Final Report

DELANEY CREEK AREA

STORMWATER MANAGEMENT MASTER PLAN UPDATE

(Known Conditions through April 2007)

Prepared for:

Stormwater Management Section
Public Works Department, Hillsborough County
601 E. Kennedy Boulevard
Tampa, FL 33602-4156

Prepared by:

AYRES ASSOCIATES

Engineers/Planners/Scientists
8875 Hidden River Parkway, Suite 200
Tampa, FL 33637-1035
Phone (813) 978-8688
Fax (813) 978-9369

May 2008
Table of Contents

EXECUTIVE SUMMARY ........................................................................................................... ES

CHAPTER 1 INTRODUCTION ............................................................................................... 1-1
1.1 Project Location and General Description .................................................................. 1-1
1.2 Background and Scope of the Project ........................................................................ 1-3
1.3 Data Collection .......................................................................................................... 1-4
1.4 Report Organization .................................................................................................... 1-6

CHAPTER 2 WATERSHED DESCRIPTION ........................................................................... 2-1
2.1 Overview .................................................................................................................... 2-1
2.2 Climate ....................................................................................................................... 2-2
2.3 Soils ............................................................................................................................ 2-2
2.4 Land Use/Cover ......................................................................................................... 2-4
2.4.1 Existing Land Use .................................................................................................. 2-4
2.4.2 Future Land Use .................................................................................................... 2-5
2.5 Physiography and Hydrology ..................................................................................... 2-6
2.6 Geology and Hydrogeology ....................................................................................... 2-7
2.6.1 Surficial Aquifer ................................................................................................... 2-8
2.6.2 Semi-Confining Zone ............................................................................................ 2-8
2.6.3 Upper Floridan Aquifer ......................................................................................... 2-9

CHAPTER 3 MAJOR CONVEYANCE SYSTEMS ................................................................... 3-1
3.1 Delaney Creek Subwatershed ...................................................................................... 3-1
3.1.1 Delaney Creek Main Channel System (model # 210xxx series) ............................... 3-1
3.1.2 Laterals .................................................................................................................. 3-1
3.1.2.1 Lateral “A” (model # 211xxx series) .................................................................. 3-1
3.1.2.2 Lateral “A-1” (model # 2115xx series) .............................................................. 3-2
3.1.2.3 Lateral “B” (model # 212xxx series) ................................................................. 3-2
3.1.2.4 Lateral “C” (model # 213xxx series) ................................................................. 3-2
3.1.2.5 Lateral “C-1” (model # 2135xx series) .............................................................. 3-2
3.1.2.6 Lateral “D” (model # 214xxx series) ................................................................. 3-2
3.1.2.7 Lateral “E” (model # 215xxx series) ................................................................. 3-2
3.1.2.8 Lateral “E-1” (model # 2155xx and 2156xx series) ........................................... 3-2
3.1.2.9 Lateral “F” (model # 216xxx series) ................................................................. 3-3
3.1.2.10 Hendries Lake System (model # 220xxx, 223xxx and 225xxx series) .............. 3-3
3.1.2.11 Brandon Town Center Mall (model # 221xxx and 217xxx series) ................. 3-3
3.1.2.12 Gornto Lake System (model # 2215xx series) .................................................. 3-3
3.1.2.13 Heather Lakes System (model # 222xxx series) ............................................. 3-4
3.1.2.14 Lumsden Road North Ditch (model # 224xxx series) ..................................... 3-4
3.1.2.15 Hickory Hammock System (model # 230xxx series) ...................................... 3-4
3.1.2.16 Isolated System (model # 227xxx series) ....................................................... 3-4
3.2 Delaney Pop-off Canal Subwatershed ......................................................................... 3-4
3.2.1 Delaney Pop-off Main Channel System (model # 240xxx series) ............................ 3-5
3.2.2 Tributaries ............................................................................................................. 3-5
3.2.2.1 Tributary “A” System (model # 243xxx series) ............................................... 3-5
3.2.2.2 Tributary “B” System (model # 247xxx series) ............................................... 3-5
3.2.2.3 Tributary “C” System (model # 2403xx and 2404xxx series) ............................ 3-5

AVRES ASSOCIATES

61-0100.06/May 2008
Delaney Creek Area Stormwater Management Master Plan Update
# Table of Contents

3.2.2.4 Tributary “E” System (model # 2420xx series) .............................................. 3-5
3.2.2.5 Tributary “F” System (model # 2425xx series) .............................................. 3-6
3.2.2.6 Tributary “G” System (model # 246xxx series) .............................................. 3-6
3.2.2.7 Tributary “H” System (model # 2465xx series) .............................................. 3-6
3.2.2.8 Tributary “I” System (model # 2478xx series) .............................................. 3-6
3.2.2.9 Aspen Cove Apartment System (model # 250xxx series) ................................ 3-6
3.2.2.10 Evergreen Estates System (model # 252xxx series) ...................................... 3-6
3.2.2.11 I-75 Ditch System (model # 253xxx series) .............................................. 3-6

3.3 North Archie Creek Subwatershed ........................................................................ 3-7
  3.3.1 North Archie Creek Main Channel System (model # 260xxx series) ................ 3-7
  3.3.2 Tributaries ..................................................................................................... 3-7
    3.3.2.1 Tributary “A” (model # 261xxx series) ....................................................... 3-7
    3.3.2.2 Tributary “B” (model # 262xxx series) ....................................................... 3-7
    3.3.2.3 Tributary “C” (model # 265xxx series) ....................................................... 3-8
    3.3.2.4 Unnamed Tributary (model # 263xxx series) ............................................. 3-8
    3.3.2.5 Tributary “D” (model # 2705xx series) ....................................................... 3-8
    3.3.2.6 Tributary “E” (model # 272xxx series) ....................................................... 3-8
    3.3.2.7 Tributary “F” (model # 273xxx series) ....................................................... 3-8
    3.3.2.8 Tributary “G” (model # 27004x series) ..................................................... 3-8

3.4 Archie Creek Subwatershed ................................................................................... 3-9
  3.4.1 Archie Creek Main Channel System (model # 280xxx and 290xxx series) ........ 3-9
  3.4.2 Tributaries ..................................................................................................... 3-9
    3.4.2.1 Tributary “A” (model # 2801xx series) ....................................................... 3-9
    3.4.2.2 78th Street Ditch (model # 2803xx series) ............................................... 3-9
    3.4.2.3 Tributary “B” (model # 2804xx series) ....................................................... 3-9
    3.4.2.4 Tributary “C” (model # 2805xx series) ....................................................... 3-10
    3.4.2.5 Tributary “D” (model # 2902xx and 2903xx series) ................................. 3-10
    3.4.2.6 Tributary “E” (model # 2900xx, 2905xx and 2906xx series) .................... 3-10
    3.4.2.7 Tributary “G” (model # 2901xx series) ..................................................... 3-10
    3.4.2.8 Isolated Basins (Cargill complex) ............................................................ 3-10

CHAPTER 4  HYDROLOGIC/HYDRAULIC MODEL METHODOLOGY .......................... 4-1

4.1 General Hydrologic/Hydraulic Model Development ........................................... 4-1

4.2 Hydrology ........................................................................................................... 4-1
  4.2.1 SCS-CN Method ............................................................................................. 4-1
  4.2.2 SCS Dimensionless Hydrograph ................................................................... 4-3
  4.2.3 Model Implementation .................................................................................... 4-4
  4.2.4 Rainfall Depth ............................................................................................... 4-5
  4.2.5 Subbasin Delineations .................................................................................... 4-5
  4.2.6 Soil Data, Land Use, and SCS Curve Number Determination ....................... 4-7
    4.2.6.1 Soil Data ................................................................................................... 4-7
    4.2.6.2 Land Use .................................................................................................. 4-7
    4.2.6.3 Runoff Curve Numbers .......................................................................... 4-7
  4.2.7 Time of Concentration .................................................................................... 4-8

4.3 Hydraulics ............................................................................................................ 4-8
  4.3.1 Major Modifications ....................................................................................... 4-8
### Table of Contents

4.3.2 Natural Channels ................................................................. 4-9
4.3.3 Conduits .................................................................................. 4-9
4.3.4 Storage Facilities ................................................................. 4-10
4.3.5 Weirs ................................................................................. 4-10
4.3.6 Orifices .............................................................................. 4-10
4.3.7 Initial Water Surface Elevations ........................................... 4-11
4.3.8 Dummy Junctions and Conduits ............................................ 4-11
4.3.9 Boundary Conditions ......................................................... 4-11
4.3.10 Numerical Instability ......................................................... 4-11
4.3.11 Model Schematic ................................................................. 4-12

**CHAPTER 5  HYDROLOGIC/HYDRAULIC MODEL CALIBRATION AND VERIFICATION** .................................................................................................................. 5-1
5.1 Boundary Conditions .................................................................... 5-1
5.2 Data Collection .......................................................................... 5-1
5.2.1 USGS Gage Stations ................................................................. 5-1
5.2.2 Precipitation Data ................................................................. 5-2
5.2.3 Surface Water Data ............................................................... 5-2
5.2.4 Antecedent Moisture Condition (AMC) .................................. 5-3
5.3 Existing Conditions Model Calibration ......................................... 5-4
5.4 Existing Conditions Model Verification ........................................ 5-6

**CHAPTER 6  EXISTING CONDITIONS LEVEL OF SERVICE** ................................................................. 6-1
6.1 Existing Conditions Model Simulation Results ............................. 6-1
6.2 Level of Service Analysis ............................................................ 6-2
6.2.1 Level of Service Methodology ................................................ 6-2
6.2.2 Establishment of Landmark Elevations .................................... 6-3
6.3 Existing Conditions Level of Service ........................................... 6-3
6.3.1 Delaney Creek Main Channel System (model # 210xxx series) ........................................................................ 6-3
6.3.2 Delaney Creek Laterals .......................................................... 6-4
6.3.2.1 Lateral “A” (model # 211xxx series) ..................................... 6-4
6.3.2.2 Lateral “A-1” (model # 21155xx series) ............................... 6-5
6.3.2.3 Lateral “B” (model # 212xxx series) ....................................... 6-6
6.3.2.4 Lateral “C” (model # 213xxx series) ....................................... 6-6
6.3.2.5 Lateral “C-1” (model # 2135xx series) ................................. 6-7
6.3.2.6 Lateral “D” (model # 214xxx series) ....................................... 6-7
6.3.2.7 Lateral “E” (model # 2150xx series) ....................................... 6-7
6.3.2.8 Lateral “E-1” (model # 2155xx/2156xx series) ....................... 6-7
6.3.2.9 Lateral “F” (model # 216xxx series) ....................................... 6-8
6.3.2.10 Hendris Lake System (model # 220xxx/223xxx/225xxx series) ................................................................. 6-8
6.3.2.11 Brandon Town Center Mall (model # 221xxx/217xxx series) ........................................................................ 6-9
6.3.2.12 Gornta Lake System (model # 2214xxx/2215xxx series) ........................................................................ 6-9
6.3.2.13 Heather Lakes System (model # 222xxx series) ....................... 6-9
6.3.2.14 Lumadon Road North Ditch (model # 224xxx series) ............ 6-9
6.3.2.15 Hickory Hammock System (model # 230xxx/233xxx series) ........................................................................ 6-9
6.3.2.16 Isolated System (model # 227xxx series) ............................. 6-10
6.3.3 Delaney Pop-off System (model # 200xxx series) ......................... 6-11
# Table of Contents

6.3.4 Delaney Pop-off Main Channel System (model # 240xxx Series) .................................................. 6-12
6.3.5 Delaney Pop-off Tributaries ........................................................................................................... 6-12
  6.3.5.1 Tributary “A” System (model # 243xxx Series) ................................................................. 6-12
  6.3.5.2 Tributary “B” System (model # 247xxx Series) ................................................................. 6-13
  6.3.5.3 Tributary “C” System (model # 244xxx Series) ................................................................. 6-13
  6.3.5.4 Tributary “E” System (model # 2420xx Series) ............................................................... 6-13
  6.3.5.5 Tributary “F” System (model # 2425xx Series) ............................................................... 6-14
  6.3.5.6 Tributary “G” System (model # 246xxx Series) ............................................................... 6-14
  6.3.5.7 Tributary “H” System (model # 2465xx Series) ............................................................... 6-14
  6.3.5.8 Tributary “I” System (model # 2478xx Series) ............................................................... 6-14
  6.3.5.9 Aspen Cove Apartment System (model # 250xxx/251xxx/254xxx series) .................. 6-14
  6.3.5.10 “Model # 248xxx” Series System ..................................................................................... 6-14
  6.3.5.11 Evergreen Estates System (model # 252xxx series) ......................................................... 6-15
  6.3.5.12 1-75 Ditch System (model # 253xxx series) .................................................................... 6-15
6.3.6 North Archie Creek Main Channel System (model # 260xxx series) .............................................. 6-15
6.3.7 North Archie Creek Tributaries .................................................................................................... 6-16
  6.3.7.1 Tributary “A” (model # 261xxx series) ................................................................................. 6-16
  6.3.7.2 Tributary “B” (model # 262xxx series) ................................................................................. 6-16
  6.3.7.3 Tributary “C” (model # 265xxx series) ................................................................................. 6-17
  6.3.7.4 Unnamed Tributary (model # 263xxx series) ......................................................................... 6-17
  6.3.7.5 Model # 264xxx/266xxx/267xxx/269xxx Series ............................................................... 6-17
  6.3.7.6 Tributary “D” (model # 2705xx series) ................................................................................. 6-17
  6.3.7.7 Tributary “E” (model # 272xxx series) ................................................................................. 6-17
  6.3.7.8 Tributary “F” (model # 273xxx series) ................................................................................. 6-18
  6.3.7.9 Tributary “G” (model # 27004xx series) ............................................................................. 6-18
  6.3.7.10 Model # 270xxx Series ........................................................................................................ 6-18
  6.3.7.11 Model # 274xxx/276xxx Series ............................................................................................ 6-18
6.3.8 Archie Creek Main Channel System (model # 2800xx/2900xx series) ............................................ 6-18
6.3.9 Archie Creek Tributaries .............................................................................................................. 6-19
  6.3.9.1 Tributary “A” (model # 2801xx series) ................................................................................ 6-19
  6.3.9.2 78th Street Ditch (model # 2803xx series) ........................................................................ 6-19
  6.3.9.3 Tributary “B” (model # 2804xx series) ................................................................................ 6-19
  6.3.9.4 Tributary “C” (model # 2805xx series) ................................................................................ 6-20
  6.3.9.5 Tributary “C-1” (model # 2806xx series) ............................................................................. 6-20
  6.3.9.6 Tributary “D” (model # 2902xx/2903xx series) .................................................................. 6-20
  6.3.9.7 Tributary “F” (model # 290xxx/2905xx/2906xx series) ................................................... 6-20
  6.3.9.8 Tributary “G” (model # 2901xx series) ................................................................................. 6-21
6.4 Model Comparisons ...................................................................................................................... 6-21
6.4.1 Comparison of Updated Model and Previous Model Results .................................................. 6-21
6.4.2 Comparison of 100-year, 1-day and 100-year, 5-day Model Results ...................................... 6-23
6.5 100-year Floodplain Delineation ................................................................................................. 6-23

**CHAPTER 7** ALTERNATIVES ANALYSIS UPDATE ........................................................................ 7-1
7.1 Flood Control Alternatives Development .................................................................................. 7-1
7.2 Implemented Flood Control Projects ...................................................................................... 7-2
7.3 Remaining Flood Control Projects – Proposed Conditions ...................................................... 7-3
# Table of Contents

7.3.1 Delaney Creek Subwatershed ..................................................................................7-3  
7.3.1.1 Delaney Creek Main Channel System (model # 210xxx series) ..................7-3  
7.3.1.2 Laterals ........................................................................................................7-4  
7.3.1.3 Hendrics Lake System (model # 220xxx series) ........................................7-6  
7.3.1.4 Hickory Hammock Lake System (model # 230xxx series) ....................7-6  
7.3.1.5 Closed Basin System (model # 227xxx series) ........................................7-7  
7.3.2 Delaney Pop-off Canal Subwatershed .................................................................7-7  
7.3.2.1 Delaney Pop-off Main Channel System (model # 200xxx and 240xxx series) 7-7  
7.3.2.2 Tributaries ..................................................................................................7-8  
7.3.2.3 Evergreen Estates System (model # 252xxx series) ..................................7-8  
7.3.3 North Archie Creek Subwatershed .................................................................7-9  
7.3.3.1 North Archie Main Channel System (model # 260xxx series) ...............7-9  
7.3.4 Archie Creek Subwatershed ...............................................................................7-10  
7.3.4.1 Archie Creek Main Channel System (model # 280xxx and # 290xxx series) 7-10  
7.3.4.2 Tributaries .................................................................................................7-10  
7.4 Project Reprioritization .........................................................................................7-11
Table of Contents

LIST OF APPENDICES

APPENDIX A: TABLES

Table 1-1 List of Environmental Permit Documents (ERP) Collected
Table 2-1 Summary Statistics of Hydrologic Soil Group Distribution
Table 2-2 Summary Statistics of Existing Land Use/Cover
Table 2-3 Summary Statistics of Future Land Use/Cover
Table 3-1 Major Conveyance Systems
Table 4-1a Runoff Curve Numbers for Urban Areas
Table 4-1b Runoff Curve Numbers for Cultivated Agricultural Lands
Table 4-1c Runoff Curve Numbers for Other Agricultural Lands
Table 4-1d Runoff Curve Numbers for Arid and Semiarid Rangelands
Table 4-2 Summary of Subbasin Hydrologic Parameters
Table 4-3 Curve Number Lookup Table for Land Use Code and Soil Hydrologic Group
Table 4-4 Overland Flow Manning’s n Values
Table 4-5 Culvert Entrance Loss Coefficients
Table 5-1 Gage Stations
Table 5-2 Precipitation and Surface Water Data Collection
Table 5-3 15-min Rainfall Data during Hurricane Frances
Table 5-4 15-min Rainfall Data during July 2004 Storm
Table 5-5 Antecedent Moisture Condition (AMC) Curve Number Lookup Table
Table 6-1 Design Storm Event Results – Existing Conditions
Table 6-2 Existing Conditions Level of Service
Table 6-3 Comparison of Updated Delaney Creek Area Watershed Model Results to Previous Model
Table 6-4 Comparison of Peak WSEL for the 100-year, 1-day and 100-year, 5-day Events
Table 7-1 Preferred Alternatives Level of Service
Table of Contents

APPENDIX B: FIGURES

Figure 1-1 Project Location Map
Figure 1-2 Watershed and Surrounding Features
Figure 1-3 Locations of ERP Reviewed
Figure 2-1 Topographic Map
Figure 2-2 Soil Map
Figure 2-3 Hydrologic Soil Groups
Figure 2-4 Existing Land Use/Cover (2006)
Figure 2-5 Future Land Use/Cover (2015)
Figure 2-6 Major Project DRI's & Vested Project Map
Figure 2-7 SWFWMD Hydrogeologic Cross-Section Map
Figure 3-1 Major Conveyance Systems Map
Figure 3-2a Delaney Creek Subwatershed Existing Connectivity Diagram (West)
Figure 3-2a Delaney Creek Subwatershed Existing Connectivity Diagram (East)
Figure 3-3 Delaney Pop-off Canal/North Archie Creek Subwatersheds Existing Connectivity Diagram
Figure 3-4 Archie Creek Subwatershed Existing Connectivity Diagram
Figure 4-1 Subbasin Delineations
Figure 4-2 Comparison of Subbasin Delineations
Figure 5-1 USGS Gage Locations
Figure 5-2 Thiessen Polygon Rainfall Distribution
Figure 5-3 Hurricane Frances Calibration: Stage – Delaney Creek
Figure 5-4 Hurricane Frances Calibration: Stage – Delaney Pop-off Canal
Figure 5-5 Hurricane Frances Calibration: Stage – North Archie Creek
Figure 5-6 Hurricane Frances Calibration: Stage – Archie Creek
Figure 5-7 July 2004 Storm Verification: Stage – Delaney Pop-off Canal
Figure 5-8 July 2004 Storm Verification: Stage – North Archie Creek
Figure 6-1 Flood Control Level of Service Diagram
Figure 6-2 Level of Service Existing Conditions
Figure 6-3 100-year Floodplain Delineation
Table of Contents

APPENDIX C: MODEL FILES (SEE ATTACHED DVD)

• Design Storms Existing Conditions
  - 2.33-year, 24-hour Simulation
  - 5-year, 24-hour Simulation
  - 10-year, 24-hour Simulation
  - 25-year, 24-hour Simulation
  - 50-year, 24-hour Simulation
  - 100-year, 24-hour Simulation
  - 10-year, 120-hour Simulation
  - 50-year, 120-hour Simulation
  - 100-year, 120-hour Simulation

• Design Storms Propose Conditions
  - 25-year, 24-hour Simulation
  - 100-year, 24-hour Simulation

• Calibration
  - Hurraicane Frances Simulation, 120-hour

• Verification
  - July 18-20, 2004 Storm Simulation, 120-hour

APPENDIX D: GIS FILES (SEE ATTACHED DVD)

• Application Dataset
  - Contours
  - Hydrology
  - Land Use
  - Soil
Table of Contents

- FEMA Streams
- Township-Range-Section
- Watershed

• Derived Dataset
  - Subbasin Delineation
  - Stormwater Junction
  - Stormwater Conduit
  - Stormwater Weir
  - Stormwater Orifice
  - Stormwater Pump
  - 100-year Floodplain Delineation

• Primary Dataset
  - Railroad
  - Street
EXECUTIVE SUMMARY

The Delaney Creek Area watershed drains approximately 34.3 square miles of land located in central Hillsborough County, Florida. It is generally bounded on the north by Palm River Road and the CSX railway, to the west by Hillsborough Bay, to the east by Valrico Road, and to the south by an imaginary line approximately 1/2 to 1/4 of a mile north of the Álafia River to the area of Buckhorn Springs, and then east along the dividing line between Townships 29 and 30 in Range 20. Ultimately, water from the watershed discharges into Hillsborough Bay of Tampa Bay via Delaney, Archie, and North Archie Creeks and the Delaney Pop-off Canal.

