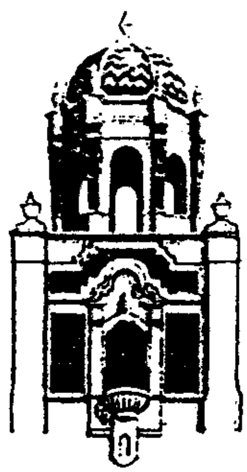
# SARASOTA COUNTY

# STORMWATER MASTER PLAN

## FINAL REPORT



MARCH 1987

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*environmental engineers, scientists, planners, & management consultants* **CDM**

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# 1.0 INTRODUCTION

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## 1.1 PURPOSE

The Sarasota County Commission and the Sarasota County Transportation Department, recognizing the inadequacies of the existing stormwater management system, authorized Camp Dresser & McKee Inc. (CDM) in October 1984 to prepare a Stormwater Master Plan. The purpose of this Master Plan is to assess the need for improvement of major drainage systems in the developed areas of the County. The objectives of this study are:

1. To assess the adequacy of primary stormwater conveyance systems in developed or developing basins;

2. To estimate the cost for public stormwater improvements as watersheds are developed to ultimate land use;

3. To prioritize the stormwater management needs of each individual basin within a framework of the needs of the entire County; and

4. To develop a plan or identify options available to the County to finance the cost of construction, operation, and maintenance of stormwater management facilities.

The County is aware that a stormwater master plan involving an in-depth analysis of the 32 previously identified basins within the study area would not be cost-effective or timely. Therefore, CDM was authorized to study in-depth, both hydraulically and hydrologically, portions of two major basins. The Stormwater Master Plan presented in this document is the result of a thorough analysis of selected portions of Alligator and Phillippi Creeks, and an extrapolation of the results to the 14 remaining non-coastal basins within the study area. Specific improvements and a suggested plan of action are provided. Additionally, financing alternatives for the previously established drainage districts are included.

## 1.2 APPROACH

This study focuses on the public stormwater system within the study area. Thus, public improvements related to the primary conveyance and outfall system will be analyzed; local and neighborhood systems will be studied only in enough detail to define their impact on the primary conveyance channels.

A detailed engineering analysis of the stormwater management system in the Alligator and Phillippi Creek basins, including hydrologic and hydraulic modeling studies of the major conveyance systems, is included. The results of this analysis will be used as a guideline for the analysis of the remaining 14 non-coastal basins.

The major tasks undertaken in the preparation of this Stormwater Master Plan are listed below:

1. Data collection
2. Drainage facility inventory
3. Land use mapping
4. Development of design storms
5. Determination of level of service
6. Study of Alligator and Phillippi Creeks
7. Assessment of remaining basins
8. Development and application of ranking system
9. Development of finance plan
10. Preparation of final report

Though the primary goal of this report is flood control, a study dealing solely with stormwater conveyance is inappropriate. Today's planners must focus not only on the flood relief, but on the effects the increased flow will have on neighboring properties and downstream waterways. These effects include degradation of downstream water quality, floodway inundation, and outfall environmental impacts. As a minimum, all stormwater management systems recommended by the in-depth study performed will meet all applicable stormwater management regulations and be permittable by the appropriate regulatory agencies.

The historical solutions to flooding problems within a basin have been referred to as "hard" solutions. These hard solutions include pipes, culverts, and in some cases, concrete-lined channels. The goal of these types of solutions is to remove the stormwater from an area as fast as possible. The primary benefit is that runoff from highly developed areas, where limited rights-of-way or erosion problems exist, can be adequately handled.

Recent stormwater management efforts have focused on "soft" drainage features. Soft features include natural drainageways, man-made lakes, canals, and grass-lined open channels. These types of features will be of primary consideration in this Master Plan because they offer benefits which may include: stormwater treatment, surficial aquifer recharge, and saltwater intrusion barriers.

# 2.0 BACKGROUND

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## 2.1 GENERAL

Sarasota County occupies approximately 590 square miles of southwestern Florida. The County is bounded on the west by the Gulf of Mexico, on the north by Manatee County, on the east by Manatee and Desoto Counties, and on the south by Charlotte County.

