APPENDIX D:
H&H

SPREADSHEET MODEL DOCUMENTATION

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1. Problem Identification

As part of the Limited Reevaluation Report (LRR) for the Tamiami Trail Modifications (TTM) of the Modified Waters Deliveries (MWD) to Everglades National Park (ENP) project it became necessary to incrementally analyze different control stages within the L-29 Borrow Canal (L-29BC). This analysis will allow benefits to be calculated as a function of stage increase and opening size. To incrementally look at the benefits that different stage constraints on Tamiami Trail would produce a simple spreadsheet model was developed that looked at volumetric change based on inflow.

2. Existing Structures and Gage Locations

Within the boundaries of this project area, five US Army Corps of Engineers (USACE) structures (S-333, S-355A, S-355B, S-334, and S-356) and 19 sets of culverts that pass water from the Levee 29 Borrow Canal (L-29BC, also referred to as L-29 Canal) south through Tamiami Trail (US 41) into North East Shark River Slough (NESRS) exist. A brief description of these features follow:

A. S-333 is a reinforced concrete, gated spillway with discharge controlled by one cable operated, vertical lift gate. The gate is operated to make releases from Water Conservation Area 3A (WCA-3A) into the Tamiami Canal (L-29BC). This structure has a maximum discharge rate of 1,350 cfs.

B. S-355A and S-355B are reinforced concrete, gated spillways with discharge controlled by one cable operated, vertical lift gate. Each structure is capable of a maximum discharge of 1000 cfs. These structures are a part of the Modified Water Deliveries to Everglades National Park (MWD) project and are designed to pass water from Water Conservation Area 3B (WCA-3B) into NESRS. This transfer of water is via the L-29BC and the combination of culverts and a new bridge being proposed by this project along Tamiami Trail. The S-355A and S-355B structures are not currently operated due to stage constraints in the L-29BC.

C. S-334 is a reinforced concrete, gated spillway with discharge controlled by one cable operated, vertical lift gate. Operation of the gate is manually controlled, and the gate is operated to make releases from the L-29BC into the L-31N canal (South Dade conveyance system). This structure has a maximum discharge rate of 1230 cfs.

D. As part of the 2002 IOP Emergency Contract the interim pump station S-356 was constructed. S-356 is a 500 cfs (4 pumps at 125 cfs each) diesel-driven pump station that pumps water from the L-31N canal into the L-29BC for the purpose of protecting the Cape Sable seaside sparrow and for returning increased seepage water from NESRS into L-31N due to the implementation of the MWD Project.

E. The 19 sets of culverts are made up of a total of 55 barrels with diameters ranging in size from 48 to 60 inches. The total discharge capacity is based on upstream and downstream stages across the Tamiami Trail.

F. ***Table 1*** lists the gages/structures used for the analysis and ***Figure 1*** shows the location of these features.

Table 1: General Structure and Gage Information

Gage/Structure	Data Type	Frequency	Statistics Type	Period of Record Start	Period of Record End	Agency
NESRS-1	Stage (ft, NGVD)	Daily	Mean	23-Jul-76	Present	USGS
NESRS-2	Stage (ft, NGVD)	Daily	Mean	26-Jul-76	Present	USGS
NESRS-3	Stage (ft, NGVD)	Daily	Mean	2-Aug-84	Present	USGS
NESRS-4	Stage (ft, NGVD)	Daily	Mean	24-Jul-85	Present	USGS
NESRS-5	Stage (ft, NGVD)	Daily	Mean	24-Jul-85	Present	USGS
Angels	Stage (ft, NGVD)	Daily	Mean	9-Apr-84	Present	SFWMD
G-3272	Well (ft, NGVD)	Daily	Mean	10-Jun-83	Present	SFWMD
G-3273	Well (ft, NGVD)	Daily	Mean	14-Mar-84	Present	SFWMD
NP-206	Stage (ft, NGVD)	Daily	Mean	1-Oct-74	Present	ENP
RG-1	Stage (ft, NGVD)	Daily	Mean	13-Jan-98	Present	ENP
R3110	Stage (ft, NGVD)	Daily	Mean	11-Oct-84	Present	ENP
S-333						
Discharge	Flow (cfs)	Daily	Mean	12-Oct-78	Present	SFMWD
Headwater	Stage (ft, NGVD)	Daily	Mean	12-Oct-78	Present	SFMWD
Tailwater	Stage (ft, NGVD)	Daily	Mean	12-Oct-78	Present	SFMWD
S-12A						
Discharge	Flow (cfs)	Daily	Mean	1-Oct-63	Present	USGS
Headwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
Tailwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
S-12B						
Discharge	Flow (cfs)	Daily	Mean	1-Oct-63	Present	USGS
Headwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
Tailwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
S-12C						
Discharge	Flow (cfs)	Daily	Mean	1-Oct-63	Present	USGS
Headwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
Tailwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
S-12D						
Discharge	Flow (cfs)	Daily	Mean	1-Oct-63	Present	USGS
Headwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
Tailwater	Stage (ft, NGVD)	Daily	Mean	1-Oct-63	Present	USGS
Rainfall	Rainfall (in)	Daily	Mean	2-Oct-63	Present	USGS
40 Mile Bend Pan Evaporation	Rain (in)	Dialy	Mean	6-Jan-40	Present	NOAA

ENP – Everglades National Park
NOAA – National Oceanographic and Atmospheric Administration
SFWMD – South Florida Water Management District

USGS – US Geographical Survey

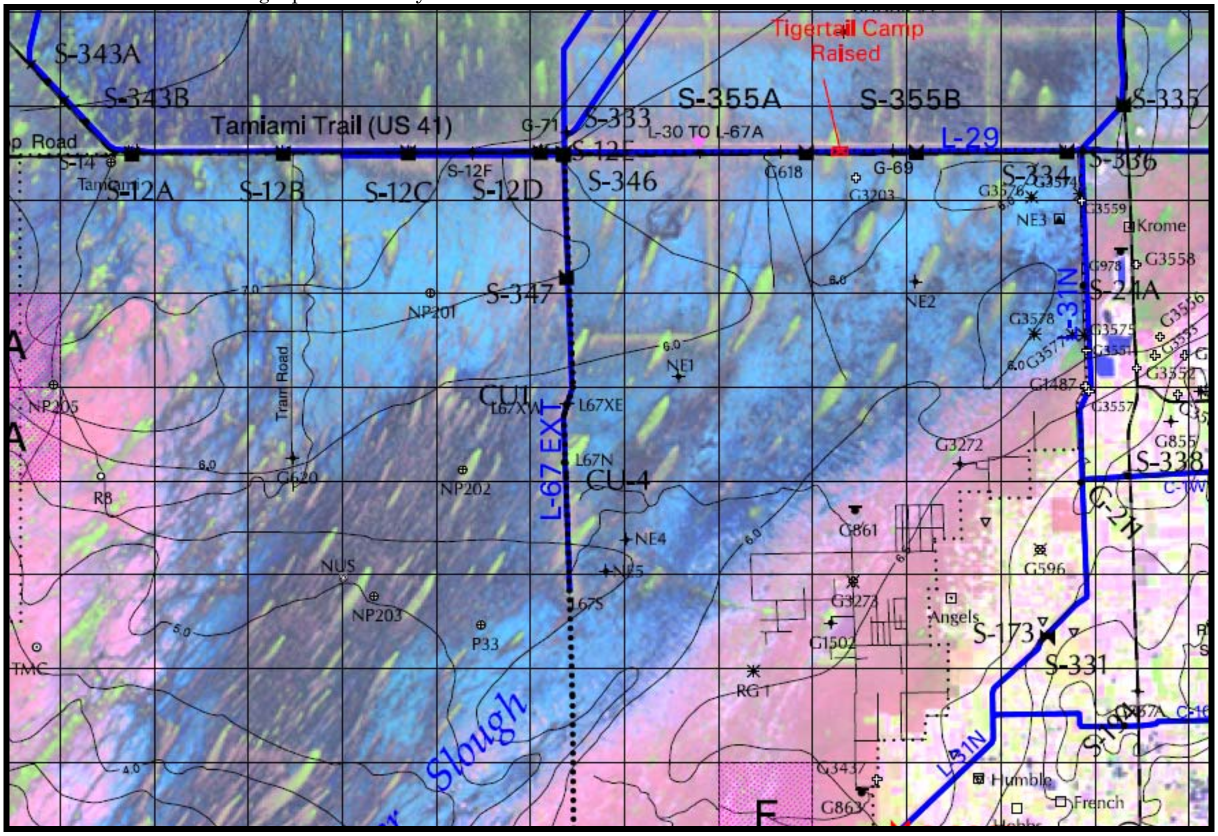

Figure 1: Structure and Gage Location Map

3. Current Operations

The discharges into the L-29BC (limited currently to S-333) are limited by stages that would cause impact to the current roadway (elevation 7.5 ft, NGVD). This elevation is based on communications with the Florida Department of Transportation (FDOT). Discharges are additionally constrained based on stages at G-3273 (elevation 6.8 ft) for the protection of the 8.5 Square Mile Area. L-29BC is used for two separate purposes:

A. Water Supply Releases: S-333 can be used in conjunction with S-334 to make water supply releases to south and east Dade County (South Dade Conveyance System). The total delivery will be the amount necessary to maintain the appropriate stages at S-331, S-25B and S-22.

B. Regulatory releases from WCA-3A to ENP are made from S-333 and the S-12's. The structures will be operated in accordance with the Interim Operation Plan for the Protection of the Cape Sable Seaside Sparrow (IOP, 2002 and later 2006). When water levels at G-3273 (a stage recorder located to the west and north of the 8.5 Square Mile Area) have been above 6.8 ft, NGVD for 24 hours, S-333 will be closed.

4. Required and Desired Water Volumes

The flow requirement of 4,000 cfs has generated considerable confusion. The Everglades National Park Protection and Expansion Act (PL 101-229) Sec 104(a)(1) did not authorize a specific flow rate but states:

> "Upon completion of a final report by the Chief of the Army Corps of Engineers, the Secretary of the Army, in consultation with the Secretary, is authorized and directed to construct modifications to the Central and Southern Florida Project to improve water deliveries into the park and shall, to the extent practicable, take steps to restore the natural hydrological conditions within the park."

The final report Part 1 Supplement 54 General Design Memorandum and Environmental Impact Statement Modified Water Deliveries to Everglades National Park, Florida June 1992, Section H. Recommended Project (page 52) defines the measures that the natural hydrologic conditions would be measured as:

> "The goal of restoring natural hydrologic conditions will be met in terms of all three of its dimensions: location, timing and volume:
>
> * Location–The historic path of Shark River Slough will be restored by bringing WCA-3B and NESRS back into the flow-way between WCA-3A and Everglades National Park
>
> * Timing–Water flows through the restored Shark River Slough will reflect natural local meteorological conditions, including the extremes of natural droughts and floods, and variations in the annual seasonal and long-term cycles.
>
> * Volume–The volume of water delivered will reflect the naturally available supplies based on local meteorological conditions, except in cases where operations of the C&SF project for other authorized project purposes necessitate increased or decreased deliveries. Natural hydroperiods will be restored."

The MWD is not authorized a specific flow but rather a volume to the extent practicable that will reflect the naturally available supplies based on local meteorological conditions. In the past confusion has revolved around the volume and timing of flows with a specific flow rate. The final report Part 1 Supplement 54 General Design Memorandum and Environmental Impact Statement Modified Water Deliveries to Everglades National Park, Florida June 1992, Section I. Environmental Analysis (page 58) states:

> "Hydrologic restoration of WCA-3B is also essential to restoring natural water conditions in the Park. Diversion of flood waters from WCA-3A into detention in WCA-3B would decrease the volume of and, in some cases, the need for regulatory water releases in to the Park from WCA-3B. This would reduce the frequency of unnatural distributions of water across SRS, and further reduce the occurrences of alligator nest flooding south of the S-12s. The ability to discharge an additional 2,000 cfs of water in to NESRS through the new S-355 structures and 1,300 cfs through S-333, would allow full restoration of historic water depths in the center of the slough, thereby causing reflooding of the short-hydroperiod marshes on the eastern slope of the slough. This would accrue a11 the wildlife benefits from increased primary and secondary productivity previously discussed. In addition, aquifer recharge, reestablishment of groundwater flows, surface water reconnection between SRS and Taylor slough, and restoration of estuarine productivity would be maximized."

The 4,000 cfs flow rate is based on the total capacity of the recommended structures of the 1992 MWD to ENP Project GDM to deliver water (Volume) into the L-29BC between structures S-333 and S-334 and then hydraulically conveyed through the Tamiami Trail (US41) embankment to ENP. This total capacity (4,000 cfs) is based combining the design discharge capacity of the following structures: S-333 (1,350 cfs), S-355A (1,000 cfs), S-355B (1,000 cfs), and S-356 (950 cfs). The 4,000 cfs represents an infrequent high flow event that is desirable for the system to be able to pass for geomorphologic changes.

