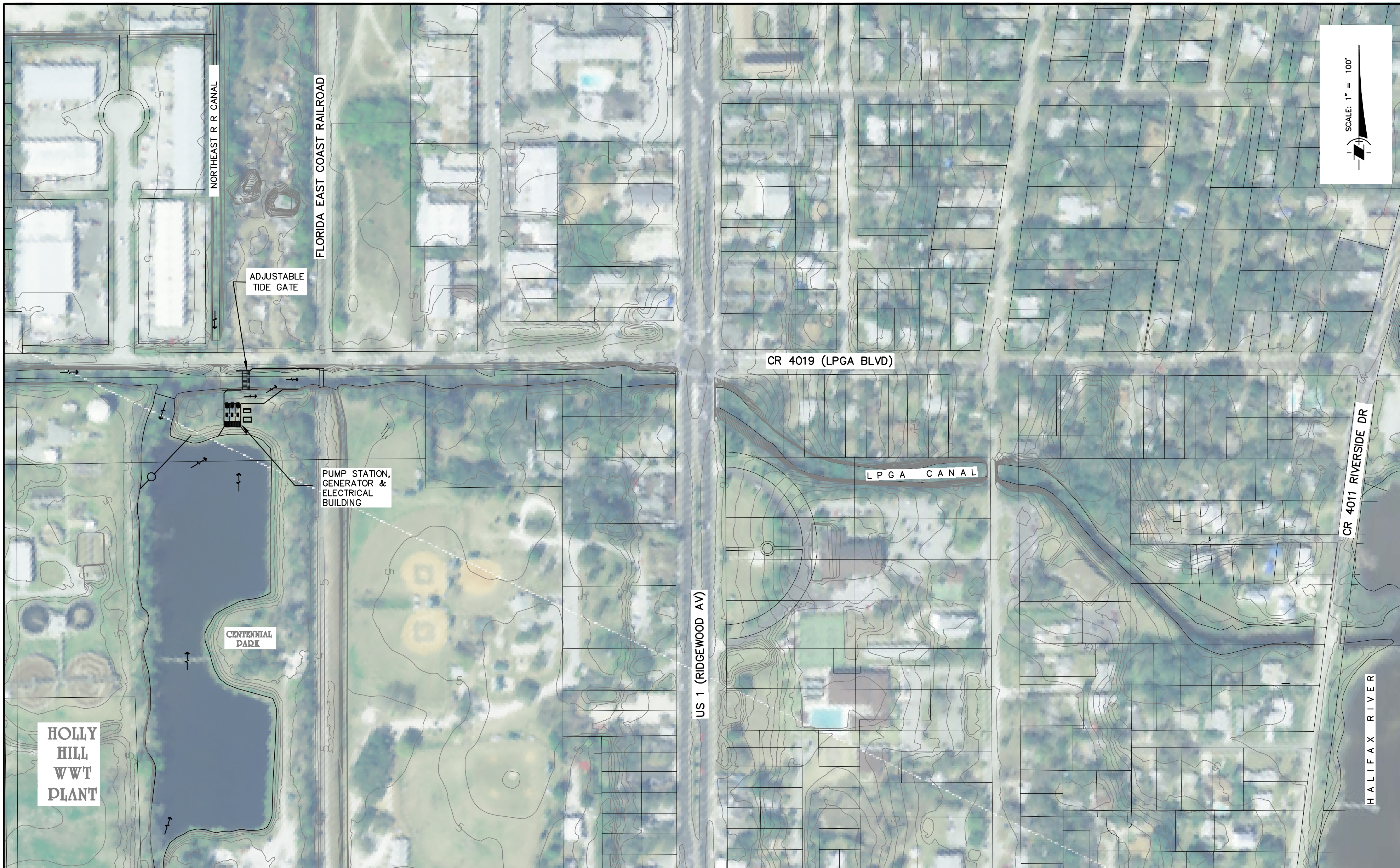


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East Volusia
Nova Canal
Flood Control

LPGA BLVD. SITE

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| APPROVED | BTB | DATE | 12-09 |

SCALE
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Figure
1-4


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Figure
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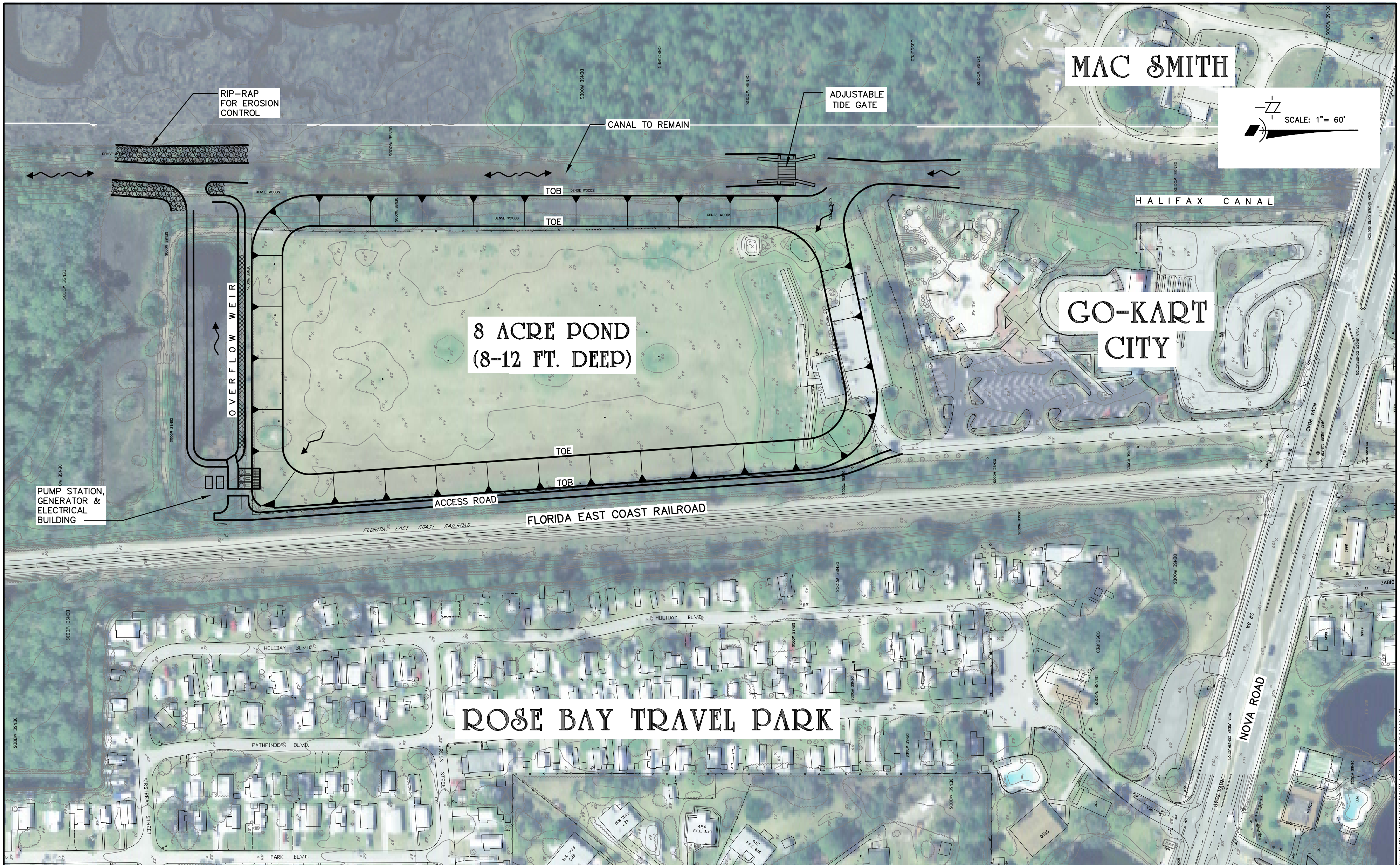


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Figure 1-6

# Section 2
# Surface Water Hydrology and Hydraulics

Section 2 presents the methodology used to develop a hydrologic and hydraulic model of the study area, and to apply the model in evaluating current conditions plus one project alternative condition. The USEPA model SWMM5 was used to simulate the surface water hydrology and hydraulics of the study area under both conditions.

The study area model was built upon previously developed models, updating these models as necessary and combining the models into a single model of the entire study area. Initial model development focused on validating the model based on comparison of model results to known high water conditions measured or reported during the May 2009 extreme storm event. The validated model was then applied to evaluate the conveyance system performance for design storms which varied by return period, storm duration, and tailwater elevation conditions. Finally, the model was applied with a project alternative at the LPGA Canal, Reed Canal and Halifax Canal. At each canal, the project alternative included a pump station and tide weir-gate designed to reduce flooding due to tidal backwater conditions and to lower water levels in advance of storms and lower flood stages during and following extreme storm events.

## 2.1 Hydrologic and Hydraulic Model

SWMM5 is a dynamic hydrologic and hydraulic model capable of performing continuous or event simulations of surface runoff and groundwater baseflow, and subsequent hydraulic conveyance in open channel and pipe systems. In this study, the focus was on extreme storm events, so the model was run on an event basis, and only surface runoff was simulated (no groundwater simulation).

The hydrologic model is based on the subdivision of the study area into hydrologic units (HUs), which are each characterized by physical characteristics such as area, imperviousness, and infiltration capacity. Rainfall is applied to the HUs, and the model calculated the quantity of rainfall converted to stormwater runoff, and the runoff rate from the HUs. The runoff from the HUs is assigned to defined loading points on the user-defined stormwater management system in the hydraulic model of the study area.

The study area hydraulic model represents the numerous canals, closed conduits and culverts that convey runoff through the stormwater management system to outlets discharging to the Halifax River. SWMM5 uses a link-node representation of the stormwater management system to dynamically route flows by continuously solving the complete one-dimensional Saint-Venant flow equations. The dynamic flow routing allows for representation of channel storage, branched or looped networks, backwater effects, free surface flow, pressure flow, entrance and exit losses, weirs, orifices, pumping facilities, rating curves, and other special structures/links. Control rules may be used to operate structures based on timing and/or stage and flow conditions within the model.

# 2.2 Study Area Model Update and Refinement

The SWMM5 study area hydrologic and hydraulic model update began with existing SWMM that had already characterized the hydrologic and hydraulic model parameters required to simulate the runoff generation in the study area and flow routing through the stormwater management system. These models were developed in earlier versions of SWMM (1958 through 2006), thus were updated to the current SWMM5 version.

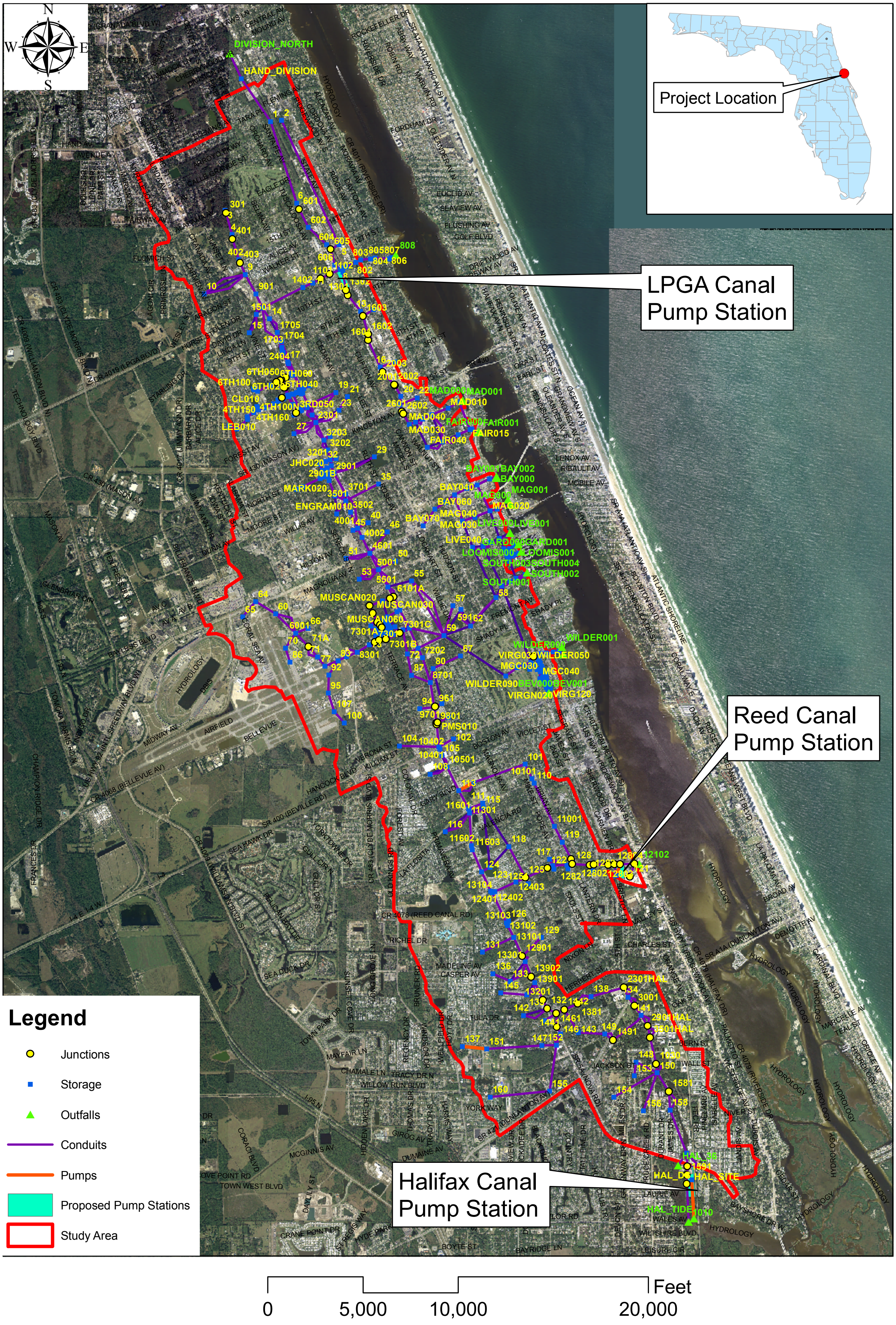
The study area model builds upon existing models that were based on NGVD 1929. Data provided by stakeholders (e.g., high water elevations) in some cases were provided based on the North American Vertical Datum (NAVD). In those cases, data were converted from NAVD to NGVD where necessary using a constant offset of +1.35 ft (NGVD = NAVD + 1.35), based on tidal benchmark differences published by the National Oceanic and Atmospheric Administration's (NOAA) National Ocean Service (NOS) between the National Geodetic Survey 's (NGS) and Center for Operational and Oceanographic Products and Services' (COOPS) datums. The offset between datums varies little over the study area.

## 2.2.1 Model Version Update

Earlier versions of SWMM (versions 3 and 4) were previously used to model the study area. These versions used separate SWMM modules for hydrology (RUNOFF module), and Hydraulics (EXTRAN module), which were linked by an interface file. SWMM 5 uses similar architecture; however, the hydrologic and hydraulic engines are modules within the same model and are run simultaneously.

