BIG CYPRESS BASIN WATERSHED MANAGEMENT PLAN

FLOOD CONTROL ELEMENT

TASK III: A - Problem Analysis

B - Alternative Evaluation and Plan Development

May 2000

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

The Big Cypress Basin operates and maintains 17 canals (approximately 163 linear miles total) and 40 water control structures in Collier County, Florida. This water management system does not have a consistent design or water management strategy. The Big Cypress Basin Board directed staff to assess the water resources of this system and to develop a regional watershed management plan. The overall study includes the following major components:

- Develop a comprehensive basin model using existing computer programs
- Calibrate the basin model to both wet year and dry year conditions
- Simulate design storms to identify problem areas for flood control
- Assess alternatives and develop a regional plan to address issues affecting flooding, water supply, and environmental quality and prepare a Capital Improvement Program

This report addresses the third and fourth components in the above list. The design storm is the 3-day, 25-year storm event for the designated urban areas, and a 3-day, 10-year storm event for the rural areas of Collier County. By definition, the urban area design storm has a four percent chance of occurrence in any given year, and is similar in magnitude to the storm experienced in August 1995.

Flood control problem areas are defined as areas where at least one of the following items occurs:

- The maximum flood elevations exceed the canal bank elevation
- The maximum flood elevation is within 1.5 feet of the canal bank elevation
- Water velocities exceed 2.5 feet per second, creating a potential for erosion

- Structures within the canal do not meet District right of way criteria

Based on the above criteria, flood control problem areas include:

- 11.8 miles out of 27.0 miles of the Golden Gate Main Canal (43%)
- 2.2 miles out of 6.0 miles of the Corkscrew Canal (37%)
- 2.5 miles out of 7.2 miles of the CR 951 Canal (35%)
- 3.2 miles out of 7.0 miles of the I-75 Canal (45%)
- 1.2 miles out of 8.0 miles of the Cypress Canal (3%)
- 6.75 miles out of 6.75 miles of the Henderson Creek Canal (100%)
- 7.7 miles out of 18.75 miles of the Miller Canal (30%)
- 10.8 miles out of 29.5 miles of the Faka Union Canal (33%)
- 1.6 miles out of 12.0 miles of the Merritt Canal (13%)

Approximately 47.6 miles out of 163 miles total in the Big Cypress Basin (29%) do not have adequate flood control for the design storm events.

A series of alternative water management strategies was developed to assess their effectiveness for flood control. The alternatives analyzed the feasibility of channel modification, flow diversion, and storage by primarily considering restoration of the predevelopment flowways, or their equivalents, to improve flow conveyance while at the same time, enhancing potentials for water supply and ecologic function.

The following predominant alternatives were identified for assessment following preliminary screening:

1. Do nothing.
2. Diversion of a portion of Golden Gate Canal flows to the Henderson Creek basin.

3. Channel and structural modifications of the Golden Gate Main Canal west of CR951.
4. Channel modification of CR951 Canal and implementation of water control measures.
5. Diversion of a portion of Corkscrew Canal flows eastward to Golden Gate Main Canal north of Golden Gate Canal Weir No. 5.
6. Cocohatchee basin east and west flowways.
7. Aquifer Storage and Recovery (ASR) of wet season flows of the Golden Gate Main Canal near CR951.
8. Modification of the C-1 Connector Canal and relocation of Miller Canal Weir No. 3.
9. Improve Henderson Creek Canal and undersized crossings.
10. Restore Camp Keais Strand flowway.

Several combinations and subsets of the above alternatives were evaluated in terms of their hydraulic efficiency and economy of cost in achieving the desired performance measures for flood protection, primarily for the designated urban areas west of a north-south line one mile east of CR951.

The combination of optimized configurations of alternatives 2, 3, 4, 7, and 9 involving channel improvement, flow diversion, structural retrofit, and ASR is recommended as the feasible plan for the designated urban areas of Collier County to achieve the desired level of flood protection. The estimated cost of the plan is $12.6 million for the Golden Gate Main Canal, $3.4 million for CR951 Canal, and $6.0 million for the Henderson Creek segment.

Improved management of surface water through flow diversion and restoration of historic flowways will further be analyzed by the application of an integrated surface and groundwater model to quantify impacts on regional water supply and ecologic functions in the wetlands and receiving estuaries. The components of the Southern Golden Gate Estates restoration plan and the Tamiami Trail flow enhancement project will be incorporated with the recommendations of this plan.

The recommendation outlined in this plan should provide the first step guideline to the Basin Board to revisit the elements of the Basin's short- and long-range capital improvement plan and to implement them within the purview of the annual budgets.

1 INTRODUCTION

1.1 Project Background

The western Big Cypress Basin (BCB) watershed (*Figure 1*) drains an approximately 1,114 square-mile area through a large network of man-made canals and natural sloughs. The BCB encompasses all of western Collier County, and small sections of Southern Lee, and Hendry Counties.

With the evolution of urban and agricultural development, the traditional surface water flow patterns in the Big Cypress Basin Watershed have undergone drastic changes. Historic flow-ways in the region followed the natural drainage emanating from the Immokalee highlands through a series of strands, sloughs and more broadly as surface sheetflows to the tidal passes of the Gulf of Mexico. These natural features consisted of a series of wetlands or swamps connected by shallow drainage ways or sloughs, and were divided by low ridges, which were dry for a portion of the year, and overtopped by water in periods of seasonal high rainfall. Characteristic of natural strands, the historic water flows were extremely slow and penetrating due to vegetation and physical geography. Hydroperiods were extended well into the winter/spring dry season. The entire western Collier region (west of the present day SR 29) is comprised of five major historical flowways or sub-basins, namely:

1. Fakahatchee-Okaloacoochee Basin,
2. Picayune-Camp Keais Basin,
3. Belle Meade Basin,
4. Rock Creek-Gordon River Basin, and

5. Cocohatchee-Corkscrew Basin.

Historically, the Corkscrew system served as the headwaters to the above basins with a portion of the Okaloacoochee Slough contributing to the Fakahatchee system. However, as land areas began to be developed, the typical “ditch and drain” development resulted in a series of canals and numerous roads that tended to overdrain the water table and drastically alter the flow patterns of the natural drainage basins. Such combinations of development events have greatly reduced the areas of functional wetlands, lowered groundwater levels, reduced aquifer recharge, and contributed to concentrating the flow of runoff instead of allowing the traditional sheetflow across the land. With the change in flow characteristics came the associated environmental impacts on the overall ecology of the uplands, wetlands and the estuaries of the region resulting in a change in the entire landscape.

Since its inception in 1977, the Big Cypress Basin Board BCBB has taken an aggressive role in conducting detailed inventories and evaluating its resources for the purpose of developing preliminary water management plans for a number of individual basins. Most of the studies inventoried and analyzed the hydrologic-hydraulic characteristics of each basin separately and recommended plans for water management of the individual basin components. However, none of the earlier studies looked at a composite plan treating the historic Western Collier watershed as an interrelated single unit.

Recognizing that much of today’s water management problems have emerged from disruption of the historic flowways, it is expected that many such problems and impacts can be minimized by trying to reassemble these historic surface flow characteristics, wherever possible. To more effectively evaluate existing conditions and assess alternative water management measures, the entire Big Cypress Basin must be considered as a single interrelated unit. The development of a comprehensive hydrologic-hydraulic model of the entire landscape of the Western BCB hydrologic region and its associated canal network as a single interrelated unit is deemed necessary to evaluate the existing system and to analyze the effectiveness of water management measures in formulating a regional watershed management plan. The other integral tool of this planning process is an ecologic assessment model to evaluate the ecologic features of the public lands under proposed water management strategies. The hydrologic-hydraulic model and the ecologic assessment methodologies are to be used interactively to assess the existing conditions, and evaluate alternative water management strategies. In 1996 the BCBB undertook the task of development of a regional watershed management plan utilizing the above approach in three phases:

- Phase I: Regional Hydrologic-Hydraulic Model Development,
- Phase II: Ecologic assessment of Public lands, and
- Phase III: Problem identification, alternative evaluation, and plan formulation.

The first two phases of the project was completed in 1998. The hydrologic-hydraulic modeling tools developed in phase I of the study are modified and have been applied in this effort to assess the hydrologic characteristics of the basin, evaluate the hydraulic capacities of the existing waterways, and develop an implementable flood control plan.

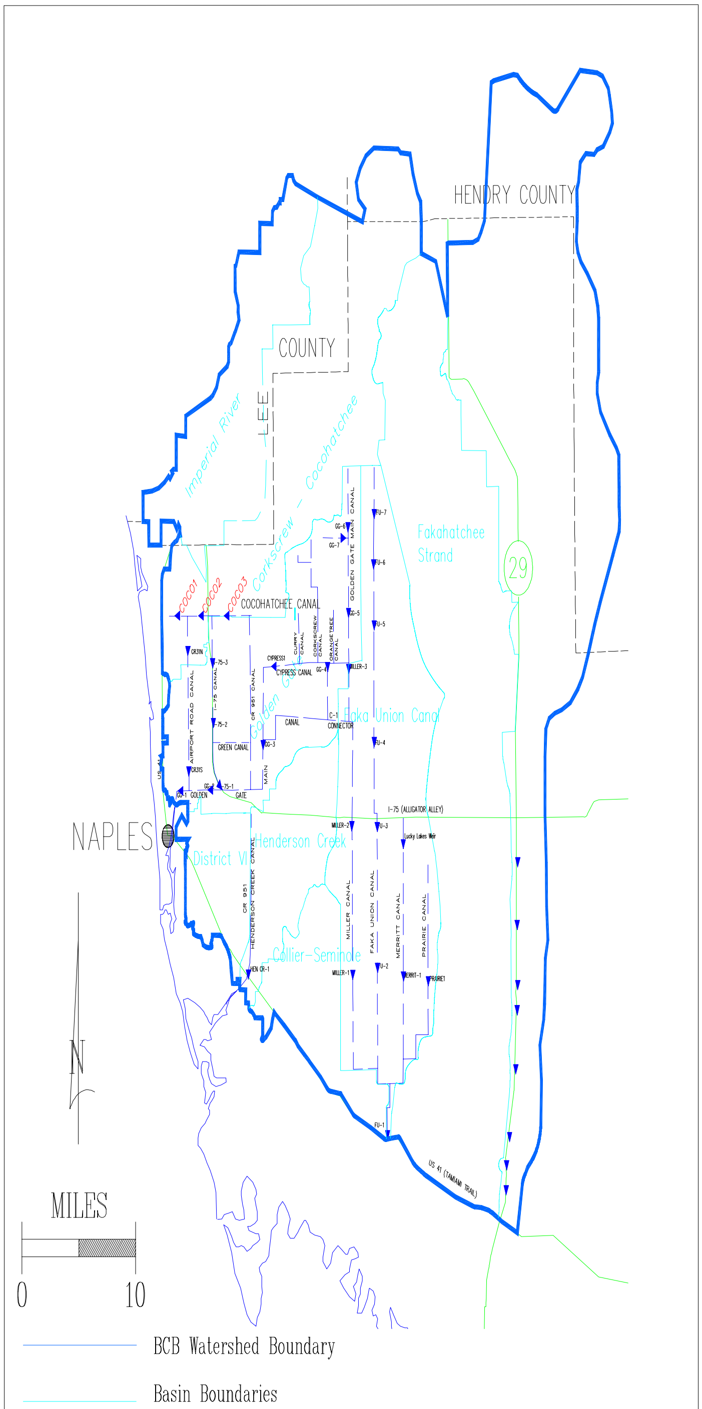


FIGURE 1. Study Area of BCB Watershed Plan

File: Figure1 Study Area.dwg

1.2 Project Goals and Objectives

The objective of Phase IIIA of the study is to evaluate the hydrologic characteristics of the major basins of the BCB watershed in terms of rainfall-runoff relationship, and to assess the conveyance capacities of the existing network of primary canals and water control structures of the watershed. The basic premise of the study will first be to incorporate the time history of meteorological data and physical information on surface hydrology, such as soils, land use and topography at various spatial scales, and integrate the dynamic processes that have influenced the overall hydrology and ecology of the Big Cypress Basin. This process will be used to generate rainfall-runoff relationship under design storms for a 25, 50 and 100-year recurrence interval events for 3-day storm duration conditions. The second step of the integrated analysis of the watershed will incorporate the hydraulic properties of the primary waterways such as the channel geometry, cross-drainage and water control structures, bed and bank material features to evaluate the conveyance capacity in meeting the level of service needs of flood control. This analysis will help identify the segments of the primary canal system that are not capable of meeting the level of service standards for flood control. Phase IIIB of the study effort involves development of and hydraulic analysis of various alternatives to formulate an economically feasible and implementable flood control plan without adverse environmental or social impacts.

2 PROBLEM IDENTIFICATION

2.1 Planning Issues

The rapid growth of Collier County during the past decades with its increased population and accompanying urban and agricultural developments has stimulated significant concerns regarding the water and environmental resources of the region. A myriad of issues relating to the water supply, flood protection, water quality and natural ecosystems of the Big Cypress Basin Watershed, have emerged from such phenomenal growth. Comprehensive planning for the wise use and development of the land and water resources of the BCB watershed requires knowledge and understanding of the issues, and the relationship existing among the many natural and man-made features that together comprise the hydrologic-hydraulic system of the watershed. An understanding of this system is of utmost importance to the BCB watershed planning program because of the interdependence of land use, surface and groundwater, any planned modification to or development of one element of the hydrologic-hydraulic system must consider the potential results and effects on all other elements of the system. Only by considering the hydrologic-hydraulic system as a whole can a sound, comprehensive watershed management plan be prepared and the water-related problems of the basin be ultimately abated. A summary of the issues pertinent to water supply and flood control specific to this study is presented below.

2.1.1 Water Supply

The BCB watershed is underlain by three major freshwater aquifers (Water Table, Lower Tamiami and Sandstone Aquifers) that provide the main source of water supply.

The Water Table and Lower Tamiami Aquifers are the primary sources of water supply and occur within the Surficial Aquifer system. The Sandstone Aquifer, part of the intermediate aquifer system is separated from the surficial system by low permeability sediments, and is only present on the northern part of the watershed. The primary source of recharge to the surficial aquifer system is rainfall. Downward movement of water through the leaky confining beds underlying the water table recharges the Lower Tamiami Aquifer. Throughout the BCB watershed, there is a direct hydraulic connection between the canal system and the upper portions of the Surficial Aquifer. Thus, rapid rate of runoff provided by the canals is a primary cause of depletion of groundwater storage. As an example, the overdrainage by canals have caused general drawdown of approximately two feet, a distance of one mile from the canals in the Faka Union Canal watershed during the early years of implementation of the canals. The drawdown of the water table has worsened considerably during the later years (Wang and Overman, 1981). Recharge from the canal has a great influence on the yield of the various public water supply wellfields within the BCB. Protection of the long-term sustained yield of the wellfields is one of the primary water supply related issues of the BCB watershed plan.

2.1.2 Flood Control

The flood control element of the BCB watershed plan is of prime concern in this study. Indeed, the rapid urban growth and agricultural developments in Collier County is not without adverse consequences. Extensive land development has caused encroachment into low lying natural storage areas resulting in occasional flooding. Due to historic wetland nature of the landscape of the BCB, flooding can be a common and natural occurrence. Historically, the seasonal high stormwater runoff of the watershed left the channels and occupied portions of the adjacent natural floodplains almost annually as a result of summer thunderstorms and tropical rainfall events. Damage from this flooding has been, to a large extent, a consequence of the failure to recognize and understand the relationships which should exist between the use of land and the hydrologic-hydraulic behavior of the stream and canal system. Unnecessary occupancy of the natural floodlands by flood-vulnerable land uses, together with development-induced changes in the flow characteristics of the streams, has substantially increased flood risks.

Comprehensive watershed planning is the first step in achieving or restoring a balance between the use of land and the hydrologic-hydraulic regimen of the watershed. To ensure that future flood damage will be held to a minimum, plans for the proper utilization of the riverine areas of the watershed must be developed so that public acquisition, land use controls, and river and open channel hydraulic engineering can be used to properly direct new development into a pattern compatible with the demands of the canal system on its natural floodlands and to achieve an adjustment or balance between land use development and floodwater flow and storage needs.

Flood damage potential and flood risk have grown from a nuisance level during predominantly natural/agricultural use of the watershed to substantial proportions as urban land use has increased. Practically all of the present flood risk can be ascribed to unnecessary location of flood damage-prone urban development in the natural floodlands. Nevertheless, in the absence of a sound watershed plan, such occupation of the floodlands may be expected to continue to increase as urban development proceeds within the watershed. Much of the floodlands, however, are as yet unoccupied by flood-vulnerable urban uses; and the opportunity still exists for limiting flood damage risk through sound land use development.

The phenomenal rate of population increase in Collier County has also resulted in encroachment of residential development into some of the critical wetlands which traditionally provide natural storage of floodwaters. The rapid pace of conversion of some low-density residential areas to higher density with resultant increase in impervious areas has led to much quicker response of flood peaks as was evidenced by the flooding of several areas along the Golden Gate Canal in 1995. The 1991, 1993, and 1995 floodings in the North Naples Bonita Springs area are also primary examples of squeezing the flood ways and outlets of the Imperial and Cocohatchee Rivers. Flood inundation photographs illustrated in Appendix A represent the gravity of the problem of flooding in the rapidly urbanizing region.

These illustrations clearly indicate that there is a crucial need to improve the flood control capabilities of the BCB waterways to ensure safety of public health and welfare from the havoc of flooding.

2.1.3 Natural Ecosystems Management

A unique combination of ecosystem dominates the landscape of the Big Cypress Basin with a vast extent of wet prairies, pine and cabbage palm flatwoods, hardwood hammocks and tidal marshes. The sloughs, strands and wet prairies carry the freshwater surface flow to the Ten Thousand Island Estuaries, one of the largest mangrove systems in Florida. Many of the large scale-developments, like the Golden Gate Estates, have played an effective role in overdraining the pristine forested and emergent wetlands, and degraded the productivity of the wetland system due to shortened hydroperiods. In addition, invasion of exotic plants like melaleuca and Brazilian pepper is beginning to pose problems to the native ecosystem and habitat. Since the hydrology of an area is the basis for structuring the type of plant and animal community that will exist, changes to the hydrology can cause a reorganization of the plant and animal community structure. The protection and management of the sensitive environmental resources like SGGE is to be achieved by public acquisition and restoration of the affected lands. Statutory changes to the Areas of Critical State Concern Program in 1993 proposed designating certain areas of Collier County as the Big Cypress Areas of Critical State Concern, and recommended: "The acquisition of Save Our Everglades CARL projects needs to be completed. The Land Selection Advisory Council should elevate the priority rankings of these projects to demonstrate the importance of these projects to the protection of the natural resources within the Big Cypress Area of Critical State Concern. The Board of Trustees should support the FDEP in using eminent domain to acquire these two CARL projects if voluntary negotiations are not successful." (District Water Management Plan; South Florida Water Management District, 1995)

2.1.4 Water Quality

Good quality of water is essential to all forms of life. In so far as the physical and chemical conditions of surface waters in the Class III freshwater bodies (recreation, fish and wildlife propagation) of the BCB area are concerned, they generally meet the acceptable state standards. The quality of groundwater is also within the FDEP's drinking water standard for potable supply. However, at issues are the quality and routing of the receiving waters of the Naples Bay, the Faka Union Bay and the Ten Thousand Islands, where enormous volumes of freshwater outflow from the Golden Gate and Faka Union Canal Systems create abnormal salinity levels throughout the year.

The discharge from the Golden Gate and the Faka Union Canals varies seasonally with a large amplitude. This results in large fluctuations in the salinity levels and current patterns with enormous shocks to the aquatic biota of the bays, and often, too little freshwater input to the surrounding saline areas. The rapid decline in the salinity to near freshwater conditions have caused prolonged salinity stresses and has eliminated or displaced a high proportion of the benthic, midwater and fish plankton communities from the Bay. Such suppressed plankton development has resulted in very low relative abundance of midwater fish and also considerable drop in shellfish harvest levels. Seagrass meadows are no longer a prevalent habitat type in these bays. Instead, bare sandy mud and algal areas predominate. The impact on commercial and recreational fisheries have been very significant.

3 PROBLEM ANALYSIS

The flood control element of the BCB watershed plan mainly focuses on the hydrologic-hydraulic (H&H) assessment of the various basins and waterways. A set of regional H&H models developed in Phase I of this Watershed Management Planning effort is used for the study. The reader is referred to the Tasks A, B, C and D Reports of the BCB Watershed Plan Phase I for details of the model development process. The BCB Watershed regional model includes components of two modeling programs known as the Linked Watershed-Waterbody (LWWM) and the United States Army Corps of Engineers one-dimensional unsteady flow hydraulic model UNET. The LWWM includes the USEPA Stormwater Management Model (SWMM), with a preprocessor for creating a SWMM input file. The LWWM utilizes information from GIS database coverage on land use, soils, topography, etc. to create an input data set for SWMM, links the SWMM output with the Water Analysis Simulation Program (WASP), and post-processes the results. For this application, only the component for creating the SWMM input file was used. GIS data of land use, soils, and topography for the BCB watershed provided spatial input to the model.

3.1 Hydrologic Submodel

The hydrologic modeling component of the Big Cypress Basin watershed plan is concerned with estimating the rate and volume of surface runoff generated from specified design storms, into the network of waterways - i.e., natural sloughs and man-made canals. The RUNOFF block of the SWMM model was used to simulate the rainfall-runoff and the upper zone ground water processes, and the interaction between ground and surface water.

The BCB regional model was set up to simulate both surface runoff and shallow groundwater influence for 127 delineated subbasins for eight basins (*Figure 2*), namely;

1. Okaloacoochee Slough – Barron River Basin,
2. Fakahatchee Strand Basin,
3. Faka Union Canal Basin,
4. Collier Seminole Basin,
5. Henderson Creek – Belle-Meade Basin,
6. District 6 Basin,
7. Golden Gate Canal Basin, and
8. Corkscrew-Cocohatchee Basin.

For each identified subbasin there is an outlet point where runoff hydrographs are generated. In order for the model to properly collate and route the various flow inputs, connectivity between land segments (subbasins) must be defined. Runoff hydrographs are applied at defined node locations. Groundwater interactions are simulated along the connecting reaches between SWMM nodes.

Hourly precipitation records were used for ten rainfall gauging stations in or near the watershed during the calibration year, 1995. Prominent physical and hydrologic parameters used were: directly connected impervious area, mannings roughness coefficient, depression storage, porosity, wilting point, saturated hydraulic conductivity, field capacity and initial upper zone moisture. Green Ampt methods were utilized for infiltration. As the vast majority of soils within the basin are of the same hydrologic soil group (HSG), category D, infiltration parameters were not area weighted by HSG during calibration, but were adjusted directly assuming 100 percent D soil.

3.1.1 SWMM RUNOFF Block

The EPA SWMM Runoff block simulates stormwater runoff from various subbasins, water quality processes, and the upper zone groundwater process and its interaction with surface water. However, in the present study, only the runoff component was modeled. The Runoff Block provides streamflow rates input to the surface water hydraulic system and groundwater system. The model uses a kinematic wave approximation to route surface runoff over pervious and impervious surfaces. The model account for infiltration (using either Horton's Equation or Green-Ampt Equation), evaporation, snowmelt, abstraction (or depression storage) and performs a simple routing through pipes or trapezoidal channels. The output from the Runoff Block is used as input to the UNET hydraulic model that routes the streamflow dynamically through the canal network system.

Using the rainfall hyetographs, the land use information, and watershed physical characteristics, SWMM generates runoff hydrographs at selected points of interest. In the Phase I of this study, Dames & Moore developed and calibrated the original SWMM input file for the watershed for continuous simulation with no flow routing. The output results of this original setup appeared to generate significantly high flows when the design storms were simulated, and resulted in surcharging in many canals. The Big Cypress Basin staff refined the earlier setup to reflect more realistic assumptions that were not previously considered and to improve the surcharging phenomena. The refining process mainly involved:

- modification of the geometric characteristics and the Manning's coefficients of the various subbasins

- change from continuous to event based simulation to model the 25, 50 and 100-year 3-day design storms
- Incorporation of future land uses.

The new future land use in the Runoff Block affected the following parameters:

- directly connected impervious areas (DCIA)
- Manning roughness
- Pervious depression storage.

Soils, topography and subbasin boundaries remained unchanged from the Dames & Moore setup. *Figure 2* presents the SWMM model schematic including subbasin delineation, hydrograph output locations, and overland (wetland) flow paths. A brief overview of the design storms computation is given in the following section.

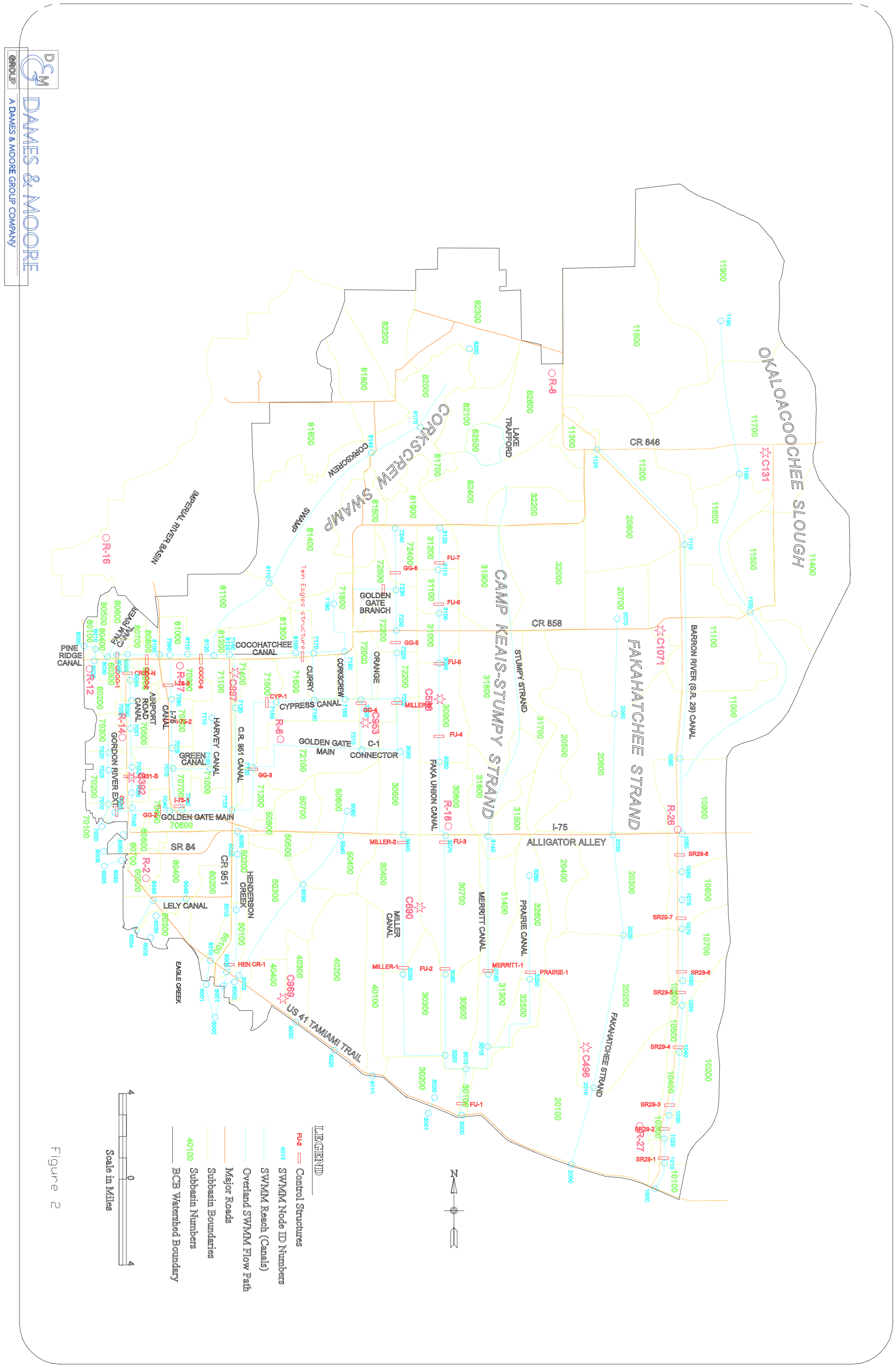

DAMES & MOORE
A DAMES & MOORE GROUP COMPANY
OKALOACOOCHEE SLOUGH
CORKSCREW SWAMP
CAMP KEAIS-STUMPY STRAND
FAKAHATCHEE STRAND
CR 846
CR 858
I-75
ALLIGATOR ALLEY
US 41 TAMIAMI TRAIL
SR 84
CR 951
IMPERIAL RIVER BASIN
COCOHATCHEE CANAL
CYPRESS CANAL
GOLDEN GATE MAIN
LELY CANAL
HENDERSON CREEK
BARRON RIVER (S.R. 29) CANAL
FAKA UNION CANAL
MERRITT CANAL
PRAIRIE CANAL
MILLER CANAL
STUMPY STRAND
LAKE TRAFFORD
PINE RIDGE CANAL
GORDON RIVER EXT.
HARVEY CANAL
GREEN CANAL
EAGLE CREEK
LEGEND
Control Structures
SWMM Node ID Numbers
SWMM Reach (Canals)
Overland SWMM Flow Path
Major Roads
Subbasin Boundaries
Subbasin Numbers
BCB Watershed Boundary
Scale in Miles

Figure 2

3.1.2 Design Storm Hydrologic Analysis

3.1.2.1 Design Storm Computation

The depth of rainfall in inches for a specific return period and storm duration is the most basic parameter needed in the design and analysis of a stormwater management system. In the BCB watershed plan, three design storms (25, 50, and 100-year 3-day) were selected for the hydrologic analysis. The procedure for the design computation is outlined in the South Florida Water Management District (SFWMD) Basis of Review Volume IV and summarized as follows.