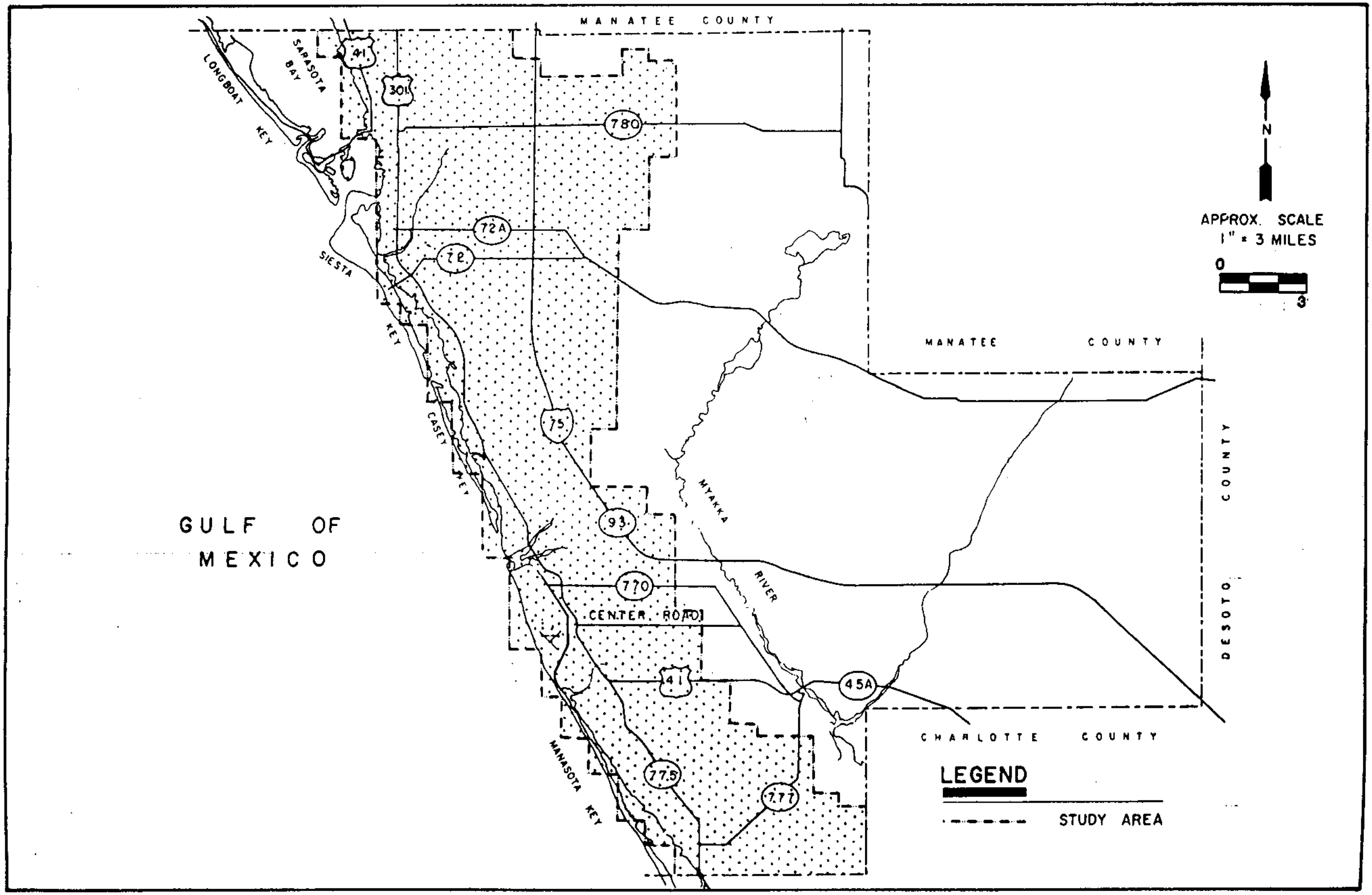
All of Sarasota County falls within an area of Florida described as the Coastal Lowlands. Changes in elevation are very gradual and the rise going away from the Gulf is barely perceptible over long stretches of landscape. Numerous depressions or shallow wet areas and sloughs of about 1 to 3 feet in depth are common. These depressions are dry or wet depending on the season, and most have no natural outlet.

The eastern portion of the County (out of the study area) drains primarily southward through the Myakka River. The western portion of the County drains through various creeks and streams to the Gulf of Mexico.

The study area for this master planning effort generally encompasses the area west of Interstate 75 along the western border of the County, as shown in Figure 2-1. The basin boundaries in this area have been delineated in a previous study and include 32 drainage basins. Of these 32 basins, 16 are coastal basins with stormwater flow draining directly to the Gulf and bays. The 16 remaining basins make up the study area for this planning effort.

The study area basins range in size from about 2 square miles to over 50 square miles. Phillippi Creek, the largest basin, occupies approximately 57 square miles, whereas Gulf Gate Canal, the smallest, occupies 1.7 square miles. Table 2-1 lists the basin names and their areas.

The scope of the study requires an in-depth study of portions of Alligator and Phillippi Creeks, with an overview examination of the remaining 14 major non-coastal basins. The Alligator and Phillippi Creek basins were




Project Study Area

FIGURE 2-1

FIGURE 2-1

# TABLE 2-1

## MAJOR NON-COASTAL BASINS

| Basin No. | Basin Name | Area (sq/mi) |
|---|---|---|
| 1 | Whitaker Bayou | 12.32 |
| 2 | Hudson Bayou | 2.15 |
| 3 | Phillippi Creek | 56.35 |
| 4 | Matheny Creek | 2.36 |
| 5 | Gulf Gate Canal | 1.67 |
| 6 | Catfish Creek | 6.44 |
| 7 | North Creek | 3.81 |
| 8 | South Creek | 20.13 |
| 9 | Shakett Creek | 10.76 |
| 10 | Curry Creek | 9.10 |
| 11 | Hatchett Creek | 5.42 |
| 12 | Alligator Creek | 9.86 |
| 13 | Woodmere | 3.90 |
| 14 | Forked Creek | 8.76 |
| 15 | Godfrey Creek | 10.74 |
| 16 | Ainger Creek | 15.44 |

178.2

selected for in-depth study due to existing conveyance problems, size, and the diverse land-use types which exist within the basins. These two basins are shown in Figure 2-2.

