BIG CYPRESS BASIN WATERSHED MANAGEMENT PLAN

Hydrologic-Hydraulic Assessment
For
Retrofit of Golden Gate Canal Weir #2

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

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Big Cypress Basin

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

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ACRONYMS AND INITIALISMS

BCB	Big Cypress Basin
DHI	Danish Hydraulic Institute
EPA	Environmental Protection Agency
FU-1	Faka Union Weir Number 1
GG-1	GoldenGate Weir Number 1
GG-2	Golden Gate Weir Number 2
GG-3	Golden Gate Weir Number 3
H&H	Hydrologic & Hydraulic
MIKE SHE/ MIKE 11	Integrated Surface Water\Groundwater Modeling System Developed By DHI
NGGE	Northern Golden Gate Estates
SFWMD	South Florida Water Management District
SGGE	Southern Golden Gate Estates
SWMM	Storm Water Management Model
UNET	Unsteady Network Hydraulic Model

EXECUTIVE SUMMARY

The existing Golden Gate Canal Weir No. 2 (GG-2) is a fixed crest weir with two small bottom opening sluice gates (5'x 6'). It is currently incapable of meeting the current water management objectives of dry season storage for water supply, and control of fresh water discharges for water quality protection of Naples Bay. Modification of this weir, including provisions for a more efficient system of operable control gates, will provide management flexibility for water conservation and flood control. Replacing this structure is an element of the Big Cypress Basin (BCB) Five-Year Plan (2002-2006). Partial funding for replacement of the structure has been provided by the Florida Legislature under the Watershed Initiative, 2004.

Surface and groundwater hydrologic assessment of the Golden Gate Canal basin and hydraulic evaluation of the conveyance capacity of the canal and structures were conducted using the BCB integrated surface water and groundwater model developed by the application of Danish Hydraulic Institute, Inc.'s (DHI) MIKE SHE\MIKE 11 modeling system. After consideration of various types of structural alternatives, a fully gated spillway with Obermeyer gates was found to be the most efficient configuration for replacement of GG-2. An assessment of the level of flood protection and general water management functions of existing GG-2 and structural modification of the proposed GG-2 were conducted by continuously simulating the hydrologic-hydraulic responses for an average hydrologic year and for the design storm event. The proposed GG-2 is designed to convey the 25-year, 5-day storm event discharge with no rise in water level beyond the existing conditions while being able to store additional water in canal and recharge the groundwater during the normal condition, and reduce the shock load of fresh water discharge to Naples Bay.

INTRODUCTION

1.1 BACKGROUND AND WATER MANAGEMENT PROBLEMS

The Big Cypress Basin of the South Florida Water Management District (District) operates and maintains 169 miles of primary canals and 46 water control structures throughout its Collier County service area. These facilities provide avenues for flood protection, enhancement of water supply and improved environmental quality. Since the early 1980s, the BCB has adopted an aggressive program to modify the water control structures in the Golden Gate Canal and its tributaries to achieve better water management objectives. The Golden Gate Canal Weir No. 2 (GG-2) has been found to be deficient in providing the desired levels of service for flood protection and conservation storage. A full-scale retrofit of the structure is outlined in the BCB five-year plan.

The present Golden Gate Canal Weir No. 2 structure is located approximately 500 feet east from the southern terminus of 64th Street SW off Golden Gate Parkway. The GG-2 is a fixed crest weir with two bottom opening sluice gates (Figure 1-1). The structure was retrofitted from a V-notch weir to a gated structure in 1985. The current GG-2 is incapable of meeting the current water management objectives of dry season storage for water supply, as well as control of fresh water discharges for water quality protection of Naples Bay. Modification of this weir, with provisions for a more efficient system of operable control gates, will provide better management flexibility for water conservation and flood control.

Replacing this structure is a part of the BCB's Five-Year Plan (2002-2006). The purpose of this assessment is to evaluate the hydraulic performance of alternative structural configurations and estimate the size of an economically feasible water control structure that will achieve the water management objectives of the Golden Gate Canal basin.

Figure 1-1: Existing Golden Gate Canal Weir No. 2

The analysis will incorporate present integrated surface and groundwater systems modeling of the existing hydrologic and hydraulic (H&H) conditions of Golden Gate Canal watershed and simulate different scenarios to evaluate hydraulic performance for the development of an economically and environmentally sound plan to retrofit GG-2. The H&H analysis will provide information on the response of improved GG-2 structure to average hydrologic conditions, design flows in terms of flood control and environmental quality protection.

1.2 OBJECTIVES

The objectives for this H&H study are as follows:

- Evaluate hydraulic performance of the existing GG-2
- Evaluate hydraulic performance of the proposed GG-2
- Demonstrate no adverse impact on flood protection in the Golden Gate Canal watershed

- Regulate and reduce the rate of fresh water discharge to the Naples Bay estuary
- Increase conservation storage for aquifer recharge during the dry season

2.0 HYDROLOGIC & HYDRAULIC MODELING

2.1 BASIN PHYSICAL FEATURES

The Golden Gate Main Canal is located in the west-central portion of Collier County. The canal system in the Golden Gate basin was built in 1960s to drain the lands for residential development in the rural area known as Golden Gate Estates. The canal drains approximately 120 square-miles, with primary land uses of agriculture, rural and urban residences and commercial development. The Golden Gate Main Canal basin is bounded by the Corkscrew-Cocohatchee basin to the north, the Gordon river Extension basin to the northwest, the District VI basin to the south, the Henderson Creek basin to the southeast, and the Faka Union Canal basin to the east (Figure 2-1). The canal flows generally southwest into Naples Bay. Presently, seven water control structures in the Golden Gate Main canal provide a controlled step-down of the water level to prevent overdrainage of the interior lands. In addition, many canals of its tributary network, namely Golden Gate side branch, Cypress, Harvey, I-75, Corkscrew, CR 951, and Airport Road canals, also have operable water control structures.

The GG-2 structure captures runoff from approximately of 109 square miles. Despite the low-relief terrain of the Golden Gate Canal basin, natural surface drainage is controlled by topography. The Immokalee Rise provides the high point for the basin where drainage begins to flow towards the southwest, and then flows in a more southerly and then westerly direction towards the Naples Bay. Ground elevations range from approximately 23 feet NGVD, in the northeastern end, to nearly 6 feet NGVD near GG-1.

2.2 MODELING METHODOLOGY

The Golden Gate Canal watershed is typical of Southwest Florida hydrology, with low relief and high water table conditions. An extensive network of drainage canals and water control structures regulate the surface and groundwater flow patterns.

Golden Gate Canal Weir #2 Drainage Area

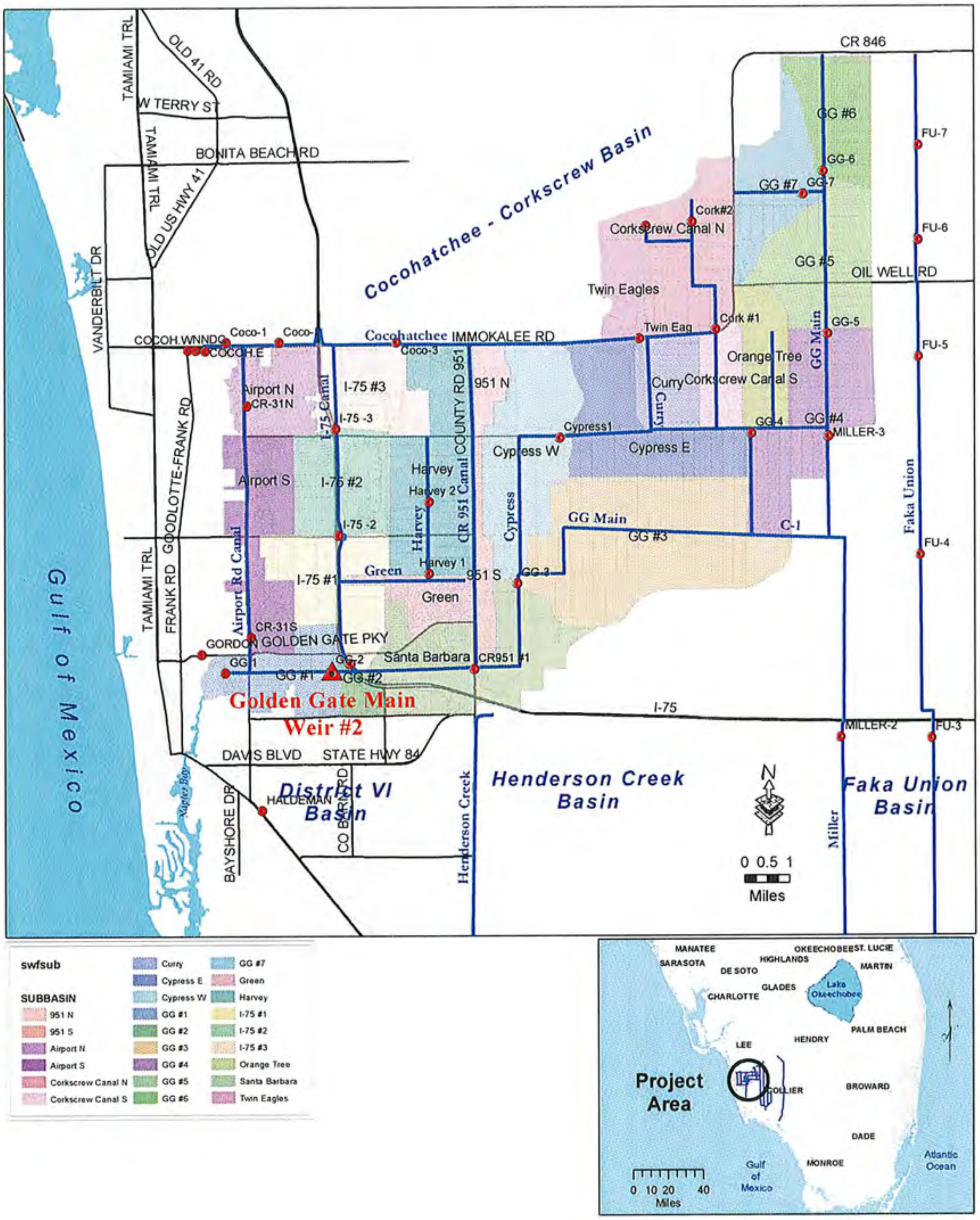


Figure: 2-1

There is significant interaction between surface water and groundwater. A set of regional hydrologic-hydraulic models were previously developed by applying the United States Environmental Protection Agency's (USEPA) Storm Water Management Model (SWMM) and the U.S. Army Corps of Engineers' unsteady state hydraulic network model (UNET) (Dames & Moore, 1998). However, the SWMM-UNET combination of models is primarily geared toward simulating the rainfall-runoff process and flood routing in open channels. Their effectiveness in assessing the effects on water supply, groundwater recharge and wetland functions is limited without the application of an integrated surface water/groundwater model.

An integrated hydrologic-hydraulic model for the BCB regional watershed was developed by the DHI Inc. to assess the impact of water management strategies on flood dynamics, wetland water levels and water supply (Christierson, 2002; DHI, 2004). The model is based on an integrated, physically distributed hydrologic modeling system – MIKE SHE, which simulates overland flow, unsaturated zone flow and groundwater flow dynamically, coupled with a river hydraulics model, called MIKE 11. The domain of the BCB model covers an area of 1194 square miles. The model is defined in State Plane 1983 Florida East coordinates and NAVD 1988, and it further subdivided spatially into 15,060 cells, with a grid dimension of 1500 feet by 1500 feet (Figure 2-2).

The integrated modeling approach provides a physical representation of the flow processes as opposed to the lumped parameter rainfall-runoff simulation process. The H&H components included in the BCB model are as follows:

- Overland sheet flow and depression storage
- Infiltration and storage in the unsaturated zone
- Groundwater flow, storage and potential heads
- Open channel flow and water levels
- Drainage effects
- Irrigation water allocation distribution
- Dynamic exchange between the unsaturated zone-groundwater (recharge)

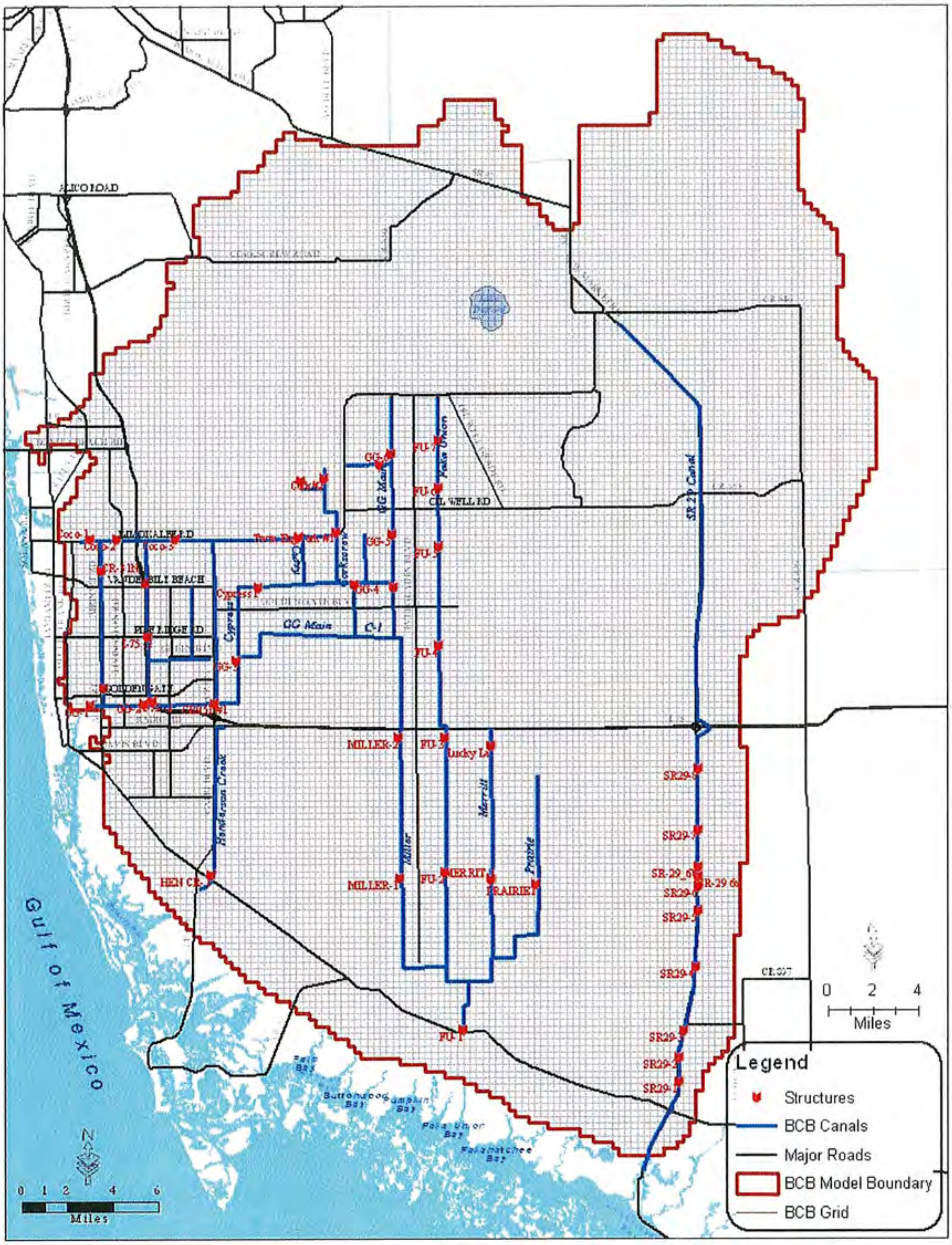


Figure 2-2: BCB Model Boundary and Major Canal Systems in the Model

The MIKE SHE/MIKE 11 modeling system couples several partial differential equations that describe flows in the overland, channel, the saturated zone and unsaturated zone to simulate the integrated process of all the principal components of the hydrologic regime, including the correlation between ground and surface waters. The physically-based flow equations to be solved include the following: (1) one-dimensional Saint-Venant flow equations for surface flow processes; (2) two-dimensional diffusive wave for overland flow; (3) one-dimensional Richard equation for unsaturated vertical infiltration; and (4) three-dimensional Boussinesq equation for saturated groundwater flow. Different numerical solution schemes are then used to solve the partial differential equations for each process. A solution to the system of equations associated with each process is found iteratively by use of different numerical solves. A schematic representation of the complete water resources system that is represented by MIKE SHE/MIKE 11 interaction model is shown in Figure 2-3.

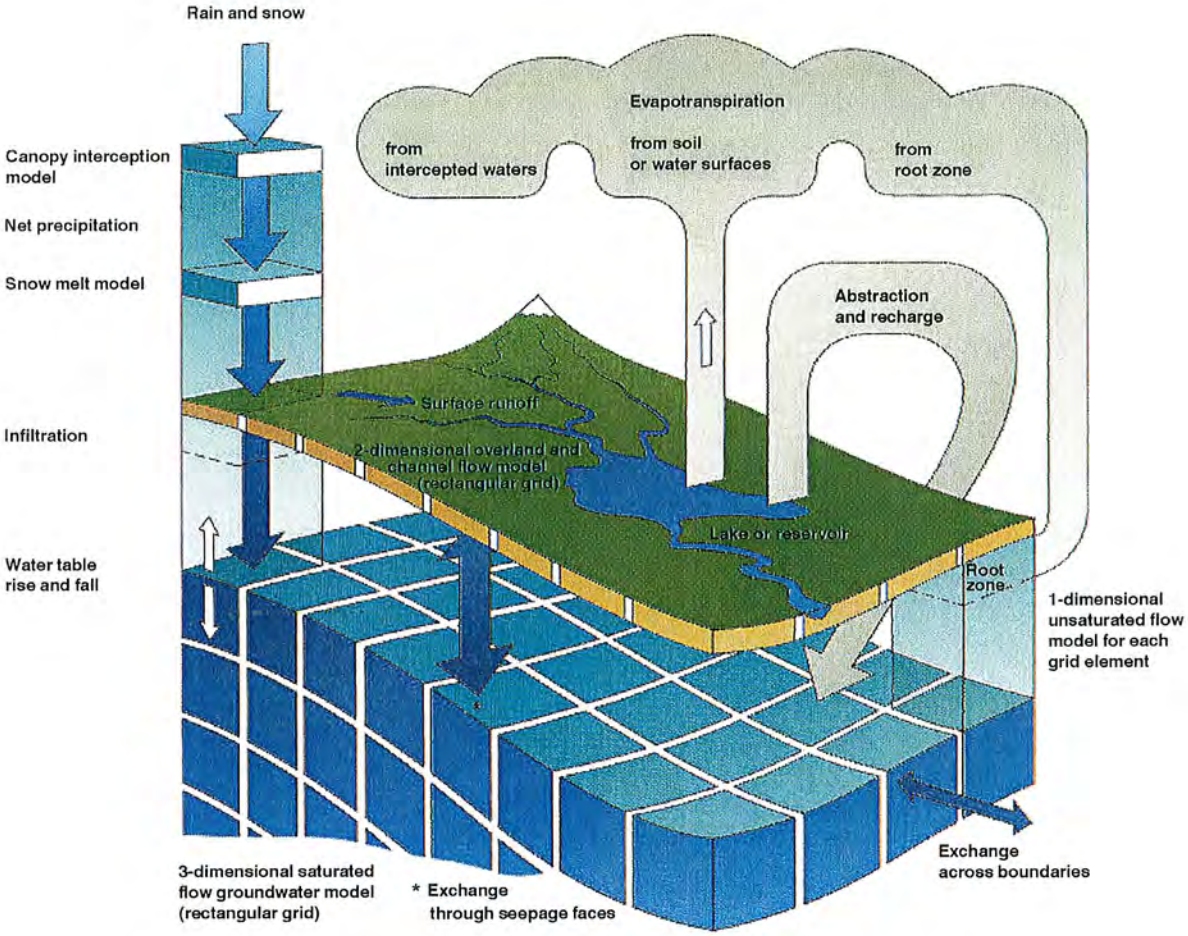


Figure 2-3: A General Configuration of an Integrative Groundwater-Surface Water Model

2.3 SURFACE WATER HYDROLOGIC MODEL

The overland flow component of the MIKE SHE model represented the rainfall-runoff processes, including the unsaturated zone and the interaction between groundwater and surface water. The BCB overland flow model was set up to simulate both surface water runoff and groundwater influence for drainage areas located in the BCB. The ground surface elevation was interpolated to 1500 feet grid based on topography generated from USGS quadrangle data and further enhanced by topographic data obtained by aerial photogrammetry, LIDAR data (2000) from Collier County and USACE cross section surveys gathered in the Golden Gate Estates (2003). The interpolated topographic digital grid input map was used to develop the conceptual model for the overland flow simulated using MIKE SHE. The topographic coverage of the BCB area describes the overland flow processes in MIKE SHE. The MIKE SHE generated overland flows acted as distributed sources for the MIKE 11 channel routing model. The topographic map used in the MIKE SHE model is shown in Figure 2-4.

In the MIKE SHE model, surface runoff occurs when water starts ponding on the surface, due to insufficient infiltration capacity of the underlying soil, proximity of the groundwater table near the ground surface, or existence of drainage flows from low-lying areas. The overland flow in MIKE SHE uses a 2-D diffusive wave approximation for computing hydrologic components, dependent on ground surface slope, surface roughness, and detention storage. These parameters are described in detail in the reports prepared by DHI (2001, 2004).

The driving forces for the integrated hydrologic model are rainfall and evapotransporation. Rainfall on the west coast of Florida, including the BCB, is typically dominated by local weather phenomena. Continuous records of rainfall for the BCB and neighboring area are available at 20 rainfall stations (Figure 2-5) for a 13-year period (1988-2000). The measured rainfall from the 20 stations (Figure 2-5) was spatially distributed using the triangulation method, Triangular Irregular Network – 10 (TIN –10). This method divides daily rainfall estimates into 2 mile by 2 mile grid cells, and then 10 by 10 sub-cells, thus the sub-cell size becomes 1056 feet by 1056 feet, as illustrated in Figure 2-6.

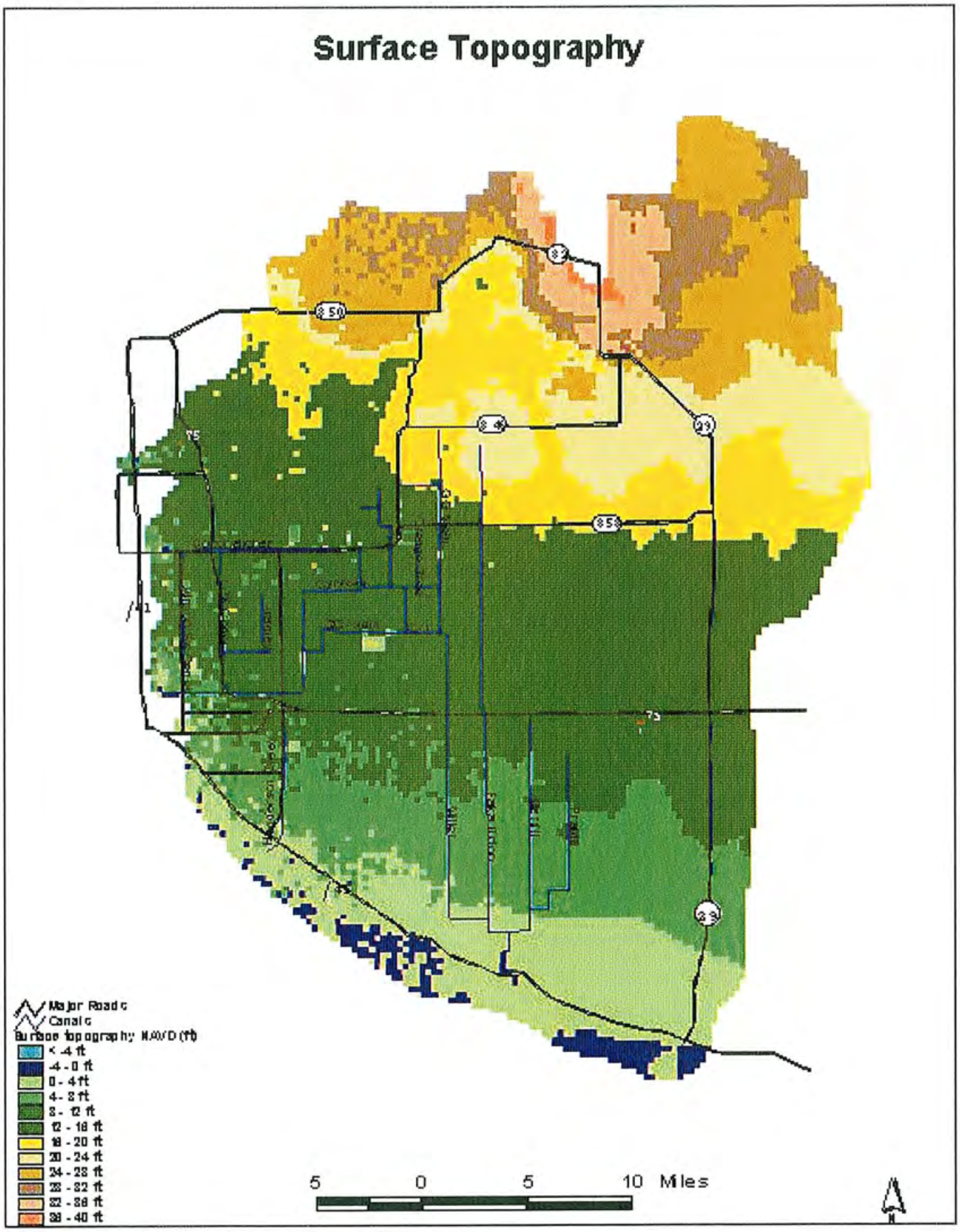

Surface Topography
850
82
884
29
858
75
41
75
29
Major Roads
Canals
Surface topography NAVD (ft)
< -4 ft
-4 - 0 ft
0 - 4 ft
4 - 8 ft
8 - 12 ft
12 - 16 ft
16 - 20 ft
20 - 24 ft
24 - 28 ft
28 - 32 ft
32 - 36 ft
36 - 40 ft
5
0
5
10 Miles
N

Figure: 2-4

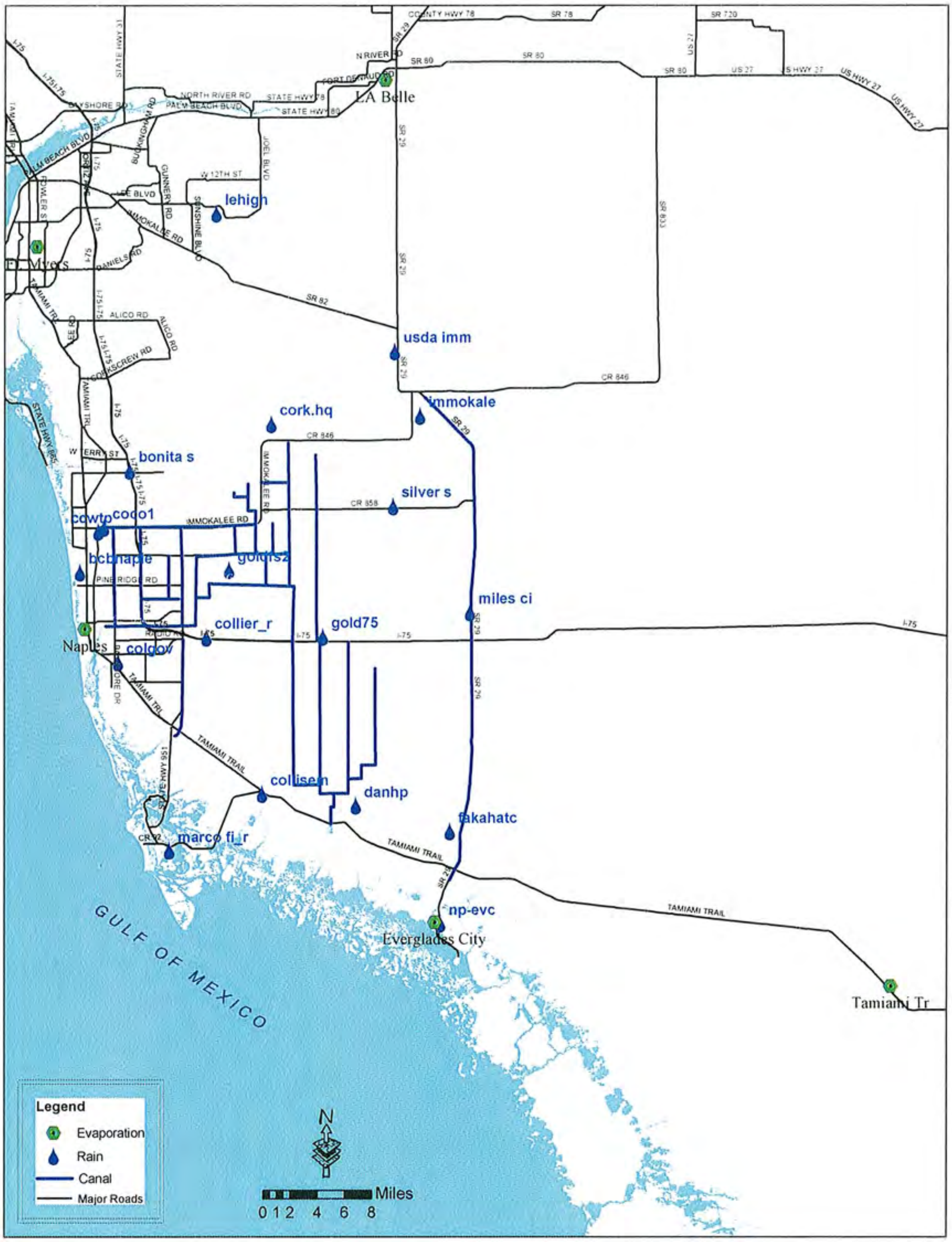


Figure 2-5: Rainfall and Evaporation Stations

Spatial Distribution of Rainfall using TIN-10

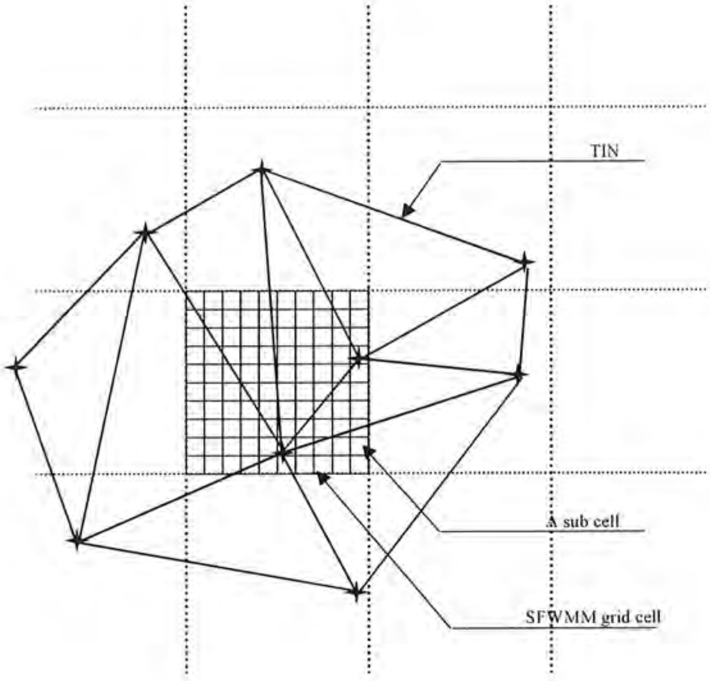


Figure 2-6

For a given day, a TIN is built whose vertices are rain gauge locations with non-missing values. For a given 2 mile by 2 mile grid cell, the above TIN is used to interpolate rainfall at the centroids of each of the 100 sub-cells covering that cell. The average of the 100 rainfall values is represented as the daily rainfall for a 2 mile by 2 mile cell. The operation is repeated to generate daily rainfall records for the entire model period.

Evapotranspiration (ET) accounts for the bulk of water loss from the modeling area. Water is lost to the atmosphere by evaporation from plant surfaces, free water surfaces, soil evaporation, and through transpiration from the plant root zone thereby reducing water available for runoff and groundwater flow. Table 2-1 gives vegetation parameters used by MIKE SHE to calculate actual evapotranspiration.

The potential ET for the BCB model was calculated by the SFWMD Simple Method, which computes the long-term historical (1965-2000) wet marsh potential ET from the evaporation stations in the model domain (Figure 2-5). Due to the difference in the roughness characteristics between marsh and grass surfaces, the crop coefficients developed were modified for use with wet marsh potential ET. Additionally, five National Oceanic and Atmospheric Administration (NOAA) stations with long-term (1965-2000) daily temperature data were thoroughly checked and patched to correct systematic errors, trends and missing values with the purpose of producing the best possible temperature dataset for ET estimates. The spatial distribution of the wet marsh potential ET values for the model domain was estimated by the TIN-10 method across the five evaporation stations. A summary of the statistics of the wet marsh potential evapotranspiration for those NOAA stations is shown on Table 2-2.

The existing condition land use distribution map used in the BCB MIKE SHE model is shown in Figure 2-7-A. The map was developed from SFWMD 2000 land use GIS coverage. The 2000 land use coverage contains 300 different land use types, simplified into 23 vegetation cover classes that are hydrologically different. Table 2-3 is the table used to convert the SFWMD 2000 land use into the MIKE SHE grid distribution map. Figure 2-7-B is the future land use map applied to the future condition analysis.

Table 2-1: Vegetation Parameters

Model Land Use Type	Growth Period	Leaf Area Index (-)	Root Depth (mm)	Crop Coef. Kc (-)	AROOT
Citrus	All year	4.5	1250	0.77-0.9047	0.25
Pasture	All year	3-4	750	0.7	0.5
Sugar Cane	All year	1-6	500-1500	0.665-1	0.25
Urban Low Density	All year	1-2	200	0.552-0.777	0.5
Urban Medium Density	All year	0.5-1	200	0.552-0.777	0.5
Urban High Density	All year	0.1-0.2	200	0.552-0.777	0.5
Truck Crops	All year	3-4.5	152-750	0.561-1	0.5
Golf Course	All year	2-3	750	0.552-0.777	0.75
Bare Ground	NA	0	0	1	0.25
Mesic Flatwood	All year	1.5-3	1219	0.246-0.82	1
Mesic Hammock	All year	2.5-4	610	0.246-0.82	1
Xeric Flatwood	All year	1-2	1219	0.221-0.738	0.5
Xeric Hammock	All year	2-3	610	0.221-0.738	0.5
Hydric Flatwood	All year	1.5-3	1219	0.237-0.79	1.5
Hydric Hammock	All year	2.5-4	610	0.237-0.711	1.5
Wet Prairie	All year	1.5-3	152	0.225-0.75	2
Dwarf Cypress	All year	1-2	152	0.22-0.734	1
Marsh	All year	2-4	152	0.254-0.845	2
Cypress	All year	2-4	1524	0.237-0.79	1
Swamp Forest	All year	3-5	1524	0.237-0.79	1
Mangrove	All year	3-4	1524	0.271-0.904	1
Water	NA	0	0	1	0.25

Table 2-2: Annual Time Series and Summary Statistics of Wet Marsh Potential Evapotranspiration in Inches Estimated at 5 NOAA Stations

Year	**La Belle**	**Ft Myers**	**Naples**	**Everglades City**	**Tamiami Trail**
1965	56.57	57.96	59.53	62.05	60.80
1966	54.92	56.94	57.94	60.51	56.16
1967	58.40	56.46	59.36	60.73	63.63
1968	57.37	57.70	58.36	60.22	59.78
1969	56.72	53.86	58.11	60.46	56.65
1970	58.85	55.86	60.22	58.52	53.54
1971	61.77	57.34	61.43	60.25	61.22
1972	59.76	59.32	60.88	58.41	58.83
1973	57.06	59.23	61.91	60.27	59.57
1974	58.07	59.90	62.95	60.58	60.10
1975	58.97	59.61	62.70	58.42	59.04
1976	57.73	59.14	62.31	60.21	56.12
1977	58.69	57.89	61.44	59.61	57.40
1978	58.38	57.57	59.82	59.58	55.98
1979	56.35	57.93	60.48	57.97	58.29
1980	57.67	58.56	60.36	58.80	59.75
1981	59.41	60.05	63.16	60.43	62.67
1982	55.33	56.76	60.70	57.69	60.47
1983	54.48	54.26	59.79	57.51	57.95
1984	55.53	56.73	58.12	60.35	56.93
1985	56.87	58.30	57.75	60.30	61.93
1986	56.85	59.85	58.34	61.27	57.20
1987	55.08	58.74	56.96	60.21	56.57
1988	56.33	60.61	58.36	63.59	57.99
1989	57.56	61.41	58.70	56.99	64.46
1990	56.37	60.83	58.71	56.90	63.73
1991	55.61	58.12	56.90	59.62	59.45
1992	54.66	58.23	57.35	57.69	59.79
1993	54.35	57.82	57.95	60.45	54.22
1994	56.24	57.11	55.85	59.39	56.36
1995	54.83	55.46	55.62	58.75	54.22

Year	**La Belle**	**Ft Myers**	**Naples**	**Everglades City**	**Tamiami Trail**
1996	54.60	57.27	58.11	62.45	58.31
1997	55.18	59.45	56.89	59.47	57.63
1998	53.60	56.51	56.33	56.20	56.44
1999	56.08	57.63	56.67	57.31	56.16
2000	55.22	58.85	57.49	58.12	56.67
Ann Ave	56.71	58.04	59.10	59.48	58.50
Stdev	1.81	1.71	2.07	1.63	2.70
Max	61.77	61.41	63.16	63.59	64.46
Min	53.60	53.86	55.62	56.20	53.54
Kr	0.158	0.179	0.176	0.190	0.179

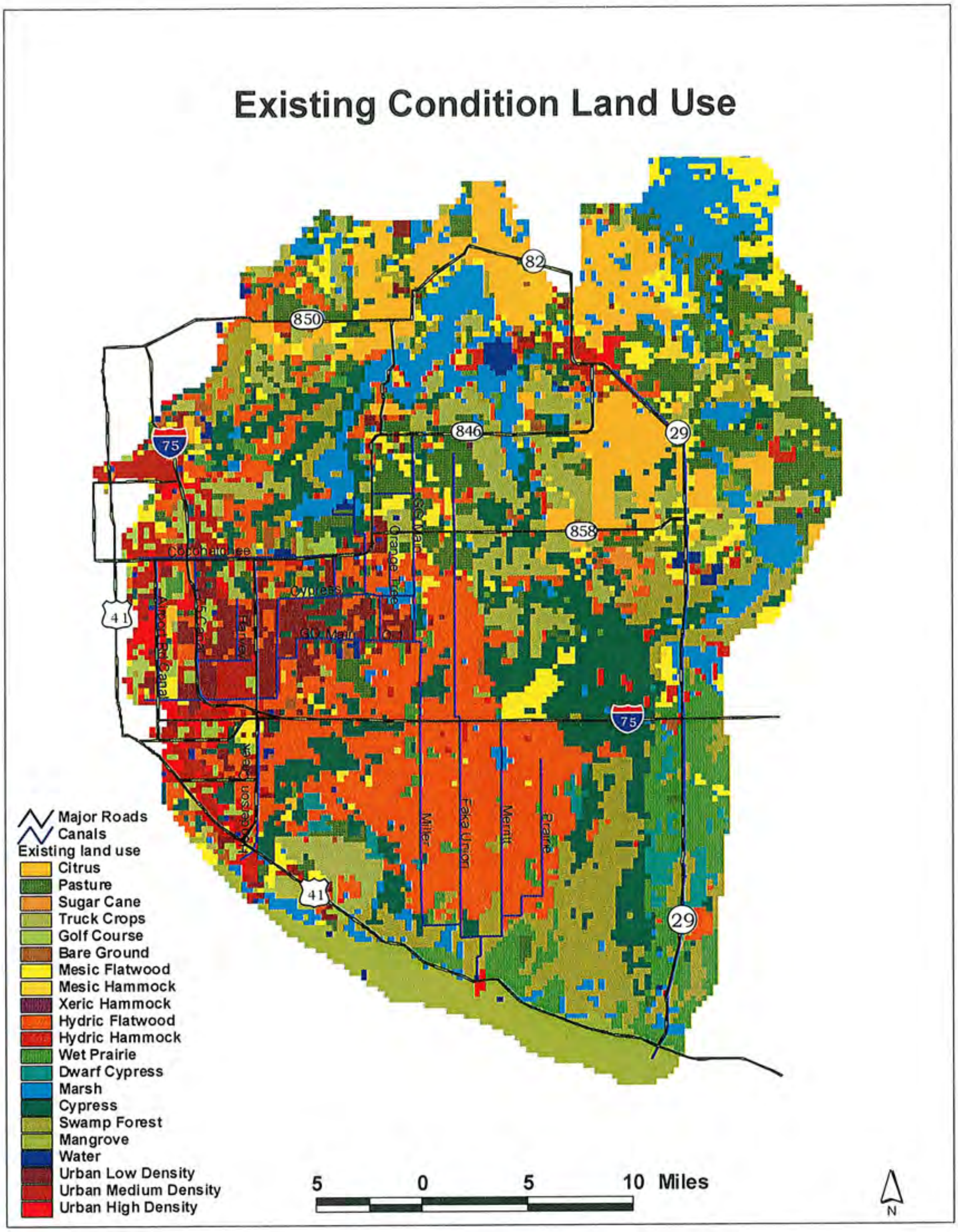

Existing Condition Land Use
82
850
846
29
75
858
41
Cocohatchee
Cypress
GG Main
Orange Tree
GG Main
Airport Rd Canal
I-75 Canal
Harvey
Henderson Creek
Miller
Faka Union
Merritt
Prairie
Major Roads
Canals
Existing land use
Citrus
Pasture
Sugar Cane
Truck Crops
Golf Course
Bare Ground
Mesic Flatwood
Mesic Hammock
Xeric Hammock
Hydric Flatwood
Hydric Hammock
Wet Prairie
Dwarf Cypress
Marsh
Cypress
Swamp Forest
Mangrove
Water
Urban Low Density
Urban Medium Density
Urban High Density
5
0
5
10 Miles
N

