JonesEdmunds

DUNEDIN
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CITY OF DUNEDIN
STORMWATER MASTER PLAN

Dunedin City Commission | June 2020

CITY OF DUNEDIN

STORMWATER MASTER PLAN

Prepared for:

Dunedin City Commission

542 Main Street

Dunedin, Florida 34698

Prepared by:

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June 2020

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06-16-2020

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LIST OF ACRONYMS

BMP	Best Management Practices
CFS	Cubic feet per second
CHLAC	Chlorophyll corrected for pheophytin
CRA	City of Dunedin Downtown Community Redevelopment Area
CRS	Community Rating System
DCIA	Directly connected impervious area
DEM	Digital Elevation Model
DOSAT	Dissolved Oxygen Saturation
DTI	Digital Topographic Information
ERCP	Elliptical Reinforced Concrete Pipe
ERP	Environmental Resource Permit
FDEM	Florida Division of Emergency Management
FDEP	Florida Department of Environmental Protection
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Maps
FIS	Flood Insurance Study
FLUCCS	Florida Land Use, Cover and Forms Classification System
GIS	Geographic Information System
GWIS	Geographic Watershed Information System
ICPR4	Interconnected Ponds and Routing Version 4
IWR	Impaired Waters Rule
LiDAR	Light Detection and Ranging
LOS	Level of Service
MHW	Mean High Water
NAVD	North American Vertical Datum
NFIP	National Flood Insurance Program
NGS	National Geodetic Survey
NOAA	National Oceanic and Atmospheric Administration
NRSC	Natural Resources Conservation Service (NRCS)
RCP	Reinforced Concrete Pipe
RTK/GPS	Real-time kinetic/global positioning system
SALIN	Salinity
SCS	Soil Conservation Service
SHW	Seasonal High Water
SWFWMD	Southwest Florida Water Management District
TN	Total Nitrogen
TP	Total Phosphorus
TSDN	Technical Support Data Notebook
USDA	United States Department of Agriculture
USGS	United States Geological Survey
WBID	Water Body Identifications
WMP	Watershed Management Plan

1 INTRODUCTION

This stormwater master plan is an update of the City of Dunedin Master Drainage Plan completed in 2003 to address drainage and water quality concerns in the City. The 2003 report included recommendations for projects to improve the drainage network throughout the City, and the City has constructed most of the projects from the 2003 plan. In addition to the project constructed, since 2003 the City has experienced growth and redevelopment requiring an update to the stormwater master plan. In 2016 the City entered into an agreement with Pinellas County to update the Curlew Creek Watershed model which covers approximately 35% of the City. Jones Edmunds was the consultant chosen to update the Curlew Creek Watershed Model. In 2017 the Dunedin City Commission initiated the Dunedin Stormwater Master Plan with Jones Edmunds to update the existing 2003 stormwater model and incorporate the updated Curlew Creek model data. This Plan involved developing digital topographic information (DTI), a watershed inventory, and a watershed model based on the DTI and inventory. The watershed model was used to develop floodplain information as well as flooding level-of-service (LOS) evaluations. Best management practices (BMPs) were developed for some of the City's LOS deficiencies as well as areas of concern as directed by City staff. The Plan also includes environmental assessments, a vulnerability assessment, a Community Rating System evaluation, and a stormwater plan for addressing development within the City of Dunedin Downtown Community Redevelopment Area (CRA). Additional details concerning the Plan include:

- **Digital Elevation Model (DEM):** The first step to create a watershed model is to develop or review the existing conditions DTI and DEM. Jones Edmunds developed a DEM of the City. The DEM provided the foundation for the watershed inventory including subbasin boundaries, storage, and conveyance. The DEM was updated to fill topographic voids in the DEM using permitted or as-built drawings. Section 3 describes the DEM.
- **Watershed Evaluation:** The other foundational element in the watershed model development is to conduct an existing conditions watershed evaluation. The watershed evaluation focuses on the feature inventory, including hydrologic (Section 4) and hydraulic (Section 5) features. The watershed evaluation includes field and desktop evaluations. A geographic information system (GIS) geodatabase was created to store information collected in the field and to identify features. The City's geodatabase is based on the Southwest Florida Water Management District's (SWFWMD) Geographic Watershed Information System (GWIS) Version 1.6 geodatabase schema. The geodatabase contains an inventory of hydraulic features such as culverts, drop structures, weirs, and channels. The feature information is based on a combination of data and plans from Environmental Resource Permit (ERP) applications and field data collection where ERP data are not available.

Photograph 1-1 Field Data Collection

- **Watershed Model:** Jones Edmunds prepared a watershed model of the City (Section 6), which simulates the City's natural and man-made drainage infrastructures' response to storms. The City of Dunedin watershed model (watershed model) was separated into 8 separate subwatersheds to capture the major conveyance systems within the City. These subwatersheds include Minnow Creek, Curlew Creek, Jerry Branch, Cedar Creek, Spring Branch, and three Coastal Zones (North, South, and Downtown) all of which drain directly to Clearwater Harbor North. The watershed model also includes portions of Pinellas County and the City of Clearwater that were included in the Curlew Creek Watershed Model that was completed in December of 2019.
- **Calibration:** Calibration provides validity to the model by demonstrating that the model accurately simulates the watershed's response (stages and flows) to historic storms. Jones Edmunds calibrated and verified the watershed model using three gauges located along Curlew Creek. The results of the calibration are discussed in section 7 of this report. The Curlew Creek and Jerry Branch subwatersheds are both represented in the gauge data used to calibrate the watershed model. While none of the other subwatersheds contained flow or stage gauges, the same calibration adjustments were made in the ungauged areas that were made in the gauged portions of the model, lending validity to the ungauged portions of the model as well.
- **Floodplains:** In order to quantify the flooding risk from rainfall in the City. The watershed model was used to develop flood-risk polygons (Section 8) for rain-induced flooding based on the 100-year, 24-hour storm and the 500-year, 24-hour storm.
- **LOS:** LOS analysis provides a "measuring stick" of the drainage network and is used as a guide for the alternatives analysis and conceptual project development. The flood protection LOS (Section 9) were assigned by Jones Edmunds to each of the City's subbasins and roadway segments based on the model results and LOS Criteria that was developed during the Curlew Creek WMP to ensure consistency across the watershed model .
- **BMPs:** The watershed model was used to develop solutions for some of the City's flooding problem areas and needed water quality improvements. Additionally, the BMP analysis (Section 10) provides a description of low impact development (LID) practices applicable to City-owned facilities and backflow-prevention devices to reduce "sunny-day flooding."
- **Environmental Assessments:** An analysis of the City's ambient water-quality data provides a picture of the water quality trends within the City and a pollutant-loading model identifies "Hot Spots" where pollutant loads are generated. This analysis is presented in Section 11, and indicates that, water quality in the City's creeks and streams is generally improving.
- **Downtown CRA Stormwater Plan:** Section 12 presents information for a potential regional stormwater approach for the Downtown CRA that will aid future improvements and redevelopment of areas inside the CRA.
- **Vulnerability Assessment:** The City of Dunedin understands the challenges and dangers that sea level rise represents to the residents and infrastructure of the City. The vulnerability assessment presented in Section 13 estimates the increases in future flood risks due to projected sea-level rise and recommendations to adapt to the increasing water levels in the Gulf of Mexico.
- **Community Rating System (CRS):** Section 14 describes how this study can be used to support and improve the City's current class 5 CRS ranking.

The Technical Support Data Notebook (TSDN) included with the project deliverables provides additional documentation for the City's Stormwater Master Plan.

Photograph 1-2 Pinellas Trail Over Curlew Creek

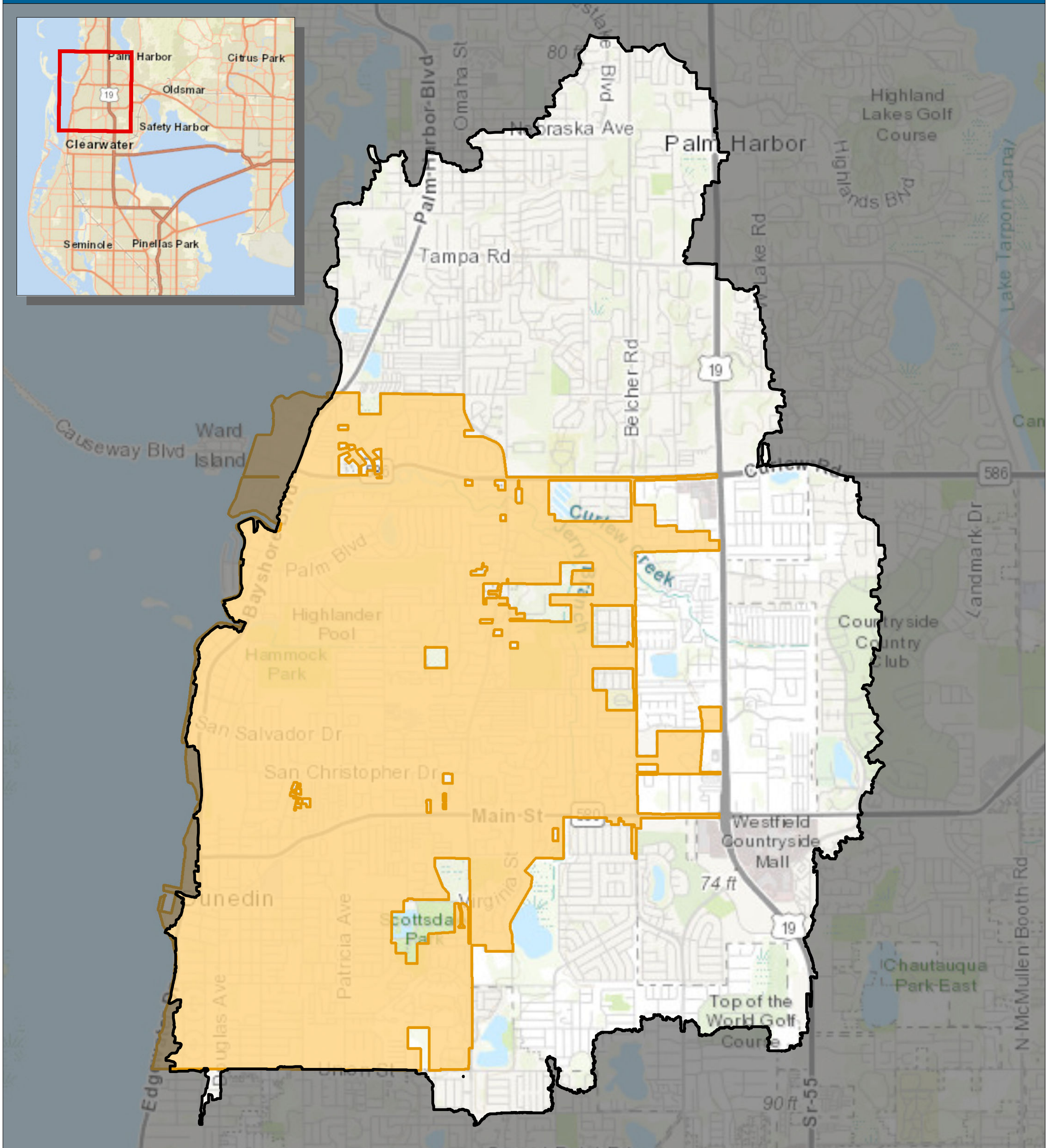
The City of Dunedin's watershed model covers an area of 19.0 square miles (m^2) – 8.1 m^2 within the City and 10.9 m^2 outside the City. The areas outside the City, developed under the Curlew Creek/Smith Bayou WMP, were maintained within the City's model to accurately reflect the influence that these tributaries have on the City's stormwater system. Both models – the City's watershed model and the Curlew Creek/Smith Bayou model (which includes Minnow Creek and Jerry Branch) – were developed by Jones Edmunds using identical procedures, resolution, and topographic information; therefore, the models are consistent and seamless.

Figure 1-1 shows a location map and the model domain.

Figure 1-1

Project Location

City of Dunedin Stormwater Master Plan



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Watershed Model

City of Dunedin Boundary

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2 WATERSHEDS AND TRIBUTARIES CHARACTERIZATION

Drainage for the City of Dunedin is provided by a combination of natural waterways, engineered open-channels, and closed conduits such as pipes. Elevations within the watershed range from approximately 108 feet (North American Vertical Datum [NAVD] 88) along the east boundary of the watershed to below sea level along the coast. A ridge runs most of the City's length along County Road 1 (CR 1)/Keene Road, and drainage is generally east to west with south to north flow in some locations. The City of Dunedin contains eight distinct watersheds – Curlew Creek and its tributary Jerry Branch, Cedar Creek, Spring Branch, Minnow Creek, and three coastal zones that do not have significant or named water features. All of the City's watersheds drain ultimately to St. Joseph Sound, and most of the watersheds outfall within the City limits (except for Spring Branch, which drains to Stevenson Creek in the City of Clearwater, and Minnow Creek, which enters the Sound near Ozona). Figure 2-1 illustrates each of the subwatersheds. Figure 2-2 shows the City's major conveyance-ways, and a description of each follows:

- **Minnow Creek** has its south headwaters originating near Curlew Road within the City. The runoff generated in this watershed is collected and conveyed to a large natural area south of the Dunedin RV Resort. The flow continues north to Shore Lake before outfalling under US Highway Alternate (US Alt) 19, Pinellas Trail, and Orange Street to Smith Bayou. Communities in this subwatershed include Laurel Oak, County Woods, and Waterford Crossing.
- **Cedar Creek** Watershed contains two major open-channel systems that drain to Cedar Creek – one originating north of San Salvador Drive between St. Catharine Drive West and St. Mary Drive, and the second originating approximately 1,300 feet east of Pinehurst Road between Royal Oak Drive and Valley Drive. Both systems appear to be entirely manmade since no evidence of natural streams is present on the earliest known aerial photographs of the area from 1928. These two main drainage features converge inside of Hammock Park, a 90-acre natural park (Photograph 2-1). The combined system conveys flow to the estuarine segment of Cedar Creek then under the Pinellas Trail and Bayshore Boulevard to the Intercoastal Waterway. This watershed includes the neighborhood of Stirling Heights and portions of Dunedin Pines.

Photograph 2-1 Hammock Park in Cedar Creek Sub watershed

Figure 2-1

Subwatersheds

City of Dunedin Stormwater Master Plan

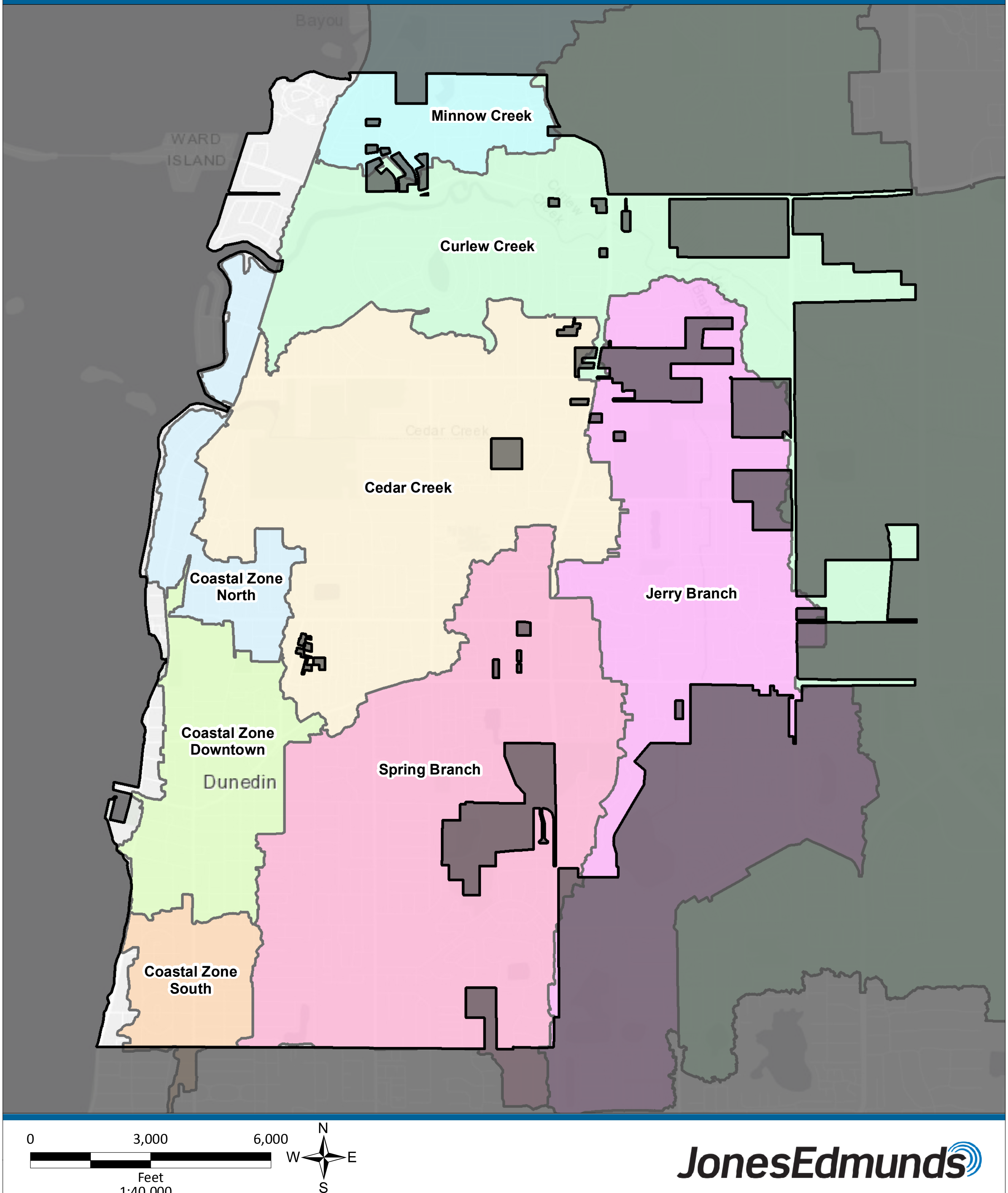

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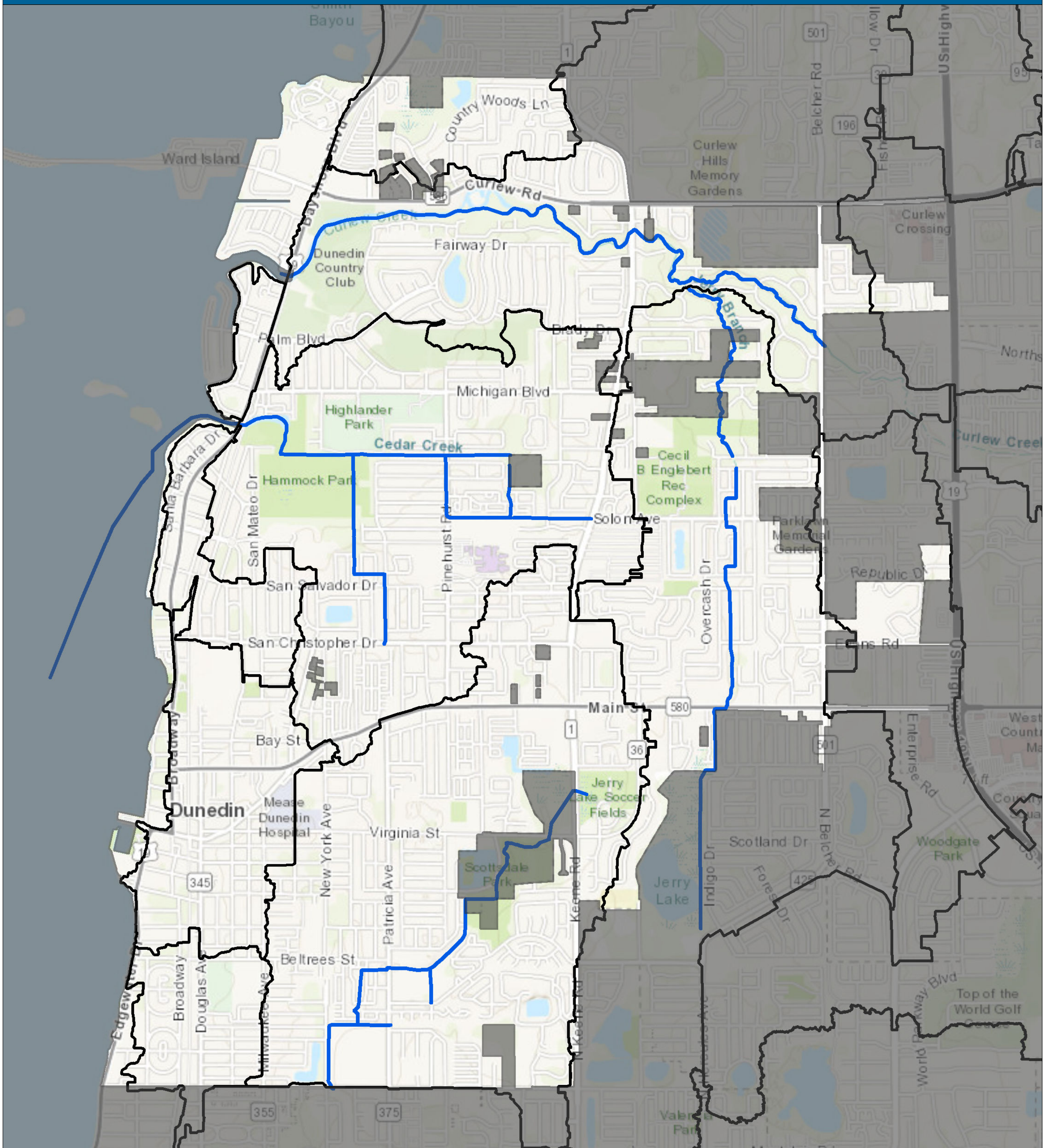
Figure 2-2

Major Conveyance

City of Dunedin Stormwater Master Plan



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Subwatersheds

Major Conveyance

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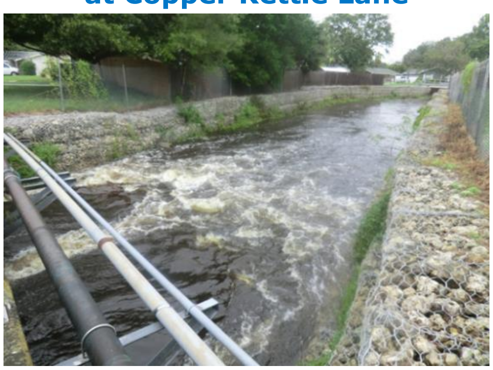
- **Coastal Zone** subwatersheds drain directly to Clearwater Harbor North. Several ridges separate the coastal zones into three units for planning purposes – Coastal Zone North is north of San Christopher Drive, Coastal Zone South begins near Belltress Street, and Coastal Zone Downtown is in the middle.
- **Curlew Creek** enters the City at Belcher Road, having originated in Clearwater, and moved through unincorporated Pinellas County. The portion of Curlew Creek contained inside the City is the most natural portion of this highly altered creek system. This portion contains naturally vegetated banks and conveys flow through mostly residential portions of the city before ultimately discharging to St. Joseph Sound approximately 0.5 mile south of Dunedin Causeway. The Curlew Creek subwatershed contains the Dunedin Country Club and Fairway Estates.

Photograph 2-2 Curlew Creek at County Road 1

Photograph 2-3 Jerry Branch at Copper Kettle Lane



- **Jerry Branch** (Photograph 2-3), a tributary to Curlew Creek, originates south of Jerry Lake conveying flow from the City of Clearwater and Pinellas County into Dunedin. Although the upper reaches along Jerry Lake contain natural channel banks, the section of channel from Copper Kettle Lane to Laurelwood Lane has been altered significantly from its natural state by the addition of gabions along its banks. This subwatershed contains the Englebert Complex a facility used by the Toronto Blue Jays and their minor league affiliates and the Piper's Glen Subdivision.

Photograph 2-4 Spring Branch South of Knollwood Drive

- **Spring Branch** originates north of Main Street in the Dunedin Pines and Golden Acres area of the City. Highly altered along its pathway, the branch alternates between natural stream segments, freshwater bodies (natural and manmade), engineered channels, and closed conduits before exiting the City at Union Street (Photograph 2-4). Lake Haven, Virginia Crossing, Somerset of Dunedin, and Dunedin Mobile Manor are some of the other neighborhoods in this subwatershed.

The City of Dunedin is mostly developed with over 70 percent of the City composed of residential area – high-density residential alone accounts for about half of the City. The other two most represented land uses are commercial and institutional areas, which account for 6 percent and 5 percent, respectively.

Table 2-1 summarizes land use within the City, and Section 4 provides more detailed information concerning each subwatershed's land use as well as soil information.

Table 2-1 Dunedin Land Use Summary

*FLUCCS Code	Description	Acres	% Area
1100	Residential Low Density	43	0.8
1200	Residential Med Density	1,077.7	20.7
1300	Residential High Density	2,589.9	49.7
1400	Commercial and Services	332.4	6.4
1500	Industrial	26.1	0.5
1700	Institutional	275.5	5.3
1800	Recreational	163.9	3.2
1820	Golf Courses	150.3	2.9
1900	Open Land	29.9	0.6
4000s	Woods	94.2	1.8
5000s	Streams, Lakes, Reservoirs, Bays, and Estuaries	96	1.8
6000s	Wetlands and Marshes	154.3	3.0
8100	Transportation	134.9	2.6
8300	Utilities	39.4	0.8
	TOTAL	**5207.6**	**100**

*FLUCCS = Florida Land Use, Cover and Forms Classification System.

3 TOPOGRAPHIC INFORMATION

The watershed evaluation used a Light Detection and Ranging (LiDAR)-based DEM. The DEM provided continuous elevation data throughout the watershed. The following subsections describe the DEM and the topographic voids that were filled. Figure 3-1 shows the DEM for the City.

3.1 DIGITAL TERRAIN MODEL(DTM)

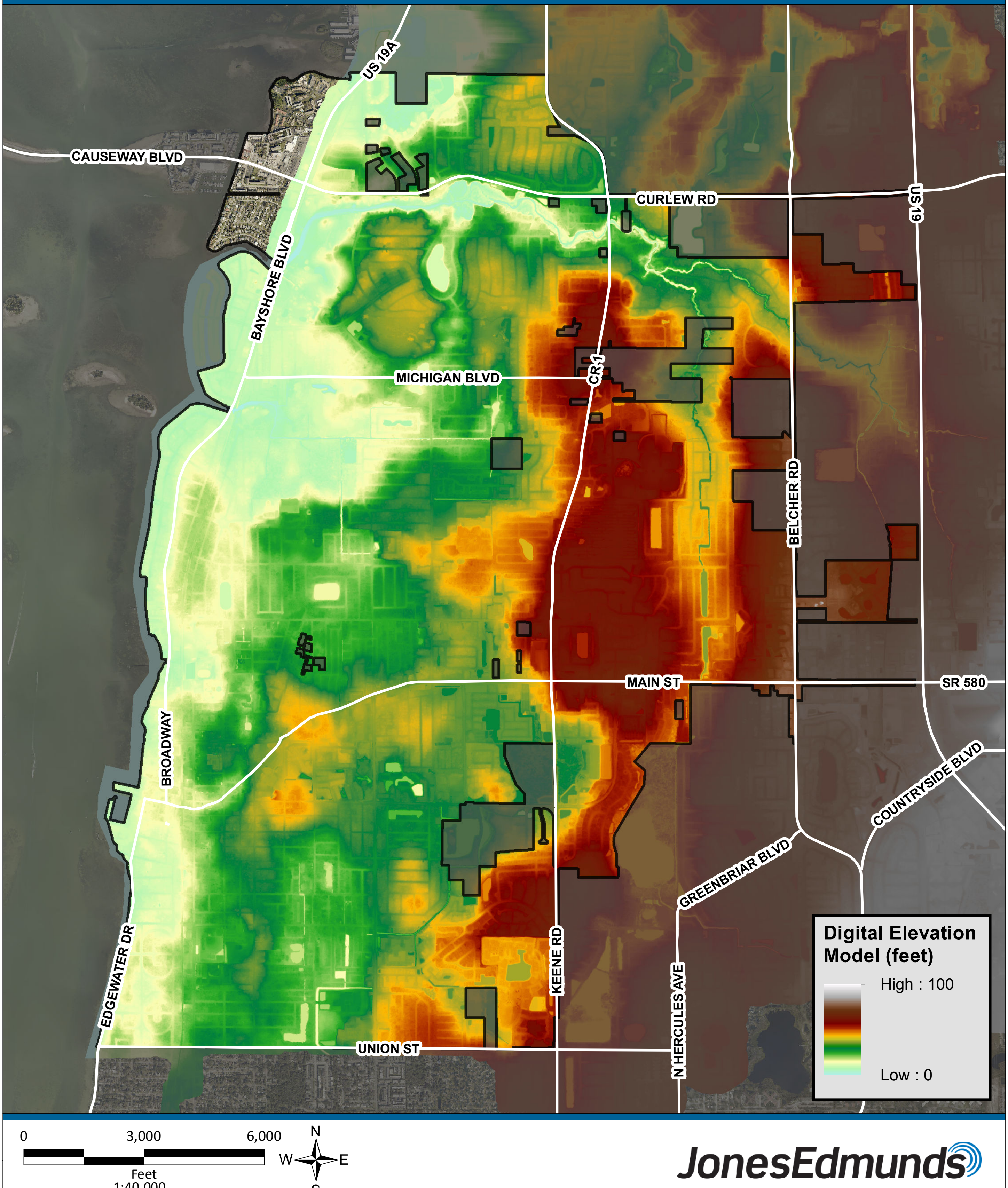
Existing digital topographic data covering the City is based on data collected by the Florida Division of Emergency Management (FDEM) in 2007 as part of the Statewide Coastal LiDAR project. The City was included as part of a larger project that covered coastal areas of Pasco County and all of Pinellas County. The LiDAR data were delivered according to FDEM specifications, which were based on the SWFWMD LiDAR specifications. In 2010, SWFWMD contracted with Earth Eye LLC to hydro-enhance the 2007 FDEM LiDAR dataset of Pinellas County. The hydro-enhancement focused on improving the 2007 LiDAR datasets to produce a DTM that is more suitable for watershed modeling. SWFWMD produced a County-wide DEM using the hydro-enhanced data. Jones Edmunds clipped the County-wide DEM to the buffered watershed boundary to produce a raster DEM that was used for the watershed model analysis.