Once the design frequency and duration are known, the area-weighted rainfall depth is estimated from an isohyetal map (see *Figures 3A – 3D*). The following example illustrates the calculation of a 25-year 3-day design storm over the Big Cypress Basin. From *Figure 3* the average 25-year 1-day depth is approximately 8 inches. From the Basis of Review, the 1-day value of 8 inches is multiplied by 1.359 to obtain a 3-day depth of 12 inches. The three design storms used in this study are summarized in the following *Table 1* and their distribution is illustrated in *Figure 4*:

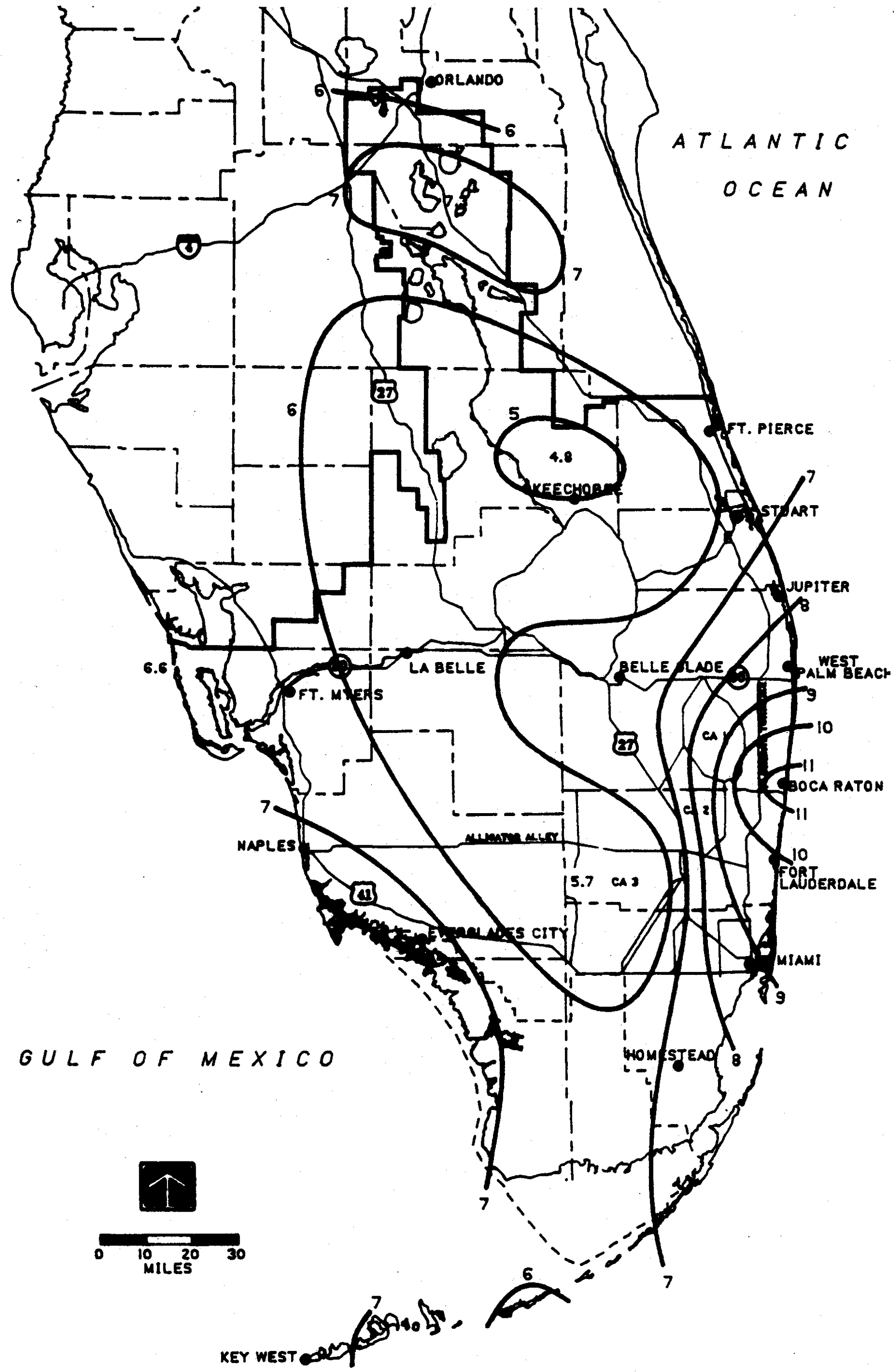


FIGURE 3-A 1-DAY RAINFALL : 10 YEAR RETURN PERIOD

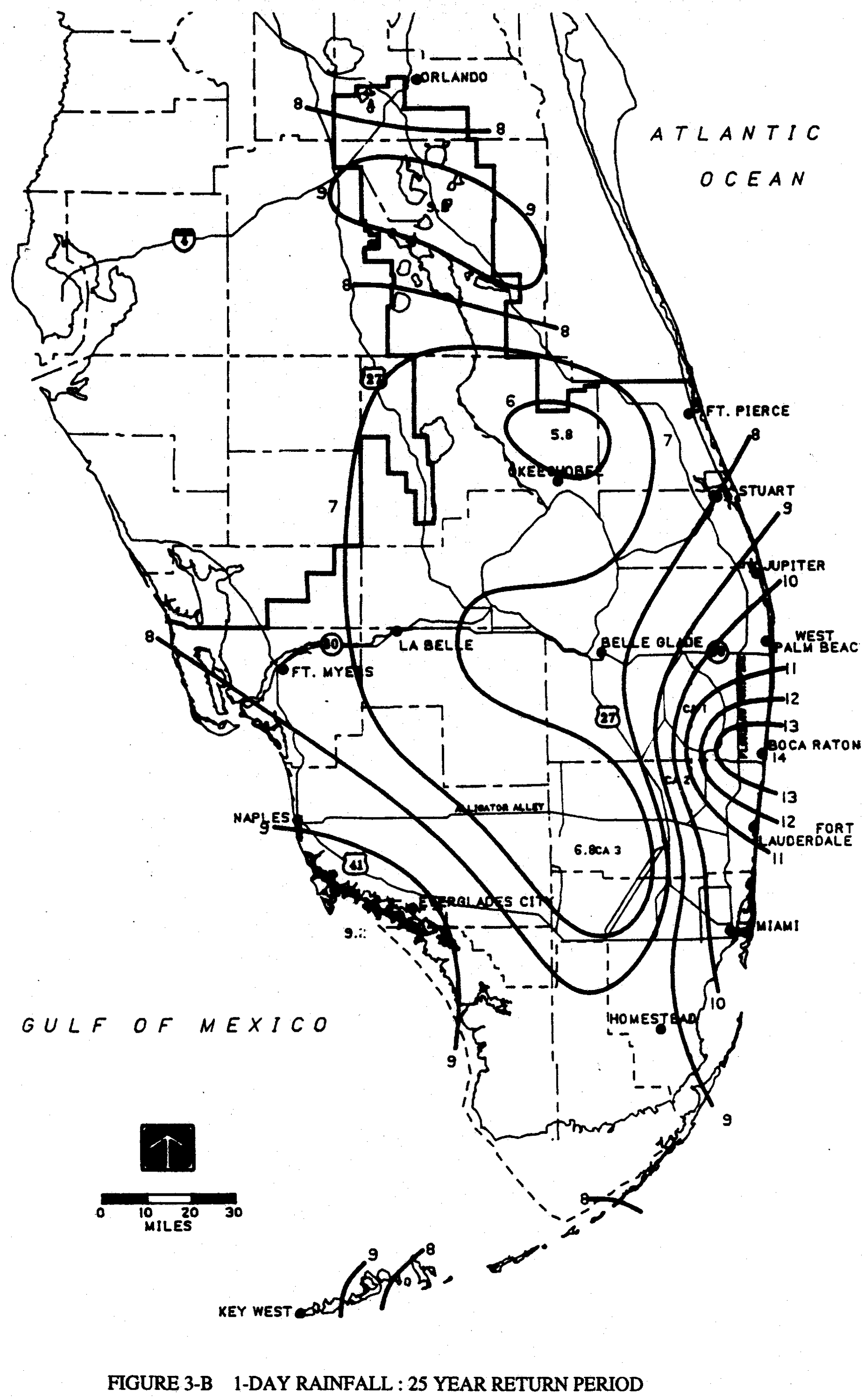


FIGURE 3-B 1-DAY RAINFALL : 25 YEAR RETURN PERIOD

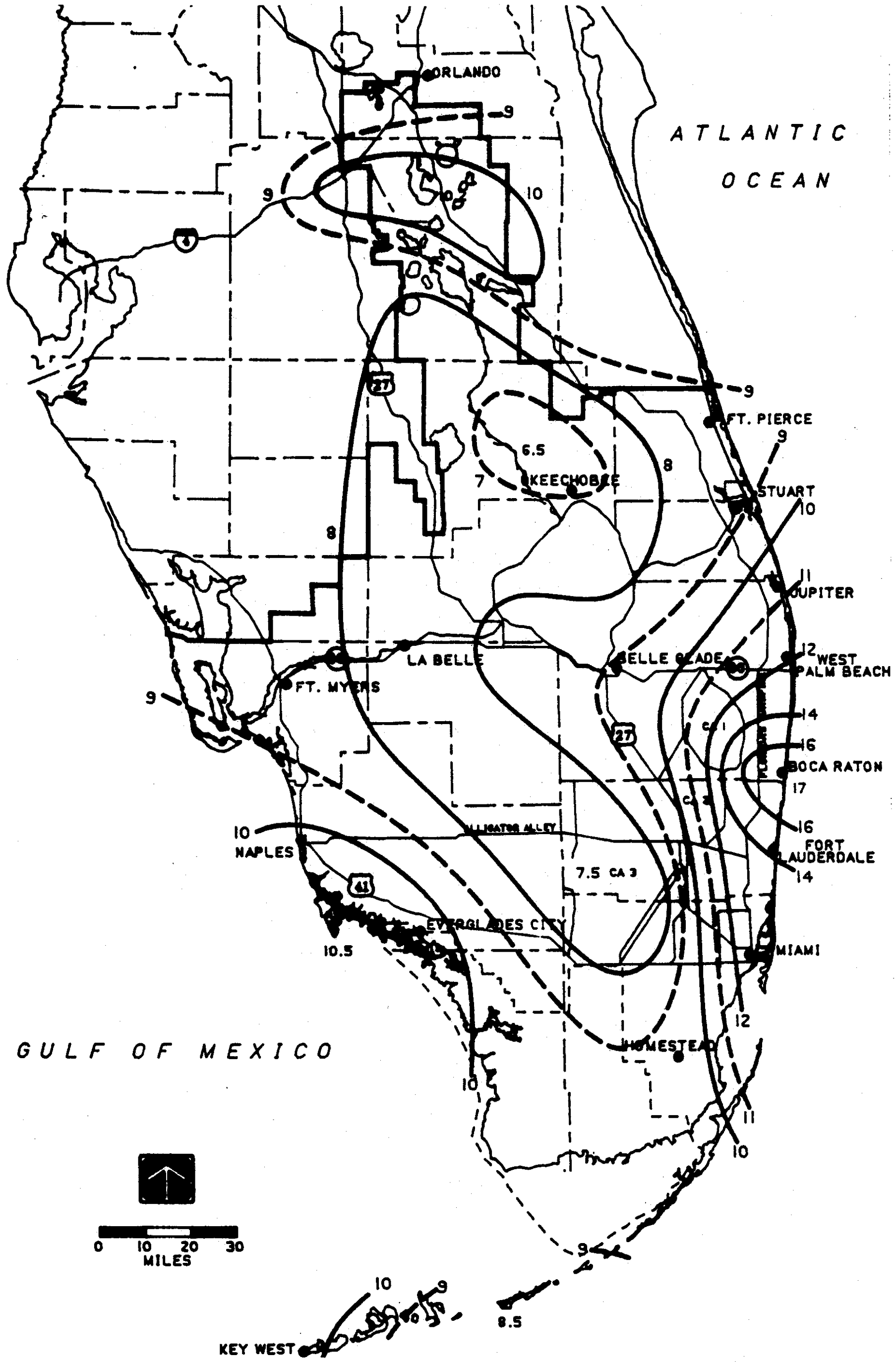


FIGURE 3-C 1-DAY RAINFALL : 50 YEAR RETURN PERIOD

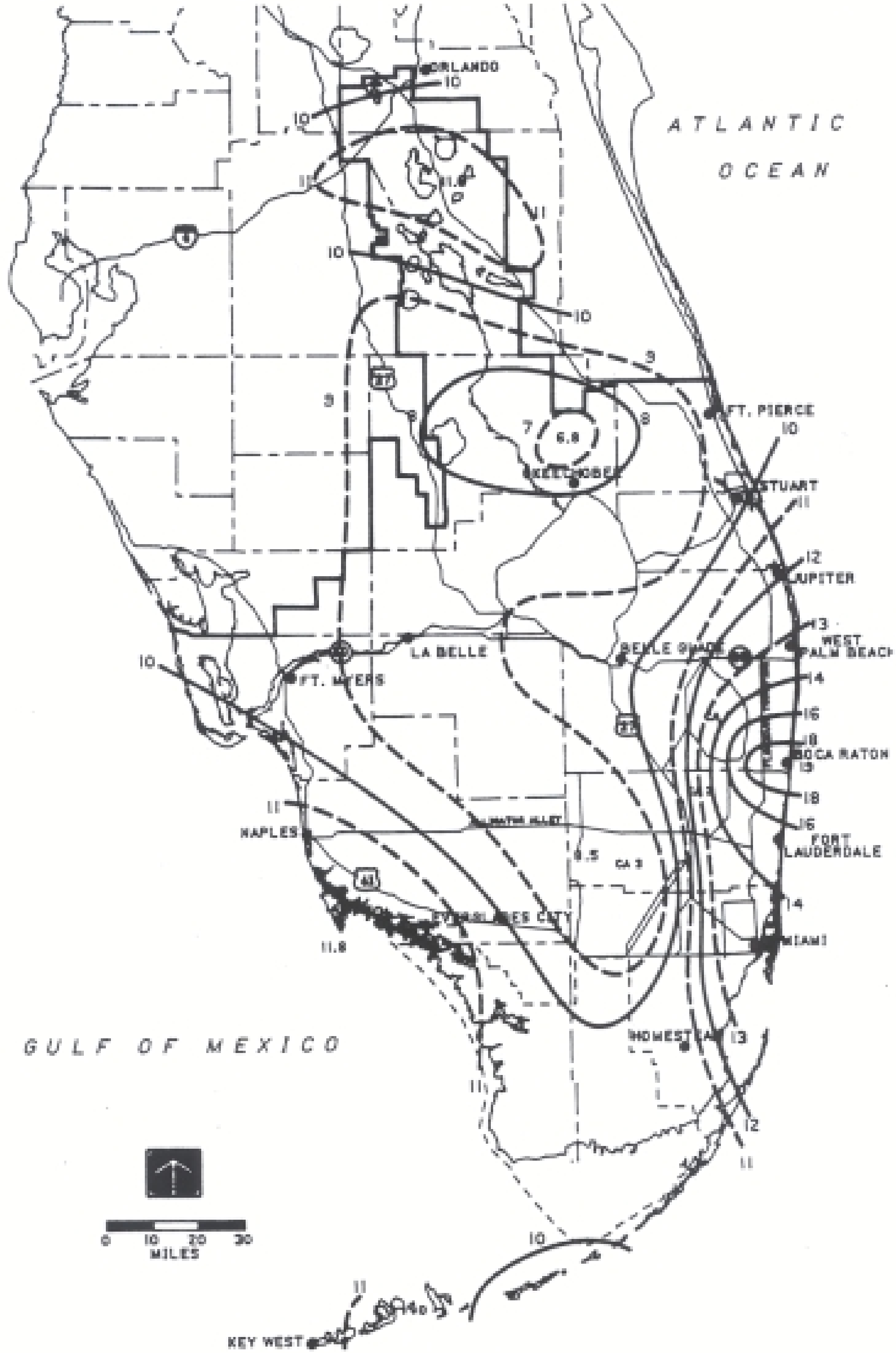


FIGURE 3-D 1-DAY RAINFALL : 100 YEAR RETURN PERIOD

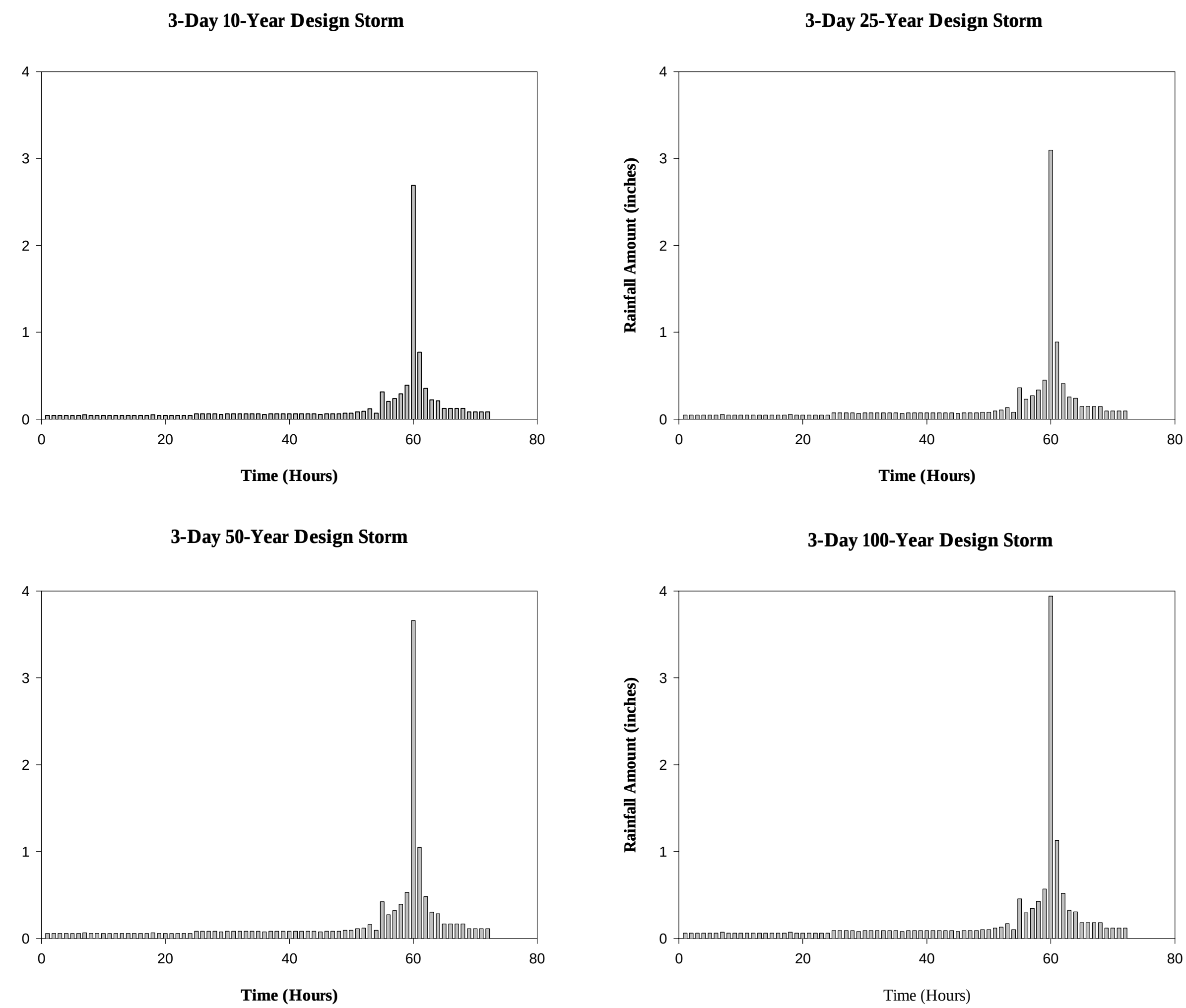


FIGURE 4. DESIGN STORM DISTRIBUTION FOR BIG CYPRESS BASIN WATERSHED

Table 1

3-Day Rainfall versus Frequency	
Frequency (years)	Rainfall (inches)
10	10
25	12
50	13
100	14

3.1.2.2 Hydrologic Model Performance

SWMM was originally developed to simulate flow in urban areas. Since its development, however, there have been numerous modification and the model has been on areas ranging from highly urban to rural (Dames & Moore, 1997). The SWMM RUNOFF Block calibrated by Dames and Moore, and refined by the BCB, was used to simulate the 3-day 25-year, 50-year and 100-year flood events under future land use condition. The SWMM model was first run to compute the approximate runoff hydrographs contribution to the canals for each storm scenarios. These inflows then become input hydrographs to the hydraulic model in order to simulate flow and stage patterns in the canals.

The evaluation of the model performance was accomplished by trial and error and by refining the geometric and hydraulic parameters of the model. Since the RUNOFF block does not include a routing mechanism, therefore the evaluation of the model performance was accomplished in concomitance with that of the hydraulic component UNET. Given a design storm, the hydrologic model is run and input into the UNET component for routing. The result of the hydrologic routing is compared to measured flow or stage data. Depending on the results the SWMM parameters are readjusted and the model is run again. This process continues until the results compare with the observed data. The main adjustments parameters are the geometric parameters on the G cards of the subbasin and the Manning's roughness coefficient. The SWMM output hydrographs for the various basins are presented in Appendix B. The pertinent SWMM parameters are summarized in *Table 2*.

Table 2

STORMWATER MANAGEMENT MODEL F FOR THE BCB WATERSHED
PRIMARY HYDROLOGIC PARAMETERS

Basin ID	Width (ft)	Area (acres)	DCIA	Slope ft/ft	imp. N	perv. N	imp. Dep. Sto	perv. Dep. Sto
30100	1.32E+04	6.32E+02	6.50E+01	2.40E-04	1.20E-02	1.50E-01	1.00E+00	2.00E+00
30200	2.03E+04	4.67E+03	9.50E+01	1.25E-04	1.20E-02	1.40E-01	1.00E-01	1.00E-01
30300	2.87E+04	8.41E+03	9.50E+01	2.50E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
30400	3.57E+04	8.87E+03	9.50E+01	1.38E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
30500	3.10E+04	7.40E+03	4.85E+00	1.46E-04	1.20E-02	2.00E-01	1.00E-01	1.50E-01
30600	1.01E+04	4.85E+03	9.50E+01	1.29E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
30700	3.16E+04	1.12E+04	9.50E+01	1.30E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
30800	2.91E+04	5.30E+03	2.00E+00	1.23E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
30900	3.62E+04	6.75E+03	2.00E+00	1.15E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
31000	1.39E+04	3.06E+03	8.77E-01	2.78E-04	1.20E-02	2.63E-01	1.00E-01	1.75E-01
31100	1.55E+04	2.44E+03	2.00E+00	3.89E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
31200	1.54E+04	2.50E+03	2.00E+00	9.77E-05	1.20E-02	3.00E-01	1.00E-01	2.00E-01
31300	1.42E+04	5.99E+03	9.50E+01	1.18E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
31400	1.61E+04	4.70E+03	9.50E+01	2.26E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
31500	7.68E+03	3.09E+03	0.0+00	2.29E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
31600	1.21E+04	5.77E+03	3.90E+01	1.30E-04	1.20E-02	1.50E-01	1.00E-01	1.00E-01
31700	9.25E+03	1.24E+03	5.00E-01	8.57E-05	1.20E-02	1.50E-01	1.00E-01	1.00E-01
31800	2.18E+04	1.08E+04	1.94E+01	1.45E-04	1.20E-02	1.40E-01	1.00E-01	1.00E-01
31900	1.80E+04	7.75E+03	1.94E+01	2.64E-04	1.20E-02	1.40E-01	1.00E-01	1.00E-01
32000	2.78E+04	1.14E+04	1.94E+01	1.72E-04	1.20E-02	1.40E-01	1.00E-01	1.00E-01
32200	1.30E+04	4.73E+03	1.94E+01	1.26E-04	1.20E-02	1.60E-01	1.00E-01	1.00E-01
32500	9.93E+03	2.33E+03	9.50E+01	1.24E-04	1.20E-02	1.00E-01	1.00E-01	1.00E-01
32600	2.33E+04	5.12E+03	9.50E+01	2.14E-04	1.20E-02	1.20E-01	1.00E-01	1.00E-01
50100	1.83E+04	1.92E+03	2.06E+00	2.26E-04	1.20E-02	2.67E-01	1.00E-01	1.94E-01
50200	1.64E+04	1.73E+03	6.68E+00	1.31E-04	1.20E-02	2.78E-01	1.00E-01	1.89E-01
50300	1.27E+04	1.05E+04	1.35E+01	1.53E-04	1.20E-02	2.61E-01	1.00E-01	3.20E-01
50400	1.49E+04	5.42E+03	5.71E+01	6.48E-05	1.20E-02	1.53E-01	1.00E-01	7.77E-01
50500	6.16E+03	1.59E+03	2.67E+01	9.20E-05	1.20E-02	2.33E-01	1.00E-01	4.58E-01
50600	1.30E+04	5.59E+03	6.50E+01	1.08E-04	1.20E-02	2.13E-01	1.00E-01	8.42E-01
50700	1.12E+04	2.84E+03	1.92E+01	6.75E-05	1.20E-02	2.72E-01	1.00E-01	3.77E-01
50800	5.87E+03	6.73E+02	5.51E+00	1.00E-04	1.20E-02	2.54E-01	1.00E-01	1.91E-01
70100	3.40E+03	4.56E+02	4.75E+01	8.57E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
70200	2.48E+04	3.00E+03	3.50E+01	1.16E-03	1.20E-02	2.00E-01	1.00E-01	1.00E-01
70300	7.33E+03	1.47E+03	4.75E+01	1.14E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
70400	2.13E+04	2.58E+03	4.00E+01	3.54E-04	8.00E-03	2.00E-01	1.00E-01	1.25E-01
70500	3.41E+04	3.48E+03	4.00E+01	3.80E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
70600	5.40E+03	6.20E+02	3.50E+00	2.00E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
70700	2.80E+04	3.48E+03	4.00E+01	2.69E-04	1.20E-02	2.50E-01	1.00E-01	1.50E-01
70800	2.40E+04	3.10E+03	4.13E+00	2.60E-04	1.20E-02	2.25E-01	1.00E-01	1.25E-01
70900	1.44E+04	2.76E+03	1.85E+01	1.20E-04	1.20E-02	2.50E-01	1.00E-01	1.50E-01
71000	1.18E+04	1.36E+03	3.50E+01	2.00E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
71100	2.83E+04	5.32E+03	1.52E+01	9.19E-05	1.20E-02	2.60E-01	1.00E-01	1.60E-01

Basin ID	Width (ft)	Area (acres)	DCIA	Slope ft/ft	imp. N	perv. N	imp. Dep. Sto	perv. Dep. Sto
71200	3.65E+04	5.24E+03	4.00E+01	2.33E-04	1.20E-02	2.35E-01	1.00E-01	1.35E-01
71300	1.68E+04	9.66E+02	2.00E+00	4.00E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
71400	9.06E+03	7.80E+02	3.50E+01	5.00E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
71500	2.87E+04	4.95E+03	1.12E+01	7.85E-05	1.20E-02	2.65E-01	1.00E-01	1.90E-01
71600	2.72E+04	4.34E+03	1.20E+00	2.54E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
71700	1.39E+04	1.86E+03	2.00E+00	1.71E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
71800	4.10E+04	4.58E+03	2.00E+00	5.28E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
71900	2.35E+04	1.35E+03	2.00E+00	5.25E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
72000	2.96E+04	2.08E+03	2.00E+00	2.78E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
72100	5.32E+04	8.81E+03	4.00E+01	2.15E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
72200	3.68E+04	4.70E+03	2.00E+00	2.25E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
72300	2.76E+04	4.05E+03	2.00E+00	2.32E-04	1.20E-02	3.00E-01	1.00E-01	2.00E-01
72400	1.77E+04	2.70E+03	1.40E+00	2.86E-04	1.20E-02	2.40E-01	1.00E-01	1.80E-01
72500	1.34E+04	2.95E+03	1.25E+00	1.02E-04	1.20E-02	2.25E-01	1.00E-01	2.00E-01
72600	6.96E+03	1.33E+03	2.00E+00	6.00E-05	1.20E-02	3.00E-01	1.00E-01	2.00E-01
80100	5.34E+03	6.64E+02	4.10E+01	1.79E-03	1.20E-02	2.00E-02	1.00E-01	1.00E-01
80200	1.49E+04	2.14E+03	3.50E+01	4.00E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80300	5.28E+03	6.56E+02	3.50E+01	1.11E-03	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80400	2.14E+03	2.46E+02	3.50E+01	1.20E-03	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80500	3.14E+03	7.81E+02	4.10E+01	4.62E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80600	6.58E+03	1.89E+03	3.50E+01	2.40E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80700	2.91E+03	8.36E+02	3.50E+01	7.20E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80800	6.34E+03	1.21E+03	4.10E+01	3.60E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
80900	1.13E+04	2.38E+03	3.50E+00	3.27E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
81000	6.98E+03	1.74E+03	3.50E+01	4.62E-04	1.20E-02	2.00E-01	1.00E-01	1.00E-01
81100	1.31E+04	8.53E+03	5.65E+01	1.41E-04	1.20E-02	1.25E-01	1.00E-01	1.00E-01
81200	4.54E+03	1.09E+03	3.50E+01	9.60E-05	1.20E-02	2.00E-01	1.00E-01	1.00E-01
81300	1.03E+04	3.36E+03	5.6375+01	7.06E-05	1.20E-02	1.63E-01	1.00E-01	1.25E-01
81400	1.73E+04	1.66E+04	5.40E+01	4.80E-05	1.20E-02	1.75E-01	1.00E-01	7.46E-01
81500	1.72E+04	4.93E+03	5.55E+00	3.60E-04	1.20E-02	2.40E-01	1.00E-01	2.36E-01
81600	2.12E+04	1.01E+04	2.21E+01	5.35E-04	1.20E-02	1.59E-01	1.00E-01	3.79E-01
81700	1.70E+04	5.52E+03	6.07E+01	3.53E-05	1.20E-02	2.01E-01	1.00E-01	8.07E-01
81800	1.06E+04	6.89E+03	1.66E+01	3.88E-04	1.20E-02	1.56E-01	1.00E-01	2.95E-01
81900	1.07E+04	3.29E+03	2.15E+01	4.69E-04	1.20E-02	1.56E-01	1.00E-01	3.66E-01
82000	1.03E+04	5.72E+03	6.36E+01	2.07E-05	1.20E-02	1.95E-01	1.00E-01	8.29E-01
82100	1.34E+04	4.30E+03	9.42E+00	4.19E-04	1.20E-02	2.30E-01	1.00E-01	2.57E-01
82200	1.10E+04	6.53E+03	8.24E+00	3.88E-04	1.20E-02	1.56E-01	1.00E-01	2.33E-01
82300	3.51E+04	1.68E+04	9.12E+00	8.93E-04	1.20E-02	1.58E-01	1.00E-01	2.18E-01
82400	1.80E+04	1.03E+04	2.14E+01	2.93E-04	1.20E-02	1.93E-01	1.00E-01	3.79E-01
82500	7.89E+04	1.81E+03	7.31E+01	1.00E-04	1.20E-02	2.07E-01	1.00E-01	9.37E-01
82600	1.48E+04	7.37E+03	1.78E+01	6.25E-04	1.20E-02	1.75E-01	1.00E-01	1.25E-01

3.2 Hydraulic Submodel

3.2.1 Modeling Criteria – Steady State versus Unsteady State Flow Simulation

Consideration of the variation of flow characteristics with time is the criterion for making the distinction between steady flow and unsteady flow. If depth, velocity, and discharge remain constant with time at a particular location on a stream, the flow is steady, if any of these characteristics vary, it is unsteady. The passage of a flood wave through a reach of river is an example of unsteady flow because the depth, velocity, and discharge are all changing with time.