Figure 2-7-A

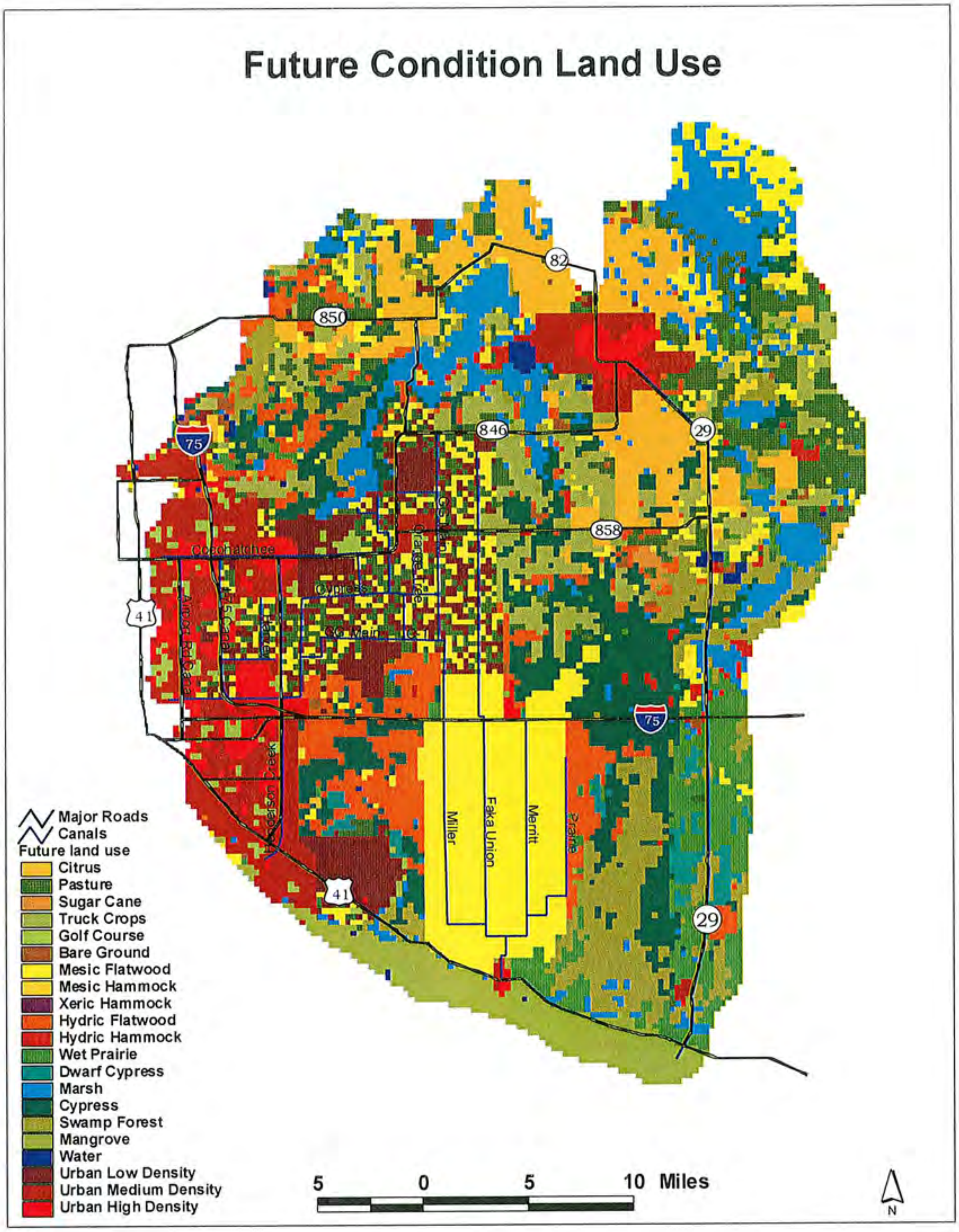

Future Condition Land Use
82
850
846
29
858
75
41
Cocohatchee
Cypress
GG Main
Miller
Faka Union
Merritt
Prairie
Major Roads
Canals
Future land use
Citrus
Pasture
Sugar Cane
Truck Crops
Golf Course
Bare Ground
Mesic Flatwood
Mesic Hammock
Xeric Hammock
Hydric Flatwood
Hydric Hammock
Wet Prairie
Dwarf Cypress
Marsh
Cypress
Swamp Forest
Mangrove
Water
Urban Low Density
Urban Medium Density
Urban High Density
5 0 5 10 Miles
N

Figure 2-7-B

Table 2-3: Land Use Types in the Model and Corresponding FLUCCS Codes

Model Land Use Type	**MIKE SHE Code**	**FLUCCS Code (Level)**
Citrus	1	220
Pasture	2	210 (3), 242
Sugar Cane	3	2156
Urban Low Density	41	110 (2), 180 (2), 192, 193, 240 (3), 241, 243, 245, 246, 250 (2)
Urban Medium Density	42	1009, 120 (2), 144, 833, 834
Urban High Density	43	130 (2), 140 (2), 150 (3), 151, 155, 170 (2), 810 (2), 820 (2), 830 (2), 152, 153, 154, 159
Truck Crops	5	214, 215
Golf Course	6	182
Bare Ground	7	160 (3), 161, 162, 163, 182, 230 (2), 261, 740 (3), 742, 744, 835
Mesic Flatwood	8	190 (3), 191, 194, 260 (3), 310 (2), 321, 330 (2), 410 (3), 411, 414, 429, 435, 440 (3), 441, 443, 710 (2), 720 (2), 741
Mesic Hammock	9	420 (3), 422, 423, 426, 427, 434, 437, 438, 439
Xeric Flatwood	10	412, 413
Xeric Hammock	11	322, 421, 432
Hydric Flatwood	12	4119, 419, 624
Hydric Hammock	13	329, 424, 425, 428, 433, 610 (3), 611, 743
Wet Prairie	14	643, 6439
Dwarf Cypress	15	6219
Marsh	16	6171, 6172, 640 (3), 641, 6411, 6412, 644
Cypress	17	620 (3), 621, 6218, 745
Swamp Forest	18	613, 614, 615, 616, 617, 630 (2)
Mangrove	19	612, 642
Water	20	166, 500 (1)

2.4 UNSATURATED ZONE MODEL

The unsaturated zone extends from the ground surface to the groundwater table. The depth is dynamic and varies throughout the year with groundwater fluctuations and rainfall. During periods of the year, the unsaturated zone may occasionally disappear in depression areas, such as wetlands, where the water table rises above ground, e.g. in wetland areas. Unsaturated flow in MIKE SHE is computed based on a simplified Richard's equation and infiltration rates depend on a number of soil parameters including hydraulic conductivity of the soil, soil retention, residual soil moisture, and water content at field capacity etc.. The model computes infiltration rates and soil moisture, which in turn affect evapotranspiration losses from the root zone and irrigation demands. Input for the model consists of soil property parameters and a soil column distribution map. The soil parameters in MIKE SHE are specified in a database and a number of soil profiles are defined using soil types from the database. The MIKE SHE soil distribution map is shown in Figure 2-8. Various physical soil parameters entered into the unsaturated zone database are given in Table 2-4.

2.5 GROUNDWATER MODEL

The geology of the area consists of a Water Table Aquifer, Lower Tamiami Aquifer and the Sandstone Aquifer. The Surficial Aquifer system includes the Water Table Aquifer and a portion of the Lower Tamiami Aquifer, extending down approximately 80 feet. The Water Table Aquifer, which is well connected with the canal systems and responds rapidly to rainfall, is the only source of recharge, and canal drainage. The Surficial Aquifer System is separated from the lower aquifers by an aquiclude. The Lower Tamiami Aquifer is the primary source of regional public water supply. However, the rapid urban development in Collier County has stressed this aquifer to its safe yield limits, and a lower Mid-Hawthorne formation is now being tapped for supplemental public water supply using reverse osmosis treatment.

- Water Table aquifer
- Lower Tamiami aquifer
- Sandstone aquifer

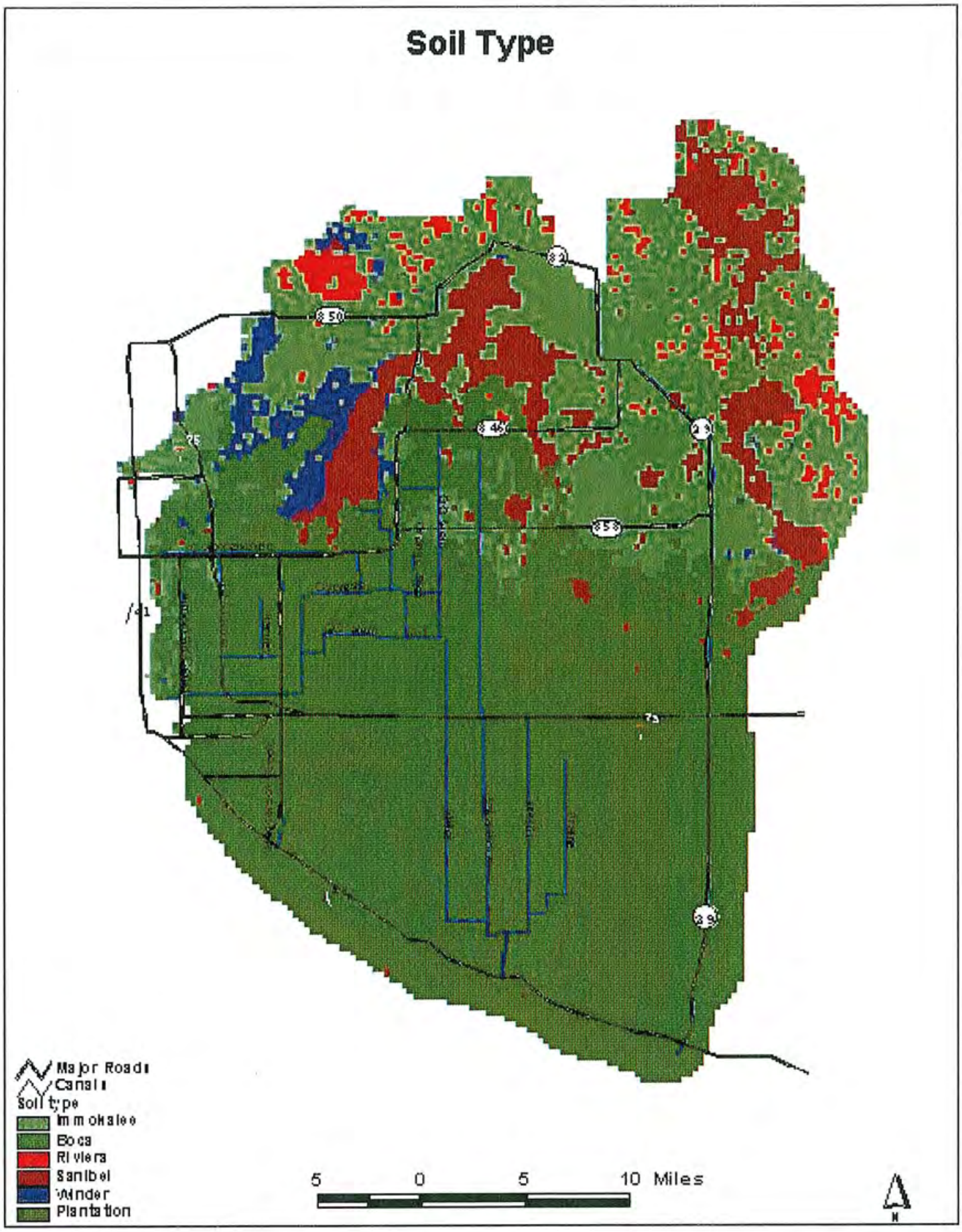

Soil Type
850
82
846
29
858
25
41
75
29
Major Roads
Canals
Soil type
Immokalee
Boca
Riviera
Sanibel
Winder
Plantation
5
0
5
10 Miles
N

Figure 2-8

Table 2-4: Soil Profile Definition and Soil Physical Parameters Entered into the Unsaturated Zone Database

Profile No. and MSHE Code	**Soil Type and Depth**	**Saturated Hydraulic Conductivity K_s [m/s]**	**Saturated Water Content Θ_s**	**Water Content at Field Capacity Θ_{fc}**	**Water Content at Wilting Point Θ_w**	**Residual Water Content Θ_r**
1	Immokalee A1 (0.0-0.1 m)	2.0e-4	0.42	0.15	0.013	0.01
	Immokalee AE (0.1-0.23 m)	1.1e-4	0.42	0.15	0.02	0.031
	Immokalee E1 (0.23-0.41 m)	8.6e-5	0.39	0.14	0.02	0.015
	Immokalee E2 (0.41-0.91 m)	1.0e-4	0.38	0.14	0.01	0.01
	Immokalee Bh1(0.91-1.27 m)	1.2e-6	0.38	0.33	0.057	0.031
	Immokalee Bh2 (1.27-1.4 m)	6.1e-6	0.38	0.28	0.05	0.043
	Immokalee Bw/Bh (1.4-30 m)	7.5e-5	0.38	0.20	0.03	0.02
2	Boca A (0.0-0.08 m)	1.1e-4	0.487	0.11	0.04	0.029
	Boca E1 (0.08-0.23 m)	9.7e-5	0.46	0.11	0.034	0.023
	Boca E2 (0.23-0.36 m)	8.0e-5	0.408	0.09	0.024	0.015
	Boca Bw (0.36-0.64 m)	5.4e-5	0.396	0.10	0.009	0.006
	Boca Btg (0.64-30 m)	8.3e-7	0.347	0.33	0.122	0.071
3	Riviera Ap (0-0.15 m)	1.2e-6	0.38	0.23	0.049	0.031
	Riviera A (0.15-0.28 m)	4.2e-5	0.52	0.22	0.047	0.02
	Riviera E1 (0.28-0.41 m)	5.0e-5	0.46	0.12	0.022	0.01
	Riviera E2 (0.41-0.64 m)	5.5e-5	0.4	0.06	0.003	0.001
	Riviera Bw (0.64-0.74 m)	3.5e-5	0.38	0.06	0.004	0.001
	Riviera Btg (0.74-30 m)	2.5e-7	0.38	0.32	0.102	0.08
4	Sanibel Oa1 (0-0.12 m)	2e-5	0.752	0.72	0.207	0.2
	Sanibel Oa2 (0.12-0.15 m)	7.8e-5	0.73	0.69	0.205	0.1
	Sanibel A1 (0.15-0.23 m)	9.4e-5	0.51	0.39	0.025	0.01
	Sanibel A2 (0.23-0.3 m)	1.7e-4	0.41	0.17	0.013	0.01
	Sanibel C1 (0.3-0.66 m)	1.4e-4	0.37	0.09	0.013	0.01
	Sanibel C2 (0.66-30 m)	1.1e-4	0.38	0.08	0.011	0.01
5	Winder A1 (0.0-0.08 m)	3.6e-5	0.374	0.26	0.024	0.014
	Winder E (0.08-0.33 m)	5.7e-5	0.37	0.15	0.008	0.004
	Winder B/E (0.33-0.41 m)	1.6e-6	0.328	0.23	0.048	0.027
	Winder Btg (0.41-0.58 m)	7.4e-6	0.43	0.40	0.153	0.101
	Winder BCg (0.58-0.74 m)	7.4e-6	0.34	0.26	0.05	0.028
	Winder C1 (0.74-0.89 m)	4.1e-6	0.332	0.27	0.038	0.021
	Winder C2 (0.89-1.04 m)	5.0e-6	0.347	0.23	0.042	0.024
	Winder C3 (0.89-30 m)	1.9e-6	0.355	0.31	0.107	0.062
6	Plantation Oap (0-0.23 m)	1.6e-4	0.86	0.56	0.164	0.1
	Plantation A/E (0.23-0.48 m)	8.4e-5	0.491	0.19	0.029	0.022
	Plantation Bw (0.48-30 m)	1.2e-4	0.392	0.10	0.003	0.002

- Mid Hawthorn aquifer

The listed aquifers are assumed to account for the exchange with the river and canal network and to constitute the major source of groundwater in the model area. The deeper Floridian Aquifer system is not considered to be recharged or add to the water available in the overlying aquifer systems. According to geological surveys in the area, negligible exchange occurs between the Mid Hawthorn and the underlying Floridian aquifers. Figures 2-9 through 2-13 are elevation maps for each layer. Figures 2-14 through 2-18 are vertical and horizontal hydraulic conductivities distribution maps for those three aquifer layers.

Groundwater flow and potential heads are computed using a 3-D finite-difference groundwater model. A conceptual geological model representing the major layers, including aquitards and aquifers, was initially set up for the watershed to adequately represent flows in the groundwater system. A number of hydrogeological parameters, e.g. hydraulic conductivity and storage coefficients, were specified and appropriate boundary conditions were established. The delineation of boundary conditions was essential for obtaining a correct water balance for the groundwater basin. Moreover, water allocation from groundwater wells will affect the water balance significantly and impact groundwater levels locally. Similarly groundwater drainage will affect water levels and the dynamics of groundwater levels, primarily in the shallow aquifers.

Some of the groundwater simulation parameters were adapted from the Collier County MODFLOW model developed earlier. A specific yield of 0.2 was used for the surficial aquifer and the storage coefficient was set at $1 \cdot 10^{-5}$ m^{-1} for the combined lower Tamiami and Sandstone aquifers. The final soil properties were determined through calibration of the model.

The boundary conditions for the confining layers were defined as an impermeable boundary. A combination of constant and variable head boundary conditions were applied for simulating the integrated surface and groundwater flow in the BCB MIKESHE model (Figure 2-19). A constant head boundary was applied along the

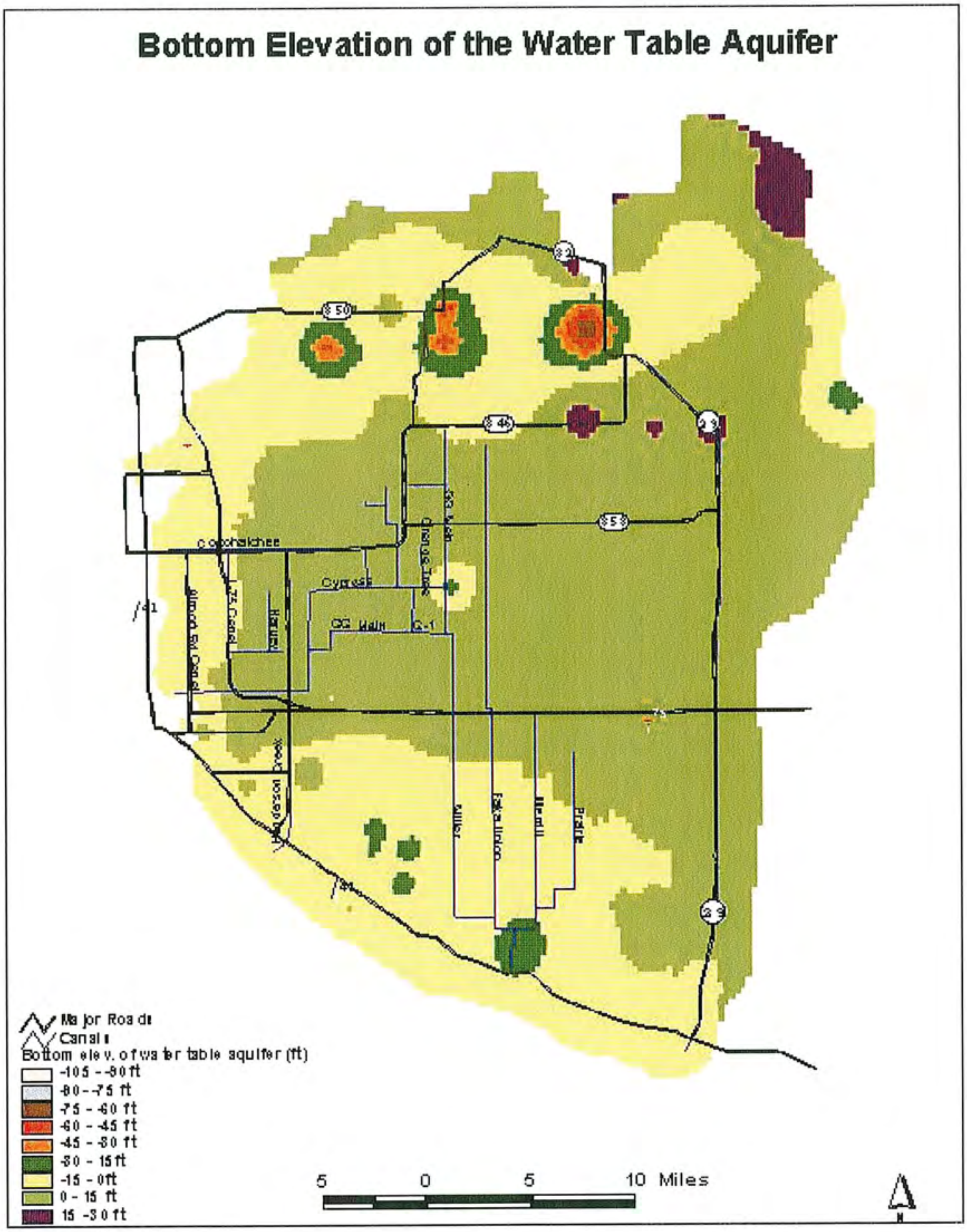

Bottom Elevation of the Water Table Aquifer
850
82
846
29
858
Cocohatchee
Cypress
CC Main
C-1
Golden Gate Main
Orange Tree
41
I-75 Canal
Airport Rd Canal
Harvey
75
Henderson Creek
Miller
Faka Union
Merritt
Prairie
Major Roads
Canals
Bottom elev. of water table aquifer (ft)
-105 - -90 ft
-90 - -75 ft
-75 - -60 ft
-60 - -45 ft
-45 - -30 ft
-30 - 15 ft
-15 - 0 ft
0 - 15 ft
15 - 30 ft
5
0
5
10 Miles
N

Figure 2-9

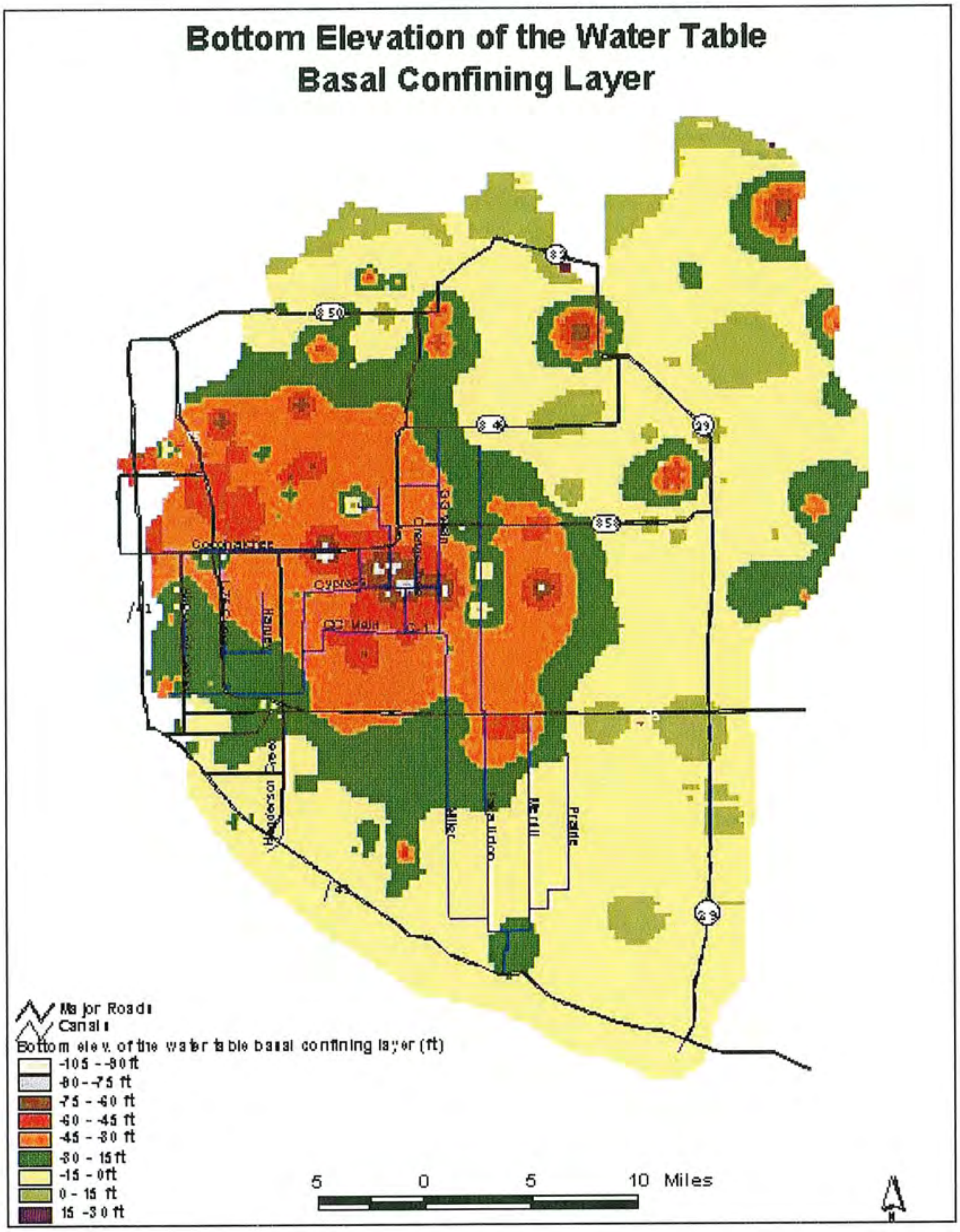

Bottom Elevation of the Water Table
Basal Confining Layer
850
84
858
Cypress
Henderson
Miller
Merrill
Prairie
Major Roads
Canals
Bottom elev. of the water table basal confining layer (ft)
-105 - -90 ft
-90 - -75 ft
-75 - -60 ft
-60 - -45 ft
-45 - -30 ft
-30 - 15 ft
-15 - 0ft
0 - 15 ft
15 -30 ft
5 0 5 10 Miles

Figure 2-10

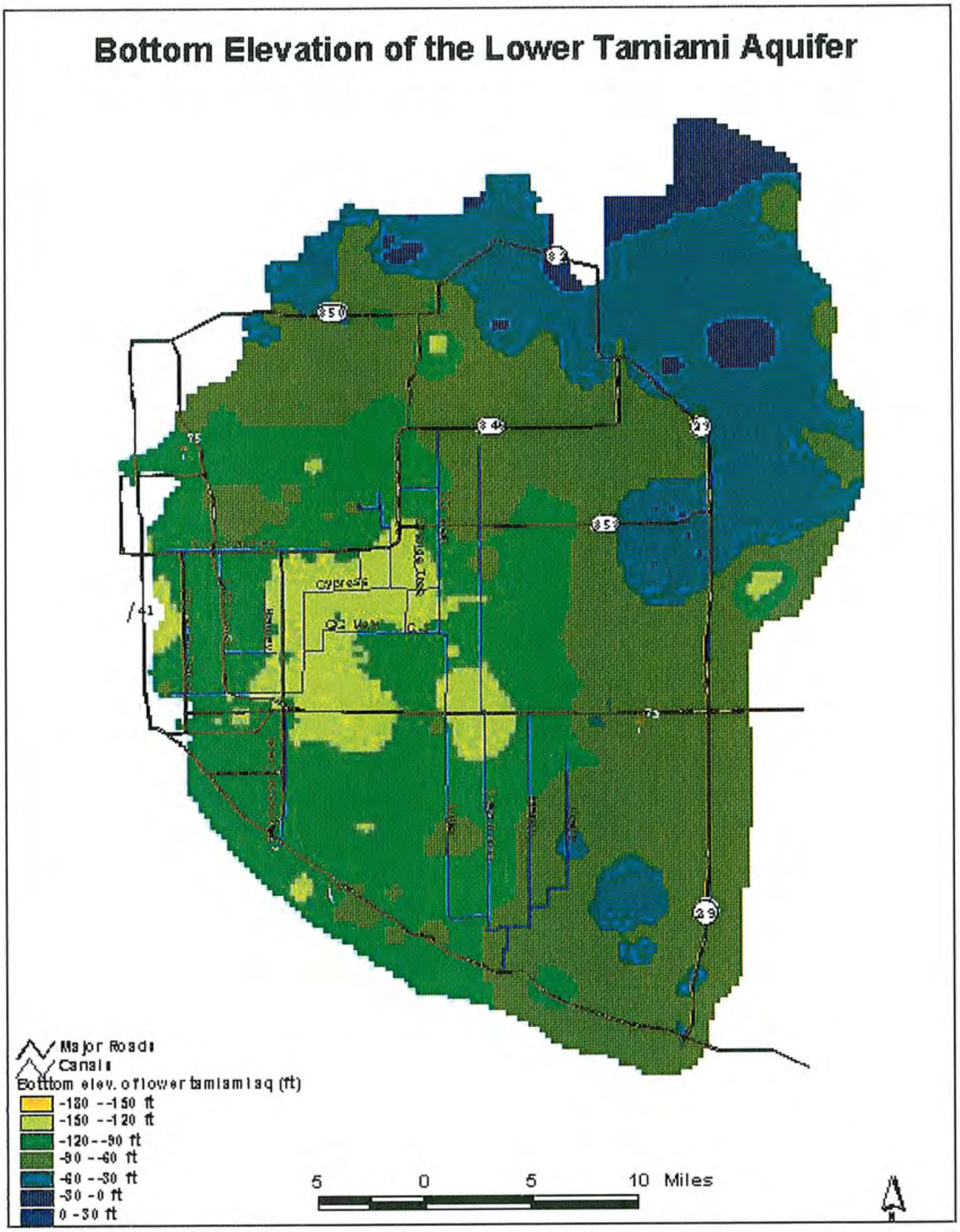

Bottom Elevation of the Lower Tamiami Aquifer
850
82
846
29
858
75
41
Cypress
Oil Well
Major Roads
Canals
Bottom elev. of lower tamiami aq (ft)
🟧 -180 - -150 ft
🟨 -150 - -120 ft
🟩 -120 - -90 ft
🟩 -90 - -60 ft
🟦 -60 - -30 ft
🟦 -30 - 0 ft
🟦 0 - 30 ft
5 0 5 10 Miles
N

Figure 2-11

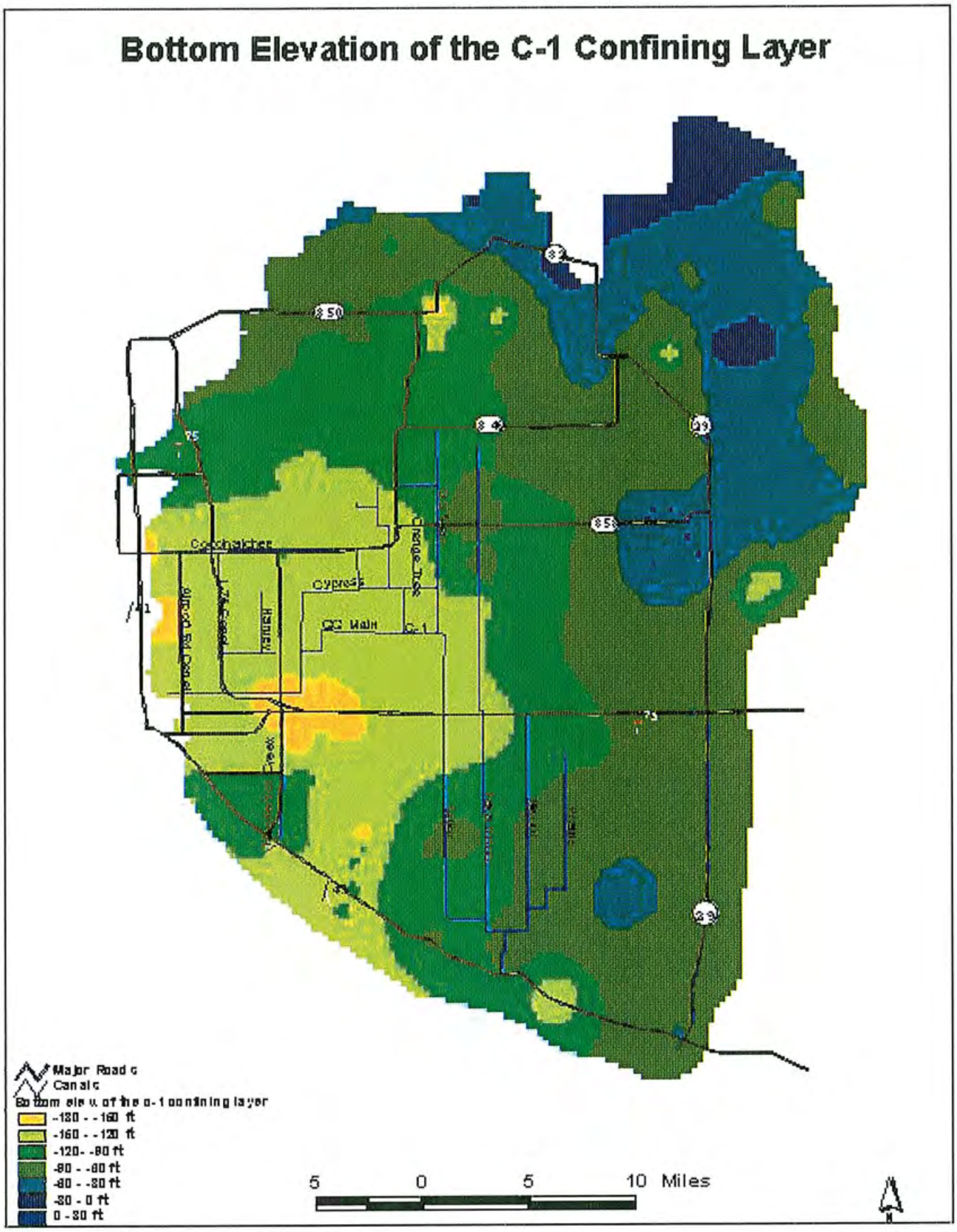

Bottom Elevation of the C-1 Confining Layer
82
850
846
29
858
75
41
Cocohatchee
Cypress
GG Main
C-1
Harvey
Orange Tree
Airport Rd Canal
I-75 Canal
Creek
Major Roads
Canals
Bottom elev. of the c-1 confining layer
-180 - -160 ft
-160 - -120 ft
-120- -80 ft
-80 - -60 ft
-60 - -30 ft
-30 - 0 ft
0 - 30 ft
5
0
5
10 Miles
N

Figure 2-12

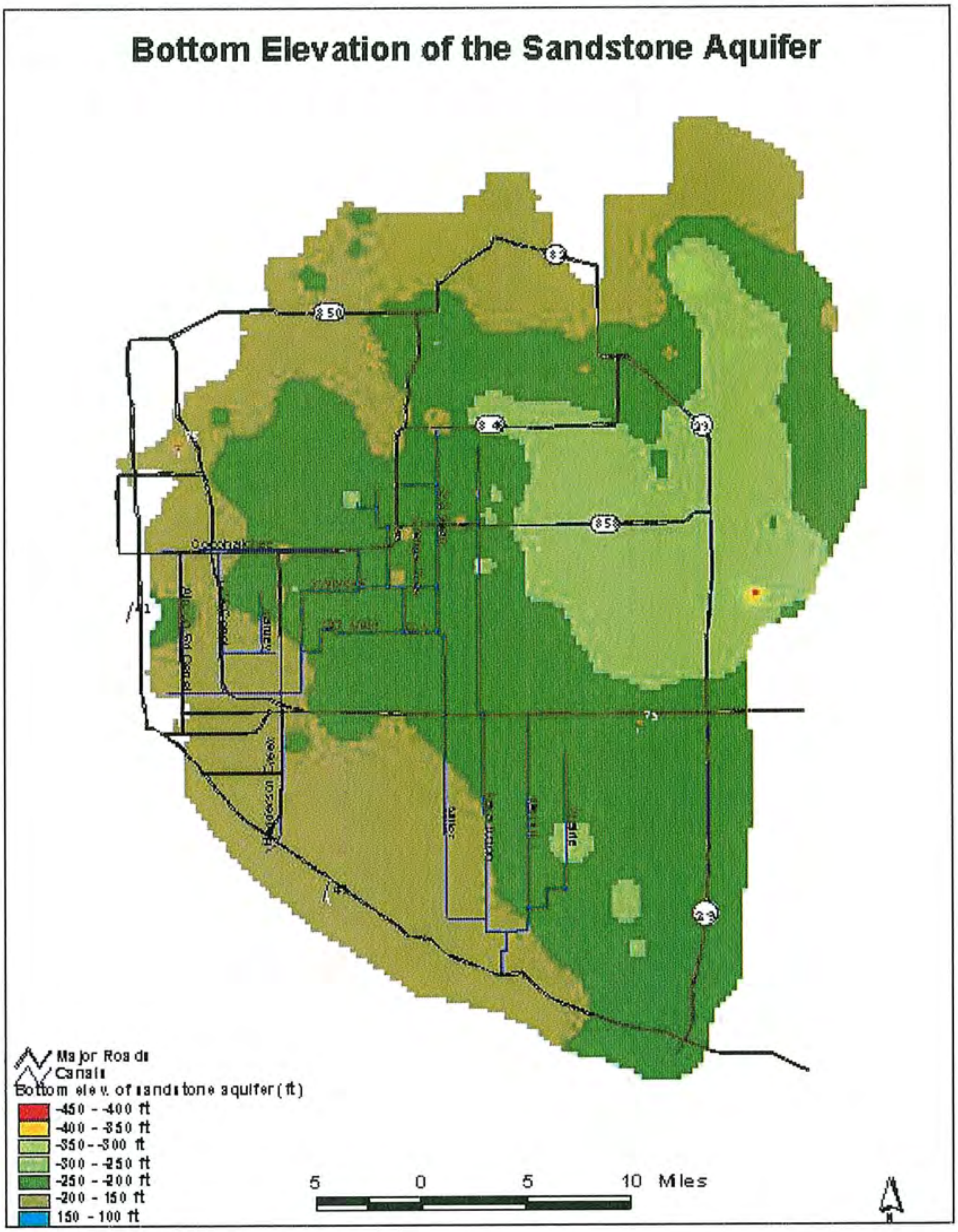

Bottom Elevation of the Sandstone Aquifer
850
82
84
29
858
75
41
29
Major Roads
Canals
Bottom elev. of sandstone aquifer (ft)
🟥 -450 - -400 ft
🟨 -400 - -350 ft
🟩 -350 - -300 ft
🟩 -300 - -250 ft
🟩 -250 - -200 ft
🟫 -200 - 150 ft
🟦 150 - 100 ft
5
0
5
10 Miles