The latest version of the Nova Road Canal SWMM from the City of Daytona Beach Stormwater Master Plan (SWMP) Update (2006) was one model used as the basis for the study area model. This model covered the area that is considered to be tributary to the Halifax River via the Reed Canal and LPGA Canal outfalls, and all outfalls in between. The remainder of the study area is considered to be tributary to the Halifax Canal. Quentin L. Hampton (QLH) provided a SWMM of this area, developed initially by Marshall Provost and Associates.

Both of these models were converted to SWMM5 using a conversion tool provided with the SWMM5 software. The converted models were then run for several design storms to demonstrate that the SWMM5 results were comparable to results generated with the older SWMM versions. Finally, the converted models were combined into a single model of the entire study area (see **Figures 2-1 through 2-4** for model schematics).

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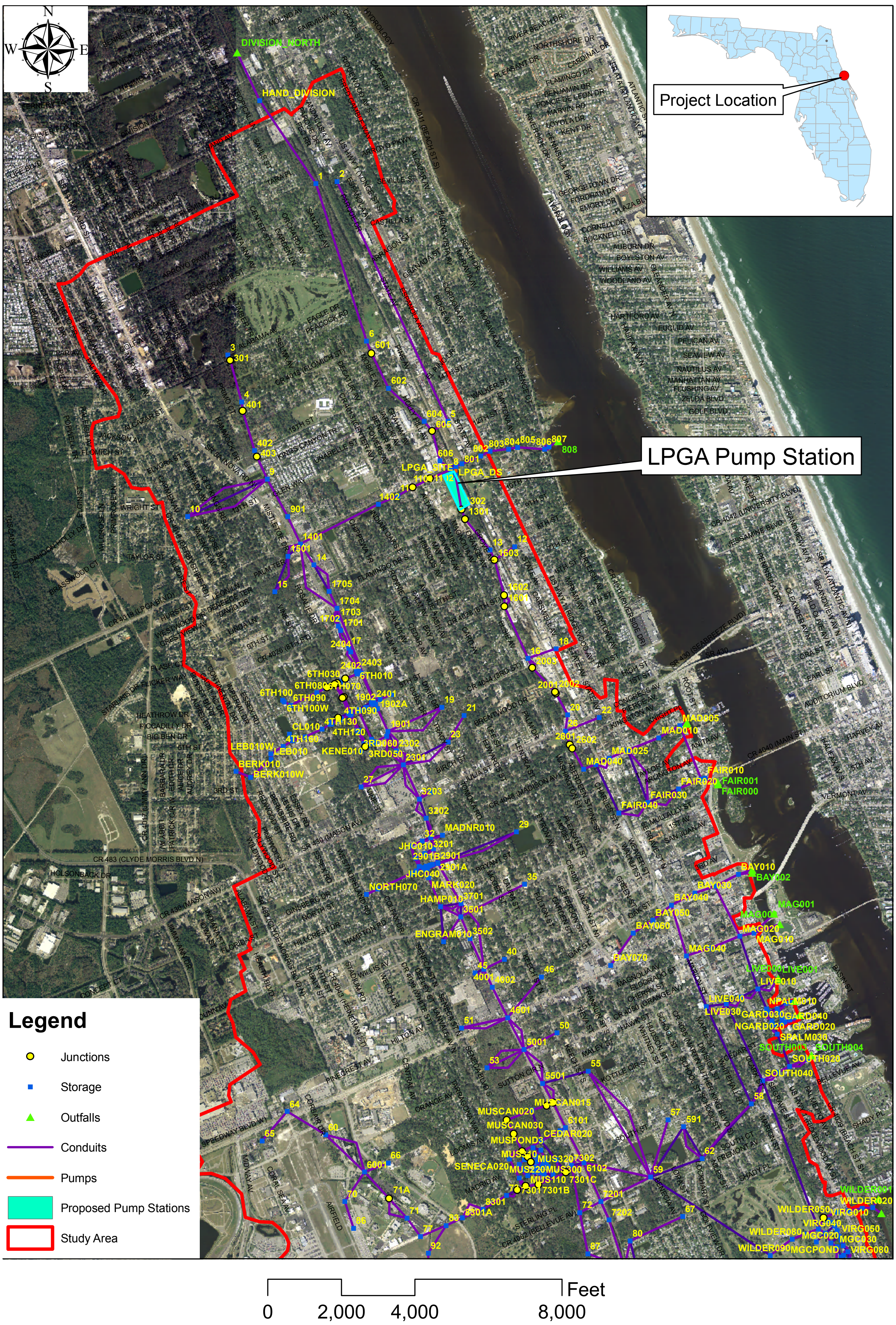
CDM

East Volusia
Regional Water Authority
(EVRWA)

**Figure 2-1**
Nova Canal Flood Control and
Integrated Water Resources Program
Nova Canal Schematic

UPDATED
12-14-09

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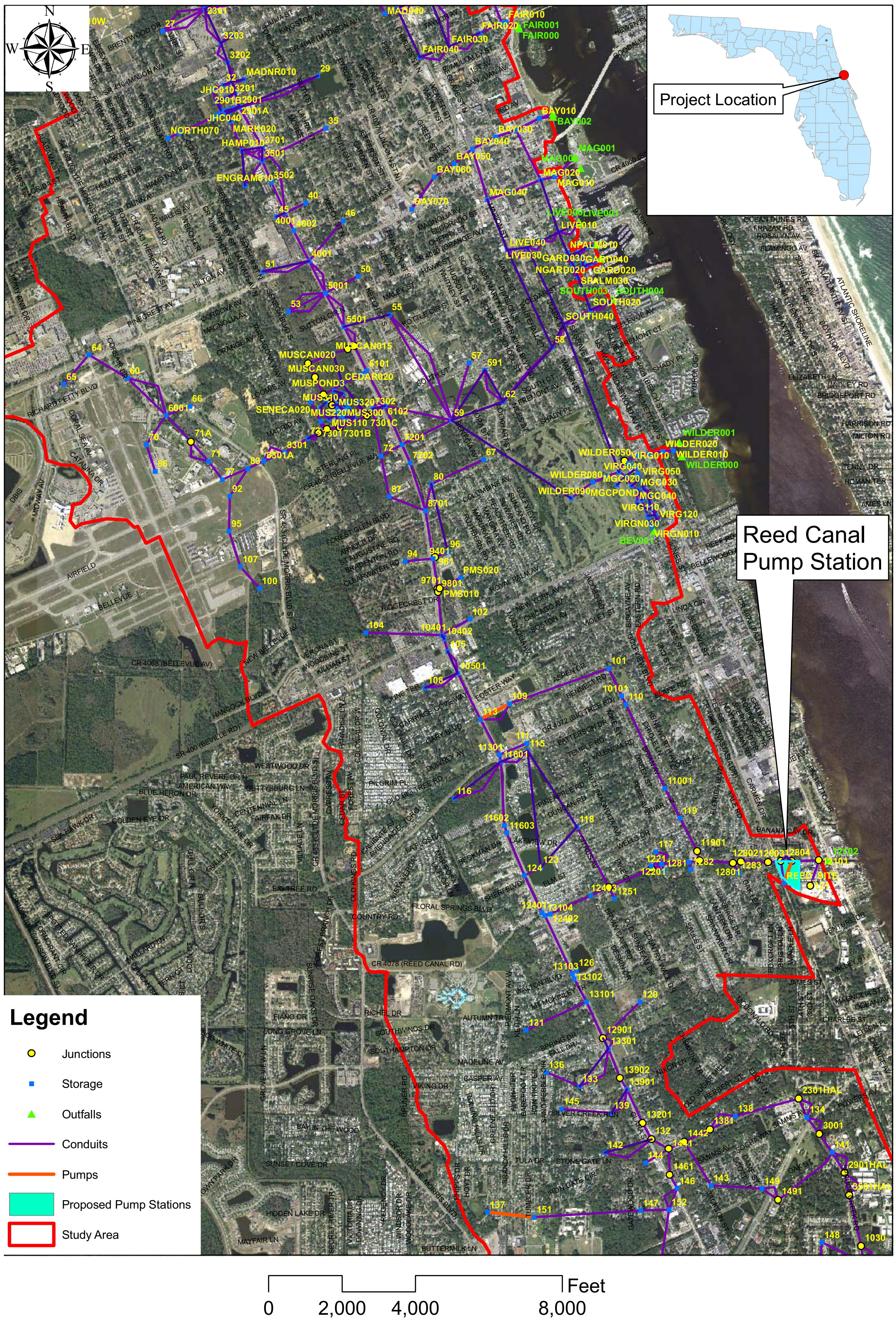
| | | | |
|---|---|---|---|
| CDM | East Volusia Regional Water Authority (EVRWA) | **Figure 2-2** Nova Canal Flood Control and Integrated Water Resources Program Nova Canal Schematic - North | UPDATED 12-14-09 |

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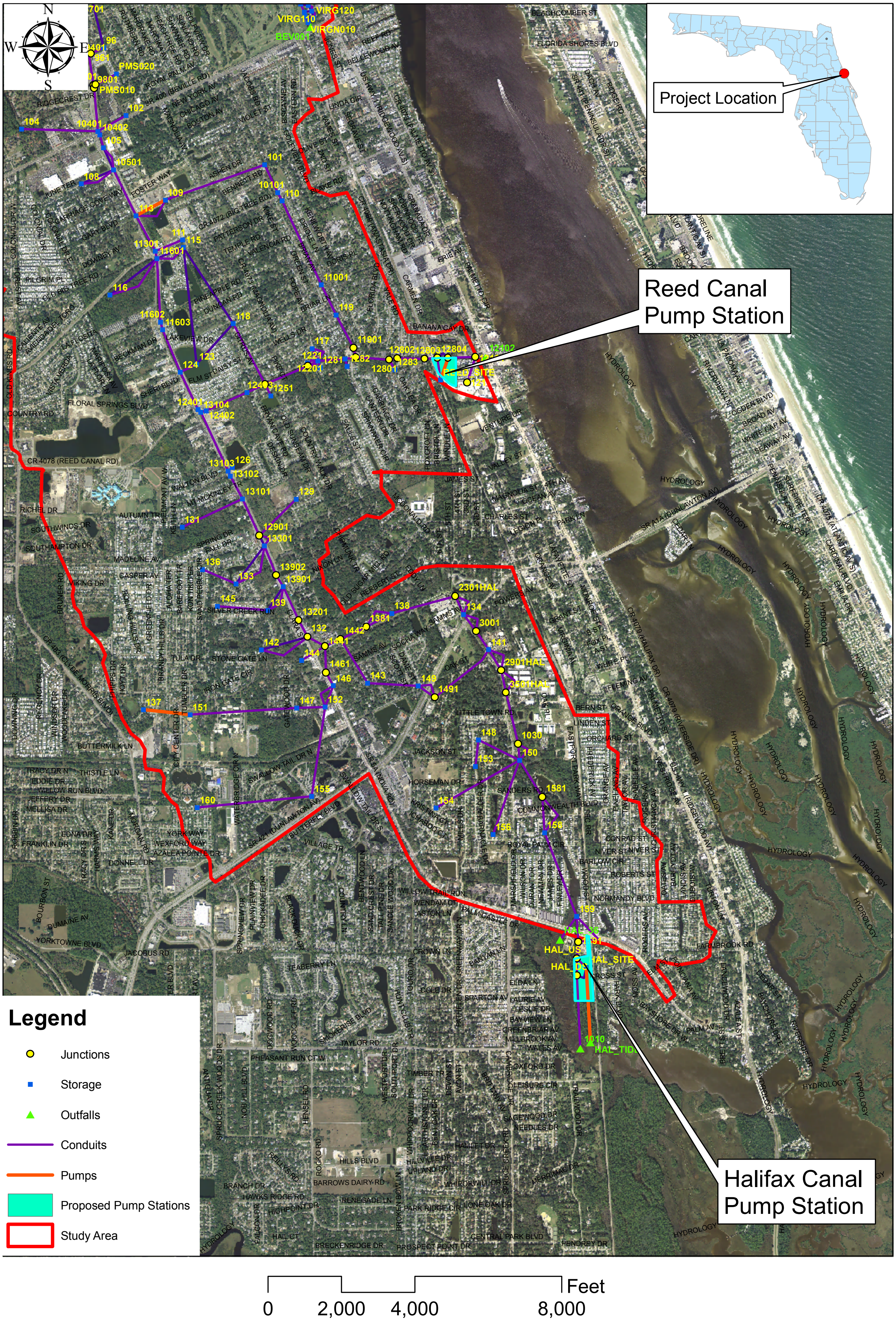
| CDM | East Volusia Regional Water Authority (EVRWA) | **Figure 2-3** Nova Canal Flood Control and Integrated Water Resources Program Nova Canal Schematic - Central | UPDATED 12-14-09 |
|---|---|---|---|

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CDM

East Volusia
Regional Water Authority
(EVRWA)

**Figure 2-4**
Nova Canal Flood Control and
Integrated Water Resources Program
Nova Canal Schematic - South

UPDATED
12-14-09

### 2.2.2 Hydrologic Model Data Review

The total study area is 23 square miles, and is subdivided into 274 HUs. The HU areas range from 0.2 acre to 517 acres, with an average of 53 acres. In addition to area, hydrologic parameters assigned to each HU include width, slope, directly connected impervious area (DCIA), overland flow roughness, initial abstraction, and infiltration rates. In general, the most critical input parameters are the infiltration rates and DCIA.

The Horton infiltration method was used for this analysis, to determine how much of rainfall is converted to runoff or infiltrates into the subsurface of pervious areas such as woods or lawns. Key input parameters are based on soil characteristics, which include maximum and minimum infiltration rates. Maximum infiltration values ranged from 2.1 to 11.4 inches per hour (in/hr), while minimum infiltration values ranged from 0.05 to 0.95 in/hr. These values were considered to be consistent with the soil types within the project area.

The DCIA represents impervious areas for which there is no infiltration and where all precipitation runs off to the primary water management system. The previously-developed models within the study area used land use distribution data to estimate the DCIA of each HU. The study area is highly developed, which is reflected in the model. The overall DCIA for the entire study area is 49 percent.

### 2.2.3 Hydraulic Model Data Review

The SWMM5 hydraulic model uses a node/link representation of the primary stormwater management system (PSMS). Generally, nodes are located at:

- The ends of culverts;
- Upstream and downstream of bridge structures;
- Points along the canals where the geometry, direction, and/or slope of the channel varies significantly;
- Canal intersections;
- Structures along the canals including pump stations, locks and gated outfalls; and
- Locations representing the HU low elevations.

The existing condition model for the study area contains 448 nodes, and 769 links conveying flows between nodes. These links include canal segments, closed conduits, culverts under roadways, weir or orifice pond outlet structures, and overflow weirs or irregular conduits representing road overflow.

Physical characteristics of the canals or other open channel conveyances in the study area include length, slope (upstream and downstream invert elevations), cross-sectional geometry and Manning's n roughness coefficient for channel and overbank. Generally, the characteristics in the historical models were used in the updated model. In several cases, channel lengths were adjusted to better reflect actual lengths. In addition, the Manning's roughness coefficient values were adjusted in several places in the model validation process.