The purpose of this study is to update the current stormwater management master plan developed by Hillsborough County around 2001. Since then, changes have occurred within the watershed and affected both hydrologic and hydraulic conditions. During this update, the Level of Service (LOS) for stormwater infrastructure is reexamined under the existing condition. Furthermore, changes in standards and reference elevation datum have been considered. These updates are summarized below. All major Environmental Resource Permits (ERP) that have significant impact on the hydraulic and storage conditions within the watershed were collected. Updated model input includes new developments reflected in collected ERPs and “As-Built” drawings through April 2007.

Hydrologic parameters have been updated within the Delaney Creek Area. Subbasin delineations were revised on the basis of latest aerial photographs and digital elevation model (DEM) from 1-foot digital contours, as well as major ERPs and “As-Built” drawings within the watershed since last model update. Existing land use coverage based on the Florida Land Use Cover Classification System (FLUCCS) 2006 was updated with latest aerial photographs. Runoff parameters, i.e., Curve Number (CN) and Time of Concentration (Tc), were recalculated for each subbasin.

Update for the hydraulic model was accomplished by incorporating collected ERPs and “As-Built” drawings. Limited field visits were also performed for certain areas to confirm the existing conditions. Storage capacities were recalculated based upon latest digital elevation model.

The updated model was calibrated with Hurricane Frances of September 2004. The results from the calibrated existing conditions model were used to evaluate the location and degree of expected flooding within the study area under the existing conditions for the 2.33-year/24-hour, 5-year/24-hour, 10-year/24-hour, 25-year/24-hour, 50-year/24-hour and 100-year/24-hour design storms. Additionally, the 10-year/120-hour, 50-year/120-hour, and 100-year/120-hour design storm simulation were also performed. The 25-year/24-hour model results were evaluated to determine the existing conditions LOS for the watershed.

Finally, the report describes the performance of the 2001 preferred alternatives under the DCA 2007 system conditions. Alternatives were developed to provide an upgrade to the existing Level of Service for areas that historically experienced flooding deficiencies. Discussion includes evaluation of those
alternative improvements already implemented by 2007 and now considered part of the current “Existing Condition”, as well as assessment of the efficacy of remaining flood control proposals.

The GIS geodatabase was constructed for the Delaney Creek Area per Hillsborough County’s guidelines on watershed management plan update. During this update, the datum for the watershed model and GIS database has been converted from existing National Geodetic Vertical Datum (NGVD29) to North American Vertical Datum of 1988 (NAVD88).
CHAPTER 1 INTRODUCTION

1.1 Project Location and General Description

The Delaney Creek Area (DCA) watershed is located in central Hillsborough County, Florida on the eastern shore of Hillsborough Bay, as shown in Figure 1-1. The watershed can be characterized as a mix of urban and industrial/commercial land uses and covers an area of approximately 34.3 square miles or about 21,938 acres. The DCA watershed has several conveyance outfalls and three boundaries within the project area. The watershed is separated into four subwatersheds: Delaney Creek, Delaney Pop-off Canal, North Archie Creek, and Archie Creek.

The watershed is generally bounded on the north by Palm River Road and the CSX railway, to the west by Hillsborough Bay, to the east by Valrico Road, and to the south by an imaginary line approximately 1/2 to 1/4 of a mile north of the Alafia River to the area of Buckhorn Springs, and then east along the dividing line between Townships 29 and 30 in Range 20. The watershed and its surrounding features are shown in Figure 1-2. Ultimately, water from the watershed discharges into Hillsborough Bay of Tampa Bay via Delaney, Archie, and North Archie Creeks and the Delaney Pop-off Canal. Both Delaney and North Archie Creeks have been highly channelized to alleviate flooding in the upper reaches of the creeks, primarily in the Brandon and Progress Village areas. In 1996, the Florida Department of Environmental Protection (FDEP) described Delaney Creek as having the worst water quality of all tributaries emptying into Tampa Bay. However, the same study indicated that trends in the creek’s water quality were improving.

The Delaney Creek subwatershed originates at a point approximately 4,000 feet north of Pauls Drive/Causeway Boulevard (S.R. 676) intersection and flows west approximately 8 miles to eventually discharge into Hillsborough Bay. The subwatershed has interconnection points with the Delaney Pop-off Canal subwatershed.

The Delaney Pop-off Canal subwatershed extends east to about U.S. Highway 301. The conveyance system consists of manmade ditches with no evidence of natural channel sections and generally flows south and west from U.S. Highway 301 to Hillsborough Bay. Major road crossings include U.S. Highway 301 at the eastern extremity, 78th Street near the middle, Madison Avenue (S.R. 676A) and U.S. Highway 41 at the western extremity. This subwatershed has interconnection points with Delaney Creek and North Archie Creek subwatersheds.

The North Archie Creek subwatershed extends east to Providence Avenue and north to the Lee Roy Selmon Expressway (formerly known as the Crosstown Expressway). This watershed is similar to the Delaney Pop-off Canal subwatershed in that it is drained by a system of manmade ditches and flows in a south and west direction to Hillsborough Bay. Some improvements and extensions to the ditch
system have been made in the eastern portions of the watershed as a result of Interstate 75 and U.S. Highway 301 construction. A portion of North Archie Creek west of 78th Street has been relocated and expanded by Gardinier, Inc. (now known as Cargill Fertilizer, Inc.). This subwatershed has interconnections between the Delaney Pop-off Canal and Archie Creek subwatersheds.

The Archie Creek subwatershed generally lies east of Hillsborough Bay, north of Riverview Drive and west of U.S. Highway 301. It is divided approximately into thirds by 78th Street and Interstate 75. The watershed is drained by a system of manmade ditches from east to west into Hillsborough Bay. A notable feature of this watershed is Cargill Fertilizer, Inc.’s large agricultural-chemical complex located west of 78th Street and east of U. S. 41. Cargill Fertilizer, Inc. has a 326-acre gypsum field to the north of the creek and south of the 238-acre cooling ponds (wastewater retention pond system) that will be expanded.

The climate in Hillsborough County is characterized as subtropical. The average annual rainfall is approximately 52 inches. The wet season is approximately four months long during the summer, usually beginning in June and ending in September. The summer is generally hot and humid with daily high temperatures in the 90's. Afternoon thunderstorms of high intensity and short duration are common during the wet season. Winter temperatures range from typical lows in the 40’s (°F) to highs in the 70’s (°F).

In general, soils in the watershed’s western, coastal segment are designated as poorly drained soils according to the classification system developed by the Natural Resources Conservation Service (NRCS) of United States Department of Agriculture (USDA) formerly known as United States Soil Conservation Service (SCS) for Hillsborough County. They are considered poorly drained in the watershed’s central region. The better-drained soils occupy the higher, eastern portions of the watershed. The DCA watershed is contained within the Gulf Coast Lowlands portion of the Midpeninsular Zone, one of the three geomorphic divisions of Florida (White, 1970).

Existing land uses within the watershed boundaries are diverse and include several large commercial uses (malls), office parks, light industries, a major port facility, major and minor roadways, residential subdivisions, golf courses and numerous types of agricultural activities. In addition, the DCA watershed contains commercial and residential sites. A few of these major urbanized locations are as follows: Evergreen Estates, Progress Village, Sanson Park, Pavilion, Waterford, Crescent Park, Lake Street Charles, Parkway Business Center, Cargill, and Starlite subdivisions.

Twenty-six projects with County vesting or Development of Regional Impact (DRI) status lie wholly or partially within the watershed. It is expected that the majority of land use changes will center on the conversion of the remaining agricultural and open land areas into mixed urban residential and commercial land uses.
While there have been no long term water quality studies on any of the watershed’s creek systems or any other water body in the basin, information for this report was gathered from a number of governmental agencies that do water quality sampling on some of these water bodies. Currently, the Hillsborough County LAKEWATCH program has several volunteers on lakes throughout the watershed. However, at the present time, no volunteers for the STREAMWATCH program exit.

Because some of the area was developed prior to the establishment of the majority of present day environmental laws, large blocks of the watershed lack stormwater treatment systems. Even though this development started early, large areas of natural systems can still be found within the watershed. The majority of these areas are around the watershed’s lakes, streams and on the coast. Because of this location, the expected listed species will be primarily wading birds, which can still use the wetland areas. Areas of significant or essential upland habitat as defined by the Hillsborough County Land Development Code (LDC) exist in the southwestern portion of the watershed. The natural areas that do exist have had significant habitat loss due to at least three factors. They are direct degradation and fragmentation by development and the introduction of exotic and invasive plants and animals.

The engineering firm of Ghioto, Singhofen and Associates, Inc. developed the previous Stormwater Management Master Plan (SMMP) for the Archie Creek watershed, in 1986. The current model and report are based on survey information provided by the 1986 report along with later surveys (1996 and 1999 performed by Hillsborough County Survey Department). The subbasin delineation was based on the latest available Southwest Florida Water Management District (SWFWMD) aerials with contours and in conjunction with “As-Built” or permitted construction plans.

Since the completion of the 1986 Archie Creek and Buckhorn Creek Stormwater Management Master Plan, the area has experienced a high volume of commercial and residential development. Some of these developments are the Falkenburg Road extension, extensive commercial development along Causeway Boulevard, the Lake Street Charles subdivision, the Parkway Business Center, Pavilion Commercial Center, and various commercial developments along 78th Street.

1.2 Background and Scope of the Project

The scope of the project includes the establishment of the existing conditions for the DCA stormwater management infrastructure in terms of the computed water surface elevations and discharge rates. Hydrodynamic mathematical models are an important tool to accomplish the analysis of quantity and quality concerns that result from urban stormwater runoff and combined sewer overflows. The Delaney Creek Area Watershed study was prepared using one of these first models, the EPA Stormwater Management Model (SWMM) and its Extended Transport (EXTRAN) Block, as modified and used by Hillsborough County Stormwater Management Section to better accommodate the specific characteristics of this county.
In this model, six standard storm events of 24-hour duration are used. The design storm events are as follows: 2.33-year (mean annual), 5-year, 10-year, 25-year, 50-year, and 100-year storm event respectively. Additionally, 120-hour storm simulations were run for 10-year, 50-year, and 100-year return frequencies at the request of SWFWMD.

The DCA major conveyance systems analysis is based on the computer model results (SWMM output files) which are graphically provided along with represented computed water surface profile. Various drainage improvements under construction or under the permitting process for construction during this study have been modeled as existing conditions and the results are reported herein.

Based on the results of the 25-year/24-hour existing condition storm event, Level of Service (LOS) analysis was performed and recommended improvements were examined for the proposed condition. These improvements include structural upgrades and non-structural improvements. All of these efforts will accomplish the required Level of Service, which is Level B in the DCA area.

1.3 Data Collection

To properly describe the existing condition of the watershed, available information was compiled from a variety of sources. These data included previous studies, existing aerial photographs and topography, latest land use coverage, recent ERP and construction plans, rainfall data, historical lake stage record, stream gage data, and flooding complaints information. The following agencies were involved during the data collection:

- Hillsborough County
- Southwest Florida Water Management District (SWFWMD)
- Florida Department of Environmental Protection (FDEP)
- Florida Department of Transportation (FDOT)
- Federal Emergency Management Agency (FEMA)
- United States Geological Survey (USGS)

In addition, site visits were conducted for certain areas to confirm the existing conditions within the Delaney Creek Area watershed.

The following is a discussion of the sources and a listing of the literature review:

Soil Survey of Hillsborough County

The soil data classifies soil types for engineering and planning purposes. This data was in the GIS format and provided by Hillsborough County.
**Land Use**
Existing land use coverage was in GIS format and provided by Hillsborough County as obtained from SWFWMD. This coverage is based on the Florida Land Use Cover Classification System (FLUCCS) 2006.

**Aerial Photography and Contour Maps**
As part of this study, latest aerial photographs (2006) and 1-foot digital contours (2004) were obtained from Hillsborough County.

**Existing Studies**
A literature search of relevant documents was performed to collect useful information pertaining to the study area. The literature search yielded the following:

- Delaney Creek Area Stormwater Management Master Plan, Hillsborough County, September 2000
- Delaney Creek Area Watershed Model Review, prepared by Singhofen & Associates, Inc. for SWFWMD, January 2005
- Flood in Southwest-Central Florida from Hurricane Frances, USGS, September 2004

**Consultant Models and Permitting Reports**
Several consultant models and permitting reports were made available through the County in areas of recent or imminent development. These data were valuable where more recent survey was desired or where private developers had investigated the local drainage network in greater detail to support their own design modeling. Although all such data was handled as unverified or provisional, Ayres utilized any certified surveyed elevations and structure data that could be reasonable verified by other data sources (e.g., topography, aerials, field review). The 2007 update model input deck includes comments to identify sources where such data have been incorporated. The following sources were used, in part, to update specific areas:

- BCI survey and model data were used as a basis to update the Delaney Pop-off drainage system near Pendola Point. Additional hydraulic detail was added between Pendola Point Road-Madison Avenue and Dover Street, in the vicinity of South 50th Street.

- Sanson Park/Canterbury Lakes Regional Detention facility structures were appropriated from the previous Watershed Management Plan proposed conditions model (2001 WMP referenced source: Parsons modeling files).

- Hills & Associates survey data and portions of H.T. Mai model input based on 2006 survey associated with Bloomingdale Apartments were utilized to update structure inverts and channels for North Archie Creek in the vicinity of Progress Boulevard between U.S. Highway 301 and South Falkenburg Road.
• H.T. Mai model input was utilized in conjunction with Cargill permitting plans to describe the Archie Creek main channel improvements (Cargill diversionary channel) and replacement culverts under Old U.S. 41.

• Genesis survey data was utilized to verify culvert inverts and locations and roadway overtopping elevations along South Falkenburg Road in the Archie Creek subwatershed.

**Environmental Resource Permits (ERP)**

Figure 1-3 and Table 1-1 illustrate the ERPs evaluated by Ayres staff during the 2007 model update.

**Problem Area Documentation**

Documentation for the reported flood prone areas was obtained through County records. These records were in GIS format and related to the complaints and locations associated with Hurricane Frances, during the month of September of 2004. In addition, a limited document search was performed through SWFWMD and USGS regarding this event.

1.4 Report Organization

This report is organized into seven chapters describing the existing condition update within the watershed:

• Chapter 1 provides an introduction and an overview of the report along with a description of objectives;
• Chapter 2 provides an overview of the watershed including major environmental features related to stormwater management;
• Chapter 3 describes the major conveyance systems within the watershed;
• Chapter 4 explains the hydrologic/hydraulic model methodology;
• Chapter 5 characterizes the hydrologic/hydraulic model calibration and verification;
• Chapter 6 describes the existing conditions level of service along with analysis and designations;
• Chapter 7 discusses flood control alternatives update.

In addition, there are various tables and figures consisting of illustrations, graphics, drawings, etc., presented at the end of the report as appendices. The computer model input/output and GIS files developed during the study are available, and are attached to this report.

Any questions, comments, or other inquiries regarding the content of this report or the project in general should be directed to the Hillsborough County Public Works Department, Engineering Division, Stormwater Management Section.
CHAPTER 2 WATERSHED DESCRIPTION

2.1 Overview

The Delaney Creek Area watershed drains approximately 34.3 square miles or 21,938 acres in central Hillsborough County, Florida (Figure 1-1). The watershed is fairly evenly mixed between manmade land uses such as residential and commercial uses and natural systems. The watershed drains into Hillsborough Bay with four main outfalls: Delaney Creek, the Delaney Pop-off Canal, Archie Creek, and North Archie Creek. The eastern and northwestern portions of the watershed are primarily residential and include portions of the City of Brandon. The western and central portions of the basin contain more of the agricultural and natural areas. The watershed is generally bounded on the north by Palm River Road and the CSX railway, to the west by Hillsborough Bay, to the east by Valrico Road and to the south by an imaginary line approximately 1/2 to 1/4 of a mile north of the Alafia River to the area of Buckhorn Springs and then east along the dividing line between Townships 29 and 30 in Range 20. Several large roads, including Interstate 75, U.S. Highways 41 and 301, State Road 60 (Brandon Boulevard) and Causeway Boulevard/Lumsden Avenue, bisect the watershed. Topography varies from a high of over 100 feet North American Vertical Datum (NAVD) in the extreme southwestern and eastern portions of the watershed to a low of sea level along the coast and at the outfalls to Hillsborough Bay as depicted in Figure 2-1.

The major natural features of the watershed are Delaney, Archie and North Archie Creeks. All the watershed’s drainage passes through these creeks and the Delaney Pop-off Canal on its way to Hillsborough Bay. Delaney Creek is approximately 8 miles long, while North Archie Creek is approximately 5 miles long. Both streams have been highly channelized over the years, primarily as a method of flood control for the residential areas of Brandon and Clair Mel. In addition to the creeks, several large lakes occur in the watershed including: Gornto, Chapman, Tenmile, Clayton, Hendrics, Hickory Hammock, and Kathy. Additionally, numerous smaller waterbodies such as Sand Pond and many borrow pits can be found. All these waterbodies are considered Class III waters with designated uses that include human recreation and the “propagation and maintenance of a healthy, well-balanced population of fish and wildlife” (Chapter 62-302.400, Florida Administrative Code).

Land uses within the watershed boundaries are diverse and include several large commercial malls, office parks, areas of light industry, major and minor roadways, residential subdivisions, golf courses, natural areas, and agriculture.
2.2 Climate

The climate of the Delaney Creek Area watershed, and Hillsborough County as a whole, is classified as humid subtropical. Annual average precipitation is around 52 inches. Approximately 60% of this total falls during the four-month rainy season that extends from June through September. This rainy season also coincides with the occurrence of most tropical storms and hurricanes. Additionally, the conditions are ripe for regular, convective afternoon and evening thunderstorms. These summer events, which can be very localized, are highly variable in both intensity and volume. The larger, normal summer storm events and those associated with tropical systems can cause flooding problems in areas where there are deficiencies in the existing stormwater or other drainage systems.

Winter rainfall is, for the most part, relatively light and generally associated with the weak cold fronts that descend from the northern part of the country and travel south through the region. However, some of the largest yearly rain events have occurred in the winter months. This is especially true in El Niño years.

The annual mean temperature in Hillsborough County is about 72°F (Fahrenheit). The mean monthly temperature ranges from a low of approximately 60°F in January to a high of approximately 82°F in August. Typically, summer temperatures range from morning lows in the high 70's and low 80's to afternoon highs that can easily reach into the mid-90's, but rarely do they exceed 100°F. Summer humidity that ranges into the mid to upper 90's can further exacerbate the situation. Conversely, typical winter low temperatures generally range above freezing into the 40's, only occasionally dropping into the low 20's and teens. High temperatures generally reach into the upper 60's or low 70's for most of the season, especially between passages of the cold fronts.

According to the National Weather Service in Ruskin, humidity does not vary as seasonally as temperature and rainfall. The Service keeps daily records for 1 and 7 o’clock A.M. and 1 and 7 o’clock P.M. The 7 A.M. time period generally records the highest humidity with the annual average at 88% with the 1 P.M. time period recording the lowest at an average of 58%.

Evapotranspiration rates vary and limited data is available for analysis. Estimates of 39 inches per year have been reported. Viessman, et al. (1977) reports the figure to be closer to 48 inches per year. Lake evaporation data often quoted for use in Hillsborough County are those reported from Lake Alfred in Polk County, supplemented by scattered data available from the Lake Padgett weather station. Studies conducted by Tampa Bay Water estimate the lake evaporation rate to average approximately 56 inches per year.

2.3 Soils

Soil distribution map is shown in Figure 2-2. According to the Natural Resource Conservation Service (NRCS) classification, there are nearly 40 different types of soils that occur within the Delaney Creek
Area. This information was developed based on GIS coverages developed by SWFWMD. Much useful information, such as drainage classification, percent slope, water table depth, permeability, natural vegetation and potential uses for development and agriculture, can be ascertained by consulting the Soil Conservation Service’s manual for Hillsborough County for each particular soil type.

Hydrologic Soil Group (HSG) is commonly used for hydrologic analysis to estimate infiltration rates and soil moisture capacities. Typically, soils are grouped into four categories, A through D, with the runoff potential increasing. The hydrologic groups are commonly used in watershed planning to estimate infiltration rates and moisture capacity. Soil properties that influence the minimum rate of infiltration obtained for bare soil after prolonged wetting are: depth to seasonally high water table, intake rate and permeability, and depth to a layer or layers that slow or impede water movement. The major soil hydrologic groups are:

- **Hydrologic Soil Group A (low runoff potential):** These soils have high infiltration rates and a high rate of water transmission even when fully saturated. They have typical infiltration rates of 10 inches/hour when dry and 0.50 inches/hour when saturated. Soil types found in the Delaney Creek Area watershed that fall into this group include: Archbold fine sand (3), Candler fine sand (7), Candler-Urban land complex (9), Fort Meade loamy fine sand (18), Gainesville loamy fine sand (19), Kendrick fine sand (23), Lake fine sand (25), Orlando fine sand (35), and Tavares-Millhopper fine sands (53 & 54).

- **Hydrologic Soil Group B (moderately low runoff potential):** These soils have moderate infiltration rates when thoroughly wet and a moderate rate of water transmission. They have typical infiltration rates of 8 inches/hour when dry and 0.40 inches/hour when saturated.