Figure 2-13

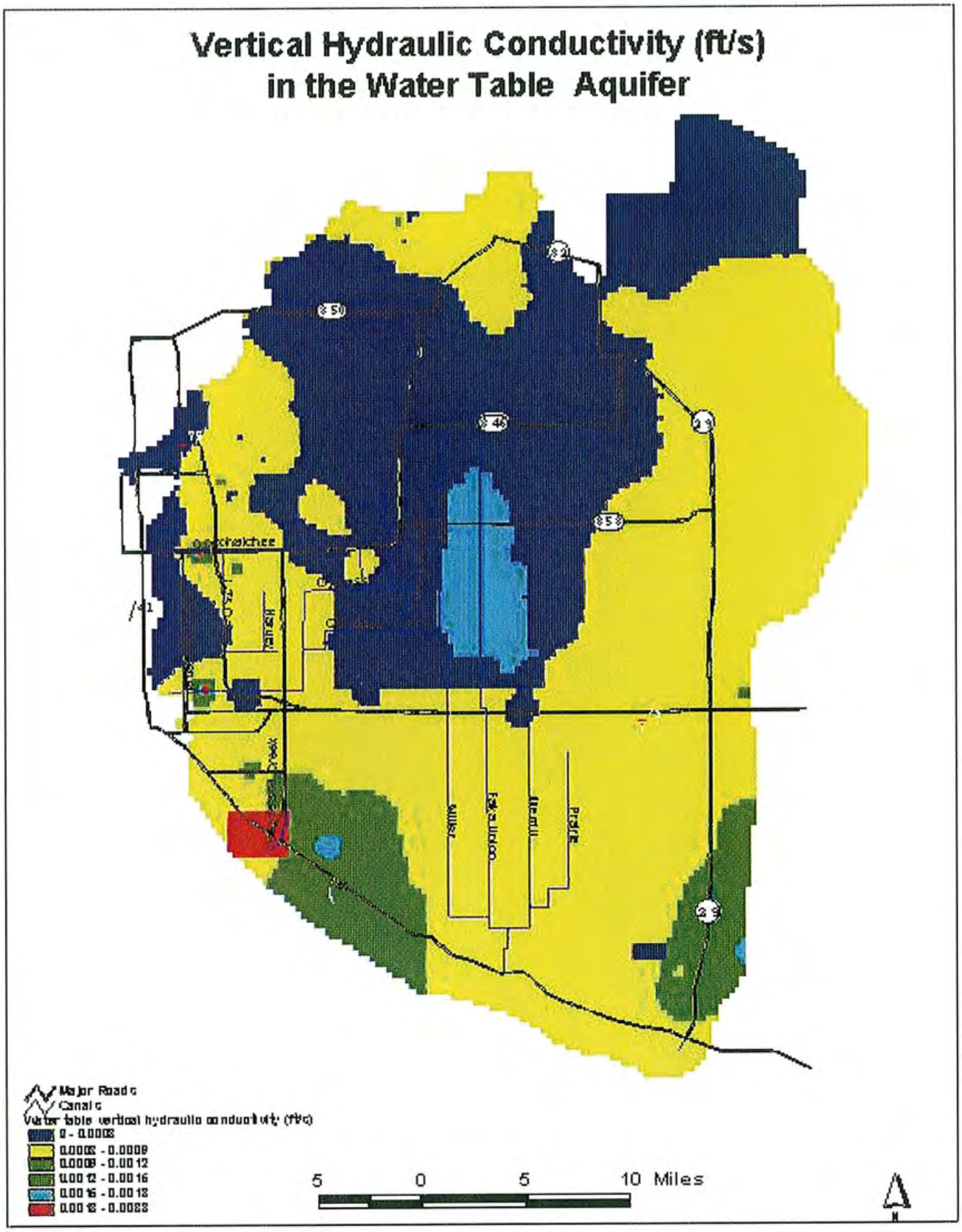

Vertical Hydraulic Conductivity (ft/s) in the Water Table Aquifer

Figure 2-14

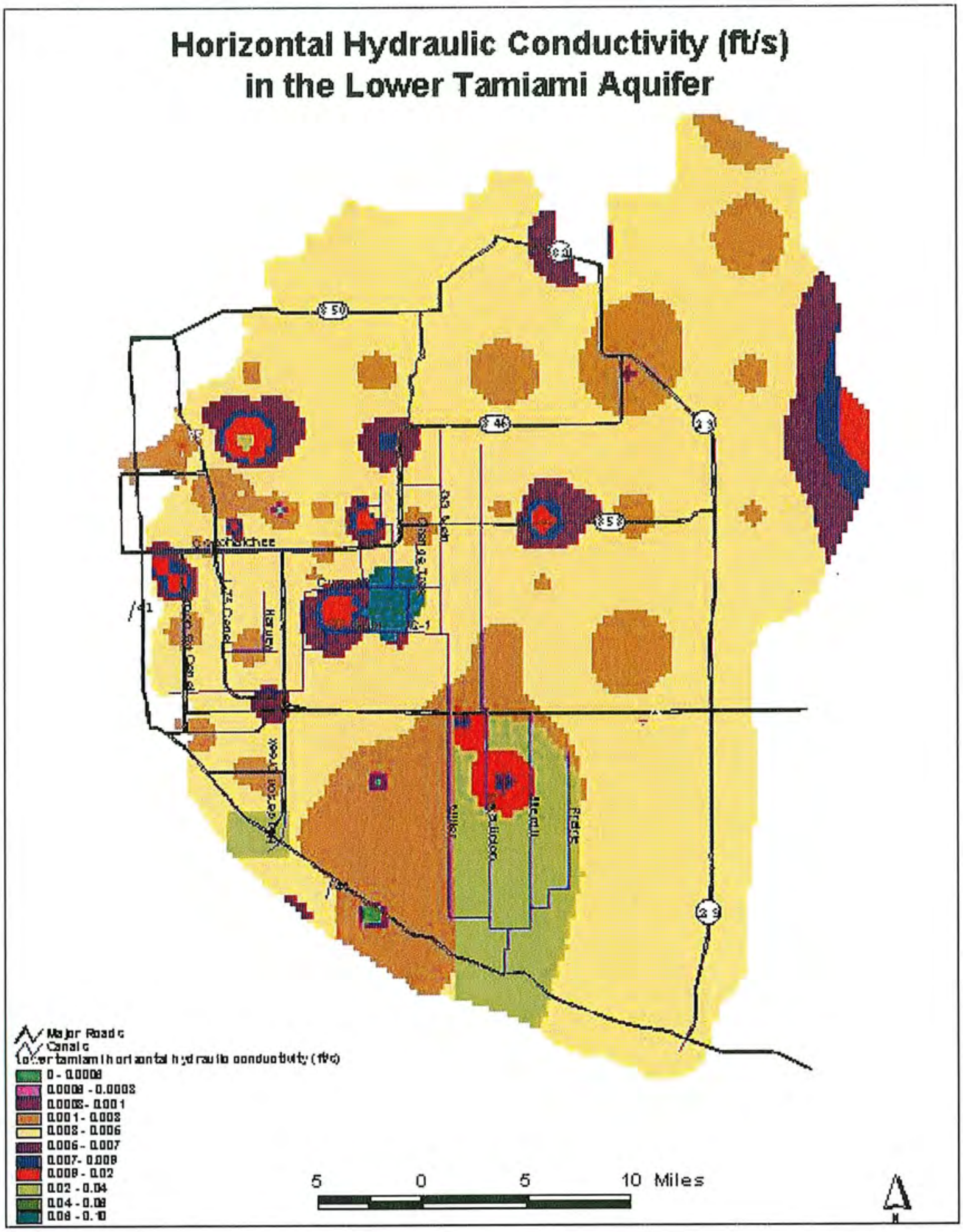
Horizontal Hydraulic Conductivity (ft/s)
in the Lower Tamiami Aquifer
Major Roads
Canals
Lower tamiami horizontal hydraulic conductivity (ft/s)
0 - 0.0003
0.0003 - 0.0008
0.0008 - 0.001
0.001 - 0.003
0.003 - 0.006
0.006 - 0.007
0.007 - 0.009
0.009 - 0.02
0.02 - 0.04
0.04 - 0.08
0.08 - 0.10
5
0
5
10 Miles
N
80
82
29
41
Miller
Merritt
Prairie

Figure 2-15

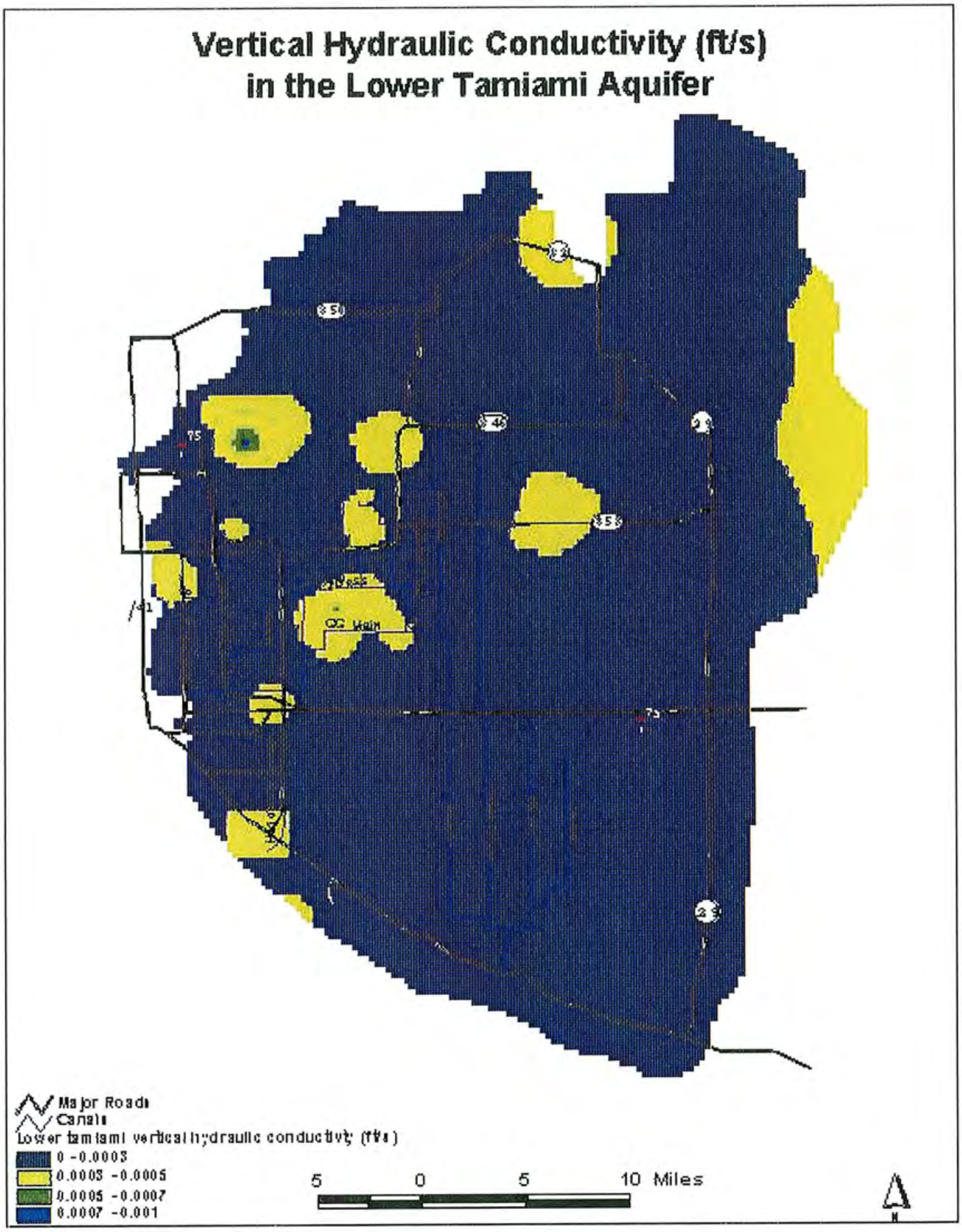

Vertical Hydraulic Conductivity (ft/s)
in the Lower Tamiami Aquifer
82
850
846
29
858
75
41
CC Main
75
29
Major Roads
Canals
Lower tamiami vertical hydraulic conductivity (ft/s)
0 - 0.0003
0.0003 - 0.0005
0.0005 - 0.0007
0.0007 - 0.001
5
0
5
10 Miles
N

Figure 2-16

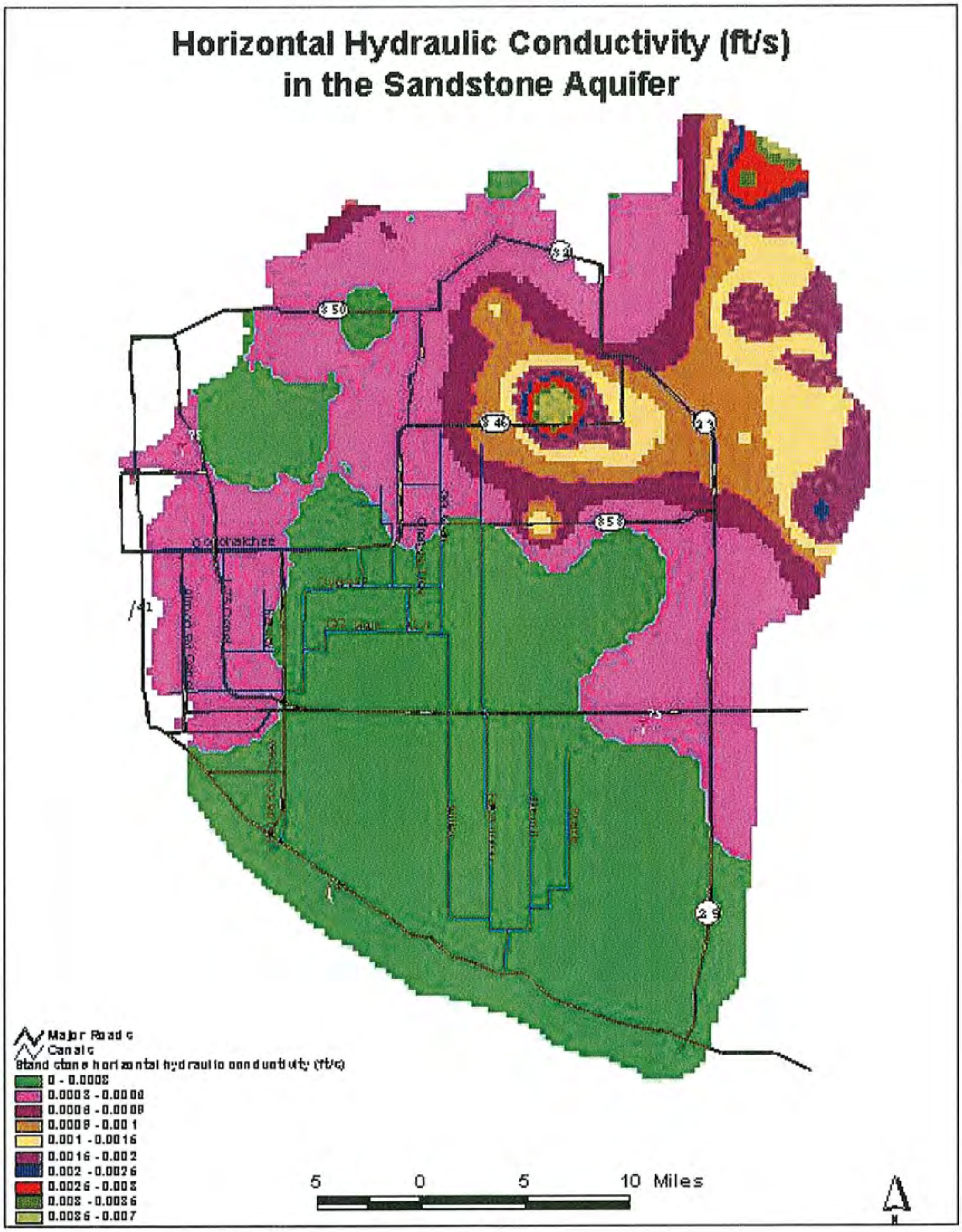

Horizontal Hydraulic Conductivity (ft/s)
in the Sandstone Aquifer
50
46
29
53
75
41
Major Roads
Canals
Sandstone horizontal hydraulic conductivity (ft/s)
0 - 0.0003
0.0003 - 0.0006
0.0006 - 0.0009
0.0009 - 0.001
0.001 - 0.0016
0.0016 - 0.002
0.002 - 0.0026
0.0026 - 0.003
0.003 - 0.0036
0.0036 - 0.007
5
0
5
10 Miles
N

Figure 2-17

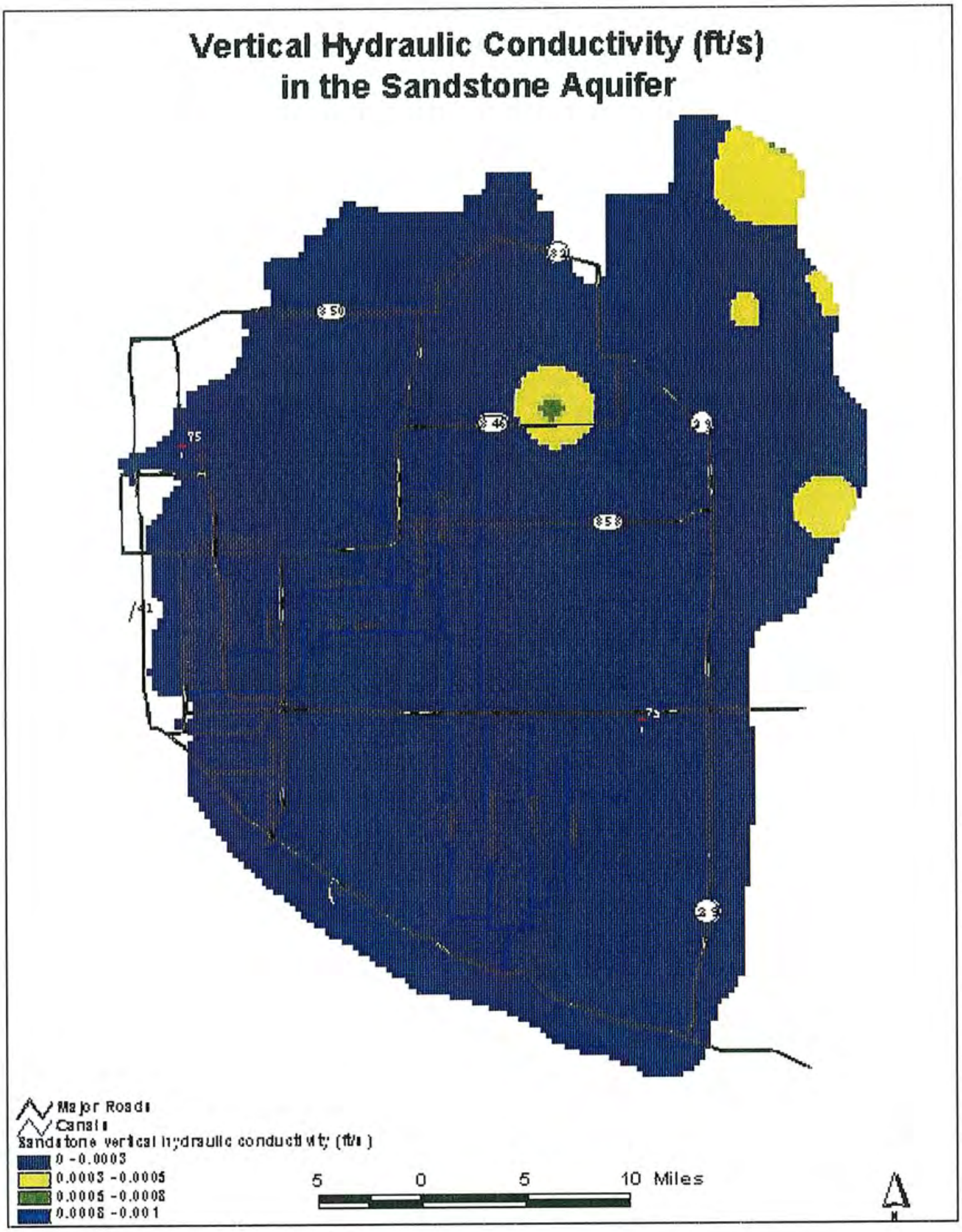
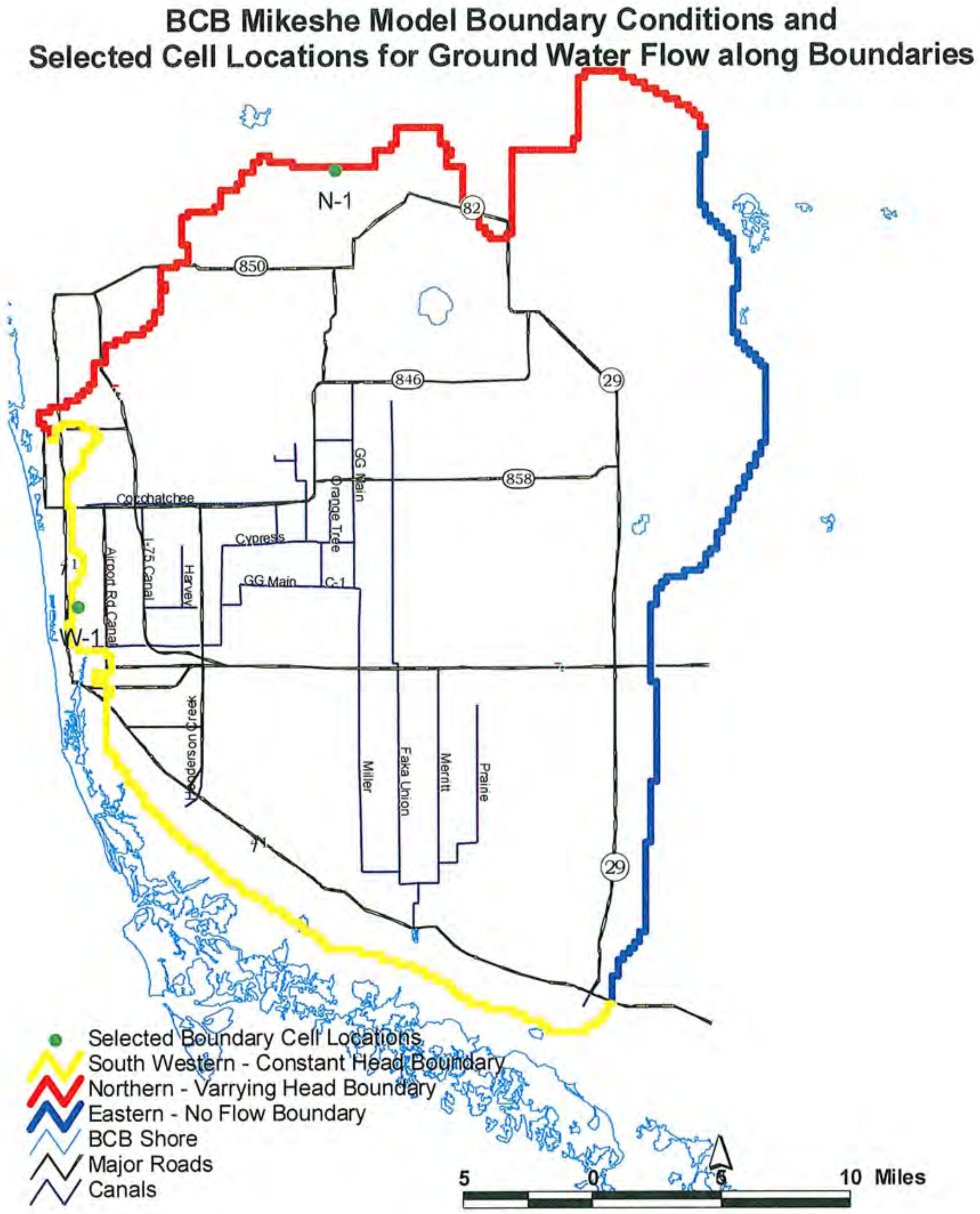

Vertical Hydraulic Conductivity (ft/s)
in the Sandstone Aquifer
850
846
858
75
41
29
75
29
Major Roads
Canals
Sandstone vertical hydraulic conductivity (ft/s)
0 - 0.0003
0.0003 - 0.0005
0.0005 - 0.0008
0.0008 - 0.001
5
0
5
10 Miles
N

Figure 2-18



BCB Mikeshe Model Boundary Conditions and Selected Cell Locations for Ground Water Flow along Boundaries

Figure 2-19

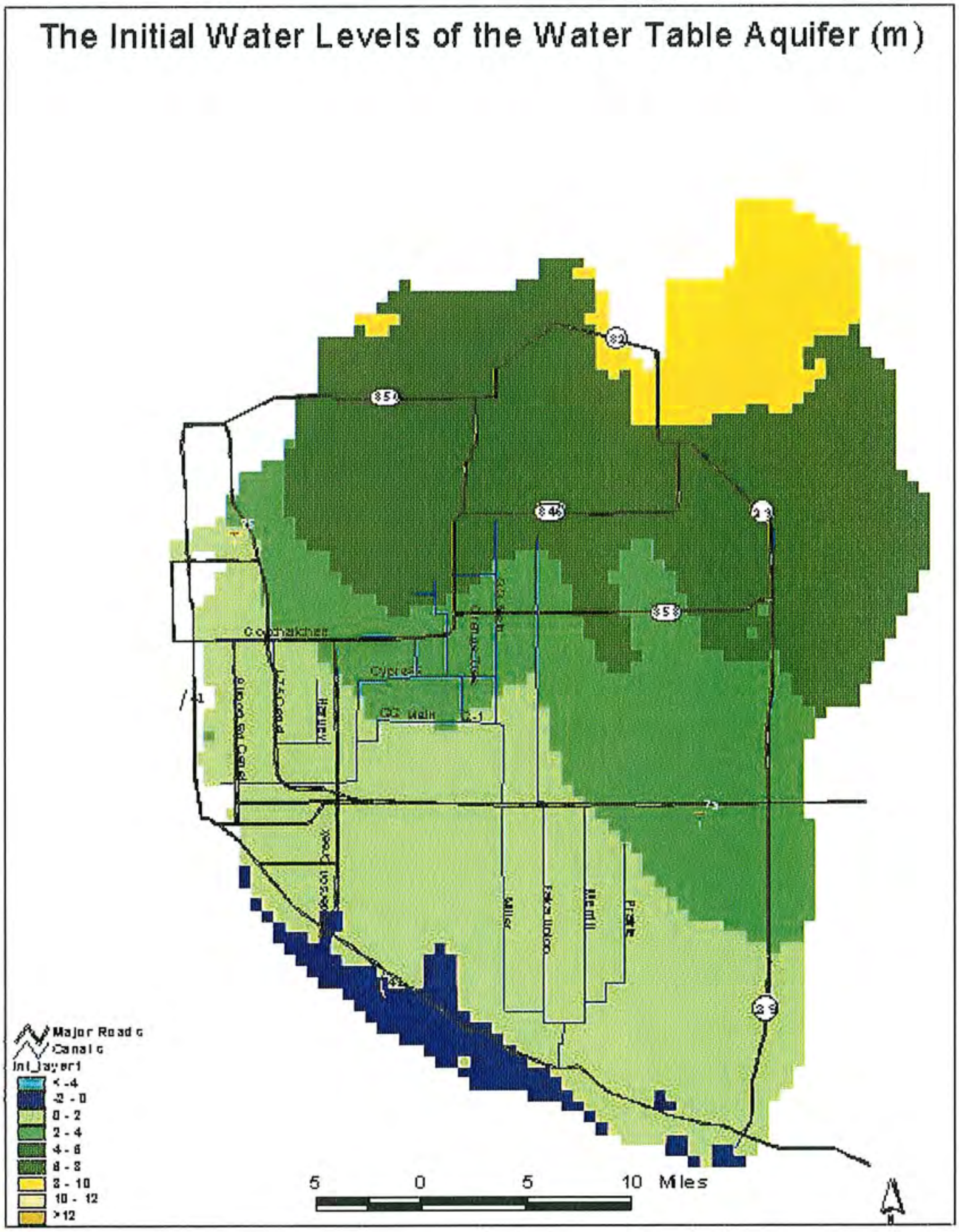

The Initial Water Levels of the Water Table Aquifer (m)

Figure 2-20

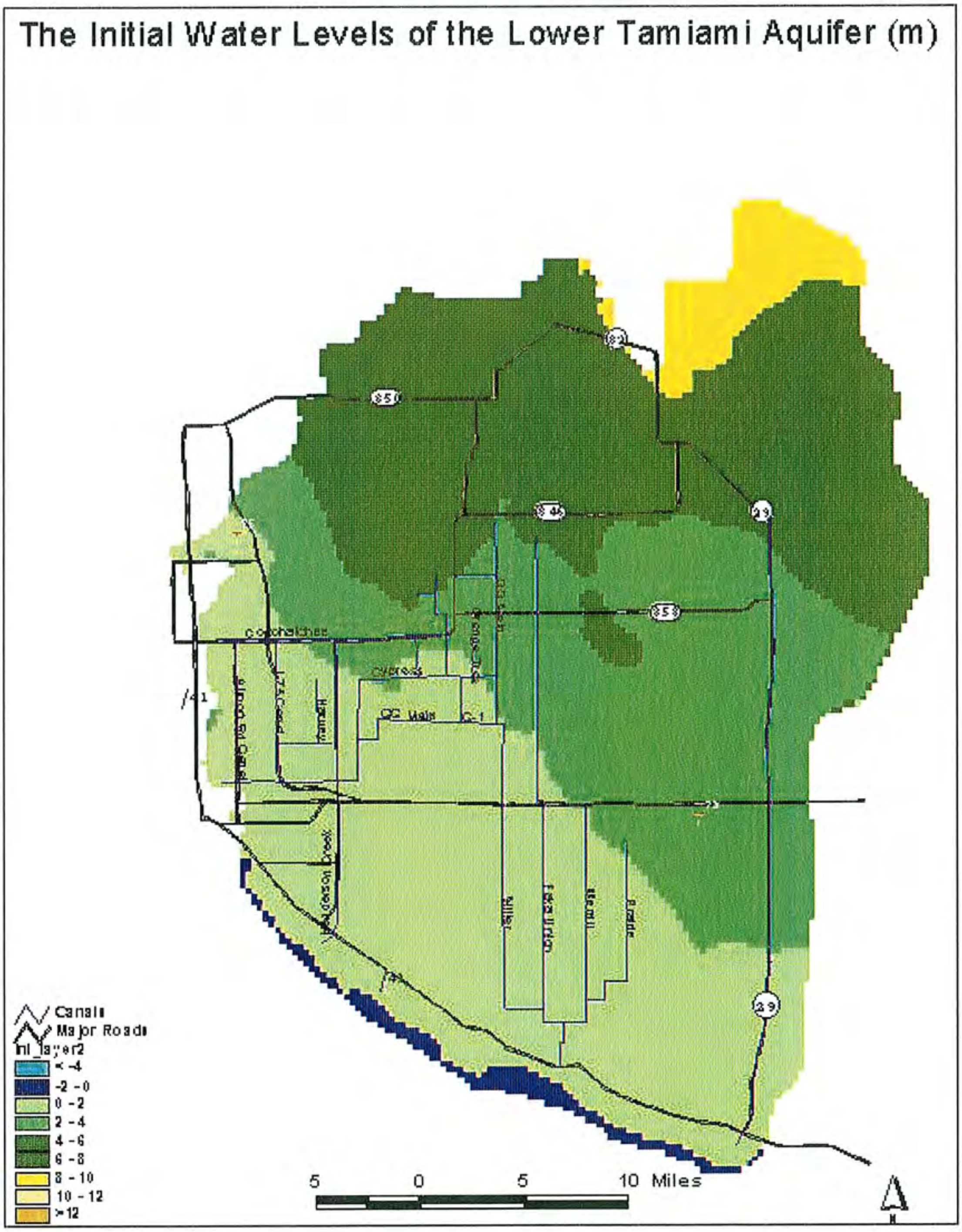

The Initial Water Levels of the Lower Tamiami Aquifer (m)
82
850
846
29
858
Cocohatchee
Cypress
CC Main
C-1
41
Henderson Creek
Miller
Faka Union
Merritt
Prairie
Canals
Major Roads
ini_layer2
< -4
-2 - 0
0 - 2
2 - 4
4 - 6
6 - 8
8 - 10
10 - 12
> 12
5
0
5
10 Miles
N

Figure 2-21

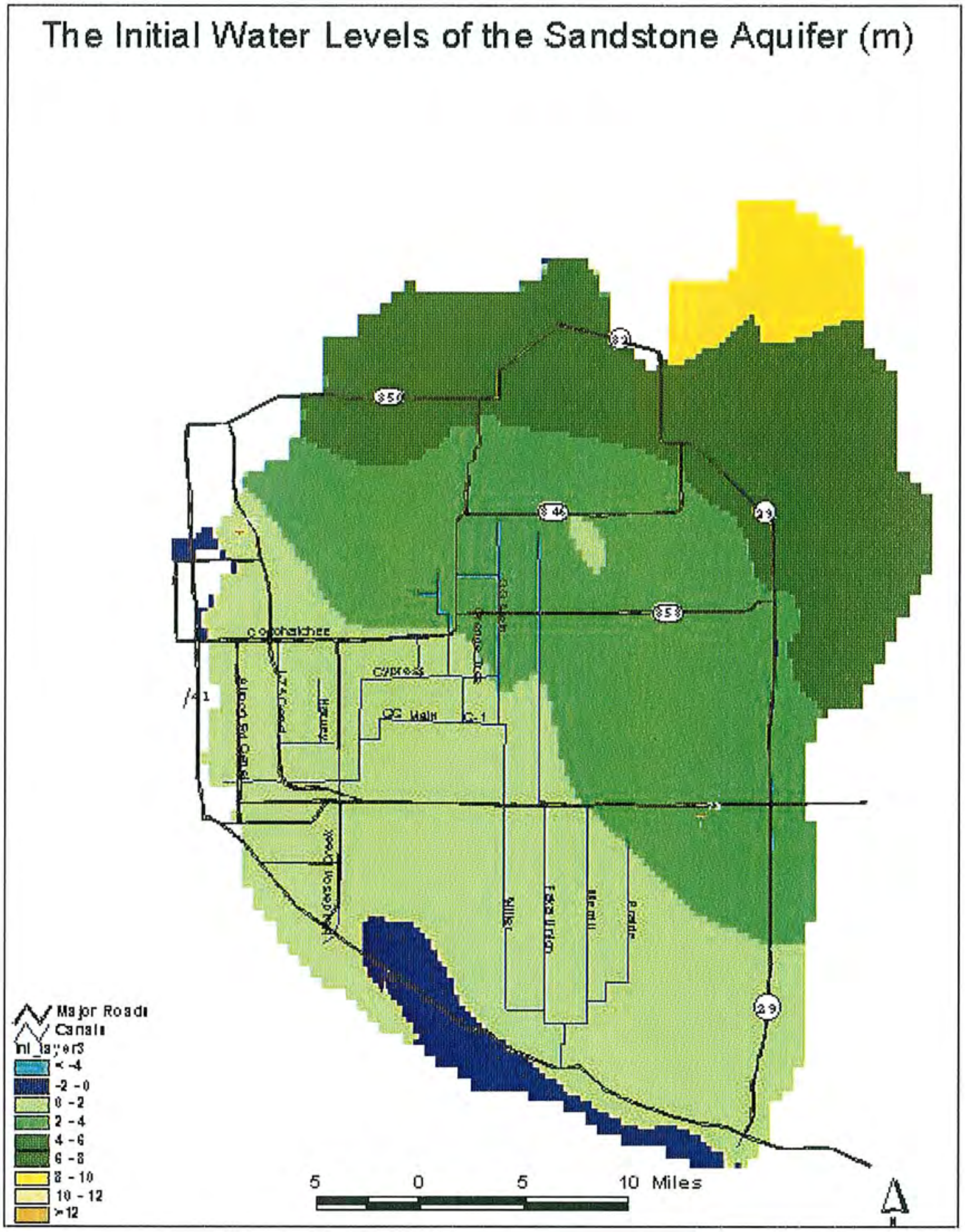

The Initial Water Levels of the Sandstone Aquifer (m)
Major Roads
Canals
ini_layer3
< -4
-2 - 0
0 - 2
2 - 4
4 - 6
6 - 8
8 - 10
10 - 12
>12
5
0
5
10 Miles
N
850
82
846
858
29
41
Cocohatchee
Cypress
GG Main
C-1
Miller
Faka Union
Merritt
Prairie
Henderson Creek

Figure 2-22

southwestern coastline. A tidal boundary condition would, in principle provide more accurate results in assessing the impacts on tidal wetlands. However, sufficient information on groundwater levels along the coastline was not available to generate transient head boundary conditions. Time-varying head boundary conditions were applied along the northern boundary generated from available groundwater level data from monitoring wells. The time series of variable heads for cells between locations with measured data were generated using triangular linear integration. A no-flow boundary condition was specified for the eastern boundary.

Initial water levels for the aquifer layers are illustrated in Figures 2-20 through 2-22.

2.6 HYDRAULIC ROUTING MODEL

Channel flows in the watershed are described by the 1-D fully hydrodynamic river/flood model MIKE 11, which couples dynamically to the integrated hydrological MIKE SHE model. All surface flowways are accounted for by the model, including canals, main rivers, channels, irrigation canals and sloughs - except surface runoff, which is handled by the MIKE SHE overland flow component.

Input for the model consists of the channel network (which is crucial in describing the channels and floodplains adequately), and surveyed cross-sections, as well as appropriate boundary conditions consistent with actual surface boundaries and bed resistance. Moreover, flow regulating structures, such as culverts, weirs and control gates that may significantly alter or modify channelized flows and stages, are specified as input to the model. Finally, the channels exchange water with the underlying aquifer. This may either be described entirely by the aquifer material properties or by a channel lining leakage coefficient as specified in MIKE 11.