## 2.2 STORMWATER REGULATIONS

Stormwater management in Sarasota County is regulated by three agencies. Overall control rests with the Florida Department of Environmental Regulation (FDER), through Chapter 17-25 of the Florida Administrative Code. Chapter 17-25 also provides FDER with the ability to delegate responsibility for permitting to local water management districts. The Southwest Florida Water Management District (SWFWMD) has been delegated as the permitting entity for southwest Florida. Additionally, the County, through the use of its Land Development Regulations, has responsibility for stormwater management.

### 2.2.1 FDER 17-25 - REGULATION OF STORMWATER DISCHARGE

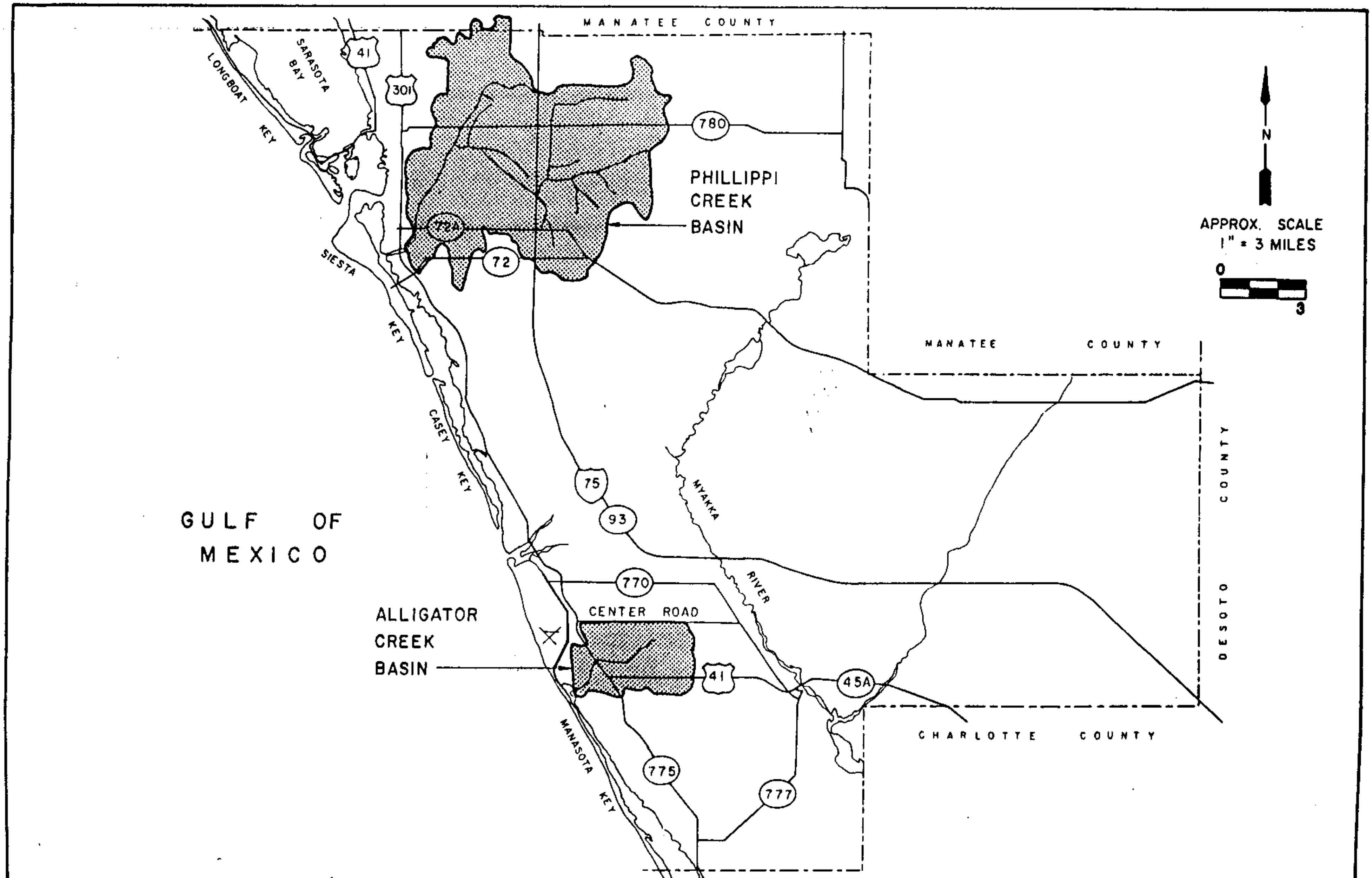
The Department of Environmental Regulation has been tasked under Chapter 17-3 of the Florida Administrative Code (FAC) to prevent pollution of state waters by the discharge of stormwater. To this end, Chapter 17-25 FAC "Regulation of Stormwater Discharge" has been promulgated. Section 17-25.025 presents design and performance standards by which stormwater discharges are governed. The salient points of this section are summarized as follows:

1. No discharge from a stormwater discharge facility shall cause or contribute to a violation of water quality standards in waters of the state.