5. Conceptual Model Layout

The spreadsheet model was developed to take into consideration two components: 1) the change in storage in the marsh that different stage constraints within L-29BC could produce based upon delivering water into NESRS and 2) the interaction with the downstream marsh and the L-29BC stage. The model is based on computing a stage at NESRS-2 based on mass balance and then using an equation to relate that stage to the L-29BC stage (***Figure 2***).

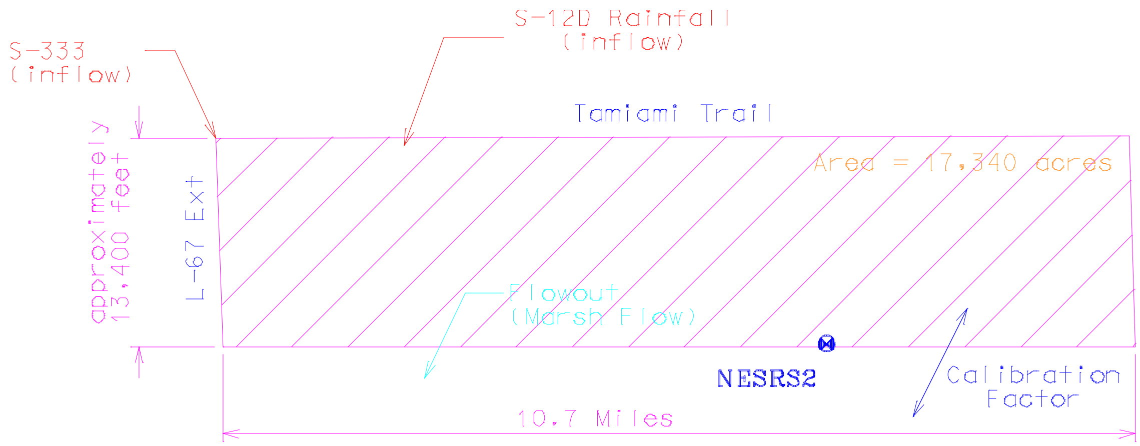

Figure 2: General Layout of Spreadsheet Model

$$\Delta S = Inflow - Outflows \pm CF$$ **Equation 1**

where

ΔS = change in stage at NESRS (ft)
Inflow = S-333 discharges + Rainfall applied to an area (ft)
Outflows = Evaporation + Flow Out (Marsh Flow) applied to an area (ft)
CF = Calibration Factor that takes into consideration unknown factors such as seepage in and out, variability in rainfall, and flow south (ft)

6. Calibration

The model was calibrated to the historic period of record (POR) from January 1, 1983 through August 15, 2007. The following historical data were used for this time period: S-333_TW, S-333_Q, NESRS2, S-12A_Q, S-12B_Q, S-12C_Q, S-12D_Q, S-12D_Rainfall, and 40 Mile Bend Evaporation. The input parameters are as follows:

A) Inflows

1) S-333

The volume of water discharged at S-333 was assumed to enter NESRS and was converted to a stage increase by the following equation:

$$I_{S\text{-}333} = (1.98*Q)/A \text{ (ft)}$$ **Equation 2**

where

$I_{S\text{-}333}$ = stage increase associated with inflow volume discharged at S-333 (ft)
1.98 = constant used to convert cubic feet per second (cfs) to acre-feet per day
Q = actual average daily discharge at S-333 (cfs)

A = area (acres), distance from Tamiami Trail to the NESRS-2 gage (13,400 feet) multiplied by the distance along Tamiami Trail (10.7 miles); equal to approximately 17,340 acres.

2) Rainfall

Rainfall was taken from the S-12D gage recorded in inches per day and was converted to feet per day.

B) Outflows

1) Evaporation

Evaporation was taken from the 40 Mile Bend Pan Evaporation gage located approximately ten miles west of the project area. This gage is recorded in inches per day and was converted to feet per day.

2) Flow Out

Flow out was computed based on a linear approximation of velocity versus stage. Velocity values were assumed as:

Stage (ft)	Velocity (ft/s)
5.5	0.001
12.0	0.015

$$Q_{flow} = \left[\frac{(0.015 - 0.01)}{(12 - 5.5)} * (NESRS2 - 5.5) + 0.001 \right] * (d * L) \qquad \textbf{Equation 3}$$

where

Q_{flow} = average daily volumetric flowrate or discharge (cfs)
NESRS2 = the stage at the NESRS-2 monitoring gage (ft)
d = the depth at NESRS2 assumed to be stage minus 5.5 feet
L = the length along Tamiami Trail (56,496 ft)

This calculation produces a range of discharges to the south out of the conceptual model from 0 and 5,500 cfs. These values were then converted to decreases in stage at NESRS-2 by the following equation:

$$O_{flow} = \frac{1.98 * Q_{flow}}{A} \text{ (ft)} \qquad \textbf{Equation 4}$$

where

O_{flow} = stage decrease related to discharge released (ft)
Q_{flow} = volumetric flowrate or discharge released (cfs)
A = area (acres)
1.98 = constant used to convert cubic feet per second (cfs) to acre-feet per day

An if' statement was used to prevent the flows from being computed below a stage of 5.5 feet (or simply put when stages are lower than 5.5 ft, then Q = 0 cfs). Early in the spreadsheet model development the values for velocity were experimented with at different ranges. However based on the nature of the model and the length of the area (56,496 ft), small variations in the velocity term created huge losses of flows or simply put created an imbalance of inflows and outflows. The final decision was based on a range that produced the smallest term in calibration factor (discussed below).

C) Calibration Factor

The CF was added in order to compensate for other unknowns in the system such as seepage in and out of the area, variability in rainfall, missing or incorrect evaporation data and flow south. The term was computed based on calculating the measured stage difference (Equation 1) at NESRS-2 and solving for the calibration factor.

$$\Delta S = I_{S\text{-}333} + I_{rain} - ET - O_{FLOW} - CF$$

where

ΔS = change in stage at NESRS (ft)

$= NESRS2_n - NESRS2_{n-1}$

Solve for CF

$$CF = I_{S\text{-}333} + I_{rain} - ET - O_{FLOW} - (NESRS2_n - NESRS2_{n-1})$$

where

CF = calibration facto

$I_{S\text{-}333}$ = stage increase associated with inflow volume discharged at S-333 (ft)

I_{rain} = stage increase from rainfall (ft)

ET = evapotranspiration, loss of water from the soil both by evaporation and by transpiration from the plants growing there (ft)

O_{FLOW} = stage decrease related to discharge released (ft)

$NESRS2_n$ = historical stage at the current time step (ft)

$NESRS2_{n-1}$ = historical stage the day before current time step (ft)

The CF was not a constant per stage (***Figure 3***) and attempts to fit a curve through the value resulted in poor matches due to the high variability. So the CF was applied for each time step then the calibration looked at the fit to the L-29BC Stage.

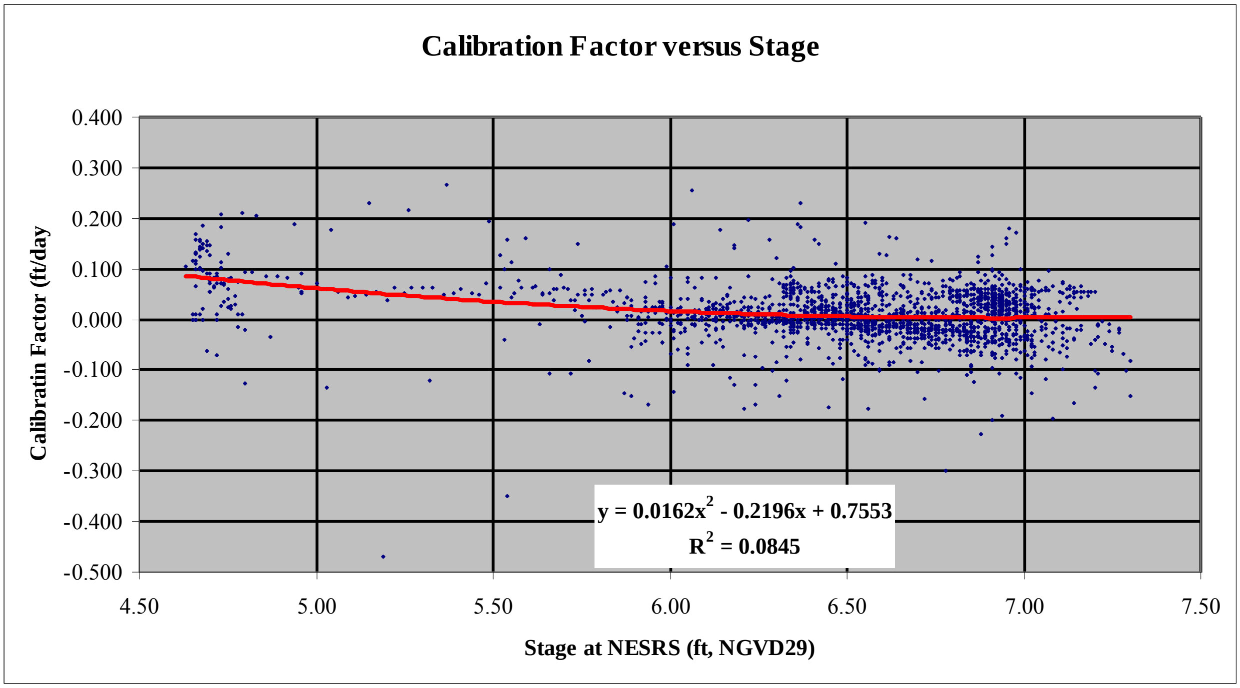

Figure 3: Calibration Factor versus Stage

D) Calibration to the L-29BC Stage.

Two approaches were investigated for developing an equation to correlate a canal stage from the NESRS-2 gage.

1) Curve Fitting Historical Data

The stage difference was computed between the S-333 tailwater recorder and the NESRS-2 gage and then plotted in regards to the discharge at S-333 (***Figure 4***).

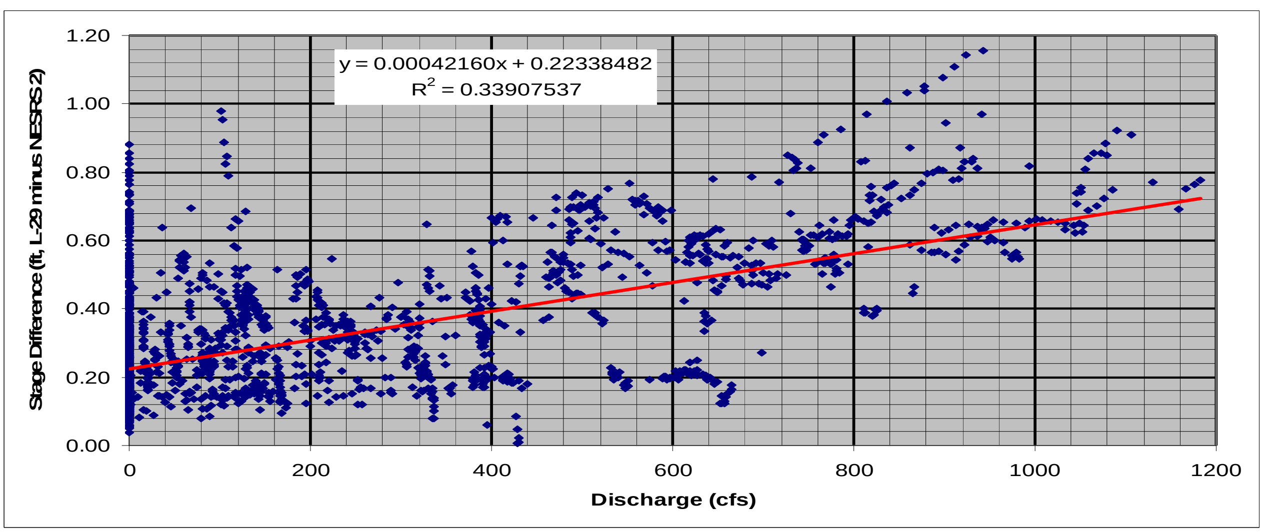

Figure 4: Historical Stage Difference Compared to Discharge

2) RMA-2 Results from the 2005 Revised General Reevaluation Report.

The results from the RMA-2 model from the 2005 Revised General Reevaluation Report (RGRR) for Tamiami Trail were used to compute a head differential or stage differential (ΔH) term which was then used to derive a canal stage based on the stage at NESRS-2 (***Figure 5***).

