SECTION 2.0

WATERSHED ASSESSMENT AND PROBLEM IDENTIFICATION

This section of the Watershed Management Plan describes the existing flooding, natural resources, and water quality conditions within the Stevenson Creek Watershed, and the watershed assessment methods used. The first portion of the section addresses the primary concern of those living within the watershed; the periodic flooding of roads and buildings. The second portion describes the natural resources available within the watershed, such as the location and quality of wildlife habitat. The third portion details the water quality assessment, including the results of the water quality sampling program and pollutant loading modeling.

2.1 FLOODING CONDITIONS ASSESSMENT

2.1.1 Watershed Model Selection

This section details the methods and procedures used to perform the assessments of watershed hydrologic and hydraulic conditions. An important component of this Watershed Management Plan for Stevenson Creek is an assessment of the current flood protection level of service provided throughout the watershed. A thorough understanding of basinwide hydrologic and hydraulic processes is necessary to determine the most effective means of alleviating the identified flooding problems. The watershed model prepared for this Management Plan was used as a planning tool to assess the existing flooding problems and susequently, to optimize the flood protection benefits of the proposed improvements.

The Advanced Interconnected Channel and Pond Routing Model, Version 2.2 (AdICPR) was chosen for the hydrologic and hydraulic modeling analysis, in part because of its ability to mathematically represent the time-dependent processes that govern flow and stage in low-relief coastal watersheds such as Stevenson Creek. Furthermore, it was necessary to select a model which has been accepted by the Federal Emergency Management Agency (FEMA) for flood insurance studies, since the results of the analysis will be used to support a request for revision of the applicable FEMA flood insurance rate maps.

2.1.2 Hydrologic Model Development

The hydrologic model used for this study is the SCS Runoff Curve Number (CN) and Unit Hydrograph Method contained within AdICPR. This method computes a runoff (flow) versus time relationship (hydrograph) for each subbasin, given a set of hydrologic input parameters.

Hydrologic parameters required for the SCS Runoff CN and Unit Hydrograph Method include the following:

- Subbasin drainage area
- Subbasin time of concentration (TC)
- Subbasin percent directly connected impervious area (DCIA)
- Subbasin weighted runoff curve number (CN) for areas outside of DCIA
- Unit hydrograph shape factor
- Rainfall data

A description of the methodologies used to derive each of these hydrologic parameters follows in the remainder of this section.

2.1.2.1 Subbasin Delineations

The Stevenson Creek watershed drainage area consists of approximately 9.82 square miles (6,286 acres) of moderately sloping terrain in west central Pinellas County that discharges to Clearwater Harbor. To provide the level of detail that was deemed necessary to analyze the primary drainage facilities within the heavily urbanized Stevenson Creek watershed, the basin was divided into a total of 307 discrete subbasins that range in size from 1 to 197 acres. The average subbasin size is 20.5 acres. The delineation of individual subbasins was dictated to a large extent by the complexity of the drainage network itself and the need to define the contributing drainage area to modeled elements of the conveyance system.

Approximately 20% and 14% of the watershed area lies within the City of Dunedin and unincorporated Pinellas County, respectively. Although these governmental bodies are not official participants in the Management Plan, approximately the same level of hydrologic and hydraulic model detail was applied in these areas. This was deemed necessary in order to accurately quantify the volumes and peak rates of runoff contributed to Stevenson Creek and its tributaries by these areas.

Figure 2.1-1 in the attached map pocket shows the delineation of the Stevenson Creek watershed into its 307 individual subbasins. Subbasin ID numbers consist of the letter “B” followed by a four-digit number. The four-digit number generally coincides with the ID number of the node at the outlet of the subbasin. In cases where two subbasins drain to the same node, the second subbasin has an ID number equal to the node number plus one. Nodes and subbasins were generally numbered in order increasing in the upstream direction. For convenience, the 307 individual subbasins were aggregated into eight major subbasins, or “subwatersheds”. Subbasin and node numbers are grouped according to the subwatershed in which they lie, as follows:

Table 2.1-1 Node / Subbasin Numbering Scheme

Subwatershed	Node / Subbasin ID Range
Lower Stevenson Creek	0000 – 0499
Lower Spring Branch	0500 – 0999
Upper Spring Branch	1000 – 1999
Hammond Branch	2000 – 2999
Middle Stevenson Creek	3000 – 3499
Lake Belleview Branch	3500 – 3999
Upper Stevenson Creek	4000 – 4139
Jeffords Street Branch	4140 – 4999

The means of defining subbasin boundaries employed a number of sources of information and methods. The principle source was 1”=200’ topographic aerial photographic mapping of the watershed provided by SWFWMD. This mapping shows overland topography, thus indicating direction of overland flow. However, because much of the watershed is developed, with a variety of residential, commercial, industrial, and institutional developments, man-made secondary drainage systems comprising swales, gutters, storm sewer systems, ditches, and detention ponds have interrupted the natural overland flow patterns within the basin and, in many cases, diverted storm runoff in directions that are not readily apparent from inspection of the topographic mapping. It is also important to note that the SWFWMD mapping within the basin is, for the most part, over 20 years old. Therefore, new development that has occurred within the watershed since that time does not appear on these maps, and drainage patterns and grades in certain areas have been altered since that time due to this new development.

The City of Clearwater drainage atlas maps provided detailed layouts of the secondary drainage systems within the incorporated city portions of the watershed and proved to be of immense value for assisting in the delineation of the subbasins. In addition, record drawings of permitted stormwater management facilities within the basin on file with the SWFWMD were reviewed and referenced in the subbasin delineation process as a means of defining the drainage systems and drainage areas for newer developments that did not show up on any of the previously listed information sources.

The final check of subbasin delineations was field reconnaissance of the watershed to confirm the initial boundaries and to inspect the drainage facilities and conditions in the basin firsthand, to investigate areas where there was no information available from the previously listed sources, and to resolve discrepancies where there was a conflict between different sources of information regarding drainage facilities.

2.1.2.2 Land Use

Existing land use conditions within the Stevenson Creek watershed were defined with the aid of a Geographic Information System (GIS) land use coverage data file provided by SWFWMD. For convenience, the 26 different land use classifications that were defined in the SWFWMD land use coverage of the basin were consolidated into the following 13 categories:

- Commercial
- Cropland and Pastureland
- Forest
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Industrial
- Institutional
- Open Land and Range Land
- Specialty Farms
- Transportation, Communications and Utilities
- Water
- Wetlands

For the purposes of this study, the medium density residential classification was redefined to include residential areas containing between 2 and 5 dwelling units per acre. In the SWFWMD classification system, this was considered to be high density. It was deemed necessary to reclassify these areas, because the actual (measured) impervious area percentages in these areas are more consistent with literature values cited for medium density residential. The significance and determination of impervious area is described in the next section. This change increased the percentage of the basin classified as medium density residential from none to the majority of the basin. Areas of multi-family developments (i.e., apartment and condominium complexes) remained classified as high density residential.

These land use coverages were then field-truthed by examination of recent aerial photography of the basin and comparing to the map to confirm that both the classifications and the areal extent are properly represented. Many adjustments were required. The majority involved the delineation of the developed portions of areas that were designated as institutional or transportation/utilities in the SWFWMD land use coverage to separate them from open land areas that occurred within these parcels to various degrees. The institutional land use classification includes parcels such as nursing homes, schools, churches, hospitals, auditoriums, and cemeteries, which can vary over a wide range in their degree of imperviousness. For the same reasons, significant open land areas within industrial and commercial tracts were delineated to segregate them from the actual developed portions of the parcels. Other changes were required in order to update the mapping to account for recent development. The resultant hydrologic land use classification map of the Stevenson Creek watershed that was used for model development purposes is shown in Figure 2.1-2. Table 2.1-2 presents a composite breakdown of land use acreages and percentages in the Stevenson Creek Watershed.