The major flowways in the BCB consist of a number of natural sloughs and an intricate system of manmade channels. The major flowways in the BCB are shown in Figure 2-23. The main channels defined in Figure 2-23 were included in the

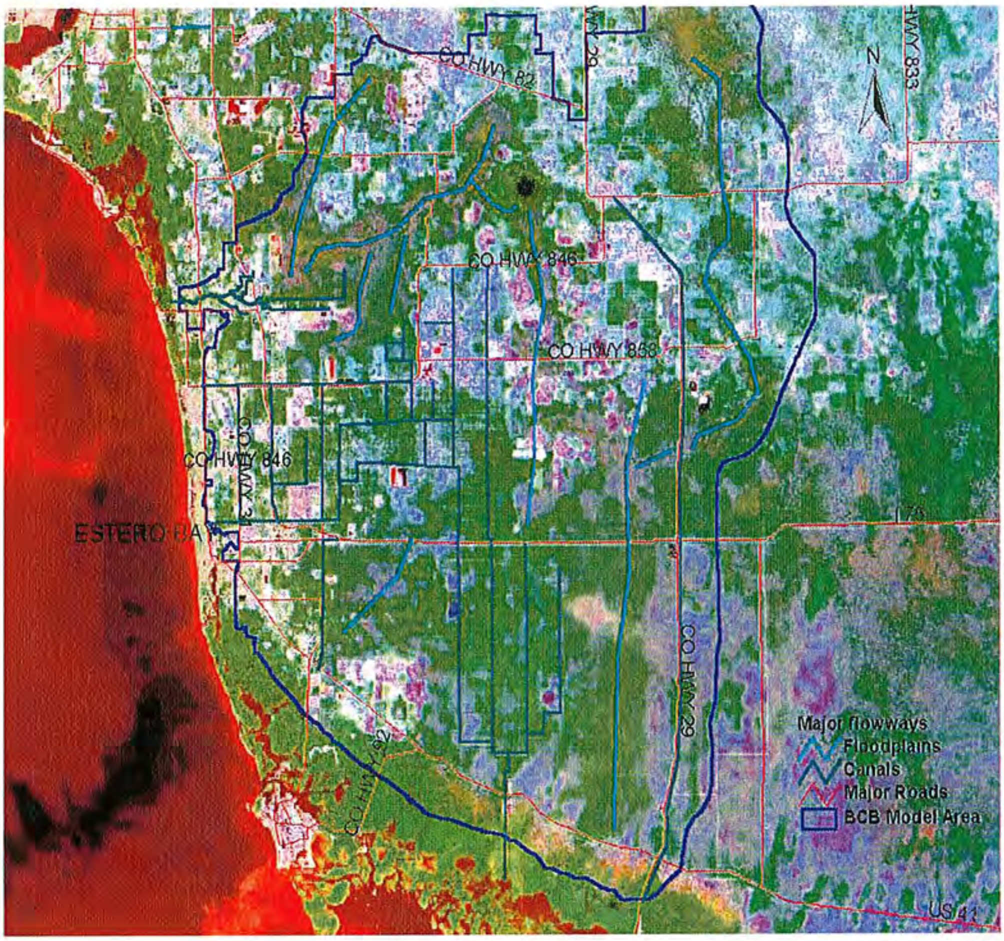


Figure 2-23: Major Flowways in the BCB Watershed

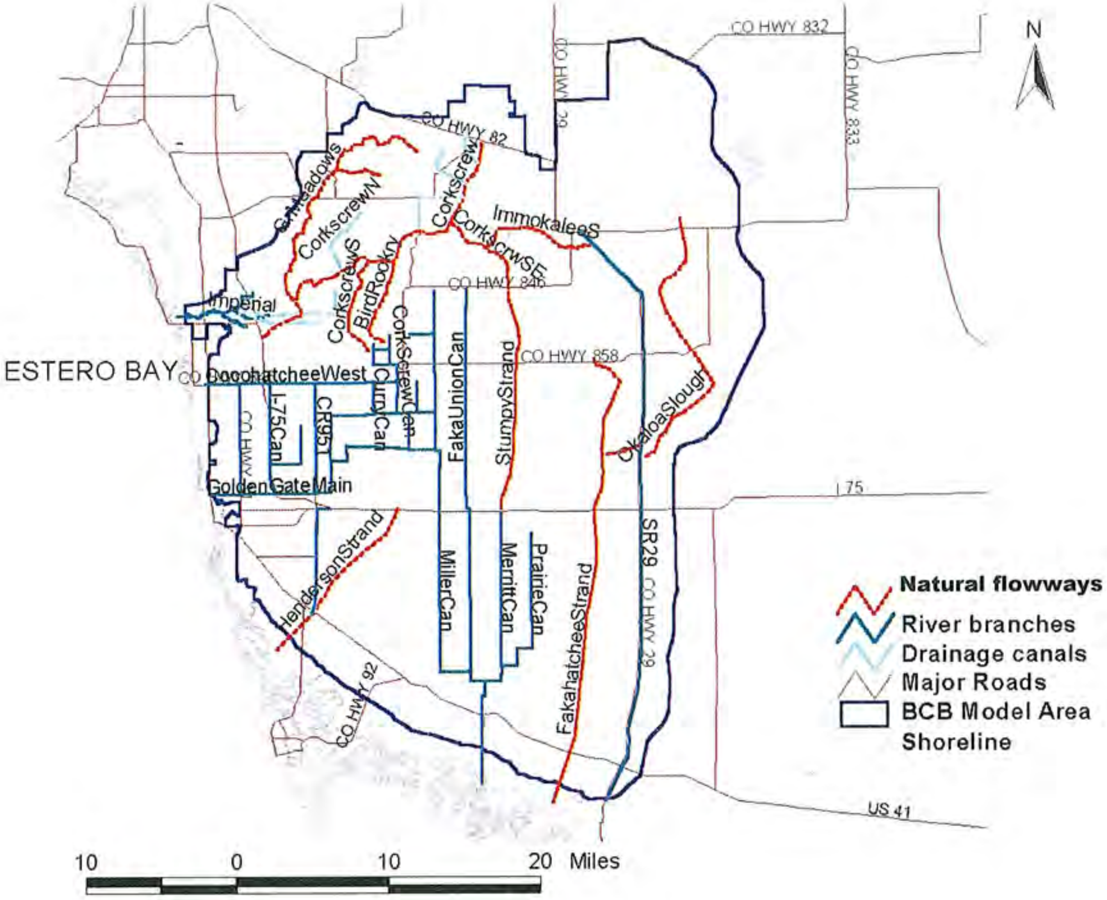


Figure 2-24: MIKE 11 Channel Network for the BCB Watershed

model, totaling 28 MIKE 11 branches. Moreover, a number of natural flowways and sloughs were defined in MIKE 11, a total of 14 branches. The final MIKE 11 branch system is presented in Figure 2-24. The conveyance and storage capacity of the channel system is described by the cross-sectional geometry of the channel branches. Cross-sections are preferably entered into the model at regular intervals of approximately 600-1600 ft (200- 500 m), if available, and as a minimum, at up- and downstream ends of each channel branch. Surveyed channel cross-sections with limited extent of the flood plains for the entire BCB channel system were available in an existing UNET model set up by Dames & Moore (1998). The cross-sections were converted to MIKE 11 format and imported directly into the model (2000). Some additional survey was carried out by COE and also incorporated in the Mike 11 model.

The BCB channel system is characterized by an intricate network of channels with a large number of control structures, culverts and bridges. In total 44 control structures are located in the BCB major canal system as outlined in Figure 2-1. Five different types of control structures are found in the BCB channel system: fixed crest weirs with underflow gates, movable crest weirs, fixed crest weirs with V-notches, fixed crest weirs with steel sheets and amil gate weirs. The structures generally prevent over-drainage from the watershed and minimize tidal effects, as well as saltwater intrusion in the canals. The dimensions and operation of the control structures are described in the operation manual, Water Control Structures, BCB (2005), and a pamphlet with operating water elevations, BCB (2006). Based on this information, the MIKE11 structure module was used for setting the structure operation in the model and, since the module is very flexible, the gates are operated close to the description in the operation manual.

As stated earlier, the interaction between surface and groundwater for detailed assessment of the impact of the alternative water resources management strategies is modeled using integrated surface water and groundwater MIKE SHE model along with the dynamic channel routing model MIKE 11. The two models are integrated in an interactive fashion in which overland flows from MIKE SHE are taken as a distributed source function for MIKE 11 which in turn provides temporal head boundary conditions (in the channels) for groundwater system modeled using MIKE SHE. The flood model includes various components of hydrologic cycle, such as overland flow, unsaturated flow and groundwater flow. The interaction between surface water and groundwater flow in the watershed areas occurs through infiltration and subsequent migration of moisture in the unsaturated zone as percolation. The percolating moisture contributes to the groundwater system as groundwater recharge.

The overland flows from the watershed areas generated from the MIKE SHE overland flow model are used as the source terms in the channel routing by the dynamic MIKE 11 model. The natural overland flows contribute to the well-defined channels as lateral distributed sources. The implicit representation of the physical process of the interaction between overland and channel flows makes the simulation using the integrated models more realistic.

2.7 CALIBRATION

The integrated surface water-groundwater management model for BCB was calibrated and validated so that the model represents actual H&H conditions prevalent in the domain. A well calibrated and validated model ensures better performance in evaluating scenarios associated with different water resources management projects. The performance of this type of integrated model will depend on a number of factors including:

- Model conceptualization
- Quantity and quality of input data
- Model parameters
- Accuracy, availability and distribution of field observations
- Mathematical/numerical model application

The model conceptualization and other factors involved in analyzing the performance of the model were described in the DHI's reports (2002, 2004). Table 2-5 summarizes major model input and parameters in MIKESHE model. The model was initially calibrated and validated for a period from 1990-1995, and further calibrated and validated to the period of 1995-2000 (DHI, 2004). These calibrated time durations cover a number of dry, wet and average meteorological condition years. The model calibration and validation demonstrated that the calibrated model was capable of reproducing field data with a reasonable confidence. A number of key calibration parameters were identified for the model, with parameters adjusted during calibration and their ranges given in Table 2-6.

The main calibration data comprise river flows and stages at a number of gauging stations and a number of monitored groundwater wells in both the shallow and deep aquifers. Stream flow records at four stations located at the outlets of the main rivers and canals were utilized for calibration. The stage and discharge station locations are outlined in Figure 2-25. Groundwater observations consist of 38 records of monitored potential head in the watershed. The wells generally cover most of the watershed and, as such, constitute a good basis for the calibration. The well locations are presented in Figure 2-26.

The rigorous calibration and validation for both surface water and groundwater system in the BCB area are illustrated in the modeling report (DHI 2002, 2004). The comparisons of observed data and simulated results at several representative locations and groundwater monitoring wells are presented in Appendix A.

Table 2-5: List of Model Input and Parameters for MIKE SHE

Model Component	Model Input	Model Parameters
MIKE SHE SZ Saturated zone flow	Geological model (lithological information Boundary conditions Drainage depth (drain maps) Wells and withdrawal rate	$K_{h,}$ Horizontal hydraulic conductivity $K_{v,}$ Vertical hydraulic conductivity S, confined storage coefficient S , unconfined storage coefficient Drainage time constant
MIKE SHE UZ Unsaturated zone flow	Map of characteristic soil types Hydraulic Conductivity Curves Retention curves	Ks, saturated hydraulic conductivity Θs Saturated water content Θ_{res} Residual water content Θ_{eff} Effective saturation water content pFc, Capillary pressure at field capacity pFw, Capillary pressure at wilting point n, Exponent of hydraulic conductivity curve
MIKE SHE ET Evapotranspiration	Time series of vegetation Leaf Area Index Time series of vegetation root depth	C_1, C_2, C_3 : Empirical parameters C_{int} : Interception parameter A_{root} :Root mass parameter Kc : Crop coefficient
MIKE SHE OC Overland and river/canal flow (MIKE11)	Topographical map Boundary conditions Digitized river/canal network River/canal cross sections	M, Overland Manning no. D , Detention storage L, leakage coefficient M, River/canal Manning no.
MIKE SHE IRR Irrigation module	Irrigated areas Irr. sources (pumps/canals/reservoirs) Distribution method (sheet, sprinkler, drip) Source capacity	Eact/Epot, crop water stress factor (target ratio between actual and potential evapotranspiration rates) Well threshold

Table 2-6: Primary Parameters Adjusted During Calibration

Model component	Calibration parameters	Parameter range
MIKE SHE SZ – Saturated zone flow	K_v: Vertical hydraulic conductivity (m/s)	$9.7 \cdot 10^{-11} - 1 \cdot 10^{-3}$
	K_H/K_v	1 - 1000
	Drainage time constant (s^{-1})	$2.9 \cdot 10^{-6} - 0.00$
	Drain level (m)	-1.62 – 13.30
	Boundary head conditions:	
	Northern Boundary (-)	Time Varying
	Eastern Boundary	No Flow
	Tidal Boundary	Fixed Head
MIKE SHE OC – Overland and river/canal flow (MIKE11)	M, Overland Manning no. $m^{1/3}/s$	0 - 2
	D , Detention storage (mm)	50 - 100
	L, leakage coefficient (s^{-1})	$9.9 \cdot 10^{-7} – 9.9 \cdot 10^{-5}$
	Canal M (Reverse of Manning's n) ($m^{1/3}/s$)	2 - 35
	Floodplains M ($m^{1/3}/s$)	2 - 35

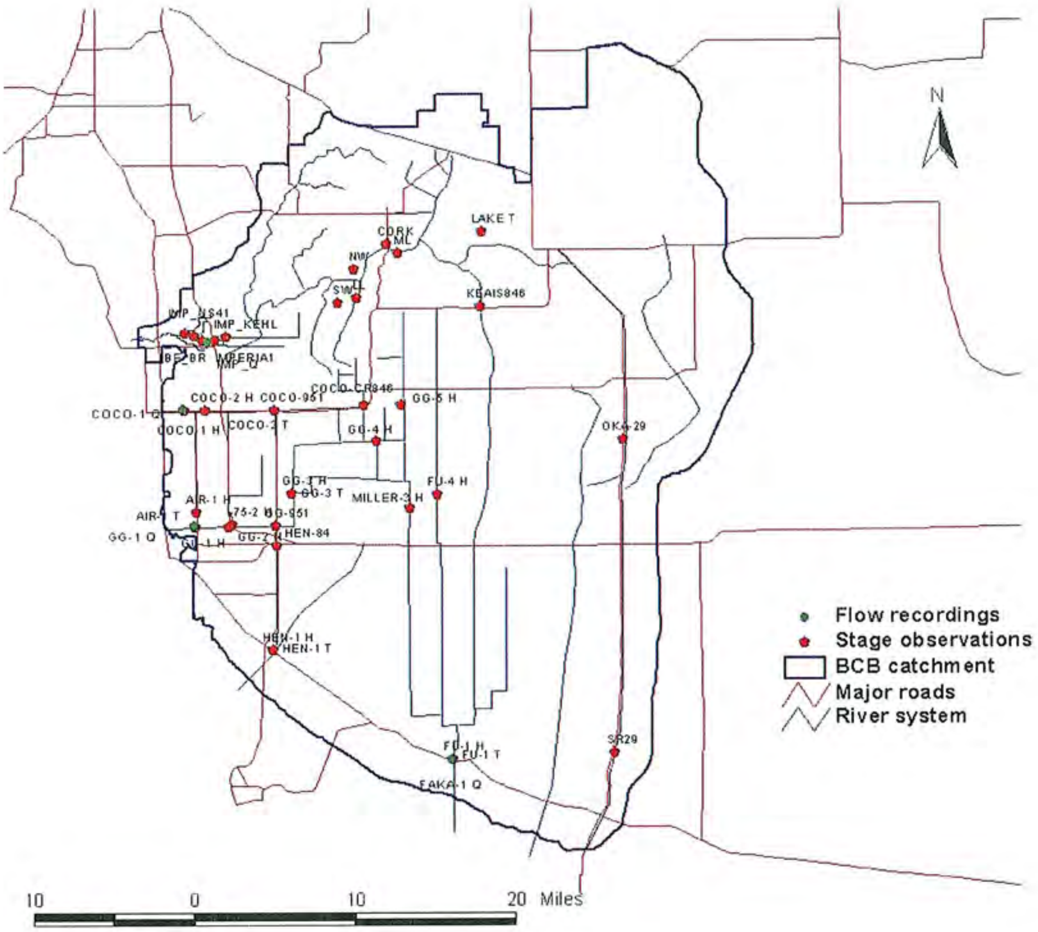


Figure 2-25: Locations of Flow and Stage Observations for 1990-1995

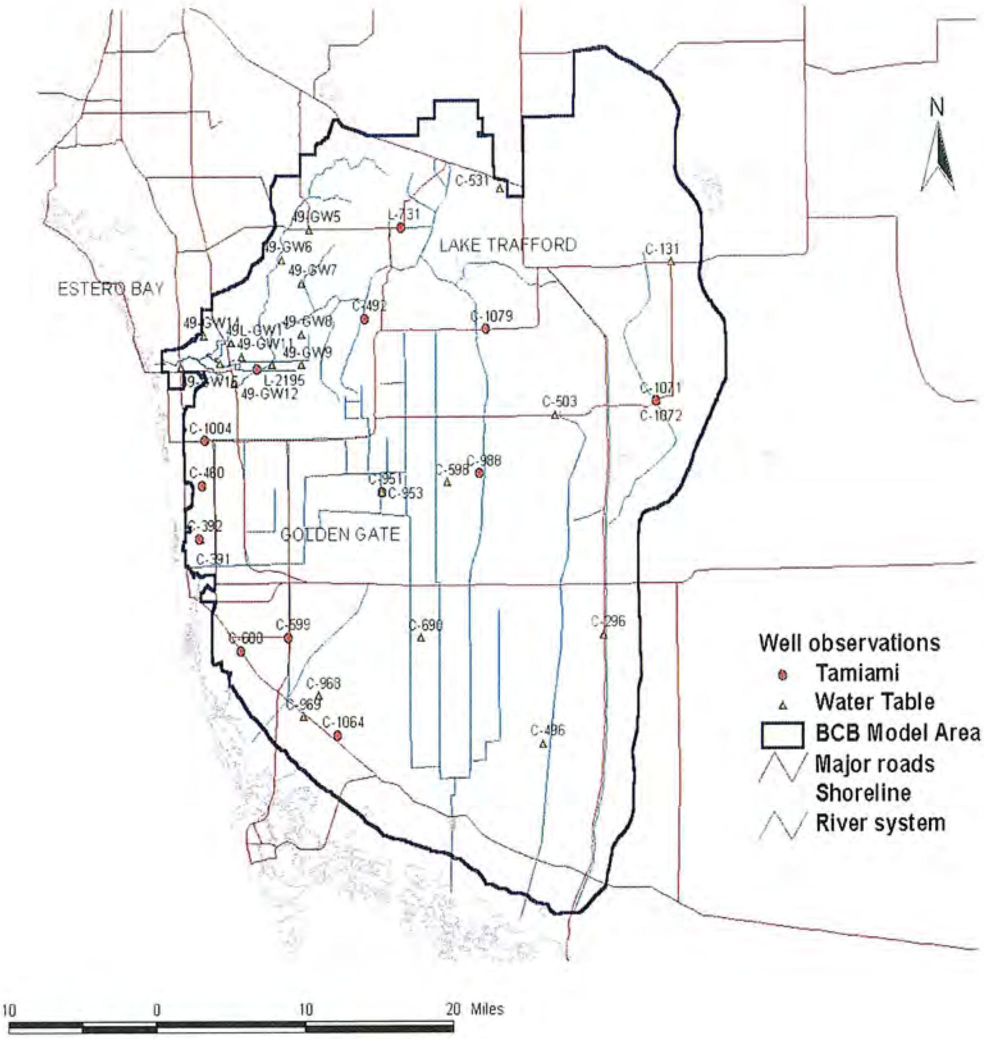


Figure 2-26: Groundwater Well Observations for 1990-2000 in the Big Cypress Basin Watershed

3.0 H&H ASSESSMENT FOR FLOOD PROTECTION

3.1 DESIGN STORM EVENT

The flood modeling for this project follows the technical recommendation guidelines proposed recently for the Project Implementation Report (PIR) of the Picayune Strand Restoration project (June, 2003, Guidelines for Design Storm Depth - Duration-Frequency; Temporal Distribution of Storm Rainfall Depth; and Antecedent Moisture Conditions Assessment for Design Storm Simulation by MIKE SHE/ MIKE 11). A 5-day, 25-year return period rainfall event was used as the design storm for H&H evaluation.

3.1.1. Spatial Distribution of Design Storm:

The spatial design storm depth-duration-frequency analysis is outlined in the SFWMD Technical Publication EMA#390 (2001) *Frequency Analysis of Daily Rainfall Maximum for Central and South Florida.* These methods are based on extensive frequency analysis of local rainfall data tested with several widely used probability distribution methods. The BCB staff has refined the spatial GIS coverages of three design storms for incorporating those data with BCB MIKESHE model. Figure 3-1 represents the spatial distribution map for a 25 year storm.

3.1.2. Temporal Distribution of Design Storm

For the one and three-day temporal distribution of design rainfall event, the applicable distribution is recommended in the SFWMD Basis of Review. This distribution is generally used for all storm water management regulatory functions in South Florida. The Basis of Review document does not specify a 5-day temporal distribution. The BCB staff reviewed the local rainfall distribution pattern of three major tropical storms affecting Collier County in the recent past at five recording rainfall stations. A composite distribution curve (Figure 3-2) was developed based on the local storm patterns and recommended for use in the BCB project.

BCB Five - Day Maximum Rainfall (in inches)
25 - Year Return Period

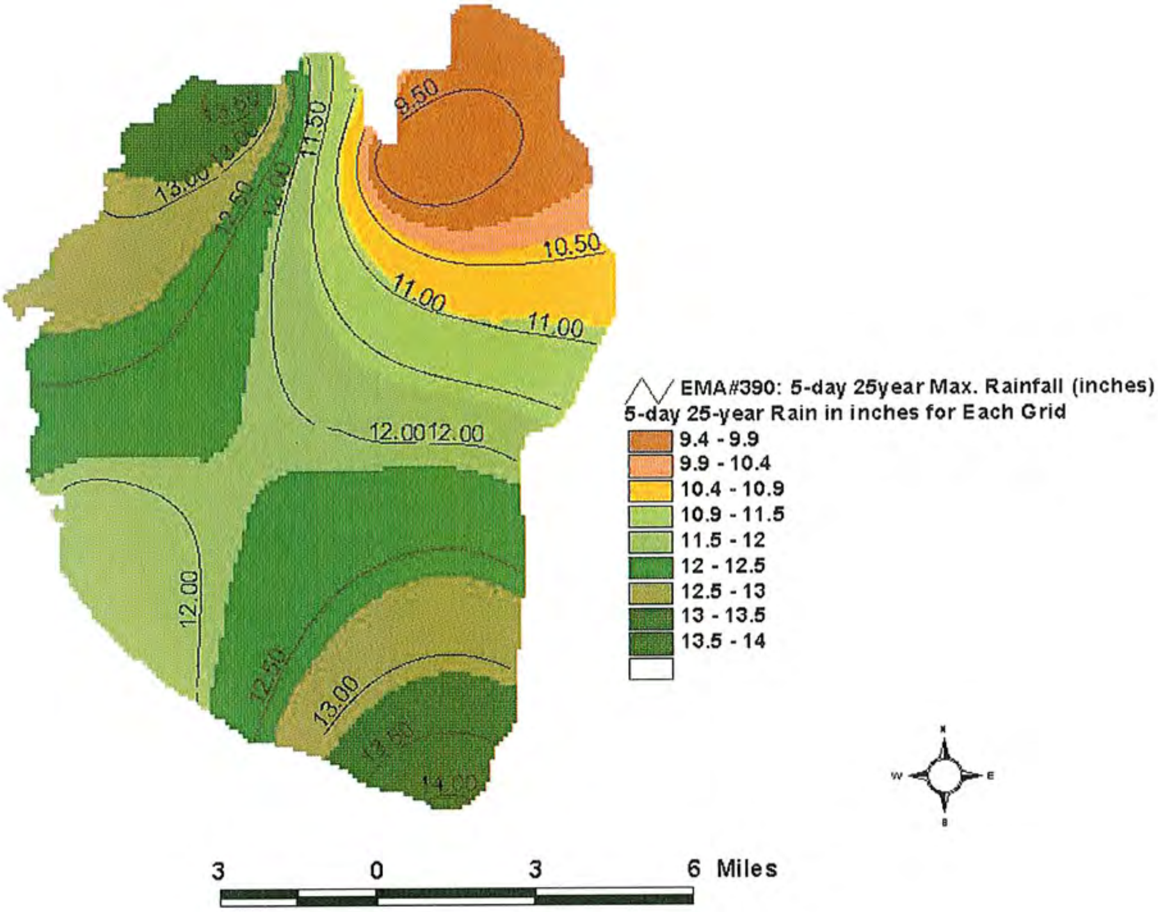


Figure 3-1

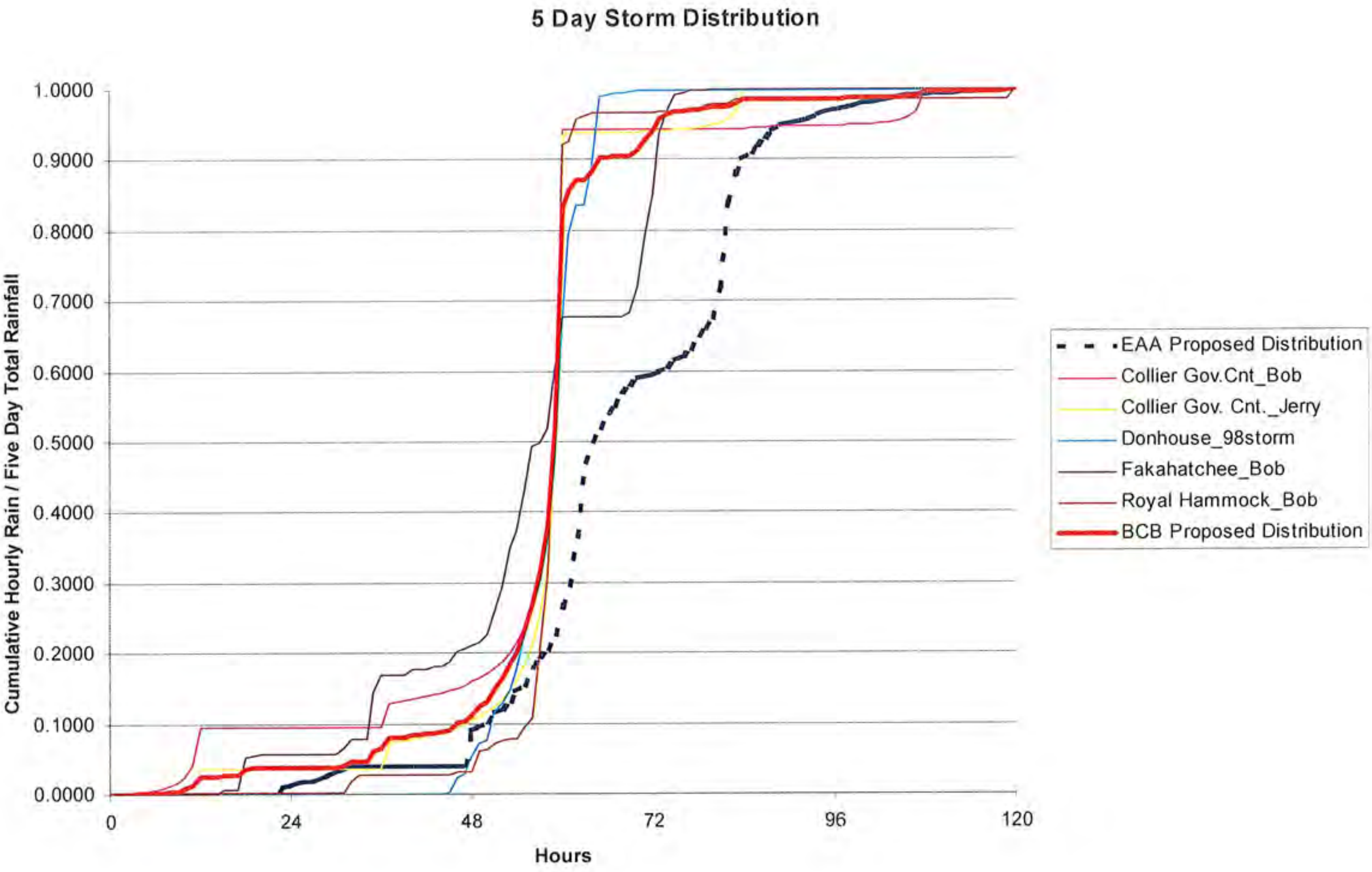


Figure 3-2

3.1.3. Antecedent Moisture Conditions (AMC) for Continuous Process Hydrologic-Hydraulic Simulation by MIKE SHE/MIKE 11

The SFWMD Basis of Review recommends the use of normal (long-term) wet season water table conditions to determine the AMC and that is used along with the design storm in designing the storm water management facilities. This method could generally be used for all design storms for the runoff curve number methods of runoff computation. These standard runoff curve numbers are based on AMCII conditions.

However, for continuous process simulation by MIKE SHE/MIKE 11, it is necessary to simulate the design storm starting at a point in time where the proper soil moisture conditions are achieved.

The BCB and DHI staff have analyzed the measured water table elevations of eleven monitor wells in the model domain to determine the beginning time frame of high average annual wet season during the simulation period of 1988-2000. The seasonal water table levels were also correlated with rainfall records. Based on this analysis, the beginning of August 1995 has been found to be a representative time frame of average annual high water table conditions. Therefore, the design flood analysis in MIKE SHE/MIKE11 model was simulated by inserting the design storm distribution beginning August 1, 1995.

3.2 PROPOSED IMPROVEMENTS

Since the basic purpose of this assessment is to formulate a conceptual design for replacement of the inefficient water control structure, only structural configuration alternatives were investigated. Conveyance capacity enhancement measures like channel modification were not explored due to limitations of economic and environmental feasibility of the project. Presently the Golden Gate Main canal has very limited right-of-way. Due to rapid urban growth and explosive real estate prices, acquisition of additional right-of-ways for widening the canal, particularly for the critical reach of the canal west of CR 951, is not economically feasible at this time.

For replacement of the present structure three different configurations of gated control structures presently utilized by the District were investigated:

- Vertical lift gates with side spillways (similar to COCO #1)
- Hinged crest radical gates (similar to GG#1)
- Obermeyer spillway gates (similar to S381 structure)

All of the above three types of gated structures are operable by automated control to achieve the desired range of objectives for flood control and maintenance of conservation pools. However, based on SFWMD and COE experience on installation, operation and overall project cost, a gated structure with Obermeyer type of gates has been proposed as the effective replacement of Golden Gate Canal Weir #2.

The Obermeyer Spillway Gate system is simply described as a row of steel gate panels supported on their downstream side by inflatable air bladders. By controlling the pressure in the bladders, the pond elevation maintained by the gates can be infinitely adjusted within the system control range (full inflation to full deflation) and accurately maintained at user-selected set-points (Figure 3-3).

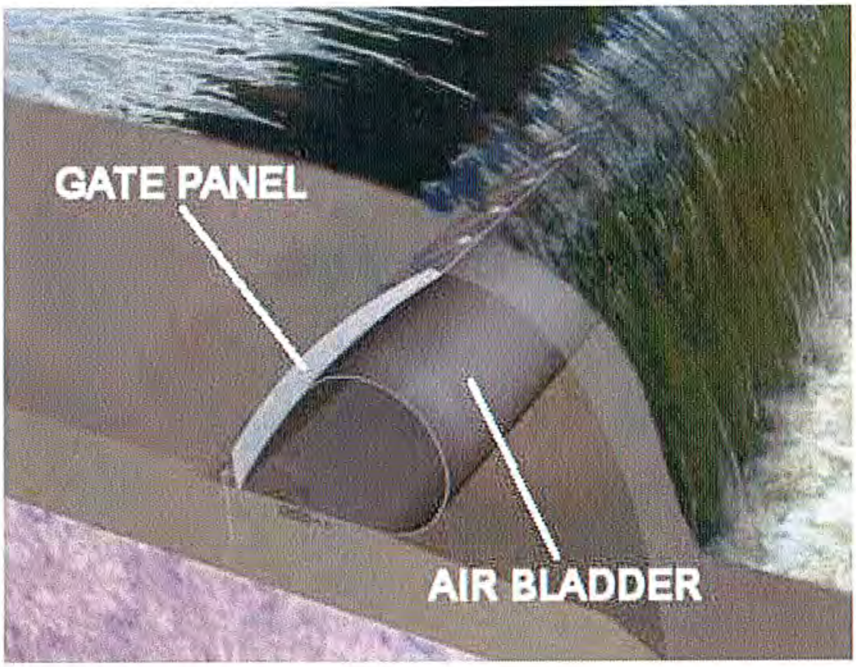


Figure 3-3: The OBERMEYER Spillway Gate System

The spillway gate system is attached to the foundation structure by stainless steel anchor bolts (epoxy or non-shrink cement grout, as design dictates). The required number of bladders are clamped over the anchor bolts and connected to the air supply pipes. When the bladder hinge flaps are fastened to the gate panels, the installation of the strong, durable and resilient crest gate system is complete.

The gaps between adjacent panels are spanned by reinforced EPDM rubber webs clamped to adjacent gate panel edges. At each abutment, an EPDM rubber wiper-type seal is affixed to the gate panel edge. This seal rides up and down the stainless steel abutment plate, keeping abutment plate seepage to a minimum. Alternatively, rubber seals may be fixed to the abutments or piers which engage the raised gate panels.

The OBERMEYER Spillway Gates can be custom designed to conform to any existing or desired spillway cross-section with a minimum profile when in the lowered position. The wedge-shaped profile of the OBERMEYER Gate System causes stable flow separation from the downstream edge of the gate without the vibration-inducing vortex shedding associated with simple rubber dams during overtopping. This results in vibration-free operation and excellent control throughout a wide range of head water elevations and gate positions.

The proposed structure will consist of three automated Obermeyer spillway gates. Each gate is 26 ft, and 8 in. wide. The total spillway width is 80 feet. The top of the hinge when the gates are down, is at elevation 0 ft NGVD (-1.27 ft NAVD). When the gates are raised, the top of the gates are at 6.3 ft NGVD (5.03 ft NAVD). Each gate can open independently, as needed, to maintain the target water surface elevation upstream of the proposed structure. The operating schedule has both a wet season setting and dry season setting. These settings are described in later sections of the report. Figures 3-4 through 3-6 give the detailed structure layout, elevation and site plan. A three dimensional layout of proposed Golden Gate structure GG-2 can be found in Appendix B.

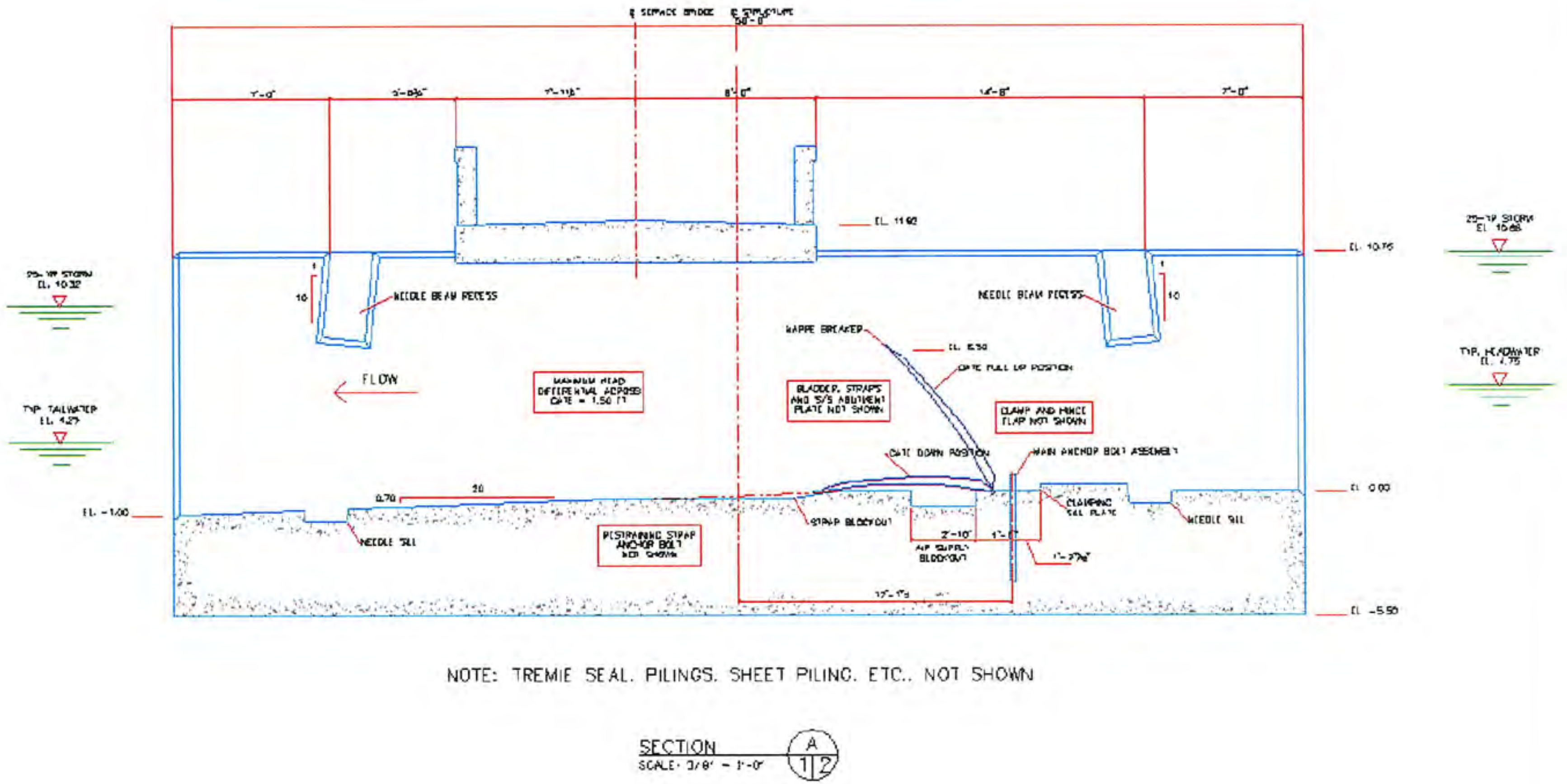
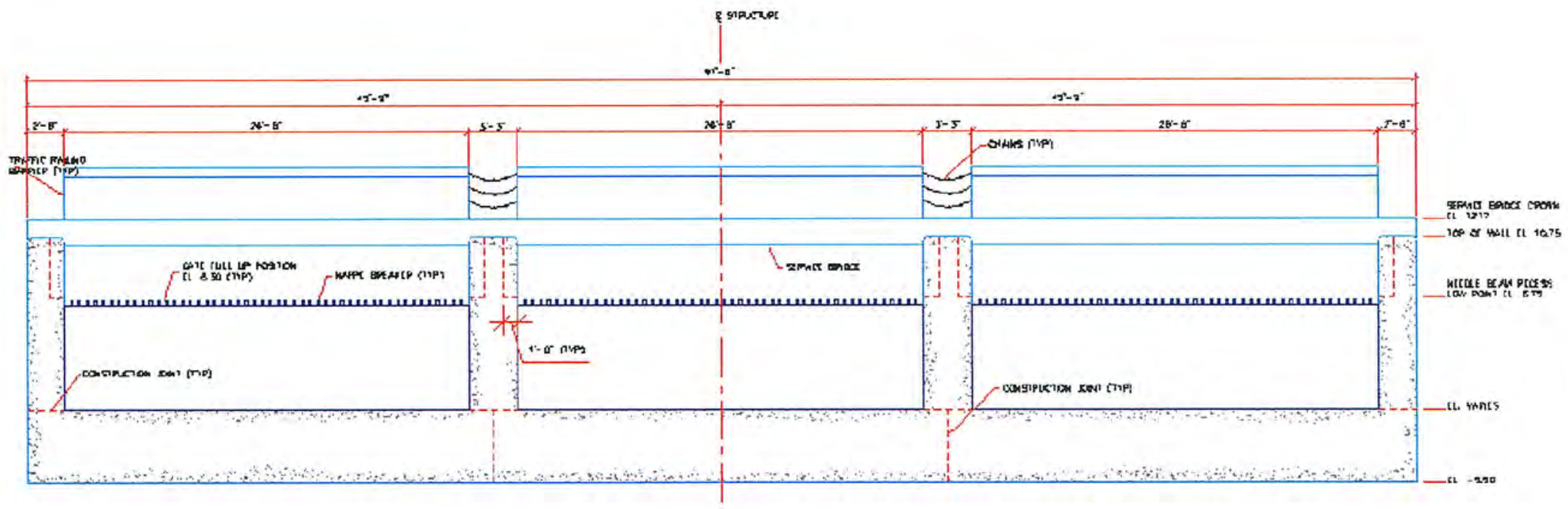


Figure 3-4: Proposed GG-2 Cross Section (Elevation in ft NGVD)



NOTE: TREMIE SEAL, PILINGS, SHEET PILING, ETC., NOT SHOWN

SECTION B 1|3
SCALE: 3/4" = 1'-0"

Figure 3-5: Proposed GG-2 Structure Elevation (ft NGVD)

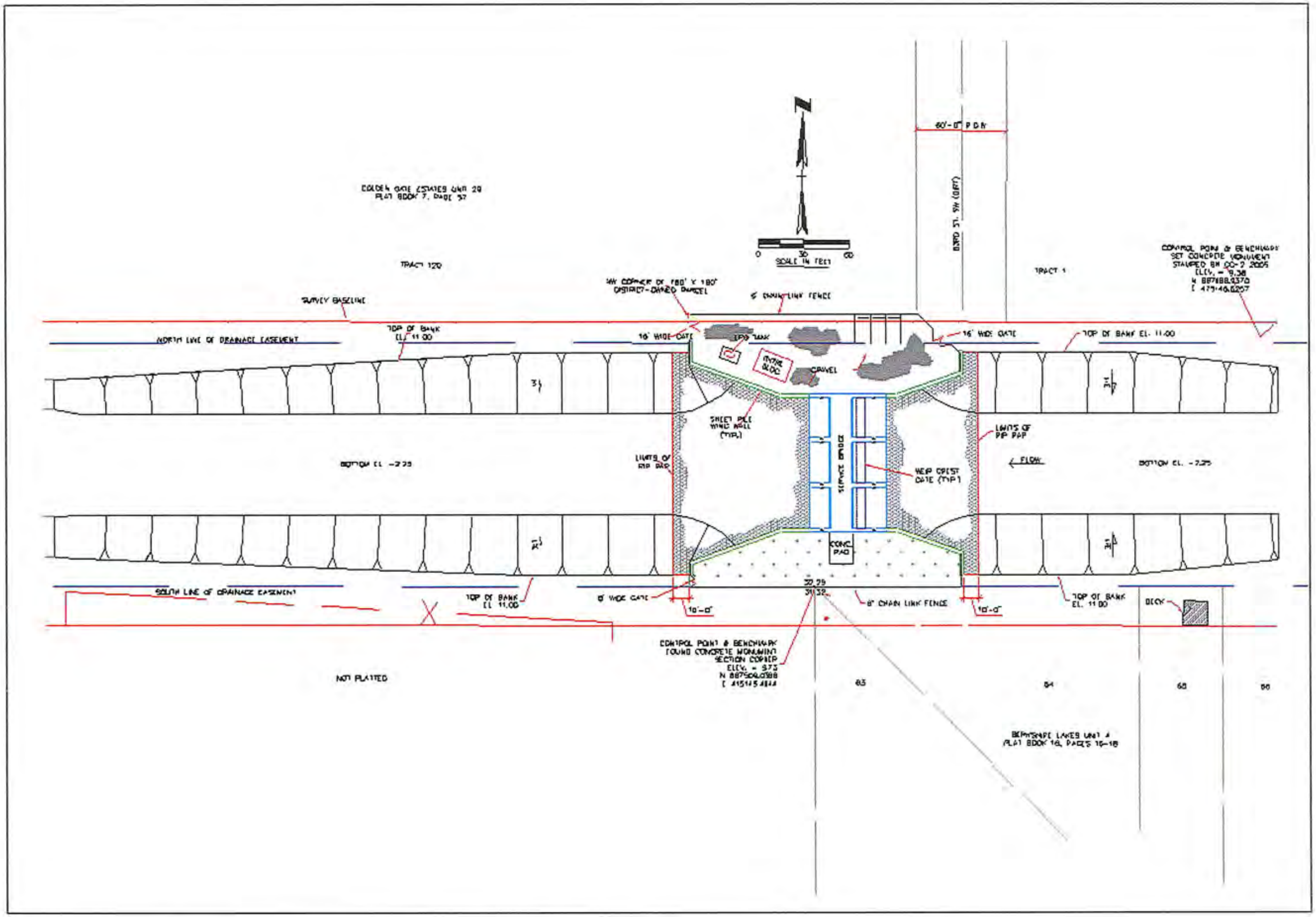


Figure 3- 6: Proposed GG-2 Structure Site Plan (Elevation in ft NGVD)

3.3 CHANNEL FLOW MODELING

The calibrated interactive MIKE SHE/MIKE 11 model was used to simulate the existing GG-2 and modified scenarios. Both existing and proposed structures were simulated for a 2-year period (1/6/93 to 8/1/95) to represent the year 1994 as the average hydrologic condition period, and for a 25-year, 5-day storm event to represent the design storm event. The two year long-term simulations were made to investigate the variations of both channel flow/stage and groundwater levels with the proposed structure performance.

