of the PMF with the higher starting stage would also meet requirements for the Hurricane season lower starting stage, i.e. 15.00ft-ngvd.

The service spillway crest elevation was set at the normal full pool stage. The 24-hour, 25-year return period rainfall was picked from SFWMD Technical Publication 81-3 May, 1981 Frequency Analysis of Rainfall Maximums for Central and South Florida. The 24-hour, 25-year rainfall amount for the Hillsboro basin was determined to be 13 inches. The ratio of 24-hour, 25-year rainfall to 72-hour, 25-year rainfall is 1.359. Thus the 72-hour, 25-year rainfall for the Site 1 impoundment is 17.7 inches. The document "Basis of Review for Environmental Resource Applications within the SFWMD, February 12 2006" sets the allowable discharge for the Hillsboro canal as 35cfs per square mile; this rate translates to 88.4cfs for the 2.6 square miles of impoundment. The length of the service spillway, 19 feet, was designed to limit the impoundment outflow to 88.8cfs during the 72-hr 25-yr storm. The resulting peak stage, 19.4ft-ngvd, was used to define the crest elevation of the adjacent auxiliary spillway.

The PMP analysis should consider an antecedent-setting event to address concerns over the sequencing of rainfall storms as described in the DCM-2. Two routings must be performed to determine governing circumstances:

a)
1. From normal pool, route 30%PMF 72-hr storm. Spillway on, but gates are inoperable.
2. Provide for a 3-day dry spell. Spillway on, gates are operable.
3. Route a 100%PMF 72-hr storm. Spillway on, gates are inoperable.
4. Provide for a 10-day dry spell. Spillway on, gates are operable.
5. Route a 30%PMF 72-hr storm. Spillway on, gates are inoperable.

b)
1. From normal pool, route 40%PMF-72hr storm. Spillway on, but gates are inoperable.
2. Provide for a 5-day dry spell. Spillway on, gates are operable.
3. Route a 100%PMF 72-hr storm. Spillway on, gates are inoperable.
4. Provide for a 10-day dry spell. Spillway on, gates are operable.
5. Route a 40%PMF 72-hr storm. Spillway on, gates are inoperable.

However, it was determined from the beginning that the proposed gated structure had the capacity (700cfs) to drawdown the impoundment stage to normal pool elevation between storms. Therefore, the single PMP storm was used for this analysis. The use of gates to return the pool to normal full pool level was also utilized by SFWMD Acceler8 consultants as referenced in the Draft Basis of Design Report (BODR) for Site 1 Impoundment submitted by Parson Brinckerhoff Quade & Douglas, Inc. (17 March 2006).

The PMP was routed through the impoundment using a 50 feet long crest for the Hillsboro auxiliary spillway. The length of the L-40 auxiliary spillway crest was optimized to 500 feet considering both, impoundment outflows as spillway overflow into Hillsboro Canal and LNWR, and resulting surcharges pool levels. Thus, the crest was set at elevation 20.4ft-ngvd corresponding to the 72-hr duration, 100-yr frequency storm surcharge plus wind setup induced by a wind speed equivalent to a Category 1 Hurricane. This design was chosen as the most appropriate and effective to limit the discharge into the Hillsboro Canal to a rate lower that what may be expected when protection of the structure would be of importance with respect to preventing a breach occurrence. This rate is also considered to have negligible impact to the basin under these conditions as all would be highly flooded combined with catastrophic wind damage caused by the design events, PMP with very high sustained wind speeds..

LNWR L-40 Levee Spillway

Early in the WPA Feasibility Study an overflow spillway on L-40 Levee (to discharge into LNWR) was ruled out based on water quality concerns and other legal issues (e.g. consent decree). The PDT discussed this complex issue at length. Some of the primary issues were the following: (1) elimination of the southern compartment from the footprint that required raising the northern compartment operating pool from 6-foot to 8-foot pool depth to compensate for lost storage volume, (2) realization that L-40 Levee height is ±26.00ft-ngvd and can be easily overtopped by wave overwash under design conditions, (3) wind-wave analyses performed earlier were insufficient and did not meet today's current standards requiring higher embankments, and (4) because of "when" L-40 Levee was constructed with original purpose, modifications of L-40 Levee would be cost ineffective, if not prohibitive—this includes raising the levee on existing peat and muck base with placement of full armoring on both sides of the levee.

The clear solution was to optimize the spillway design to lower maximum surcharge pool levels and to discharge on a very infrequent basis as steady overflow or as true overtopping--discounting wave splash and overwash as an insignificant volume compared to scale of project to LNWR--into LNWR. What must also be restated is that as the reservoir has a High HPC, it must be able to safely pass the IDF using the PMP event, i.e. ±55 inches.

The proposed project design now includes two separate spillway features, one that discharges into Hillsboro Canal and the other into LNWR. Because of water quality concerns, it is not desired to have excess reservoir water spill over into LNWR under any conditions. However, under infrequent extreme conditions, the allowance for such may prove to be very cost effective for the project. The overwash, i.e. spray and sporadic wave overflow, from wave action will occur for some storm conditions, though all are fairly infrequent and only under constant high winds for a finite duration of time when the reservoir is at or nearly full during the off-season operational pool levels, or highly surcharged during the Hurricane season above the normal full operational pool level. To prevent a spillway overflow into LNWR except under the most extreme conditions, the spillway crest elevation will be set above the 100-year surcharge stage routed with the starting pool level of 18.00ft-ngvd giving 19.79ft-ngvd, plus additional height to prevent a Category 1 Hurricane level wind speed of 1-hour duration, i.e. 75mph 1-minute average converted to 60.3mph of 1-hour duration, from “setting up” a water level that would overflow/overtop into LNWR. Wind setup has been calculated to be 0.58 feet giving a crest elevation of 19.79ft-ngvd + 0.58 = 20.37ft-ngvd—20.40ft-ngvd. Also, it is worth noting that the dual crested spillway discharging into the Hillsboro Canal has crest elevations of 1.0 feet and 2.4 feet below that of the L-40 Levee spillway so that discharge occurs into Hillsboro Canal first.

Off-season Overflow Expectations

A simple expected frequency of overflow into LNWR via the L-40 Levee spillway can be calculated as follows for the off season operational pool level at 18.00ft-ngvd:

- Rainfall is based on a 1 in 100-year return interval event, or 0.01.
- Probability of hurricane hit on Palm Beach County, 0.144.
  (A Dynamic Probability Model of Hurricane winds in Coastal Counties of the United States, Journal of Meteorology, May 2001)
- Alternative probability return interval of 25-year wind, or 0.25.
  (Updated Wind-Hazard Data and Wind-Damage Functions for Use in FEMA Benefit-Cost Analyses Submitted in Support of Hazard Mitigation Grant Program Applications in Florida, FEMA, March 2005).
- Probability of Storm reaching north versus south (wind direction), say 0.75. A lower probability of 0.67 would normally be used for directional variability that may cause impact, but 0.75 used to allow a little conservativeness in determination.
- Assigning a probability of occurrence outside of Hurricane Season, say 0.25.

Multiplying probabilities with 25-year wind provides a probability of 0.0005 for occurrence. If we consider that the 1 in 100-year storm that supplies rainfall also provides the wind speed, then probability for any given year is 0.002. In other words, expectation for overflow into LNWR is on the order of 1 in 500 years during the off season.

Hurricane-season Overflow Expectations

A simple expected frequency of overflow into LNWR via the L-40 Levee spillway can be calculated as follows for the off season operational pool level at 15.00ft-ngvd:

- Rainfall is based on a 1 in 10,000-year return interval event, or 0.0001.
- Probability of intense hurricane hit on Palm Beach County, 0.144.
  (Used same probability as Category 1 under off-season overflow expectations)

- Probability of Storm reaching north versus south (wind direction), say 0.75. A lower probability of 0.67 would normally be used for directional variability that may cause impact, but 0.75 used to allow a little conservativeness in determination.

Multiplying probabilities provides a probability of 0.00001 for occurrence. If we consider that the 1 in 10,000-year storm that supplies rainfall also provides the wind speed, then probability for any given year is 0.00008. In other words, expectation for overflow into LNWR is on the order of 1 in 12,000 years during Hurricane season.