The vertical reference for all data is NAVD 88. Some of the data from plan sets used to develop the hydraulic parameters that were originally in elevation data in National Geodetic Vertical Datum (NGVD) 29 were converted to NAVD 88 using a conversion value of -0.85 foot. This value was determined using the average US Geological Survey (USGS) VERTCON conversion value across the City. The geographic coordinate system (GCS) North American 1983 HARN GCS was used for all data developed as part of this plan.

Figure 3-1

Digital Elevation Model

City of Dunedin Stormwater Master Plan



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3.2 Topographic Voids and Areas of New Development

Jones Edmunds identified and filled topographic voids due to development in the watershed model domain. Topographic voids due to development are those areas where the digital topographic information does not accurately describe the existing topography because of new development. These topographic updates (voids filling) were accomplished by digitizing contours and spot elevations from as-built drawings, creating a DTM from the digitized data, and replacing the void areas with the updates. Figure 3-2 shows an example of topographic void and filling (ID 1 in table), and Table 3-1 provides a list of topographic voids that were filled.

Figure 3-2 Topographic Void and Filling

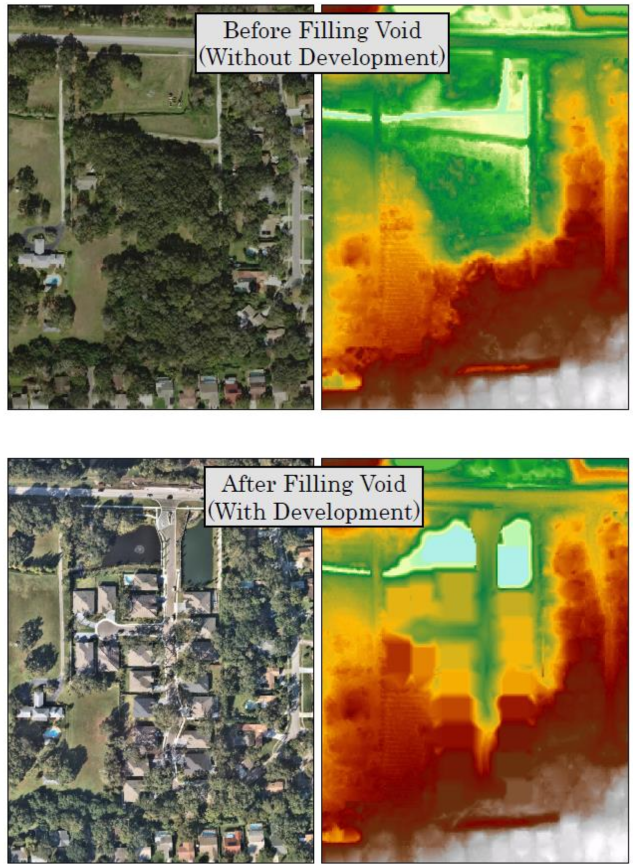


Table 3-1 Areas of Ground Cover Changes

ID	ERP Number	Project Name
1	042652_000	Aberdeen Oaks
2	042462_001	Dunedin Commons
3	035291_001	Dunedin Isles Drainage Improvements
4	0 41696_000	Glenn Moor Subdivision
5	N|A	*Gramercy Court

* As-Built Drawings provided by the City of Dunedin.

4 HYDROLOGIC INVENTORY

The hydrologic inventory comprises datasets that define and characterize the City's subwatersheds and their associated subbasins. The hydrologic inventory, together with hydrologic parameters added during the modeling phase (Section 6), become the hydrologic model, which is the first part of the watershed model. The second part of the model is the hydraulic component. The hydrologic model was developed within the Interconnected Ponds and Routing Version 4 (ICPR4) framework and will simulate infiltration and runoff resulting from rainfall events.

The level of detail in the subbasin delineation within the City was driven primarily by the level of detail in the hydraulic model described in Section 4.2. In general, a subbasin was delineated for each node in the model network. Exceptions include manhole nodes that receive piped inflows but no surface runoff.

4.1 Subbasin Delineation Process

Jones Edmunds developed subbasin (catchment) delineations for the City's watersheds in general accordance with the SWFWMD *Watershed Management Program Guidelines and Specifications* (2011). Subbasins were typically delineated around all major drainage conveyances and significant detention systems.

4.2 Subbasin Characterization

The study area is subdivided into 1055 subbasins ranging from 0.2 to 105.9 acres with an average size of 11.2 acres. For modeling and reporting, subbasins were aggregated into eight subwatersheds based on the main hydrologic feature into which they ultimately drain. Table 4-1 summarizes the subbasin statistics. Figure 4-1 shows the subbasin and subwatershed delineations. The largest subbasin contains Jerry Lake and it's contributing area.

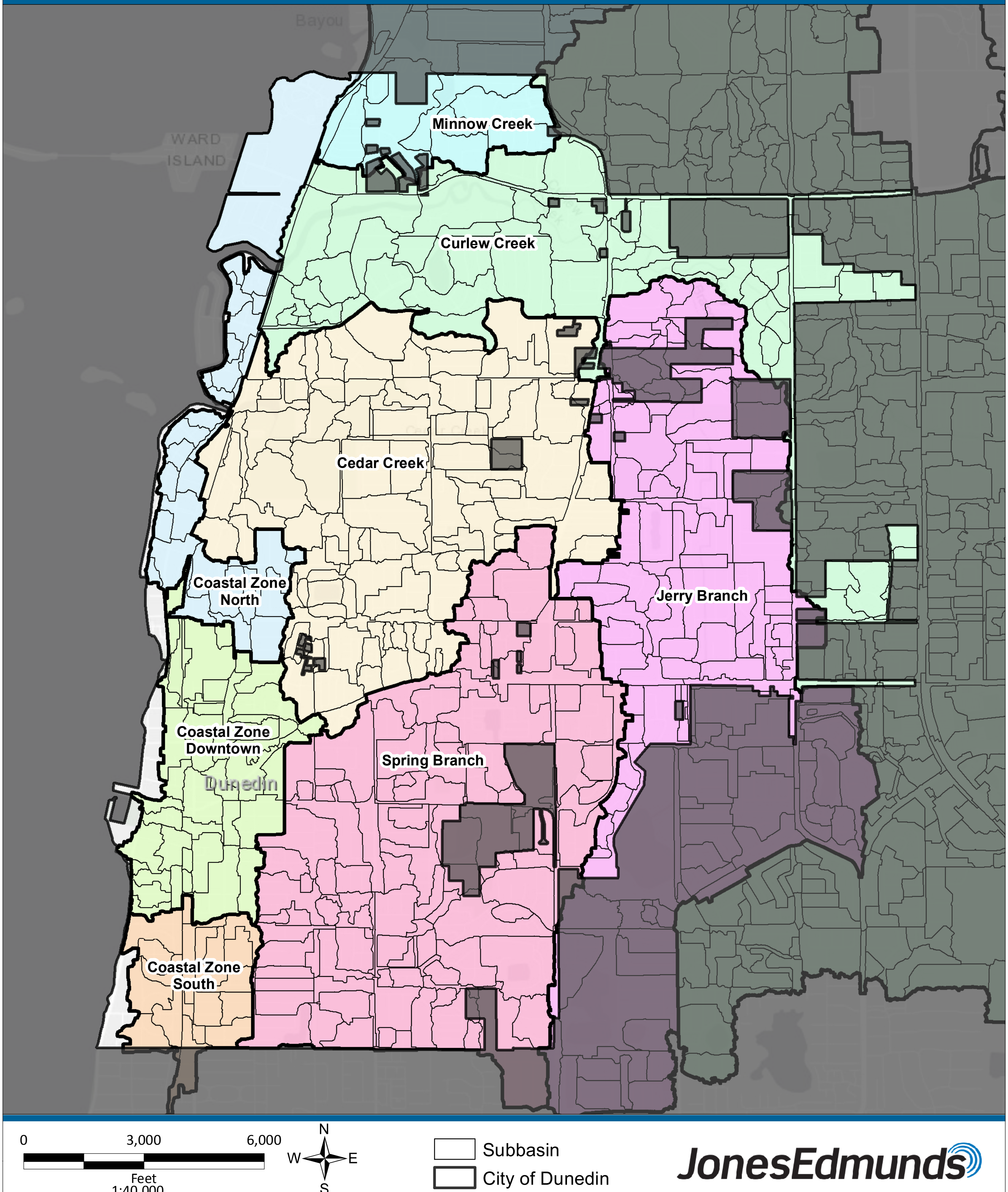
Table 4-1 Summary Statistics of Subbasin Sizes

Count	1,055
Minimum	0.22 acres
Maximum	105.88 acres
Average	11.25 acres
Standard Deviation	10.08 acres
Total Area	12,196 acres

Figure 4-1

Subbasins and Subwatersheds

City of Dunedin Stormwater Master Plan



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4.3 Land Use Characterization

Land use characterization was developed using the 2011 SWFWMD land use (based on FLUCCS). Table 4-2 provides tabulated land use information for each subwatershed. The land use information presented in this section provides general information regarding development intensity, which is the primary variable, along with soils, that determine runoff potential. Land use is often used as a measure of imperviousness; however, an impervious layer was developed for the City's watershed model (Section 6). Figure 4-2 presents the SWFWMD land use distribution within the subwatersheds.

Table 4-2 Summary of Land Use by Subwatershed

Subwatershed	FLUCCS Code	Description	Acres	% Area
Cedar Creek	1100	Residential Low Density	6.7	0.6
	1200	Residential Med Density	367.1	30.2
	1300	Residential High Density	461.3	38.0
	1400	Commercial and Services	40.1	3.3
	1700	Institutional	98.4	8.1
	1800	Recreational	51.4	4.2
	1820	Golf Courses	48.2	4.0
	4000	Woods	45.9	3.8
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	12.7	1.1
	6000	Wetlands and Marshes	65.8	5.4
	8100	Transportation	14.8	1.2
	8300	Utilities	2.5	0.2
		TOTAL	**1,214.8**	
Coastal Zone Downtown	1200	Residential Med Density	28.4	7.0
	1300	Residential High Density	210.9	51.9
	1400	Commercial and Services	74.4	18.3
	1500	Industrial	23.1	5.7
	1700	Institutional	34.8	8.6
	1800	Recreational	7.0	1.7
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	1.7	0.4
	8100	Transportation	12.2	3.0
	8300	Utilities	14.1	3.5
		TOTAL	**406.7**	

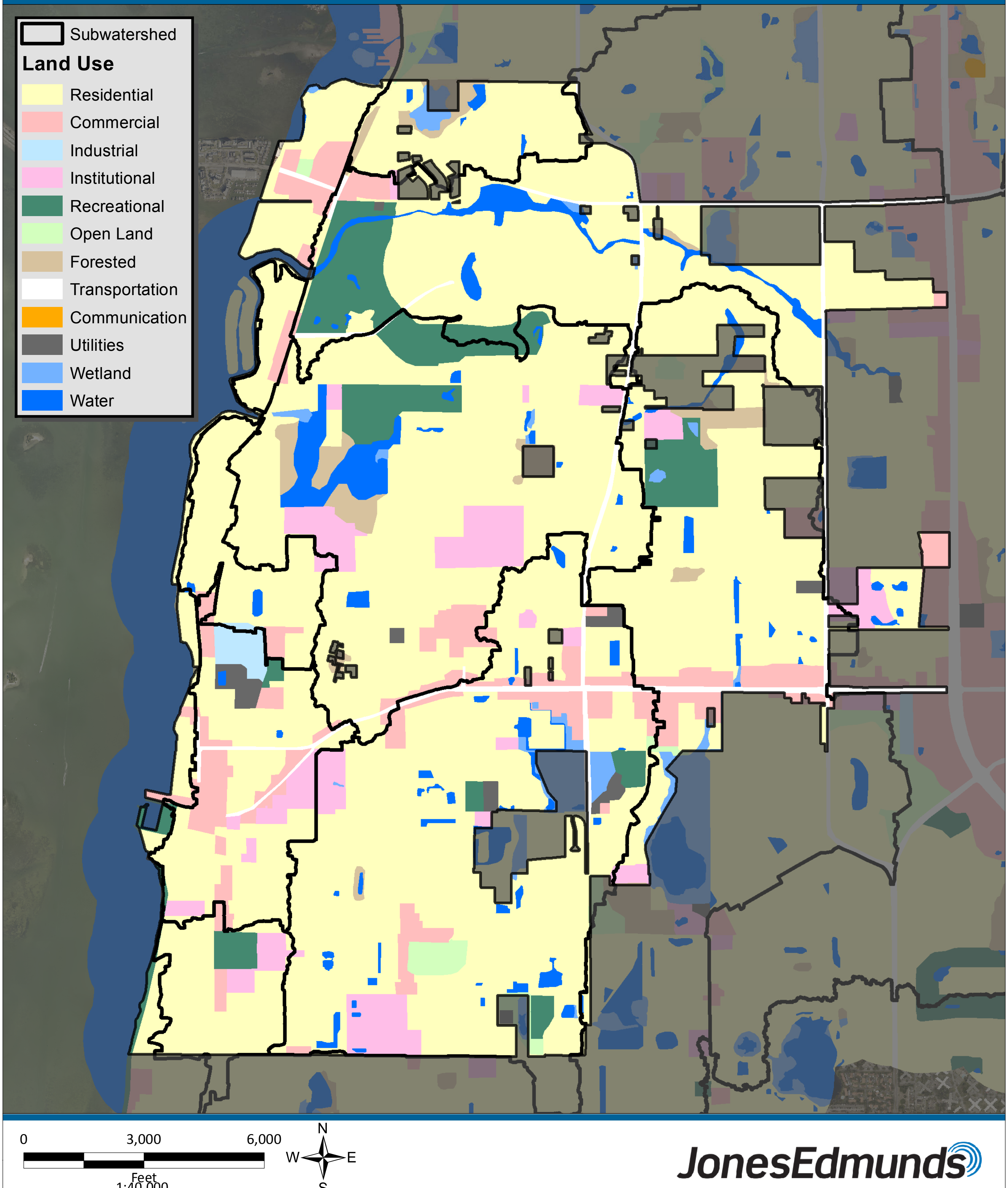
Subwatershed	FLUCCS Code	Description	Acres	% Area
Coastal Zone North	1300	Residential High Density	215.8	87.8
	1400	Commercial and Services	19.0	7.7
	1500	Industrial	3.0	1.2
	1700	Institutional	4.1	1.7
	1800	Recreational	0.4	0.2
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	2.8	1.2
	6000	Wetlands and Marshes	0.8	0.3
		TOTAL	**245.9**	
Coastal Zone South	1300	Residential High Density	136.8	74.0
	1400	Commercial and Services	9.1	4.9
	1700	Institutional	20.0	10.8
	1800	Recreational	19.0	10.3
		TOTAL	**184.8**	
Curlew Creek	1100	Residential Low Density	6.5	0.7
	1200	Residential Med Density	486.2	54.9
	1300	Residential High Density	106.4	12.0
	1400	Commercial and Services	29.5	3.3
	1700	Institutional	19.7	2.2
	1800	Recreational	5.9	0.7
	1820	Golf Courses	102.2	11.5
	1900	Open Land	0.2	0.0
	4000	Woods	3.1	0.4
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	21.4	2.4
	6000	Wetlands and Marshes	41.2	4.7
	8100	Transportation	63.1	7.1
	8300	Utilities	0.9	0.1
		TOTAL	**886.2**	

Subwatershed	FLUCCS Code	Description	Acres	% Area
Jerry Branch	1100	Residential Low Density	24.6	3.2
	1200	Residential Med Density	87.7	11.5
	1300	Residential High Density	437.8	57.3
	1400	Commercial and Services	61.2	8.0
	1700	Institutional	20.4	2.7
	1800	Recreational	52.9	6.9
	1900	Open Land	2.1	0.3
	4000	Woods	25.6	3.4
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	12.0	1.6
	6000	Wetlands and Marshes	17.6	2.3
	8100	Transportation	18.6	2.4
	8300	Utilities	3.9	0.5
		TOTAL	**764.4**	
Minnow Creek	1200	Residential Med Density	108.4	55.0
	1300	Residential High Density	57.8	29.3
	1700	Institutional	0.7	0.4
	1900	Open Land	0.1	0.1
	4000	Woods	16.6	8.4
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	4.3	2.2
	6000	Wetlands and Marshes	9.2	4.7
	8100	Transportation	0.2	0.1
		TOTAL	**197.1**	
Spring Branch	1100	Residential Low Density	5.3	0.4
	1300	Residential High Density	963.2	73.7
	1400	Commercial and Services	99.2	7.6
	1700	Institutional	77.5	5.9
	1800	Recreational	27.3	2.1
	1900	Open Land	27.4	2.1
	4000	Woods	3.0	0.2
	5000	Streams, Lakes, Reservoirs, Bays, and Estuaries	41.1	3.1
	6000	Wetlands and Marshes	19.8	1.5
	8100	Transportation	26.1	2.0
	8300	Utilities	18.0	1.4
		TOTAL	**1,307.7**	

Figure 4-2

Land Use

City of Dunedin Stormwater Master Plan



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4.4 Soil Characterization

Photograph 4-1 Exceptionally Well-Drained Soils Present in Large Portions of the Watershed

Soil information from the Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) dataset are presented in this section. The NRCS hydrologic groupings provide a convenient way to review relative infiltration rates throughout the City; however, the actual infiltration rates, rather than the hydrologic groupings, were used to develop the hydrologic model parameters (Section 6).

Soils within the City range from poorly drained to well-drained, some excessively well-drained, with saturated hydraulic conductivities of up to 78.4 feet per day. In general, soils in the east and northwest parts of the City are more well-drained and soils in the southwest portion of the City are poorly drained.

Table 4-3 provides the area of each hydrologic soil group within each subwatershed, and Figure 4-3 shows the soil classifications within the City of Dunedin.

Table 4-3 Soil Acreage by Subwatershed Within the City

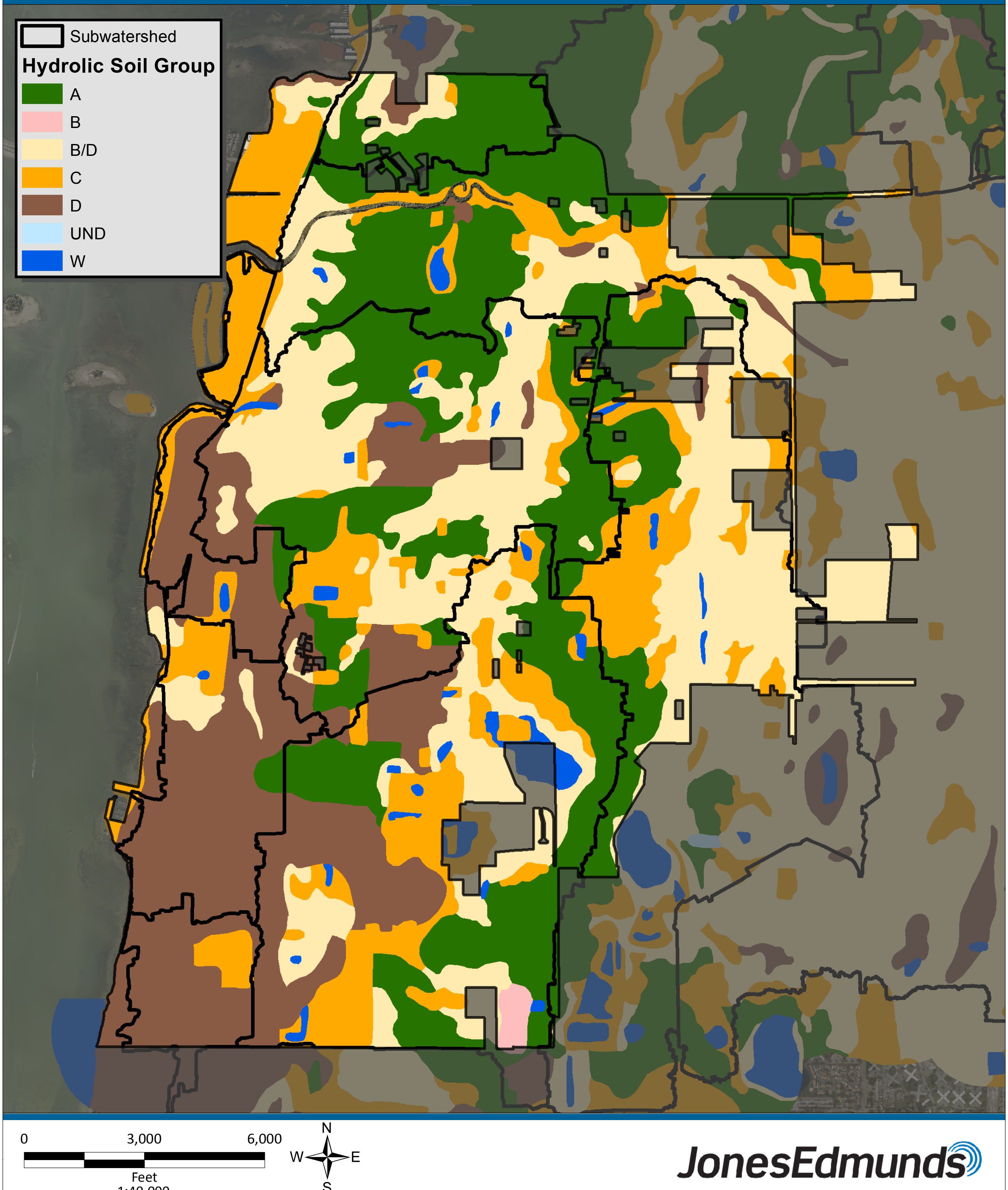
Subwatershed	Hydrologic Soil Group Acreage					
	A	B	B/D	C	D	W
Minnow Creek	146.0	0	40.4	2.3	8.4	0
Cedar Creek	395.3	0	457.8	139.9	207.8	14.0
Coastal Zone Downtown	11.4	0	35.4	37.4	321.5	1.0
Coastal Zone North	12.5	0	0.0	83.8	146.9	2.6
Coastal Zone South	0	0	0	27.8	157.0	0
Curlew Creek	379.7	0	319.5	145.7	17.2	7.9
Jerry Branch	200.9	0	364.8	183.7	6.6	8.4
Spring Branch	389.1	20.0	283.5	296.6	280.6	37.9
Total	1,534.9	20.0	1,501.5	917.4	1,146.0	71.8

Notes: W = Water.

Figure 4--3

Soils

City of Dunedin Stormwater Master Plan



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5 HYDRAULIC FEATURE INVENTORY

Jones Edmunds completed a hydraulic feature inventory beginning with a desktop reconnaissance and continuing through windshield surveys and field data collection. Follow-up visits were made after the survey to confirm feature locations and information. The following subsections describe the hydraulic feature inventory.

5.1 Hydraulic Feature Inventory Development Process

Potential hydraulic features were initially identified using features provided by the City of Dunedin's stormwater asset-management database. Additional stormwater asset data were provided by Pinellas County and the City of Clearwater for areas outside the City. Jones Edmunds reviewed plans from a number of sources including SWFWMD ERPs and Florida Department of Transportation (FDOT) for additional hydraulic feature information.

The *City of Dunedin Master Collection System Plan* completed in 2006 was used as a quality control measure to corroborate pipe sizes, material types, and other characteristics; however, all model data were developed independently from the 2006 study.

Photograph 5-1 Hydraulic Feature

The locations of other potential hydraulic features were found by analyzing the DTM, Google Street View, and aerial imagery. Once all potential hydraulic features were identified, Jones Edmunds' Project Engineer completed a prescreening process in the field. This preliminary field reconnaissance identified more than 700 hydraulic structure points and 31 cross-sections to be inventoried inside the City. The features were then verified by the Project Manager. This process allowed the Project Engineer to identify the locations of assumed hydraulic features and to verify other subsurface drainage before starting field reconnaissance and survey.

Once the prescreening process was completed, field crews took photographs and captured the hydraulic characteristics of all accessible hydraulic features that were previously identified by Jones Edmunds. This field effort followed the survey plan and was conducted in three parts – data collection by Jones Edmunds staff, survey by Degrove Surveyors, and survey by Hyatt Surveyors. The field data collection effort contained over 2,000 points (x, y, and z) and included 547 structures and 23 cross-sections. Appendix A provides a sample hydraulic feature form. Hydraulic feature forms are part of the HyperText Markup Language (HTML) folder and were provided digitally. Hydraulic feature forms were generated using a custom Microsoft Access application. Forms were created for all features inventoried during the field reconnaissance task. Table 5-1 summarizes the hydraulic features surveyed for this project and the field crew that collected the data.

Table 5-1 Hydraulic Features Inventories by Data Source Within the City

Survey	817
ERP/FDOT Plans	677
Asset Inventory	89
Estimated	8
Total	1,591

Figure 5-1 shows these features spatially. All survey data were uploaded into the Hydraulic Element Point Feature class within the GWIS geodatabase.

5.2 Establishment of Elevation Control for Watershed

The Degrove Surveyors' real-time kinetic/global positioning system (RTK/GPS) survey results were obtained by a methodology of RTK/GPS observations, trigonometric leveling, differential leveling, and/or a combination thereof.

RTK/GPS-derived survey results are based on RTK corrections from the FDOT Florida Permanent Reference Network Station ID "FLEM" or "FLIS." RTK Corrections were verified against National Geodetic Survey (NGS) Horizontal and/or Vertical Control Points "GRAY T" and DUNEDIN D and NGS Horizontal Control Points "V733", "872 6819 G", and "872 6819 H."

The Hyatt Surveyors' data points were collected using a combination of survey methodologies including calibrated virtual reference station (VRS) GPS and robotic total station. All elevations were referenced to NAVD 88, and the horizontal positions are referenced to the North American Datum of 1983 (NAD83), State Plane Coordinates, and Florida West Zone (0902).

Jones Edmunds' GPS/RTK observations used NGS monument AL0136 and AL6289 for elevation control.

5.3 Summary of Conveyance Features by Subwatershed and Type

Significant hydraulic conveyance features in the watershed model include channels, culverts, drop-structures, overland weirs, and structural weirs. Table 5-2 summarizes hydraulic inventory counts by structure type for the study area – this table excludes overland weirs.

Table 5-2 Hydraulic Features Inventories by Type

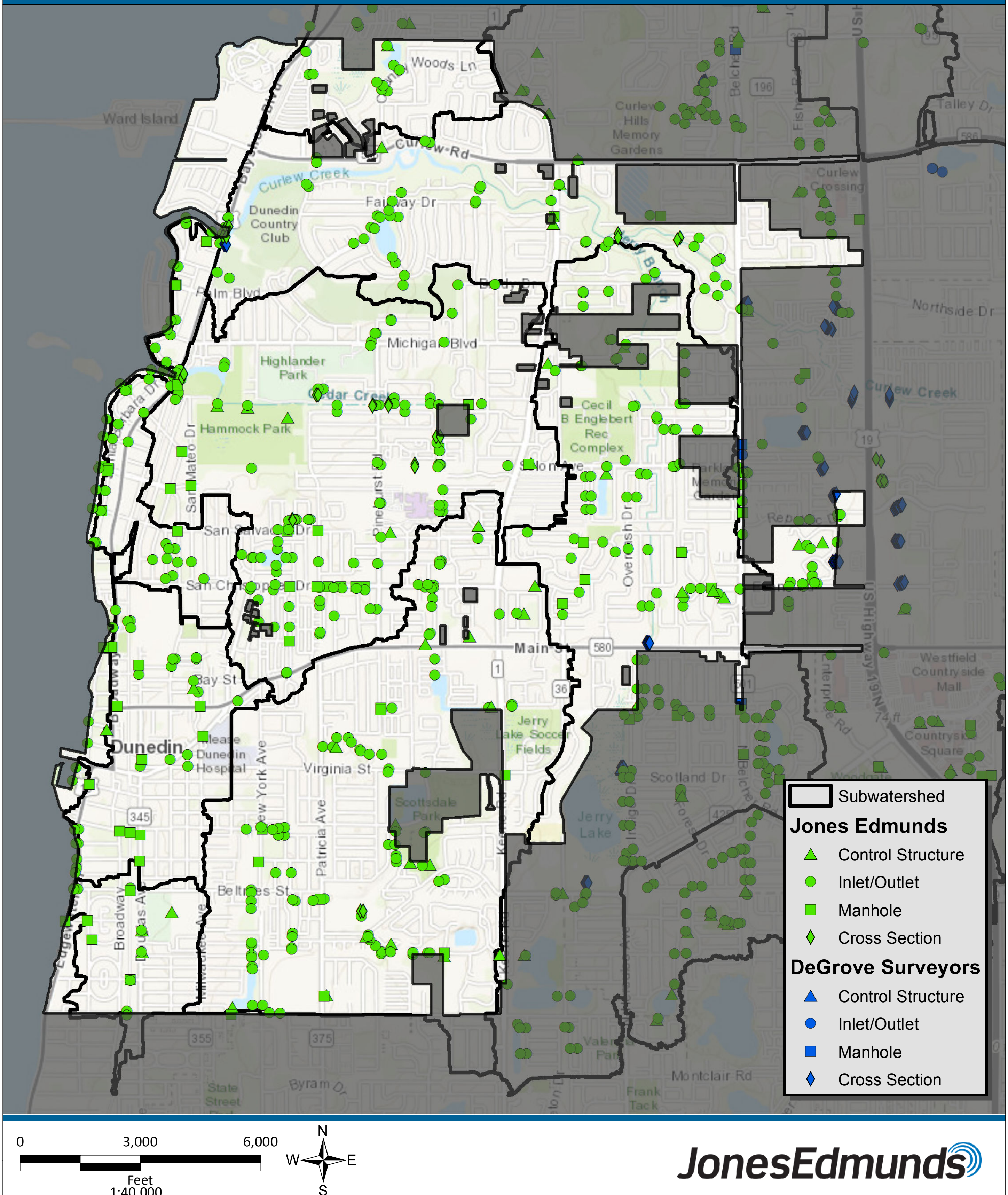
Pipe	854
Channel	350
Weir	113
Drop Structure	292
Pumps	2
Total	1,611

Note: A "drop structure" feature is composed of a control structure and pipe and is counted as two structures.