The simulated channel flows and stages with the proposed structure were compared with those under the existing condition to evaluate the performance for flood control, freshwater flow reduction to Naples Bay and change in groundwater levels. The comparison of the existing and proposed condition has been conducted at the structure locations of GG-1, GG-2, GG-3, GG-4, CY-1, I-75-1, Airport Road Bridge, FU-1 and COCO 1 in the BCB canal system. The features of those structures are summarized in Table 3-1. The existing and proposed gate operating schedule of GG-2 are summarized in Table 3-2.

The flood profile along Golden Gate Canal for the 25-year storm runoff under existing and proposed GG-2 conditions simulated by MIKE 11 model is presented in Figure 3-4. The water surface profile indicates that the 25-year stages in the large part of the canal do not stay within the banks. Although the elimination of the fixed crest weir and addition of large gates will provide larger draw-down capabilities, the conveyance capacity of the canal is limited by the existing size of the canal. Hence, with larger storms, like a design 25-year storm, the hydraulic analysis indicates that the weir modification will not result in significant reduction of flood stage from the existing levels. The modified GG-2 will not have adverse impact on the current levels flood protection in the Golden Gate watershed.

Figures 3-5-A through 3-6-I are the flow and stage hydrographs for 25-year design storm. The peak flow and stage at selected locations are also summarized in Table 3-3.

Table 3-1: Summary of Existing Canal Crossings in the BCB Canal System

Structure Name	**Location**	**Structure Description**	**Structure Invert Elevation**	
			NAVD in ft	**NGVD in ft**
GG-1	Golden Gate canal	Three 26.8' movable crest spillway	Crest elevation: 3.73' NAVD Invert of gate: -2.27' NAVD	Crest elevation: 5.0' NGVD Invert of gate: -1.0' NGVD
GG-2	Golden Gate canal	One 105' fixed crest weir, two 6'x5' gates	Crest elevation: 3.73' NAVD Invert of gate: -2.27' NAVD	Crest elevation: 5.0' NGVD Invert of gate: -1.0' NGVD
GG-3	Golden Gate canal	One 100' fixed crest weir, two 6'x5' gates	Crest elevation: 6.23' NAVD Invert of gate: -1.57' NAVD	Crest elevation: 7.5' NGVD Invert of gate: -0.5' NGVD
GG-4	Golden Gate canal	One 100' fixed crest weir, two 6'x5' gates	Crest elevation: 8.27' NAVD Invert of gate: 1.27' NAVD	Crest elevation: 9.5' NGVD Invert of gate: 2.5' NGVD
I-75-1	I-75 canal	One 96' fixed crest weir, two 6'x5' gates	Crest elevation: 4.73' NAVD Invert of gate: -0.27' NAVD	Crest elevation: 6.2' NGVD Invert of gate: 1.0' NGVD
CY-1	Cypress Canal	One 42' fixed crest weir, two 5'x4' gates	Crest elevation: 8.23' NAVD Invert of gate: 1.27 NAVD	Crest elevation: 9.5' NGVD Invert of gate: 2.5 NGVD
AirPort Bridge	Golden Gate Canal and Airport road	Bridge with piers	Invert elevation: -5.87' NAVD	Invert elevation: -4.6' NGVD
FU-1	Faka Union Canal	One 200 feet fixed crest weir	Crest elevation: 0.73' NAVD	Crest elevation: 2.0' NGVD
COCO-1	Cocohachee Canal	Two-bay gated spillway with vertical-leaf roller gates (10'x7'each)	Top of Spillway: 4.23' NAVD Gate invert elevation: -2.27' NAVD	Top of Spillway: 6.5' NGVD Gate invert elevation: -1.0' NGVD

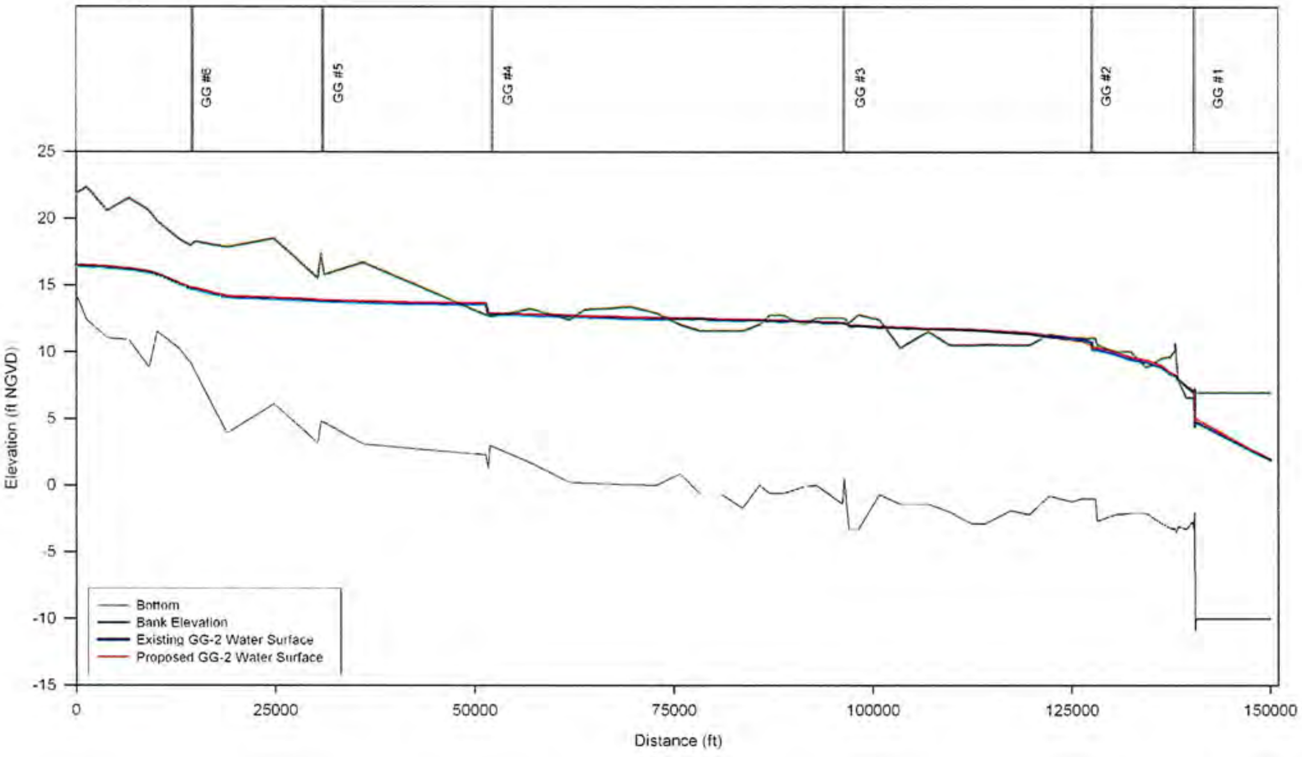
Table 3-2: GG-2 Operating Schedule

	Elevation in NGVD			
	Wet season water level (ft NGVD)		**Dry season water level (ft NGVD)**	
	Open Elevation	**Close Elevation**	**Open Elevation**	**Close Elevation**
Existing (Manual Operating)	5.5	5.0	6.0	5.25
Proposed (Automated with manual override)	6.0	5.5	7	5.75

Table 3-3: Summary of Maximum Flow and Stage During a 5-Day, 25-Year Design Storm

Canal Crossings	**Mike11 Chainage**	**Existing Condition**		**Proposed Condition**	
		Peak Flow (cfs)	**Stage NGVD (feet)**	**Peak Flow (cfs)**	**Stage NGVD (feet)**
GG#1	42805	4425	6.97	4625	7.00
GG#2	38875	3821	10.72	4122	10.46
GG#3	29376	2009	12.11	2067	12.07
GG#4	15916	792	12.78	809	12.80
I-75-1	12201	1294	11.03	1296	10.84
CY #1	6749	472	13.08	481	13.11
AirPort Bridge	42060	4506	8.16	4680	8.16
FU-1	45922	3111	5.86	3113	5.86
COCO-1	15212	1339	9.81	1338	9.81

Figure 3-7. Maxium Water Surface Profile of Golden Gate Canal During a 5 Day 25 Year Storm