Table 2.1-2 Basin Land Use Percentages

Land Use Classification	Total Area (acres)	Percentage of Basin (%)
Commercial	610	9.7
Cropland and Pastureland	5	0.1
Forest	50	0.8
Low Density Residential	56	0.9
Medium Density Residential	3861	61.4
High Density Residential	459	7.3
Industrial	42	0.7
Institutional	182	2.9
Open Land and Range Land	573	9.1
Specialty Farms	10	0.2
Transportation, Communications, and Utilities	170	2.7
Water	205	3.3
Wetlands	63	1.0
Total	**6286**	**100.0**

(Figure 2.1-2)

2.1.2.3 Impervious Areas

Impervious areas are those surfaces such as rooftops and pavement, which impede the infiltration of runoff into the soil. Directly connected impervious area (DCIA) is defined as those impervious areas that are connected directly to the subbasin outfall (i.e., node) without flow occurring over a pervious surface. In the AdICPR model, DCIA is an optional parameter which, if used, is treated as a separate subarea within each subbasin. A separate hydrograph is computed for the DCIA, which is not subject to any initial abstraction (i.e., infiltration). Any impervious area which is not directly connected to the subbasin outfall is then used in computing a weighted curve number (CN) as described in the next section.

For the Stevenson Creek watershed model, literature values of total percent imperviousness for the different land use classifications were utilized. Measurements of impervious coverage within representative samples of each land use classification were conducted using aerial photography, in order to confirm the literature values. The split between DCIA and non-directly connected impervious areas was determined through the model calibration process, as discussed in subsequent sections.

2.1.2.4 Runoff Curve Number

By superposition, the hydrograph computed using the DCIA is added by AdICPR to the hydrograph computed using a weighted runoff curve number (CN). This results in a single composite hydrograph, which is then input into the corresponding node of the hydraulic model.

In the SCS method, the CN is a function of S, the potential maximum infiltration after runoff begins (in inches):

$$CN = \frac{1000}{S+10}$$

The standard SCS method uses the following empirical relationship:

$$I_a = 0.2S$$

Where I_a is the initial abstraction, which includes all losses before runoff begins. It includes water intercepted by vegetation, shallow depression storage, evaporation, and infiltration. For design storm conditions in watersheds containing sandy soils, experience and literature on the subject indicate that infiltration losses comprise the majority of the initial abstraction.

The weighted CN is therefore primarily a function of the non-directly connected impervious coverages (based on land use), and the infiltration capacity of the soils. For this study, a land use/ hydrologic soils group intersection analysis was performed using the ArcView/ArcInfo Geographic Information Systems (GIS) software.

The Soil Conservation Service (SCS) (now the Natural Resources Conservation Service) of the U.S. Department of Agriculture has mapped soils in Pinellas County in its publication “Soil Survey of Pinellas County, Florida” (1972). This soil survey also provides generalized information on the hydrologic properties of the soil classifications.

A standard method of soils classification is the hydrologic soils group. Soils are grouped into four hydrologic soil groups A through D. These groups are commonly used in hydrologic analyses to estimate infiltration rates and soil moisture capacities. Descriptions of these soil groups are:

Hydrologic Soil Group A (low runoff potential): Soils that have high infiltration rates even when thoroughly wetted and a high rate of water transmission. Typical maximum infiltration rate of 10 inches per hour when dry and 0.5 in/hr when saturated.

Hydrologic Soil Group B (moderately low runoff potential): Soils that have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Typical maximum infiltration rate of 8 in/hr when dry and 0.4 in/hr when saturated.

Hydrologic Soil Group C (moderately high runoff potential): Soils that have a slow infiltration rate when thoroughly wetted and a slow rate of water transmission. Typical maximum infiltration rate of 5 in/hr when dry and 0.25 in/hr when saturated.

Hydrologic Soil Group D (high runoff potential): Soils having very slow infiltration rates when thoroughly wetted and a very slow rate of water transmission. Typical infiltration rate of 3 in/hr when dry and 0.10 in/hr when saturated.

In many cases in Florida, dual hydrologic soil group classifications (A/D or B/D) are assigned to soils that, during the wet season, are saturated throughout much of the soil column due to a high surficial water table. Thus, during this time of year, infiltration is impeded and the soil acts as a D soil. However, during the rest of the year, when the water table is lower, the soil acts as an A or B soil.

Figure 2.1-3 presents a composite hydrologic soil group classification map of the Stevenson Creek watershed. It can be seen from the map that soils within the basin are predominantly classified as A and B/D. Group A soils occur on upland sand ridges and group B/D soils occur on low lying areas and flat areas with poor natural drainage around natural waterbodies and wetlands. A composite breakdown of the hydrologic soil group acreage and percentages for the basin is as follows:

Table 2.1-3 Hydrologic Soil Group Acreages and Percentages

Hydrologic Soil Group	Total Area (acres)	Percentage of Basin (%)
A	2424	38.6
B	21	0.3
B/D	2158	34.3
C	1114	17.7
D	370	5.9
Water	199	3.2
Total	**6286**	**100.0**

(Figure 2.1-3)

The following table presents the Runoff Curve Number Computation Matrix used by the ArcView/ArcInfo Geographic Information Systems (GIS) software to compute the overall area-weighted curve number for each subbasin. In this method, a unique curve number is assigned to each possible combination of hydrologic soil group and land use category. The computer program then computes an area-weighted value based on the percentages of these soil/land use combinations found within the particular subbasin.

Table 2.1-4 Runoff Curve Number Computation Matrix

Land Use Classification	Hydrologic Soil Group	Runoff Curve Number
Commercial	A	89
Commercial	B	92
Commercial	C	94
Commercial	D	95
Commercial	B/D	94
Cropland and Pastureland	A	49
Cropland and Pastureland	B	69
Cropland and Pastureland	C	79
Cropland and Pastureland	D	84
Cropland and Pastureland	B/D	79
Forest	A	36
Forest	B	60
Forest	C	73
Forest	D	79
Forest	B/D	73
High Density Residential	A	77
High Density Residential	B	85
High Density Residential	C	90
High Density Residential	D	92
High Density Residential	B/D	90
Industrial	A	81
Industrial	B	88
Industrial	C	91
Industrial	D	93
Industrial	B/D	91
Institutional	A	81
Institutional	B	87
Institutional	C	91

Table 2.1-4 (Continued) Runoff Curve Number Computation Matrix

Land Use Classification	Hydrologic Soil Group	Runoff Curve Number
Institutional	D	93
Institutional	B/D	91
Low Density Residential	A	50
Low Density Residential	B	65
Low Density Residential	C	76
Low Density Residential	D	87
Low Density Residential	B/D	76
Medium Density Residential	A	61
Medium Density Residential	B	75
Medium Density Residential	C	83
Medium Density Residential	D	86
Medium Density Residential	B/D	83
Open Land and Rangeland	A	39
Open Land and Rangeland	B	61
Open Land and Rangeland	C	74
Open Land and Rangeland	D	80
Open Land and Rangeland	B/D	74
Specialty Farms	A	59
Specialty Farms	B	74
Specialty Farms	C	82
Specialty Farms	D	86
Specialty Farms	B/D	82
Transportation, Communications, and Utilities	A	78
Transportation, Communications, and Utilities	B	83
Transportation, Communications, and Utilities	C	88
Transportation, Communications, and Utilities	D	90
Transportation, Communications, and Utilities	B/D	88
Water	W	100
Wetland	A-D	98
Wetland	W	100

Overall CN values were calculated for each subbasin using the above matrix and GIS intersection analysis. In order to derive the percent impervious area for each subbasin, a "pervious area" CN was created for each subbasin based only on its soil type and its pervious area land use. For example, medium density residential was shown as open land and range land to represent the lawns and open space. By comparing the overall CN to the pervious area CN, a total percent impervious area for each subbasin was back-calculated using using an excel spreadsheet. Then, based on the percent of total impervious as DCIA (derived through the calibration process), the curve numbers were recomputed using only the unconnected impervious area. The excel spreadsheet used for this analysis is included in Appendix A.

2.1.2.5 Time of Concentration

The time of concentration (TC) is a measure of the time scale of the runoff hydrograph and is a function of subbasin slope and surface roughness. Conceptually, it is the time required for the hydrologically most distant point in the subbasin to begin contributing flow to the subbasin outlet, following the commencement of rainfall. The time of concentration for each subbasin within the Stevenson Creek watershed was computed using the Kinematic Wave approach as outlined in SCS Technical Release 55. In this method, the TC is computed as the sum of travel times for sheet flow, shallow concentrated flow, and open channel flow. The reader is referred to chapter 3 of SCS Technical Release 55 in Appendix E for further details on this method.