Figure 5-1

Hydraulic Feature Survey

City of Dunedin Stormwater Master Plan



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6 WATERSHED MODEL DEVELOPMENT

Jones Edmunds created a watershed-wide hydrologic and hydraulic model covering the City and its inflow tributaries using ICPR4. This Section describes modeling methodology and parameter development for the watershed model.

6.1 HYDROLOGIC AND HYDRAULIC MODEL METHODOLOGY

The hydrologic component of the model simulates runoff flows, and the hydraulic component routes these flows through constructed stormwater management facilities and natural topographic features to determine flood stages. Five 24-hour design storms were simulated, including return frequencies of 2.33 (mean-annual), 10, 25, 50, and 100 years. Jones Edmunds selected ICPR4 for hydrologic and hydraulic modeling based on a variety of factors such as applicability to the watershed and the local engineering community's familiarity with the software. This model is Federal Emergency Management Agency (FEMA)-accepted for flood insurance studies within SWFWMD.

We developed the model schematic from the hydro-network developed during the Watershed Evaluation phase of the project (see Section 5). In some instances, we revised the model schematic to better represent a specific feature within the watershed. In these instances, the model schematic may no longer match the hydro-network developed during the watershed evaluation. Throughout the model development, we reviewed the model for missing interconnections and added connections that may have been excluded initially. Appendix B includes the model schematic, which is stored in the *Model* feature dataset within GWIS 1.6.

6.2 HYDROLOGY

Subbasin areas, times of concentration (t_c), and infiltration parameters were developed as model inputs. Portions of the City contain sandy, exceptionally well-drained soils with a relatively deep Seasonal High-Water Table (SHWT); therefore, runoff volumes were calculated by applying the Vertical Layers infiltration method as implemented in ICPR4. This infiltration method uses the Green-Ampt method for each vertical soil layer and considers interactions between the layers. The runoff volume is then distributed over the duration of the simulation at rates calculated according to the NRCS Unit Hydrograph Method. Important parameters needed to calculate the runoff volume include the directly connected impervious area (DCIA) and the infiltration parameters. Runoff rates and timing are controlled by the hydrograph shape factor and the t_c. The subsections below describe the methodologies that Jones Edmunds applied to develop the hydrology parameters.

6.2.1 TOTAL IMPERVIOUS AREAS AND DCIAS

Jones Edmunds created a total impervious surface layer for the City by combining various sources of existing information with newly created impervious data generated to provide complete coverage. Existing polygon sources included Pinellas County's stormwater utility data and building footprints together with the waterbodies feature class from the 2007 FDEM LiDAR project. Jones Edmunds supplemented these data by creating a roads polygon layer from the roadway breaklines collected as part of the 2007 FDEM Coastal LiDAR project and manually digitizing impervious areas for large commercial properties, mobile home parks, and multi-family residential communities not contained in the in the County's datasets. Digitizing was accomplished using the Pinellas County 2018 aerials. Jones Edmunds then assigned assumed DCIA values to the records in the total impervious surface layer based on several factors such as building type (e.g., commercial or residential), usage (parking versus roadway), and data source (e.g., waterbodies). Jones Edmunds then calculated total percent impervious and DCIA for each subbasin in the watershed model.

6.2.2 INFILTRATION LOSSES

Jones Edmunds downloaded the NRCS soil data for the watershed model from the US Department of Agriculture (USDA) Web Soil Survey by uploading a buffered outline of the watershed. The soil data were then submitted to SWFWMD, who returned a look-up table of values that contained all necessary soil parameters for the Vertical Layers infiltration method in ICPR4. This look-up table was then imported into the model.

6.2.3 TIME-OF-CONCENTRATION

Jones Edmunds calculated the t_c using the methods outlined in the NRCS *Technical Release 55* (TR-55). We determined the longest flow path in each subbasin using a combination of GIS techniques and manual review. We excluded from this analysis any storage or conveyance areas that would be considered in the hydraulics model to avoid routing flow in both the hydrologic and hydraulic components of the model. Sheet flow was assumed for the first 100 feet of the flow path. The remainder of the flow path was considered shallow concentrated, open channel, or pipe flow. Roughness values were assigned for sheet flow, and pervious/impervious classifications were assigned to the remaining shallow concentrated portion of the flow path. Travel times were then calculated using the methods described in TR-55. A minimum velocity of 0.1 foot per second (fps) was applied as well as a total minimum travel time of 10 minutes. The travel times were reviewed for consistency and adjusted as needed to ensure consistency with the t_c values in the model.

6.2.4 UNIT HYDROGRAPH

The NRCS Unit Hydrograph Method was used to distribute runoff volume over the duration of the storm. Runoff rates and timing are controlled by the hydrograph shape factor and the t_c, with lower peak factors. The standard peak factor of 256 recommended by SWFWMD was used for all subbasins. This peak factor is reasonable because of the watershed's high development intensity – which would tend toward higher peak factors – is offset by the watershed's low relief.

6.2.5 RAINFALL SIMULATIONS

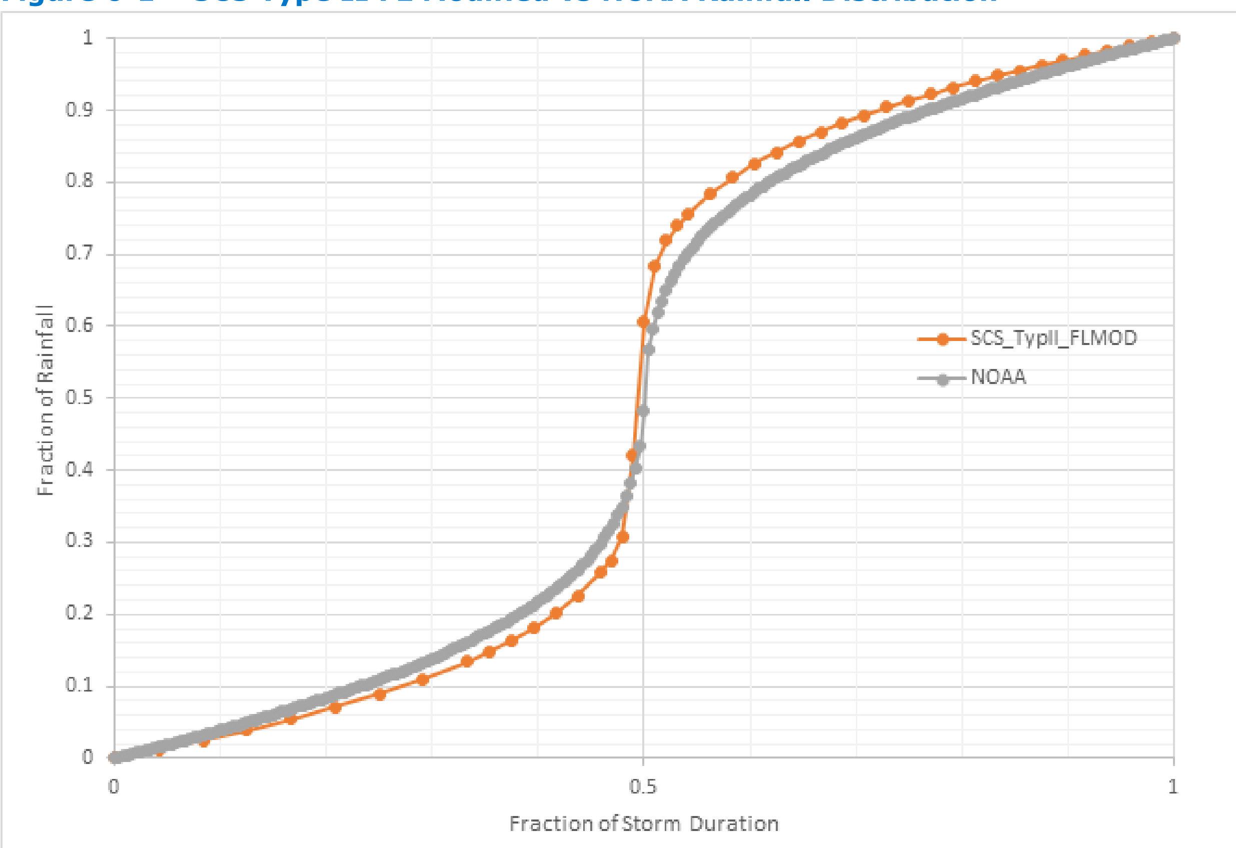
Jones Edmunds modeled the 2.33- (mean-annual), 10-, 25-, 50-, and 100-year frequency 24-hour storm events in ICPR using the Soil Conservation Service (SCS) Type-II Florida-Modified Rainfall Distribution. Table 6-1 lists the rainfall volumes for these storms, which were derived from rainfall isohyet maps provided in SWFWMD's *ERP Information Manual* (SWFWMD, 1996).

Table 6-1 Design Storm Rainfall Volumes

Simulation	Return Frequency	Duration	Distribution	Rainfall Volume
2.33YR24HR	2.33 Year	24-Hour	Type II FL Modified	4.5
10YR24HR	10 Year	24-Hour	Type II FL Modified	7.5
25YR24HR	25 Year	24-Hour	Type II FL Modified	9.0
50YR24HR	50 Year	24-Hour	Type II FL Modified	10
100YR24HR	100 Year	24-Hour	Type II FL Modified	12.0
500YR24HR	500Year	24-Hour	Type II FL Modified	15.0

Jones Edmunds also tested the City's watersheds' responses to design storms specified in current National Oceanic and Atmospheric Administration (NOAA) guidance as currently recommended by NRCS. Although these storms have a greater depth than the SWFWMD recommended storms, the intensity curve is sloped slightly more gently than the standard SCS Type-II Florida-Modified Rainfall Distribution (shown in Figure 6-1); therefore, the results, reflected in peak flood stages, are not that dissimilar to the results obtained using the SCS Type-II Florida-Modified Rainfall Distribution with standard rainfall depths. For this reason, the model and all subsequent evaluations – floodplains, LOS, and BMP evaluations – were based on the more locally accepted design storms.

Figure 6-1 SCS Type II FL Modified vs NOAA Rainfall Distribution



6.3 Hydraulics

6.3.1 Conveyance Features

Conveyance features within the watershed include closed conduits (pipes), structural weirs, drop-structures, overland weirs, and open channels – both natural and manmade. Modeled closed conduits include an overflow connection as needed to simulate flow occurring from water levels breaching ground surface during storms.

Photograph 6-1 Closed Conduit Discharging to Manmade Channel

Table 6-2 Hydraulic Conveyance Features

Hydraulic Feature	Count
Pipes	854
Channels	156
Drop Structures	192
Overflow Channels	156
Structural Weirs	102
Overland Weirs	1,364

Table 6-2 shows the hydraulic conveyance features contained in the watershed model.

Invert elevations and dimensions were entered into the model based on field reconnaissance, as-built drawings, design plan sets, and survey data. Pipe lengths for the model were calculated within GIS based on the distance between surveyed inverts or as shown on as-built documents.

6.3.2 Overland Weirs

Overland flows occur at saddles along basin boundaries, over man-made berms, or over roads. Flows over these landscape features were estimated with the weir equation. Weirs in ICPR4 representing subbasin saddles are linked to irregular cross-sections developed using the LiDAR-based DTM. A Jones Edmunds GIS-based tool was used to extract the cross-sections that represent the geometry of the saddle captured in the LiDAR. We developed the cross-section lines from the subbasin boundaries, which were typically delineated along the ridge between subbasins and would provide inter-basin connections during extreme storm events. The cross-section lines were horizontally smoothed in GIS to avoid overestimating the true weir length. We then extracted elevations along the lines from the 5-foot-by-5-foot DEM. Next, we exported the station-elevation relationship for each cross-section. We thinned (generalized) the station-elevation data using the Douglas-Peucker technique with a tolerance of 0.1 foot. This reduced the number of points needed to characterize each cross-section. As a quality control (QC) measure, we compared the cross-sectional area before and after thinning to confirm that no significant changes occurred in the cross-sectional area. We also reviewed a plot comparing the original cross-section and the thinned cross-section to confirm that no errors occurred during the thinning process and that cross-sectional geometry was essentially the same.

6.3.3 Storage Representation

Storage is represented in ICPR4 by stage-area relationships at model nodes. Jones Edmunds calculated stage-area relationships for each subbasin using a GIS-based tool that we developed. The stage-area tool extracts volume and area from the DTM at user-specified intervals (0.1-foot intervals were used for the watershed model). The extraction interval varies based on an error tolerance that the user specifies.

Storage areas in the watershed include permitted detention or retention ponds, other man-made water features, and lakes. Most of the permitted storage areas have a control structure such as a drop structure, pipe, or structural weir. Storage also exists in the channel overbanks. Channel storage was removed from node storage using exclusion polygons delineated in ArcGIS. These polygons define the channel area occupied by flow conveyance and therefore are not available for storage.

Photograph 6-2 Typical Pond Storage Area

Jones Edmunds set starting water levels in storage areas using control structure information, aerial imagery, LiDAR data, seasonal high water (SHW) indicators visible on aerials or other datasets (water, vegetation, ground cover), and tidal (boundary) information. The following summarizes the approach used to set the starting water level at various types of features in the watershed:

- Nodes representing stormwater features with control structures, such as wet detention ponds, were set to start at the elevation of the lowest modeled control elevation.
- Nodes representing storage features such as lakes or wetlands were set to start at the highest water level observed in aerial imagery or the apparent wetland limits where water was not present.
- Nodes representing the outfalls to the Intracoastal Waterway were set to start at the boundary elevation representing NOAA's Ozona gauge (0.76 foot NAVD 88). This elevation is based on the mean high water (MHW) levels provided in the SWFWMD *Watershed Management Program Guidelines and Specifications* (2002). This tailwater was translated upstream until superseded by initial conditions set by one of the other methods presented in this list.

- Nodes representing other channels and stormwater conveyance features were set with the initial condition based on the assumption that the stormwater system was drained to the lowest invert directly downstream of each node – or in other words, based on the assumption that the stormwater system was drained dry.

6.3.4 BOUNDARY CONDITIONS

A fixed boundary is set at the downstream node of each coastal outfall. The fixed boundary, which is set at 0.76 foot NAVD 88 representing MHW at the NOAA Ozona gauge. At Spring Branch, the outfall is set as a variable time-stage boundary based on the model results for the City of Clearwater's Stevenson Creek model. Jones Edmunds reviewed the watershed boundaries and topography for adjacent basins and determined several locations where water likely flows out of the watershed during high flow events. None of the flow boundaries contribute appreciable flow, with the largest being 21 cubic feet per second (cfs) (peak for the 100-year storm) at an overland flow location between Curlew Creek and Alligator Creek.

Photograph 6-3 Typical Coastal Model Boundary

7 MODEL CALIBRATION AND VERIFICATION

Jones Edmunds calibrated and verified the Curlew Creek and Jerry Branch portions of the model to three gauges as part of the Curlew Creek/Smith Bayou WMP. The results show that the model is well calibrated. The simulation quality proof offered by the calibration and verification also extends to the ungauged portion of the watershed, such as Cedar Creek and Spring Branch, since these watersheds share similar hydrologic and hydraulic characteristics to the gauged portion and the models were prepared using identical methods and constants. The following subsections describe the available data as well the approach and results for the calibration and verification. Final calibration- and verification-stage graphs are provided below. Appendix C includes the complete set of graphs including the initial comparisons for stage and flow before calibration.

7.1 CALIBRATION DATA

USGS maintains three continuous-recording stream gauging stations within the study area, providing an excellent stage and flow record for model calibration and verification. Table 7-1 lists the stations and locations.

Table 7-1 USGS Gauges

Site Number	Site Name	Period of Record
02309415	Curlew Creek at Evans Road	08-10-1999 to Present
02309421	Curlew Creek at Belcher Road	06-13-2002 to Present
02309425	Curlew Creek at CR 1	06-13-2002 to Present

NexRad RADAR rainfall estimates, available from SWFWMD on a 2-kilometer (km) grid, were applied to more accurately account for the calibration and verification events' spatial and temporal distributions. Rainfall distributions were developed at 15-minute increments for each of the NexRad grids. For the model hydrology, the NexRad rainfall distributions were applied to each basin based on the intersection of the basin's centroid with the NexRad grid cells.

7.2 MODEL CALIBRATION

Jones Edmund calibrated the model using the rainfall record and recorded stages for Hurricane Irma. With the eye passing about 40 miles east of Pinellas County, Hurricane Irma was the first hurricane experienced by Pinellas County since 2004. Hurricane Irma was chosen as the calibration event for several reasons:

- As in the design simulation, detention ponds were typically full due to approximately 5 inches of rainfall in the previous 2 weeks.
- Rainfall depth was reasonably uniform across the watershed.
- The event produced a discrete, well-defined response in the USGS gauges in the study area.

NexRad rainfall volumes varied from 3.6 inches for Pixel 100785 at the west edge of the watershed to 4.3 inches for Pixel 100788 at the east edge of the watershed. Total rainfall depths for the 19 NexRad pixels (grid cells) had a standard deviation of 0.2 inch.

Jones Edmunds ran the model simulation from September 9 through September 11, 2017, which was approximately the time required for water levels to recover at the USGS gauge locations. We compared the model results to gauge data and then adjusted the appropriate model parameters to obtain a better fit.

7.2.1 Initial Comparison

Jones Edmunds compared the initial simulation results to the gauge data (Appendix C) and found that at the start of the simulation the model was reporting initial water levels that were lower than the recorded data at Gauges 02309421, 02304915, and 02309425. During the simulation, the water levels quickly reached simulated stage levels. The simulated maximum stage was 0.3 foot lower than the recorded maximum at Gauge 02309421 and 0.6 foot lower than the recorded maximum at Gauge 02304915. The maximum simulated stage at Gauge 02309425 was 0.2 foot higher than measured.

The simulated discharge versus the calculated discharge was also compared for all gauges. The flows at the gauges compared reasonably well to the flow values calculated for each gauge with differences ranging between 10 and 20 percent. Although calibration and verification comparisons are provided for both discharge and stage, stage is the primary benchmark used for calibration.

7.2.2 Calibration Adjustments

Before adjusting the model parameters, Jones Edmunds sought to determine the cause of the differences in elevations between the model starting elevations and the initial elevations reported by the gauges. We began by reviewing the source data for the model features near each gauge. While reviewing the data near Gauge 02309415, we determined that the 5-foot-x-10-foot box culvert that discharges on the north side of Evans Road does not match the channel bottom but rather discharges to a “sump” in the Type U energy dissipator at the end of the box culvert. Water must then rise to a minimum elevation of 54.1 feet (NAVD 88) before continuing downstream. The initial stage for Node ND4095 was originally set at elevation 51.5 feet, the invert of the box culvert; therefore, the initial stage for the node was changed to 54.1 feet to match the invert of the downstream channel. Jones Edmunds conducted field visits to Gauge Nos. 02309421 and 02309425 to determine the cause of the differences between the modeled and measured data. During the field visit to Gauge 02309425 Jones Edmunds found a weir just downstream of the gauge, which helped to explain the initial stage differences noted between the simulated and measured stages (Photograph 7-1).

Photograph 7-1 USGS Gauge 02309425

Jones Edmunds staff also visited Gauge 02309421, Jones Edmunds' staff found a rip-rap weir just downstream of the gauge intake location (Photograph 7-2). Jones Edmunds collected a cross-section at the weir and added the weir to the model to minimize the difference between the initial stage of the model and the measured stage at Gauge 02309421.

Photograph 7-2 USGS Gauge 02309421

After making these adjustments, the simulated and gauge elevations were compared again. Table 7-2 and Table 7-3 present stage and flow comparisons and Figure 7-1 through Figure 7-3 graphically show the stage results. Appendix C provides flow comparisons.

Table 7-2 Hurricane Irma Stage Comparison

Gauge	Node	Simulated (ft)	Measured (ft)
2309415	ND4095	56.6	56.5
2309421	NA0185	28.2	28.1
2309425	NA0080	12.5	12.1

Table 7-3 Hurricane Irma Flow Comparison

Gauge	Link	Simulated (cfs)	Measured (cfs)
2309415	RD4095C	95	116
2309421	RA0185W	616	503
2309425	RA0090C	699	772

Figure 7-1 Final Calibration (Hurricane Irma) – Curlew Creek at Evans Road

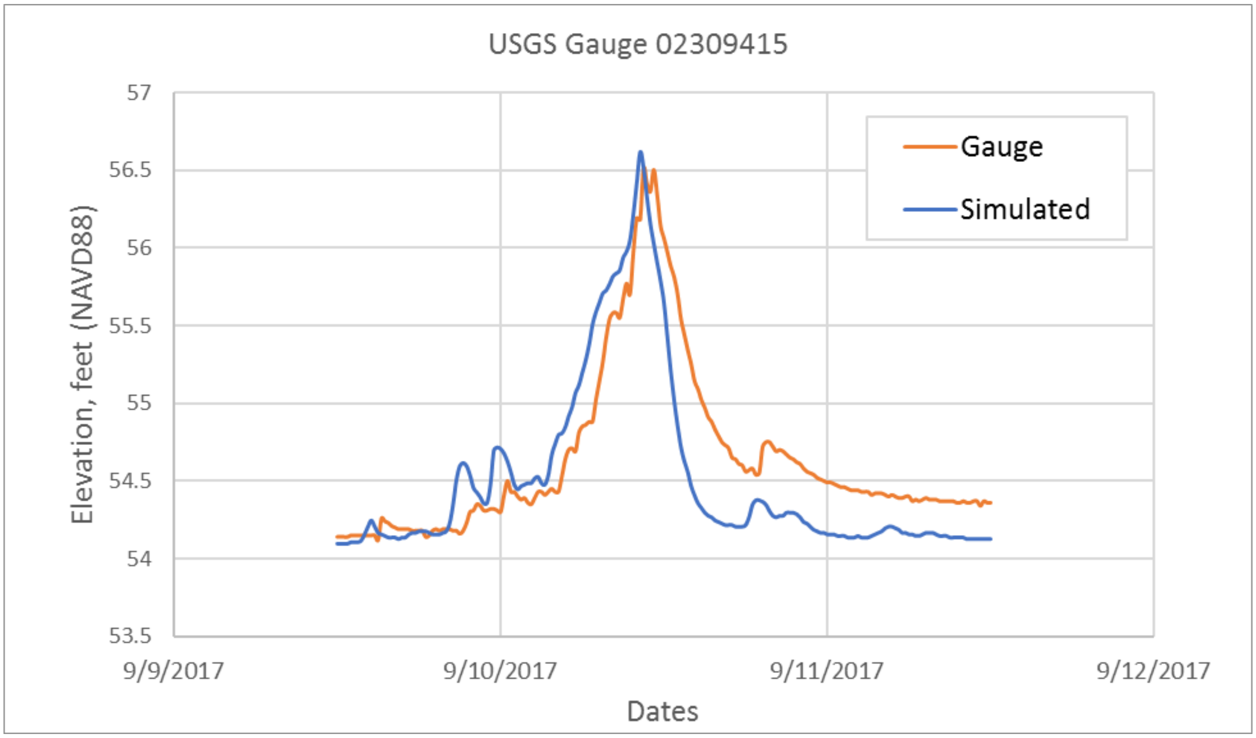


Figure 7-2 Final Calibration (Hurricane Irma) – Curlew Creek at Belcher Road

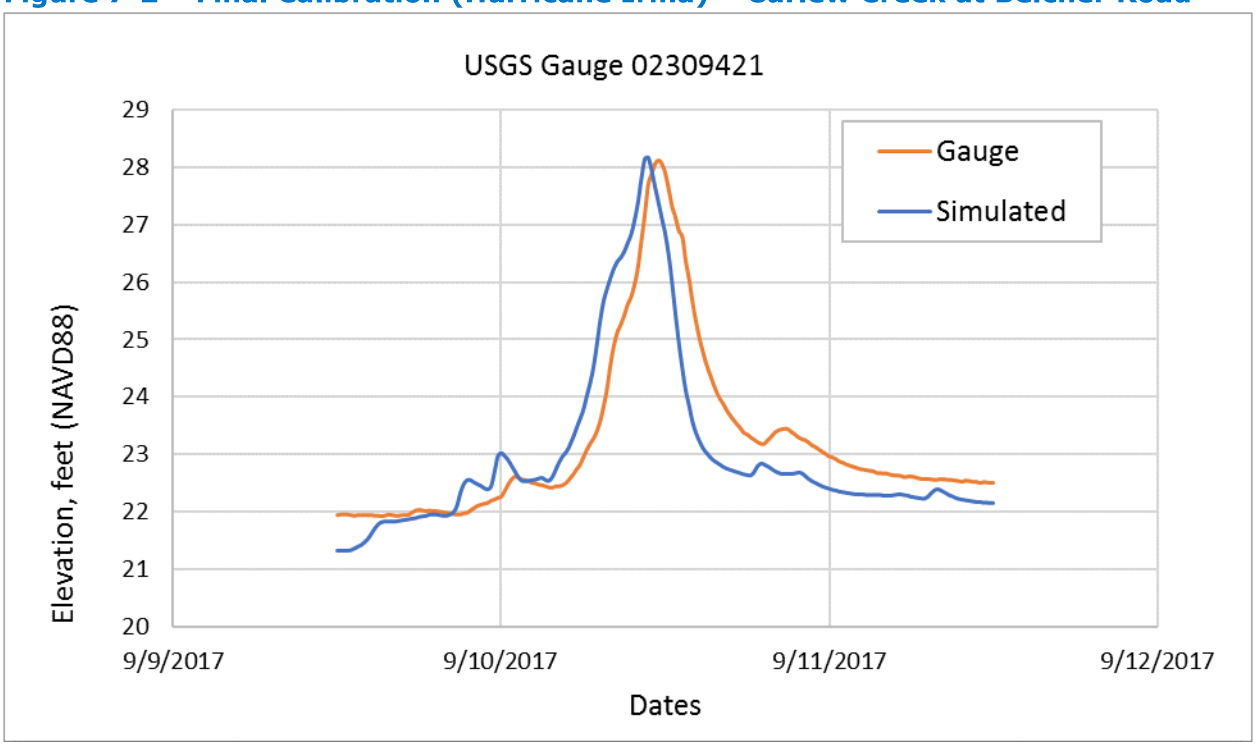
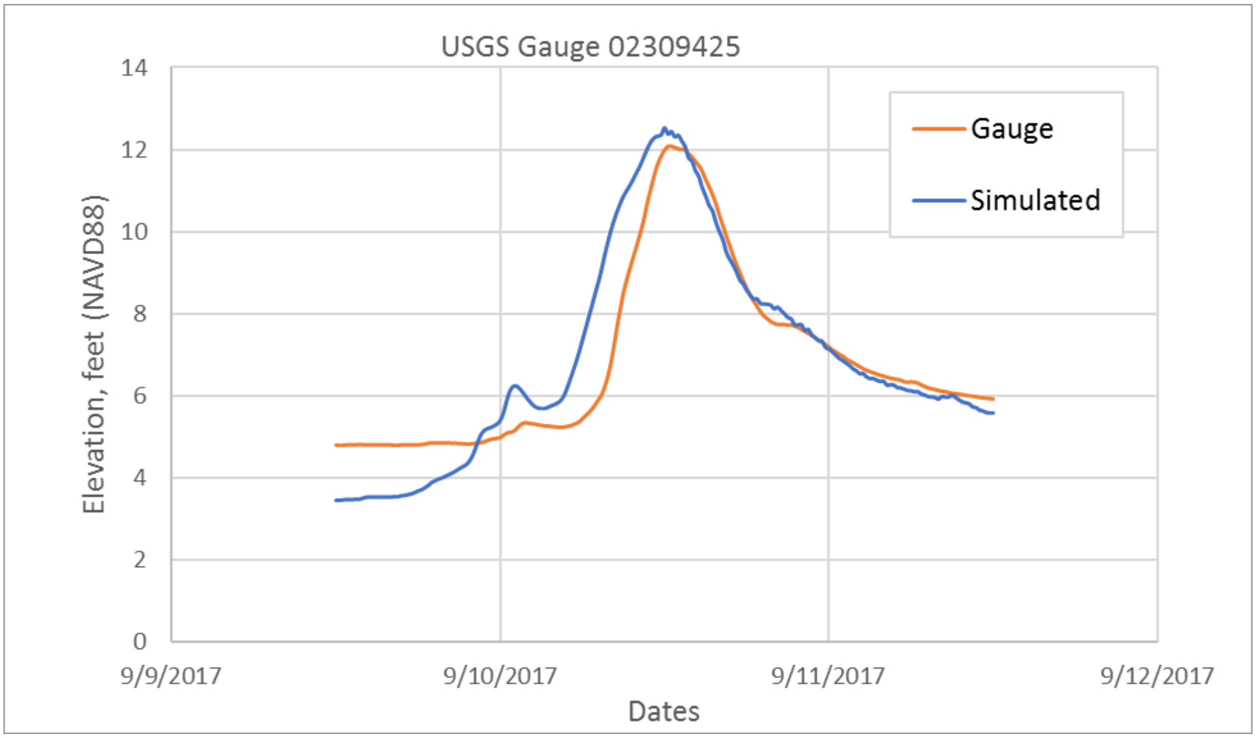


Figure 7-3 Final Calibration (Hurricane Irma) – Curlew Creek at CR 1



7.3 Model Verification

Jones Edmunds simulated a second storm to verify that the calibration adjustments will produce reasonable results for other storms. Jones Edmunds used Hurricane Hermine as the verification event. Hurricane Hermine produced heavy, intense rainfall and runoff in the Pinellas County area as it made landfall on August 30, 2016, approximately 150 miles north of the watershed. To determine the period for the verification simulation, we examined rainfall for the days leading up to Hurricane Hermine and found that approximately 1 inch of rain fell across the watershed on the afternoon before the storm. This rain event, which ended approximately 18 hours before the beginning of the initial rain from Hurricane Hermine, was included in the simulation to capture the soil storage loss before the beginning of the verification event.