Stage-surface area relationships are often assigned at nodes to reflect low-lying areas in the HUs. This is particularly suitable for areas such as the study area. Stage-storage area relationships are necessary in relatively flat models where flood waters may overflow the channel banks and fill low-lying areas. An accounting of the volume of these areas is needed for both accurate flood elevation predictions as well as peak flow estimates.

Generally, the stage-surface area relationships in the historical models were used in the updated model. Stage-area data at the northern end of the Railroad Ditch was modified based on review of available LiDAR data, as part of the model validation, to better represent ponding in that area and subsequent overflow north to Thompson Creek (Laurel Creek and Riviera Oaks subdivision area).

Closed conduit and culvert characteristics include length, slope (upstream and downstream invert elevations), width and depth, Manning's roughness coefficient, and inlet and outlet loss coefficients. The data from the historical models were used in the initial model development, and were updated as necessary to reflect known PSMS improvements or improvements discovered as part of the model validation process.

## 2.3 Model Validation

For purposes of this study, validation refers to the process where results produced by the model are compared to measured/observed parameters to demonstrate that the predicted values (flows and stages during a rainfall event) match reasonably well. In this case, the focus was on the May 2009 event, which delivered up to 27.8 inches of rain to the study area over a 5-day period (May 18 through May 22). Measured data used in the evaluation of model performance include flow and stage time series at three USGS stations on the major outfall canals (LPGA Canal, Reed Canal, Halifax Canal) and measured or estimated high water elevations for the City of Ormond Beach, City of Holly Hill, City of Daytona Beach, City of South Daytona, and City of Port Orange.

### 2.3.1 Rainfall Data

Rainfall data for the validation storm were obtained from several sources, including the National Climatic Data Center (NCDC), Wunderground, and localized measurements taken by the stakeholders at critical infrastructure sites. The rainfall data were analyzed and ultimately two data sets were utilized. Rainfall data with an hourly interval obtained from the NCDC for the Daytona International Airport were applied to the southern portion of the study area. A rainfall distribution based on the daily depths measured at an NCDC station in the City of Ormond Beach was applied to the northern portion of the study area. The daily depths for this area were further discretized to hourly inputs by applying the same daily distribution observed at the airport. The total rainfall depths for the north and south areas were 27.8 and 20.6 inches, respectively.

### 2.3.2 Flow and Stage Data

Hourly flow and stage data were available at a total of three locations, one each on the LPGA Canal, Reed Canal, and Halifax Canal.

The USGS gage on the LPGA Canal (ID 02247509) is located at the upstream end of the culvert under US 1 (Ridgewood Avenue). Stage measurements were available for the entire event, but flow data ended on the morning of May 20. The USGS has rated this gage on average 8 times per year since installation in December of 2000.

The USGS gage on the Reed Canal (ID 02248025) is located at the downstream end of the railroad crossing approximately 1.5 miles upstream from the canal discharge point to the Halifax River. Like the LPGA Canal, the Reed Canal has measured stages available for the entire event, but flow data ended on the morning of May 20.

The USGS gage on the Halifax Canal (ID 02248030) is located at the downstream end of the culvert under Nova Road. Both stage and flow measurements were available for the entire event.

### 2.3.3 Downstream Boundary Conditions

There were no available tidal boundary stage data at any of the major canal outfalls to the Halifax River. Available tidal data nearest to the study area was collected by NOAA for the following stations:

- 8721147 Ponce De Leon Inlet South, FL
- 8720954 Ormond Beach, FL

Real-time verified stage measurements were available for Ponce Inlet, and predicted values were obtained for the Halifax River at Ormond Beach.

The difference in predicted timing and amplitude between the stations was analyzed and vertical and temporal offsets were developed. It was determined that the tides in the Halifax River at Ormond Beach lag those at Ponce Inlet by approximately four hours with peak stages approximately 75 percent of those experienced at the inlet. Utilizing the offsets, the values measured during the storm event were superimposed onto the Ormond Beach station. Using a combination of the predicted values at Ormond for pre-storm conditions, and the manipulated values to represent the storm surge, a dynamic tidal boundary condition was developed for the Halifax River at Ormond Beach for the month of May 2009 (see **Appendix A** for a visual representation).

There are numerous outfalls to the Halifax River represented in the model, varying spatially from the Halifax Canal in the south to the LPGA Canal in the north. Dynamic tidal boundary conditions were developed based on distance-weighted vertical and temporal offsets applied to the Ormond Beach and Ponce Inlet information (see Appendix A for charts showing the tidal conditions). **Table 2-1** shows the relative distances of the outfalls from the inlet and the temporal lag.

**Table 2-1 Relative Distances of Outfalls**

| Location | Distance (miles) | Distance Ratio | Time Lag (hours) |
|---|---|---|---|
| **Ponce Inlet** | 0 | 0.00 | 0.0 |
| **Halifax Canal** | 5.8 | 0.34 | 1.3 |
| **Reed Canal** | 7.6 | 0.44 | 1.8 |
| **Live20** | 11.5 | 0.66 | 2.7 |
| **LPGA Canal** | 14.3 | 0.83 | 3.3 |
| **Ormond Beach** | 17.3 | 1.00 | 4.0 |

It should be noted that dynamic boundary conditions were explicitly developed for the three major canals being analyzed, as well as one additional intermediate location (Live20). These four boundary conditions were then applied to all outfalls to the Halifax River based on spatial proximity.

## 2.3.4 High Water Mark Data

Estimates of high water elevations during the May 2009 were provided by the EVRWA stakeholders.

The City of Ormond Beach provided Arc GIS shapefile coverages which included locations of structural flooding during the event. Many of these structures were at the north end of the Railroad Ditch Canal, at Hand Avenue. There were also several structures at Arroyo Road at the headwaters of the Northwest Canal. Shapefile attributes included surveyed slab elevations and depth of flooding, which were added to calculate the high water elevations.

The City of Holly Hill provided one high water mark, at the Public Works building. This is located near the LPGA Canal, between Alma Road and the confluence of the Railroad Ditch canal and the LPGA Canal.

The City of Daytona Beach provided high water estimates at several locations. These were primarily located off of the Nova Canal system running north and south along Nova Road between the LPGA Canal and the Reed Canal.

The City of South Daytona provided high water estimates along the Reed Canal and the tributary Stevens Canal. These included peak stages in the canals, as well as high water elevations on several streets.

For the City of Port Orange, a spreadsheet was provided that listed addresses of flood damage with estimated flooding depths. However, first floor structure elevations were not included or available. Consequently, ground elevations near the flooded structures were estimated from available LiDAR coverage of the area.

### 2.3.5 Validation Model Results

Results of the validation performed as part of this study are presented in **Table 2-2**, which compares the modeled peak stages to the stages reported by the stakeholders, as well as **Figures 2-5 through 2-10**, which compare the modeled flow and stage timeseries to the measured USGS data. It should be noted that several of the USGS gages experienced equipment failures due to the elevated flows experienced during the May 2009 event. The modeled peak stages compare well to the available data and including the stakeholder observations. Peak flows are close for the rated locations at LPGA and Halifax Canals. The flows at Reed Canal vary, but the City of South Daytona believes that the model-predicted higher flows are generally in the correct range.

Other than rainfall, the original hydrologic parameters were all maintained throughout the validation process. The only hydraulic parameters adjusted during the validation process were center channel Manning's roughness coefficients in some canal and stream reaches. Several links were also added to better represent connectivity and conveyance during the validation storm.

## 2.4 Model Design Storm Evaluation

Once the model was validated to the May 2009 event, the validated model was applied for a number of design storms. Design storms with two different durations (24-hour and 96-hour) were evaluated for frequencies ranging from the 2.33-year (mean annual) to the 100-year return periods. All of these combinations of storm frequency and duration were also evaluated for two different downstream boundary conditions, including the 1-year and 100-year stillwater elevation.

### 2.4.1 Rainfall Data

Surface water modeling was performed using 24-hour and 96-hour design storm distributions from the St. Johns River Water Management District (SJRWMD) (see **Figures 2-11 and 2-12**). The rainfall depths applied for the mean annual, 10-year, 25-year, and 100-year design storm events with 24-hour duration were determined to be 5.2 inches, 8.0 inches, 9.5 inches, and 13.0 inches, respectively. The depths applied for the mean annual, 10-year, 25-year, and 100-year events with 96-hour duration were determined to be 7.0 inches, 10.5 inches, 13.0 inches, and 16.5 inches, respectively. These rainfall depths were distributed evenly across the study area HUs.

## Table 2-2 Comparison of SWMM Output and Measured High Water Marks/Estimated Ground Elevations - May 2009 Storm Event

| Community | SWMM Node Location | Physical Location | SWMM Node(s) | Ground or High Water Elevation (ft NGVD) | SWMM Peak Water Elevation (ft NGVD) | Difference (ft ) | Comment |
|---|---|---|---|---|---|---|---|
| Ormond Beach | Most upstream node of Railroad Ditch | Hand Avenue | 1<br>6 | 6.8 - 8.6 | 7.1<br>7.1 | Varies | Estimated water elevations at homes on Hand Avenue and Sauls Street |
| | Most upstream node of Callegrande Ditch | Arroyo Parkway | 3 | 5.7 - 9.9 | 9.5 | Varies | Estimated water elevations at two homes at Arroyo Parkway |
| Holly Hill | LPGA Canal between Alta Road and railroad | Public Works offices | 8<br>1102 | 7.0 | 6.7<br>6.8 | -0.2 to -0.3 | |
| Daytona Beach | Off Nova Canal north of Reed Canal | Hawk Street and Hudson Street | 50 | 9.0 | 9.4 | 0.4 | |
| | Off Nova Canal north of Reed Canal | Cedar Street near tennis courts | 55 | 9.4 | 9.3 | 0.0 | |
| | Nova Canal north of Reed Canal | Sutton Place apartment complex | 5501 | 7.4 - 8.4 (parking lot) | 9.3 | Not applicable | News report said water was knee-high in parking lot |
| | Off Nova Canal north of Reed Canal | South Street and Willie Drive near B6 pond | 59 | 8.8 | 9.3 | 0.5 | |
| | Off Nova Canal north of Reed Canal | Sunland Road near Shady Place | 67 | 9.2 | 9.2 | 0.1 | |
| | Off Nova Canal north of Reed Canal | Golfview Drive and Columbine Avenue | 67 | 9.3 | 9.2 | -0.1 | |
| | Nova Canal north of Reed Canal | Woodcliff Drive and Nova Road | 8701 | 8.9 | 9.2 | 0.3 | |
| | Nova Canal north of Reed Canal | 0.2 miles north of Nova Road and Beville Road | 9801 | 9.0 | 9.2 | 0.2 | |
| South Daytona | Reed Canal downstream of US 1 | Reed Canal at mouth | 12102 (outfall) | 3.9 | 3.6 | -0.3 | Canal crest |
| | Reed Canal at Stevens Canal | Reed Canal at Stevens Canal | 128 | 5.5 | 5.9 | 0.4 | Canal crest |
| | Reed Canal at Stevens Canal | Lantern Drive | 128 | 5.9 | 5.9 | 0.0 | |
| | Reed Canal at Nova Canal | Reed Canal at Nova Canal | 13104 | 8.2 | 7.8 | -0.4 | Canal crest |
| | Nova Canal north of Reed Canal | Nova Canal just south of Beville Road | 10402 | 9.0 | 9.1 | 0.1 | Canal crest |
| | Stevens Canal | Green Street between Briar Lane and Millbrook Lane | 119<br>11901 | 5.7 | 6.5<br>6.1 | 0.3 to 0.8 | |
| | Stevens Canal | Intersection of Aspen Drive and Bennett Road | 101<br>10101 | 7.0 | 7.4<br>7.2 | 0.2 to 0.4 | |
| | Stevens Canal | Canal crest at Violet Street and Bennett Road | 101 | 7.7 | 7.4 | -0.3 | Canal crest |
| | Stevens Canal | Western Road and Violet Street | 101 | 8.3 | 7.4 | -0.9 | |
| Port Orange | Nova Canal south of Reed Canal | Sugar House Drive | 13301 | 7.5 - 8.4 (ground) | 8.3 | Not applicable | Water in two homes |
| | Off Nova Canal south of Reed Canal | Moonstone Court | 13901 | 7.4 - 8.4 (ground) | 8.2 | Not applicable | Up to 8 inches interior flood damage |
| | Off Nova Canal south of Reed Canal | Jackson Street between Moonstone Ct and Springwood Sq | 13901 | 8.4 (ground) | 8.2 | Not applicable | Not applicable |
| | Halifax Canal | Canalview Blvd between Jackson St and Ryanwood Dr | 1442<br>1381 | 7.6 (ground) | 8.2<br>8.1 | Not applicable | Up to 6 inches interior/exterior flooding |
| | Off Halifax Canal south of Canalview Blvd | Dame Street between Oak Street and Dianne Street | 149 | 7.9 - 8.4 (ground) | 8.1 | Not applicable | 1 - 2 inches interior, 4 -6 inches in garage |
| | Off Halifax Canal south of Canalview Blvd | Donna Street and Dianne Street | 149 | 8.4 (ground) | 8.1 | Not applicable | 4 inches interior/garage/living room |
| | Halifax Canal east of Ryanwood Dr | Eddy Lane | 2301HAL<br>138 | 5.5 - 6.4 (ground) | 6.8<br>6.9 | Not applicable | 1 inch water in home |
| | Halifax Canal east of Ryanwood Dr | Judges Lane | 2301HAL<br>138 | 6.4 (ground) | 6.8<br>6.9 | Not applicable | 1 -2 inches interior flooding |
| | Halifax Canal north of Powers Ave | Ruth Street south of Powers Avenue | 2301HAL | 6.4 (ground) | 6.8 | Not applicable | Not applicable<br>3 - 6 inches water in home |
| | Off Halifax Canal west of Nova Road | W Samms Ave west of Nova Road | 152 | 9.4 (ground) | 9.0 | Not applicable | Not applicable |
| Volusia County: No high water marks provided | | | | | | | |