2. Detention basins shall provide the capacity for the specified treatment volume within 72 hours following the storm.

3. Filtration systems shall have pore spaces such that the permeability of the filter is greater than or equal to the surrounding soil.

FIGURE 2-2




In-depth Study Basins

FIGURE 2-2

4. Filtration systems shall be designed with a safety factor of at least 2 unless it can be affirmatively demonstrated that a lower safety factor is sufficient.

5. Retention basins shall again provide the capacity for the given volume of stormwater within 72 hours after the rainfall event.

6. Sediment and erosion control will conform to best management practices.

7. Regional stormwater facilities shall provide treatment equivalent to retention, or detention and filtration, of the runoff from the first inch of rainfall; or, as an option, the first one-half inch of runoff for areas less than 100 acres.

8. Wetlands used as part of the stormwater management system shall maintain the normal range of water level fluctuation. Detention within the wetland area shall be at least 120 hours, with no more than one-half of the volume discharged within the first 60 hours.

Thus, Chapter 17-25 sets the standards for stormwater quality control. It provides for treatment utilizing retention or detention with filtration. Additionally, it provides for the utilization of wetland areas for stormwater detention and treatment. Section 17-25.09 establishes the agencies to which the authority to permit stormwater discharges has been delegated. In southwest Florida, including Sarasota County, this agency is the Southwest Florida Water Management District.

## 2.2.2 SWFWMD CHAPTER 40D-4

The Southwest Florida Water Management District has been given the authority under Chapter 17-25 to issue all surface water management permits. Within this responsibility, SWFWMD has promulgated Chapter 40D-4 "Management and Storage of Surface Waters."

An application to SWFWMD for a surface water management permit requires

1. Site Information - including location, topography, existing runoff and land use, wet season high water table, the 100-year flood plain, and vegetation description.

2. Master Drainage Plan - including basin boundaries, structures, easements, and seasonal water levels.

3. Drainage Calculations - including design storm characteristics, stage-storage and stage-discharge calculations, minimum building and road elevations, and the area of pervious, impervious, and water bodies.

A construction or operations permit will be issued only if the following major conditions are met by the proposed surface water management system. The issuance of the permit shall:

1. Not cause adverse water quality and quantity effects or cause adverse impacts on wetlands, fish, wildlife, or other natural resources;

2. Assure that design and performance standards are maintained to require that:

a. off-site discharge be limited to discharges which will not cause adverse effects downstream. The amount permittable is limited to the peak discharge from the site under pre-construction conditions unless otherwise determined in a previous permit action.

b. off-site discharge shall be computed using the SCS Type II Modified distribution for a 24-hour storm event with a 25-year frequency.

c. habitable structures should be at or above the 100-year flood elevation and encroachment into the 100-year flood plain minimized.

d. all projects must be designed to meet all applicable state water quality standards as set forth in Chapter 17-3 and Rule 17-4.242.

e. wetland areas and other environmentally sensitive areas shall be protected.

f. gravity control devices shall be designed such that 50 percent of the detention volume shall be discharged in one day.

### 2.2.3 SARASOTA COUNTY REGULATIONS

Sarasota County also has jurisdiction over stormwater management as outlined in Section B4, Article 4, of the County Land Development Regulations. Generally, the requirements closely follow both FDER and SWFWMD regulations. The Regulations call for:

1. Design for the peak discharge due to the 25- and 10-year storm for a major and minor stormwater system respectively.

2. Drainage systems to provide for the attenuation and retention of stormwater from the site. The rate of runoff after development must be less than or equal to pre-development conditions.

3. Drainage systems to be designed to treat the runoff from the first inch of rainfall where discharge is to a freshwater body.

### 2.2.4 SUMMARY

The Sarasota County Stormwater Master Plan is governed by all of the regulations that have been discussed in this section. Any recommended alternatives will, by necessity, meet or exceed all of these applicable stormwater regulations.

The above section is not intended to discuss all of the major drainage regulations but is presented to summarize the more important issues. For a more in-depth discussion, the applicable state and local publications should be consulted.

## 2.3 BASIS OF ANALYSIS

The purpose of this section is to present the basis for the hydraulic and hydrologic analysis to be used in this Master Plan.

### 2.3.1 MODELING APPROACH

The two basins that will be studied in-depth have been modeled both hydraulically and hydrologically. Two models were used in this phase of the investigation. The first model, "MSSM," is an adaptation of the United States Environmental Protection Agency's (USEPA) Stormwater Management Model (SWMMIII) RUNOFF portion. This adaptation includes provisions for:

lake storage; an improved routing capability (in which the major type of channels, e.g. double-trapezoidal channels, can be modeled); and an improved Horton's infiltration calculation equation parameter which allows the incorporation of a maximum infiltration volume.