Final Overflow Expectations

Although overflow/overtop has been looked at from a quasi-steady state flow, it is not meant to debunk the notion that wave over-wash and wave themselves may sporadically overtop the L-40 Levee spillway at a more frequent interval. They will. However, the time duration of major rain and especially wind storm events prevent the total volume of water crossing over from being significant than what can be expected under an essentially steady flow that can add volume continuously over a period of days (versus hours for waves). Also, this water coming from the top of the water column will necessarily consist of rainwater (non-runoff) as a major constituent, though stirred with water normally routed to the impoundment (e.g. urban runoff, excess LNWR water, seepage collected, etc).

Reverse Overflow Considerations

It has been suggested that a 6-foot cut into the L-40 Levee may provide a high probability that reverse flow—LNWR into Site 1 reservoir—may occur with disastrous results. A wind setup calculation utilizing a LNWR stage of 18.00ft-ngvd was performed with results indicating a setup of 3.34 feet could be expected. Parameters and discussion of parameters used in the analysis are provided as follows:

- Average natural grade, 14.00 ft-ngvd. This was assumed to provide a conservative somewhat shallow depth across the effective fetch. This is in acknowledgment that shallower depths create the highest fetches due to unbalance boundary stresses. The basis of natural grade selection was on examination of the 100-foot grid point DEM created for south Florida that references currently the best available data.
- Stillwater water stage, 18.00 ft-ngvd. This stage was suggested by SFWMD by teleconference call held 23 June 2006. Reference of Figure AA-1 illustrates this to be a reasonable assumption.
- Design water depth assumed, 4 feet. Along with the shallow depth assumption, it is acknowledged that the L-40 Borrow canal along the perimeter is deeper and would distribute backflow with less hydraulic resistance than reverse counter flow across the vegetated plain. This is believed to offset the possibility of added setup for the directional point of setup (assumed to be the spillway) on the relatively rounded concave arc formed by the L-40 Levee. If winds were directed as a ray

anywhere else, this assumption would be another conservative factor added to the analysis. This in itself reduces the probability of occurrence specifically for the reservoir.

- Effective fetch for LNWR, 2 miles (10,560 feet). This is in acknowledgment of dense emergent vegetation and tree islands that are encountered as one progressively moves toward the center of the refuge from the outer southeastern boundary (from the reservoir site).
- Applied design wind speed, 119mph. This is the DCM-2 recommended design wind speed for use in determining sound design for a High HPC reservoir.
- Bretschneider's model referenced in DCM-2 was used to determine wind setup. This model allows for exposed ground upstream of setup—invoking conservation of mass theory, same as stating that mass is balanced.

Stated previously, the calculates for setup resulted in a vertical setup of 3.34 feet to stage 21.34ft-ngvd, almost a foot over the L-40 Levee spillway crest elevation of 20.40ft-ngvd. The next question that arises is what does this mean? Assuming a 1-foot hydraulic head over the spillway, a weir coefficient of 3.0, and proposed spillway length of 500 feet, the inflow rate into the reservoir becomes ±1,500cfs. However, history through hurricane data collection over a century tells us that wind setup is a phenomena measured by hours, not days. In the New Seasonal Operating Schedule subsection, reference to USACE GDM is made that levees should be constructed to withstand 3 to 4 hours of hurricane exposure impacts. Now, assuming a conservative same wind direction and velocity duration of 6 hours for LNWR setup, the volume added in the reservoir due to overflow of LNWR setup would be 32,400,000cuft (743acft). Since the reservoir has a 1,660ac surface area, added stage to the reservoir would be 0.45 feet. This is considered to be inconsequential to the Site 1 reservoir embankment integrity with respect to the newly proposed seasonal operation schedule. Basis for this judgment is the understanding (based on model results) that extreme wind velocities used for design are the dominant parameter in wave development at Site 1. An increase of 0.45 feet of water depth would not significantly alter wave development under these extreme conditions.

It has been suggested to perform a full routing and wind-wave analysis to demonstrate insignificance of the reverse flow. This can be easily deduced through reasoning and knowledge of wind setup numbers only. Since the overtopping of the L-40 spillway occurs over a 6-hour timeframe with the 100-year wind, it cannot realistically be expected for the same wind to occur again in a sequential manner (back-to-back) as the design parameter for time duration is 3 to 4 hours as established by USACE GDM (prior sub-section). Therefore, assuming the average wind speed drops down to a Category 1 Hurricane after the previous 6 hours (eye-wall has passed). Wind setup for the Category 1 wind speed with PMP and additional LNWR surcharged pool is on order of 0.60 feet. However, wind setup without additional surcharge from LNWR, but still including the PMP with full 100-year wind speed is 2.22 feet. Added embankment height required due to water depth induced with "negative" wind setup is: + 0.60 feet + 0.45 feet – 2.20 feet = -1.15. In other words, the lesser wind immediately following the 100-year event prevents reverse overflow from being an increase of risk to embankment integrity under reasonable (and DCM-2) use of design parameters. This is a simplistic, though very conservative analysis because with the lower wind speed after the first 6-hours, wave height would diminish accordingly since we are dealing with shallow water waves and not mature deepwater sea waves.

Maximum Surcharge Pool Levels

Thus, the PMF maximum surcharge pool levels for the two starting normal full pool levels are the following:

- Hurricane season five-foot surcharge pool level = 19.58ft-ngvd
- Off-season eight-foot surcharge pool level = 21.52ft-ngvd

Table A.A-3 Inflow Design Flood Analysis

Probable Maximum Precipitation (PMP) 72-Hour Rainfall	55.7	inches
SFWMD 25-year, 24-hour Rainfall	13.0	inches
SFWMD 25-year, 72-hour Rainfall	17.7	inches
Basin Area	2.6	sq miles
Normal Pool Elevation (Service Spillway Crest Elevation)	18.0	ft-ngvd
Service Spillway Length	19.0	feet
Auxiliary Overflow Spillway Crest Elevation, Hillsboro (25yr, 72hr storm surcharge)	19.4	ft-ngvd
Auxiliary Spillway Crest Length, Hillsboro	50.0	feet
Auxiliary Spillway Crest Elevation, L40 (100yr, 72hr storm surcharge plus wind setup)	20.4	ft-ngvd
Auxiliary Spillway Crest Length, L40	500	feet
Peak IDF Instantaneous Inflow	27,418	cfs

Average ground elevation based on LIDAR data.

Table A.A-4 Selection of L-40 Auxiliary Spillway Crest Length

L40 Auxiliary Spillway length (ft)	Maximum IDF Pool Elevation (ft-ngvd)	Peak Outflow (cfs) L40	Peak Outflow (cfs) Hillsboro
0	21.93	0	996
50	21.85	248	954
100	21.78	463	922
150	21.73	655	897
300	21.62	1148	843
500	21.52	1683	797
1000	21.36	2674	724
2000	21.20	4110	656
4000	21.09	6566	608
6000	21.01	8301	578