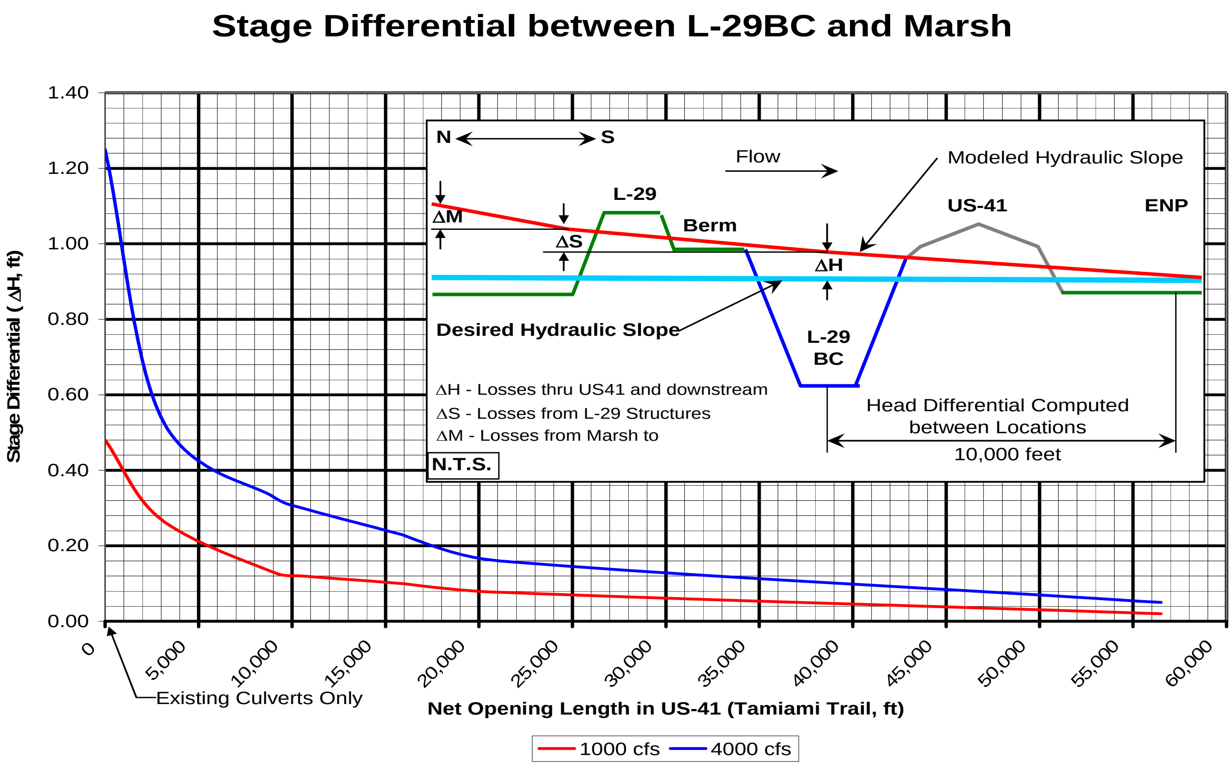

Figure 5: Computed RMA-2 Stage Differential between Marsh and L-29 Borrow Canal

3) Calibration Results

Both methods produce reasonable results in matching the general trends of canal stages (***Figure 6***). The RMA-2 calibration run was only off on average by minus 0.123 ft (***Figure 7***) when compared to the historically delivered flows. For alternative comparison analysis though it was decided to remain with the RMA-2 calculations so that the ΔH term could be easily manipulated per alternative. In addition the RMA-2 modeling looked at higher flowrates up to the target of 4,000 cfs, where historical data did not get over 1400 cfs.

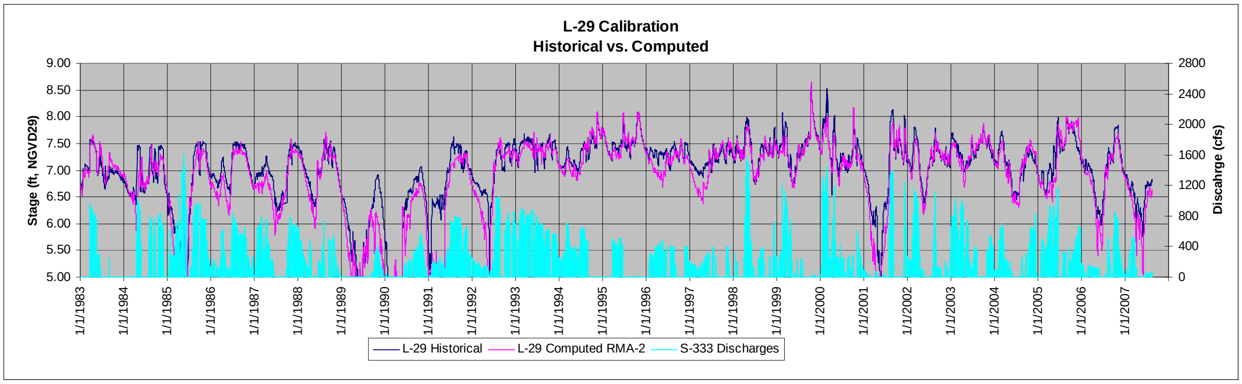

Figure 6: Computed Stage verus Historical L-29 Borrow Canal

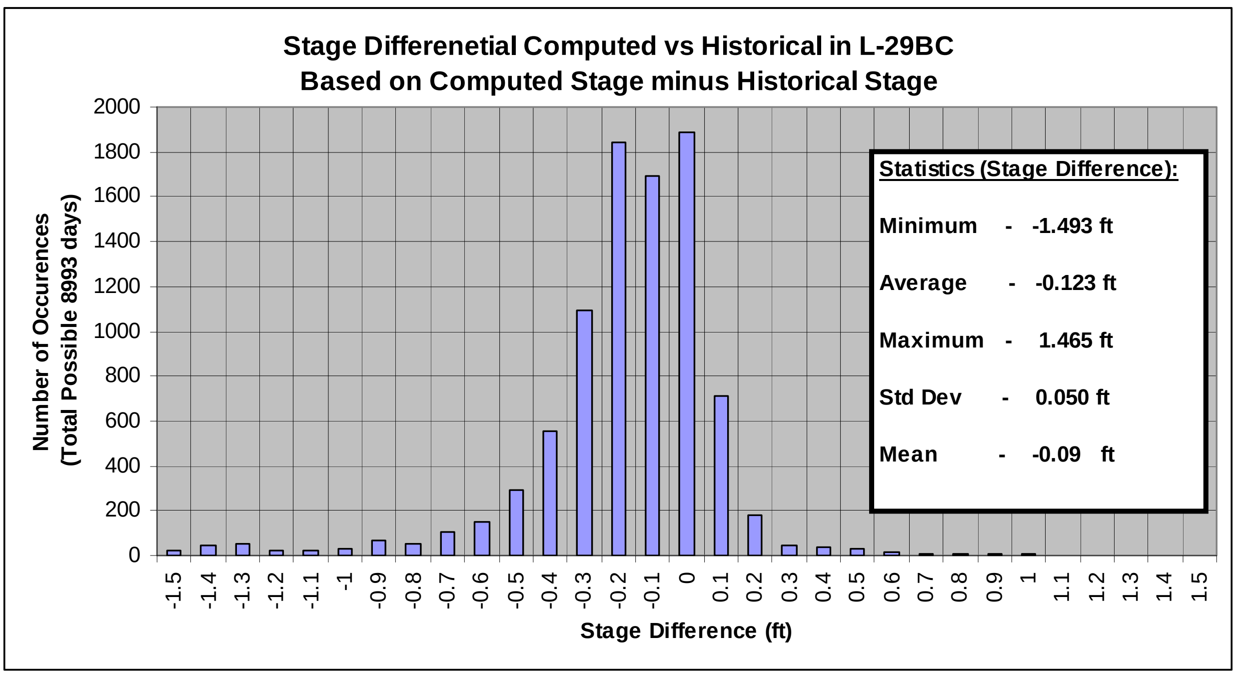

Figure 7: Histogram of Computed versus Historical Stage Differentials in L-29 Borrow Canal

7. Alternative Modeling Strategy

To model alternatives and the effect of different stage constraints the following assumptions were made for all alternatives:

A) Calibration Factor (CF)

The CF remained constant. The goal of the model was to determine the increased flow volumes under different L-29BC stage constraints and opening configurations.

B) Inflows at S-333

Inflows at S-333 were computed based on summing the historical flows delivered to ENP (S-12A, S-12B, S-12C, S-12D, and S-333 minus S-334) and multiplying by 55 percent (***Figure 8***). The 55 percent value was the target flow distribution for the MWD to ENP project. This method was chosen to avoid an operational model that would take more time to develop and that would simply use the effect of meeting the target distribution of 55 percent of the flows to the east. In short if the capability existed for distributing the flows 45 percent to the west and 55 percent to the east then this volume in correlation with different stage constraints on the L-29BC would produce these results for the different alternatives.

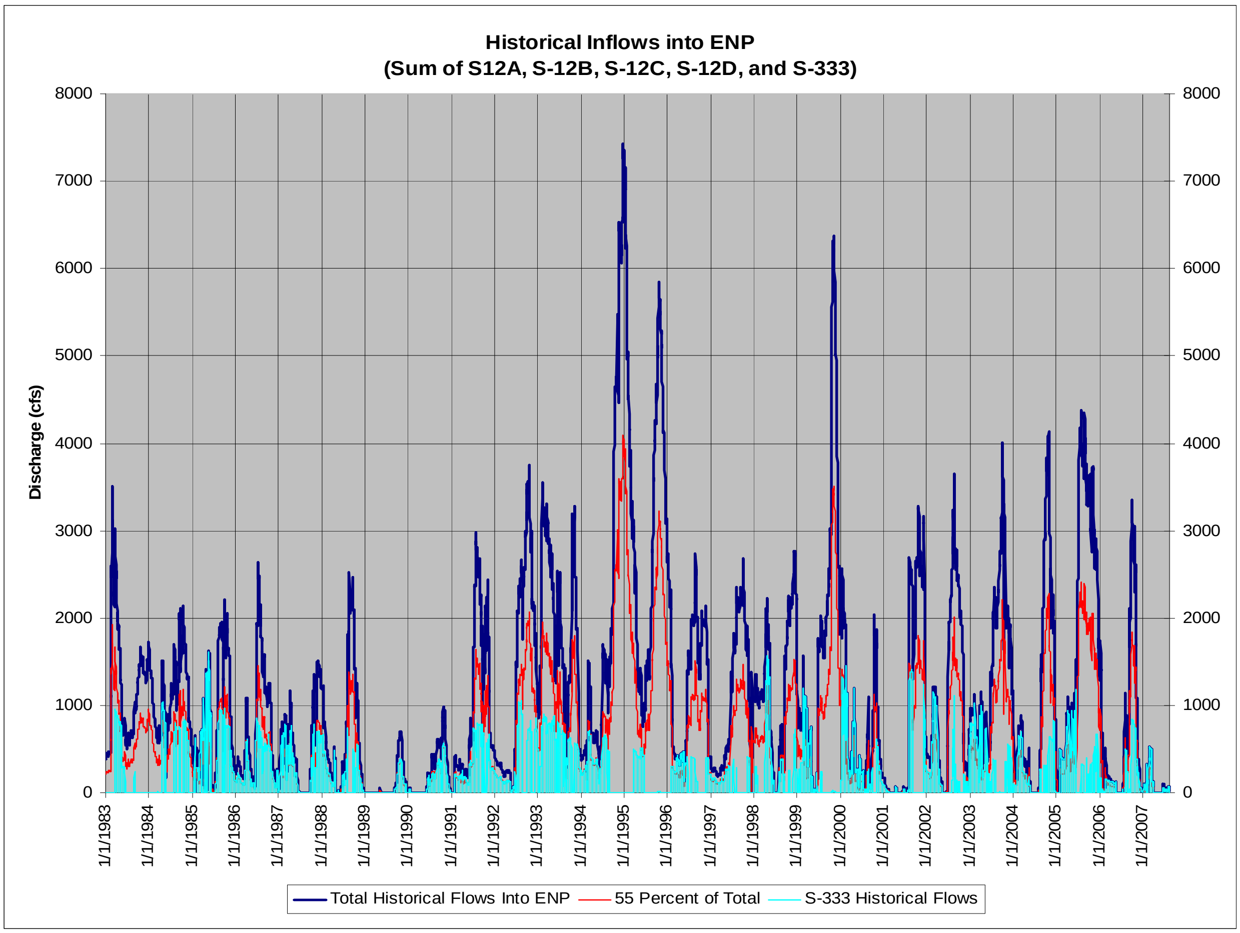

Figure 8: Historical Flows Delivered to ENP

To gain a perspective of the historical water availability of the system a daily flow duration curve (***Figure 9***) was developed from the period of record from 1 January 1983 through 15 August 2007 (approximately 25 years of data or 8,993 days). This curve counts the number of days that discharges actually exceeded a certain value. From the curve only approximately 3.25 percent of the days (292 days out of 8,993 days) actually saw a total delivery [S-12A + S-12B + S-12C +S-12D + (S-333 – S-334)] greater than 4,000 cfs to ENP. When 55 percent of the total is computed then only 8 days out of 8,993 days actually have a possibility of discharging 4,000 cfs to the east based on historical discharges.