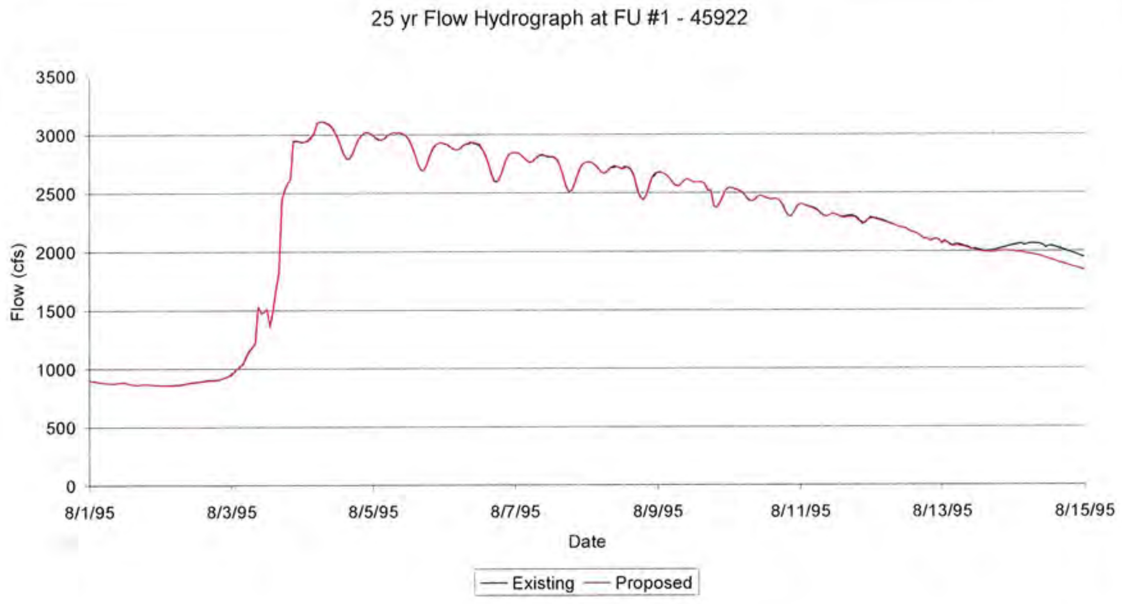


Figure 3-8-A

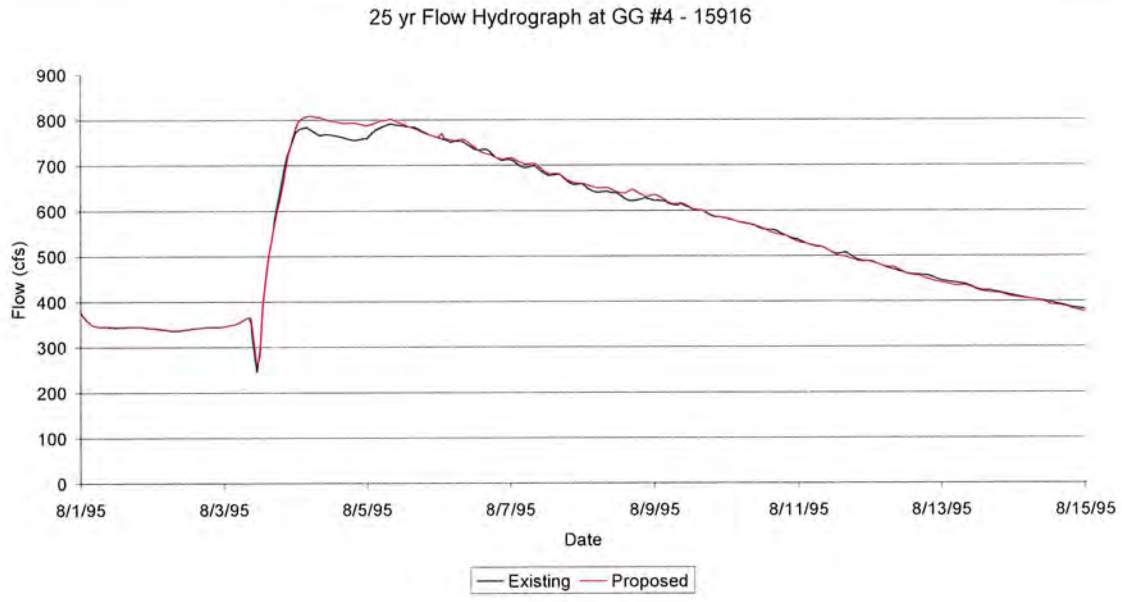


Figure 3-8-B

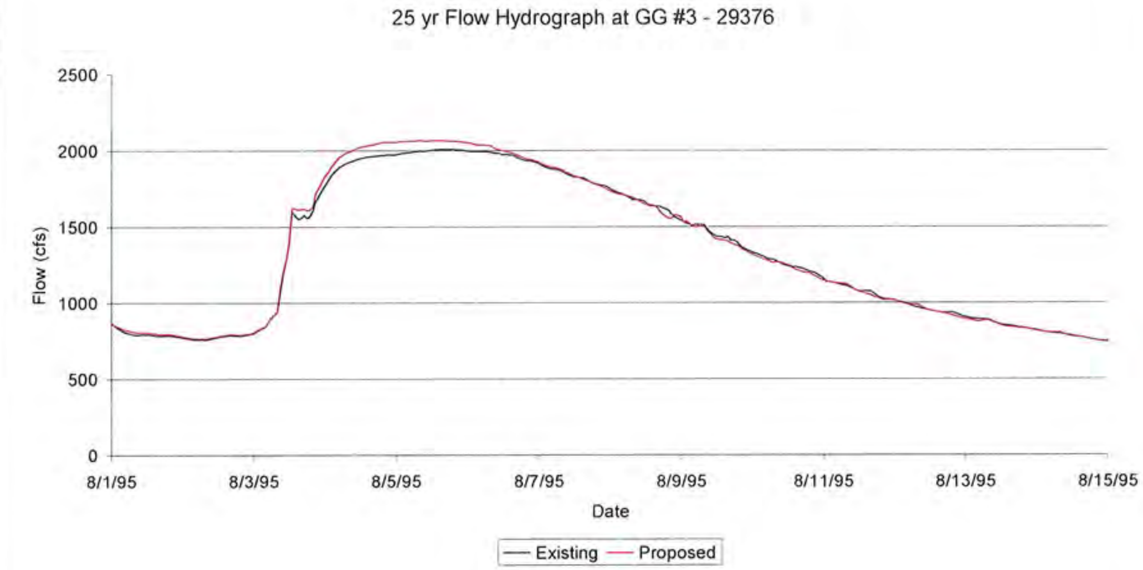


Figure 3-8-C

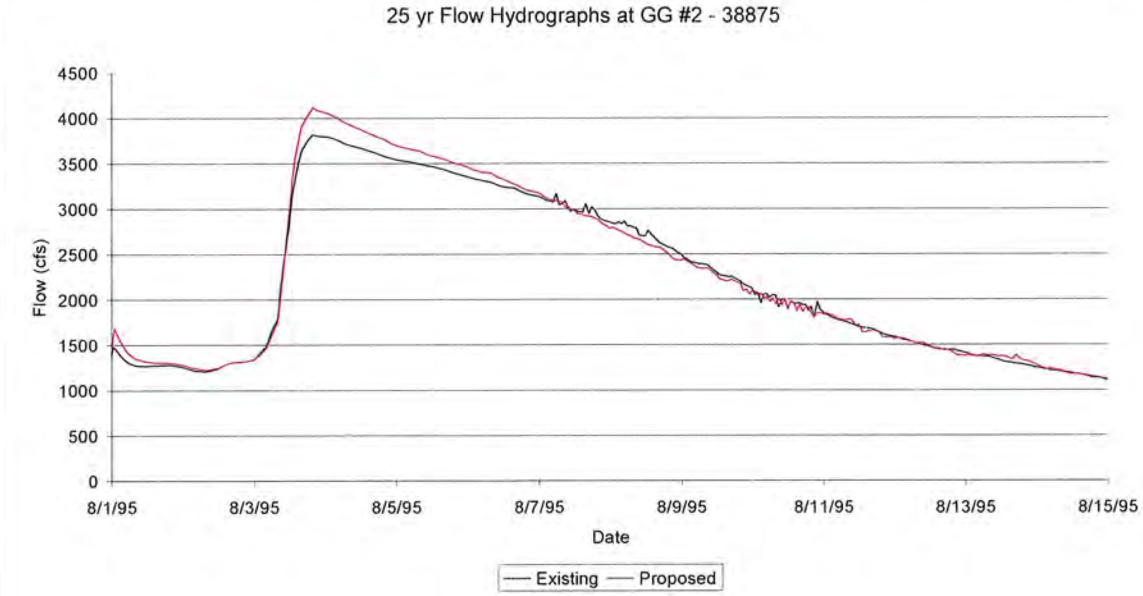


Figure 3-8-D

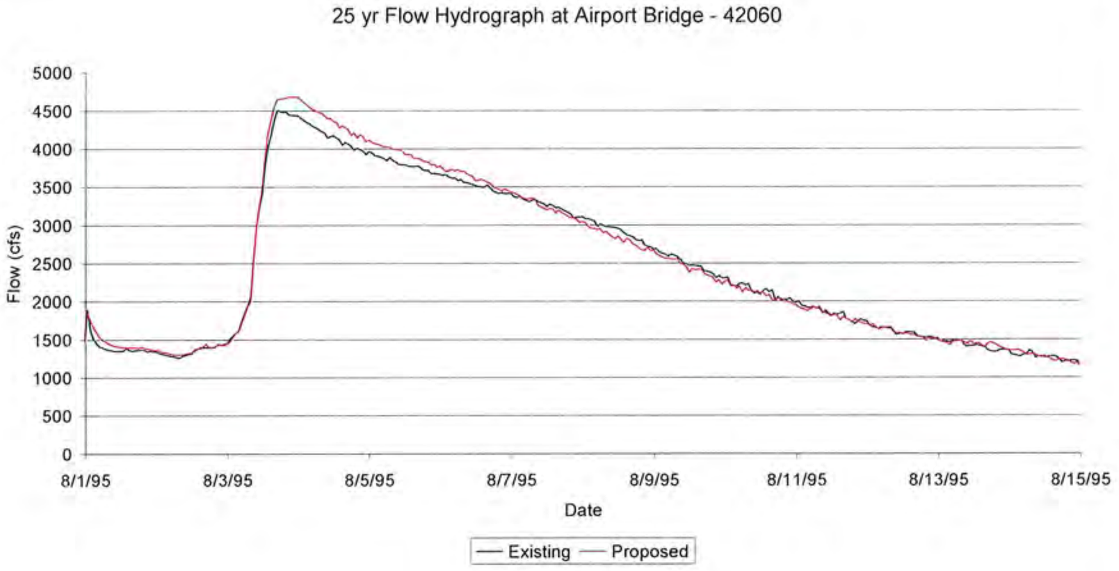


Figure 3-8-E

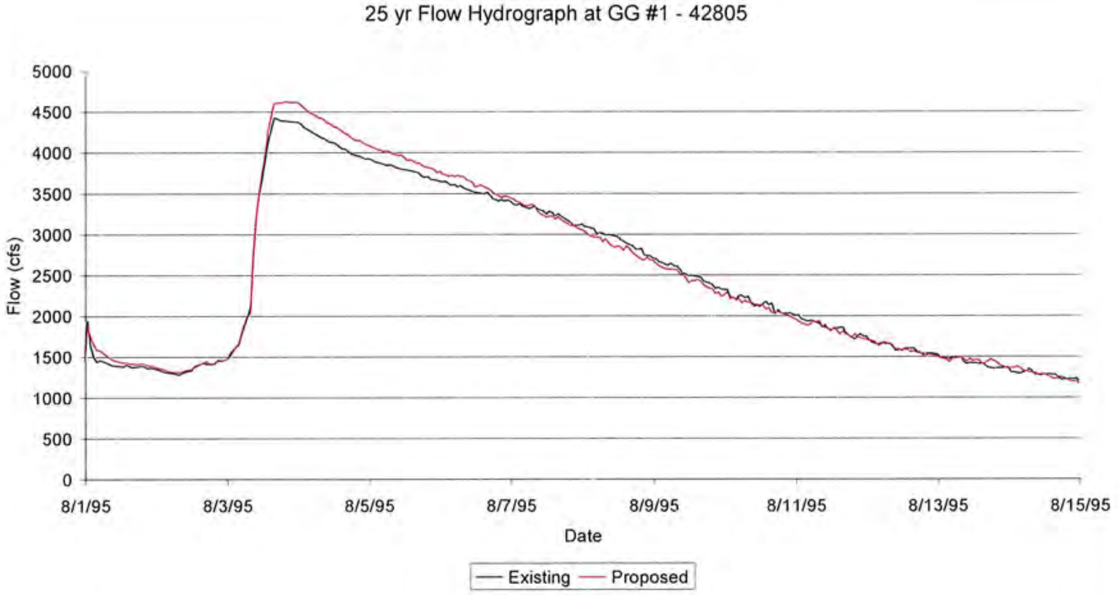


Figure 3-8-F

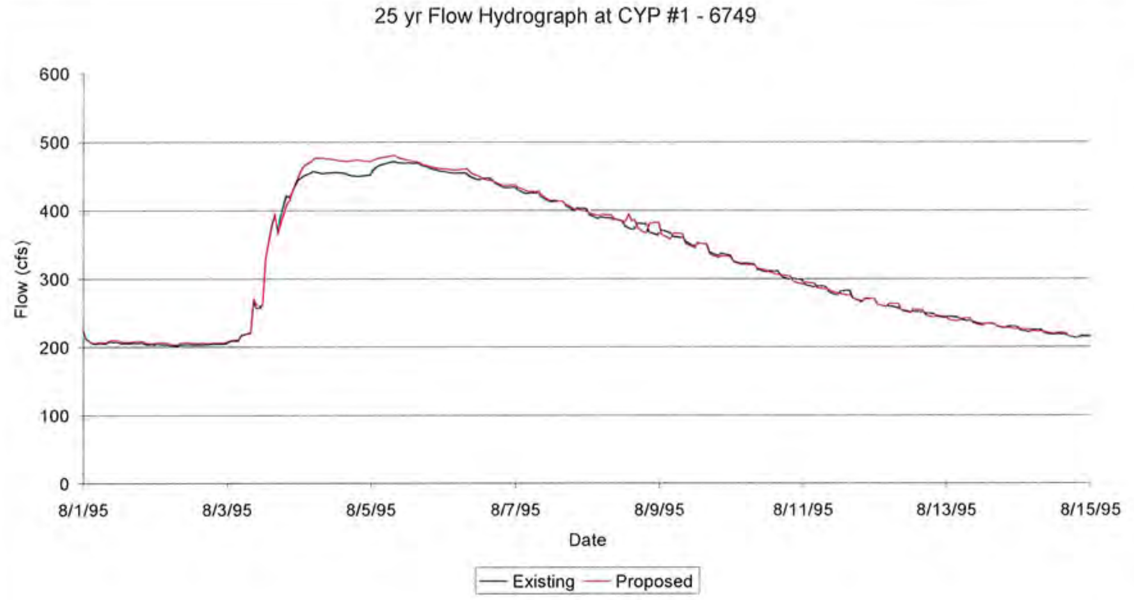


Figure 3-8-G

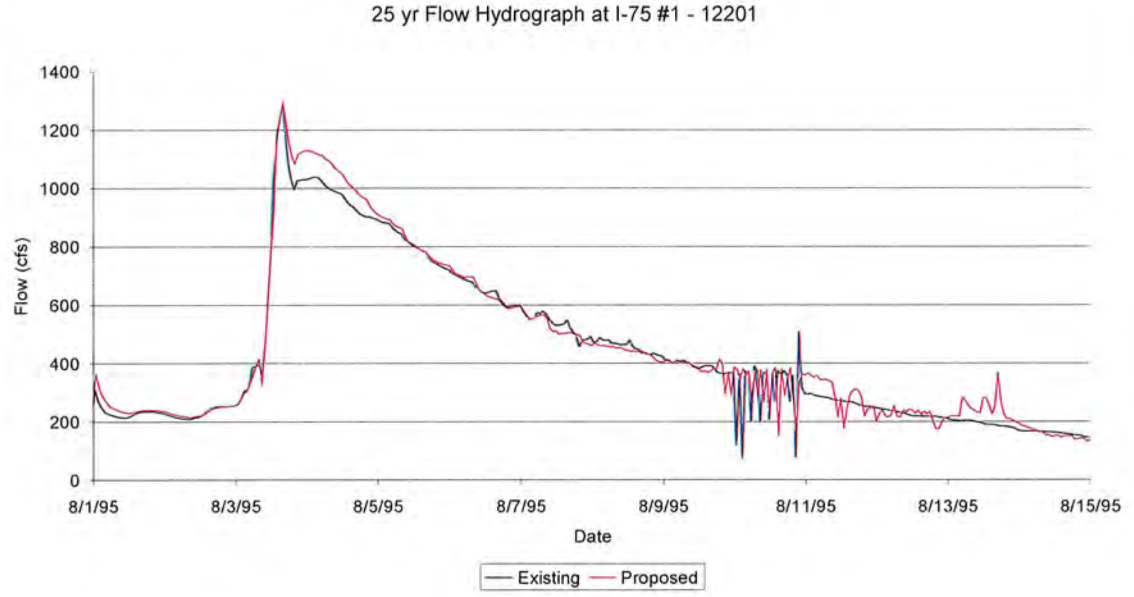


Figure 3-8-H

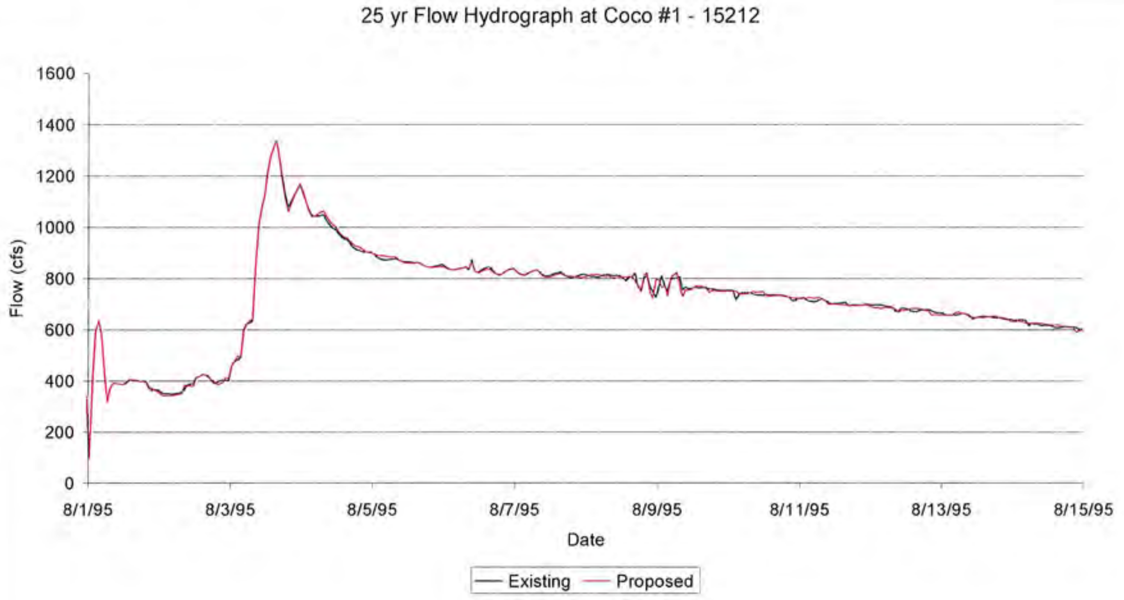


Figure 3-8-I

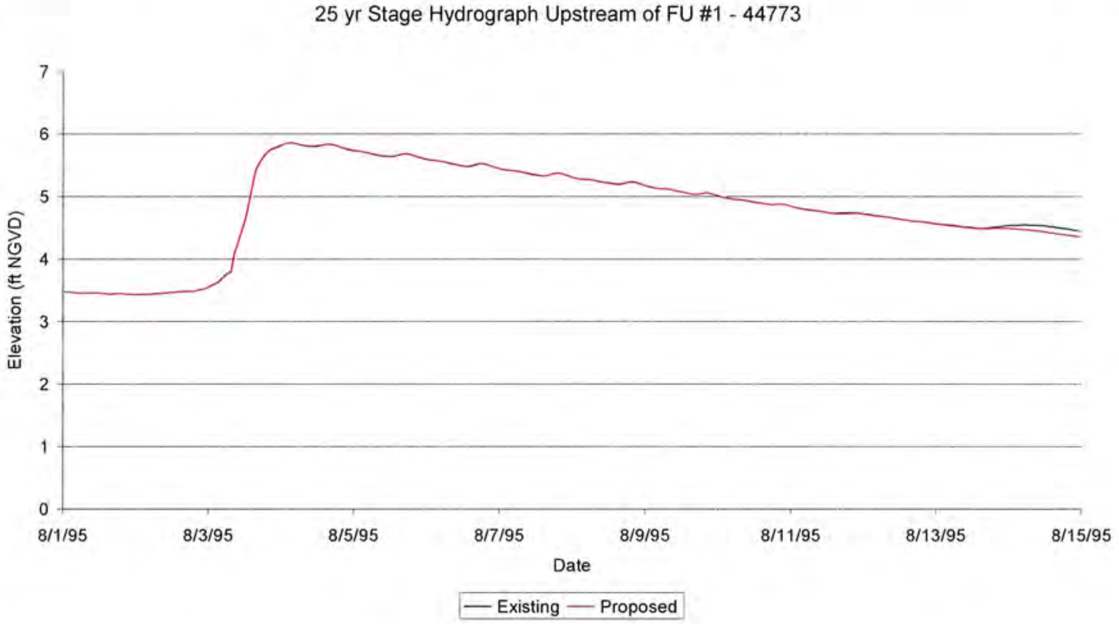


Figure 3-9-A

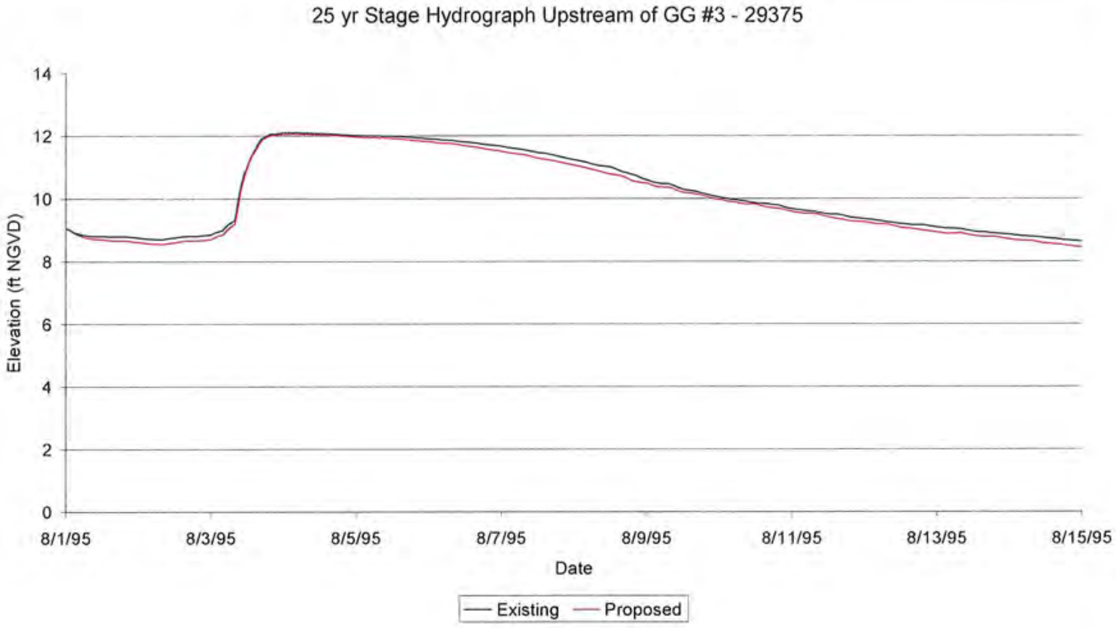


Figure 3-9-C

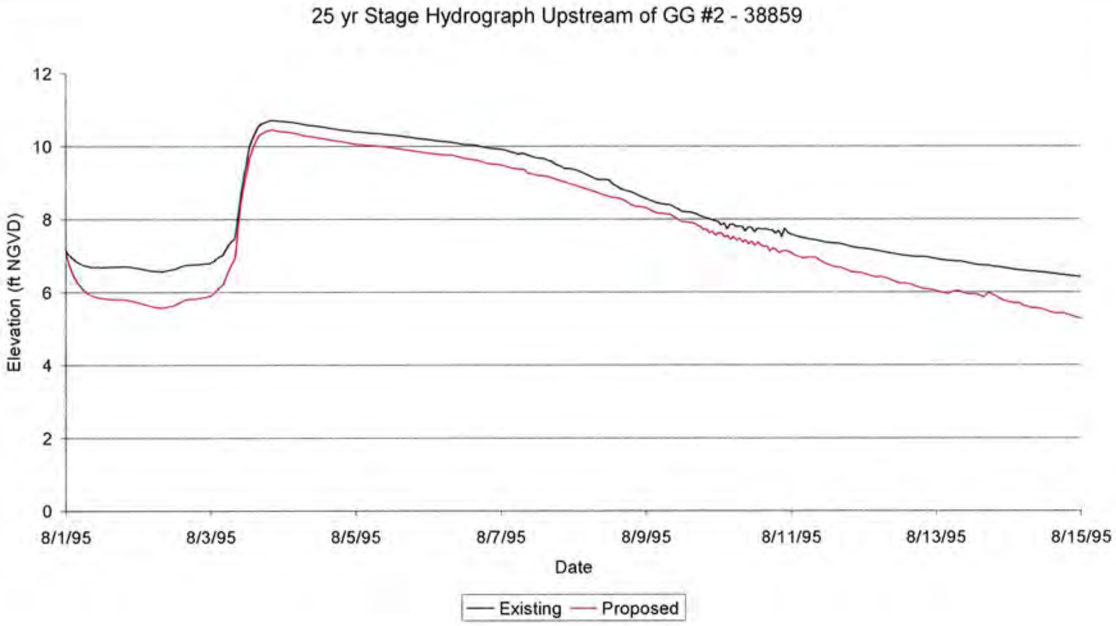


Figure 3-9-D

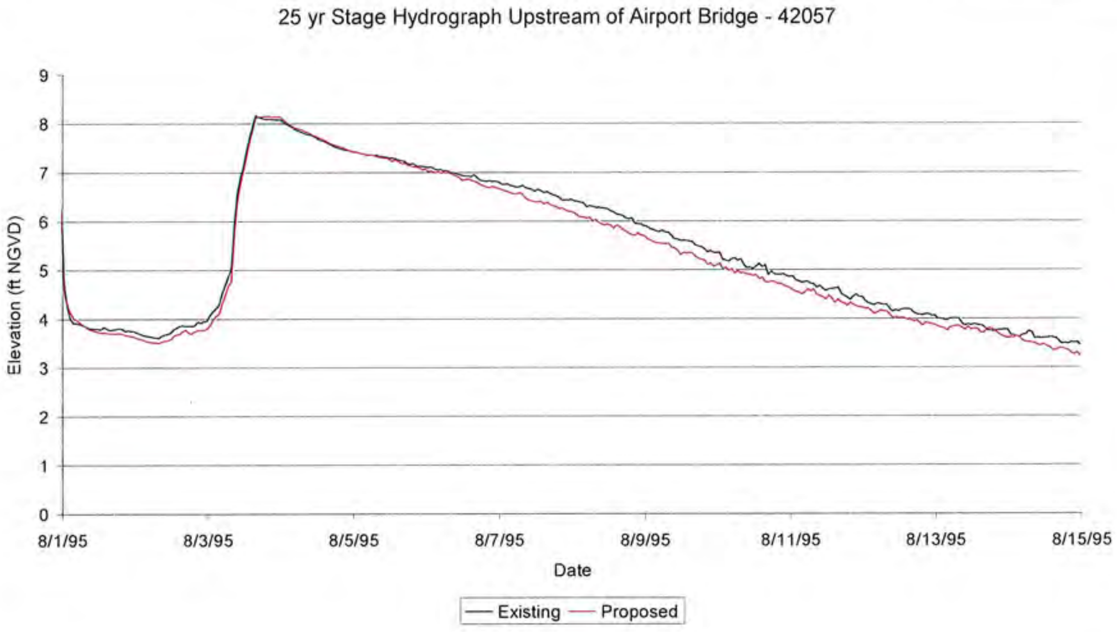


Figure 3-9-E

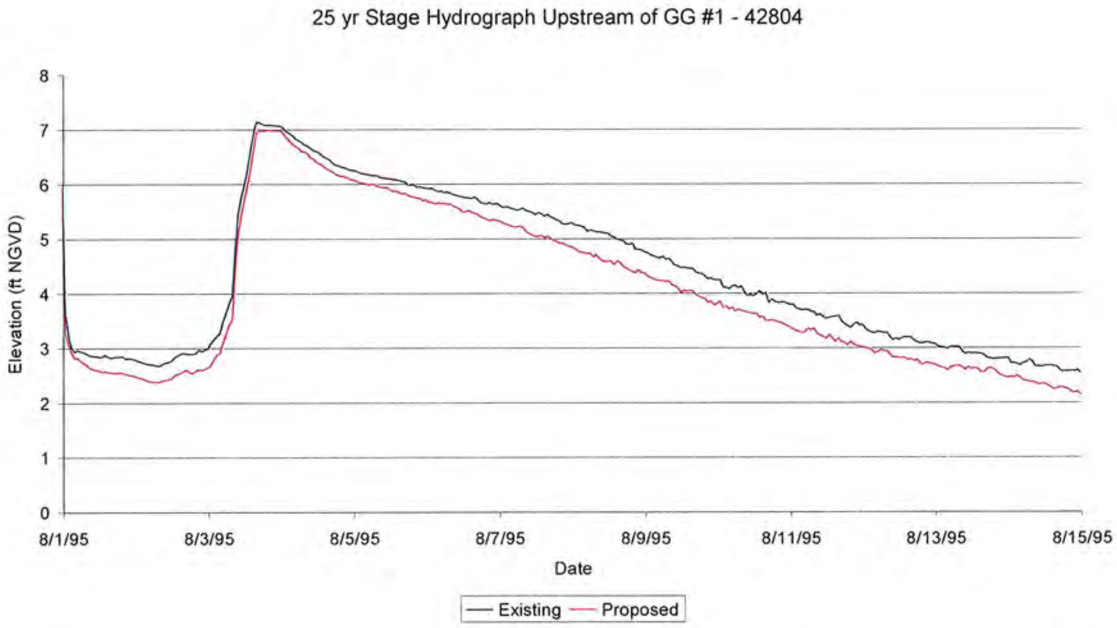


Figure 3-9-F

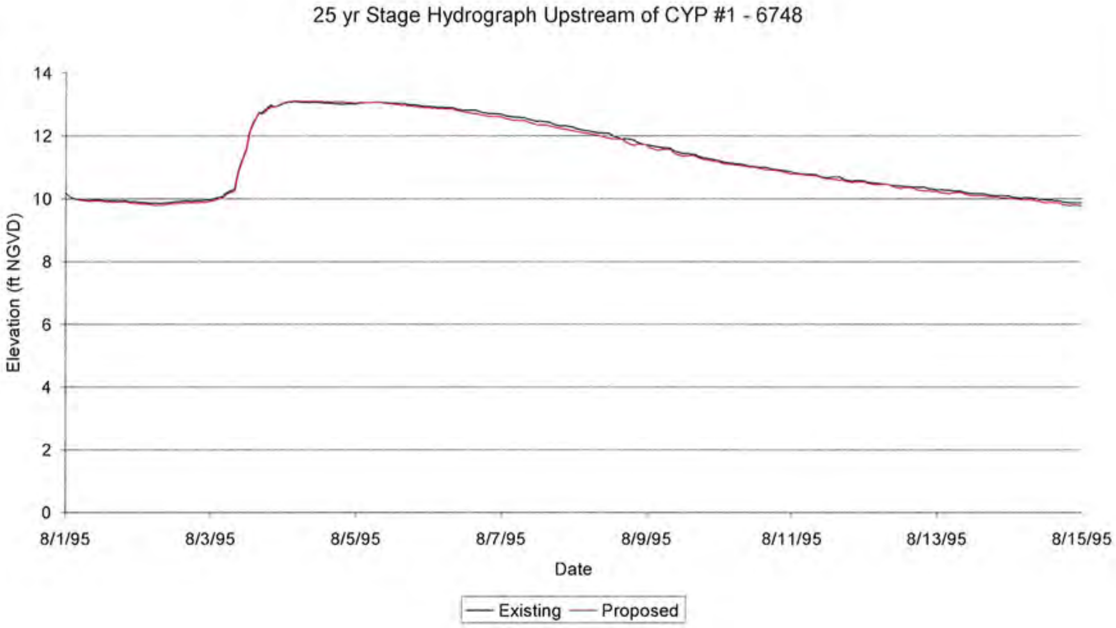


Figure 3-9-G

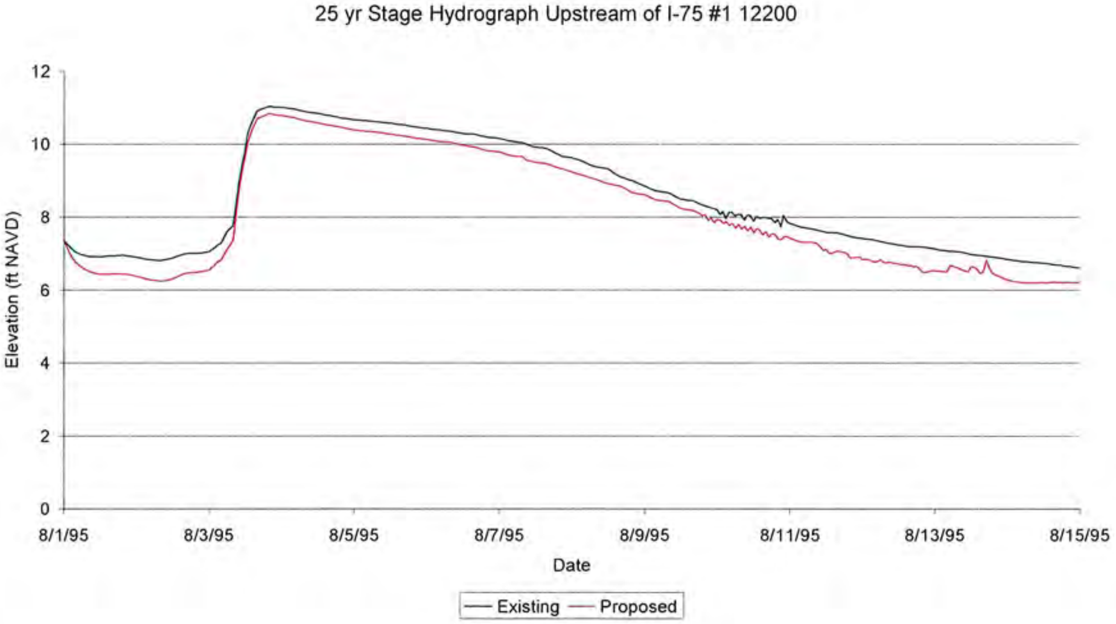


Figure 3-9-H

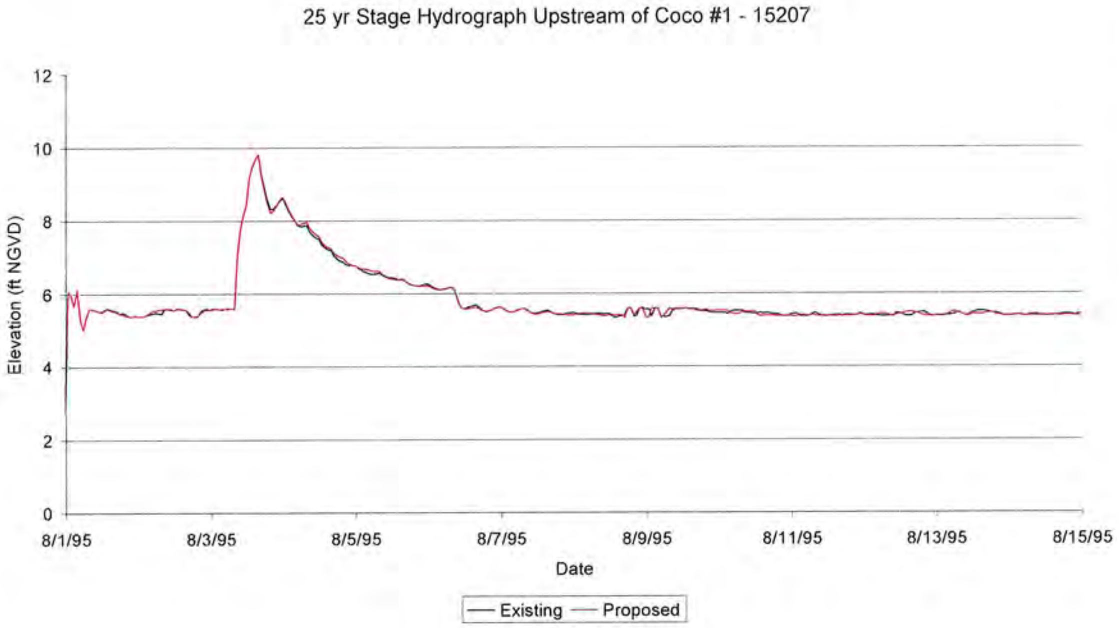

25 yr Stage Hydrograph Upstream of Coco #1 - 15207
12
10
8
6
4
2
0
Elevation (ft NGVD)
8/1/95
8/3/95
8/5/95
8/7/95
8/9/95
8/11/95
8/13/95
8/15/95
Date
Existing
Proposed

Figure 3-9-I

4.0 H&H ASSESSMENT FOR AVERAGE HYDROLOGIC CONDITION

Comparison of the change of surface and groundwater levels between the existing and proposed canal modification provides a measure of whether the proposed canal improvements impact the water levels in the surrounding areas, and how. Continuous simulations were performed for a 26 months period between 6/1/1993 and 7/31/1995. The hydrologic performances of a one year period between 5/1/1994 to 4/30/1995 was represented as the average meteorological condition of the Golden Gate Canal basin. The effect on surface and groundwater resources of the area under this average condition, resulting from the proposed modification and operation of the structure, are described below.

4.1 AVERAGE ANNUAL SURFACE AND GROUNDWATER CHANGE

4.1.1. Groundwater

The integrated MIKE SHE / MIKE 11 model proved to be a very useful tool to investigate the impact and change caused by the proposed GG-2 improvement. The effects of water level change in the water table aquifer caused by the structural improvements can be visualized by comparing the annual groundwater head difference between the existing and proposed improvement conditions. Figure 4-1 shows that the modification of GG-2 structure will cause groundwater level increases by 0.10 to 0.35 feet in the vicinity of the canal between GG-2 and GG-3 for an average year. The temporal effects are examined as well by comparing the time variations of groundwater levels at eight (8) selected locations listed in Figure 4-1. Figures 4-2-A through Figure 4-2-H represent water table hydrographs from 5/1/1994 through 4/30/1995 at those selected locations.

4.1.2 Surface Water

Figures 4-3-A through 4-3-I are simulated flow hydrographs and Figures 4-4-A through 4-4-I are simulated stage hydrographs at selected locations. Table 4-1 list the simulated total volume of flow pass through the GG-1 structure for existing and improved GG-2 conditions.

4.2 SURFACE AND GROUNDWATER LEVEL CHANGES DURING WET SEASON

4.2.1 Groundwater

During the simulated average wet season of 5/1/94 to 10/14/94, the proposed GG-2 structure will maintain the current water level condition. Figure 4-5 represents the average ground water level difference between proposed condition and existing condition during wet season. The difference maps show that during wet season there is no significant change to groundwater for the proposed structure improvement. The detailed temporal variations of groundwater levels during wet season at selected locations are given in Figures 4-2-A through 4-2-H.

4.2.2 Surface Water

Figure 4-6 compares the water surface profiles of Golden Gate between existing and proposed GG-2 structures in the middle of wet season, indicating almost no change in the stage of canal water during the middle of the wet season 9/1/1994. The detailed simulated flow and stage hydrographs for both existing and proposed GG-2 conditions at selected points in the Golden Gate Canal system can be found in Figures 4-3-A through 4-4-F.

4.3 SURFACE AND GROUNDWATER LEVEL CHANGES DURING DRY SEASON

4.3.1 Groundwater

During the simulated dry season of 10/15/94 through 4/30/95, the proposed GG-2 structure will enhance groundwater storage between the upstream of GG-2 and downstream GG-3, as illustrated in Figure 4-7. The detailed temporal variations of groundwater level during dry season at selected locations are given in Figures 4-2-A through 4-2-I.

4.3.2 Surface Water

Figure 4-18 compares the water surface profiles between existing and proposed GG-2 structures in the middle of dry season. The total volume of runoff at GG-1 during simulated dry season is given in Table 4-1. Figures 4-3-A through 4-4-F are simulated flow and stage hydrographs at selected points in the BCB canal system.

Table 4 -1: Simulated Total Volume of Flow Discharge through Structure GG-1

	Total Volume of Flow (Million Gallons)	
	Existing GG-2 Condition	**Proposed GG-2 Condition**
Average Year (5/1/94 – 4/30/95)	**87,094**	**86,836**
Dry Season (10/16/94 – 4/30/95)	**35,344**	**35,001**

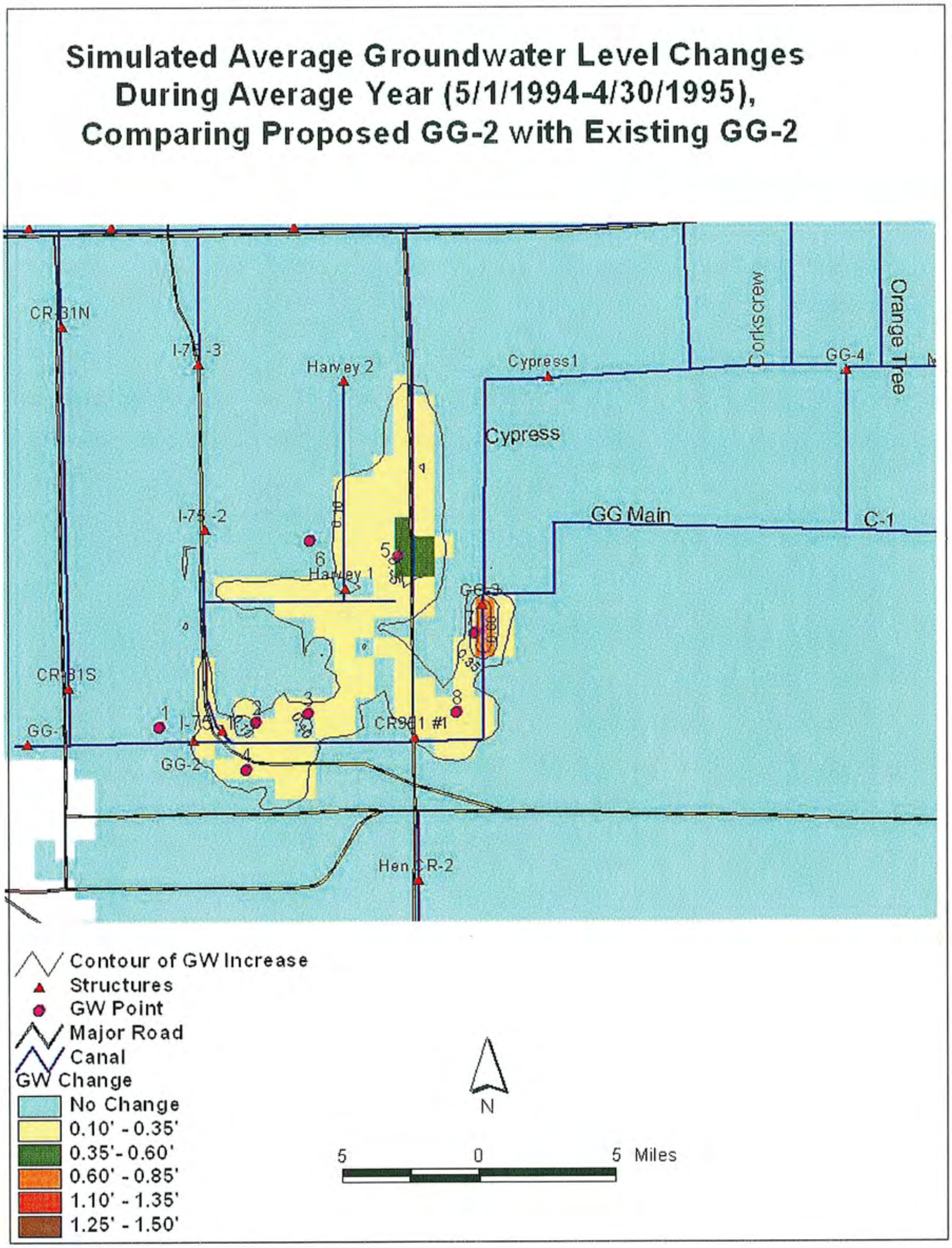

Simulated Average Groundwater Level Changes During Average Year (5/1/1994-4/30/1995), Comparing Proposed GG-2 with Existing GG-2

Figure 4-1

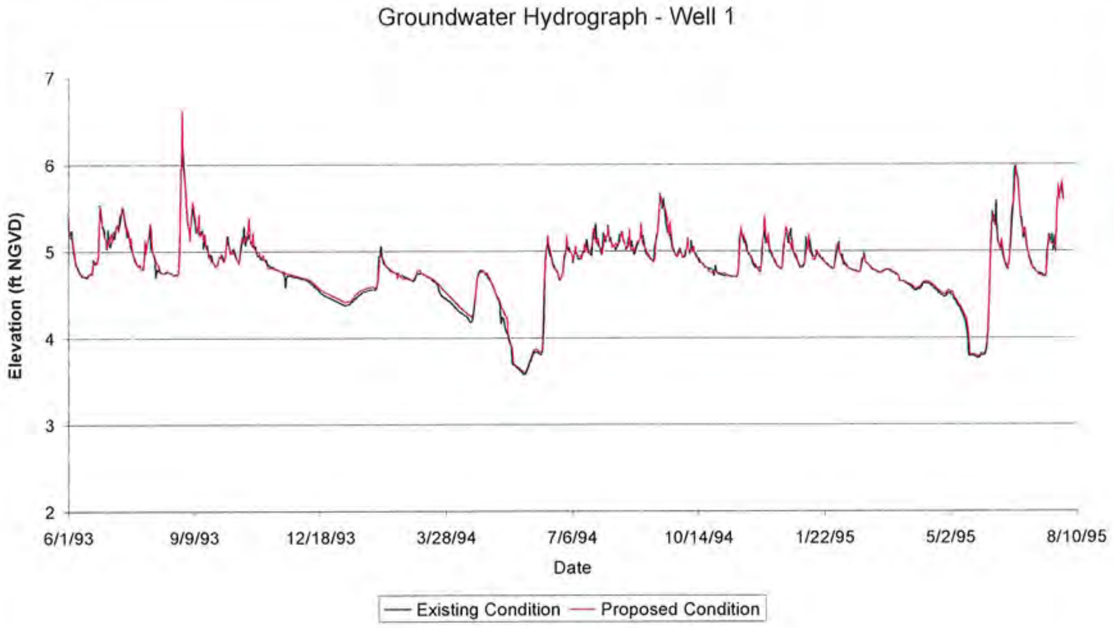


Figure 4-2-A

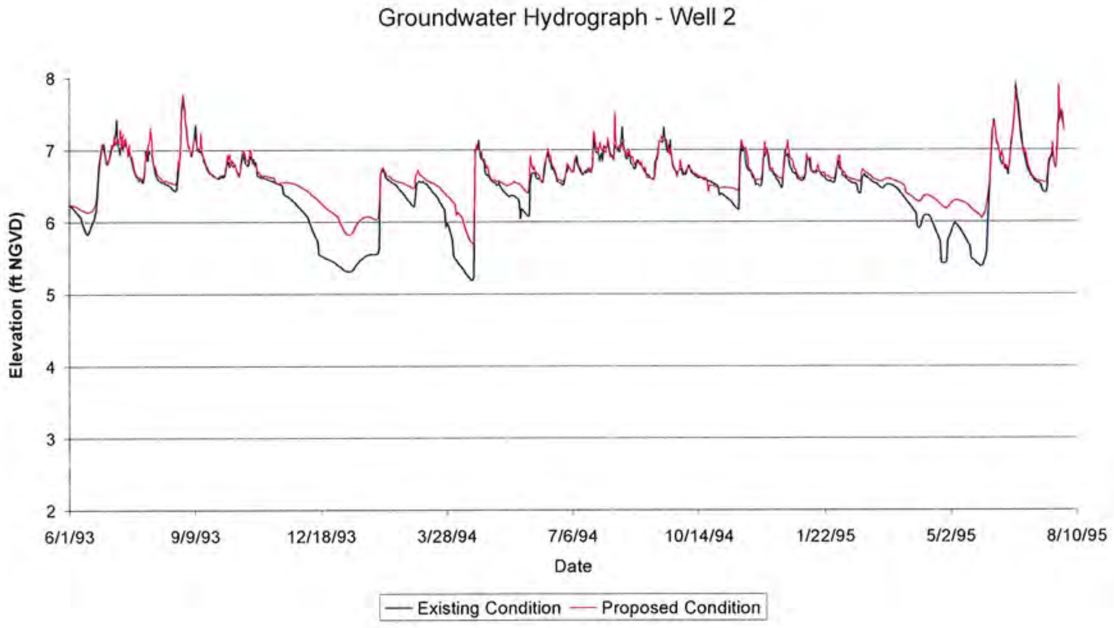


Figure 4-2-B

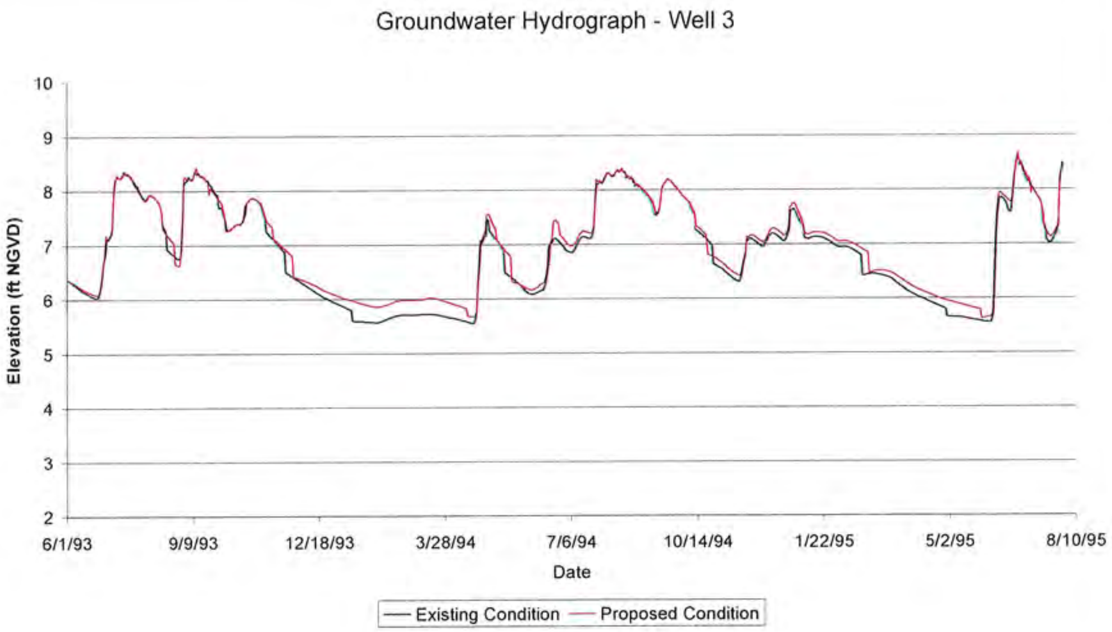


Figure 4-2-C

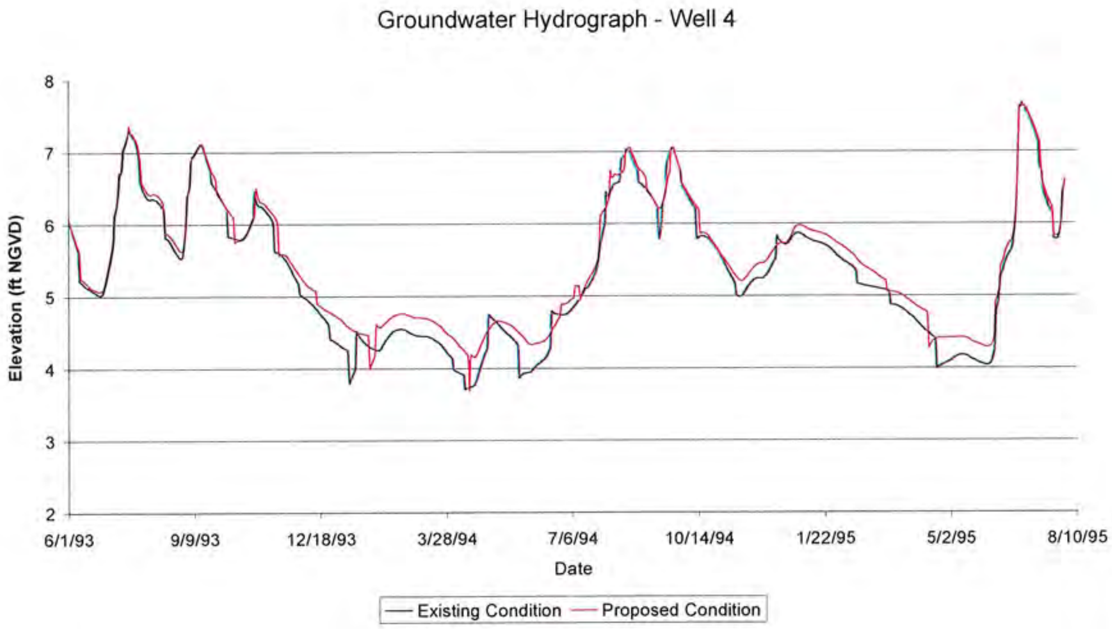


Figure 4-2-D

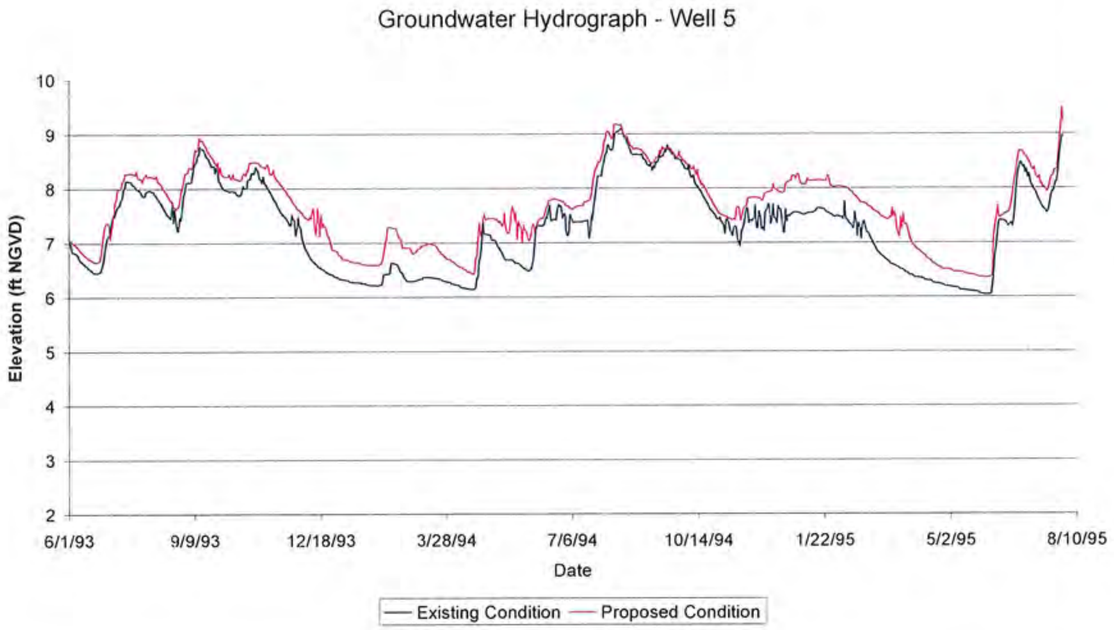


Figure 4-2-E

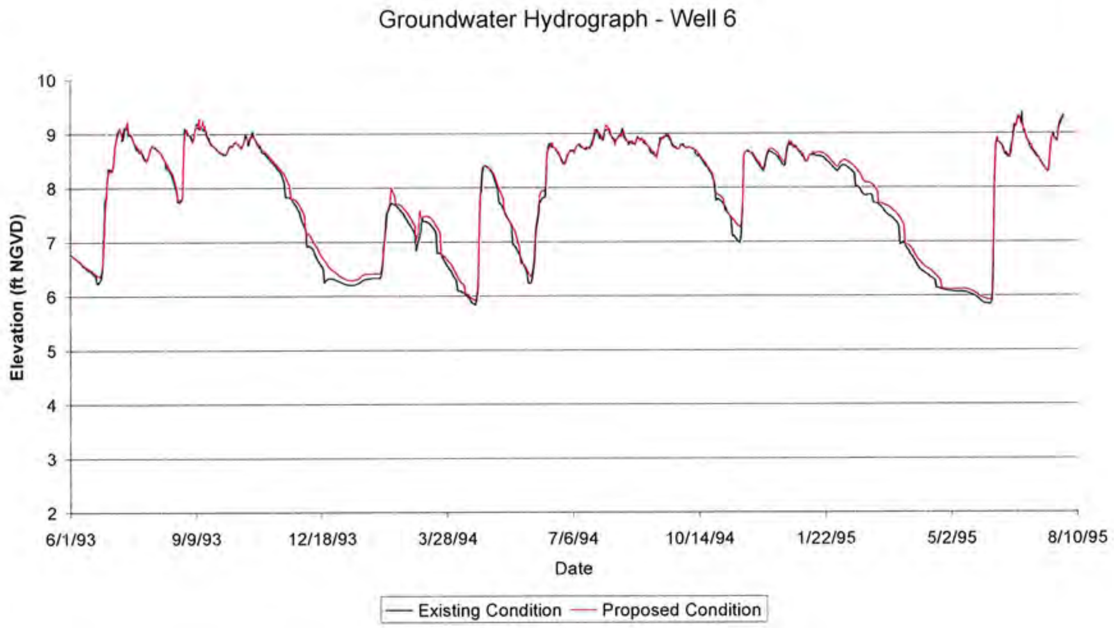


Figure 4-2-F

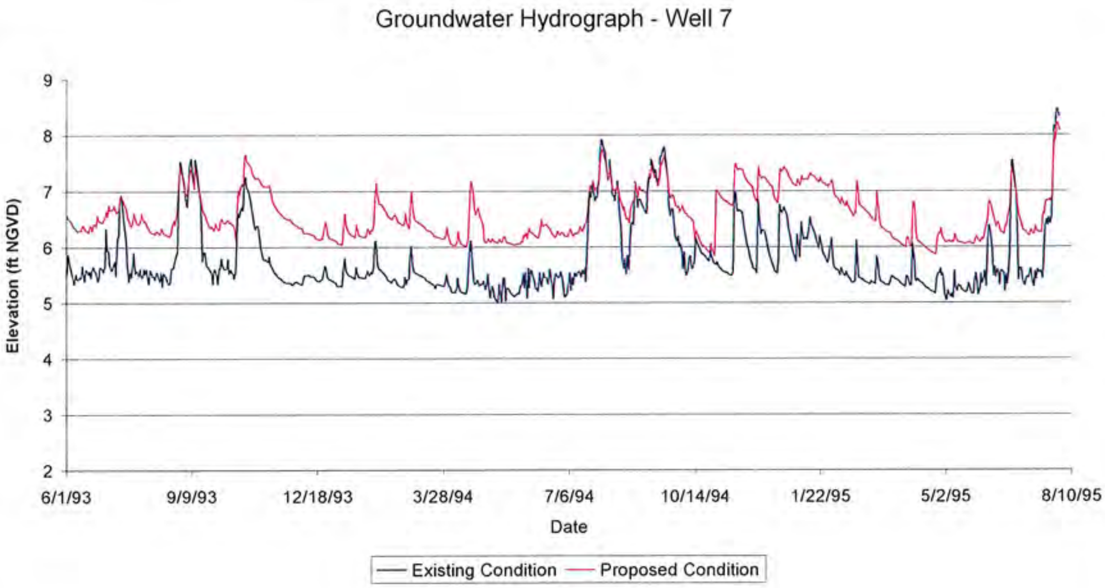


Figure 4-2-G

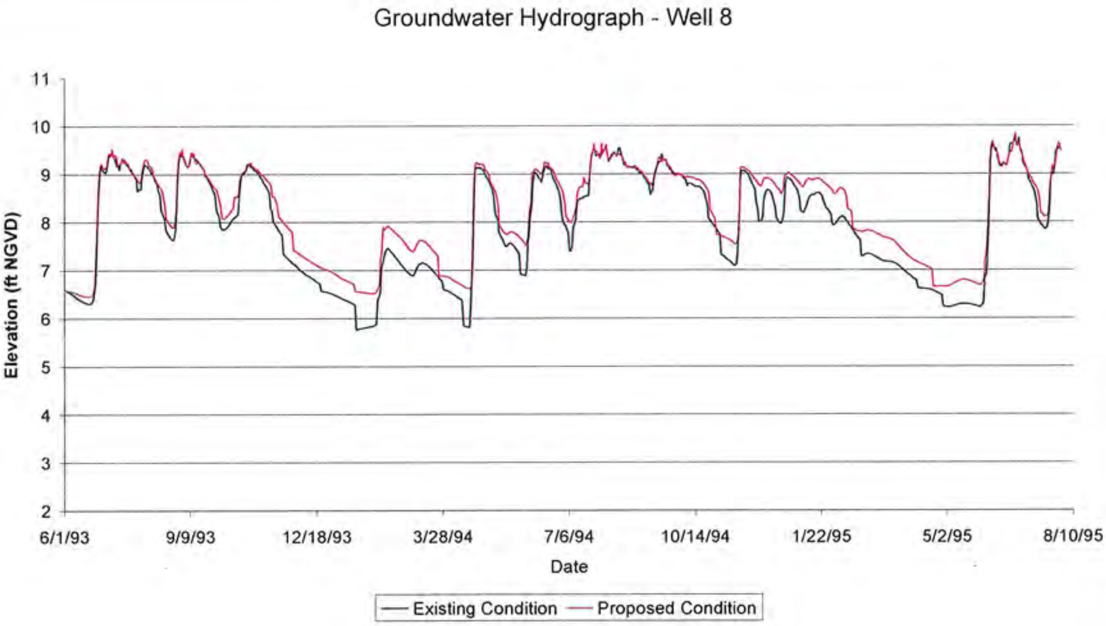


Figure 4-2-H

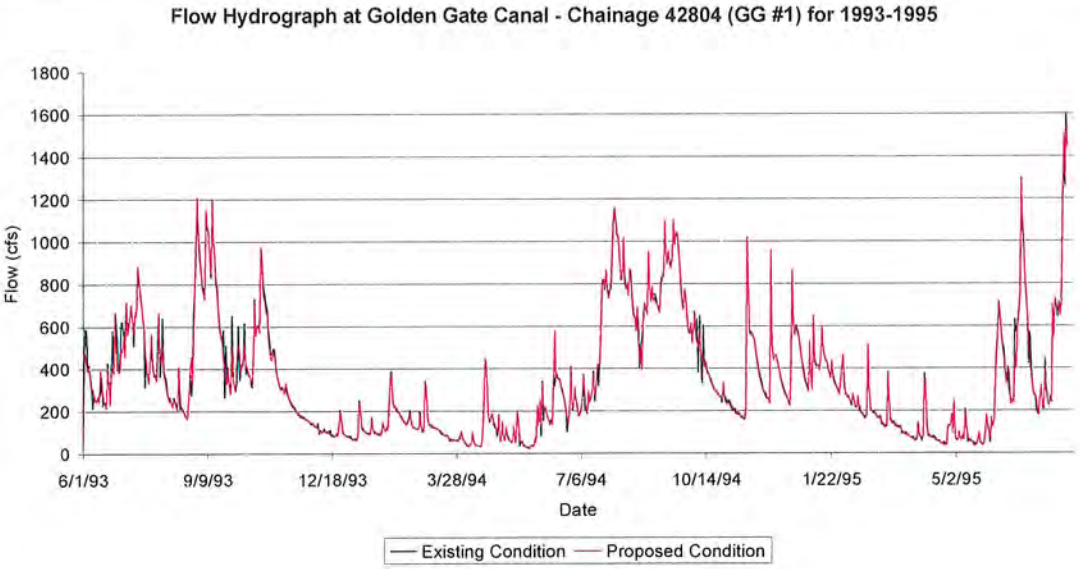


Figure 4-3-A

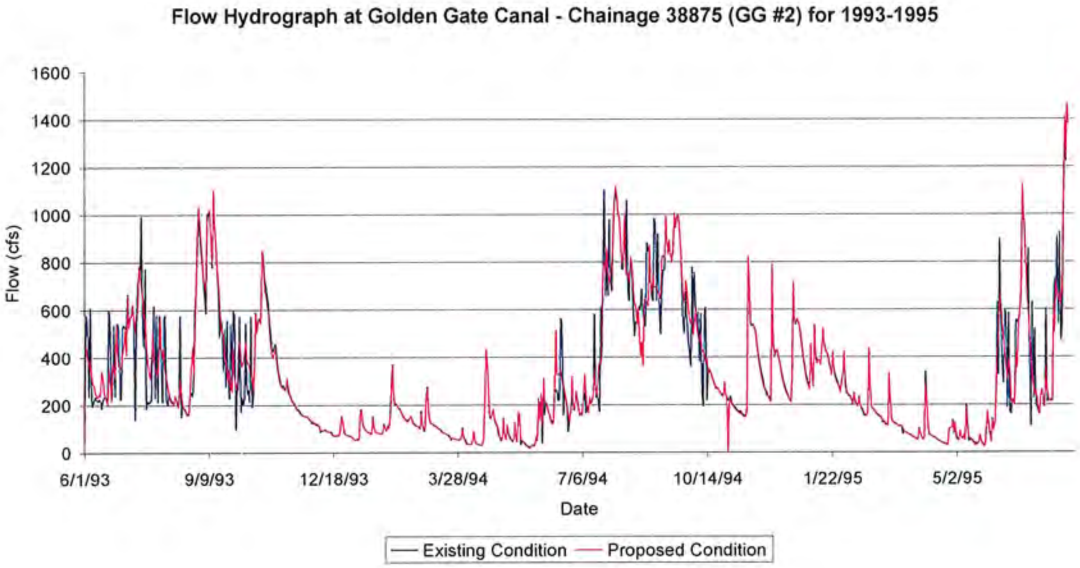


Figure 4-3-B

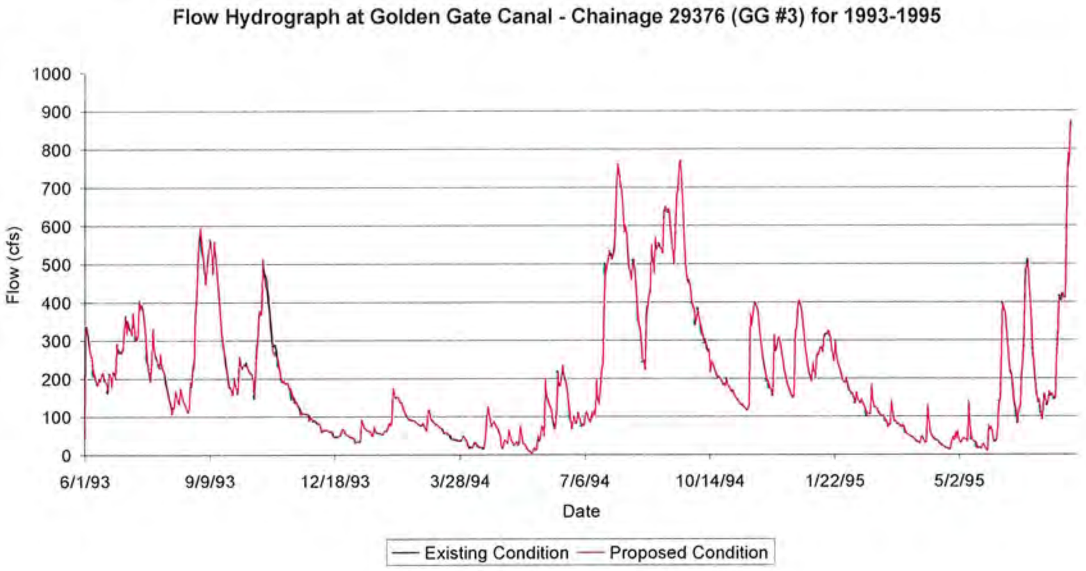


Figure 4-3-C

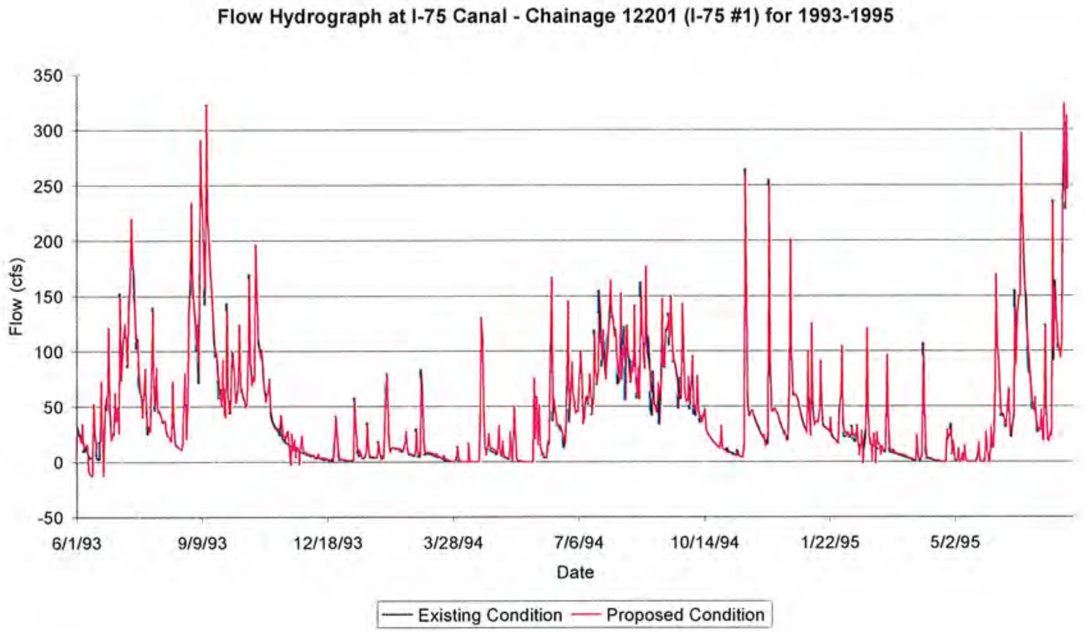
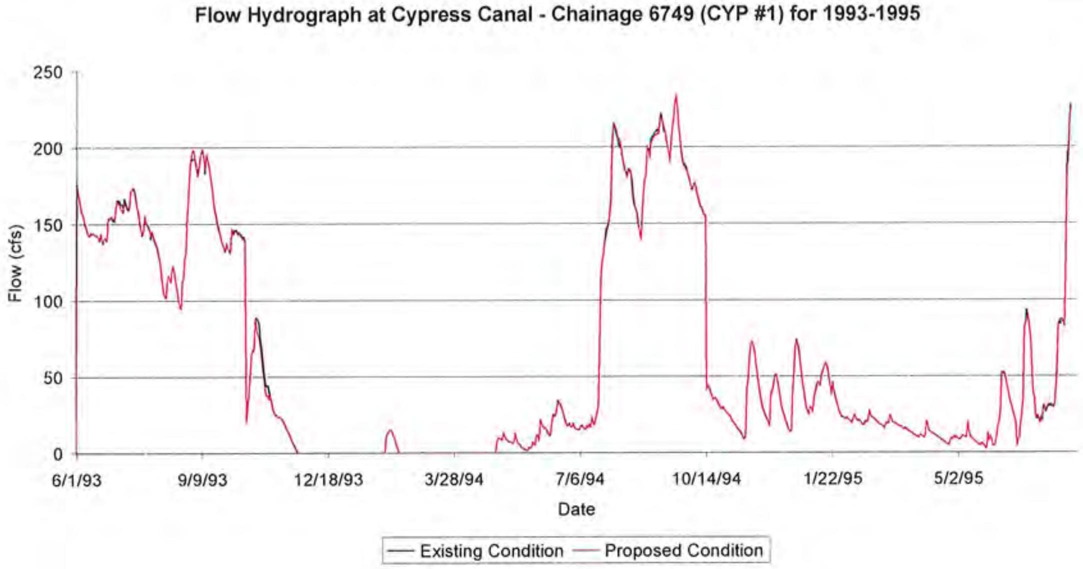