2.1.3 Hydraulic Model Development

The hydraulic model used for this study was AdICPR, which contains a one-dimensional unsteady flow hydraulic routing model. This model uses a node-reach representation of the drainage system. AdICPR receives hydrograph input at specific nodes by file transfer from the hydrologic model. The model performs dynamic routing of stormwater flows through the defined storm drainage system to the points of outfall in the receiving waterbody. The program will simulate branched or looped networks; backwater due to tidal or non-tidal conditions; free-surface flow; pressure flow or surcharge; flow reversals; flow transfer by weirs, orifices, and pumping facilities; and storage. Types of reaches that can be simulated include pipes, weirs, open channels of regular or irregular cross section, bridges, and drop structures (weir and pipe in series).

Simulation output takes the form of water surface elevations and discharges at each node and reach within the model network, reported at user-specified time intervals.

2.1.3.1 Drainage Facility Inventory

The initial and most important step in the development of the hydraulic model of the Stevenson Creek watershed was the inventory of the drainage structures along the primary drainage system. This information provides the foundation for the model representation of the hydraulic system. The drainage facility inventory of the study area was compiled from an array of sources and methods. Hydraulic data for culverts, storm sewers, bridges, control structures, and watercourse cross sections were obtained from city drainage atlas mapping, development plans, roadway plans, previous studies, and field surveys. Data collected included elevations, lengths, dimensions, construction materials, channel vegetation, structure entrance and exit conditions, and other pertinent features. The following is a discussion of the sources and methods used to collect this information:

Existing Maps, Plans and As-Builts

The City of Clearwater maintains 1”=100’ scale drainage atlas maps that include considerable detail of locations, dimensions, and invert elevations of drainage facilities located within the incorporated city limits. These maps are periodically updated as new development occurs or new information is obtained, and provide a great deal of detail within the areas they cover, including secondary storm sewer systems. Invert elevations are not always indicated, however, and there is no information shown for the unincorporated areas within the basin. The City of Dunedin storm atlas provides a similar degree of mapping of drainage facilities.

It was previously noted that the SWFWMD stormwater management facility permit files were researched to collect information on facilities constructed within the Stevenson Creek watershed since the inception of the permit regulations in 1984. The City of Clearwater, Pinellas County, and the City of Dunedin also provided record drawings of drainage and flood control works that have been constructed within the basin. In addition, the Stevenson Creek Sediment Evaluation with Cross Sections completed by the City of Clearwater in August 1999 provided many of the irregular cross sections used in the model. The following list contains the most noteworthy plans, maps and as-builts used as sources of existing hydraulic model input data used in the development of the Stevenson Creek hydraulic model:

- Pinellas-Anclote Basin Aerial Photography with Contours. March 1977, May 1979, and December 1985, Southwest Florida Water Management District.
- City of Clearwater Stormwater Atlas. 1996-1999, City of Clearwater Public Works Administration, Engineering.
- City of Dunedin Stormwater Atlas. 1999, City of Dunedin Engineering Section
- Stevenson Creek Sediment Evaluation with Cross Sections. August 1999, City of Clearwater Public Works Administration, Engineering.
- Spring Branch of Stevenson's Creek Drainage Basin Field Survey Notes. March 1979, Pinellas County Engineering Department.
- Stevenson's Creek Watershed Channel Improvements, Phase 1 As-Built Plans. February 1993, W.K. Daughtery Consulting Engineers, Inc.
- Stevenson Creek – Channel Improvements – Phase II Plans. March 1995, Camp, Dresser and McKee.
- Highland Avenue Stormwater Improvements As-Built Plans. November 1998, HDR Engineering, Inc.
- The Mall From Union Street to the South Terminus Record Drawings. July 1994, Seminole Engineering.
- 3rd Avenue Drainage Improvements Record Drawings. March 2000, City of Dunedin Department of Public Works and Utilities.
- Villas at Renaissance Square (Old Sunshine Mall) Construction Plans. January 1999. Florida Design Consultants, Inc.
- Various MSSW and ERP permit information on file with the Southwest Florida Water Management District.

<u>*Previous Studies*</u>

The Stevenson Creek watershed has been the subject of several stormwater management studies in the last 20 years that served as valuable sources of information on the existing structures and channels along the primary drainage system of the basin. These sources were used to varying degrees to supplement and/or confirm other sources to provide the required AdICPR model input data to describe the primary drainage system and facilities in the watershed.

The first of these studies was a storm drainage basin study conducted in 1981 by Henningson, Durham, and Richardson, Inc. (HDR) for Pinellas County. This study was a part of the countywide comprehensive stormwater drainage master plan, which has since been adopted within the Pinellas County Comprehensive Plan in its Drainage Element (1989). To conduct its study of the Stevenson Creek basin, HDR used the SCS TR-20 computer model for hydrologic analysis. Runoff hydrographs were developed for each subbasin in the watershed, using existing and comprehensive future land use, with the 25-year, 6-hour duration storm event of 5.5 inches total precipitation used as the basis for design. The U.S. Army Corps of Engineers HEC-2 model was used to perform backwater analyses to determine flood profiles for each of the design storm events. The Pinellas County Study also served as the basis for a Master Drainage Plan for the Spring Branch Watershed, prepared in 1983 by the City of Dunedin Public Works Department. For this study, Parsons ES used selected information from the original Pinellas County field survey notes as needed to augment the structure and cross section information within the City of Dunedin portion of Spring Branch.

In 1988, a Preliminary Master Plan for Stevenson Creek was prepared by W.K. Daughtery, Inc. for the City of Clearwater. In the 1988 study, a three-phase channel improvement project was recommended for the main channel of Stevenson Creek. The modeling methodology employed TR-20 to generate runoff hydrographs for the 5-, 10-, 25-, 50-, and 100-year, 24 hour duration storm events. The hydrographs were then input to a one-dimensional, steady-flow water surface profile model (WSP-2) to calculate the flood elevations. In 1990-1991, Camp, Dresser, and McKee, Inc. (CDM) prepared a limited re-study of Stevenson Creek under which the TR-20/WSP-2 model was converted to AdICPR, Version 1.4. The AdICPR model was then used to support the detailed design of portions of the preliminary master plan project, as well as a FEMA Letter of Map Revision (LOMR). This re-study by CDM also included analysis of alternatives for floodplain enhancements within the Glen Oaks Golf Course. To date, two of the three phases of the original project have been constructed, consisting of the channel segments between Betty Lane and Jeffords Streets. The previously referenced as-built plans for these channel improvements were relied upon extensively in the development of the model.

The following list summarizes the previous and on-going studies of the Stevenson Creek Watershed which were consulted in the development of this watershed management plan:

- Summary Report for Stevenson's Creek Preliminary Master Plan. August, 1988, W.K. Daughtery Consulting Engineers, Inc.
- City of Clearwater 1997 Watershed Management Plan, Volume II. Post, Buckley, Schuh & Jernigan
- Stevenson Creek Channel Improvements, Phase 2 – Joint Dredge and Fill Permit Application. April 1992, Camp, Dresser, and McKee
- Technical information submitted to FEMA in support of request for Stevenson Creek Letter of Map Revision. May 1998, Camp, Dresser and McKee.
- Pinellas County Storm Drainage Basin Study Technical Appendix – Spring Branch of Stevenson Creek Basin and Coastal Zone 4 Basin. October 1981, Henningson, Durham & Richardson, Inc.
- Repetitive Loss Report – City of Clearwater, FL. October 1998, FEMA
- Master Drainage Plan: Spring Branch Watershed. September, 1983, City of Dunedin Public Works Department, Water Resources.
- Stevenson Creek Sediment Characterization and Removal Feasibility Study - Final Report. August 1998, BCI, Inc.

- Stevenson Creek Estuary – Preliminary Restoration Plan. October 2000, U.S. Army Corps of Engineers, Jacksonville District

Field Reconnaissance and Field Survey

Field reconnaissance of the Stevenson Creek watershed was conducted by driving and walking the basin to determine drainage patterns, map drainage facilities, measure facility dimensions inspect the condition of the drainage facilities in the field, assemble a photographic log of drainage facilities, and to confirm information collected from the previously discussed sources and/or resolve differences between different sources.

Following completion of the field reconnaissance, Parsons ES developed a field survey plan to establish the size, location, dimensions and inverts of drainage structures, and to define current stream channel and floodplain cross sections within the Stevenson Creek watershed based on the specific needs of the hydraulic model. The emphasis of this survey was placed on areas within the basin where there were no available sources of as-built record drawings and surveys. The survey program was conducted by Harry Marlow, Inc., under the direction of Parsons ES. In addition, the City of Dunedin conducted limited surveys of selected structures and cross sections for use in the portion of the model that lies within the limits of the City of Dunedin.