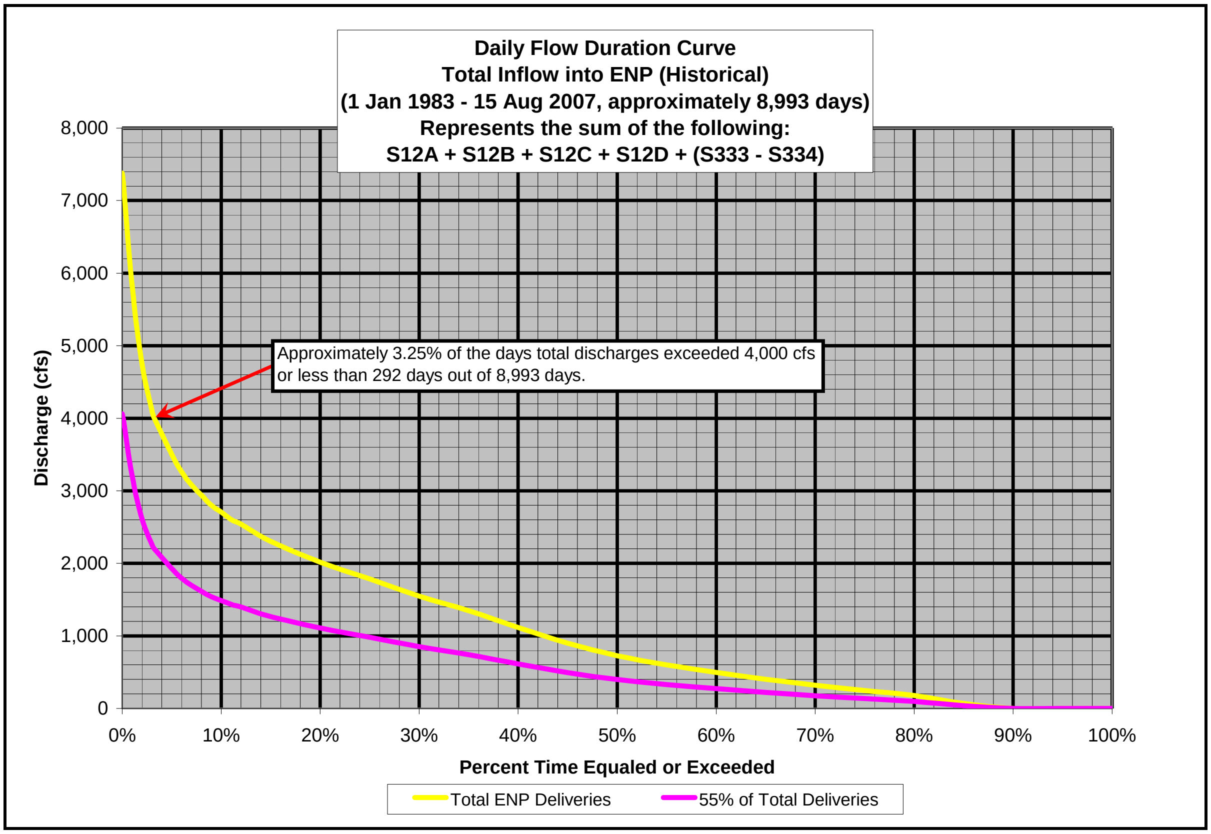

Figure 9: Historical Daily Flow Duration Curve of Total Inflows into ENP

C) Rainfall

Remained constant.

D) Evaporation

Remained Constant.

E) Marsh Outflow

Same calculation as the calibration model.

F) Marsh Headloss Factor

One of two factors that changed per alternative. This factor was based on the RGRR analysis from 2005 where multiple alternatives were analyzed under different flow regimes. From this analysis two curves were developed based on head differential between the marsh and the L-29BC and flow (***Figure 5***). It was assumed that a linear approximation could be used between the curves. The head differential between the marsh and the L-29BC for existing conditions was 0.22ft when the discharge was 0 cfs, and ΔH = 1.2 ft when the discharge was 4,000 cfs. Then if the flowrate was equal to 800 cfs, the ΔH term would be equal to:

$$\Delta H = \left[\left(\frac{Q_{S-333} - 0cfs}{4{,}000cfs - 0cfs}\right) * \left(1.2\,ft - 0.22\,ft\right)\right] + 0.22\,ft \qquad \textbf{Equation 5}$$

$$\Delta H_{800cfs} = 0.416\,ft$$

where

ΔH = marsh headloss factor (ft)
Q = volumetric discharge (cfs)

G) Stage Constraint on the L-29 Borrow Canal

The stage constraint controlled whether or not flows could be discharged into the model. If the stage in the L-29BC was higher than the constraint then flows went to zero. It should be noted that for the lower stage constraints this produced daily flows that might produce high discharges. However, these high discharges would then create a stage that would turn off the flows for several days. From a real time operational perspective, weekly adjustments are made to the structures to target a specific flow. If the spreadsheet model ran weekly average flows then one would get a better perspective of how water would be discharged into NESRS. This happens because the spreadsheet model simply looks at distributing 55 percent of the total flows into the L-29BC, not small increments of the percentage. The goal was to keep the model simplistic so that it would run quickly.

H) Relocation of L-67 Extension to Blue Shanty Canal

This alternative followed a similar analysis as the other alternatives but divided the area within NESRS into two separate areas: 1) the area east of the Blue Shanty Canal which used the NESRS-2 and 2) the area west of the Blue Shanty Canal used NESRS-1 (***Figure 10***). In this alternative flows were initially distributed proportionally east and west of the newly relocated L-67 Extension Levee (72% and 28%, respectively). Once the eastern side (NESRS-2) violated the stage constraint then all flows were delivered west of Blue Shanty Canal. This rule allowed the plan to deliver the full potential of flows into NESRS.

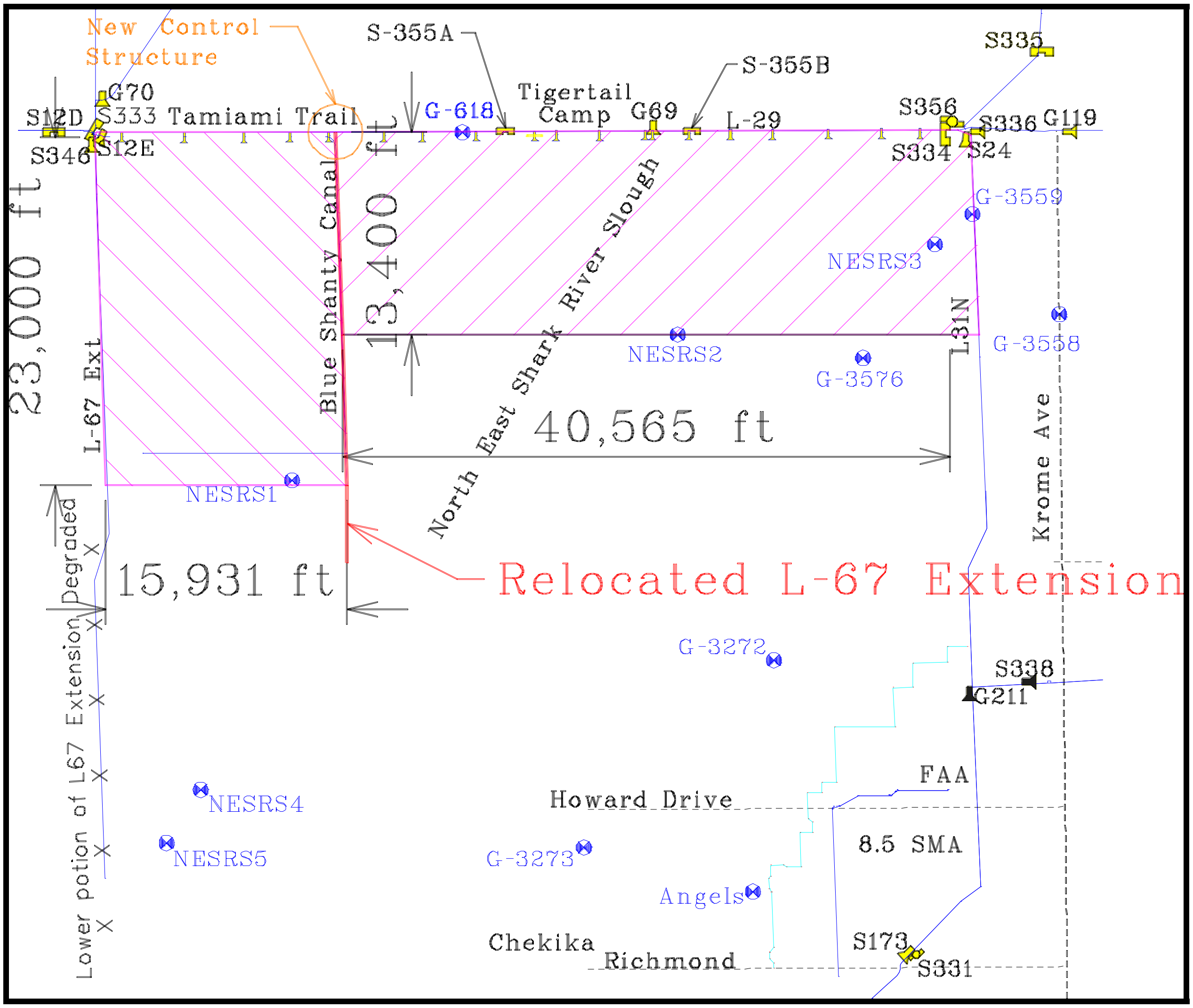

Figure 10: General Representation of L-67 Ext Plan

8. Alternatives:

A) Alternative 1–No Roadway Improvements

All alternatives in this category had an L-29BC stage constraint of 7.5 feet.

1) Alternative 1.1-No Action

This alternative represents the existing conditions of the system, 19 sets of existing culverts. This alternative was used as the basis for which all other alternatives were compared. The marsh headloss factor used for this alternative was ΔH=0.22 ft for 0 cfs and ΔH=1.2 ft for 4,000 cfs (***Table 2***).

2) Alternative 1.2–Existing Culverts with Spreader Swales

To increase the efficiency of the culverts downstream spreader swales were constructed, assumed to have a 30 foot bottom width and 1000 foot length centered on the culvert. In terms of efficiency it was assumed that the spreader swales would increase the efficiency by 10 percent on the lower end of discharges and 12 percent on the higher end of discharges. This factor was applied by changing the head differential between the marsh and the L-29BC of (ΔH=0.2 ft for 0 cfs and ΔH=1.06 ft for 4,000 cfs). Best professional judgment was used for the selection of the reduction value.

3) Alternative 1.3–Existing Culverts with 19 Additional Culverts and Swales

No improvements to the road were made but 19 sets of culverts were added to the roadway. Each new set of culverts would contain three pipes. Each pipe would be five-foot diameter reinforced concrete. All culverts existing and new would have a downstream spreader swale constructed similar to Alternative 1.2. The head differential between the marsh and the L-29BC for this alternative was based on ***Figure 5*** with a net opening of approximately 600 feet and using the net reduction of 10 and 12 percent reduction as done in Alternative 1.2; ΔH=0.2 ft for 0 cfs and ΔH=0.94 ft for 4,000 cfs.

4) Alternative 1.4a-1 Mile Eastern Bridge

No improvements to the road but a one mile bridge would be constructed on the eastern end of the project area where the 2005 RGGR proposed to construct the one mile bridge. The head differential between the marsh and the L-29BC for this alternative was; ΔH=0.2 ft for 0 cfs and ΔH=0.42 ft for 4,000 cfs.

5) Alternative 1.4b-1 Mile Western Bridge

No improvements to the road but a one-mile bridge would be constructed on the western end of the project area within the area 2005 RGGR proposed to construct the 2-mile bridge. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.2 ft for 0 cfs and ΔH=0.42 ft for 4,000 cfs.

6) Alternative 1.5-Raise Western Section of Road and 1-Mile Western Bridge

Same as Alternative 1.4b except road in this vicinity was raised to 13.00 ft crown. A one-mile bridge would be constructed on the western end of the project area within the area 2005 RGGR proposed to construct the 2-mile bridge. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.2 ft for 0 cfs and ΔH=0.42 ft for 4,000 cfs.

B) Alternative 2–Roadway Improvements to Raise Road Crown to 11.05 ft

All alternatives in this category had an L-29BC stage constraint of 8.0 ft.

1) Alternative 2.1–Raise only Low Points of Existing Roadway

Same as Alternative 1.1 except the stage constraint was changed to 8.0 feet for the L-29BC.

2) Alternative 2.2.1–Raise Low Points and Add 19 Additional Culverts and Swales

Same as Alternative 1.3 except the stage constraint was changed to 8.0 feet for the L-29BC.