- **Hydrologic Soil Group C (moderately high runoff potential):** These soils have low infiltration rates when thoroughly wet and a low rate of water transmission. They have typical infiltration rates of 5 inches/hour when dry and 0.25 inches/hour when saturated. Soil types found in the Delaney Creek Area watershed that fall into this group include Gypsum land (20), Pomello fine sand (41), Pomello-Urban land (42), Seffner fine sand (47), Street Augustine fine sand (44), Street Augustine-Urban land complex (45), and Zolfo fine sand (61).

- **Hydrologic Soil Group D (high runoff potential):** These soils have very slow infiltration rates when thoroughly wet and a very low rate of water transmission. They have typical infiltration rates of 3 inches/hour when dry and 0.10 inches/hour when saturated. Soil types found in the Delaney Creek Area watershed that fall within this group include: Basinger/Holopaw/and Samsula soils/depressional (5), Kesson muck (24), and Myakka fine sand (30).

Dual classifications (e.g., A/D or B/D) can be assigned to soils that exhibit substantially different hydrologic characteristics during the wet and dry seasons or if extensively and effectively drained such as by groundwater interception trenches or deep, artificial channels. During the wet season, these soils become saturated throughout much of the soil column due to elevated water table conditions.
Infiltration is thus impeded and the soils exhibit Group D infiltration and runoff rates. During the dry season when the water levels recede, infiltration rates increase and runoff rates decline to Group A or Group B levels. Soil types that fall within the B/D classification found in the Delaney Creek Area watershed are: Chobee loamy fine sand (10), Felda fine sand (15), Floridana fine sand (17), Immokalee fine sand (21), Malabar fine sand (27), Myakka fine sand (29), Myakka-Urban land complex (32), Ona fine sand (33), Pinellas fine sand (38), Street Johns fine sand (46), Smyrna fine sand (52), Wabasso fine sand (57) and Winder fine sands (59 & 60).

Arents soils, both nearly level (4) and very steep (39), as well as Quartzipsammements, nearly level (43), are not assigned a hydrologic soils group due to the highly disturbed nature of these typically urban or mined soils.

Soils can also be classified as either hydric or non-hydric, which relates to whether the soils had wetland or upland origins, respectively. Those soils designated as hydric develop under anaerobic conditions in wetland areas and generally contain a large amount of organics. They are poorly to very poorly drained or depressional in nature, and are associated with a high seasonal water table. In contrast, those soils that are non-hydric lack these characteristics and are associated with upland or transitional areas. Soil types with the hydric classification found within the Delaney Creek Area watershed are Basinger/Holopaw/and Samsula soils/depressional (5), Chobee loamy fine sand (10), Felda fine sand (15), Floridana fine sand (17), Kesson muck (24), Malabar fine sand (27), Myakka fine sand (30), Street Johns fine sand (46) and Winder fine sand (60). All other types would be considered non-hydric.

The distribution of hydrologic soil groups is illustrated in Figure 2-3 and Table 2-1.

### 2.4 Land Use/Cover

This information was gathered from two primary sources. The existing land use was determined by using 2006 SWFWMD GIS database and based on the Florida Land Use and Cover Classification System (FLUCCS). The future land use information is from the Hillsborough County Planning Commission and reflects the predicted land use for the year 2015.

#### 2.4.1 Existing Land Use

The Delaney Creek Area watershed is diverse in terms of land uses, being made up of a combination of residential, agricultural, commercial and natural systems. The latest aerial photographs (2006) and major ERP plans since 2000 were incorporated to furnish the existing land use update. The modified 2006 land use/cover within the watershed is presented in Figure 2-4. Commercial, industrial, institutional and highway/utility land uses constitute approximately 4,800 acres in the watershed. These land uses are found primarily along the basin's major roads - U.S. Highways 41 and 301, State Road 60 and Causeway BoulevaRoad. Residential land uses comprise nearly 6,800 acres and include areas of low, medium and high density.
These uses are scattered throughout the basin, with the bulk of the medium density areas in the extreme eastern portion of the watershed. Many of the watershed’s residential areas tend to be older subdivisions with little or no stormwater treatment being provided. The lots are typically between a quarter of an acre to an acre in size. Agricultural land uses encompass about 1,800 acres in the watershed. The majority of the agricultural uses are made up of pasture and croplands found in the central portion of the watershed around U.S. Highway 301 and Interstate 75.

Other land uses in the agricultural category are feeding operations, specialty farms, tree crops, tropical fish farms and other open lands (rural). Undeveloped lands, including open land and water, and upland and wetland natural systems account for around 6,400 acres of the watershed. Again, these areas are scattered throughout the basin. Included in this grouping are both saltwater and freshwater forested and non-forested wetlands, forested uplands, waterbodies and open lands. The remainder of the watershed is made up of recreational land uses and mining or mining related activities, primarily the gypsum operations in the southwest portion of the watershed. As shown in Figure 2-6, there are numerous vested projects or Developments of Regional Impact (DRI) projects within the watershed. These occur primarily in the area of the basin’s major roads, Interstate 75 and U.S. Highway 301 in the central portion of the watershed and in the eastern portion of the area. Much of the significant or essential upland wildlife habitats also can be found in the coastal area, with the remainder existing in the southwest quarter of the watershed area.

Table 2-2 indicates that of the nearly 22,000+ acres in the watershed, 15,000+ acres or 70% of the watershed has been developed. A composite breakdown of acreage and percentage for each type of land use within the watershed is also presented in the table.

2.4.2 Future Land Use

Figure 2-5 (adopted from Hillsborough County Stormwater Management Master Plan, September 2000) shows the Planning Commission’s projected land use for the year 2015. Due to the highly developed nature of the Delaney Creek Area watershed, not many changes in land use are predicted by Hillsborough County’s Planning Commission projections for the year 2015. The majority of predicted changes will be associated with the agricultural and open land areas and will most likely change over to a mixed urban use of residential and light commercial land uses.

Table 2-3 shows a composite breakdown of acreage and percentage for each type of future land use within the watershed. As seen from Table 2-3, several land uses that occurred in the existing land use table (Table 2-2) are not found in Table 2-3. In some cases, this is because that land use has been converted to another type. For example, it can be expected that at some point in the future that much of the agriculture use will be developed for residential, commercial or similar land uses. Open land would fall under this category as well. However, other designations such as commercial and highway utilities are not shown. These land uses, designated by an asterisk (*), still exist in the watershed but due to the way the Planning Commission compiles their information, they are incorporated into...
another land use category. For example, the R-20 category in Figure 2-5 includes commercial land uses, but for the purposes of Table 2-3, the R-20 designation has been included in the high-density residential category. There is no accurate way to remove the commercial land uses from this designation. Wetlands are treated in a similar manner. With present day wetland protection regulations, it is not likely that the over 1,500 acres of wetland will disappear. However, the Planning Commission does not treat wetlands as a land use; they are a form of land cover. Because of this, again they have been incorporated into another land use category. The acreage indicated in the table are only those natural areas that are under conservation easement or other type of preservation.

2.5 Physiography and Hydrology

The Delaney Creek Area watershed lies within the Gulf Coast Lowlands physiographic unit as defined by White (1970). This unit is part of the Central or Mid-Peninsular physiographic zone, one of three in Florida. Land elevations in the watershed vary between a high of over 100 feet NAVD in the eastern portions of the watershed to a low of sea level along the coast and at the three outfalls. These elevations are shown on Figure 2-1.

Surface flows are generally from east to west or southwest toward Hillsborough Bay following the natural topography within the basin. Hydrologically, surface flows originate for the most part through stormwater runoff with very little influence from groundwater flows, with the exception of the two major creek systems.

The watershed has four major outfalls, the three creeks and the Delaney Pop-off Canal, which discharge into Hillsborough Bay. In addition to these outfalls, each of the creek systems has major stormwater conveyance systems associated with them.

The Delaney Creek system is the largest of the three, with eight major conveyance systems. The westernmost of these is Tributary “A”, a ditched natural system that has its origins to the south of Palm River Road and to the east of Maydell Drive. This tributary drains an area of approximately 550 acres. A series of seven manmade “laterals” then drain into Delaney Creek. All but the final lateral enters the creek on the north side. They aid in routing flood/stormwater away from the residential areas of Clair Mel and west Brandon. The first three of these laterals, denoted “A”, “B”, and “C”, are used to drain the Clair Mel area between 70th Street and Hobbs Road (90th Street). Excavated materials from these laterals may have been used as fill in the construction of the adjacent residential areas of Clair Mel. The next three, laterals “D”, “E”, and “F”, drain mostly undeveloped lands between Interstate 75 and U.S. Highway 301. However, this area will be occupied by the Crosstown Center in the near future. The seventh and final lateral, designated as “G”, enters Delaney Creek from the south between Providence Road and Interstate 75.

The North Archie Creek system is similar to the one for Delaney Creek. It has seven major conveyance systems associated with it. The most downstream of these, Tributaries “B” and “C”, drain
a large area west of the intersection of U.S. 301 and Interstate 75 and east of 78th Street. Draining the area west of Interstate 75 are Tributaries “E”, “F”, and “G”. To the north, Tributaries “I” and “J” drain the area north of the intersection of U.S. Highway 301 and Interstate 75 between those two main roads.

Finally, the system for the Delaney Pop-off Canal drains the area between the two creek systems. Like the North Archie Creek system, it is made up of seven major conveyance systems. Tributaries “A”, “E” and “F” drain that part of the watershed to the west of 78th Street. Of the portion of the watershed remaining to the east of 78th Street, Tributaries “B” and “C” drain the northern portion, with Tributaries “G” and “H” draining the southern section.

### 2.6 Geology and Hydrogeology

The Delaney Creek Area watershed is underlain by a thick sequence of sedimentary strata divided into an upper zone of unconsolidated sediments and lower zone of consolidated carbonate rock.

At land surface, undifferentiated sediments including silt, sand, and clay, form surficial deposits that vary in thickness from less than 10 feet in coastal areas to over 100 feet in paleokarst depressional areas or in sand ridges. Typical thickness of the surficial deposits varies from 20 to 50 feet. In lower lying areas near lakes and streams, thin layers of organic material mix with the surficial deposits. Pleistocene-aged silts and clays form the base of the undifferentiated sediments.

Underlying the unconsolidated material is a series of Tertiary-aged limestones and dolomites that form the carbonate platform of peninsular Florida. The sequence of carbonate rocks includes, in descending order, the following formations: Tampa Member of the Hawthorn Group, Suwannee Limestone, Ocala Group, Avon Park, Oldsmar and Cedar Key Formations. A lithographic change from limestone and dolomite to a sequence of gypsiferous dolomite begins in the lower portion of the Avon Park Formation and continues into the Oldsmar and Cedar Key Formations. The top of this lithologic change marks the middle confining unit of the Floridan aquifer system. The middle confining unit is generally considered the base of the freshwater production zone of the Upper Floridan aquifer.

The Tampa Member of the Hawthorn Group is a tan-colored carbonate and sand mixture, which can contain variable amounts of clay. The Tampa Member can be fossiliferous and may also contain phosphate grains and chert. The Tampa Member ranges from 50 to 150 feet in thickness. The Suwannee Limestone consists of two rock types; the upper portion is tan-colored crystalline, limestone containing prominent gastropod and pelecypod molds and the lower portion is cream-colored limestone containing foraminifera and pellets of micrite in a finely crystalline limestone matrix. The Suwannee Limestone varies from 150 to 300 feet in thickness.

The Ocala Group contains a series of limestones that are generally soft, friable, porous and fossiliferous. This unit is late Eocene in age and ranges in thickness from 90 to 300 feet. The Avon Park Formation
comprises brown, highly fossiliferous, and soft to well-indurated, chalky limestone and a gray to brown, very fine microcrystalline dolomite. The Avon Park Formation ranges from 300 to 500 feet in thickness.

The hydrogeologic flow system of the Tampa Bay region contains two distinct groundwater reservoirs: the unconfined surficial aquifer and the semi-confined Upper Floridan aquifer. The Upper Floridan aquifer is under water table conditions in areas where the clay confining layer is discontinuous or absent. A general hydrogeologic cross-section of the Tampa Bay region is shown in Figure 2-7.

2.6.1 Surficial Aquifer

The surficial aquifer is comprised primarily of unconsolidated deposits of fine-grained sand with an average thickness of 30 feet. Due to the karst geology of the region, thickness of the sand is highly variable. The depth of the water table ranges from near land surface to several tens of feet below land surface. Rainfall is the primary influence on water table elevation; with annual highs in most years occurring during the end of the wet season (September through October) and annual lows occurring near the end of the dry season (May through June). The direction of groundwater flow varies locally and is significantly influenced by the topography of the land surface. The hydraulic gradient (change of elevation per unit length) in the area typically ranges from a few feet per mile to about 10 feet per mile. The permeability of the surficial aquifer is generally low and water withdrawn from this aquifer is used most often for lawn irrigation and watering livestock. Surficial aquifer wells typically yield less than 20 gallons per minute.

2.6.2 Semi-Confining Zone

Below the surficial aquifer is a semi-confining unit comprised of clay, silt and sandy clay that has the ability to retard the movement of water between the overlying surficial aquifer and the underlying Upper Floridan aquifer. The confining materials are comprised of blue-green to gray, waxy, plastic, sandy clay and clay. The upper portion of the Arcadia Formation (Hawthorn Group) typically forms the semi-confining layer.

Leakage from the surficial aquifer into the Upper Floridan aquifer occurs by infiltration across the semi-confining layer or through fractures or secondary openings in the semi-confining unit caused by chemical dissolution of the underlying limestone. Due to the highly karstic nature of the geologic system, the clay semi-confining layer can be absent in one area but tens of feet thick just a short distance away. These localized karst features, in which the clay semi-confining layer is breached or missing, significantly increases hydraulic connection between the two aquifers (Hancock and Smith, 1996).
2.6.3 Upper Floridan Aquifer

The Upper Floridan aquifer consists of a continuous series of carbonate units that include portions of the Tampa Member of the Hawthorn Group, Suwannee Limestone, Ocala Limestone, and the Avon Park Formation. Groundwater within the Upper Floridan aquifer is typically under artesian conditions within the project area.

Near the base of the Avon Park Formation lies the middle confining unit of the Floridan aquifer, an evaporite sequence of very low permeability that is composed of gypsiferous dolomite and dolomitic limestone.

The middle confining unit generally delineates the boundary between the freshwater Upper Floridan aquifer and the brine-saturated Lower Floridan aquifer. The evaporites function as a lower confining unit and retard vertical flow across the boundary. In general, the permeability of the Upper Floridan aquifer is moderate in the Tampa Member and Suwannee Limestone, low in the Ocala Limestone and very high in portions of the Avon Park Formation. The limestone and dolomite beds produce significant quantities of water due largely to numerous solution openings along bedding planes and fractures. The Ocala Limestone yields limited amounts of water and may be considered a semi-confining layer within the Upper Floridan aquifer. Overall, the Ocala Limestone tends to act as a semi-confining zone between the overlying Tampa/Suwannee Formations and the underlying Avon Park Formation. Transmissivity of the Avon Park Formation is very high due to the fractured nature of the dolomite zones.

Ground water flow in the Floridan aquifer originates as rainfall that percolates downward from the surficial aquifer. In areas where the Upper Floridan aquifer outcrops, this recharge can be direct. Recharge rates are generally higher in the northern portion of the County. However, recharge can be highly variable throughout the area, due to karst geology and induced leakage caused by ground-water withdrawals. The regional hydraulic gradient and direction of flow in the Upper Floridan aquifer is generally toward the south and west.
CHAPTER 3 MAJOR CONVEYANCE SYSTEMS

The existing condition system performance for the major conveyance systems is contained in this chapter. The description of major conveyance systems in the DCA watershed has been segmented into four subwatersheds: Delaney Creek, Delaney Pop-off Canal, North Archie Creek, and Archie Creek, as shown in Table 3-1 and Figure 3-1.

3.1 Delaney Creek Subwatershed

The Delaney Creek subwatershed originates at a point approximately 4,000 feet north of Pauls Drive/East Lumsden Road (State Road 676) intersection and flows west approximately 8.0 miles to eventually discharge out to Hillsborough Bay.

3.1.1 Delaney Creek Main Channel System (model # 210xxx series)

The Delaney Creek Main Channel System is considered with this study starting from south of Brandon Town Center Mall area, at the Hendrics Lake System confluence with Hickory Hammock System (Model Junction ID 210361) to the outfall location in Hillsborough Bay. The Delaney Creek Main Channel is a well-defined channel which crosses Interstate 75, U.S. Highway 301, the Crosstown Expressway, 86th Street South, 78th Street South, Causeway Boulevard, 70th Street South, Maydell Drive, 37th Avenue South, the CSX Transportation System (twice) and U.S. Highway 41, ultimately discharging into Hillsborough Bay just north of Pendola Point Road/Port Sutton area.

3.1.2 Laterals

The Delaney Creek main channel has various laterals that converge into the main channel. Most of the laterals are manmade and run north and south behind back lots of residential subdivisions. The laterals for the Delaney Creek subwatershed flow from north to south into the main channel. Several of the laterals are manmade ditches that were excavated originally for agricultural purposes but now serve as drainage ditches for various subdivisions as mentioned in the main channel system.

3.1.2.1 Lateral “A” (model # 211xxx series)

A natural tributary joins Delaney Creek approximately 3,000 feet west of Maydell Drive. This tributary drains the area north of Causeway Boulevard and flows in a north to south direction, crossing Causeway Boulevard, Maydell Drive, 20th Avenue South, 16th Avenue South, and 12th Avenue South.
3.1.2.2  Lateral “A-1” (model # 2115xx series)

A manmade ditch system joins Delaney Creek approximately 1,300 feet East of Maydell Drive. This tributary drains the area south of Causeway Boulevard and flows in a south to north direction, crossing 36th Avenue South and 32nd Avenue before the confluence with Delaney Creek.

3.1.2.3  Lateral “B” (model # 212xxx series)

Lateral “B” is a large lateral ditch draining into Delaney Creek from the north at a 90° turn. Drainage from the western Clair Mel City area is collected between 78th Street and Causeway Boulevard in this lateral. The channel flows south and crosses Tidewater Trail, Robindale Road, and 12th Avenue South to the confluence with Delaney Creek.

3.1.2.4  Lateral “C” (model # 213xxx series)

Lateral “C” is located less than 1,000 feet upstream of the 78th Street crossing. The channel generally flows south and drains a portion of the Clair Mel City area. The channel flows south and crosses Tidewater Trail and Rideout Road twice before its confluence with Delaney Creek.

3.1.2.5  Lateral “C-1” (model # 2135xx series)

Lateral “C-1” is located approximately 800 feet west of Hobbs Road and drains the Green Ridge subdivision which is located to the north of Delaney Creek. Residential homes are located on the east side of the lateral along with a power transmission line easement. The channel flows south and crosses Tidewater Trail to the confluence with Delaney Creek.

3.1.2.6  Lateral “D” (model # 214xxx series)

Lateral “D” is located south of Adamo Drive and receives drainage from the Tampa Central Park commercial site. The channel is generally parallel to U.S. Highway 301 on the east and flows south crossing Palm River Road to the confluence with Delaney Creek.

3.1.2.7  Lateral “E” (model # 2150xx series)

Lateral “E” originates south of the CSX railroad located in the Interstate Park of Commerce. The channel generally flows south, crossing Adamo Drive and Palm River Road to the confluence with Delaney Creek.

3.1.2.8  Lateral “E-1” (model # 2155xx and 2156xx series)

Lateral “E-1” receives flow from an unnamed lake and adjacent developed area located north of State Road 60. Drainage from the unnamed lake flows through a swamp on the west side of Interstate 75 before entering Lateral “E-1”. This agricultural channel has been excavated and spoil material has been
placed adjacent to the channel. Falkenburg Road has been constructed recently to connect Causeway Boulevard (Lumsden Road) to Adamo Drive (State Road 60).

3.1.2.9 Lateral “F” (model # 216xxx series)

Lateral “F” originates south of the Crosstown Expressway and flows north through open pastures and meets with the main channel near Falkenburg Road. This agricultural channel has been excavated and spoil material has been placed adjacent to the channel.

3.1.2.10 Hendries Lake System (model # 220xxx, 223xxx and 225xxx series)

The Hendries Lake System begins east of Parsons Avenue between Brandon Boulevard (State Road 60) and Lumsden Road. A freshwater marsh is located north of Hendries Lake and is connected to it through a small ditch. The marsh receives drainage from areas east of Bryan Road under extreme rainfall events. Hendries Lake is connected to Clayton Lake with a concrete culvert crossing under Parsons Avenue. Clayton Lake then discharges into Doctor’s Pond (Clayton Lake Addition) crossing Vonderburg Drive. From Doctor’s Pond, the flow continues west crossing Kings Avenue to manmade ponds south of Oakfield Drive between Pauls Drive and Kings Avenue. From this location, the water’s path changes towards the south on a natural channel ultimately reaching the Brandon Town Center area between Town Center Boulevard and Gornto Lake Road after crossing Pauls Drive, Lakewood Drive, Providence Road, and Town Center Boulevard, respectively.

3.1.2.11 Brandon Town Center Mall (model # 221xxx and 217xxx series)

The Brandon Town Center Mall system consists of two internal systems that are made up of interconnected ponds and pipes flowing south to join with the Delaney Creek main channel. One system meets with the main channel behind the car dealership property on the west. The second system runs southeast near the Gornto Lake Drive/Brandon Town Center Boulevard intersection on the east and joins with the main channel.