2.1.3.2 Channel Cross Section and Floodplain Definition

A variety of sources of channel cross section and drainage structure data were utilized in the formulation of the AdICPR model. Parsons ES used what was judged to be the most current, detailed, and representative source of information available for any particular reach of the open channel system. In addition, the extension of channel cross-sections into the floodplain regions of the Stevenson Creek floodway was necessary for many of the surveyed cross-sections obtained from the various sources. In these instances, the SWFWMD one-foot contour topographic maps were used to scale off the floodplain extensions.

One of the most important requirements of the hydraulic model representation of open channel reaches is the selection of an appropriate channel roughness coefficient, or Manning's n, value. This is an index of the resistance to flow, primarily due to friction, that is dependent upon factors such as the extent and type of channel vegetation, channel bottom material, channel irregularity, channel alignment, obstructions, and depth of flow. Selection of an appropriate Manning's n value for a particular channel reach is a subjective procedure that is facilitated by past experience. As a first step in this procedure, Parsons ES personnel took color photographs of all open channel reaches within the model. Published guides and past modeling experience were then applied in the selection of channel and floodplain Manning's n values. These values were either confirmed or adjusted during the model calibration process using the rating curves of the streamflow gaging stations and the measured high water elevations.

2.1.3.3 Storage Node Stage-Area Relationships

In order to represent the attenuating effects of storage on the hydrographs computed by the hydrologic model, it is important that all significant stormwater storage areas and their hydraulic controls features be well defined within the model. This is especially important in the Stevenson Creek watershed where much of the drainage throughout the basin is controlled by lakes (man-made and natural) and natural depression areas. The AdICPR model allows the user to specify a variable stage-area relationship at any model node that defines the storage properties at that point, be it a pond, lake, wetland, or other water body.

In the development of the Stevenson Creek watershed hydraulic model, Parsons ES used, where available, record drawings and permits as a means of establishing stage-area relationships of constructed stormwater management facilities. For all others, including the natural lakes, ponds, and wetlands within the watershed, the stage-area relationships were determined by direct measurement of SWFWMD one-foot topographic maps with a planimeter.

2.1.3.4 Model Schematic

The hydraulic model (AdICPR) of the Stevenson Creek watershed consists of a network of open channel segments, culverts, bridges, storm sewers, weirs, lakes, ponds, and wetlands that compose the primary drainage system within the basin. AdICPR uses a link-node concept to idealize the "real world" drainage system. A node is a discrete location in the drainage system where conservation of mass (continuity) is maintained. Links, or "reaches" are the connections between nodes and are used to convey water through the system. The entire network of nodes and reaches forms the hydraulic model network and serves as the computational framework for AdICPR.

The first step in development of a model schematic was to identify the primary drainage system and all drainage facilities within it. This task was accomplished through the research and review of all available sources of information that have been previously described, and through field reconnaissance of the entire watershed. All such information was compiled and a watershed drainage map developed which depicts the primary drainage system. Figure 2.1-1, included in the attached map pocket, presents the drainage map of the Stevenson Creek watershed and the model node locations which were used to define the primary conveyance network of the basin. General guidelines which were followed in development of the model schematic are:

- Nodes are required at the upstream and downstream ends of any structure (e.g. culvert, bridge, weir, etc.).
- Storage elements (ponds, lakes, wetlands, etc.) are specified as nodes.
- Nodes are located at tributary confluence locations.
- Nodes are placed at locations where there is a major surface water inflow to the conveyance system.
- Points of change in channel geometry and/or slope are specified as node locations.

- Nodes are added at locations within the network to define the hydraulic gradeline in greater detail.

When constructing a model network, unique identifiers must be assigned to all nodes and reaches. The node-numbering scheme is the same as the subbasin numbering scheme previously described, except that node identifiers begin with the letter "N". By convention, the identification of a reach between two nodes is the same as the name of the upstream node. Reach identifiers begin with a letter prefix pertaining to the type of reach as follows:

Reach Type	**Prefix**
Channel	C
Pipe	P
Weir	W
Drop Structure	DS
Bridge	BR
Overland Flow Channel	COF
Overland Flow Weir	WOF

A node-reach schematic of the AdICPR model of the Stevenson Creek watershed is provided in Figure 2.1-4, contained within the attached map pocket. This schematic represents the hydraulic network as it is modeled. Storage nodes and type of reaches are indicated as they are represented within the AdICPR model. The schematic includes "overflow" reaches which model the overland flow paths that floodwaters can follow when flow rates exceed the capacities of the primary drainage system. In all, the model includes 383 nodes, 252 pipe reaches, 176 open channel reaches, 171 irregular cross sections, 98 weir reaches, 39 drop structure reaches, and 6 bridge reaches.

2.1.3.5 Boundary Conditions

The AdICPR model requires specification of hydraulic boundary conditions at all outfall points of the model schematic. In the Stevenson Creek watershed application, the primary basin outlet is located at the downstream face of the most downstream bridge at Edgewater Drive (Alt. US 19). A constant tailwater boundary condition representing the water elevation of Clearwater Harbor, a tidal waterbody, was specified at elevation 2.5 feet above mean sea level, 1929 National Geodetic Vertical Datum (NGVD) for all model simulations. This elevation was intended to approximate a "spring tide" condition within the harbor. It should be noted that for flow rates above approximately 3,000 cfs (which applies to the peak rates of flow for all design storms), the existing conditions stages within the Stevenson Creek estuary were not found to be sensitive to changes in the tidal boundary condition over a range of 0 to 3 feet NGVD.

An "overflow" boundary node exists at the northern end of the Jeffords Street Branch where floodwaters can "pop off" to the Allens Creek Basin when the flows exceed the capacity of the storm sewers. A third boundary node exists at the Clearwater Executive Airport, where a 36" diameter CMP culvert under the runways diverts a portion of the flow from the headwaters of Hammond Branch east into the Alligator Creek Basin. The tailwater elevations at these overflow boundary nodes are allowed to fluctuate with normal depth in the boundary link. Modeling results from the completed Allens Creek and Alligator Creek Watershed Plans were consulted to determine whether flooding was predicted in those watersheds which would influence stages in the corresponding Stevenson Creek boundary nodes. These model results indicated that the design flood elevations are well below the Stevenson Creek "pop-off" elevations, therefore it was not necessary to code in the time/stage boundary conditions based on these other models.

2.1.4 Model Calibration and Verification

2.1.4.1 Introduction

Model calibration refers to the adjustment of model parameters within reasonable limitations so that the model results (i.e., streamflows and water elevations) are in reasonable agreement with a set of measured data. A reasonable range of values for the adjustment of model parameters is established through the review of literature references, and adjustments outside of those ranges are made only if some unusual hydrologic or hydraulic condition exists. Ideally, the model is calibrated to more than one different storm event in order to represent a variety of volumes, intensities, and distributions. It is also desirable to calibrate to recorded flow and stage information at different locations within the watershed.

The two primary data requirements for model calibration and verification are gaged rainfall and streamflow for the study area. When selecting a calibration storm event, the rainfall and streamflow data must be sufficiently documented in appropriate time intervals so that variations in rainfall intensity and the associated runoff can be accurately simulated. Data should be recently acquired so that the current land use and hydraulic conditions existing in the study area are accurately represented. Additionally, because of the non-uniform spatial distribution inherent in Florida rainfall patterns, it is desirable that precipitation data be collected at more than one location within a large study area.

2.1.4.2 Streamflow and Precipitation Gaging Data

Prior to the initiation of this watershed management plan, no known measured streamflow data existed for Stevenson Creek. Therefore, it was necessary to collect streamflow data for calibration of the hydrologic/hydraulic computer model. This was accomplished by establishing two streamflow gaging stations within the Watershed. Continuous rainfall, stage, and streamflow data were collected under the direction of Parsons ES by Hydrogage, Inc. at Stevenson Creek near Drew Street and at Spring Branch near King’s Highway. The locations of the stations are shown on Figure 2.1.5.

(Figure 2.1-5)

The streamflow data was collected in digital format from May 12, 2000 to October 12, 2000. Stream stage (water level) was recorded at 15-minute intervals using a calibrated pressure transducer. Rainfall amounts were recorded at 15-minute intervals using a standard tipping-bucket rain gage. The rainfall data recorder utilized a telemetry system to alert Hydrogage of storm events in progress. Streamflow measurements were made over a wide range of flow conditions using standard USGS stream gaging procedures. These measurements were then correlated to recorded stage elevations to establish a flow versus stage rating curve at each location.