The second model that has been used in this study is the HEC-2 model. This model, developed by the Hydrologic Engineering Center, provides a steady-state determination of the backwater effects of a storm on a particular stream section. The HEC-2 model, although not a dynamic model, has been used extensively to calculate the backwater effects on a non-steady state system such as Alligator and Phillippi Creeks. It is recognized by the Federal Emergency Management Agency as the model of choice for the performance of flood plain determination.

Use of the MSSM and HEC-2 models have provided a determination of the maximum discharge at various points along the stream channel. The normal procedure when analyzing the output from the runoff model would be to calibrate the model using streamflow data previously collected. This procedure cannot be used in the Alligator Creek basin due to the lack of a recording gage within the basin. Phillippi Creek was gaged (1980-1982). The gaged stream flow has been used for the calibration of Phillippi Creek, and the results were extrapolated over the entire study area. Additionally, the United States Geological Survey (USGS), WRI 82-42 (Lopez & Woodham, 1983), as well as recent work in Manatee County, has been used in the selection of the parameters for input and the calibration of the model.

### 2.3.2 INVESTIGATION REGIME

A modeling effort such as the one used in this investigation requires a vast amount of input data to adequately represent complex natural and man-made interception and conveyance systems. This input data is discussed briefly in the following paragraphs and in detail in subsequent chapters.

## Land Use

The current land use of the areas to be investigated has been established as part of the previous Basin and Subbasin Delineation effort. Generally, aerial maps of the area were reviewed, and the type and extent of the various land use types were defined. The resulting values were tabulated and can be found in the Basin and Subbasin Delineation Report. For the modeling effort in the two study basins, the previously defined values were reviewed and adjusted to reflect changes in land use patterns within the past year. Future land use was estimated by a review of the previous effort, consultation of the County's Future Land Use Plan - Apoxsee, and the incorporation of information on new developments, particularly Palmer Ranch.

## Physical Characteristics

The area, length of the main channel, overland flow length, soil type, etc. for each of the basins and subbasins were defined through the use of soil surveys, topographic maps, SWFWMD aerial maps with contours, subdivision plans, Florida Department of Transportation design and construction drawings, etc.

## Modeling Parameters

A discussion of the modeling parameters and the basis for their selection is found in Appendix A, as well as Sections 5.4.2 through 5.4.4 for Alligator Creek, and Sections 6.4.2 through 6.4.4 for Phillippi Creek. The selection of parameters is subjective in that there are few absolutes; rather there are acceptable value ranges within which the skilled modeler can select appropriate values. Additionally, studies that have previously been performed have been reviewed and incorporated as an aid to the selection process. Calibration of the model provides a verification of the selected parameters.

## 2.4 ALTERNATIVE SELECTION PROCESS

The ultimate purpose of the Sarasota County Stormwater Master plan is to identify alternatives that will alleviate the flooding problems that are currently experienced within the basins. The basin simulation efforts that are discussed in later chapters identify remedial action alternatives to alleviate these flooding problems. Once these alternatives have been identified, it is necessary that some method of selecting the best alternative be devised. For this reason an alternative selection process has been devised.

The alternative selection process that was developed for this study consists of two phases. Initially, the various stormwater management options that are developed in Sections 5.0 and 6.0 are investigated to determine their applicability in solving the flooding problems. Those options that are found to adequately solve the flooding problems are then analyzed by the use of a ranking system. The ranking system analyzes the alternatives on the basis of engineering and cost criteria.

### 2.4.1 THE RANKING SCHEME

There are several objectives that must be met to ensure that fair and equitable consideration is given to each of the stormwater control options. The evaluation objectives are:

1. To ensure that all criteria and constraints are applied to each alternative unilaterally.

2. To ensure that sufficient evaluation is conducted for all alternatives.

3. To provide a method which will identify and exclude non-viable options from further consideration.

The ranking procedure incorporates a matrix evaluation procedure whereby the applicable criteria acceptable to all alternatives are listed across the top, and the alternatives are listed along the left side of the matrix. Each alternative is assigned a ranking based on its relative performance within each criteria. The final ranking for the engineering criteria is obtained by pulling together the individual rankings to form a composite criteria rank.

### 2.4.2 ENGINEERING ANALYSIS CRITERIA

The engineering analysis criteria used in the ranking system are:

1. Reliability
2. Public support
3. Master Plan agreement
4. Implementability
5. Environmental impacts
6. Degree or need

These criteria are defined as follows:

1. Reliability - An alternative is considered reliable if all of the flow conveyance conditions can be met with no breakdown of the system. This is not to say that the alternative must afford protection from those floods above the 25-year design storm, but that the system shall perform as designed.

   The ranking values for this criterion are:

   0 - very low reliability
   1 - low reliability
   2 - reasonable reliability
   3 - high reliability
   4 - very high reliability
   5 - excellent reliability

2. Environmental Impact - By necessity, all conveyance and detention system structures must be designed in accordance with all applicable stormwater design regulations. However, within the guidelines, there is room for a certain degree of latitude. It is the goal of this Master Plan to provide stormwater control with minimal environmental impact.