3.1.2.12 Gornto Lake System (model # 2215xx series)

The Gornto Lake system is part of the Hendries Lakes main system and consists of Lake Gornto, Lake Chapman, and Tenmile Lake. Tenmile Lake has a small swamp along a portion of its southern boundary. In addition, temperate and upland areas are located on the southeast side of Tenmile Lake. Improved pasture borders a major portion of both Gornto and Chapman Lakes. A weir outfall for Tenmile Lake exists at the south part of the lake. Flows over this weir are conveyed through a swale/ditch system to a 60-inch diameter culvert under State Road 60 and then into a lake located within the Old Times Square development. This lake serves as a detention pond and is controlled by an overflow structure. Under extreme storm events, the drainage area located north of CSX Transportation System discharges to Lake Chapman via a concrete culvert crossing the railroad tracks and Camp Florida Road.
3.1.2.13 Heather Lakes System (model # 222xxx series)

Heather Lakes is a major tributary to the Hendrics Lake system. The confluence point is downstream of its Providence Road crossing. Heather Lakes is primarily a residential development located north and south of Lumsden Road between Pauls Drive and Providence Road. The development consists of a series of interconnected lakes, storm sewers, and culverts. The entire lake system crosses Lumsden Road at two locations. Both crossings by-pass the natural channels located on the north and south side of Lumsden Road (the south channel is named Lateral “G” in this study). Easterly, the location connects the two Heather Lakes directly via a concrete culvert. Westerly, Lumsden Road crossing runs along Providence Road collecting the roadway drainage north of a controlled structure, which discharges to Delaney Creek along the northeast side of the development. From north Heather Lakes, the flow path is along Providence Road heading north to the confluence downstream of Delaney Creek (Hendrics Lake System), crossing Providence Road.

3.1.2.14 Lumsden Road North Ditch (model # 224xxx series)

The Lumsden North ditch consists of an area north of Lumsden Road at a gas station near the Lumsden Road/Kings Avenue intersection. The ditch flows west until it reaches Pauls Drive and then heads north into the Delaney Creek channel.

3.1.2.15 Hickory Hammock System (model # 230xxx series)

Lateral “G” flows east to west from Kings Avenue to the crossing under Lumsden Road for a distance of approximately 9,500 feet. Providence Road crosses this lateral approximately 1,300 feet east of the Lumsden Road crossing. Lateral “G” receives drainage from areas east of John Moore Road and Hickory Lake as well as areas to the south of Hickory Lake. The only outlet to this system is through a pair of 18-inch diameter culverts under Kings Avenue which ties into Lateral “G”.

3.1.2.16 Isolated System (model # 227xxx series)

The isolated basins consist of the area east of Kingsway Road and the Hickory Hammock Lake system. These basins encompass various subdivisions and commercial sites like Brentwood Hills, the Nativity Catholic School, and Salondino Park. The subbasins are not connected to the Delaney Creek main channel but to several systems like the Delaney Pop-off Canal, Archie Creek, and North Archie Creek.

3.2 Delaney Pop-off Canal Subwatershed

The Delaney Pop-off Canal subwatershed originates in the Tampa Triangle area and receives drainage from Evergreen Estates, Aspen Cove Apartments, Crescent Commercial Park, Madison Estates Mobile Home Park, Sanson Park, Waterford, and the Pavilion subdivisions.
3.2.1 Delaney Pop-off Main Channel System (model # 240xxx series)

The Delaney Pop-off Main Channel System starts from U.S. Highway 301 near Tampa Triangle to the outfall location at Hillsborough Bay. The Delaney Pop-off Main Channel is a manmade canal that was constructed to alleviate the flooding problems of the Delaney Creek area located north of Delaney Pop-off Canal.

3.2.2 Tributaries

The tributaries of the Delaney Pop-off Canal subwatershed consist mostly of manmade ditches along the back lots of residential areas like Fortuna Acres and Green Ridge Estates. The remaining tributaries on the east side of U.S. Highway 301 consist of the Interstate 75 drainage ditch and several ditch systems located in subdivisions of the Pavilion areas.

3.2.2.1 Tributary “A” System (model # 243xxx series)

Tributary “A” originates south of 36th Avenue and also collects flow from a small residential area north of 36th Avenue. The channel flows approximately 2,500 feet south to the confluence with the Delaney Pop-off main channel.

3.2.2.2 Tributary “B” System (model # 247xxx series)

Tributary “B” originates in the Sanson Park subdivision located south of Causeway Boulevard and east of 78th Street. The channel generally flows east for approximately 2,500 feet to eventually meet with the Delaney Pop-off channel and Tributary “C” from the west.

3.2.2.3 Tributary “C” System (model # 2403xx and 2404xxx series)

Tributary “C” originates south of Causeway Boulevard near the Boulevard Villas subdivision, which is located south of Causeway Boulevard and east of U.S. Highway 301. The channel flows south from this point along the U.S. Highway 301’s east roadside ditch and then west along Falkenburg Road to the confluence with the Delaney Pop-off main channel at South 86th Street (Wellborn Way). The portion of Tributary “C” along Falkenburg Road collects drainage from the Pavilion and Waterford subdivision.

3.2.2.4 Tributary “E” System (model # 2420xx series)

Tributary “E” originates to the west of 78th Street and south of Madison Avenue. The channel generally flows west for approximately 2,600 feet to eventually meet with the Delaney Pop-off channel at the Madison Estates Mobile Home Park.
3.2.2.5 Tributary “F” System (model # 2425xx series)

Tributary “F” flows west and collects drainage from a mobile home park to the east of Marc Drive located between the backyards of 50th and 51st Avenue lots.

3.2.2.6 Tributary “G” System (model # 246xxx series)

Tributary “G” is located just west of U.S. Highway 301 and south of the Waterford subdivision. The channel flows west for approximately 3,000 feet before its confluence with the Delaney Pop-off Canal.

3.2.2.7 Tributary “H” System (model # 2465xx series)

Tributary “H” originates in the Waterford subdivision and flows west approximately 2,000 feet away from the Delaney Pop-off Canal. The channel collects drainage from the subdivision via treatment ponds located north of the channel.

3.2.2.8 Tributary “I” System (model # 2478xx series)

Tributary “I” originates west of U.S. Highway 301 and flows east along the Falkenburg Road ditch into Crescent Park. The channel’s confluence with the Delaney Pop-off Canal is approximately 3,500 feet away from Interstate 75.

3.2.2.9 Aspen Cove Apartment System (model # 250xxx series)

The Aspen Cove Apartment System originates south of Causeway Boulevard and west of the Falkenburg Road/Crosstown Expressway exit. This system collects the drainage for the Aspen Cove Apartment complex through various treatment ponds and eventually discharges south into the Delaney Pop-off Canal after going through Crescent Park’s large treatment pond.

3.2.2.10 Evergreen Estates System (model # 252xxx series)

The Evergreen Estates System originates in the Evergreen Estates subdivision and collects drainage from the various treatment ponds before it crosses Causeway Boulevard near South Falkenburg Road.

3.2.2.11 I-75 Ditch System (model # 253xxx series)

The I-75 Ditch System originates at the Causeway Boulevard crossing and collects drainage before it meets Aspen Cove Apartment System near South Falkenburg Road.
3.3 North Archie Creek Subwatershed

The North Archie Creek subwatershed is comprised of new subdivisions and commercial parks like Sterling Ranch, Brandon Lakes, Waterford, Parkway Business Center, and Progress Village. The Interstate 75 and U.S. Highway 301 drainage systems make up a third of this subwatershed.

3.3.1 North Archie Creek Main Channel System (model # 260xxx series)

The North Archie Creek Main Channel originates in the Providence Lakes subdivision and the area west of Providence Road. It flows for approximately 4,600 feet and then turns south to meet with Tributary “F” at a borrow pit pond near Sherwood complex. At the Evergreen Estates Outfall, the channel crosses Interstate 75 and Tributary “D” from west to east. The channel continues to move west going under U.S. 301 Highway and Interstate 75 again on the north side of Madison Avenue. After passing through Interstate 75, North Archie Creek goes under Madison Avenue on west of Interstate 75, and then keeps heading west through the Progress Village subdivision. From here, the creek crosses 78th Street and flows on the north side of the Cargill gypstack before crossing Old U.S. Highway 23 and U.S. Highway 41 to its outfall into Hillsborough Bay.

3.3.2 Tributaries

The tributaries of the North Archie subwatershed consist mostly of manmade ditches along the back lots of residential areas such as Progress Village west of U.S. Highway 301. The remaining tributaries on the eastern side of U.S. Highway 301 consist of the Interstate 75 drainage ditch and several ditch systems located in subdivisions of the Bloomingdale Hills, Providence Oaks, Sterling Ranch, and Brandon Lakes areas.

3.3.2.1 Tributary “A” (model # 261xxx series)

Tributary “A” originates north of Madison Avenue and west of the Falkenburg Road extension in a wetland located north of Fir Drive. The channel flows south to its confluence with the North Archie Main channel by way of interconnected stormwater pipes along the east side of 78th Street.

3.3.2.2 Tributary “B” (model # 262xxx series)

Tributary “B” originates south of Madison Avenue and west of the Falkenburg Road extension in the Parkway Business Center. The channel flows south and converges with the North Archie Main channel approximately 1,600 feet away. This tributary is an open channel on top of a series of underground stormwater pipes. The outfall of the stormwater pipe is approximately 200 feet north of the 82nd Street double Conspan.
3.3.2.3 Tributary “C” (model # 265xxx series)

Tributary “C” originates south of Madison Avenue and west of the Falkenburg Road extension in the Parkway Business Center. The channel flows south to its confluence with the North Archie Main channel approximately 1,600 feet away.

3.3.2.4 Unnamed Tributary (model # 263xxx series)

This tributary originates north of Madison Avenue and west of Interstate 75 near the U.S. Highway 301 exit and the Crescent Park commercial site. The channel flows south and converges with the North Archie Main channel south of Foxworth Road, which is approximately 2,600 feet away.

3.3.2.5 Tributary “D” (model # 2705xx series)

Tributary “D” originates south of Lumsden Avenue in the proposed Florida Corporate Center. This tributary serves as Interstate 75’s ditch. It flows south along the east side of Interstate 75 and west of Robert Tolle Road to its eventual confluence with the main channel near the west side of the borrow pit pond located in the Sherwood Apartment complex.

3.3.2.6 Tributary “E” (model # 272xxx series)

Tributary “E” originates approximately 1,400 feet south of Bloomingdale Avenue located across from a mobile home park near the Bayou Crossing Apartment Complex. It serves as the ditch for the west side of Duncan Road which flows north collecting drainage from the Bayou Crossing Apartment complex on the east side of Duncan Road. This tributary eventually meets with the main channel near the west side of the borrow pit pond located in the Sherwood Apartment complex after it crosses a box culvert under Bloomingdale Avenue.

3.3.2.7 Tributary “F” (model # 273xxx series)

Tributary “F” originates in the Bloomingdale Hills subdivision, west of Providence Road. This tributary collects flow from two sub-tributaries that run north and parallel to each other for approximately 1,150 feet. One of the sub-tributaries originates in the Bloomingdale Hills wetland, which also collects drainage from the Providence Oaks subdivision. The other sub-tributary originates in the Bloomingdale Hills Park subdivision pond located south of Bloomingdale Avenue. Both sub-tributaries ultimately join the Bloomingdale Avenue Outfall channel flowing west to meet with Providence Lakes Outfall from the north at the borrow pit pond near Sherwood complex.

3.3.2.8 Tributary “G” (model # 27004x series)

Tributary “G” originates in the Sterling Ranch subdivision to the west of Providence Road. This tributary generally flows west and then turns south to meet with Tributary F” from the south at the borrow pit pond near Sherwood complex.
3.4 Archie Creek Subwatershed

Half of the Archie Creek subwatershed consists of commercial areas like the Cargill complex and the Parkway Business Center. The other half is the Lake Street Charles, Starlite, Ashley Oaks, Suntree Estates, and McMullen Farms subdivisions. Cargill occupies about a third of the subwatershed. The subwatershed generally starts at the Ashley Oaks subdivision and discharges out to the Bay near the north part of the Cargill facility.

3.4.1 Archie Creek Main Channel System (model # 280xxx and 290xxx series)

The Archie Creek Main Channel System is considered in this study to start from east of Krycul Avenue near the Ashley Oaks subdivision, at the Lake Street Charles System confluence with Lateral “D” (Model Junction ID 290006) to the outfall location at Hillsborough Bay. The Archie Creek Main Channel is a well-defined channel which crosses Krycul Avenue, Bucks Ford Drive, Interstate 75, 78th Street, the Rinker Company entrance, Old U.S. Highway 23, the CSX Transportation System (twice) and U.S. Highway 41. It ultimately discharges into Hillsborough Bay just north of the Cargill area.

3.4.2 Tributaries

The tributaries are comprised mostly of manmade ditches that originate from wetland areas like Tributary “A” and “C”. The other tributaries were originally excavated for agricultural use but now serve as drainage channels for areas like the Parkway Business Center, Starlite, Lake Street Charles, and Ashley Oaks subdivisions.

3.4.2.1 Tributary “A” (model # 2801xx series)

A natural tributary joins Archie Creek approximately 650 feet east of the Rinker Company’s entrance. This tributary drains the area north of Cargill’s cooling ponds and west of 78th Street near Progress Boulevard, as it flows in a north to south direction crossing 78th Street.

3.4.2.2 78th Street Ditch (model # 2803xx series)

A ditch system joins Archie Creek approximately 4,000 feet south of Archie Creek. This tributary drains an area south of Eagle Palm Drive and east of 78th Street as it flows in a south to north direction through several residential and commercial (junkyard) driveways before the confluence with Archie Creek.

3.4.2.3 Tributary “B” (model # 2804xx series)

Lateral “B” is a large lateral ditch draining into Archie Creek from the north. Drainage from the Business Parkway area is collected east of 78th Street and west of Interstate 75 in this lateral. The
channel flows south and crosses Falkenburg Road and Eagle Palm Drive to the confluence with Archie Creek near the Premier Beverage facility.

3.4.2.4 Tributary “C” (model # 2805xx series)

Lateral “C” is located approximately 1,350 feet north of Riverview Drive near an eagle/gopher tortoise preserve and wetland area. The channel generally flows north and drains a portion of the Parkway Business area. The channel crosses only one dirt road that has limited access.

3.4.2.5 Tributary “D” (model # 2902xx and 2903xx series)

Lateral “D” is located approximately 350 feet south of Colonial Lake Drive immediately south of the Pond “AL,” and drains Lake Street Charles located to the north of it. The channel flows south to the confluence with Archie Creek.

3.4.2.6 Tributary “F” (model # 290xxx, 2905xx and 2906xx series)

Lateral “F” originates at the intersection of Springbrook Drive and Brandon Circle and splits into two flow directions. The portion that receives drainage from the Starlite subdivision flows west, while the portion collecting drainage of the fish hatchery flows south into the Lake Street Charles system. The channel flowing east to west crosses several wetlands located west of the bermed dirt road near the Starlite subdivision.

3.4.2.7 Tributary “G” (model # 2901xx series)

Lateral “G” originates south of the Menard Avenue located in the FDOT pond adjacent to Interstate 75. The channel generally flows north and parallel to Interstate 75 on the east side to connect with Archie Creek.

3.4.2.8 Isolated Basins (Cargill complex)

These isolated basins consist of the area east of U.S. Highway 41 on the Cargill complex and west of U.S. Highway 41. The Cargill gypstack has its own drainage system. There is no connection to Archie Creek.
CHAPTER 4 HYDROLOGIC/HYDRAULIC MODEL METHODOLOGY

Several computer software products and analysis techniques have been used to develop the current model for all the County watershed studies, including the Delaney Creek Area watershed (DCA). This chapter provides a general description of these methods and approaches.

4.1 General Hydrologic/Hydraulic Model Development

The United States Department of Agriculture’s Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), Runoff Curve Number (CN) method is used to generate runoff hydrographs from rainfall data and watershed parameters. This method estimates expected stormwater runoff based on soil and land cover characteristics as well as watershed flow path and slope characteristics. Runoff hydrographs are developed using the NRCS Dimensionless Unit Hydrograph method.

Inflow hydrographs are generated at corresponding node-basin junctions. Discharges are routed through the system using a modified version of the U. S. Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) version 4.31b, Hillsborough County’s version of SWMM (HCSWMM). The EXTRAN block of SWMM provides a hydrodynamic channel routing model.

4.2 Hydrology

In the Hillsborough County version of SWMM, the SCS-CN method, rather than the nonlinear reservoir method, is used to calculate the runoff hydrographs.

4.2.1 SCS-CN Method

The SCS-CN method is one of the most popular methods for computing the volume of surface runoff for a given rainfall event from small watersheds. Kent (1973) described and examined this method in details. The SCS-CN method is based on the water balance equation and two fundamental hypotheses. The first hypothesis states that the ratio of the actual amount of direct runoff to the maximum potential runoff is equal to the ratio of the amount of actual infiltration to the amount of the potential maximum retention. The second hypothesis states that the amount of initial abstraction is some fraction of the potential maximum retention. Expressed mathematically, the water balance equation and the two hypotheses, respectively, are:
\[ P = I_a + F + P_E \quad (4-1) \]

\[ \frac{P_E}{P - I_a} = \frac{F}{S} \quad (4-2) \]

\[ I_a = \lambda S \quad (4-3) \]

where:

\[ P = \text{total precipitation, inch;} \]

\[ I_a = \text{initial abstraction, inch;} \]

\[ F = \text{cumulative infiltration excluding } I_a, \text{ inch;} \]

\[ \lambda = \text{non-dimensional parameter;} \]

\[ P_E = \text{direct runoff, inch;} \text{ and} \]

\[ S = \text{potential maximum retention or infiltration, inch.} \]

The current version of the SCS-CN method assumes \( \lambda \) equal to 0.2 for usual practical applications. As the initial abstraction component accounts for surface storage, interception, and infiltration before runoff begins, \( \lambda \) can take any value ranging from 0 to 1. Combining (4-1) and (4-2), we can write an equation for \( P_E \) as follows:

\[ P_E = \frac{(P - I_a)^2}{P - I_a + S} \quad (4-4) \]

If \( \lambda = 0.2 \), then combining Equations (4-3) and (4-4) will result in

\[ P_E = \frac{(P - 0.2S)^2}{P + 0.8S} \quad (4-5) \]

By studying the relationships of many different watersheds, the SCS further introduced a dimensionless number, CN, called the curve number. The curve number and S are related by

\[ S = \frac{1000}{CN} - 10 \quad (4-6a) \]

or

\[ CN = \frac{1000}{S + 10} \quad (4-6b) \]

The curve number is a function of land use, cover, soil classification, hydrologic conditions, and antecedent moisture conditions. The variation in infiltration rates of different soils is incorporated in
curve number selection through the classification of soils into four hydrologic soil groups: A, B, C, and D. These groups, representing soils having high, moderate, low, and very low infiltration rates:

Group A: soils have low runoff potential and high infiltration rates even when thoroughly wet (fully saturated). They consist chiefly of deep, well-drained sands or gravels and have a high rate of water transmission (greater than 0.30 in/hr).

Group B: soils have moderate infiltration rates when thoroughly wet and consist chiefly of moderately deep to deep, moderately well-drained to well-drained soils with moderately fine to moderately coarse texture. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).

Group C: soils have low infiltration rates when thoroughly wet and consist mainly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).

Group D: soils have high runoff potential. They have very low infiltration rates when thoroughly wet and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table and shallow depths over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr).

Runoff curve numbers for urban areas, cultivated and other agricultural lands, and arid and semiarid rangelands are shown in tables Table 4-1a through Table 4-1d.

### 4.2.2 SCS Dimensionless Hydrograph

The SCS dimensionless hydrograph is a synthetic unit hydrograph in which the discharge is expressed by the ratio of discharge Q to peak discharge Q₀ and the time by the ratio of time t to the time of rise of the unit hydrograph, T₀. The unit peak discharge is calculated by:

\[
U_p = \frac{KA}{T_p} \quad (4-7)
\]

where:

- \(U_p\) = unit peak discharge, cfs/inch;
- \(A\) = drainage area, mile²;
- \(K\) = hydrograph shape factor, ranges from 300 for flat swampy areas to 600 in steep terrain. SCS standard K value = 484, K=256 is used in this study.
- \(T_p\) = time to peak, in hours.

\[
T_p = \frac{t_p}{2} + t_p \quad (4-8)
\]
where: \[ t_r = \text{storm duration, hours;} \]
\[ t_p = \text{drainage area lag, hours.} \]
\[ t_p = 0.6 T_c \] (4-9)

where: \[ T_c = \text{time of concentration, hours.} \]

The figure below shows the definition of \( U_p, T_p \), for a triangular unit hydrograph used in Hillsborough County version of SWMM.

The peak discharge for a given rainfall is calculated by
\[ Q_p = U_p P_E \] (4-10)

where:
\[ Q_p = \text{peak discharge, cfs and } P_E \text{ is calculated with Equation (4-5).} \]

### 4.2.3 Model Implementation

The convolution method is used to yield the direct runoff hydrograph. The convolution equation is:
\[ Q_n = \sum_{m=1}^{n=M} P_{Em} U_{n-m+1} \] (4-11)

where:
\[ P_{Em} = \text{excess rainfall of } m^{th} \text{pulse, inch;} \]
\[ U_{n-m+1} = \text{unit direct runoff at time } n\Delta t \text{ of } m^{th} \text{rainfall pulse, interpolated from the figure above, cfs/ inch;} \]
\[ \Delta t = \text{time step, minutes}; \]

\[ Q_p = \text{total runoff at time } n\Delta t, \text{ cfs}; \]

\[ M = \text{total pulses of excess rainfall.} \]

### 4.2.4 Rainfall Depth

The rainfall depths used for the 2.33-year, 5-year, 10-year, 25-year, 50-year, and 100-year events were taken from the SWFWMD isohyetal maps published in the Environmental Resource Permitting Information Manual (ERPM). The rainfall depths shown are for the 24-hour event as shown below:

<table>
<thead>
<tr>
<th>Storm Event – 24-hour Duration</th>
<th>2.33-year</th>
<th>5-year</th>
<th>10-year</th>
<th>25-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall Depth (inch)</td>
<td>4.5</td>
<td>5.5</td>
<td>7.0</td>
<td>8.0</td>
<td>10.0</td>
<td>11.0</td>
</tr>
</tbody>
</table>

These design storm events utilized the SCS Florida Modified Type II rainfall distribution as specified by Hillsborough County and by SWFWMD.