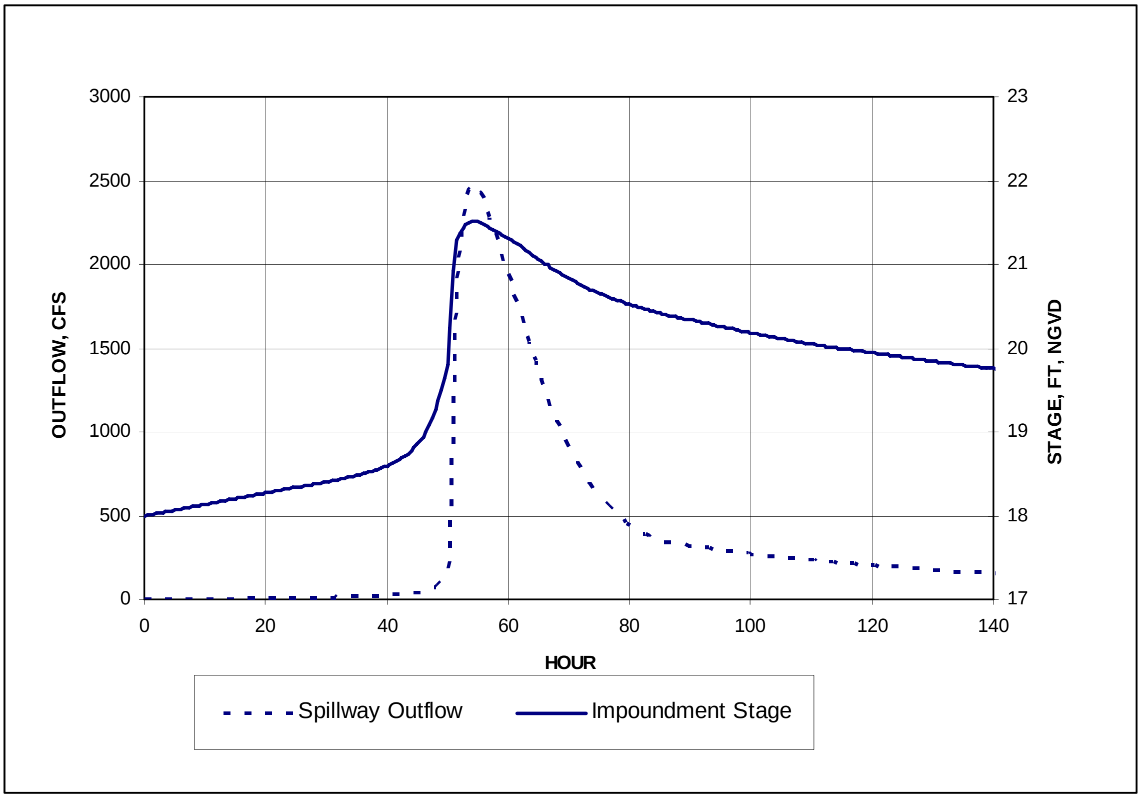

Figure A.A-3 Routing 72hr PMP with Service (19ft) and Auxiliary Spillways (50ft, Hillsboro and 500ft, L-40)

Progression Table 3

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP	19.58				
8-foot (HHPC)	PMP	21.52				
8-foot (LHPC)	100-yr	19.79				

A.A.3.2.7 Appropriate Fetch Length

Fetch Length Results

Fetch length when determined independently of other methods of setup and wave height calculations is used to determine wind setup or as input into numerical or spreadsheet-type models.

With removal of the L-508I Internal Levee feature from the preliminary Final PIR plan, a new fetch length is required for wind setup calculations. Using USACE EM recommended methodology (reference in DCM-2); the fetch length was determined to be 2.8 miles, little less than the combined two-cell alternative that included fetch lengths of 1.8 feet and 1.6 feet—3.4 miles. Difference of lengths is a result of cater-cornered analysis of the former separated cells to determine longest theoretical fetch length.

Attached References

Refer to attachment Wave Run-up and Levee Over-wash Analysis report for in-depth data, background information, procedural discussion, and results. DCM-2 has been provided for the reviewer’s convenience too.

A.A.3.2.8 Determine Appropriate Design Wind Speed

Three Wind Speeds

Three design wind speeds were determined for different requirements of applicability. As discussed in the New Seasonal Operating Schedule sub-section, the two seasonal operating normal pools supports the allow ability of determining two “reasonable” wind speeds for application in determining wind and wave impacts. These two wind speeds were selected acknowledging the High HPC placed on the reservoir and more specifically the D-1 Embankment. Also, to ensure that an unmodified L-40 Levee meets Low HPC criteria as presented in DCM-2, a third wind speed is also required.

Wind Speed Results

The three wind speeds used for analyses are provided as follows:

- For the 5-foot Hurricane season operational normal pool, the design 100-year wind speed was applied per DCM-2. This design wind speed is as calculated 119.1mph.
- For the 8-foot off-season operational normal pool, the design Category 1 Hurricane wind speed was applied. This design wind speed is as calculated 68.3mph.
- For the L-40 Levee with an accepted lower risk in case of failure, a wind speed of 60mph was used per DCM-2 for a Low HPC.

Design wind speeds as assigned are the 1-hour average wind speed. The 100-year wind speed is thus converted from the 3-second gust, the Hurricane referenced wind speed is thus converted from the 1-minute average, and the 60mph design wind speed is what it is.

<u>Attached References</u>

Refer to attachment Wave Run-up and Levee Over-wash Analysis report for in-depth data, background information, procedural discussion, and results. DCM-2 has been provided for the reviewer's convenience too.

Progression Table 4

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP	19.58	100-yr			
8-foot (HHPC)	PMP	21.52	Cat 1			
8-foot (LHPC)	100-yr	19.79	60mph			

A.A.3.2.9 Determine Wind Setup

<u>Wind Setup Results</u>

With removal of the L-508I Internal Levee feature from the preliminary Final PIR plan, calculated vertical setup results in a higher setup than the same plan with two-cell reservoir individually determined. This is because wind setup models account for the linear proportionality effect fetch length has on wind setup. Also, what should be noted (that most reviewers find counter-intuitive) is the phenomenon that shallower waters provide higher wind setup expectations (with all other physical boundary characteristics remaining the same). The reason for this is the relationship between the balance of boundary surface stresses and the creation of boundary layer effects on movement of fluids against other fluids of different viscosity, or fixed boundary with surface roughness.

The vertical wind setup determine through analyses are provided as follows:

- For the 5-foot Hurricane season operational normal pool with PMF routing surcharge stillwater level stage of 19.58ft-ngvd and application of the design 100-year wind speed, wind setup was calculated to be 2.22 feet.
- For the 8-foot off-season operational normal pool with PMF routed surcharge stillwater level stage of 21.52ft-ngvd and application of the Category 1 Hurricane wind speed of 68.3mph, wind setup was calculated to be 0.62 feet
- For the L-40 Levee with an accepted lower risk in case of failure, the 8-foot pool with 100-year routing surcharge stillwater level pool of 19.40 and application of the design 60mph wind speed, wind setup was calculated to be 0.58 feet.

Wind setups for the less than 100-year wind speeds were calculated based on the average value provided by Zeider Zee and Sibul formula models as recommended in cases where Bretschneider does not provide a verifiable solution. Note that design wind speeds as assigned are the 1-hour average wind speeds. The 100-year wind speed is thus converted from the 3-second gust, the Hurricane referenced wind speed is thus converted from the 1-minute average, and the 60mph design wind speed is what it is. Also, to demonstrate the difference of effect pool depth has on setup, a wind setup with the surcharge pool stage of 21.52 with application of the design 100-year wind speed was calculated. The resulting wind setup was 1.88 feet, 0.34 feet less than the Hurricane season PMF surcharge pool stage of 19.58ft-ngvd.

<u>Attached References</u>

Refer to attachment Wave Run-up and Levee Over-wash Analysis report for in-depth data, background information, procedural discussion, and results. DCM-2 has been provided for the reviewer’s convenience too.

Progression Table 5

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP	19.58	100-yr	2.22		
8-foot (HHPC)	PMP	21.52	Cat 1	0.62		
8-foot (LHPC)	100-yr	19.79	60mph	0.58		

A.A.3.2.10 Determine Wind-Wave Heights

<u>Wind-Wave Height Results</u>

With removal of the L-508I Internal Levee feature from the preliminary Final PIR plan, calculated wave height results in higher waves than the same plan with two-cell reservoir individually determined. This is because in the case for this reservoir and assigned design wind speeds, waves are fetch-limited and do not reach deepwater wave lengths or heights. Thus, with additional fetch, additional wave height is achieved or is increased. Wind-wave heights were calculated using the numerical STWAVE model as required by Jacksonville District for final product analyses in the case of the High HPC reservoirs. The L-40 Levee wave heights were calculated through spreadsheet based on steps provided in DCM-2. Since the spreadsheet method simulates idealize conditions—perfectly smooth reservoir bottom with no frictional value—the results are a little higher than what would be expected in the field, in comparison to STWAVE model results.