Figure 4-3-D



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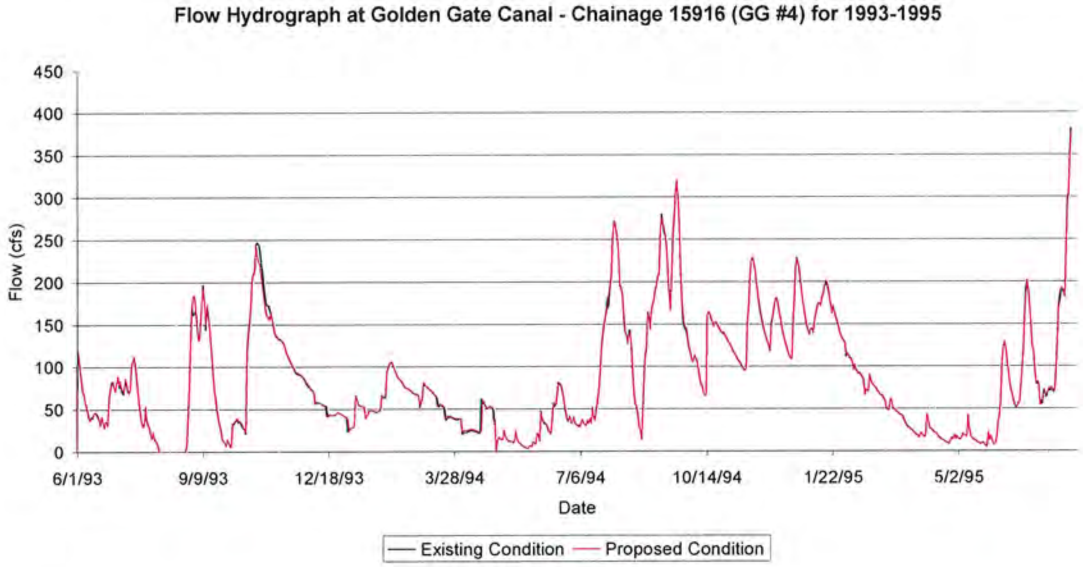
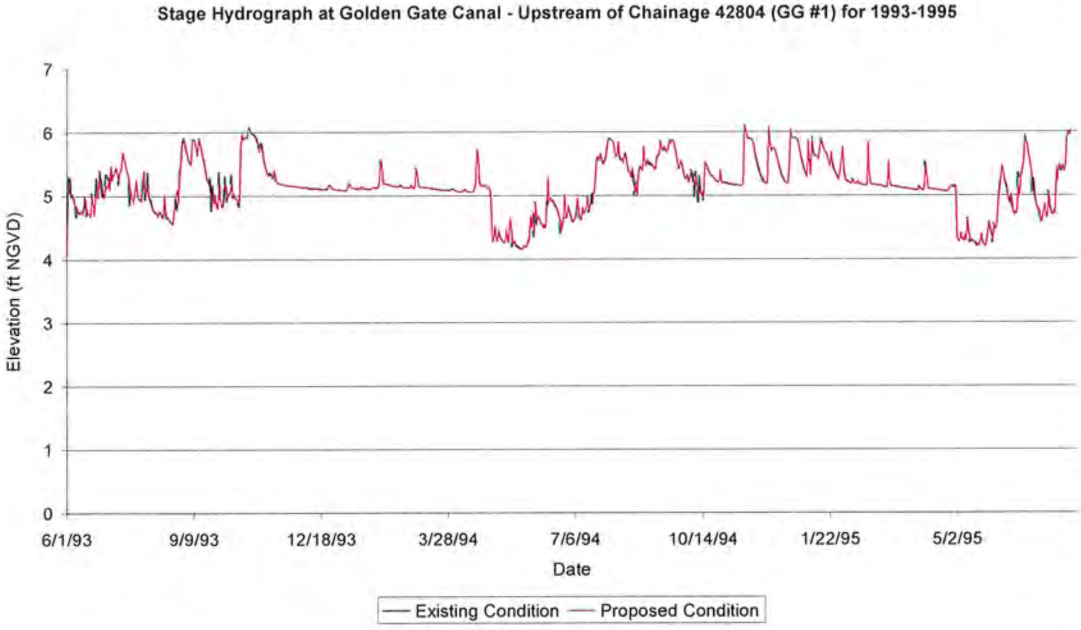


Figure 4-3-F



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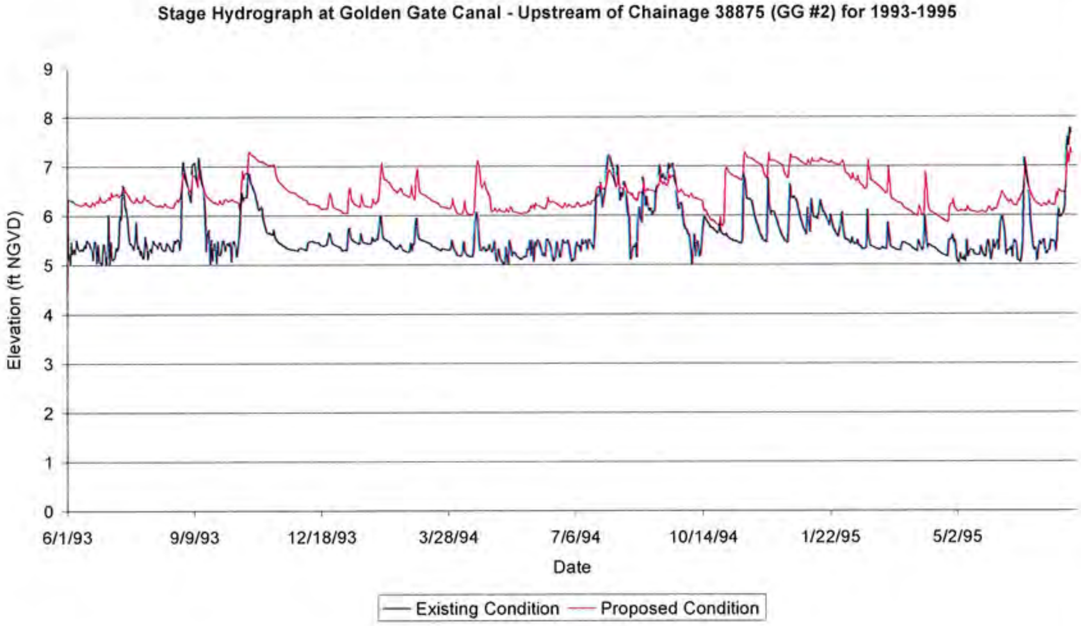

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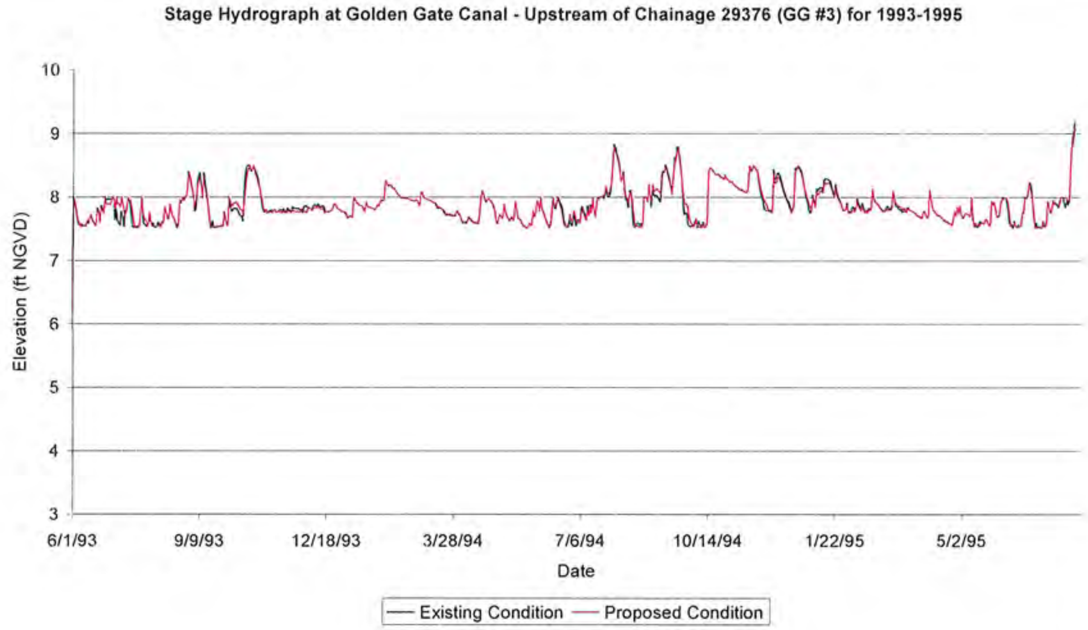

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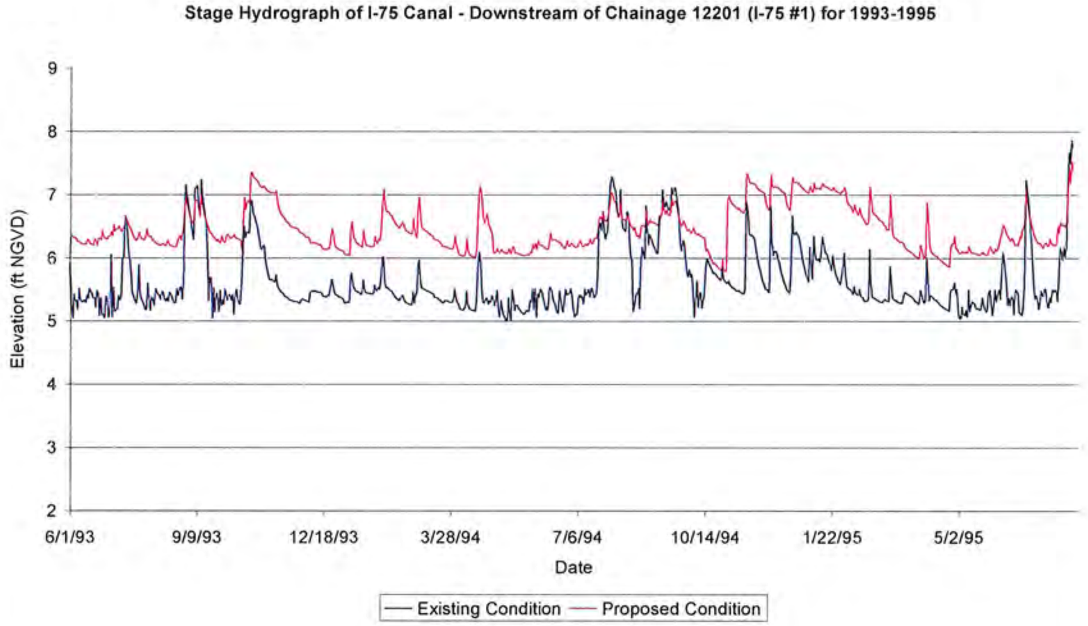


Figure 4-4-D

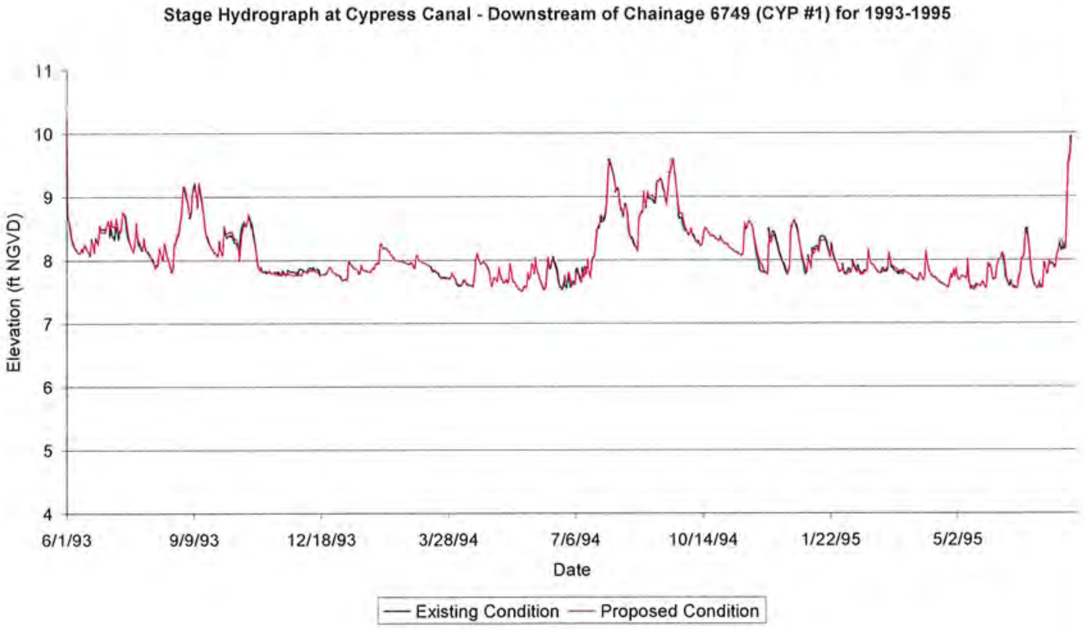
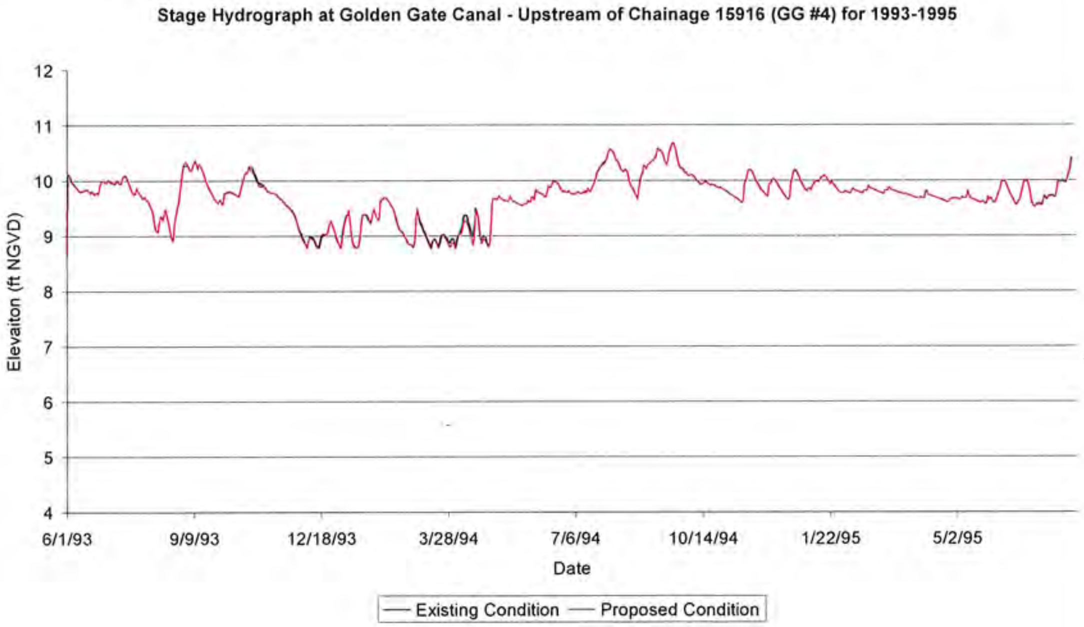


Figure 4-4-E



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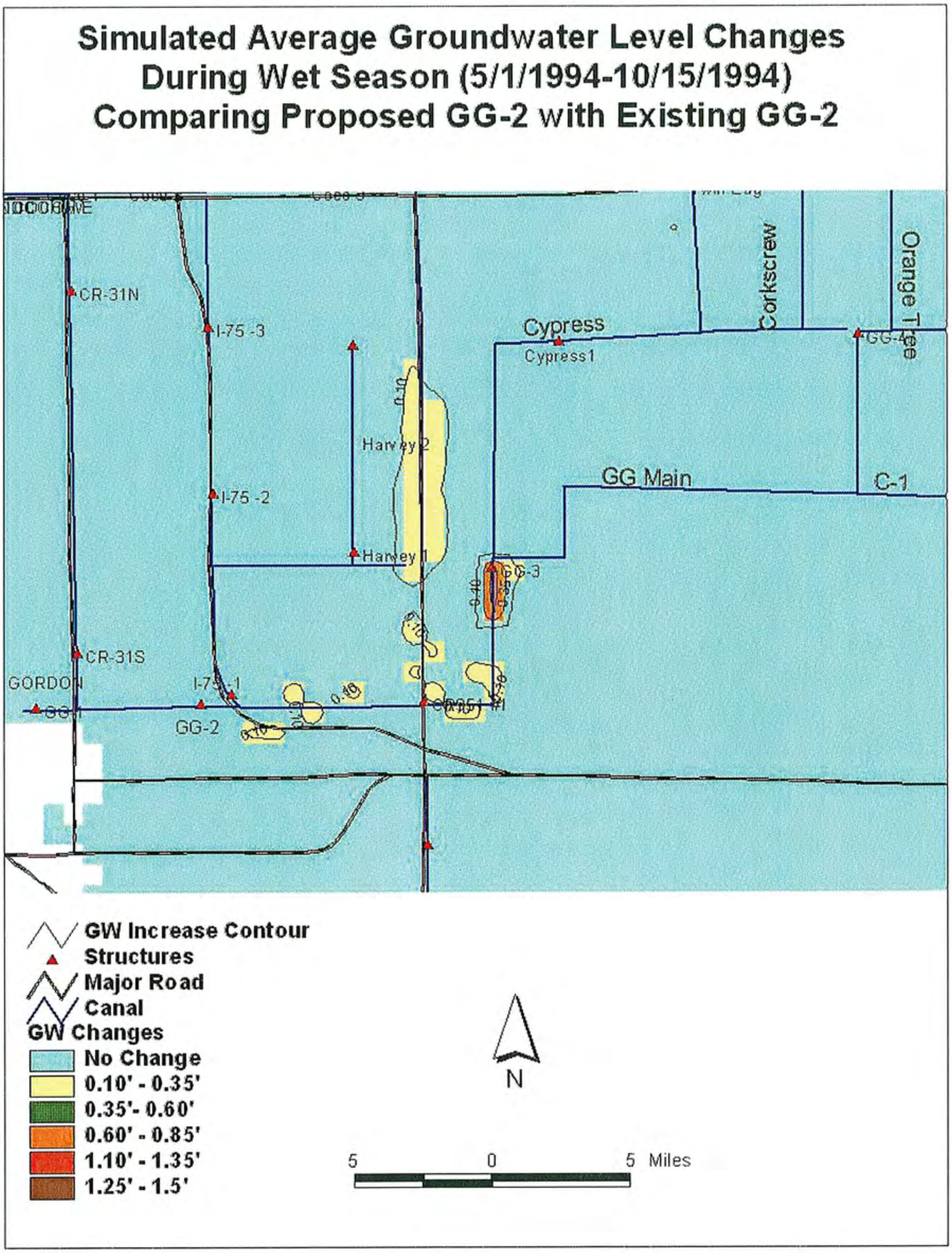
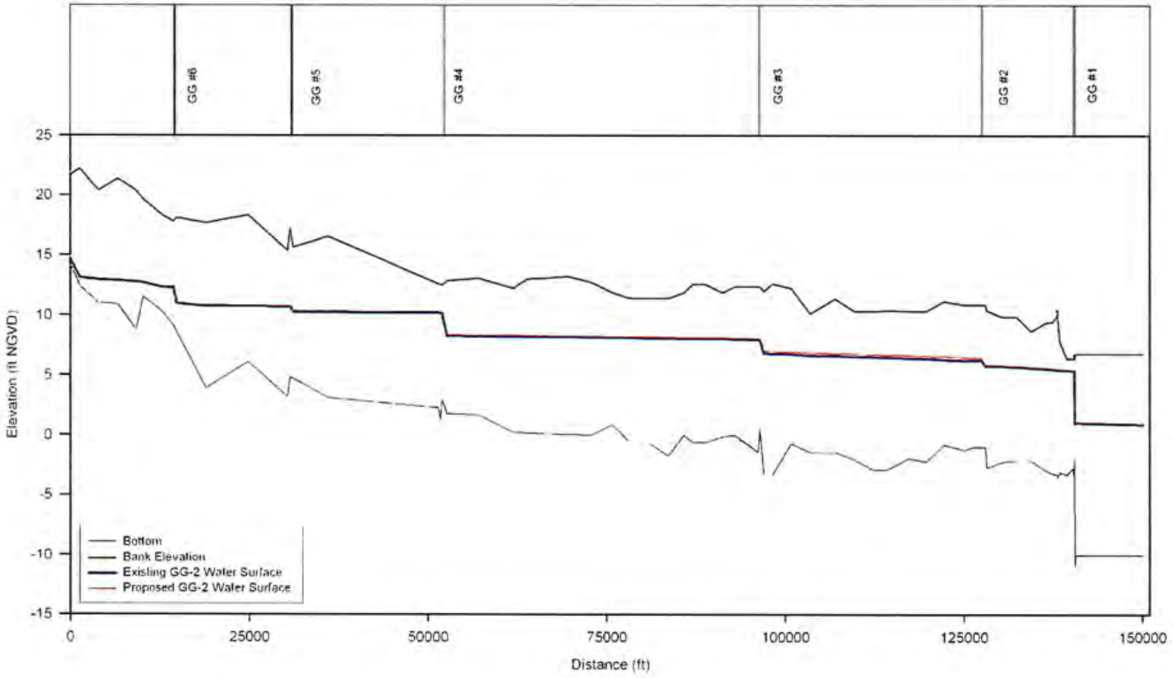
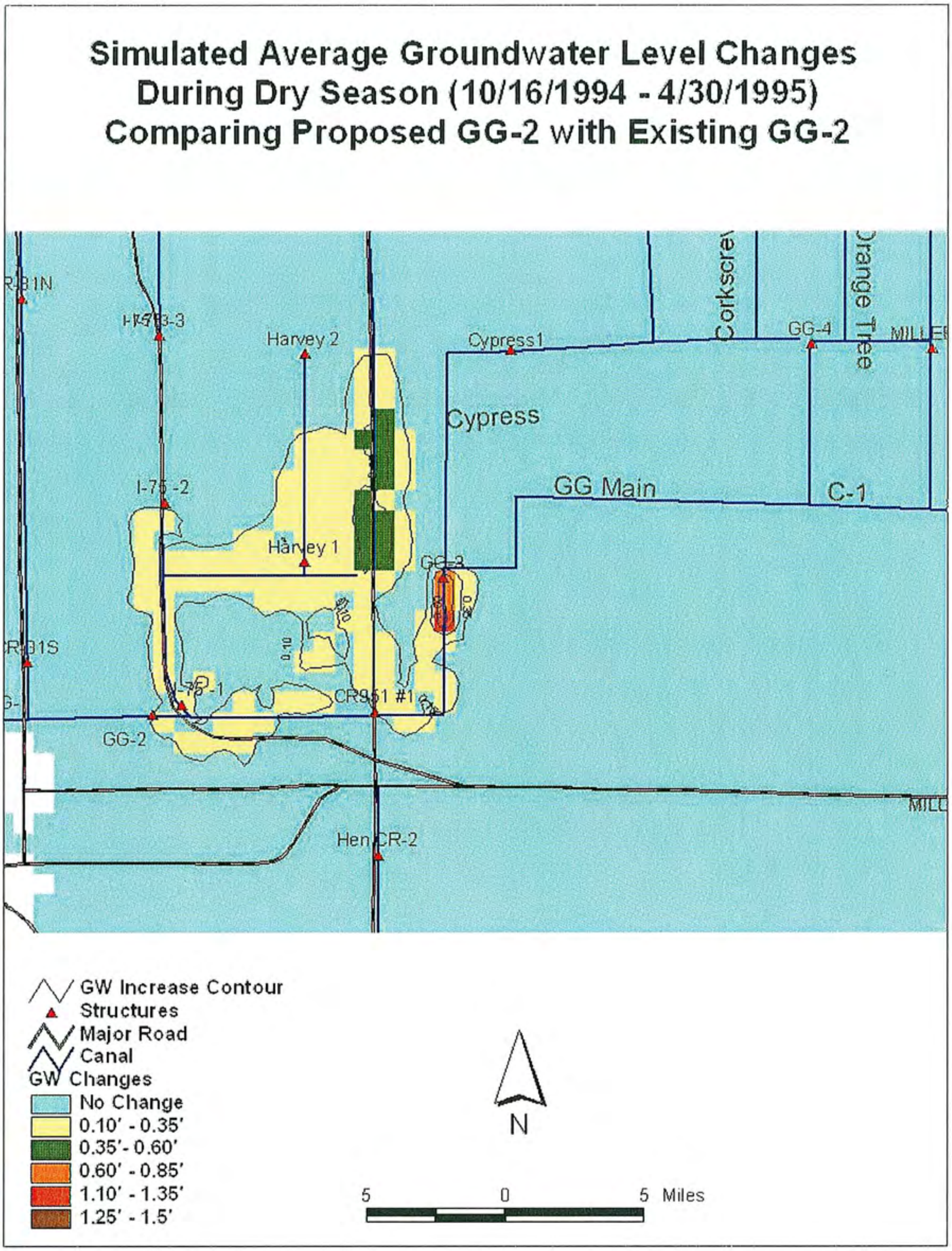

Simulated Average Groundwater Level Changes
During Wet Season (5/1/1994-10/15/1994)
Comparing Proposed GG-2 with Existing GG-2
CR-31N
I-75-3
Cypress
Cypress1
Corkscrew
Orange Tree
GG-4
Harvey 2
GG Main
C-1
I-75-2
Harvey 1
GG-3
0.10
0.40
CR-31S
GORDON
I-75-1
GG-2
GW Increase Contour
Structures
Major Road
Canal
GW Changes
No Change
0.10' - 0.35'
0.35'- 0.60'
0.60' - 0.85'
1.10' - 1.35'
1.25' - 1.5'
N
5
0
5 Miles

Figure 4-5

Figure 4-6. Water Surface Profile of Golden Gate Canal During the Middle of the Wet Season 9-1-1994

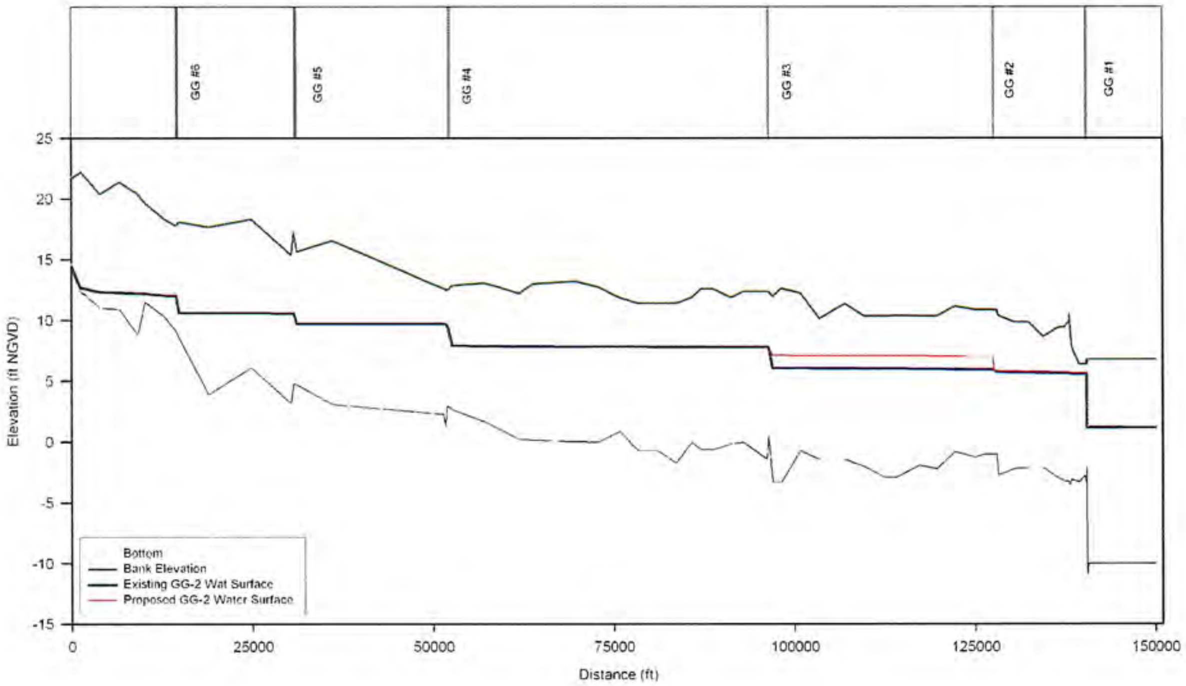




Simulated Average Groundwater Level Changes During Dry Season (10/16/1994 - 4/30/1995) Comparing Proposed GG-2 with Existing GG-2

Figure 4-7

Figure 4-8. Water Surface Profile of Golden Gate Canal During the Middle of the Dry Season 2-1-1995



5.0 SUMMARY AND CONCLUSIONS

The optimized configuration for replacement of GG-2 is a three-bay gated automated Obermeyer spillway with total spillway length of 80 feet. Each gate can be operated with variable top elevations for more versatile hydraulic performance. Operation can be either automatic or manual control. Different operational scenarios of GG-2 were evaluated in terms of their hydraulic performance. The operating control elevations for the wet and dry season water surface elevation are set as follows:

Wet season: gates open at 6.00 ft NGVD(4.73 ft NAVD) and close at 5.5 ft NGVD (4.23 ft NAVD).

Dry season: gates open at 7.00ft NGVD (5.73ft NAVD) and close at 5.75 ft NGVD (4.48 ft NAVD).

The construction for replacement of the structure has been budgeted for FY 2006. The major advantages of GG-2 replacement are summarized as:

1. Fixed crest weir replaced with movable automated spillway will greatly enhance the water management flexibility.
2. The proposed structure will not have adverse impacts on the existing levels of flood protection in the Golden Gate Canal watershed.
3. The proposed structure and its operation will reduce the total volume of fresh water discharge to Naples Bay.
4. The proposed structure will increase groundwater storage in the upstream structure area during dry season.

6.0 REFERENCES

1. Dames & Moore, 1998. BCB Watershed Plan, Task D – Model Development. Prepared for SFWMD, BCB Board under Contract no. C-7703, Naples, Florida.

2. SFWMD, 1990. Permit Information Manual Volume IV, West Palm Beach, Florida.

3. Danish Hydraulic Institute, Inc. (DHI), 2002. Big Cypress Basin Integrated Hydrologic-Hydraulic Model

4. BCB, 2000. BCB Watershed Management Plan, Task IIIA Problem Analysis – Flood Control Element. SFWMD, Naples, Florida.

5. BCB, 2002. Five-Year Plan, 2002-2006. BCB of the SFWMD.

6. BCB, 2005. Water Control Structures. BCB of the SFWMD.

7. BCB, 2006. Operation Schedule of Water Control Structures. BCB of the SFWMD.

8. SFWMD, 1999. SFWMD Permit Information Manual, Volume V, Criteria Manual for Use of Works of the District. West Palm Beach, Florida.

9. US Army Corps of Engineers and SFWMD, 2004. Final Integrated Project Implementation Report and Environmental Impact Statement

10. Chandra S. Pathak, 2001. Technical Publication EMA # 390, Frequency Analysis of Daily Rainfall Maxima for Central and South Florida.