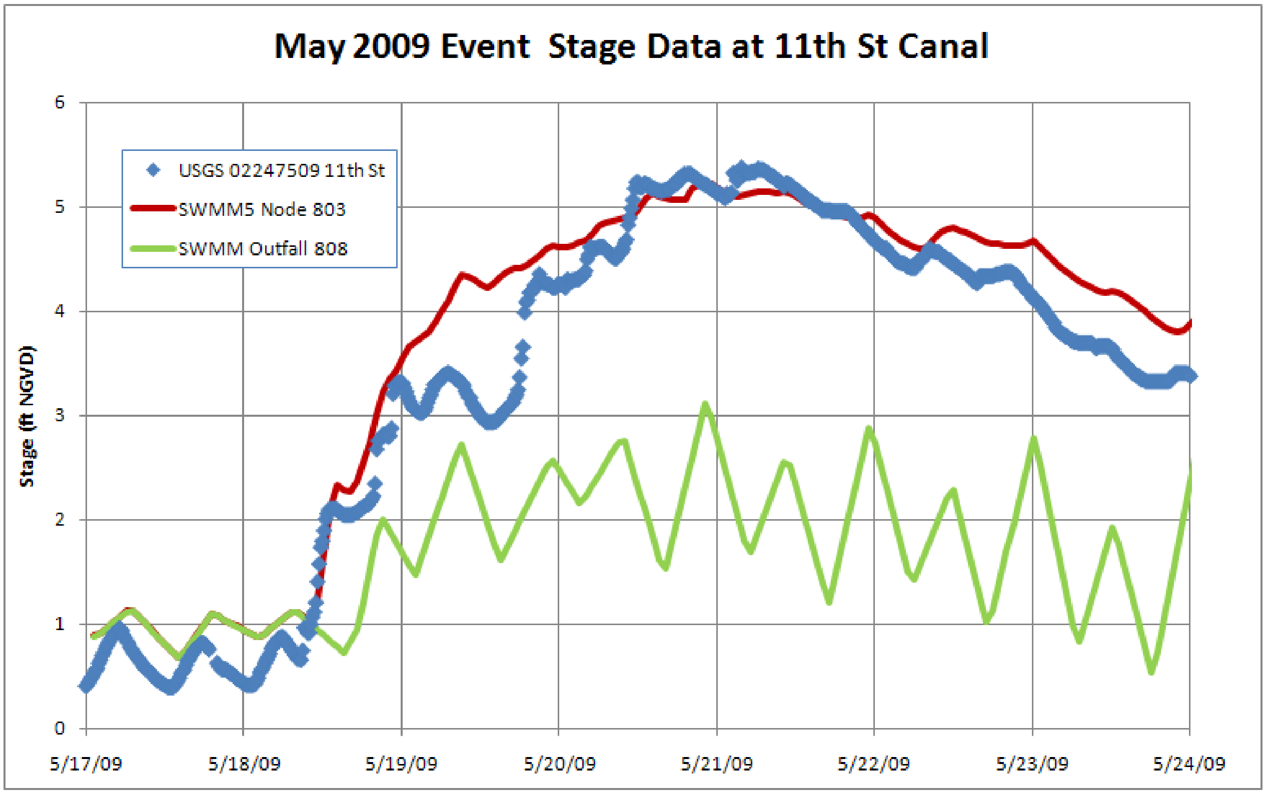


**Figure 2-5 LPGA (11th Street) Canal Stages**

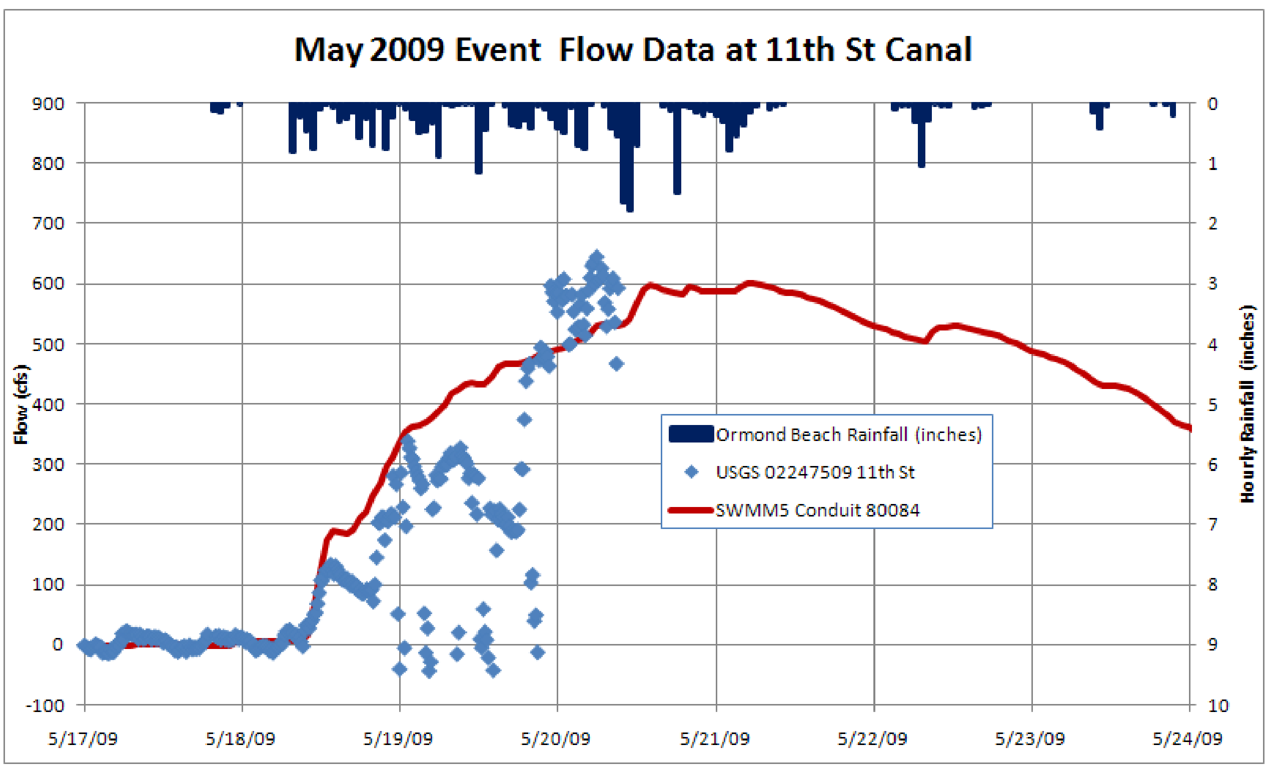


**Figure 2-6 LPGA (11th Street) Canal Flows**

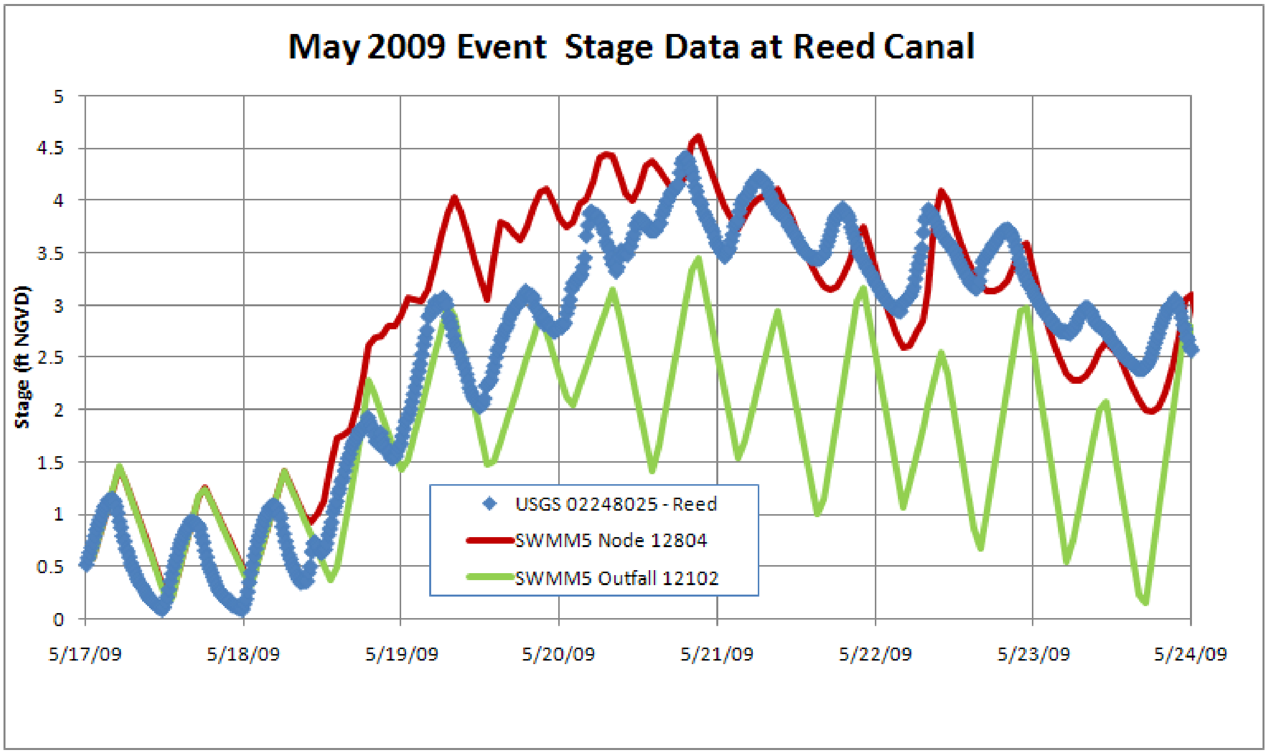


**Figure 2-7 Reed Canal Stages**

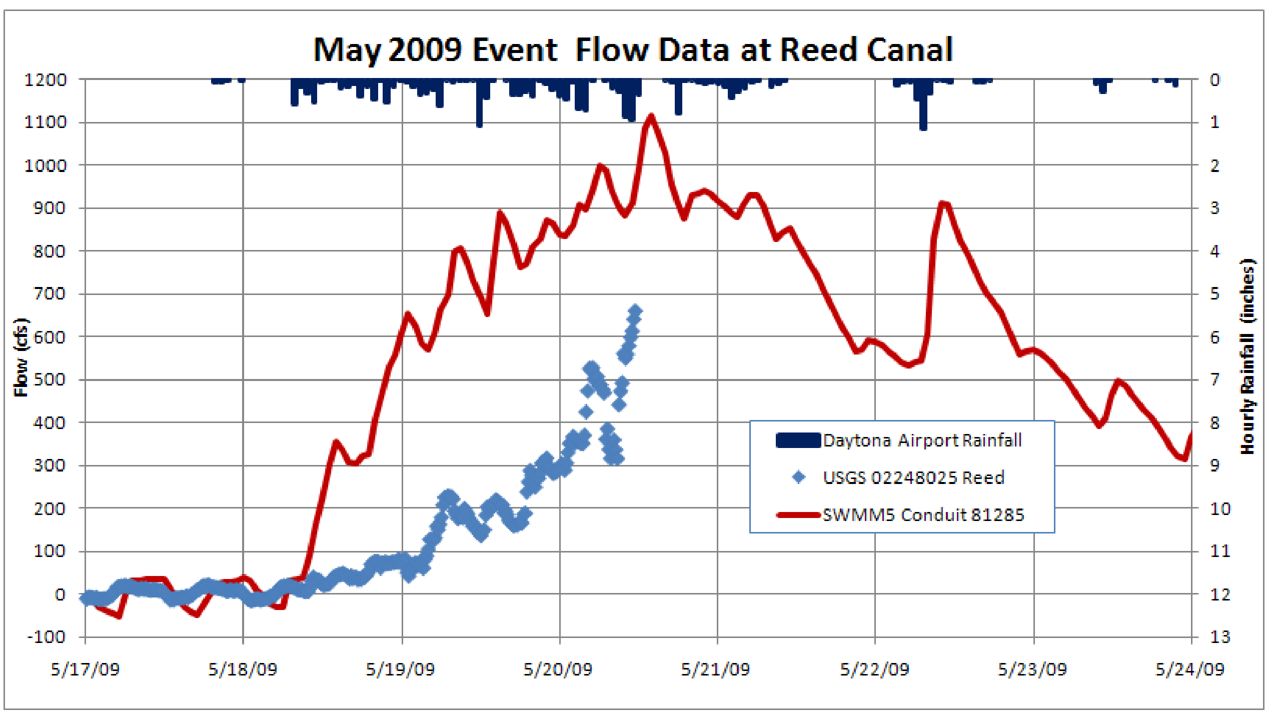


**Figure 2-8 Reed Canal Flows**

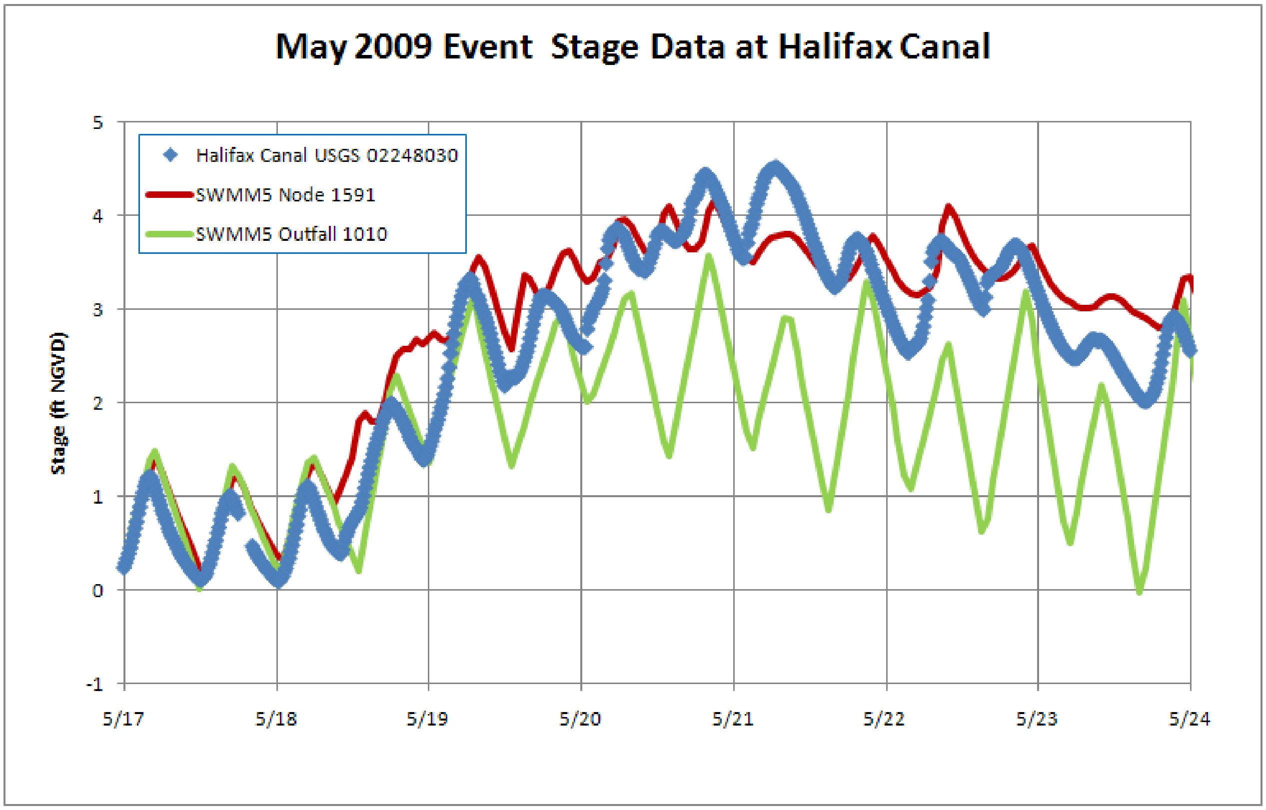


**Figure 2-9 Halifax Canal Stages**

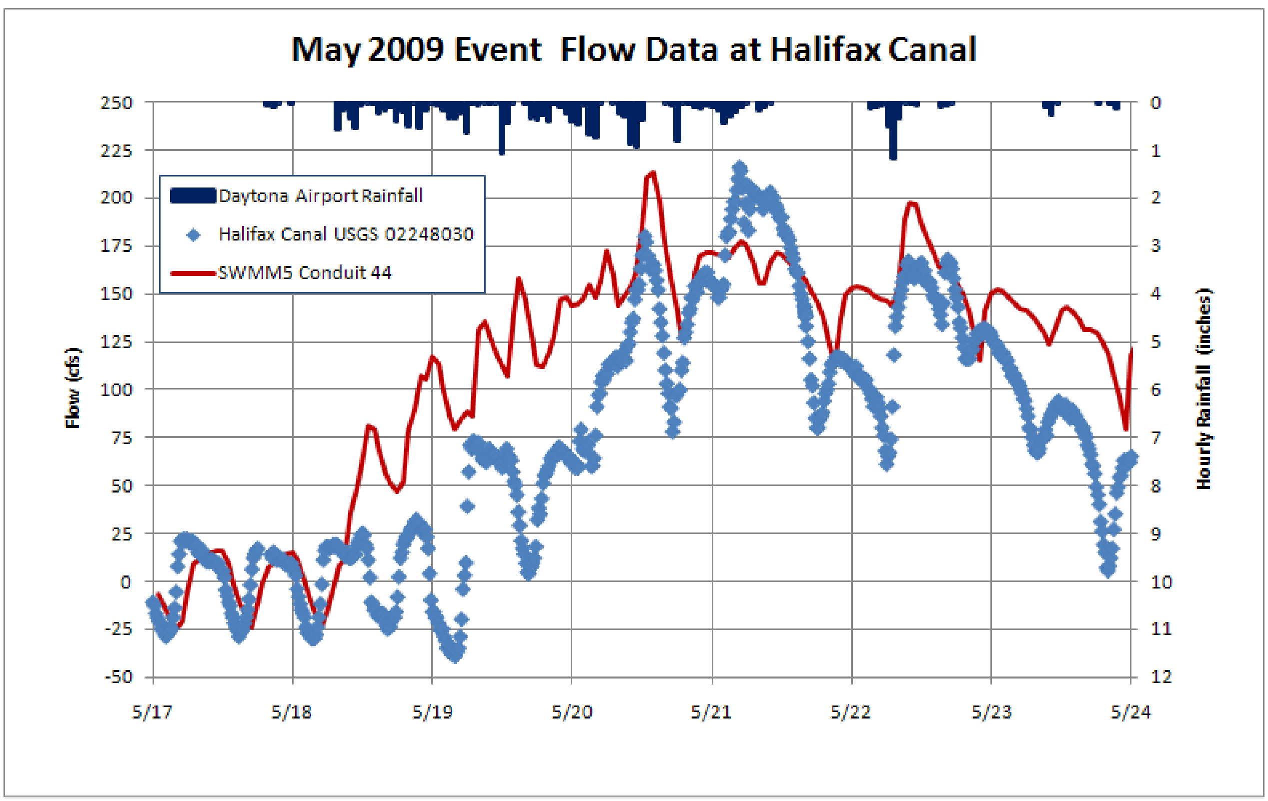


**Figure 2-10 Halifax Canal Flows**

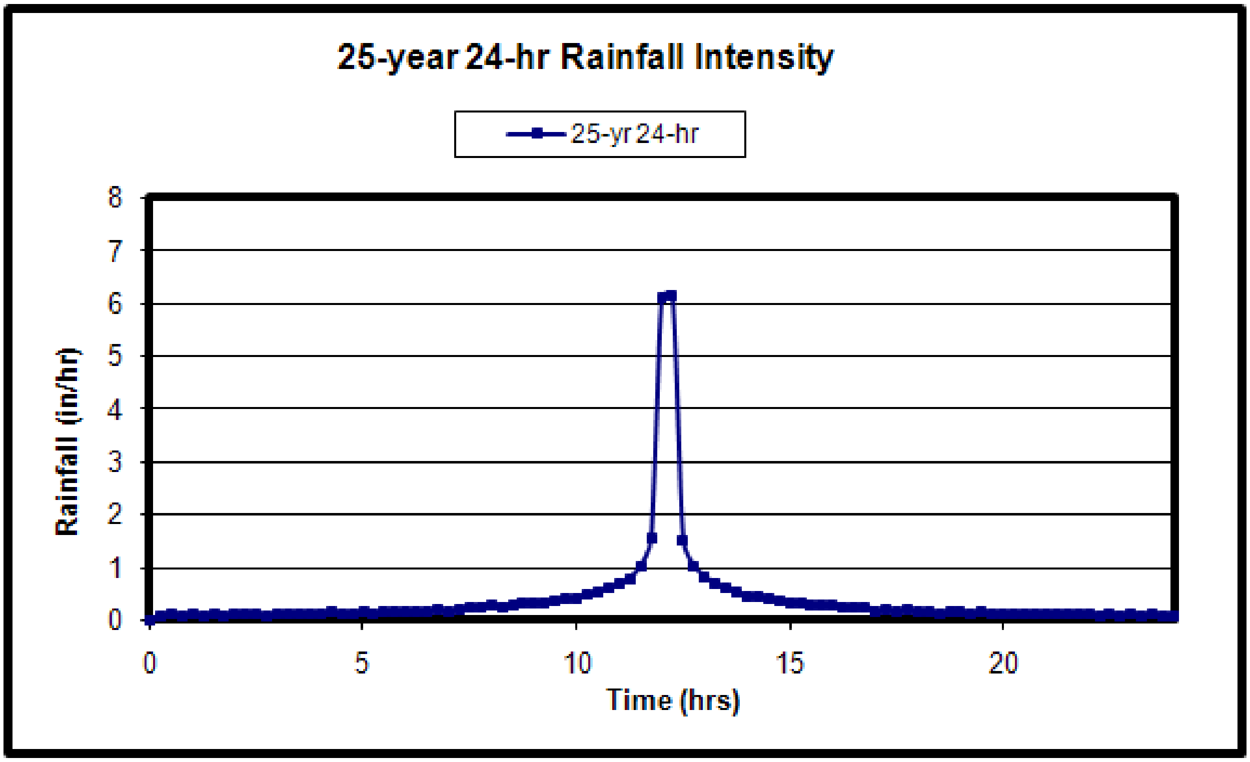


**Figure 2-11 SJRWMD 25-year, 24-hour Design Storm Distribution**

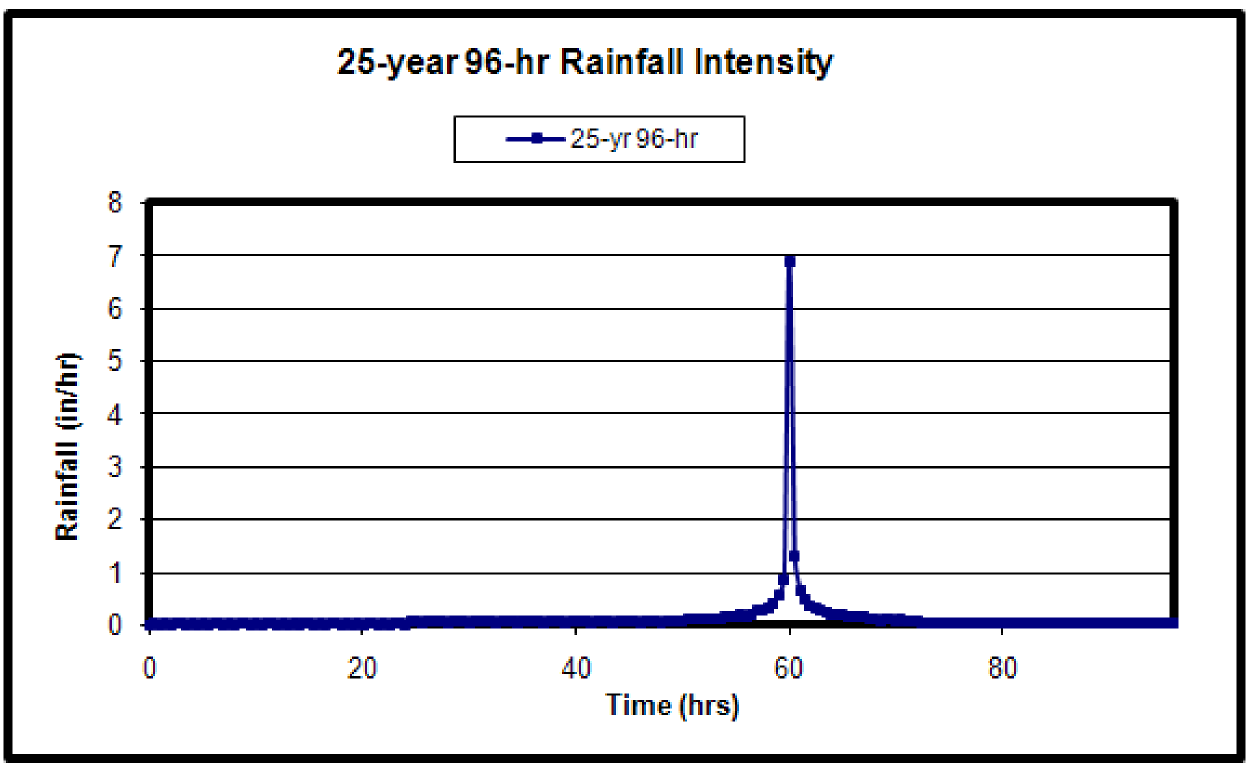


**Figure 2-12 SJRWMD 25-year, 96-hour Design Storm Distribution**

## 2.4.2 Downstream Boundary Conditions

Two boundary conditions were evaluated for design storm simulations: the 1-year and the 100-year stillwater elevations. According to FEMA, the stillwater elevation is the maximum storm-induced water-surface elevation, and is primarily a combination of the normal astronomic tide and the storm surge. The 10-, 50-, 100-, and 500-year stillwater elevations published in the 2003 FEMA Flood Insurance Study for Volusia County as shown in **Table 2-3** were used to develop the 1-year and 100-year boundary conditions.

**Table 2-3 FEMA Coastal Stillwater Elevations**

| Location | Stillwater Elevation[a] (ft NGVD) | | | | | |
|---|---|---|---|---|---|---|
| | 1-yr[b] | 10-yr | 25-yr[b] | 50-yr | 100-yr | 500-yr |
| Halifax/ICW from south side SR40 to just S of Daytona northern corp limit | 1.4 | 2.7 | 3.5 | 4.2 | 5.0 | 7.8 |
| Halifax/ICW from S of Daytona northern corp limits to Dunlawton BLVD | 2.2 | 3.7 | 4.5 | 5.2 | 6.0 | 8.8 |
| ICW from Dunlawton BLVD to North of New Smyrna | 3.0 | 4.7 | 5.5 | 6.2 | 7.0 | 9.8 |

[a] Source: FEMA Volusia County FIS, 2003
[b] Interpolated or Extrapolated Values