Jones Edmunds reviewed the gauges to identify the appropriate simulation period. Gauges 2309415 and 2309421 showed a quick response and recovery from the rainfall due to Hurricane Hermine, but Gauge 02309425 at the downstream end of the watershed did not recover fully until approximately 6 days after the storm. The model simulation begins at August 29, 2016, at 1200 hours and ends at August 31, 2016, at 0600 hours to capture the storm peak. NexRad rainfall volumes varied from 2.0 inches for Pixel 101259 at the northwest edge of the watershed to 5.2 inches for Pixel 99366 at the southeast edge of the watershed with a standard deviation of 0.9 inch. We compared the model results to gauge data to verify the model calibration.

Table 7-4 and Table 7-5 summarize the results of the verification; Figure 7-4 through Figure 7-6 provide comparison graphs. Review of the stage/time graph for Gauge 02309415 shows that the stage increased 0.94 foot between 1915 and 1930 hours to stage 58.73 (the highest stage reported during the simulation period). Then the stage decreased by 1 foot between 1930 and 1945 hours to 57.73. This sudden rapid increase and decrease in reported stage does not match the overall shape of the stage versus time hydrograph. Flow data were not reported for Gauge 02309415 during the simulation event possibly due to questionable stage data. Overall, the model matched Gauge 02309425 and Gauge 2309421 well regarding flow and stage. These verification results confirm that the City's model is well calibrated.

Table 7-4 Hurricane Hermine Stage Comparison

Gauge	Node	Simulated (feet)	Measured (feet)
2309415	ND4095	57.9	58.7
2309421	NA0185	30.1	30.8
2309425	NA0080	13.1	12.8

Table 7-5 Hurricane Hermine Flow Comparison

Gauge	Link	Simulated (cfs)	Measured (cfs)
2309415	RD4095C	95	Not Reported
2309421	RA0185W	953	859
2309425	RA0090C	699	772

Figure 7-4 Verification (Hurricane Hermine) – Curlew Creek at Evans Road

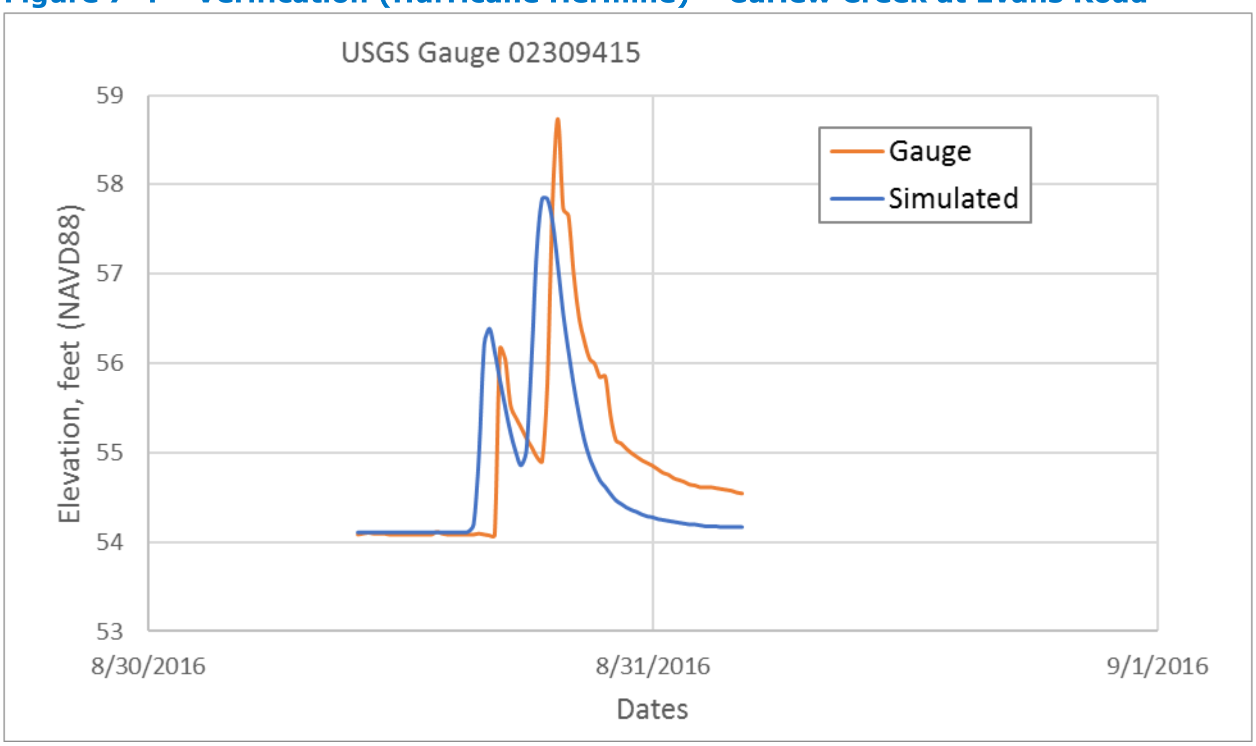


Figure 7-5 Verification (Hurricane Hermine) – Curlew Creek at Belcher Road

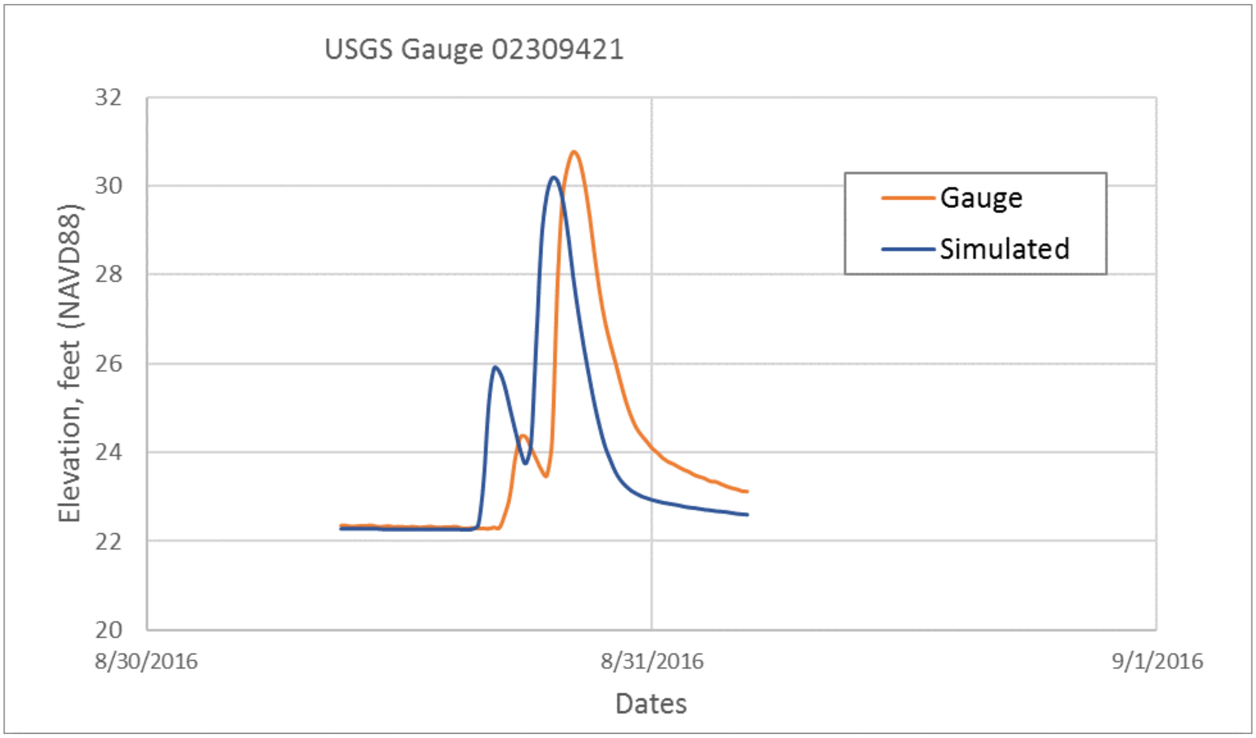
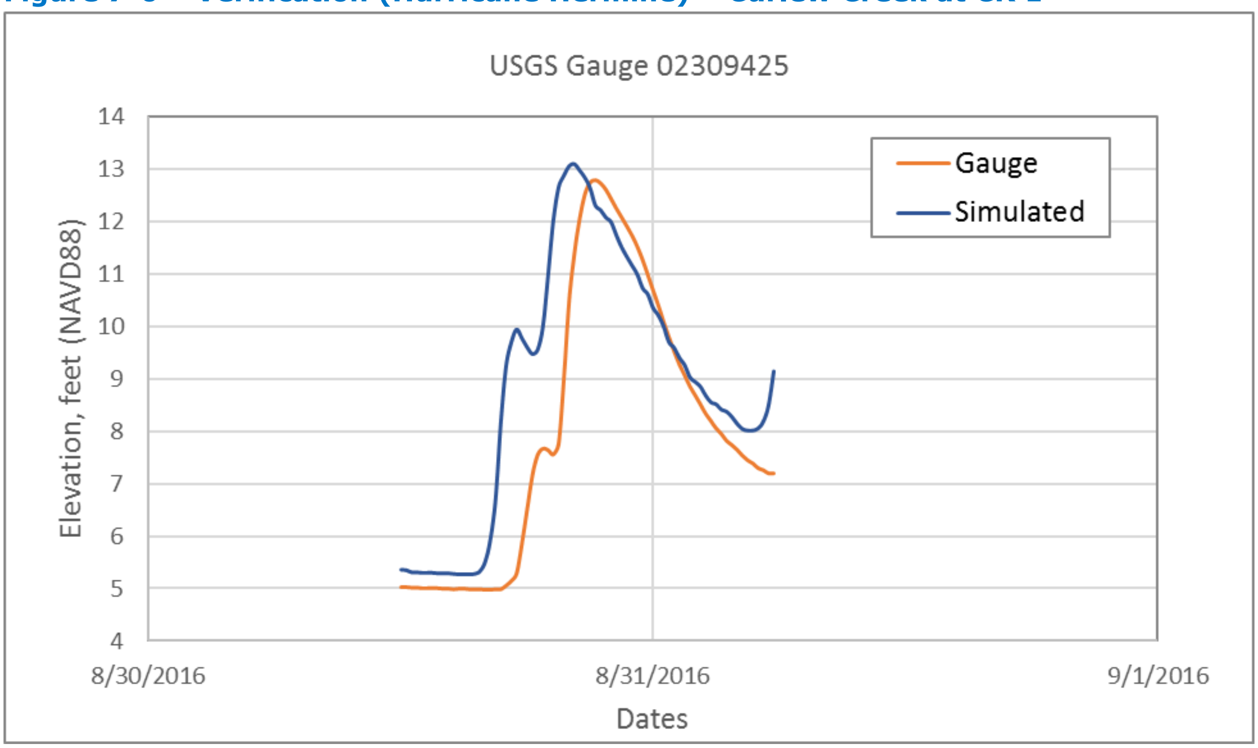


Figure 7-6 Verification (Hurricane Hermine) – Curlew Creek at CR 1



8 FLOODPLAIN ANALYSIS

Jones Edmunds created floodplains for the 100-year, 24-hour storm by mapping the existing conditions floodplains using results from the model described earlier in this Plan. The floodplains include sloped-surface floodplains in the channels where needed and also in some of the overland flow weirs as required to smooth the floodplain transitions for one basin to the next. Floodplains were mapped by creating a 5-foot resolution water-surface grid. The 100-year floodplains were "cleaned" to remove small islands and fill small gaps. "Small" was defined as 2,500 square feet or less consistent with FEMA's *Guidelines and Standards for Flood Risk Analysis and Mapping*.

Before mapping the 100-year/24-hour storm, Jones Edmunds tested a 5-day event to confirm that the 24-hour storm was the appropriate duration event to map. Although some of the closed-basins showed a slightly higher peak for the 5-day storm, the 24-hour event produced the majority of peaks for this watershed. Hydrographs in this watershed generally show a sharp vertical rise and a similar steep decline, which are characteristic of "flashy" systems where significantly increased flows occur immediately following rainfall and a relatively quick return to pre-rain conditions shortly after the end of the rain event. The 24-hour storm will produce higher peak stages than multi-day storms of the same return period in "flashy" systems.

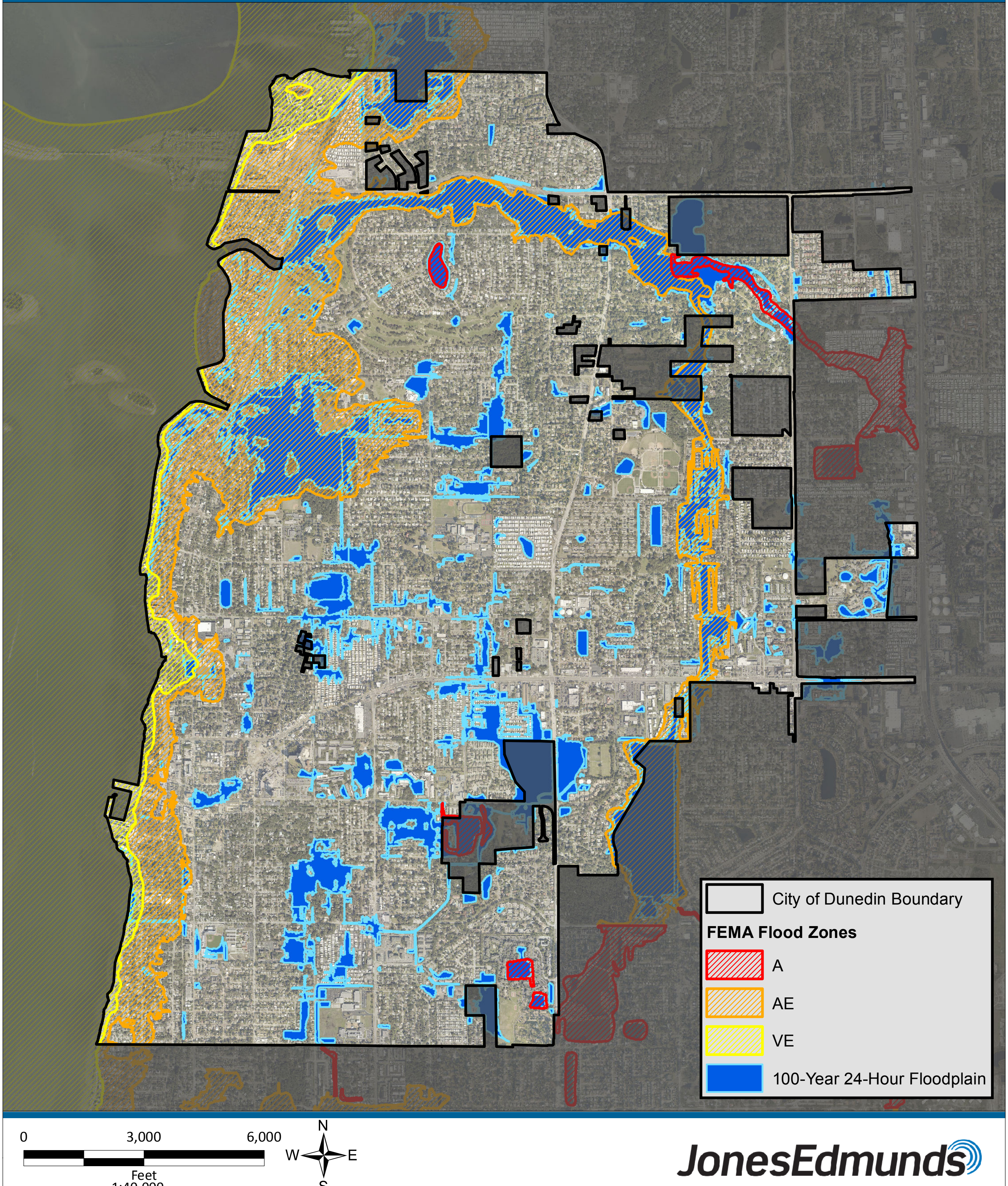
Jones Edmunds compared the 100-year floodplain prepared for this project to the existing FEMA inland (non-coastal) floodplains (Figure 8-1) effective August 18, 2009. With better topography and a more detailed study throughout the watershed, the floodplains prepared for this project appear to provide a more accurate representation of the watershed's inland flood risk areas. Many of the watershed's effective floodplains in the inland areas are based on approximate methods and an older topographic data source.

Appendix D provides a poster-sized map of the City's floodplains.

Figure 8-1

100-Year/1-Day Floodplains Compared to FEMA Effective Floodplains

City of Dunedin Stormwater Master Plan



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9 FLOODING LEVEL-OF-SERVICE

Jones Edmunds performed a flooding LOS analysis. The results of the LOS evaluation helped to identify the locations and severity of flooding problems within the City. The LOS evaluation has four components: 1) Roadway and Drainage Facility, 2) Major Channel, 3) Structural, and 4) Critical Facilities.

The LOS criteria, methodologies, and results are described for each component in the following sections.

9.1 Roadway and Drainage Facility LOS Standard

Jones Edmunds based the roadway and drainage facilities LOS on Section 3.5.4.2 of the *Pinellas County Stormwater Manual* (Pinellas County, 2017) (Figure 9-1). Jones Edmunds used the evacuation routes shapefile from the County's GIS library and FDOT data to identify the Evacuation Routes and Arterial and High-Use (annual average daily traffic [AADT] >1,500) Roads.

Figure 9-1 Excerpt from the Pinellas County Stormwater Manual

3.5.4.2. Design Frequency

The Table below summarizes the minimum design frequencies for use on County transportation facilities. These design frequencies are generally referenced in the FDOT Drainage Manual however no open or closed conveyance systems shall be designed below the 10-year frequency. The most current versions of the references listed in Section 3.5.5.3 shall be utilized when designing drainage systems for transportation projects.

Facility	Frequency
Roadside, Median and Collector ditches or swales	10-year
Outfall ditches, Major Channels and Canals	25-year
Storm Drains	10-year
Bridges and Cross Drains on Evacuation Routes	100-year
Bridges and Cross Drains on Arterial and High-Use (ADT > 1,500) Roads	50-year
Bridges and Cross Drains on all Other Roads and Facilities	25-years
Roadside Ditch Culverts	10-year

The frequencies cited by the table above are minimums. Higher design frequencies shall be used where Level of Service (LOS) requirements prevail or Master Plan Goals warrant use of such. Examples of this are situations where structural flooding LOS concerns may entail consideration of the 100-year design frequency and requirement to complete design in accordance with future improvements identified by Stormwater Master Plans or Watershed Management Plans.

The proposed project shall also demonstrate that the inlet spacing is sufficient to maintain the spread in accordance with the standards in the Florida Greenbook.

Using these criteria, evacuation routes were graded as passing or deficient based on flooding during the 100-year/24-hour storm (12-inch rainfall depth). Arterial and High-Use (ADT >1,500) Roads LOSs were graded as passing or deficient based on flooding during the 50-year/24-hour storm (10-inch rainfall depth). All other roadway LOS were graded as passing or deficient based on flooding during the 10-year/24-hour storm (7.5-inch rainfall depth). Each basin was assigned a passing or deficient LOS based on each of the criteria, so a basin may have a passing grade for Evacuation Route LOS and a deficient grade for either or both the Arterial and High-Use Road LOS or the Local Road LOS. Figure 9-2 through Figure 9-4 show the results of the roadway LOS. Appendix E contains a list of LOS-deficient sub-basins and the affected roadways within each sub-basin.

The results of the roadway LOS analysis are contained in the LOS feature dataset in the project database.

9.2 Major Channel LOS Standard

Major channels were graded as passing or deficient based on conveyance of the 25-year/24-hour storm within the top-of-bank of the channel. While a channel system being graded as deficient may not result in property damage, this LOS standard allows the City a measuring stick for the performance of the major open drainage systems in the City. The results of the channel LOS is provided in the LOS feature class contained in the LOS feature dataset in the project database.

9.3 Structural LOS Standard

Each sub-basin was also assigned a structural LOS based on structures within the sub-basin intersecting the 100-year 24-hour floodplains. The City contains 1,436 structures that intersect the 100-year/24-hour floodplains. Figure 9-5 illustrates the results of the Structural LOS. Appendix E contains a table of the LOS-deficient subbasins and the number of buildings intersecting the floodplains for each basin.

The results of the structural LOS analysis are contained in the LOS feature dataset in the project database.

9.4 Critical Facilities LOS Standard

Jones Edmunds also evaluated and assigned a LOS for the critical facilities. These facilities were developed from a combination of data provided from the City and downloading and analyzing the latest health care sites, essential government facilities, schools, fire stations, and shelters shapefiles from the County's GIS library. The facility LOS is passing if the facility structure does not intersect the 100-year/24-hour floodplain and road access to the facility is protected during the 100-year/24 storm. Table 9-1 provides a list of the critical facilities and the results of the LOS analysis. The results are also shown in Figure 9-6 The results of the critical facility LOS analysis are contained in the LOS feature dataset in the project database.

Table 9-1 Critical Facilities

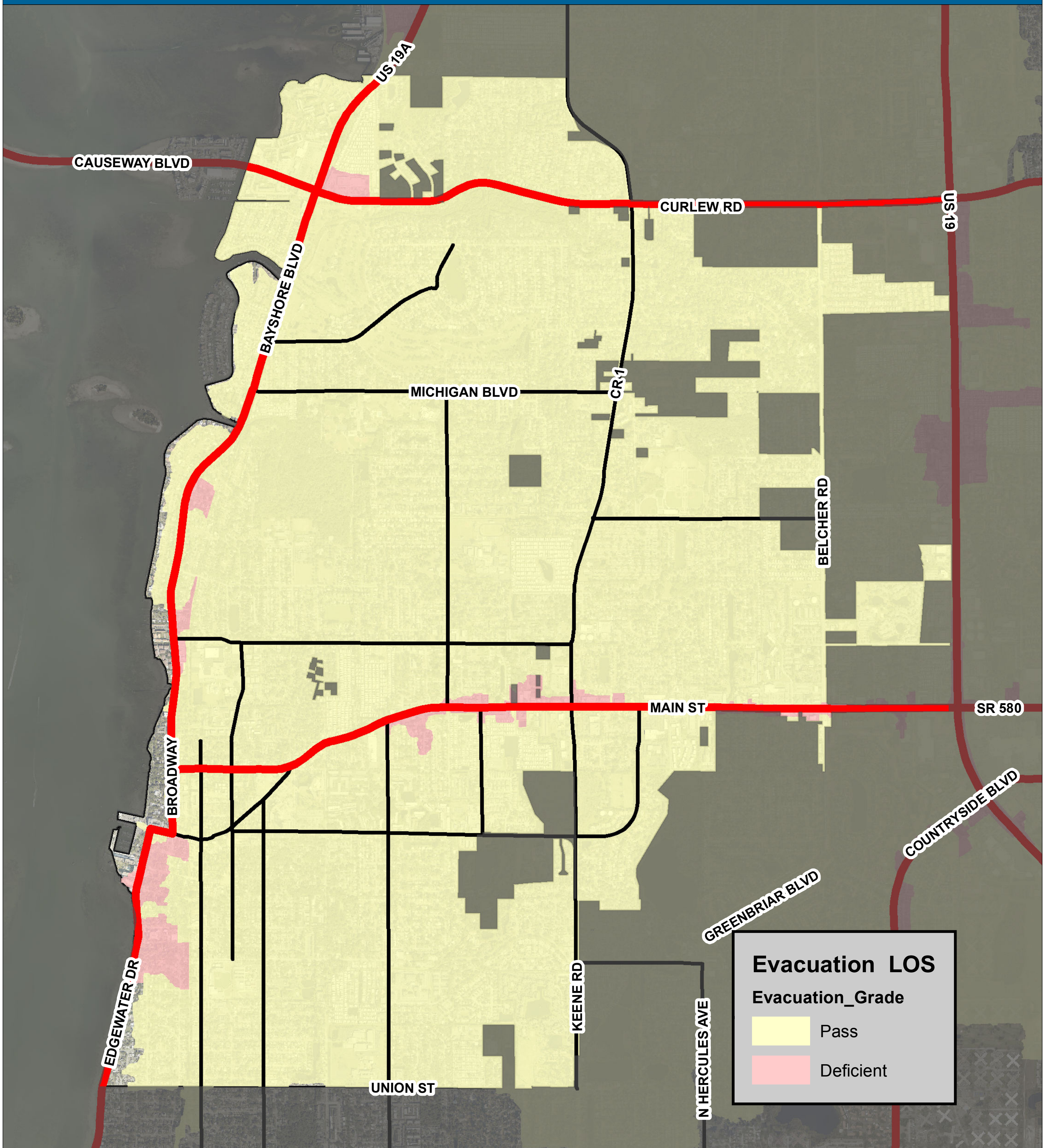
ID	Facility Name	Grade
1	Fire Station 62	Pass
2	Curlew Care Home #1	Pass
3	Bayou Gardens Dunedin	Deficient
4	Garrison-Jones Elementary	Pass
5	Dunedin Fire Admin	Pass
6	Dunedin City Hall	Deficient
7	Fire Station 60	Pass
8	Fire Station 61	Pass
9	Grand Villa of Dunedin	Pass
10	Villas at Lakeside Oaks	Pass
11	Park Place of Dunedin	Pass
12	Villa Anna Assisted Living Facility	Pass
13	Manor Care of Dunedin	Deficient
14	Cross Terrace Rehabilitation Center	Pass
15	Heather Haven	Pass
16	Dunedin ALF	Deficient
17	Mease Assisted Living	Pass
18	Heather Haven II	Pass
19	Mease Hospital - Dunedin	Pass
20	Lakeside Manor	Pass
21	Wild Flower Inn	Pass
22	Mease Manor Memory Care	Pass
23	Mease Continuing Care	Pass
24	Squire Community Home	Pass
25	Dunedin Highland Middle School	Pass
26	Dunedin Elementary School	Pass
27	Dunedin Community Center	Pass
28	Dunedin Academy	Pass
29	Open Door School	Pass
30	Cornerstone Christian School	Pass
31	Our Lady Of Lourdes Catholic School	Pass
32	Academie Da Vinci	Pass
33	Dunedin High	Pass
34	Curtis Fundamental Elementary	Pass
35	San Jose Elementary	Pass
36	DUNEDIN SOLD WASTE	Pass
37	Reclaimed Storage	Pass
38	DUNEDIN WATER TREATMENT PLANT	Pass
39	DUNEDIN PARK MTCE DIVISION	Pass
40	ENGLEBERT/VANECH RECREATION COMPLEX	Pass
41	DUNEDIN WASTEWATER TREATMENT PLANT	Pass
42	DUNEDIN COMMUNITY CENTER	Pass

ID	Facility Name	Grade
43	DUNEDIN PUBLIC LIBRARY	Pass
44	DUNEDIN SENIOR CENTER	Pass
45	PINELLAS COUNTY SHERIFF & VEHICLE MAINT	Pass
46	DUNEDIN (OLD) FIRE STATION NO 61	Pass
47	DUNEDIN NATURE CENTER	Pass
48	MARTIN LUTHER KING, JR RECREATION COMPLEX	Pass
49	DUNEDIN PUBLIC SERVICES	Pass
50	DUNEDIN FINE ARTS & CULTAL CENTER	Pass
51	Blue Jays Facilities	Deficient
52	Blue Jays Facilities	Deficient
53	Blue Jays Facilities	Pass
54	Blue Jays Facilities	Pass
55	Reclaimed Storage	Pass
56	Reclaimed Storage	Pass
57	Blue Jays Training Facility	Pass
58	DUNEDIN EOC	Pass
59	DUNEDIN GOVERMENT CENTER (Future)	Pass

