ATLANTIC BEACH STORMWATER MASTER PLAN UPDATE

City of Atlantic Beach | November 2018

ATLANTIC BEACH

STORMWATER MASTER PLAN UPDATE

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1 INTRODUCTION

1.1 PROJECT SCOPE

The City of Atlantic Beach experienced flooding during Hurricane Irma, Hurricane Matthew, and the November 2015 nor'easter. The flooding from these extreme events prompted the City to solicit assistance and hire Jones Edmunds to complete an update of the City's Stormwater Master Plan (SWMP). The City had previously completed similar SWMP projects (1995, 2002, and 2012) that this project will build upon.

The results of the Master Plan Update are summarized in six sections:

- Section 2: Reviews and summarizes previous SWMP results for data that were useful for completing this Master Plan Update.
- Section 3: Summarizes a literature review of ongoing sea-level rise measures being taken by other municipalities across the southeast. It provides recommendations to the City for managing future sea-level rise.
- Section 4: Summarizes the updated existing condition hydrologic and hydraulic (H&H) model.
- Section 5: Summarizes the future conditions H&H models for projected 2030 and 2045 hydrologic conditions.
- Section 6: Identifies locations for stormwater capital improvement projects and develops capital improvement projects to improve the existing stormwater system.
- Section 7: Presents a 10-year Stormwater Capital Improvement Plan (CIP) for the City that includes budget-level costs.

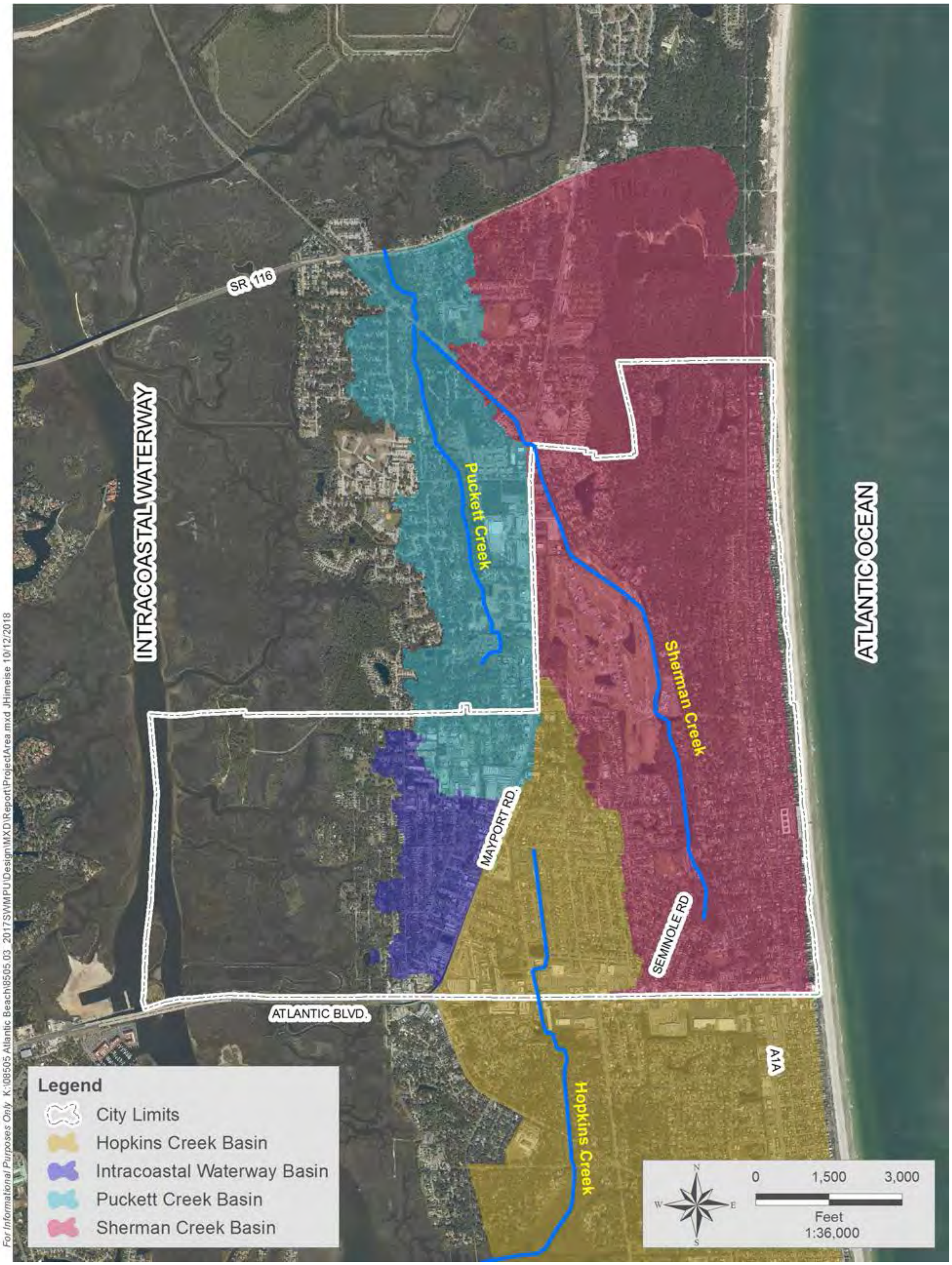
Hanson Professional Services Inc. provided additional engineering support with the H&H modeling tasks and capital improvement project development. They provided a peer-review of the H&H modeling parameters and results. They also developed capital improvement project alternatives for four of the problem areas that were investigated.

1.2 BACKGROUND

The City covers an area of approximately 4 square miles in the northeast corner of Duval County (Figure 1-1). The City consists of mostly medium- and high-density residential land uses with pockets of commercial and services areas along Atlantic Boulevard and Mayport Road. Most of the City was developed before modern stormwater regulations requiring on-site retention/detention systems for flood protection and water quality treatment were in place. This is a major cause of the flooding issues within the City today.

The City is bound to the north by the City of Jacksonville, to the east by the Atlantic Ocean, to the south by the City of Neptune Beach, and to the west by the Intracoastal Waterway (ICW). The City primarily drains through stormwater pipe collection systems that route water to larger ditch/creek systems. The City's stormwater discharges to the ICW via direct discharge from ditches, by way of Hopkins Creek or Sherman-Puckett Creek. A majority of the City drains through Hopkins Creek or Sherman-Puckett Creek, which flow through a combination of ditches and large culvert crossings owned and maintained by other entities before discharging to the ICW.

Figure 1-1 Study Area



2 SUMMARY OF PREVIOUS STUDIES

The City completed previous stormwater master planning efforts in 1995, 2002, and 2012. The original study was completed in 1995 by CH2M Hill and subsequent master plan updates were completed in 2002 and 2012 by CDM. The City tasked Jones Edmunds with reviewing the previous master reports and summarizing the information from the studies that was beneficial to developing the updated H&H models and developing the 10-year CIP.

2.1 ATLANTIC BEACH STORMWATER MASTER PLAN, 1995

As part of the original City of Atlantic Beach's SWMP effort completed in 1995, CH2M Hill completed the following tasks according to the final report provided by the City:

- Identified and summarized the City's major drainage basins including the Selva Marina Lagoon, Hopkins Creek, Puckett Creek, Sherman Creek, ICW, and Atlantic Ocean basins. They also split each of the major basins into smaller subbasins and summarized the drainage characteristics and flooding issues in each subbasin.
- Identified flooding problem areas based on historical information and H&H modeling. The H&H modeling was completed using a stormwater management model (SWMM) to model the larger creek systems and Interconnected Pond Routing (ICPR) for the smaller pipe systems. CH2M Hill used a copy of the County's stormwater model as the starting point and added detail where necessary. Survey data were collected for drainage features within the City.
- Completed a review of statewide floodplain level-of-service and water quality level-of-service criteria and developed level-of-service criteria for the City.
- Presented a detailed summary of the H&H model results and inputs for basins that were found to have serious flooding problems.
- Completed a water quality pollutant loading analysis and presented the inputs and results of the analysis.
- Developed and presented a flood protection and water quality ranking criteria for the stormwater problem areas and ranked the subbasins with problem areas using the criteria.
- Developed conceptual layouts and budget level costs for recommended stormwater improvements for six subbasins in the City.

The information from this study was used as a historical reference for problem areas that were identified in the 1995 Report, remained unresolved stormwater concerns at the time of this study, and were looked into further as a part of this study. Proposed improvements from the 1995 Report in each of these areas were reviewed and summarized in Section 6 of this report.

2.2 ATLANTIC BEACH STORMWATER MASTER PLAN UPDATE, 2002

As part of the City's SWMP effort completed in 2002, CDM completed the following tasks according to the final report provided by the City:

- Evaluated existing data sources and identified/classified existing known stormwater problem areas.

- Identified data sources that could be integrated into a stormwater geographic information system (GIS). Converted the data into a GIS database design.
- Developed and presented a prioritized stormwater CIP with problem area descriptions, proposed improvements, and planning level costs for 11 problem areas.
- Completed a detailed analysis of the Hopkins Creek drainage basin. The analysis included an existing data review, problem area identification, H&H modeling, summary of existing drainage basin characteristics, modeling sensitivity analysis to extreme tide conditions, and stormwater improvements alternatives analysis with three alternatives.
- Developed a ranking system for the CIP projects and ranked the proposed CIP projects for the 10-year CIP.
- Developed stormwater criteria for development and redevelopment.

The 2002 SWMP Update does not provide a significant source of information not presented in the more recent 2012 Study. The modeling and other technical data from 2002 is largely superseded by that of the 2012 Study. Several sites were identified in the 2002 Update listed as CIPs that are also listed as such in the 2012 Study. The 2002 analysis and proposed alternatives were useful in providing historical conditions at the sites and serve as a starting point for the proposed solutions in the current CIP Update. The detailed evaluation of Hopkins Creek was particularly useful while developing the improvements proposed in this report.

2.3 Atlantic Beach Stormwater Master Plan Update, 2012

As part of the City's SWMP effort completed in 2012, CDM completed the following tasks according to the final report provided by the City:

- Collected data to support project tasks including H&H modeling and development of a stormwater CIP.
- Identified 35 problem areas within the City and proposed local-scale capital improvement projects for 13 of these areas with planning-level cost estimates. The analysis completed for these areas was at a lower level-of-detail and did not require additional H&H modeling due to the smaller scope of the proposed projects.
- Completed more in-depth alternatives analyses for two priority areas with regional drainage issues that included detailed H&H modeling, conceptual alternatives, planning level cost estimates, and recommendations.
- Completed a review of the City's stormwater operations and maintenance program. Proposed an operations and maintenance schedule based on stormwater feature types. Completed a capacity analysis of the City's ditches based on H&H modeling and the rational method for the 25-year/24-hour storm to determine whether the ditches have adequate capacity.
- Evaluated low-impact development best management practices.
- Reviewed the City's regulations for onsite runoff controls. As part of this, a pilot analysis was completed for a small portion of the City to determine the effectiveness of the City's regulations.
- Performed a review of the City's GIS data and completed a GIS needs assessment.
- Completed a review of sea-level-rise projections and potential impacts of sea-level-rise on the City.

- Completed a review and provided recommendations for the City's National Flood Insurance Program Community Rating System score.

The 2012 Master Plan Update served as a starting point to the current update by outlining large-scale drainage patterns and conveyances and identifying problem areas throughout the City. The list of problem areas helped identify problem areas in the city that may require more detailed analysis or may be developed as CIPs. The list from CDM was not considered exhaustive as some areas on the list have already been addressed and areas not mentioned on the list may have become problematic since 2012. CDM's work to assess the City's GIS needs was not scoped to be revised in this update and will remain the most recently updated formal assessment of the City's GIS program. The 2012 report section focused on sea-level-rise will be updated with the most current literature and data. Also, the H&H modeling completed during the 2012 Master Plan Update served as the starting place for the current update project and was enhanced where necessary.

3 CLIMATE CHANGE SEA-LEVEL-RISE REVIEW AND RECOMMENDATIONS

Jones Edmunds performed a literature review regarding the Southeastern communities' response to current and future sea-level-rise impacts from climate change. Case studies of three communities that have made remarkable efforts are presented in the following subsections. Receiving the most recognition among the three, the Compact took a detailed, collaborative, and high budget approach. Similarly, the City of Satellite Beach also took a detailed and high budget approach, but did not partner with nearby communities. Spartanburg Water took a non-quantitative and lower budget approach, also without partnership. Our recommendations for the City are based on lessons learned from these communities and the findings from the master plan development.

3.1 CASE STUDIES

3.1.1 SOUTHEAST FLORIDA REGIONAL CLIMATE CHANGE COMPACT (THE COMPACT)

The Compact has received various local, national, and international awards and recognitions, including from the White House, for its progress and efforts and is referenced as a case study example for academic purposes and the assessment tools developed (Compact, 2012). The recognition attracted support and resources from within and outside the region (Compact, 2018).

Adopted in January 2010, the Compact is a partnership agreement between Broward, Miami-Dade, Monroe, and Palm Beach Counties to coordinate mitigation and adaptation strategies in response to climate change (Compact, 2018). The formation started in 2009 with the Southeast Florida Regional Climate Leadership Summit, where representatives of those counties discussed the local challenges and threats from global climate change and the need for regionalized action efforts (i.e. the Compact) (Compact, 2018). The Staff Steering Committee consists of representatives from the counties, the 109 cities of the region, scientists from major research universities of the region, South Florida Water Management District (SFWMD), National Oceanographic Atmospheric Administration (NOAA), U.S. Army Corps of Engineers (USACE), and others (Compact, 2018). The involvement of the agencies and professionals from academia provides a technical foundation for regional climate issues (Compact, 2018).

In the early stages, the Compact created a Technical Ad Hoc Working Group to establish a uniformly accepted upper and lower bounds of sea-level-rise projection for the region in 2012 until 2060 (Compact, 2012). The members of the group are experts from academia and government agencies. The Unified Sea-Level-Rise Projection (USLRP) was updated in 2012 to incorporate new studies and data (Compact, 2012). This is important for planning purposes across the counties.

The Southeast Florida Regional Climate Action Plan (RCAP) was also finalized in 2012 and formally adopted in the spring of 2014 (Compact, 2018). One hundred and ten action items are outlined to achieve a 5-year goal of reducing greenhouse gas emission, adapting to climate change, and improving resiliency (Compact, 2012). To encourage and support participation, the Compact provides a tool on their website (http://www.southeastfloridaclimatecompact.org/regional-climate-action-plan) for the public to create a customized implementation plan. During this time, the region also coordinated in federal and state legislative programs, including the state legislature (Community Planning Act HB 720) that established the Adaptation Action Area (AAA) designation for areas susceptible to climate impacts (Compact, 2018).

Currently, the Compact continues to work on the implantation of the RCAP and other greenhouse gas mitigation and climate adaption strategies (Compact, 2018).

3.1.2 City of Satellite Beach

The City of Satellite Beach, a small coastal city, was the first local government on the east-central Florida coastline to plan for sea-level-rise effects (Parkinson, 2010). The small size of the City and its active public engagement add to the uniqueness of the effort.

Recognizing the risks, the effort started with the Climate Ready Estuaries Pilot Project. The City authorized this project in fall 2009 to assess its vulnerability to sea-level-rise and initiate mitigation planning (Parkinson, 2010). Through the Indian River Lagoon National Estuary Program, the project was funded by the U.S. Environmental Protection Agency (EPA) 2009 Climate Ready Estuaries (CRE) Program (Parkinson, 2010). A technical report of the assessment was produced, stating that approximately 5 percent of the City would be flooded with sea-level-rise of 2 feet and 25 percent would be flooded with a sea-level-rise of 4 feet. (Parkinson, 2010) Using findings from this assessment, the Sea-Level-Rise Subcommittee of the City's Comprehensive Planning Advisory Board (CPAB) formulated and submitted to the City Council its policy recommendations in 2010 for response strategies and updates to the City's Comprehensive Plan (Adaptation Clearing House, 2010). After several revisions in response to private property rights concerns and the City Council's request, the recommended amendments were adopted in August 2013. Policies that aim to identify and protect Adaptation Action Areas are included in the Comprehensive Plan (LaRue & City of Satellite Beach, 2017).

From July 2014 to June 2015, the City performed the Community Resiliency Project. Funded by the Florida Department of Environmental Protection (FDEP) Coastal Partnership Community Resiliency Grant, this project aimed to identify problem areas and/or criteria for Adaptation Action Areas, set the foundation for AAAs to be used as a tool by the community and the City Council to improve resilience, and engage the public in brainstorming strategies and setting action and resources allocation priorities (Baker, 2016). Through public workshops and the online tool MetroQuest, the City collected community inputs and encouraged conversation between the officials and the community (Baker, 2016). Using this tool, the respondents sorted by priority the City's vulnerabilities, response strategies, and initiative opportunities (Baker, 2016). This public engagement effort successfully collected 3 months of inputs from 479 validated respondents (Baker, 2016). Areas of focus and strategies for future actions were identified (Baker, 2016). Efforts from this project guided the CPAB to adopt new AAA policies into the Comprehensive Plan and implement policies in appropriate areas (Baker, 2016).

Following this project, a Sustainability Board was also formed to focus on sustainability issues and develop a Sustainability Action Plan for the City (Baker, 2016). The Board is responsible for maintaining a living Sustainability Action Plan that serves as a guide to the City from policymaking to infrastructure developments (Eichholz & Lindeman, 2017). The guidance focuses on meeting the City's requirements regarding economic, social, and environmental needs (Eichholz & Lindeman, 2017).

From 2016 to 2017, the City received a Sea Grant to develop an infrastructure plan in GIS format to include sea-level-rise and public education (Baker, 2016).

3.1.3 Spartanburg Water

Spartanburg Water, located in South Carolina, is a public water and wastewater utility, and the collective entity of Spartanburg Water System and Spartanburg Sanitary Sewer District Commission (Vogel et al., 2016). The drought and extreme rainfall conditions of the area prompted the utility to plan for climate change effects on its already problematic water supply and water quality (Vogel et al., 2016). Spartanburg Water is unique because a low budget approach was taken and a quantitative vulnerability assessment was not conducted, but its adaptation actions were based on climate change information and recent extreme climate observations.

Primarily vulnerable to droughts and inland flooding, the 39 weeks of exceptional drought conditions in 2008 resulted in the reservoirs dropping to historic low levels (Vogel et al., 2016). In the same year, Spartanburg Water launched a wastewater collection system rehabilitation program to correct water and wastewater lines leakage problems and left old pipes in place for additional flow capacity for future intensive conditions (Vogel et al., 2016). The Utility also made an effort to save water in its routine operation, successfully reducing the unaccounted-for water from 18 percent in 2001 to 11 percent in 2009 (Vogel et al., 2016).

To raise awareness and to be educated on the climate change latest development, the Utility encourages staff to engage in water conferences and committees, utility councils, and professional delegations (Vogel et al., 2016). These workshops and networking opportunities allow the staff to learn from the other utilities' experience in overcoming extreme climate challenges (Vogel et al., 2016). Upon return, the staff are asked to disseminate the information with other staff to spread awareness and knowledge (Vogel et al., 2016). These experiences have a catalytic effect within the Utility in recognizing the need to address climate change impacts and to consider its programs and activities (Vogel et al., 2016). In 2010, the Utility began incorporating climate change planning into its programs, management actions, and culture to mitigate vulnerabilities (Vogel et al., 2016).

Outside the company, the Utility also invests in public outreach and education events (Vogel et al., 2016). To enhance the effectiveness of the effort, the Utility carefully tailors its communications to ensure the acceptability of the message by its audience (Vogel et al., 2016). For example, instead of using the term *climate change*, *immediate and future effects of droughts and flooding* is used to avoid being perceived as politically loaded (Vogel et al., 2016).

In 2015, the Utility became a pilot community for EPA's Climate Resilience Evaluation & Awareness Tool (CREAT) – a utilities planning tool for identifying potential climate change impacts and evaluating adaptation options (Vogel et al., 2016). The tool helped the Utility to understand the projected future climate condition and to better incorporate the issues into programs and management actions (Vogel et al., 2016).

3.2 Observations and Recommendations

Based on the literature review, Jones Edmunds summarized a few observations and recommendations.

3.2.1 Use Vulnerability Assessment as the Starting Point

Vulnerability assessment is the logical first step in the adaptation planning process. By better understanding the risk, the community can effectively identify Adaptation Action Areas to focus on, make informed decisions, and support adaptation strategies incorporations into programs and policies.

The options are quantified and non-quantified assessment. The case studies demonstrate that both options can be successful and that the selection should be based on budget and needs. Communities such as the City of Satellite Beach and the Compact chose quantified assessment to determine a projected sea-level-rise level that the vulnerability analysis and adaptation planning could be based on. This is useful for planning and implementations that require a reference point or clear guidance, especially in policy making such as Building Code regulations. However, more resources and time are required to perform the research and generate the technical report. Other entities such as Spartanburg Water proceeded with known climate change information and observations from past extreme events. Using such information, it focuses on fixing known issues, such as pipe leakage, and preparing for future events in a general sense, such as leaving original pipe in place for use as additional flow capacity.

3.2.2 Leverage Extreme Events as Motivation

Jones Edmunds recommends including adaptation actions in the response plan after extreme events such as floods and hurricanes. These windows of opportunities can be used to galvanize support and resources for climate change adaptation efforts.

3.2.3 Establish Focus Committee with Members from Different Sectors

A diversified committee has the advantage of gaining insights from experts regarding different aspects of the issue, access to data and resources from different entities, and funding opportunities.

The Compact and the City of Satellite Beach collaborated with technical agencies, local universities, stakeholders, and administrative departments and councils to establish committees. As a result, they have a technical foundation to support the assessment and planning process and inputs from stakeholders. By contrast, Spartanburg Water did not collaborate and relied on information learned from staff engagement in water conferences and committees, utility councils, and professional delegations.

3.2.4 Collaborate with Others for Resources and Capabilities

Climate change impacts are not limited by political boundaries; neighbors are likely to be in a similar situation. Collaborating efforts and sharing resources and capabilities with neighbors can make planning and implementation more efficient and effective.

The Compact collaborated to establish USLRP values and AAA designation, and the City of Satellite Beach collaborated with the public and experts on the Community Resiliency Project. Those projects resulted in an impactful outcome.

3.2.5 Use Public Outreach and Engagement to Build Community Support

Public outreach raises awareness of the issue and encourages public involvement. In turn, the adaptation actions are tailored to the community's needs and local support is engendered for the adaptation efforts. Effectiveness can be enhanced when the campaign is planned according to local context and sentiment.

All three case studies include active public outreach in the plans. The City of Satellite Beach in particular had remarkable success with its MetroQuest tool in collecting public input on focus area priorities and response strategies selection. Public buy-in and support was ensured for the adaptation actions. This success is in contrast with its Comprehensive Plan recommendation process earlier, where several revisions were required before being accepted. Similarly, Spartanburg Water made an effort within and outside the Utility. Considering local context and sentiment, the Utility carefully crafts outreach communication outside the Utility to avoid language that could be perceived as politically loaded. Within the Utility, it encourages staff to become educated on the issue, which became the catalyst of its adaption actions.

3.2.6 Adjust Adaptation Plan

Priorities, needs, and available resources change as the conditions evolve and lessons are learned from experimentation. Adaptation policies and strategies should be flexible and evolve with these changes.

Other than regularly reevaluating the Comprehensive Plan, the Compact and the City of Satellite Beach have other living plans that evolve to guide policymaking and program developments. For example, the Compact updates its USLRP when new data become available and the City of Satellite Beach has a Sustainability Action Plan that is actively updated by the Sustainability Board. Spartanburg Water gathers new information by engaging in water conferences and committees, utility councils, and professional delegations.

3.2.7 Incorporate Adaptation Planning into Existing Policies and Programs

Regulatory policies and mitigation programs can be revised to implement adaptation actions. This includes ordinances, building codes, utility fees, zoning, and hazard mitigation planning. Adaptation consideration can also be incorporated into remediation programs for other community issues, such as decaying infrastructure. Incentives and deterrence can be used to encourage population and development to locate outside of vulnerable areas.

The establishment of the AAA designation by the Compact and its incorporation in the Comprehensive Plan is a major achievement. The Compact, the City of Satellite Beach, and other communities use the designation as a planning tool.

4 EXISTING CONDITIONS H&H MODEL DEVELOPMENT

Jones Edmunds developed an updated existing conditions H&H model using Streamline Technologies, Inc. Interconnected Channel and Pond Routing Model Version 4.03.02 (ICPR4). The City provided Jones Edmunds with EPA SWMM5 model files for the Sherman-Puckett Creek and Hopkins Creek watersheds developed by CDM as part of the 2012 SWMP Update. These models were developed at a coarse regional basin scale, which is not ideal for developing capital improvement project alternatives for localized flooding issues. Jones Edmunds used these models as the starting place for developing the Updated Existing Conditions Model.

We exported the existing model input and spatial data from SWMM5 into a Microsoft Access Geodatabase that was developed by Southwest Florida Water Management District (SWFWMD). SWFWMD has developed an XML export routine for this database schema that allows the data to be exported directly into ICPR4 from the database. We made the following updates to the Existing Conditions Model within the database using ArcGIS before exporting the model to ICPR4.

4.1 Existing Conditions Model Hydrologic Updates

4.1.1 Basin Delineation Updates

Jones Edmunds exported the basins from the EPA SWMM model files provided by the City and combined them into one shapefile so that they could be reviewed spatially in GIS. This shapefile served as the starting point for updating the basins. The existing models included a total of 32 basins covering an area of approximately 4,900 acres with an average area of 150 acres. A majority of the modeled area was outside the City's limits and was included to accurately characterize tailwater conditions in the creeks, which are controlled by hydraulic structures owned and maintained by the adjacent municipalities.

We used the following data sources in GIS to review the existing basins against:

- A 5-foot-by-5-foot digital elevation model (DEM) generated from the 2007 City of Jacksonville Light Detection and Ranging (LiDAR) data.
- The City's GIS stormwater asset database.
- 2017 Florida Department of Transportation (FDOT) aerial imagery.
- As-built and construction plan data provided by the City.

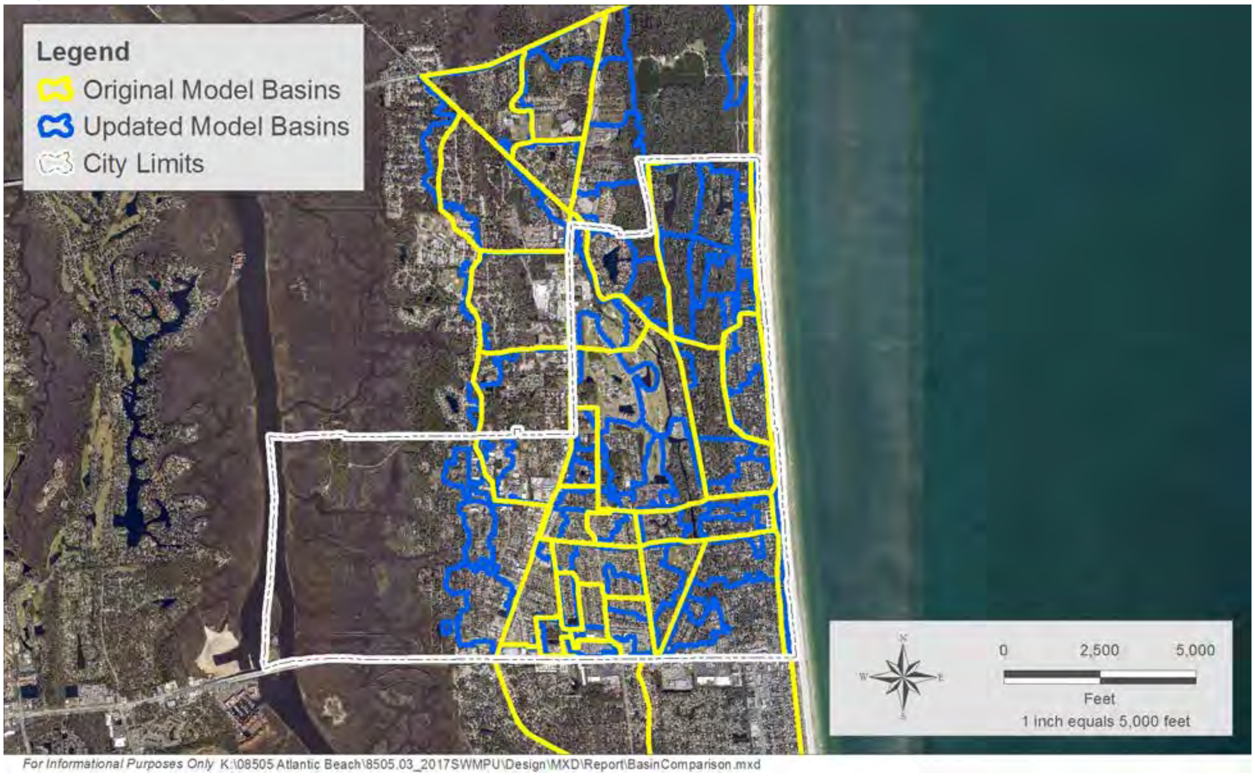
We made updates to the basins where:

- Gaps or overlaps existed and runoff was being double-counted or excluded from the model.
- Basin boundaries were not following topographic divides in the DEM.
- Basin boundaries were not in line with manmade subsurface stormwater drainage features.
- New developments or stormwater facilities were constructed since the previous model was developed.

- The level-of-detail was not appropriate to accurately characterize conditions where known flooding issues existed.
- Large basins contained multiple large hydraulic conveyance features within them.

The updated model contained 78 basins covering approximately 5,000 acres with an average area of 64 acres. More than twice as many basins are included than in the original models with the average basin area reduced by more than half. Also, the level of detail was increased further within the City limits with 61 of the basins falling within the City with a total area of approximately 1,800 acres and an average area of approximately 30 acres. The basin areas within the City ranged from 1 acre to 104 acres. Figure 4-1 compares the updated basins against the original model basins.

Figure 4-1 Updated Basins



4.1.2 Curve Number Calculations

We used the Natural Resource Conservation Service's (NRCS) Technical Release 55 (TR-55) curve number (CN) methodology to generate runoff in the ICPR4 model. CNs were generated for each of the basins using data from the following sources:

- NRCS soil survey.
- 2009 St. Johns River Water Management District (SJRWMD) Land Use and Land Cover (LULC).
- 2016 Duval County Parcels.
- 2017 FDOT aerial imagery.
- University of Florida's Institute of Food and Agricultural Sciences (UF IFAS) Florida Soil Characterization Data Retrieval System database.

We used these data sources with estimates of impervious area to calculate an area-weighted CN for each basin. We performed an intersection within ArcGIS of the model basins, NRCS soils data, and SJRWMD LULC to develop hydrologic response units that were used to calculate the weighted CN. We assigned an open land CN value and an estimated impervious area percentage for each response unit.

For response units with undeveloped land use categories that did not contain impervious area such as wetlands, forests, or open land, the CN for each response unit was assigned using standard open land TR-55 CN values varied by the NRCS soil hydrologic group. We assigned these undeveloped response units an impervious area percentage of 0 percent. Response units with undeveloped land use categories accounted for approximately 30 percent of the total modeled area and 25 percent of the modeled area within the City limits.

We assigned an open land CN calculated based on the available soil storage for the hydrologic response units with developed land uses. We calculated the open land CN for each soil polygon based on the estimated depth to seasonal high water table included in the NRCS soils data table. We used this methodology for the developed land use areas because they accounted for 70 percent of the total modeled area and 85 percent of the developed polygons fell into split hydrologic soil groups “A/D” or “C/D.” Estimating the CN value based on the available soil storage provides a physical based estimate for the split hydrologic group response units rather than assuming one soil group over the other, which may not be accurate. We measured the impervious area and assigned impervious area percentages for each of these response units in GIS using the 2017 FDOT aerial imagery.

We aggregated the response units by model basin once the impervious area percentage and open land CN values were assigned to the response units. An area-weighted CN was calculated for each basin using a CN of 98 for all impervious areas and the assigned open land CN values for non-impervious areas. The average weighted CN for the modeled basins was 80 with a minimum CN of 67 and a maximum CN of 95. High CN values are expected in the City due to the amount of impervious area from development and high water table conditions.

4.1.3 RAINFALL

Jones Edmunds used rainfall depth-frequency data from the NOAA Atlas 14 Volume 9 Version 2 to determine rainfall depths for the mean-annual (2.33-year), 10-year, 25-year, and 100-year/24-hour events. Table 4-1 summarizes the rainfall depths used.

Table 4-1 Rainfall Depths

Rainfall Event	Rainfall Depth (inches)
Mean Annual/24-hour	5
10-Year/24-Hour	7.3
25-Year/24-Hour	9.2
100-Year/24-Hour	12.6

We used the Florida Modified Rainfall Type II rainfall distribution to generate synthetic storm hyetographs for the 24-hour events. This is the standard rainfall distribution used by permitting agencies in the Atlantic Beach jurisdiction and has proven reasonable for determining flood risk.

4.2 Existing Conditions Model Hydraulic Updates

Jones Edmunds made updates to or verified all existing modeled hydraulic parameters and added several hydraulic elements with new parameters to the model to match the increased level-of-detail of the updated model basins. The number of model links was doubled from 151 in the existing model to 303 in the updated model and the number of nodes was increased from 113 to 160.

4.2.1 Pipe and Drop Structure Updates

We increased the number of modeled pipe and drop structure links from 40 in the existing model to 111 in the updated model. The length of modeled pipe was nearly quadrupled, increasing from 4,893 feet in the existing model to 18,976 feet in the updated model. Pipe size, shape, material, and invert parameters were verified against the GIS asset data, as-built plans, field observations, and construction plan sets for all pipe links carried over from the existing model. Generally, new pipe links were parameterized using data from the GIS asset data, as-built plans, and construction plan sets. We converted invert data to the North America Vertical Datum 1988 (NAVD88) from the National Geodetic Vertical Datum 1929 (NGVD29) using a 1.10-foot conversion factor.

Jones Edmunds collected engineering grade survey for 13 structures where existing elevation data were not available. The elevation data were collected using Real Time Kinematic (RTK) Geographic Positioning System (GPS) equipment in December 2017. Vertical and horizontal coordinates were collected for each surveyed structure.