The 100-year tidal stillwater elevation was utilized as published. The 1-year stillwater elevation was then derived using a power equation regression of the published FEMA stillwater elevations. As shown on **Figure 2-13,** the regression yielded very good approximations ($R^2$ values of 0.99).

It should be noted that a 1-year stillwater elevation of 2.0 feet NGVD was utilized for the LPGA canal to provide a conservative analysis and evaluation of this area of the watershed. It is expected that a higher stillwater flood stage may occur for the northern portion of the study area based on the fact that the LPGA canal discharges to the Halifax River at the downstream end of the stillwater zonal boundary, near the northern corporate limit of Daytona Beach where the regression analysis shows the 1-year stillwater increasing from 1.4 feet to 2.2 feet NGVD. Based on the spatial location as well as the fact that the causeways at US 92 and Seabreeze have been removed, a slightly higher tidal stillwater boundary condition was used for the LPGA Canal.

## 2.4.3 Design Storm Model Results

Design storm model results are summarized in **Appendix B** and **Appendix C** of the report. Appendix B includes peak stage results and Appendix C includes peak flow results.

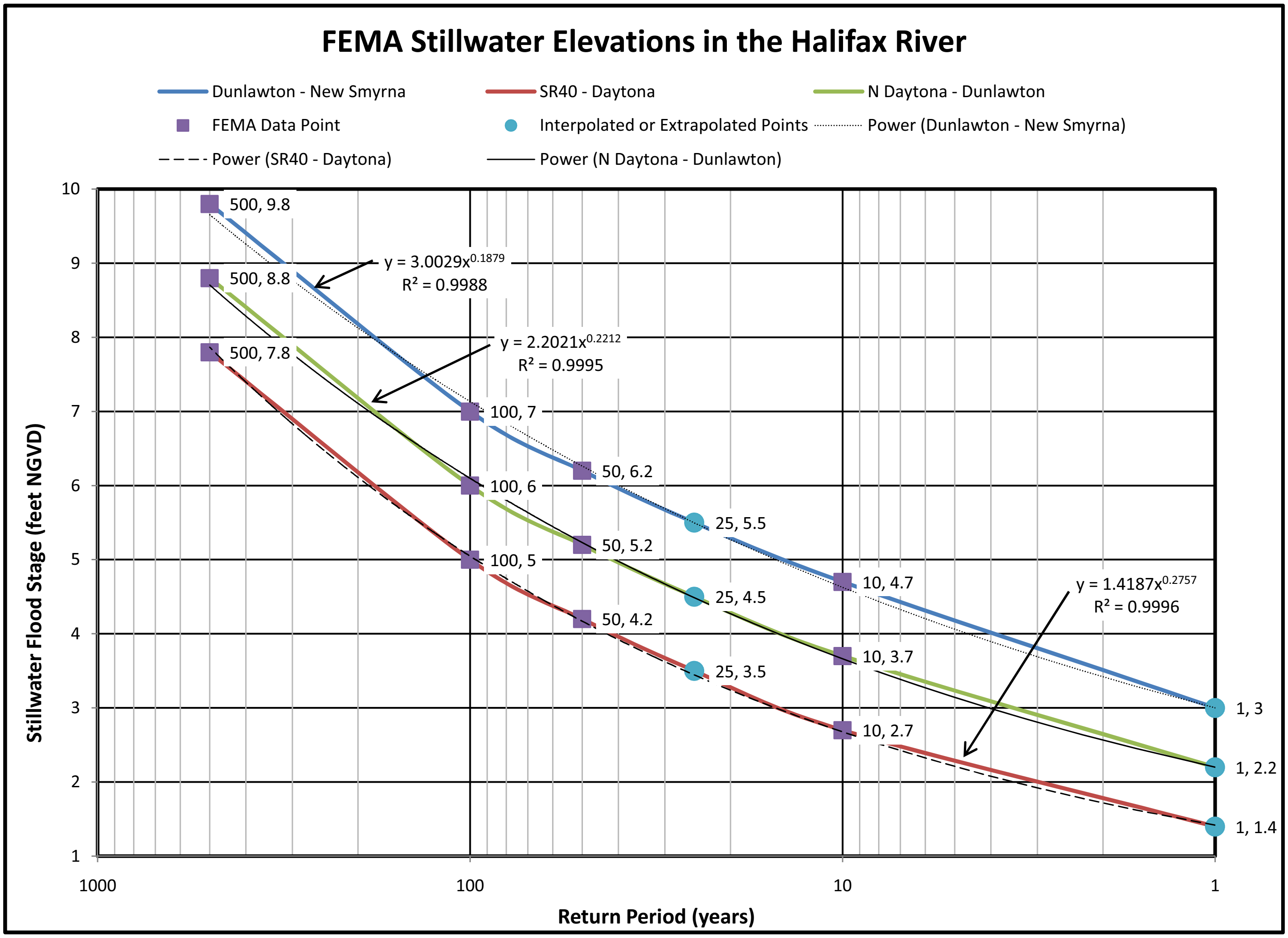


**Figure 2-13**
**FEMA Tidal Stillwater Elevations in the Halifax River**

In Appendix B, peak water stages at selected locations in the primary stormwater management system are tabulated. Separate tables are presented for the following areas:

- LPGA Canal
- LPGA Canal branches (Railroad Ditch Canal, Southeast Canal, Northwest Canal)
- Reed Canal (includes Stevens Canal)
- Halifax Canal
- Nova Canal (includes Nova Road locations between LPGA and Halifax Canals)

Each table presents peak stages for the mean annual, 10-year, 25-year and 100-year design storms for one of the areas, and either the 1-year or 100-year stillwater downstream boundary condition. Consequently, a total of 10 tables (5 areas, 2 downstream boundary conditions) are included.