5-day storm simulations were also run for 10-year, 50-year, and 100-year return frequencies at the request of SWFWMD. The rainfall depths cited in the chart below are for the 5-day event:

<table>
<thead>
<tr>
<th>Storm Event – 5-day Duration</th>
<th>10-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall Depth (inch)</td>
<td>11.3</td>
<td>15.9</td>
<td>17.8</td>
</tr>
</tbody>
</table>

These design storm events utilized rainfall depths and distributions specified in Tables 3 and 4 of the “Southwest Florida Water Management District’s Watershed Management Program Guidelines and Specifications” (August 2002) for Watershed Management Programs.

### 4.2.5 Subbasin Delineations

The determination of the subbasin boundaries within the watershed was made on the basis of the existing physical features such as the drainage areas (topography), storage areas, and conveyance elements (pipes, control structures, etc.), which make up the system network. A number of sources of information were used to define the individual subbasins based on this network. The main source was the County’s latest 1-foot digital contours in 2004. This information was used to determine or verify most of the subbasin delineations and the overland connections between subbasins. During the model update, Ayres staff also collected the latest aerial photographs as well as major ERP plans (“As-Built”)
of new developments and roadway projects within the watershed since 2000, to verify and update the
delineations from the original model.

In some instances, development or re-development have occurred after the date of completion of the
original model. Development in the watershed sometimes resulted in the construction of stormwater
ponds, or additional conveyance features (pipe systems gutters, ditches, etc.), which potentially altered
the indicated flow patterns as determined from the aerial contour maps. In these cases, construction
plans (when obtainable) were used to assist in updating the subbasin delineations, by taking into
account land use alteration activities. Available data used to assist in the development of the subbasin
delineations included State and County roadway plans, private development plans, and County
inventories of stormwater collection systems and road crossings. A limited review of the permitted
activities on file with SWFWMD and the Hillsborough County Planning and Growth Management
Department to evaluate potential developments of significance was also performed.

In addition, a limited field verification of the subbasin delineations was conducted to resolve conflicting
information from data sources, to inspect for additional large developments and for potential new large
scale construction not yet permitted, to verify outer limits of the watershed area, and to resolve
questions related to subbasin delineations. The latest aerial photographs were also used as a tool to
indicate potential updates of the subbasin delineations. These aerial photographs were obtained from
Hillsborough County and were dated 2006.

The aerial photographs indicated new constructions, land use/cover changes, additional creek
crossings, and potential alterations to the drainage patterns indicated in the 1981 aerial contour maps.
It should be noted that the details of the subbasin delineations represented in this report reflect
planning level functions. Should it become necessary to evaluate specific individual developments, a
higher degree of definition may be required.

A QA/QC process was performed when finalizing the subbasin delineations. The TIN (Triangular
Irregular Network) through ArcGIS 3D rendering was overlaid with the delineations to resolve the
potential inconsistencies, such as conflicts with ridgelines, depression areas, etc.

Figure 4-1 shows the subbasin delineations in GIS environment and Figure 4-2 compares the
subbasin delineations during the update. The file is stored as personal geodatabase (*.mdb) format,
with an associated polygon attribute table. In addition, each subbasin was given a unique 6-digit
character nomenclature as specified by the Hillsborough County Stormwater Management Master Plan
Hydrologic and Hydraulic Model Set-Up Standard (Hillsborough County Stormwater Management
Section, Engineering Division, Public Works Department, 2/11/99). The attribute table was then
enhanced to include the necessary data fields to bring it into compliance with the GIS data format
outlined by the Southwest Florida Water Management District Watershed Data Management System
for Engineering (SWFWMD Engineering and GIS Section, January 2000). Table 4-2 summarizes the
hydrologic parameters for each subbasin within the watershed.
4.2.6 Soil Data, Land Use, and SCS Curve Number Determination

4.2.6.1 Soil Data

SWFWMD GIS soil coverage was used to obtain soil information for the Delaney Creek Area watershed. The SWFWMD coverage was developed from data in the SCS Soil Survey of Hillsborough County, Florida, 1989. Each soil polygon in the GIS coverage is associated with an attribute that designates its soil identification number. A database table was used to associate soil identification numbers with their corresponding Hydrologic Soil Groups (HSG). Hydrologic soil groups in the watershed consist of six designations A, B, C, D, B/D, and Water. The HSG A soils have a high infiltration rate and low runoff potential. HSG B soils are moderately well drained and have a moderate infiltration rate. HSG C soils have slow infiltration rates and may contain a layer of fine textured soil, which impedes the downward movement of water. HSG D soils include poorly drained, very silty/clayey/organic soils or soils with high groundwater tables. Dual hydrologic classification B/D includes soils which have a seasonal high water table but can be drained. The first hydrologic soil group designates the drained condition and the second hydrologic soil group designates the undrained condition of the soil. The hydrologic soil groups used in the analysis were shown in Figure 2-2.

4.2.6.2 Land Use

The updated SWFWMD GIS Land Use Coverage (2006) was used to represent the existing conditions land use. Each land use polygon in the GIS coverage is associated with an attribute that designates a classification from the Florida Land Use Classification Code System (FLUCCS) - also known as the Florida Land Use, Cover and Forms Classification System (FLUCFCS). As impervious area increases, runoff usually increases. SWFWMD has been regulating quantity of stormwater runoff since 1984. The objective of regulation has been to prevent peak runoff rates under the developed conditions from exceeding peak runoff rates associated with the predevelopment conditions. The Land Use/Land Cover data used in the analysis were shown in Figure 2-3.

4.2.6.3 Runoff Curve Numbers

The SCS Runoff Curve Number (CN) method was used to generate runoff from rainfall. The method estimates runoff on the basis of soil and land cover characteristics. Runoff curve numbers are related to land use and hydrologic soil group. Land use polygon, hydrologic soil group polygon, and subbasin delineation polygon coverages were overlaid utilizing GIS intersection techniques. This procedure generated unique polygons within subbasin polygons that were assigned a land-use, subbasin number, and soil type. These polygons were aggregated into a weighted CN for each subbasin using a database script and a lookup table. The procedure calls for a polygon element within a subbasin to be assigned a CN value based on soil type and land use (Table 4-3). Based on this, a composite (area weighted) CN for each subbasin was calculated and assigned.
4.2.7 Time of Concentration

The time of concentration is defined as the time for the runoff to travel from the hydraulically most distant point in the drainage basin to the point of interest, usually the basin outlet. TR-55 provides the methodology for calculating Tc in three types of flow regimes: sheet flow (overland flow), shallow concentrated flow, and open channel flow. The time of concentration for each subbasin was calculated based on the guidelines specified in the Hillsborough County Stormwater Technical Manual. The components that make up the travel time for each subbasin were derived from the following description:

<table>
<thead>
<tr>
<th>Flow Regime</th>
<th>Method/Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overland Flow</td>
<td>Kinetic Wave Equation</td>
</tr>
<tr>
<td>Shallow Concentrated Flow - Paved</td>
<td>SCS Equations Relating Velocity to Watercourse Slope</td>
</tr>
<tr>
<td>Shallow Concentrated Flow - Unpaved</td>
<td>SCS Equations Relating Velocity to Watercourse Slope</td>
</tr>
<tr>
<td>Channel Flow</td>
<td>Assume 2 ft/sec</td>
</tr>
<tr>
<td>Pipe Flow</td>
<td>Assume 3 ft/sec</td>
</tr>
</tbody>
</table>

The selection of Manning’s coefficients for the calculation of overland flow travel time is provided in Table 4-4.

4.3 Hydraulics

4.3.1 Major Modifications

A modification of the U.S. EPA SWMM 4.31b, Hillsborough County version of SWMM, has been used to compute water surface elevations and discharges at links and nodes shown on the junction/reach schematic diagram. The SWMM EXTRAN block was used for hydraulic routing. The most significant modifications to EPA SWMM 4.31b included directly integrating the SCS method to generate runoff hydrographs, entrance and exit head loss coefficient, and conduit stretch factor.

The exit head loss coefficient is usually set to 1.0. The entrance head loss coefficient is selected based on values in Table 4-5.

Other minor changes included the increase of dimensions of a number of key parameters, enhancements of the inputs and the outputs and error trapping. Input enhancements included a provision for specifying reach numbers for orifices and weirs and another for using elevations rather than depths above invert for weir data. Several output enhancements have been provided including a
provision for printing a summary file showing both computed peak discharge values and water surface elevations.

Elliptical and arch pipes are included in the current County version SWMM model. Natural channels are represented in EXTRAN as conduits with irregular cross-section data. The cross-section data is input as ground shots (elevations and stations across the channel) in a format similar to that of HEC-2 (U.S. Army Corps of Engineers) cross-section data. EXTRAN uses the cross-section data only to obtain the shape geometry. It uses invert elevations input on the conduit records to determine the channel slope. A natural channel is thus treated as a prismatic conduit with an irregular shape.

4.3.2 Natural Channels

The data for the channel geometry was derived mostly from the channel cross-section survey data. Natural channel reaches were evaluated for out of bank conveyance capability based on aerial photographs, field photographs of the actual channel, and field evaluations. In some cases, channel cross-sections were modified to account for encroachment into the conveyance portion of the channel by buildings or other obstructions. In those cases, the portion of the channel outside of the conveyance area was treated as floodplain storage with no conveyance capability. Thus, each channel was evaluated for a friction loss that related to the roughness conditions at the bottom, and the right and left out of bank. Channel roughness (Manning’s coefficients) values were evaluated from literature sources provided by Hillsborough County, sources obtained from the USGS based on Manning’s coefficient studies for natural channels, and from previous experience with channel systems in Hillsborough County. Initial Manning’s roughness values were either confirmed or adjusted during the calibration phase of the study.

4.3.3 Conduits

The way the Hillsborough County version of the EXTRAN block calculates friction loss in conduits differs from the original program by the EPA. The original version had an input for the friction loss coefficient that was the only basis for the total head loss in a conduit. It was necessary to adjust this coefficient to account for the other minor losses of the conduit such as entrance, exit, and conduit transitions. In addition, if a conduit experienced instability during a simulation, an equivalent conduit (elongated) would possibly have to be used. In such cases, the friction loss coefficient would again have to be manipulated to account for the additional “length” of the new conduit. The County version of EXTRAN has four additional data fields that allow the user to input entrance, exit, conduit transitions and an elongation factor. This allows for the preservation of the friction loss and the input data, and thus the Manning’s “n” value represents the roughness of the conduit only. The creation of the equivalent conduit is internal to the model.
4.3.4 Storage Facilities

The EXTRAN model allows the user to input variable relationships between stage and area. These areas can represent the flood storage created by depressions, lakes, wetlands, retention/detention ponds, or out of bank storage. This relationship is assigned to a specific junction within the model schematic. This storage is important in the Delaney Creek Area because many areas which contain lakes experience out of bank conditions during high flow. During the model update, the stage-area curves were established with the latest 1-foot contours dated 2002. The relationship between stage and area was derived from a spatial analysis of the Digital Elevation Model (DEM) generated through GIS Spatial Analyst rendering. These data were then input into the computer model to represent the basin storage available during storm simulations. In those cases where development occurred after the contours and aerial mapping, construction drawings were employed to estimate storage facilities. Also, a nominal storage was defined for manhole junctions where necessary to stabilize the model.

4.3.5 Weirs

The overtopping of roadways at channel crossings was simulated using broad crested weirs as the conveyance mechanism. The weir invert elevations were obtained from County’s latest 1-foot digital contours dated 2004.

The width of the weir was scaled from the aerial contour maps. After preliminary simulations were made, the weir widths were evaluated to verify or modify these initial values. Hillsborough County specified that the weir coefficients for roadway overtopping should be 2.0.

In some areas of the watershed, broad crested weirs were used to simulate flow that may occur in an overland fashion from subbasin to subbasin. The weir invert elevations were obtained from the SWFWMD aerial contour maps. Hillsborough County specified that the weir coefficients for the basin-to-basin interconnections should be 1.0. The data for weir invert elevations, and widths used in conjunction with control structures were obtained from survey data, construction plans, or field estimates.

Control structure weir information was also verified with construction plans and supplemented for missing ones or new constructed ones. A coefficient of 3.2 is selected for a structure weir.

4.3.6 Orifices

The use of orifices to model flow conditions in a master planning level study is generally not practical. The normally small amount of flow conveyed through the orifices in control structures is not significant enough to affect the peak stages and discharges predicted for the design storm events.
4.3.7 Initial Water Surface Elevations

The initial water surface elevations were estimated by evaluating the invert elevations of the channel bottom and conduit inverts. For design storm simulations, it was assumed that the starting water elevation was equal to highest invert of the channels/conduits/control structures downstream of the junction. This procedure provided static water elevations within the hydraulic system that provided zero flow at the beginning of the design storm simulation, and allowed for discharge immediately upon introduction from the runoff hydrographs.

4.3.8 Dummy Junctions and Conduits

The practice of utilizing dummy or imaginary conduits within the EXTRAN input data was done to eliminate artificial warning flags in the output files. EXTRAN will generate a warning flag in the output file for any junction that does not have a conduit equal to the junction invert. It will also generate a flag for any junction that has conduits whose crown is lower than the adjacent conduit inverts. For both of these cases, dummy pipes were added to the input file to keep the output files clear from warning flag clutter. The dummy conduits were noted as such in the input data files. Dummy storage areas are required at any junction that connects two or more conduits or weirs. In these cases, the area was non-variable with depth and assigned to be 4356.0 square feet. This was done for model stability. All dummy storage junctions were noted as such in the input data files.

4.3.9 Boundary Conditions

The DCA watershed has four major outfalls, the three creeks, and the Delaney Pop-off Canal, which discharge into Hillsborough Bay. The boundary conditions at the outflow points were set to be 1.6 feet NAVD.

4.3.10 Numerical Instability

The EXTRAN model solves the Saint-Venant equations that describe unsteady flow in channels based on three different numerical methods: the explicit finite difference method, the implicit finite difference method, and the iteration method. In this study, method three, the iteration method was used. The advantages of this method are: 1) better stability, 2) faster execution, and 3) easier debugging. However, this method is still subject to numerical instability caused by accumulated round-off error and it is difficult to predict the conditions that cause numerical instability. Large time steps, short conduit lengths, steep bottom slopes for conduits, and low storage at junctions are frequently associated with numerical instability. Achieving numerical stability requires numerous adjustments to the model input data. Such adjustments include the use of equivalent pipes with longer lengths, decreased time step, adjusting roughness and the addition of storage at the junctions.

The equivalent pipe formula used to calculate the adjustments is as follows:
\[ n_e = n_p \frac{I_p^{1/2}}{I_e^{1/2}} \quad (4.12) \]

where

- \( n_e \) = Manning roughness of equivalent pipe, dimensionless;
- \( I_e \) = Computed equivalent length, ft;
- \( n_p \) = Actual Manning roughness of the pipe, dimensionless;
- \( I_p \) = Actual length of the pipe, ft.

### 4.3.11 Model Schematic

The hydraulic model of the watershed consists of all of the features that make up the primary conveyance network. These features include lakes, wetlands, pipes, natural channels, and control structures. The EXTRAN model uses a conduit junction concept to idealize the hydraulics of the system. The junctions within the model are the discrete locations within the watershed where the conservation of mass is maintained. These represent the storage and stage related elements of the model. The conduits are the connections between the junctions. These represent the flow and conveyance related elements of the model.
CHAPTER 5 HYDROLOGIC/HYDRAULIC MODEL CALIBRATION AND VERIFICATION

The calibration and verification of a stormwater model typically involves comparing simulated stages, flows and/or volumes of water with observed data for a recent and significant storm event. This chapter contains the data collection, hydrological/hydraulic model calibration and verification procedure conducted for the Delaney Creek, Delaney Pop-off Canal, North Archie Creek, and Archie Creek subwatersheds existing conditions. The goal is to develop a hydrological/hydraulic model that reflects the observed conditions in these subwatersheds and can be used to predict system performance for future events, and to evaluate alternative projects within the watershed.

The calibration process includes simulating a monitored event by first adjusting the hydrologic input parameters within an acceptable range and using the measured rainfall depth and distribution, and then comparing computed water surface elevations and flows to the measured data collected at gauging stations. The hydrodynamic model parameters are then adjusted in a series of runs so that the computed and measured values closely match and almost mimic one another. The model is considered well calibrated when the simulated stages, flows, and volumes are in reasonable range with the recorded data at the established gage stations. The model is considered with “best fit” parameters accordingly and verified by simulating one or more other recent storm event(s) independent of the event used during the calibration.

As mentioned in Chapter 1, the Delaney Creek Area is a highly developing area within which many apartment and housing complexes, transportation, and stormwater management projects have been developed. Because of continuing development, rainfall/runoff information that is not recently collected may not be very reliable to conduct the stormwater management study under existing conditions. The model calibration for this study was performed using rainfall and streamflow gage data from Hurricane Frances during September 4-7, 2004, which had a dramatic impact on the watershed. The July 18-20, 2004 storm was selected for the model verification purpose.

5.1 Boundary Conditions

The major outfalls within the study area all discharge into Hillsborough Bay. The water level boundary conditions at the outflow points were set to be 1.6 feet NAVD.

5.2 Data Collection

5.2.1 USGS Gage Stations
Four gage stations are available in the Delaney Creek Area. All of these stations were installed and are being operated by United States Geological Survey (USGS) as described in details in Table 5-1. Locations of these gage stations are shown in Figure 5-1.

5.2.2 Precipitation Data

As discussed in an earlier section, the selected storm events used in this study for model calibration and verification are as follows:

- September 4-7, 2004 Hurricane Frances with an approximate duration of 60 hours
- July 18-20, 2004 storm lasting approximately 60 hours

The rainfall gage data was obtained from the USGS with 15-minute interval. Three rainfall gages, 02301750, 02301745, and 02301740 were recorded for Hurricane Frances and two of those, 02301745 and 02301740, have 15-minute rainfall data available for the July 2004 storm as shown in Table 5-2. Before assigning rainfall to the subbasins, the available rainfall data was thoroughly reviewed for reliability and consistency.

Areal assignment of rainfall depth from the gages to the individual subbasins was accomplished using the Thiessen Polygon method. Connecting lines were drawn to form polygons around each gage. Subbasins mostly within a rain gage polygon were assigned the rainfall depth from that gage. Figure 5-2 presents the Thiessen Polygon distribution for Hurricane Frances.

Table 5-3 and Table 5-4 summarize the rainfall distribution for Hurricane Frances and the July 2004 storm, respectively.

5.2.3 Surface Water Data

Streamflow data was obtained from the USGS with 15-minute interval for four gage stations within the watershed during Hurricane Frances. During the Hurricane Frances, gage 02301750 recorded a peak stage of 16.29 ft NAVD with an estimated peak flow of 721 cfs; gage 02301745 recorded a peak stage of 9.22 ft NAVD with an estimated peak flow of 399 cfs; gage 02301740 recorded a peak stage of 19.77 ft NAVD with an estimated peak flow of 353 cfs; gage 02301738 recorded a peak stage of 10.85 ft NAVD with an estimated peak flow of 138 cfs.
5.2.4 Antecedent Moisture Condition (AMC)

An important aspect of the hydrologic model during the calibration process is the establishment of antecedent soil moisture conditions. The numerous lakes and retention ponds are not the only storage elements that retain precipitation and runoff during storm events. The unsaturated portion of the soil profile acts as a storage reservoir for the water, which infiltrates the ground. In Florida, where the water table is usually very shallow, the available soil moisture holding capacity can vary over a wide range depending on the seasonal elevation of the water table. It became apparent during model calibration that the antecedent water table elevation (i.e., elevation at the beginning of the storm event) is an important factor, which impacts the magnitude and temporal distribution of runoff. That is, the behavior of infiltration excess and saturation excess runoff are very different and was evident during the calibration efforts.

The index of watershed wetness used with the runoff estimation method is Antecedent Moisture Condition (AMC). Traditionally, rainfall in antecedent periods of 5 days prior to a storm is commonly used as indices of watershed wetness. AMC is generally classified on an index of 1 to 3; an increase in an index means an increase in the runoff potential, which is summarized by the table below. Design storm events generally assume AMC-2.

<table>
<thead>
<tr>
<th>AMC Class</th>
<th>Description</th>
<th>5-Day Antecedent Rainfall Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Dormant Season</td>
</tr>
<tr>
<td>AMC-1</td>
<td>Low runoff potential. Dry soils.</td>
<td>Less than 0.5&quot;</td>
</tr>
<tr>
<td>AMC-2</td>
<td>Average runoff and soil moisture.</td>
<td>0.5&quot; to 1.1&quot;</td>
</tr>
<tr>
<td>AMC-3</td>
<td>High runoff potential. Saturated soils.</td>
<td>Greater than 1.1&quot;</td>
</tr>
</tbody>
</table>

Much of the Delaney Creek Area is characterized by relatively flat topography and high ground water table. Consequently, rainfall depths less than those presented by the table above may result in runoff responses similar to a higher AMC. Based on the available information, Hurricane Frances calibration event is classified as AMC-2 and the July 2004 storm is classified as AMC-1.

Runoff Curve Numbers (CN) requires adjustment for AMC other than normal AMC-2. These adjustments are typically made using lookup tables such as Table 5-5. In practice, intermediate CN and AMC values are interpolated.
5.3 Existing Conditions Model Calibration

In the Hillsborough County version of SWMM (HCSWMM), most of the required input data simply describe the geometry and size of the hydrologic and hydraulic units of the subdivided study area. These data, such as the subbasin areas, channel widths, lengths and cross drain dimensions, are known quantities and are subject to very little interpretation. A few of the input requirements, however, are not derived from measurable qualities and have larger space for modification.