Figure 9-2

Evacuation Routes and High-Use Roads

City of Dunedin Stormwater Master Plan



0 3,000 6,000

Feet

1:40,000

N W E S

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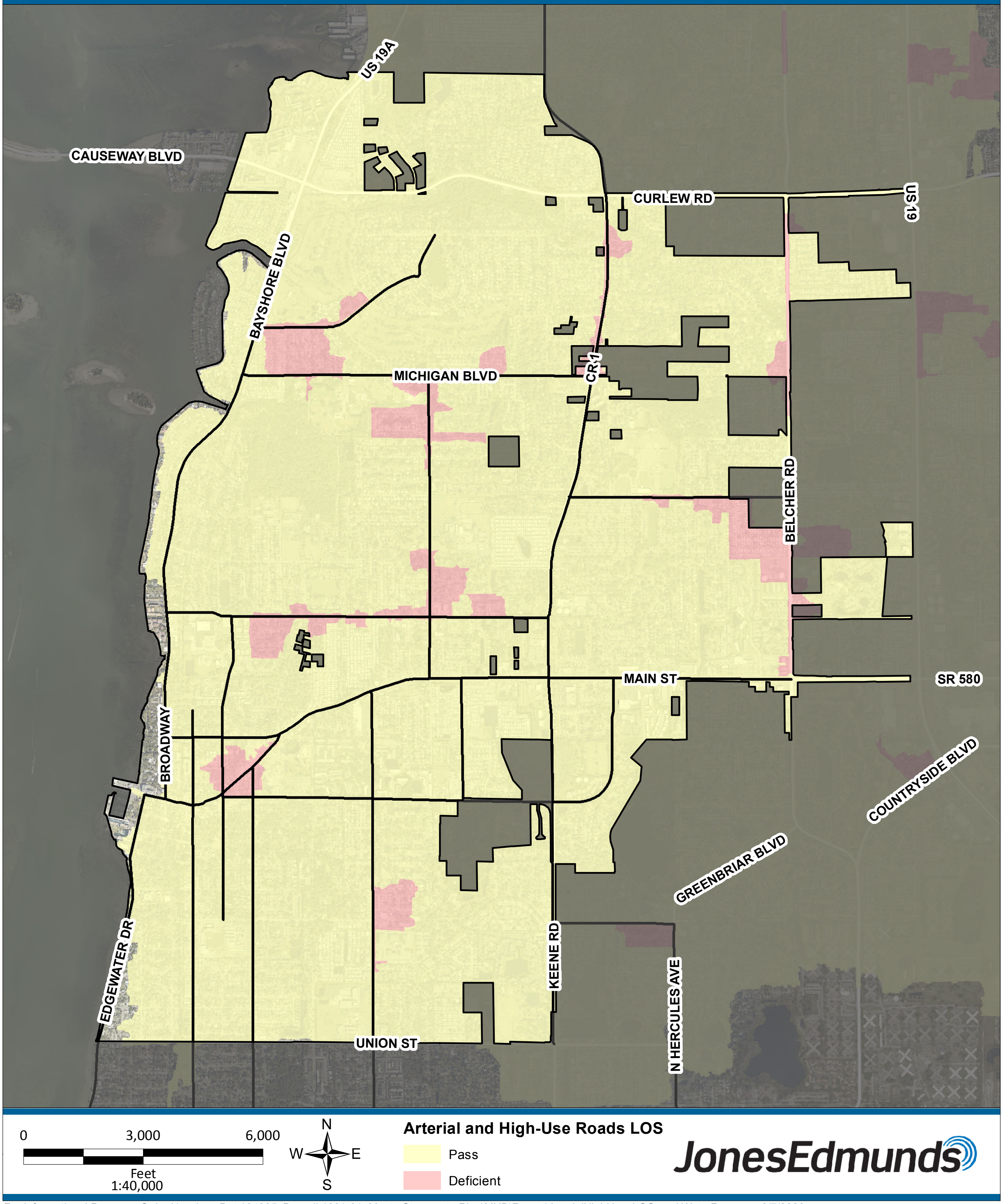
Figure 9-3

Arterial and High-Use Roads LOS

City of Dunedin Stormwater Master Plan



Arterial and High-Use Roads LOS

Pass

Deficient

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Figure 9-4

Local Roads LOS

City of Dunedin Stormwater Master Plan

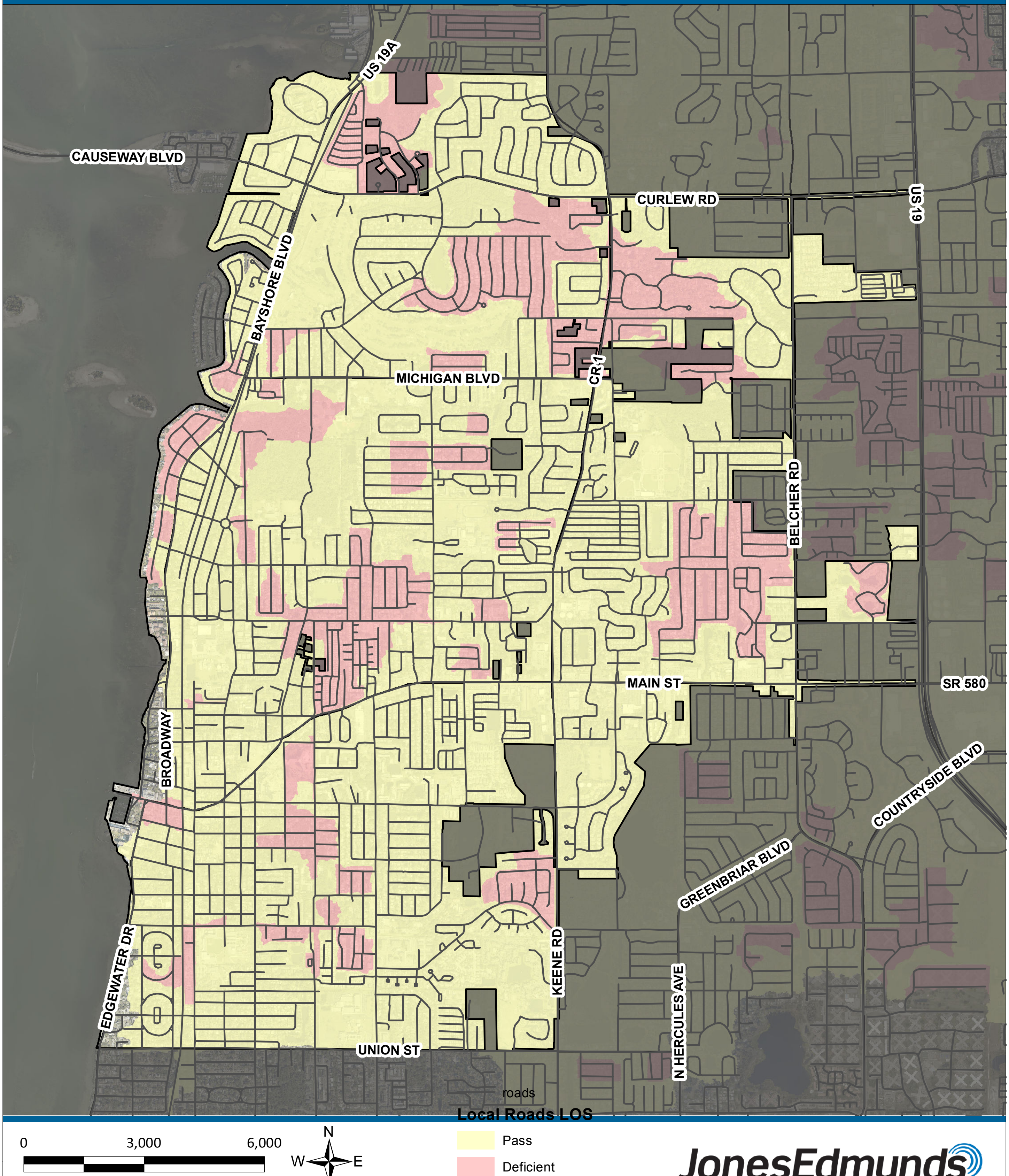

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Figure 9-5

Structural LOS

City of Dunedin Watershed Management Plan

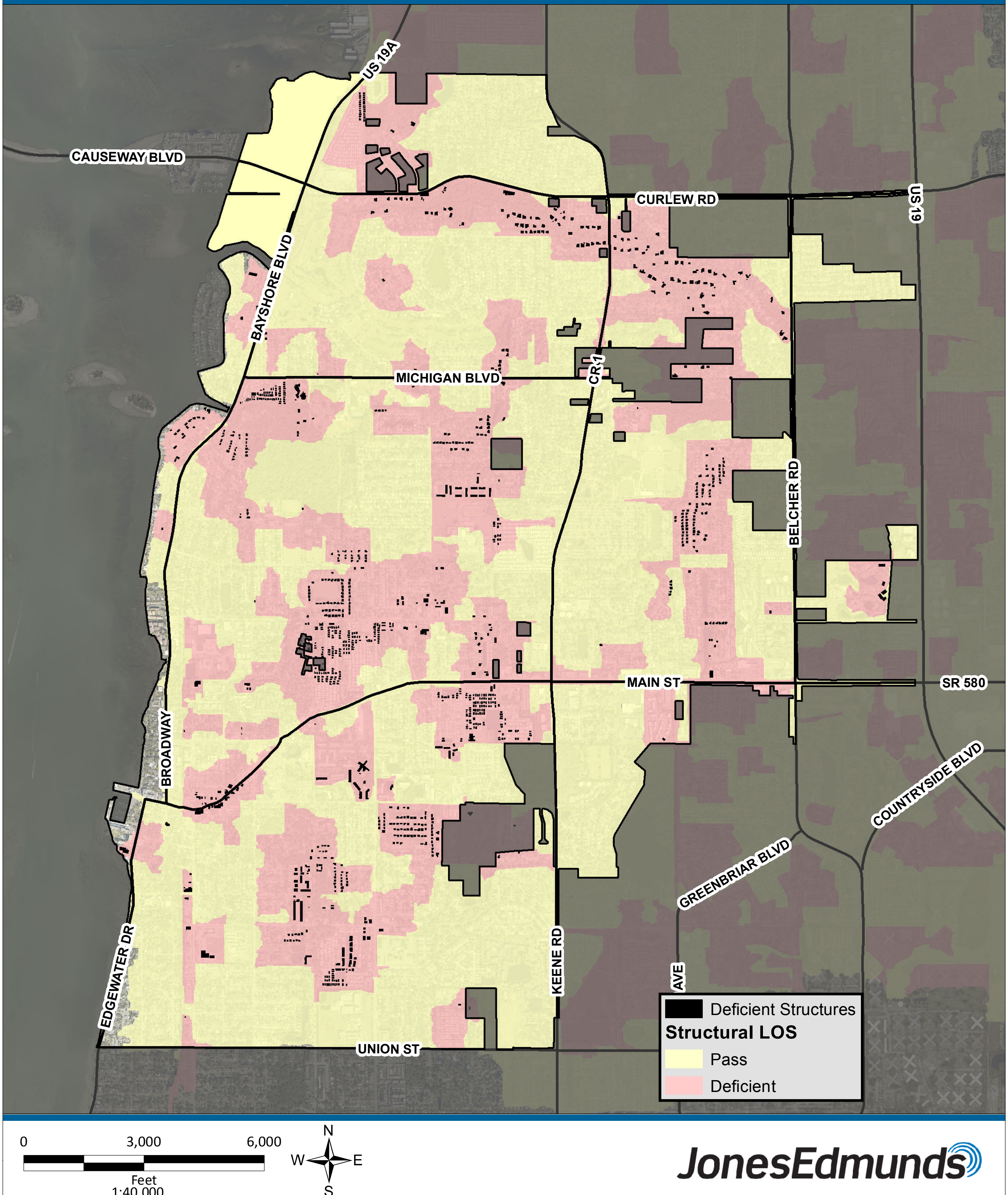
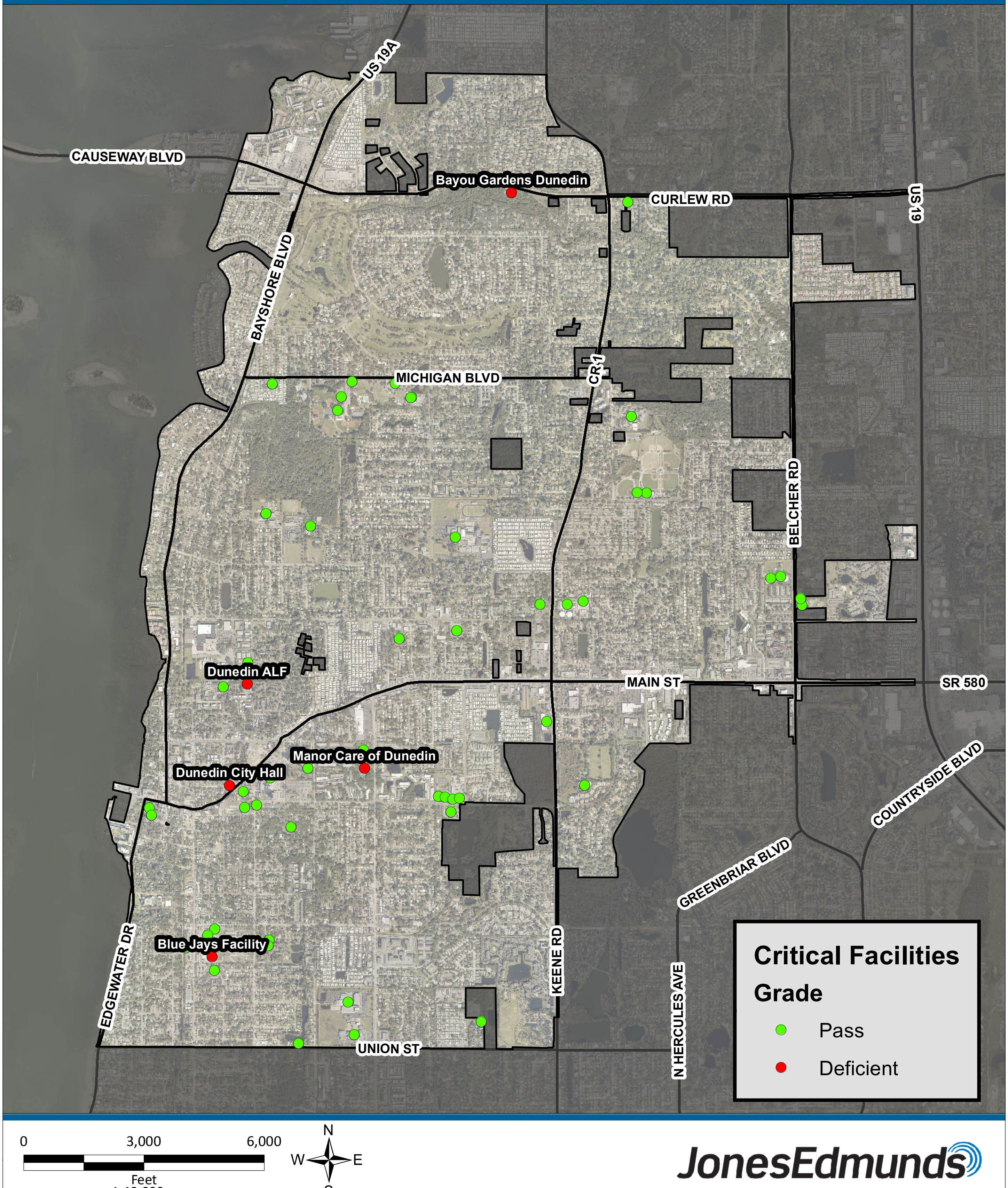


Figure 9-6

Critical Facilities LOS

City of Dunedin Stormwater Master Plan



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10 BMP ANALYSIS

Jones Edmunds conducted a BMP analysis to identify projects that provide flood control and water quality benefits within the City of Dunedin. Projects were developed for eight structural BMPs targeting specific flooding or water quality improvement locations and two additional conceptual BMPs defining general recommendations applicable throughout the City. One BMP (BMP #4) was carried over from the Curlew Creek WMP; therefore, BMPs contained within the City from both studies are contained within the City's masterplan report and project rankings. The City completed a Master Drainage Plan in 2003 that contained a list of recommended projects. Table 10-1 provides a list of the projects from the 2003 Stormwater Masterplan indicating that most of the projects have been constructed. Footnotes below the table provide the status of projects not completed.

Table 10-1 2003 Stormwater Projects

Project No.	Project Location	Completed
1	Union Street (West Outfall)	Yes
2	City's Proposed Local Drainage Improvements	***See Below**
3	San Christopher Drive	Yes
4	President Street	Yes
5	Orangewood Drive	Yes
6	Sperry Lake	Yes
7	Coastal Outfalls < 36" Diameter	No[1]
8	Coastal Outfalls> 36" Diameter	No[1]
9	East of Patricia, North of Union	No[2]
	Local Drainage Improvement Projects	
C-1	Richmond and Diane	Yes
C-2	Bay Street	Yes
C-3	Fairway and Sarazan	Yes
C-4	San Christopher and Bass	Yes
C-5	Dunedin Isles	Yes
C-6	Orangewood and Douglas	Yes
C-6A	Richmond and Highland	Yes
C-6B	Highland and Union	Yes
C-7	San Mateo Drive	Yes
C-8	Amberlea Subdivision	No[3]
C-9	Hillside Park	No[2]
C-10	Lake Suemar and Patricia	Yes
C-11	Heather Drive	Yes
C-12	Lakewood Estates	Yes
C-12A	Manor Drive South	Yes
C-13	Brady Road Bridge	No[4]
C-14	St. Catherine Drive	Yes

[1] Pollution controls on coastal outfalls have not been constructed Citywide; however, these improvement measures remain valid options.
[2] This area currently not identified as flood prone.
[3] This project should remain under consideration as the Amberlea Subdivision contains flood prone areas.
[4] The flooding issue at Brady Drive is currently under investigation as part of a separate project

10.1 STRUCTURAL BMPS

Table 10-2 lists the projects discussed in this section that are all effective at providing reduced flood stages, water quality improvements, or natural systems improvements, with several projects providing multiple benefits.

Table 10-2 BMP Projects

Rank	Name	Flood Control	Water Quality	Natural Systems
1	Buena Vista Drive Drainage Improvements	X		
2	San Charles Drainage Improvements	X	X	
3	Santa Barbara Drive Drainage Improvements	X		
4	Palm Boulevard and Douglas Avenue Drainage Improvements	X		
5	Stirling Links Drainage Improvements	X	X	X
6	Main Street Drainage Improvements	X		
7	Michigan Boulevard Drainage Improvements	X		
8	Lyndhurst Street Drainage Improvements		X	X

Several meetings were held during this study where City staff directed Jones Edmunds' attention to drainage issues around the City. This list of issues was supplemented with the identified LOS deficiencies (see Section 9) and BMPs that had been identified in the earlier City-wide Stormwater Master Plan. In selecting the eight locations for BMP development, Jones Edmunds considered the severity of the issue together with the ease with which the issue could be resolved. To find solutions we conducted modeling on considerably more than eight locations keeping an account of how the various potential projects compared to each other in terms of potential flooding impacts and the relative benefits verses the magnitude of the effort to complete the project. Some potential project ideas were abandoned because of feasibility concerns; for example, a water quality improvement pond had been contemplated just south of San Christopher Drive and Patricia Avenue but was abandoned due to concerns over subsidence at the proposed project site. Because of this approach, a certain amount of project ranking is built into the approach – in other words, a non-feasible or low-benefit-per-dollar project did not make the final list of eight projects. Therefore, all eight projects are feasible and provide adequate benefits relative to the magnitude of the project.

While the eight BMPs provide targeted flood relief addressing specific LOS issues within the City, most of the City's structures and roadways that are at risk for flooding are due to larger regional drainage issues mainly caused by the limited capacity in natural creeks and streams or more local issues caused by low topographic relief in areas where building at grade would not be allowed today. These areas would require very large capital investment to reduce the flood risk. The suggested citywide backflow prevention, which can be phased in as sea level rises, will also provide flood relief for areas suffering from tide-induced flooding.

All structural BMPs were tested for how well they improved LOS violations. We conducted the evaluations for LOS improvement using the 100-year storm for structural flooding and evacuation routes, as well as the 50-year and 10-year storms as appropriate, depending on the type of roadway. In cases where the project does not solve the LOS, benefits are reported in terms of the highest return period storm that does not flood the roadway. In most cases, where the BMP was directed at roadway LOS violations, the LOS violation was solved. The projects were tested for downstream impacts using the 25-year storm, consistent with permitting requirements. All storms tested have a duration of 24 hours. Figure 10 shows an overview of the BMP project locations. Jones Edmunds developed construction cost estimates for each of the conceptual projects presented in this section. These opinions of probable cost are presented in Appendix F and Appendix G provides a BMP ranking matrix.

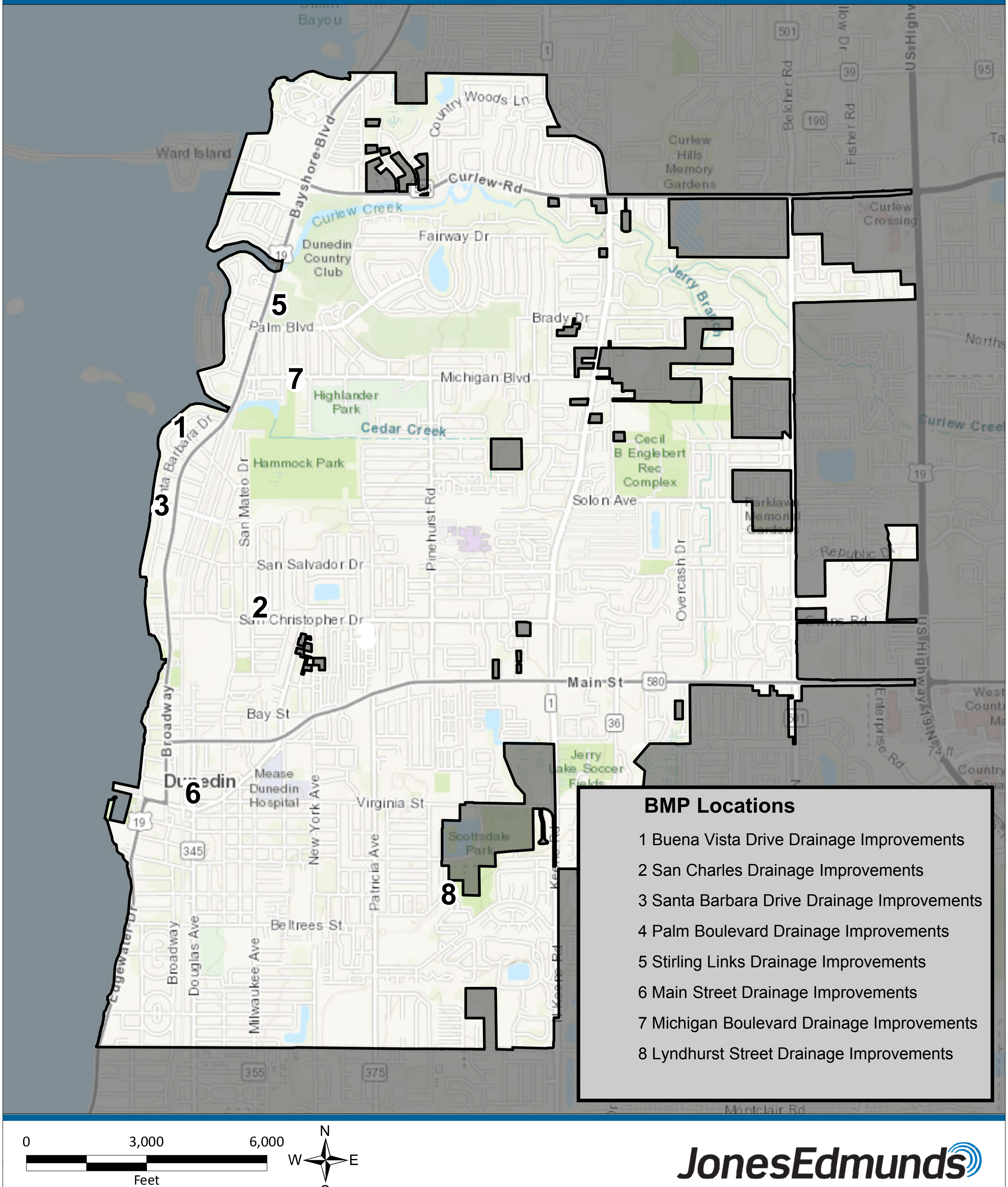
The eight BMP projects are described in the following sections. Each project is independent of the other projects, and no sequencing is required. A figure illustrating the project concept and benefits is provided for each project.

Photograph 10-1 Marina Near the City of Dunedin

Figure 10

BMP Locations

City of Dunedin Stormwater Master Plan



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10.1.1 Buena Vista Drive Drainage Improvements

Runoff from approximately 15 acres is collected along Buena Vista Drive and outfalls to Clearwater Harbor North via two separate drainage systems. The first system outfalls via two 15-inch reinforced concrete pipes (RCPs) approximately 200 feet northwest of the intersection of North Buena Vista Drive and Santa Barbara Drive. The second system outfalls through a 15-inch RCP northeast of the first system. Model results indicate that large portions of Buena Vista Drive and Mira Vista Drive do not meet the set LOS for local roadways. In addition to rain-induced flooding, tide levels occasionally exceed roadway grades leading to street flooding. As sea levels continue to rise, coastal communities like Dunedin have experienced increased sunny day flooding events. The proposed project recommends connecting the two outfall systems to better distribute stormwater flows, increase the capacity of the outfall pipes, and provide backflow prevention at each outfall. Figure 10 -1 illustrates this proposed BMP.

The following improvements are included as a part of this project:

1. Install new 18-inch RCP along Buena Vista Drive.
1. Replace two existing 15-inch outfall pipes with two 18-inch outfall pipes.
2. Install backflow preventers on both outfall systems; two 18-inch and one 15-inch backflow preventers are required.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $302,300.

Project Benefits

The proposed project lowers the stage about 0.3 foot in the project area and removes eight structures from the 100-year riverine floodplain along or adjacent to Buena Vista Drive. The proposed conditions model results show that water surface elevations (WSEs) are lowered during the 10-year 24-hour storm enough to meet the prescribed LOS for all local roads in the project area. Installing backflow preventers will also reduce the potential for sunny day flooding in the project area. This project ranks 1 out of the 8 projects considered.

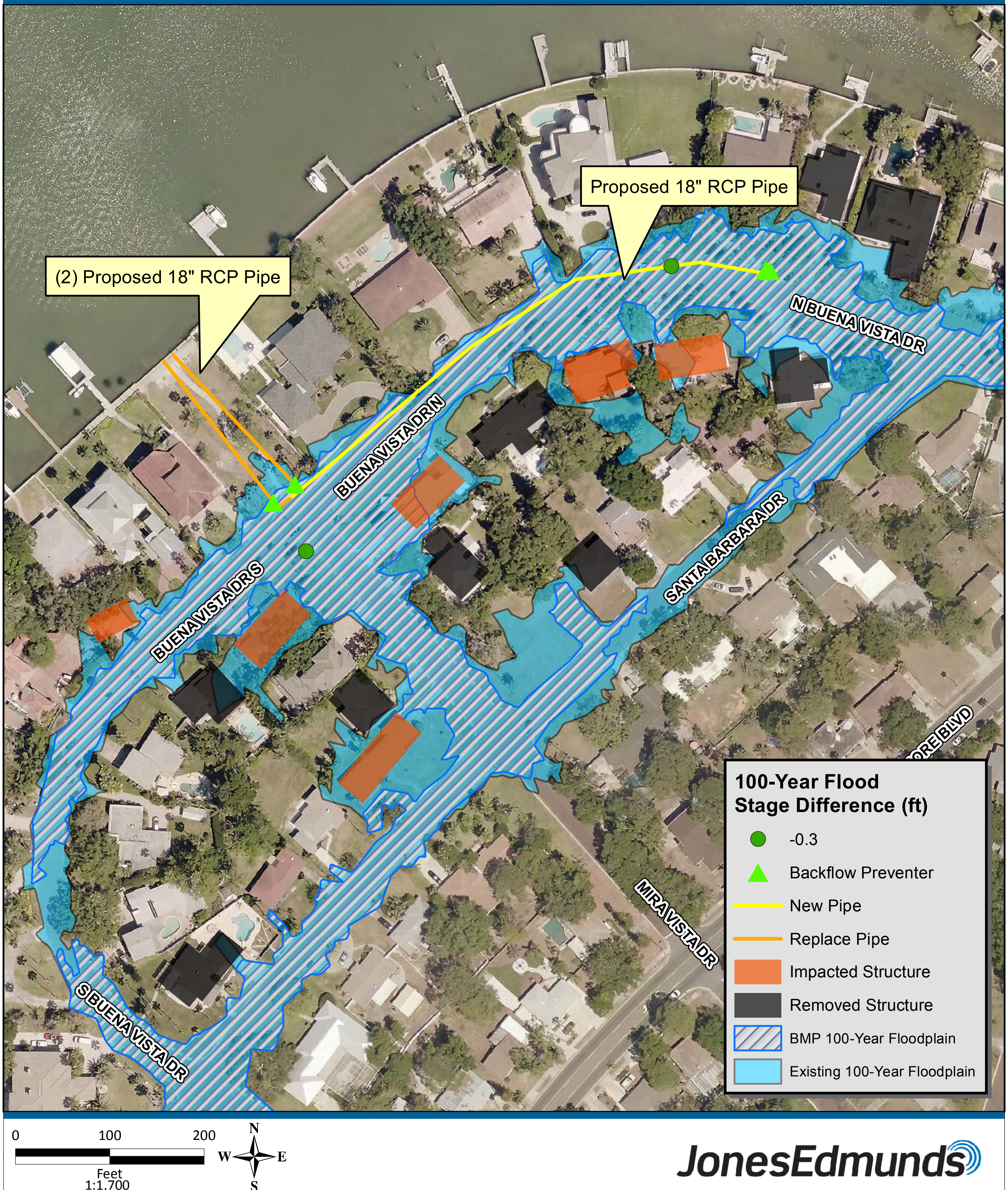
Project Considerations

The proposed improvements are inside the existing ROW or on a City-owned park, where the two new outfall pipes are proposed. An ERP will be required for the proposed project. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required. While not specifically included in this project the design engineer should consider ways to improve water quality as a part of the project.