Most real flows are unsteady; but most flow analyses assume steady flow. Practically, many real flows can be approximated by steady state flow. This is true in those cases where the goal is to maintain essentially steady flow for long periods of time such as irrigation canals. However, the principle reasons for the predominance of steady-flow analysis is that it is simple to understand and it requires less computational power. In a typical floodplain H&H study, the peak discharges used for computing water surface profiles are obtained from hydrographs produced by a basin rainfall-runoff model or from frequency analysis. The peak discharge is often used regardless of the fact that it represents an unsteady-flow condition. The widely used HEC-2 model justifies its steady flow assumption by the fact that flood waves rise and fall gradually. An observer standing on the bank of a stream watching a flood wave pass would merely see evidence of the rise and fall of the water surface. As long as the change in flow occurs gradually, reasonable results can generally be obtained with steady-flow analytic methods.

There are at least four cases in which a steady-flow model may not provide adequate results for flow simulation, and the unsteady state analyses should be used in the BCB interconnected canal system :

1. Abrupt variation flow. In the case of the control structure from close to fully open in a finite time, the time–dependent term of the complete unsteady-flow has a significant effect, the steady state model can not simulate this scenario.
2. Backwater effects - As a part of the BCB interconnected water management canal system, there are another four canals directly connect to Cocohatchee Canal, as illustrated in Figure 2, the flow in Cocohatchee Canal has significant backwater effects and steady state analysis can not simulate this scenario.
3. Looping effects - In the Cocohatchee Canal, there is a pronounced "loop effect" in the relationship between discharge and elevation resulting from a flat channel slope. The loop is due primarily to the difference between water surface slopes associated with the rising and falling sides of the flood wave as it passes a point. Steady state analysis can not simulate this "loop effects".
4. Storage effects - The steady-flow analysis can only evaluate the effects of changes in the ability of a canal to convey water and does not reflect the effect of changes in the ability of the canal to store water. The effect of change in storage is a primary consideration in the water management

strategies to be evaluated in the BCB watershed management plan development.

3.2.2 Model Description

The unsteady state hydraulic analysis model UNET simulates one-dimensional unsteady flow through a full network of open channels. One basic element of a full network problem which the model can simulate is that of split flow into two or more channels. For subcritical flow, the division of flow depends on the stages in each of the receiving channels. These stages are a function of channel geometry and downstream backwater effects.

A second basic element of a full network problem is the combination of flow, termed the dendritic problem. This is considered to be a simpler problem than the flow split, because flow in each tributary is dependent only on the stage in the receiving stream. The full network is the most general problem. It includes single channels, dendritic systems, and fully looped systems (another commonly used term for the full network). The system may include flow bifurcations, crossing canals, four-node junctions, and storage areas.

Another facet of the full network model is storage areas; lake-like regions that can either provide water to, or divert water from, a channel. This is a split flow problem, although in this case, the storage area water surface elevation will control the volume of water diverted. Storage areas can be the upstream or downstream boundaries for a river reach. In addition, the river can overflow laterally into the storage areas over a gated spillway, weir, levee, through a culvert, or a pumped diversion.

In addition to solving the one-dimensional unsteady flow equations in a network system, UNET provides the user with the ability to apply several external and internal boundary conditions, including flow and stage hydrographs, gated and uncontrolled spillways, bridges, culverts, and levee systems. All input, output and calculations are performed in U.S. Foot-Pound Units.

To facilitate model application, cross sections are input in a modified HEC-2 forewater format (upstream to downstream). Over the past decade, a large number of BCB channels have been modeled using HEC-2 backwater analysis. Those data files can be adapted to UNET format. Boundary conditions for UNET can be input from any existing HEC-DSS (HEC, 1990b) data base. For most problems, particularly those with large numbers of hydrographs and hydrograph ordinates, HEC-DSS is advantageous because it eliminates the tabular input of hydrographs and creates an input file that can be easily adapted to a large number of scenarios. Hydrographs and profiles that are computed by UNET are output to HEC-DSS for graphical display and for comparison with observed data. Guidance for numerical modeling of river hydraulics is given in River Hydraulics (USACOE, 1993).

Due to the complexity and inherent model instabilities encountered in numerical solution of unsteady flow analysis, the BCB canal network was formulated in four models which, as in the physical system, acted independently of each other. The models were referred to as:

COFA - Cocohatchee, Faka Union, and Golden Gate Canal system connected via the Corkscrew/Curry Canals

HEN - Henderson Creek

BAR - Okaloacoochee-Barron River Canal system

GORDON - Gordon River.

COFA is by far the most complex of the BCB models with its system of interacting watersheds, canals, and water control structures with stop-logs, gates, and an overland runoff overflow to the Imperial River and Fakahatchee Strand Basins. This model has more than 1,400 cross sections and 50 reaches.

The COFA model was configured to allow multidirectional flow, cross flow between basins, and simulation of looped networks. SWMM computed flows served as input to UNET via the HEC-DSS program. Reaches were used to describe uninterrupted flow path from an upstream boundary or connecting reach to a downstream boundary or connecting reach. Within each defined UNET reach, cross-section geometry for canals was defined by modifying previously developed HEC-2 data input. For sloughs and strands, cross-sections were generated from one-foot contour maps developed from aerial photographic surveys and, in some cases, interpolated from USGS topographic maps with spot survey information. Data on structures were gleaned from the available HEC-2 input decks, or from design records. Operable gates were modeled as elevation controlled boundary conditions, specifying closed-gate and open-gate water surface elevations and orifice dimensions along with a gate opening rate.

3.2.3 Detailed Routing Configuration

The Big Cypress Basin routing configuration has been developed using three primary sources: The Big Cypress Basin database archives and staff knowledge on long-term operation of the canal system, the Collier County Stormwater Management Program Basin Hydrologic and Hydraulic Assessments, Collier County Stormwater Management

Atlas Maps, and a variety of individual hydrologic and hydraulic assessments conducted by others within the watershed boundaries. The UNET model has been configured to allow multidirectional flow, cross flow between basins, and simulation of looped networks. Due to the size of the watershed and the network complexity, four separate models were developed to allow more manageable sized models, and reduce the required run times. *Figure 5* represents the UNET model schematics for the BCB hydraulic network. Information on subbasin outline, hydrograph input, canals and overland flow reaches, and water control structures is elaborated in *Figure 5*. Development of the hydraulic model is described in detail below.

Figure 5

3.2.4 Hydraulic Model Setup

The construction of the hydraulic model requires input of data related to model linkages, as described above, detailed channel and floodplain cross-section, hydraulic structure configurations, hydrograph input locations, initial modeling conditions, upstream and downstream boundary conditions, and internal boundary conditions. Methods utilized for the development of these data for the Big Cypress Basin model are described in detail below.

3.2.4.1 CSECT

The primary function of the UNET's CSECT program module is to establish a routing connectivity (interior boundary conditions), define cross section geometry, and compute appropriate free and submerged flow rating curves for user-defined constrictions such as culverts, bridges and weir structures. The data input structure closely resembles that of HEC-2 forewater analysis files, with data format fixed and right justified. Alphanumeric record identification characters occupy columns 1 through 3, followed by 10 fields of data occupying eight columns each (with the exception of Field 1 which occupies only five columns). Channel and structure data developed by others for HEC-2 modeling of various portions of the Big Cypress Basin canal network, have been transcribed into CSECT records for the UNET modeling effort. Record development is outlined below.

3.2.4.2 UNET Boundary Conditions

The UNET Boundary Condition (*.BC) file contains information on the time step to be used in the model and other job control parameters, identification of DSS hydrographs to be read, initial conditions, upstream and downstream boundary conditions, and internal boundary conditions. Initially the entire system was set up as one model.

The model time step varied from 30 minutes for BAR and GORDON models to one minute for the most complex of the models, COFA.

3.2.4.3 Initial Conditions

Initial Conditions development consisted of assigning initial flows for each reach. For the simpler models initial conditions were assumed, and then refined by trial and error. If the simulating computations did not converge, then the initial conditions were modified until there were no initial condition errors and there was obvious continuity in the reach connections. The approach to initial conditions for the complex COFA model was, however, different. High downstream boundary condition elevations were input to the model so that all the reaches would initially flood. Concurrently, an assumed minimal flow rate of one cfs was assigned to all reaches. The model was then run for 90 hours to generate an initial conditions file. The initial conditions file served as the basis for a new model run with a reasonable lower downstream boundary condition. The process was repeated with incremental reductions of downstream boundary stages and a final initial condition file saved.

3.2.4.4 Upstream and Downstream Boundary Conditions

Upstream boundary conditions were hydrographs read from a DSS input file. The upstream boundary hydrographs were input at reaches where there were no upstream connections to other reaches. These upstream points as well as lateral inflow points are shown on *Figure 5*.

Downstream boundaries were typically defined as constant head boundaries at an elevation near mean sea level.

3.2.5 Evaluation Hydraulic Performance of the Canals

The calibrated unsteady-state hydraulic model was used to simulate the flow conveyance characteristics existing network of canals and several natural sloughs throughout the BCB watershed. Problems were often encountered in the application of UNET to such a large complex basin. Although the model was set up within the dimensions of the software, numerous numerical solution stability problems were encountered. The cross sectional data provided indicate that the canal bottoms are very irregular in some areas. Many canals were not dug according to the configurations proposed in the GAC plans. Shoaling and aquatic weed bio-mass deposition over the years have also contributed toward elevating the canal beds. Even though the data is believed to be accurate, such highly irregular profiles are typically difficult to model. The large network of interconnections including canals and wetlands also provided challenges. Problems were overcome by selecting small time steps and performing multiple runs within a one year period. The COFA model was exceptionally difficult, whereas the simple one-reach model of Gordon River was easier to simulate. The complex COFA model was run in 2-week increments, with the final conditions at the end of each run saved as a hot start file for the initial conditions of a subsequent run.

The manually operated adjustable gates were modeled as automated elevation controlled gates because records of their daily operation were not available in all cases. Streamflow records suggest that the modeled operation assumptions were not always representative of actual operation. The model would greatly benefit from the collection of reliable gate operation data specifying daily gate position.

The water surface profiles resulting from the evaluation of the hydraulic model for the various canals are presented in Appendix D.

The COFA model was configured to allow multidirectional flow, cross flow between basins, and simulation of looped networks in the following canals:

1. Cocohatchee Canal with natural sloughs,
2. Golden Gate Main Canal,
3. Corkscrew Canal,
4. Curry Canal,
5. Orange Tree Canal,
6. Cypress Canal,
7. Green Canal,
8. Harvey Canal,
9. I-75 Canal,
10. Airport-Road Canal, and
11. CR 951 Canal.

The HEN segment of the BCB Hydraulic Network Model includes the main stem of the Henderson Creek. The modeled cross-sections of this canal do not include the recent modifications of the canal implemented by the Forest Glenn of Naples Development. The modified cross-sections will be incorporated into the next update of the model.

The BAR segment of the BCB Hydraulic Network Model includes the natural sloughs of the Okaloacoochee Slough, and approximately 21 miles of the Barron River Canal. The development of the water surface profiles of this system is not complete at the time of preparation of this report. The GORDON segment of the BCB Hydraulic Network Model was framed using cross-section data surveyed in the early 1980s. A detailed hydraulic model of the Gordon River with newly surveyed cross-section data is being developed as a part of the on-going Gordon River Extension Basin Study performed under a three-party agreement between Collier County, City of Naples, and the Big Cypress Basin.

SWMM computed flows served as input to UNET via the HEC-DSS program. Reaches were used to describe uninterrupted flow path from an upstream boundary or connecting reach to a downstream boundary or connecting reach. Within each defined UNET reach, cross-section geometry for canals was defined by modifying previously developed HEC-2 data input. For sloughs and strands, cross-sections were generated from one-foot contour maps developed from aerial photographic surveys and in some cases, by interpolation from USGS topographic contour maps supplemented with spot elevation surveys. Data on structures were gleaned from the available HEC-2 input decks, or from design records. Operable gates were modeled as elevation controlled boundary conditions, specifying closed-gate and open-gate water surface elevations and orifice dimensions along with a gate opening rate.

Hydraulic performance of the canals simulated in the COFA model under level-of-service standard storm (25-year, 3-day) conditions indicate severe inadequacy of design conveyance in many segments of the canals. A summary of the hydraulically deficient segments is illustrated in Table 3. The impact of rapid land use changes to urban built-up condition is conspicuously evident in the Golden Gate Main Canal, Henderson Creek Canal, I-75 Canal, and Cypress Canal. Of particular importance is the Golden Gate Main Canal where the entire segment of the canal west of CR 951 does not meet the level-of-service standard for a design 25-year flood.

The hydraulic capacity of the Cocohatchee Canal has been substantially enhanced by the three phases of channel modification. Three small segments of this canal east of the Florida Rock Quarry are still observed to be deficient in conveyance capacity and are proposed to be improved by the fourth phase of channel modification in FY 2000.

Many sections of the so-called rural canals, namely Miller, Faka Union, Merritt, and Prairie Canals in Southern Golden Gate Estates are also inadequate to provide protection against flooding even from a 10-year storm. These deficiencies are the combined results of inadequate channel cross-section, undersized culverts and bridges, and in several instances, encroachment to the natural flowways by roads and other land-use developments.

Table 3

Hydraulically Deficient Segments of Big Cypress Basin Primary Canals to Meet 25-Year Peak Flood Protection		
Canal	**Stationing (Approximate)**	**Description**
Golden Gate Main Canal	62,500 ~ 65,000	Between Golden Gate Boulevard and 5th Street NW
	69,000 ~ 80,000	Between 5th Street NW and White Boulevard
	86,300 ~ 135,000	Upstream of Golden Gate Weir No. 3 to Golden Gate Weir No. 1
Corkscrew Canal	20,500 ~ 32,500	Between 43rd Avenue NE and CR 846
CR 951 Canal	3,000 ~ 16,000	Between CR 846 and Golden Gate Boulevard
I-75 Canal	23,000 ~ 40,000	Pine Ridge Road to Golden Gate Main Canal
Cypress Canal	34,700 ~ 41,000	Golden Gate Boulevard to Golden Gate Main Canal
Henderson Creek	44,800 ~ 82,000	Entire Canal South of Forest Glenn of Naples
Miller Canal	21,500 ~ 48,000	Stewart Boulevard to T
	95,000 ~ 110,000	Between 28th St. SE to Miller #3
Faka Union Canal	0 ~ 30,000	Between Faka #1 to Stewart Blvd
	44,000 ~ 62,000	Between Faka #2 to Faka #3
	96,000 ~ 101,000	Between Golden Gate Blvd to Faka #4
	144,500 ~ 149,500	Upstream of Faka #7
Merritt Canal	13,000 ~ 21,000	Junction with Prairie to T

3.3 Summary Observation of Hydraulic Performance

Hydrologic-hydraulic assessment of the western Big Cypress Basin has been performed utilizing a set of unsteady state H&H models developed for formulating a comprehensive watershed management plan for the region. The primary canal network operated by the Big Cypress Basin and several natural sloughs were simulated to evaluate their hydraulic capacities to convey peak runoff generated by 10-, 25-, 50- and 100-year storms.

Assessment of the simulated existing condition of the watershed indicates that the canals largely control the average hydrology of the watershed. The canals also intercept shallow groundwater outflow, and have continually lowered the water table. The characteristics of the rainfall-runoff response continue to change significantly as land use changes occur. Flash flooding has been a common occurrence in many urbanizing areas in the Cocohatchee, Golden Gate, and Henderson Creek Basins. Of particular importance is the urban segment of the Golden Gate Main Canal west of CR 951 where the unabated pace of growth in the impervious area of this canal basin has resulted in drastic reduction of times of concentration and consequent flash flooding. Most of the Golden Gate Canal System was designed to convey a 10-year recurrence interval peak storm runoff. With rapid urban growth and the level-of-service requirement for protection from a 25-year storm runoff for the urban areas, it is imperative that the conveyance capacity of many of the primary canals be enhanced in the very near future.

4 ALTERNATIVE DEVELOPMENT

4.1 Criteria

The hydrologic-hydraulic assessment of the simulated existing condition of the watershed and the primary canal network and water control structures indicate that approximately 47.6 miles out of 163 miles of canals presently operated and maintained by the Big Cypress Basin, or 29 percent of the network, does not meet the desired level of service criteria for flood control. The rapid urbanization and resulting increase in impervious areas in most of the primary canal basins have continued to result in flash flooding. It is, therefore, imperative that the improvement of the hydraulic conveyance capacities of the canal segments that do not meet the LOS standards should be a priority element of the watershed plan.

In formulating alternative water management strategies to achieve the desired conveyance capacity of the hydraulically deficient canal segments were defined by the following criteria:

- The maximum flood elevations exceed the canal bank elevation.
- The maximum flood elevation is within 1.5 feet of the canal bank elevation.
- Water velocities exceed 2.5 feet per second, creating a potential for erosion.
- Structures within the canal do not meet District right-of-way criteria.

In addition to the objectives of flood prevention, the alternatives are to be chosen to achieve the following goals of the watershed management plan.

- Maintain or improve existing levels of flood protection in the developed and developing areas consistent with Collier County Comprehensive Plan.
- Restore historic surface water flow characteristics on conservation and public lands.
- Improve water retention and aquifer recharge potential.
- Reduce threats of saltwater intrusion.
- Reduce excessive freshwater discharge impacts on downstream estuaries.
- Provide basis for off-site mitigation opportunities.
- Enhance natural system functions and values on publicly owned and conservation lands.

Within the purview of this task, however, the alternatives are developed solely geared toward maintenance or improvement of the flood protection in the designated urban area of the watershed. Another range of alternatives, in addition, but not limited to the flood control alternatives will also be considered to meet the realm of the above goals. The thrust of the alternatives will concentrate on restoring the predevelopment flowways or their equivalent for enhancing the water supply and environmental potential of the region, specifically rerouting flows from the 'historic high' regions of the north, like the Corkscrew Regional Ecosystem Watershed to the southern portions of the region, namely the Fakahatchee, Southern Golden Gate Estates, and Belle Meade basins. The key assumption in developing such alternatives is that the lands in the above-referenced systems are eventually expected to be in public ownership.

4.2 Alternatives

The alternatives for flood protection will concentrate on the feasibility of channel modification, flow diversion, and storage by primarily considering restoration of the predevelopment flowways, or their equivalents, to improve flow conveyance while at the same time, enhancing potentials for water supply and ecologic function.

The following predominant alternatives were identified for assessment following preliminary screening:

1. Do nothing.
2. Diversion of a portion of Golden Gate Canal flows to the Henderson Creek basin.
3. Channel and structural modifications of the Golden Gate Main Canal west of CR 951.
4. Channel modification of CR 951 Canal and implementation of water control measures.
5. Diversion of a portion of Corkscrew Canal flows eastward to Golden Gate Main Canal north of Golden Gate Canal Weir No. 5
6. Cocohatchee basin east and west flowways.
7. Aquifer Storage and Recovery (ASR) of wet season flows of the Golden Gate Main Canal near CR 951.
8. Modification of the C-1 Connector Canal and relocation of Miller Canal Weir No. 3.
9. Improve Henderson Creek Canal and undersized crossings.
10. Restore Camp Keais Strand flowway.

Alternative Water Management Strategies
BCB Watershed Management Plan
Flood Control Element

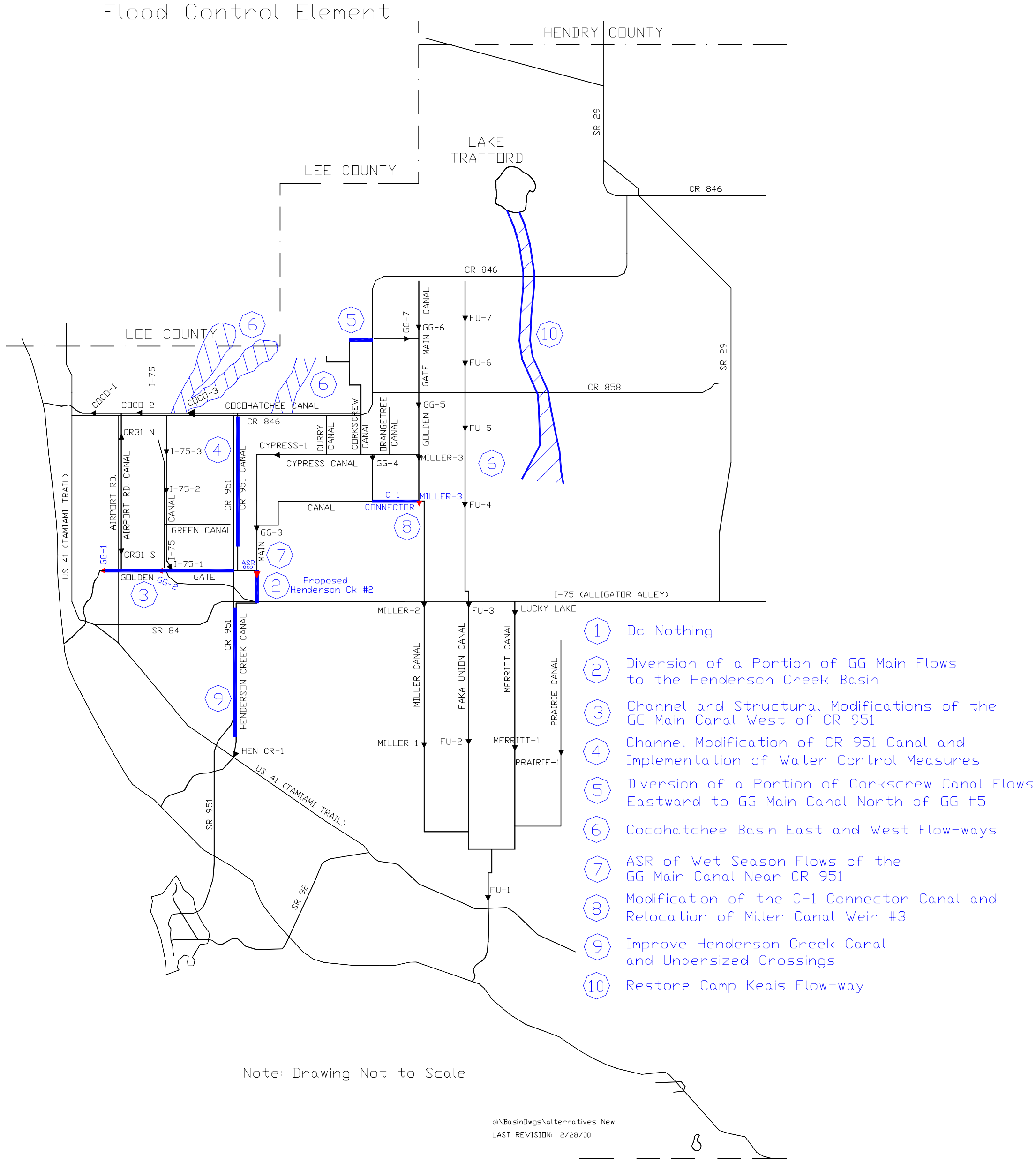


Figure 4-1

Several combinations and subsets of the above alternatives will be evaluated in terms of their hydraulic efficiency and economy of cost in achieving the desired performance measures for flood protection, primarily for the designated urban areas west of a north-south line one mile ease of CR 951. The alternatives for flood protection of the deficient reaches of the BCB primary canals in the rural areas are anticipated to be not only limited to channel modification, flow diversion and storage, but will also incorporate the restoration of the historic flowways and elements of the on-going and proposed ecosystem restoration projects in the Basin. The effectiveness of those alternatives will be evaluated by the application of an integrated surface and groundwater model.

5 ALTERNATIVE EVALUATION

5.1 Hydraulic Performance of Alternatives

The performance of the alternatives identified following preliminary screening were analyzed by application of the set of the calibrated hydraulic models. To evaluate the response of the realm of scenarios considered under a particular alternative, a steady-state analysis of design storm flood routing was performed interactively first by application of the Corps of Engineers' flood analysis program HECRAS. Once the desired performance of the water management strategy in achieving the levels of reduction in flood stages were perceived, dynamic routing was performed by application of the unsteady-state model UNET to observe the attenuation of floodwave and its impact on the entire interconnected canal network. The results of the hydraulic assessments of the alternatives are presented below.

5.1.1 Alternative 1: Do Nothing

In this hydraulic assessment scenario, a projected 2020 land use is applied with no changes in the infrastructure except the on-going channel and structural modification in the Cocohatchee Canal basin. Under this scenario, the rapid pace of urban development in the watershed and associated increase in impervious areas will continue to cause flash flooding, particularly in the urban corridor adjacent to Golden Gate Main and Henderson Creek Canals. Within the purview of the responsibilities assigned to the Basin for safeguarding human health and welfare, the "Do Nothing" alternative is not acceptable.

5.1.2 Alternative 2: Diversion of a Portion of Golden Gate Canal Flows to the Henderson Creek Basin

The historic flowways of the Henderson Creek have been disrupted by nearly 50 years of road and drainage development. Some of these flows have been intercepted by the Golden Gate Canal and others have been disrupted, leading to flooding in low lying areas encroached by development and adverse environment impact to the estuaries of Naples Bay and Rookery Bay. One of the key objectives of the Big Cypress Basin Watershed Management Plan is to restore this important flowways to reduce flooding and minimize adverse impacts to estuaries.

The BCB Watershed Management Plan has considered implementation of a diversion canal to connect Golden Gate Main Canal and Henderson Creek across I-75, as illustrated in Figure 4-1. The objective of this alternative is to divert a portion of the peak flows from the Golden Gate Canal to the Henderson Creek Canal, to restore the historic flowways of the Henderson Creek Basin, and to reduce flooding along the urbanized areas of Golden Gate Main Canal. It will additionally reduce voluminous freshwater shock load to the Naples Bay Estuary.

The United States Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM) program is used to simulate the hydrologic characteristic. The US Army Corps of Engineers' Unsteady State hydraulic network model UNET is applied to simulate flows in the connected dendritic system of Golden Gate Main Canal and Henderson Creek as shown in Figure 5.1.2-1. The existing regional COFA model results are used as input boundary condition for this alternative modeling study.

The hydraulic design criteria includes a comparison of different configurations of canal cross section and weir type and raising of the berms to keep the flood stage below the canal banks with a free board of 1.5 feet, and a non-scouring flow velocity not great than 2.5 feet per second.

An approximately 1-mile long diversion canal needs to be excavated to connect Golden Gate Main Canal with Henderson Creek Canal. The proposed canal geometry has a bottom width of 30 feet, a side slope 2:1 and a longitudinal slope of 0.5 feet per mile, with an inlet elevation is 1.5 feet NGVD at the confluence with Golden Gate Main Canal, and 1.0 feet NGVD at the South of the I-75. A water control structure of 70 feet long fixed crest weir with crest elevation 9.0 feet NGVD is proposed at the beginning of the diversion canal so that the flow in the Golden Gate Canal can be diverted to Henderson Creek only at the extreme high stage condition.

The hydraulic performance of the alternative was simulated for the design 25-year 3-day storm event; as well as for the 10-year, 50-year and 100-year rainfall scenarios. For the existing Golden Gate Canal and Henderson Creek Canal system and with the diversion canal implementation, the simulated stage and flow conditions at inlet of the diversion canal (upstream of the proposed structure Weir No. 2) and at the I-75 culvert are presented in Figures 5.1.2-2 through 4. The simulated water surface profiles of Golden Gate Main Canal and Henderson Creek Canal are illustrated in Figures 5.1.2-5 through 12. Table 5.1.2-1 summarizes the simulated maximum flow rate can be diverted from Golden Gate Main Canal to the Henderson Creek.

Table 5.1.2-1: Simulated Maximum Flow From Golden Gate Main Canal to Henderson Creek Through Diversion Canal

Storm Event	Maximum Flow Diverted Golden Gate Canal to Henderson Creek
10-year	100
25-year	122
50-year	171
100-year	213

The modeling results indicate that in order for the 1 mile diversion canal to convey the 25-year, 3-day storm event without overtopping canal bank, 1.5 feet high berm is needed for both sides of the bank. The estimated cost for this alternative is summarized in Table 5.1.2-2.