3) Alternative 2.2.2a–Raise Low Points and Add 1 Mile Eastern Bridge

Same as Alternative 1.4a except the stage constraint was changed to 8.0 feet for the L-29BC.

4) Alternative 2.2.2b-Raise Low Points and Add 1 Mile Western Bridge

Same as Alternative 1.4b except the stage constraint was changed to 8.0 feet for the L-29BC.

5) Alternative 2.2.3-Raise Low Points and Add 2 Mile West and 1 Mile East Bridge

This alternative was the 2005 RGRR plan with a lowered stage constraint in the L-29BC (8.0 feet). The head differential between the marsh and the L-29BC for this alternative was ΔH=0.06 ft for 0 cfs and ΔH=0.3 ft for 4,000 cfs.

C) Alternative 3–Roadway Improvements to Raise Road Crown to 11.55 ft

All alternatives in this category had an L-29BC stage constraint of 8.5 ft.

1) Alternative 3.1–Raise only Low Points of Existing Roadway

Same as Alternative 1.1 except the stage constraint was changed to 8.5 feet for the L-29BC.

2) Alternative 3.2.1–Raise Low Points and Add 19 Additional Culverts and Swales

Same as Alternative 1.3 except the stage constraint was changed to 8.5 feet for the L-29 Borrow Canal.

3) Alternative 3.2.2a–Raise Low Points and add 1 Mile Eastern Bridge

Same as Alternative 1.4a except the stage constraint was changed to 8.5 feet for the L-29 Borrow Canal.

4) Alternative 3.2.2b-Raise Low Points and Add 1 Mile Western Bridge

Same as Alternative 1.4b except the stage constraint was changed to 8.5 feet for the L-29BC.

5) Alternative 3.2.3-Raise Low Points and add 2 Mile West and 1 Mile East Bridge

Same as Alternative 2.2.3 except the stage constraint was changed to 8.5 feet for the L-29BC.

D) Alternative 4–Roadway Improvements to Raise Road Crown to 12.75 ft

All alternatives in this category had an L-29BC stage constraint of 9.7 feet.

1) Alternative 4.1–Raise only Low Points of Existing Roadway

Same as Alternative 1.1 except the stage constraint was changed to 9.7 feet for the L-29BC.

2) Alternative 4.2.1–Raise Low Points and Add 19 Additional Culverts and Swales

Same as Alternative 1.3 except the stage constraint was changed to 9.7 feet for the L-29BC.

3) Alternative 4.2.2a–Raise Low Points and Add 1 Mile Eastern Bridge

Same as Alternative 1.4a except the stage constraint was changed to 9.7 feet for the L-29BC.

4) Alternative 4.2.2b-Raise Low Points and Add 1 Mile Western Bridge

Same as Alternative 1.4b except the stage constraint was changed to 9.7 feet for the L-29BC.

5) Alternative 4.2.3-Raise Low Points and Add 2 Mile West and 1 Mile East Bridge

This alternative was the 2005 RGRR plan and the same as Alternative 2.2.3 except the stage constraint was changed to 9.7 feet for the L-29BC.

6) Alternative 4.2.4–Construct a 10.7-Mile Bridge (2005 RGRR)

Removed the existing Tamiami Trail (US Highway 41) throughout the project area and replaces it with a 10.7 Mile Causeway. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.01 ft for 0 cfs and ΔH=0.05 ft for 4,000 cfs.

E) Alternative 5–Structural Alternatives and/or Road Realignment

All alternatives in this category had an L-29BC stage constraint of 9.7 feet.

1) Alternative 5.1–Northern Alignment of Alternative 14 from 2005 RGRR

This alternative located the two-mile an one-mile bridge alternative to the north of the current location of the existing Tamiami Trail placing the roadway and bridges entirely onto the L-29 levee. The L-29 levee would be removed and three bridges would be constructed as part of the access curves to transition too and from the levee back onto Tamiami Trail. The top elevation of the road would be 12.75 feet. The bottom cord elevation of the bridges would be 14.75 feet. Water quality treatment of stormwater runoff was required. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.06 ft for 0 cfs and ΔH=0.3 ft for 4,000 cfs.

2) Alternative 5.2–Northern Alignment with 1-Mile Bridge

This alternative was similar to alternative 5.1 except there was less bridging. A one mile bridge would be constructed on the west side of Tamiami Trail to the north of the current location of the existing Tamiami Trail, placing the roadway and bridges entirely onto the L-29 levee. The top elevation of the road would be 12.75 feet. The bottom cord elevation of the bridges would be 14.75 feet. Water quality treatment of stormwater runoff was required. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.2 ft for 0 cfs and ΔH=0.42 ft for 4,000 cfs.

3) Alternative 5.3–Northern Alignment with 1-mile Bridge and Relocation of L-67 levee-Crown 13.0 feet

This alternative would concentrate all increased water stages and all road work between S-333 and the Blue Shanty Canal near the Everglades Safari. A one-mile bridge would be constructed between Osceola Camp and Everglades Safari, aligned along the existing L-29 Levee. There would need to be additional bridging to connect the new bridge to the existing road alignment. The L-29 levee would have to be degraded and compacted to make it a suitable sub-grade for the roadway. The road elevation itself would have to be a minimum of 13 feet (National Geodetic Vertical Datum (NGVD) at the crown. This alternative included modifications to L-67A, L-67C, and L-29 levees and L-67A canal to promote water flow from WCA-3A into a small portion of WCA-3B and then under the raised portion of Tamiami Trail and into NESRS. The proposed structural changes would include water conveyance features added in the L-67A Levee, degrading a portion of the L-67C and L-29 levees, and plugging portions of the L-67A Canal to promote sheetflow from WCA-3A, through WCA-3B and into NESRS. The proposed modifications also included plugs in the L67A Canal, with different degrees of backfilling, to investigate the changes in canal flow patterns, as well as, any adverse impacts to recreational boating and fishing. In addition, this plan included the construction of a new boat ramp to maximize recreational access while the canal plug studies are being completed. Construction of temporary levees along the current north-south alignment of the Blue Shanty Canal in southwestern WCA-3B and northern NESRS in ENP, and a new gated water control structure in the L-29 Canal at the temporary levee alignment. The levee to the south and the levee to the north would be constructed to elevation 13 feet, NGVD. The levee would have 4 to 1 side slopes for maintenance until it is removed at a later date. The road would have to be raised to cross the levee which would put the crown at 15 feet, NGVD over the levee. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.2 ft for 0 cfs and ΔH=0.6 ft for 4,000 cfs.

4) Alternative 5.4-Raise Low Points and Add 1 Mile Western Bridge

This alternative would concentrate all increased water stages and all road work between S-333 and the Blue Shanty Canal near Everglades Safari. A one-mile bridge would be constructed between Osceola Camp and Everglades Safari, aligned along the existing road. The remainder of the road within this section would be raised to a minimum elevation of 13 feet, NGVD at the crown. The road cross section would be similar to Alternative 4.2.3. The section of the L-29 Levee opposite this new bridge would be removed. This alternative would include moving the L-67 Extension eastward to the Blue Shanty Canal edge. The levee to the south and the levee to the north would be constructed to elevation 13 feet NGVD. The road would have to be raised to cross the new levee which would put the crown at 15 feet NGVD over the levee. The head differential between the marsh and the L-29BC for this alternative was ΔH=0.2 ft for 0 cfs and ΔH=0.6 ft for 4,000 cfs.

5) Alternative 5.5-Pump Stations along L-29

This alternative suggested adding pump stations. There was no determination of the size of the station or the amount of water it would have to continually pump and therefore was not modeled. In order for the pump station concept to work, the road would still require raising the road and providing an outlet for water to pass through the road.

Table 2: Spreadsheet Model Controls

Alt	ALTERNATIVES	L-29 DESIGN STAGE (FEET)	RMA-2 Control Controls Hydraulic Slope between NESRS-2 and L-29BC	
1	**No roadway improvements**		**0 cfs**	**4000 cfs**
1.1	no action	7.5	0.22	1.20
1.2	spreader swales	7.5	0.20	1.06
1.3	add culvert sets (19 - 3x5ft dia) with swales **(2)**	7.5	0.20	0.94
1.4a	1-mile eastern bridge	7.5	0.20	0.42
1.4b	1-mile western bridge	7.5	0.20	0.42
1.5	raise western road section and 1-mile western bridge	7.5	0.20	0.42
2	**Roadway improvements - Crown 11.05ft**			
2.1	raise low points	8.0	0.22	1.20
2.2	*Roadway improvements with increased opening*			
2.2.1	raise low points, add culverts	8.0	0.20	0.94
2.2.2a	raise low points, add 1-mile eastern bridge	8.0	0.20	0.42
2.2.2b	raise low points, add 1-mile western bridge	8.0	0.20	0.42
2.2.3	raise low points, add 2-mile + 1-mile bridges	8.0	0.06	0.30
3	**Roadway improvements - Crown 11.55ft**			
3.1	raise road	8.5	0.22	1.20
3.2	*Roadway improvements with increased opening size*			
3.2.1	raise road, add culverts	8.5	0.20	0.94
3.2.2a	raise road, add 1-mile eastern bridge	8.5	0.20	0.42
3.2.2b	raise road, add 1-mile western bridge	8.5	0.20	0.42
3.2.3	raise road, add 2-mile + 1-mile bridges	8.5	0.06	0.30
4	**Roadway improvements - Crown 12.75ft**			
4.1	raise road	9.70	0.22	1.20
4.2	*Roadway improvements with increased opening size*			
4.2.1	raise road, add culverts	9.70	0.20	0.94
4.2.2a	raise road, add 1-mile eastern bridge (RGRR)	9.70	0.20	0.42
4.2.2b	raise road, add 1-mile western bridge (RGRR)	9.70	0.20	0.42
4.2.3	raise road, add 2-mile + 1-mile bridges (RGRR)	9.70	0.06	0.30
4.2.4	10.7-mile skyway (RGRR)	9.70	0.01	0.05
5	**Structural alternatives and/or road realignment**			
5.1	northern alignment of Alt 14	9.70	0.06	0.30
5.2	northern alignment with 1-mile bridge	9.70	0.20	0.42
5.3	northern alignment with 1-mile bridge and relocation of L-67 levee - Crown 13.00ft	9.70	0.3 West 0.22 East	0.6 West 1.2 East
5.4	current alignment with 1-mile bridge and relocation of L-67 levee - Crown 13.00ft	9.70	0.3 West 0.22 East	0.6 West 1.2 East
5.5	pump stations along L-29	9.70	-	-

9. Spreadsheet Model Results:

A) Average Annual Discharge into North East Shark River Slough

The annual discharge into NESRS for each year was computed from 1983 to 2006 (***Figure 11)*** and then the average annual discharge was calculated for each alternative (***Figure 12***). It should be noted that based on the average annual discharge the different alternatives ranged from 176,559 to 471,587 acre-feet per year (a spread of 275,028 acre-feet per year). One should be careful using only average annual volumes delivered because it does not accurately reflect all of the constraints on the system. These constraints range from available volume of water, amount of rainfall, and stage constraint on the system. From ***Figure 12*** it can be seen that the stage constraint on the roadway plays a significant factor in the deliveries of water during the wet season. As the stage constraint increases then the ability to meet a more natural wet season hydroperiod becomes achievable. For example with a 7.5 foot constraint during the 1995 year NESRS was hydrated enough to prevent the release of flows, however