The HCSWMM includes hydrologic (RUNOFF) and hydraulic (EXTRAN) computational blocks. Hurricane Frances calibration parameters within the hydrologic portion of HCSWMM include:

- Time of Concentration (Tc): Effects start time of runoff response and hydrograph shape. Accounts for the travel time of unconcentrated flow.
- Runoff Curve Number (CN): Effects total runoff volume. Accounts for soil, land use, and Antecedent Moisture Condition characteristics.

The HCSWMM EXTRAN Block performs complex flow routing through channels, culverts, and reservoirs. Like the hydrologic portion, EXTRAN was calibrated to match peak flow rate, runoff volume (at a particular location), and flow hydrograph shape. In addition, EXTRAN was calibrated to match observed stages. Calibration parameters within the EXTRAN Block include:

- Manning’s Friction Coefficient (n): Effects flow rate and stage. Accounts for roughness, irregularities, and obstructions within the conveyance system.
- Starting Water Surface Elevation: Effects stage and runoff volume. Accounts for surface storage not included in the fixed initial abstraction parameter described above. Examples of this surface storage are those wetlands, lakes, and ponds that have capacity to accept runoff prior to discharging into the main conveyance system.

These parameters were first approximated with values derived from local data (e.g., aerial topographic photographs and soil surveys), but their final values were ultimately determined through model calibration. After a fundamental hydrologic and hydraulic check, a calibration process is conducted to evaluate the general reliability of the model for producing reasonable results.

Hurricane Frances during September 4-7, 2004 was selected for calibrating the existing conditions model due to its significance and the availability of recorded data.

The studied four subwatersheds cover an area of approximately 34.3 square miles. Total rainfall for each of the subwatershed was not uniformly distributed. Distribution ranges between 8.80 inches at the Delaney Creek gage in the northern part of the watershed, 7.01 inches at the Delaney Pop-off Canal gage south to Delaney Creek gage, and 7.92 at the North Archie Creek gage located in southern portion of the watershed (see Figure 5-1 and Table 5-2).
The objective of calibration is to arrive at a better match between the simulated and measured stages and discharges displayed as the calculated hydrographs versus the recorded data in the figures. Adjustments to the infiltration parameters (that causes increase or decrease in flow rates) are made during the period of runoff to that end. Similarly, adjustments to the total infiltration capacity affect the runoff volume, may shift the peak time for runoff, and alter the recession limb of the hydrograph. Based on a given set of calibration criteria, the model is adequately calibrated when the observed and calculated hydrographs of the Hurricane Frances are in close agreement. The model is then assumed ready for further verification using one or more other recent storm events.

**Figure 5-3** through **Figure 5-6** illustrates the comparisons of water levels between model simulation and observed values at the four gage stations.

In **Figure 5-3**, a good correlation is seen at the Delaney Creek gage for the first 30 hours of simulation. At this point, the simulated system begins to recover as rainfall dies down, but the gaged system remains high. Beyond this point, the simulated stage response to subsequent rainfall is very similar to the gaged system response, but at the “recovered” lower elevation. A possible explanation is that the system experienced blockage downstream of the gage from first-flush debris or silts, which the simulated model cannot mimic.

**Figure 5-4** presents the Delaney Pop-off Canal system’s simulated and recorded stages. Correlation is quite good.

The North Archie Creek system response to the calibration event is presented as **Figure 5-5**. Although the simulated peak stage is reasonably close to the gaged peak stage, the simulated system recedes more quickly than the gaged system. System debris blockage is once again a likely cause, although this cannot be proven. Overall correlation is sufficient to produce reliable design event model results.

Archie Creek calibration, presented in **Figure 5-6**, shows good stage correlation and is anticipated to produce reliable results. This area proved difficult to calibrate initially, due to uncertainties regarding the status of the Cargill outfall diversion. As it turned out, the system was in a state of partial construction during the calibration event (determined through careful inspection of aerial photography from that time period). Temporary construction road pipe crossings are part of the calibration model only, but are removed for updated model design event simulations.

Although not presented as figures in this report, gaged and simulated flow were also compared at these locations. Peak flow correlation was within 10% for Delaney Creek and North Archie Creek. Simulated peak flow was about 20% low compared to the gage for Delaney Pop-off although the response pattern was fairly good. Archie Creek gaged and simulated flow correlated well until the stage exceeded the channel banks. The modeled cross-section defines the overbank conveyance and reported significant increased flow, but the USGS rating curve does not extend above the true top of bank and reported a fairly constant flow rate beyond that point.
5.4 Existing Conditions Model Verification

The July 18-20, 2004 storm was selected for verifying the existing conditions model using the calibrated model. Total rainfall for each of the subwatershed was not uniformly distributed. Rainfall distribution ranged between 5.97 inches at the Delaney Pop-off Canal gage and 6.07 inches at the North Archie Creek gage located south of the watershed (see Table 5-2). Due to lack of rainfall data for Delaney Creek gage and small difference between the two known rainfall series, the Thiessen Polygon distribution for Hurricane Frances was modified using visual judgment.

Figure 5-7 and Figure 5-8 illustrates the comparisons of water levels between model simulation and observed values at two gage stations with records.

The Delaney Pop-off verification event stage comparison is presented in Figure 5-7. The stage is consistently over predicting here for the verification event. The earlier event date, at the very beginning of the wet season, may justify lower starting elevations for storage systems throughout the watershed in addition to the assumed AMC-I condition. Additionally, actual soil infiltration rates will be higher under field conditions while the surficial water table has not yet risen to its seasonal high. The simulating model cannot account for infiltration within depressions, detention areas and sandy-bottomed channels once the runoff hydrographs are generated and routing begins. Simulated flow rates (not shown) were also higher than gaged flow rates but exhibited rising and falling patterns consistent with the gaged flow. Response was notably sharper in the simulated system for both rising and falling flow rates.

Figure 5-8 presents the comparison of gaged and simulated stage for the North Archie Creek system. Good correlation of both stage and flow (not shown) were achieved for this event.
CHAPTER 6 EXISTING CONDITIONS LEVEL OF SERVICE

Based on the Hillsborough County Stormwater Drainage Manual and the SWFWMD’s Environmental Resource Permitting (ERP) Manual, a standard design storm is defined by duration, rainfall depth, and distribution, for a specific return period.

There are six standard design storms used to analyze the flooding impact in the Delaney Creek Area. The standard design storms used in this study are the 100-year, 50-year, 25-year, 10-year, 5-year, and 2.33-year (mean annual). The duration and distribution set by SWFWMD criteria are 24-hour and SCS-type II Florida Modified, respectively. AMC-2 (normal condition) is also set by the same SWFWMD criteria. It should be noted that design elevations might be exceeded for longer duration, higher volume storms of the same frequency and under very wet conditions. The total amount of rainfall for a particular frequency was determined by using the SWFWMD rainfall map, which may vary with physical location inside the watershed. As stated in Chapter 4, the rainfall intensities used for the design storm events are:

- 100-year/24-hour duration, total rainfall depth 11.0 inches
- 50-year/24-hour duration, total rainfall depth 10.0 inches
- 25-year/24-hour duration, total rainfall depth 8.0 inches
- 10-year/24-hour duration, total rainfall depth 7.0 inches
- 5-year/24-hour duration, total rainfall depth 5.5 inches
- 2.33-year/24-hour duration, total rainfall depth 4.5 inches

In addition, three 120-hour storm simulations were performed at the request of SWFWMD. The rainfall intensities used for these storm events are:

- 100-year/120-hour duration, total rainfall depth 17.8 inches
- 50-year/120-hour duration, total rainfall depth 15.9 inches
- 10-year/120-hour duration, total rainfall depth 11.3 inches

Initial lake elevations used in the stormwater management model at the start of the design storm event were determined from the recorded data provided by SWFWMD. Generally, the seasonal high water elevations were adopted for obtaining the most conservative results.

6.1 Existing Conditions Model Simulation Results

Hydrograph for each subbasin is generated by the hydrologic model routing through the hydrodynamic model to calculate time-varying stages and discharges. The updated Delaney Creek Area’s stormwater management model results for the 2.33-year/24-hour, 5-year/24-hour, 10-year/24-hour, 25-year/24-hour...
hour, 50-year/24-hour, 100-year/24-hour, and 100-year/120-hour design storm events are listed in Table 6-1. This table presents peak flood elevations at junctions in the junction-reach diagram.

6.2 Level of Service Analysis

This section describes the methodology utilized in defining the Level of Service (LOS) for the various subbasins within the Delaney Creek Area. Figure 6-1 contains a graphical representation of the LOS diagram of the watershed.

6.2.1 Level of Service Methodology

The Hillsborough County Comprehensive Plan, Stormwater Element contains definitions for the LOS flood protection designations. These definitions specify that a storm return period, storm duration, and a letter designation are required to define a level of flood protection. The flood level of service designations contained in the Comprehensive Plan are A, B, C, and D. A is the highest service level and D is the lowest. However, these criteria are somewhat subjective in what is termed as “significant” flooding. Therefore, for the purposes of this study, an interpretation of this definition is assigned to the LOS categories. The following contains the interpretation of the Comprehensive Plan definitions used in the LOS analysis.

Hillsborough County has recently updated the LOS definitions to be used throughout the project area as interpreted in the table below. These definitions are for the 25-year/24-hour storm event. The desired LOS for Hillsborough County is Level B.

<table>
<thead>
<tr>
<th>Level</th>
<th>Hillsborough County Comprehensive Plan Definition</th>
<th>Master Plan Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>No significant street flooding. All lanes are drivable.</td>
<td>No flooding.</td>
</tr>
<tr>
<td>B</td>
<td>Minor street flooding. At least one lane is drivable.</td>
<td>Street Flooding is more than 3” and 6” or less above crown of road.</td>
</tr>
<tr>
<td>C</td>
<td>Street flooding. Flooding depth above the crown of the road is less than one foot.</td>
<td>Street Flooding is more than 6” and 12” or less above crown of road.</td>
</tr>
<tr>
<td>D</td>
<td>No limitation on flooding.</td>
<td>Street Flooding is more than 12” above crown of road.</td>
</tr>
</tbody>
</table>

It was decided that drivable refers to less than or equal to three (3) inches of water above the crown of the road. It was also decided that one (1) lane passable means one (1) lane in each direction for a four (4) lane road or larger, or one (1) lane along the center of the road for a two (2) lane road.
The LOS designations in the Comprehensive Plan assumed that the sites (ground level surrounding adjacent property) are higher than the roads and that the houses are higher than the roads and the sites’. The Comprehensive Plan contains estimated Adopted (existing) and Ultimate (proposed) LOS designations for several watersheds in Hillsborough County.

* This is not always the case. It is possible to have a subbasin where the road does not flood (LOS A) or has minor flooding (LOS B) yet the site, and even the structure may flood. These situations are noted in Table 6-2 and Section 6.3.

6.2.2 Establishment of Landmark Elevations

Landmark elevations must first be determined in order to evaluate the LOS. These elevations refer to landmarks contained in the LOS definitions, including roads, sites, and structures. Landmark elevations are established for every subbasin in the watershed. These landmarks then serve as a tool for determining the LOS for the subbasin and on a broader scale, the system and the watershed. The landmark elevations established for LOS analysis are the critical or lowest landmark elevations in a subbasin. The critical landmark elevations are reflective of the worst case flooding that could occur in a subbasin. These are obtained from survey data and from topographic analysis. Every subbasin in the watershed is examined for the critical structure, site and road elevation. Table 6-2 contains landmark elevations determined for each subbasin of the Delaney Creek Area.

6.3 Existing Conditions Level of Service

Using flood protection LOS designation criteria contained in the previous section, the landmark elevations for each subbasin were compared to the computed results of the hydraulic model. The objective is to present both the areas and major structures where the computer model indicates insufficient storm conduit capacity exists and flooding occurs along the Delaney Creek Area watershed channel alignments. In general, LOS determinations will be segmented into each conveyance system described in Chapter 3. Diagnosis of major factors is also provided for each deficient LOS subbasin. Table 6-2 and Figure 6-2 illustrate the LOS exhibit of Delaney Creek Area. Overall speaking, more than 92% of DCA subbasins have a LOS of B and above.

6.3.1 Delaney Creek Main Channel System (model # 210xxx series)

Most subbasins of the Delaney Creek Main Channel System have a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

*Street flooding during the 25-year/24-hour design storm event:*
• Subbasin 210010 (LOS C) – Hartford Street will experience flooding due to low topography.

• Subbasin 210040 (LOS D) – South 50th Street will experience flooding due to insufficient storm conduit capacity; Trenton Street will experience flooding due to low topography.

• Subbasin 210045 (LOS C) – 36th Avenue South will experience flooding due to insufficient storm conduit capacity.

• Subbasin 210080 (LOS C) – South 54th Street will experience flooding due to low topography and insufficient storm conduit capacity.

• Subbasin 210100 (LOS C) – 34th Avenue South may experience flooding due to low topography.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 210010 – Channel surrounding residential areas are predicted both site and structure flooding due to low topography.

• Subbasin 210040 – Industrial site and structures adjacent to Trenton Street are predicted flooding due to low topography.

• Subbasin 210060 – A couple of structures north to the main channel may experience flooding due to low topography.

• Subbasin 210080 – Some residential sites are predicted flooding due to low topography and insufficient channel capacity.

• Subbasin 210125 – Several houses will experience site and structure flooding east of Maydell Drive due to lack of storm conduit system.

• Subbasin 210280 – Utility site is predicted flooding due to insufficient channel capacity.

• Subbasin 210290 – Open land will experience flooding due to insufficient channel capacity.

• Subbasin 210300 – Open land will experience flooding due to insufficient channel capacity.

6.3.2 Delaney Creek Laterals

6.3.2.1 Lateral “A” (model # 211xxx series)

The majority of Delaney Creek Lateral “A” subbasins has a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:
Street flooding during the 25-year/24-hour design storm event:

- Subbasin 211030 (LOS D) – Haven Oak Circle may experience flooding due to low topography.
- Subbasin 211060 (LOS D) – Maydell Drive is predicted flooding due to insufficient storm conduit capacity.
- Subbasin 211080 (LOS C) – 20th Avenue South is predicted flooding due to insufficient storm conduit capacity.
- Subbasin 211165 (LOS D) – South 66th Street is predicted flooding due to lack of storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 211060 – A house or two will experience site flooding southeast of Maydell Road/20th Avenue intersection.
- Subbasin 211116 – Several houses will experience slight site and structure flooding northwest of Maydell Road/16th Avenue intersection.
- Subbasin 211160 – A house or two will experience slight site and structure flooding east to South 66th Street.

6.3.2.2 Lateral “A-1” (model # 2115xx series)

Two thirds of Delaney Creek Lateral “A-1” subbasins do not meet the LOS B criteria for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 211510 (LOS C) – 32nd Avenue South is predicted flooding due to insufficient storm conduit capacity.
- Subbasin 211530 (LOS D) – 36th Avenue South is predicted flooding due to insufficient storm conduit capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 211510 – Several houses will experience site flooding.
- Subbasin 211530 – Several houses will experience site and structure flooding.
6.3.2.3  Lateral “B” (model # 212xxx series)

More than half of the Delaney Creek Lateral “B” subbasins do not meet the LOS B criteria for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 212030 (LOS C) – South 70th Street/24th Avenue South intersection is predicted flooding due to insufficient storm conduit capacity.
- Subbasin 212060 (LOS C) – South 70th Street near the channel area is predicted flooding due to insufficient channel capacity.
- Subbasin 212100 (LOS D) – Robindale Road at channel crossing is predicted flooding due to insufficient channel capacity.
- Subbasin 212130 (LOS D) – Destin Drive/South 76th Street at the channel crossing is predicted flooding due to insufficient channel capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 212130 – Several houses will experience site flooding near the Destin Drive and South 76th Street crossing.

6.3.2.4  Lateral “C” (model # 213xxx series)

Half of Delaney Creek Lateral “C” subbasins have a LOS D for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 213010 (LOS D) – Tidewater Trail at channel crossing is predicted flooding due to insufficient channel capacity.
- Subbasin 213040 (LOS D) – Ridein Road at channel crossing is predicted flooding due to insufficient channel capacity.
- Subbasin 213060 (LOS D) – Rideout Road at channel crossing is predicted flooding due to insufficient channel capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

There is no site nor structure flooding predicted within this system.
6.3.2.5 Lateral “C-1” (model # 2135xx series)

All Delaney Creek Lateral “C-1” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.2.6 Lateral “D” (model # 214xxx series)

All Delaney Creek Lateral “D” subbasins have a LOS A or B for the 25-year/24-hour design storm event with exception of one. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 214010 (LOS C) – Palm River Road downstream of the pond is predicted flooding due to insufficient storm conduit capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 214000 – The pastureland west of the commercial plaza will experience flooding due to insufficient channel capacity.

6.3.2.7 Lateral “E” (model # 2150xx series)

All Delaney Creek Lateral “E” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.2.8 Lateral “E-1” (model # 2155xx/2156xx series)

The majority of Delaney Creek Lateral “E-1” subbasins have a LOS A for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 215532 (LOS D) – Horace Avenue is predicted flooding at South Falkenburg Road crossing due to insufficient storm conduit capacity.
- Subbasin 215543 (LOS D) – Sand Street is predicted flooding due to insufficient storage/lack of storm conduit system.
- Subbasin 215544 (LOS C) – Elizabeth Pl. is predicted flooding due to insufficient storage/lack of storm conduit system.
Subbasin 215545 (LOS C) – Pride Road is predicted flooding due to insufficient storage/lack of storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

There is no site nor structure flooding predicted within this system.

6.3.2.9 Lateral “F” (model # 216xxx series)

All Delaney Creek Lateral “F” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.2.10 Hendrics Lake System (model # 220xxx/223xxx/225xxx series)

Most Hendrics Lake System subbasins have a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 220220 (LOS D) – Oak Regency Lane is predicted flooding due to insufficient storage/lack of storm conduit system.
- Subbasin 223060 (LOS D) – Lakeside Drive is predicted flooding due to insufficient storage.
- Subbasin 225016 (LOS C) – Oakfield Drive is predicted flooding due to insufficient storage.
- Subbasin 225110 (LOS D) – Buckingham Pl. is predicted flooding due to insufficient storage/lack of storm conduit system.
- Subbasin 225120 (LOS D) – West Robertson Street is predicted flooding due to insufficient storage/lack of storm conduit system.
- Subbasin 225150 (LOS D) – South Moon Avenue is predicted flooding due to insufficient storm conduit capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 220050 – Open land is predicted flooding due to insufficient storm conduit capacity.
- Subbasin 223050 – Reservoir adjacent houses will experience site and structure flooding due to insufficient storage.
- Subbasin 223060 – Reservoir adjacent houses will experience site flooding due to insufficient storage.
• Subbasin 225120 – The commercial building/parking lot at Buckingham Pl./West Robertson Street intersection will experience flooding due to insufficient storage/lack of storm conduit system.

6.3.2.11 Brandon Town Center Mall (model # 221xxx/217xxx series)

All Brandon Town Center Mall subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.2.12 Gortno Lake System (model # 2214xxx/2215xx series)

All Gortno Lake System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event except one location below:

Site, structural flooding during the 25-year/24-hour design storm event:
• Subbasin 221499 – Pasture land is predicted flooding due to insufficient channel capacity.

6.3.2.13 Heather Lakes System (model # 222xxx series)

All Heather Lakes System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.2.14 Lumsden Road North Ditch (model # 224xxx series)

All Lumsden Road North Ditch subbasins have a LOS A for the 25-year/24-hour design storm event except one. The EXTRAN model predicts LOS deficiency at the following location during the 25-year/24-hour design storm event:

Street flooding during the 25-year/24-hour design storm event:
• Subbasin 224050 (LOS C) – South Kings Avenue at the West Lumsden Road crossing may experience flooding due to insufficient storm conduit capacity.

6.3.2.15 Hickory Hammock System (model # 230xxx/233xxx series)

Most Hickory Hammock System subbasins have a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:
• Subbasin 230180 (LOS D) – John Moore Road and Hickory Lake Drive are predicted flooding due to insufficient storm conduit capacity.
• Subbasin 230197 (LOS D) – Dewolf Road is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 230200 (LOS D) – Julie Lane/Bryan Road crossing is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 233000 (LOS C) – John Moore Road/Julie Lane crossing is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 233015 (LOS D) – East Lumsden Road is predicted flooding due to insufficient storage/lack of storm conduit system.

*Site, structural flooding during the 25-year/24-hour design storm event:*

• Subbasin 230180 – Some houses near Hickory Lake may experience site and structure flooding due to insufficient storm conduit capacity.

• Subbasin 230195 – A couple of houses will experience site and structure flooding due to insufficient storage.

• Subbasin 230197 – The house west of the pond will experience site flooding due to insufficient storage.

• Subbasin 230200 – Several houses around the pond will experience site flooding due to insufficient storage. A couple of houses close to Clarissa Drive/Bryan Road intersection will experience site and structure flooding due to insufficient storage.

• Subbasin 233000 – Several houses around the pond will experience site and structure flooding due to insufficient storage.

• Subbasin 233015 – Several houses around the pond will experience site and structure flooding due to insufficient storage.

6.3.2.16 Isolated System (model # 227xxx series)

Nearly one-third isolated system subbasins do not meet the LOS B criteria for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

*Street flooding during the 25-year/24-hour design storm event:*

• Subbasin 227020 (LOS D) – Bryan Road is predicted flooding due to insufficient storage/lack of storm conduit system.
• Subbasin 227027 (LOS C) – Holly Ter. and Fairmont Drive are predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227030 (LOS D) – Fig Tree Lane is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227035 (LOS C) – Beverly Boulevard is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227050 (LOS C) – Lithia Pinecrest Road/East Lumsden Road intersection is predicted flooding due to insufficient storage/storm conduit capacity.