4.2.2 Weir Updates

We increased the number of modeled weir links from 41 in the existing model to 121 in the updated model. Three of the modeled weir links represent structural weirs and the remaining 118 represent overland flow connections between basins. Overland flow connections were represented using irregular weir cross-sections that were extracted from the 5-foot-by-5-foot DEM in GIS. These connections allow water to flow between basins when node elevations exceed the surface elevations along the hydrologic basin boundaries. This prevents water from artificially stacking up or *glass-walling* along the boundary.

4.2.3 Channel Updates

We increased the number of modeled channel links from 68 in the existing model to 71 in the updated model. We used channel cross-section and invert data from the existing model where available and used the DEM and construction plan/as-built plan data to develop parameters for new channel links.

4.2.4 Node Storage

We extracted a stage-area relationship for nodes that represent the primary storage unit in each basin using a Python-based script tool, the 5-foot-by-5-foot DEM and a shapefile of the model basins. We excluded the extents of the modeled channel reaches from the basins shapefile used to extract the stage-area relationships to ensure that the channel storage volume was not double-counted in the model. Nominal stage-area relationship values were assigned to secondary or junction nodes that were not the primary source of storage volume in the basins.

4.2.5 Topographic Voids

We identified two locations where the 2007 LiDAR DEM was significantly different from current ground conditions and would cause model results to be inaccurate. These locations are referred to as topographic voids. Figure 4-2 shows the two topographic void areas identified as the Atlantic Beach Country Club and the Hopkins Creek Regional Stormwater Facility.

Figure 4-2 Topographic Void Areas

For each of these areas we adjusted H&H model parameters to accurately reflect current ground conditions based on construction plan sets and environmental resource permitting (ERP) drainage calculation information downloaded from the SJRWMD's permitting website. We did not update the project DEM to reflect the current ground conditions in these areas because it was not a cost-effective use of the City's funds and the main goals of the Master Plan Update would not be enhanced by doing this.

4.2.6 Boundary Conditions

We set the boundary condition for the Existing Conditions Model to elevation 2.0 feet NAVD88 based on a calculation of the estimated mean high high water (MHHW) elevation. We calculated this elevation based on tide gauge data from FDEP's Bar Pilots Dock St. Johns River tide station (ID 872-0218) and historical MHHW elevation data from FDEP's Pablo Creek tide station (ID 872-0267) (Figure 4-3). The MHHW elevation for the Bar Pilots Dock station based on historic tide data is 1.95 feet NAVD88, and the MHHW elevation based on historic data for the Pablo Creek station is 1.48 feet NAVD88. These elevations were calculated based on tidal data collected from 1983 to 2001, which is likely under-predicting current tidal conditions due to rising sea levels. To adjust for this, we downloaded tide stage data from January 1, 2017, to December 31, 2017, for the Bar Pilots Dock station and calculated the MHHW elevation for this dataset. The calculated MHHW elevation for 2017 was 2.3 feet NAVD88, which is approximately 0.3 foot greater than the MHHW elevation based on historic data from this station. To calculate the boundary condition of 2.0 feet NAVD88, we took the average of the MHHW elevations based on historic data at the two tide stations, which was approximately 1.7 feet NAVD88 and added 0.3 foot.

Figure 4-3 Tide Station Locations

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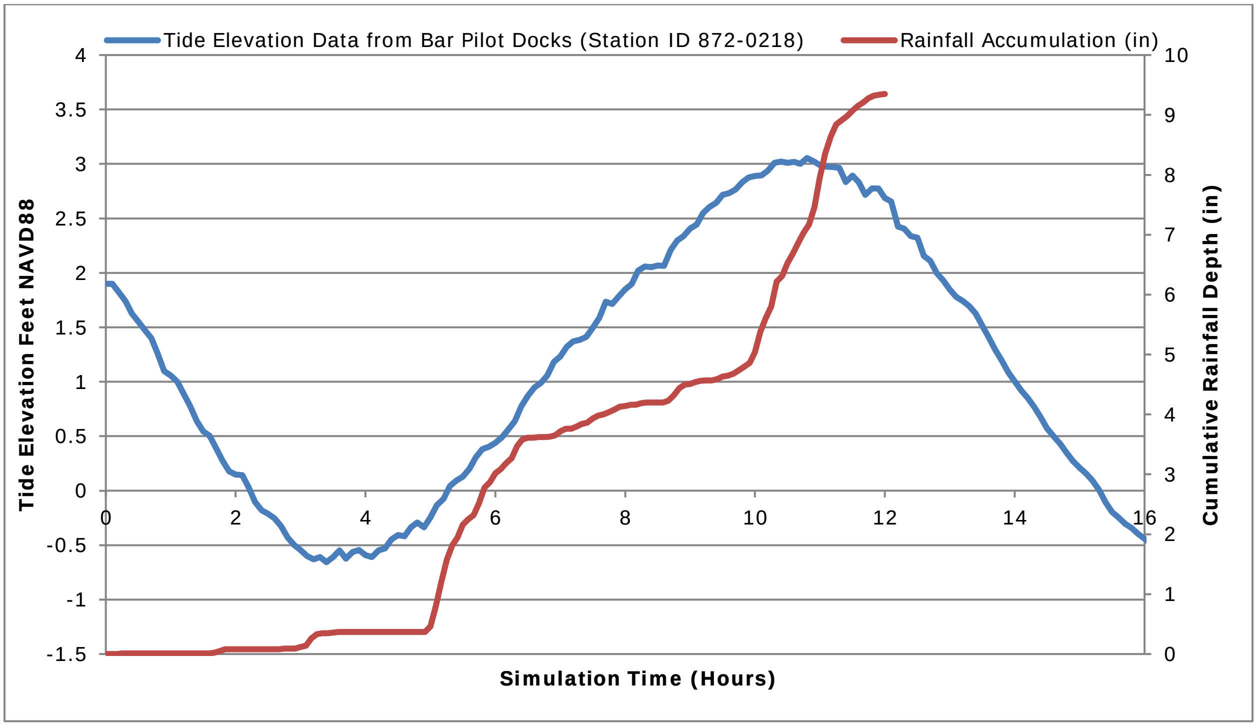
4.3 Existing Conditions Model Verification

We verified the Updated Existing Conditions Model results against the November 8, 2015, rainfall event that occurred during a nor'easter, which caused extreme tidal conditions in addition to approximately 9.3 inches of rainfall within a 12-hour period. According to the City, more than 100 residences experienced flooding above finished floor elevations (FFEs) resulting from this event.

We downloaded 5-minute interval rainfall data from Wunderground.com for the Atlantic Beach/Mayport Weather Station (Station ID KFLATLAN3) and used the data to develop a rainfall distribution for the November 8, 2015, event. We also downloaded observed tide gauge time-stage data from the Bar Pilots Dock St. Johns River tide station and used the data to develop time-stage boundary condition data and to set node initial stages to reflect actual boundary stages when the rainfall began. Figure 4-4 shows a plot of the cumulative rainfall distribution and the boundary time-stage data. According to the data, approximately 9.3 inches of rain fell with a peak boundary stage in the ICW of approximately 3.0 feet NAVD88.

Figure 4-4 Verification Event Rainfall Data and Boundary Time-Stage Data



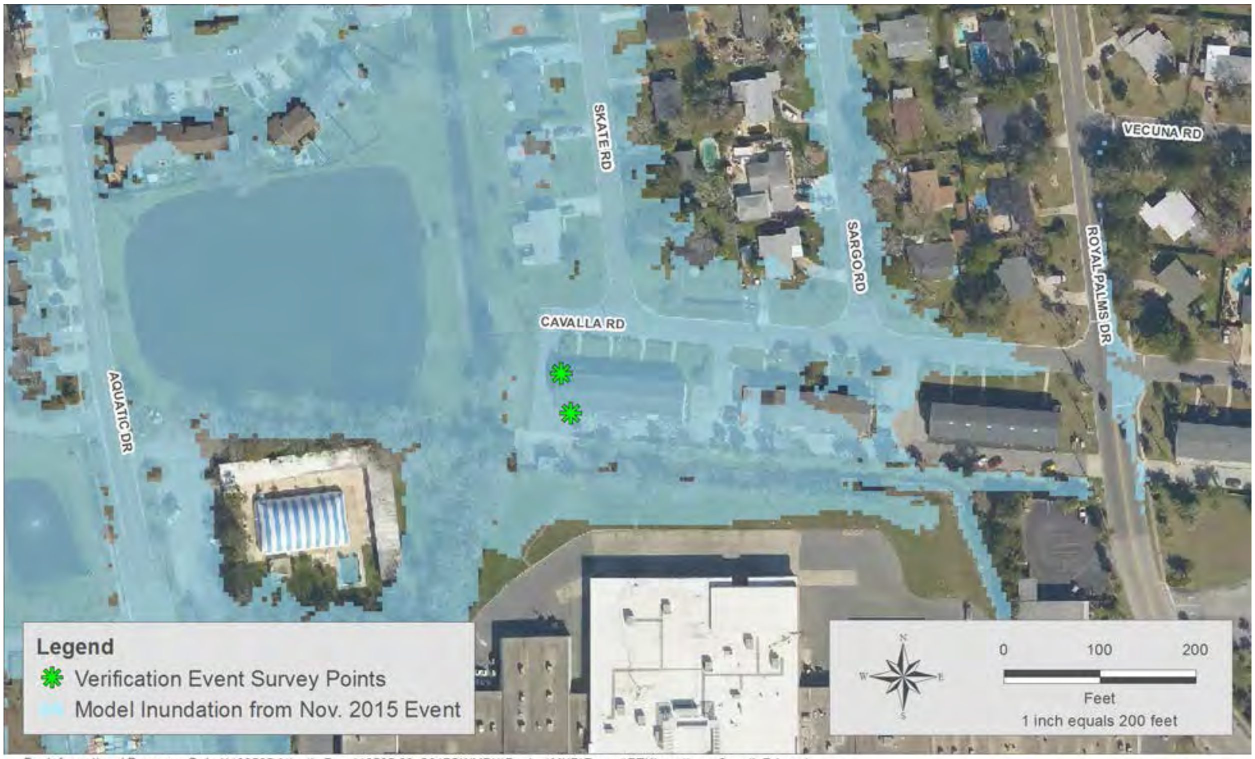
We simulated this event using the ICPR4 model, the Wunderground rainfall distribution, and the tide gauge time-stage data for a boundary condition. The City provided us with several photographs of the damage that occurred resulting from the flooding throughout the City. We collected RTK survey elevations at three of these locations, which allowed us to compare peak modeled stages to estimated peak stages from the photographs and survey elevations.

The first location we compared modeled stages to observed stages was at 846 Cavalla Road, which is the townhome at the west end of Cavalla Road bordered by the Cavalla Road Ditch to the south and the larger Aquatic outfall ditch to the west. Photograph 4-1 and Photograph 4-2 show debris lines that were left behind on the sides of this building after the water receded. We collected the RTK elevation data shown in Figure 4-5 at the northwest corner of the building in Photograph 4-1 and just south of the back door in Photograph 4-2. The modeled peak stage in this area for the November 8, 2015, event was 6.4 feet NAVD88, which is approximately 9 inches above the surveyed elevation at the northwest corner of the building and approximately 13 inches above the surveyed elevation at the back door. Based on the debris lines in the photographs compared to the modeled peak stage elevations, we believe that the model was accurately representing the drainage system in this area.

Photograph 4-1 Northwest Corner of 846 Cavalla Road after Nov. 8, 2015, Event

Photograph 4-2 Back Door at 846 Cavalla Road after Nov. 8, 2015, Event

Figure 4-5 RTK Survey Location at 846 Cavalla Road



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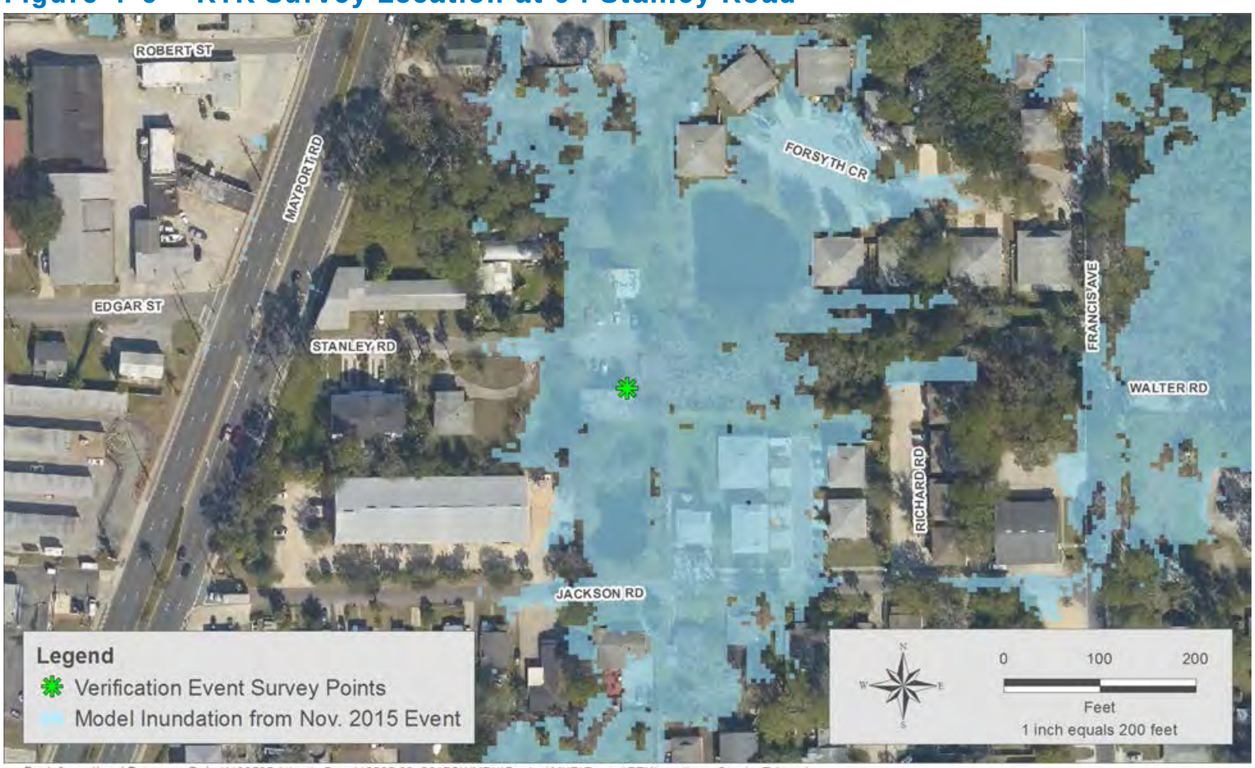
We also compared modeled versus observed peak stages from the November 8, 2015, event at the apartment complex on the south end of Stanley Road. Photograph 4-3 shows the flooding and debris lines left at 94 Stanley Road. Figure 4-6 shows an RTK survey point on the concrete pad in front of the northeast corner of the building. The modeled peak elevation for the November 8, 2015, event at this location was 11.3 feet NAVD88, which is approximately 14.5 inches above the RTK surveyed elevation on the concrete pad. Photograph 4-3 shows a debris line along the front of the building that appears to be very close to the 14.5-inch modeled depth. We believe that the model was accurately representing the drainage system in this area based on the debris line in the photograph compared to the modeled peak stage.

Photograph 4-3 Flooding at 94 Stanley Road after Nov. 8, 2015, Event

Figure 4-6 RTK Survey Location at 94 Stanley Road



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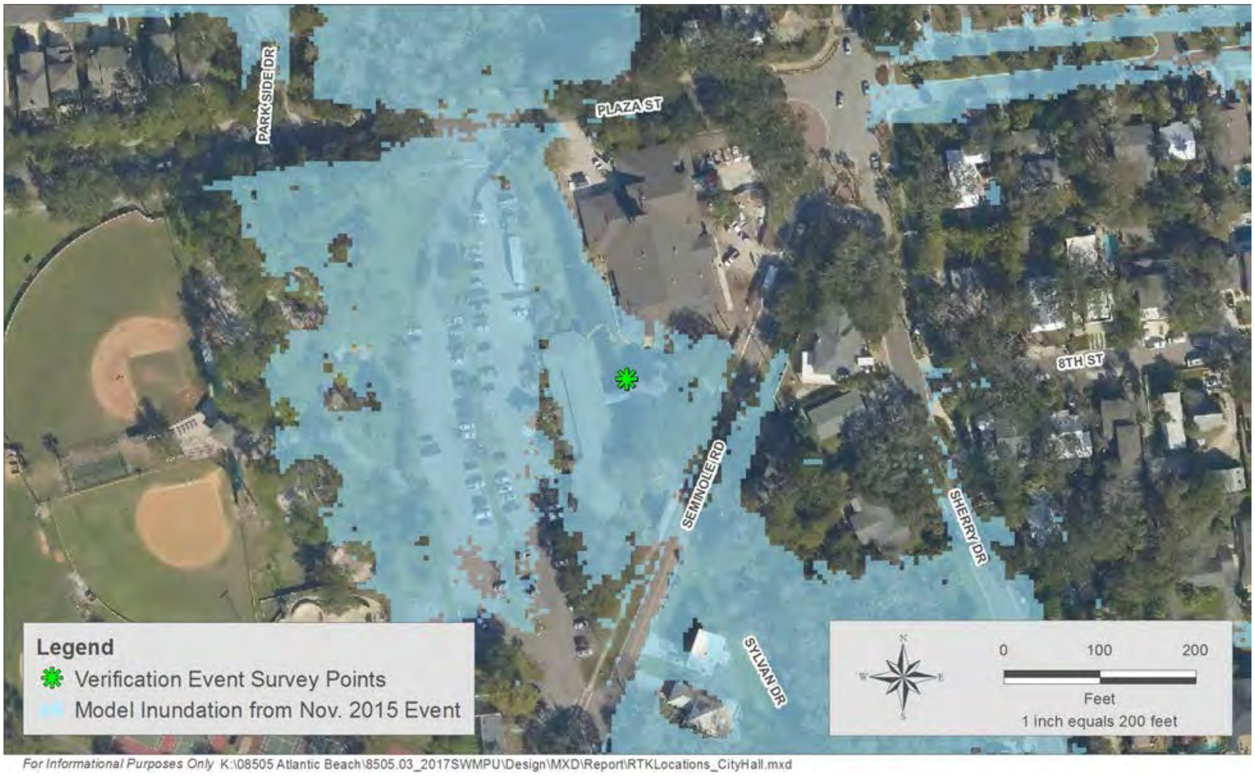
The third location where we compared modeled versus observed peak stages from the November 8, 2015, event was at the staff gauge on Sherman Creek at Atlantic Beach City hall. Photograph 4-4 shows the debris line on the staff gauge left from the flooding. From the photograph, the top of the debris line appears to be at approximately 7.9 on the staff gauge. Figure 4-7 shows the RTK survey elevation on the concrete deck, which corresponds to 9.7 on the staff gauge. The RTK elevation collected at this point was 8.5 feet NAVD88, which is approximately 1.2 feet below the staff gauge elevation. This means that the elevation of the top of the debris line shown in the photograph was at approximately elevation 6.7 feet NAVD88. The modeled peak stage in this location for the November 8, 2015, event was 6.9 feet NAVD88, which is within 0.2-foot of the observed peak stage on the staff gauge. This is well within industry standard acceptable tolerance ranges for considering a model validated.

Photograph 4-4 Staff Gauge after Nov. 8, 2015, Event

Figure 4-7 RTK Survey Location at City Hall Staff Gauge



Based on the comparisons of modeled versus observed peak stages at these three locations throughout the City, we were confident that the updated existing conditions ICPR4 model was representative of the City's drainage system well within the industry standard tolerance of ±0.5 foot.

5 FUTURE CONDITIONS H&H MODEL DEVELOPMENT

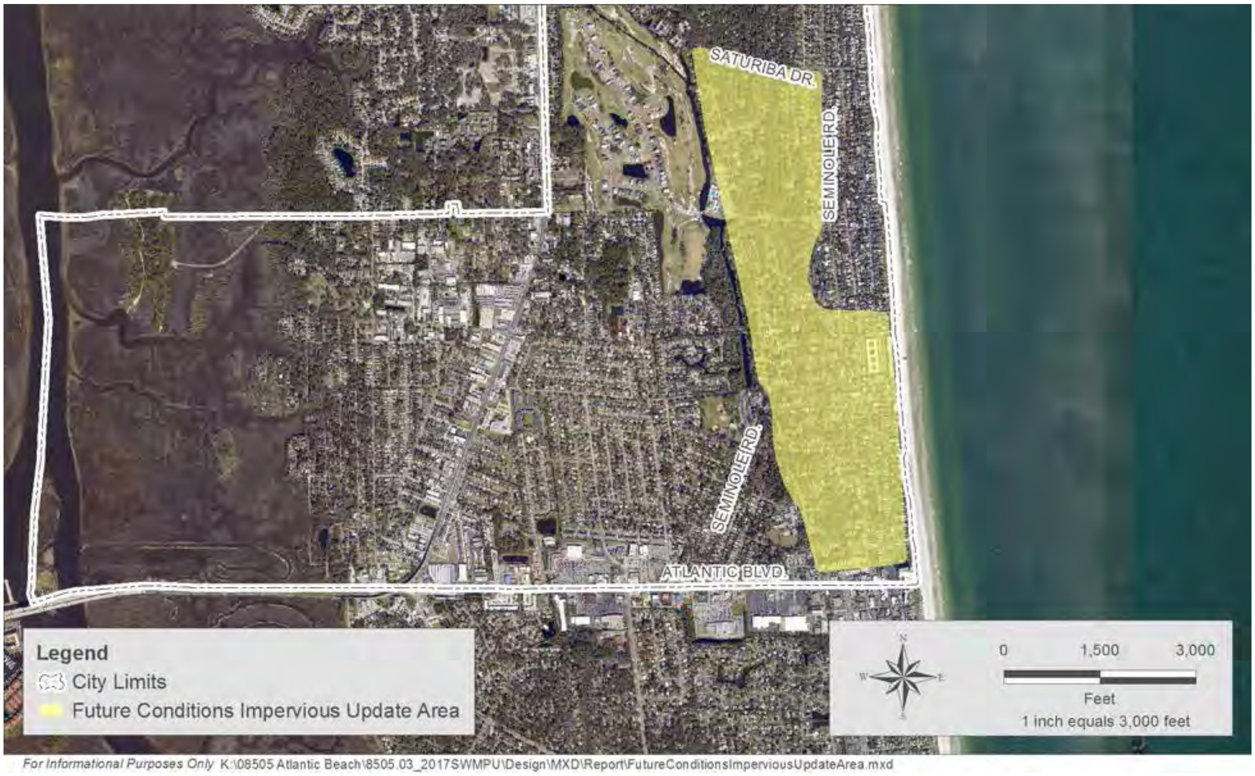
The City of Atlantic Beach is a coastal community completely surrounded by tidally influenced waterbodies that could significantly impact future drainage conditions within the City if sea levels continue to rise and sea-level-rise projections come to fruition. Also, the City has experienced areas of redevelopment, which is likely to continue. This redevelopment results in increased amounts of impervious area that will impact hydrologic conditions. Considering future drainage conditions when developing capital improvement projects that have service life spans up to 50 years is important. If feasible and cost effective, capital improvement projects can be sized appropriately to maintain or enhance the drainage level-of-service provided in projected drainage conditions.

We developed 2030 and 2045 Drainage Conditions Models to see what future drainage conditions within the City may be. We used these Future Conditions Models when developing capital improvement projects to determine if maintaining or improving the drainage level-of-service under future drainage conditions is feasible. To develop these models, we adjusted H&H parameters to reflect projected increases in impervious area from future development, increased boundary conditions and node initial conditions from rising sea levels, and reduction in soil storage from rising sea levels.

5.1 FUTURE IMPERVIOUS AREA UPDATES

We updated the basin CNs to reflect hydrologic conditions resulting from projected future increases in impervious area for each of the Future Conditions Models. Figure 5-1 shows the changes that were implemented in the residential area of the City. According to City staff, this area is where the City has experienced increases in imperviousness from developers buying large lots, splitting them into two separate lots, and building them out to about 50 percent imperviousness. For the 2030 model, we increased the impervious area to the full build-out 50 percent impervious for 40 percent of the lots that are not currently fully built-out. For the 2045 model, we increased 45 percent of the remaining 60 percent of the lots that are not currently built-out to 50 percent impervious. This method assumes that two-thirds of the remaining lots that are not already at 50 percent impervious area will be built-out by 2045 with a majority of the development occurring by 2030 and the development rate decreasing from 2030 to 2045 as lots available for increased imperviousness become scarcer. The new impervious area was applied to the basins spatially, so that it was correctly assigned based on the amount of parcels available for redevelopment in each basin. Overall, we added 12 acres of impervious area for the 2030 Conditions Model and 19 acres of impervious area for the 2045 Conditions Model.

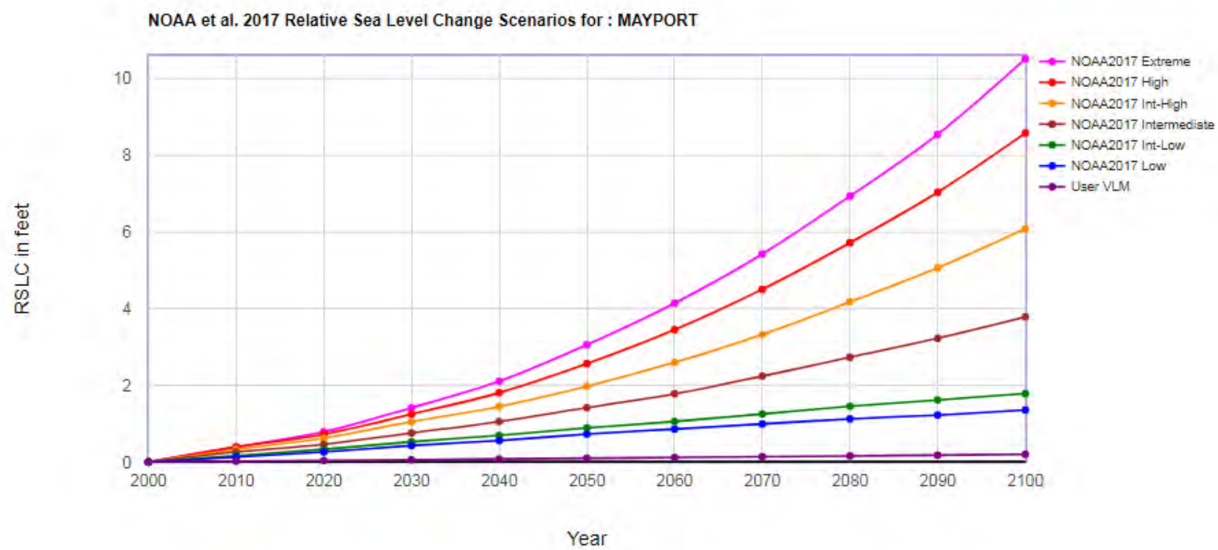
Figure 5-1 Future Conditions Impervious



5.2 Boundary Condition Updates From Sea-Level-Rise

Figure 5-2 shows the increased static tailwater condition elevations for each of the Future Conditions Models based on the 2017 NOAA Relative Sea-Level-Rise curves. These curves were developed specifically for the NOAA tide gauge at Mayport. These are the most recent and widely accepted predictions of sea-level-rise available. We used the intermediate curve, which resulted in an increase in boundary stage of 0.35 foot for the 2030 Conditions Model and an increase of 0.8 foot for the 2045 Conditions Model. Initial conditions for nodes directly connected to the boundary condition were also updated to match these projected increases.

Figure 5-2 NOAA Relative Sea-Level-Rise Curves for Mayport



Enter Project Name
Scenarios for MAYPORT
User Entered VLM: 0.00200 feet/yr
All values are expressed in feet

Year	User Entered VLM	NOAA2017 Low	NOAA2017 Int-Low	NOAA2017 Intermediate	NOAA2017 Int-High	NOAA2017 High	NOAA2017 Extreme
2000	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2010	0.02	0.13	0.17	0.26	0.33	0.40	0.40
2020	0.04	0.27	0.33	0.46	0.63	0.72	0.79
2030	0.06	0.43	0.53	0.76	1.05	1.25	1.41
2040	0.08	0.56	0.69	1.06	1.45	1.81	2.10
2050	0.10	0.73	0.89	1.42	1.98	2.57	3.06
2060	0.12	0.86	1.06	1.78	2.60	3.45	4.14
2070	0.14	0.99	1.26	2.24	3.32	4.50	5.42
2080	0.16	1.13	1.45	2.73	4.18	5.72	6.93
2090	0.18	1.23	1.62	3.23	5.06	7.03	8.54
2100	0.20	1.36	1.78	3.79	6.08	8.58	10.51

5.3 CN Updates From Soil Storage Reduction

We adjusted basin CNs to reflect hydrologic conditions with decreased soil storage from higher groundwater tables created by rising sea levels. We expect that groundwater levels will be higher because of consistently higher tides holding water back in the soil column. This will reduce the amount of soil storage available for runoff to infiltrate into and increase the amount of runoff during storm events. The decrease in soil storage will be more marked in areas directly adjacent to the coastline and will be reduced farther inland.

We assumed that locations directly connected to the boundary condition will experience groundwater table increases equivalent to the increases in boundary conditions from sea-level-rise in Section 5.2. We also assumed that the increase in groundwater table elevation will decrease at a linear rate and that the increases would become negligible and be 0 at 1 mile inland from the boundary condition. These assumptions were based on our engineering judgement. A detailed groundwater model would be required to understand the effects of sea-level-rise on groundwater table elevations but is outside the scope and budget of this project. We completed a search, but were not able to identify any research that had already been completed in this region to estimate the relationship between the increases in groundwater level from sea-level-rise versus distance from the boundary condition.

We calculated the increase in groundwater table elevation throughout the watershed based on this linear relationship. We used the increase in groundwater table elevation and the UF IFAS soil properties to calculate the soil storage reduction that would occur and the resulting total soil storage available for runoff infiltration. We then used TR-55 CN relationships to calculate the resulting open land CNs that would result from the reduced soil storage capacity. Finally, we recalculated weighted CNs for all of the basins that included the impervious area estimates discussed in Section 4.1.2 and the future increases in impervious area discussed in Section 5.1. The average CN increased from 81 in the Existing Conditions Model to 84 in the 2030 Conditions Model and 89 in the 2045 Conditions Model.

5.4 Future Conditions Model Results

We simulated the 10-, 25-, and 100-year/24-hour storm events using the ICPR4 2030 and 2045 Conditions Models. Table 5-1 summarizes the changes in peak stage results throughout the City. Figure 5-3, Figure 5-4, and Figure 5-5 compare the existing, 2030, and 2045 conditions inundation extents for the 10-, 25-, and 100-year storm events. As expected, modeled peak stage increases were greater for nodes closer to the boundary condition and decreased further inland. Also, modeled node peak stage increases were greatest in the 10-year event followed by the 25-year and 100-year events respectively, because the volume required to impact peak stages increases as stages increase. Low-lying storage areas fill up and the area of inundation spreads out and becomes larger, requiring more volume to produce the same relative peak stage increase than at lower stages.

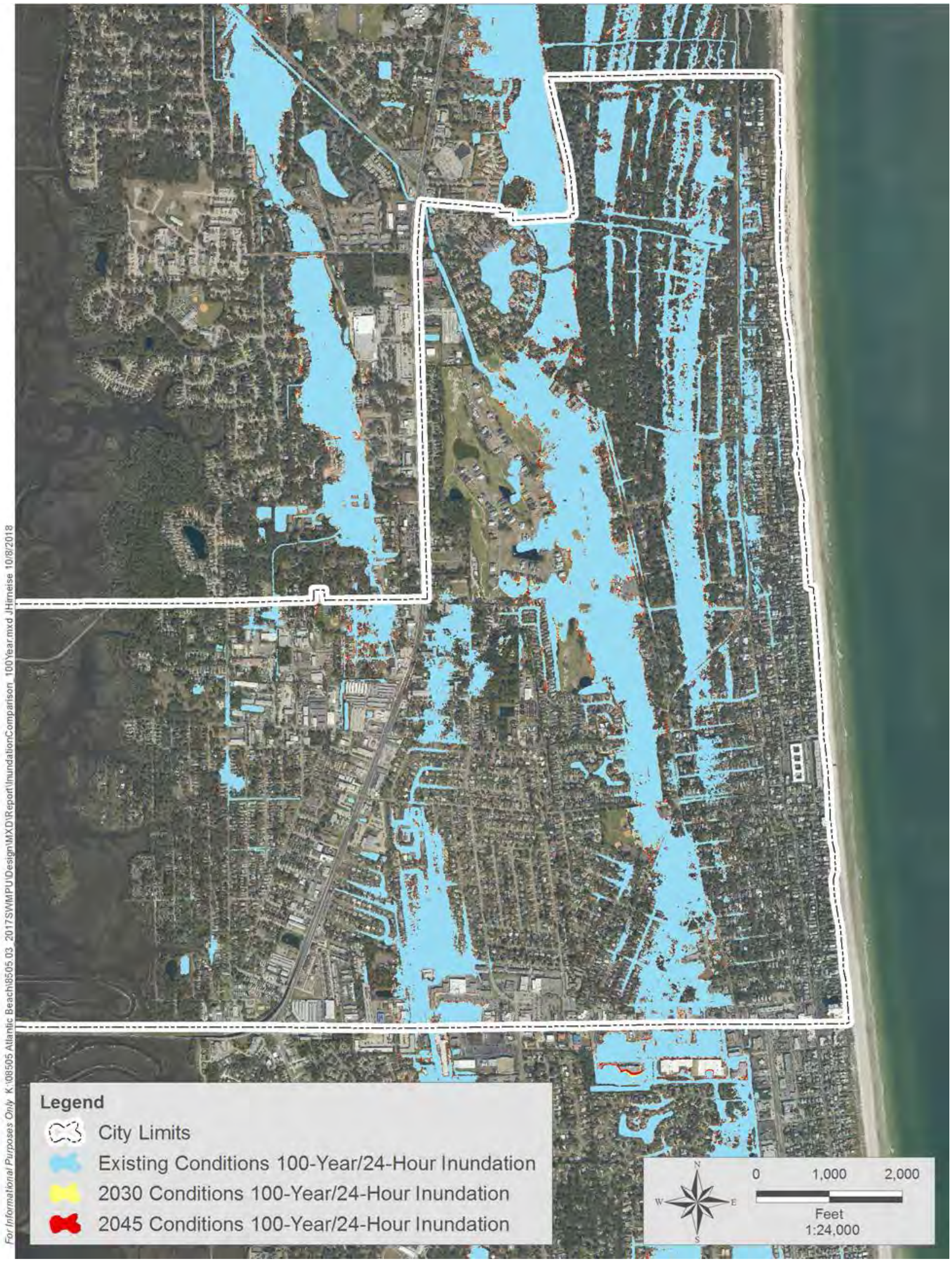
Table 5-1 Peak Stage Comparison Summary Table

	2030 Conditions	2045 Conditions
Average Peak Stage Increase From Existing Conditions (feet)		
10-Year/24-Hour	0.15	0.3
25-Year/24-Hour	0.1	0.2
100-Year/24-Hour	0.06	0.1
Maximum Peak Stage Increase From Existing Conditions (feet)		
10-Year/24-Hour	0.5	1.1
25-Year/24-Hour	0.3	0.7
100-Year/24-Hour	0.2	0.4

Figure 5-3 10-Year/24-Hour Inundation Comparison

Figure 5-4 25-Year / 24-Hour Inundation Comparison

Figure 5-5 100-Year / 24-Hour Inundation Comparison



6 CAPITAL IMPROVEMENT PROJECT AREAS

Based on a review of the H&H model results, historical information from previous City SWMP efforts, and conversations with City staff, the following problem areas were identified for developing capital improvement projects. For each of the identified problem areas, this section provides a detailed description of the problem area, a summary of the improvements considered, a description of the recommended improvements, and an engineering opinion of probable cost.