The results for existing conditions are also summarized in **Tables 2-4 and 2-5**. Tables 2-4 and 2-5 show the "Level of Service" (LOS) under existing conditions for the 24-hour and 96-hour design storms, respectively. The LOS is based on comparison between modeled stages and the "indicator elevation," which is typically a roadway elevation, based on the model representation of road overflow or examination of LiDAR elevation data. At each location, the table lists "100-Yr" if the modeled stage is less than the indicator elevation for all design storms; otherwise, the value in the table shows the design storm at which LOS is not met (e.g., "< 25-Yr" means that the modeled peak stage is greater than the indicator elevation for the 25-yr design storm and more extreme events). Each of the tables shows LOS results for both the 1-year and 100-year stillwater downstream boundary conditions.

Review of the values in Table 2-4 shows that the indicator elevations are exceeded for the 10-year, 24-hour design storm with 1-year stillwater boundary (and in some cases the mean annual design storm) at a number of locations. These include most of the canal branches off of the LPGA Canal (Railroad Ditch Canal, Southeast Canal, and Northwest Canal), the upstream end of Stevens Canal, and the Nova Canal system at Madison Avenue and between West International Speedway Boulevard and Woodcliff Drive. Moonstone Court (east of Nova Road) has a particularly low road elevation that is exceeded even for the mean annual event.

**Table 2-4 Nova Canal Flood Control and Integrated Water Resources Project Baseline Conditions – 24-hour Design Storm Level of Service**

| Canal System | Location | Level of Service | |
|---|---|---|---|
| | | 1-Yr Stillwater | 100-Yr Stillwater |
| LPGA Canal | Upstream of Riverside Dr | 100-Yr | < MA |
| | 11th St Canal at Railroad Ditch and Southeast Canal | 100-Yr | < 100-Yr |
| LPGA Canal Branches | Railroad Ditch at Walker | < MA | < MA |
| | Railroad Ditch at Flomich | < 10-Yr | < 10-Yr |
| | Railroad Ditch headwater (Hand Ave) | < 10-Yr | < MA |
| | Southeast Canal at 10th St | < MA | < MA |
| | Southeast Canal at 8th St | < 10-Yr | < 10-Yr |
| | Southeast Canal at 6th St | < 10-Yr | < 10-Yr |
| | Southeast Canal at 3rd St | < 10-Yr | < 10-Yr |
| | Southeast Canal at Mason | < 100-Yr | < 100-Yr |
| | Northwest Canal at 13th St | < 100-Yr | < 100-Yr |
| | Northwest Canal at 15th St | < 10-Yr | < 10-Yr |
| | Northwest Canal at Flomich | < 10-Yr | < 10-Yr |
| | Northwest Canal upstream of Alabama; Calle Grande Ditch headwater (Arroyo Blvd) | < 10-Yr | < 10-Yr |
| Reed Canal | Reed Canal at Nova Road | 100-Yr | < 100-Yr |
| | Downstream end of Stevens Canal | < 10-Yr | < MA |
| | Stevens Canal at Ridge Drive | < 25-Yr | < 10-Yr |
| | Stevens Canal at Big Tree Road | 100-Yr | < 100-Yr |
| | Upstream end of Stevens Canal | < 10-Yr | < MA |
| Halifax Canal | Near Jackson Street | < 100-Yr | < MA |
| | Oak Street | < 25-Yr | < MA |
| | Powers Avenue | < 25-Yr | < MA |
| | Ryanwood Ave | < 100-Yr | < 25-Yr |
| Nova Canal | 10th Street | < 100-Yr | < 25-Yr |
| | Madison Avenue | < 10-Yr | < 10-Yr |
| | George W. Engram Boulevard | < 100-Yr | < 100-Yr |
| | West International Speedway Boulevard | < 10-Yr | < 10-Yr |
| | Museum Boulevard/Navy Canal | < 10-Yr | < 10-Yr |
| | Bellevue Avenue | < 10-Yr | < 10-Yr |
| | Woodcliff Drive | < 10-Yr | < 10-Yr |
| | Beville Road | < 100-Yr | < 100-Yr |
| | Big Tree Road | < 100-Yr | < 100-Yr |
| | Moonstone Court | < MA | < MA |
| | Madeline Avenue | < 100-Yr | < 100-Yr |
| Note: MA = Mean Annual | | | |

### Table 2-5 Nova Canal Flood Control and Integrated Water Resources Project Baseline Conditions – 96-hour Design Storm Level of Service

| Canal System | Location | Level of Service | |
|---|---|---|---|
| | | 1-Yr Stillwater | 100-Yr Stillwater |
| LPGA Canal | Upstream of Riverside Dr | 100-Yr | < MA |
| | 11th St Canal at Railroad Ditch and Southeast Canal | 100-Yr | < 25-Yr |
| LPGA Canal Branches | Railroad Ditch at Walker | < MA | < MA |
| | Railroad Ditch at Flomich | < 10-Yr | < 10-Yr |
| | Railroad Ditch headwater (Hand Ave) | < MA | < MA |
| | Southeast Canal at 10th St | < MA | < MA |
| | Southeast Canal at 8th St | < 10-Yr | < 10-Yr |
| | Southeast Canal at 6th St | < 10-Yr | < 10-Yr |
| | Southeast Canal at 3rd St | < 10-Yr | < 10-Yr |
| | Southeast Canal at Mason | < 100-Yr | < 100-Yr |
| | Northwest Canal at 13th St | < 25-Yr | < 25-Yr |
| | Northwest Canal at 15th St | < 10-Yr | < 10-Yr |
| | Northwest Canal at Flomich | < 10-Yr | < 10-Yr |
| | Northwest Canal upstream of Alabama; Calle Grande Ditch headwater (Arroyo Blvd) | < 10-Yr | < 10-Yr |
| Reed Canal | Reed Canal at Nova Road | 100-Yr | < 100-Yr |
| | Downstream end of Stevens Canal | < MA | < MA |
| | Stevens Canal at Ridge Drive | < 25-Yr | < 10-Yr |
| | Stevens Canal at Big Tree Road | 100-Yr | < 100-Yr |
| | Upstream end of Stevens Canal | < 10-Yr | < MA |
| Halifax Canal | Near Jackson Street | < 25-Yr | < MA |
| | Oak Street | < 25-Yr | < MA |
| | Powers Avenue | < 10-Yr | < MA |
| | Ryanwood Ave | < 100-Yr | < 25-Yr |
| Nova Canal | 10th Street | < 25-Yr | < 25-Yr |
| | Madison Avenue | < 10-Yr | < 10-Yr |
| | George W. Engram Boulevard | < 100-Yr | < 100-Yr |
| | West International Speedway Boulevard | < 10-Yr | < 10-Yr |
| | Museum Boulevard/Navy Canal | < 10-Yr | < 10-Yr |
| | Bellevue Avenue | < 10-Yr | < 10-Yr |
| | Woodcliff Drive | < 10-Yr | < 10-Yr |
| | Beville Road | < 100-Yr | < 25-Yr |
| | Big Tree Road | < 100-Yr | < 25-Yr |
| | Moonstone Court | < MA | < MA |
| | Madeline Avenue | 100-Yr | < 100-Yr |
| Note: MA = Mean Annual | | | |

Comparison of the results in Table 2-4 for the 1-year and 100-year stillwater boundary elevations shows that the LOS is lower in a number of areas for the 100-year stillwater boundary. These include the LPGA Canal at and below the confluence with the Railroad Ditch Canal and Southeast Canal, the Railroad Ditch headwater at Hand Avenue, all locations at the Reed Canal and Halifax Canal, and the Nova canal system at 10th Street. In some cases (e.g., LPGA Canal at Riverside Drive, Halifax Canal at Oak Street), the 100-year stillwater elevation is actually greater than the indicator elevation.

A comparison of the results for the 24-hour design storms (Table 2-4) and 96-hour design storms (Table 2-5) shows that the LOS is in most cases the same for the two design storms. If there is a difference, the LOS is generally lower for the 96-hour event. Lower LOS for the 96-hour event occurs at one or more locations in the LPGA Canal, Railroad Ditch, Northwest Canal, Halifax Canal and Nova Canal.

Flow data for the various design storms and stillwater boundary conditions are presented in Appendix C. These are presented primarily for informational purposes and are not discussed here. Where appropriate, changes in flow with the project alternative will be discussed in Section 2.5.3.

## 2.5 Model Project Alternative Evaluation

A project alternative was evaluated which includes a combination tide gate and pump station at each of the three major outfall canals. This is a “pump only” option as opposed to a pump and force main in parallel to increase flow rate above gravity capacity. The general concept of each project is to reduce the peak stages and duration of flooding by preventing tidal backflows into the conveyance system, pumping down the system before or at the start of major storm events, and increasing capacity of the system during high tailwater conditions but limit gravity flow out. Operating rules and appropriate pump capacities were evaluated initially based on the May 2009 rainfall and tidal conditions. The project was then evaluated for the various design storm durations and return periods, for both 1-year and 100-year stillwater downstream tidal boundary.

### 2.5.1 Project Components

For each of the three major outfall canals, a project was defined as part of the overall project alternative. Each canal project was represented in the SWMM5 with the following elements:

- Tide weir-gate. The tide weir-gate was defined as a rectangular structure located in the canal. Based on stakeholder discussion, the tide weir-gate is expected to typically operate in the open position. The weir-gate is then closed under specified design conditions. The weir-gates would either inflate or rotate from the bottom of the canal to allow a range of tidal control along with gravity outflow.

- Diversion channel. This channel conveys water from the upstream side of the weir-gate to the pond at the project site.

- Pond at project site. A pond is included at the project site. This pond represents the wet well from which water is pumped.

- Pump station. The pump station is located at the project site, and will pump water from the proposed ponds to downstream of the gate.

- Emergency spillway. An emergency spillway is provided to promote additional flow conveyance when the pump is inoperable or not sufficient to pass peak flows for extreme events.

Per the Scope of Services, evaluations focused on these improvements, and the project team also identified potential bottleneck culverts in the canal system.

Each pump station was sized based on the peak flow through the site during the May 2009 event. While it is understood that pumping at an even greater rate (than the May 2009 peak flow) would potentially provide greater upstream benefits, it could also result in downstream impacts.

The operation of the tide weir-gate was evaluated to determine appropriate control rules. For all three canal projects, the weir-gate operation specified that the weir-gate would go fully closed any time that the water stage immediately upstream of the tide gate was just below the 1-year stillwater tidal boundary elevation. This could occur due to high downstream boundary stages and/or a relatively large storm event under the normal range of boundary stages.

The model was run to evaluate the benefits of re-opening the weir-gates during and after an extreme storm event to enhance the capability of the system to pass peak storm flows. The analysis suggested that re-opening the weir-gates would provide little benefit for several reasons. One is that the pumped flow is introduced back into the canal just downstream of the weir-gate, and under an open weir-gate condition, could actually flow back through the weir-gate and to the pump station (recirculate). The inclusion of the emergency spillway also limited any benefit that the weir-gate re-opening would achieve. Therefore, the weir-gate was not re-opened in the model runs. In actuality, it is expected that the weir-gates would re-open when the downstream boundary stage reaches a normal low water level (see **Table 2-6** for critical operational parameters).

**Table 2-6 Critical Operational Parameters for Pump / Tide Weir-Gate**

| Parameter | LPGA Canal Pump Station | Reed Canal Pump Station | Halifax Canal Pump Station |
|---|---|---|---|
| Pump Size (cfs) | 600 | 1125 | 300 |
| 1-yr Stillwater Elevation (ft-NGVD) | 2.0 | 2.2 | 3.0 |
| Pump-on Elevation (ft-NGVD) | 2.0 | 2.2 | 3.0 |
| Pump-off Elevation (ft-NGVD) | -2.0 | -2.0 | -1.0 |
| Emergency Spillway Elevation (ft-NGVD) | 4.5 | 4.7 | 5.5 |
| Emergency Spillway Length (ft) | 40.0 | 40.0 | 40.0 |
| Gate Close Elevation (ft-NGVD) | 2.0 | 2.2 | 3.0 |
| Gate D/S Re-Open Elevation (ft-NGVD) | 0.0 | 0.0 | 0.0 |

The pump station was modeled representing a single pump that would turn on at a specified water elevation in the pond at the project site, and then shut off at another specified water elevation. The “pump on” elevation generally coincides with the gate closure water elevation, and “pump off” elevation is several feet higher than the bottom of the pond on the project site. In actuality, the pump station will have multiple pumps that may turn on and off at staggered elevations. The difference in the actual operational protocol of the pumps is not expected to significantly change the model results. The pump capacity at each project location was chosen based on simulation results for the May 2009 storm event.

The emergency spillway is represented as a weir in the model. At all three projects, the spillway crest was set at an elevation of 2.5 feet higher than the 1-year stillwater elevation / gate closure / “pump on” elevation. This elevation was selected based on several model iterations evaluating different crest elevation options.

## 2.5.2 Results for May 2009 Event

The project alternative was initially evaluated using the historical May 2009 storm event and tidal boundary conditions. The existing conditions model was modified to incorporate the weir-gates, diversion channels, project site ponds, pumps, and emergency spillways discussed earlier. Thus, all three projects (LPGA canal, Reed Canal, and Halifax Canal) are all represented in a single model.

The summary results for the May 2009 event, with and without the project alternative, are presented in **Tables 2-7 through 2-11**. Each table represents a separate part of the overall study area primary stormwater management system. In each table, “indicator elevation” values are listed at selected model nodes, along with the peak stages with and without the project, and the changes in peak water elevations.

Table 2-7 shows results for the LPGA Canal main stem. The table indicates that under these storm conditions, the project alternative provided minimal benefit to the main stem. This is in part due to the fact that the pumping rate does not exceed the existing peak flow for this event and that downstream improvements are not in place to allow greater flow to the Halifax River. Also, stormwater is being pumped to the downstream side of the weir-gate but may move back upstream over the emergency spillway.