• Subbasin 227054 (LOS D) – Lithia Pinecrest Road/East Lumsden Road intersection is predicted flooding due to insufficient storage/storm conduit capacity.

• Subbasin 227080 (LOS D) – Hummingbird Lane is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227084 (LOS D) – Lorea Lane is predicted flooding due to insufficient storage/lack of storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 227027 – Several houses will experience site flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227030 – Many houses around the pond will experience site flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227050 – The institutional site will experience site and structure flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 227080 – Several houses along the Hummingbird Lane will experience site and structure flooding due to insufficient storage/lack of storm conduit system.

6.3.3 Delaney Pop-off System (model # 200xxx series)

The majority of Delaney Pop-off “200xxx” System subbasins have a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:
• Subbasin 200025 (LOS D) – Pendola Point Road is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 200095 (LOS D) – Portland Street is predicted flooding due to insufficient channel capacity.

*Site, structural flooding during the 25-year/24-hour design storm event:*

• Subbasin 200095 – Utility site is predicted flooding due to insufficient channel capacity.

• Subbasin 200110 – Pastureland is predicted flooding due to insufficient channel capacity.

• Subbasin 200120 – Open land is predicted flooding due to insufficient channel capacity.

• Subbasin 200315 – Open land is predicted flooding due to insufficient storage.

• Subbasin 200320 – Open land is predicted flooding due to insufficient storage.

• Subbasin 200330 – Rangeland is predicted flooding due to lack of storage.

6.3.4 Delaney Pop-off Main Channel System (model # 240xxx Series)

All Delaney Pop-off Main Channel System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are a couple of locations where site flooding is predicted:

*Site, structural flooding during the 25-year/24-hour design storm event:*

• Subbasin 240085 – Open land is predicted flooding due to insufficient storage/lack of storm conduit system.

• Subbasin 240390 – Open land is predicted flooding due to insufficient storage/lack of storm conduit capacity.

6.3.5 Delaney Pop-off Tributaries

6.3.5.1 Tributary “A” System (model # 243xxx Series)

All Delaney Pop-off Tributary “A” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are several locations where site flooding is predicted.

*Site, structural flooding during the 25-year/24-hour design storm event:*
• Subbasin 243020 – Pastureland is predicted flooding due to insufficient channel capacity.

• Subbasin 243120 – Several houses near South 74th Street will experience flooding due to insufficient storm conduit capacity.

• Subbasin 243150 – Southwest corner house will experience site flooding due to insufficient storm conduit capacity.

• Subbasin 243510 – Pastureland is predicted flooding due to insufficient storage.

• Subbasin 243520 – Pastureland is predicted flooding due to insufficient storage.

6.3.5.2 Tributary “B” System (model # 247xxx Series)

The majority of Delaney Pop-off Tributary “B” System subbasins have a LOS A or B for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

• Subbasin 247060 (LOS D) – Tammell Road is predicted flooding due to insufficient conduit capacity.

• Subbasin 247070 (LOS C) – Clifford Sample Drive is predicted flooding due to insufficient storage/lack of storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

There is no site nor structure flooding predicted within this system.

6.3.5.3 Tributary “C” System (model # 244xxx Series)

All Delaney Pop-off Tributary “C” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.5.4 Tributary “E” System (model # 2420xx Series)

All Delaney Pop-off Tributary “E” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.
6.3.5.5 Tributary “F” System (model # 2425xx Series)

All Delaney Pop-off Tributary “F” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.5.6 Tributary “G” System (model # 246xxx Series)

All Delaney Pop-off Tributary “G” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there is one location where site flooding is predicted:

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 246025 – Pastureland is predicted flooding due to insufficient storage.

6.3.5.7 Tributary “H” System (model # 2465xx Series)

All Delaney Pop-off Tributary “H” System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.5.8 Tributary “I” System (model # 2478xx Series)

All Delaney Pop-off Tributary “I” subbasins have a LOS A or B for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.5.9 Aspen Cove Apartment System (model # 250xxx/251xxx/254xxx series)

All Aspen Cove Apartment System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are a couple of locations where site flooding is predicted:

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 250550 – Open land is predicted flooding due to insufficient storage.

- Subbasin 254050 – The parking lot is predicted flooding due to insufficient storm conduit capacity.

6.3.5.10 “Model # 248xxx” Series System

All “model # 248xxx” Series System subbasins have a LOS A for the 25-year/24-hour design storm event with exception of one subbasin. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:
Street flooding during the 25-year/24-hour design storm event:

- Subbasin 248045 (LOS D) – Bellewater Boulevard is predicted flooding due to insufficient storage/lack storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 248045 – Several houses around the pond are predicted site flooding due to insufficient storage.

6.3.5.11 Evergreen Estates System (model # 252xxx series)

All Evergreen Estates System subbasins have a LOS A for the 25-year/24-hour design storm event except one. The EXTRAN model predicts LOS deficiency at the location below during the 25-year/24-hour design storm event.

Street flooding during the 25-year/24-hour design storm event:

- Subbasin 252050 (LOS C) – Alamba Avenue may experience flooding due to insufficient channel capacity.

6.3.5.12 I-75 Ditch System (model # 253xxx series)

All I-75 Ditch System subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.6 North Archie Creek Main Channel System (model # 260xxx series)

All subbasins of the North Archie Creek Main Channel System have a LOS A or B for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are several locations where site flooding is predicted:

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 260230 – Several houses will experience site flooding due to insufficient channel capacity.
- Subbasin 260240 – Pastureland will experience flooding due to insufficient channel capacity.
- Subbasin 260250 – Pastureland on the channel sides will experience flooding due to insufficient channel capacity.
- Subbasin 260285 – Pastureland will experience flooding due to insufficient storage.
• Subbasin 260312 – Pastureland will experience flooding due to lack of storage.

• Subbasin 260315 – Pastureland will experience flooding due to insufficient storage.

### 6.3.7 North Archie Creek Tributaries

#### 6.3.7.1 Tributary “A” (model # 261xxx series)

All subbasins of the North Archie Creek Tributary “A” have a LOS A or B for the 25-year/24-hour design storm event except one subbasin. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

*Street flooding during the 25-year/24-hour design storm event:*

• Subbasin 261060 (LOS D) – Gumwood Avenue will experience flooding due to insufficient storage.

*Site, structural flooding during the 25-year/24-hour design storm event:*

There is no site nor structure flooding predicted within this system.

#### 6.3.7.2 Tributary “B” (model # 262xxx series)

Half of the North Archie Creek Tributary “B” subbasins do not meet the LOS B criteria for the 25-year/24-hour design storm event. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

*Street flooding during the 25-year/24-hour design storm event:*

• Subbasin 262000 (LOS D) – Endive Avenue will experience flooding due to insufficient storm conduit capacity.

• Subbasin 262020 (LOS D) – Allamanda Avenue will experience flooding due to insufficient storm conduit capacity.

• Subbasin 262030 (LOS C) – South 83rd Street will experience flooding due to insufficient storm conduit capacity.

*Site, structural flooding during the 25-year/24-hour design storm event:*

• Subbasin 262000 – Many houses along Endive Avenue will experience flooding due to insufficient storm conduit capacity.

• Subbasin 262020 – A couple of houses may experience site flooding due to insufficient storm conduit capacity.
6.3.7.3  Tributary “C” (model # 265xxx series)

All North Archie Creek Tributary “C” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.7.4  Unnamed Tributary (model # 263xxx series)

All North Archie Creek Unnamed Tributary subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.7.5  Model # 264xxx/266xxx/267xxx/269xxx Series

All these subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there is several locations predicted site flooding as below.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 266000 – Forest will experience flooding due to insufficient storm conduit capacity.
- Subbasin 266010 – Forest will experience flooding due to lack of storm conduit system.
- Subbasin 266020 – Forest will experience flooding due to lack of storm conduit system.

6.3.7.6  Tributary “D” (model # 2705xx series)

All North Archie Creek Tributary “D” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.7.7  Tributary “E” (model # 272xxx series)

All North Archie Creek Tributary “E” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there is one location where site flooding is predicted.

Site, structural flooding during the 25-year/24-hour design storm event:

- Subbasin 272000 – A couple of houses will experience site flooding due to insufficient storage.
6.3.7.8 Tributary “F” (model # 273xxx series)

All North Archie Creek Tributary “F” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are several locations where site flooding is predicted:

*Site, structural flooding during the 25-year/24-hour design storm event:*

- Subbasin 273010 – There is a house may experience site flooding due to insufficient channel capacity.
- Subbasin 273020 – There is a house may experience both site and structure flooding due to insufficient channel capacity.
- Subbasin 273800 – The commercial site east to the wetland will experience flooding due to insufficient storage capacity.

6.3.7.9 Tributary “G” (model # 27004x series)

All North Archie Creek Tributary “G” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.7.10 Model # 270xxx Series

All these subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.7.11 Model # 274xxx/276xxx Series

All these subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.8 Archie Creek Main Channel System (model # 2800xx/2900xx series)

All subbasins of the Archie Creek Main Channel System have a LOS A or B for the 25-year/24-hour design storm event, except two. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

*Street flooding during the 25-year/24-hour design storm event:*


• Subbasin 290045 (LOS C) – Mint Julep Circle will experience flooding due to insufficient storm conduit capacity.

• Subbasin 290055 (LOS C) – Krycul Avenue will experience flooding due to insufficient storm conduit capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 290015 – Northwest area will experience flooding due to insufficient storage.

• Subbasin 290055 – A house or two may experience flooding due to insufficient storm conduit capacity.

6.3.9 Archie Creek Tributaries

6.3.9.1 Tributary “A” (model # 2801xx series)

All Archie Creek Tributary “A” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.9.2 78th Street Ditch (model # 2803xx series)

All 78th Street Ditch subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there are several locations where site flooding is predicted.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 280360 – TECO easement will experience flooding due to lack of storm conduit system/insufficient storage.

• Subbasin 280385 – Open land will experience flooding due to insufficient channel capacity.

• Subbasin 280397 – Open land will experience flooding due to insufficient channel capacity.

6.3.9.3 Tributary “B” (model # 2804xx series)

All Archie Creek Tributary “B” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street flooding to occur during the 25-year/24-hour design storm event. However, there is one location where site flooding is predicted.

Site, structural flooding during the 25-year/24-hour design storm event:
• Subbasin 280405 – TECO easement will experience flooding due to insufficient channel capacity.

6.3.9.4 Tributary “C” (model # 2805xx series)

All Archie Creek Tributary “C” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.9.5 Tributary “C-1” (model # 2806xx series)

All Archie Creek Tributary “C-1” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.3.9.6 Tributary “D” (model # 2902xx/2903xx series)

All Archie Creek Tributary “D” subbasins have a LOS A or B for the 25-year/24-hour design storm event except one subbasin. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

• Subbasin 290311 (LOS D) – Lake Saint Charles Boulevard will experience flooding due to insufficient storage/lack of storm conduit system.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 290311 – There is a house or two near the pond will experience flooding due to insufficient storage/lack of storm conduit system.

6.3.9.7 Tributary “F” (model # 290xxx/2905xx/2906xx series)

All Archie Creek Tributary “F” subbasins have a LOS A or B for the 25-year/24-hour design storm event except one subbasin. General locations where the EXTRAN model predicts LOS deficiencies are summarized below:

Street flooding during the 25-year/24-hour design storm event:

• Subbasin 290612 (LOS C) – Springbrook Drive will experience flooding due to insufficient channel capacity.

Site, structural flooding during the 25-year/24-hour design storm event:

• Subbasin 290572 – Shrub land will experience flooding due to lack of storm conduit system.
6.3.9.8 Tributary “G” (model # 2901xx series)

All Archie Creek Tributary “G” subbasins have a LOS A for the 25-year/24-hour design storm event. The EXTRAN model predicts no major street, site, or structural flooding to occur during the 25-year/24-hour design storm event.

6.4 Model Comparisons

6.4.1 Comparison of Updated Model and Previous Model Results

Comparisons have been made of the simulated peak water surface elevations produced by the 2003 version of the DCA model obtained from Hillsborough County and the 2007 updated version developed by Ayres Associates. Comparisons were made for both the 25-year/24-hour and 100-year/24-hour design events. Numerous changes have been made for the 2007 update including but not limited to:

- Land use changes - impacting computed curve numbers, times of concentrations and subsequently runoff rates and volumes
- Completed County’s stormwater Capital Improvement Projects (CIPs)
- Level of subbasin discretization and hydraulic detail
- Alterations to model connectivity (in some cases as corrections and in most cases for physical revisions to the drainage system)
- Incorporation of updated and more accurate topographic information, resulting in improved simulation of out-of-channel storage, flooded subbasin storage, and intersubbasin flow connections
- Addition of numerous overland flow paths to represent roadway overtopping, and depression area or pond overbanking during severe storm events

As a result of these model revisions, simulated flood stages at most model nodes will be impacted to some degree. In areas where flood-control CIPs have been constructed or significant channel maintenance activities have been performed, a notable drop in flood stage can be observed. In other areas, the inclusion of upstream overtopping reaches or the increase of impervious area in volume sensitive subbasins has resulted in an increased predicted flood stage. In a small number of areas, changes in predicted flood stage have occurred because of necessary corrections to connectivity or the addition of missing entrance loss and exit loss coefficients for closed conduits.
Table 6-3 presents the comparison of previous and current model flood stage predictions. Some notable areas of change are described below:

- Delaney Creek’s main channel system in the vicinity of the Crosstown Expressway had vertical channel walls defined in the previous model with no out-of-channel storage. Incorporation of out-of-bank storage using the latest topographic data has improved flood stages in this area and upstream of this area by an average of 1.5 feet.

- Delaney Creek subbasins located between Brandon Town Center Drive and I-75 have realized drastic reductions (>4 feet) in simulated flood stage through the incorporation of inter-basin weir paths.

- Reductions in flood stage from 1.5 feet to 3 feet are seen throughout most of the Delaney Creek tributaries, attributable to the lower HGL in the main channel.

- Delaney Pop-off Canal’s lower main channel (200xxx) exhibits a reduced peak flood stage in the updated model of 2 to 3 feet. This is primarily due to the inclusion of previously unaccounted out-of-bank storage curves.

- Incorporation of the Canterbury Lakes Regional Detention Facility reduces flood stages in the Sanson Park area (247xxx, 2401xx, and 2402xx) from 0.2 to 1.8 feet.

- Some notable rises in peak stage at the Falkenburg Road nodes 240310 through 240370 are the results of entrance/exit loss coefficients being added. The LOS remains “A”.

- Addition of out-of-channel storage curves using updated topography has lowered the predicted peak flood stage along North Archie Creek in the vicinity of I-75 and Progress BoulevaRoad Upstream areas (nodes 270xxx) also benefit from the lowered main channel stages.

- Closed North Archie subbasins (272xxx) have generally exhibited higher predicted flood stages due to topography-based raising of overtopping elevations. The LOS remains “A.” Similar effects are seen for several 273xxx subbasins.

- Archie Creek nodes associated with Cargill holding ponds have undergone significant changes as Cargill operations progress. Large changes in simulated stage are mostly due to higher initial stages and higher control elevations defined in the model. These systems are closed systems and do not impact the free flowing channel system.

- The Archie Creek lower main channel and its tributaries exhibit lower predicted peak stages downstream of 78th Street South as a result of Cargill’s outfall channel diversion. Main channel peak stages just upstream of 78th Street South increase as a result of corrected model connectivity.
6.4.2 Comparison of 100-year, 1-day and 100-year, 5-day Model Results

Table 6-4 presents a comparison of the predicted peak flood stages associated with the 100-year, 1-day and 100-year, 5-day events. The 1-day design event generally will produce a single system peak. The 5-day design event incorporates a higher volume of rainfall but distributes it as light, saturating rain for much of the first day and then simulates two distinct high-intensity episodes. Most free-flowing systems will exhibit a fairly similar peak stage with either event. Distinctly higher 5-day peak stages generally indicate a rate-limited, closed or volume sensitive subbasin.

As revealed in Table 6-4, the differences in 1-day and 5-day peak stage ranges from -9.10 feet to +3.67 feet, with an average difference of 0.32 feet. The vast majority of this variability is seen in the Delaney Creek system (22xxxx), which exhibits an average difference of 0.77 feet.

6.5 100-year Floodplain Delineation

Areal extent of 100-year floodplains was mapped for the 100-year return frequency flood simulations using HCSWMM model peak stage simulation results and best available topographic data. With knowledge of the reach type, nodal flood elevations were transitioned from subbasin to subbasin and appropriately attributed as estimated or determined flood stages depending on whether reaches were overland weirs or explicitly modeled channel reaches. Figure 6.3 presents the extent of mapped floodplain boundaries for the DCA watershed.

Attribute data for floodplain mapping included Flood Zone designation according to the following guidelines:

Zone AE - flood insurance rate zones that correspond to the 100-year floodplains that are determined in the Flood Insurance Study by detailed methods. In most instances, Base Flood Elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone. Mandatory flood insurance purchase requirements apply. The proposed AE floodplain boundaries for the DCA watershed generally represent level-pool flood stages associated with wetlands, lakes, ponds, and subbasins with significant depression storage. Along defined channels, AE floodplain boundaries represent the anticipated flood profile along the reach, which varies along its length and typically follows the channel slope.

Zone A - flood insurance rate zones that correspond to the 100-year floodplains that are determined in the Flood Insurance Study by approximate methods. Because detailed hydraulic analyses are not performed for these areas, no Base Flood Elevations or depths are shown within this zone. Mandatory flood insurance purchase requirements apply. The proposed A floodplain boundaries for the DCA watershed generally represent one of three conditions:
1. Wetlands or water features mapped in the National Wetlands Inventory (NWI) or 2006 FLUCCS coverages provided by the District but not meeting G&S threshold criteria for subbasin delineation and individual storage curve generation, but assumed to be fully saturated;

2. Small stormwater management areas (SMSAs) not explicitly modeled due to G&S threshold criteria, but assumed to be filled with water;

3. Transitional conveyances between flooded subbasins connected by overland or overbanking weirs, where notable changes in flood elevation are observed between the upstream and downstream storage features – best engineering judgment was applied.

Based on the 100-year floodplain delineation, Zone AE areas are approximately 7,220 acres and Zone A areas are about 314 acres.
CHAPTER 7 ALTERNATIVES ANALYSIS UPDATE

The objective of this chapter is to describe the performance of the 2001 preferred alternatives under the DCA 2007 system conditions. Alternatives were developed to provide an upgrade to the existing Level of Service for areas that historically experienced flooding deficiencies. Discussion includes evaluation of those alternative improvements already implemented by 2007 and now considered part of the current “Existing Condition”, as well as assessment of the efficacy of remaining flood control proposals.

7.1 Flood Control Alternatives Development

Flood control alternatives were developed based on the previously calibrated model simulations of the DCA watershed’s primary drainage system. Locations with potential flooding concerns were identified by applying the County’s adopted flooding level of service criteria (LOS) and noting where the LOS criteria were not being met.

As a primary level of analysis, known flooding concerns in the study area were evaluated based on a review of historical flood complaint information that was compiled from the recorded available data. The staff of the Hillsborough County Stormwater Section developed this system as a means of recording and monitoring response activities to residential flooding-related complaints that are received. The nature and extent of these flood concerns was quite varied, but could be aggregated into three general classifications:

- Minor drainage problems,
- Maintenance problems, and
- Flooding problems

Minor drainage problems are considered as those, which are not associated with the main drainage system, but with the minor drainage system, such as roadside ditches, street drains and lot lines swales. Generally, these complaints are associated with complaints requiring completion of vegetative control activities, as well as ditch and culvert cleaning.

Maintenance problems are those requiring repairs or prevention of eroded natural and/or manmade channel banks, culvert failure, impaired drainage, or where there is a risk to public works such as roadways.

Flooding problems are considered as complaints of property and street flooding associated with the primary drainage system of the DCA watershed. Focus was directed to this latter category.
Based on the 2001 model results, Hillsborough County staff identified a set of flooding deficiencies that were to be addressed within the DCA Watershed Management Plan.

### 7.2 Implemented Flood Control Projects

At the time of the last report, Hillsborough County was in a land acquisition process for implementing the Delaney Creek Stormwater Management Improvements Phase I project on the Delaney Creek main channel system. Phase I channel improvements have since been completed and are included as an “Existing Condition” for the 2007 DCA SMMP update. Channel improvements produced a lowered flood profile of 1-2 feet throughout the system.

Maydell Drive Bridge upgrade is completed and included in the “Existing Condition” of the 2007 model. Previous 9 x 17.5 foot concrete box culvert was replaced with a 12 x 32 foot Conspan.

In addition, the Sanson Park flooding problems were anticipated to be alleviated through the Canterbury Lakes Regional Stormwater Detention Facility Project that was under review for the Delaney Pop-off Canal area. The regional facility and channel improvements upstream and downstream of the detention pond are also complete and reflected as an “Existing Condition” in the current drainage model. The regional detention facility and channel improvements resulted in a 1-2 foot reduction in flood stages over a substantial area.

Also, the 78th Street South twin 4.58’ x 7’ arch metal pipes along the Delaney Pop-off main channel were replaced with reinforced concrete pipes as part of Capital Improvement Project (CIP) #47025, which was in the permitting stage during the previous modeling study. It is now included in the “Existing Condition” of the 2007 model.