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APPENDIX A

CALIBRATION RESULTS

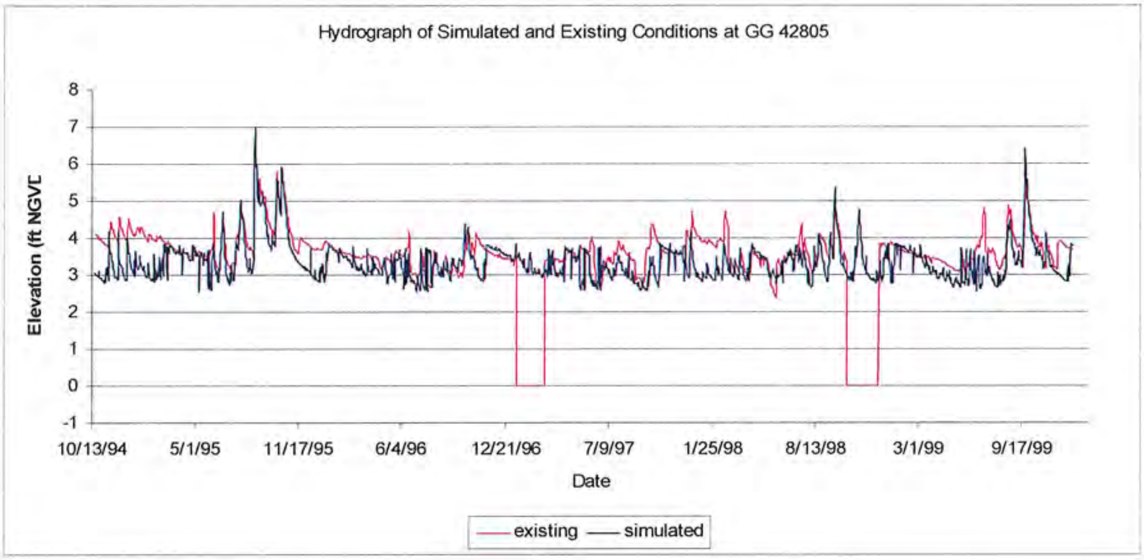


Figure A-1: Simulated and observed headwater at GG #1 from 1995-2000

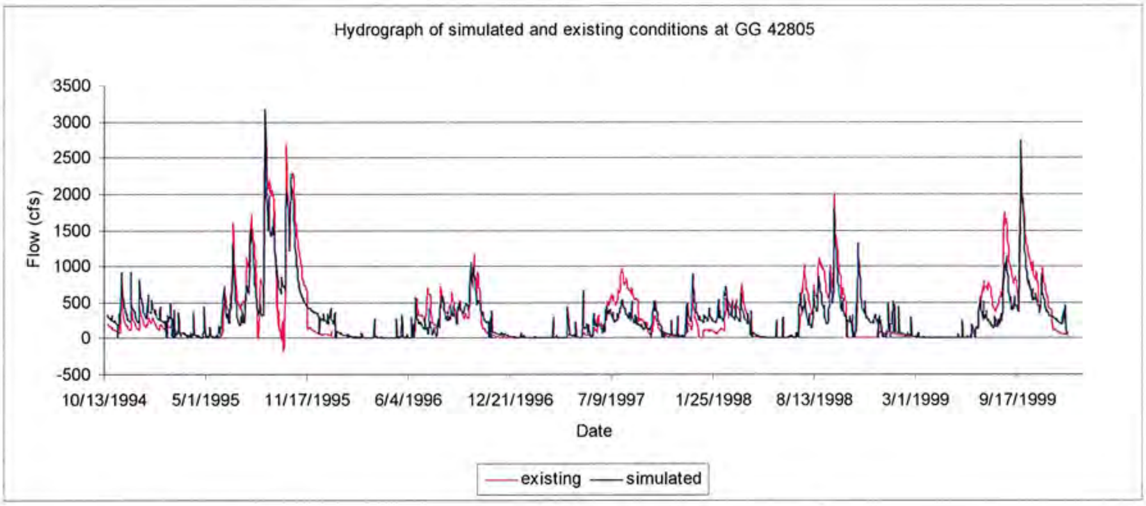
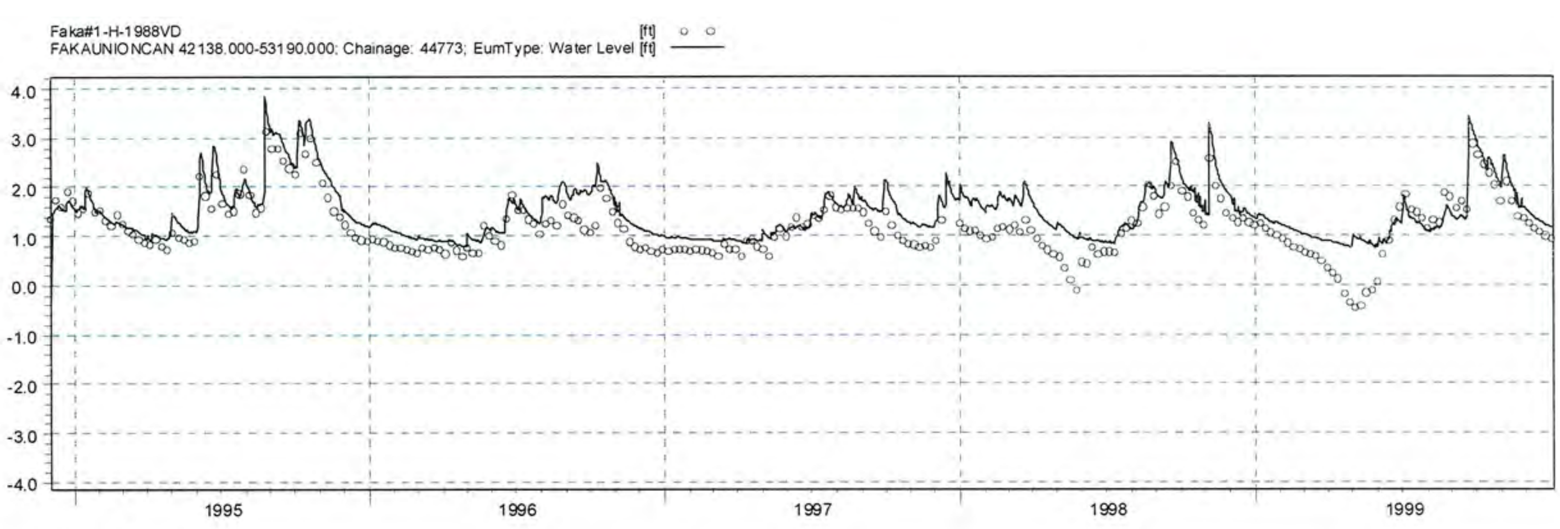
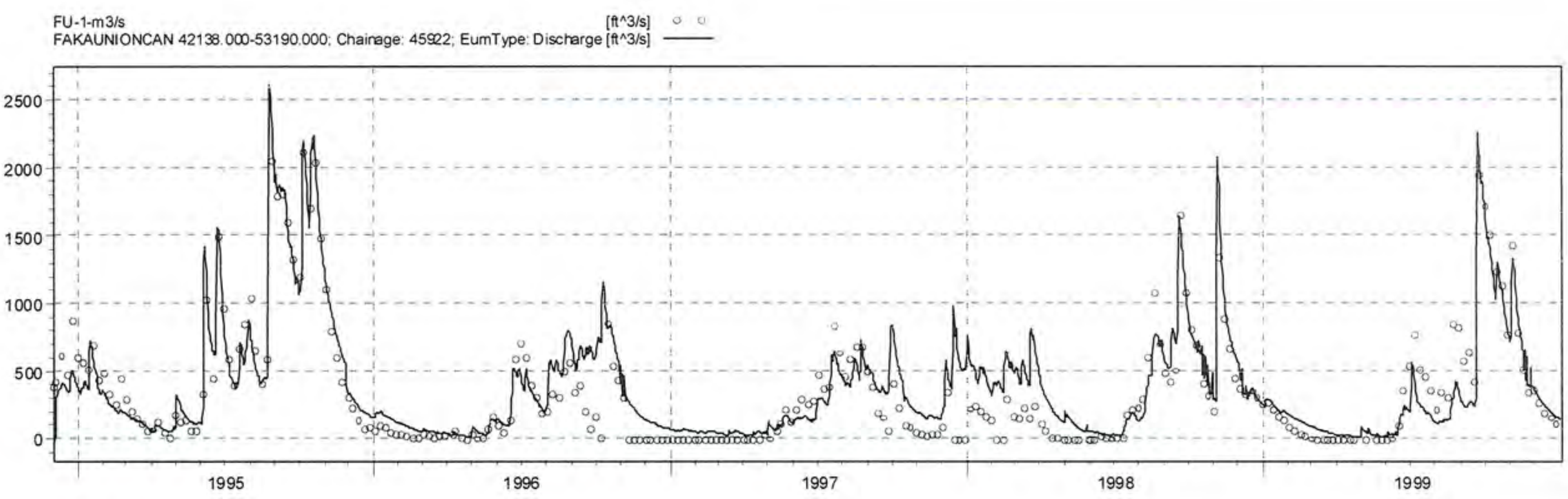


Figure A-2: Simulated and observed headwater at GG #1 from 1995-2000



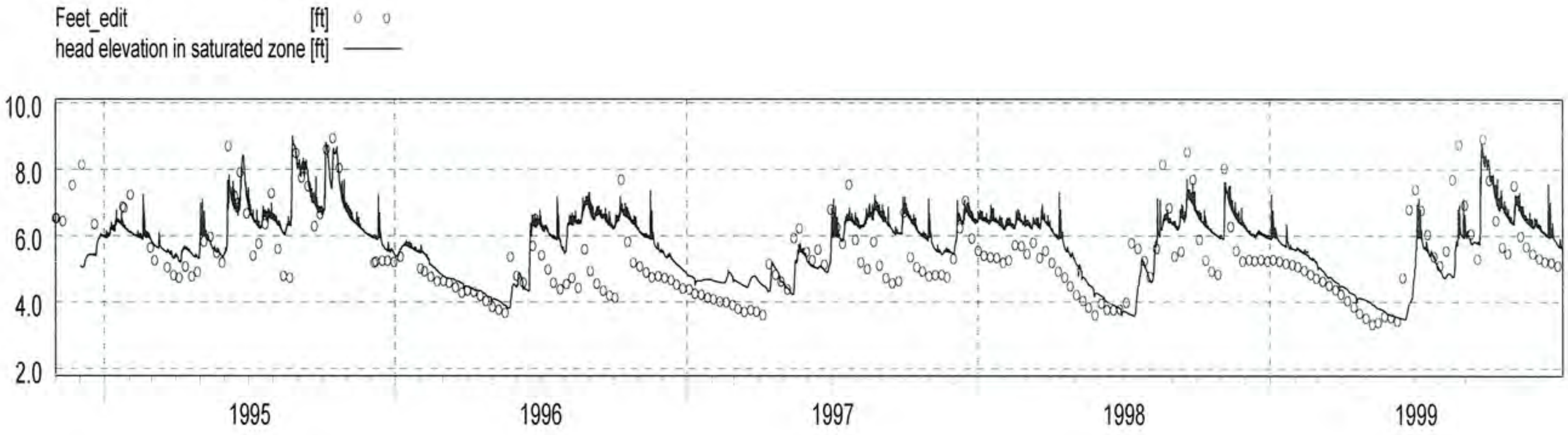
ME=-0.260438
MAE=0.312554
RMSE=0.406389
STDres=0.311968
R(Correlation)=0.860849
R2(Nash_Sutcliffe)=0.553116

Figure A-3: Headwater Stage at FU-1 Weir during Calibration Period



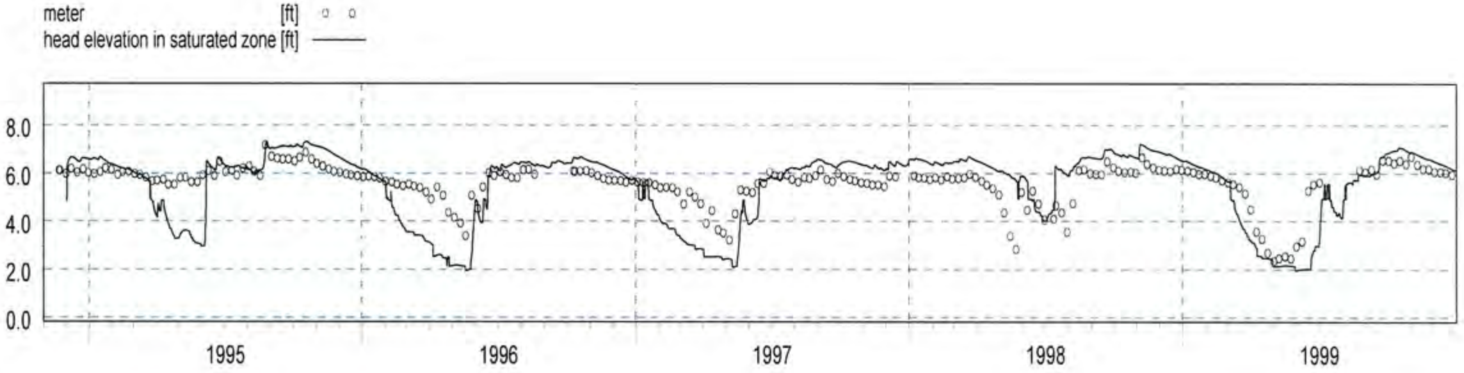
ME=-35.0216
MAE=136.07
RMSE=191.502
STDres=188.273
R(Correlation)=0.908787
R2(Nash_Sutcliffe)=0.817164

Figure A-4: Discharge at FU-1 Weir during Calibration Period



ME=-0.428054
MAE=0.76222
RMSE=0.961212
STDres=0.860638
R(Correlation)=0.6986
R2(Nash_Sutcliffe)=0.305965

Figure A-5: Simulated and Observed Groundwater Level at Well C-690



ME=0.0847068
MAE=0.790851
RMSE=1.05606
STDres=1.05266
R(Correlation)=0.741699
R2(Nash_Sutcliffe)=-0.440119

Figure A-6: Simulated and Observed Groundwater Level at Well C-496

(Fakahatchee Strand south of I-75)

APPENDIX B

SKETCHES FOR GOLDEN GATE STRUCTURE GG#2

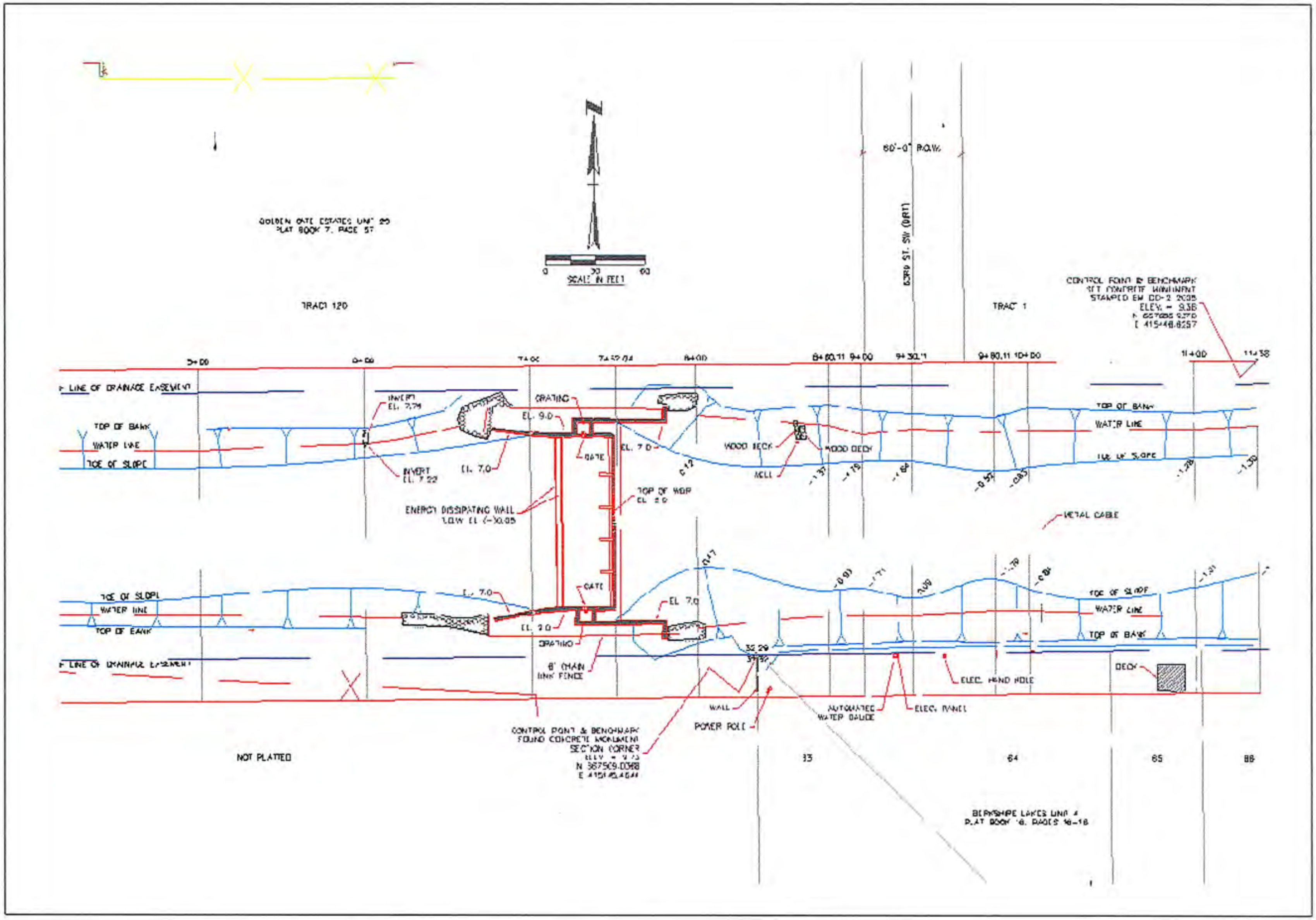


Figure B-1: Existing GG-2 Structure Site Plan (Elevation in ft NGVD)

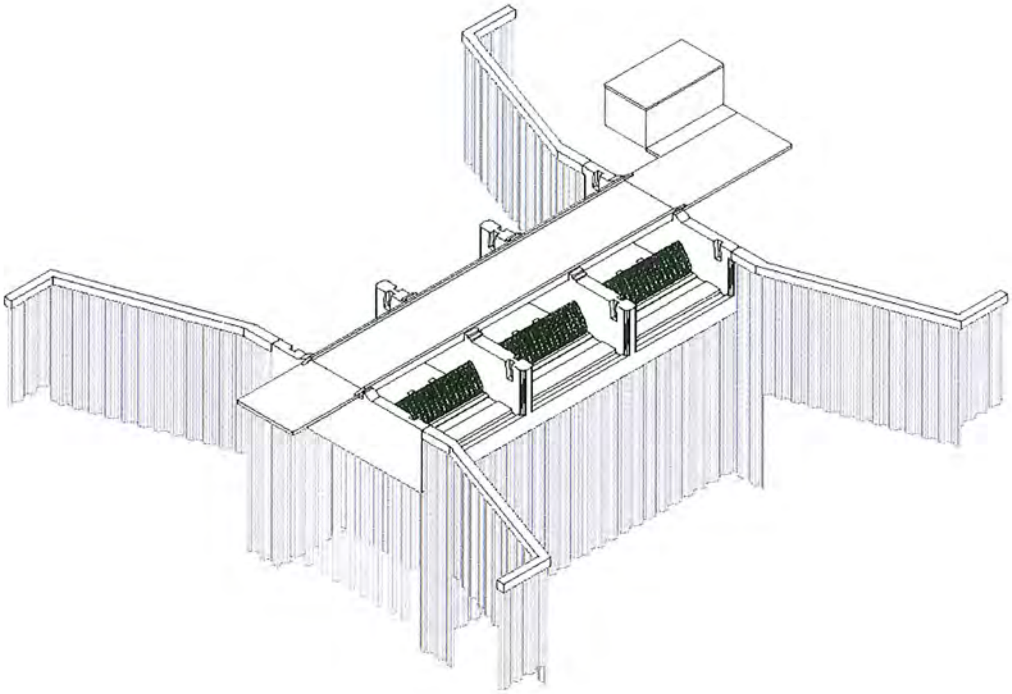
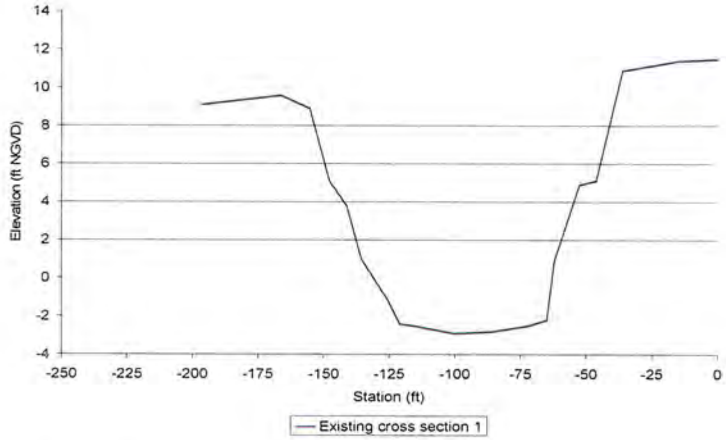


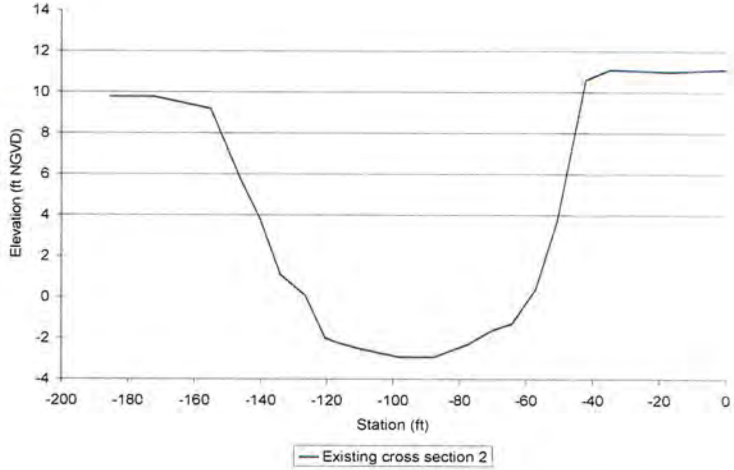
Figure B-2 3-D: Layout of Proposed Golden Gate Structure GG-2

APPENDIX C

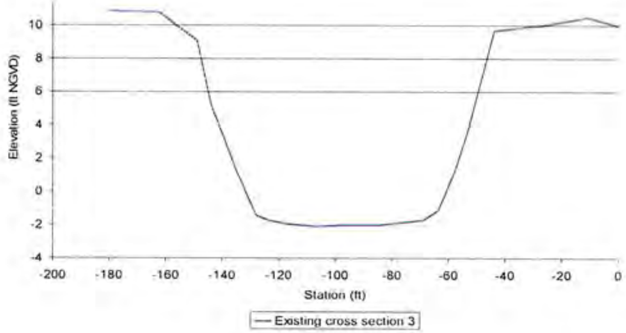
CANAL CROSS SECTIONS



Golden Gate #2 Cross Section 1 (Chainage 139684)



Golden Gate #2 Cross Section 2 (Chainage 139784)



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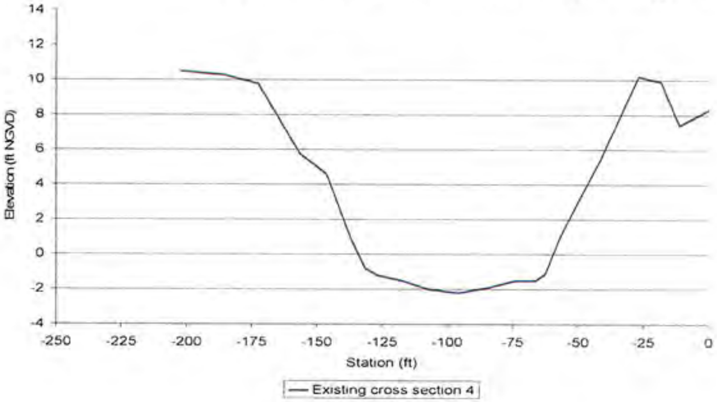

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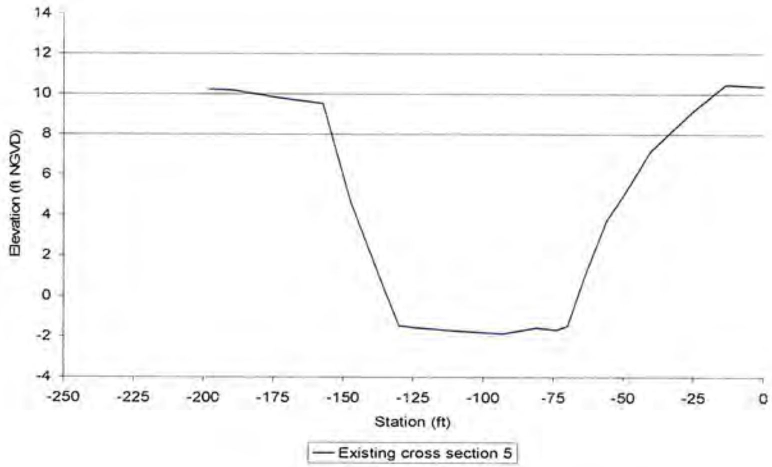

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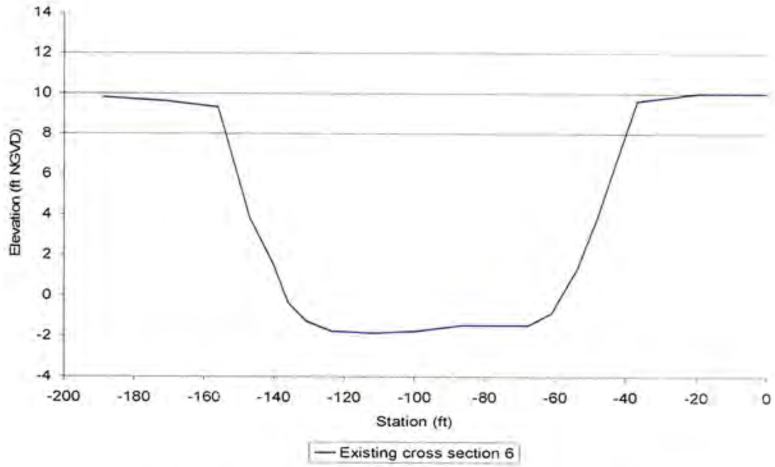
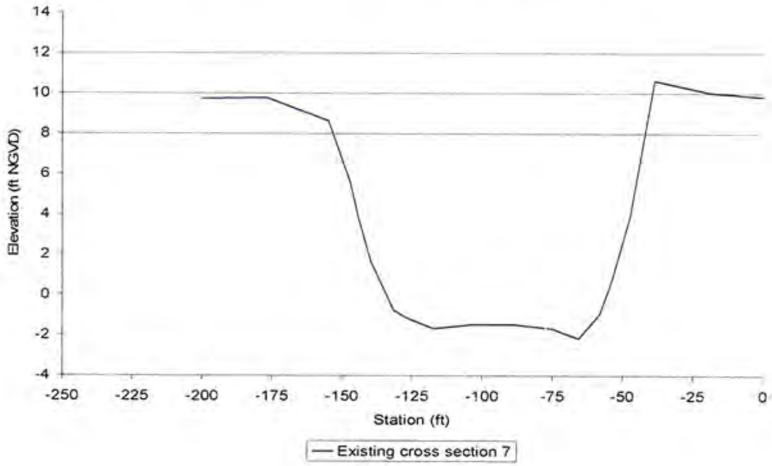
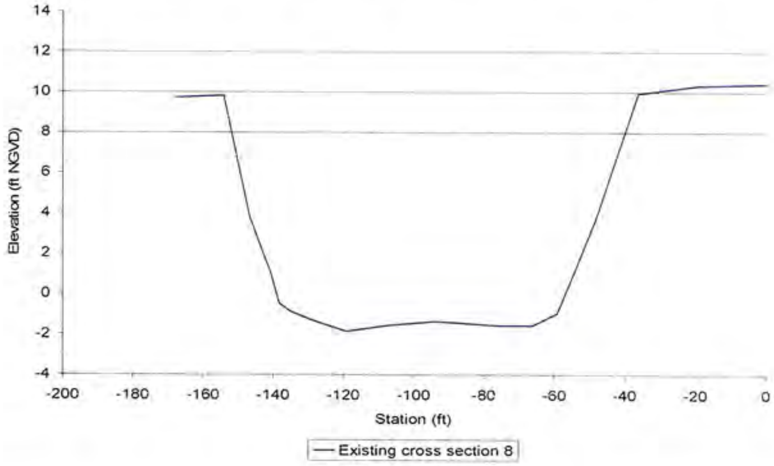


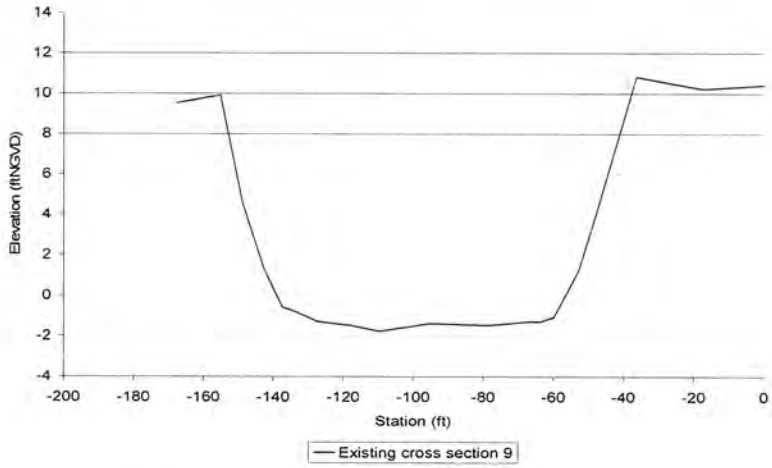
Golden Gate #2 Cross Section 6 (Chainage 139964)



Golden Gate #2 Cross Section 7 (Chainage 139984)



Golden Gate #2 Cross Section 8 (Chainage 140084)



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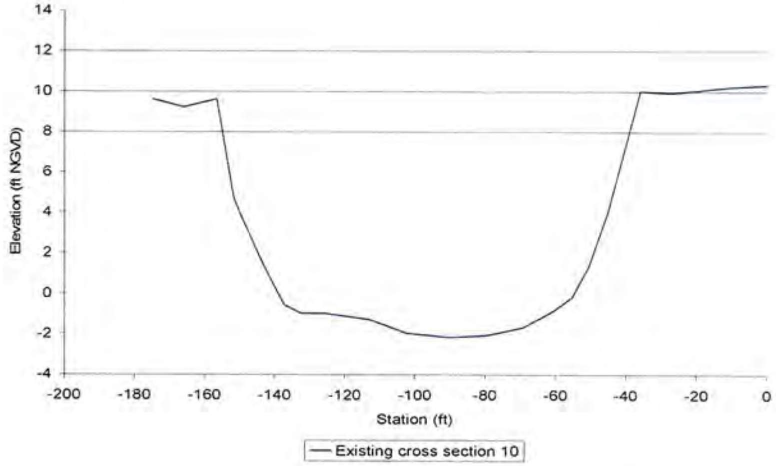
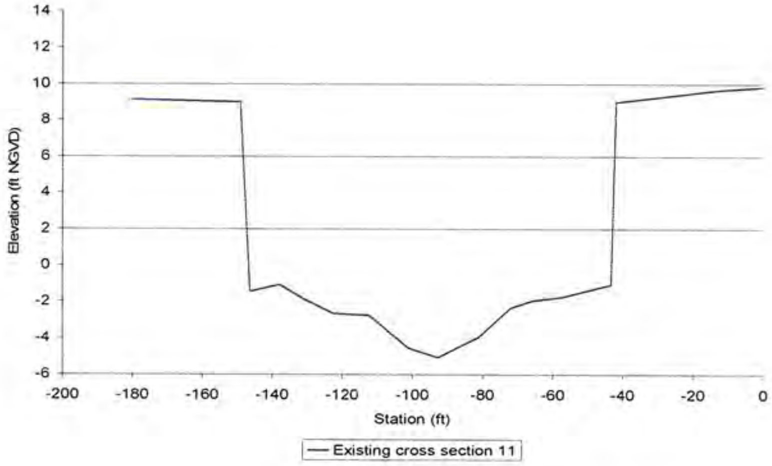
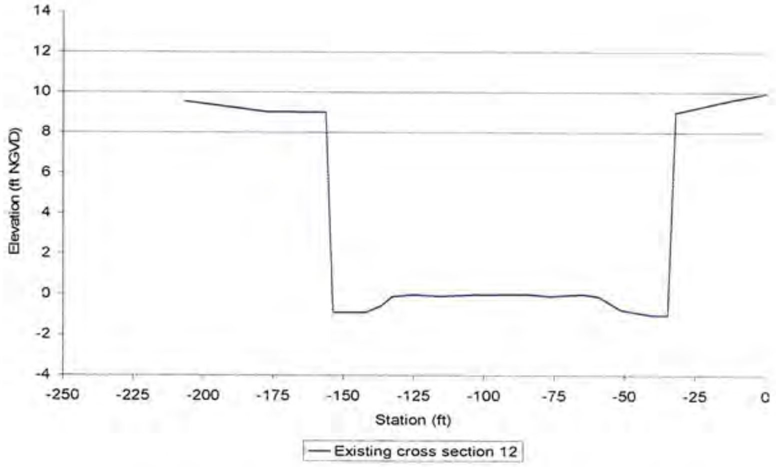


Golden Gate #2 Cross Section 10 (Chainage 140284)



Golden Gate #2 Cross Section 11 (Chainage 140384)



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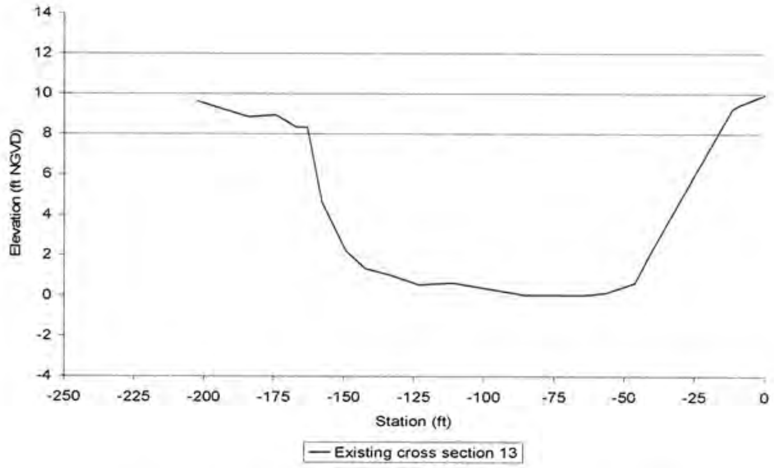

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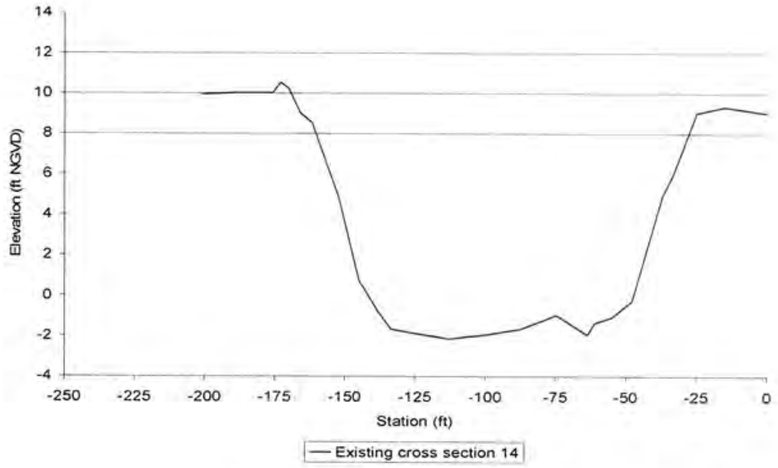

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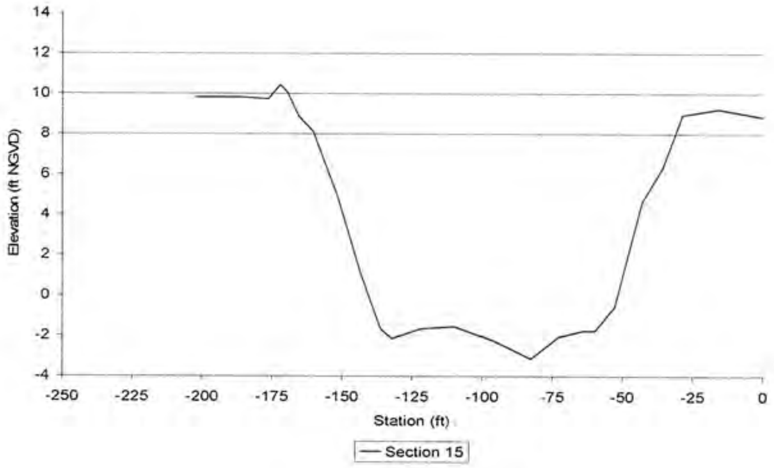

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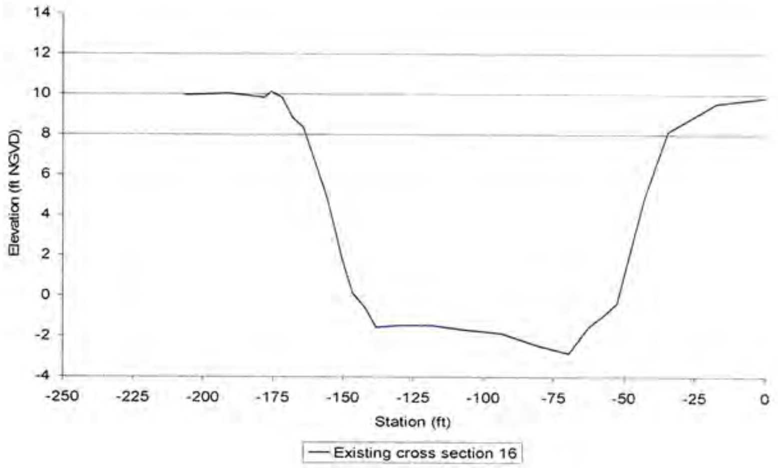

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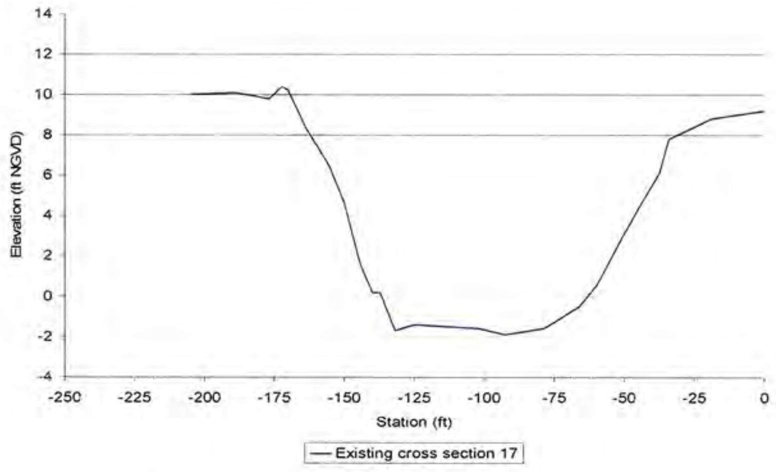

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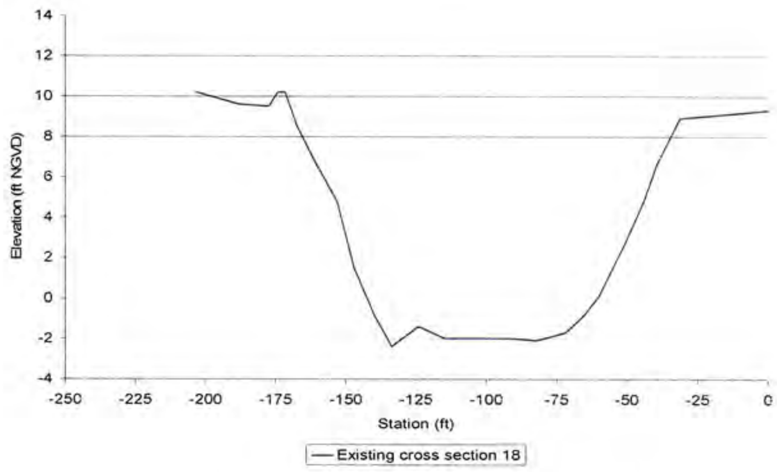

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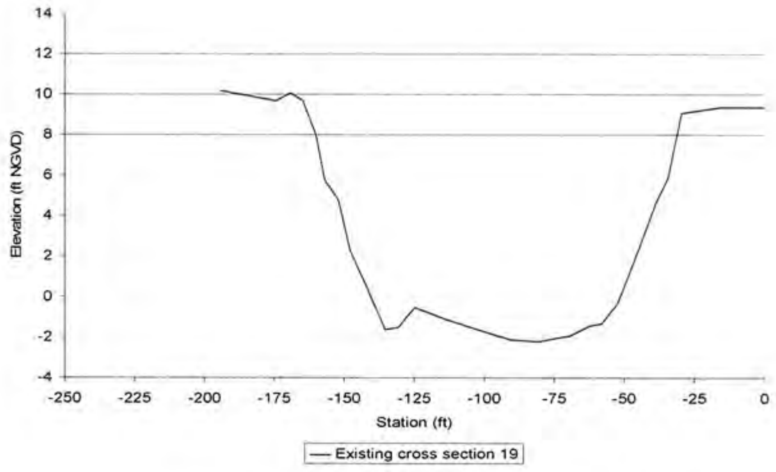

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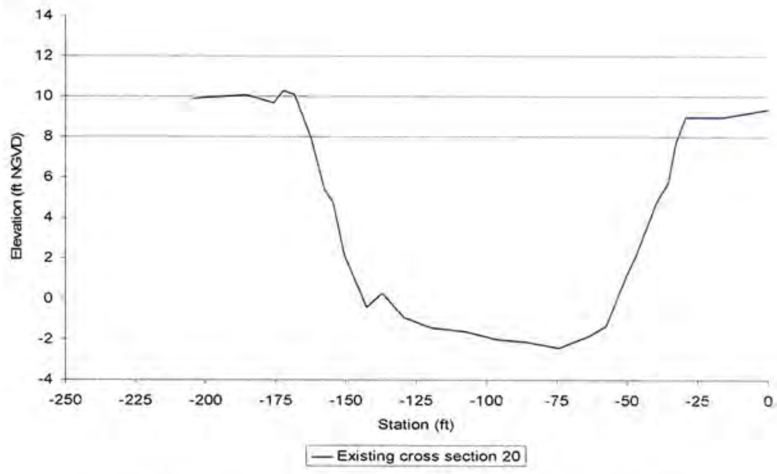

Golden Gate #2 Cross Section 20 (Chainage 140822)

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