   The ranking values for this criterion are:

   0 - severe environmental impact
   1 - substantial environmental impact
   2 - moderate environmental impact
   3 - marginal environmental impact
   4 - negligible environmental impact

3. Public Awareness/Acceptance - The measure of the public's acceptance is a complex but essential task. The public may impede, alter, or eliminate any alternative which it deems unacceptable. Should severe public opposition or lack of support be anticipated, a low ranking would ensue. The rankings are:

   0 - unacceptable
   1 - undesirable
   2 - moderately acceptable
   3 - acceptable
   4 - highly acceptable
   5 - enthusiastic

4. Master Plan Agreement - The objective of this Stormwater Master Plan is to integrate the various stormwater control measures throughout the study area into a manageable and functional system.

Ranking for the Master Plan Agreement factors are:

0 - disagreement
1 - slight agreement
2 - moderate agreement
3 - acceptable agreement
4 - substantial agreement
5 - total agreement

5. Implementability - Implementability reflects the ease at which the selected alternative can be instituted. Many factors affect the construction of a facility, including technical availability, environmental considerations, permitting considerations, funding questions, land acquisition problems, and political policy decisions. The accumulation of many of these hard to define factors may create a significant impact on the viability of an alternative. The rankings are:

0 - not feasible
1 - slightly feasible
2 - moderately feasible
3 - reasonably feasible
4 - feasible
5 - very feasible

The alternatives were ranked according to the selected criteria. A composite ranking was derived. A high ranking value indicates that the alternative is preferred over those with a lower value.

2.4.3 COST ANALYSIS

In addition to the engineering considerations, the primary factor governing the acceptability of a flood relief alternative is the cost. An alternative may be technically feasible and provide adequate protection, but at a cost that is prohibitive. In any case, the cost to benefit ratio must be acceptable. That is, the value of the benefits must be greatest when compared to the cost.

There are basically two types of costs associated with each of the alternatives: capital costs, and operation and maintenance costs.

1. Capital Costs - Capital costs are those costs associated with the original construction or purchase and installation of an alternative. For the purposes of this report, the components of capital costs are land purchases, retention structures, and conveyance structures. An example of these costs associated with an alternative might be:

   land purchase - of R/W for pond or conveyance system
   retention system - weirs, fences, earthwork, etc.
   conveyance structures - channels, pipes, bridges, etc.

2. Operation and Maintenance Costs (O&M) - O&M costs are those costs required to keep the facility operating in the manner in which it was designed to function. O&M costs usually associated with stormwater facilities include pond sediment removal, filterbed cleaning, weed removal, channel maintenance, and inlet and pipe cleaning, etc.

### 2.4.4 SUMMARY

The final ranking of alternatives is based on the use of both the engineering and cost analyses. A second matrix integrating capital and O&M costs with the engineering parameters yields a final ranking of alternatives. For an illustrative example of this ranking technique, refer to the alternative selection process in Sections 5.0 and 6.0.

## 2.5 LEVEL OF DRAINAGE PERFORMANCE/STORMWATER SERVICE LEVELS

The objective of this subsection is to present criteria which will specify the level of service to be used for evaluation of stormwater drainage systems in Sarasota County. The criteria will identify the acceptable depth and frequency of flooding of buildings (residential, commercial, and industrial), adjoining green space or open space, roadways, and agriculture that will occur during a storm of design proportions. The level of service outlined in the following subsections will be used in the subsequent detailed analysis and recommendations for improvements of the drainage system.

### 2.5.1 GENERAL

Rainfall within a basin can either be absorbed or infiltrated into the soil, held in depression storage, or can flow as runoff downgradient to the conveyance system. The conveyance system, composed of natural channels, pipes, culverts, etc., passes the flow downstream to its ultimate outfall. Sarasota County's major drainage systems are natural, albeit "improved," channels which flow to one of the tidally controlled bays, and then into the Gulf of Mexico. Generally, as the rainfall increases, the flow to the system increases with time accounting for the time it takes the runoff to reach the point of inflow to the conveyance system. Thus, the peak inflow to the system will normally occur after the peak rainfall has occurred. This delay in the occurrence of peak runoff is known as basin lag and is distinct for each basin or subbasin.

The runoff from each subbasin reaches the inlet to the conveyance system with a distinct hydrograph shape and timing. This flow is then attenuated as it is routed downstream toward its ultimate outfall. Although the immediate stream channel may possibly pass the flow from each individual subbasin without flooding, the flow from all subbasins (with their individual hydrographs added and attenuated) may exceed the channel's capacity.

A basic criterion in any stormwater management study is the level of performance that is expected for the drainage system. Since it is not feasible to completely contain, within natural stream channels, the runoff from all storms of any duration and volume, it is implicit that some amount of out-of-bank flooding be permissible when runoff exceeds a pre-established design storm. Sarasota County has adopted a 24-hour, 25-year storm for use in the analysis and design of major stormwater system components.

Theoretically, it is possible for a stormwater conveyance and detention system to be designed and constructed which would pass the runoff from the design storm with absolutely no flooding. However, in most natural channels, a storm with a return period of 2 to 5 years will reach a bankfull condition, which would normally be a threshold for out-of-bank flooding. The cost of a drainage system which would prevent any flooding for larger storms may be prohibitive. Thus, it is imperative that some attainable goal for flood prevention be formulated, and subsequent analysis and design of stormwater management systems conform to this goal.