Wind-wave height results are provided as the significant wave height, or the top1/3 wave height to be expected. Waves in nature are irregular (versus laboratory) that occur over a spectrum often referred to as a Rayleigh distribution for convenience. The significant wave labeled as irregular is inputted into the wave run-up analysis to determine ultimately the overtopping, or better described as overwash, quantification required to examine embankment heights. The significant wave heights determined through analyses are provided as follows:

- For the 5-foot Hurricane season operational normal pool with PMF routing surcharge stillwater level stage of 19.58ft-ngvd and application of the design 100-year wind speed, the significant wave height was calculated to be 4.49 feet.
- For the 8-foot off-season operational normal pool with PMF routed surcharge stillwater level stage of 21.52ft-ngvd and application of the Category 1 Hurricane wind speed of 68.3mph, the significant wave height was calculated to be 3.15 feet
- For the L-40 Levee with an accepted lower risk in case of failure, the significant wave height is not available to the writer, but calculated embankment height required to meet the Low HPC is illustrated under the final results provided in tabular form.

Attached References

Refer to attachment Wave Run-up and Levee Over-wash Analysis report for in-depth data, background information, procedural discussion, and results. DCM-2 has been provided for the reviewer's convenience too.

Progression Table 6

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP	19.58	100-yr	2.22	4.49	
8-foot (HHPC)	PMP	21.52	Cat 1	0.62	3.15	
8-foot (LHPC)	100-yr	19.79	60mph	0.58	-	

A.A.3.2.11 Determine Wave Run-up and Overwash

Model Application

Wave Run-up and overtopping, or better described as overwash, calculations were performed using the numerical ACES model. The ACES model provides quantification of overwash for smooth (plated armor) and rough sloped (riprap) surfaces. For soil cement "stepped" alternative, a simple local non-shareable sub-routine was inserted into code to provide the necessary adjustments for convenience. This slight model modification can be done analytically by others without need of sub-routine addition.

Overwash Rate and Acceptability

The acceptable overwash rate for earthen embankments may be defined in three ways: (1) based on the research provided by the Dutch Technical Advisory Committee on Flood Defense (TACFD) findings, (2) based on risk and uncertainty, what is the probability of event occurring, that failure may occur, and uncertainty that model numbers provide, and (3) both decision making tools weighing in for final design.

Dutch Guidance

TACFD provides the following overtopping rates for indication of erosive forces:

- 0.001cfs/lf for sandy soil with poor grass cover, reported as 0.1L/s/m.
- 0.011cfs/lf for clayey soil with good grass cover, reported as 1.0L/s/m.
- 0.107cfs/lf for clay cover with grass according to requirements for outer slope, reported as 10L/s/m.

The problem for the engineering team is the possible interpretations of the guidance as provided (e.g. does it represent an overtopping rate that includes small damage or no damage to the downstream slope?). What durations of time (for continuous exposure) were investigated to lead to their conclusions? Reasons for need of answers to these types of questions lie in the fact that the overtopping events for the project are necessarily infrequent. For instance, USACE is supportive of repairing damage to the embankment's downstream slope if required after the storm has passed (interior slope is armored). What is of paramount interest is that the integrity of the embankment is maintained throughout the design storm event.

Risk and Uncertainty

A risk and uncertainty paper was written for the CERP Everglades Agricultural Area project that included a detail risk and probabilities analysis (see attachment *Risk Assessments for CERP Impoundments*). This paper is considered equally applicable to the Site 1 project because the region is homogeneous enough to share many of the meteorological events and probabilities. One conclusion made was that designing for a zero overtopping rate in combination with the design events (PMP + 100-yr wind) would not--in practical terms--reduce the risk of a breach or embankment failure when compared to other causes that are much more probable. For instance, the number used to quantify the probability of a failure due to piping was assigned a 0.0001 (10^{-4}). The probability of the design event for overtopping is that of 0.000001 (10^{-6}). This means that failure due to piping is 100 times more likely than the design event to occur and/or cause failure. When approaching a design for safety and cost-effectiveness, these results are viewed highly with practicality.

There are many uncertainties other than the probabilistic view provided as well.
One uncertainty of note is the ability of the downstream slope to withstand erosive actions under overtopping flows that include spray in the quantified resulting rates. Literature research has proved challenging in that all prior studies found were based on true overtopping (or classic spill over) versus wave overtopping as overwash. Theoretical avenues, including calculation of downstream velocities, are not applicable because of the irregularity of waves contributing to the wash, as well as time steps required if modeled with a program such as UNET. This particular uncertainty is in all probability the largest and the one that has the most impact on acceptable overtopping rate decision making. Another variable that contributes to the overall uncertainty of erodibility is the index of the local material to be used for construction. Together, these particular uncertainties have more effect on decision making than all others combined, though they are not quantifiable.

Other uncertainties include, but are not limited to simulated rainfall distribution curves in relation to cyclonic events (with design wind speeds), lag time between expected design wind speed and maximum surcharge pool, and local vegetal resistance to overtopping flows and sustainability.

Probability

Since there are limitations in our ability to predict and protect structures against every possible event (or combination of events), risk and the management of risk are key elements in the design and construction of all structures, particularly impoundments. Consequently, a risk assessment for these impoundments is an essential step in the design process to prevent these structures from being over or under designed. To perform a risk assessment, sufficient analyses must be conducted to provide a clear understanding of the exposure to the array of risks associated with the proposed facility, the need for risk reduction measures, the potential level of risk reduction for each measure, and the need for additional investigations to validate the key assumptions.

In order to adequately evaluate risk, we must first identify and understand the various failure modes for the structure with the recognition that this identification/understanding process is the foundation upon which the risk assessment is built. This assessment must be performed using considerable care and skill to avoid significant distortions in risk estimates. Another risk factor that must be considered is the evaluation and comparison of failure probabilities from an initiating event or events, and the case of the multiple or combined probabilities. For example, the probability of various, specific, events which may lead to failure must be considered along with the combined probability of two or more events occurring simultaneously. For a specific project configuration, the geotechnical or structural characteristics of the embankment materials may govern the overall design as opposed to the hydrologic/hydraulic conditions of the facility. By considering an array of that clearly defines the specific initiating modes of failure as well as the combination of events that may occur, the evaluation of risk based methodologies will be more easily understood, evaluated, and appropriately mitigated.

For impoundments, one of the key components leading to a safe design which is completely based on an accurate assessment of risk is the determination of the embankment height. Since dams are capable of placing human life at risk or causing catastrophic damage, should they fail, they must be designed with appropriate superiority, appurtenance works, and regulation schedules so that the dam will safely pass the IDF. However, the requirements of ER 1110-8-2 (FR) only require that the dam "safely pass", not fully contain, the IDF. Therefore, a dam may sustain damage during this rare event as long as it retains its structural integrity.

When determining superiority, two major elements to consider are overtopping and over-wash, which are defined as follows:

- Overtopping. A static and/or dynamic event which occurs when the height of the pool and/or associated waves exceed the maximum height of the embankment.

- Over-wash. A dynamic event where the maximum height of the wave does not exceed the embankment height, but the wave run-up and/or wind borne spray associated with the energy dissipation of the breaking waves exceed the embankment height.

The differentiation between these two events is of extreme importance, particularly for impoundments with a relatively large spatial extent and exposure to severe wind events, which can create significant dynamic pool conditions. To completely prevent all over-wash during an IDF event, embankment superiorities must be extremely high which adds significant cost to projects. However, ER 1110-8-2 states that "zero over-wash is not always required under infrequent high pool conditions, but it is required that over-wash will not be of such a magnitude and duration as to threaten the safety of the dam." In other words, infrequent over-wash events may not warrant absolute prevention since these pulsed loads may not lead to dam failure. Therefore, if the design over-wash event occurs, then the embankment height may be optimized to minimize costs while ensuring the structural integrity of the impoundment under the extreme event. For example, a large earthen embankment constructed of highly erosive materials may require zero over-wash in order to maintain structural integrity. However, if the embankment is constructed of non-erodible materials, such as monolithic or roller compacted concrete, the dam may be able to tolerate not only over-wash but also overtopping.