Table 5.1.2-2: Preliminary Cost Estimate of Alternative 2

Volume of Excavation:	91,000 cubic yards	$1,092,000
Volume of Berm:	5,280 cubic yards	
Fixed Crest Weir:	70 feet long	$500,000
Total Cost:		**$1,592,000**

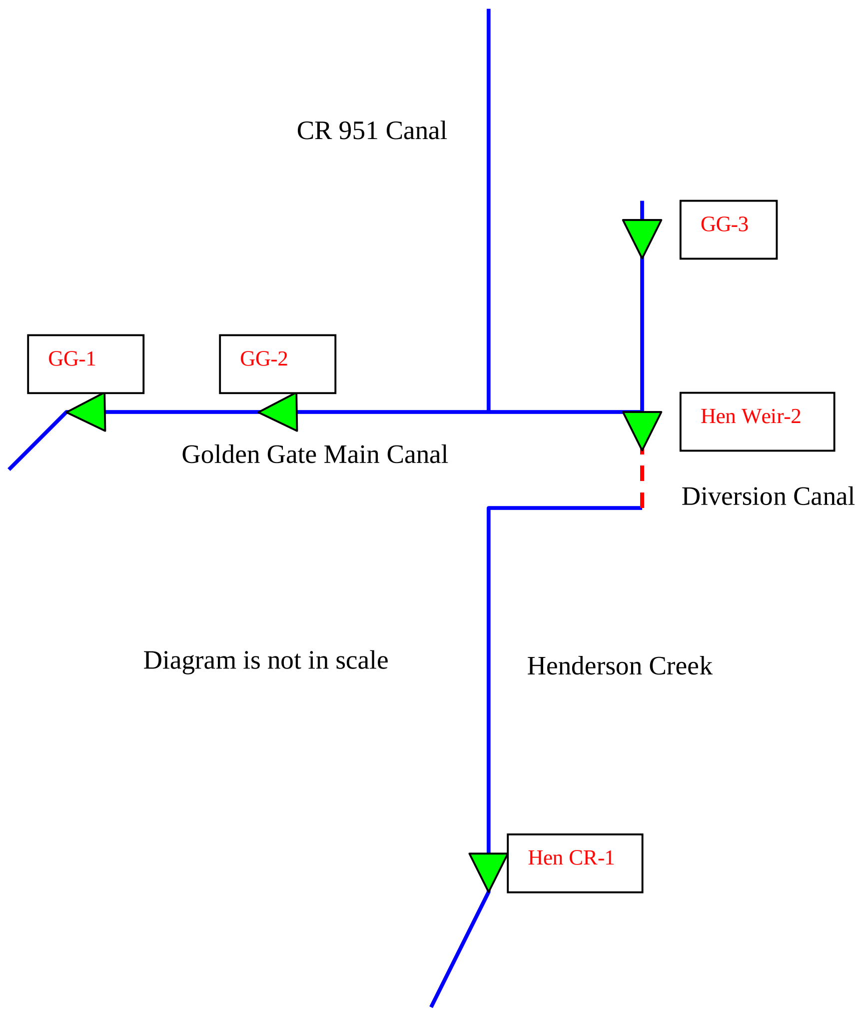


Figure 5.1.2-1. Unet Model Schematic for Golden Gate Main Canal and Henderson Creek Division Plan

Figure 5.1.2 -2

SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 10-YEAR DESIGN STORM

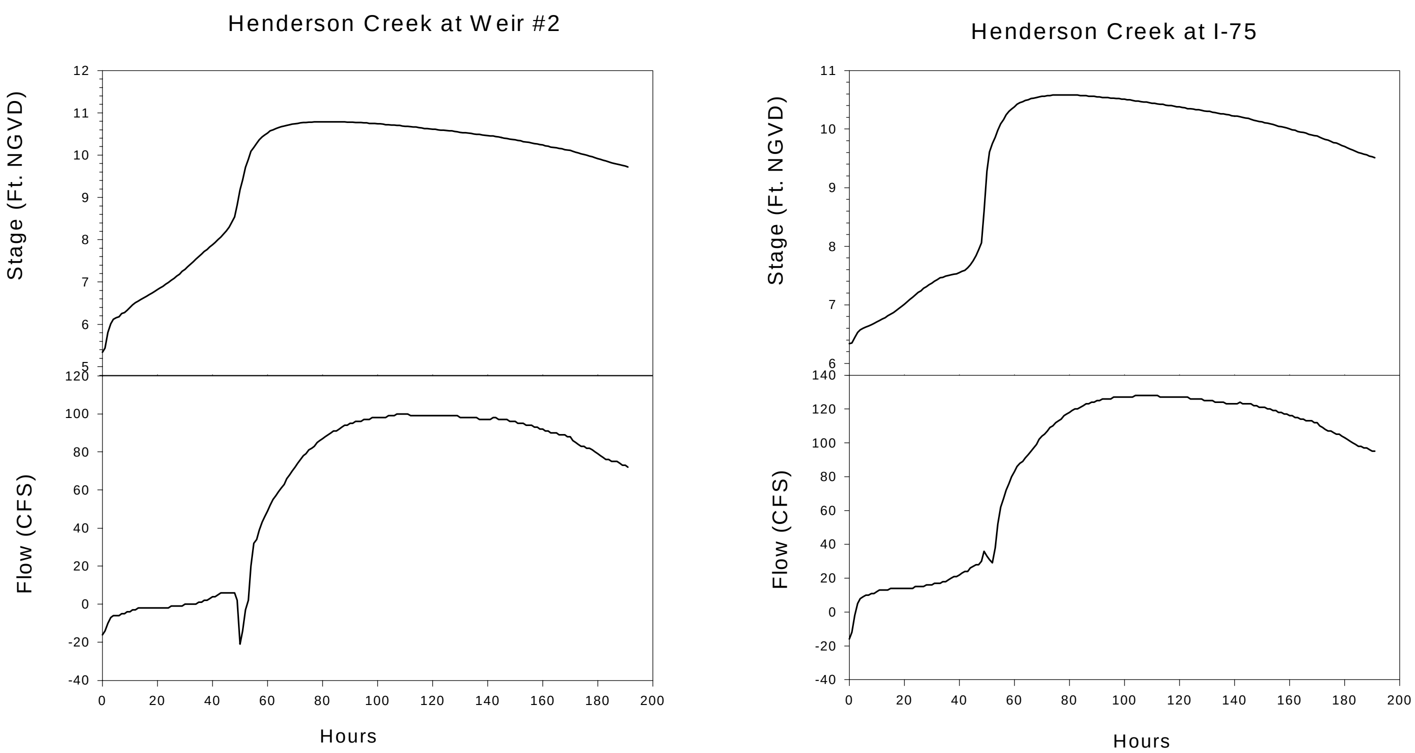


Figure 5.1.2 -3

SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 25-YEAR DESIGN STORM

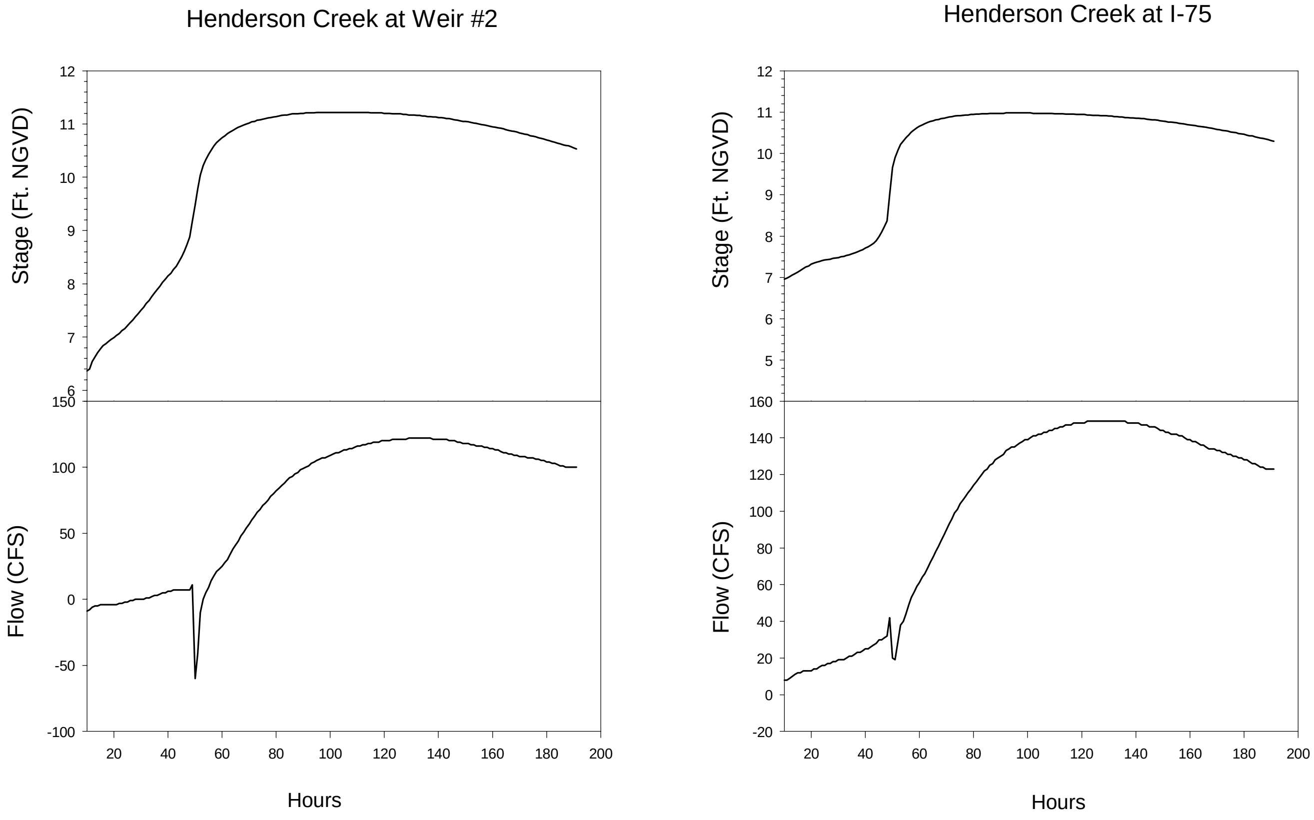


Figure 5.1.2 - 4

SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 50-YEAR DESIGN STORM

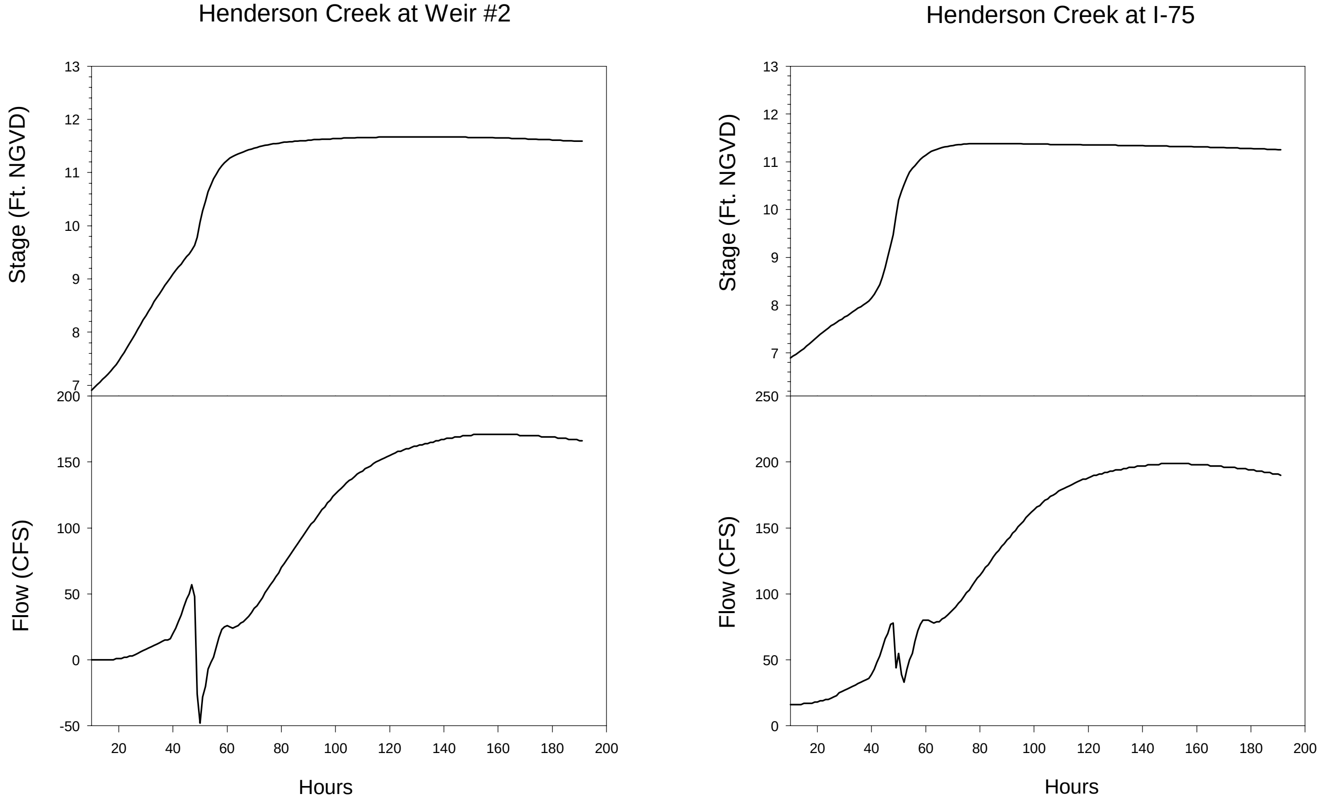


Figure 5.1.2 - 5

Alternative 2

Dynamically Simulated Water Surface Profile for

Golden Gate Main Canal

3-Day 10-Year Design Storm

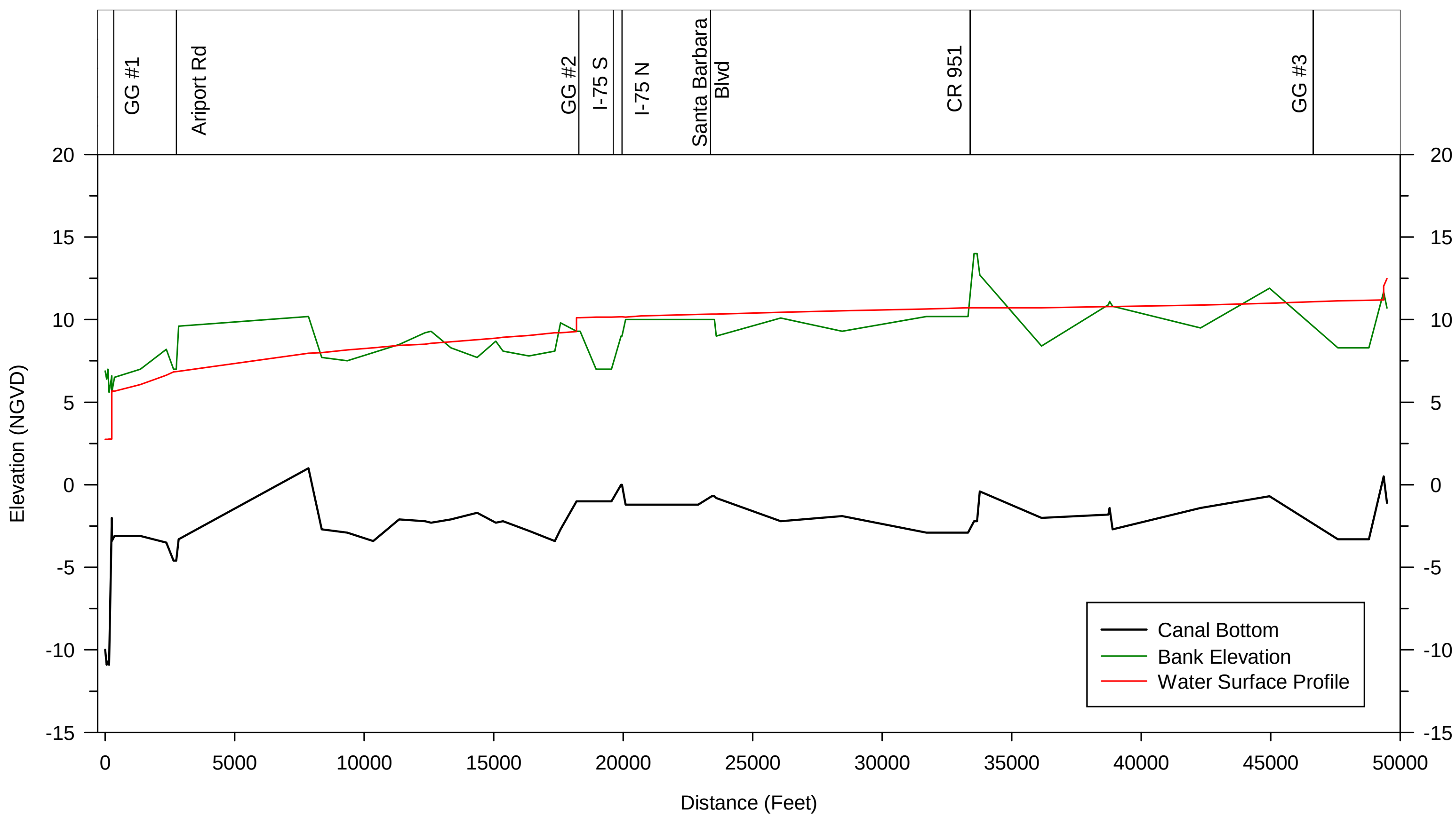


Figure 5.1.2 - 6

Alternative 2
Dynamically Simulated Water Surface Profile for
Golden Gate Main Canal
3-Day 25-Year Design Storm

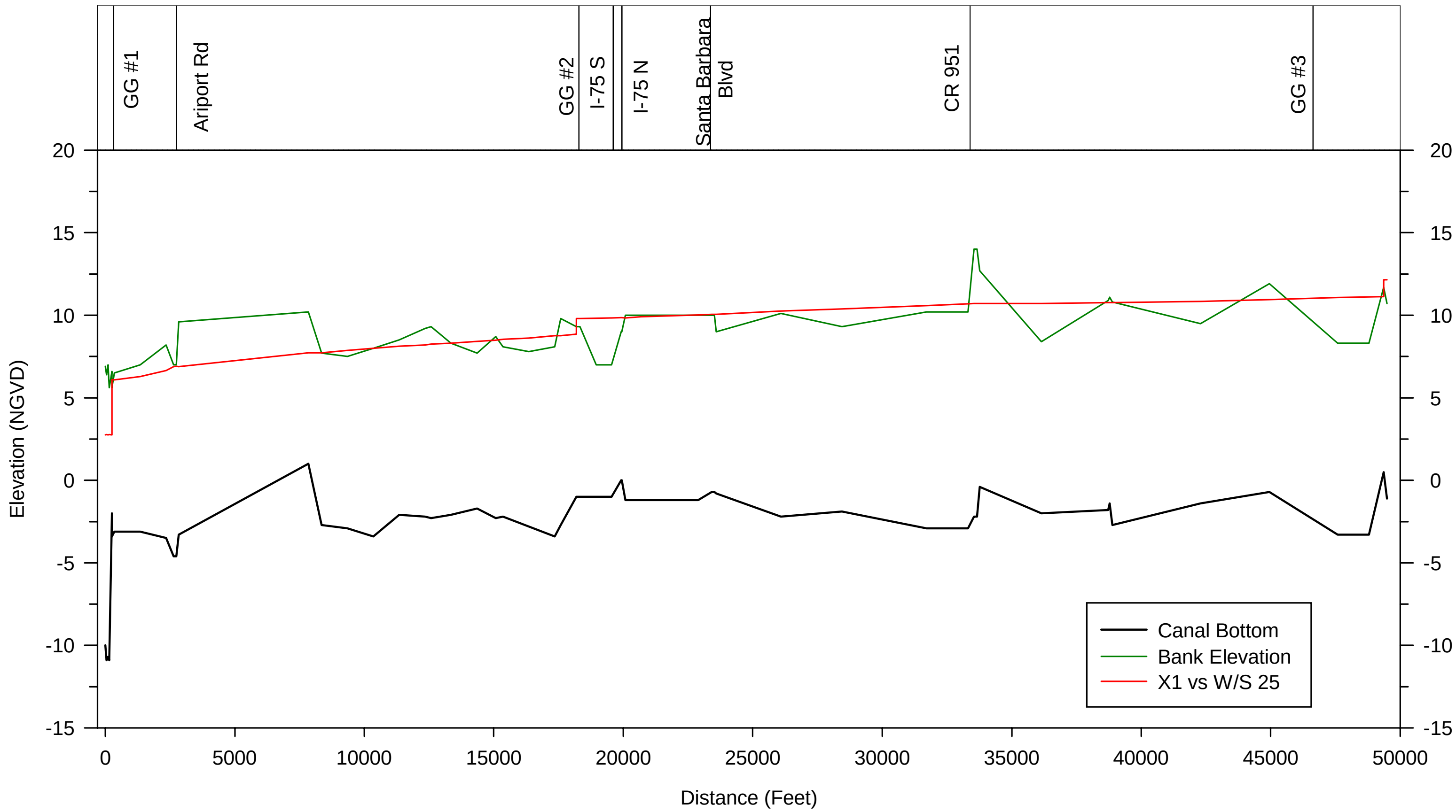


Figure 5.1.2 - 7

Alternative 2
Dynamically Simulated Water Surface Profile for
Golden Gate Main Canal
3-Day 50-Year Design Storm

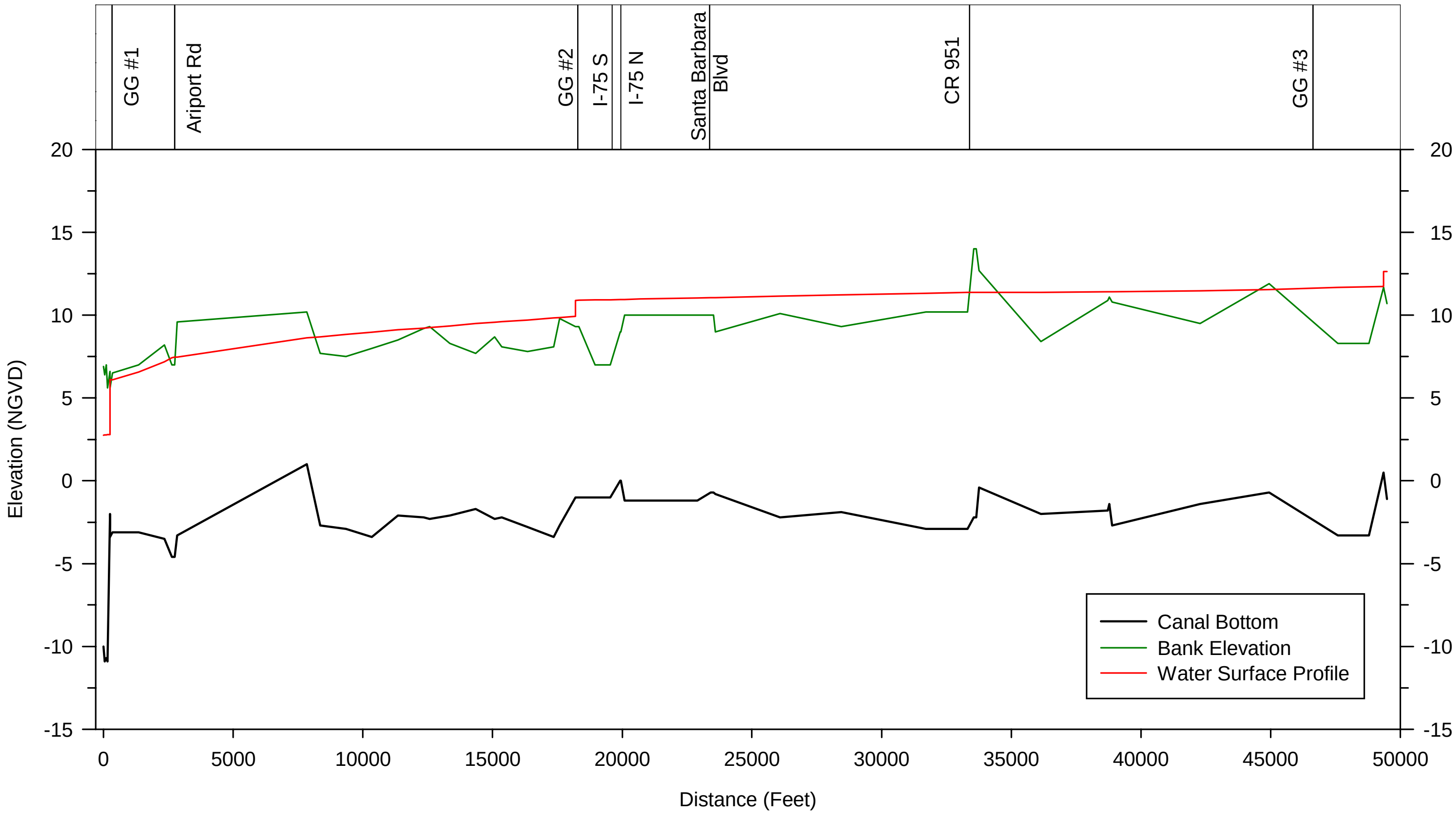


Figure 5.1.2 - 8

Alternative 2
Dynamically Simulated Water Surface Profile for
Golden Gate Main Canal
3-Day 100-Year Design Storm