The following paragraphs define and justify the allowable limits of flooding which are economically feasible, yet offer sufficient protection for the safety and welfare of the general public and private property. Limits for flooding will be derived for streets, open or green space, and residential and non-residential structures.

### 2.5.2 STREET FLOODING

The perception of the general public as to what constitutes street flooding is somewhat different than that of a stormwater design engineer. The public is subjectively concerned with perceived inconvenience, whereas the engineer is objectively evaluating vehicle passability, and public safety and welfare. Streets are an integral part of the drainage system in that they convey water from adjoining properties to a point of collection, i.e. the catch basin.

It is considered essential that arterial roadways remain passable to the extent that at least half of the normal travel lanes are free from standing water. This will ensure that there is at least one lane of travel passable at all times, and that there will be no interference with the travel of emergency vehicles.

The objective test for vehicle passability is the determination of the depth above the street surfaces at which water will reach the lower door and/or floor pan and enter the vehicle. Generally, the pan and door heights are a minimum of 6 inches above pavement. Roads are typically designed and constructed to have a crown along the centerline with slopes of approximately 1 inch for every 4 feet of width from the centerline (FDOT drainage manual). A conveyance system that allows no more than 6 inches of water to be standing in the curb or shoulder area will generally allow for the safe passage of vehicles along a roadway. Thus, for the purposes of this stormwater management study, flow to a depth of 6 inches or less on the outer edge of the street surface will be considered acceptable. This 6-inch depth will cause the street to be almost fully covered with water. This will allow the safe passage (one lane) of emergency vehicles.

### 2.5.3 STRUCTURE FLOODING

Current Sarasota County development codes call for the building of all habitable structures above the 100-year flood plain. It is envisioned that this restriction will remain in effect in the future and that all permitted construction will be above this flood level. Construction in the years preceding the adoption of this code has resulted in many structures that are below the 25-year design storm flood level. The objective of this Stormwater Management Study will be to eliminate structure flooding of the first habitable floor in private residences as a result of the 25-year design storm. The scope of the study precludes the management of floods that are tidally induced or occur in properties other than those along the major drainage systems. All methods of flood protection will be considered, including the purchase of residences within the 25-year flood zone if it is determined to be the most cost-effective method.

Many commercial and industrial properties may also be within the 25-year design storm flood plain. Another objective of this Stormwater Management Study is to prevent flooding of all commercial and industrial properties where the flooding would interfere and impede the intended use of the property. This may require structural stormwater management measures to lower flooding below the ground floor level of buildings. However, when no residential or street flooding is occurring in the same area, an alternative that will be considered in controlling the flooding of commercial and industrial buildings will be the use of floodproofing techniques.

### 2.5.4 OPEN OR GREEN SPACE FLOODING

Green space can be considered that portion of the surface vegetation that is covered generally by grass and other non-harvestable crops. These green space areas include swales, yards, parks, road medians, etc. Usually green space can be flooded for a short time with no deleterious effects.

Green space surrounding major drainage systems normally floods to some extent whenever storm discharge exceeds the channel capacity. The composite 25-year design storm hydrograph at any point may have peak flows greater than the capacity of the channel, resulting in flooding. Limiting flooding of open or green space in residential, commercial, or industrial areas to a depth of between 12 and 18 inches, unless the area is specifically intended to convey or detain water (e.g., swales, retention ponds, and ditches along street frontages), is an objective of this study. The maximum flood level in green areas is such that there will be no threat to public health or safety, or permanent impediment to the intended use of property. Within agricultural areas which are not intended for development, the current level of floodplain flooding during the design storm is satisfactory providing no adverse public safety effects occur.

## 2.5.5 SUMMARY

In summary, some flooding due to the 25-year, 24-hour design storm may be expected. This is because the 25-year storm substantially exceeds the capacity of most channel systems. However, the stormwater management alternatives presented in the subsequent sections of this report will modify or reduce the flooding conditions along the major drainage channels such that (1) street flooding will be allowed up to a depth of 6 inches in the shoulder or curb area, (2) the flooding of the first habitable floor of private residences will be eliminated, (3) flooding of commercial/ industrial properties will be controlled to a point that the intended use of the property is not impeded, and (4) where allowable, flooding of green space will be allowed to a depth between 12 and 18 inches. In no case will flooding along a major channel be permissible if it endangers the public health and safety or substantially impedes the intended use of property. These guidelines will serve as the basis for evaluation of stormwater management systems throughout Sarasota County.