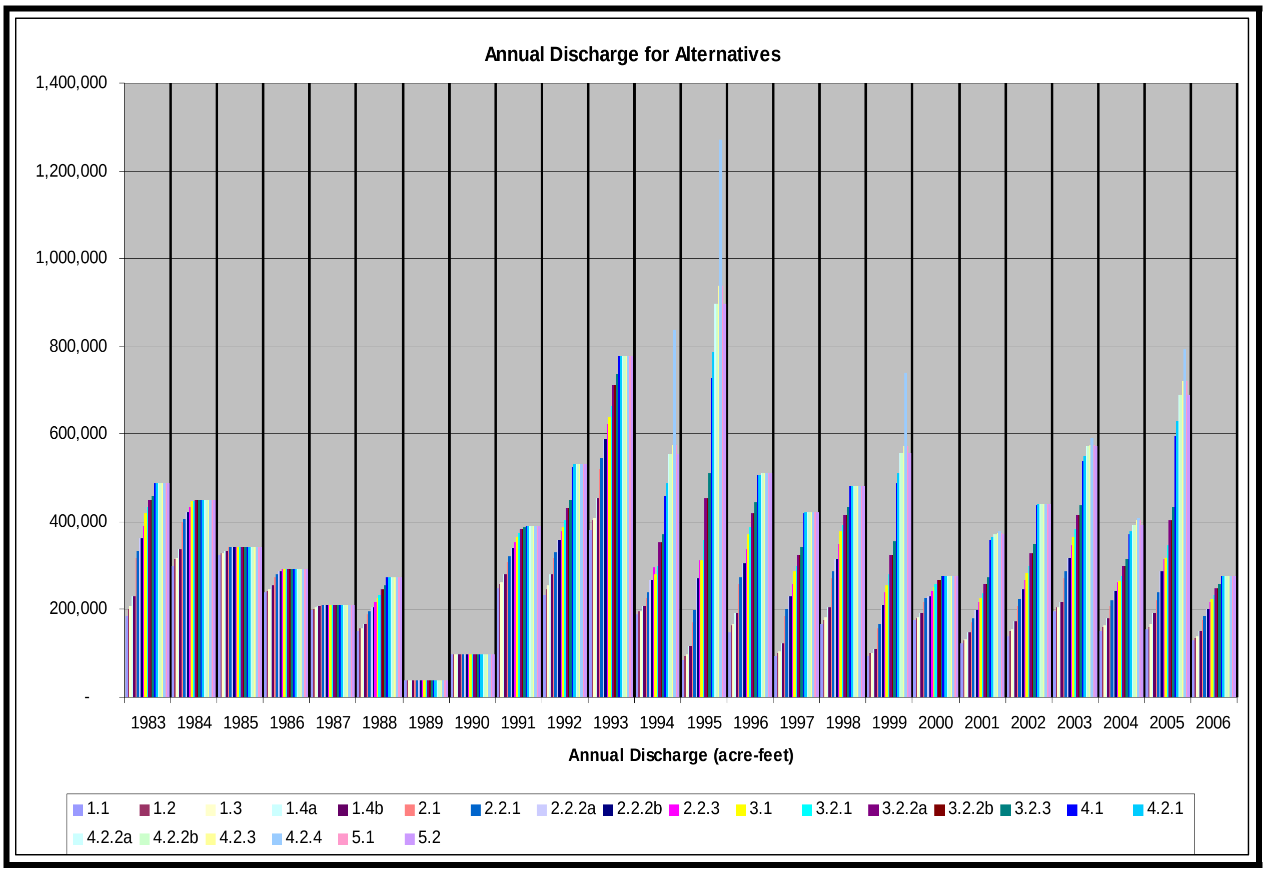

Figure 11: Annual Discharge for Alternatives

the annual discharge for 1995 was considerable less than average annual discharge. As the stage constraint increased however you see that the annual discharge for 1995 increase. In order to restore the natural hydroperiod within NESRS the system needs to be unconstrained to allow flows during all events.

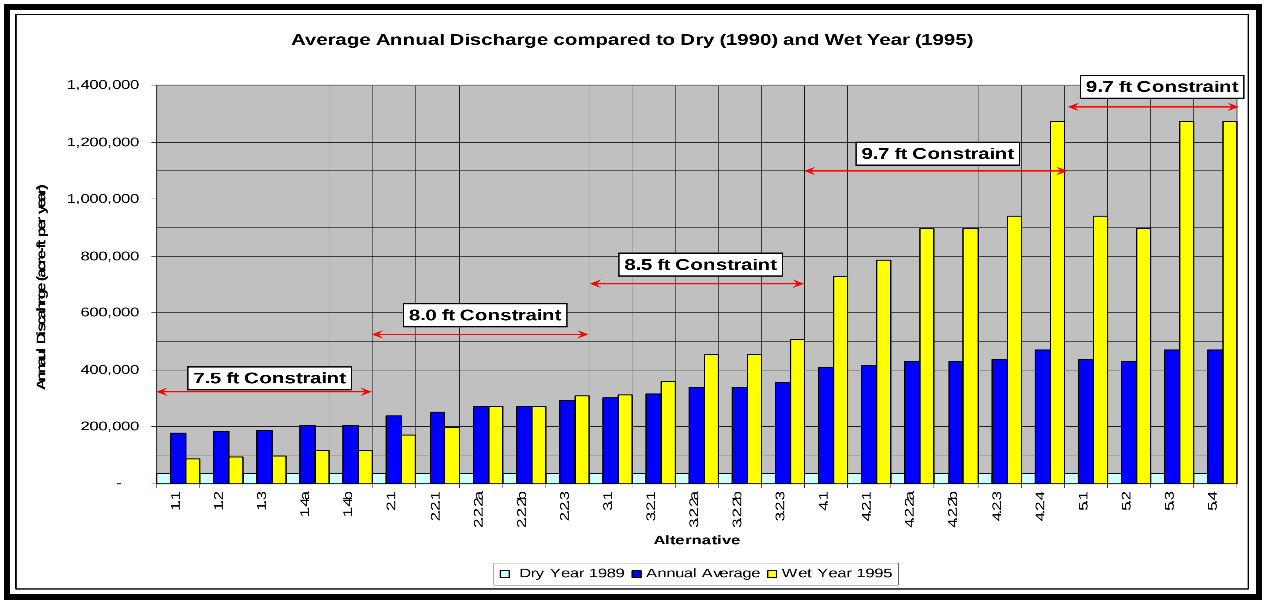

Figure 12: Average Annual Discharge Compared to Dry and Wet Year

B) Computed Stages at NESRS-2

The model computed a stage at NESRS-2 (***Figure 13***). One interesting point of note from the plot was that there were no significant differences during the dry year months. This finding can be easily explained by how the model assumed water moved through the system. This analysis was not an operations model looking at the best timing to deliver water; it simply looked at a specific day in the year and if the flows west to east could be redistributed then a certain stage would result. In short during the dry months all alternatives delivered basically the same volume of water resulting in similar stages. Similar results were seen during dry wet years.

In addition, a daily stage duration curve was produced that compares historical stages and modeled output (***Figure 14***) between the one-mile bridge with various Tamiami Trail stage constraints and historical data for the monitoring gage NESRS2. This figure shows that based on the model assumptions used that the bridge only increases the stages approximately 55 percent of the time. No difference was seen for the other 45 percent based on modeling assumptions used in the delivery of water to NESRS.

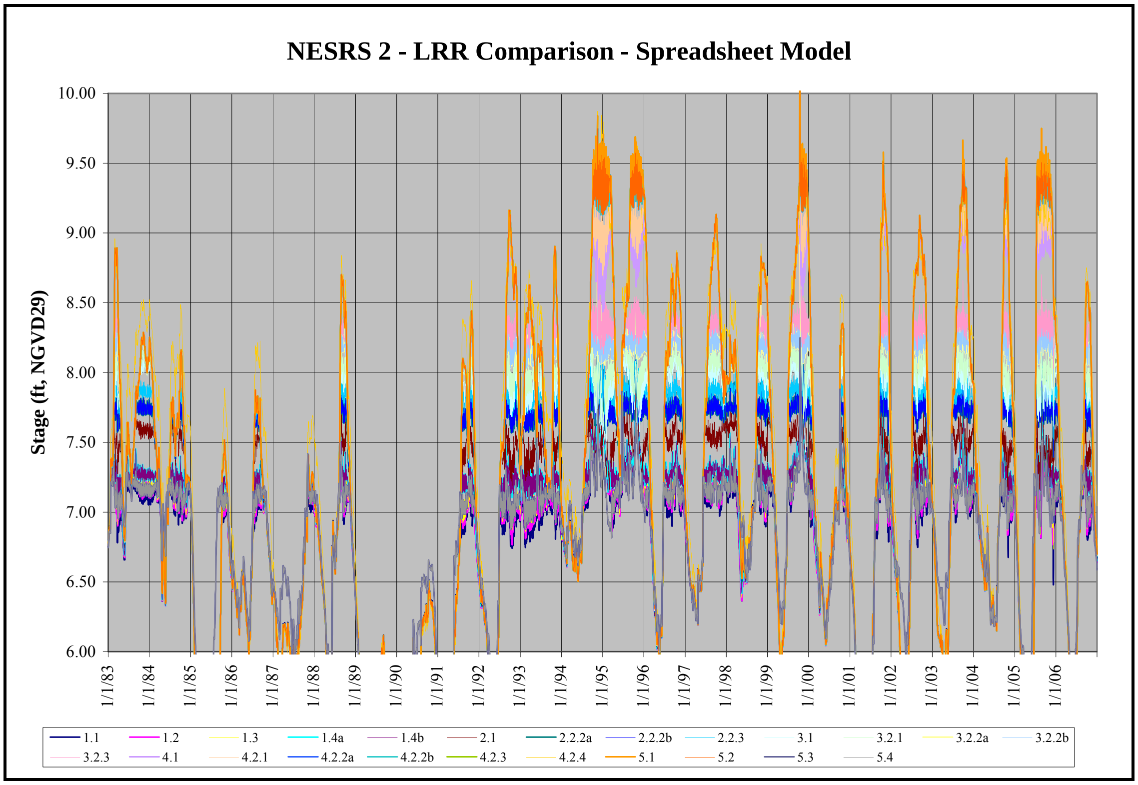

Figure 13: Computed stages for All Alternatives at NESRS2

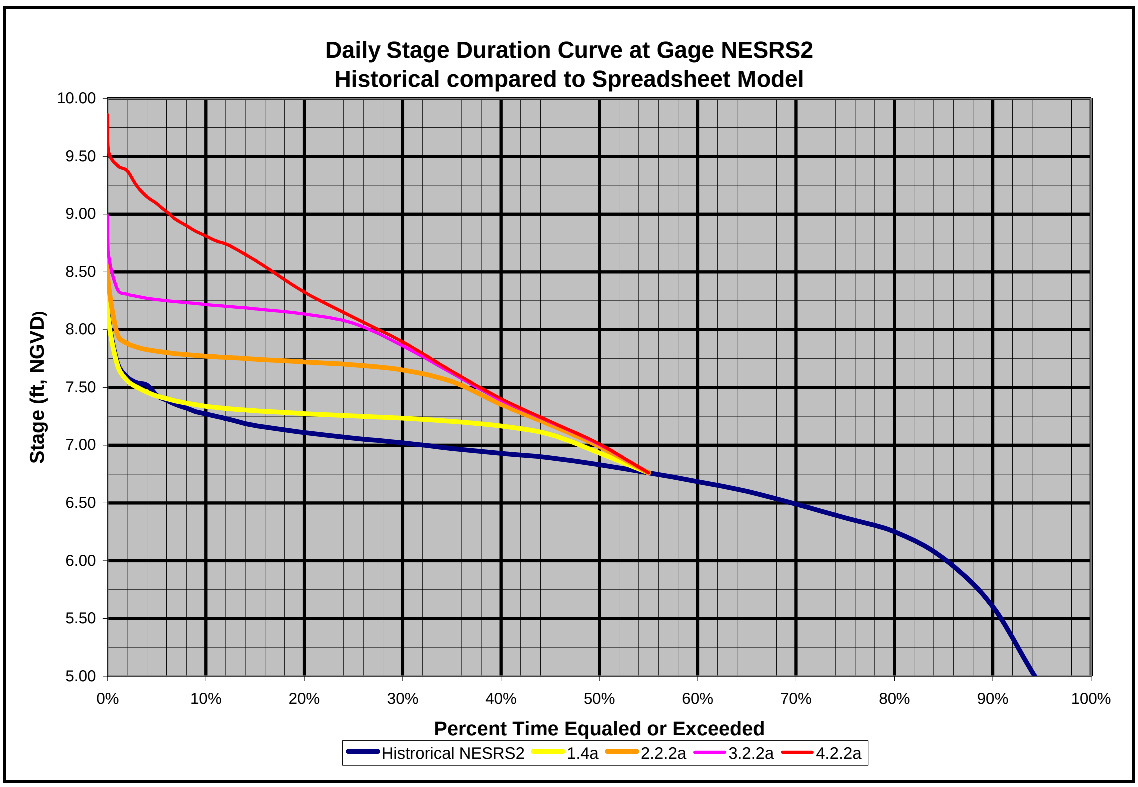

Figure 14: Daily Stage Duration Curve for Monitoring Gage NESRS-2

C) Computed Stages in L-29 Borrow Canal

The model computed a stage in L-29BC (***Figure 15***). From an analysis standpoint Alternatives 5.3 and 5.4 (which utilized movement of the L-67 Extension to the Blue Shanty Canal) in this plot showed the canal stage to the east of the new levee. The L-29BC stage to the west of the Blue Shanty Canal if shown would track slightly higher than the 10.7-mile bridge (Alternative 4.2.4). This slight increase was explained from a simple mathematical standpoint that when the same volume of water was distributed over 10.7 miles it had less stage difference than if it was distributed over approximately three miles.