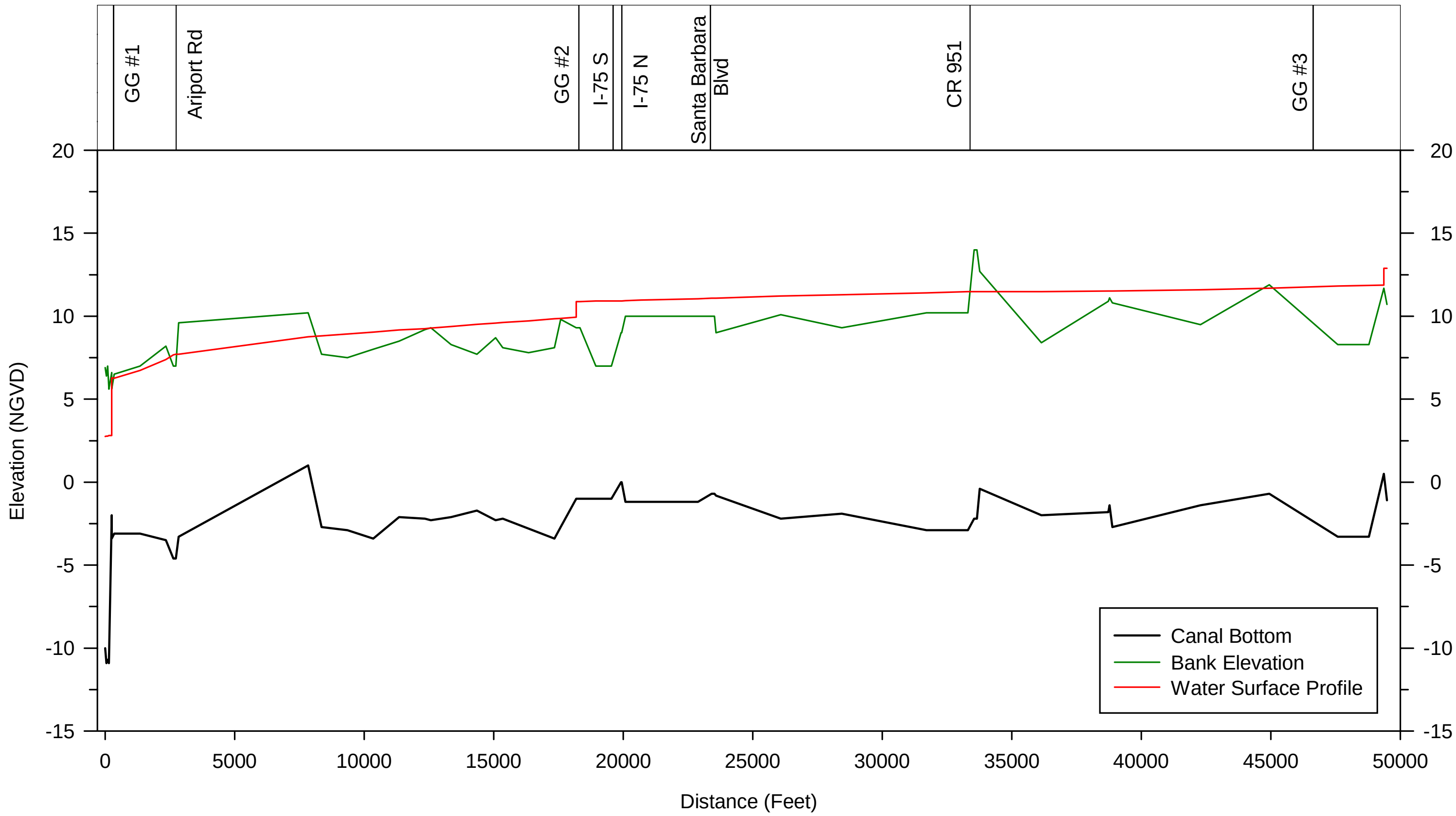


Figure 5.1.2 - 9

Alternative 2
Dynamically Simulated Water Surface Profile for

Henderson Creek Canal

3-Day 10-Year Design Storm

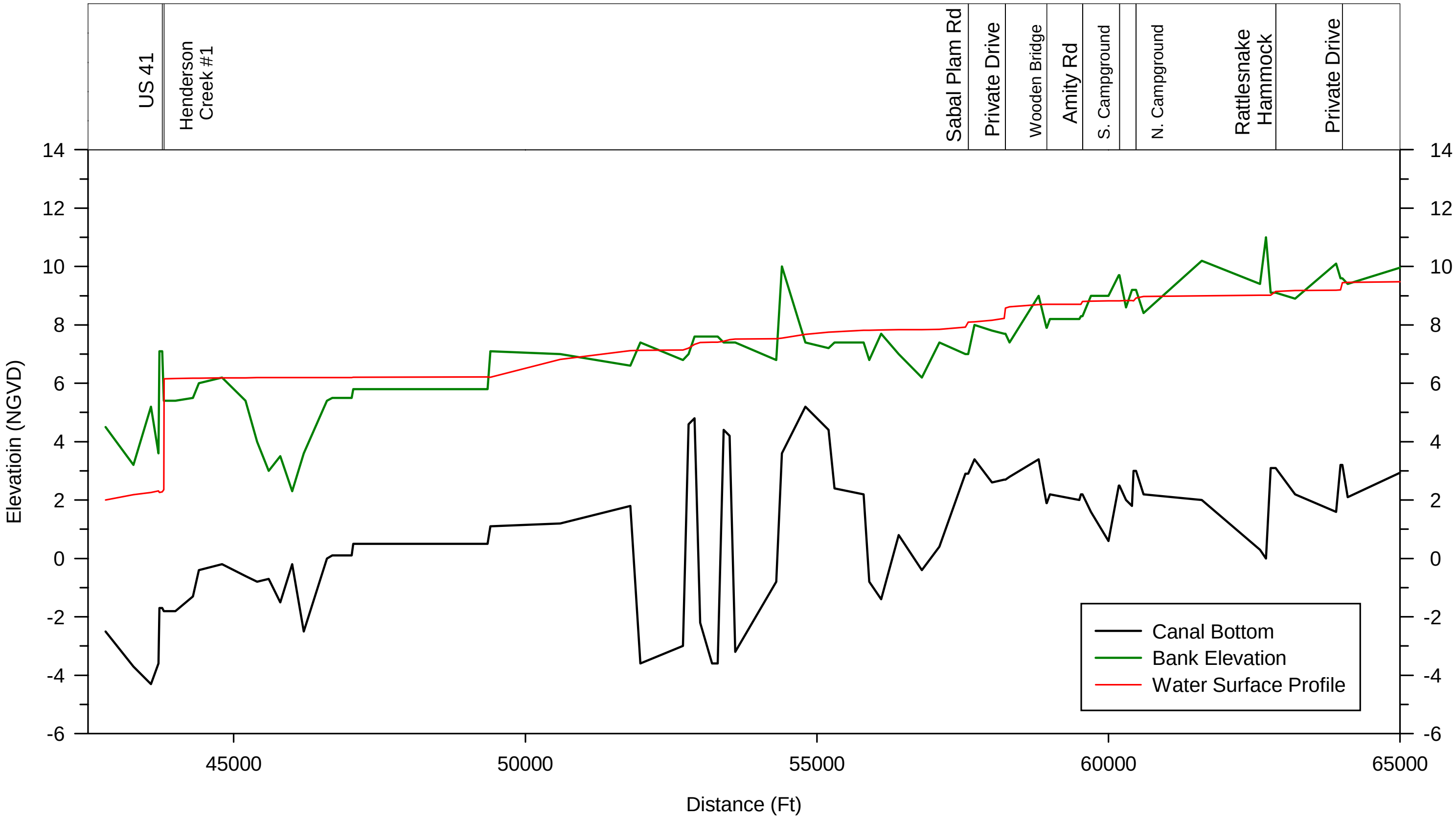


Figure 5.1.2 -9

Alternative 2
Dynamically Simulated Water Surface Profile for

Henderson Creek Canal

3-Day 10-Year Design Storm

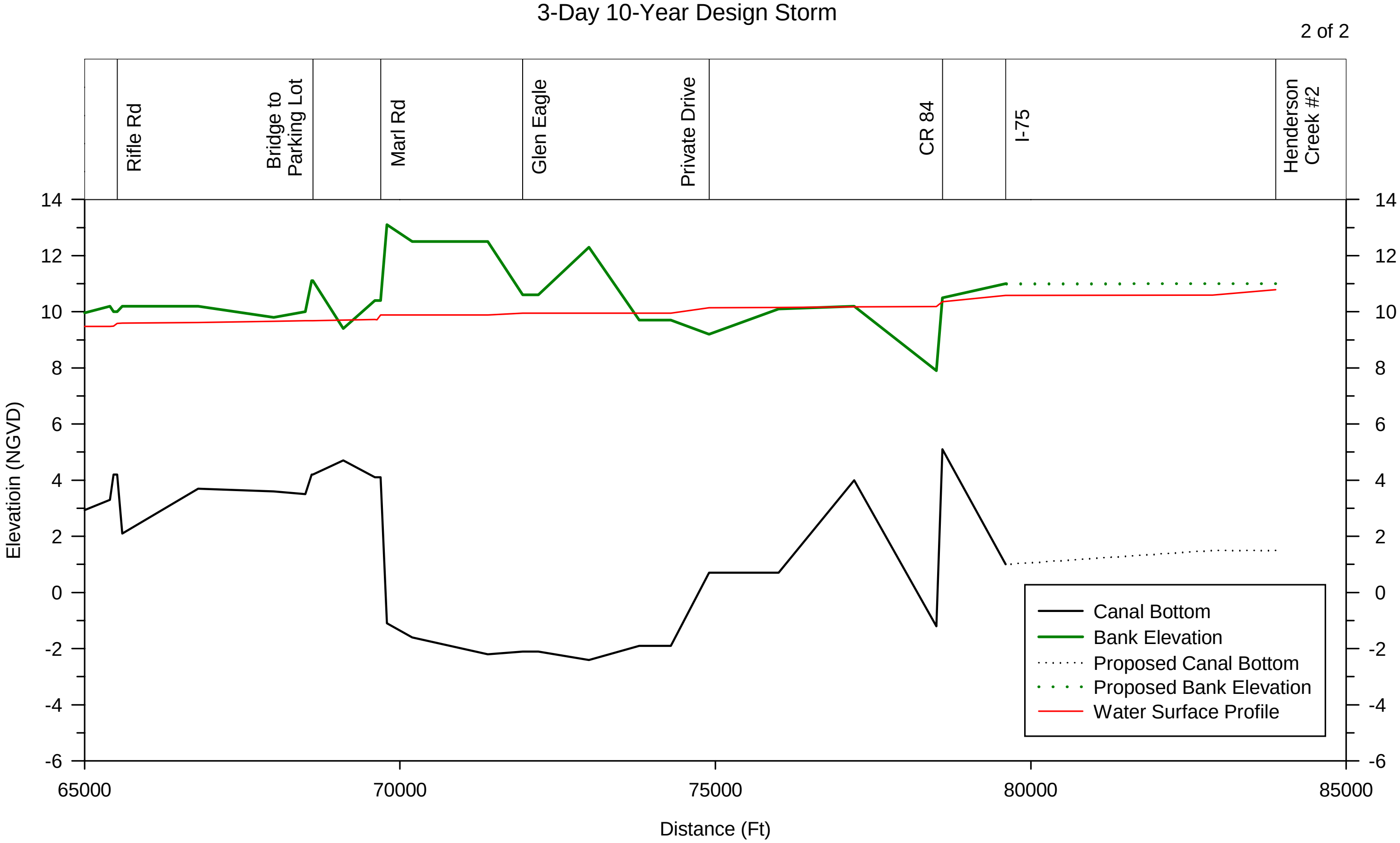


Figure 5.1.2 - 10

Alternative 2
Dynamically Simulated Water surface Profile for

Henderson Creek Canal

3-Day 25-Year Design Storm

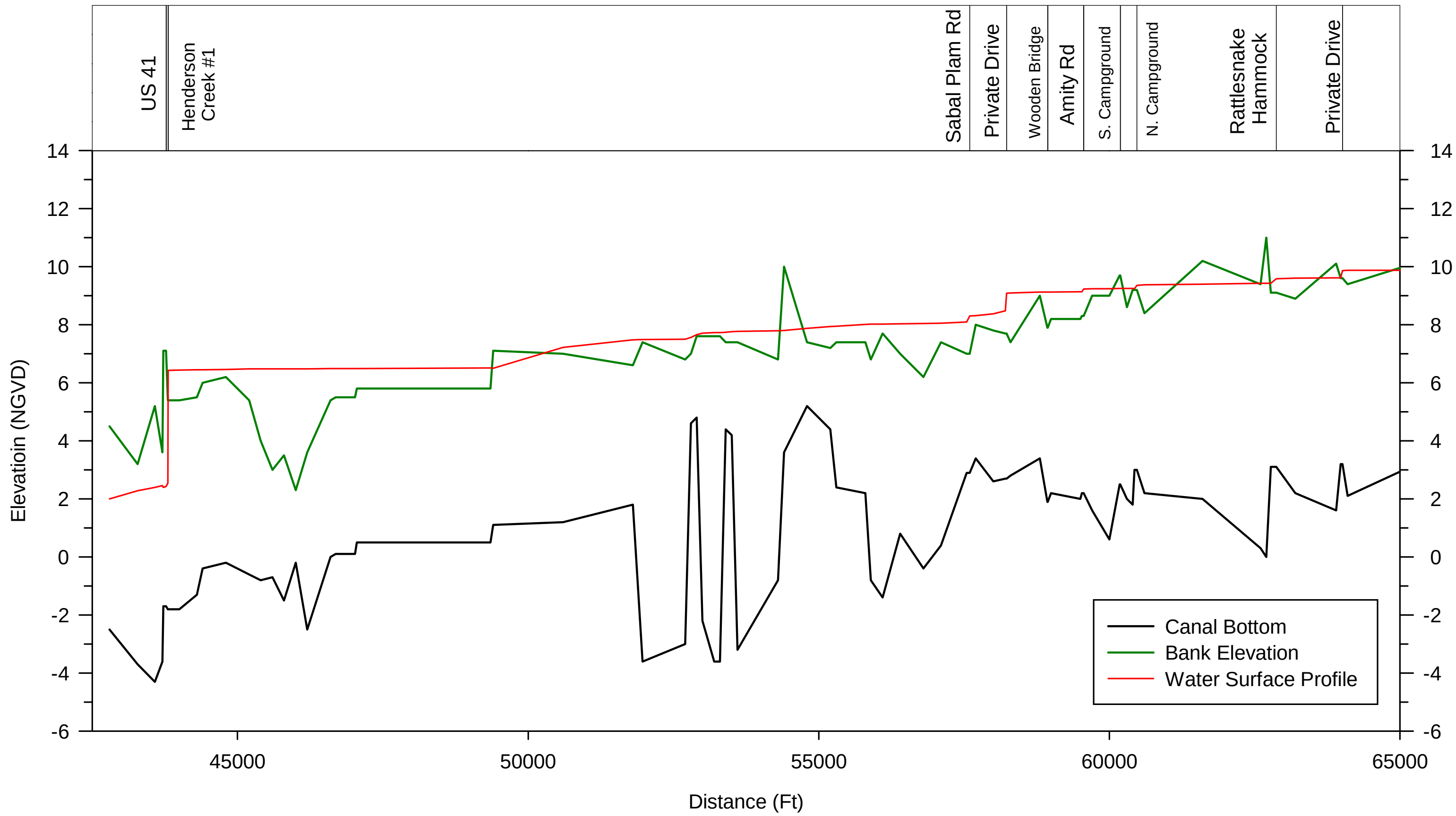


Figure 5.1.2 - 10

Alternative 2
Dynamically Simulated Water Surface Profile for

Henderson Creek Canal

3-Day 25-Year Design Storm

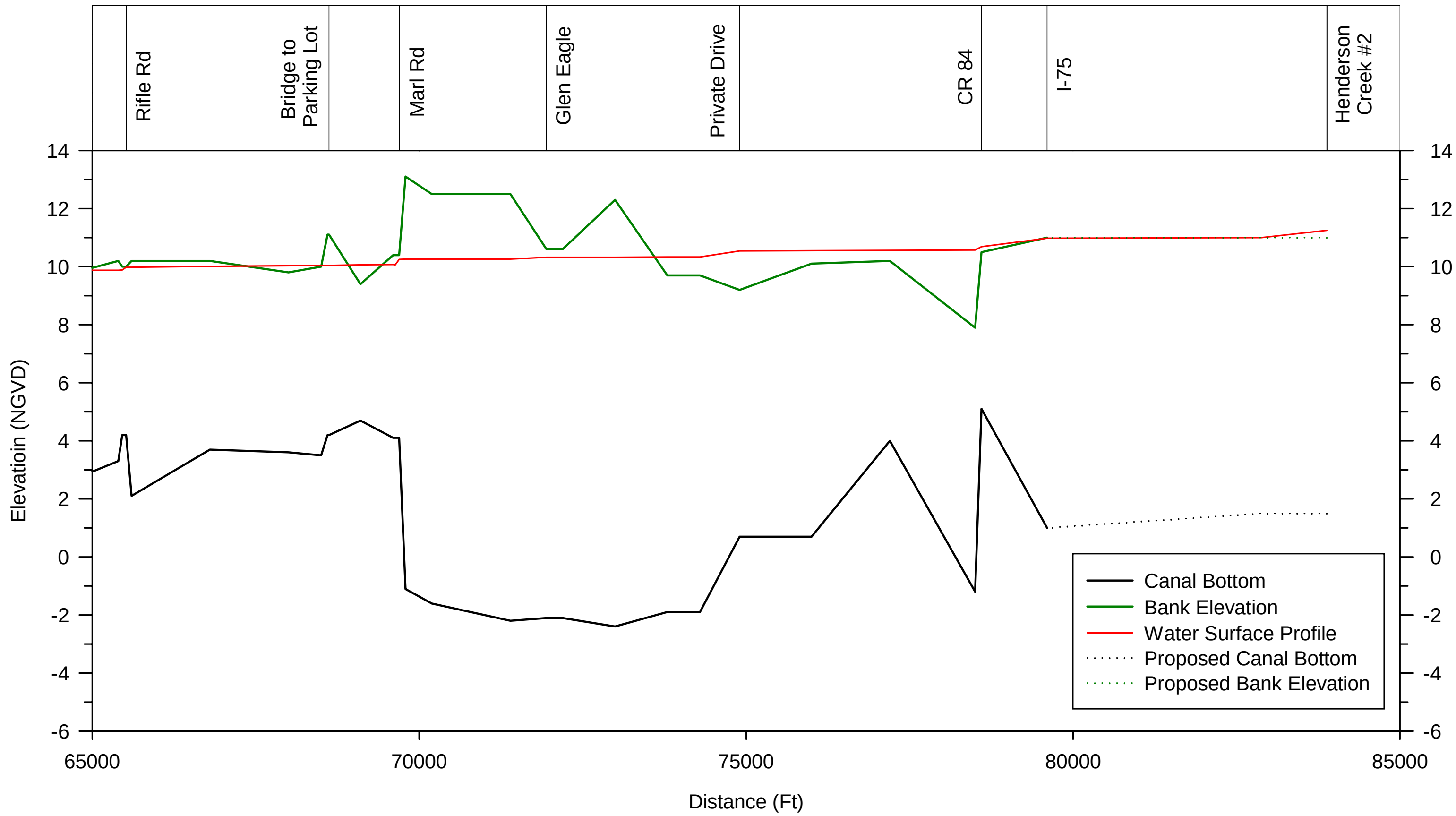


Figure 5.1.2 - 11

Alternative 2
Dynamically Simulated Water Surface Profile for

Henderson Creek Canal

3-Day 50-Year Design Storm

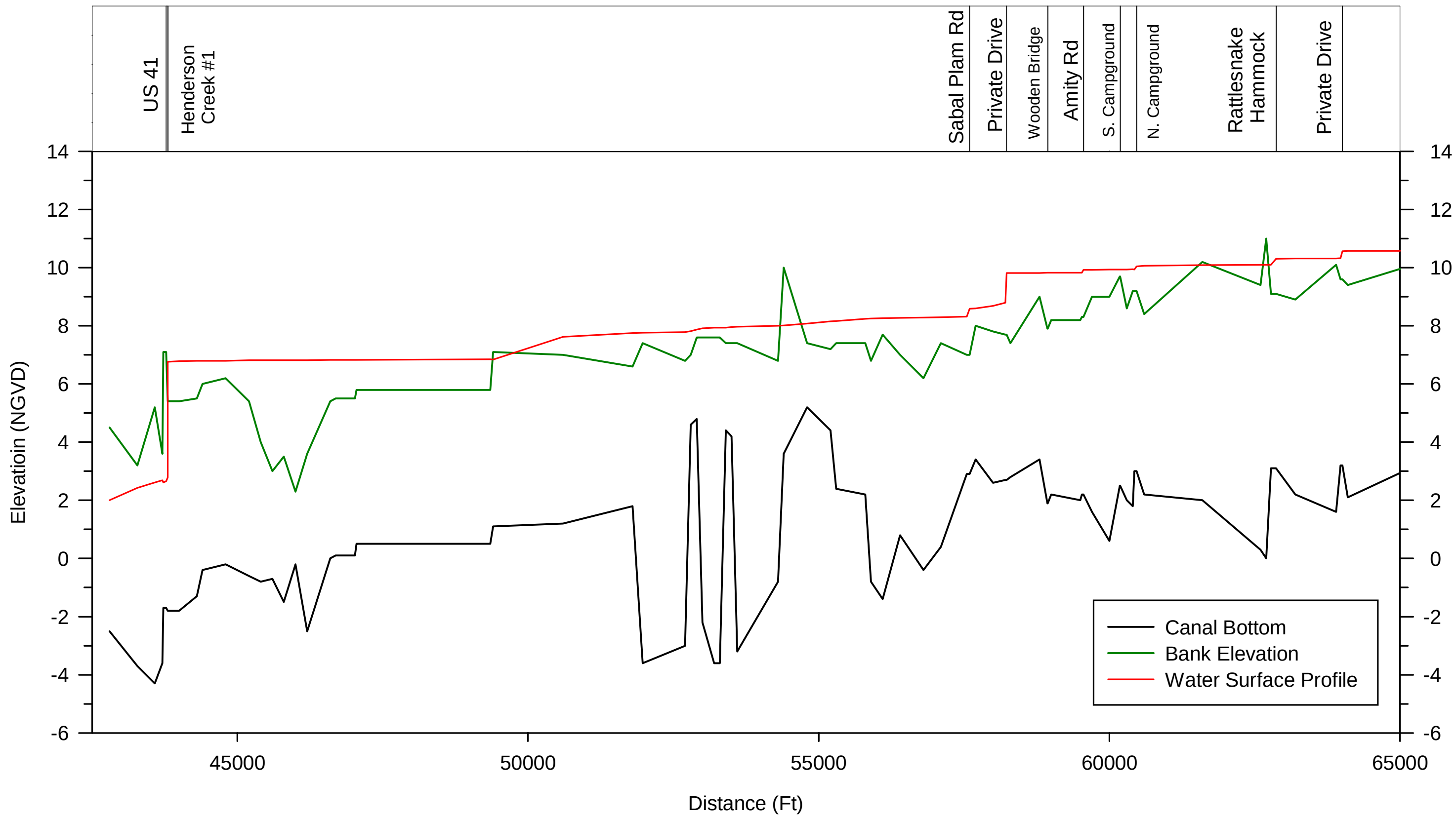


Figure 5.1.2 - 11

Alternative 2
Dynamically Simulated Water Surface Profile for
Henderson Creek Canal
3-Day 50-Year Design Storm

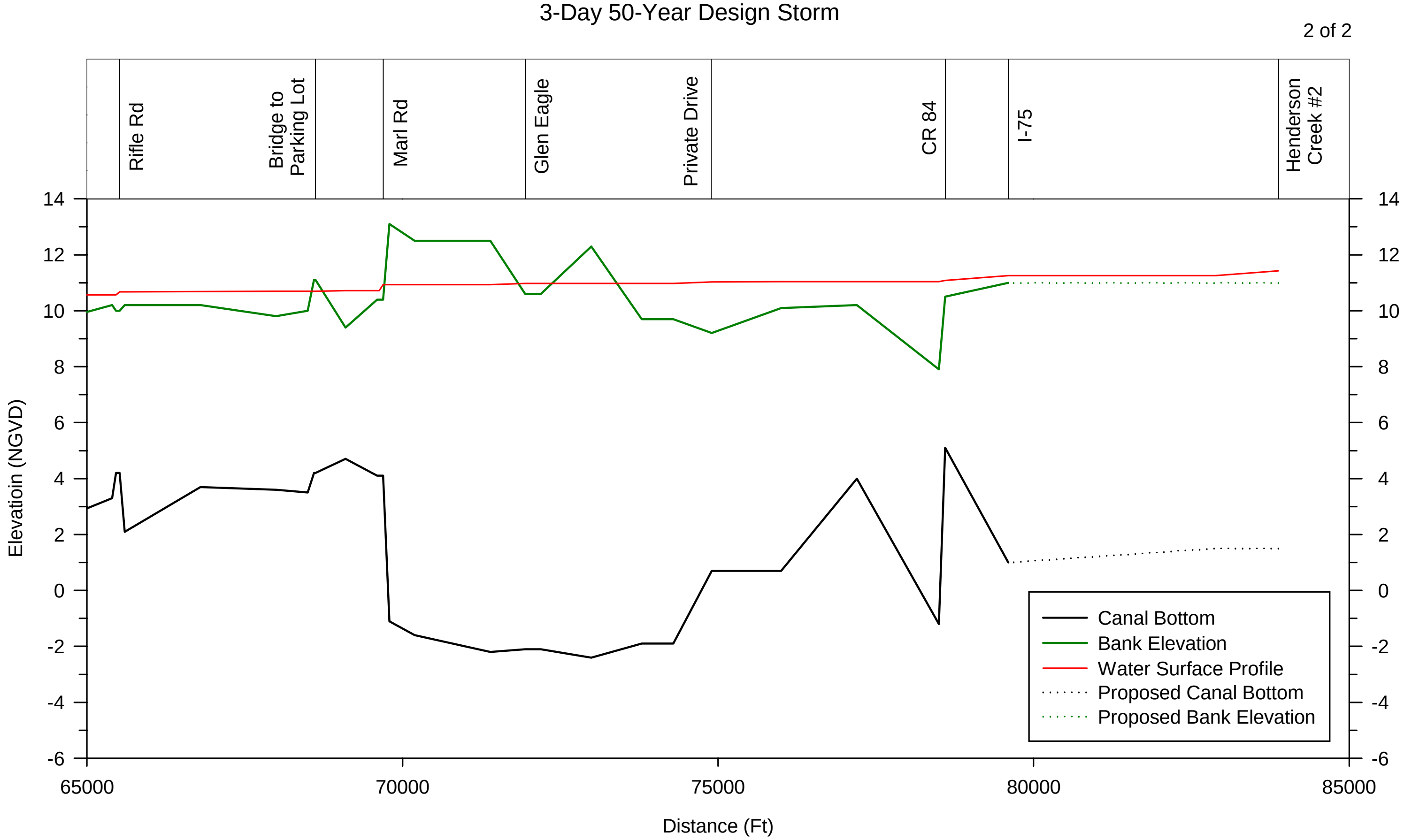


Figure 5.1.2 - 12

Alternative 2
Dynamically Simulated Water Surfile Profile for

Henderson Creek Canal

3-Day 100-Year Design Storm

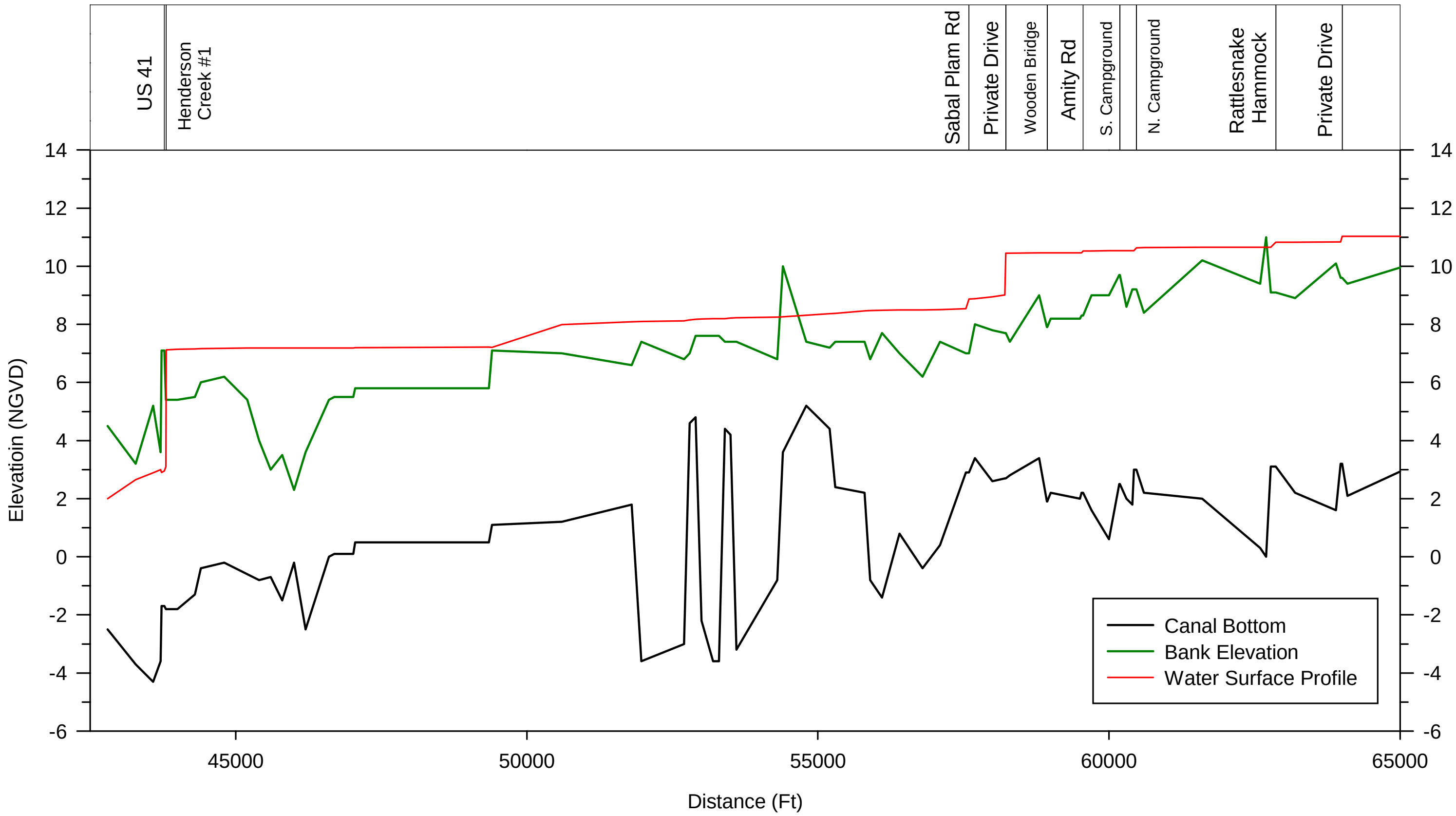
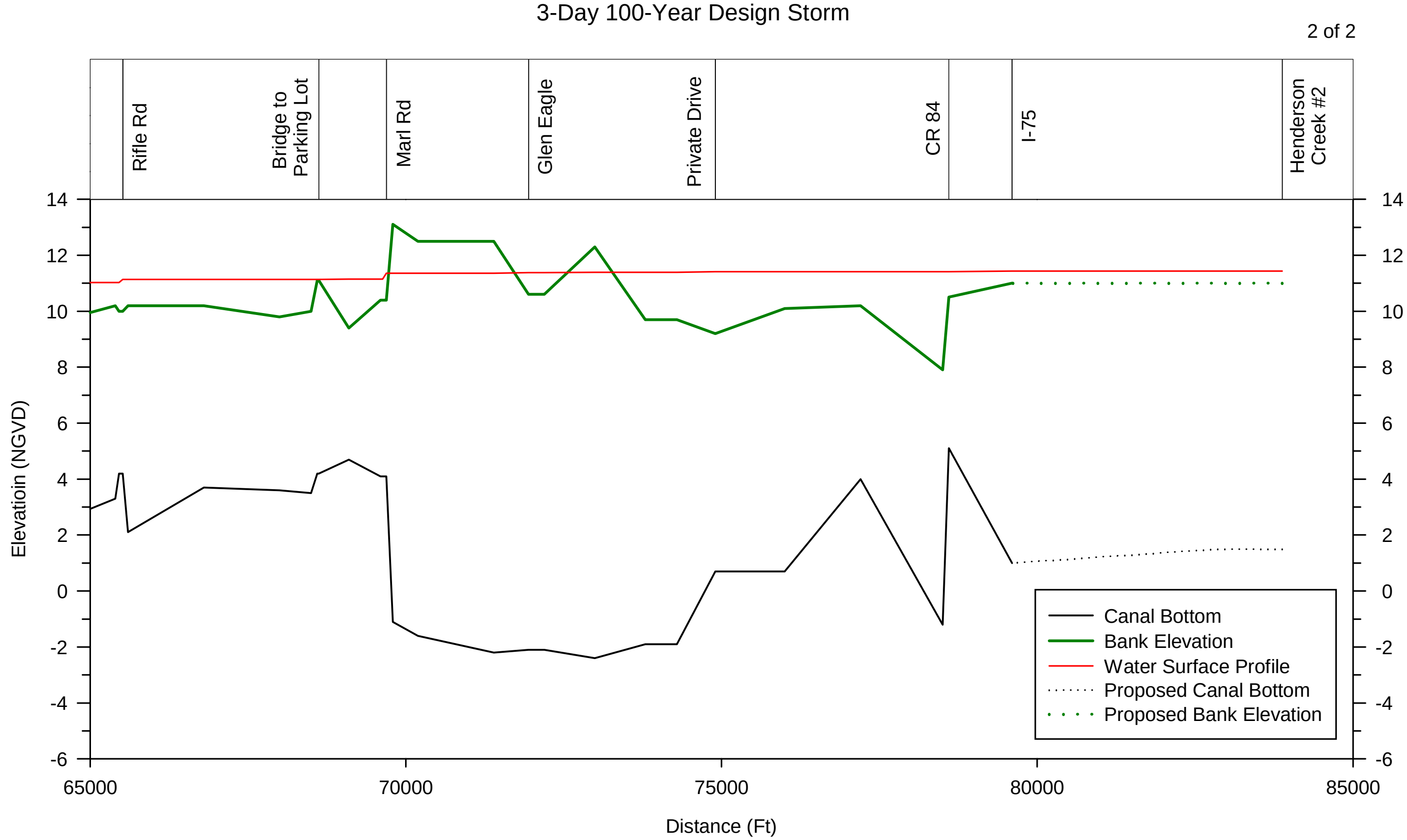


Figure 5.1.2 - 12

Alternative 2
Dynamicall Simulated Water Surface Profile for

Henderson Creek Canal

3-Day 100-Year Design Storm



5.1.3 Alternative 3: Channel and Structural Modifications of the Golden Gate Main Canal West of CR951

The Golden Gate Canal watershed, particularly the area west of CR 951 and the northern Golden Gate Estates is undergoing rapid urban development. The increase in impervious areas has resulted in drastic reduction of the time of concentration of storm runoff and has led to flash flooding in many parts of the Golden Gate Canal basin. Also, the continual overdrainage from this basin has led to associated problems in ground water level decline, invasion of exotic vegetation species, and adverse impact on the estuaries. The Big Cypress Basin Board has been apprised of the problems of this basin, and since 1985, the Board has undertaken an aggressive program to retrofit the old, ineffective water control structures.

The H&H assessment for the Golden Gate Main Canal, under alternative water management scenarios was carried out first by setting up a preliminary steady state hydraulic model of the canal. The Golden Gate Main Canal is configured in a dendritic system. The dendritic system includes CR 951 Canal as it is connected to Golden Gate Main Canal. The U. S. Corps of Engineers HEC-RAS model is used to represent the conceptual model of the dendritic canal system. The schematic diagram of the interconnected canal reaches represented in HEC RAS model is shown in Figure 5.1.3-1. The stage boundary conditions and design flows for the existing Golden Gate Main Canal are depicted in Figure 5.1.3-2. The existing canal crossing structures and the potential head losses are summarized in Table 5.1.3-1.

Henderson Creek is connected to the dendritic system of the Golden Gate Main canal to evaluate different improvement scenarios as shown in Figure 5.1.3-3. The stage boundary conditions and design flows for the Golden Gate Main Canal and Henderson Creek are shown in Figure 5.1.3-4. The channel improvement scenarios include excavation and modifications of hydraulic structures. The channel excavation is based on a slope of half foot per mile. The target invert elevations at the end points of different reaches with design discharge are shown in Figure 5.1.3-5. The target cross sections are used for different reaches for different alternatives to achieve better water management objectives. Golden Gate Main Canal is modified to simulate different geometric and hydraulic options. The scenarios simulated to investigate hydraulic performance of the canals are summarized in Table 5.1.3-2. The quantities of excavation for different reaches of the canal are summarized in Table 5.1.3-3. The configurations of the scenarios with diversion of 200 cfs of flows toward Henderson Creek and an additional 500 cfs of flows from Golden Gate Main Canal to use for Aquifer Storage and Recovery (ASR) are shown in Figures 5.1.3-6 and 5.1.3-7 respectively.