Jones Edmunds completed this analysis for the following areas:

- Hopkins Creek between Atlantic Boulevard and Plaza Street.
- Stanley Road, Dora Drive, and Simmons Road drainage system.
- West Plaza.
- Mary Street and Stewart Street.
- Constrictions outside the City's jurisdiction.

Hanson Professional Services completed this analysis for the following areas:

- 100/200/300 Blocks of Seminole Road South.
- Johansen Park.
- 9th/10th/11th/12th Streets.
- Salt Air and Howell Park.

6.1 HOPKINS CREEK BETWEEN ATLANTIC BOULEVARD AND PLAZA STREET

6.1.1 DESCRIPTION OF AREA

Figure 6-1 shows the Hopkins Creek area between Atlantic Boulevard and Plaza Street, which has well-documented flooding issues dating back several decades. Figure 6-2, which includes aerial imagery from 1943, shows that this area was developed within the historic floodplain and has significantly encroached upon and reduced the capacity of the natural flow-way that existed before development. This area sits at the confluence of Hopkins Creek, the Cavalla Road ditch, and the Saratoga-Forrestal ditch. These ditches drain approximately 350 acres of highly developed residential and commercial land that was built before modern stormwater regulations that required peak-flow attenuation and stormwater treatment. This is also a very low-lying tidally influenced area with roadway elevations as low as 4 feet NAVD88 and residential FFEs between 6 and 6.5 feet NAVD88.

The effect of tidal conditions in the ICW on drainage in this area was analyzed as part of the 2002 SWMP Update completed by CDM. The analysis showed that extreme tidal conditions did not have significant impact on modeled peak stages in rainfall events with return periods greater than 10 years. This is because Atlantic Beach is approximately 1.7 miles upstream of the ICW, which allows the tidal effects to be absorbed by the system (CDM, 2002). Jones Edmunds also completed a sensitivity analysis using the ICPR4 model to test the sensitivity of the system to tidal conditions. We simulated the November 8, 2015, rainfall event with observed tailwater conditions in the ICW and with tailwater conditions at the mean low low water level. The differences in modeled peak stages upstream of the box culvert under Atlantic Boulevard between these simulations were less than 0.1 foot.

One stormwater pond was constructed in this area to serve the Aquatic Drive townhome development. During high flow events, the pond and ditch banks are overtopped and the pond takes on water from the adjacent ditches. This pond was also designed and constructed before modern stormwater regulations and has a 19-inch-by-30-inch elliptical pipe outfall with inverts below the MHHW elevation. The City has modified this structure and installed a weir on the face of the upstream end of the pipe allowing them to pump the pond water level down before large storm events to maximize the storage volume available for stormwater runoff.

If this area were developed to meet today's stormwater permitting regulations, we estimate that 10 to 15 percent, or approximately 35 to 50 acres of the total 350-acre contributing area, would need to be set aside for stormwater ponds to provide peak flow attenuation and treatment. Currently, this area has less than 10 acres or 20 to 30 percent of the pond area required under current stormwater regulations.

The lack of storage volume in the upstream contributing area causes stormwater to run off at a very high rate. Runoff accumulates in the ditches and quickly exceeds the flow capacity of the drainage ditches and culvert crossings along the ditches. This causes the water to back up in the ditch system and spill out into the surrounding area, effectively using the roadways and homes as storage instead of stormwater ponds.

All of these factors have combined to create very poor and even dangerous drainage conditions in this area. During the November 8, 2015, rainfall event discussed in Section 4.3, flood elevations exceeded the FFE of more than 100 residences with some having water at or above electrical outlets. This event caused millions of dollars in damages and was a hazard to public health and safety. The flooding resulted from an extreme storm event, but less-extreme, shorter-duration storms regularly cause stormwater to pond on Aquatic Drive and begin to encroach on residential properties. Figure 6-1 shows the modeled peak inundation for this area in the mean-annual, 10-year, and 25-year return period storm events.

Figure 6-1 Existing Drainage Condition Summary

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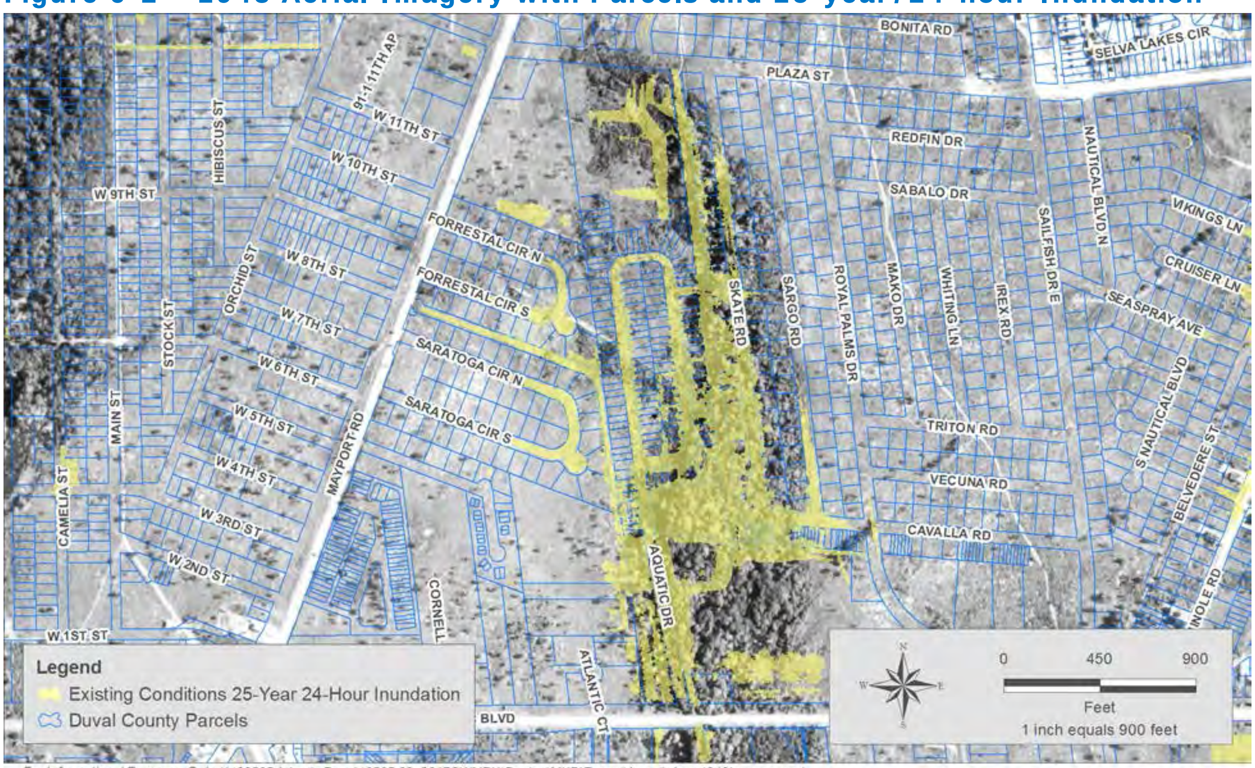
Figure 6-2 1943 Aerial Imagery with Parcels and 25-year/24-hour Inundation



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6.1.2 Improvements Considered

This area was reviewed in the City's 1995 SWMP and 2002 SWMP Update. The 1995 Master Plan identified it as a problem area, but did not propose any alternatives to alleviate the flooding problems noting that the cost would be significant and that coordination with adjacent communities such as the City of Jacksonville and Neptune Beach would be required and would be difficult to achieve. As part of the 2002 SWMP Update, CDM developed a detailed model of this area and proposed three alternatives to improve drainage conditions. Alternative 1 was to build a regional stormwater pond on several vacant lots west of Aquatic Drive just north of Atlantic Boulevard to divert water from Hopkins Creek during high flow events. A smaller version of this alternative, the Hopkins Creek Regional Stormwater Facility, has been constructed by the City. The second alternative was to be constructed with Alternative 1 and included culvert capacity improvements at the Cutlass Road crossing of Hopkins Creek. The third alternative, which CDM did not recommend, was creating a new outfall from Hopkins Creek to the ICW. This outfall would require a large pump system because not enough gradient across the landscape is available to drain via gravity outfall. The benefit from this proposed project was not considered worth the $12.2 million estimated cost in 2002. CDM also considered the benefits of installing backflow prevention in Hopkins Creek to diminish the impacts of high tide conditions. Based on the analysis, they determined that backflow prevention only provides flood stage benefits during small storm events with extreme high tides, and that backflow prevention actually increased stages in larger storm events because of the headloss added by the measures. They did not recommend using backflow prevention measures in Hopkins Creek.

We investigated several other alternatives to alleviate flooding in this area as part of this Master Plan Update. We used the updated ICPR4 model to test the benefit provided from the following improvements:

- Improving box culvert and channel capacity in Hopkins Creek downstream of the Aquatic Drive pond.
- Constructing additional on-line storage capacity along Hopkins Creek and the Cavalla Road ditch.
- Combining culvert and channel capacity improvements downstream of the Aquatic Drive pond and constructing additional on-line storage capacity.
- Purchasing homes adjacent to the Aquatic Drive pond, expanding and deepening the pond, and installing a pump station on the pond to pump down the water level before storm events for increased storage capacity.
- Improving culvert and channel capacity with pond improvements and a pump station at the Aquatic Drive pond.
- Constructing a new large pipe outfall down Royal Palm Drive that would intercept flow to Hopkins Creek and the Cavalla Road ditch from the residential area to the east and route it to the upstream end of the Atlantic Beach box culverts. This would divert flow away from the existing ditch systems.
- Constructing a backflow prevention structure on Hopkins Creek and pumping down the ditch system before large storm events to maximize the storage capacity in the ditch.
- Reducing the amount of runoff through low-impact development stormwater measures such as permeable pavement or bioswales.

6.1.3 Recommended Improvement

After reviewing the benefit from all of the alternatives listed in Section 6.1.2, we recommend that the City makes the following improvements to the drainage system in this area:

- Replace the existing 6-foot-by-8-foot box culvert under the west entrance to the Atlantic Village shopping center with double 6-foot-by-8-foot box culverts or larger to match the hydraulic capacity of the downstream culverts under Atlantic Boulevard. This crossing was identified as a major constriction based on the modeling results.
- Stabilize and widen Hopkins Creek from Atlantic Boulevard to the confluence with the Cavalla Road ditch. This includes a vertical wall rectangular typical section with a channel width of 30 feet, or the maximum width possible given the constraints of the site. For this analysis, use of sheet piling to construct the new channel section was assumed. Sheet piling is expensive and may be difficult to place because of the overhead powerlines adjacent to the ditch along Aquatic Drive. Additional site-specific analysis will be needed to determine the exact channel improvements that are feasible and the ideal vertical wall stabilization method that should be used.
- Deepen the existing Aquatic Drive stormwater pond and install a stormwater pump station to draw the pond water level down before large storm events and provide more storage volume in the pond. This will also require re-establishing the pond berm to a consistent elevation, removing the existing outfall pipe on the pond, and installing a new outfall structure. For costing this alternative and modeling the flood benefits, we assumed that the pond would have a bottom elevation of approximately -5 feet NAVD88 and that the pond would be drawn down to an elevation of -1 foot NAVD88 before major storm events.
- Acquire adjacent properties to the north and/or south of the existing Aquatic Drive pond and expand the footprint of the pond to increase the pond storage. If the property south of the pond is acquired, reshaping of Hopkins Creek to reduce head-loss from sharp bends in the existing ditch should also be evaluated. If the City plans to pursue this option following the deepening of the pond and construction of the stormwater pump station, they will need to ensure that the pump station is sized adequately to accommodate the new storage volume before the pond expansion.
- Replace the existing 4-foot-by-6-foot box culvert under Cutlass Drive with double 4-foot-by-6-foot box culverts or larger.

Figure 6-3 provides the location and layout of all of the proposed improvements.

Figure 6-3 Proposed Improvements

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Due to budget constraints and costs of the proposed improvements, we recommend that the City implement these improvements using a phased approach based on available City capital improvement funds and grant funding opportunities. The order in which these improvements should be implemented will depend on available funding. The City is applying for $2,000,000 in funding for the Aquatic Drive area from the Federal Emergency Management Agency's (FEMA) Hazard Mitigation Grant Program (HMGP) for money that will be released to mitigate flood hazards following Hurricane Irma. If the City is awarded this funding, the plan is to complete Phase 1 of the improvements in this area by implementing the following improvements:

- Improve the culvert capacity at the west entrance to the Aquatic Village shopping center.
- Deepen the Aquatic Drive stormwater pond, install a stormwater pump station, replace the existing outfall structure, and re-establish the pond berm.
- Implement minor improvements to Hopkins Creek to enhance flow capacity. This does not include reshaping the ditch to have vertical walls with a greater channel width as proposed above.

Once Phase 1 has been constructed, future phases can be implemented depending on available funds and property acquisition opportunities. The improvements have been prioritized in the proposed 10-year CIP in Section 7 of this Report.

Exhibits 1A, 1B, and 1C in Appendix A, Capital Improvement Projects Exhibits, summarize the Phase 1 improvements including peak stage reductions. Exhibits 2A, 2B, and 2C in Appendix A summarize all of the improvements and the peak stage reductions if all of the improvements were implemented.

Table 1 in Appendix B, Engineer's Opinion of Probable Cost, provides an opinion of probable cost in 2018 dollars for the recommended improvements that will be completed in Phase 1. Individual estimates are also provided in Tables 2, 3, and 4 in Appendix B for each of the additional recommended improvements that should be completed following Phase 1. Property acquisition costs were not included in the detailed cost estimates, but were estimated based on the Just Market Value from the Duval County Property Appraiser's website. The property south of the pond has a just market value of approximately $600,000, and the properties north of the pond have a combined just market value of approximately $1,200,000. Cost estimates were developed based on ASTM E2516, Standard Classification for Cost Estimate Classification System, Class 4 cost ranges. The estimated cost to complete design and construction of the Phase 1 improvements is $1,700,000 to $2,800,000.

6.2 Stanley Road, Dora Drive, and Simmons Road Drainage System

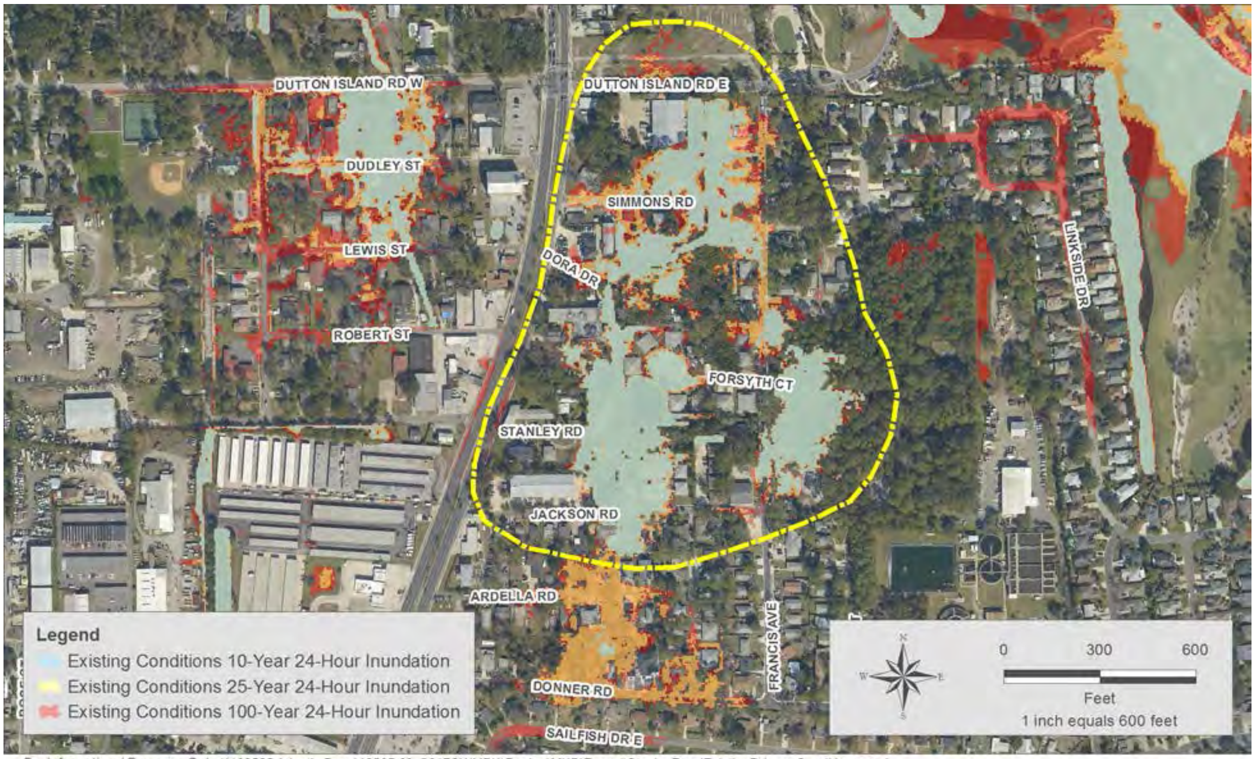
6.2.1 Description of Problem Area

Figure 6-4 shows the Stanley Road, Dora Drive, and Simmons Road drainage area, which consists of approximately 40 acres of medium- and low-density residential, commercial, and open land land uses. This area drains to a ditch system that runs 1,200 feet north to south from Dutton Island Road to Stanley Road. The ditch outfalls through a 15-inch pipe at the end of Stanley Road.

Drainage issues have been well documented in this area and are identified by the City as a drainage priority area. Residents in the apartments at the end of Stanley Road experienced flooding in their homes and widespread roadway flooding occurred during the November 8, 2015, rainfall event. Photograph 6-1 and Photograph 6-2 show the flooding that occurred in this area during the November 8, 2015, event.

Most of this area was developed before modern stormwater regulations were in place, and the City's stormwater system was not constructed with enough storage or hydraulic capacity to handle the runoff. The existing 15-inch outfall pipe is undersized for the amount of flow in the ditch, which causes water to backup and pond in low-lying areas upstream. This causes residential property to flood and hazardous roadway conditions. Figure 6-4 shows the modeled peak inundation for this area in the mean-annual, 10-year, and 25-year return period storm events.

Figure 6-4 Existing Drainage Condition Summary



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Photograph 6-1 Flooding On Simmons Road During November 2015 Event

Photograph 6-2 Flooding at 94 Stanley Road During November 2015 Event

6.2.2 Improvements Considered

This area was previously reviewed as part of the Hopkins Creek study area in the 2002 SWMP Update project completed by CDM. Most of the analysis and alternatives proposed in the Hopkins Creek basin focused on the area between Cutlass Drive and Atlantic Boulevard, but CDM did propose purchasing the apartment complexes at the end of Stanley Road that have previously flooded. In the 2002 report, CDM estimated a purchase price of $300,000 and noted that this would be the optimal option for this area. No additional alternatives were presented in the previous master plan reports for alleviating the flooding issues at this location.

We investigated several other alternatives to alleviate flooding in this area as part of this Master Plan Update. We used the updated ICPR4 model to test the benefit provided from the following improvements:

- Replacing the 15-inch outfall pipe at Stanley Road with a larger diameter pipe to provide more outflow capacity.
- Constructing an on-line storage basin at the end of Stanley Road east of the 15-inch outfall pipe.

- Constructing an on-line storage basin in the open lot at the end of Dora Drive.
- Constructing an on-line storage basin in the open lot directly north of Dutton Island Road east of Mayport Road.
- Constructing an on-line storage basin on the City-owned parcels north of Jordan Park on Francis Avenue.
- Constructing an on-line storage basin on the City-owned parcels north of Jordan Park on Francis Avenue and upsizing the 15-inch outfall pipe at Stanley Road.

6.2.3 Recommended Improvement

After reviewing the benefit from all of the alternatives listed in Section 6.2.2, we recommend two options for the City's consideration:

- Figure 6-5 shows Option 1, which includes replacing the existing 15-inch pipe from Stanley Road to Donner Road with a 36-inch pipe along the existing corridor. Based on spatial data for the 2016 Duval County Parcels, a 12-foot drainage easement appears to run along the proposed pipe corridor between existing homes. This easement will need to be verified before pursuing this option. Removing the existing pipe and installing a new 36-inch pipe along this corridor will be extremely challenging and disruptive to many residents who live on the adjacent parcels. The removal of several large trees, privacy fences, and other privately-owned structures will also be required to complete this alternative. Other corridors for the new pipe were considered but are not feasible due to a lack of gravity head from the upstream end of the pipe to the elevation of the existing pipe on Donner Road where the new pipe would tie into. Additionally, Francis Avenue has recently been reconstructed and cannot be disturbed to install a new pipe under it. Exhibits 3A, 3B, and 3C in Appendix A summarize the Option 1 improvements including peak stage reductions.

Figure 6-5 Proposed Option 1 Alternative Layout

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- Figure 6-6 shows Option 2, which would require purchasing the two repetitive loss properties at 91 and 94 Stanley Road, as recommended in the 2002 Report. A history of several instances of flooding in these residences is available. These properties could be purchased by the City if the owners were willing to sell and the land could be put into conservation, or the City could have them graded to provide more floodplain storage for the area.

Figure 6-6 Repetitive Loss Properties at Stanley Road

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Table 5 in Appendix B provides an opinion of probable cost in 2018 dollars for recommended Option 1. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. For construction and engineering services for the proposed Option 1 improvement, the cost is estimated to be $400,000 to $640,000. The estimated cost of acquiring the properties at the end of Stanley Road as proposed in Option 2 is $600,000 based on the Just Market Value of the properties obtained from the Duval County Property Appraiser's website.

6.3 West Plaza

6.3.1 Description of Problem Area

The West Plaza ditch is an approximately 750-foot-long tidally-influenced ditch that runs north to south behind the homes on Gladiola Street (Figure 6-7). The ditch collects runoff from the surrounding low-density residential and open land area and discharges it into the ICW. The ditch is connected under West Plaza by a 15-inch pipe. Elevations in the bottom of the ditch range from 0.0 to 1.0 foot NAVD88, the roadway crown elevation on West Plaza is approximately 3.0 feet NAVD88, and the elevations of residents' yards on Gladiola Street are approximately 2.5 to 3.0 feet NAVD88 based on the 2007 LiDAR DEM.

The ditch is inundated daily during high tides. The roadside swales, residents' yards, and Gladiola Street are inundated intermittently during more extreme high tides causing resident complaints of nuisance flooding and potentially damaging flooding if extreme rainfall occurs during high tide events. Figure 6-8 shows the inundation in this area resulting from tide elevations between 1 and 3 feet NAVD88.

Figure 6-7 West Plaza Ditch

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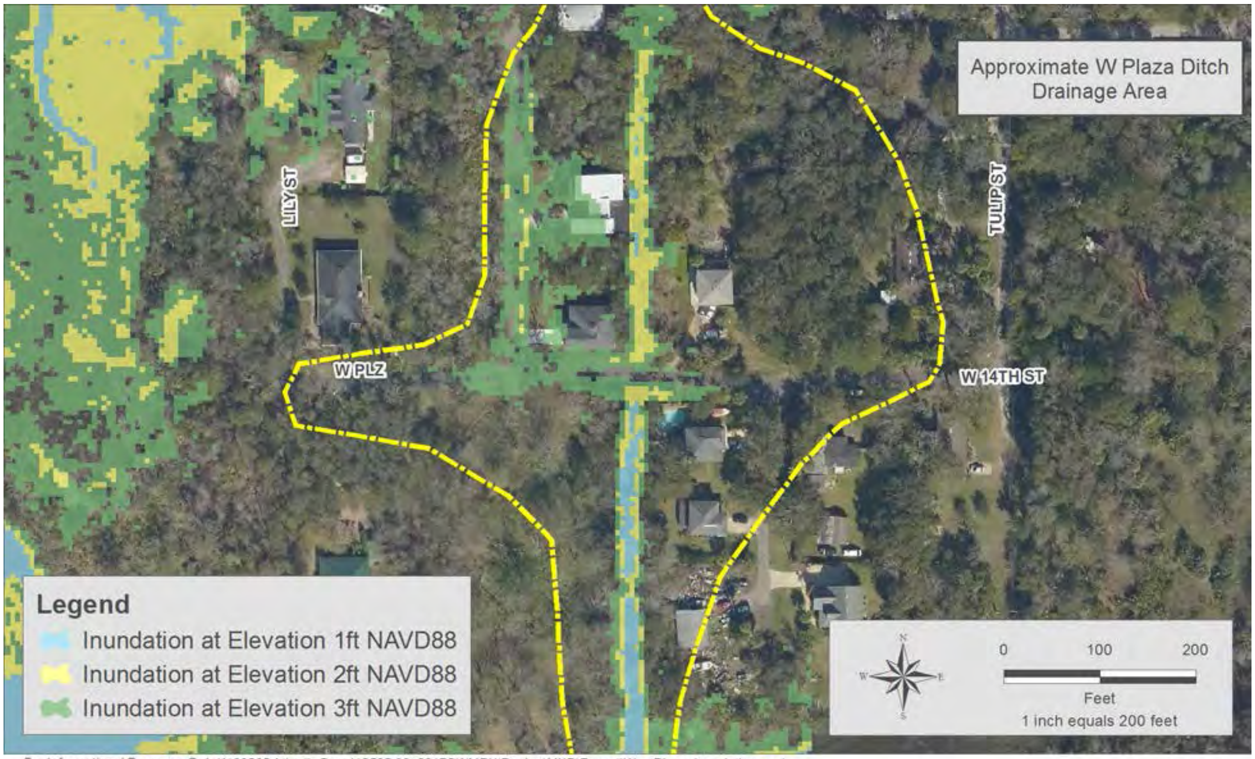
Figure 6-8 Inundation at Tide Elevations Between 1 and 3 Feet NAVD88



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6.3.2 Improvements Considered

The West Plaza drainage area was identified as a nuisance drainage problem and improvements were proposed as part of the 2012 SWMP Update completed by CDM. The improvements proposed in the plan were centered on reducing nuisance flooding at the intersection of West Plaza and Carnation Street, but no improvements were proposed for the flooding on Gladiola Street during high tides. The CDM report states that the City had already made several improvements in the West Plaza and Gladiola area and that this area would be monitored to ensure the completion of improvement projects.

Nuisance flooding issues on Gladiola Street have persisted since the 2012 SWMP Update and the City has continued to receive complaints from residents. We investigated the feasibility of placing a tidal backflow preventer valve on the pipe under West Plaza Street to reduce nuisance flooding from normal tidal conditions.

6.3.3 Recommended Improvement

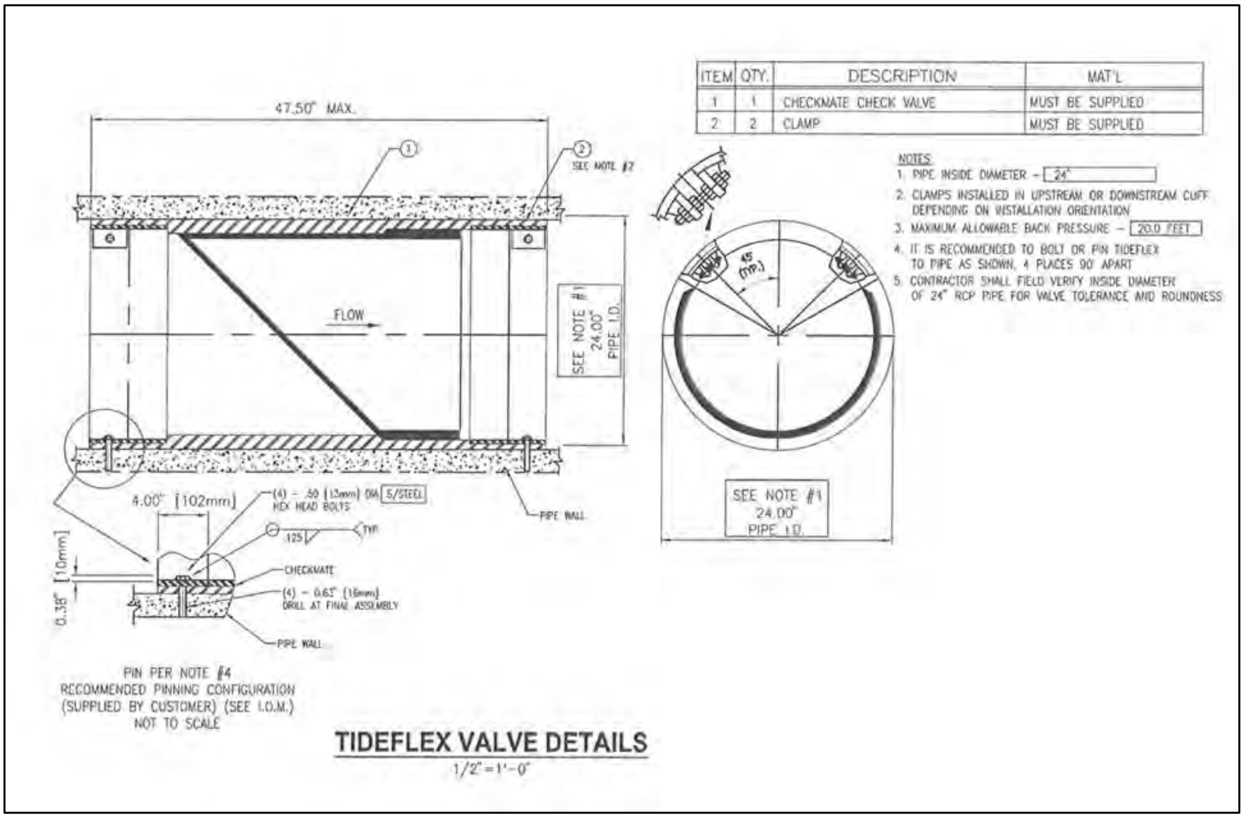
Figure 6-9 shows Jones Edmunds' recommendation of placing a Tideflex inline backflow preventer valve immediately upstream or downstream of a new manhole installed on West Plaza. This would effectively cut off the flow of tidal water into the portion of the ditch north of West Plaza until tides overtopped West Plaza at approximately elevation 3.0 feet NAVD88. Water would only be able to flow out of the ditch north of West Plaza through the pipe when water surface elevations north of West Plaza are higher than water elevations south of West Plaza. Placing the valve adjacent to a manhole will allow City crews to easily access the valve to remove sediment and perform routine maintenance on the valve. Figure 6-10 provides an example schematic of what the Tideflex valve looks like. This improvement would help to eliminate nuisance flooding from normal high tides but will not prevent flooding or change peak flood stages during extreme rainfall events.

We recommend that the City confirm through a field visit or survey data collection that water does not enter the ditch from the ICW on the north end of the ditch. This cannot be confirmed from the 2007 LiDAR DEM and this portion of the ditch is not publicly accessible. If water is able to flow into the ditch from the north end, the ditch would need to be plugged to isolate it from the ICW. This alternative will not be viable if water is able to enter the ditch on the north end.

Table 6 in Appendix B provides an opinion of probable cost in 2018 dollars for the conceptual capital improvement. Cost estimates were developed based on ASTM E2516 Class 4 costs ranges. For construction and engineering services for the proposed improvement, the cost is estimated to be $53,000 to $86,000.

Figure 6-9 West Plaza Proposed Alternative Layout

Figure 6-10 Tideflex Inline Valve Schematic



6.4 MARY STREET AND STEWART STREET

6.4.1 DESCRIPTION OF PROBLEM AREA

Mary Street is a short dead-end roadway immediately south of Dutton Island Road along the northwest boundary of the City. It is accessed from Stewart Street and contains approximately ten duplex housing units. The drainage area for the subbasin containing Mary Street is relatively small at 2.3 acres, and runoff generated within the subbasin collects in roadside swales on Mary Street and ultimately ponds in the low area in and around the intersection of Mary Street and Stewart Street.

Currently, the drainage infrastructure within the subbasin consists of swales and small-diameter driveway culverts. Jones Edmunds identified two directions of positive outfall for the subbasin – one culvert on the south side of Stewart Street connecting a swale to the subbasin to the west and one culvert/swale combination at the north end of Mary Street connecting to the swale along the south side of Dutton Island Road. However, during a field visit to this site, we noticed that both of the culvert connections have been constricted due to crushed ends and sediment/vegetation buildup, and the swale has localized high points restricting flow. As a result, water must stage up and flow over high points along the basin boundary.

An open ditch is in the subbasin directly east of the problem area and is the main drainage outlet for the surrounding industrial development. The ditch ultimately drains north to the ICW through a series of culverts, ditches, and ponds. The drainage system west of the problem area consists of a series of roadside swales and driveway culverts, which also ultimately discharge into the ICW. The drainage system to the north of the problem area consists of a series of roadside swales and culverts. Figure 6-11 shows the 25-year/24-hour inundation in this area.

Figure 6-11 Mary Street Existing Conditions

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6.4.2 Improvements Considered

Jones Edmunds reviewed the City's 1995 SWMP and the subsequent SWMP Updates written in 2002 and 2012 to determine whether the intersection of Mary Street and Stewart Street had previously been analyzed for improvements. The City's 2012 SWMP Update identified the intersection of Mary Street and Stewart Street as a problem area due to standing water in the intersection. The 2012 SWMP Update ranked problem locations provided by the City based on criteria designed to identify areas with the highest potential impact to the City and developed conceptual capital improvements for the top 15 locations. However, the Mary Street/Stewart Street area ranked outside the top 15 locations; therefore, no further analysis was provided.