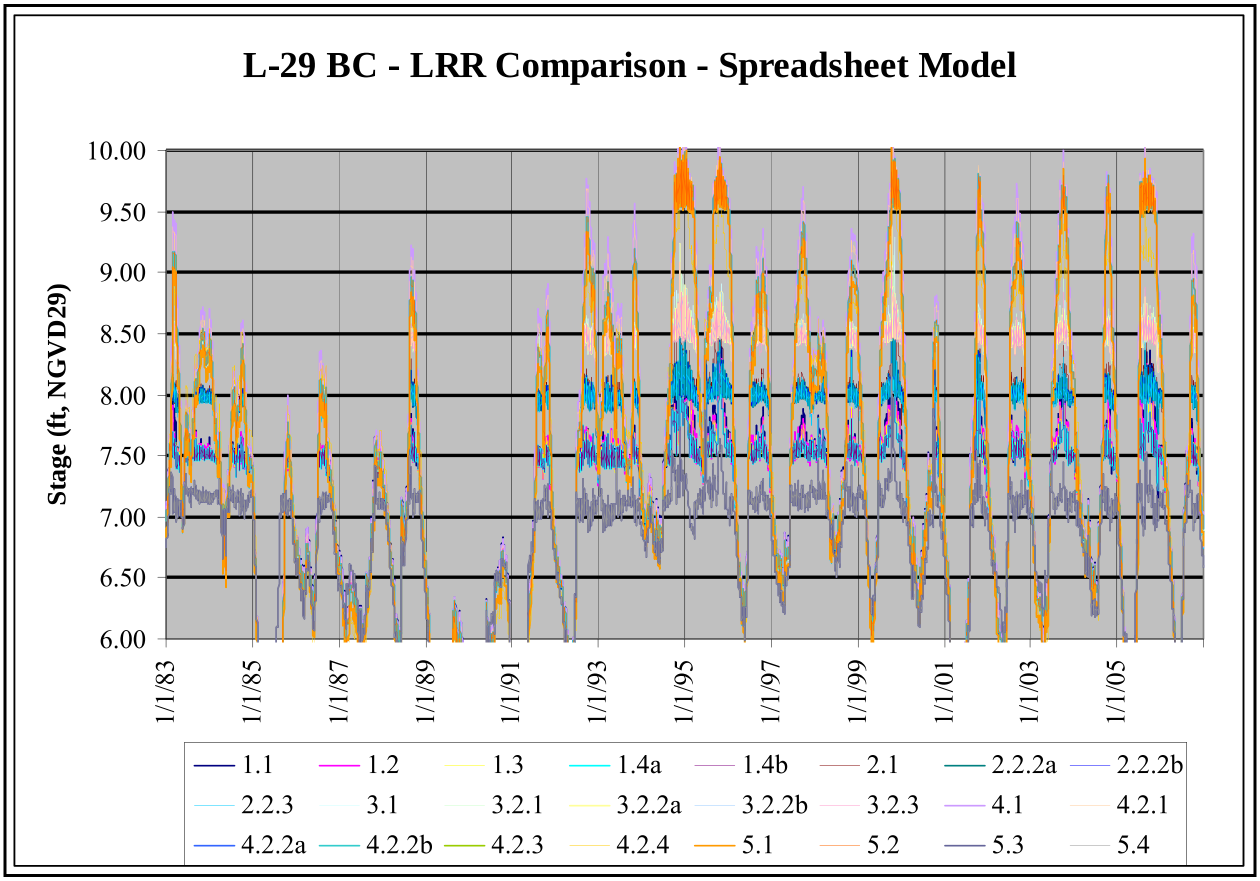

Figure 15: Computed Stages in L-29BC for Alternatives

In addition, a daily stage duration curve was produced that compares historical stages and modeled output (***Figure 16***) between the 1 mile bridge with various Tamiami Trail stage constraints and historical data. This figure shows that based on the model assumptions used that the bridge only increases the stages approximately 55 percent of the time. No difference was seen for the other 45 percent based on modeling assumptions.

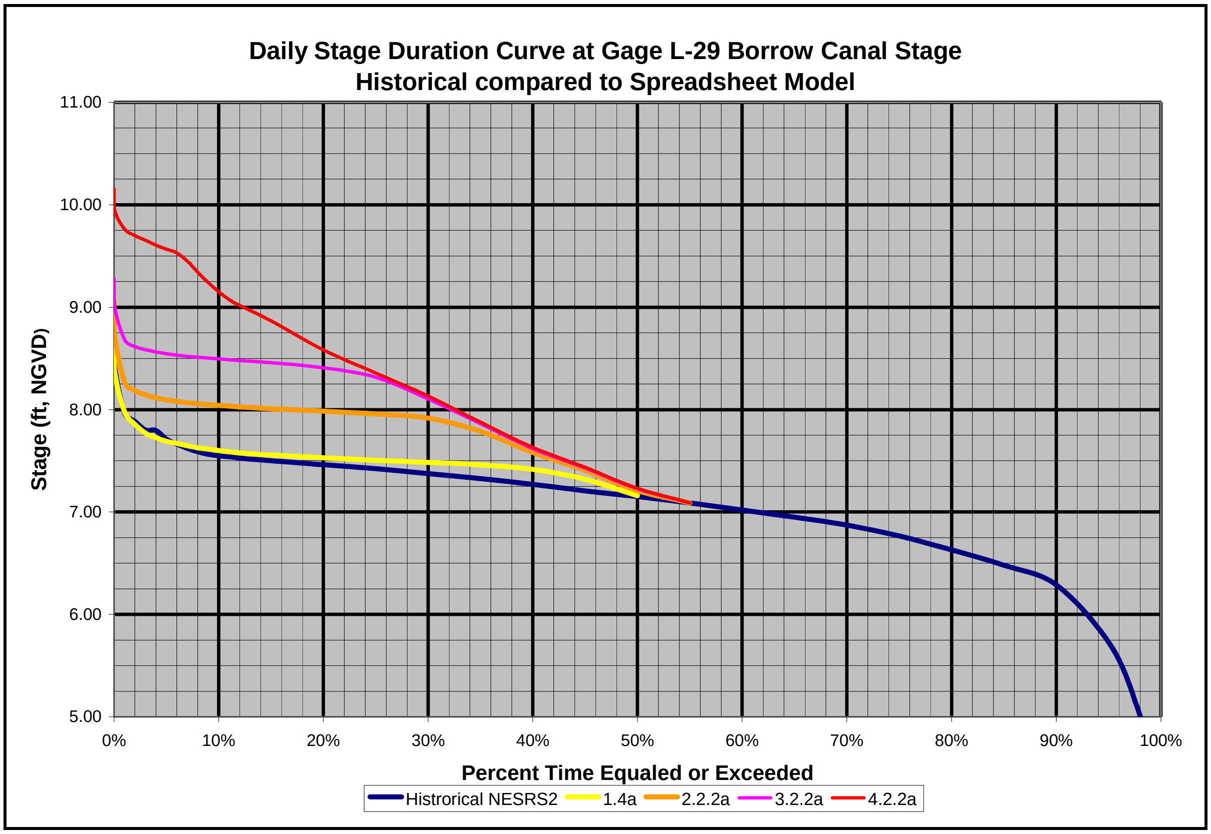

Figure 16: Daily Stage Duration Curve at the L-29 Borrow Canal

D) Correlations to Other Gages in North East Shark River Slough

For ecological evaluations correlations between NESRS-2 gage and other locations were performed based on historical data. ***Table 3*** depicts the equation and the R-squared value for these correlations.

Table 3: Correlation Equations and R Squared Value

Gage	Equation	
NESRS-1	y = 0.8504x + 1.1354	$R^2 = 0.919$
NESRS-3	y = 1.359x - 2.45	$R^2 = 0.9023$
NESRS-4	y = 0.8329x + 1.1346	$R^2 = 0.8048$
NESRS 5	y = 0.8989x + 0.5680	$R^2 = 0.8414$
R3110	y = 1.2745x - 4.2109	$R^2 = 0.3075$
NP-206	y = 1.205x - 2.5325	$R^2 = 0.4587$
RG-1	y = 0.141x5 - 4.671x4 + 60.951x3 - 390.92x2 + 1231.3x - 1519.3	$R^2 = 0.5149$
G-3273	y = 1.3224x - 2.7102	$R^2 = 0.7855$

10. Spreadsheet Model Assumptions and Uncertainty:

The spreadsheet model was developed in order to analyze the ecological effects of NESRS that different stage constraints and bridge sizes on Tamiami Trail would produce. This spreadsheet analysis/model looked at the area within NESRS in a simplified manner and the following general assumptions were made for all alternatives:

a) The area between Tamiami Trail (north side), the NESRS2 monitoring gage (south side), L-67Ext (west side), and L-31N (east side) could be defined as a simple storage area. As water was added/subtracted to the area the stage would increase/decrease based on a mass balance approach.

b) To compute the inflow volumes historical deliveries were used to prevent having to develop an operational model. This general assumption looked at the total deliveries into ENP [S12A + S12B + S12C + S12D + (S333 – S334)] and provided 55 percent of this volume into NESRS as long as the L-29BC was at a lower stage than the constraint for Tamiami Trail. If the L-29 stage was above the constraint flows were assumed to be zero. To smooth out the results for comparison purposes a seven day rolling average was used to compute the discharges into NESRS. For example, Alternative 1.2, during the period of 1 through 14 April 1995 computed flows (cubic feet per second, cfs) based on 55 percent of the volume were: 0, 1356, 0, 0, 1253, 0, 1435, 0, 0, 0, 1252, 0, 1172, and 0. In operations of the real system however we target a weekly flow volume to prevent the open/closing of the structure and to maintain a more steady flow. The computed 7 day running average produced results of: 420, 614, 398, 398, 577, 373, 578, 578, 384, 384, 563, 384, 551, and 346.

c) If the flow volume was not delivered to NESRS then it was assumed it was discharged via the S-12's to NWSRS. This assumption produced no net change to the Water Conservation Area 3A stage compared to historical conditions.

d) Bridge locations did not influence the ability of the spreadsheet model to deliver water. The spreadsheet model only consider topography in a very simplistic manner in regards of allowing flow out of the model and in terms of computing volumetric change. In reality the location of the bridge in conjunction with major sloughs would increase the volume of water delivered into NESRS. However this determination was beyond the scope of the spreadsheet model. It should be noted a separate analysis was used for Performance Measure 2.C (Flows into Northeast Shark Rive Slough provided via Bridge), see Appendix E for a description of the analysis.

e) A linear equation based on flow versus stage difference between L-29BC and NESRS2 was used to compute the stage in L-29BC. The basis for this linear equation was results from the RMA-2 modeling from the 2005 RGRR for Tamiami Trail Modifications.

The spreadsheet model does a very good job of interpreting the general trends that increased inflows will produce within NESRS as measured at the NESRS2 monitoring gage. However, stage predictions should not be considered absolutes from this analysis. This analysis is a simplification of a very complicated system developed for a comparison purposes between all of the different alternatives.

11. RMA-2 Model Results from 2005 RGRR fro the Tamiami Trail Modifications:

A) Objective of RMA-2 Modeling

The RMA-2 model was not used to determine the DHW but was used to evaluate the effects of bridge width and location when all other variables are held constant. The objective of this modeling analysis was to evaluate the velocity distribution south of the Tamiami Trail (US 41) and stage impacts that different bridge configurations will produce in North East Shark River Slough (NESRS). The goal of the Tamiami Trail Bridge is not only to pass an increased amount of flow into NESRS but also to create a more natural flow pattern (sheet flow) into NESRS. Velocities in excess of 0.1 ft/sec within ENP are assumed to be excessive and destructive to the ridge and slough processes of the Everglades. The RMA-2 model will was used to determine the stage impact in the L-29BC due to flow expansion losses based on different bridge widths.

B) RMA-2 Model Parameters

Conditions within ENP were modeled using RMA2, the depth-averaged hydrodynamic model of the Corps' TABS-MD modeling system. The model solves the depth-averaged (2D) nonlinear Navier-Stokes equations using an eddy viscosity turbulence closure. The Newton-Raphson iterative approach is used to solve the nonlinear equations. The model uses a fully implicit Galerkin finite element formulation, allowing for time steps as large as the variation in boundary forcing dictates.

1) Materials Specification

Six different material types were assigned within the model based on land features (***Table 4***). These land features varied from the marsh to the L-29 Borrow Canal.

Table 4: RMA-2 Model Material Types

Material Number	Land Type	Manning's N-Value
1	Marsh	Variable with Depth
2	L-29BC	0.035
3	Culverts thru Tamiami	0.045
4	Just downstream of Cu	Variable with Depth
5	Just downstream of S-	Variable with Depth
6	marsh along L-31N	Variable with Depth

2) Roughness Specification

Table 4 lists the corresponding land type with the Manning's N-value used. Where the variable with depth coefficient was used, the model utilized an equation for bottom roughness as a function of water depth equation. The mathematical form of the dependence of the Manning's friction coefficient with depth is

$$n = \frac{n_0}{d^{\alpha}} + n_v e^{-d/d_0} \qquad \text{Equation (4)}$$

Where, d = water depth (ft)
n_0 = scaling friction factor for depth dependence
n_v = scaling factor for exponential decay dependence (vegetative effects)
α = exponent on depth dependence
d_0 = reference depth for exponential decay

Figure 17 illustrates the depth dependence curve for the four material types that use this function. All four material types with a variable n-value used the same depth dependence curve.