The designs for improvements of the Golden Gate Main Canal were investigated to determine their effectiveness in terms of hydraulic performance. The hydraulic simulations were done for the existing conditions and for the design scenarios listed in Table 5.1.3-2. The simulated hydraulic profiles are shown in Figures 5.1.3-8 through 5.1.3-12. It is observed that the improvement only through cutting for a width of 85 feet, side slope 2:1, and longitudinal cutting slope of 0.5 feet per mile does not ensure a no-flooding condition in the reaches of the canal. In addition to the cutting and improvement, the diversion of an amount of flow of 200 cfs to Henderson Creek and 500 cfs to use for

ASR would ensure a no-flooding condition along the reaches of the Golden Gate Main Canal.

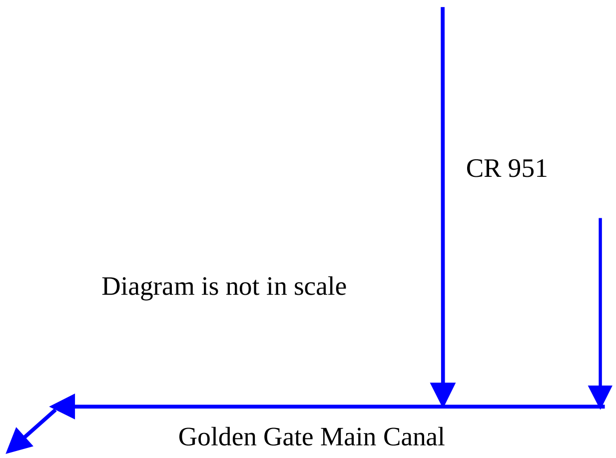


Figure 5.1.3-1. Model Configuration for the existing Golden Gate Main Canal

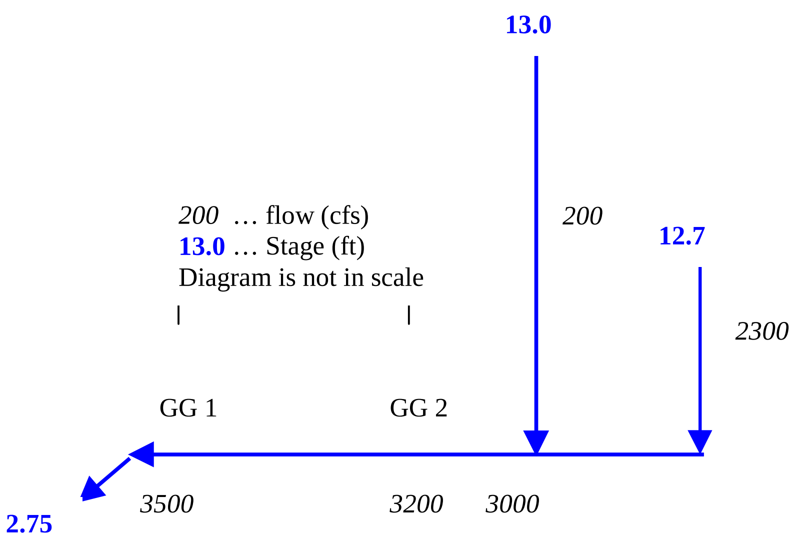


Figure 5.1.3-2. Stage boundary conditions and design flows for the existing condition

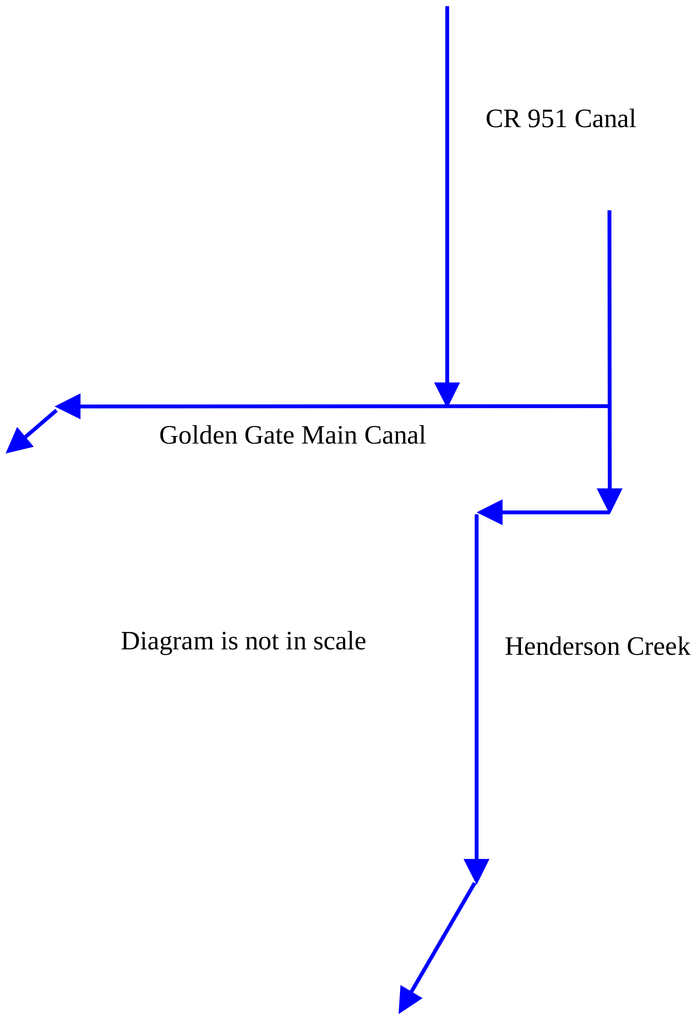


Figure 5.1.3-3. Interconnected reaches of Golden Gate Main and Henderson Creek.

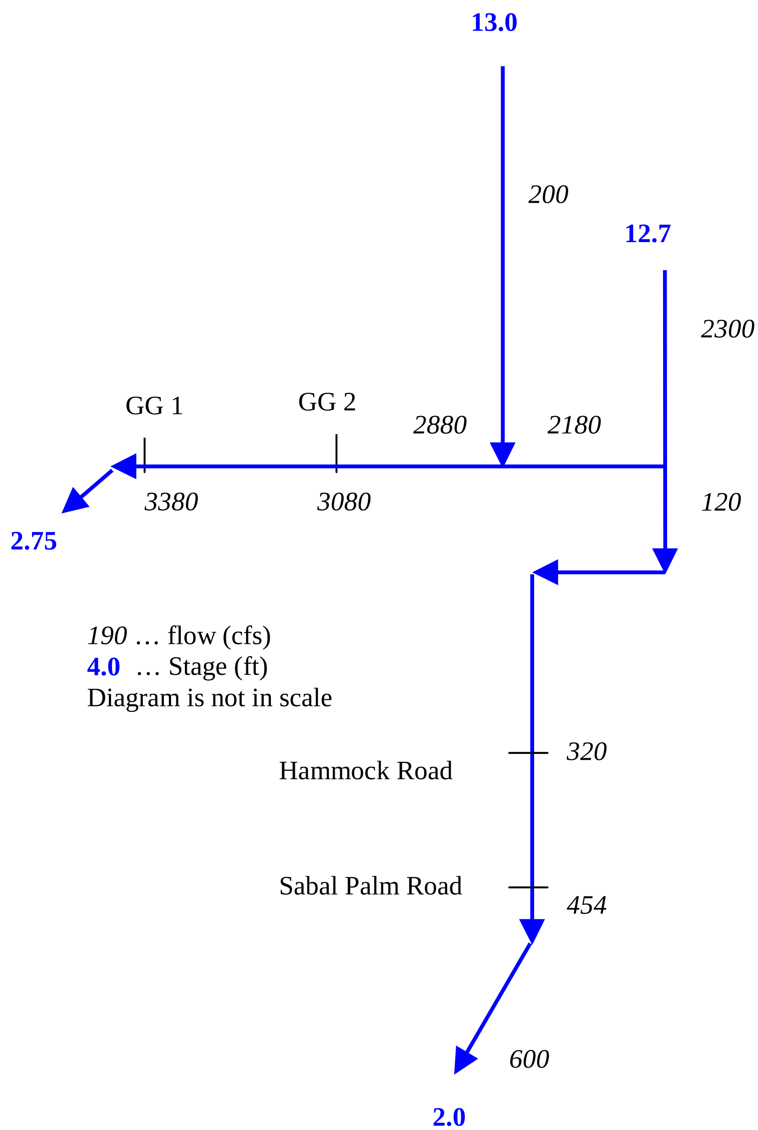


Figure 5.1.3-4. Stage boundary conditions and design flows in the reaches

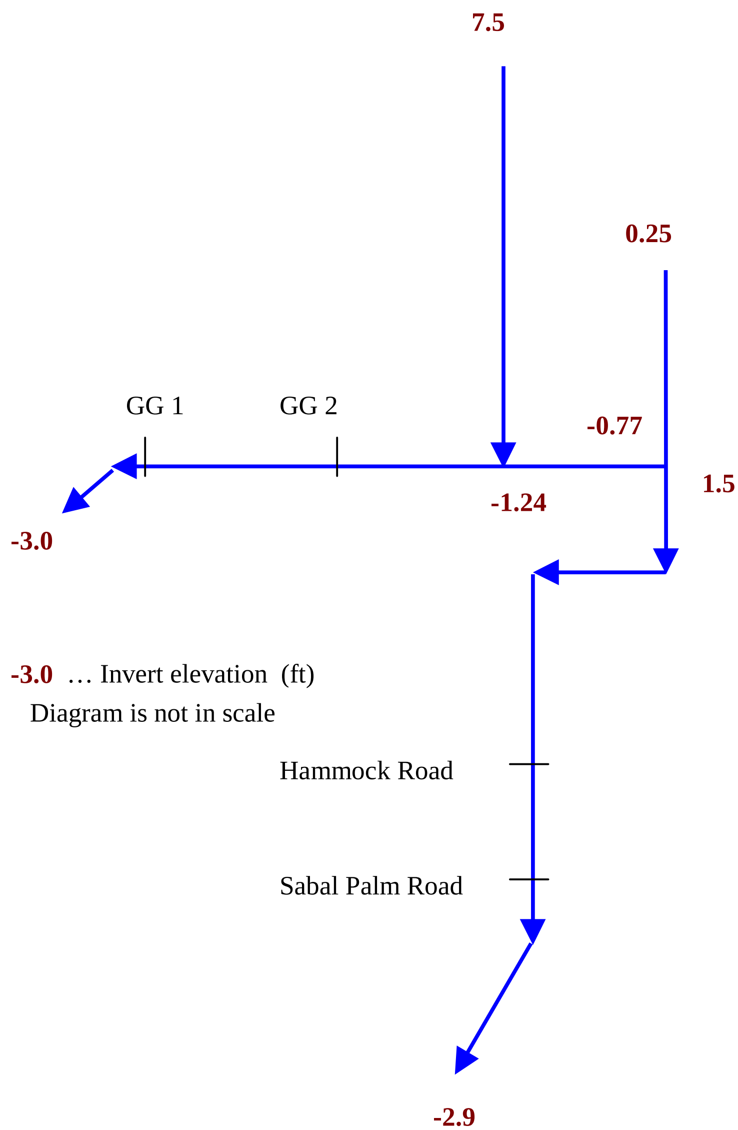


Figure 5.1.3-5. Modified channel with target invert elevations after improvements.

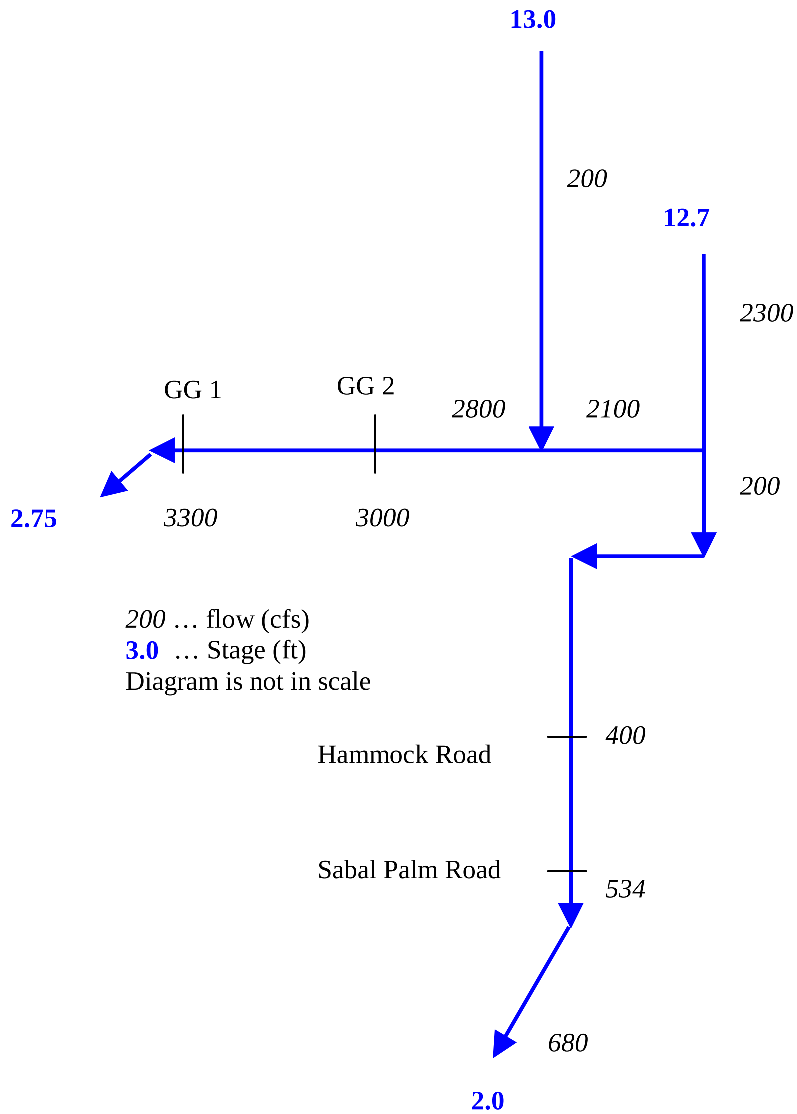


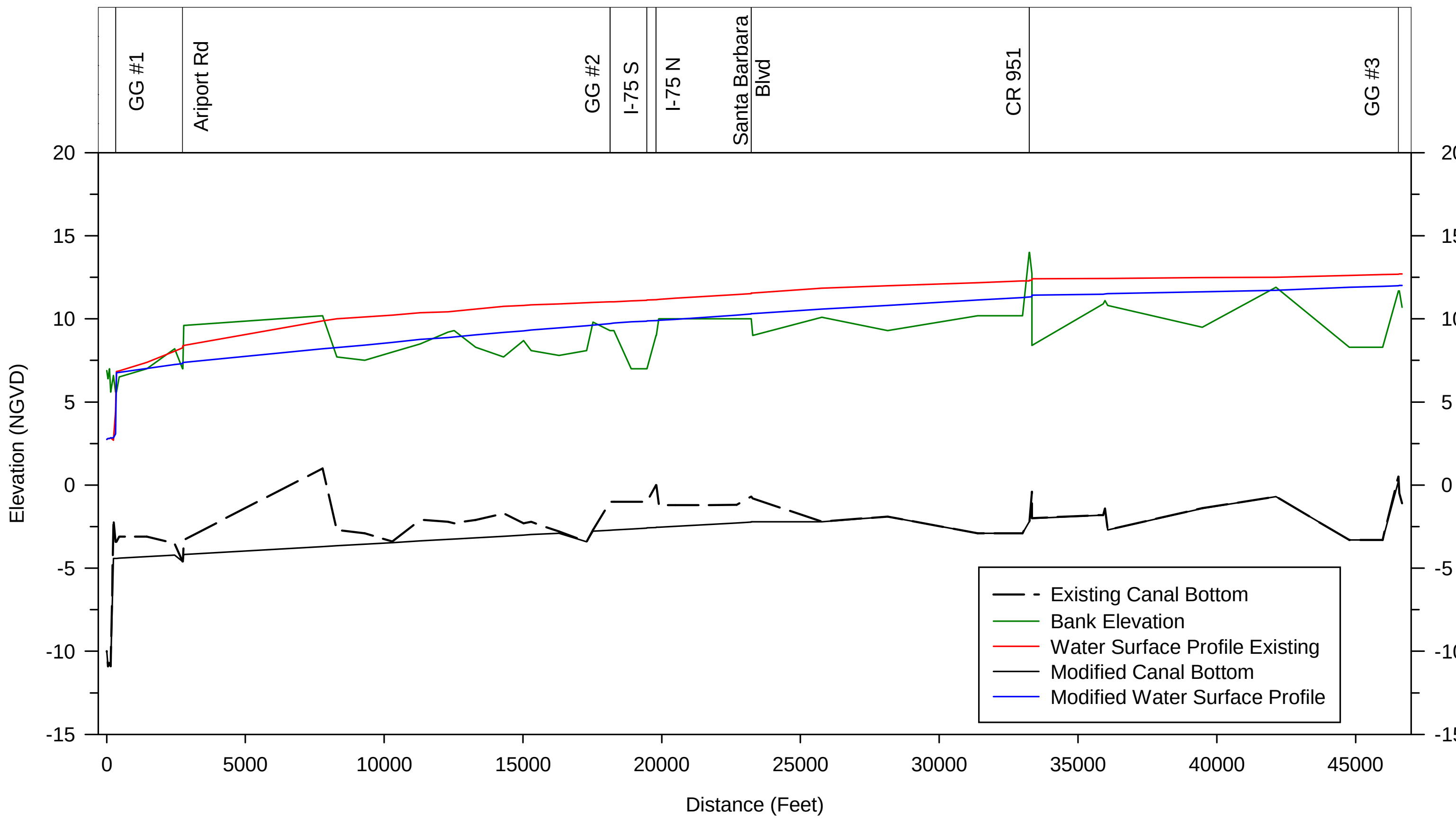
Figure 5.1.3-6. Design flows after diversion of 200 cfs toward Henderson Creek

Figure 5.1.3-8

Hec-Ras Results for

Existing and Modified Golden Gate Main Canal

3-Day 25-Year Design Storm



ShareonBCBserve\BCB H&H Model..\Figure8

Figure 5.1.3-9

Hec-Ras Results for

Existing, Modified Canal and Modified GG#1 Golden Gate Main Canal

3-Day 25-Year Design Storm

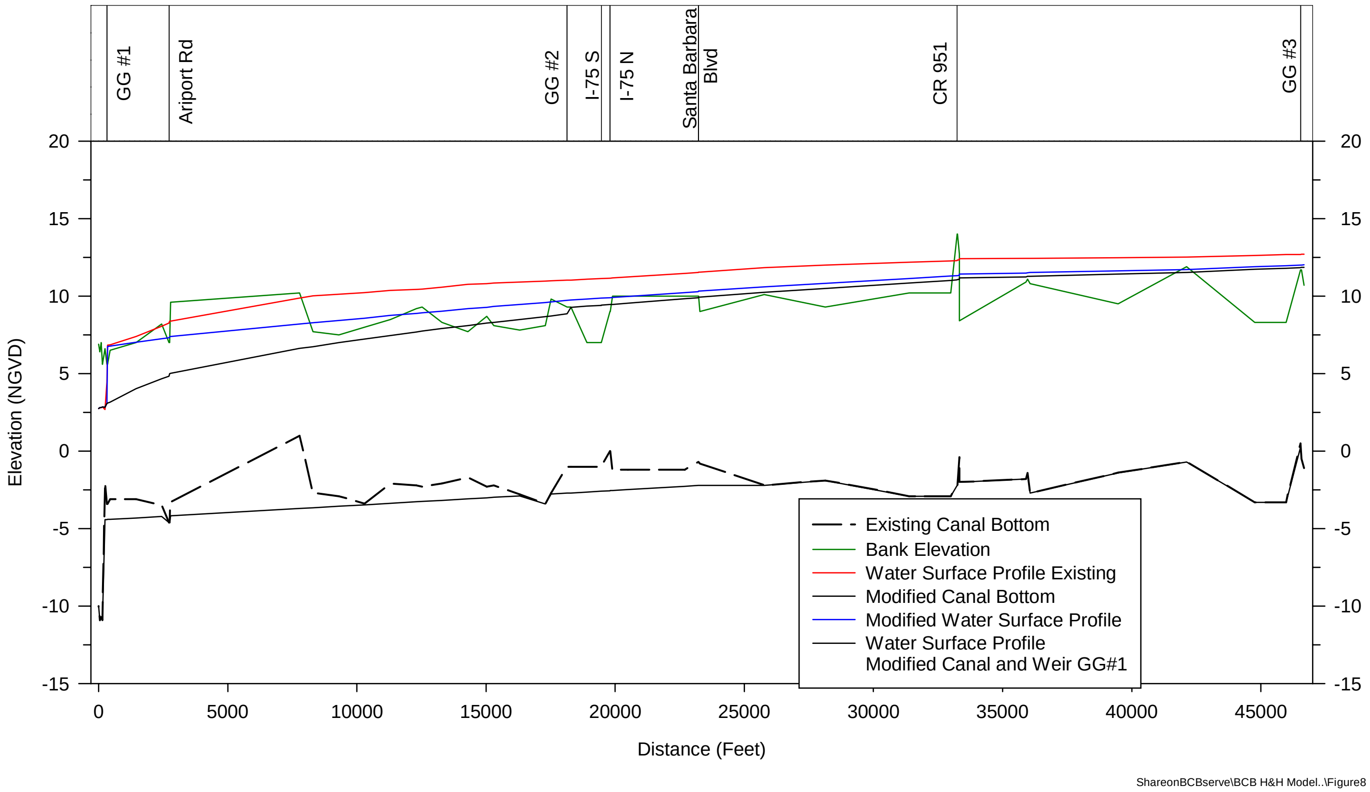


Figure 5.1.3-10

Hec-Ras Results for

Modified Golden Gate Main Canal and Weirs GG#1 & GG#2

3-Day 25-Year Design Storm

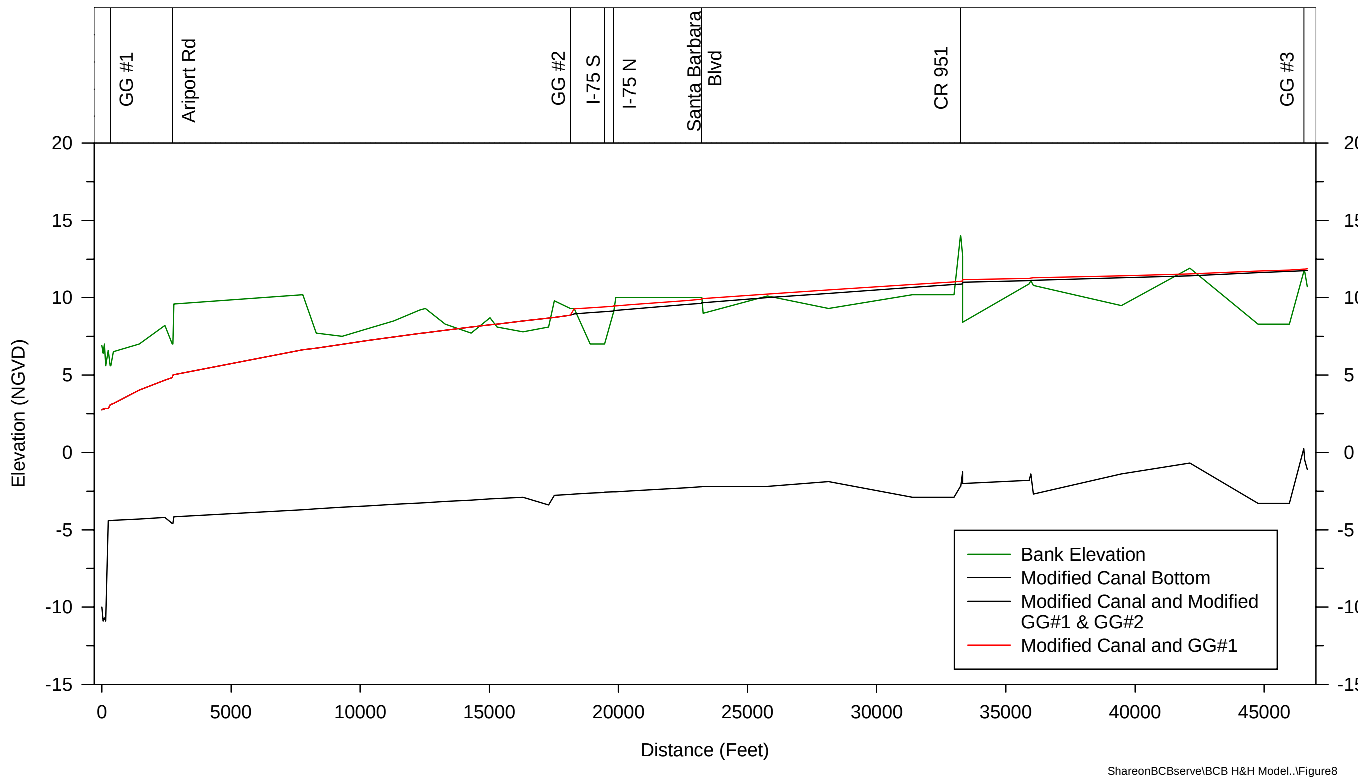


Figure 5.1.3-11

Hec-Ras Results for

Improved Canal with Structural Modifications (GG#1 & GG#2)

and Diversion of 200 cfs to Henderson Creek

3-Day 25-Year Design Storm

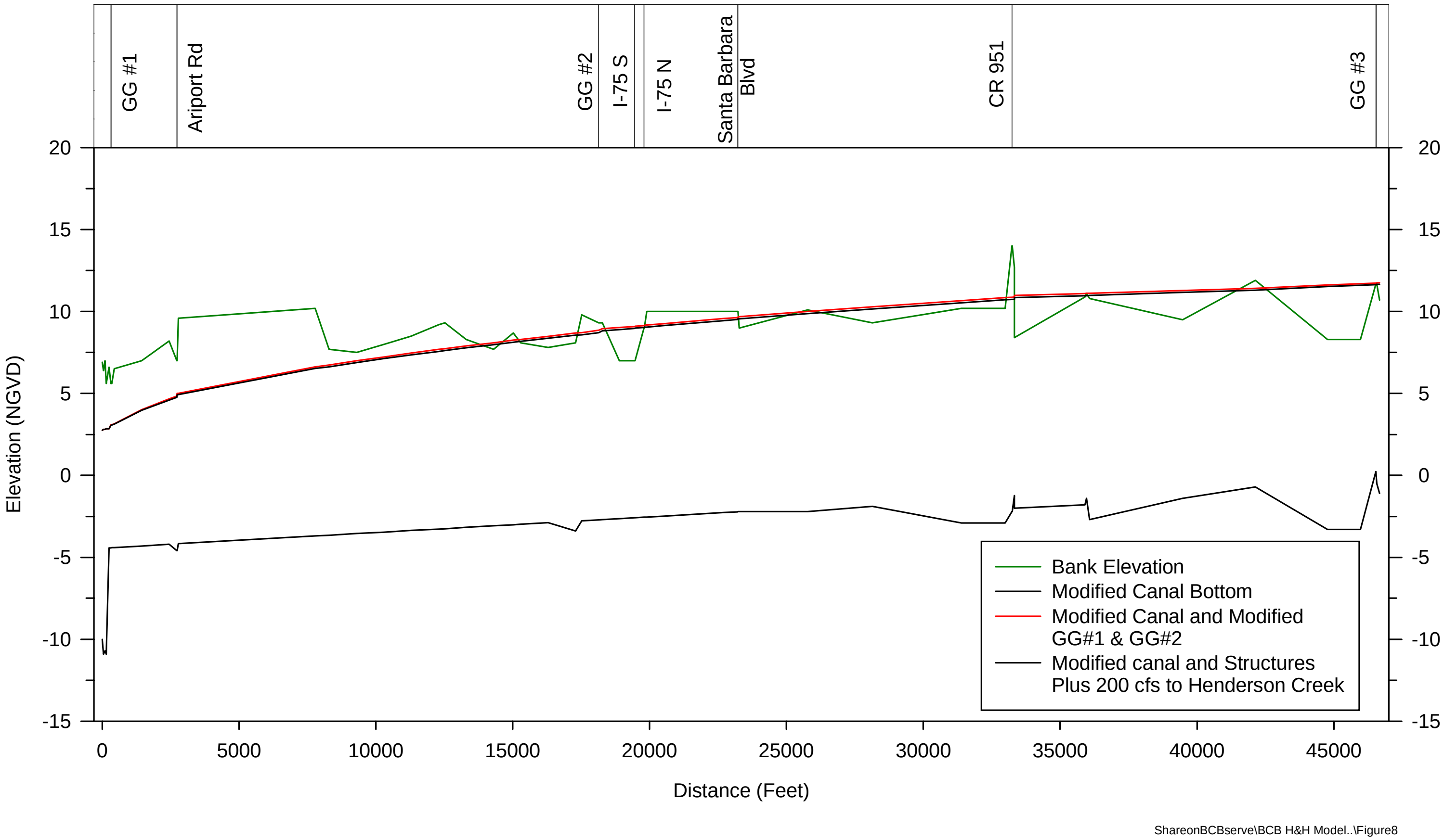


Figure 5.1.3-12

Hec-Ras Results for

Improved Golden Gate Main Canal with divirson of Flows

to Henderson Creek (200cfs) and ASR (500cfs)