Jones Edmunds considered several options to mitigate flooding in this area during our analysis, including:

- Replacing drainage structures.
- Installing new drainage structures.
- Improving swale and open channel conveyance.
- Adding storage volume.

We had to consider impacts to the adjacent subbasins should stormwater improvements increase flows or volumes in either direction. Flooding has been reported in the subbasins to the east and southwest of the Mary Street/Stewart Street intersection. The 2012 SWMP

Update ranked the two problem areas – Main Street between Levy Street and Stewart Street, and Mealy Street industrial area and Dudley Street west of Donner Park – as Numbers 8 and 9, respectively. Because of the location of the problem area relative to the previously identified adjacent areas of concern, we determined that routing water either east or west would not be ideal. Additionally, we performed a site visit to the problem area to inspect the existing system and investigate possible alternatives to mitigate flooding. During the field reconnaissance, the originally intended outfall direction for Mary Street appeared to be to the north through the combined ditch/swale system and to the west through the culvert on the south side of Stewart Street. As the originally intended stormwater outfalls were determined to be north and west, combined with the difficulty of routing stormwater east without impacting adjacent homes and the lack of constructability, we recommend restoring the stormwater system through maintenance measures that include replacing existing culverts, expanding swales, and adding new culverts to improve hydraulic connectivity.

6.4.3 Recommended Improvement

The following improvements are recommended for the Mary Street/Stewart Street intersection to alleviate flooding during large storm events:

- Replace the 12-inch reinforced concrete pipes (RCPs) under the driveway on the south side of Stewart Street, east of Mary Street.
- Install two 12-inch RCPs under Stewart Street, one on each side of Mary Street.
- Replace the 12-inch outfall culvert on the northwest side of Mary Street.
- Replace the driveway culverts on both sides of Mary Street with 12-inch RCP.
- Replace corrugated metal pipe on the southeast corner of Stewart Street and Main Street with 12-inch RCP.
- Clean and regrade swales on the south side of Stewart Street and both sides of Mary Street.

Figure 6-12 and Exhibit 4 in Appendix A show the layout of the proposed improvement and a summary of the peak stage and inundation duration benefits. Due to the limited storage capacity within the basin and limited space for storage expansion, the modeled peak stages for the proposed condition are only slightly lower than those in the existing conditions model. However, the proposed improvements allow the water to drain out of the basin at a higher rate and lower elevation, thereby greatly reducing the amount of time the roadways within the basin are inundated. The existing conditions model shows inundation lasting the entire duration of the simulation due to the overland weir elevations leaving the basin being greater than the lowest points of the roadway centerline. However, the roadway centerline is only inundated for a maximum of 2 hours in the proposed conditions model simulations.

Figure 6-12 Mary Street Improvement Layout

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A right-of-way permit will be required to complete the construction detailed in the recommended improvement. The majority of the proposed work would occur within City-owned easements and rights-of-way. However, a number of driveways, sidewalks, and roadways would require repair after installation of the proposed drainage improvements. Overhead utility lines throughout the area may also inhibit movement of large equipment during construction and some portions of the overhead utility system may require relocation. A City-operated lift station is also immediately east of the proposed improvements; therefore, sanitary sewer utility conflicts may arise.

Table 7 in Appendix B provides an opinion of probable cost in 2018 dollars for the conceptual capital improvement. Cost estimates were developed based on ASTM E2516 Class 4 costs ranges. For construction and engineering services for this conceptual alternative, the cost is estimated to be $280,000 to $450,000.

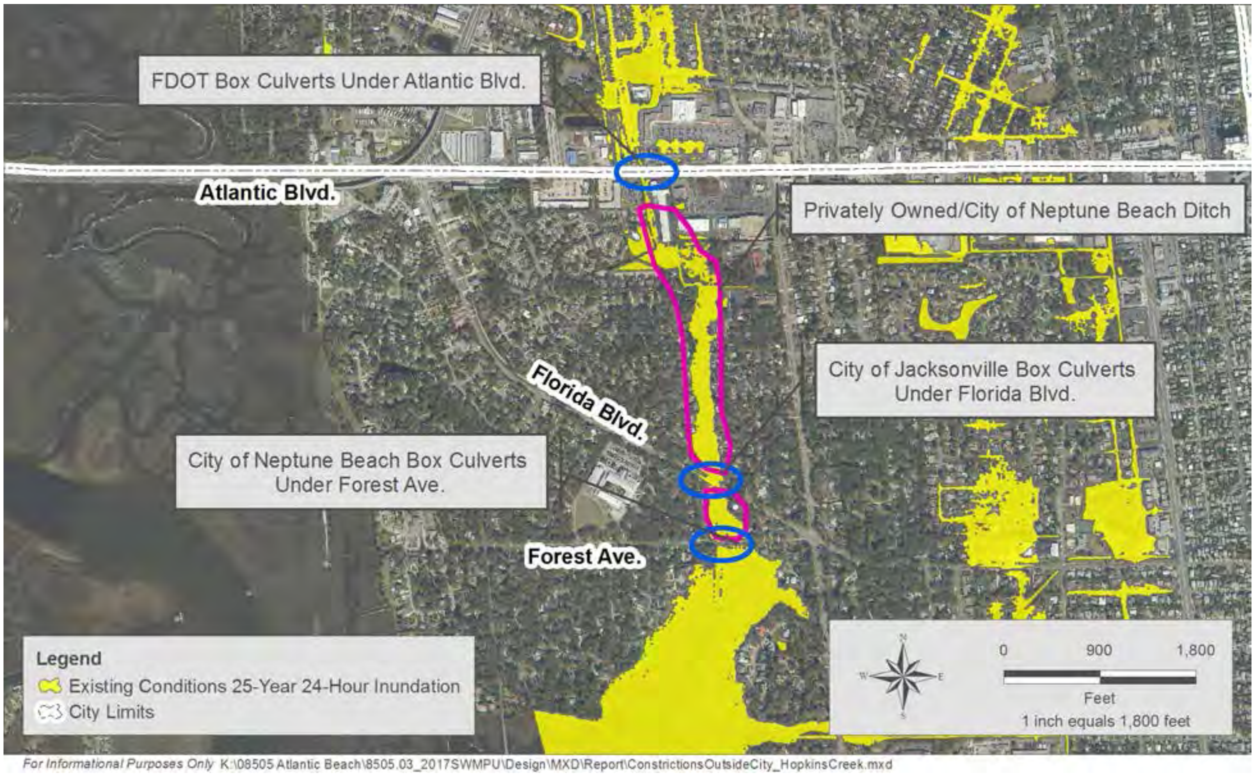
6.5 Constrictions Outside The City's Jurisdiction

6.5.1 Description of Problem Area

6.5.1.1 Hopkins Creek

Hopkins Creek and Sherman Creek outfall to the ICW through drainage features that are not owned, operated, or maintained by the City, and in some cases these features are undersized, restricting flow leaving the City. Hopkins Creek outfalls from the City to the south through box culverts under Atlantic Boulevard owned by FDOT and proceeds through a combination of ditches and culverts owned by private entities, the City of Neptune Beach, or the City of Jacksonville. Figure 6-13 shows the location and ownership of drainage features outside the City on Hopkins Creek.

Figure 6-13 Hopkins Creek Drainage Features Outside of the City



Upgrades to the hydraulic capacity in Hopkins Creek upstream of the Atlantic Boulevard box culverts are required before any of the constrictions outside the City become the critical choke-points in the Hopkins Creek drainage system. According to the updated existing conditions model, the double 6-foot-by-8-foot box culverts under Atlantic Boulevard are not a significant flow constriction in the storm events with return periods less than the 100-year event. Head losses across these culverts are less than 0.3 foot in the mean-annual, 10-, and 25-year events, but in the 100-year event the head loss increases to approximately 0.7 foot (8 inches) because more significant flow constrictions are in the ditch immediately upstream of these culverts that have less hydraulic capacity. The head loss across these culverts could increase in the smaller events if the flow capacity in the upstream ditch were increased causing this crossing to become the primary constriction. These considerations are important when developing upstream alternatives.

Modeled head losses in Hopkins Creek from the downstream end of the box culverts at Atlantic Boulevard to the upstream end of the box culverts at Florida Boulevard range from 0.8 foot in the mean-annual event to 1.0 foot in the 100-year event. The north half of this ditch is privately-owned and the south half is maintained by the City of Neptune Beach. Any hydraulic capacity upgrades to these ditches would have to be made in combination with upstream hydraulic capacity upgrades for the benefit to be realized within City limits. The existing conditions stormwater model assumes that these ditches are well maintained. If the hydraulic capacity of these ditches becomes severely inhibited by debris or other obstructions, they could potentially create a restriction that impacts peak stages in the City.

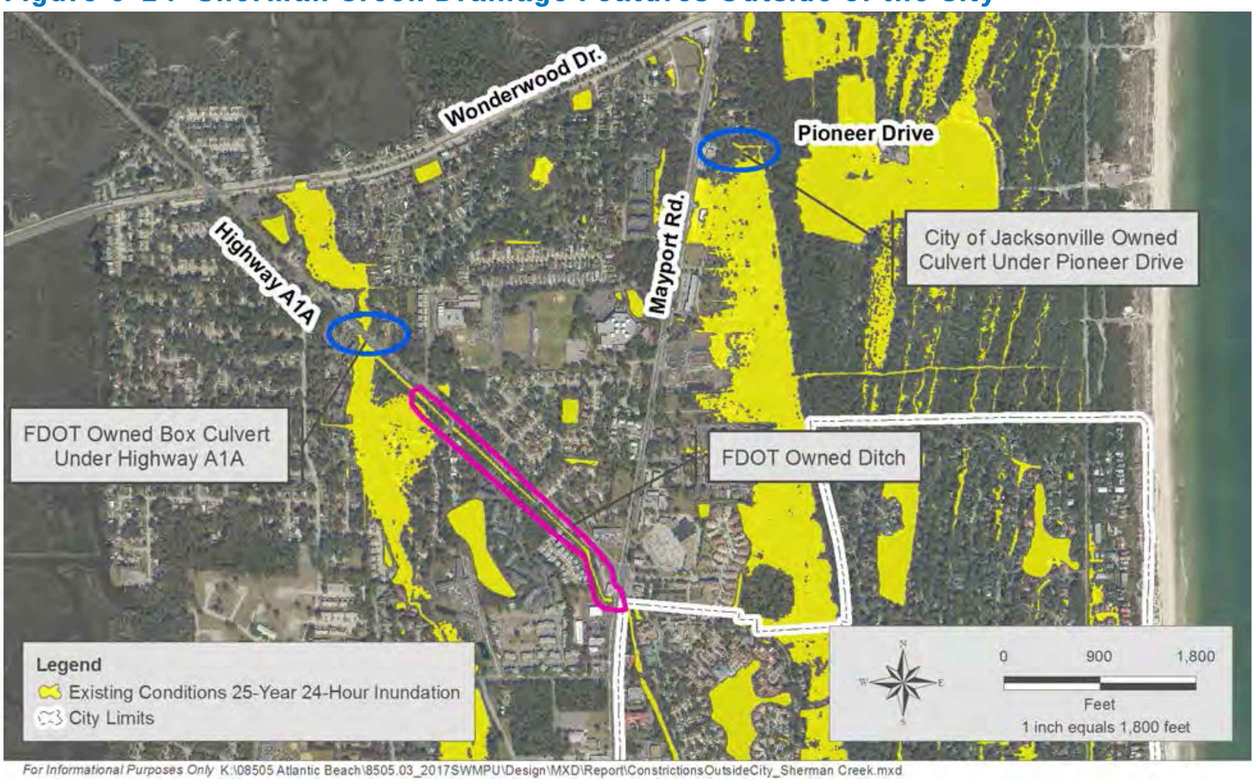
Modeled head loss in Hopkins Creek from the upstream end of the culverts at Florida Boulevard to the downstream end of the culverts under Forest Avenue range from 0.2 foot in the mean-annual event to 0.6 foot in the 100-year event with the majority occurring at the double 6-foot-by-8-foot culverts under Forest Avenue. These culverts are owned and maintained by the City of Neptune Beach, and the head loss across them in the 100-year event is approximately 0.3 foot. These culverts are significantly smaller than the double 6-foot-by-14-foot culverts at Florida Boulevard.

The hydraulic features south of Forest Avenue do not create enough restrictions and are far enough away from the City not to cause significant impacts to drainage conditions in the City.

6.5.1.2 Sherman Creek

Sherman Creek outfalls on the north end of the City through two separate outfalls. The main flow-way of Sherman Creek, which runs through the Atlantic Beach Country Club, outfalls from the City through the FDOT box culverts under Mayport Road and continues through a combination of FDOT ditches and culverts before discharging again under Mayport Road through another FDOT box culvert. Sherman Creek has a secondary outfall that drains the large wetland system north and east of Fleet Landing. This system receives water from the Oceanwalk ditch, which drains a large portion of the City. This system outfalls north of the City through several 48-inch to 54-inch corrugated metal culvert crossings in poor condition owned by the City of Jacksonville or private entities. Figure 6-14 shows the location and ownership of drainage features outside the City on Sherman Creek.

Figure 6-14 Sherman Creek Drainage Features Outside of the City



The outfall of the main Sherman Creek flow-way that runs through the Atlantic Beach County Club is severely restricted by the FDOT ditch downstream of Mayport Road and the 6-foot-by-10-foot box culvert under the section of Highway A1A that runs northwest to Wonderwood Drive. Head loss across the box culvert ranges from approximately 2.9 feet in the 100-year event to approximately 0.6 foot in the mean-annual event. This culvert is severely undersized for storm events exceeding the mean-annual event. A significant amount of headloss also occurs over the 1,150-foot stretch of modeled ditch immediately downstream of the double 8-foot-by-10-foot box culverts leaving the City under Mayport Road. The headloss across this ditch ranges from 0.6 foot to 0.8 foot in storm events greater than the mean-annual event.

The wetland system northeast of Fleet Landing, which is the tailwater condition for the Oceanwalk ditch, outfalls through a 48-inch corrugated metal culvert under Pioneer Drive that is owned and maintained by the City of Jacksonville. Downstream of this culvert, a series of three driveway culverts range in size from 48 inches to 54 inches, with some of them being poorly maintained. These culverts drain into a double 6-foot-by-8-foot culvert crossing under Mayport Road. The headloss from the upstream end of the culvert under Pioneer Drive to the upstream end of the box culverts under Mayport Road ranges from 1.5 feet in the mean-annual event to 2.7 feet in the 100-year event. These culverts are severely undersized for the flow leaving the system at this location.

6.5.2 Improvements Considered

Improvements to drainage features outside the City were not investigated in detail as part of the previous master plan projects. They were noted as potential issues outside the City's jurisdiction and were therefore not looked into. The City asked Jones Edmunds to identify the potential constrictions outside the City and to conduct a quick analysis to determine if improving these constrictions would have significant impacts on flood stages within the City. This analysis identified constrictions outside the City that impact drainage within the City and provides rough estimates of potential flood stage reductions if improvements to these constrictions were made. This analysis also identifies the entities that the City would need to engage to make improvements at these structures to reduce flood stages within the City.

6.5.2.1 Hopkins Creek

The following improvements to drainage features outside the City jurisdiction were considered for Hopkins Creek:

- Hydraulic capacity improvements to the FDOT-owned culverts under Atlantic Boulevard.
- Hydraulic capacity improvements to the City of Jacksonville-owned culverts under Florida Boulevard.
- Hydraulic capacity improvements to the City of Neptune Beach-owned culverts under Forest Avenue.

6.5.2.2 Sherman Creek

The following improvements to drainage features outside the City jurisdiction were considered for Sherman Creek:

- Hydraulic capacity improvements to the 6-foot-by-10-foot FDOT-owned box culvert that drains Puckett Creek to the ICW under Florida Highway A1A.
- Hydraulic capacity improvements to the 1,150-foot stretch of FDOT-owned ditch immediately downstream of the double 8-foot-by-10-foot box culverts leaving the City under Mayport Road.
- Hydraulic capacity improvements to the 48-inch corrugated metal culvert under Pioneer Drive that is owned and maintained by the City of Jacksonville.

6.5.3 Recommended Improvements

6.5.3.1 Hopkins Creek

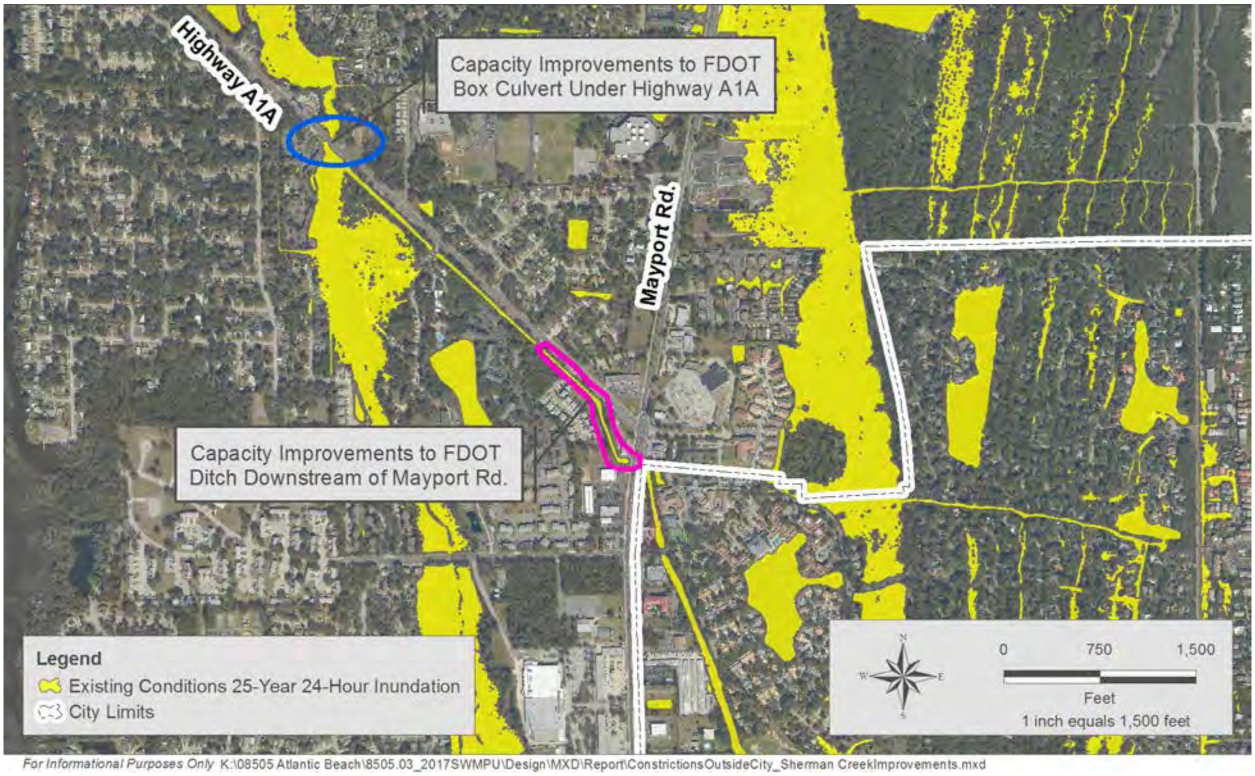
None of the hydraulic improvements outside the City that were analyzed for Hopkins Creek made significant changes to peak flood stages within the City because the existing 6-foot-by-8-foot box culvert in Hopkins Creek under the west entrance to the Aquatic Village shopping center within the City is the primary constriction in Hopkins Creek. None of the proposed capacity improvements will have an impact in the City until capacity improvements are made to this culvert and to Hopkins Creek north of Atlantic Boulevard.

We did not analyze the effects of the maintenance condition of the hydraulic features outside the City's jurisdiction as a part of this analysis. The maintenance condition of the FDOT box culverts under Atlantic Boulevard and the ditch downstream of Atlantic Boulevard could impact drainage conditions in the City. We recommend that the City inspects these features regularly and coordinates with the Owners to ensure that they are maintained and properly functioning.

6.5.3.2 Sherman Creek

Based on our analysis, we recommend that the City coordinates with FDOT to improve the hydraulic capacity of the FDOT ditch downstream of Mayport Road and the culvert crossing under Highway A1A. Our modeling shows that these features are undersized and create significant headloss and impact peak flood stages in Sherman Creek within the City. Improving the culvert and ditch capacities could reduce peak flood stages by approximately 4 to 6 inches in Sherman Creek within the City. This improvement allows water to drain out of the City more efficiently and reveals new upstream constrictions at several of the City-owned culvert crossings along Sherman Creek. As part of this analysis, we also modeled increasing the capacities of these City-owned Sherman Creek culvert crossings to Howell Park with the improvements to the FDOT features. The model results from this scenario resulted in stage reductions of approximately 8 inches in the 10- and 25-year/24-hour events in the Howell Park/Salt Air area. Figure 6-15 summarizes the improvements made for this scenario.

Figure 6-15 Sherman Creek Improvements



These improvements also create hydraulic capacity in Sherman Creek and allow for hydraulic improvements to reduce flooding in the upper reaches of the Sherman Creek contributing area to be made without causing adverse downstream impacts. The proposed improvements in subsequent sections of this Report for the Johansen Park, 9th/10th/11th/12th Streets, and Salt Air/Howell Park areas should not be considered until improvements are made to the FDOT system because they will cause peak flood stages to increase downstream in Sherman Creek.

If the City and/or FDOT pursue these improvements along Sherman Creek, analyzing the effects of tidal influence on the City's drainage when conveyance capacity is improved is important. Improving the capacity may increase the City's vulnerability to extreme high tide conditions by allowing the tide a more efficient flowpath into the City. Backflow prevention with the capacity improvements may need to be considered if tidal influences on the City's drainage may worsen with increased capacity along Sherman Creek.

Cost analyses and flood reduction exhibits for the proposed improvements outside the City were not completed. These improvements would not be funded by the City, and the modeling completed was meant to provide a rough estimate of potential flood reduction benefits from these alternatives. If the City were to pursue these alternatives with FDOT or the other entities, a more thorough analysis would need to be completed to determine exactly what improvements are feasible and their associated benefits.

6.6 100/200/300 BLOCKS OF SEMINOLE ROAD

6.6.1 DESCRIPTION OF PROBLEM AREA

Seminole Road has a low area that experiences frequent flooding between the 100 and 300 blocks north of Atlantic Boulevard near the Park Street intersection. Runoff from the road and adjacent parcels is intended to drain into several ditch-bottom inlets along the west side of the road in the pervious areas between the road and the sidewalk. The inlets are piped via 10 to 12-inch Orangeburg pipes to the downstream end of the box culvert under Seminole Road near the City Hall complex.

According to the 2012 SWMP Update, the City receives complaints of standing water in this area. Figure 6-16 depicts the approximate limits of the project study area and the existing inundation results from the ICPR model for the 2045 development scenario.

Figure 6-16 Project Study Area and Existing Conditions Inundation Results

6.6.2 IMPROVEMENTS CONSIDERED

In the 1995 Master Plan, this area fell within Subbasin SM-D and within Basins D2 and D3 of the detailed delineation. The study primarily focused on issues occurring east of Seminole Road along Sturdivant Avenue, citing high tailwater at the Howell Park outfall as the primary source of street flooding. No specific improvements for the flooding along Seminole Road were identified in the Plan.

In the 2002 SWMP Update, this area was outside the area of focus.

The 2012 SWMP Update specifically addressed this location and identified undersized pipes along Seminole Road as the primary source of the street flooding. Recommended improvements consisted of replacing selected runs of pipe with larger diameter pipes, regrading some areas of the right-of-way to form shallow swales, adding driveway culverts to help convey flow, and adding ditch-bottom inlets to allow runoff to enter the new, larger pipe.

Several improvement scenarios were considered as part of this study. Conceptually they consisted of increasing the conveyance capacity to the outfall, connecting to other outfalls, and increasing the capacity of downstream features to lower the tailwater of the system. One option was to connect the Seminole Road system to the Belvedere Street outfall pipe, which has a larger and more direct connection to the existing outfall. Another consideration was replacing and upsizing the run of pipe along Seminole Road. As mentioned in previous studies, these pipes are undersized and due to their age and material could be deforming and/or clogging. A combination of these options with a drastic increase in the pipe sizes was modeled to test the sensitivity of the system to pipe size changes.

6.6.3 Recommended Improvement

Recommended improvements along Seminole Road and Belvedere Street:

- Replace 10-inch, 12-inch, and 15-inch pipes along Seminole Road from Park Street to Palm Avenue with 36-inch pipe.

Exhibits 5A, 5B, and 5C in Appendix A depict the improvements resulting from these alterations for the mean-annual, 10-, and 25-year/24-hour design events.

Completion of the work will primarily occur within City-owned rights-of-way; however, to facilitate construction, temporary impacts to some adjacent properties may be necessary. The existing 10-inch pipes are under the sidewalk on the east side of the road; therefore, upsizing them will require the reconstruction of the sidewalk and numerous driveway turnouts after the pipe is installed. Several inlets and manholes along the proposed run will also need to be replaced and an additional survey of water/sewer services will need to be performed to determine whether this alternative is feasible.

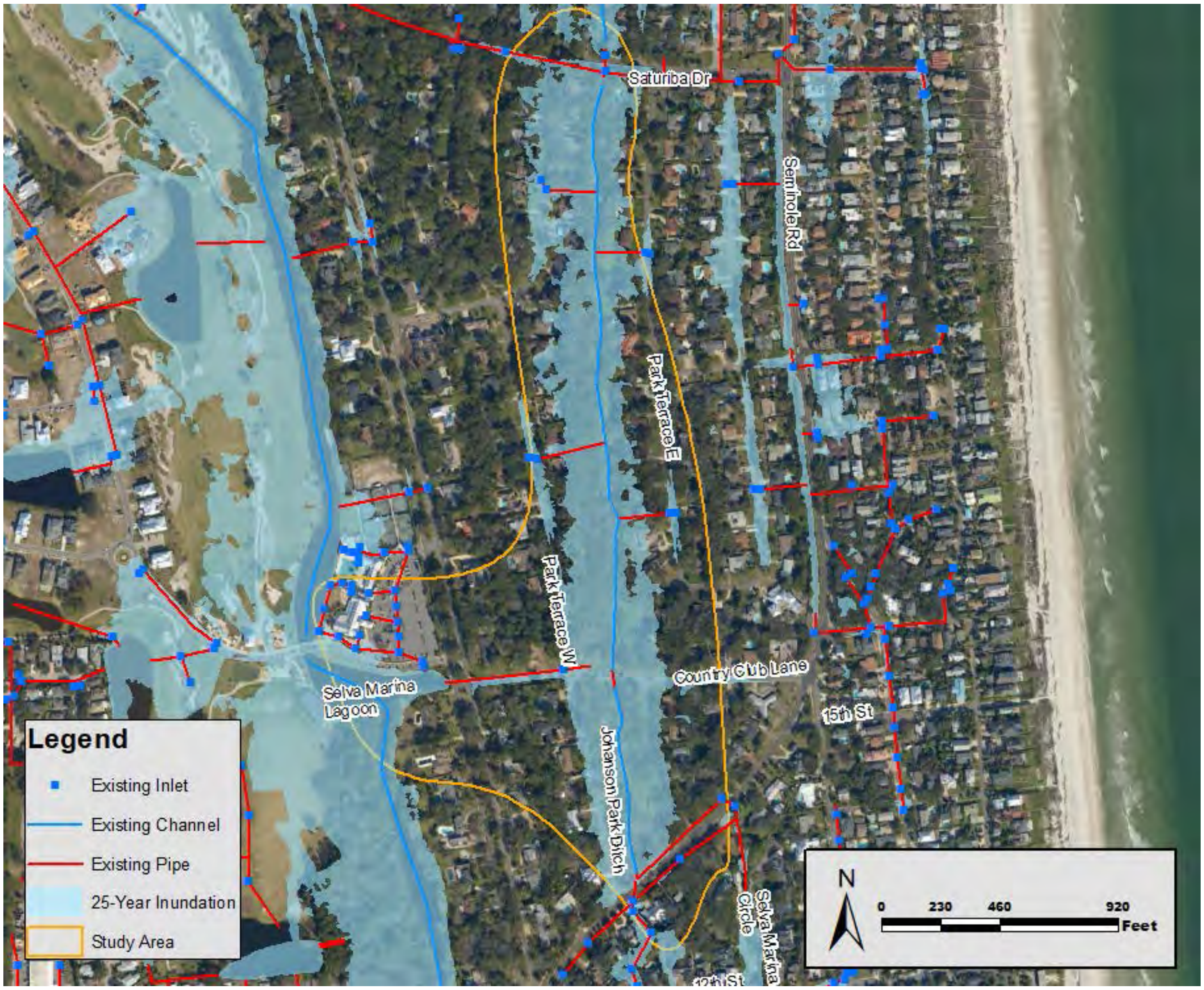
Table 8 in Appendix B provides an opinion of probable cost in 2018 dollars for this conceptual improvement. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. Construction and engineering for improvements are estimated between $900,000 and $1,500,000. This cost range includes removing and replacing one lane of Seminole Road the entire length of the pipe. This was included in the estimate to account for the possibility of the pipe needing to be placed under Seminole Road due to conflicts with other utilities in the area. If this is not the case and the pipe is able to be kept out of the roadway, the cost of this improvement will be reduced.

6.7 JOHANSEN PARK

6.7.1 DESCRIPTION OF PROBLEM AREA

The problem area is between Park Terrace West and Park Terrace East from Seminole Road north to Saturiba Drive. According to the 2012 SWMP Update, the area discharging to the Johansen Park area is too large for the existing ditch capacity. Figure 6-17 shows the approximate study area and the inundation from the 25-year/24-hour design event.

Figure 6-17 Project Study Area and Existing Conditions Inundation Results



6.7.2 IMPROVEMENTS CONSIDERED

In the 1995 Master Plan, this area fell within Subbasin SM-F and within Basin F3 of the detailed delineation. The study primarily focused on flooding issues occurring in and around Selva Marina Circle and at a low spot on 12th Street, as well as the intersection of 15th Avenue and Seminole Road.

The runoff from Selva Marina Circle is routed north across Seminole Road and then southwest to Previn Johansen Park ditch. The flow is carried north in the open channel to Country Club Lane, where it is directed west to Selva Marina Lagoon. The recommended solution was to install a 19-by-30-inch pipe from Selva Marina Circle to the Previn Johansen Park ditch.

In the 2002 SWMP Update, this area was in Problem Area No. 2, which stated that 47 acres of tributary area served by the Selva Marina system entirely discharged into an existing ditch at Johansen Park before eventually discharging to the Selva Marina Canal. The recommended solution was reducing the tributary area being discharged to the Johansen Park ditch and rerouting the storm sewer that runs from Selva Marina Circle to Johansen Park. The plan suggested the current storage of the Johansen Park ditch could be increased by adding a v-notch weir or an orifice plate.

The 2012 SWMP Update does not specifically address this location and therefore has no recommended solutions.

As part of this Update, an increase in conveyance capacity of the connection from Johansen Park to the Selva Marina Canal/Sherman Creek was tested. This pipe begins on the north side of Country Club Lane and runs due west to a headwall on the west side of Selva Marina Drive near the Atlantic Beach Country Club. Another connection to the Canal is at the north end of the park along Saturiba Drive but due to its length and proximity to private property, improvements to it were not considered.

6.7.3 Recommended Improvement

Recommended improvements along Country Club Lane to Sherman Creek:

- Replacing 24-inch culvert under Country Club Lane with 48-inch pipe.
- Replacing Country Club Lane 24-inch outfall pipe with 48-inchpipe.

Exhibits 6A, 6B, and 6C in Appendix A depict the flood reductions resulting from these improvements for the mean-annual, 10-, and 25-year/24-hour design events.

Completion of the work will primarily occur within City-owned rights-of-way; however, to facilitate construction, temporary impacts to some adjacent properties may be necessary. The existing pipe is along the north side of Country Club Lane but construction will likely impact the roadway and connecting driveways. Overhead power lines may also be a constraint. Additionally, the 24-inch pipe headwall on the west side of Selva Marina is shown on private property according to 2016 Duval County parcel data.

As mentioned in Section 6.5.3 of this Report, the completion of this project under existing conditions will cause peak flood stages to increase downstream in Sherman Creek. This project should not be considered until improvements are made to the FDOT-owned constrictions outside the City along Sherman Creek.

Table 9 in Appendix B provides an opinion of probable cost in 2018 dollars for this conceptual improvement. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. Construction and engineering costs for Phase 1 improvements are estimated between $280,000 and $450,000.

6.8 9TH / 10TH / 11TH / 12TH STREETS

6.8.1 DESCRIPTION OF PROBLEM AREA

This problem area is from 9th Street to 12th Street spanning from Seminole Road to East Coast Drive. The 1995 SWMP and 2002 SWMP Update reported flooding problems in this area due to flat terrain and an inadequate drainage system. Inundation occurs in a low area on the east side of Seminole Road that impacts the streets and several properties in the area. The existing collection system has two primary outfall connections to Sherman Creek, one at 11th Street and a smaller one at 9th Street. The 12th street system previously accepted flow from Johansen Park via a 12-inch pipe from the north. This pipe has become clogged and is essentially non-functioning. Figure 6-18 depicts the approximate limits of the project area.

Figure 6-18 Project Study Area and Existing Conditions Inundation Results

6.8.2 IMPROVEMENTS CONSIDERED

In the 1995 Master Plan, this area fell within Subbasin SM-E/SM-F and within the Subsections F4, E2, E3, and E4. The study ranked SM-E as the second worst and SM-F as the fourth worst areas out of 20 in terms of problem flood areas. According to the report, the main reason that SM-E was having flooding problems was because the entire basin's outfall was a single 18-inch pipe and replacement of this was recommended with at least a 48-inch pipe. For the SM-F Basin, the study concludes that flat slopes and small pipe sizes are the problem for directing stormwater out of the area. The report gave SM-E and SM-F a flood protection level-of-service score of “C” with a level-of-service “D” being the worst.