Design flood conditions for the CERP impoundments include an IDF based on the hazard classification of the impoundment and potential impacts due to flooding. Unlike typical reservoirs, CERP impoundments are formed by a fully encircling embankment that will be filled by pumping and direct precipitation. For these impoundments, there are no gravity (tributary) inflows. In addition to precipitation and pumped inflows, considerations are also given to wind setup and wave run-up for determining the final embankment heights due to annual tropical storm and hurricane events.

Corps of Engineers Dam Safety criteria requires that any dam “safely pass” the probable maximum flood event. In South Florida, this is typically associated with either a large tropical storm or a series of smaller tropical storms which produce large amounts of rain. For the EAA project, several scenarios were evaluated and a probability matrix for the South Florida region was developed to identify the IDF based on numerical modeling of the hydrologic conditions for CERP impoundments which has been clearly defined in the CERP Design Criteria Memorandum (DCM-2), titled, “Wind and Precipitation Criteria for Freeboard.” The methodology described in DCM-2 is the accepted industry standard for IDF development of the PMF and subsequent calculation of wind set-up and wave run-up values. A study was conducted through the Hydrologic Engineering Center (HEC) to determine the risk of overtopping given the proposed embankment height and the increased residual risk for other, lower embankment heights. While specifically utilizing data for study of the EAA project, similar generalizations can be drawn that are applicable to other facets of the Site 1 project.

Once the appropriate design events are established, the overall superiority can be developed through statistical and probabilistic evaluations of precipitation and wind events in conjunction with wave height/run-up estimates generated from accepted numerical models. Assessing the probability of the maximum precipitation/wind blown reservoir stage was conducted by computing the joint probability of these 2 events occurring simultaneously. Data available for building probability distributions of wind speed and precipitation were:

1) Hurricanes hitting Florida, 1894 to present (with data gaps)
2) Daily precipitation and maximum wind speed at West Palm Beach (1965-2004) and Ft. Lauderdale (1973 – 2004) from the NCDC
3) TP-49 reported 100-year, 3-day precipitation of 16 inches and ASCE reported 100-year wind speed of 125 mph (3-second gust)

From the data available, frequency curves were developed to estimate the probabilities for the worst case scenarios. Initially, the wind and precipitation were assumed to be independent, which is the simplest analysis to conduct. Based on this assumed independence, the probability of seeing any combination of rainfall and wind speed is simply the product of their separate probabilities:

$$P\ (\text{rain}>r \text{ and wind}>w) = P\ (\text{rain}>r) * P\ (\text{wind}>w)$$

Following this logic, the computations for the hydrologic matrix are summarized in **Table A.A-5**, below. Based on this data, it is obvious that Case 1 is by far the least probable (maximum rainfall paired with a rare wind event) and that Case 3 is most common (rare wind event paired with normal reservoir pool). However, regulations govern that a high hazard dam such as the Site 1 reservoir must be able to safely pass the IDF. Therefore, for this project, Case 1 governs the design from a hydrologic standpoint.

Table A.A-5 Probability of Wind/Rain Combinations

Case	3-day rainfall	Probability	Max. Wind Speed	Probability	Joint Probability
1	PMP, 57.5in	.0024% or 1 in 42000	100-year, 125 mph (3-sec gust), 102 mph (1-hour dur)	1% or 1 in 100	1 in 4,200,000
2	100-year, 16in	1% or 1 in 100	Category 5 hurricane, 156 mph (3-sec gust), 124 mph (1-hour dur)	.21% or 1 in 490	1 in 49,000
3	normal pool 0in	99.9% or ~1 in 1	PMW, 200 mph (3-sec gust), 161 mph (1-hour dur)	.0024% or 1 in 42000	1 in 42,000
4	Hurricane Easy 39in	.032% or 1 in 3100	Hurricane Easy, 125 mph (3-sec gust), 102 mph (1-hour dur)	1% or 1 in 100	1 in 310,000

While this is a simple analysis, it is not the most precise since it assumes that the variable of interest is impacted by only one other variable. However, the variable of interest for the Site 1 is the maximum reservoir stage and it is impacted by two variables: rainfall and wind. However, a frequency curve showing the probability of any stage provides the most complete answer since there are multiple combinations of wind and rain that could produce the same maximum reservoir stage and the probabilities of all of those combinations must be summed to accurately state the probability of that stage.

Based on the results from these analyses, the **Figure A.A-4** shows the relationship between embankment superiority and flow rate at the inner rim of the embankment.

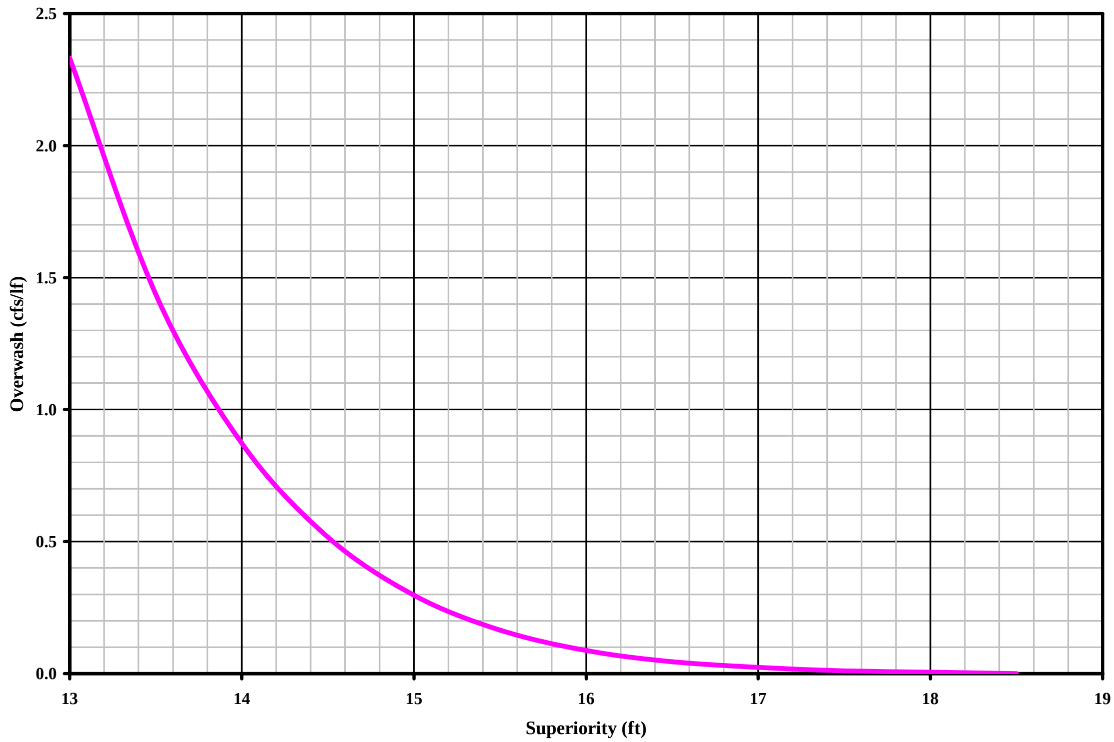

Figure A.A-4 Superiority versus Flow Rate at Embankment Inner Rim

The asymptotic relationship of the curve in **Figure A.A-4** is indicative of diminishing return of over-wash to superiority and considerations should be given to assess the probability of failure due to geotechnical or structural conditions. The probability of dam failure has been studied by several researchers including Cullen (1990), International Committee on Large Dams (ICOLD, 1997), Foster et al. (1998), and Foster et al. (2000). In general, the probability of failure was determined through a statistical examination of over 45,000 existing dams worldwide. ICOLD determined that the probability of failure for all embankment dams ranged from 1.0x10-3 to 4.0x10-3 and that the probability of failure from internal erosion or piping was twice as likely as through overtopping. Foster et al. (1998) developed similar findings with a cumulative probability range from 6.3 x 10-4 to 3.5 x 10-3. Foster et al. (1998) suggested that the probability of failure from internal erosion or piping was similar to that of overtopping. For failures caused by internal erosion or piping, Foster et al. (2000) developed more specific probability estimates of various dam types through a detailed review of available design/construction information. **Table A.A-6** lists the average annual probability of dam failure for the most common types of embankment dams studied.