3-Day 25-Year Design Storm

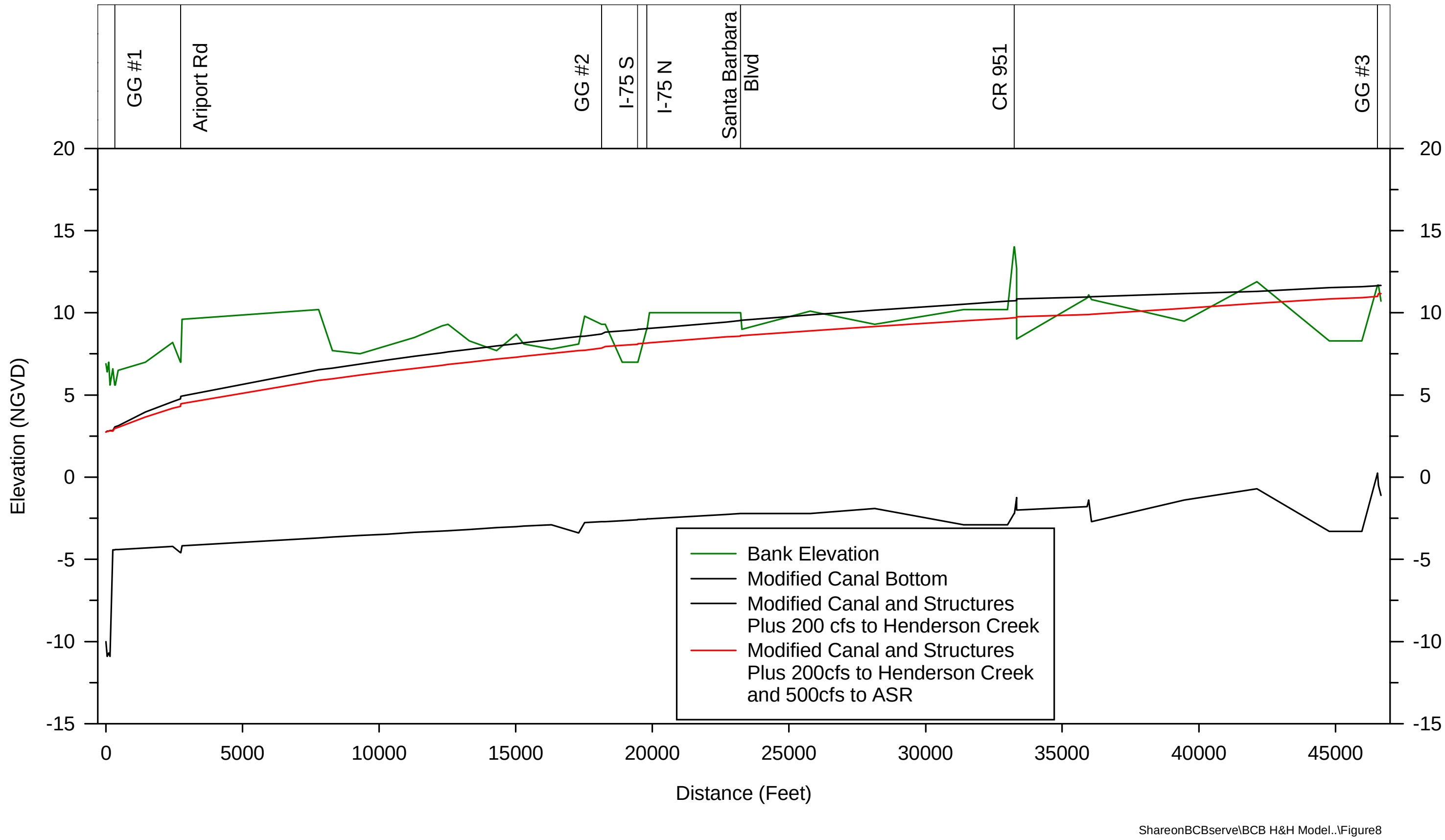


Table 5.1.3-1. Summary of existing Golden Gate Main Canal crossings

Model Section	Approximate miles upstream of GG Weir No. 1	Location	Structure description	Invert elevation	Potential Head Loss (ft)
1206	8.75	GG Weir No. 3	One 100 ft Broad Crested Weir Two 6' by 5' Gate	7.5 -0.5	0.01
1126	6.23	CR 951	Bridge with piers	-2.2	0.03
1086	4.34	Santa Barbara Boulevard	Bridge with piers	-0.7	0.02
1078	3.69	I-75 North	Bridge with piers	0	0
1076	3.62	I-75 South	Bridge with piers	-1	0.02
1072	3.37	GG Weir No. 2	One 105 ft Broad Crested Weir Two 6' by 5' Gate	5 -1	0.01
1017	0.46	Airport Road	Bridge with piers	-4.6	0.12
1012	0	GG Weir No. 1	One 112 ft Broad Crested Weir	2	2.39

Structural Modification and Flow

Channel Improvement, Scenarios for Alternative 3 Diversion

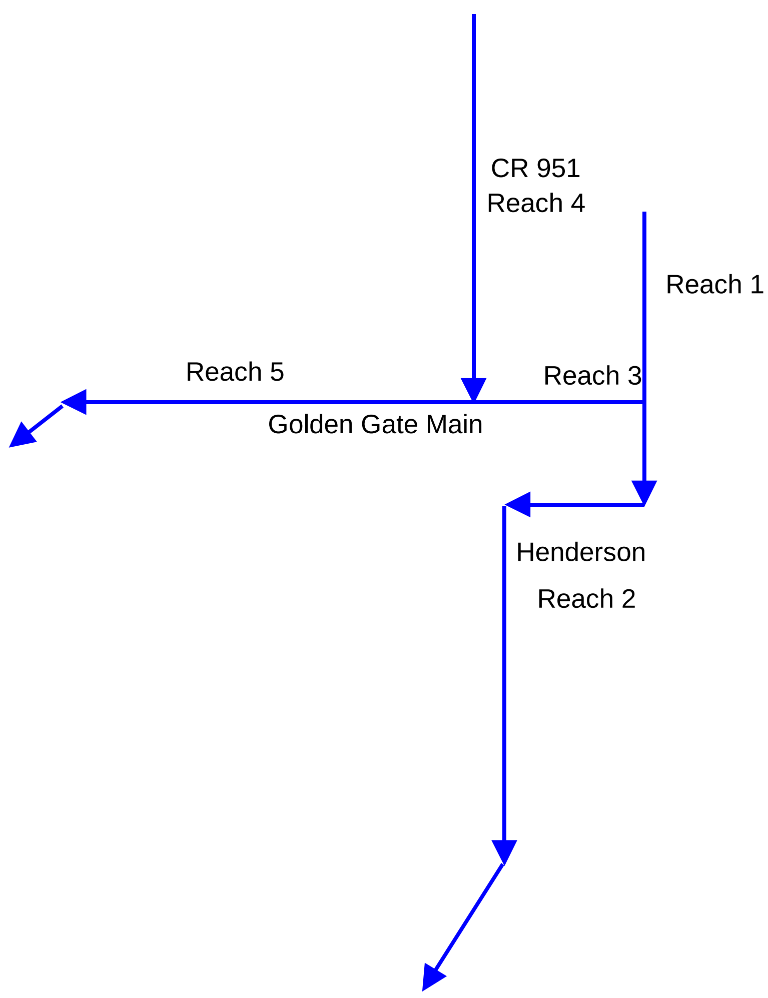
Table 5.1.3-2. Scenarios to simulate hydraulic performance of canal reaches

Sub-Alternative	Canal	Description[b]
3-1	Existing condition	No modification in the channels and structures
3-2	CR 951[c]	30 ft bottom width
	Golden Gate Main	85 ft bottom width
	Henderson Creek	30 ft bottom width
3-3	Golden Gate Main	Weir GG 1 modified with six 10 ft by 8 ft gate (crest at -3 ft) Broad crested weir 55 ft long besides gate with crest at 3.5 ft
3-4	Golden Gate Main	In addition to the changes in alternative 3, weir GG 2 is modified by adding six 10 ft by 8 ft gate in place of one of two 5 ft by 6 ft existing gates
3-4a	Golden Gate Main	In alternative 4 flow is reduced by 200 cfs
3-4b	Golden Gate Main	In alternative 4 flow is reduced by 700 cfs

[a] Sub-Alternatives 3-3 to 3-4b include changes in alternative 3-2 for Henderson Creek, CR 951, and Golden Gate Main east of
[b] Modified based on 2 (horizontal) : 1 (vertical) side slope and 0.5 ft per mile cutting slope along the channel
[c] CR 951 canal improvements include excavation and structural modifications as described in detail in a separate report

Table 5.1.3-3. Summary of Excavation and Cost for Golden Gate Main Canal

Canal	Bottom Width (ft)	Side Slope	Excavation Quantity (cubic yard)	Estimated Cost @ $12.00/yd^3
Golden Gate Main				
Reach 1	85	2:1	47,430	$569,160
Reach 3	85	2:1	16,805	$201,660
Reach 5	85	2:1	323,650	$3,883,800
Total for Golden Gate Main			387,885	$4,654,620
Total			387,885	$4,654,620



5.1.4 Alternative 4: Channel Modification of CR951 Canal and Implementation of Water Control Measures

The County Road 951 Canal subbasin drains approximately 1,746 acres. This canal was originally constructed as a source of fill material for the adjacent roadway, and is now a part of the Big Cypress Basin primary canal system. The canal is approximately seven miles long and connects the Cocohatchee Canal with the Golden Gate Main Canal. During expansion of County Road 951 in 1988 and construction of the North County Regional Water Treatment Plant, two V-notch weir water control structures in the canal were removed. The required level of service for flood protection in this canal and subbasin is to convey runoff from the 25-year, 3-day storm event.

The on-going Big Cypress Basin capital improvement effort determined that the County Road 951 Canal cannot currently convey the runoff from the 25-year, 3-day storm event within District canal criteria. In addition, current development plans and flood protection plans in South Lee County call for excess storm water from South Lee County to flow into a planned system of regional flow ways that will restore historical flow connections. Additional improvements to the County Road 951 Canal will be needed to convey the future flows and runoff conveyed by these proposed flow ways.

Hydrologic and Hydraulic computer modeling, using the regional models developed as part of the capital improvement plan, assessed flood protection and general water management functions of the existing conditions and various alternative improvement scenarios for the design storm event. Peak discharges in the existing canal, without any improvements to the regional flow ways, range from 54 cubic feet per second at the north end to 129 cubic feet per second at the south end. Peak discharges in the improved canal, after regional flow way improvement, range from 171 cubic feet per second at the north end to 223 cubic feet per second at the south end. The proposed improvements are designed to convey these larger discharges without a significant increase in the water surface elevations. The proposed improvement plan includes the following improvements in the primary canal system.

- Enlarge the canal cross section in the north two mile reach
- Relocate two utility pipe line crossings
- Remove/replace nine culvert crossings
- Construct two water control structures
- Landscape the canal bank and right of way

The proposed improvements include modifications to both private and public infrastructure/facilities. Collier County owns the utility pipeline crossings and one culvert crossing. The other eight culvert crossings are privately owned. The detailed improvement plans for channel modifications with structural improvements supported by Hydrologic-Hydraulic modeling analyses are described in a separate report titled "Hydrologic-Hydraulic (H&H) Assessment of CR951 Canal Improvement."

5.1.5 Alternative 5: Diversion of a Portion of Corkscrew Canal Flow to Golden Gate Main Canal

The Corkscrew Canal and its side ditches, including the Curry Canal, comprise a dendritic network of poorly drained canals that provides an interbasin transfer of water from the Corkscrew-Cocohatchee basin to Golden Gate basin. Virtually uncontrolled flows through these canals have led to overdrainage of portions of the Bird Rookery Swamp ecosystem and loss of dry season surface and shallow aquifer storage. The flood control capacity of these canals is also limited, as was evident during the recent wet seasons of 1991, 1992, and 1995. The Basin has proposed entering into an agreement with Collier County to construct two water control structures by utilizing wetlands mitigation funds from the Livingston Road construction project. Some preliminary drainage improvement work in the secondary canals has been implemented by the Collier County Stormwater Management Department.

Hydraulic evaluation of the Golden Gate Main Canal, performed earlier in this report, indicates availability of considerable storage for surface water in the canal reach upstream of Golden Gate Canal Weir No. 5. The topographic characteristics of this Northern Golden Gate Canal subbasin also have the potential of diverting some wet season flows from the Corkscrew-Cocohatchee Canal basin. The feasibility of this alternative will be analyzed by the application of the Integrated Surface and Groundwater Model to assess the impact on groundwater recharge, in addition to the level of reduction of floodstages.

5.1.6 Alternative 6: Cocohatchee Basin East and West Flowways

An evaluation of the flooding problem in the Bonita Springs area, performed as a part of the South Lee County Watershed Management Plan (SFWMD, 1999), recommended restoration of several regional flowways as shown in Figure 5.1.6-1. This plan identified two major flowways in the vicinity of the CR 951 Canal headwater area known as Cocohatchee west and east flowways. A conceptual construction and maintenance plan recently proposed by the consultants of the South Lee County Watershed Plan has outlined a flow conveyance corridor for the Cocohatchee west flowway. This plan utilizes lands of CREW Trust and of several proposed developments to create a flowway through wetlands, preserve areas, lakes and vegetative buffer areas to alternate flood peaks. In our present assessment, additional conveyance improvement along the east flowway was considered to reduce flow to the Imperial River basin. Improvement to this flowway will involve removal/control of vegetation and clearing of the ditch between the Florida Rock quarry and Bonita Bay East development.

Hydrologic analysis for reduction of flow toward the Imperial basin and conveyance analysis through the improved Cocohatchee east flowway to the Cocohatchee Canal was performed by application of the Environmental Protection Agency Storm Water Management Model (SWMM). The Extended Transport (EXTRAN) block of SWMM was used to quantify the flow through the East Flowway. The East Flowway extends from the end of Corkscrew Swamp to Cocohachee Canal about one mile east of CR 951 Canal.

The design storm used for the Cocohatchee East Flowway is a 25-year, 3-day event. Intensity-duration-frequency distribution of the design rainfall event was obtained from the South Florida Water Management District (SFWMD) Permit Information Manual Volume IV (1990). A 25-year, 3-day rainfall magnitude of 11 inches was used for the design storm simulation. The SWMM EXTRAN simulation results for a 10-day outflow hydrograph through Cocohatchee east flowway are shown in Figure 5.1.6-2. The simulated results indicate a peak flow of 154 cfs toward Cocohatchee through East Flowway.

The Cocohatchee West Flowway extends from the end of Corkscrew Swamp to Cocohatchee Canal about one mile west of CR 951. The UNET model was used to simulate flow through Cocohatchee West Flowway. The same design storm for a 25-year, 3-day event was used to simulate West Flowway using UNET. The simulated results are shown in Figure 5.1.6-3. The results indicate a peak flow of 594 cfs toward Cocohatchee Canal through West Flowway. Both the flow hydrograph inputs through the improved flowways were incorporated into the UNET hydraulic model of the Cocohatchee-Golden Gate Canal System.

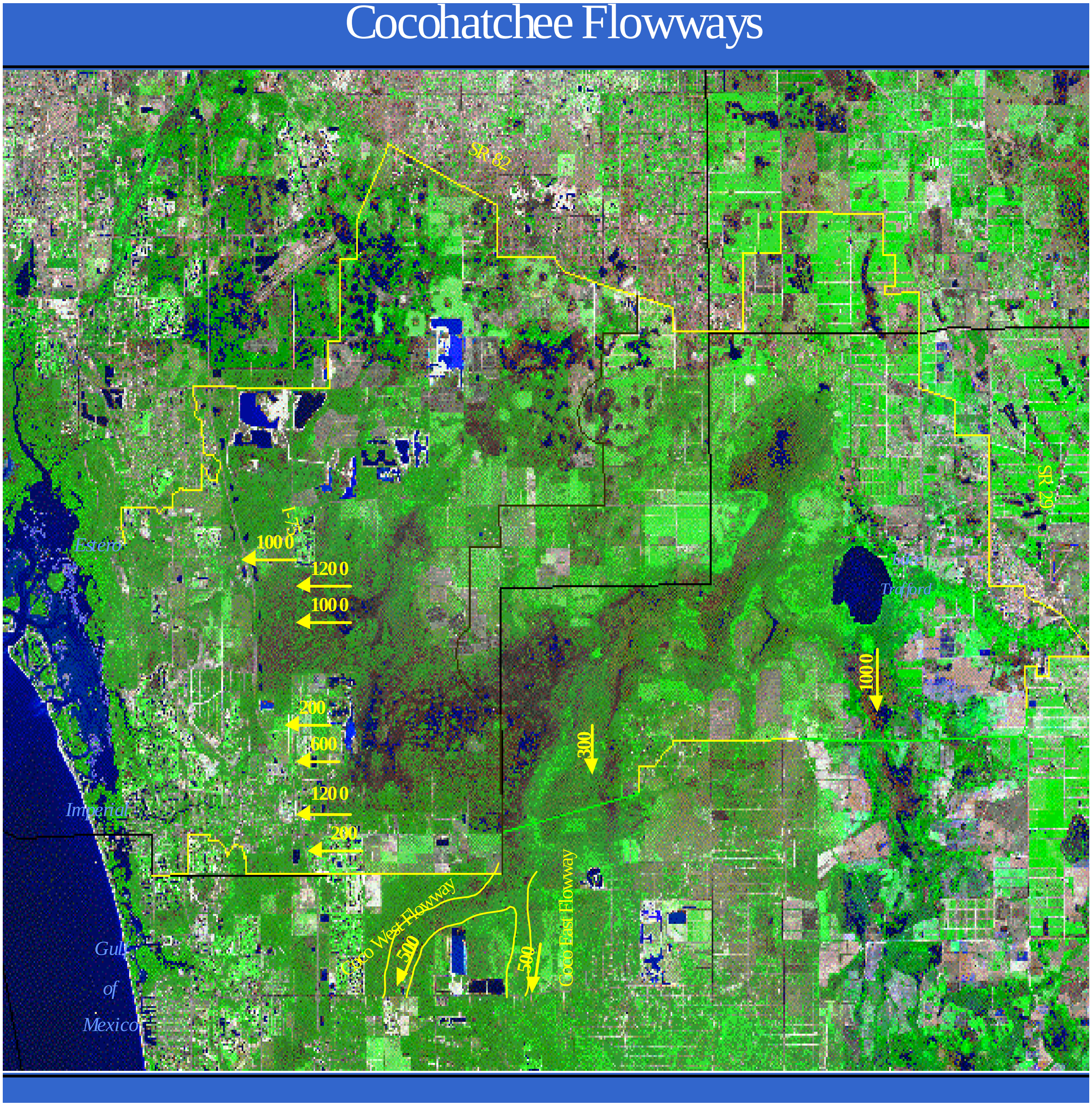


Figure 5.1.6-1 Regional Flowways Conveyance Recommended in the South Lee County Watershed Management Plan

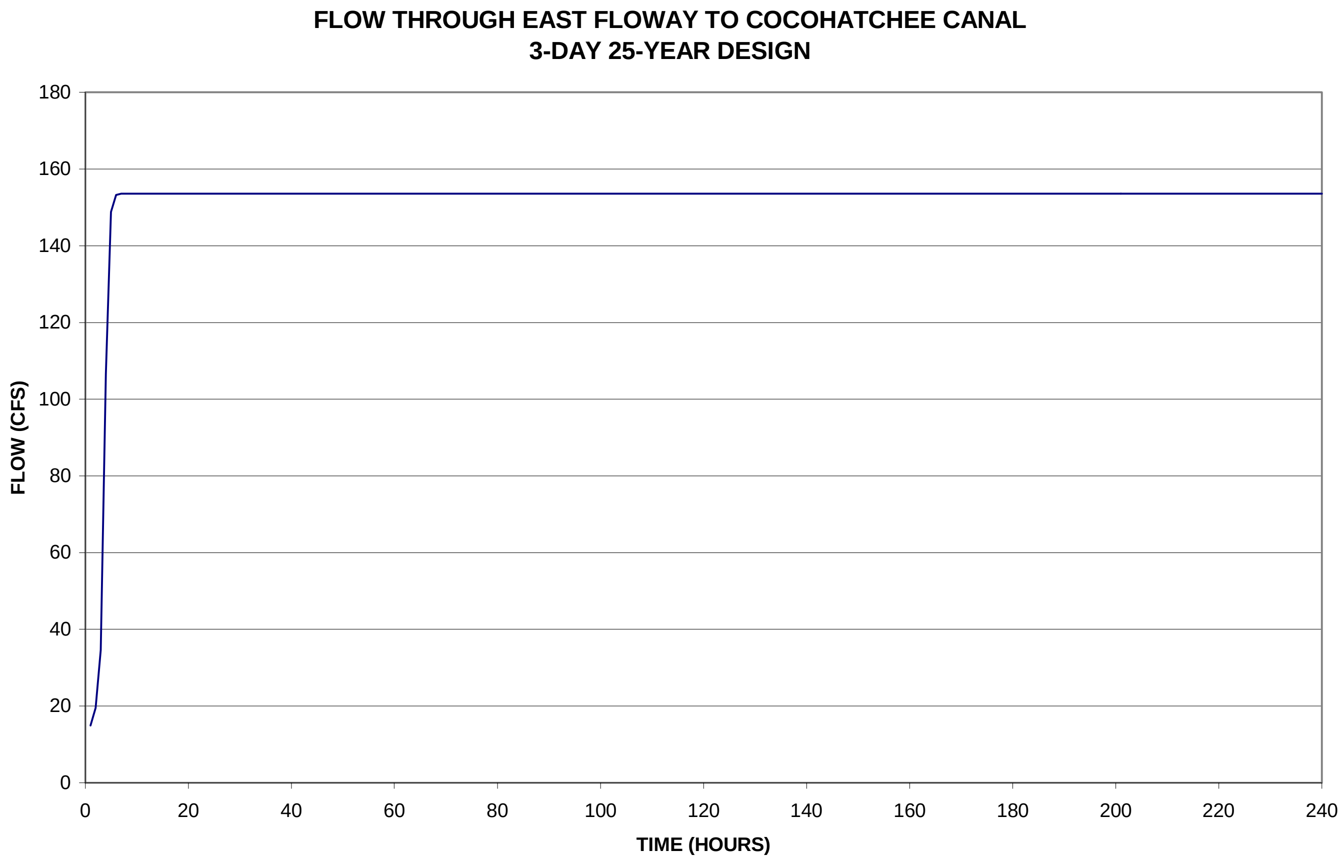

FLOW THROUGH EAST FLOWAY TO COCOHATCHEE CANAL
3-DAY 25-YEAR DESIGN
FLOW (CFS)
180
160
140
120
100
80
60
40
20
0
0
20
40
60
80
100
120
140
160
180
200
220
240
TIME (HOURS)

Figure 5.1.6-2

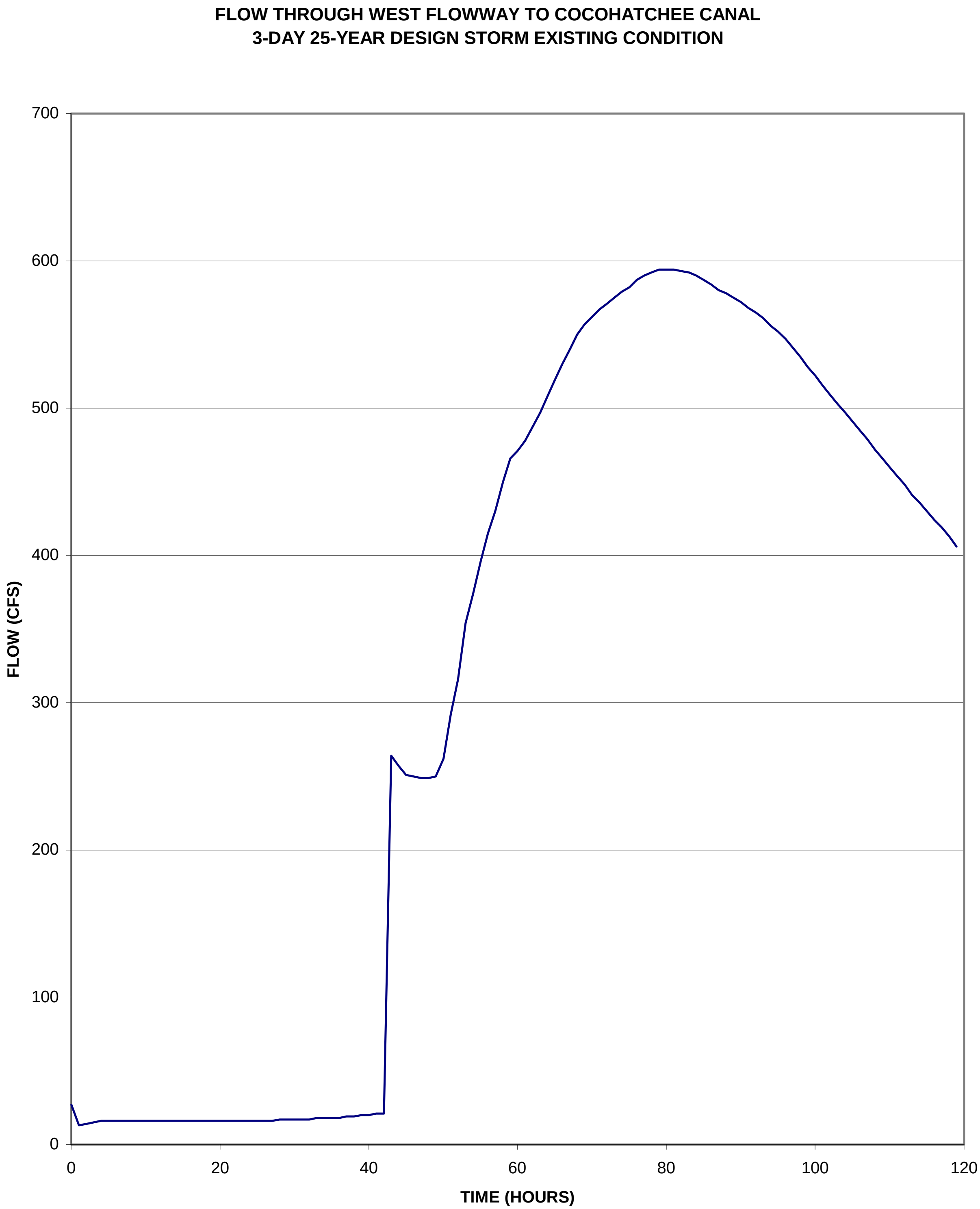

FLOW THROUGH WEST FLOWWAY TO COCOHATCHEE CANAL
3-DAY 25-YEAR DESIGN STORM EXISTING CONDITION
FLOW (CFS)
0
100
200
300
400
500
600
700
0
20
40
60
80
100
120
TIME (HOURS)

Figure 5.1.6-3

5.1.7 Aquifer Storage and Recovery (ASR) of Wet Season flows of Golden Gate Main Canal near CR 951

The District is aggressively exploring a new paradigm of aquifer management that integrates the ability of aquifers to store, treat and convey water, thereby achieving a reliable, cost-effective water supply for all water users, including natural systems. Certain technical issues remain to be resolved, and the regulatory framework requires changes to accommodate this new technology, however, the direction seems clear - aquifer storage is the key to cost-effective, integrated management of seasonally available surface water supplies and groundwater systems.

In 1986 the Big Cypress Basin Board sponsored a study to determine engineering feasibility for implementing ASR to meet seasonal water supply demands in Western Collier County. One recommendations from that study was to consider storing a portion of the wet season flows of the Golden Gate Main Canal near Collier county South Water Treatment Plant, near CR 951. Such a facility would reduce the fresh water shock load to the Naples Bay estuary, reduce canal stages in the reach of the Golden Gate Main Canal West of CR 951, and supplement seasonal water supply demand during the dry season.

According to the ASR well specifications outlined in the Lower West Coast Water Supply Plan (LWCWSP), an 1,800 million gallon facility consisting of a set of 900 feet deep, 16-inch diameter wells are considered to provide 9 MGD for 200 days of the year. Available cost data indicate that typical unit costs for water utility ASR systems now in operation range from $200,000 to $600,000 per MGD of recovery capacity (LWCWSP, 2000). Considering a conservative unit cost of $500,000 per MGD capacity, the cost of the Golden Gate Canal ASR facility is estimated at $4.5 million. The effectiveness of the

ASR in reduction of the design flood profile is illustrated in Figures 5.1.3 - 8 through 12, combined with the element of alternative 3.

5.1.8 Alternative 8: Modification of the C-1 Connector Canal and Relocation of Miller Canal Weir No. 3

C-1 connector canal links the Golden Gate Main canal with Miller Canal at a location approximately one mile south of Golden Gate Boulevard (See Figure 4-1). The canal was not completely excavated to accommodate flow conveyance capacity although the Gulf American Corporation drainage plans envisioned it to be a regular canal with cross-section 14 feet bottom width, 10 feet deep with 1:1 side slope. The dynamically simulated flow and stage hydrographs at Miller No. 3 structure and at the outlet of C-1 Connector Canal to Miller Canal under existing condition for the 3-Day, 10-year, 25-year, 50-year, and 100-year storm events are presented in Figures 5.1.8-1 through 4. A summary of the simulated maximum flows and stages is also presented in Table 5.1.8-1.

Table 5.1.8-1: Simulated Maximum Flow and Stage at the Outlet of C-1 Connector Canal to Miller Canal and at the structure Miller No.3 for 10, 25,50 and 100-year Design Storm

Design Storm	C1-Connector to Miller Canal		Miller No.3 Structure	
	Flow (cfs)	Stage (feet)	Flow (cfs)	Stage (feet)
10-year	106	11.03	20	11.8
25-year	123	11.41	54	12.08
50-year	134	11.75	94	12.34
100-year	147	12.05	141	12.6

Due to its strategic linkage between the Golden Gate and Faka Union Canal Systems, modification of the C-1 Connect Canal and relocation of the existing water control structure Miller No. 3 will provide important water management features for relief of floodwater as well as for Groundwater recharge avenues to the adjacent wellfields of the city of Naples and Collier County Utilities. A more detailed hydraulic analysis need to be performed.