The 2002 SWMP Update Final Report identified two areas near 9th, 10th, 11th, and 12th Streets as places with flooding issues. The first is Selva Marina Circle and the second is the intersection of 13th Street and Ocean Boulevard. The Report states that flooding issues in Selva Marina Circle are caused by flat terrain and a hydraulically inadequate drainage system that serves the area. Also stated is that the flooding issues at 13th Street and Ocean Boulevard are directly related to the flooding problems in Selva Marina. The suggested improvements to this area include reducing the contributing area to the outfall by redirecting flow west and upgrading the existing stormwater conveyance system.

The 2012 SWMP Update did not address this area as a particularly high flooding area.

As part of this Study, the focus in this area was primarily on strategically upsizing the existing conveyance pipes and the existing outfall at 11th Street because upgrades to the connections at 12th Street and 9th Street would impact private property. Based on previous studies and model results, the pipes along 12th/11th Streets and Seminole Boulevard appear to be undersized. Several iterations of upsizing these pipes resulted in the following recommended improvement.

6.8.3 Recommended Improvement

Depending on funding availability, splitting the recommended improvements into two projects is a possibility. One would improve the 12th/11th Streets outfall pipe, and the other would improve the Seminole Road conveyance.

Recommended Phase 1 improvements along 11th and 12th Streets to Sherman Creek include:

- Replacing the 48-inch outfall pipe from Sherman Creek to Seminole Road with double 48-inch pipes.
- Replacing the 11th Street 24-inch pipe with a 48-inch pipe from Seminole Road to 12th Street.
- Replacing the 12th Street 18-inch pipe with a 48-inch pipe.

Exhibits 7A, 7B, and 7C in Appendix A depict the inundation reductions resulting from these improvements for the mean annual, 10-year, and 25-year/24-hour design events.

Completion of the work will primarily occur within City-owned rights-of-way; however, to facilitate construction, temporary impacts to some adjacent properties may be necessary. The existing pipes are in the center of 11th Street and under the curb of 12th Street. Replacing them will require reconstruction of the roadway and numerous driveways. The 48-inch pipe is also close to Lift Station 425. Detailed survey and utility locations will need further investigation to determine the feasibility of this alternative.

Table 10 in Appendix B provides an opinion of probable cost in 2018 dollars for this conceptual improvement. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. Construction and engineering costs for Phase 1 improvements are estimated between $680,000 and $1,110,000.

Recommended Phase 2 improvements along Seminole Road to Sherman Creek include:

- Replacing the 36-inch pipe along Seminole Road from 9th Street to 11th Street with a 60-inch pipe.
- Replacing the 10th Street 30-inch pipe with a 48-inch pipe from the low point to Seminole Road.
- Replacing the 9th Street 24-inch pipe with a 48-inch pipe from the low point to Seminole Road.

Exhibits 8A, 8B, and 8C in Appendix A depict the inundation reductions that result from these improvements for the mean-annual, 10-year, and 25-year/24-hour design events.

Completion of the work will primarily occur within City-owned right-of-way; however, to facilitate construction, temporary impacts to some adjacent properties maybe necessary. The existing pipes are in the center of 9th Street and 10th Street; therefore, replacing them will require reconstruction of the roadway and will likely impact numerous driveways. The Seminole Road pipe runs along the side of the roadway, but impacts to the road can be expected when installing a large diameter pipe. Impacts to existing utilities are also a possibility.

As mentioned in Section 6.5.3 of this Report, the completion of this project under existing conditions will cause peak flood stages to increase downstream in Sherman Creek. This project should not be considered until improvements are made to the FDOT-owned constrictions outside the City along Sherman Creek.

Table 11 in Appendix B provides an opinion of probable cost in 2018 dollars for this conceptual improvement. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. Construction and engineering costs for Phase 2 improvements are estimated between $710,000 and $1,160,000.

6.9 Salt Air / Howell Park

6.9.1 Description of Problem Area

This problem area is east of the Seminole Road south problem area and drains to the Howell Park outfall via a piped collection system that terminates at the north end of Pine Street. The boundary on the south side is Sturdivant Street and the inundation occurs primarily along Pine, Poinsettia, Sylvan, and Sturdivant Streets. Figure 6-19 depicts the approximate limits of the project area as well as the 25-year/24-hour inundation results from the ICPR model for the 2045 development scenario. As shown, several properties are impacted in this scenario.

Figure 6-19 Project Study Area and Existing Conditions Inundation Results

6.9.2 IMPROVEMENTS CONSIDERED

In the 1995 Master Plan, this area is within Subbasin SM-D and within Basins D1, D2, and D3 of the detailed delineation. The 1995 Master Plan primarily focused on issues occurring along Sturvidant Avenue, citing high tailwater at the Howell Park outfall as the primary source of the street flooding. The study did not have recommendations to reduce the peak flood elevations within Howell Park specifically. The study focused on stormwater collection system interconnections and pipe size increases with the goal of reducing street flooding. Figure 6-20 summarizes recommendations for Basins SM-A and SM-D:

Figure 6-20 Summary of Recommendations from the 1995 Master Plan Report

Table 7-1
Recommendations for SM-A and SM-D

Specific Goals	1) Eliminate flow restriction on Sturdivant Ave. for system improvement. – by P.W. ≈ 1996 2) Reduce yard and street flooding on Ahern St. → Not Completed 3) Reduce yard and street flooding near Pine St. and Sturdivant Ave. – completed 2006 4) Eliminate flow restriction on Poinsettia St. – completed ≈ 1997 5) Reduce yard and street flooding at Magnolia St. and Pine St. – completed ≈ 1997, 2006 6) Reduce yard and street flooding on 2nd St. → Not Completed
Recommended Solution **(See Figure 7-1)**	1) Replace 12 inch diameter pipe near Sherry with 24 inch diameter pipe. 2) Install new 19 x 30 inch pipe from Sherry Dr. to East Coast Dr. 3) Install 19 x 30 inch pipe from Pine St. to Poinsettia St. 4) Install 29 x 45 inch pipe from Sturdivant St. to David St. 5) Install 30 inch diameter pipe from Magnolia to Poinsettia St. 6) Reroute flow from 2nd St. north to SM-B Subbasin.
Notes	Improvements should be made concurrently within these subbasins but are independent of other subbasin proposals with the exception of item 6, which <u>cannot</u> be done without the completion of SM-B proposals. 6) Costs for this modification are included with the SM-B proposals.
Cost to Implement :	**$ 284,000**

According to the handwritten notes in the document, several of the improvements were completed and some not. Figure 6-21 (Figure 7-1 from the 1995 Master Plan) depicts the locations of the proposed improvements.

Figure 6-21 Proposed Improvements from the 1995 Master Plan Report

The 2012 SWMP Update specifically addressed this location. The area was identified as *Problem Area No. 3: Salt Air Drainage Upgrades* and the problem was described as poor drainage and standing water resulting from historic roadside swales being cut off by driveway connections. Recommended improvements consisted of regrading the shallow swales and adding or cleaning out existing driveway culverts to help convey flow. The study also suggested adding Geoweb material in select areas to facilitate parking along the grass swales. Figure 6-22 shows these improvements along Magnolia and Poinsettia Streets (Figure 3-2 in the 2012 SWMP Update).

Figure 6-22 Proposed Improvements from the 2012 SWMP Update Report

Before modeling alternatives for improvements in this area, note that the 10- and 25-year events are both over the top of the headwall at the north end of Pine Street where the system discharges to the open channel through Howell Park. After the headwall is overtopped, little can be done to the collection system to improve or reduce the flooding of the streets upstream of this point. Several options were considered during this study to test the response of the system. The downstream culverts at Seminole Road and Plaza were doubled in size to determine if a meaningful stage reduction in Howell Park could be achieved. This had little effect because the downstream FDOT ditch and culvert are the controlling constrictions along Sherman Creek. We also modeled what could be considered a levee in the area of the headwall combined with a backflow preventer on the Pine Street outfall pipe. This provided some benefit, but the upstream collection system lacks the storage that would be needed to contain the runoff from the design events. A small pump combined with the levee could provide a significant benefit but would overwhelm the Howell Park systems and therefore was not modeled. Due to the connection to the Seminole Road system at David Avenue, improvements along Seminole Road do provide some benefit to this area, especially in the mean-annual design storm.

6.9.3 Recommended Improvement

Based on the alternatives investigated, local improvements to the collection system in this area do not provide significant enough flood reduction to be recommended. The improvements on Seminole Road provide some relief, but the downstream conveyance improvements to City-owned culvert crossings on Sherman Creek with the hydraulic capacity improvements to the FDOT ditch and culvert described in Section 6.5.3 of this Report provide the greatest reduction to flood stages in the Salt Air/Howell Park area. Capacity improvements to the City-owned culvert crossings alone will not result in significant flood stage reductions in this area. The downstream constriction outside the City must be removed for the benefits to be realized upstream. If the downstream constrictions outside the City are removed, the City should improve the hydraulic capacity at the 11th Street, Plaza, and Seminole Road culvert crossings of Sherman Creek.

Exhibits 9A, 9B, and 9C in Appendix A depict the inundation reductions that result from these improvements for the mean-annual, 10-year, and 25-year/24-hour design events.

Table 12 in Appendix B provides an opinion of probable cost in 2018 dollars for this conceptual improvement. Cost estimates were developed based on ASTM E2516 Class 4 cost ranges. Construction and engineering costs are estimated between $950,000 and $1,550,000.

7 TEN-YEAR CAPITAL IMPROVEMENT PLAN

The proposed stormwater capital improvement projects were prioritized based on the following considerations to develop the 10-year Stormwater Capital Improvement Plan (Table 7-1):

- Conversations with City staff about the City's drainage priorities.
- The cost versus benefit of the proposed drainage improvement projects.
- The City's current and projected future stormwater funding sources.

The costs presented in Table 7-1 are budget level costs in 2018 dollars. We based the budget costs on the high-end cost estimate ranges we developed. The cost ranges are presented in Appendix B. The ranges include contingencies to account for potential unforeseen factors that could increase project costs and to account for potential increases in construction costs beyond the Consumer Price Index. Costs for Improvements in the medium-term and long-term planning windows will likely need to be revisited and updated before they are implemented to account for changes in construction costs from 2018.

We anticipate that the order the projects are completed will depend on funding availability. The total cost of implementing all of these projects is significant. If the City expects to complete all of these projects, the City may need to increase stormwater capital improvement project funding. To meet the funding needs to complete this Plan, we recommend the City consider:

- Continuing to pursue grant funding opportunities.
- Pursuing funding from legislative appropriation.
- Coordinating with FDOT and other adjacent entities to improve hydraulic constrictions outside the City.
- Reviewing its stormwater utility fee rate structure and consider increasing the fee and/or implementing a tiered rate structure.
- Bonding options once the existing bonds are paid off from the Core City Drainage Improvement project.

We believe completing the projects in the 10-year CIP will address the neighborhood scale flooding issues in the City that were investigated as part of this SWMP Update. The projects presented in this 10-year plan are intended to provide improvements from storm events up to the 25-year frequency. The projects are not intended to provide additional protection from rainfall events beyond the 25-year frequency and storm surge events.

Table 7-1 10-Year Capital Improvement Plan

Planning Window	Priority Range Total Cost	Capital Improvement Project	Project Cost
Short-term (0 to 3 Years)	$ 4,400,000	Aquatic Drive Improvements Phase 1: • Culvert improvements at Aquatic Village shopping center • Deepening the Aquatic Drive pond • Installing a stormwater pump station • Replacing the existing outfall structure • Minor improvements to the Hopkins Creek	$2,760,000
		100/200/300 blocks of Seminole Road South	$1,510,000
		West Plaza backflow prevention	$86,000
Medium-term (4 to 7 Years)	$ 17,500,000	Stanley Road: Option 1: New 36 inch outfall pipe Option 2: Property acquisition	 $640,000 $600,000
		Mary Street improvements	$450,000
		Aquatic Drive Improvements Phase 2: • Culvert improvements at Cutlass Drive • Hopkins Creek capacity improvements[2] • Adjacent property acquisition • Aquatic Drive pond expansion following property acquisition	 $450,000 $13,630,000 $1,800,000 $475,000
Long-term (8 to 10 Years) [1]	$ 4,000,000	Salt Air/Howell Park Improvements	$1,170,000
		9th/10th/11th/12th Street Improvements Phase 1	$1,110,000
		9th/10th/11th/12th Street Improvements Phase 2	$1,160,000
		Johansen Park Improvements	$450,000

[1] These projects are contingent upon downstream improvements being made to FDOT ditch and culvert on Sherman Creek.

[2] The cost for this alternative could be reduced depending on the type of channel modifications completed. This cost reflects using sheet piling, which is the most expensive alternative. Other methods for reconstructing channel are possible and should be investigated in detail once property acquisition is complete.

8 REFERENCES

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Appendix A

Capital Improvement Project Exhibits

Design Event	Peak Stage Summary — Peak Water Surface Elevations (feet NAVD88)																				
	Node NHK 10050			Node NHK 10060			Node NHK 10101			Node NHK 10110			Node NHK 10111			Node NHK 10120			Node NHK 10131		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.54	4.47	-0.07	4.55	1.84	-2.71	4.81	4.82	0.01	4.37	4.21	-0.16	4.27	4.03	-0.24	3.78	3.9	0.12	3.71	3.81	0.1
10-Year, 24-Hour	5.66	5.35	-0.31	5.66	4	-1.66	5.66	5.37	-0.29	5.56	5.12	-0.44	5.51	5	-0.51	4.58	4.78	0.2	4.43	4.58	0.15
25-Year, 24-Hour	6.24	5.86	-0.38	6.24	5.86	-0.38	6.24	5.87	-0.37	6.19	5.72	-0.47	6.14	5.63	-0.51	5.3	5.34	0.04	5.02	5.05	0.03

Exhibit 1A

Aquatic Drive, Cavalla Road, Skate Road Ph. 1 Alternative

City of Atlantic Beach

Section 38 Township - 0 Range 0

Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions Mean Annual, 24-Hour Inundation
- Existing Conditions Mean Annual, 24-Hour Inundation

Flood Mitigation BMP Type: Enhanced Drainage Mean Annual Results

Estimated Cost:
$ 1,600,000 - $ 2,700,000

N

0 150 300
Feet
1 inch equals 300 feet

JonesEdmunds

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Design Event	Node NHK 10050			Node NHK 10060			Node NHK 10101			Node NHK 10110			Node NHK 10111			Node NHK 10120			Node NHK 10131		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.54	4.47	-0.07	4.55	1.84	-2.71	4.81	4.82	0.01	4.37	4.21	-0.16	4.27	4.03	-0.24	3.78	3.9	0.12	3.71	3.81	0.1
10-Year, 24-Hour	5.66	5.35	-0.31	5.66	4	-1.66	5.66	5.37	-0.29	5.56	5.12	-0.44	5.51	5	-0.51	4.58	4.78	0.2	4.43	4.58	0.15
25-Year, 24-Hour	6.24	5.86	-0.38	6.24	5.86	-0.38	6.24	5.87	-0.37	6.19	5.72	-0.47	6.14	5.63	-0.51	5.3	5.34	0.04	5.02	5.05	0.03

Peak Stage Summary — Peak Water Surface Elevations (feet NAVD88)

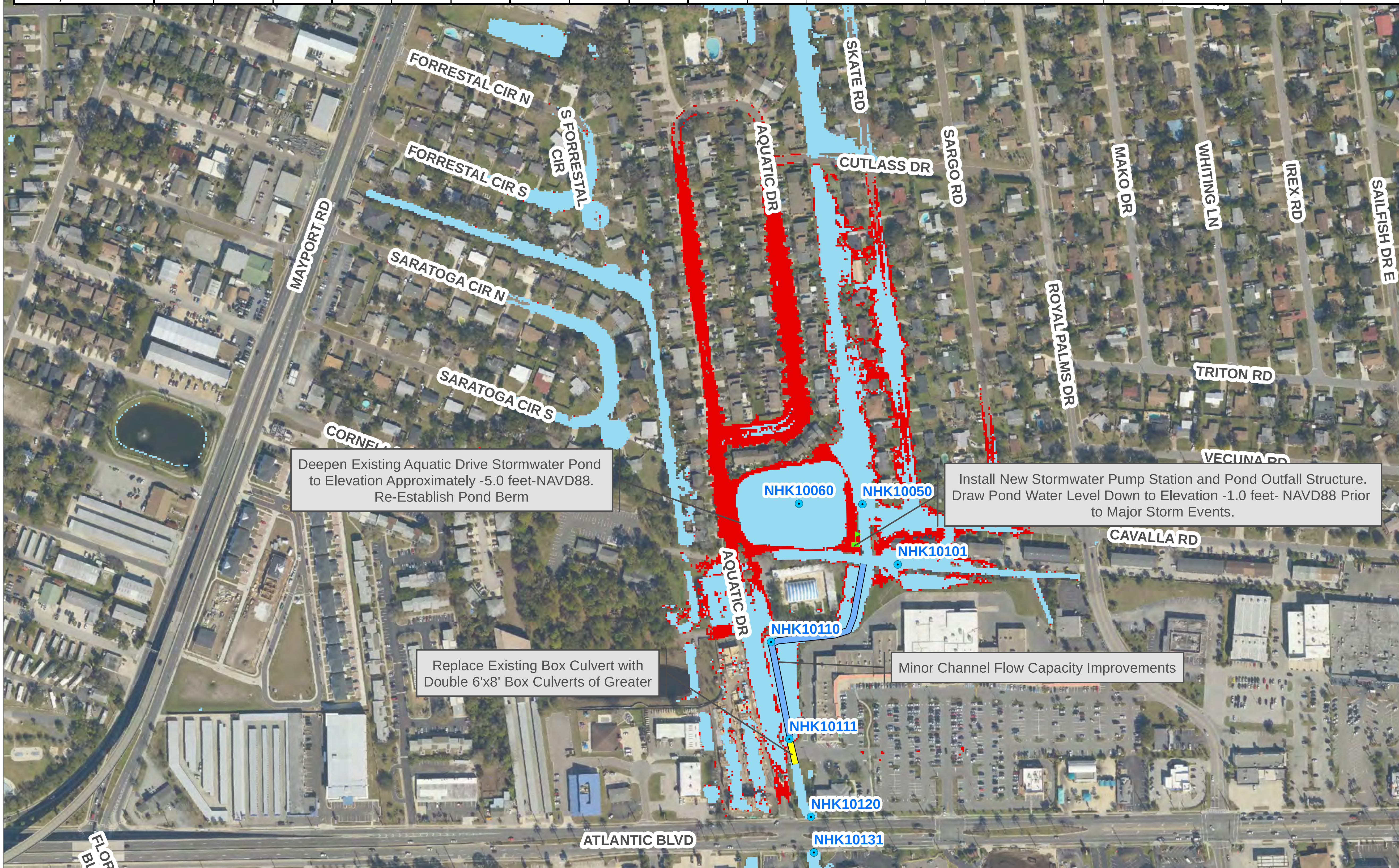
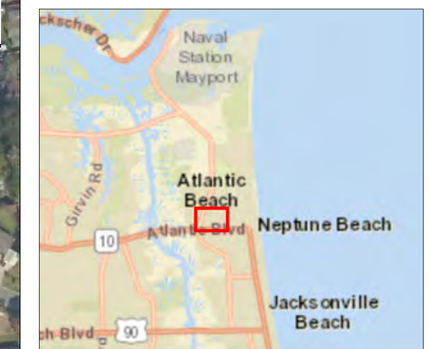


Exhibit 1B

Aquatic Drive, Cavalla Road, Skate Road Ph. 1 Alternative

City of Atlantic Beach

Section 38 Township - 0 Range 0



Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions 10-Year, 24-Hour Inundation
- Existing Conditions 10-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
10-Year Results

Estimated Cost:
$ 1,600,000 - $ 2,700,000

N

0 150 300
Feet
1 inch equals 300 feet

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Exhibit 1C

Aquatic Drive, Cavalla Road, Skate Road Ph. 1 Alternative

City of Atlantic Beach

Section 38 Township - 0 Range 0

Design Event	Peak Stage Summary																				
	Peak Water Surface Elevations (feet NAVD88)																				
	Node NHK 10050			Node NHK 10060			Node NHK 10101			Node NHK 10110			Node NHK 10111			Node NHK 10120			Node NHK 10131		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.54	4.47	-0.07	4.55	1.84	-2.71	4.81	4.82	0.01	4.37	4.21	-0.16	4.27	4.03	-0.24	3.78	3.9	0.12	3.71	3.81	0.1
10-Year, 24-Hour	5.66	5.35	-0.31	5.66	4	-1.66	5.66	5.37	-0.29	5.56	5.12	-0.44	5.51	5	-0.51	4.58	4.78	0.2	4.43	4.58	0.15
25-Year, 24-Hour	6.24	5.86	-0.38	6.24	5.86	-0.38	6.24	5.87	-0.37	6.19	5.72	-0.47	6.14	5.63	-0.51	5.3	5.34	0.04	5.02	5.05	0.03

Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions 25-Year, 24-Hour Inundation
- Existing Conditions 25-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
25-Year Results

Estimated Cost:
$ 1,600,000 - $ 2,700,000

N

0 150 300
Feet
1 inch equals 300 feet

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Design Event	Node NHK 10030 Pre	Node NHK 10030 Post	Node NHK 10030 Change	Node NHK 10040 Pre	Node NHK 10040 Post	Node NHK 10040 Change	Node NHK 10041 Pre	Node NHK 10041 Post	Node NHK 10041 Change	Node NHK 10050 Pre	Node NHK 10050 Post	Node NHK 10050 Change	Node NHK 10051 Pre	Node NHK 10051 Post	Node NHK 10051 Change	Node NHK 10060 Pre	Node NHK 10060 Post	Node NHK 10060 Change
Mean Annual, 24-Hour	4.78	4.23	-0.55	4.76	4.19	-0.57	4.74	4.16	-0.58	4.54	4.01	-0.53	4.58	4.1	-0.48	4.55	2.3	-2.25
10-Year, 24-Hour	6.1	5.52	-0.58	6.08	5.47	-0.61	6.06	5.43	-0.63	5.66	5.09	-0.57	5.69	5.21	-0.48	5.66	3.5	-2.16
25-Year, 24-Hour	6.43	5.99	-0.44	6.4	5.93	-0.47	6.37	5.88	-0.49	6.24	5.45	-0.79	6.27	5.52	-0.75	6.24	5.46	-0.78

Peak Stage Summary — Peak Water Surface Elevations (feet NAVD88)

Design Event	Node NHK 10101 Pre	Node NHK 10101 Post	Node NHK 10101 Change	Node NHK 10110 Pre	Node NHK 10110 Post	Node NHK 10110 Change	Node NHK 10111 Pre	Node NHK 10111 Post	Node NHK 10111 Change	Node NHK 10120 Pre	Node NHK 10120 Post	Node NHK 10120 Change	Node NHK 10131 Pre	Node NHK 10131 Post	Node NHK 10131 Change
Mean Annual, 24-Hour	4.81	4.58	-0.23	4.37	3.97	-0.4	4.27	3.93	-0.34	3.78	3.81	0.03	3.71	3.73	0.02
10-Year, 24-Hour	5.66	5.29	-0.37	5.56	5.03	-0.53	5.51	4.98	-0.53	4.58	4.76	0.18	4.43	4.57	0.14
25-Year, 24-Hour	6.24	5.46	-0.78	6.19	5.4	-0.79	6.14	5.35	-0.79	5.3	5.12	-0.18	5.02	4.89	-0.13

Exhibit 2A

Aquatic Drive, Cavalla Road, Skate Road All Improvements

City of Atlantic Beach

Section 38 Township - 0 Range 0

Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions Mean Annual, 24-Hour Inundation
- Existing Conditions Mean Annual, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
Mean Annual Results

Estimated Cost:
$ 10,500,000 - $ 17,100,000

N

0 250 500
Feet
1 inch equals 500 feet

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Peak Stage Summary

Peak Water Surface Elevations (feet NAVD88)

Design Event	Node NHK 10030			Node NHK 10040			Node NHK 10041			Node NHK 10050			Node NHK 10051			Node NHK 10060		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.78	4.23	-0.55	4.76	4.19	-0.57	4.74	4.16	-0.58	4.54	4.01	-0.53	4.58	4.1	-0.48	4.55	2.3	-2.25
10-Year, 24-Hour	6.1	5.52	-0.58	6.08	5.47	-0.61	6.06	5.43	-0.63	5.66	5.09	-0.57	5.69	5.21	-0.48	5.66	3.5	-2.16
25-Year, 24-Hour	6.43	5.99	-0.44	6.4	5.93	-0.47	6.37	5.88	-0.49	6.24	5.45	-0.79	6.27	5.52	-0.75	6.24	5.46	-0.78

Peak Stage Summary

Peak Water Surface Elevations (feet NAVD88)

Design Event	Node NHK 10101			Node NHK 10110			Node NHK 10111			Node NHK 10120			Node NHK 10131		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.81	4.58	-0.23	4.37	3.97	-0.4	4.27	3.93	-0.34	3.78	3.81	0.03	3.71	3.73	0.02
10-Year, 24-Hour	5.66	5.29	-0.37	5.56	5.03	-0.53	5.51	4.98	-0.53	4.58	4.76	0.18	4.43	4.57	0.14
25-Year, 24-Hour	6.24	5.46	-0.78	6.19	5.4	-0.79	6.14	5.35	-0.79	5.3	5.12	-0.18	5.02	4.89	-0.13

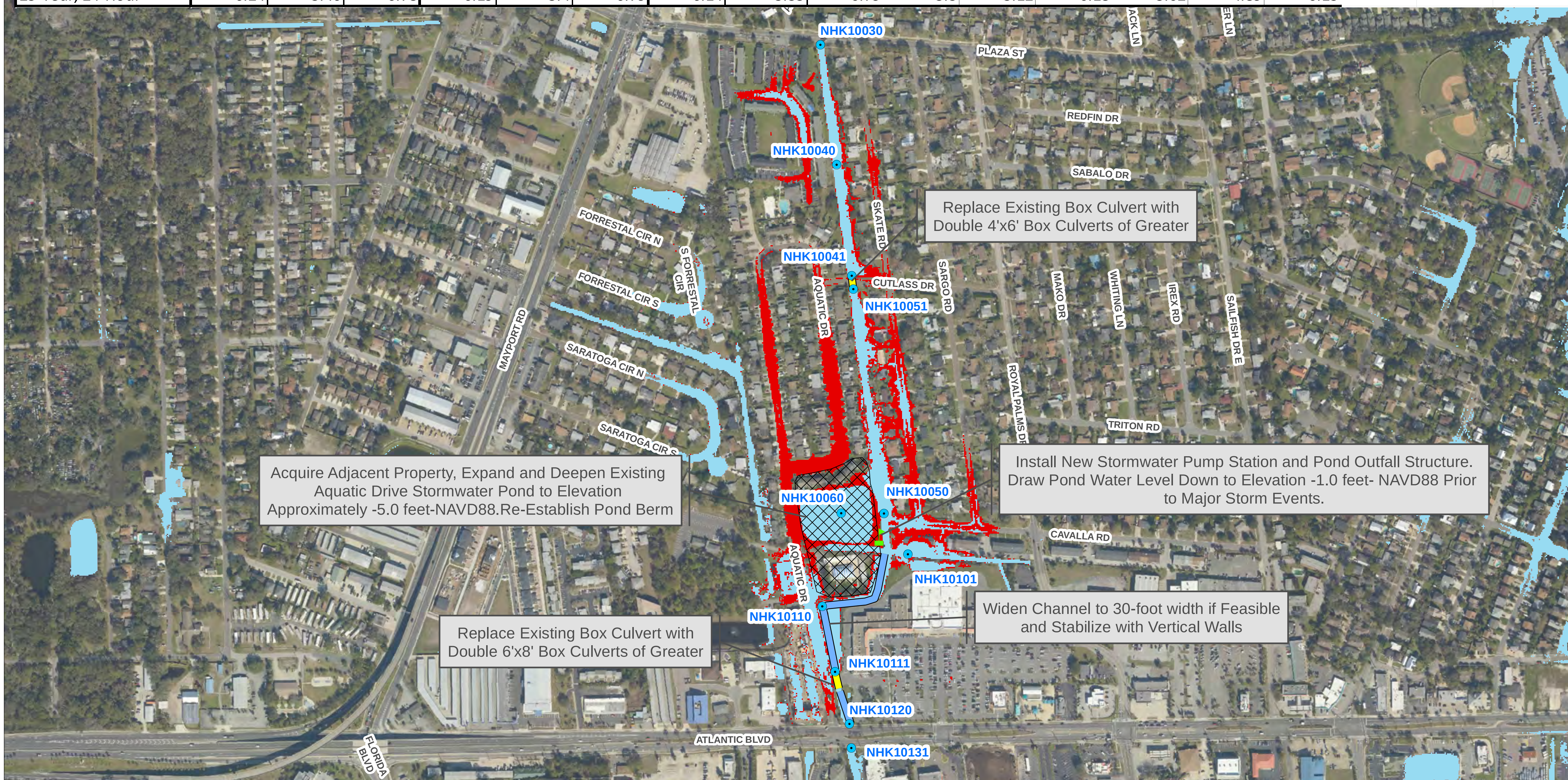


Exhibit 2B

Aquatic Drive, Cavalla Road, Skate Road All Improvements

City of Atlantic Beach

Section 38 Township - 0 Range 0

Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions 10-Year, 24-Hour Inundation
- Existing Conditions 10-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
10-Year Results

Estimated Cost:
$ 10,500,000 - $ 17,100,000

N

0 250 500
Feet
1 inch equals 500 feet

JonesEdmunds

Design Event	Peak Stage Summary																	
	Peak Water Surface Elevations (feet NAVD88)																	
	Node NHK 10030			Node NHK 10040			Node NHK 10041			Node NHK 10050			Node NHK 10051			Node NHK 10060		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.78	4.23	-0.55	4.76	4.19	-0.57	4.74	4.16	-0.58	4.54	4.01	-0.53	4.58	4.1	-0.48	4.55	2.3	-2.25
10-Year, 24-Hour	6.1	5.52	-0.58	6.08	5.47	-0.61	6.06	5.43	-0.63	5.66	5.09	-0.57	5.69	5.21	-0.48	5.66	3.5	-2.16
25-Year, 24-Hour	6.43	5.99	-0.44	6.4	5.93	-0.47	6.37	5.88	-0.49	6.24	5.45	-0.79	6.27	5.52	-0.75	6.24	5.46	-0.78

Design Event	Peak Stage Summary														
	Peak Water Surface Elevations (feet NAVD88)														
	Node NHK 10101			Node NHK 10110			Node NHK 10111			Node NHK 10120			Node NHK 10131		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	4.81	4.58	-0.23	4.37	3.97	-0.4	4.27	3.93	-0.34	3.78	3.81	0.03	3.71	3.73	0.02
10-Year, 24-Hour	5.66	5.29	-0.37	5.56	5.03	-0.53	5.51	4.98	-0.53	4.58	4.76	0.18	4.43	4.57	0.14
25-Year, 24-Hour	6.24	5.46	-0.78	6.19	5.4	-0.79	6.14	5.35	-0.79	5.3	5.12	-0.18	5.02	4.89	-0.13

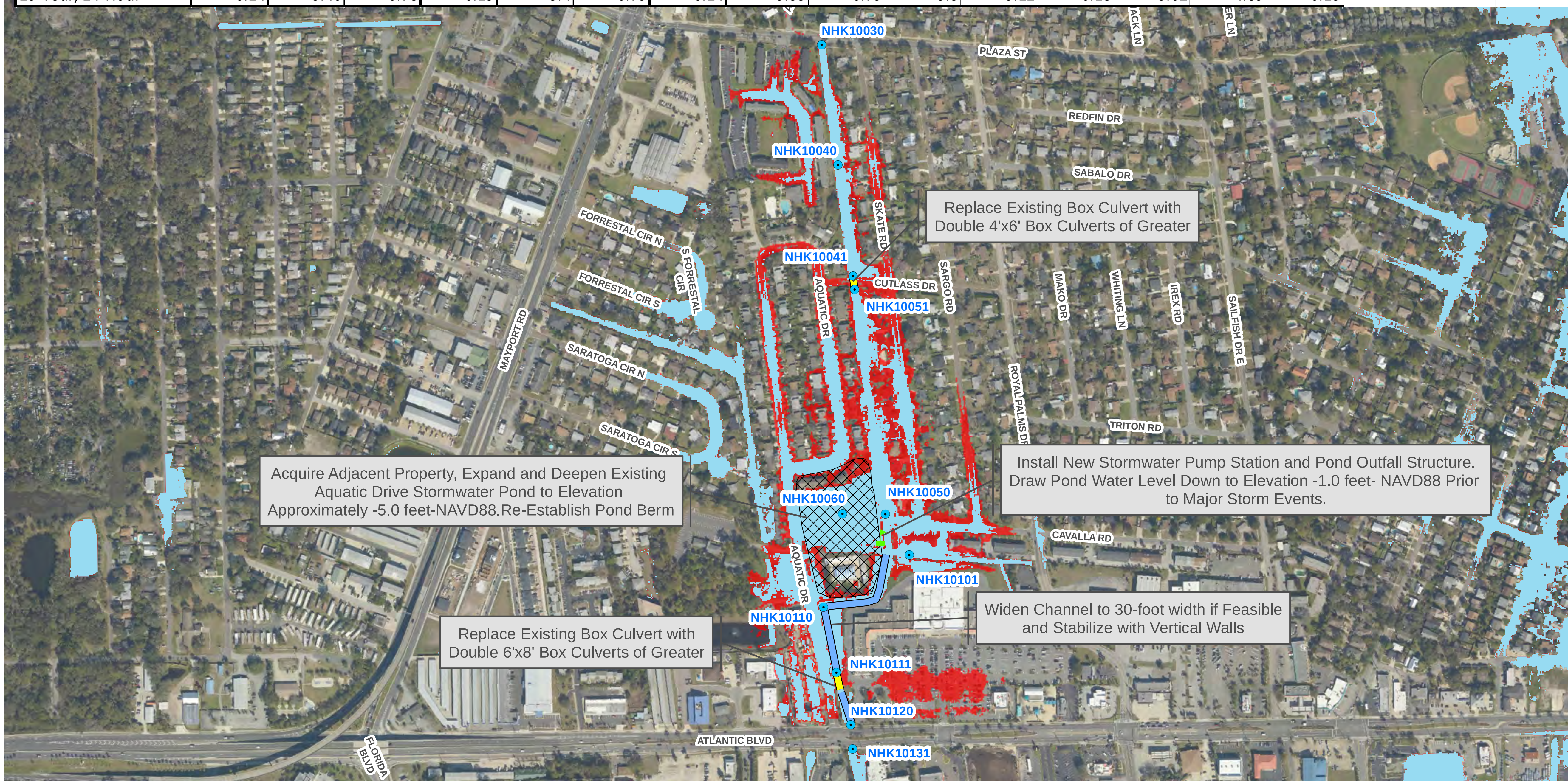


Exhibit 2C

Aquatic Drive, Cavalla Road, Skate Road All Improvements

City of Atlantic Beach

Section 38 Township - 0 Range 0

Legend

- Pump Station and Outfall
- Model Node
- Channel Improvments
- Culvert Capacity Improvements
- Proposed Conditions 25-Year, 24-Hour Inundation
- Existing Conditions 25-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
25-Year Results