In addition to the precipitation and streamflow data, storm event flow-weighted composite water samples were collected with the aid of automatic samplers at each station. The water quality monitoring program is described in detail in Section 2.3.2.

2.1.4.3 Selection of Calibration/Verification Storm Events

July 15, 2000

Due to the relatively short period of record, only a handful of storm events was available for calibration of the model. Fortunately for this analysis, the period of record captured a large storm that occurred on July 15, 2000. This storm produced 7.60” and 5.21” of total rainfall within a ten-hour period at the Drew Street and King’s Highway sites, respectively. As evident from Figure 2.1-6, this storm consisted of two separate rainfall events occurring approximately 6 to 7 hours apart. The first event occurred from about 7:00 AM to 10:00 AM and produced 3.23” at the Drew Street site and 1.69” at the King’s Highway site. At 2:00 PM, the second event occurred, producing 4.37” at the Drew Street site and 3.52” at the King’s Highway site. All of the rainfall in the afternoon event fell within a 2-hour period from 2:00 PM to 4:00 PM. The peak 15-minute intensity occurred at about 2:30, averaging 7.44 and 4.04 inches per hour at the Drew Street and King’s Highway sites, respectively. This storm produced widespread flooding throughout Stevenson Creek and the City of Clearwater. Following this event, staff from

(Figure 2.1-6)

Parsons ES and the City marked visible high water elevations at 21 locations throughout the basin, which City survey personnel later surveyed (three of the elevations marked in the field as "questionable" were later discarded).

Because the hydrologic model chosen for the analysis is intended to be used strictly as a single-event model, the larger, afternoon rainfall event was chosen as a calibration event. By inspection of Figure 2.1-6, it is evident that both Stevenson Creek and Spring Branch responded to the two storms as separate and distinct events. It should be noted, however, that the first event served to substantially increase the soil moisture leading into the second event. Prior to the morning of July 15, hydrologic conditions within the watershed were still extremely dry due to the several months of drought during the spring of 2000.

June 12, 2000

In order to compliment and verify the large storm event of July 15, a smaller storm was chosen in order to verify the calibration parameters, and to "fine-tune" the DCIA percentages. For these purposes, the storm of June 12, 2000 was chosen, which produced 0.85 and 0.54 inches of rainfall at the Drew Street and King's Highway stations, respectively. The antecedent moisture conditions preceding this event were extremely dry. The total rainfall amounts were therefore small enough to make the assumption that all runoff due to this storm was contributed by the DCIA, and thus all rainfall in areas outside the DCIA was lost to infiltration.

2.1.4.4 Calibration Parameters and Results

When calibrating a computer model, certain model input parameters are held constant while others are adjusted in attempt to produce reasonable agreement with measured data. Those parameters that are adjusted are referred to as calibration parameters. Since it is desirable to limit the number of calibration parameters to the fewest possible, those parameters that can be easily measured or calculated normally are not adjusted. Calibration parameters are typically those coefficients and or indexes that are not easily determined from known quantities and/or established procedures. The two primary calibration parameters chosen for the Stevenson Creek model are runoff peak rate factor for the hydrologic component, and channel Manning's roughness for the hydraulic component of the model.

When calibrating a model which uses the SCS unit hydrograph and runoff curve number method, it is often necessary to adjust the runoff curve number (CN) in order for the model to match the measured runoff volume. This is primarily due to the fact that the selected calibration storm events may have antecedent moisture conditions which differ significantly from typical design values found in the literature, which are conventionally based on either average, or average wet season, conditions. Runoff volumes can be very sensitive to CN, especially in southwest Florida where the sandy soils and seasonally high water table can result in a wide range of CN values depending on the time of year (i.e, wet season versus dry season). However, no CN adjustment was necessary for the Stevenson Creek calibration since the runoff volumes predicted for the July 15th storm using CN's corresponding to average antecedent moisture conditions matched the observed volumes fairly well. This is due to the likelihood that the first of the two rainfall events on July 15th served to increase the soil moisture conditions from dry conditions to something closer to typical design conditions.

Channel Mannings Roughness

The primary means of calibrating the AdICPR hydraulic model was through adjustment of the Mannings Roughness Coefficient, or Mannings "n". This is an index of the resistance to flow through a channel, primarily due to friction, that is dependent upon factors such as the extent and type of channel vegetation, channel bottom material, channel irregularity, channel alignment, obstructions, and depth of flow. Adjustments to the channel Mannings "n" were made to the cross sections in the vicinity of the streamflow gages until the simulated flow versus stage rating curve provided a reasonable match with streamflow measurements made during a number of different storm events and flow conditions. No adjustments were made to the Manning's roughness in the vicinity of the Spring Branch at King's Highway station however, since within the range of the measurements, the flow appears to be controlled by the box culverts under King's Highway (refer to Figure 2.1-7). The simulated rating curve at this location provided a reasonable fit to the measured data without any adjustment to the Mannings "n" values.

Figure 2.1-7 Box Culverts Downstream of King's Highway Monitoring Station

At the Drew Street gage in Stevenson Creek, it was necessary to increase the in-bank Mannings "n" from the initial estimate of 0.07, to 0.09 for the cross section immediately downstream of the gage in order to match the measured data. In a field visit in March of 2000, this section was observed to be heavily vegetated (refer to Figure 2.1-8), although the degree of vegetation in the channel generally decreased in the sections further downstream through the golf course. Of course, the degree of vegetation varies not only with location, but also with time of year and point in the maintenance cycle.

Figure 2.1-8 Vegetation in Channel Downstream of Drew Street Monitoring Station

Figures 2.1-9 and 2.1-10 illustrate the simulated flow versus stage rating curves at Drew Street and King's Highway, respectively. Note that the dependant variable (flow) is plotted on the X-axis. The measured values are plotted for comparison.

Following calibration of the Peak Rate Factor (described below), the channel Mannings "n" values were adjusted at several other locations in the watershed in order to match the surveyed high water elevations of the July 15th flood event.

Peak Rate Factor

In the SCS Unit Hydrograph Method, a composite unit hydrograph is constructed of several incremental triangular unit hydrographs that adhere to the basic SCS triangular unit hydrograph equation:

$$q_p = \frac{K'AQ}{T_p}$$

(Figure 2.1-9 and 2.1-10)

Where q_p = peak runoff rate in cfs,
A = area in square miles,
Q = rainfall excess depth in inches,
K’ = runoff peak rate factor

The runoff peak rate factor, K’, includes a unit conversion and a hydrograph shape factor which determines the respective fractions of the runoff volume occurring under the rising and falling limbs of the hydrograph. The value of K’ has been shown to vary widely, from 100 in extremely flat basins in South Florida, up to 600 in steep, mountainous terrain. It has been suggested that K’ varies with watershed surface storage and/or average watershed slope. However, since no definitive relationship between subbasin slope and K’ has been proposed, it is often used as a calibration parameter. In the absence of gage data, the SCS recommends using a value of 256 for Florida, and 484 for most of the rest of the country. It can be shown that a K’ of 484 corresponds to 3/8 and 5/8 of the runoff volume occurring on the rising and falling limbs of the hydrograph, respectively. A K’ of 256 corresponds to 1/5 and 4/5 of the runoff volume occurring on the rising and falling limbs, respectively. The lower peak rate factors produce longer recession limbs with a smaller peak discharge rate for the same unit volume of runoff. It should be noted that these empirical values were derived from regional studies of small groups of similar watersheds, and are not necessarily applicable when applied outside their experimental ranges.

For the Stevenson Creek watershed, a value of 256 was used as the initial estimate of K’ for all subbasins, in accordance with the SCS recommendation for Florida. However, this value tended to produce simulated hydrograph shapes that suggested that the K’ value was too low. The K’ was increased to 484 for several of the steeper subbasins throughout the watershed, including several along the main channel of Stevenson creek which had average slopes exceeding 3%. Those subbasins with average slopes of less than about 1% were left with their original K’ of 256. This adjustment produced better agreement with the observed hydrograph shapes. Figures 2.1-11 and 2.1-12 provide a comparison between the simulated and measured hydrographs for the storm of July 15, 2000.