Figure 10-1

Buena Vista Drive Drainage Improvements

City of Dunedin Stormwater Master Plan



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10.1.2 San Charles Drive Drainage Improvements

Runoff from approximately 13 acres is collected and conveyed via roadway curb and gutters toward two inlets at the intersection of San Charles Drive and San Roy Drive South. The flow is then routed to the drainage system along San Christopher Drive via a 15-inch RCP. Model results indicate that San Charles Drive and San Roy Drive South do not meet the 10-year LOS set for local roadways. This project proposes increasing the capacity of the outfall for this drainage basin (Figure 10-2).

The following improvements are included as a part of this project:

1. Replace the existing 15-inch RCP with new 24-inch RCP.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $103,400.

Project Benefits

This project lowers WSE along San Charles Drive and San Roy Drive South enough to meet the prescribed LOS. The proposed project lowers the stage for one node 0.8 foot. Although four nodes show increases in stage, these increases are contained in the existing drainage system along San Christopher Drive and do not result in adverse impacts downstream of the project area. This project ranks 2 out of the 8 projects considered.

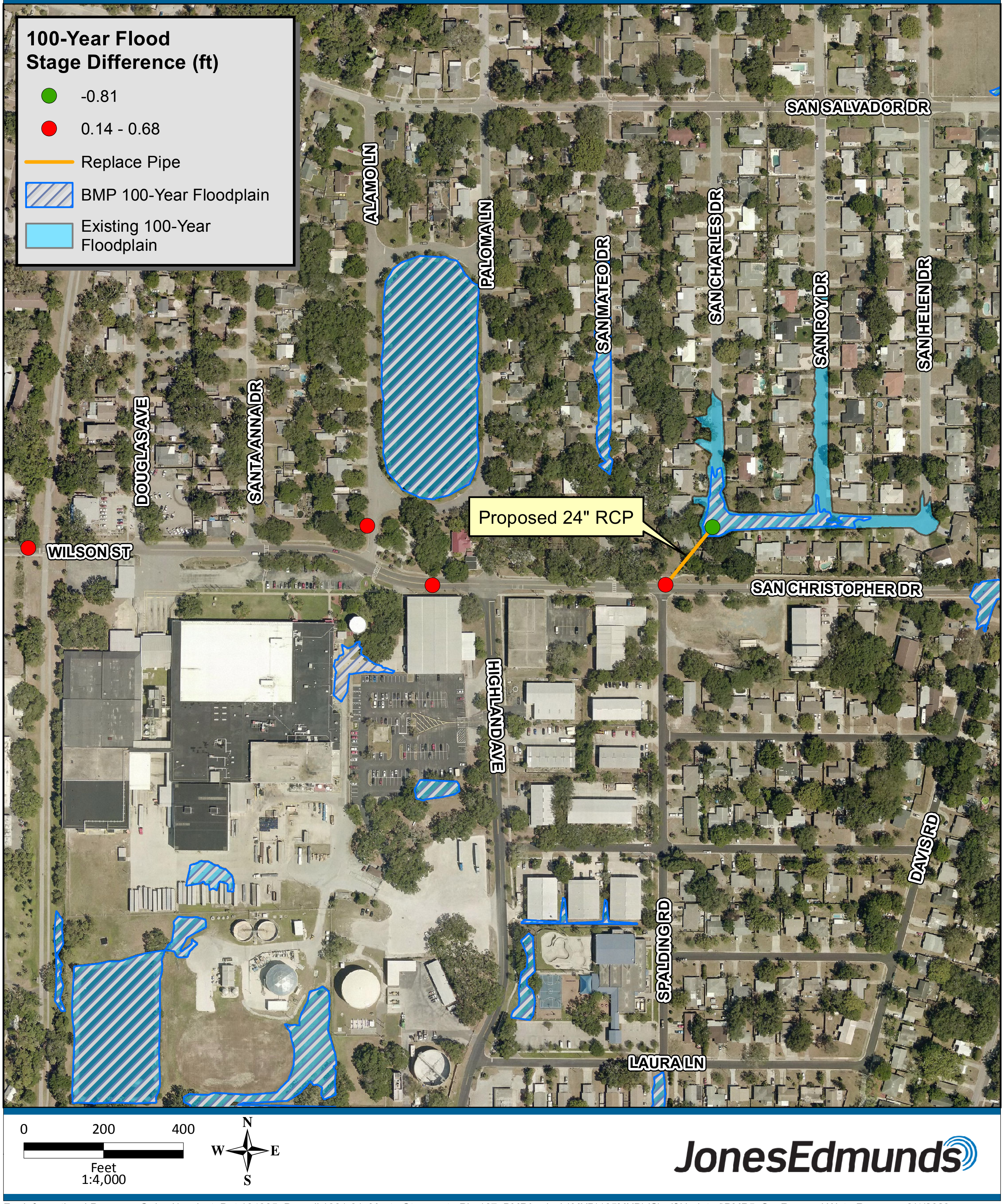
Project Considerations

If the 15-inch pipe is located in an easement, then the proposed 24-inch pipe will very likely fit within the existing easement; however, new or expanded permanent easements may be required. An ERP may be required for the proposed project. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required. While not specifically included in this project the design engineer should consider ways to improve water quality as a part of the project.

Figure 10-2

San Charles Drive Drainage Improvements

City of Dunedin Stormwater Master Plan



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10.1.3 Santa Barbara Drive Drainage Improvements

Runoff from approximately 34 acres is collected along Santa Barbara Drive and outfalls to Clearwater Harbor North via four separate drainage systems along Santa Barbara Drive. The first (northernmost) system outfalls through a 15-inch RCP approximately 150 feet south of the intersection of South Buena Vista Drive and Santa Barbara Drive. Moving south, the second system outfalls through an 18-inch lined CMP pipe on the west side of the intersection of Cevera Drive and Santa Barbara Drive. The third system outfalls through a 24-inch RCP and a 30-inch RCP that outfall at San Jose Park, a small City-owned park along Santa Barbara Drive. The fourth system outfalls via a 15-inch lined CMP approximately 450 feet south of the park. Model results in this area indicate the Santa Barbara Drive, South Buena Vista Drive, and Cevera Drive do not meet the LOS for local roadways. Sunny day flooding due to high tides has also been reported in the systems along Santa Barbara Drive. The proposed project recommendation is to connect the four outfall systems, increase the capacity of the two outfall pipes that convey flow through San Jose Park, and provide backflow prevention.

Figure 10-3 illustrates this BMP project. The following improvements are included as a part of this project:

1. Install new 24-inch RCP along Santa Barbara Drive from approximately 150 feet south of the intersection of South Buena Vista Drive and Santa Barbara Drive to Cevera Drive.
2. Install new 29- x 45-inch Elliptical Reinforced Concrete Pipe (ERCP) along Santa Barbara Drive from Cevera Drive to San Jose Park.
3. Install new 24-inch RCP running along Santa Barbara Drive from approximately 450 feet south of the entrance to San Jose Park to the entrance of San Jose Park.
4. Replace one 24-inch RCP and one 30-inch RCP with two 34- x 53-inch ERCPs from the entrance of San Jose Park to Clearwater Harbor North.
5. Install backflow preventers on all outfall systems.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $1,035,300.

Project Benefits

This project lowers stages along Santa Barbara Drive and adjacent areas by 0.1 foot to 1.26 feet and removes three structures from the 100-year floodplain. The proposed conditions model results show that WSEs are lowered during the 10-year 24-hour storm enough to meet the prescribed LOS for all local roads in the project area. The backflow preventers should eliminate sunny day flooding in the project area. This project ranks 3 out of the 8 projects considered.

Project Considerations

The proposed improvements are planned inside the existing ROW and City-owned park. An ERP will be required for the proposed project. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required. While not specifically included in this project the design engineer should consider ways to improve water quality as a part of the project. While not specifically included in this project the design engineer should consider ways to improve water quality as a part of the project.

Figure 10-3

Santa Barbara Drive Drainage Improvements

City of Dunedin Watershed Management Plan

JonesEdmunds

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10.1.4 Palm Boulevard and Douglas Avenue Drainage Improvements

The Palm Boulevard and Douglas Avenue Drainage Improvements project concept was developed as part of the Curlew Creek/Smith Bayou WMP. Street flooding has historically been an issue at the intersection of Palm Boulevard and Douglas Avenue. Palm Boulevard is classified as an arterial road and serves as the primary west exit for the Fairway Estate community. Douglas Avenue is the primary route available leading to an evacuation route for the Fairway Estates area of Dunedin. Model results indicate that two structures along Douglas Avenue are LOS deficient, and both Palm Boulevard and Douglas Avenue are LOS deficient. The proposed project, shown in Figure 10-4 recommends increasing the capacity under Palm Boulevard and the system that conveys flow to the swale parallel to the Pinellas Trail that flows directly to Curlew Creek.

The following improvements would increase the size of the conveyance facilities: Replace the 19-inch-x-30-inch RCP with new 29-inch-x-45-inch RCP under Palm Boulevard. Replace the two 19-inch-x-30-inch RCPs with two new 29-inch-x-45-inch RCPs from the north side of Palm Avenue to the channel parallel to the Pinellas Trail.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $709,400.

Project Benefit

This project lowers the stage by 0.48 foot along Palm Boulevard and by 0.42 foot along Douglas Avenue. The proposed project removes two structures from the 100-year floodplain and provides the prescribed LOS for Palm Boulevard and Douglas Avenue.

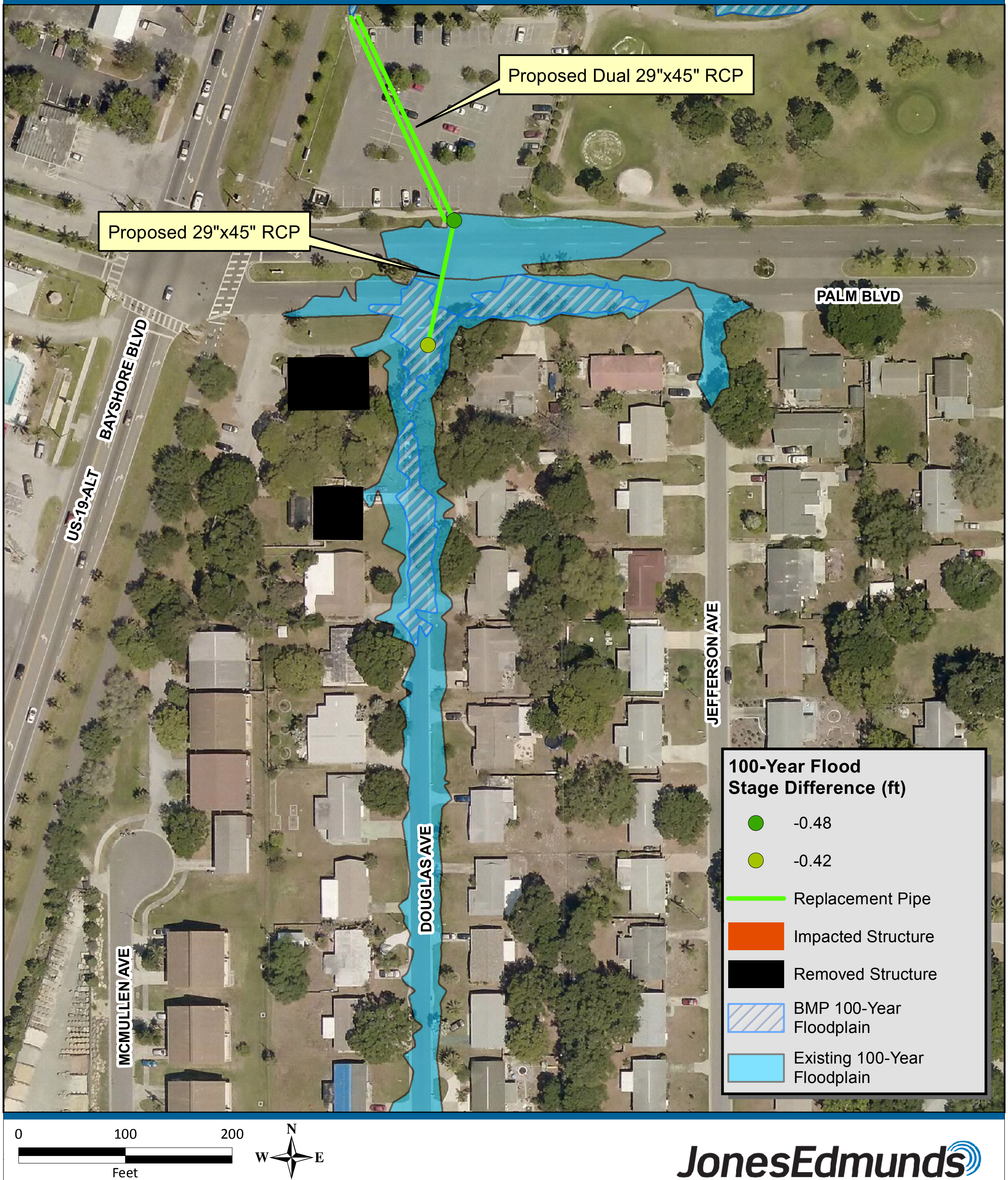
Project Considerations

The proposed improvements are planned within the existing ROW; however, new or expanded permanent easements may be required. An ERP permit will be required for the proposed project, and although model results show no flood impacts, a preapplication meeting with SWFWMD is recommended. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer as required. This project ranks 4 out of the 8 projects.

Figure 10-4

Palm Blvd. and Douglas Ave. Drainage Improvements

City of Dunedin Stormwater Master Plan



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10.1.5 Stirling Links Drainage Improvements

The City of Dunedin is redeveloping the 27-acre, 18-hole Stirling Links Golf Course into a park. Stirling Links is at the northeast corner of the intersection of Palm Boulevard and US Alt 19. Street flooding has historically been an issue at the intersection of Palm Boulevard and Douglas Avenue. Palm Boulevard East of ALT 19 is classified as an Arterial Road and serves as the primary exit for the Fairway Estates community. Douglas Avenue is classified as a Local Road between Palm Boulevard and Michigan Boulevard. Model results indicate that two structures along Douglas Avenue are LOS deficient, and both Palm Boulevard and Douglas Avenue are LOS deficient. Redeveloping Stirling Links provides an opportunity to solve the flooding at the intersection of Palm Boulevard and Douglas Avenue and to also provide water quality improvements. Figure 10-5 shows the proposed project recommends constructing two ponds on the Stirling Links site while preserving the driving range and other ancillary facilities. The project also increases the capacity of the system under Palm Boulevard.

The following improvements are included as a part of this project:

1. Construct a 4.2-acre pond for attenuation and water quality improvement.
2. Construct a 0.75-acre pond for water quality improvement.
3. Replace the existing 19- x 30-inch RCP under Palm Boulevard with new 34- x 53-inch RCP.
4. Replace existing double 19- x 30-inch outfall pipes with new double 34- x 53-inch RCP.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $2,009,800.

Project Benefits

Stages in the project area are lowered from 0.36 to 0.83 foot during the 100-year 24-hour storm. The proposed project removes two structures from the 100-year floodplain and provides the prescribed LOS for Palm Boulevard and Douglas Avenue. Together the ponds provide treatment for about 37 acres and are expected to treat approximately 22 acre-feet of stormwater on an annual average basis, resulting in the pollutant removals shown in Table 10-3 and Table 10-4. This project ranks 5 out of the 9 projects considered.

Table 10-3 Annual Average Load (Pounds) at 0.75-acre Pond

Pollutant	Load Received	Load Removed
TN	58	25
TP	11	10
TSS	1,110	999
BOD	86	86

Table 10-4 Annual Average Load (Pounds) at 4.2-acre Pond

Pollutant	Load Received	Load Removed
TN	201	87
TP	38	34
TSS	5,608	5,047
BOD	774	774

Project Considerations

Portions of this project (BMP#6) overlap with the Palm Boulevard and Douglas Avenue Drainage Improvements (BMP#5) presented in Section 10.1.5. Both projects have similar flood control benefits but BMP#5 does not include water quality improvement benefits. Additionally, pipe sizes are larger for BMP#6 since BMP#6 discharge to a higher tailwater because of the pond.

The 0.75-acre pond is located to capture and provide treatment for the runoff from 10.8 acres that is currently not treated and discharges directly to Curlew Creek. The 4.2-acre pond is sized to handle additional runoff and could be incorporated into larger-scale improvements at the golf course that could include additional recreational and environmental education opportunities. During the Curlew Creek WMP project a larger-scale regional stormwater management area was considered that would include an offline treatment facility for Curlew Creek; however, this regional facility would not be feasible on the pond site currently proposed as part of BMP#6 because the larger pond is not located in the lower areas of the Stirling Links site. The current proposed pond site is situated to minimize conflicts with existing and future amenities that may be planned by the City; however, as plans are refined over time the pond siting should be reevaluated to determine if the regional facility can be accommodated.

If constructed in its currently proposed location and without these additional drainage improvements to route more runoff to the pond, then the pond size could be limited to around 2 acres and still provide similar water quality benefits; however, final pond sizing should also consider fill material needs that could be used to provide flood protection along the southern bank of Curlew Creek to reduce nuisance flooding to the Dunedin Golf Club.

The proposed improvements are planned within property currently owned by the City or in the existing ROW. An ERP will be required for the proposed project, and although model results show no flood impacts, a preapplication meeting with SWFWMD is recommended. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required. Figure 10-4 partially shows a City-owned 2.5-acre parcel adjacent to Curlew Creek. Although not a part of the proposed project described herein, the potential exists to use this area for environmental improvement and education, depending on the City's redevelopment plan. Unlike the proposed project, which maintains current site recreational activities with space for additional features, the 2.5-acre site would need to be wholly dedicated to environmental improvement to fit meaningfully sized facilities on the site.

Figure 10-5
Sterling Links
Drainage Improvements
City of Dunedin
Watershed Management Plan

100-Year Flood Stage Difference (ft)

- -0.83
- -0.78
- -0.45
- -0.36
- New Pipe
- Replace Pipe
- Impacted Structure
- Removed Structure
- BMP 100-Year Floodplain
- Existing 100-Year Floodplain
- Sterling Links Golf Course

N E S W

0 200 400
Feet
1:4,000

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10.1.6 MAIN STREET DRAINAGE IMPROVEMENTS

Runoff from approximately 78 acres, much of it originating from State Road (SR) 580, is collected and conveyed along Main Street through downtown Dunedin. The storm sewer along Main Street increases in size as it crosses downtown. When the system reaches Douglas Avenue, the flow is picked up in two 30-inch RCPs along Main Street and a 48-inch system that runs north along Douglas Avenue to Monroe Street. The flow is then conveyed along Monroe Street in a 48-inch RCP to the intersection of Monroe Street and US Alt 19 where the pipe size increases to a 54-inch RCP that carries the flow to Clearwater Harbor North. Model results indicate that numerous structures are inundated during the 100-year/24-hour storm event along the Main Street corridor between Park Street and Douglas Avenue. A section of Main Street, an Arterial Roadway, is also LOS deficient during the 50-year/24-hour storm. Figure 10-6 shows the proposed project includes additional capacity along Grant Street and increases the capacity of the existing system along Monroe Avenue. The following improvements are included as a part of this project:

1. Install new 36-inch RCP along Grant Street.
2. Install new 42-inch RCP along Highland Avenue and Grant Street.
3. Replace the existing 54-inch RCP along Douglas Avenue with 58- x 91-inch ERCP.
4. Replace the existing 54-inch RCP along Monroe Street with 58- x 91-inch ERCP.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $2,289,000.

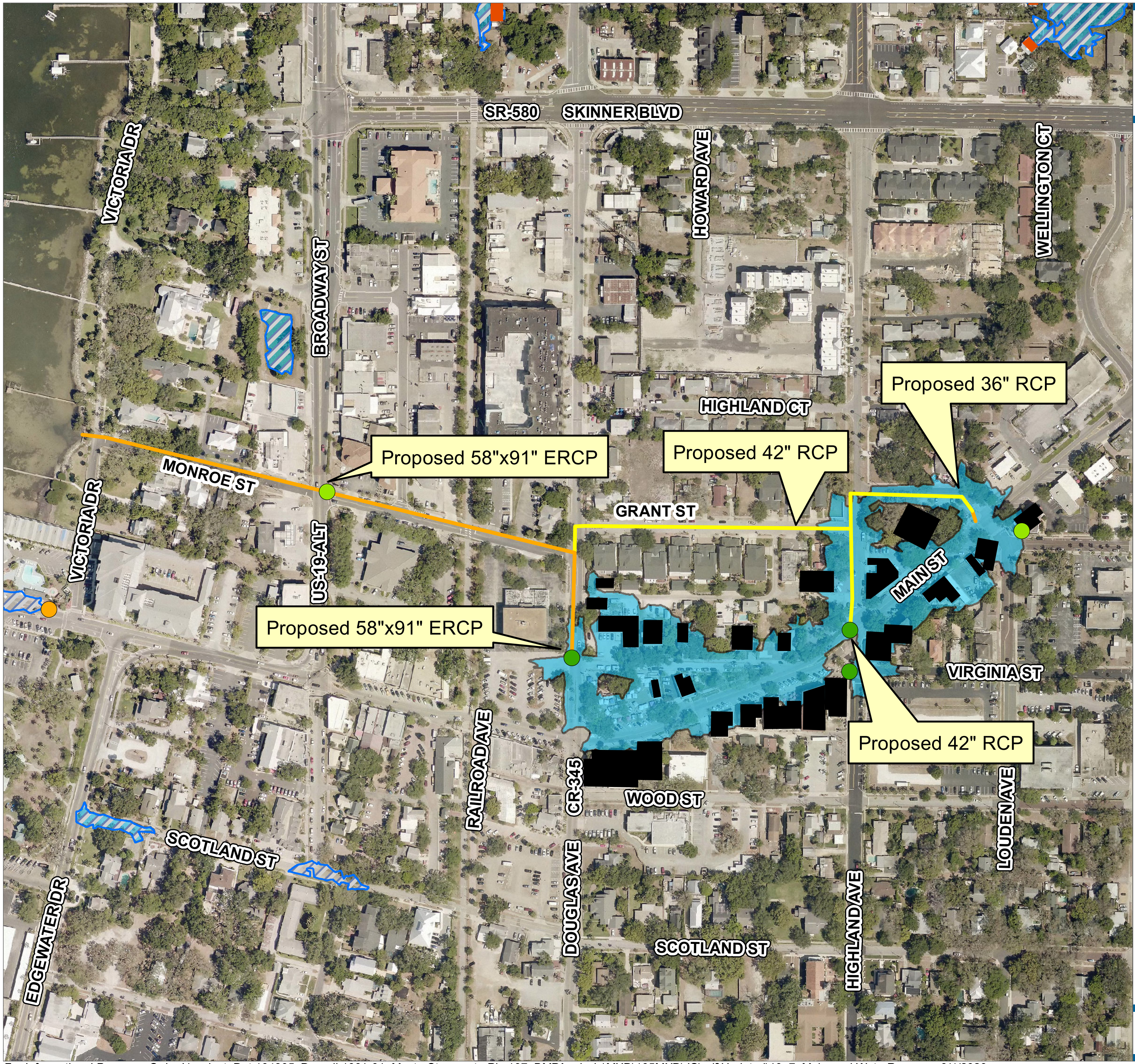
Project Benefits

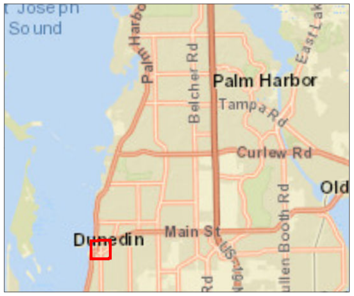
This project lowers stages for eight nodes ranging from 0 foot to 3.34 feet along the Main Street corridor. The proposed improvements remove 23 structures from the 100-year floodplain along or adjacent to Main Street. The recommended improvements also reduce WSEs along Main Street during the 50-year/24-hour storm enough that Main Street provides the prescribed LOS. This project ranks 6 out of the 8 projects considered.

Project Considerations

Due to the potential for disruptions to businesses that may occur because of this project, a more rigorous evaluation concerning the cost versus the benefits should be undertaken before implementing this project. The proposed improvements are generally planned within the existing ROWs; however, a new or expanded permanent easement may be required at the outfall where the project exits the ROW west of Victoria Drive. An ERP will be required for the proposed project, and although model results show no flood impacts, a pre-application meeting with SWFWMD is recommended. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required. During the Preliminary Engineering phase of the project, the City may consider including CDS units into the design to improve water quality. The City may also wish to consider installing an exfiltration system inside Pioneer Park and\or along Grant Street between Highland Avenue and Douglas Avenue to take advantage of the well-drained soils in this area for water quality improvement.

Figure 10-6
Main Street
Drainage Improvements
City of Dunedin
Stormwater Master Plan





100-Year Flood Stage Difference (ft)

- -3.34 - -3.14
- -1.66 - -1.34
- -0.12
- New Pipe
- Replacement Pipe
- Impacted Structure
- Removed Structure
- BMP 100-Year Floodplain
- Existing 100-Year Floodplain

N S W E

0 200 400
Feet
1:4,000

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10.1.7 Michigan Boulevard Drainage Improvements

Model results indicate that Michigan Boulevard from Yale Avenue and Woodward Avenue does not meet the prescribed LOS for an Arterial Road. Runoff from approximately 65 acres is collected and conveyed via the drainage system in the south ROW for Michigan Boulevard that discharges to Hammock Park. Figure 10-7 shows the project proposes installing a secondary collection in the north ROW along Michigan Boulevard that conveys flow to Cedar Creek.

The following improvements are included as a part of this project:

1. Install new 24-inch pipe along Michigan Boulevard from Yale Avenue to Harvard Avenue.
2. Install new 29- x 45-inch ERCP along Boulevard Avenue from Harvard Avenue to Woodward Avenue.
3. Install two new 24- x 38-inch ERCPs along Michigan Boulevard from Woodward Ave to US Alt 19.
4. Replace the existing 18-inch pipe under Michigan Boulevard with 34- x 53-inch ERCP.
5. Install new 34- x 53-inch ERCP from south of Michigan Boulevard along US Alt 19 to Cedar Creek.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $2,247,600.

Project Benefits

This project lowers stages for seven nodes ranging from 1.06 feet to 0.15 foot along Michigan Boulevard and adjacent areas. The proposed improvements lower WSEs enough to meet the 50-year LOS for Arterial Roads.

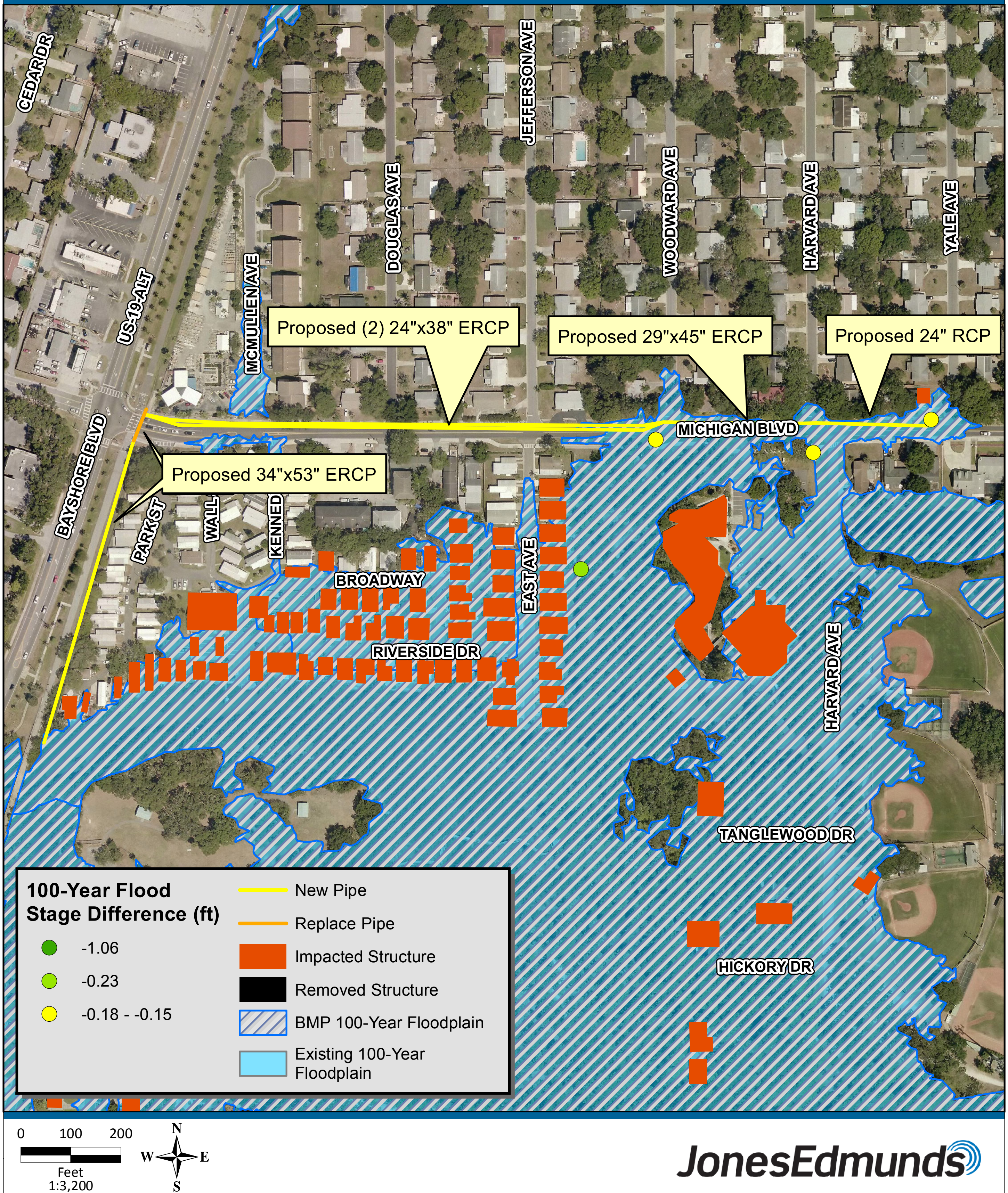
Project Considerations

The proposed improvements are shown inside of the existing ROW, but new or expanded permanent easements may be required. An ERP will be required for the proposed project, and although model results show no flood impacts, a pre-application meeting with SWFWMD is recommended. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer as required. While not specifically included in this project the design engineer should consider ways to improve water quality as a part of the project. This project ranks 7 out of the 8 projects considered.