Table A.A-6 Probability of Failure for Embankment Dams

Dam Type	Probability
Homogeneous Earthfill – Average Annual Probability	2.09x10-3
Earthfill with filter – Average Annual Probability	1.9x10-4
Zoned Earthfill – Average Annual Probability	1.9x10-4
Zoned Earth and Rockfill – Average Annual Probability	1.5x10-4
Concrete or other Face on Earthfill – Average Annual Probability	6.9x10-4
Concrete Core with Earthfill – Average Annual Probability	<1.3x10-3

Similar relationships for failure can also be drawn for other construction material types such as monolithic concrete or RCC. Structural design guidance moved to a reliability approach in the early 1980s and design methodology used by USACE and nationally recognized specifications and standards specify appropriate nominal load values and combination of loads to be used in design along with load factors to be applied to different categories of loads and resistance factors applied to material strengths. These loading requirements have evolved gradually and have been studied using probabilistic methodology.

Therefore, the relative probabilities of failure for a zoned earth embankment or a RCC embankment are on the order of 1.9x10-4 or 1.0x10-4, respectively. As indicated in **Table A.A-6**, the relative probability of the design hydrologic event has been determined to be on the order of 1 in 4,200,000 or 2.4x10-7 (noting that this event in itself may not result in failure but merely represents the probability of occurrence). As noted earlier, with an embankment and superiority established at a level for full containment of the IDF, the relative probabilities of a geotechnical or structural circumstance causing uncontrolled release is more likely than the probabilities of over-wash with the combined extreme events (resulting in failure). From this comparison it appears that the probabilistic criteria for determining the crest height should not be more restrictive than that commonly accepted for geotechnical or structural reliability.

Recognizing that failure of the facility is more likely to occur due to a latent geotechnical or structural defect than to a rare hydrologic event, 17 feet of superiority represents a much more acceptable level of over-wash for the Site 1 reservoir.

Acceptable Overtopping (as Overwash) Rate

Jacksonville District made a decision for the Site 1 Impoundment project that the overtopping rate of 0.1cfs/lf was the limit of acceptability. This decision is not a general statement, but to be specifically applied only to this project. The decision was based on both, acknowledgement of research and wide acceptance of results thereof by the Dutch, and on the risk and uncertainty analysis that provided insight to event probabilities.

Attached References

Refer to attachment Wave Run-up and Levee Over-wash Analysis report for in-depth data, background information, procedural discussion, and results. DCM-2 has been provided for the reviewer's convenience too. Also, refer to attachment Risk Assessments for CERP Impoundments specifically for risk and uncertainty analyses.

Progression Table 7

Normal Pool	Rainfall Event	Surcharge Stage	Wind Speed	Wind Setup	H_S Wave Height	Max OW /Height
		(ft-ngvd)		(ft)	(ft)	(cfs/lf)
5-foot (HHPC)	PMP	19.58	100-yr	2.22	4.49	0.051[1]
8-foot (HHPC)	PMP	21.52	Cat 1	0.62	3.15	< 0.001[2]
8-foot (LHPC)	100-yr	19.79	60mph	0.57	-	< 0.007[3]

Note: 1. 17-foot Perimeter Embankment with stepped soil cement armoring.
2. 17-foot Perimeter Embankment with stepped soil cement armoring.
3. 16-foot L-40 Levee with smooth soil cement armoring. Levee was modeled with a 1 on 3 slope. Since it is now known that the slope is shallower (closer to 1 on 4), overwash will be less than that recorded here.

A.A.3.2.12 Select Embankment Height and Design

D-1 Embankment

Embankment Height

The selected design height of the D-1 Embankment is 17 feet above average natural grade. On basis of LIDAR data, average natural grade is assumed to 10.00ft-ngvd placing embankment crest at 27.00ft-ngvd. Although the acceptable overtopping rate (as overwash) is 0.1cfs/lf, the highest overtopping rate under design conditions is ½ that at 0.051cfs/lf. Because of the shape of the reservoir, lower overtopping rates are expected for various reaches of the embankment (not L-40 Levee). This embankment height is considered appropriate for Site 1 with a High HPC given the newly proposed operations schedule implemented for the reservoir that incorporates seasonal pools based on probabilities of design events.

A check on design height to ensure it meets minimum height required for High HPC earthen embankments by USACE Regulations was performed (ER 1110-8-2(FR), Paragraph 9.c). The designed height exceeds minimum USACE requirements as provided calculations demonstrate.

- Starting with an 8-foot pool for normal pool stage (18.00ft-ngvd).
- Add PMF surcharge storage height of 3.52 feet (21.52ft-ngvd).
- Add 5-foot superiority to surcharge pool stage (26.52ft-ngvd).
- Design height 17.00 feet (27.00ft-ngvd) > 16.52 feet (26.52ft-ngvd).

Embankment Design

The embankment will have side slopes 1-on-3 with a crest width of 12 feet. Revetment includes armored sloped surfaces on pool side and grass on the downstream side. Revetment also includes a footer on pool side for toe protection. Armoring will consist of a composite surface of soil cement. Smooth soil cement plate from and including toe to height representing wave impact zone (13.00ft-ngvd to 14.00ft-ngvd). Stepped soil cement, 12" rise, 96" horizontal, forms from wave impact zone to crest of 27.00ft-ngvd.

Cost-Effectiveness of Design

A cost-effective analysis was performed between the selected embankment design and one higher with smooth armor plate. The result was both costs were essentially the same, i.e. well within the margin of error in cost estimation procedures. Primarily for aesthetics reasons the lower 17-foot embankment was selected (smaller height in flatlands more pleasing to the eye than taller structure).

<u>L-40 Levee</u>

Levee Height

The L-40 Levee exists with a crest elevation of ±26.00ft-ngvd. This is one-foot below the proposed D-1 Embankment height of 27.00ft-ngvd. It was decided by the Jacksonville District that L-40 Levee has a Low HPC, thus a lower design criteria. This criterion was established in the DCM-2 guidance and was used to verify its capability of meeting same criteria with the off-season higher normal pool of 18.00ft-ngvd. Performing the IDF with the 100-yr storm and applying a 60mph design wind speed, the resulting overtopping rate of less than 0.007cfs/lf. This number is conservative in that a steeper slope of 1-on-3 was modeled versus the known existing slope of 1-on-4. Acknowledging this rate as it is, it was considered acceptable by the Jacksonville District for a Site 1 levee reach with a Low HPC.

A check on design height to ensure it meets minimum height required for Low HPC earthen embankments by joint USACE and non-Federal sponsor agreement was performed (DCM-2, Section 2.2.2). The designed height exceeds minimum DCM-2 requirements as provided calculations demonstrate.

- Starting with an 8-foot pool for normal pool stage (18.00ft-ngvd).
- Add IDF surcharge storage height of 1.79 feet (19.79ft-ngvd).
  (design IDF for Low HPC is the 100-yr 72-hr storm).
- Add 3-foot superiority to surcharge pool stage (22.79ft-ngvd).
- Existing height 16.00 feet (26.00ft-ngvd) > 12.79 feet (22.79ft-ngvd).

Levee Design

The levee will retain its side slopes 1-on-4 with the existing crest width. Revetment includes armored sloped surfaces on pool side and grass on the downstream side. Revetment also includes a footer on pool side for toe protection. Armoring will consist of smooth soil cement plate of two thicknesses: thinner plate outside of wave impact zone and thicker plate within wave impact zone (13.00ft-ngvd to 21.50ft-ngvd).

A.A.3.3 SG-526B Service and Auxiliary Overflow Spillway (Hillsboro Canal)

Service Spillway

The emergency overflow spillway included in the preliminary Final PIR plan was not assigned a structure number and consisted of a 50-foot concrete spillway. It was single crested with a crest elevation set to contain the 25-yr 72-hr storm without discharge. This in effect eliminated the need to do a pre- post-project runoff comparison calculation and did not contribute to discharge that was later allowed per DCM3 agreement. The volume rate of discharge allowed is established in "Basis of Review for Environmental Resource Applications within the SFWMD, February 12 2006" that sets the allowable discharge for the Hillsboro canal as 35cfs per square mile. This constraint was used to design the "service spillway" portion of the spillway structure. See the Routing PMP with Determination of Appropriate Spillway(s) sub-section for additional information on methodology in determining hydraulic function.