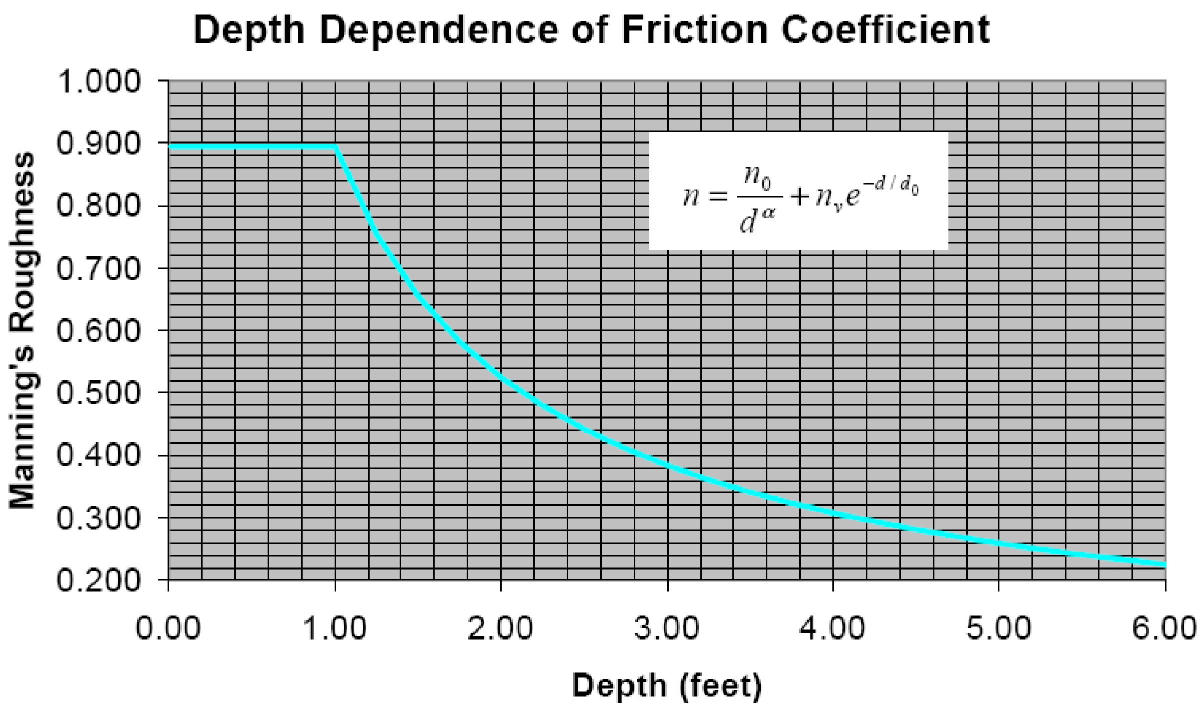

Figure 17: RMA-2 Depth Dependence Friction Coefficient

3) Topography
The model topography was developed from the best available data within the area. These sources included the USGS Helicopter Survey, the USGS Topometric Truck Survey, the SFMWD 5 foot Contour, and NHAP aerial photography (1950s-1960s). In addition, several Corps of Engineers surveys of L-29 Borrow Canal were used to approximate the canal invert. The accuracy of the data is approximately 0.5 feet.

4) Culvert Locations
Culvert locations were approximated as gaps through Tamiami Trail. These locations were set to the same elevation as the marsh downstream of the culvert. To account for the increased area and ease of flow, the Manning's n-value was set higher than what would be typically used for a culvert structure. Based on limitations of the model to not exceed a 50 percent change in area between elements (the base grid along the south side of Tamiami Trail is 200 feet by 200 feet), the culverts were approximated as 12.5 feet wide. All culvert structures were approximated to the same width.

5) Boundary Conditions
The model uses two types of boundary conditions, 1) boundary discharge lines and 2) boundary headlines. Boundary discharge lines were defined for all inflow points along the northern boundary of the model

representing all structures. A boundary headline was used along the southern boundary to specify the starting water surface elevations from gage P-36. To determine the flows and stage for the model runs, a frequency analysis using the Log Pearson Type III Distribution was performed on the West Bookend Run (CSOP Alternative 2 dated 010405 v5.5.4). The West Bookend Run was chosen because it was the most environmentally aggressive plan that put the largest volume of water into North East Shark River Slough. Steady state simulations were performed for the following return period discharges: 1, 2, 5, 10, 20, 25, 50, and 100 year events.

6) Structure Locations
All structures and culverts were located in the general proximity of the real world coordinates plus or minus 100 feet based on the mesh configuration of the model. The new weirs on the L-29 levee are based on the centerline locations of the CSOP model runs for Water Conservation Area 3B.

C) RMA-2 Model Results

Several different results were analyzed from the RMA-2 Model output as part of the benefits analysis. A brief description follows for each set of information.

1. For each alternative, the velocity at the center of the bridge for the 1-year and 100-year computed flows was compared to the marsh velocity at a distance of approximately 10,000 feet downstream of the road from the 10.7-mile bridge option. Velocities for these return periods are depicted in ***Figure 18*** and ***Figure 19***. The target is to minimize the difference in velocity between the bridge and the marsh. The higher velocities produced by the shorter bridge are extremely destructive to the ridge and slough environment of the Everglades immediately south of the Tamiami Trail.

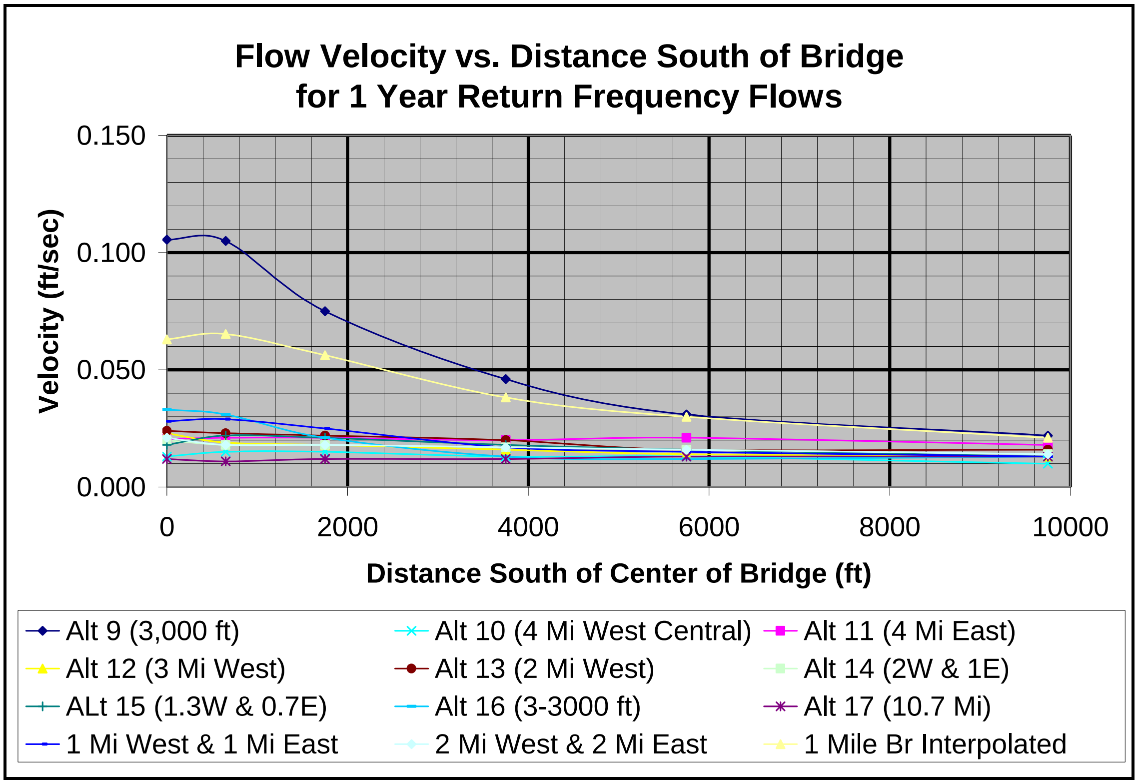

Figure 18: Velocity south of Tamaimi Trail 1-Yr Return Frequency

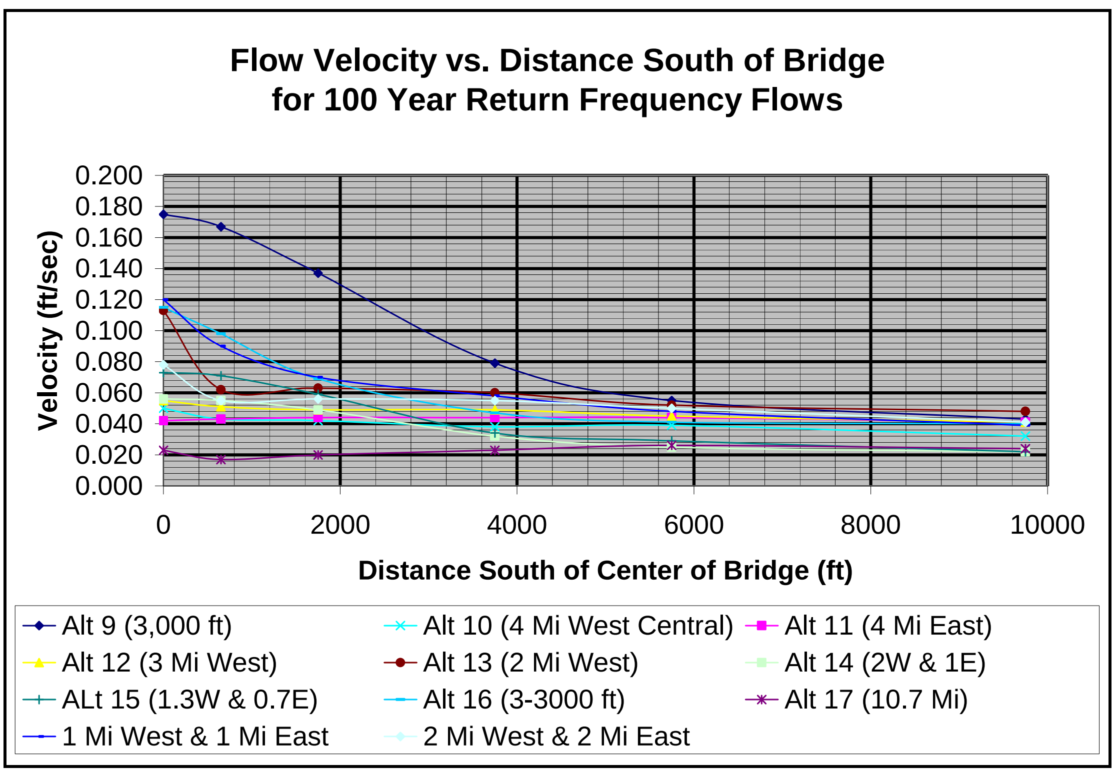

Figure 19: Velocity south of Tamaimi Trail 100-Yr Return Frequency

2. For each alternative the area with velocities above 0.1 feet per second was computed. This allowed for a comparison of which alternatives would produce the least amount of impacted area (***Table 5)***. The calculations for the area are based on the area immediately south of Tamiami Trail and east of S-333.

Table 5: RMA-2 Analysis of Area of Impact of Velocity Greater than 0.1 ft/sec

		Acres Above
	No Action	187
Alt 9	3000 Foot	411
Alt 10	4 Mi Central	98
Alt 11	4 Mi East	105
Alt 12	3 Mi West	181
Alt 13	2 Mi West	220
Alt 14	2 Mi West & 1 Mi East	295
Alt 15	1.3 Mi West & 0.7 Mi East	300
Alt 16	Three - 3,000 foot	330
Alt 17	10.7 Mi	8

3. The backwater effect that the marsh produces is the main controlling factor in the stage in the L-29BC. Each bridge alternative analyzed as part of the Tamiami Trail RGRR/SEIS would produce a minimum amount of head loss across the embankment. For example in the Draft RGRR/SEIS in 2003, the recommended alternative had a 3,000-foot bridge to convey water south. The differences are the net opening of the bridge and the expansion losses created by the marsh as the water moves south and away from the bridge opening. To show the impact of embankment capacity (size of openings for culverts or bridge) vs. marsh resistance, a plot was generated from the RMA-2 model runs comparing the stage difference between the L-29BC and 10,000 feet downstream (ΔH) in the marsh for the various opening lengths considered (***Figure 5*** note existing culverts are indicated as zero bridge length in this graph). This clearly shows that bridge length affects the getaway capacity of the downstream marsh, and the longer the bridge the more efficient the marsh is at moving water south into North East Shark River Slough (NESRS). The L-29BC acts as a stage equalizer upstream of the roadway embankment and this increased stage is then propagated into WCA-3B as water is discharged through the S-355s and potentially other passive structures (ΔS) in L-29 (resulting in a stage increase for WCA-3B of $\Delta H + \Delta S$)

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