**Table 2-7 Existing and Alternative 1 - Peak Stages for May 2009 Storm LPGA Canal System**

| *May 2009 Storm Event- LPGA Canal System* | | | | | |
|---|---|---|---|---|---|
| | SWMM5 | Indicator Elevation | Existing Peak Stage | Alternative 1 | |
| Location | Node | (ft NGVD) | (ft-NGVD) | Peak Stage (ft-NGVD) | Change (ft) |
| 11th St Canal Outfall | 808 | - | 3.2 | 3.2 | 0.0 |
| Upstream of Riverside Dr | 806 | 4.6 | 3.6 | 3.6 | 0.0 |
| Upstream of Daytona Road | 804 | 12.3 | 4.5 | 4.5 | 0.0 |
| Upstream of US 1 (Ridgewood) | 802 | 11.7 | 5.2 | 5.2 | 0.0 |
| 11th St Canal at Railroad Ditch and Southeast Canal | 8 | 6.7 | 6.7 | 6.6 | 0.0 |
| Upstream of Alta Drive | 1101 | 8.7 | 7.8 | 7.7 | 0.0 |
| Upstream of Center Avenue | 1402 | 8.6 | 8.1 | 8.1 | 0.0 |
| 11th St Canal at Nova Road | 1401 | 10.9 | 8.7 | 8.7 | 0.0 |

Results for the LPGA Canal branches (Railroad Ditch Canal, Southeast Canal, and Northwest Canal) are presented in Table 2-8. Both the Southeast and Railroad Ditch Canal branches show some benefit as peak stages are reduced by 0.1 to 0.2 foot. Benefits are less than 0.1 foot for the Northwest Canal.

**Table 2-8 Existing and Alternative 1 – Peak Stages for 2009 Storn LPGA Canal Branch Systems**

| *May 2009 Storm Event- LPGA Canal Branch Systems* | | | | | |
|---|---|---|---|---|---|
| | SWMM5 | Indicator Elevation | Existing Peak Stage | Alternative 1 | |
| Location | Node | (ft NGVD) | (ft-NGVD) | Peak Stage (ft-NGVD) | Change (ft) |
| Railroad Ditch at Walker | 604 | 5.2 | 7.0 | 6.9 | -0.1 |
| Railroad Ditch at Flomich | 6 | 5.9 | 7.1 | 7.1 | -0.1 |
| Railroad Ditch headwater (Hand Ave) | 1 | 5.4 | 7.1 | 6.9 | -0.2 |
| Southeast Canal at 10th St | 1301 | 5.5 | 7.1 | 7.0 | -0.1 |
| Southeast Canal at 8th St | 1603 | 6.5 | 7.4 | 7.3 | -0.1 |
| Southeast Canal at 6th St | 1601 | 6.4 | 7.4 | 7.3 | -0.1 |
| Southeast Canal at 3rd St | 2003 | 6.7 | 7.4 | 7.3 | -0.1 |
| Southeast Canal at Mason | 26 | 7.5 | 7.4 | 7.3 | -0.1 |
| Northwest Canal at 13th St | 9 | 8.4 | 9.1 | 9.1 | 0.0 |
| Northwest Canal at 15th St | 402 | 7.6 | 9.3 | 9.3 | 0.0 |
| Northwest Canal at Flomich | 4 | 7.9 | 9.4 | 9.3 | 0.0 |
| Northwest Canal upstream of Alabama; Calle Grande Ditch headwater (Arroyo Blvd) | 3 | 7.5 | 9.5 | 9.48 | 0.0 |

Table 2-9 lists the results for the Reed Canal and Stevens Canal branch off of Reed Canal. For the Reed Canal main stem, peak stage reductions range from 0.2 foot to 1.7 feet, with maximum benefit immediately upstream of the gate. On the Stevens Canal branch, the reduction in peak stage ranges from 0.1 to 0.4 foot.

**Table 2-9 Existing and Alternative 1 - Peak Stages for May 2009 Storm Reed Canal System**

| *May 2009 Storm Event- Reed Canal System* | | | | | |
|---|---|---|---|---|---|
| | SWMM5 | Indicator Elevation | Existing Peak Stage | Alternative 1 | |
| Location | Node | (ft NGVD) | (ft-NGVD) | Peak Stage (ft-NGVD) | Change (ft) |
| Reed Canal Outfall | 12102 | - | 3.6 | 3.6 | 0.0 |
| Upstream of US 1 | 12101 | 7.5 | 3.9 | 4.0 | 0.1 |
| Proposed Pond | REED_SITE | -- | -- | 2.2 | -- |
| Downstream of RR / Upstream of Proposed Gate | 12804 | 11.5 | 4.7 | 3.1 | -1.7 |
| Reed Canal at Stevens Canal | 128 | 8.8 | 5.9 | 5.2 | -0.6 |
| Saul Drive | 122 | 9.6 | 6.6 | 6.3 | -0.4 |
| Reed Canal at Nova Road | 13104 | 9.2 | 7.8 | 7.6 | -0.2 |
| Stevens Canal at Ridge Drive | 11001 | 6.9 | 6.5 | 6.2 | -0.4 |
| Stevens Canal at Big Tree Road | 10101 | 8.6 | 7.2 | 7.0 | -0.2 |
| Upstream end of Stevens Canal | 101 | 6.6 | 7.4 | 7.3 | -0.1 |

The Halifax Canal results are presented in Table 2-10. The results show peak stage reduction at almost all locations, ranging from 0.3 foot at Nova Road (near the project site) to no change at several locations (e.g., Powers Avenue).

**Table 2-10 Existing and Alternative 1 - Peak Stages for May 2009 Storm Halifax Canal System**

| *May 2009 Storm Event- Halifax Canal System* | | | | | |
|---|---|---|---|---|---|
| | SWMM5 | Indicator Elevation | Existing Peak Stage | Alternative 1 | |
| Location | Node | (ft NGVD) | (ft-NGVD) | Peak Stage (ft-NGVD) | Change (ft) |
| Outfall | 1010 | - | 3.7 | 3.7 | 0.0 |
| Pump Station Site | HAL_SITE | -- | -- | 3.0 | -- |
| Nova Road d/s | 1591 | 11 | 3.8 | 3.5 | -0.3 |
| Nova Road u/s | 159 | 11 | 3.8 | 3.6 | -0.2 |
| Near Jackson Street | 150 | 5.3 | 5.4 | 5.3 | -0.1 |
| Oak Street | 141 | 6 | 6.3 | 6.3 | 0.0 |
| Powers Avenue | 2301HAL | 6.6 | 6.8 | 6.8 | 0.0 |
| Ryanwood Ave | 1381 | 8.5 | 8.1 | 8.0 | -0.1 |
| Nova Road | 132 | 9.1 | 8.2 | 8.1 | 0.0 |

Table 2-11 shows results for the Nova Canal system. There is no peak stage benefit from Navy Canal northward, but there is some peak stage reduction south of Navy Canal. In that area, the benefit is generally 0.1 to 0.2 foot.

**Table 2-11 Existing and Alternative 1 - Peak Stages for May 2009 Storm Nova Canal System**

| *May 2009 Storm Event- Nova Canal System* | | | | | |
|---|---|---|---|---|---|
| | SWMM5 | Indicator Elevation | Existing Peak Stage | Alternative 1 | |
| Location | Node | (ft NGVD) | (ft-NGVD) | Peak Stage (ft-NGVD) | Change (ft) |
| 10th Street | 14 | 8.3 | 8.8 | 8.7 | 0.0 |
| 3rd Street | 1901 | 10.8 | -- | 9.1 | -- |
| Mason Avenue | 3203 | 10.9 | 9.3 | 9.2 | 0.0 |
| Madison Avenue | 3201 | 8.7 | 9.3 | 9.3 | 0.0 |
| George W. Engram Boulevard | 3501 | 9.5 | 9.3 | 9.3 | 0.0 |
| West International Speedway Boulevard | 4601 | 8.9 | 9.3 | 9.3 | 0.0 |
| Sutton Place Apartment Complex | 5501 | * | 9.3 | 9.3 | 0.0 |
| Museum Boulevard/Navy Canal | 6102 | 8.4 | 9.3 | 9.3 | 0.0 |
| Bellevue Avenue | 7201 | 8.4 | 9.3 | 9.3 | 0.0 |
| Woodcliff Drive | 8701 | 8.4 | 9.2 | 9.2 | 0.0 |
| Beville Road | 10401 | 9.4 | 9.1 | 9.1 | 0.0 |
| Big Tree Road | 11601 | 9.4 | 8.9 | 8.9 | -0.1 |
| Reed Canal Road | 12401 | 9.4 | 7.8 | 7.7 | -0.2 |
| Moonstone Court | 13901 | 6.5 | 8.2 | 8.1 | -0.1 |
| Madeline Avenue | 12901 | 9.4 | 8.3 | 8.2 | -0.1 |

## 2.5.3 Results for Design Storms

As mentioned earlier, design storm model results are summarized in Appendix B and Appendix C of the report. Appendix B includes peak stage results and Appendix C includes peak flow results. The tables in these appendices show the model results with and without the project alternative, and tabulate differences in peak stage and flow.

The results for existing conditions are also summarized in **Tables 2-12 and 2-13**. Tables 2-12 and 2-13 compare the "Level of Service" with and without the project, for the 24-hour and 96-hour design storms, respectively. The LOS is based on comparison between modeled stages and the "indicator elevation," which is typically a roadway elevation, based on the model representation of road overflow or examination of LiDAR elevation data. Each of the tables shows LOS results for both the 1-year and 100-year stillwater downstream boundary conditions. Locations at which the LOS is higher with the project alternative have been highlighted in the table.

## Table 2-12 Nova Canal Flood Control and Integrated Water Resources Project Baseline and Alternative 1 Conditions – 24-hour Design Storms Level of Service

| Canal System | Location | 1-Year Stillwater | | 100-Year Stillwater | |
|---|---|---|---|---|---|
| | | Existing | Alternative 1 | Existing | Alternative 1 |
| LPGA Canal | Upstream of Riverside Dr | 100-Yr | 100-Yr | < MA | < MA |
| | 11th St Canal at Railroad Ditch and Southeast Canal | 100-Yr | 100-Yr | < 100-Yr | < 100-Yr |
| LPGA Canal Branches | Railroad Ditch at Walker | < MA | < 10-Yr | < MA | < MA |
| | Railroad Ditch at Flomich | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Railroad Ditch headwater (Hand Ave) | < 10-Yr | < 10-Yr | < MA | < MA |
| | Southeast Canal at 10th St | < MA | < 10-Yr | < MA | < MA |
| | Southeast Canal at 8th St | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at 6th St | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at 3rd St | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at Mason | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | Northwest Canal at 13th St | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | Northwest Canal at 15th St | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Northwest Canal at Flomich | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Northwest Canal upstream of Alabama; Calle Grande Ditch headwater (Arroyo Blvd) | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| Reed Canal | Reed Canal at Nova Road | 100-Yr | 100-Yr | < 100-Yr | 100-Yr |
| | Downstream end of Stevens Canal | < 10-Yr | < 10-Yr | < MA | < 10-Yr |
| | Stevens Canal at Ridge Drive | < 25-Yr | < 100-Yr | < 10-Yr | < 10-Yr |
| | Stevens Canal at Big Tree Road | 100-Yr | 100-Yr | < 100-Yr | 100-Yr |
| | Upstream end of Stevens Canal | < 10-Yr | < 10-Yr | < MA | < 10-Yr |
| Halifax Canal | Near Jackson Street | < 100-Yr | < 100-Yr | < MA | < MA |
| | Oak Street | < 25-Yr | < 100-Yr | < MA | < MA |
| | Powers Avenue | < 25-Yr | < 25-Yr | < MA | < MA |
| | Ryanwood Ave | < 100-Yr | < 100-Yr | < 25-Yr | < 100-Yr |
| Nova Canal | 10th Street | < 100-Yr | < 100-Yr | < 25-Yr | < 25-Yr |
| | Madison Avenue | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | George W. Engram Boulevard | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | West International Speedway Boulevard | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Museum Boulevard/Navy Canal | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Bellevue Avenue | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Woodcliff Drive | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Beville Road | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | Big Tree Road | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | Moonstone Court | < MA | < MA | < MA | < MA |
| | Madeline Avenue | < 100-Yr | 100-Yr | < 100-Yr | < 100-Yr |

Note: Highlighted cell represents nodes with improved LOS for Project Alternative 1 conditions

**Table 2-13 Nova Canal Flood Control and Integrated Water Resources Project Baseline and Alternative 1 Conditions – 96-hour Design Storms Level of Service**

| Canal System | Location | 1-Year Stillwater | | 100-Year Stillwater | |
|---|---|---|---|---|---|
| | | Existing | Alternative 1 | Existing | Alternative 1 |
| LPGA Canal | Upstream of Riverside Dr | 100-Yr | 100-Yr | < MA | < MA |
| | 11th St Canal at Railroad Ditch and Southeast Canal | 100-Yr | 100-Yr | < 25-Yr | < 100-Yr |
| LPGA Canal Branches | Railroad Ditch at Walker | < MA | < 10-Yr | < MA | < MA |
| | Railroad Ditch at Flomich | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Railroad Ditch headwater (Hand Ave) | < MA | < 10-Yr | < MA | < MA |
| | Southeast Canal at 10th St | < MA | < 10-Yr | < MA | < MA |
| | Southeast Canal at 8th St | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at 6th St | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at 3rd St | < 10-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Southeast Canal at Mason | < 100-Yr | 100-Yr | < 100-Yr | < 100-Yr |
| | Northwest Canal at 13th St | < 25-Yr | < 25-Yr | < 25-Yr | < 25-Yr |
| | Northwest Canal at 15th St | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Northwest Canal at Flomich | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Northwest Canal upstream of Alabama; Calle Grande Ditch headwater (Arroyo Blvd) | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| Reed Canal | Reed Canal at Nova Road | 100-Yr | 100-Yr | < 100-Yr | 100-Yr |
| | Downstream end of Stevens Canal | < MA | < 10-Yr | < MA | < MA |
| | Stevens Canal at Ridge Drive | < 25-Yr | < 25-Yr | < 10-Yr | < 10-Yr |
| | Stevens Canal at Big Tree Road | 100-Yr | 100-Yr | < 100-Yr | 100-Yr |
| | Upstream end of Stevens Canal | < 10-Yr | < 10-Yr | < MA | < 10-Yr |
| Halifax Canal | Near Jackson Street | < 25-Yr | < 100-Yr | < MA | < MA |
| | Oak Street | < 25-Yr | < 25-Yr | < MA | < MA |
| | Powers Avenue | < 10-Yr | < 25-Yr | < MA | < MA |
| | Ryanwood Ave | < 100-Yr | < 100-Yr | < 25-Yr | < 100-Yr |
| Nova Canal | 10th Street | < 25-Yr | < 25-Yr | < 25-Yr | < 25-Yr |
| | Madison Avenue | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | George W. Engram Boulevard | < 100-Yr | < 100-Yr | < 100-Yr | < 100-Yr |
| | West International Speedway Boulevard | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Museum Boulevard/Navy Canal | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Bellevue Avenue | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Woodcliff Drive | < 10-Yr | < 10-Yr | < 10-Yr | < 10-Yr |
| | Beville Road | < 100-Yr | < 100-Yr | < 25-Yr | < 100-Yr |
| | Big Tree Road | < 100-Yr | < 100-Yr | < 25-Yr | < 100-Yr |
| | Moonstone Court | < MA | < MA | < MA | < MA |
| | Madeline Avenue | 100-Yr | 100-Yr | < 100-Yr | < 100-Yr |

Note: Highlighted cell represents nodes with improved LOS for Project Alternative 1 conditions

Review of the values in table 2-12 shows that an improvement in LOS for the 24-hour event, 1-year stillwater boundary condition occurs in a number of locations. Most evident is improvement in the LPGA Canal branches, with LOS improvement on the Railroad Ditch Canal, Southeast Canal and Northwest Canal. Reed Canal, Halifax Canal, and Nova Canal each show LOS improvement at one or two of the locations.