Auxiliary Spillway

The crest elevation of the "auxiliary spillway" portion of the spillway was set by equating it with the 25-yr 72-hr storm routed surcharge pool stage, at 19.40ft-ngvd. This may be raised slightly to accommodate the 100-yr 72-hr event in the plans & specification phase of the project without need of modifying embankment heights. The decision on final storm accommodation will be made jointly between the non-Federal sponsor and USACE. The width of the spillway was designed in conjunction with SG-526C L-40 Spillway and the PMF storm sequence. It was decided to constrain the discharge into Hillsboro Canal with a 50-foot auxiliary spillway to no more than 800cfs, even though in case of a PMP event the Hillsboro Canal is expected to be out of bank. But, for typical lesser storms, including that of the 100-yr 72-hr storm event, the discharge will be much less and will not reduce level of service flood protection. Should level of service be encroached by operation of the spillway, diesel motors for the SG-525A Inflow Pump Station are capable of return pump operations until discharge no longer requires active management.

Design Parameters

Spillway Description: Combination 2-crested non-gated overflow spillway.

Service Crest Elevation:	18.00	ft-ngvd
Service Crest Length:	19.0	feet
Auxiliary Crest Elevation	19.40	ft-ngvd
Auxiliary Crest Length	50.0	feet
Downstream Slope of Spillway	1 on 3	
Energy Dissipation Method:	Concrete Off-Setted Baffle Blocks	
Discharge Point	Basin directly connected with Hillsboro Canal	

Main Material of Construction	Concrete
Structure Toe Protection	5-foot Horiz. Toe at 0.00ft-ngvd (7 feet underwater) with Riprap

A.A.3.4 SG-526C Auxiliary Overflow Spillway (L-40 Levee)

Background

See LNRW L-40 Levee Spillway in the Routing PMP with Determination of Appropriate Spillway(s) sub-section for more background information on this newly proposed structure.

The structure was designed to cost effectively pass the IDF routed with the PMP event safely as required in USACE Regulations for a High HPC reservoir. Care was taken in the design of the crest elevation to reduce potential impacts to LNWR, including the discharging of excess storage first into Hillsboro Canal. The reservoir will be able to accept the 100-yr 72-hr storm surcharge pool with a Category 1 Hurricane wind speed generated setup before spillway discharge as overflow takes place. However, wave splash and some waves themselves will pass over for short durations measured in hours though, but not days. It is currently thought that this design is appropriate with due consideration for the environment and legal issues regarding such. If under further review, the spillway crest must be raised, the spillway length will exponentially increase to maintain D-1 Embankment height of 17 feet.

Design Parameters

Spillway Description: Single crested non-gated overflow spillway.

Auxiliary Crest Elevation	20.40 ft-ngvd
Auxiliary Crest Length	500.0 feet
Downstream Slope of Spillway	1 on 4
Energy Dissipation Method:	None on Slope, Riprap at Toe.
Discharge Point	LNWR into L-40 Borrow pool.
Main Material of Construction	Smooth Soil Cement Plate Armor, both sides of L-40 Levee.
Structure Toe Protection	5-foot Horiz. Toe at bottom of L-40 Borrow pool, xx.yyft-ngvd (z feet underwater) with Riprap.

A.A.3.5 SG-525A Inflow Pump Station

Background

In the preliminary Final PIR plan, the SG-525A Pump Station includes a total capacity of 1,360cfs. However, based on SFWMD Acceler8 consultant's water budget analyses, the inflow pump station is proposed to have a reduced total capacity of 650cfs for inflow (600cfs) and seepage management (50cfs) purpose.

The Restudy identified through SFWMM (2X2) modeling efforts a 700cfs inflow pump station when Site 1 footprint contained a southern compartment with an overall storage depth of 6 feet. During the WPA Feasibility Study effort, the capture of up to 500cfs from NSID was supported when storage volume was available. This increased the inflow pump capacity to 1,200cfs. Also during WPA efforts, the southern compartment was removed from the plan with an increase change in operational pool depth required of the northern compartment from 6 feet to 8 feet. With this additional pool, an added seepage pump capacity was identified as requiring an additional 150cfs capacity to be collocated with the inflow pump station. This increased the inflow pump station total capacity to 1,350cfs with seepage management function. Another 150cfs was added to the inflow design based on PDT discussions to offset expected decrease of basin storage volume with loss of lands for the future CERP Ag Reserve Impoundment project. Final size of inflow pump station recommended in WPA Draft Feasibility Study and Draft Site 1 Impoundment PIR was a pump station inflow capacity of 1,500cfs.

Additional analysis between the Draft PIR and the preliminary Final PIR reports demonstrated a need to separate the seepage management function from the inflow pump station that provided the Final PIR design of a 1,360cfs inflow pump station and a separate 150cfs seepage pump station.

Current Analyses with Capacity Requirements

The non-Federal sponsor under the Acceler8 program had additional analyses performed that demonstrated a lesser need for 1,360cfs capacity with recommendation of a pump station with total capacity of 600cfs (Draft BODR). Upon review by the Jacksonville District, USACE supports Acceler8 recommendation with modifications. It has been decided that at this level of design, the inflow pump station will be reduced to 600cfs for inflow operation with an additional 50cfs pump for continuous control of Hillsboro Canal stages due to Reservoir seepage, as well as seepage collected in the Hillsboro Canal from LNWR and WCA-2A. The separate seepage pump station also remains in the plan with a capacity of 150cfs as identified in the preliminary Final PIR. The new SG-525 Inflow Pump Station data sheet is provided herein. Also, as stages on SG-525B Seepage Pump Station have been altered, this data sheet has been provided here as well.

Hillsboro Canal Improvement

No Improvement Required

Hillsboro Canal currently has an approximate conveyance capacity of ±800cfs in the vicinity of the reservoir as measured from S-39 800cfs design discharge capacity and from survey cross-sections. If the inflow pump station in the preliminary Final PIR plan was constructed as recommended with 1,360cfs inflow capacity, to implement its function fully the Hillsboro Canal would require improvement through excavation of additional cross sectional width. This requirement is noted to prevent excessive drawdown of the canal during pumping operations and canal bank erosion created through an increase in velocities above the stable bank threshold, i.e. >2.5fps. Indeed the Hillsboro Canal Improvement was included in the preliminary Final PIR. However, as the now proposed inflow pump station includes smaller 650cfs inflow capacity, no requirement to improve Hillsboro Canal is recommended.

Borrow Area

The Hillsboro Canal may still be improved, though not for conveyance purposes, but as a borrow area to gain suitable material for construction of the D-1 Embankment. For borrow purposes, the south canal bank along Lox Road will be left intact. The north canal bank may be excavated landward with some deepening of the canal north of the thalwag or centerline allowed.

A.A.3.6 SG-527A Fixed Weir

Background

Originally, the SG-527A Fixed Weir was designed to control stages within the D-1 Borrow canal with discharge into the Hillsboro Canal for return pump into the reservoir by the SG-525A Inflow Pump Station. The design included a notched or combination weir with two crest elevations that helped maintain a stage beneficial for the provided littoral shelf to gain flora and fauna habitat benefits, but provided ability to convey higher flows efficiently when needed into the Hillsboro Canal. Since flow over the weir would be fairly constant, a stilling basin was included in the design based on the California Institute of Technology (CIT) design concept. However, with the new seepage management plan that provides a separate SG-525B Seepage Pump Station with a separate controlled seepage canal stage that is 0.5 feet to 1.0 feet lower on average than Hillsboro Canal, the structure now serves a new function.

New Function

The SG-527A Fixed Weir now serves to provide ultimate control of seepage canal stages in times of power outages. With SG-525B Seepage Pump Station equipped with efficient electric pumps (efficiency is attained by low maintenance and with continuous running as opposed to event use with multi on-off cycles), it is necessary to provide either backup generators capable of powering the pumps or provide an alternate means of control. The weir structure provides the latter with combination use of the diesel motors found in the SG-525A Inflow Pump Station and normal flood control of the Hillsboro Canal basin, i.e. use of the Gated G-56 Spillway coastal structure.