The Evergreen Estates system experienced frequent street and yard flooding which the County has addressed through the construction of the 5,300 gallon-per-minute (capacity) Alambra Avenue pump station and the 1,540 gallon-per-minute (capacity) Ventura Avenue pump station, which diverts flows to Delaney Creek’s main channel. These pump stations were constructed as CIP #41102 and have been incorporated into the 2007 “Existing Condition” model. It is assumed that they provide notable relief of flooding from typical summer rainfall events. The simulated 25-year/24-hour design storm, however, produces runoff rates which greatly exceed the pump stations’ capacity.

Lastly, the proposed 1-acre detention facility for 78th Street South, located within the Archie Creek subwatershed north of Riverview Drive, has been completed as CIP# 41086. It is reflected as an “Existing Condition” for the 2007 DCA SMMP update and results in an LOS of “A” for the associated portion of 78th Street South.

Remaining flood control projects are described in the following section.
7.3 Remaining Flood Control Projects – Proposed Conditions

Flood control projects recommended, but not yet implemented, are described in the following subsections. The simulated “Existing” and “Proposed” condition LOS for the 2007 updated model were compared for the 25-year/24-hour design storm event. A tabular summary of results is presented as Table 7-1. Subbasins with LOS designations below “B” but exhibiting a change in LOS category resulting from proposed improvements are highlighted.

7.3.1 Delaney Creek Subwatershed

The Delaney Creek subwatershed originates at a point approximately 4,000 feet north of the Pauls Drive/Causeway Boulevard (S.R. 676) intersection and flows west approximately 8.0 miles through several residential and commercial areas to its eventual discharge into Hillsborough Bay.

7.3.1.1 Delaney Creek Main Channel System (model # 210xxx series)

The Delaney Creek Main Channel System incorporates the Clair Mel City area located between 54th Street South and the Crosstown Expressway in the east/west direction and Palm River Road and Causeway Boulevard in the north/south direction. This area includes Laterals “A” (211xxx model series), “B” (212xxx model series), “C” (2130xx model series), and “C-1” (2135xx model series). Clair Mel City has historically provided a low level of flood protection based on previous computer model results as well as numerous resident complaints for street and yard flooding.

Alternative Components:

In addition to the channel cross-section improvements already implemented between McKay Bay and the Crosstown Expressway, the following proposed elements were added to the 2007 updated model:

- U.S. 41 Bridge Upgrade – In order to correct overtopping of the U.S. 41 bridge during 50-year and higher design events (even after channel improvements), the addition of an 8’ x 12’ concrete box culvert to the three existing concrete box culverts is proposed at this location.

- 70th Street South Bridge Upgrade – The channel improvement upstream of this location resulted in an increase in flow rate and consequently of the head loss at 70th Street bridge structure. The computer model results reflected the necessity to decrease the water surface elevation upstream of this location in order to provide the adequate tailwater conditions for Laterals “B”, “C”, and “C-1” within the Clair Mel City area. There are flooding complaint records for a nearby location which can be attributed to the undersize structure – cross-section of 70th Street South Bridge. The existing concrete bridge pile is proposed to be replaced with a 12 x 32 foot Conspan structure (same size and shape as Maydell Drive structure).

The effectiveness of these improvements in stage reduction along the main channel is shown in Table 7-1. System LOS improves by one full category at several locations.
7.3.1.2 Laterals

The laterals for the Delaney Creek subwatershed flow from north to south into the main channel. Several of the laterals are manmade ditches that were excavated originally for agricultural purposes but serve as drainage ditches for the various subdivisions mentioned in the main channel system.

7.3.1.2.1 Lateral “A” (model # 211xxx series)

- Channel cross-section improvements are proposed between the confluence with Delaney Creek and 20th Avenue South. The proposed cross-section for this approximately 0.90 miles (4,800 feet) of natural channel alignment has been designed to preserve the existing top of banks location and reshape the bank slopes to a 3 (horizontal) to 1 (vertical) on both sides.

- The existing single and double 48-inch CMPs between Haven Oak Circle and the private driveway east of Maydell Drive crossing will be replaced with 5 x 16 foot concrete box culverts. The previous existing conditions model results reflect an overtopping water surface elevation at these locations, which is confirmed by the recorded flood complaints of this area.

The effectiveness of this improvement in stage reduction and relief of flooding conditions within the Lateral “A” drainage system is not impressive, as shown in Table 7-1. The raising of the road profiles at Maydell Drive should be considered.

7.3.1.2.2 Lateral “B” (model # 212xxx series)

- Channel cross-section improvements between the confluence with Delaney Creek and Robindale Road. The proposed cross-section for this approximately 0.7 miles (3,780 feet) of natural channel alignment has been designed to increase the Lateral “B” main channel width and reshape the bank slopes to a 2 (horizontal) to 1 (vertical) on both sides. All this work should be performed within the existing 80-foot drainage right-of-way. There is also proposed a 1,260 foot channel cross-section improvement on the ditch along Balfour Circle. The remaining 0.6 miles (3,288 feet) of natural channels within the Lateral “B” system shall be cleaned of nuisance vegetation and debris.

- Culverts under Tidewater Trail and Robindale Road are undersized resulting in road overtopping. The construction of a neighborhood drainage system that will provide adequate flood relief to the Clair Mel City residents in the vicinity of Lateral “B” will require the complete rehabilitation of the conveyance system, along the main channel that serves this subdivision. In addition to the natural channel cleaning and cross-section improvements mentioned at the beginning of this sub-section, the culvert replacement at Tidewater Trail and Robindale Road is part of the Lateral “B” flood control improvements. The proposed culvert replacements are as follows:

  - 12th Avenue South and South 76th Street location - the existing 44 x 66 inch ECMP will be replaced with 4 x 12 foot concrete box culvert.
- Robindale Road location - the existing twin 30 x 54 inch ERCPs will be replaced with 4 x 12 foot concrete box culvert.

- Tidewater Trail location - the existing twin 36 x 60 inch ERCPs will be replaced with 5 x 12 foot concrete box culvert.

- Balfour Circle location - the existing 48 inch CMP will be replaced with 4 x 6 foot concrete box culvert.

The effectiveness of this improvement in stage reduction and relieving the flooding conditions within the Lateral “B” drainage system is shown in **Table 7-1**.

7.3.1.2.3 Lateral “C” (model # 213xxx series)

- Channel cross-section improvements between the confluence with Delaney Creek and Rideout Road are proposed for this approximately 0.85 mile (4,480 feet) of natural channel alignment, increasing the Lateral “C” main channel width and reshaping the bank slopes to a 2 (horizontal) to 1 (vertical) on both sides. All this work should be performed within the existing 80-foot drainage right-of-way.

- Culverts under Tidewater Trail, Ridein Road, and Rideout Road are undersized, resulting in road overtopping. The construction of a neighborhood drainage system to provide flood relief to the Clair Mci City residents in the vicinity of Lateral “C” will require the complete rehabilitation of the conveyance system along the main channel that serves this subdivision. In addition to the natural channel cross-section improvements mentioned at the beginning of this sub-section, the following culvert replacements at Tidewater Trail, Ridein Road, and Rideout Road are proposed:

  - Tidewater Trail location - the existing twin 36” x 54” ERCPs will be replaced with a 5’ x 14’ concrete box culvert.

  - Ridein Road location - the existing twin 30” x 54” ERCPs will be replaced with 4’ x 12’ concrete box culvert.

  - Rideout Road location - the existing 34” x 53” ERCP will be replaced with 4’ x 8’ concrete box culvert.

The effectiveness of this improvement in stage reduction and relieving the flooding conditions within the Lateral “C” drainage system is shown in **Table 7-1**.

7.3.1.2.4 Lateral “C-1” (model # 2135xx series)

No flooding problems are identified from the model simulations nor were any complaints recorded within this drainage system.
7.3.1.2.5 Lateral “D” (model # 214xxx series)

The flooding concern at this location was identified from model simulations. The computed water surface exceeds the Palm River Road crossing overtopping elevation for the 25, 50, and 100-year design storms respectively. Despite model results, no flooding complaints have been recorded at this time; therefore, no improvement is proposed for this location.

7.3.1.2.6 Lateral “E” (model # 215xxx series)

No flooding problems were identified during the model simulations nor were any complaints recorded within this drainage system.

7.3.1.2.7 Lateral “E-1” (model # 2155xx series)

A flooding concern area, which was identified by both the model and complaint records, is along the Lateral “E-1” system between the Palm River Road crossing and Frank Adamo Drive (State Road 60). It was determined based on model results and field observations that the Palm River Road culvert is undersized, an existing 6’ x 5’ concrete box culvert, compared to the four 48” x 72” ERCPs upstream. Flooding complaints were recorded at a location upstream of the Palm River Road crossing. The area north (upstream) of the Adamo Drive crossing has also experienced flooding concerns based on complaint records. The proposed culvert replacements are as follows:

- Palm River Road location - the existing 6’ x 5’ concrete box culvert will be replaced with 6’ x 16’ concrete box culvert.
- Frank Adamo Drive (State Road 60) location - one additional 4’ x 6’ concrete box culvert to the existing twin 4’ x 6’ concrete box culverts.

The effectiveness of this improvement in stage reduction and relieving the flooding conditions within the Lateral “E-1” drainage system is shown in Table 7-1.

7.3.1.3 Hendrics Lake System (model # 220xxx series)

No flooding problems are identified from the model simulations. Although flooding complaints were recorded within this drainage system, they appear to be a local drainage problem, which cannot be resolved with improvements in the major channels.

7.3.1.4 Hickory Hammock Lake System (model # 230xxx series)

There was only one flooding problem identified from the updated model simulations. Flooding complaints were reportedly recorded within some of the subbasins and appear to be a local drainage problem. These flooding concerns cannot be resolved due to a lack of outfall for these subbasins.
7.3.1.5  Closed Basin System (model # 227xxx series)

Flooding complaints have been recorded within this drainage system; they appear to be a local drainage problem, which cannot be resolved with improvements in the major channels. However, lack of storm conduit system is the essential cause and should be resolved with local drainage improvements.

7.3.2  Delaney Pop-off Canal Subwatershed

The Delaney Creek Pop-off Canal subwatershed extends east to about U.S. Highway 301. The conveyance system consists of manmade ditches with no evidence of natural channel sections and generally flows south and west from U.S. Highway 301 to Hillsborough Bay. Major road crossings include U.S. Highway 301 at the eastern extremity, 78th Street near the middle, and Madison Avenue (State Road 676A) and U.S. Highway 41 at its western extremity.

7.3.2.1  Delaney Pop-off Main Channel System (model # 200xxx and 240xxx series)

Although road overtopping is not prolific along the Pop-off Canal’s main channel, there are several locations where improvements are necessary due to the water surface elevation impact on the major tributary systems.

The results of computer simulation identify the existing Old U.S. Highway 41 crossdrain as a significant headloss along the main channel system. Another location that shows a significant difference between upstream and downstream water surface elevations is identified at Madison Avenue. The large losses are attributable to the size and depth of the channel south of Madison Avenue. With the flow increase caused by the upstream improvements described later on this chapter, the Madison Avenue crossdrain will require an increase in capacity. The main purpose of the additional capacity at Madison Avenue is to improve the level of service on Tributary “F” located 600 feet upstream of Madison Avenue. Finally, the 78th Street South crossdrain shows significant headloss according to the computer model results.

- The existing 60” corroded metal pipe at Old U.S. Highway 41 is proposed to be replaced with an 8’ x 12’ concrete box culvert.
- An additional 5’ x 8’ concrete box culvert is proposed at Madison Avenue.
- The channel cross-section is proposed to be improved upstream of Madison Avenue for 4,000 linear feet of ditch alignment by matching the cross-section characteristics upstream of the current ditch narrowing. This improvement is also part of the Tributary “F” alternative solutions and will lower the water surface elevation upstream of Madison Avenue.
- Install a 25 foot weir structure on the Fortuna Acres subdivision pond located west of the Delaney Pop-off main channel and north of the Madison Avenue crossing.

The water surface elevation decreases at Old U.S. Highway 41 for the 25-year/24-hour design storm event with this upgrade. The proposed level of service of the adjacent subbasins will be upgraded from
B to A for the 25-year design storm event. The additional culvert at Madison Avenue will also lower the water surface elevation for the 25-year design storm event.

7.3.2.2 Tributaries

The tributaries of the Delaney Pop-off subwatershed consist mostly of manmade ditches along the back lots of residential areas like Fortuna Acres and Green Ridge Estates. The remaining tributaries on the east side of U.S. Highway 301 consist of the Interstate 75 drainage ditch and several ditch systems located in the Pavilion areas.

7.3.2.2.1 Tributary “F” (model # 2425xx series)

The Tributary “F” System incorporates the neighborhood east of Fortuna Acres subdivision. This channel runs east to west for approximately 1,300 feet until it reaches its confluence with the Delaney Pop-off Canal.

As identified in the Existing Condition chapter of this report, this area provides a reasonable level of flood protection with downstream improvements already implemented, but still predicts a significant extent of shallow yard flooding for a 25-year event. There is a history of numerous resident complaints for street and yard flooding.

- Channel improvements are proposed east of the main channel and north of Madison Avenue. The channel cross-section improvement on Tributary “F” begins to the west of 78th Street at the eastern end and ends at the confluence with the Pop-off Canal main channel 600 feet north of Madison Avenue. The improved ditch is approximately 1,240 feet length. The new cross-sections were designed to reshape the bank slopes to 2 (horizontal) to 1 (vertical) on both sides.

- Culvert upgrades are proposed as follows:
  - Replace the existing double 14” RCP with 3’ x 3.5’ concrete box culvert at the private driveway located on the west right-of-way of 74th Street South.
  - Replace the existing Palm Drive double 29” x 45” ERCP with 3.5’ x 6’ concrete box culvert.

The impact of the improvements described above is presented in Table 7-1.

7.3.2.3 Evergreen Estates System (model # 252xxx series)

The Evergreen Estates System incorporates the Evergreen Estates subdivision located between Falkenburg Road and U.S. Highway 301 and north of Causeway Boulevard. As identified in the Existing Conditions chapter of this report, this area provides a low level of flood protection based on computer model results as well as the numerous resident complaints for street and yard flooding.

In order to alleviate the flooding concerns in this residential area, a flow diversion to Delaney Creek’s main channel was proposed as a solution. The County has since constructed two pump stations, the
Alambra Avenue and Ventura Avenue pump stations, which divert flows to Delaney Creek’s main channel downstream of the Crosstown Expressway concrete pile bridge (junction #210240). These pump stations have been incorporated into the 2007 “Existing Condition” model but did not improve the 25-year LOS.

7.3.3 North Archie Creek Subwatershed

The North Archie Creek subwatershed extends as far east as Providence Avenue and as far north as the Crosstown Expressway. This subwatershed is similar to the Delaney Creek Pop-off Canal subwatershed in that it is drained by a system of manmade ditches and flows in a south and west direction to Hillsborough Bay. Some improvements and extensions to the ditch system have been made in the eastern portions of the subwatershed as a result of Interstate 75 and U.S. Highway 301 construction. A portion of North Archie Creek west of 78th Street has been relocated and expanded by Gardinier, Inc.

7.3.3.1 North Archie Main Channel System (model # 260xxx series)

Progress Village has reported a long history of frequent and long duration flooding. Historical observations indicate that numerous roads in the Progress Village area will flood during the 10-year and 25-year events. Some of the lower roadways will experience as much as 12 inches of flooding with the 25-year events and may persist for several days. The problems experienced in Progress Village (the 261xxx and 262xxx series in the model) are mainly caused by the higher water level in the main channel. The updated model results do not show flooding problems in the main channel but confirm flooding in the residential areas. Historically, field observations confirm that the channels downstream of 82nd Street have problems with vegetation and silt build-up.

The 82nd Street road crossing is located at the south end of Progress Village along North Archie Creek. This crossing has been identified as a separate problem from Progress Village because of its close proximity to an elementary school. The residents reported that the road and adjacent sidewalk become inundated during summer storms. This is the only access to the school and poses a significant safety hazard. Road Substantial vortexing occurs at the upstream end of the existing culverts.

The primary proposed improvement discussed in the previous SMMP for the 82nd Street Road Crossing and Progress Village was cleaning and snagging of channels downstream of 82nd Street. Previous report narrative concluded that this maintenance activity would totally solve the problem, as the key was lowering the water level of the receiving channel. These channel improvements were not, however, incorporated into the final “Proposed Condition” model which was updated by Ayres in 2007.

The following structural upgrade was included in the proposed model:

- Upgrade the culvert at Old U.S. Highway 41 from the existing double 5’ x 13’ culvert to three 6.5’ x 13’ culverts.

As reflected in Table 7-1, upgrading the Old U.S. Highway 41 had nominal impact on flood levels. The channel improvements appear to be the more critical element.
7.3.4 Archie Creek Subwatershed

Half of the Archie Creek subwatershed consists of commercial areas like the Cargill complex and the Parkway Business Center. The other half contains areas like the Lake Street Charles, Starlite, Ashley Oaks, Suntree Estates, and McMullen Farms subdivisions. Cargill occupies about a third of the subwatershed. The subwatershed generally starts at the Ashley Oaks subdivision and discharges out into the Bay near the north part of the Cargill facility.

7.3.4.1 Archie Creek Main Channel System (model # 280xxx and # 290xxx series)

The Archie Creek Main Channel System incorporates the Cargill complex, Parkway Business Center, Lake Street Charles subdivision, and the Ashley Oaks subdivision. This area includes Tributary “A” (2801xx model series), 78th Street Ditch (2803xx model series), “B” (2804xx model series), and “C” (2805xx model series). As identified in the Existing Condition LOS discussion, the Ashley Oaks area near Krycul Avenue provides a low level of flood protection based on computer model results as well as the resident complaints for street and yard flooding.

The following improvements have been proposed to improve flood stages:

- Channel cross-section improvements (approximately 1,125 LF) are proposed between Bucks Ford Drive in the Lake Street Charles subdivision and Mint Julep Circle in the Ashley Oaks subdivision. The new cross-sections were designed to preserve the existing/or proposed top of banks location and to reshape the bank slopes to a 2 (horizontal) to 1 (vertical) on both sides; increasing the channel capacity to match the downstream cross-sections of Archie Creek.

- Upgrade the existing 26” x 34” ERCP at Mint Julep Circle located on the west side of the Ashley Oaks treatment pond to a double 36” RCP.

- Upgrade the existing 26” x 36” ERCP at Mint Julep Circle located on the east side of the Ashley Oaks treatment pond and west of the Ashley Oaks entrance to a double 36” RCP.

- Upgrade the existing 15” x 24” ERCP under Krycul Avenue located north of the Ashley Oaks entrance to a 42” RCP.

As shown in Table 7-1, this alternative can significantly lower the water level in the main channel and in the problem areas. This alternative has little effect on the downstream channel.

7.3.4.2 Tributaries

The tributaries are comprised mostly of manmade ditches that originate from wetland areas like Tributary “A” and “C”. The other tributaries were originally excavated for agricultural uses but now serve as drainage channels for areas like the Parkway Business Center, Starlite, Lake Street Charles, and Ashley Oaks subdivisions.

7.3.4.2.1 Tributary “A” (model # 2801xx series)
The Tributary “A” System incorporates the Rinker complex, Parkway Business Center, and the Trinity College of Florida. In the previous WMP, this area reported a low level of flood protection for 78th Street South based on computer model results, with street flooding indicated. Changes to the main channel outfall, constructed by Cargill, have provided a more favorable hydraulic profile that considerably improved this situation. The proposed condition model includes only one other improvement.

- The existing 24” x 48” ECMP will be replaced by a 36” RCP under 78th Street located north of Eagle Palm Drive of the Parkway Business Center entrance.

This project is no longer recommended, as it has negligible impact on water surface elevation upstream of the culvert and the updated 25-year LOS under existing conditions is now “A”.

7.3.4.2.2 78th Street Ditch (model # 2803xxx series)

A pond was proposed along the 78th Street ditch system and has been constructed by the County as CIP #41086. It is been included as an “Existing Condition” in this 2007 model update.

7.4 Project Reprioritization

Flood control projects are reprioritized are prioritized as follows:

Priority 1: Delaney Creek Lateral “B” and Lateral “C” channel cross-section improvements and culverts upgrade as discussed in sections 7.3.1.2.2 and 7.3.1.2.3.

Priority 2: Delaney Creek Main Channel U.S. 41 Bridge Upgrade.

Priority 3: Delaney Creek Lateral “A” channel cross-section improvements as discussed in section 7.3.1.2.1.

Priority 4: Local drainage improvements, such as pumps, are suggested for closed systems following:

- Delaney Creek
  - Lateral “A”: 211165
  - Lateral “E-1”: 215543, 215544, and 215545
  - Hendrics Lake System: 220220, 225110, and 225120.
  - Hickory Hammock System: 230197, 230200, 233000, and 233015
  - Isolated System: 227020, 227027, 227030, 227035, 227080, 227084

- Delaney Pop-off Canal
  - 200xxx series: 200025
  - Tributary “B”: 247070
- 248xxx series: 248045

- Archie Creek
  - Tributary “F”: 290311

Priority 5: Alternatives evaluation is necessary for the following subbasins:

- Delaney Creek
  - Lateral “A-1”: 211510 and 211530
  - Lateral “D”: 214010
  - Lateral “E-1”: 215532
  - Hendrics Lake System: 223060, 225016, and 225150
  - Lumsden Road North Ditch: 224050

- Delaney Pop-off Canal
  - 200xxx series: 200095
  - Tributary “B”: 247060
  - Evergreen Estates System: 252050

- North Archie Creek
  - Tributary “A”: 261060
  - Tributary “B”: 262000, 262020, and 262030

- Archie Creek
  - Main Channel: 290045 and 290055
  - Tributary “F”: 290612
APPENDIX A: TABLES
APPENDIX B: FIGURES
APPENDIX C: MODEL FILES

(See attached DVD)
APPENDIX D: GIS FILES

(See attached DVD)