Figure 5.1.8 - 1

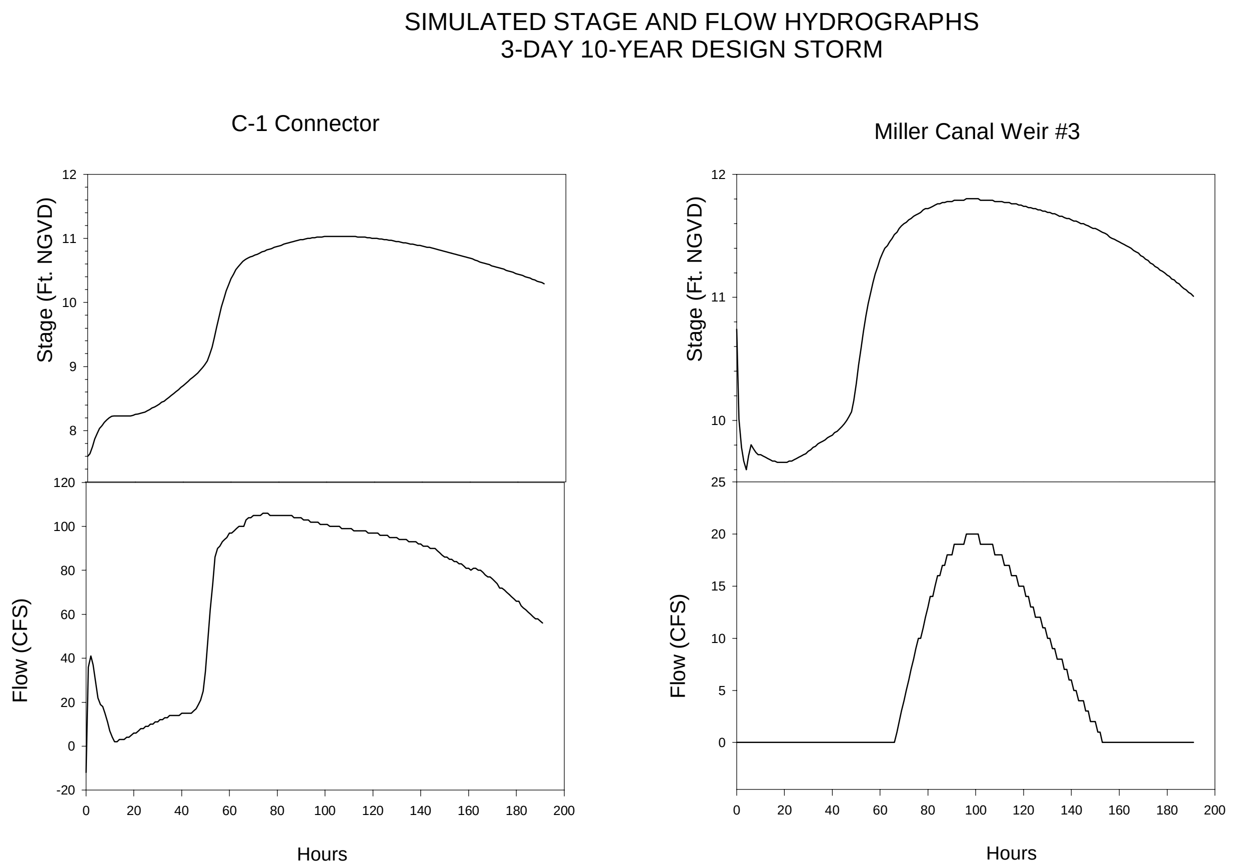
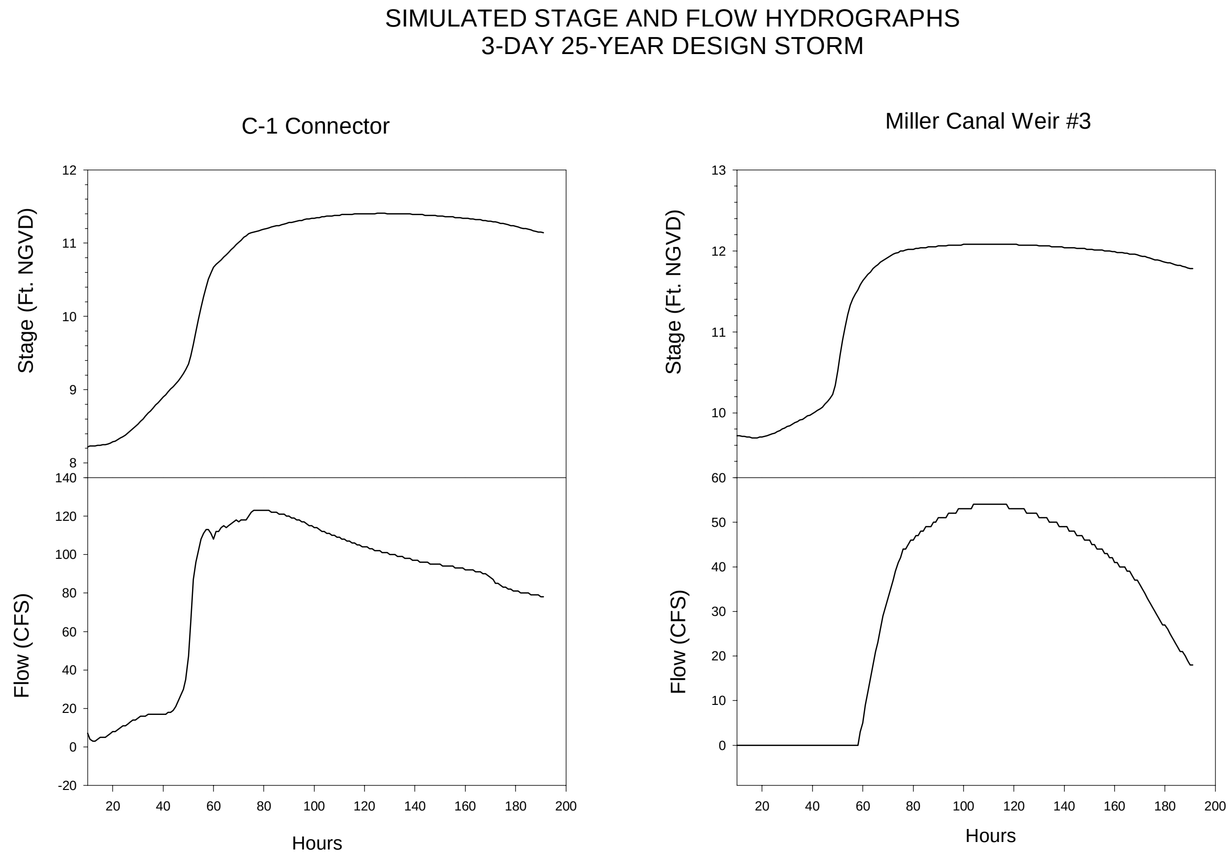

SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 10-YEAR DESIGN STORM
C-1 Connector
Miller Canal Weir #3
Stage (Ft. NGVD)
Flow (CFS)
Hours
12
11
10
9
8
120
100
80
60
40
20
0
-20
25
15
5
0
20
40
60
80
100
120
140
160
180
200

Figure 5.1.8 - 2



SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 25-YEAR DESIGN STORM
C-1 Connector
Miller Canal Weir #3
Stage (Ft. NGVD)
Flow (CFS)
Hours

Figure 5.1.8 - 3

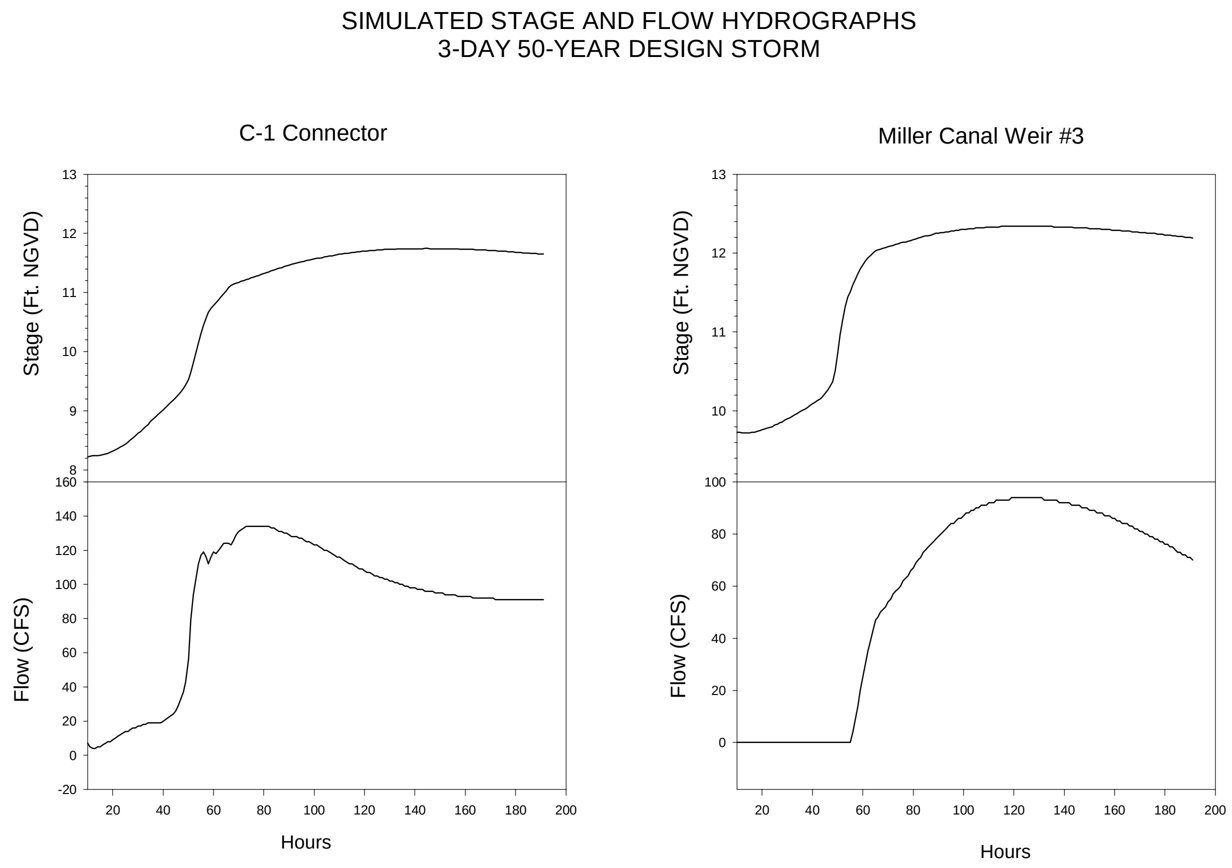
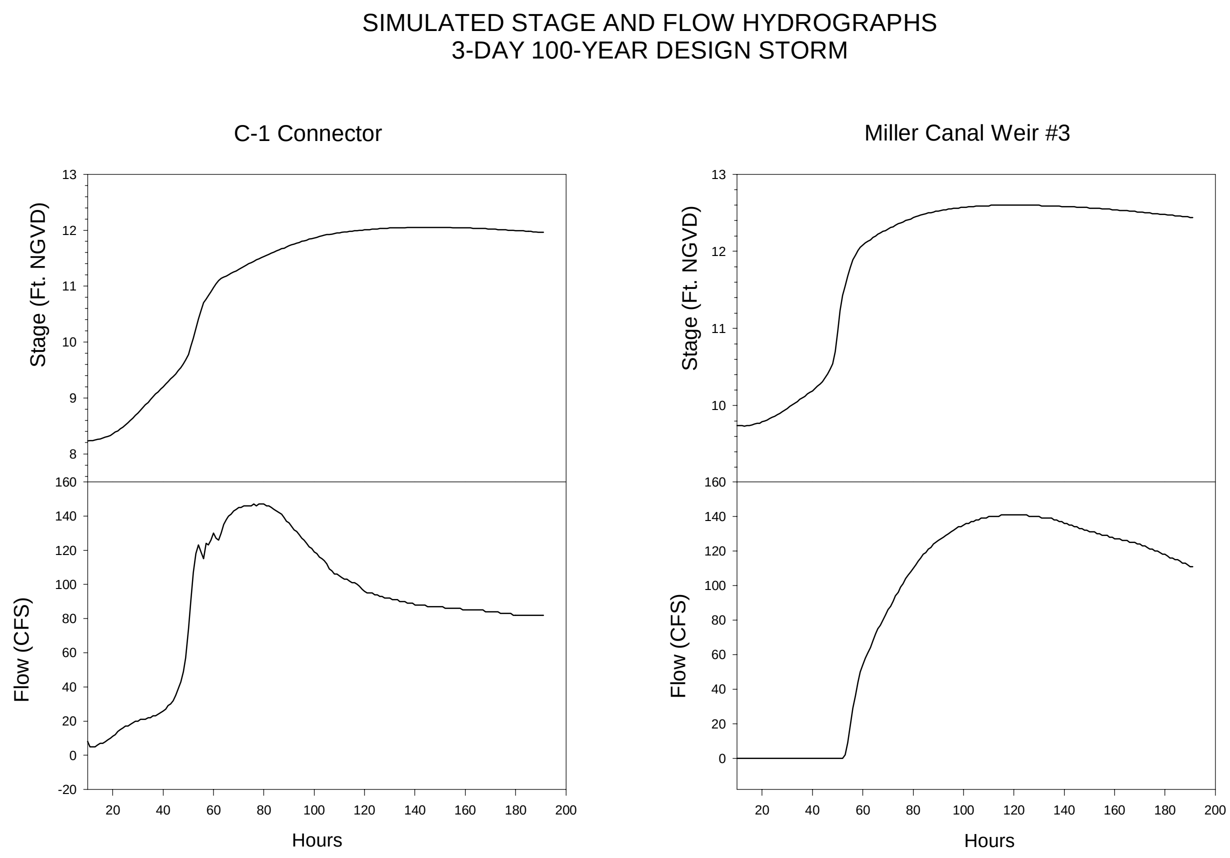

SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 50-YEAR DESIGN STORM
C-1 Connector
Miller Canal Weir #3
Stage (Ft. NGVD)
Flow (CFS)
Hours
13
12
11
10
9
8
160
140
120
100
80
60
40
20
0
-20
20
40
60
80
100
120
140
160
180
200

Figure 5.1.8 - 4



SIMULATED STAGE AND FLOW HYDROGRAPHS
3-DAY 100-YEAR DESIGN STORM
C-1 Connector
Miller Canal Weir #3
Stage (Ft. NGVD)
Flow (CFS)
Hours
13
12
11
10
9
8
160
140
120
100
80
60
40
20
0
-20
20
40
60
80
100
120
140
160
180
200

5.1.9 Alternative 9: Improvement Henderson Creek Canal

Henderson Creek Canal drains the Henderson Creek Basin located in the south central portion of Collier County and is bounded by CR 951 to the west, the Rookery Bay estuary to the southwest, the US-41 Outfall Swale 2 Subbasin to the south, the Seminole Park Outlet Subbasin to the southeast, the Faka Union Canal Basin to the east, and the Golden Gate Basin to the north. The majority of the Henderson Creek Basin is wetlands. The basin is extremely flat and any surface runoff is naturally directed to the southwest where it enters the Rookery Bay estuary via Henderson Creek. The existing canal crossing structures and the potential head losses are summarized in Table 5.1.9-1.

The alternative evaluation process for improvement of the Henderson Creek Canal was accomplished by setting up a HECRAS configuration of the existing canal and the water control structure network. To split and divert flow (from Golden Gate Main Canal), Henderson Creek is connected to the Golden Gate Main Canal as shown in Figure 5.1.9-1. The design flows and boundary conditions are discussed in alternative 3 that includes Henderson Creek for diverting flows from Golden Gate Main Canal. The improvement scenario includes cutting the canal for a bottom width of 30 feet, a side slope 2:1, and a longitudinal slope of 0.5 feet per mile. The quantities of excavation for different reaches of the canal are summarized in Table 5.1.9-2.

The spatially varying effects of the improvements in Golden Gate Main Canal have influence on the hydraulic conditions of Henderson Creek. The simulated hydraulic profiles for the existing and modified conditions are shown in Figure 5.1.9-2. The structural improvements for better drainage have not been considered. It is observed that the cutting and improvement of the channel would achieve proper drainage without significant flooding.

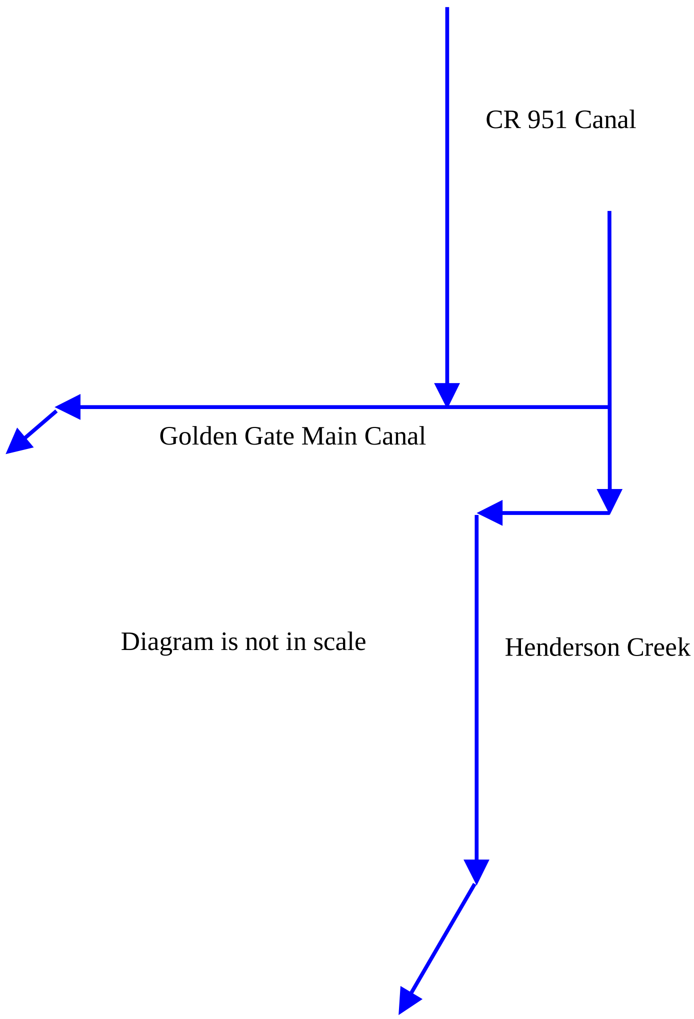


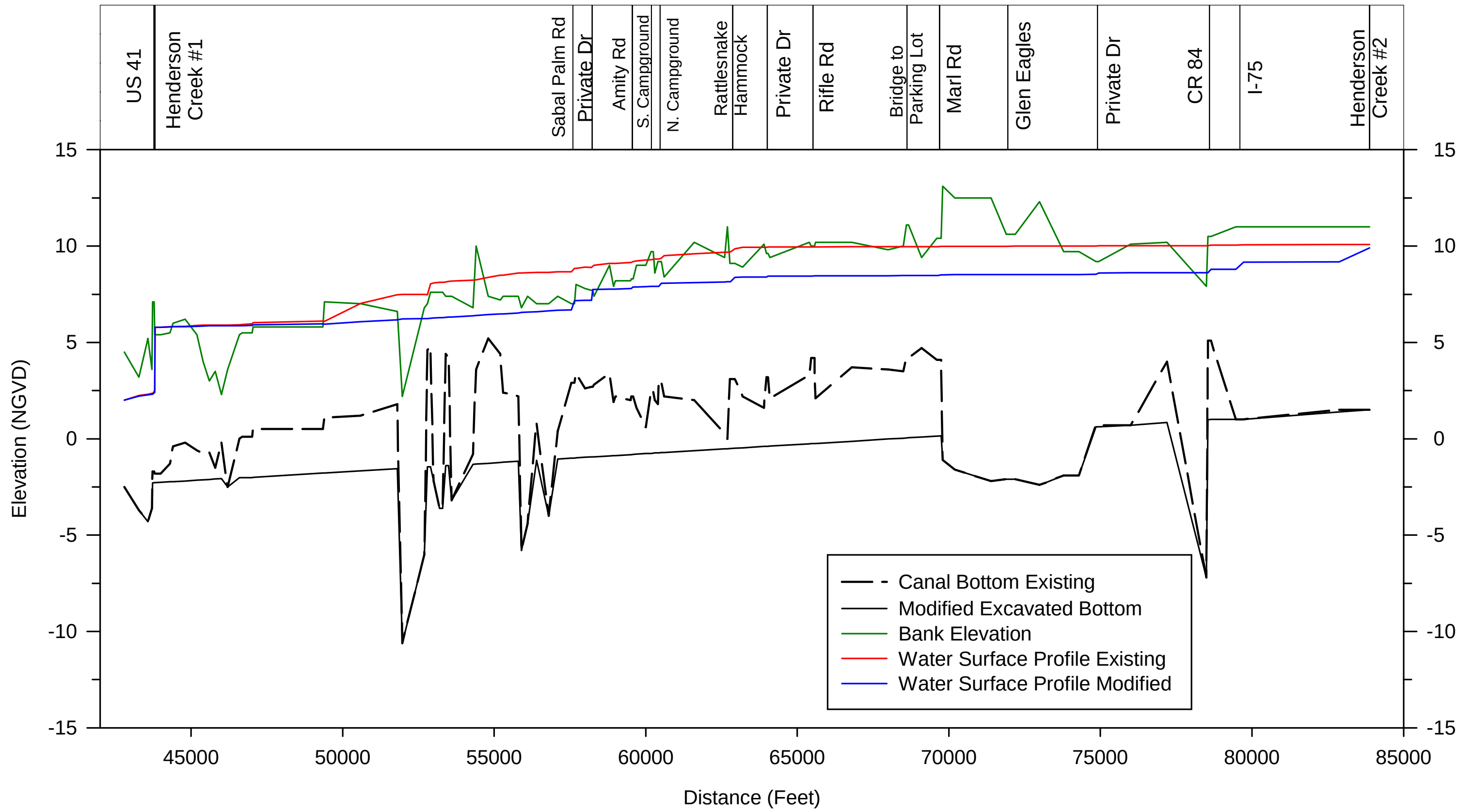
Figure 5.1.9-1. Interconnected reaches of Golden Gate Main and Henderson Creek.

Figure 5.1.9-2

Hec-Ras Results for

Existing and Modified Henderson Creek Canal

3-Day 25-Year Design Storm



File:ShareonBCB Serve\BCB H&H Model..\Figure 2

Table 5.1.9-1. Summary of existing Henderson Creek crossings

Model Section	Approximate miles south of I-75	Location	Structure description	Invert elevation	Potential Head Loss (ft)
79600	0	I-75	One 8' by 8' Box Culvert	5	0.03
78600	0.18	CR 84	One 10' by 6' Box Culvert	5	0.02
74900	0.91	Private Driveway	Three 6' diameter CMP	2.9	0.01
71945	1.47	Glen Eagles	One 24' by 9' Box Culvert	-2.1	0.02
69695	1.88	Marl Road	Five 6' diameter CMP	4.1	0.01
68620	2.07	Bridge to Parking Lot	Bridge with piers	4.2	0
65518	2.66	Rifle Road	One 24' by 8' Box Culvert	3	0
64009	2.95	Private Driveway	Four 5' diameter CMP	3.2	0.01
62868	3.18	Rattlesnake Hammock Road	Five 6' diameter CMP	3.1	0.17
60472	3.62	North Campground	One 20' by 9' Box Culvert	3	0.05
60188	3.67	South Campground	Bridge with piers	2.5	0.02
59557	3.79	Amity Road	Three 10' diameter CMP	2	0.04
58232	4.04	Private Driveway	Four 5' diameter CMP	2.7	0.06
57596	4.17	Sabal Palm Road	One 24' by 9' Box Culvert	3	0.17
43805	6.71	Henderson Creek # 1	One 62' Broad Crested Weir One 4' by 4' Gate One 12' by 4' Gate	5 0.5 3.5	3.26
43773	6.72	US 41	Bridge with piers	-1.7	0.11

Table 5.1.9-2. Summary of Excavation and Cost for Henderson Creek

Canal	Bottom Width (ft)	Side Slope	Excavation Quantity (cubic yard)	Estimated Cost @ $12.00/yd3
Henderson Creek	30	2:1	254,745	$3,056,940

5.1.10 Restore Camp Keais Strand Flowway

Camp Keais Strand is a large natural flowway that conveys water from Lake Trafford to the Fakahatchee Strand State Preserve through the Florida Panther National Wildlife Refuge, and the Southern Golden Gate Estates. The historic flowway of the strand has been altered (constricted and bisected) by many farm roads, ditches, dikes and farm fields, and by the invasion of exotic vegetation like melaleuka and Brazilian peppers.

Beginning at the north end of the strand, there is an existing historic roadway that served as the connection from Immokalee to the Big Corkscrew Island. The elevation of the roadway is approximately two feet higher than the natural ground, with no visible stormwater conveyance structure across it. This northern roadway acts as the first of a series of berms which blocks outflow from Lake Trafford into the Camp Keais Strand. The next major impediment of flow conveyance is the Immokalee Road (CR 846) which has only one 105 feet wide bridge. Although there is a considerable southward gradient during the periods of high lake levels, no significant flow conveyance is achieved due to heavy infestation of emerging vegetation across this bridge.

Downstream of CR 846 there is at least one roadway that connects both sides of the banks of the strand. The next crossing to the south is a 125 feet wide bridge on Oil Well Road (CR 858). Continuing southward down the strand there are also several farm crossings with inadequate cross drainage structures.

As a part of the South Lee County Watershed Management Planning effort, preliminary hydraulic analyses were performed to investigate the feasibility of improving the flowway for reduction of flood stages in the Imperial-Cocohatchee basin. Detailed topographic information for an approximately 70 square-mile area in and adjacent to the strand is being procured in FY 2000. A comprehensive hydraulic analysis will be performed incorporating the new topographic information to assess the effectiveness of this alternative in achieving the desired levels of flood protection in the Imperial-Cocohatchee basin, and the impact on the ecologic functions of the strand.

6 RECOMMENDED PLAN

On the basis of the above hydrologic-hydraulic and economic evaluations, and assessment of the impacts of the ten alternative water management strategies, it is observed that the optimized configurations of Alternatives 2, 3, 4, 7, and 9 prove to be the most feasible solutions for achieving the desired levels of flood protection for **the designated urban areas of the watershed**. The plan will involve channel modification for approximately 6.5 miles of Golden Gate Main Canal, 2 miles of CR 951 Canal, 6.75 miles of the Henderson Creek Canal; construction of a diversion canal connecting Golden Gate Main Canal with Henderson Creek; construction of two low-head water control structures on the CR 951 Canal; retrofit of Golden Gate Canal Weirs No. 1 and 2; and implementation of an Aquifer Storage and Recovery facility near CR 951. A network configuration of the recommended plan is presented in Figure 6-1.

ELEMENTS OF THE RECOMMENDED
FLOOD CONTROL PLAN

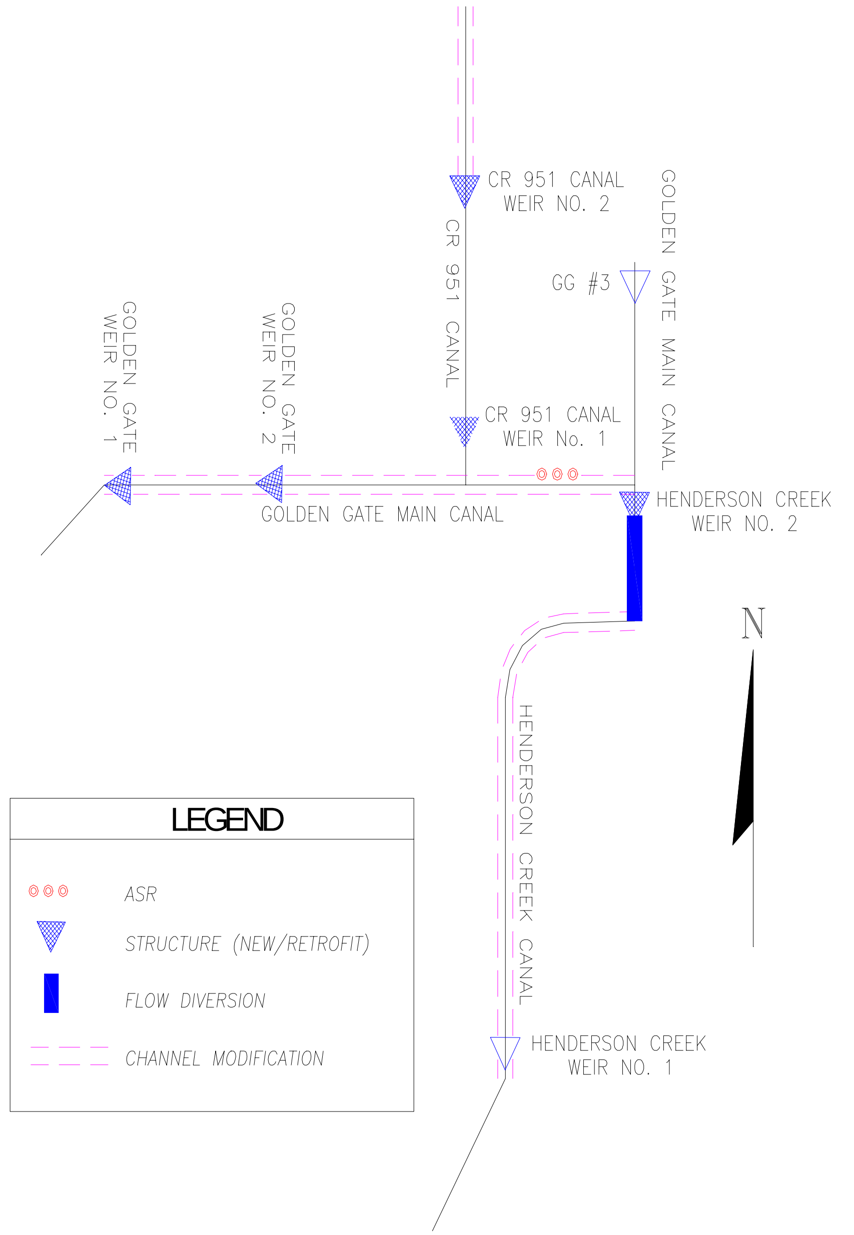


Figure 6-1

Recommended Plan.dwg

The overall estimated cost of the recommended plan elements are as follows:

Golden Gate Main Canal:		
Channel Modification		$4,654,620
Water Control Structure Retrofit		$3,500,000
ASR Facility		$4,500,000
	Total:	$12,654,620
CR 951 Canal:		
Channel Modification		$1,687,000
Water Control Structure Retrofit		$400,000
Culvert Replacement		$1,300,000
	Total:	$3,387,000
Henderson Creek:		
Channel Modification		$3,055,000
Diversion Channel		$1,592,000
Culvert Replacement		$1,400,000
	Total:	$6,047,000

Improved management of surface water through flow diversion, storage, and restoration of the historic flowways will further be analyzed by the application of an integrated SW and GW model. Such an integrated modeling effort would enable quantify impacts on regional water supply, water quality and ecologic functions of wetlands and the receiving estuaries. The components of the SGGE restoration plan, and the on-going critical restoration projects, namely the Tamiami Trail Flow Enhancement Plan, Lake Trafford Restoration, and the Southern CREW Restoration Plans will be incorporated with the recommendations outlined in this plan.

The recommendation outlined in this plan should provide a first step guideline to the Basin Board to revisit the elements of the Basin's short- and long-range capital improvement plans, and to implement them within the purview of the annual budgets.