(Figure 2.1-11 and 2.1-12)

Once the model was calibrated to match the observed hydrograph shape, the Mannings “n” was adjusted for the remaining cross sections throughout the watershed, in order to provide a closer match to the surveyed high water elevations. The following table compares the simulated and the measured high water elevations of the flood of July 15, 2000:

Table 2.1-5 Comparison of Measured and Simulated High Water Elevations

Storm of July 15, 2000

Node ID	Node Location	Peak Stage (ft NGVD '29)		Difference (ft)
		Measured	Simulated	
1000	Spring Branch at King’s Highway gage	12.51	12.48	-0.03
1040	Spring Branch at Union Street (u/s)	16.45	16.36	-0.09
2030	Hammond Branch Pond at King’s Highway	13.16	13.47	0.31
2130	Flagler Ditch at Saturn Avenue	61.10	61.11	0.01
2300	Smallwood Circle and Rosewood Road	45.98	46.08	0.10
3000	Stevenson Creek at Drew Street gage	14.12	14.14	0.02
3040	Stevenson Creek at Cleveland Street (u/s)	18.04	17.81	-0.23
3090	Stevenson Creek at Court Street (d/s)	18.61	18.83	0.22
3100	Stevenson Creek at Court Street (u/s)	18.97	18.88	-0.09
3120	Stevenson Creek at Druid Road (d/s)	20.76	20.60	-0.16
3125	Stevenson Creek at Druid Road (u/s)	20.67	20.82	0.15
3700	Lake Belleview outfall structure at Lakeview	40.22	40.13	-0.09
4020	Stevenson Creek at Browning Street (u/s)	30.05	30.21	0.16
4040	Stevenson Creek at Lakeview Road (u/s)	31.16	31.14	-0.02
4080	Stevenson Creek at St. Thomas Drive (u/s)	39.53	39.52	-0.01
4090	Stevenson Creek at Belleair Road (d/s)	39.80	39.58	-0.22
4100	Stevenson Creek at Belleair Road (u/s)	40.08	40.12	0.04
4140	Jeffords Street Branch at Highland Avenue	28.73	28.73	0.00
4160	Jeffords St Branch at Skyview and Jeffords	31.14	30.70	-0.44
4190	Jeffords Street Branch at Barry and Tuscola	30.80	30.67	-0.13

All of the simulated high water elevations were within 0.50 feet of the measured values, which is within the accuracy of computation for this type of analysis, and exceeds FEMA's minimum requirements for flood insurance studies. All except two of the values were with 0.25 feet of the measurements.

Model Verification and DCIA

The storm of June 12, 2000 was simulated in order to verify the calibration parameters and to fine-tune estimates of DCIA as a percentage of total impervious area. As mentioned in previous sections, total impervious area within each subbasin was estimated based on land use and aerial photography. A 50/50 split between directly and non-directly connected impervious areas was initially assumed. Since runoff volumes resulting from high intensity, large volume storm events normally are not sensitive to the percentage of total impervious as DCIA, a small, less intense storm event was chosen for DCIA determination. Since the storm of June 12 was a low volume, low intensity storm that had extremely dry antecedent moisture conditions, all runoff from this storm can reasonably be attributed to DCIA. The initial model simulation slightly under predicted the volume from this storm. The percentage of total impervious as DCIA was then raised from 50% to 65%, which resulted in a better fit with the measured hydrographs (refer to Figures 2.1-13 and 2.1-14). The 15% impervious area that was transferred into DCIA was subtracted from the calculations of weighted CN, resulting in slightly lower CN values. Then, the calibration storm of July 15 was simulated with the new CN and DCIA values. As expected, the results from the larger storm event did not change significantly.

2.1.5 Flooding Conditions Assessment

Upon the successful completion of the development and calibration of the detailed hydrologic and hydraulic model, the next step of the watershed management planning process was to apply the model to assess the performance of the basin drainage network for a given set of design storm events. Results of these simulations were then analyzed with respect to the flooding level of service criteria, in order to identify the locations and the severity of potential flooding problems.

(Figure 2.1-13 and 2.1-14)

2.1.5.1 Design Storm Events

The basis for the evaluation of flooding conditions in the watershed were the 10, 25, 50, and 100-year return period, 24-hour duration design storm events. The total rainfall for these events are 7.5, 9.0, 10.5, and 12.0 inches, respectively, as determined from the SWFWMD Environmental Resource Permitting (ERP) Information Manual (February 1996). The 500-year, 24-hour storm was also simulated in order to be consistent with FEMA mapping guidelines. The rainfall of 14.8 inches for the 500-year storm was determined through a probability analysis of the more frequent design storm events. The SCS Type II Florida-Modified rainfall distribution was used to develop the design storm hyetographs for the all return periods.

2.1.5.2 Existing Flooding Conditions

Using the described design storm events as the basis for simulations, the AdICPR model was run to generate predictions of basinwide flooding conditions for the existing conditions throughout the Stevenson Creek watershed. The results of these model simulations are summarized in Appendix D, Table D.1. This table lists the model nodes in sequence starting from the downstream end and the corresponding model predictions of 10-, 25-, and 100-year flood elevation at those locations. Also listed are the locations and descriptions of the adjacent roadways and the low road elevation at each structure. Note that the low road elevation does not necessarily occur at the exact location of the structure itself, since sags along the road profile, as determined from SWFWMD topographic mapping and/or roadway construction drawings, may occur several hundred feet away in certain instances. When a peak flood elevation exceeds the low road elevation, it indicates that the roadway will be inundated at that location by a depth corresponding to the difference in elevations.

Comparison with Previous Studies

The following table compares the simulated 100-year flood elevations and peak discharge rates at five key locations within Stevenson Creek to the previous analyses of the watershed:

Table 2.1-6 Comparison of 100-year Peak Discharge Rates and Flood Stages

Location	W.K. Daugherty 1988 Preliminary Master Plan, Future Conditions		CDM 1997 FEMA Submittal		Parsons ES Existing Conditions	
	100-yr Flow (cfs)	100-yr Stage (ft, NGVD)	100-yr Flow (cfs)	100-yr Stage (ft, NGVD)	100-yr Flow (cfs)	100-yr Stage (ft, NGVD)
Douglass Avenue	5,859	4.6	n/a	n/a	4,697	7.73
Palmetto Street	4,603	11.0	2,190	8.67	3,300	10.14
Drew Street Gage	3,718	15.5	2,030	12.75	2,848	15.70
S. of Court Street	3,075	22.8	1,280	19.58	2,588	22.89
Jeffords Street	2,574	28.0	1,724	25.00	1,998	28.24

Because the modeling completed for this study was calibrated to measured stage and flow data, and the level of detail in the model network is an order of magnitude higher than the modeling completed for the previous studies, differences in the results are to be expected. Comparison of the peak discharge rates shows that those predicted by the Watershed Management Plan model consistently fall between those predicted by the other two studies. Upstream of Palmetto Street, the flood elevations predicted by the Parsons ES model generally compare closely with the W.K. Daugherty future conditions model, which includes the phase 1, 2, and 3 improvements (of which only phases 1 and 2 were constructed). In these same locations, the flood elevations predicted both in this study and in the W. K. Daugherty study are approximately three feet higher than those reported in the 1997 CDM study. While the reason for this large difference was not readily discernable, it should be mentioned that the 100-year project conditions flood levels reported in the 1997 CDM FEMA submittal are 0.5’ to 2.5’ lower than the 25-year project conditions flood levels reported in the 1992 CDM dredge and fill permit application (not shown). In addition, when comparing flood elevations at Douglass Avenue and Palmetto Street, it should be noted that neither of the two previous studies included the modeling of bridges at Edgewater Drive and the Pinellas Trail. In addition, the CDM study did not include the bridge at Douglass Avenue.

2.1.5.3 Flooding Level of Service Criteria

For the development of the comprehensive watershed management plan for the Stevenson Creek watershed, it was necessary to establish flood protection level of service (FPLOS) criteria by which flooding problems can be identified and alternatives for flood control can be evaluated. Determination of appropriate FPLOS criteria is therefore a very important part of the watershed planning process. Once the design storm flood elevations have been determined through the comprehensive modeling procedures, the FPLOS criteria then become the primary factor in determining the cost taxpayers must bear in order to remedy the existing flooding problems. Thus, the FPLOS criteria become the point at which the cost of infrastructure improvements is balanced against the public’s desire to further reduce the flooding in the watershed.