Estimated Cost:
$ 10,500,000 - $ 17,100,000

N

0 250 500
Feet
1 inch equals 500 feet

JonesEdmunds

For Informational Purposes Only K:\08505 Atlantic Beach\8505.03_2017SWMPU\Design\MXD\Report\Exhibits\AquaticExhibit_2C.mxd JHirneise 10/9/2018

Design Event	Peak Stage Summary											
	Peak Water Surface Elevations (feet NAVD88)											
	Node NHK 10010			Node NHK 10000			Node NHK 10210			Node NHK 10200		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	7.1	7.78	0.68	10.61	8.8	-1.81	10.61	9.73	-0.88	10.61	9.88	-0.73
10-Year, 24-Hour	9	9.88	0.88	10.98	10.45	-0.53	11.04	10.62	-0.42	11.04	10.63	-0.41
25-Year, 24-Hour	10.8	10.65	-0.15	11.13	10.98	-0.15	11.39	11.2	-0.19	11.39	11.2	-0.19

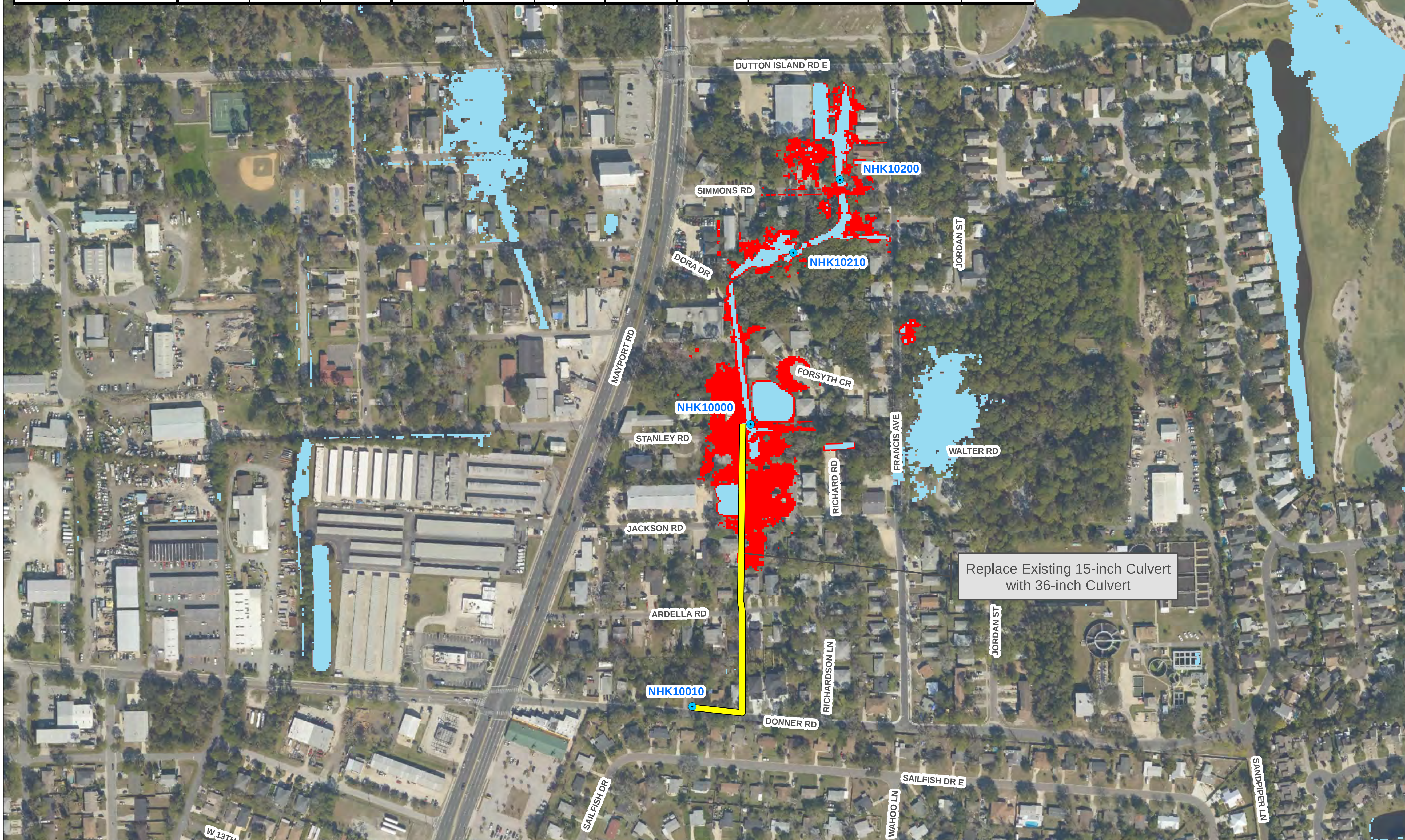
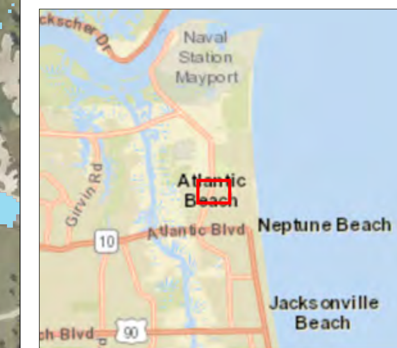


Exhibit 3A

Stanley Road, Dora Drive, Simmons Road Alternative

City of Atlantic Beach

Section 17 Township - 2S Range 29E



Legend

- Model Node
- Culvert Capacity Improvements
- Existing Conditions Mean Annual, 24-Hour Inundation
- Proposed Conditions Mean Annual, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
Mean Annual Results

Estimated Cost:
$ 390,000 - $ 640,000

N

0 150 300
Feet
1 inch equals 300 feet

JonesEdmunds

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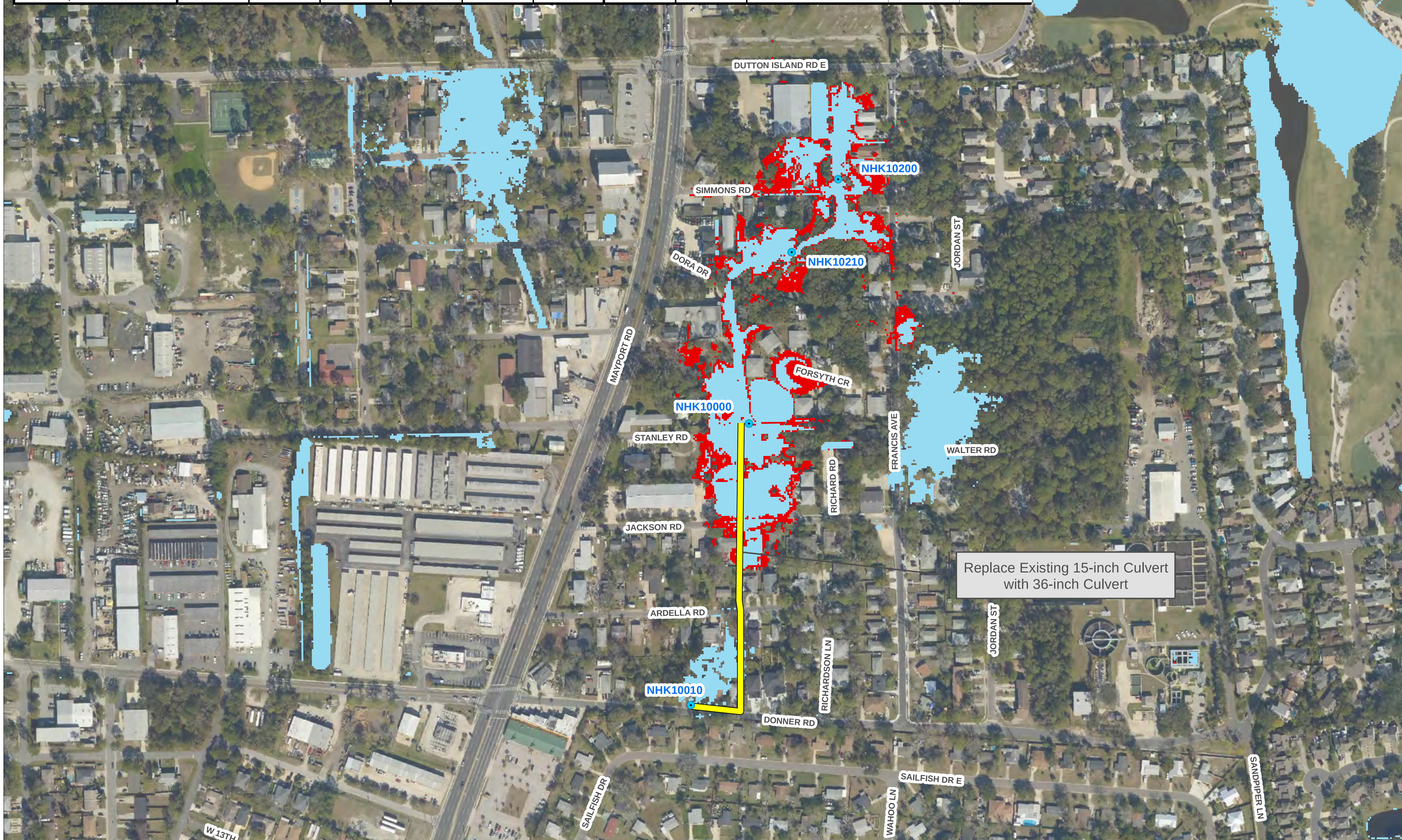
Exhibit 3B

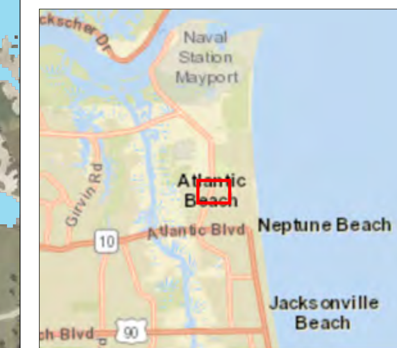
Stanley Road, Dora Drive, Simmons Road Alternative

City of Atlantic Beach

Section 17 Township - 2S Range 29E

Design Event	Peak Stage Summary											
	Peak Water Surface Elevations (feet NAVD88)											
	Node NHK 10010			Node NHK 10000			Node NHK 10210			Node NHK 10200		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	7.1	7.78	0.68	10.61	8.8	-1.81	10.61	9.73	-0.88	10.61	9.88	-0.73
10-Year, 24-Hour	9	9.88	0.88	10.98	10.45	-0.53	11.04	10.62	-0.42	11.04	10.63	-0.41
25-Year, 24-Hour	10.8	10.65	-0.15	11.13	10.98	-0.15	11.39	11.2	-0.19	11.39	11.2	-0.19





Legend

- Model Node
- Culvert Capacity Improvements
- Existing Conditions 10-Year, 24-Hour Inundation
- Proposed Conditions 10-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
10-Year Results

Estimated Cost:
$ 390,000 - $ 640,000

N

0 150 300
Feet
1 inch equals 300 feet

JonesEdmunds

For Informational Purposes Only K:\08505 Atlantic Beach\8505.03_2017SWMPU\Design\MXD\Report\Exhibits\StanleyRdExhibit_3B.mxd JHirneise 10/9/2018

Design Event	Peak Stage Summary											
	Peak Water Surface Elevations (feet NAVD88)											
	Node NHK 10010			Node NHK 10000			Node NHK 10210			Node NHK 10200		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
Mean Annual, 24-Hour	7.1	7.78	0.68	10.61	8.8	-1.81	10.61	9.73	-0.88	10.61	9.88	-0.73
10-Year, 24-Hour	9	9.88	0.88	10.98	10.45	-0.53	11.04	10.62	-0.42	11.04	10.63	-0.41
25-Year, 24-Hour	10.8	10.65	-0.15	11.13	10.98	-0.15	11.39	11.2	-0.19	11.39	11.2	-0.19

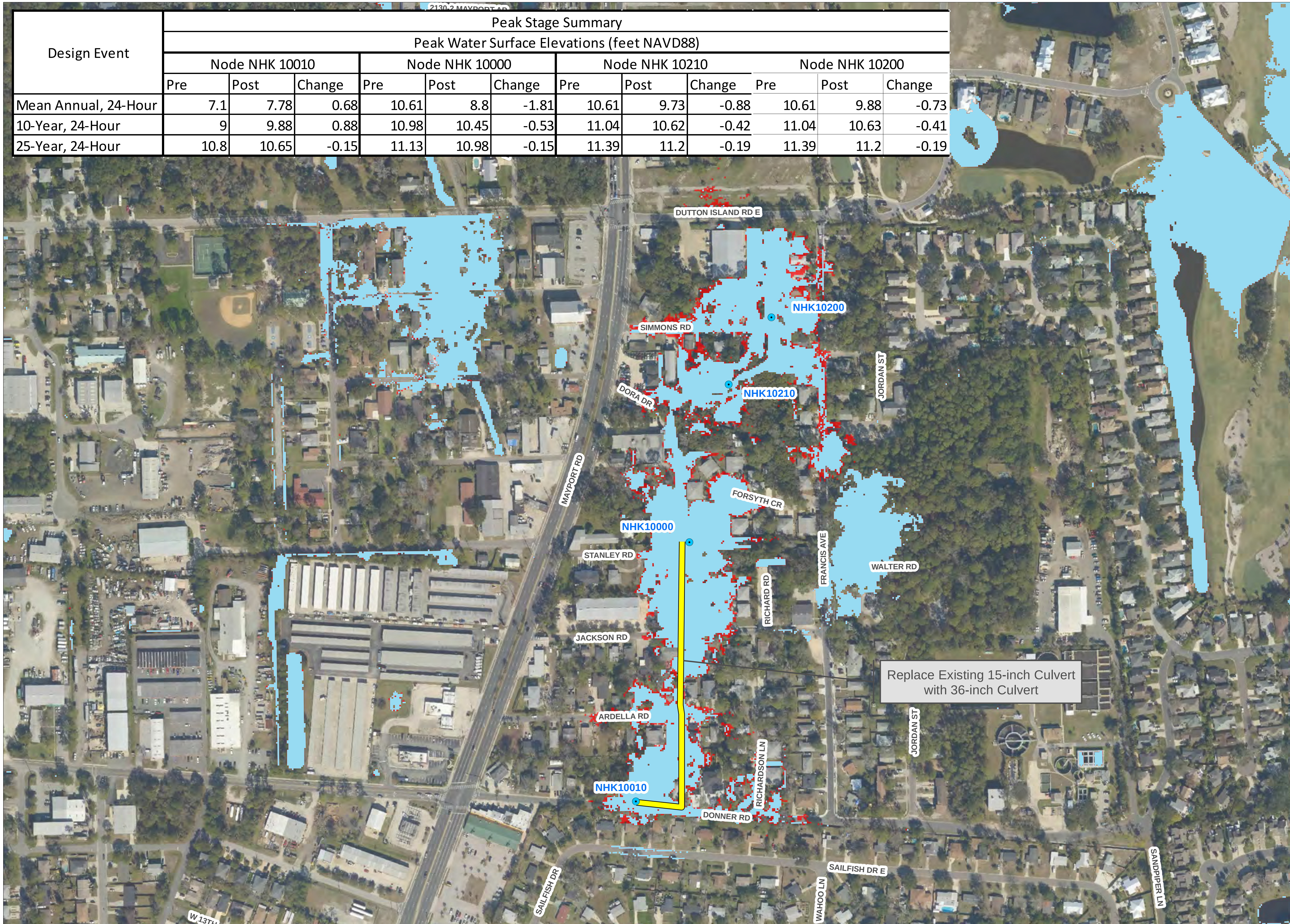
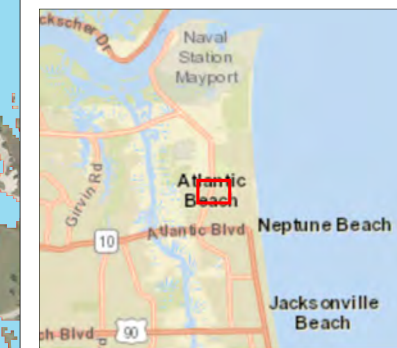


Exhibit 3C

Stanley Road, Dora Drive, Simmons Road Alternative

City of Atlantic Beach

Section 17 Township - 2S Range 29E



Legend

- Model Node
- Culvert Capacity Improvements
- Existing Conditions 25-Year, 24-Hour Inundation
- Proposed Conditions 25-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage
25-Year Results

Estimated Cost:
$ 390,000 - $ 640,000

N

0 150 300
Feet
1 inch equals 300 feet

JonesEdmunds

For Informational Purposes Only K:\08505 Atlantic Beach\8505.03_2017SWMPU\Design\MXD\Report\Exhibits\StanleyRdExhibit_3C.mxd JHirneise 10/9/2018

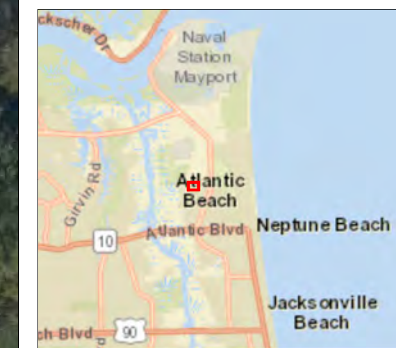
Drainage Impacts for Mary Street Alternative			
Design Event	Peak Water Surface Elevation (feet NAVD 88)		
	Node NSP30110		
	Pre	Post	Change
10-year, 24-hour	10.97	10.85	-0.12
25-year, 24-hour	10.99	10.91	-0.08
100-year, 24-hour	11.02	10.98	-0.04
Design Event	Proposed Conditions Duration of Roadway Centerline Inundation (hours)		
	Node NSP30110		
10-year, 24-hour	1.4		
25-year, 24-hour	1.7		
100-year, 24-hour	2.0		
Note: Existing Centerline Inundation Duration > 24 HRS For All Events			

Exhibit 4

Mary and Stewart Street

City of Atlantic Beach

Section 17 Township -2S Range 29E



Legend

- Existing Inlet
- Existing Culvert
- Proposed New 12" RCP Culvert
- Replace Existing Culvert with 12" RCP
- Proposed Swale Improvements
- Existing Conditions 25-Year, 24-Hour Inundation

Flood Mitigation BMP
Type: Enhanced Drainage

Estimated Cost:
$ 270,000 - $ 430,000

N

0 50 100
Feet
1 inch equals 100 feet

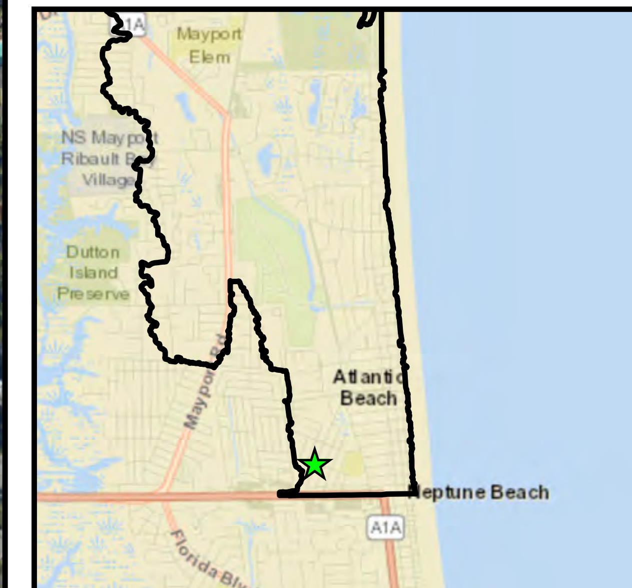
JonesEdmunds

For Informational Purposes Only Q:\08505_AtlanticBeach\StormwaterMasterPlanUpdate\Report\BMP_Alternatives\Mary_Street\Alt_MarySt.mxd JHirneise 10/10/2018

Exhibit 5A

Seminole South

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed Mean Annual Flooding
- Existing Mean Annual Flooding

Mean Annual Results

N

0 165 330 660 Feet

Peak Stage Summary

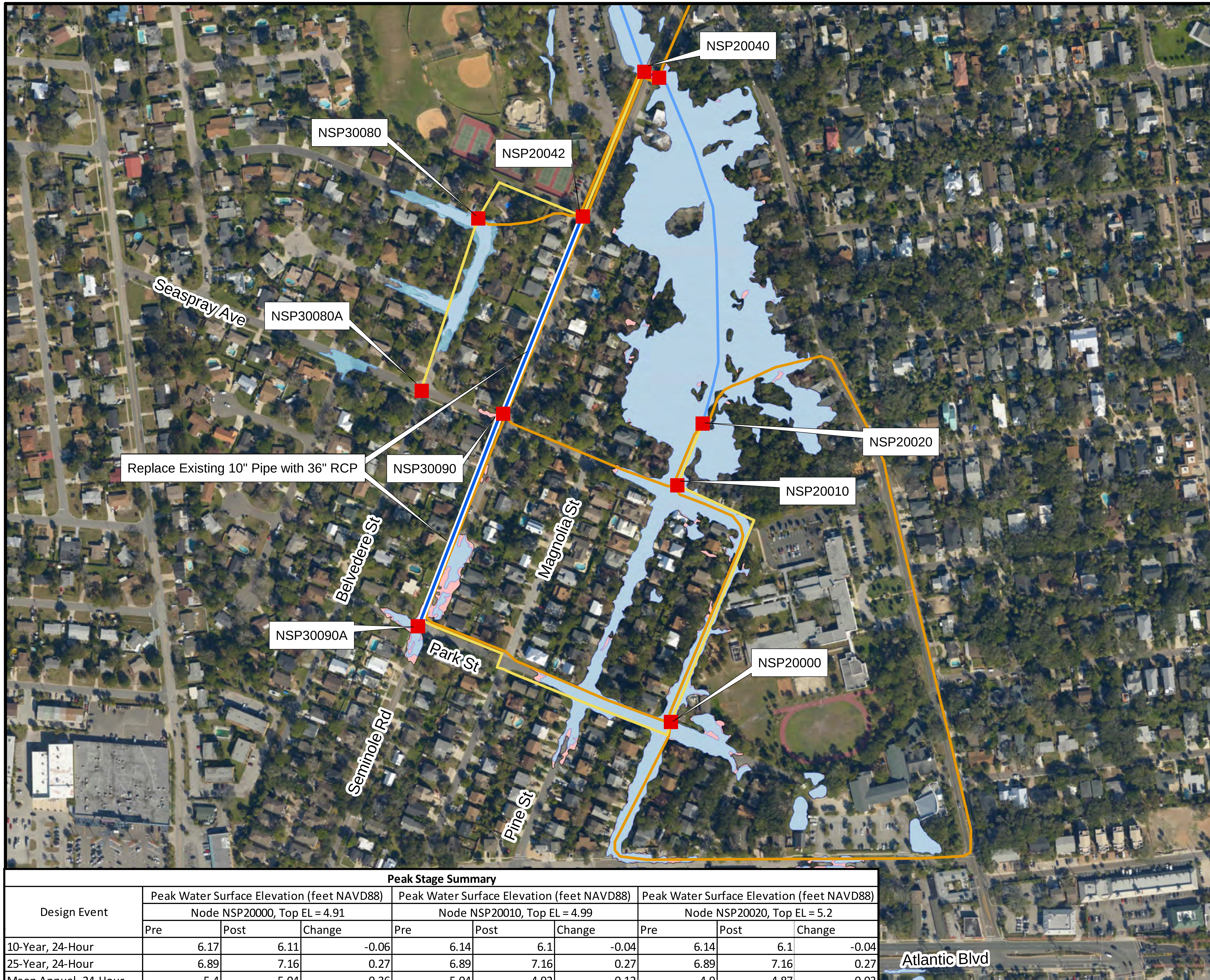
Design Event	Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)		
	Node NSP20000, Top EL = 4.91			Node NSP20010, Top EL = 4.99			Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.11	-0.06	6.14	6.1	-0.04	6.14	6.1	-0.04
25-Year, 24-Hour	6.89	7.16	0.27	6.89	7.16	0.27	6.89	7.16	0.27
Mean Annual, 24-Hour	5.4	5.04	-0.36	5.04	4.92	-0.12	4.9	4.87	-0.03

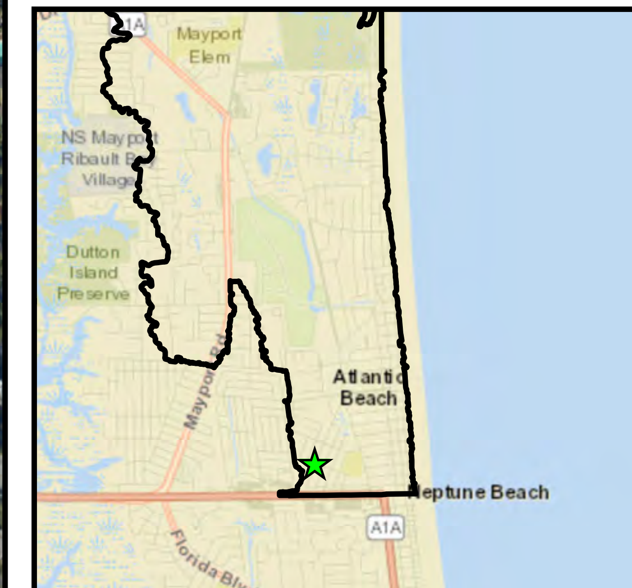
HANSON

Exhibit 5B

Seminole South

Section 16&17 Township 2S Range 29E





Legend

- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed 10 Yr Flooding
- Existing 10 Yr Flooding

10-year, 24-hour Results

N

0 155 310 620 Feet

HANSON

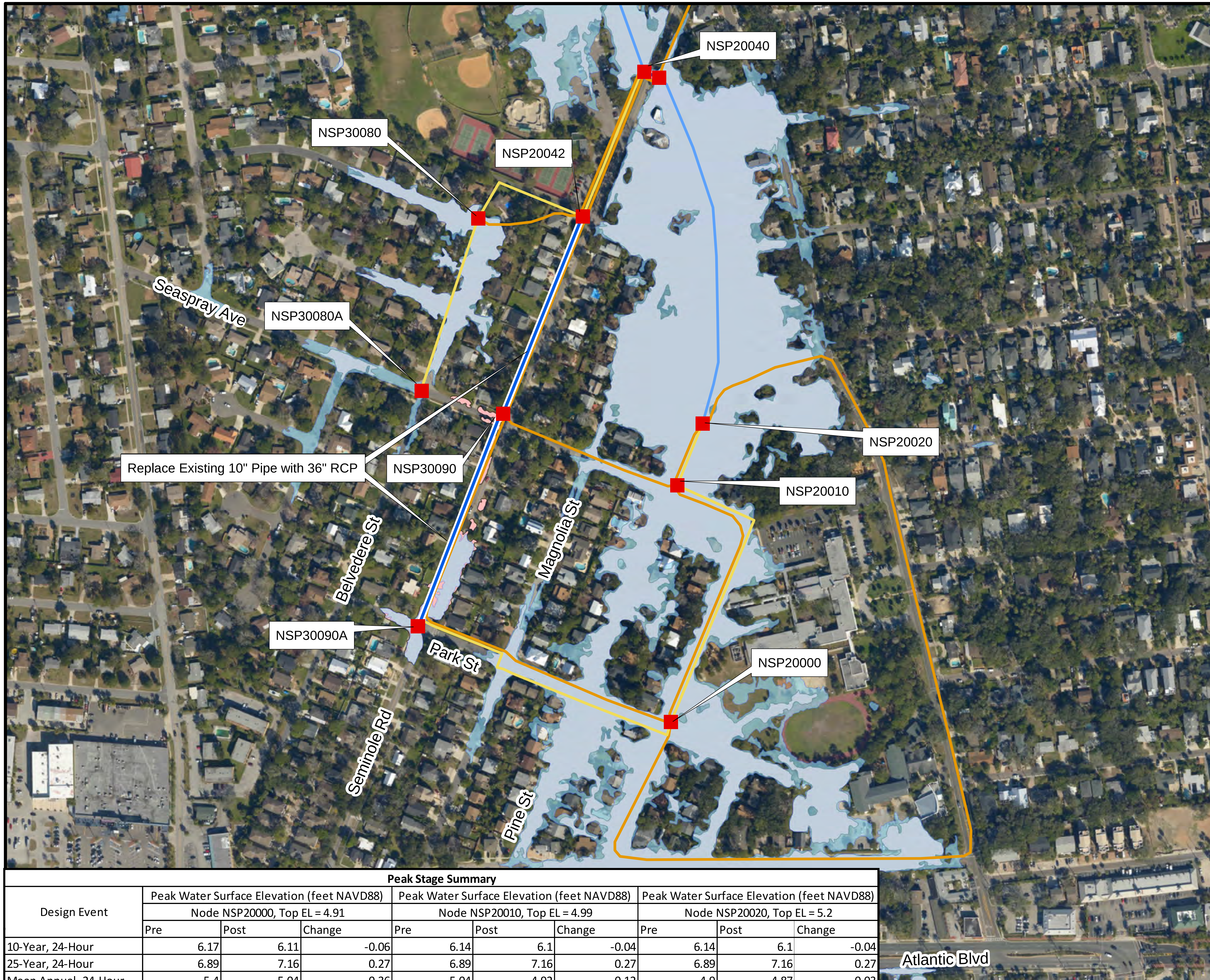
Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)		
	Node NSP20000, Top EL = 4.91			Node NSP20010, Top EL = 4.99			Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.11	-0.06	6.14	6.1	-0.04	6.14	6.1	-0.04
25-Year, 24-Hour	6.89	7.16	0.27	6.89	7.16	0.27	6.89	7.16	0.27
Mean Annual, 24-Hour	5.4	5.04	-0.36	5.04	4.92	-0.12	4.9	4.87	-0.03

Exhibit 5C

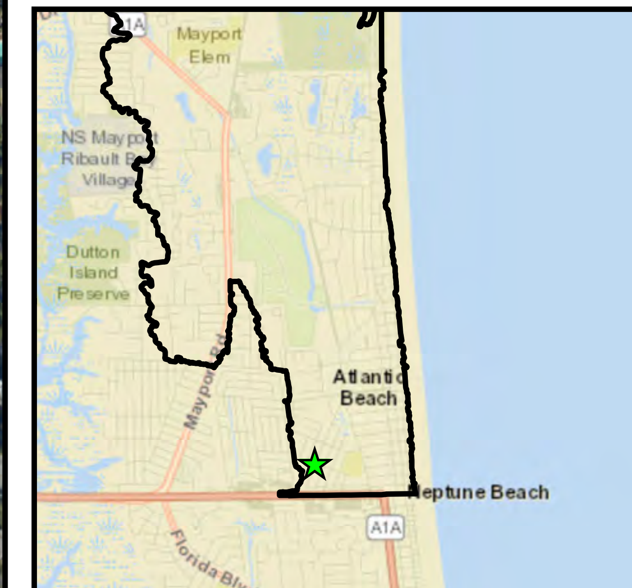
Seminole South

Section 16&17 Township 2S Range 29E



Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)		
	Node NSP20000, Top EL = 4.91			Node NSP20010, Top EL = 4.99			Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.11	-0.06	6.14	6.1	-0.04	6.14	6.1	-0.04
25-Year, 24-Hour	6.89	7.16	0.27	6.89	7.16	0.27	6.89	7.16	0.27
Mean Annual, 24-Hour	5.4	5.04	-0.36	5.04	4.92	-0.12	4.9	4.87	-0.03



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed 25 Yr Flooding
- Existing 25 Yr Flooding

25-year, 24-hour Results

N

0 150 300 600 Feet

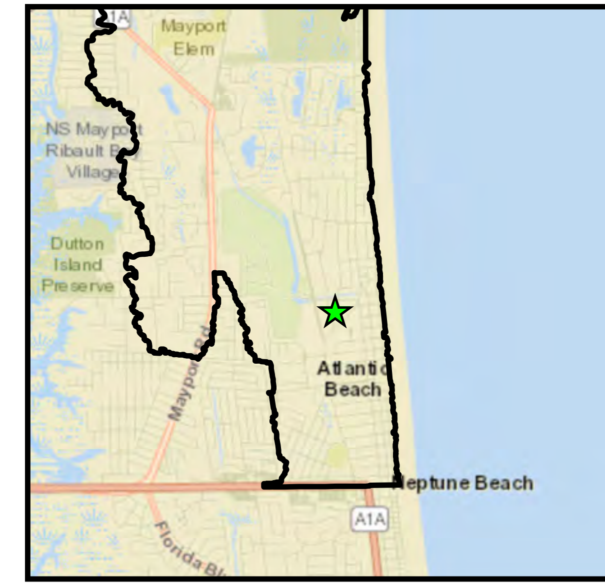
HANSON

Peak Stage Summary						
Design Event	Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)		
	Node NSP20101, Top EL = 6.2			Node NSP20121, Top EL = 6.2		
	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.5	6.97	-0.53	7.5	6.97	-0.53
25-Year, 24-Hour	7.78	7.53	-0.25	7.78	7.53	-0.25
Mean Annual, 24-Hour	6.67	5.77	-0.9	6.67	5.64	-1.03

Exhibit 6A

Johansen Park

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed Mean Annual Flooding
- Existing Mean Annual Flooding

Mean Annual Results

N

0 330 660 1,320 Feet

HANSON

Peak Stage Summary						
Design Event	Peak Water Surface Elevation (feet NAVD88)			Peak Water Surface Elevation (feet NAVD88)		
	Node NSP20101, Top EL = 6.2			Node NSP20121, Top EL = 6.2		
	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.5	6.97	-0.53	7.5	6.97	-0.53
25-Year, 24-Hour	7.78	7.53	-0.25	7.78	7.53	-0.25
Mean Annual, 24-Hour	6.67	5.77	-0.9	6.67	5.64	-1.03