Figure 10-7

Michigan Avenue Drainage Improvements

City of Dunedin Stormwater Master Plan



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10.1.8 Lyndhurst Street Drainage Improvements

Lyndhurst Street serves as the main point of access to three single-family residences northeast of the intersection of Cedarwood Drive and Lyndhurst Street. An open-channel system currently conveys runoff to four 43-inch x 45-inch corrugated metal pipes (CMPs) approximately 200 feet east of the intersection of Cedarwood Drive and Lyndhurst Street. During the field reconnaissance portion of this project, these pipes were observed to be severely rusted and creating the potential for a safety hazard. Also, erosion and sedimentation has historically been an issue within the City's streams and for the channel system downstream of Lyndhurst Street. Figure 10-8 shows that this project replaces the existing CMP pipes with a concrete box culvert and provides a sediment trap downstream of Lyndhurst Street.

The following improvements are included as a part of the project:

1. Replace four existing 43-inch x 45-inch CMPs under Lyndhurst Street with a new 4-foot x 8-foot concrete box culvert (CBC).
2. Construct a 0.3-acre sediment trap.

A detailed opinion of probable construction cost is provided in Appendix F. The estimated probable construction cost for this project is $143,500.

Project Benefits

This project eliminates the safety hazard caused by the deteriorating culvert crossing at the residential access point and provides water quality improvement. Modeling results show that the proposed project results in no changes in stage upstream or downstream of the project area, indicating that the proposed CBC provides the capacity to convey the flow in the system without causing adverse impacts upstream or downstream of the proposed project. With a capacity of around 700 CY, the sediment sump will capture a portion of the bedload in Spring Branch's primary tributary. This project ranks 8 out of the 8 projects considered.

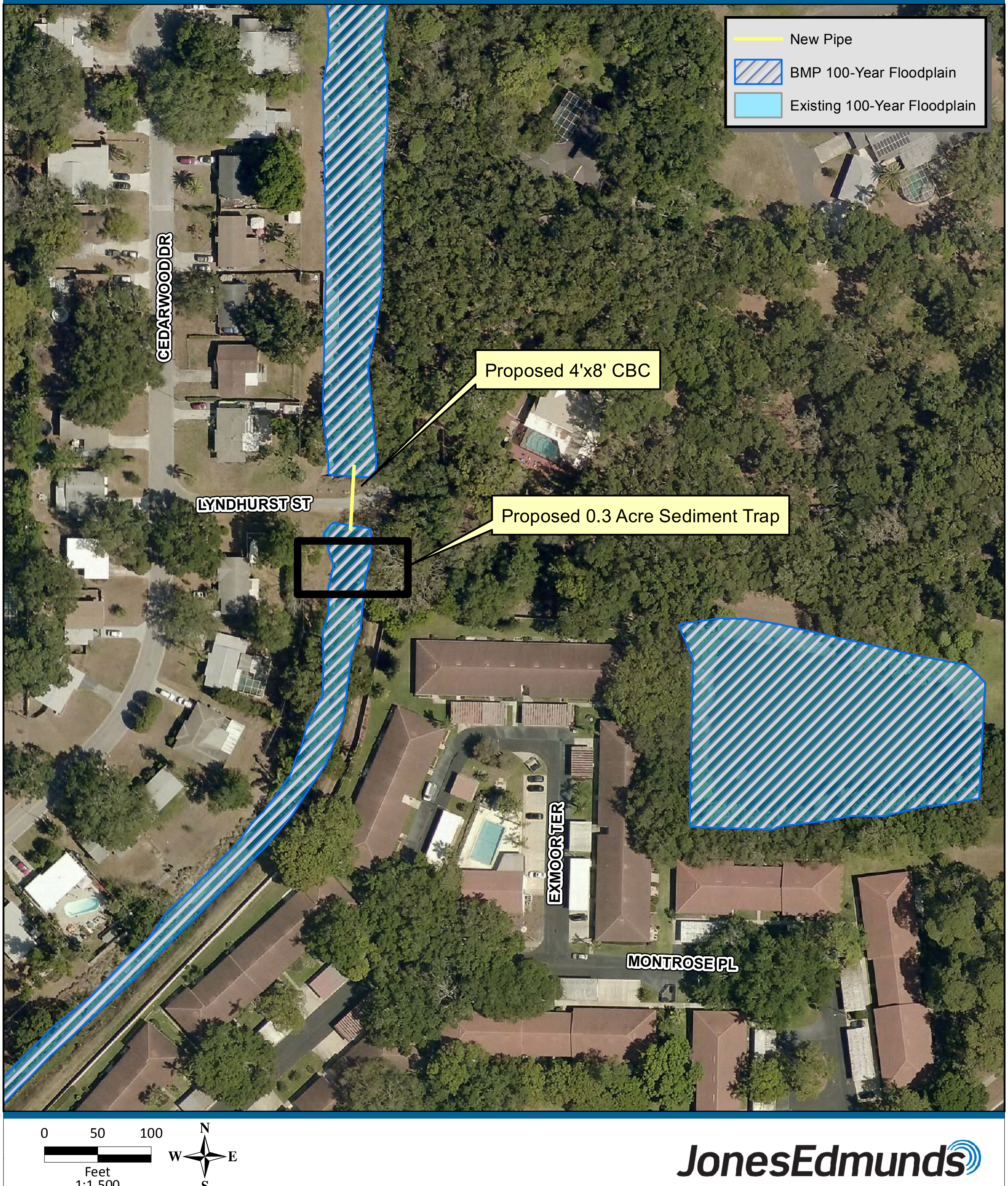
Project Considerations

The proposed CBC is planned in the existing right-of-way (ROW), and the sediment sump is on a small City-owned parcel (outlined in blue on Figure 10-8); however, a construction easement may be needed to construct the sediment sump. One edge of the sediment sump parcel should be reserved for maintenance access, and the sediment sump weir should be designed to maintain flood flows without increases to stage. An ERP will be required for this project, and although model results show no flood impacts, a preapplication meeting with SWFWMD is recommended. During preliminary design, the design engineer should review all appropriate soil considerations such as stable subgrade, bearing capacity, groundwater conditions, and contamination and seek recommendations from a geotechnical engineer or other professional as required.

Figure 10-8

Lyndhurst Street Drainage Improvements

City of Dunedin Stormwater Master Plan



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10.2 Program Recommendations

In addition to the nine structural BMPs recommended in the previous subsections, two program recommendations are included – LID practices applicable to City-owned facilities and backflow prevention devices to reduce sunny-day flooding. These program recommendations are described in the following sections.

10.2.1 LID on City-Owned Facilities

This BMP considers implementing LID practices on existing City-owned facilities and ROWs, where practical. As the first City in the State to achieve Platinum Level as a Certified Green Local Government, Dunedin is a leader in taking action to ensure an environmentally sustainable future. LID practices are environmentally sustainable systems that use or imitate natural processes resulting in infiltration, evapotranspiration, or stormwater reuse to reduce runoff and protect water quality. The US Environmental Protection Agency (EPA) summarizes LID as follows:

> *LID is an approach to land development (or re-development) that works with nature to manage stormwater as close to its source as possible. LID employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product. There are many practices that have been used to adhere to these principles such as bioretention facilities, rain gardens, vegetated rooftops, rain barrels and permeable pavements. By implementing LID principles and practices, water can be managed in a way that reduces the impact of built areas and promotes the natural movement of water within an ecosystem or watershed. Applied on a broad scale, LID can maintain or restore a watershed's hydrologic and ecological functions. (EPA, 2020)*

Conventional stormwater controls typically control and treat runoff using a single stormwater pond at the hydraulic low point of the site; whereas, LID systems promote volume attenuation and treatment nearer to the stormwater runoff sources by using stormwater retention, detention, infiltration, treatment, and harvesting systems. One goal of LID is to improve the overall effectiveness and efficiency of stormwater management relative to conventional systems. These improvements include lower peak runoff rates, lower runoff volumes, and lower pollutant loads discharged from the site. Allowing less stormwater to leave the site also removes a part of the burden from downstream stormwater systems, thus benefiting regional drainage.

Numerous guidance documents and information are available covering the various site planning, design criteria, and other information needed to incorporate LID into site development or redevelopment. Examples include the *Pinellas County Stormwater Manual* (Pinellas County, 2017), Sarasota County *Low Impact Development Guidance Document* (Sarasota County, 2015), and EPA's Green Infrastructure website (https://www.epa.gov/green-infrastructure). The City can use these and other similar references to plan LID controls that can be added on existing City-owned facilities and ROWs.

Many LID practices are geared toward promoting infiltration. LID practices that promote infiltration reduce stormwater generation at the source by preserving and promoting opportunities for infiltration on site. Pollutants contained in the volume of runoff that infiltrates are completely removed from surface water; therefore, removal efficiencies for infiltration practices are calculated as the percent of runoff that infiltrates. Infiltration also provides for groundwater recharge, which adds a number of benefits including increased groundwater supply, protecting aquatic habitats, reducing saltwater intrusion, among others. Some of the infiltration-based LID practices appear to be some of the more cost-effective strategies to incorporate LID into City-owned properties.

Soil conditions are an important consideration when selecting LID practice that incorporate infiltration. Soils within the City of Dunedin consist primarily of four of the hydrologic soil groups, as classified by NRCS: Group A (well-drained), C (somewhat poorly drained), B/D (moderately well-drained when dry, not well-drained when wet), and D (poorly drained). The soils are listed in order of suitability for infiltration practices, with Group A being the most suitable and Group D being the least suitable. Figure 4-3 provides a mapping of these hydrologic soil groups. The B/D hydrologic soil group within the City is classified as such due to a shallow SHWT; therefore, performance of infiltration-dependent LID applications will be constrained under wet conditions in areas with these soil types.

Although potential stormwater infiltration capacity and rates may be constrained by the SHW level and hydrologic soil group, some type of infiltration-dependent LID practices can be designed to perform effectively under most site conditions in the City.

Four infiltration-related practices suitable within the City are described in the following subsections.

10.2.2 Reduce Directly connected Impervious Areas (DCIA)

DCIAs transfer runoff volume and associated loads directly to the outlet without providing an opportunity for infiltration. Disconnecting impervious areas directly from the outlet and allowing flow to occur over pervious areas where infiltration can occur can provide some of the most cost-effective practices to promote infiltration. Examples include redirecting roof downspouts from paved surfaces or using curb cuts to drain parking and driveway areas toward pervious surfaces. Care should be taken not to create erosion problems by redirecting drainage. Although not required, disconnecting impervious areas can be accomplished in conjunction with the other LID practices, where the drainage is directed toward a LID feature – examples include directing downspouts to a rain garden or directing drainage from curb cuts to a grassed swale.

10.2.3 Bioretention Systems

Bioretention systems are shallow depressions used to capture, treat, and infiltrate stormwater runoff.

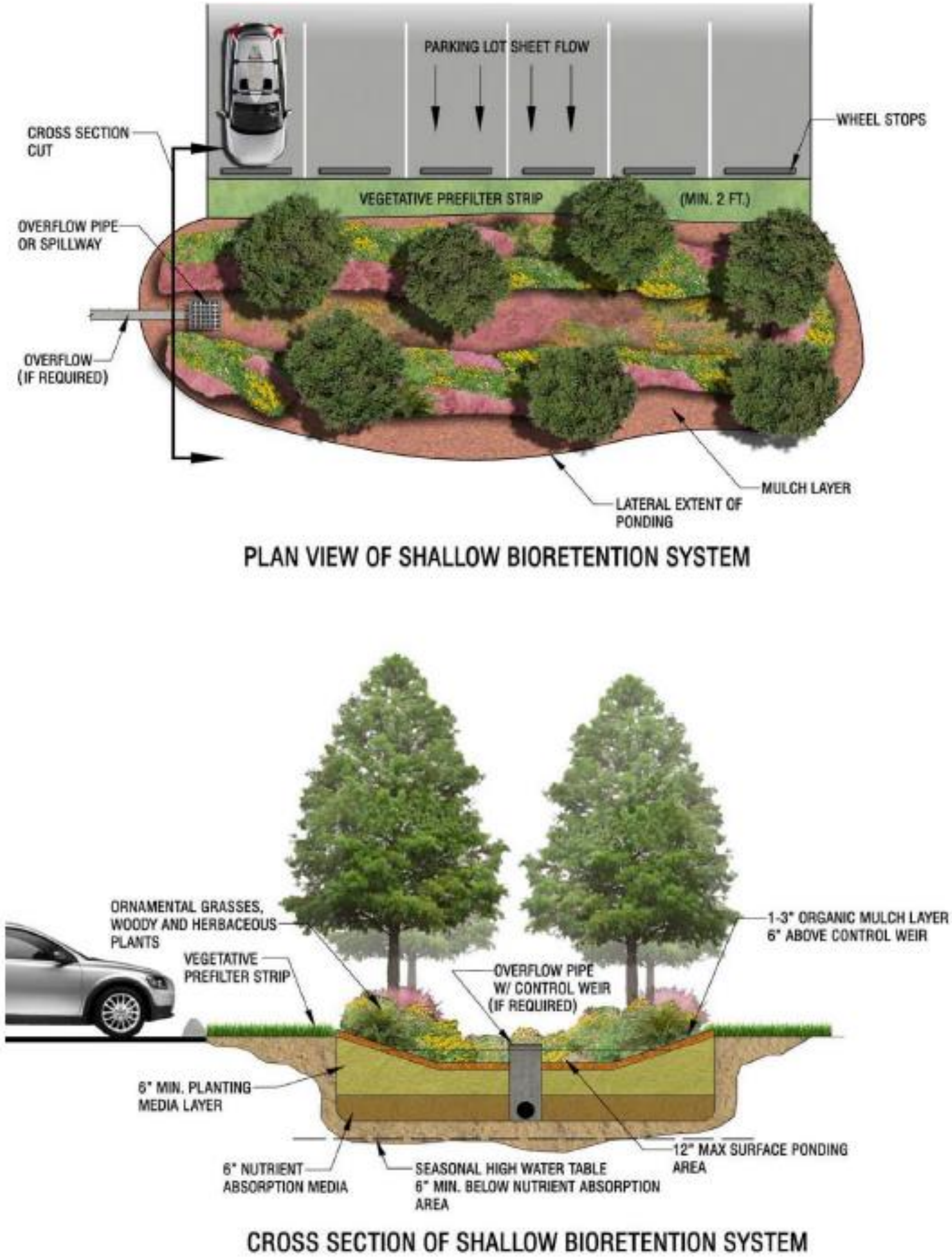
Although bioretention systems can contain optional components to improve infiltration and pollutant removal efficiency, all bioretention systems should contain a:

- Storage area – The storage area consists primarily of a retention volume about 1 foot deep with adequate additional depth to allow overflow to occur only though the dedicated outlet to prevent bank erosion.
- Overflow structure – An inlet, pipe, or spillway allowing rainfall events that exceed the retention volume to bypass the system.

- Organic mulch layer – A 2- to 3-inch layer to promote plant heath as well as to add organic matter to the soil and attenuate heavy metals
- Planting soil/filter bed – A layer providing at least 6 inches of soil for planting as a sorption site for pollutants and a matrix for soil microbes, which aid in nutrient recycling.
- Woody and herbaceous plants – Florida-friendly plants to provide a carbon source, encourage microbial activity, and improve infiltration rates.

Energy-dissipation mechanisms may be needed at the inlet, especially if the retention area is receiving a concentrated inflow. Additionally, a prefilter strip (such as grassed area) between the contributing area and retention area can filter out coarse sediments and reduce erosion potential. Bioretention areas should be constructed at least 6 inches above the SHW level. Figure 10-9 illustrates a typical bioretention area.

Figure 10-9 Cross-Section and Plan View of Bioretention System



Source: Duval County LID Manual, 2013.

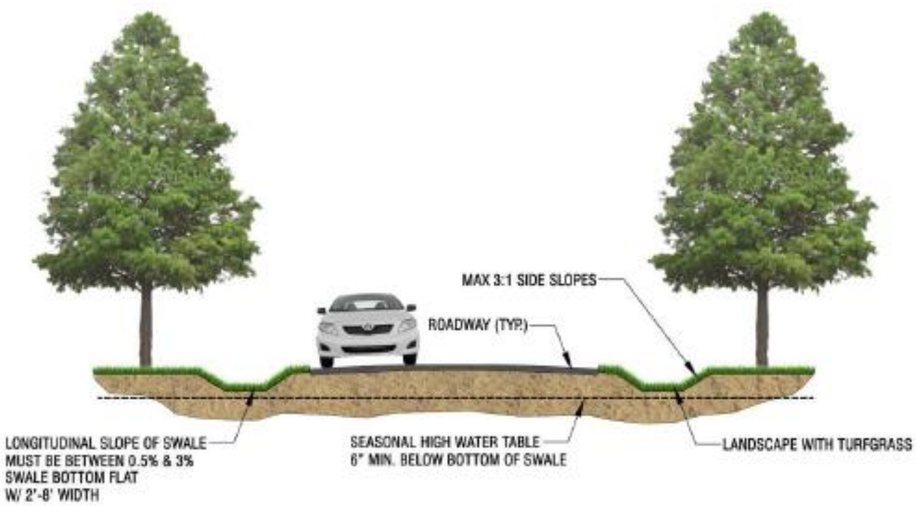
Rain gardens are a type of bioretention system. Although the terms “bioretention basin” and “rain garden” are largely interchangeable (http://buildgreen.ufl.edu/Fact_sheet_Bioretention_Basins_Rain_Gardens.pdf), “rain garden” normally connotates a smaller bioretention area, often on an individual lot; whereas “bioretention basin” is used to describe larger basins. Generally, rain gardens have smaller contributing areas, may be slightly shallower than bioretention basins, and do not typically have overflow.

In cases where the SHW level is too high for bioretention, a similar practice – Detention with Biofiltration – can be used. Detention with biofiltration shares many of the same components as bioretention areas; however, the storage area is separated from the water table by a liner with underdrains used to pull the water through a filter media, typically sand. Both bioretention and biofiltration systems' performance can be enhanced with filter media, such as woodchips or other commercially available products.

10.2.4 Grassed Swale

Grassed swales are the "original" LID practice. Popular as a lower-cost drainage solution in cases where ROW widths allow for their construction, grassed swales also provide for infiltration and all the associated benefits such as runoff and pollutant load reduction and groundwater recharge. Grassed swales require a SHW level at least 6 inches below the bottom and should only contain standing or flowing water after rainfall events. A minimum bottom width of 2 feet is desirable for maintenance and a maximum of 8 feet wide to avoid forming erosion channels. A reasonable volume recovery time target is 72 hours. Ditch-blocks can be used to retain additional runoff for infiltration; however, when blocked, the swale would behave more like a retention pond, and retention pond design criteria may be more appropriate. Either way, grassed swales are an excellent practice to promote infiltration. Figure 10-10 illustrates a typical swale system.

Figure 10-10 Cross-Section View of Swale System



Source: Duval County LID Manual, 2013.

10.2.4.1 Exfiltration Trenches

Much of Dunedin is underlain by sandy soils. Many of these soils are very well drained and have surficial aquifer water levels low enough to support exfiltration. An exfiltration system retains stormwater runoff allowing the runoff to infiltrate. The system consists of a perforated pipe encased in aggregate. Stormwater flows through the perforated pipe and infiltrates through the trench. Exfiltration is a very effective infiltration strategy; however, proper functioning requires a SHW level of at least 2 feet below the bottom and infiltration rates that can recover the treatment volume within a reasonable time. Some areas of Dunedin have soil and SHW conditions such that exfiltration systems could accommodate flows from existing stormwater systems as well as locally derived flows.

10.2.5 BACKFLOW PREVENTER RECOMMENDATIONS

Photograph 10-2 Typical Existing Outfall Pipe to Tidal Waters

Sea level is rising at an increased rate leading to more frequent sunny day flooding due to tides rising in storm sewer systems and inundating streets and connected properties. In addition to the usual drawbacks of flooding, the salt in tidewater damages the City's roadways as well as vegetation, including the City's ROWs and private yards.

Backflow-prevention devices or check valves can be used to prevent tidewater from backing up into drainage systems to mitigate tidal flooding, while still allowing the outfall to drain stormwater runoff when the tide recedes. The City may need to install increasing numbers of backflow-prevention devices, and this BMP evaluates the three most common type of check valves for stormwater applications, including hinged-flap gates, duckbill check valves, and inline backflow preventors. Each backflow prevention type is described in the following sections along with their unique advantages, disadvantages, and maintenance considerations. All types of backflow prevention should be carefully evaluated in outfalls connected to upstream wetlands and waterways, since upstream habitats may depend on tidal cycling; additionally, backflow preventors will restrict aquatic life movements.

10.2.5.1 Hinged-Flap Gates

Hinged-flap gates are cast iron, aluminum, or stainless-steel flaps or doors that open when the upstream water level is greater than the downstream level. These devices are installed at the end of a pipe or on a concrete headwall. Because of their disadvantages, listed below, the choice of which type of check valve to use is between duckbill and inline check valves.

Advantage

- Relatively inexpensive as compared to the other types of check valves depending on their material.

Disadvantages

- Corrosion, barnacle growth, or debris can reduce effectiveness.
- Can get stuck open negating the purpose or stuck closed causing flooding.

Maintenance Considerations

- Periodic inspection of flap to remove any trapped debris.
- Downstream end of flap must be kept free of sediment and debris.
- Depending on gate material and hardware, corrosion protection may be needed.
- Gate removal for maintenance or debris removal requires heavy equipment.

10.2.5.2 Duckbill Check Valves

Similar to flap gates, duckbill check valves function according to the water level on each side of the valve. Duckbill check valves, which are synthetic rubber flaps, are normally closed. When the water level is higher on the upstream side (thus the pressure is higher) the flap is forced open to let the water out. When the water pressure on the downstream side of the valve is greater, the flap stays closed, preventing water from flowing upstream. These check valves can be installed on a headwall or seawall at stormwater discharge points or on the end of a pipe. Duckbill check valves come in a variety of models that can be slip-on mounted or ring clamped to the end of the pipe.

Advantages

- Less inexpensive (initially) than inline backflow preventors.
- No mechanical parts that can fatigue or corrode.
- Installation on the end of the pipe provides for easier installation, maintenance, and inspection compared to in-line check valves.

Disadvantages

- Requires higher pressure difference (head) upstream than inline check valves to fully open – thus higher headloss which can make upstream rainfall flood depths greater
- If exposed to the elements (not submerged) can dry and crack over time locking into an open or closed position.
  Connection to pipe end treatment subject to failing.

Maintenance Considerations

- Periodic inspection of flap to remove any trapped debris.
- Downstream end of flap must be kept free of sediment and debris.
- Maintenance can be accomplished without specialized equipment.

10.2.5.3 Inline Backflow Preventors

Photograph 10-3 Inline Backflow Preventors Typically Installed in the Last Manhole Before Outfall

Inline backflow preventors, such as the Inline Checkmate Valves by Red Valve, are made of synthetic rubber and open when the water pressure on the upstream side of the valve flap is greater than the pressure on the downstream side. When the pressure is greater on the downstream side, the valve seals shut inside the pipe preventing flow from travelling back up the pipe. The valves are placed inside of a pipe on the downstream side of an existing stormwater structure. Since these valves can be installed upstream of the discharge points, maintenance requirements are significantly reduced.

Advantages

- No mechanical parts that can fatigue or corrode.
- Require less pressure difference (head) to operate than the duckbill check valve.
- Less maintenance requirement compared to duckbill check valve or flap gate.

Disadvantages

- More difficult to inspect and maintain because of installation inside of the pipe.

Maintenance Considerations

- Periodic inspection of flap to remove any trapped debris.
- Downstream end of flap must be kept free of sediment and debris.
- Cleaning of the downstream side could require valve removal or flushing with high-pressure water.

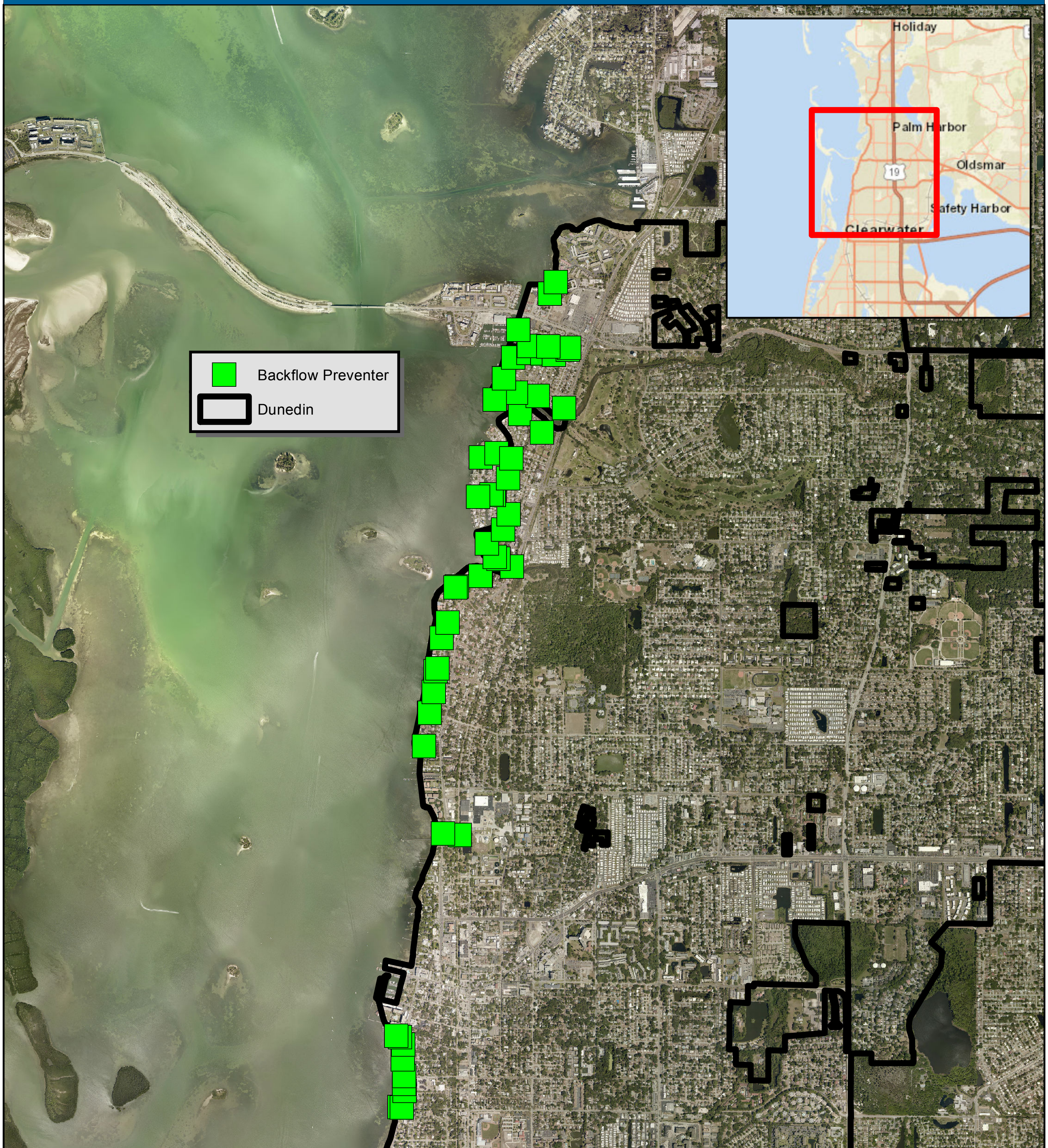
10.2.5.4 Conclusion

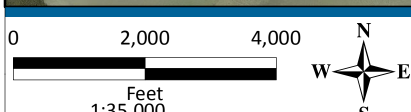
Due to reliability, low maintenance requirements, and low head-loss, inline check valves have become the industry-preferred stormwater backflow prevention measure and are recommended. Duckbill check valves can be viable alternatives in limited cases where inline check valves are cost prohibitive from a capital outlay perspective; however, the full life-cycle costs should be considered. Figure 10-11 shows where back flow preventers could be installed to minimize sunny-day flooding.

Figure 10-11

Backflow Preventer Program Recommendations

City of Dunedin Stormwater Master Plan





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11 ENVIRONMENTAL ASSESSMENTS

Environmental assessments were led by Janicki Environmental and included a water quality trend analysis and a pollutant loading model. Complete details and results are provided in Appendices H (trend analysis) and I (loading model). Each is summarized below.