The primary FPLOS criterion is that there is to be no structural flooding (i.e., homes and businesses) for events up to and including the 100-year flood. This criterion refers to the low floor slab elevation (lowest inhabited floor) as the point of structural flooding. This is consistent with FEMA guidelines, and has been widely adopted by counties and municipalities across the state and the country.

The secondary set of FPLOS criteria specifies allowable thresholds for street flooding. The criteria is based upon residential, collector, and arterial roadways being passable for the 10, 25, and 100-year flood events, respectively. For the purposes of this study, "passable" is defined as 6" of flooding or less at the lowest edge of pavement in a travel lane. No flooding is allowable on hurricane evacuation routes for events up to and including the 100-year flood. The following table summarizes the proposed FPLOS criteria for Stevenson Creek:

Table 2.1-7 -- Proposed Flood Protection Level of Service (FPLOS) Criteria for the Stevenson Creek Basin

FPLOS Category	Allowable Flooding Depths
1. Residential Streets, 2 lanes	Up to 6" Allowed for 10-year flood
2. Collector Roads, 2-4 lanes	Up to 6" Allowed for 25-year flood
3. Arterial Roads, 4 or more lanes	Up to 6" Allowed for 100-year flood
4. Hurricane Evacuation Routes	No flooding up to 100-year flood
5. Habitable Structures	No flooding up to 100-year flood

In the next section, the above FPLOS criteria is used to identify and develop potential projects to reduce the design flood elevations to within the allowable flooding depths. However, experience shows that improving infrastructrue to reduce flood depths to meet a desired level of service is not always cost effective. For this reason, project costs must be weighed against the relative benefits. Of course, multiple-use facilities provide benefits in addition to flood control which must also be considered, as will be discussed in subsequent sections. Projects which do not provide benefits commensurate with the costs will not be recommended for funding.

Furthermore, it is not always technically feasable to reduce flood depths to meet a desired level of service. For example, many structures in low-lying areas near the coast in Clearwater were erected prior to the community's participation in the National Flood Insurance Program, which first began in 1968. Many of these structures are subject to saltwater inundation from the 100-year storm (hurricane) surge from the Gulf of Mexico. Although beyond the scope of this Watershed Management Plan, it is highly unlikely that an infrastructure project designed to protect against the 100-year storm surge in this area would be technically feasable.

In instances such as these where it is not cost effective or technically feasable to lower the floodplain, alternatives to flood level reduction should be considered. One such alternative would be a program which allows the city, in cooperation with FEMA, to assist willing property owners in the floodproofing, elevating, or purchase of flood prone structures.

2.1.5.4 Flooding Problem Identification

The limits of the 100-year floodplain were generated by delineating the predicted flood elevations on SWFWMD 1”=200’ one foot contour aerial maps. Within the lower tidal portions of Stevenson Creek, the mapped floodplain limits are governed by FEMA’s computed storm surge elevations, as they are higher than those predicted to occur from riverine (freshwater) flooding due to rainfall and runoff. The storm surge floodplain intersects with the riverine floodplain near the Palmetto Street crossing of Stevenson Creek.

The floodplain limits were digitized for incorporation into the GIS, as illustrated in Figure 2.1-15 in the rear map pocket. The 500-year floodplain was mapped per FEMA requirements and is shown for informational purposes only. Although the hydrologic and hydraulic modeling necessarily included the entire watershed, including areas outside the corporate limits of the City of Clearwater, floodplain limits within the City of Dunedin were not mapped. Riverine flood profiles were plotted for the 10, 25, 50, 100, and 500-year design storm events for Stevenson Creek and Spring Branch, as shown on Figures 2.1-16 through 2.1-24 at the end of this section.

Flood protection level of service deficiencies were identified in the watershed area within the corporate limits of the City of Clearwater. The street flooding level of service deficiencies for the basin are tabulated in Appendix D, Table D.1. Shaded cells indicate locations that currently do not meet the proposed FPLOS criteria. The locations of the street FPLOS deficiencies are shown graphically on Figure 2.1-15.

In order to determine the number and location of structure FPLOS deficiencies, habitable structures within the limits of the 100-year floodplain were identified. A determination was then made as to which homes and businesses were likely to be susceptible to flooding for the 100-year event. This determination was based on the assumption that the floor elevations for slab-on-grade type structures are at least 6 inches above adjacent ground as shown on the SWFWMD aerials. Using this proceedure, 470 structures were identified as potentially flood susceptible. In order to more accurately approximate the actual number of structure FPLOS deficiencies, surveyed finished floor information was compiled for the majority of these structures. Surveyed elevations for approximately 75 of the structures were obtained as part of the 1988 W.K. Daugherty study. An additional 272 structures were surveyed by Harry Marlow, Inc., as part of this study. The remaining 123 structure elevations were estimated from the SWFWMD topography and the surveyed elevations of nearby structures.

Of the 470 structures entered into the database, 263 structures were found to have finished floor elevations below the 100-year flood level. This is, by definition, a structure flooding level of service deficiency. As indicated on Figure 2.1-15, the majority of these structures are located within the primary floodplains of the Stevenson Creek and Spring Branch main channels. However, many of the identified FPLOS deficiencies result from inadequate secondary drainage systems that feed into the main channels.

Table D.2 in Appendix D provides a complete listing of the structure flooding level of service evaluation for the Stevenson Creek Watershed. A summary of the identified structure FPLOS deficiencies in terms of both structures and total dwelling units is provided below as Table 2.1-8. Because many of the structures are multi-family buildings (duplexes, apartment buildings, etc), the number of flood-susceptible dwelling units is larger than the number of flood susceptible structures.

Table 2.1-8 Summary of Structure Flooding Level of Service Deficiencies

Subwatershed	Identified 100-year Flood-Susceptible Structures	Identified 100-year Flood-Susceptible Dwelling Units
Spring Branch	32	32
Lower Stevenson Creek	32	32
Middle Stevenson Creek	49	105
Upper Stevenson Creek	85	85
Hammond Branch	6	6
Lake Bellevue Branch	14	23
Jeffords Street Branch	45	51
Total	**263**	**334**

Subwatershed 1: Spring Branch

The Spring Branch Subwatershed was found to contain 32 structure FPLOS deficiencies. The majority of these (27) are in the area of Byram Drive and Flora Drive between King’s Highway and Highland Avenue, and along Huntington Lane, east of King’s Highway. These FPLOS deficiencies are located in a depressional area bordering the historical Spring Branch channel. At Woodlawn Terrace, east of Douglass Avenue, an additional four structure FPLOS deficiencies were identified in an isolated depressional area drained by an inadequate secondary storm sewer system. These two areas were identified in the 1997 City of Clearwater Watershed Management Plan, through interviews with City staff in the Engineering Department, as “areas of concern for flooding”. Sunset Point Baptist Church was also identified as a level of service deficiency. Four residential street FPLOS deficiencies and eight collector road FPLOS deficiencies were identified, including Douglass Avenue, King's Highway, Sunset Point Road (two locations) and Highland Avenue. Although not FPLOS deficiencies of the City of Clearwater, there are estimated to be 10-15 flood-susceptible homes along Betty Lane and Macomber Avenue, north of Sunset Point Road within unincorporated Pinellas County. The locations of the City of Clearwater FPLOS deficiencies are shown on Figure 2.1-15.

Subwatershed 2: Lower Stevenson Creek

In the Lower Stevenson Creek Subwatershed, 32 structure FPLOS deficiencies were identified. The majority of these (27) are located along the main channel of Stevenson Creek between Douglass Avenue and Palmetto Street. These homes and businesses all have floor elevations below 9.5 feet NGVD, and are susceptible to flooding from both riverine flooding (rainfall and runoff) and storm (hurricane) surge. The 100-year storm surge elevation, as determined from the FEMA Flood Insurance Rate Maps, is elevation 11.0, NGVD. In addition to the structures susceptible to both flooding sources, there are several additional structures between Edgewater Drive and Palmetto Street which are susceptible to storm surge flooding only, which were not included. Other structure FPLOS deficiencies in Lower Stevenson Creek include two utility buildings adjacent to the Creek in the Clearwater Country Club. Also identified were two relatively isolated structure FPLOS deficiencies in the North Greenwood area and one near the intersection of Palmetto Street and King's Highway. The latter three FPLOS deficiencies are far removed from the creek, and are the result of inadequate secondary drainage systems. Residential street FPLOS deficiencies were identified at seven locations, and collector road FPLOS deficiencies were identified at six locations, including Douglass Avenue, Fairmont Avenue (two locations), Palmetto Street (two locations), and North Greenwood Avenue (refer to Figure 2.1-15).