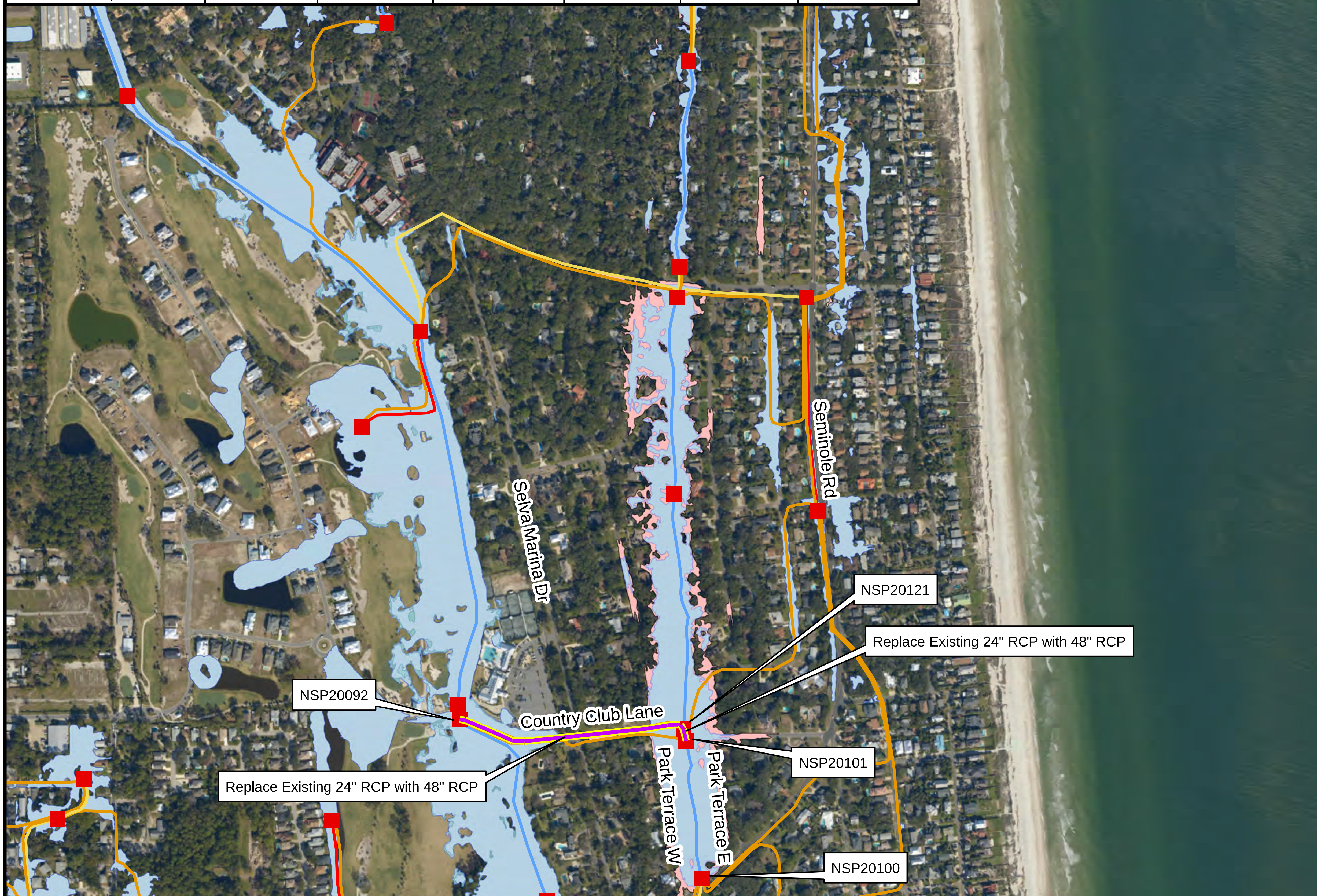
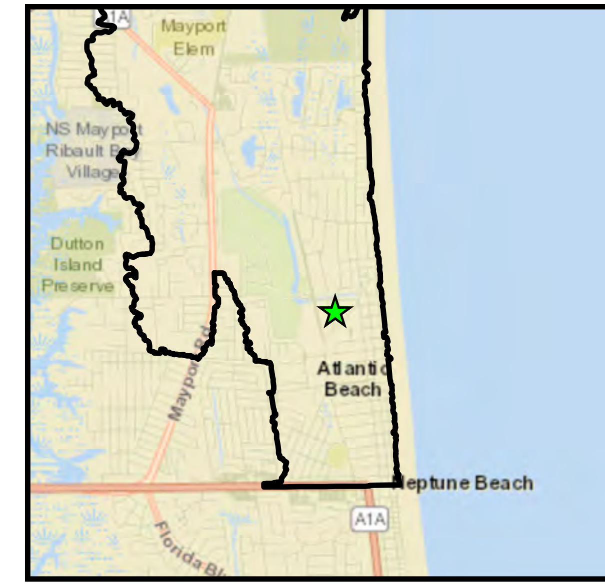


Exhibit 6B

Johansen Park

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed 10 Yr Flooding
- Existing 10 Yr Flooding

10-year, 24-hour Results

N

0 330 660 1,320 Feet

HANSON

Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88) Node NSP20101, Top EL = 6.2			Peak Water Surface Elevation (feet NAVD88) Node NSP20121, Top EL = 6.2		
	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.5	6.97	-0.53	7.5	6.97	-0.53
25-Year, 24-Hour	7.78	7.53	-0.25	7.78	7.53	-0.25
Mean Annual, 24-Hour	6.67	5.77	-0.9	6.67	5.64	-1.03

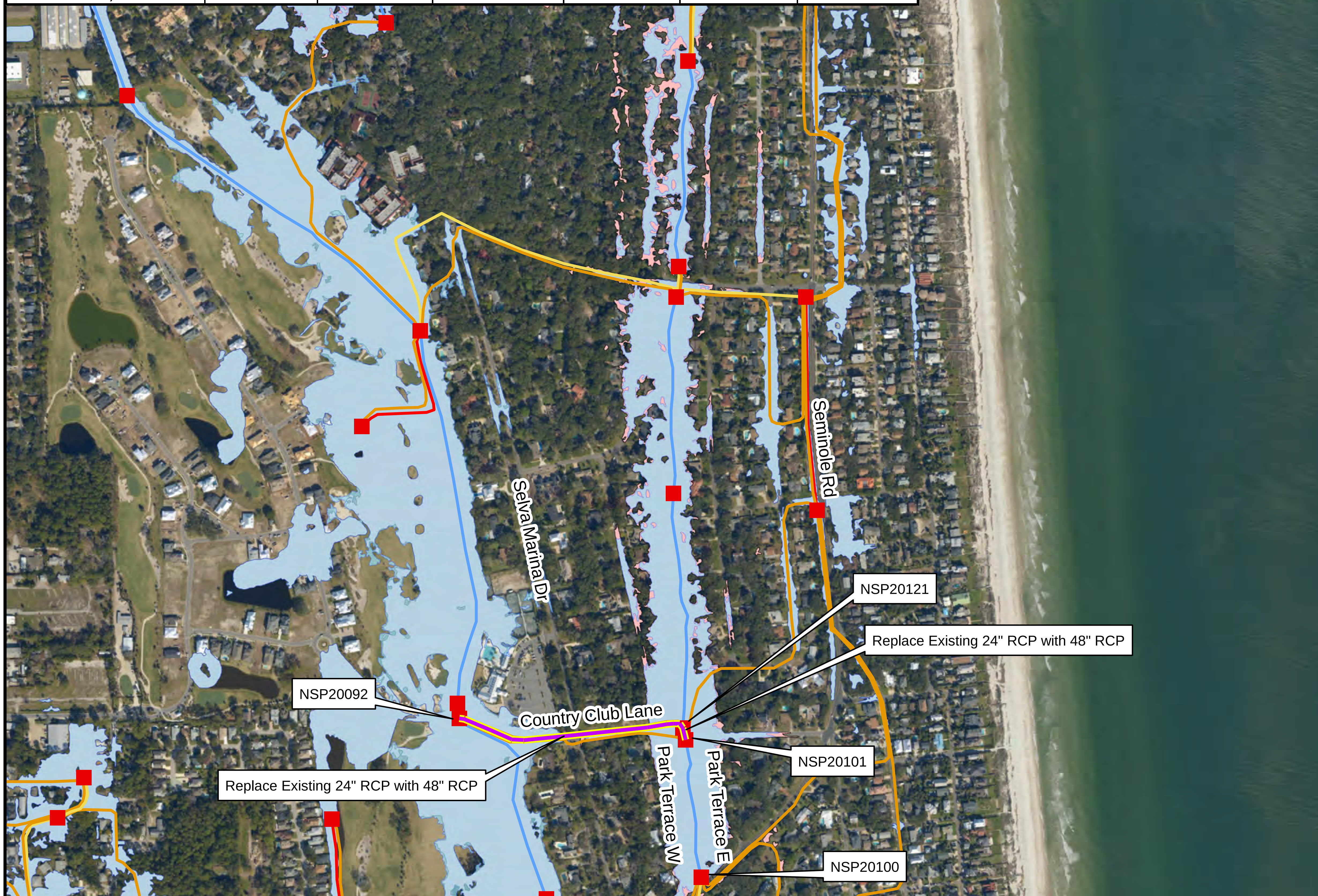
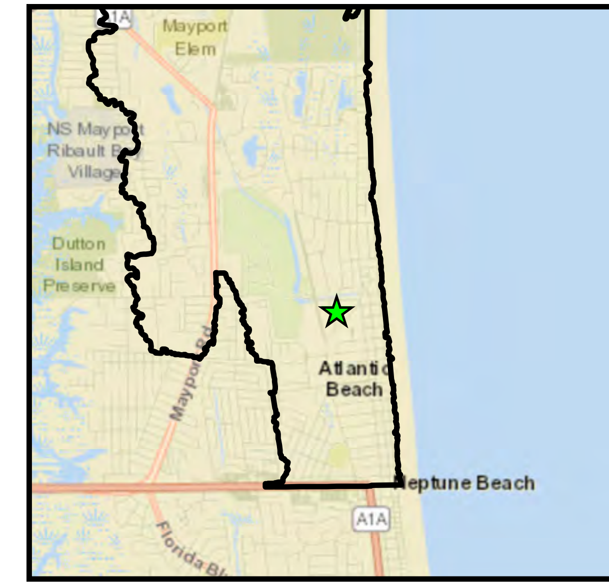


Exhibit 6C

Johansen Park

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed 25 Yr Flooding
- Existing 25 Yr Flooding

25-year, 24-hour Results

N

0 330 660 1,320 Feet

HANSON

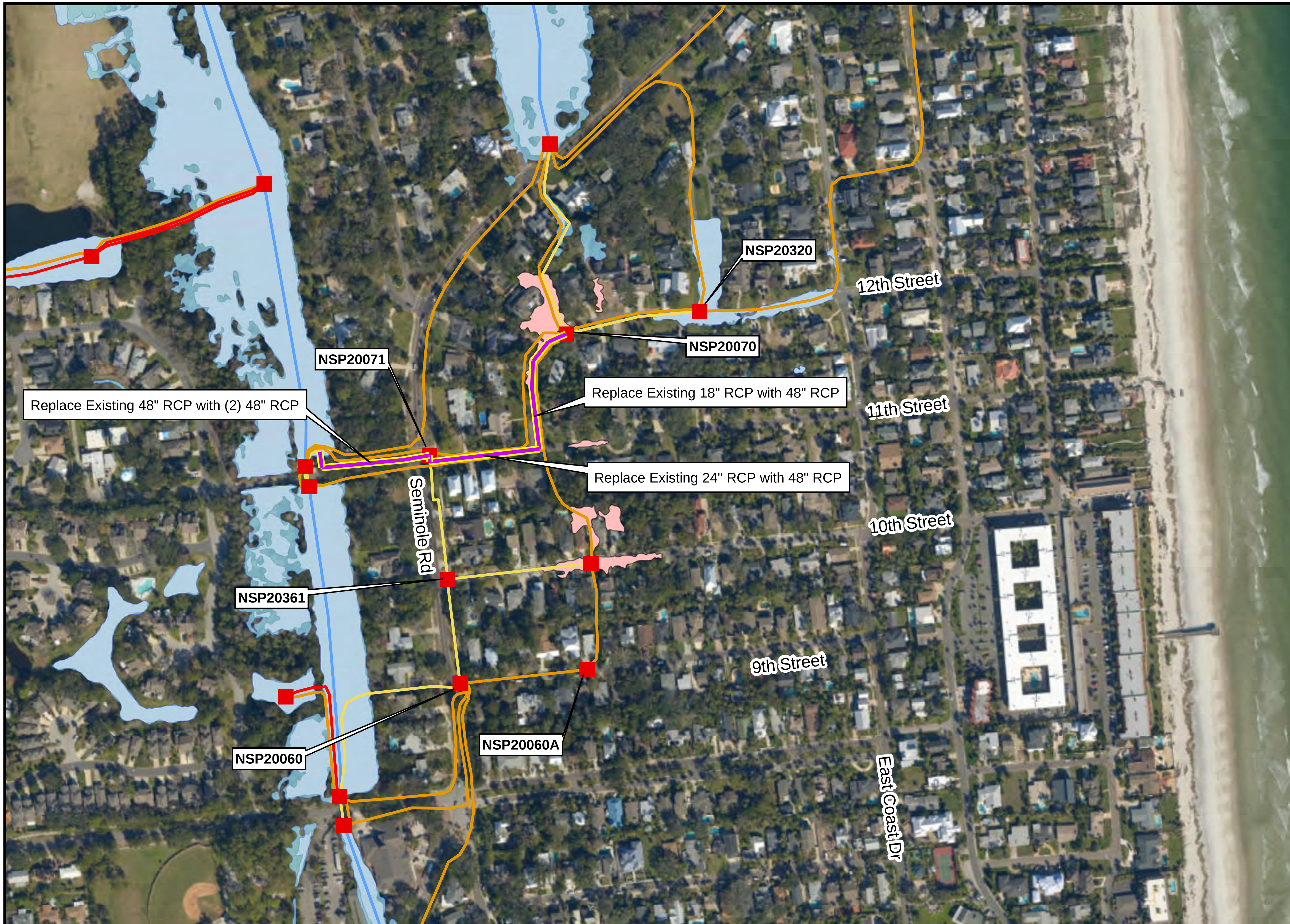
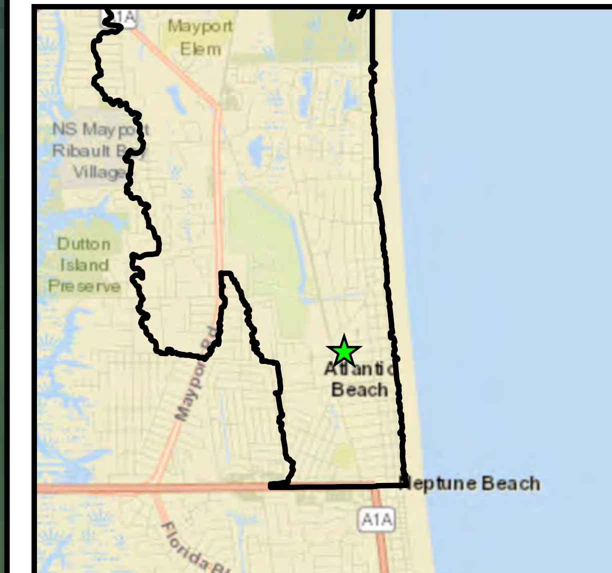


Exhibit 7A

9th, 10th, 11th, & 12th Streets Phase I

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase One
- Proposed Mean Annual Flooding
- Existing Mean Annual Flooding

Mean Annual Results

HANSON

Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)																				
	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

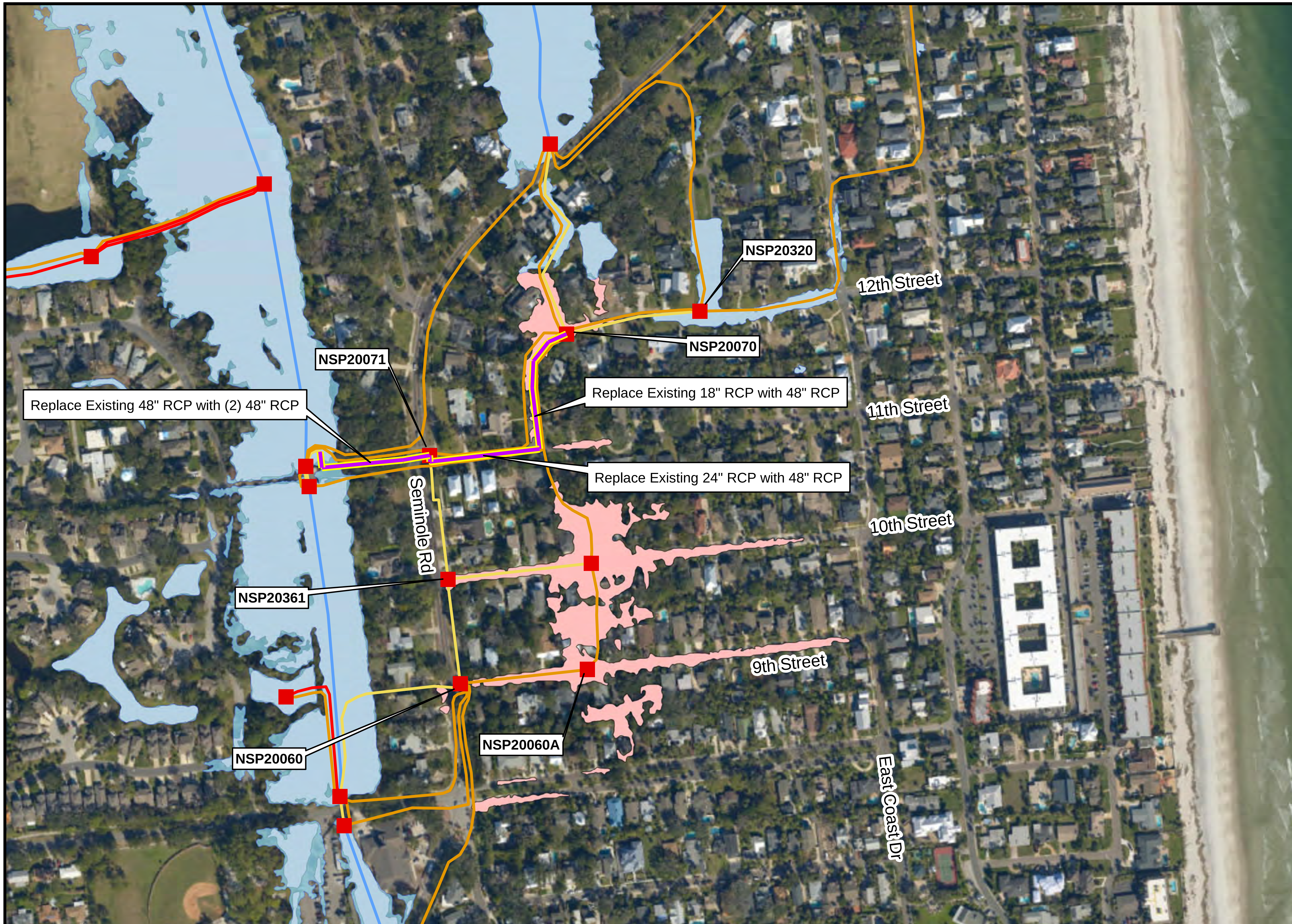
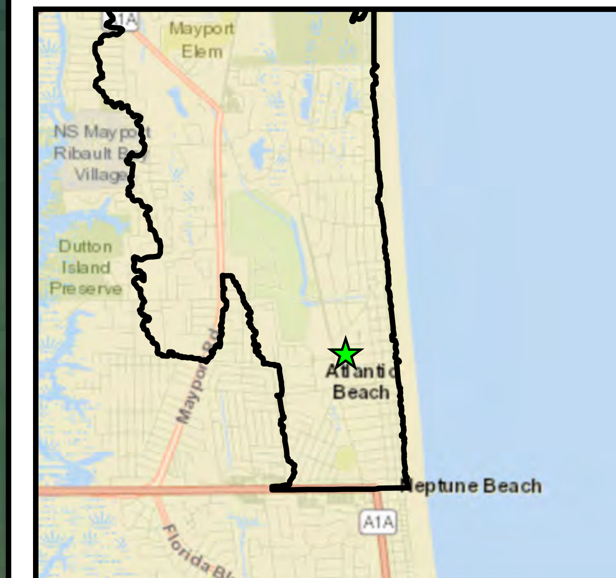


Exhibit 7B

9th, 10th, 11th, & 12th Streets Phase 1

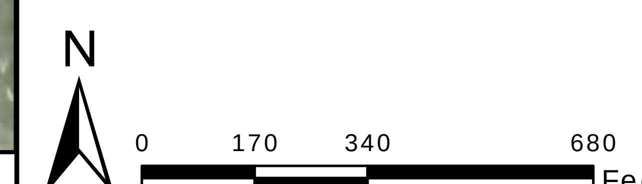
Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase One
- Proposed 10 Yr Flooding
- Existing 10 Yr Flooding

10-year, 24-hour Results



HANSON

Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)																				
	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

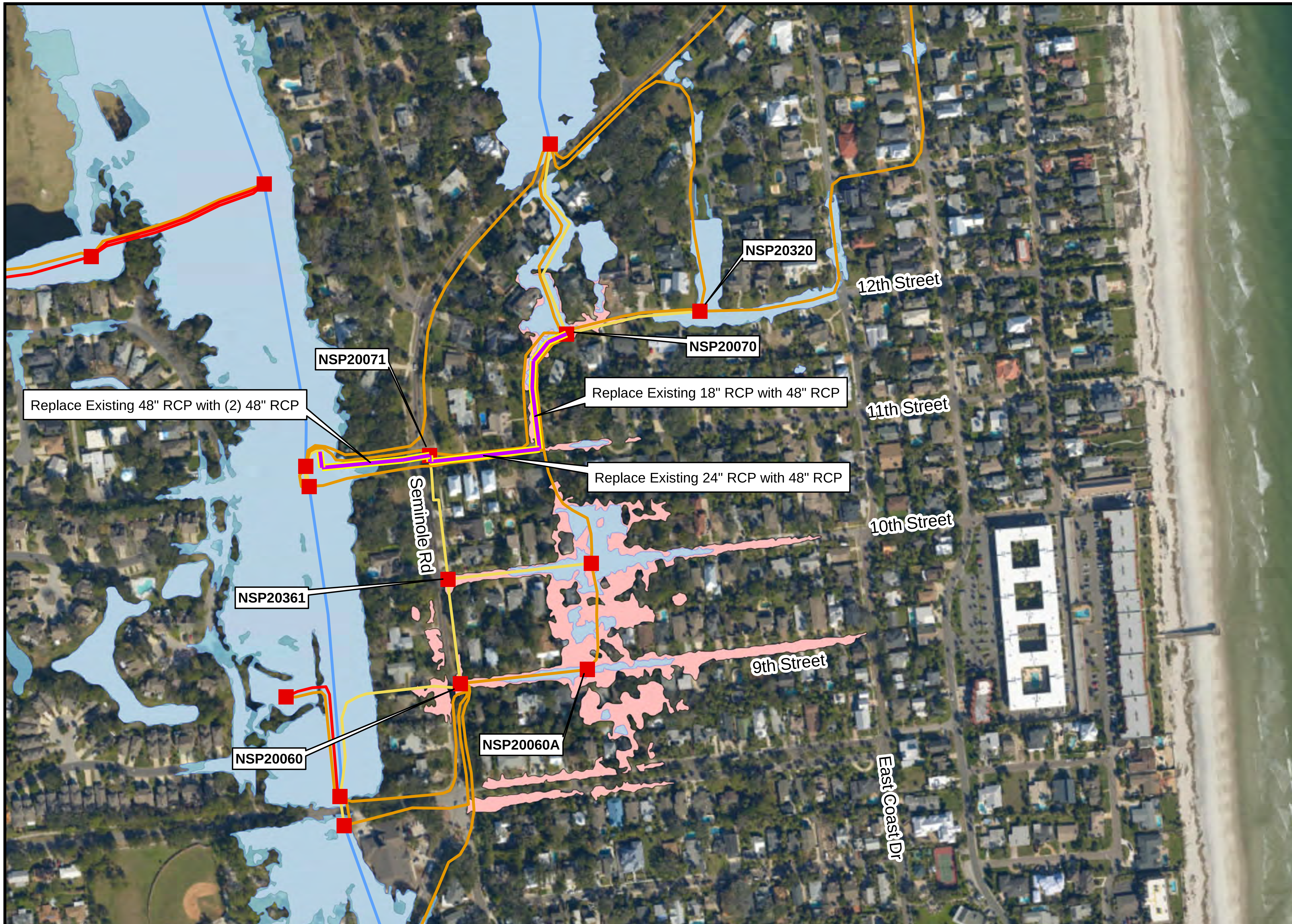
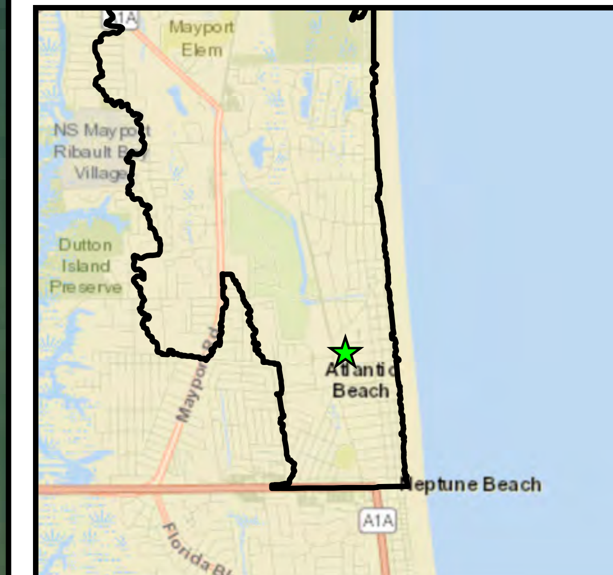


Exhibit 7C

9th, 10th, 11th, & 12th Streets Phase 1

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase One
- Proposed 25 Yr Flooding
- Existing 25 Yr Flooding

25-year, 24-hour Results

N

0 170 340 680 Feet

HANSON

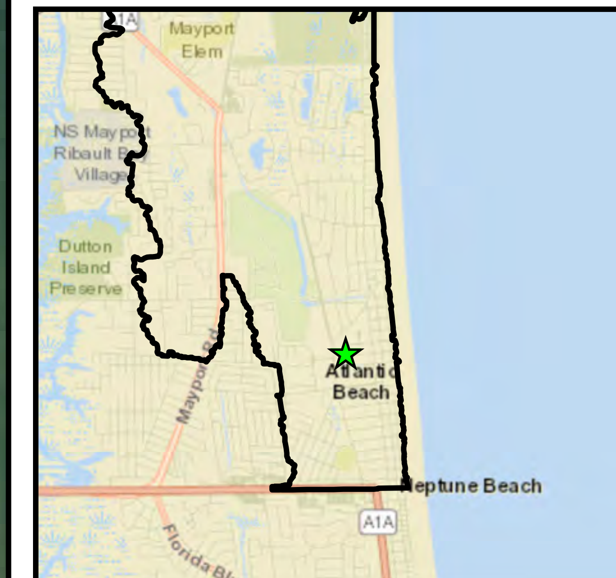
Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)																				
	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

Exhibit 8A

9th, 10th, 11th, & 12th Streets Phase 2

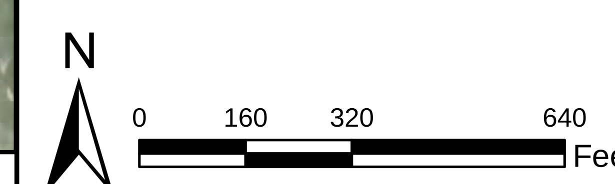
Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase Two
- Proposed Mean Annual Flooding
- Existing Mean Annual Flooding

Mean Annual Results



Peak Stage Summary

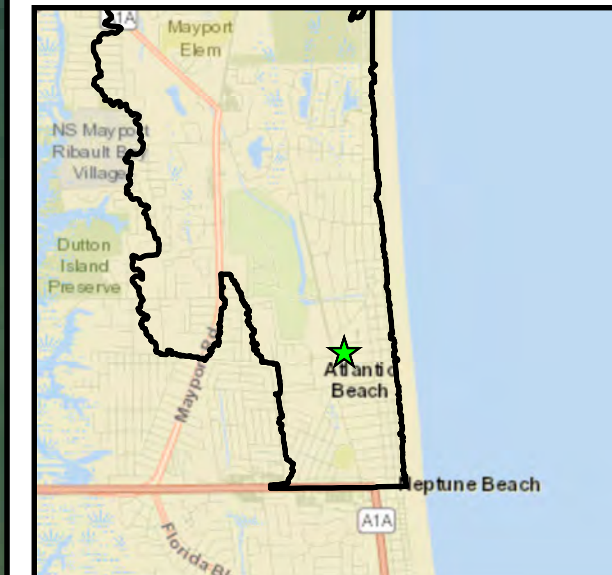
Design Event	Peak Water Surface Elevation (feet NAVD88)																				
	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

HANSON

Exhibit 8B

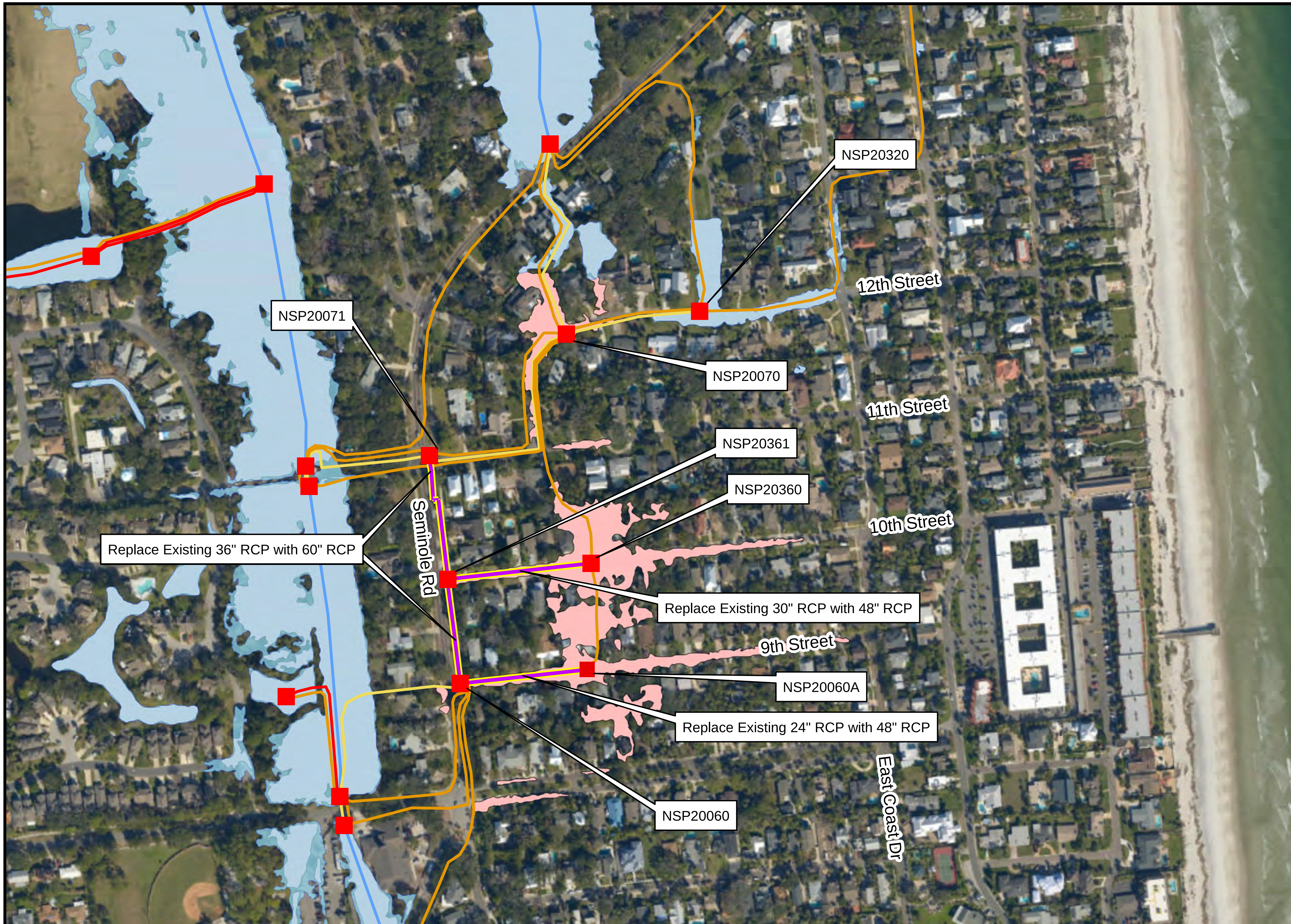
9th, 10th, 11th, & 12th Streets Phase 2

Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase Two
- Proposed 10 Yr Flooding
- Existing 10 Yr Flooding



10-year, 24-hour Results

N

0 170 340 680 Feet

HANSON

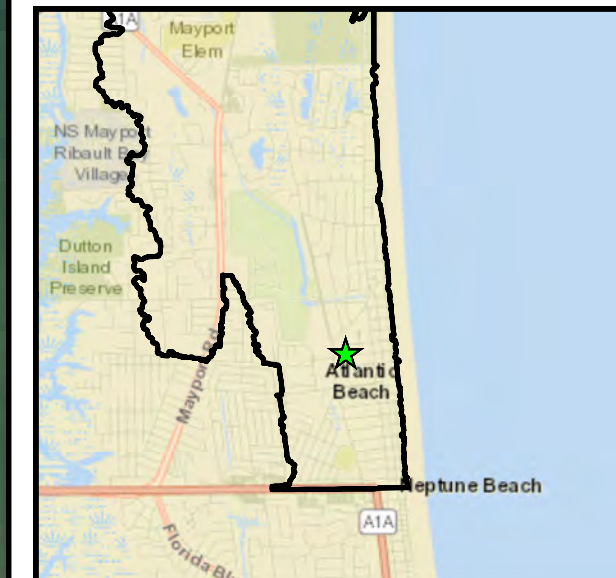
Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88)																				
	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