11.1 TRENDS ANALYSIS

The trend analysis included data from the Florida Department of Environmental Protection (FDEP) Impaired Waters Rule (IWR) and the City of Dunedin's ambient water quality monitoring program. The results show generally improving water quality within the City. The combined IWR/City dataset were queried for all available data between 1990 and 2018 (data before 1990 are rare and sporadic) for Total Phosphorus (TP), Total Nitrogen (TN), Chlorophyll corrected for pheophytin (CHLAC), Salinity (SALIN) and Dissolved Oxygen Saturation (DOSAT). Bacteria was excluded from the trend analysis since the variability in the results does not lend itself to trend analysis.

The City of Dunedin intersects 13 Water Body Identifications (WBIDs) as defined by FDEP. These WBIDs roughly represent the contributing area of a waterbody and are used by FDEP for determining if a waterbody is meeting its designated use, developing TMDLs, and restoration plans. Figure 11-1 shows the 13 waterbodies.

Figure 11-1 WBIDs Near City of Dunedin

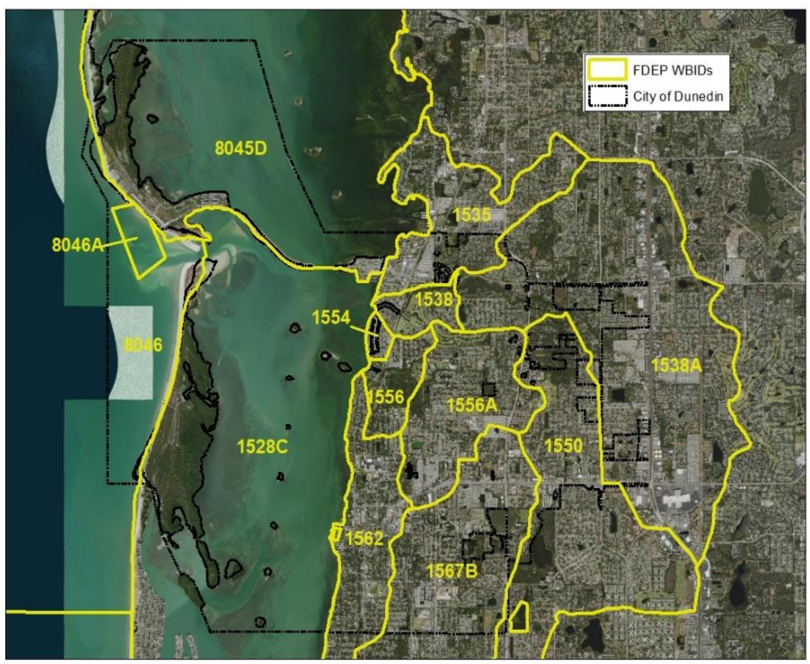


Table 11-1 lists the waterbodies associated with the City of Dunedin and analyzed water quality parameters with "improving" conditions, "degrading" conditions, or "no statistically significant long-term trend." Table 11-1 shows that nearly all results indicated either no statistically significant trends or "improving" water quality conditions in the long-term. Three results indicated "degrading" trends. These included decreasing dissolved oxygen saturation levels in WBID 1550 (Jerry Branch) and WBID 1556A (Cedar Creek) and increasing salinity levels in WBID 1538A (Curlew Creek freshwater segment).

Table 11-1 Waterbodies Associated with the City of Dunedin

WBID	FDEP Basin	Improving	Degrading	No Trend
1538A	Curlew Creek Freshwater Segment	TN, TP	SALIN	DOSAT, CHLAC
1550	Jerry Branch		DOSAT	SALIN, TN, TP
1535	Direct Runoff To Gulf (Minnow Creek)	CHLAC, TP		DOSAT, SALIN, TN
1567B	Spring Branch	DOSAT, SALIN, TN, TP		CHLAC
1538	Curlew Creek Tidal	DOSAT, TN, TP		CHLAC, SALIN
1556	Cedar Creek (Tidal)	DOSAT, CHLAC, SALIN, TN		TP
1556A	Cedar Creek	SALIN, TN	DOSAT	TP

Appendix H provides complete details.

11.2 Pollutant Loading Model

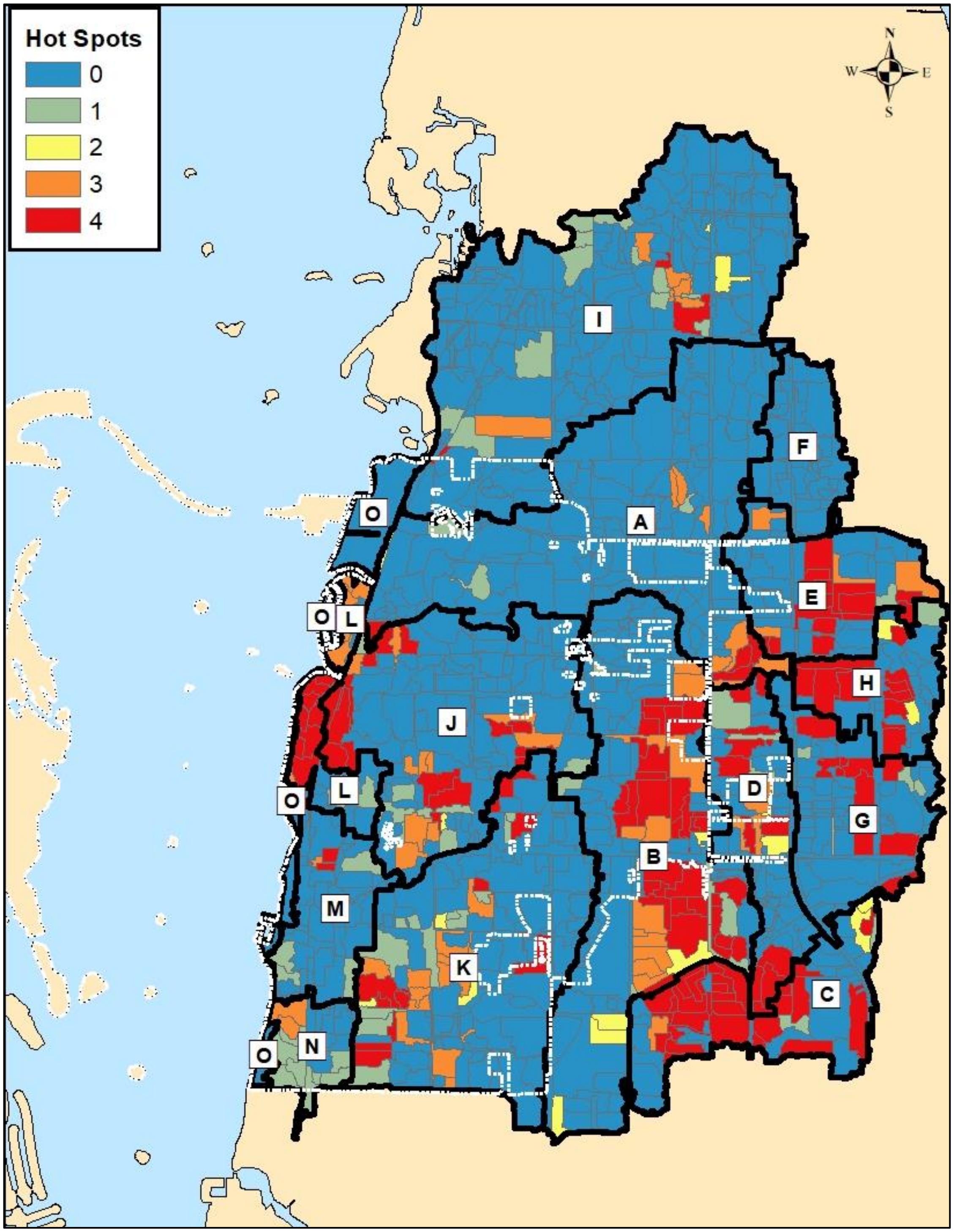
The pollutant loading estimates were prepared using the Spatially Integrated Model for Pollutant Load Estimates (SIMPLE) developed by Jones Edmunds (2005). The model provides loading estimates from various inputs, with land use categories, soil classification, and local rainfall having the largest influence. The model is capable of determining loads continuously (monthly) or seasonally based on annual average rainfall. For this effort, we chose the seasonal model. Modeled parameters include TN, TP, total suspended solids (TSS), and biochemical oxygen demand (BOD). These parameters cover the major contributors to the TMDLs identified in relation to this watershed. We conducted loading evaluations for surface water only; loadings to groundwater were not evaluated.

Mean annual load contributions were calculated for each source – baseflow, direct runoff (stormwater), point source, and septic tanks by basin. Direct runoff dominates the sources followed by baseflow. Septic loads comprise the smallest fraction in each basin; however, in Jerry Branch, the load is comparable in magnitude of TN and TSS to the point source discharge in Curlew Creek.

Loads were developed for each catchment layer, making it possible to identify areas contributing the highest unit area loads (pounds per acre per year [lb/acre/year]) for each of the parameters, or "Hot Spots." Displaying the quartile ranks with the highest unit area loads in red allows for the identification of "hot spots" where load reductions would be beneficial. Appendix I provides a series of maps (Figures 13 through 16) for TN, TP, BOD, and TSS, respectively. Figure 11-2 combines the four pollutants by counting the number of times a subbasin falls with the highest quartile by parameter. The legend shows the color corresponding to the number of pollutants that fall within the highest quartile. Subbasins with no (zero) analyzed pollutants within the highest quartile are shaded blue, subbasins with all four analyzed pollutants in the highest quartile are shared red, and so on. This map identifies areas contributing the highest unit area loads of more than one pollutant. The results from the "Hot Spot" loading analysis should be coupled with the results from the hydrologic analysis to identify areas where both issues could be addressed providing the most "bang for the buck" when planning for Capital Improvement Projects (CIPs).

Appendix I provides complete details for the pollutant loading model.

Figure 11-2 Nutrient Loading Hot Spot Locations



12 DOWNTOWN REDEVELOPMENT AREA

Burgess & Niple led the development of a regional stormwater approach for the Downtown CRA that will aid future development. Most existing stormwater treatment systems within the CRA are small "postage stamp" stormwater management facilities, and the City seeks to fulfill the stormwater treatment needs for redevelopment with more effective regional BMPs while easing land requirements for stormwater management for individual parcels.

The CRA covers approximately 215 acres; 372 parcels (82 acres) within this footprint have been identified for redevelopment. An 85-percent impervious threshold is assumed for the post-development condition. The BMP TRAINS modeling software was used to analyze the reduction in nutrient loadings achieved by 13 BMP alternatives. The required nutrient loading removal rates, based on the difference between the "Pre" and "Post" condition runoff for included parcels within the CRA, are listed in Table 12-1.

Table 12-1 CRA Total Required Nutrient Loading Removal

TN Removal Required (kg/yr)	421
TP Removal Required (kg/yr)	67

Note: kg/yr = kilograms per year.

Two existing and 11 new BMP alternatives were reviewed in combination to achieve the City's redevelopment goals. Appendix J provides the results and recommendations from the evaluation, and Table 12-2 summarizes the three BMP configuration options. Each of these options would meet or exceed nutrient loading removal requirements.

Table 12-2 BMP Configuration Options

BMP	BMP Description	Option 1	Option 2	Option 3
EX-1	Existing Bay Street Pond	X	X	X
A	City Hall Dry Retention			X
B	Grant Street Exfiltration	X	X	
A-B	Retention-Grant Street Exfiltration BMP Train			
C	Pioneer Park Exfiltration	X		X
D	Main Outfall NSBB			
C-D	Pioneer Park Exfiltration-NSBB BMP Train			
E	Monroe Outfall NSBB			
EX-2	Existing Washington Street FDOT Pond	X		
F	Expand Washington Street FDOT Pond		X	X
G	Washington Outfall NSBB			
F-G	Expand Washington Street FDOT Pond-NSBB			
H	Stirling Links (Compensatory Wet Detention)			X
	Preliminary Capital Cost	$213,200	$282,500	$810,200
	TN Removed	464	439	429
	TP Removed	74	69	75

Note: NSBB = Nutrient-separating baffle box.

Appendix J provides complete details.

13 VULNERABILITY ASSESSMENT

Collective Water Resources (Collective) performed a flooding vulnerability assessment for the City of Dunedin (City). Collective was tasked with estimating the increases in future flood vulnerability throughout the City due to projected sea level rise (SLR) that could exacerbate three flood hazards: extreme high tides, storm surge, and stormwater runoff. As part of the vulnerability assessment, Collective identified three assets within the City that could be encroached (or exposed) by flooding for each of the flooding hazards: property, structures, and roadways. Based on the exposure analysis, vulnerability was ranked for individual properties and roadways according to the characteristics of each of these assets with respect to the degree each could be affected (potential impact) - and the ability to cope with impacts (adaptive capacity). Stormwater adaptation strategies were also developed for the City to consider in its long-range planning.

13.1 Flooding Hazards

The potential increase in stormwater runoff, or rainfall-induced flooding, and King Tide events from sea level rise were analyzed by Collective. Jones Edmunds evaluated potential increase in flooding from coastal storm surge with increased sea levels. For each of the three types of flooding two planning scenarios were analyzed corresponding with 1-foot and 2-feet of SLR. These SLR scenarios were selected to represent future tidal conditions that could occur within the following time frames according to the National Oceanic and Atmospheric Administration (NOAA, 2017):

- One foot of sea level rise relative to 2020 could occur between 2039 and 2070.
- Two feet of sea level rise relative to 2020 could occur between 2052 and after 2100.

These time frames are based on conditions at the St. Petersburg tidal station (8726520) for the three sea level rise scenarios that are being utilized in Pinellas County's ongoing Vulnerability Assessment (Pinellas County, 2016)

Collective evaluated and mapped the potential increase in rainfall-induced flooding from the 100-year/24-hour storm associated with the two SLR scenarios by adjusting the hydrologic and hydraulic models developed by Jones Edmunds for the City of Dunedin and Curlew Creek watershed. The model revisions accounted for changes in tidal boundary conditions, initial water surface elevations, and water table depths due to SLR.

King Tide is a common term used to refer to the highest predicted tide of the year experienced in coastal areas. Current King Tides also provide an example of what future, daily water levels may be like with sea level rise. The average highest annual tide at the Clearwater Beach station (Station 8726724) for the past decade is 2.9 feet (North American Vertical Datum of 1988, or NAVD88). For the vulnerability assessment, two future King Tide scenarios were evaluated by adding 1-foot and 2-feet of SLR to the average highest annual tide, or 3.9 feet and 4.9 feet, respectively.

Jones Edmunds evaluated the potential increase in storm surge and wave runup associated with the two SLR scenarios along 15 coastal transects located within the City's limits. The analysis utilized the effective transects and stillwater elevations from FEMA's Flood Insurance Study (FIS) for Pinellas County. Generally, the coastal flooding depths increased between one and two feet for the 1-foot SLR scenario compared to effective coastal Base Flood Elevations (BFEs) and between two and four feet for the 2-feet SLR scenario.

13.2 Asset Exposure

Collective evaluated three asset categories within the City that could be encroached by, or exposed to, flooding: property, structures (as building footprints), and roadways. Each asset category was assessed to determine whether individual assets could be exposed to potential future flooding. Assets were marked as exposed to each of the flooding scenarios based on whether it was is within the flood inundation area (for rainfall-induced flooding and King Tide scenarios) or topographically lower than the most-landward BFE determined at each transect (for storm surge scenarios).

13.3 Vulnerability Assessment

Vulnerability, as defined by NOAA, is defined as the "*potential for loss of or harm/damage to exposed assets largely due to complex interactions among natural processes, land use decisions, and community resilience" (NOAA,* 2010). How vulnerable an exposed asset is depends on its potential impact, sensitivity to the impact, and adaptive capacity.

During a vulnerability analysis, normally properties that are not exposed to a hazard have no potential impact. The degree to which a property could experience negative impacts due to a hazard is influenced by its sensitivity. Properties, including the structures thereon, and roadways exposed to flooding, were graded as either "high," "medium," or "low" sensitivity for each flooding threat/SLR scenario based on structure-level characteristics and the services provided as well as roadway use.

Adaptive capacity is the ability to cope with impacts with minimal disruptions and costs. A property's adaptive capacity to flooding was determined based on exposure to flooding as well as available information about when structures within the property were built compared to availability of regulatory flood elevations to support the City's floodplain management. For roadways, minimum design standards, as defined by Pinellas County (see Figure 9-1) were assumed to apply and were used to grade adaptive capacity. Similar to potential impact, properties were graded as either "high," "medium," or "low" adaptive capacity for each flooding threat/SLR scenario.

13.4 Findings

Key results of the vulnerability assessment of properties within the City are as follows:

- Rising sea levels will increase flooding vulnerability within the City.
- The majority of Dunedin's essential facilities have no or low vulnerability to flooding.
- The majority of properties with medium or high vulnerability to flooding are single family homes.
- Storm surge intensified by SLR will likely present the greatest vulnerability for properties within the City.
- The majority of properties vulnerable to storm surge flooding are highly vulnerable.
- All properties with high or medium vulnerability to rainfall-induced flooding for either SLR scenario already are exposed to flooding under current conditions. The amount of flooding experience by these properties will increase with rising sea levels.

- The majority of properties with high or medium vulnerability to rainfall-induced flooding with SLR, have older structures constructed prior to BFE requirements for floodplain management.
- While the number of properties that have a high or medium vulnerability to potential King Tide flooding for either SLR scenario is relatively low, this type of flooding will occur more frequently (e.g. such as annual basis).

With respect to roadways, key take-aways from the assessment are as follows:

- Roadway flooding can result in properties becoming inaccessible or isolated.
- Rising sea levels will likely increase flooding vulnerability of roadways within the City. This increased vulnerability includes new roadways that will be flooded due to rising sea levels as well as additional length of roadways that currently flood. Given the topography throughout the City, the total length of roadways with increasing vulnerability is generally minimal.
- Increasing depths of flooding on already vulnerable roadways will exacerbate damage and losses and impact the ability of both emergency vehicles and routine traffic to access these roadways.
- Storm surge with 2-feet of SLR will likely result in the entire evacuation route along the City's coastline being inundated.
- Additionally, storm surge with 2-feet of SLR will impact access to essential facilities.
- Rainfall-induced flooding predominately impacts local roadways, which is to be expected since they are not typically designed to this flood frequency (100-year).
- Rainfall-induced flooding compounded with SLR will impact access to essential facilities and further impair accessibility to City Hall.
- While the length of roadways to potential King Tide flooding for either SLR scenario is relatively low, this type of flooding will occur more frequently.

13.5 Stormwater Adaptations

Stormwater adaptations that are available to address major flood vulnerability needs and priorities include both structural and non-structural measures. Based on the results of the vulnerability assessment the following measures should be considered in the City's long-term planning efforts:

- Install backflow preventors for City's stormwater management system outfalls.
- Consider adopting future conditions in the design of stormwater improvements.
- Increase freeboard requirements for all new or substantially improved structures.
- Establish seawall elevation requirements within the City's land development code to be implement for all new or repaired seawalls, both public and private, that addresses SLR.
- Promote low impact development practices on City-owned properties and incentivize these practices for private developments, similar to Pinellas County's approach.
- Maintenance and customer service aspects of the City's stormwater program should also be considered.

It is also important to mention that flood adaptation tends to focus on water quantity. However, a truly sustainable resiliency program will also incorporate water quality, ecological, and long-term sustainability considerations. Aging infrastructure will also further stress the City’s resources related to adaptation. Integrating considerations for infrastructure that is reaching the end of its useful life is another helpful strategy.

Please refer to Appendix K for complete details.

14 COMMUNITY RATING SYSTEM

The City of Dunedin participates in the National Flood Insurance Program's (NFIP) CRS. This is a voluntary program in which communities are encouraged to implement floodplain management activities beyond the minimal requirements for the NFIP. By participating in the program, homeowners can receive discounted flood insurance premium rates.

Photograph 13-1 Saint Joseph Sound in Dunedin

These rate reductions are due, in part, to the City's initiatives that help to:

- Reduce flood damage to insurable property.
- Strengthen and support the insurance aspects of the NFIP.
- Encourage a comprehensive approach to floodplain management.

The City's recent watershed study identified areas that are at increased flood risk from a 100-year storm event. The study identified similar flood risks as the effective flood information depicted on FEMA's Flood Insurance Rate Maps (FIRM) as well as additional areas resulting from a more detailed study. This study and the accompanying stormwater model provide the City with a framework for developing projects to help reduce the impacts of flooding and evaluate the potential impacts of future development.

This section evaluates how this study can be used to support and improve the City's current class 5 CRS ranking. The study fits with the overall goal of the CRS by identifying and helping to reduce flood damage to insurable property and providing a framework for implementing a comprehensive approach to floodplain management.

The following sections highlight how this study can enhance certain elements within the various activities of the CRS.

14.1 Activity 320 – Map Information Service

Map products from this study can provide additional information that the City can leverage for this CRS activity. Data products include floodplains not identified in the current FEMA maps. In addition, one of the by-products of the floodplain delineation for this study is a flood depth grid.

14.1.1 Recommendations

The City should be logging the map information service inquiries and **make sure that this additional information is being provided to property owners and residents** who call, email, or visit the City. The City also needs to publicize to the community that this service is available, along with the types of information that the City can provide with regard to flood risks.

14.1.1.1 Potential Credited Elements

- MI3 (Problems not shown on the FIRM).
- MI4 (Flood depth).

14.2 Activity 410 – Flood Hazard Mapping

This activity credits communities with mapping flood hazards that are above and beyond the risk level depicted on the FEMA maps. CRS credit criteria include regulating based on the new floodplains, which may require an ordinance change. The floodplain also must be submitted to FEMA so that the local FIRM may eventually be revised.

Although this study is more detailed than what is depicted on the FEMA maps, the amount of changes to the floodplains will not significantly affect the credits. In some areas, the FEMA floodplains are reduced and cannot be credited. Although, the study results in additional areas mapped and will be used to develop projects or evaluate new developments, these areas will not necessarily be submitted to FEMA for insurance rating. For example, flooding information on a roadway will guide the City on how to develop projects to mitigate losses but will not necessarily need to be submitted to FEMA for rating insurance.

14.2.1 Recommendations

This increase in the extent of floodplain is minimal and will not impact this activity.

14.2.1.1 Potential Credited Elements

None.

14.3 Activity 430 – Higher Regulatory Standards

The floodplain results from this study have the potential to affect several elements within Activity 430 of the CRS. These elements include regulating development with respect to:

- Development limitations (DL) – prohibiting fill, buildings, and/or storage of materials in the Special Flood Hazard Areas (SFHAs).
- Freeboard (FRB) – freeboard requirements for new buildings constructed in the SFHA.
- Foundation protection (FDN) – engineered foundations.
- Cumulative substantial improvements (CSI) – counting improvements cumulatively.
- Protection of critical facilities (PCF) – protecting facilities that are critical to the community.
- Enclosure limits (ENL) – limiting enclosures below the base flood elevation (BFE).

14.3.1 Development Limitations

14.3.1.1 Prohibiting Fill

The City currently permits fill provided that the fill is designed to be stable under conditions of flooding including rapid rise and rapid drawdown of floodwaters, prolonged inundation, and protection against flood-related erosion and scour. Evaluation of the City's ordinance did not reveal considerations for compensatory storage. Compensatory storage provides an equal volume of storage to replace what is lost due to buildings or the placement of fill. This approach is particularly more effective in inland areas not affected by coastal flooding. The CRS credits this element for regulations that require new development to provide compensatory storage at hydraulically equivalent sites up to a ratio of 1.5:1. Currently, Pinellas County requires 1:1 compensatory storage for development, redevelopment, or fill outside of floodways.

14.3.1.2 Storage of Hazardous Materials in the Floodplain

The City has regulations prohibiting the discharge of hazardous wastes into the storm sewer systems and public waters, mainly to satisfy National Pollutant Discharge Elimination System (NPDES) discharge requirements. The regulations do not specifically prohibit the on-site storage of certain hazardous materials with respect to the floodplain. The City may choose to revise its regulations to be consistent with Pinellas County. Pinellas County prohibits structures used for the manufacture or storage of hazardous materials in the floodplain or floodway.

14.3.2 Freeboard

City regulations already have freeboard standards above the minimum requirements of the NFIP. The City currently requires at least 1 foot of freeboard above the BFE. This requirement is consistent with the recent changes in the Florida Building Code requiring a minimum 1-foot freeboard.

14.3.3 Foundation Protection

The City may choose to revise its regulations using one of three approaches credited in the CRS. These approaches include:

- Requiring engineered foundations and no buildings on fill (FDN1).
- Buildings on compacted fill must be protected from erosion and scour, with compensatory storage (FDN2).
- Buildings on compacted fill must be protected from erosion and scour, but no compensatory storage (FDN3).

The City currently meets the Florida Building Code requirements with respect to foundation protection (FDN3). Regulations require buildings on compacted fill be protected from erosion and scour. Additional credit can be achieved if the City decides to include compensatory storage requirements for new developments in the floodplain.

14.3.4 Cumulative Substantial Improvements

The City currently requires substantial improvements to be tracked for 5 years. Over time, a home's value increases – more so if improvements have been done. The potential cost of the losses also increases along with this increased value of the home. The City may choose to increase this requirement to 10 years to encourage better protection for these investments and reduce losses.

14.3.5 Protection of Critical Facilities

The City currently prohibits construction of certain critical facilities such as hospitals and nursing homes in the coastal storm area. In addition, the Florida Building Code requires that these facilities be elevated or protected to the higher of the BFE plus 2 feet or the 500-year flood elevation.

14.3.6 Enclosure Limits

The City defines break-away walls as *Shall be limited to lattice work or decorative screening for aesthetic purposes which are not part of the structural support of the building and which are so designed as to breakaway under abnormally high tides or wave action without damage to the structural integrity of the building on which they are used or any buildings to which they might be carried by floodwaters*. In addition, basement floors that are below grade on all sides shall be elevated to or above the BFE plus 1 foot or the design flood elevation, whichever is higher. Basement floors that are below grade on all sides are prohibited.

14.3.7 Local Drainage Protection

According to the City's floodplain ordinance 105-41.3.1 (Rules and Regulations to Effect Purpose of Building Code – Finished Grade):

(A) The finished grade elevations of the individual lots shall be shown on the drainage plan, these elevations will normally drain the lot from the rear property line to the street, along the common property lines. Where topography or other features make impractical such lot drainage, an alternate lot drainage plan will be submitted for the approval of the building official.

(B) The drainage plan shall include the finished minimum floor elevations of all structures which may be constructed. The minimum finished floor elevation shall be at least 18 inches above the centerline of the abutting roadway. Proposed finished floor elevations of structures shall ensure adequate fall of the building's sanitary sewer line, and ensure that surface water flows will not cause damage nor enter any portion of the structure on the lot or abutting properties unless drainage easements are provided.

Additional language is also provided for subdivisions and general site improvements to guide flood waters away from buildings. These requirements should provide the City with credit for elements LDP1, LPD2 and LPD3. The City may consider revising the language to use the crown of the road instead of the centerline.

14.3.8 Recommendations

The City should consider revising the ordinance to explicitly include compensatory storage for all or certain high-risk areas. The City may also consider revising the regulations to include prohibiting storage of hazardous materials within the regulatory floodplains. These changes will reduce the potential damages from flooding, enhance the protection of the natural floodplains, and bring the City's regulations more in line with the County's minimum requirements.

Other revisions to the ordinances that the City may consider include:

- Change the cumulative substantial improvement period from 5 to 10 years.
- Revise Ordinance 105-41.3.1 to reference the crown of road instead of the centerline of road for drainage purposes.

14.3.8.1 Potential Credited Elements

- Compensatory storage (DL1b).
- Prohibitions on storage of materials (DL3).
- Buildings on compacted fill must be protected from erosion and scour, with compensatory storage (FDN2). Credit for this element is dependent on achieving credit for element DL1b.
- Cumulative Substantial Improvements (CSI).

14.4 Activity 450 – Stormwater Management

The data resulting from this study will provide the City with an improved framework for evaluating future developments. The City may receive additional credit for implementing stormwater management regulations through an adopted watershed master plan. The results of this study can provide the data necessary for implementing such a plan.

14.4.1 Stormwater Management Regulations (SMR)

Current City regulations require developments to manage runoff up to the 25-year storm. The City requires pre- and post-development hydrology calculations, and post-development must be limited to pre-development levels. In addition, the developments should be designed to pass the 100-year flows from all off-site upstream areas without damaging effects.

14.4.1.1 Size of Development (SZ)

The City currently implements the above drainage criteria for developments that are 0.5 acre or more. This corresponds to a credit score of 90 points for this CRS element. The City should continue with this exceptional level of protection for its residents and businesses.

14.4.1.2 Design Storms (DS)

The City currently qualifies for the DS2 credit of 36 points for managing runoff for the 25-year design storm. This is comparable to Pinellas County's regulations and other municipalities within the SWFWMD.

14.4.1.3 Low-Impact Development

Current City regulations describe LID techniques as one of the possible means to manage stormwater within the City. LID features are subject to approval of the City Engineer. This regulation does not specifically require the use of LID. The City may consider revising the regulation to encourage more use of LID practices, consistent with the City comprehensive plan goals.

14.4.2 WATERSHED MANAGEMENT PLAN

This element credits the City with implementing a WMP that incorporates the effects of future conditions, sea level rise, protection of natural systems, and providing a dedicated funding source for implementing the Plan.

This portion of the study is currently underway and will be evaluated for incorporation into the overall comprehensive planning efforts by the City. This will subsequently help the City to get additional CRS credit for this activity.

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