Subwatershed 3: Middle Stevenson Creek

In terms of the concentration of flood-susceptible dwelling units, the most severe flooding problems in the watershed occur within the Middle Stevenson Creek subwatershed. Approximately 49 structures consisting of 105 ground-floor dwelling units along the banks of Middle Stevenson Creek were identified that are subject to inundation from the 100-year design flood. Many of the structures are multi-family buildings. All of the FPLOS deficiencies identified in this subwatershed are adjacent to the main channel of the Creek between Drew Street and Jeffords Street. The majority (60) of these structure FPLOS deficiencies occur in the area immediately to the west of Glen Oaks Golf Course. To provide an idea of the magnitude of the flood problem in this area, 24 dwelling units were also found to be vulnerable to the 25-year flood. In addition, three homes were found susceptible to the 10-year flood, including two homes in the 600 block of Betty Lane, which have been the subject of documented structure flooding. The most notable street FPLOS deficiency occurs at Cleveland Street, a hurricane evacuation route, at its crossing of Stevenson Creek. The predicted flood depths are more than two feet above the roadway for the 100-year flood. The roadway would also be impassable for a 25-year flood, and minor flooding is predicted for a 10-year flood. Residential street FPLOS deficiencies were identified at five locations, including Pierce Street, Franklin Street, Betty Lane (2 locations), and Mark Drive.

Although the construction of the Phase 1 and Phase 2 improvements in the 1990’s significantly improved the conveyance capacity of Stevenson Creek from Drew Street to Court Street, floodplain enhancements within the Glen Oaks Golf Course were never constructed to the level originally envisioned in the 1990-1991 re-study of Stevenson Creek by CDM. In addition, the effectiveness of the constructed conveyance improvements has been greatly diminished due to a lack of adequate maintenance. Layers of sediment, and at times dense vegetation, have occupied portions of the improved channel. This problem was observed to be most severe between Drew Street and Cleveland Street.

Subwatershed 4: Upper Stevenson Creek

Upper Stevenson Creek encompasses the portion of the Creek targeted for conveyance improvements in Phase 3 of the 1988 W.K. Daugherty study. This phase was never constructed, primarily due to environmental concerns over the proposed improvement method of hardlining (paving) the channel with concrete. In the current analysis, the Upper Stevenson Creek subwatershed was found to contain 85 structure FPLOS deficiencies. All of these lie within the floodplain of the main channel of Stevenson Creek, with the largest concentration of them occuring on both sides of the Creek between Jeffords Street and Lakeview Avenue, and along Hillcrest Avenue south of Lakeview Road (see Figure 2.1-25).

Figure 2.1-25 Flooding on Hillcrest Avenue, South of Lakeview Road
Storm of July 15 , 2000

However, the most severe and frequent flooding in the subwatershed occurs along Evergreen Avenue between Bellevue Boulevard and St. Thomas Drive, where seven homes were found susceptible to the 10-year flood. According to the City's complaint logs, at least two of these structures flooded during the recent storm of July 15, 2000. The flooding problem at this location is due to a 54" CMP culvert placed within the Stevenson Creek Channel that lacks the capacity to convey even the mean annual storm event. Residential street FPLOS deficiencies were identified at seven locations, and collector road FPLOS deficiencies were identified on Belleair Rd and Highlands Avenue.

Subwatershed 5: Hammond Branch

Hammond Branch was found to contain six structure FPLOS deficiencies, the fewest of all the subwatersheds. All of the FPLOS deficiencies are a result of inadequate secondary drainage systems. A single structure FPLOS deficiency on Hibiscus Street near King's Highway occurs when the storage capacity of the Hibiscus Pond is exceeded due to inflows from a 54" RCP culvert from the Highland Avenue drainage system. The existing outfall of Hibiscus Pond, consisting of a gunited 30" CMP, is not adequate to handle the inflows from the Highland's Avenue culvert. An unrelated problem occurs on Smallwood Circle (southeast of Highland and Palmetto), where two structure FPLOS deficiencies were identified. These homes were built in a depressional area surrounding a small city park, which floods frequently during moderate to heavy rains. The depression is drained by an undersized, failing 24" CMP culvert which runs under existing homes on Elmwood Street and Smallwood Circle. In addition, three FPLOS deficiencies were identified on Linwood Drive near Sharondale Drive, on the north side of the CSX railroad that runs parallel to Flagler Drive. This problem area is the result of floodwaters from the Flagler Drive ditch backing up through twin 48" RCP culverts under the CSX railroad. The 48" culverts were intended to convey runoff from the subbasin to the north into the Flagler ditch. During flood events, however, the culverts flow in the opposite direction, directing floodwaters from the south side of the railroad, across the flood susceptible properties bordering Linwood Drive. The problem is exacerbated by the fact that the floor elevations of the homes are only two to three feet above the invert of the Flagler Drive Ditch.

Residential street FPLOS deficiencies were identified at twelve locations in the Hammond Branch Subwatershed. Collector road FPLOS deficiencies were identified at nine locations, including three locations on King's Highway, four locations on Highland Avenue, and two locations on Palmetto Street.

Subwatershed 6: Lake Bellevue Branch

Incrementally over past several decades, the Lake Bellevue Branch has been almost completely enclosed in storm sewer. Unfortunately, little of the storm piping was designed to adequately handle major a storm event. In the Lake Bellevue Branch Subwatershed, 14 structures consisting of 23 ground-floor dwelling units were found susceptible to inundation from the 100-year design flood. Of this total, five are businesses located on Missouri Avenue immediately south of Turner Street. Missouri Avenue itself constitutes an arterial road FPLOS deficiency at this location. This flooding problem is due to the fact that upstream of Missouri Avenue, the Lake Bellevue Branch flows into a 54" diameter culvert which lacks the capacity to convey the flow resulting from even a mean-annual storm event. Further upstream in the subwatershed, four structure FPLOS deficiencies occur at Pinellas Street, east of Greenwood Avenue, and one on Tuskawilla Street, west of Greenwood Avenue. These FPLOS deficiencies are also the result of undersized culverts placed within the historic channel of the Lake Bellevue Branch. Upstream of Lake Bellevue, four flood-susceptible structures containing a total of 13 dwelling units were identified on Wildwood Way, west of the CSX railroad. These structures were built in a low area southwest of the Lake. The drainage for this area is towards Lake Bellevue, but flows are restricted by undersized culverts beneath the CSX railroad tracks. In addition to the single arterial road FPLOS deficiency on Missouri Avenue, the Lake Bellevue Branch contains five collector road FPLOS deficiencies on Druid Road, South Greenwood Avenue (two locations), Lakeview Road, and Belleair Road, and six residential street FPLOS deficiencies.

Subwatershed 7: Jeffords Street Branch

Within the Jeffords Street Branch Subwatershed, 45 structures containing 51 dwelling units were found susceptible to the 100-year flood. The majority of these structures (35 of 45) are located within a contiguous floodplain area that encompasses the Jeffords Street ditch and the three interconnected lakes in the vicinity of Duncan Avenue and Jeffords Street. Historical 1926 aerial photography shows a wetland existed where these structures and lakes are today. It is likely that material dredged to form the three lakes was used as fill to form the building lots and roadways in the former wetland. However, the area is still extremely floodprone, as evidenced by the numerous documented complaints of flooding, and the mapped extent of the 100-year floodplain. The remaining structure FPLOS deficiencies (10 structures/16 dwelling units) were also constructed in a former wetland, in the area now bounded by Spencer Avenue, Turner Street, Duncan Avenue, and Druid Road. These structures flood on a very frequent basis, as seven are below the 10-year flood level and nine of the ten are below the 25-year flood level. According to the City's complaint logs, at least two of these structures flooded during the recent storm of July 15, 2000. Collector road FPLOS deficiencies were identified on Highland Avenue, Lake Avenue (two locations), and Cleveland Street at Duncan Avenue. In addition, seven residential street FPLOS deficiencies were identified, most of which lie within the large contiguous floodplain area that encompasses the Jeffords Street Ditch.

- Figure 2.1-17 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-18 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-19 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-20 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-21 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-22 Stevenson Creek Flood Profile, Existing Conditions
- Figure 2.1-23 Spring Branch Flood Profile, Existing Conditions

- Figure 2.1-24 Spring Branch Flood Profile, Existing Conditions