New Design

The original design of the structure called for concrete wall with apron and stilling basin. The new structure consists of concrete capped steel sheet piles with walled sides to prevent side flow by-pass. Energy dissipation is accomplished with appropriate riprap sized for flows up to 2cfs/lf of weir.

A.A.3.7 SG-527B Gated Culverts

The only change for the SG-527B Gated Culverts is the method of construction. Originally, dewatering through well point system and gatewell base slab cast directly on soil/rock application was thought adequate. However, based on further investigation and review, the use of cofferdam is now included in recommended construction methodology and is included in new cost estimate.

A.A.4 Hydraulic Data Sheets

A.A.4.1 SG-525A Inflow Pump Station (Page 1 of 2)

Revisions:
- WPA Draft Feasibility Study – Original submission
- 27 May 2005 – Optimized pump mix "large" to "medium" and relocation of electric seepage pumps.
- 12 May 2006 – Final hydraulic data provided for Plan & Specifications effort.

Location: Hillsboro Canal, approximately 3,400 feet east of S-39 spillway
State Plane X, Y Coordinate[1] : 894020, 733125

Purpose/Operational Intent: Reservoir--Storage for Water Supply and WCA Seepage Control.
- Pump station pumps excess stormwater from the Hillsboro Canal when available into the Site 1 Impoundment. Sources include: LWDD's E-1W-S, E-1 and E-2, NSID, and LNWR via Hillsboro Canal.
- Seepage intercepted by Hillsboro Canal will be return-pumped to Impoundment as necessary to maintain Hillsboro Canal control stages (see Operating Manual)

Design Condition: Inflow Control 650 cfs

Pump Station Capacity Criteria
- The total pump capacity was determined primarily through Acceler8 Daily Water Budget analyses reviewed by USACE. Pump capacity captures 85% of historic flow rates through G-56 since year 1978.

Number of Pumps: 5
Pump Mix Type, Number and Size:
- Diesel 4 @ 150 cfs
- Electric 1 @ 50 cfs

Mix Criteria:
- The pump station will have 5 bays, 4 with matching pumps.
- The pump mix allows for intermediate flow values.
- The pump mix was redesigned to provide medium pump sizes that use typical suction bell intakes versus large pumps with Formed Suction Intakes (FSI).
- Small 50 cfs pump added for seepage and topping off reservoir when nearly full 30% to 40% of the time.

Control: Remote by SCADA or Local

SG-525A Inflow Pump Station (Page 2 of 2)

Design Heads

Normal (7.00 HW to 18.00 TW, ft-NGVD)	11.00 feet
Maximum (5.50 HW to 18.00 TW, ft-NGVD)	12.50 feet

Intake Water Surface Elevations

Maximum Non-Pumping	12.00 ft-NGVD
Maximum Pumping	12.00 ft-NGVD
Start Pumping	7.70 ft-NGVD
Normal Drawdown	5.5 to 7.5 ft-NGVD
Minimum Drawdown	5.00 ft-NGVD
Minimum Non-Pumping	4.50 ft-NGVD
Channel Invert (approx, design to determine need)	-3.00 ft-NGVD

Discharge Water Surface Elevations

Maximum Non-Pumping	22.50 ft-NGVD
Maximum Pumping	18.00 ft-NGVD
Normal Pumping	18.00 ft-NGVD
Minimum Pumping	6.00 ft-NGVD
Minimum Non-Pumping	6.00 ft-NGVD
Channel Invert (est., design to determine need)	4.00 ft-NGVD

Notes:

1) [1] XY coordinates system used is NAD 83, Florida east, state plane.
2) All elevations are in feet, NGVD (National Geodetic Vertical Datum of 1929)
3) Diesel generator is required for control station and electric pumps in cases of power outage.
4) Data Compiled from: S-39 TW records and WPA Alternative hydrograph evaluations.

A.A.4.2 SG-525B Seepage Pump Station (Page 1 of 2)

Revisions:
- 31 May 2005 – Original submission
- 12 May 2006 – Final hydraulic data provided for Plan & Specifications effort.

Location: Hillsboro Canal, approximately 3,400 feet east of S-39 spillway

Purpose/Operational Intent:
- Control C-508N Seepage Canal stage.

Design Condition: Seepage Control 150 cfs

Pump Station Capacity Criteria
- Pump capacity was primarily based on results from SEEP2D analysis and WPA FS MODFLOW modeling. The quantity of seepage intercepted and required for control was based on extended sensitivity analysis with an additional factor-of-safety of 1.5 applied to required pump station capacity.

Number of Pumps: 2
Pump Mix Type, Number and Size:
- Electric 2 @ 75 cfs

Mix Criteria:
- The pump station will have 2 bays with matching pumps.
- The pump mix uses pump sizes that are duplicated throughout the system.

Control: Remote by SCADA or Local

Design Heads

Normal (7.00 HW to 18.00 TW)	11.00	feet
Maximum (6.50 HW to 18.00 TW)	11.50	feet

SG-525B Seepage Pump Station (Page 2 of 2)

Intake Water Surface Elevations		
Maximum Non-Pumping	12.00	ft-NGVD
Maximum Pumping	12.00	ft-NGVD
Start Pumping	7.30	ft-NGVD
Normal Drawdown	6.80	ft-NGVD
Minimum Drawdown	6.30	ft-NGVD
Minimum Non-Pumping	4.50	ft-NGVD
Channel Invert	-10.00	ft-NGVD

Discharge Water Surface Elevations		
Maximum Non-Pumping	22.50	ft-NGVD
Maximum Pumping	18.00	ft-NGVD
Normal Pumping	18.00	ft-NGVD
Minimum Pumping	7.00	ft-NGVD
Minimum Non-Pumping	6.00	ft-NGVD
Channel Invert (est., design to determine need)	6.00	ft-NGVD

Notes:

1) [1] XY coordinates system used is NAD 83, Florida east, state plane.
2) All elevations are in feet, NGVD (National Geodetic Vertical Datum of 1929)
3) Diesel generator is required for control station and electric pumps in cases of power outage.
4) Data Compiled from: S-39 TW records and WPA Alternative hydrograph evaluations.

SG-527A Fixed Weir

Ungated Weir Structure

Revision	5 September 2001 - Original Submission
	25 June 2006 - Modified to straight capped sheetpile weir
XY Coord	901210 730810
Location	SE corner of Hillsboro Impoundment on C-525N seepage canal
Purpose	Ultimate control of seepage canal stage during power outage
Notes	1. Weir is a straight single crested weir
	2. Riprap requirements have not been verified with Geotech.

Design Conditions			
	Discharge (CFS)	200	cfs
	Headwater Elevation	8.75	ft,NGVD
	Tailwater Elevation	8.00	ft,NGVD
Maximum Expected Stages			
	Headwater Elevation	11.50	ft,NGVD
	Tailwater Elevation	11.50	ft,NGVD
Maximum Head Difference			
	Maximum Headwater Elevation	9.00	ft,NGVD
	Minimum Tailwater Elevation	4.50	ft,NGVD
Weir Data			
	Weir Crest Elevation	8.00	ft,NGVD
	Weir Type	Broad Crested	
	Weir Breadth	2.00	feet
	Overall Crest Length	100.0	feet
	Minimum Tieback Wall Elevation	11.25	ft, NGVD
	Weir Control	Non-gated	
Canal Data			
	Side Slopes Cotangent	3	(varies)
	Upstream Bottom Width	40.00	feet
	Upstream Bottom Elevation	-10.00	ft,NGVD
	Downstream Bottom Width	40.00	feet
	Downstream Bottom Elevation	-9.00	ft,NGVD
Riprap Requirements			
	Riprap Protected Area	1,000	sq-ft
	Riprap Thickness	2.0	feet
	Riprap Bedding Thickness	1.0	feet
Control Protection Elevation		11.75	ft,NGVD

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