Exhibit 8C

9th, 10th, 11th, & 12th Streets Phase 2

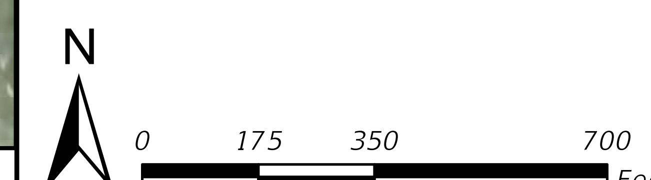
Section 16&17 Township 2S Range 29E



Legend

- STAGE/AREA NODES
- CHANNEL LINK
- DROP STRUCTURE
- PIPE
- WEIR LINK
- Proposed Pipes - Phase Two
- Proposed 25 Yr Flooding
- Existing 25 Yr Flooding

25-year, 24-hour Results



Peak Stage Summary

Design Event	Node NSP20070, Top EL = 7.8			Node NSP20071, Top EL = 9.66			Node NSP20320, Top EL = 8.00			Node NSP20361, Top EL = 9.56			Node NSP20360, Top EL = 9.19			Node NSP20060A, Top EL = 9.82			Node NSP20060, Top EL = 9.45		
	Peak Water Surface Elevation (feet NAVD88)																				
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	7.81	6.62	-1.19	5.67	6.01	0.34	8.35	8.38	0.03	7.73	6.6	-1.13	8.64	7.11	-1.53	8.36	6.78	-1.58	8.36	6.78	-1.58
25-Year, 24-Hour	7.9	7.66	-0.24	6.37	6.63	0.26	8.42	8.45	0.03	7.96	7.55	-0.41	8.74	8.24	-0.5	8.67	7.8	-0.87	8.67	7.8	-0.87
Mean Annual, 24-Hour	7.67	5.65	-2.02	5.25	5.1	-0.15	8.24	8.28	0.04	6.86	5.2	-1.66	7.93	5.41	-2.52	7.27	5.33	-1.94	7.26	5.33	-1.93

HANSON

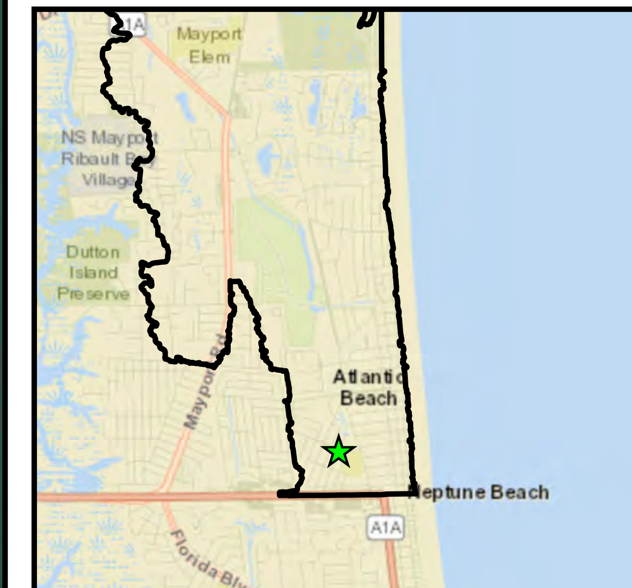
Exhibit 9A

Howell Park/Salt Air

Section 16&17 Township 2S Range 29E

Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88) Node NSP20000, Top EL = 4.91			Peak Water Surface Elevation (feet NAVD88) Node NSP20010, Top EL = 4.99			Peak Water Surface Elevation (feet NAVD88) Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.23	0.06	6.14	6.08	-0.06	6.14	6.07	-0.07
25-Year, 24-Hour	6.89	6.71	-0.18	6.89	6.71	-0.18	6.89	6.7	-0.19
Mean Annual, 24-Hour	5.4	5.84	0.44	5.04	5.27	0.23	4.9	5.09	0.19



Legend

- Proposed Pipes
- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed Mean Annual Flooding
- Existing Mean Annual Flooding

Mean Annual Results

N

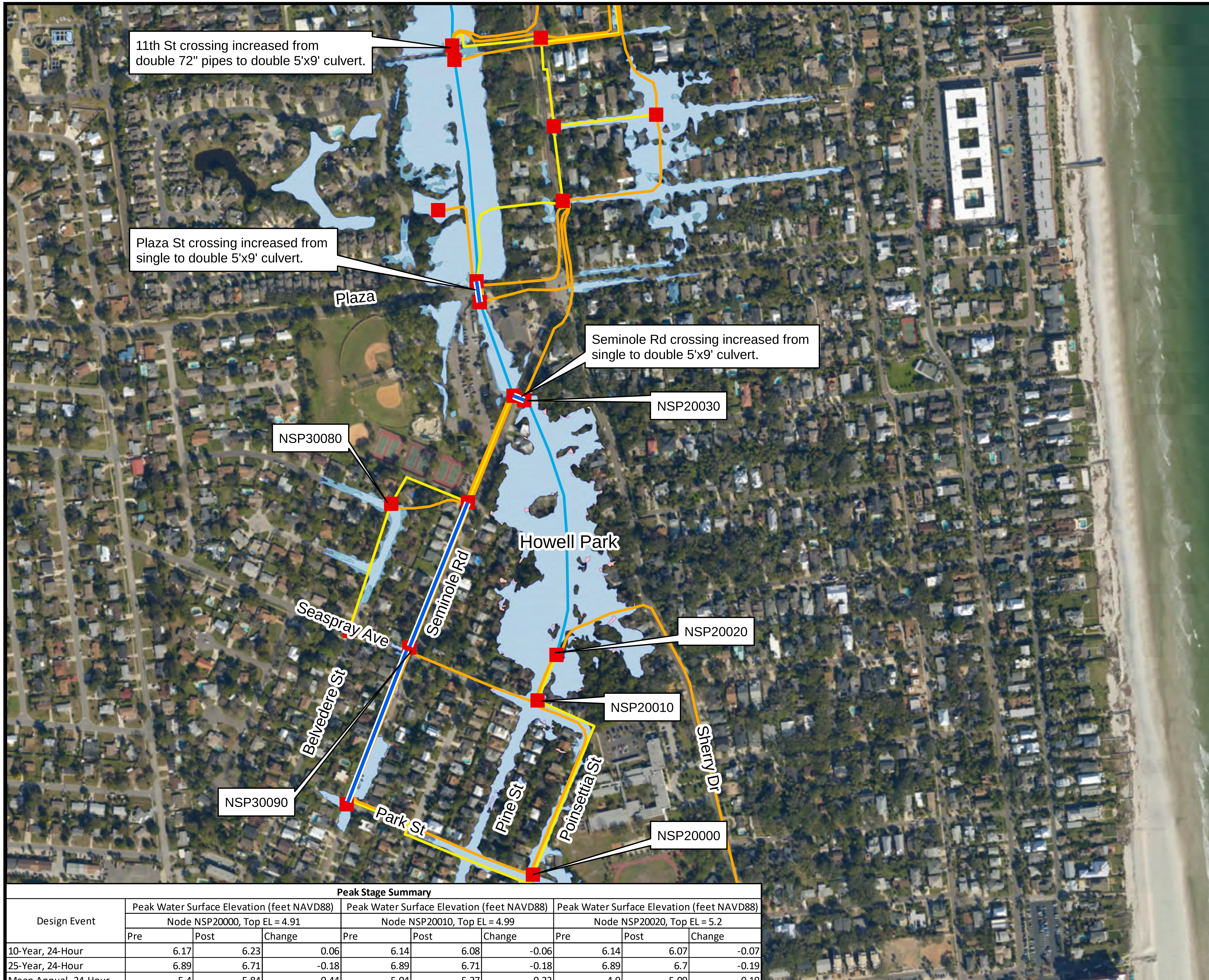
0 240 480 960 Feet

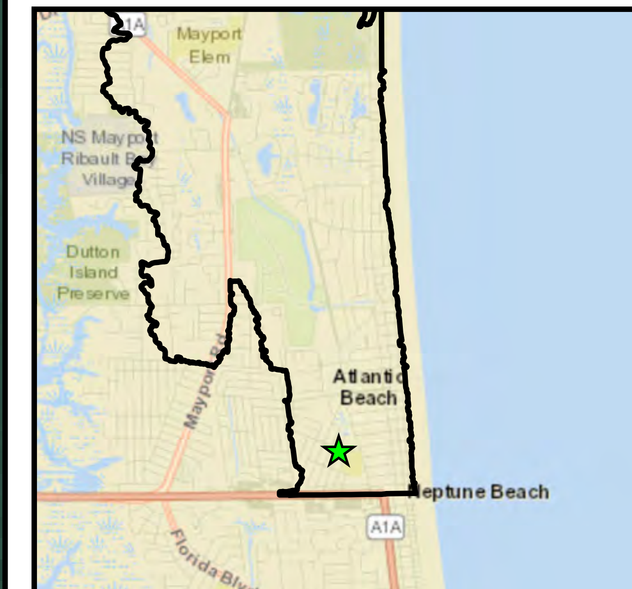
HANSON

Exhibit 9B

Howell Park/Salt Air

Section 16&17 Township 2S Range 29E





Legend

- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed Pipes
- Proposed 10 Yr Flooding
- Existing 10 Yr Flooding

10-year, 24-hour Results

N

0 225 450 900 Feet

Peak Stage Summary

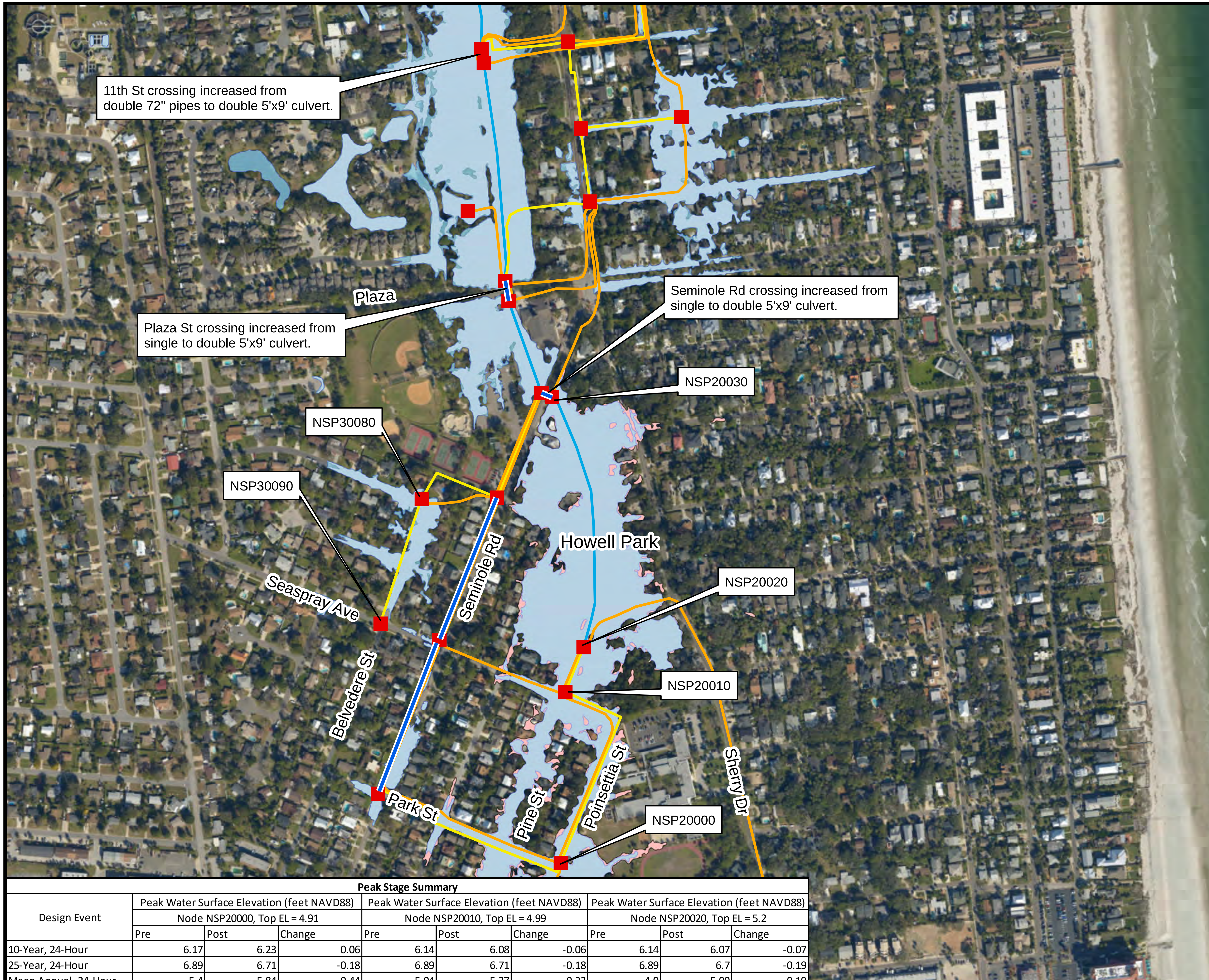
Design Event	Peak Water Surface Elevation (feet NAVD88) Node NSP20000, Top EL = 4.91			Peak Water Surface Elevation (feet NAVD88) Node NSP20010, Top EL = 4.99			Peak Water Surface Elevation (feet NAVD88) Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.23	0.06	6.14	6.08	-0.06	6.14	6.07	-0.07
25-Year, 24-Hour	6.89	6.71	-0.18	6.89	6.71	-0.18	6.89	6.7	-0.19
Mean Annual, 24-Hour	5.4	5.84	0.44	5.04	5.27	0.23	4.9	5.09	0.19

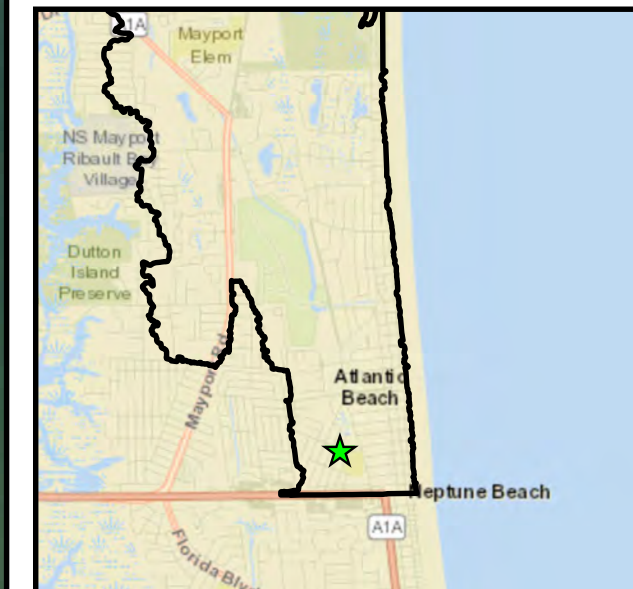
HANSON

Exhibit 9C

Howell Park/Salt Air

Section 16&17 Township 2S Range 29E





Legend

- Proposed Pipes
- STAGE/AREA NODES
- CHANNEL LINK
- PIPE
- WEIR LINK
- Proposed 25yr Flooding
- Existing 25 Yr Flooding

25-year, 24-hour Results

N

0 230 460 920 Feet

HANSON

Peak Stage Summary

Design Event	Peak Water Surface Elevation (feet NAVD88) Node NSP20000, Top EL = 4.91			Peak Water Surface Elevation (feet NAVD88) Node NSP20010, Top EL = 4.99			Peak Water Surface Elevation (feet NAVD88) Node NSP20020, Top EL = 5.2		
	Pre	Post	Change	Pre	Post	Change	Pre	Post	Change
10-Year, 24-Hour	6.17	6.23	0.06	6.14	6.08	-0.06	6.14	6.07	-0.07
25-Year, 24-Hour	6.89	6.71	-0.18	6.89	6.71	-0.18	6.89	6.7	-0.19
Mean Annual, 24-Hour	5.4	5.84	0.44	5.04	5.27	0.23	4.9	5.09	0.19

Appendix B

Engineer’s Opinion of Probable Cost

Table 1 – Aquatic Drive Phase 1

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway/Civil						
Mobilization (10%)	LS	1	$111,970	$89,600	$145,600	$112,000
Maintenance of Traffic (5%)	LS	1	$55,985	$44,800	$72,800	$56,000
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$10,000	$8,000	$13,000	$10,000
Aquatic Village Box Culvert Replacement						
Removal of Existing Pavement	SY	280	$25	$5,600	$9,100	$7,000
Removal of Existing Curb	LF	130	$25	$2,700	$4,300	$3,300
Removal of Existing Box Culvert	LS	1	$15,000	$12,000	$19,500	$15,000
6-Foot-x-8-Foot Double Box Culvert with Headwalls	LF	120	$1,700	$163,200	$265,200	$204,000
Curb Replacement	LF	130	$15	$1,700	$2,800	$2,100
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	280	$16	$3,700	$6,000	$4,600
Superpave Asphalt Concrete (Traffic C)	TN	31	$90	$2,300	$3,700	$2,800
Temporary Flow Bypass	LS	1	$20,000	$16,000	$26,000	$20,000
Sodding	SY	150	$3	$400	$600	$400
Dewatering	LS	1	$25,000	$20,000	$32,500	$25,000
Pond And Pump Station Construction						
Pond Excavation	CY	25,000	$10	$200,000	$325,000	$250,000
Pond Sod	SY	4,000	$3	$8,000	$13,000	$10,000
Outfall Structure	EA	1	$4,000	$3,200	$5,200	$4,000
Outfall Pipe	LF	30	$82	$2,000	$3,300	$2,500
Pump/Wet Well	LS	1	$400,000	$320,000	$520,000	$400,000
Dewatering	LS	1	$75,000	$60,000	$97,500	$75,000
Minor Channel Improvements	LF	700	$120	$67,200	$109,200	$84,000
Overall Items						
Contingency Amount (40%)	LS	1	$515,080	$412,100	$669,700	$515,100
Design/Permitting/Data Collection (25%)	LS	1	$321,925	$257,600	$418,600	$322,000
			Total	**$1,701,000**	**$2,763,000**	**$2,125,000**

Note: Refer to Exhibit 1 in Appendix A for details of this project.

Table 2 – Cutlass Drive Culvert

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$16,440	$13,200	$21,500	$16,500
Maintenance of Traffic (5%)	LS	1	$8,220	$6,700	$10,800	$8,300
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Cutlass Drive Box Culverts						
Removal of Existing Pavement	SY	150	$25	$3,100	$5,000	$3,800
Removal of Existing Curb	LF	150	$25	$3,100	$5,000	$3,800
Removal of Existing Culvert	LS	1	$15,000	$12,000	$19,500	$15,000
4-Foot-x-6-Foot Double Box Culvert with Headwalls	LF	100	$1,100	$88,000	$143,000	$110,000
Curb Replacement	LF	150	$15	$2,000	$3,200	$2,400
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	150	$16	$2,000	$3,300	$2,500
Superpave Asphalt Concrete (Traffic C)	TN	17	$90	$1,200	$2,000	$1,500
Temporary Flow Bypass	LS	1	$20,000	$16,000	$26,000	$20,000
Sodding	SY	150	$3	$400	$600	$400
Dewatering	LS	1	$25,000	$20,000	$32,500	$25,000
Overall Items						
Contingency Amount (40%)	LS	1	$75,680	$60,600	$98,500	$75,700
Design/Permitting/Data Collection (25%)	LS	1	$53,550	$42,900	$69,700	$53,600
			Total	**$276,000**	**$448,000**	**$344,000**

Note: Refer to Exhibit 2 in Appendix A for details of this project.

Table 3 – Aquatic Channel Improvements

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$552,510	$442,100	$718,400	$552,600
Maintenance of Traffic (5%)	LS	1	$276,255	$221,100	$359,200	$276,300
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Channel Improvements						
Channel Excavation	CY	3,600	$6	$17,300	$28,100	$21,600
Vertical Channel Wall Armoring (Sheet Piling)	LF	1,900	$2,700	$4,104,000	$6,669,000	$5,130,000
Channel Bottom Armoring	SY	3,100	$96	$238,100	$386,900	$297,600
Existing Pipe Connections to Channel Armoring	EA	2	$9,200	$14,800	$24,000	$18,400
Sodding	SY	1,000	$3	$2,000	$3,300	$2,500
Temporary Flow Bypass	LS	1	$50,000	$40,000	$65,000	$50,000
Overall Items						
Contingency Amount (40%)	LS	1	$2,541,600	$2,033,300	$3,304,100	$2,541,600
Design/Permitting/ Data Collection (25%)	LS	1	$1,588,500	$1,270,800	$2,065,100	$1,588,500
			Total	**$8,388,000**	**$13,630,000**	**$10,485,000**

Note: Refer to Exhibit 2 in Appendix A for details of this project.

Table 4 – Pond Expansion

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$19,160	$15,400	$25,000	$19,200
Maintenance of Traffic (5%)	LS	1	$9,580	$7,700	$12,500	$9,600
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Pond Expansion Construction						
Clearing and Grubbing	AC	0.5	$10,000	$4,000	$6,500	$5,000
Site Demolition	LS	1	$35,000	$28,000	$45,500	$35,000
Regular Excavation	CY	18,000	$5	$71,300	$115,900	$89,100
Dewatering	LS	1	$50,000	$40,000	$65,000	$50,000
Pond Sod	SY	3,000	$3	$6,000	$9,800	$7,500
Overall Items						
Contingency Amount (40%)	LS	1	$88,160	$70,600	$114,700	$88,200
Design/Permitting/Data Collection (25%)	LS	1	$55,100	$44,100	$71,700	$55,100
			Total	**$292,000**	**$474,000**	**$364,000**

Note: Refer to Exhibit 2 in Appendix A for details of this project.

Table 5 – Stanley Road

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$25,840	$20,800	$33,700	$25,900
Maintenance of Traffic (5%)	LS	1	$12,920	$10,400	$16,900	$13,000
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Culvert Replacement						
Removal of Existing Pavement/Sidewalk	SY	175	$25	$3,600	$5,800	$4,400
Removal of Existing Storm Pipe	LF	550	$75	$33,100	$53,700	$41,300
Clearing and Grubbing	AC	0.5	$15,000	$6,000	$9,800	$7,500
Manhole, Type-P < 10 Feet	EA	1	$4,500	$3,600	$5,900	$4,500
Pipe Culvert, 36-Inch RCP	LF	1,000	$150	$120,000	$195,000	$150,000
Mitered End Section, 36-Inch	EA	1	$3,500	$2,800	$4,600	$3,500
Type C DBI, J-Bot, < 10 Feet	EA	4	$6,000	$19,200	$31,200	$24,000
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	150	$16	$2,000	$3,300	$2,500
Superpave Asphalt Concrete (Traffic C)	TN	17	$90	$1,200	$2,000	$1,500
6-Inch Concrete Driveway	SY	25	$57	$1,200	$2,000	$1,500
Temporary Flow Bypass	LS	1	$10,000	$8,000	$13,000	$10,000
Sodding	SY	1,050	$3	$2,200	$3,600	$2,700
Overall Items						
Contingency Amount (40%)	LS	1	$118,920	$95,200	$154,700	$119,000
Design/Permitting/Data Collection (25%)	LS	1	$74,325	$59,600	$96,800	$74,400
			Total	**$392,800**	**$638,300**	**$491,000**

Note: Refer to Exhibit 3 in Appendix A for details of this project.

Table 6 West Plaza

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$3,280	$2,700	$4,300	$3,300
Maintenance of Traffic (10%)	LS	1	$3,280	$2,700	$4,300	$3,300
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Backflow Preventer Construction						
Removal of Existing Pavement	SY	30	$25	$700	$1,100	$800
Removal of Existing Storm Pipe	LF	25	$25	$600	$1,000	$700
Manhole, Type-P < 10 Feet	EA	1	$4,500	$3,600	$5,900	$4,500
Pipe Culvert, 18-Inch RCP	LF	25	$70	$1,500	$2,400	$1,800
Headwall	EA	2	$2,000	$3,200	$5,200	$4,000
Tideflex Inline Backflow Valve	EA	1	$10,000	$8,000	$13,000	$10,000
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	30	$16	$400	$700	$500
Superpave Asphalt Concrete (Traffic C)	TN	3	$90	$300	$400	$300
Temporary Flow Bypass	LS	1	$5,000	$4,000	$6,500	$5,000
Sodding	SY	50	$3	$200	$300	$200
Overall Items						
Contingency Amount (40%)	LS	1	$15,760	$12,700	$20,600	$15,800
Design/Permitting/Data Collection (25%)	LS	1	$9,850	$8,000	$12,900	$9,900
			Total	**$52,800**	**$85,800**	**$66,000**

Note: Refer to Figures in Section 6.3.3 for details of this project

Table 7 – Mary Street

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$16,300	$20,000	$30,000	$17,000
Maintenance of Traffic (5%)	LS	1	$8,150	$6,600	$10,700	$8,200
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Mary Street Drainage Improvement Construction						
Removal of Existing Culverts	EA	12	$1,000	$10,000	$16,000	$12,000
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	50	$16	$1,000	$2,000	$1,000
Superpave Asphalt Concrete (Traffic C)	TN	39	$90	$4,000	$6,000	$4,000
Remove Existing Asphalt and Concrete	SY	700	$25	$15,000	$24,000	$18,000
Swale – Excavation	CY	300	$5	$2,000	$3,000	$2,000
Concrete Sidewalk and Driveway 4-Inch	SY	400	$50	$16,000	$26,000	$20,000
12-Inch RCP	LF	408	$80	$27,000	$43,000	$33,000
12-Inch Mitered End Section	EA	32	$2,000	$52,000	$84,000	$64,000
Sodding	SY	1,500	$3	$4,000	$6,000	$4,000
Overall Items						
Contingency Amount (40%)	LS	1	$75,280	$70,000	$100,000	$76,000
Design (25%)	LS	1	$66,050	$60,000	$90,000	$67,000
			Total	**$280,000**	**$450,000**	**$340,000**

Note: Refer to Exhibit 4 in Appendix A for details of this project.

Table 8 – Seminole Road South

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway/Civil						
Mobilization (10%)	LS	1	$61,100	$48,900	$79,500	$61,100
Maintenance of Traffic (5%)	LS	1	$30,550	$24,500	$39,800	$30,600
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Culvert Replacement						
Clearing and Grubbing	AC	1.1	$10,000	$8,800	$14,300	$11,000
Removal of Existing Pavement/Sidewalk	SY	4,550	$25	$91,100	$148,000	$113,800
Removal of Existing Storm Pipe	LF	1,350	$25	$27,100	$44,000	$33,800
Pipe Culvert, 36-Inch RCP	LF	1,350	$126	$136,100	$221,200	$170,100
Mitered End Section, 36-Inch	EA	1	$3,500	$2,800	$4,600	$3,500
Type E DBI, J-Bot, > 10 Feet	EA	10	$8,800	$70,400	$114,400	$88,000
6-Inch Concrete Driveway	SY	950	$67	$51,000	$82,900	$63,700
4-Inch Concrete Sidewalk	SY	900	$50	$36,000	$58,500	$45,000
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	2,700	$16	$34,800	$56,600	$43,500
Superpave Asphalt Concrete (Traffic C)	TN	297	$90	$21,500	$34,900	$26,800
Sodding	SY	2,700	$2.50	$5,500	$8,900	$6,800
Overall Items						
Contingency Amount (40%)	LS	1	$281,080	$224,900	$365,500	$281,100
Design/Permitting/Data Collection (25%)	LS	1	$175,675	$140,600	$228,500	$175,700
			Total	**$928,000**	**$1,508,000**	**$1,160,000**

Note: Refer to Exhibit 5 in Appendix A for details of this project.

Table 9 – Johansen Park

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$18,220	$14,700	$23,800	$18,300
Maintenance of Traffic (5%)	LS	1	$9,110	$7,400	$12,000	$9,200
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
Johansen Drainage Improvements						
Clearing and Grubbing	AC	0.2	$10,000	$1,300	$2,100	$1,600
Removal of Existing Pavement/Sidewalk	SY	139	$25	$2,800	$4,600	$3,500
Removal of Existing Storm Pipe	LF	570	$25	$11,500	$18,600	$14,300
Pipe Culvert, 48-Inch RCP	LF	570	$160	$73,000	$118,600	$91,200
Mitered End Section, 36-Inch	EA	1	$7,000	$5,600	$9,100	$7,000
Curb Inlet, > 10 Feet, J-Bottom	CY	47	$1,160	$43,500	$70,600	$54,300
Class 1 Concrete, Headwalls and Concrete Flumes	SY	139	$16	$1,900	$3,000	$2,300
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	TN	15	$90	$1,200	$1,900	$1,400
Superpave Asphalt Concrete (Traffic C)	SY	630	$2.50	$1,300	$2,100	$1,600
Sodding						
Overall Items	LS	1	$83,880	$67,200	$109,100	$83,900
Contingency Amount (40%)	LS	1	$52,425	$42,000	$68,300	$52,500
Design/Permitting/Data Collection (25%)	LS	1	$18,220	$14,700	$23,800	$18,300
			Total	**$277,600**	**$451,100**	**$347,000**

Note: Refer to Exhibit 6 in Appendix A for details of this project.

Table 10 – 9th, 10th, 11, and 12th Street Phase 1

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway/Civil						
Mobilization (10%)	LS	1	$44,950	$36,000	$58,500	$45,000
Maintenance of Traffic (5%)	LS	1	$22,475	$18,000	$29,300	$22,500
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
9th, 10th, 11th, and 12th Street Phase 1 Improvements						
Clearing and Grubbing	AC	0.3	$10,000	$2,700	$4,300	$3,300
Removal of Existing Pavement/Sidewalk	SY	1,200	$25	$24,000	$39,000	$30,000
Removal of Existing Storm Pipe	LF	1,445	$25	$29,000	$47,100	$36,200
Pipe Culvert, 48-Inch RCP	LF	1,445	$160	$185,000	$300,600	$231,200
Manhole, > 10 Feet, J-Bottom	EA	6	$10,000	$48,000	$78,000	$60,000
Curb Inlet, J-Bottom > 10 Feet	EA	3	$7,000	$16,800	$27,300	$21,000
Type C DBI, J-Bottom, > 10 Feet	EA	2	$8,800	$14,100	$22,900	$17,600
Class 1 Concrete for Double 48-Inch Headwall	CY	10	$1,160	$9,700	$15,800	$12,100
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	1,200	$16	$15,600	$25,300	$19,400
Superpave Asphalt Concrete (Traffic C)	TN	132	$90	$9,600	$15,500	$11,900
Sodding	SY	700	$2.50	$1,500	$2,400	$1,800
Overall Items						
Contingency Amount (40%)	LS	1	$206,800	$165,500	$268,900	$206,800
Design/Permitting/Data Collection (25%)	LS	1	$129,250	$103,500	$168,100	$129,300
			Total	**$683,000**	**$1,110,000**	**$854,000**

Note: Refer to Exhibit 7 in Appendix A for details of this project.

Table 11 – 9th, 10th, 11th, and 12th Street Phase 2

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway/Civil						
Mobilization (10%)	LS	1	$47,120	$37,800	$61,400	$47,200
Maintenance of Traffic (5%)	LS	1	$23,560	$18,900	$30,700	$23,600
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$5,000	$4,000	$6,500	$5,000
9th, 10th, 11th, and 12th Street Phase 2 Improvements						
Clearing and Grubbing	AC	0.3	$10,000	$2,500	$4,100	$3,100
Removal of Existing Pavement/Sidewalk	SY	1,483	$25	$29,700	$48,300	$37,100
Removal of Existing Storm Pipe	LF	1,338	$25	$26,800	$43,600	$33,500
Pipe Culvert, 48-Inch RCP	LF	1,338	$160	$171,300	$278,400	$214,100
Manhole, > 10 Feet, J-Bottom	EA	5	$10,000	$40,000	$65,000	$50,000
Curb Inlet, J-Bottom > 10 Feet	EA	5	$7,000	$28,000	$45,500	$35,000
Type C DBI, J-Bottom, > 10 Feet	EA	5	$8,800	$35,200	$57,200	$44,000
6-Inch Concrete Driveway	SY	200	$67	$10,800	$17,500	$13,400
4-Inch Concrete Sidewalk	SY	33	$50	$1,400	$2,300	$1,700
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	1,250	$16	$16,200	$26,300	$20,200
Superpave Asphalt Concrete (Traffic C)	TN	138	$90	$10,000	$16,200	$12,400
Sodding	SY	667	$2.50	$1,400	$2,300	$1,700
Overall Items						
Contingency Amount (40%)	LS	1	$216,800	$173,500	$281,900	$216,800
Design/Permitting/Data Collection (25%)	LS	1	$135,500	$108,400	$176,200	$135,500
			Total	**$716,000**	**$1,163,500**	**$895,000**

Note: Refer to Exhibit 8 in Appendix A for details of this project.

Table 12 Salt Air Howell Park

Item Description	Unit	Quantity	Unit Price	Low Estimate	High Estimate	Cost
Roadway / Civil						
Mobilization (10%)	LS	1	$60,050	$48,100	$78,200	$60,100
Maintenance of Traffic (10%)	LS	1	$60,050	$48,100	$78,200	$60,100
Prevention, Control, and Abatement of Erosion and Water Pollution	LS	1	$10,000	$8,000	$13,000	$10,000
Cutlass Drive Box Culverts						
Clearing and Grubbing	AC	1.1	$10,000	$8,800	$14,300	$11,000
Removal of Existing Pavement/Sidewalk	SY	540	$25	$10,800	$17,600	$13,500
Removal of Existing Culvert	LS	3	$15,000	$36,000	$58,500	$45,000
5-Foot-x-9-Foot Double Box Culvert with Headwalls	LF	300	$1,100	$264,000	$429,000	$330,000
Optional Base Group 9 (10-Inch Limerock) (LBR 100)	SY	400	$16	$5,200	$8,500	$6,500
Superpave Asphalt Concrete (Traffic C)	TN	44	$90	$3,200	$5,200	$4,000
Temporary Flow Bypass	LS	3	$20,000	$48,000	$78,000	$60,000
Sodding	SY	170	$3	$400	$700	$500
Dewatering	LS	3	$40,000	$96,000	$156,000	$120,000
Overall Items						
Contingency Amount (40%)	LS	1	$288,280	$230,700	$374,800	$288,300
Design/Permitting/Data Collection (25%)	LS	1	$180,175	$144,200	$234,300	$180,200
			Total	**$952,000**	**$1,547,000**	**$1,190,000**

Note: Refer to Exhibit 9 in Appendix A for details of this project