The results in Table 2-13 for the 96-hour design storm generally show the same results with respect to improvement in LOS. Greatest improvement occurs in the LPGA Canal branches, with some improvement in the Reed Canal, Halifax Canal, and Nova Canal.

A summary of average peak stage reductions for various locations in the study area is presented in **Table 2-14**. In this table, the average value represents an arithmetic average of the change in elevation at the selected locations in the Appendix B tables. For each location, change in peak stage is tabulated for each combination of design storm duration, stillwater elevation, and design storm return period.

Several observations can be made based on the tabulated results. One is that the benefit is greatest (i.e., greatest reduction in peak stages) for the mean annual event, with less peak stage reduction for more extreme events. Another is that the project can actually increase peak stages downstream of the gate, as is the case for the LPGA Canal and Reed Canal. However, this is generally limited to the 25-year event and less extreme events, such that the LOS downstream of the gate does not get worse. Further the increased stages are below the indicator elevations. The greatest overall benefit at all locations tends to be associated with the events with 96-hour duration and 100-year stillwater boundary elevation. Much of this benefit can be attributed the weir-gate preventing tidal backflow into the system in the project alternative condition.

Comparisons of peak flows, with and without the project, are presented in the Appendix C tables. It should be noted that the peak flows downstream of the gate structures are generally higher with the project, particularly for the more frequent design storms (e.g., mean annual). As discussed earlier, however, the downstream stage increases tend to be small and do not result in a lower LOS. It should also be noted that the peak flows from the headwaters of the Railroad Ditch Canal are generally lower for the project alternative, which would be considered a benefit to the Laurel Creek area. Model results show the peak flow to the north increasing for the 25-year and 100-year design storms with 96-hour duration and 100-year stillwater downstream boundary, when the project alternative is included. Further review of the model shows that this is correct, but only because the project is effectively pumping down the system storage and preventing overflow to the north until the peak rainfall occurs, whereas the model without the project alternative has earlier discharge to the north, resulting in high tailwater conditions north of Hand Avenue when the peak rainfall occurs.

**Table 2-14 Nova Canal Flood Control and Integrated Water Resources Project Baseline Conditions – Average Stage Reductions and Increases for Design Storm Simulations**

| Location | Design Storm Duration | Downstream Stillwater Elevation | Change in Maximum Stage (feet) with Alternative for Various Design Storm Return Periods | | | |
|---|---|---|---|---|---|---|
| | | | MA | 10-Year | 25-Year | 100-Year |
| LPGA Canal East of Proposed Gate | 24-hr | 1-Year | 0.3 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | 0.2 | 0.1 | 0.1 | 0.0 |
| | 96-hr | 1-Year | 0.2 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | 0.2 | 0.1 | 0.0 | 0.0 |
| LPGA Canal West of Proposed Gate | 24-hr | 1-Year | -0.5 | -0.2 | -0.1 | 0.0 |
| | | 100-Year | -0.4 | -0.2 | -0.2 | -0.1 |
| | 96-hr | 1-Year | -0.4 | -0.2 | -0.1 | -0.1 |
| | | 100-Year | -0.5 | -0.3 | -0.2 | -0.2 |
| Railroad Ditch Canal | 24-hr | 1-Year | -0.6 | -0.2 | -0.1 | -0.1 |
| | | 100-Year | -0.5 | -0.3 | -0.2 | -0.1 |
| | 96-hr | 1-Year | -0.5 | -0.2 | -0.2 | -0.1 |
| | | 100-Year | -0.8 | -0.5 | -0.3 | -0.3 |
| Southeast Canal | 24-hr | 1-Year | -0.3 | -0.2 | -0.1 | -0.1 |
| | | 100-Year | -0.3 | -0.2 | -0.2 | -0.1 |
| | 96-hr | 1-Year | -0.3 | -0.2 | -0.1 | -0.1 |
| | | 100-Year | -0.4 | -0.3 | -0.3 | -0.2 |
| Northwest Canal | 24-hr | 1-Year | 0.0 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | -0.1 | -0.1 | -0.1 | 0.0 |
| | 96-hr | 1-Year | 0.0 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | -0.1 | -0.1 | -0.1 | -0.1 |
| Reed Canal East of Proposed Gate | 24-hr | 1-Year | 0.4 | 0.2 | 0.1 | -0.1 |
| | | 100-Year | 0.3 | 0.1 | 0.0 | -0.1 |
| | 96-hr | 1-Year | 0.3 | 0.3 | 0.1 | -0.1 |
| | | 100-Year | 0.3 | 0.0 | 0.0 | -0.1 |
| Reed Canal West of Proposed Gate | 24-hr | 1-Year | -0.6 | -0.2 | 0.1 | 0.3 |
| | | 100-Year | -0.5 | -0.3 | -0.2 | -0.1 |
| | 96-hr | 1-Year | -0.6 | 0.1 | 0.2 | 0.3 |
| | | 100-Year | -0.5 | -0.3 | -0.2 | -0.2 |
| Stevens Canal | 24-hr | 1-Year | -0.2 | -0.1 | 0.0 | 0.0 |
| | | 100-Year | -1.0 | -0.3 | -0.2 | -0.2 |
| | 96-hr | 1-Year | -0.2 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | -0.9 | -0.3 | -0.3 | -0.3 |
| Halifax Canal | 24-hr | 1-Year | -0.3 | -0.2 | -0.1 | -0.1 |
| | | 100-Year | -0.2 | -0.2 | -0.2 | -0.2 |
| | 96-hr | 1-Year | -0.2 | -0.1 | -0.1 | -0.1 |
| | | 100-Year | -0.4 | -0.4 | -0.4 | -0.3 |
| Nova Canal | 24-hr | 1-Year | 0.0 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | -0.2 | -0.1 | -0.1 | 0.0 |
| | 96-hr | 1-Year | 0.0 | 0.0 | 0.0 | 0.0 |
| | | 100-Year | -0.2 | -0.1 | -0.1 | -0.1 |

## 2.6 Conclusions

The study area model was updated and refined using the most recent version of the EPA Stormwater Management Model (SWMM5). The model was validated to the May 2009 event, used to evaluate existing system capacity for various combinations of design storm return period, design storm duration, and downstream stillwater boundary elevations, and used to evaluate a project alternative for reducing peak stages and improving level of service. As indicated by the model results for existing conditions the existing primary stormwater management systems within the Nova Canal Watershed do not provide adequate flood control for large design storm events.

The alternative included a project on each main outfall canal (LPGA Canal, Reed Canal and Halifax Canal) to the Halifax River. The project at each canal consists of an operable weir-gate and a pump station (“pump only”), with weir-gate closure and pumping initiated at a specified water elevation. The design also included an emergency spillway in case the pumps are inoperable or insufficient, and flow would then be passed through the open weir-gates and over the spillway.

Pump sizing was based on the peak flow at each site during the May 2009 storm event. Ultimately the total flood control benefits will rely on how much flow can be conveyed to the Halifax River (River) outfall while not negatively impacting downstream areas. Since both the LPGA and the Reed Canal sites are designed to use the existing (unimproved) canals and structures downstream of the pump stations for conveyance to the River, the capacity of the pump stations must be limited so as not to negatively impact these downstream structures and flood-prone areas. The benefits that can be provided by the Halifax Canal site are also limited by flow capacity, but at this site the capacity constraint is on the upstream drainage system to convey flow to the pump station. Apparent means to increase the capacities of the canals to the River include improving the downstream (or upstream) drainage systems or by adding outfalls (such as force mains) to complement the existing canal flows without inducing negative downstream impacts.

As discussed throughout the project, flooding within the watershed is not eliminated by the proposed systems, but flood control benefits can be provided via the proposed pump stations and weir-gates at the outfall canals (LPGA Canal, Reed Canal, and Halifax Canal) as indicated by the model results.

An overall solution would require three additional project outfalls along with upgrades at the existing three outfalls, and other in-system improvements to reduce bottlenecks and addition of storage systems for flood attenuation and treatment. The three additional outfalls could be 1) near Laurel Creek, 2) from the City of Daytona Beach North Street pond, and 3) from the Daytona Beach Samuel Butts Park Pond. Ultimately the overall system would be linked to the water supply options being considered by the stakeholders.

The proposed pump and weir-gate systems provide the following benefits:

1. Reduced peak stages;
2. Shorten flood durations;
3. Capability to pump down the system in advance of an approaching storm;
4. Weir-gates to prevent tidal back surge; and
5. Capability to pump west for additional water supply and/or water quality benefits.

The benefits provided by the proposed systems are more significant for more moderate design storms (e.g., the mean annual, 5- and 10-year design storms). For much larger storms such as the May 2009 event (based on rainfall depths and duration), the proposed pump and weir-gate systems provide limited to moderate flood control benefits.

Specific conclusions regarding the modeling results are as follows:

- The greatest overall peak stage reduction under project alternative 1 conditions occurs at locations including:
  - LPGA Canal upstream of proposed gate location
  - Railroad Ditch Canal
  - Southeast Canal
  - Stevens Canal
- Less overall peak stage reduction, but large local peak stage reduction near the project sites, occurred in the Reed Canal and Halifax Canal.
- Minimal overall peak stage reductions occurred in the Northwest Canal and Nova Canal system.
- Some peak stage increases are predicted for areas downstream of the gates at the LPGA and Reed Canals. However, the peak stage increase is minimal for the most extreme events and does not adversely affect the defined LOS in the downstream areas. Right-of-way will need to be confirmed for any increased stages.
- Regardless of design storm duration or downstream stillwater boundary, the greatest peak stage reduction is achieved for the smallest evaluated design storm (mean annual) and more intense storms exhibit less peak stage reduction benefit.
- Regardless of design storm return period, the greatest peak stage reduction occurs in the 96-hour storm duration and 100-year downstream stillwater boundary. The LPGA Canal project also reduces the quantity of flow discharged to the north into the Tomoka Basin.

- Significant benefits were realized at the Halifax site during elevated tailwater conditions (see Appendix B).

# Section 3
# Additional Flood Control Alternatives and Conceptual Plan

## 3.1 Introduction

Following the analysis of the Phase 1 improvements, it was determined that the proposed Phase 1 improvements (Alternative 1 pump stations and tide gates) did not yield significant flood stage reductions. This is due to several factors, including the fact that the three main outfalls at LPGA, Reed, and Halifax Canals were all flowing at gravity capacity during the May 2009 event and tide levels were not significant in causing backwater to reduce gravity flow capacity. Following additional analysis and consultation with the JPA Partners, it was determined that an additional evaluation phase (Phase 3) should be performed to define and evaluate alternative conceptual program components to provide more significant flood control benefits within the study area prior to completing the Phase 2 Design and Permitting tasks.

As part of the Phase 3 additional alternatives analyses, various flood control components were evaluated individually and in combination to achieve greater comprehensive flood control benefits within the study area for the EVRWA. The potential flood reduction benefits provided by the project components were compared to the estimated capital costs to implement the components into the Nova Canal Flood Control and Integrated Water Resource Program. This section presents the individual program components, combinations of components and alternatives, flood reduction benefits (depth, area, and flood damage cost reduction), and capital costs.

## 3.2 Project Component Descriptions and Development

An initial project alternative (Alternative 1) was evaluated as part of the Phase 1 study, and included combination tidal weir-gates and pump stations at the LPGA, Reed and Halifax Canals. This configuration was a “pump only” option as opposed to a pump and force main to the Halifax River. The general concept of each canal component is to reduce the peak stages and duration of flooding by preventing tidal backflows into the conveyance system, pumping down the system before or at the start of major storm events, and increasing capacity of the system during high tailwater conditions that limit gravity outflow. Operating rules and appropriate pump capacities were evaluated initially based on the May 2009 rainfall and tidal conditions. The alternative was then evaluated for the various design storm durations and return periods for both 1-year and 100-year stillwater downstream tidal boundary conditions.

In addition to the flood control components considered in Alternative 1, additional conceptual project components within the study area were developed. Existing stormwater and other potential surface water storage facilities, as well as ongoing watershed projects that are currently committed to or in construction were also considered in developing these components. The flood control components for

Alternative 1 and the additional conceptual project components are presented in **Figure 3-1**. Detailed conceptual layouts for the project components are provided in **Appendix D**.

Project components evaluated typically consist of one or more of the following elements:

- **Tidal weir-gate**. The tidal weir-gate is defined as a rectangular structure located in the canal. Based on JPA discussion, the tidal weir-gate is expected to typically operate in the open position. The weir-gate is then closed under specified design conditions such as elevated tidal conditions or to allow pumping down the system in advance of a storm. Closing (raising) the weir gates could also allow diversion of the surface water for water supply purposes. The weir-gates would operate by either inflating or rotating from the bottom of the canal to allow a range of tidal control along with gravity outflow.

- **Diversion channel at pump station site**. This channel conveys water from the upstream side of the weir-gate to the pond at the pump station site.

- **Pond at pump station site**. A pond is included at the pump station site. This pond represents the wet well from which water is pumped.

- **Storage pond**. In addition to ponds at pump station sites, additional storage ponds may also be considered in order to capture and store stormwater runoff for additional flood control.

- **Force main**. A pressurized pipe system extending from the pump station to the downstream receiving water (typically the Halifax River for this project).

- **Pump station**. The pump station is located at the project site, and will pump water from the proposed pond. The "pump only" option pumps to downstream of the weir-gate, while the "pump with force main" option pumps through a force main routed to a downstream water body (Halifax River).

- **Emergency spillway**. An emergency spillway is provided at the pump station pond to promote additional flow conveyance when the pump is inoperable or not sufficient to pass peak flows for extreme events.

### 3.2.1 LPGA Canal Project Component

The LPGA Canal project component was originally configured in the Phase 1 Study as part of Alternative 1. Several options were evaluated to determine this location in Phase 1, which was chosen by the City of Holly Hill to be a pump station at the existing pond in Centennial Park, and a tide weir-gate located just downstream. The pump station was operated in the pump only (no force main) configuration. Specific operational parameters for the LPGA Canal component considered as part of Alternative 1 are described in Section 2.