# 3.0 LAND USE CHARACTERISTICS

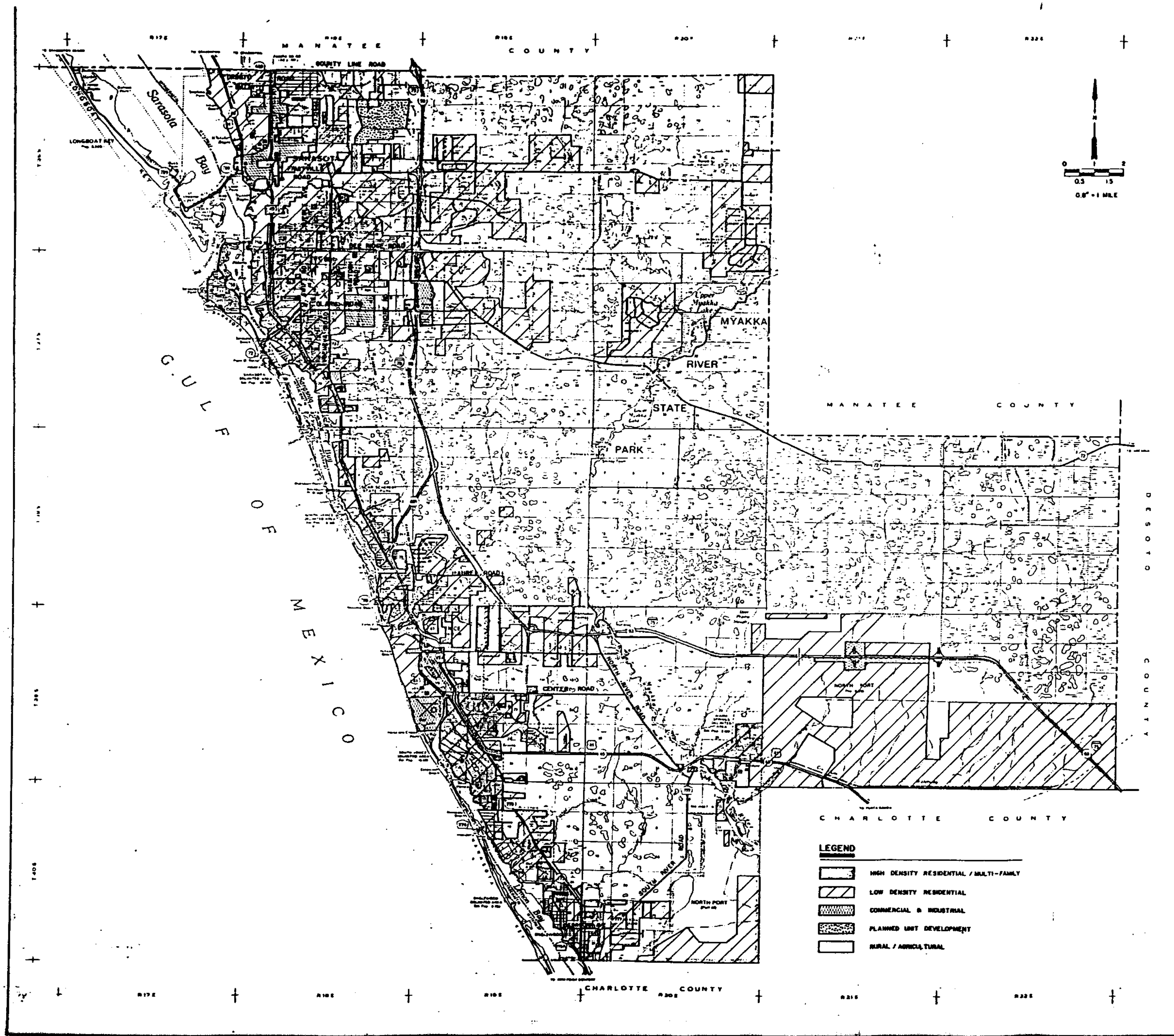
# 3.0 LAND USE CHARACTERISTICS

It is generally agreed that urbanization causes a change in the intensity and duration of the stormwater runoff hydrograph. Areas that are predominantly rural tend to have hydrographs that have relatively flat peaks and a long duration. Urban areas, on the other hand, generally have higher peaks with a shorter duration. This is due primarily to the amount of pervious area within the basins and subbasins. It is therefore necessary that the type of land use be established for each basin and subbasin to be modeled. To this end, land use maps for both the current and future conditions have been established. It must be noted that these maps are not intended to replace or supplement existing Sarasota County Planning Department land use maps and/or data, but are merely an aid to the hydrologic modeler in modeling parameter identification.

## 3.1 CURRENT LAND USE

The current land use map (Figure 3-1, Plate 3-1) has been compiled using the aerial photography available as part of the previous Basin and Subbasin Delineation and Refinement project, and a knowledge of the area. Five land use categories have been defined for this mapping effort. They are: (1) commercial/industrial, (2) high density residential/multi-family, (3) low density residential/single family, (4) planned unit development, and (5) open/green space and rural.

As can be seen on the map, the major historical thrust of development has been along the west coast and barrier islands. High intensity urbanization has occurred in and around the City of Sarasota, and to a lesser extent around the Cities of Venice and Englewood. Generally, development decreases as you move eastward, with only minimal and scattered development occurring east of Interstate 75, which has been limited primarily to single family residential development.




BASED ON SARASOTA COUNTY GENERAL HIGHWAY MAP

SHEET NO 1

CAMP DRESSER & McKEE INC.

CDM

MAPPING STUDY

EXISTING LAND USE MAP

SARASOTA COUNTY, FLORIDA

PROJECT NO 9250-5

SHEET NO 1

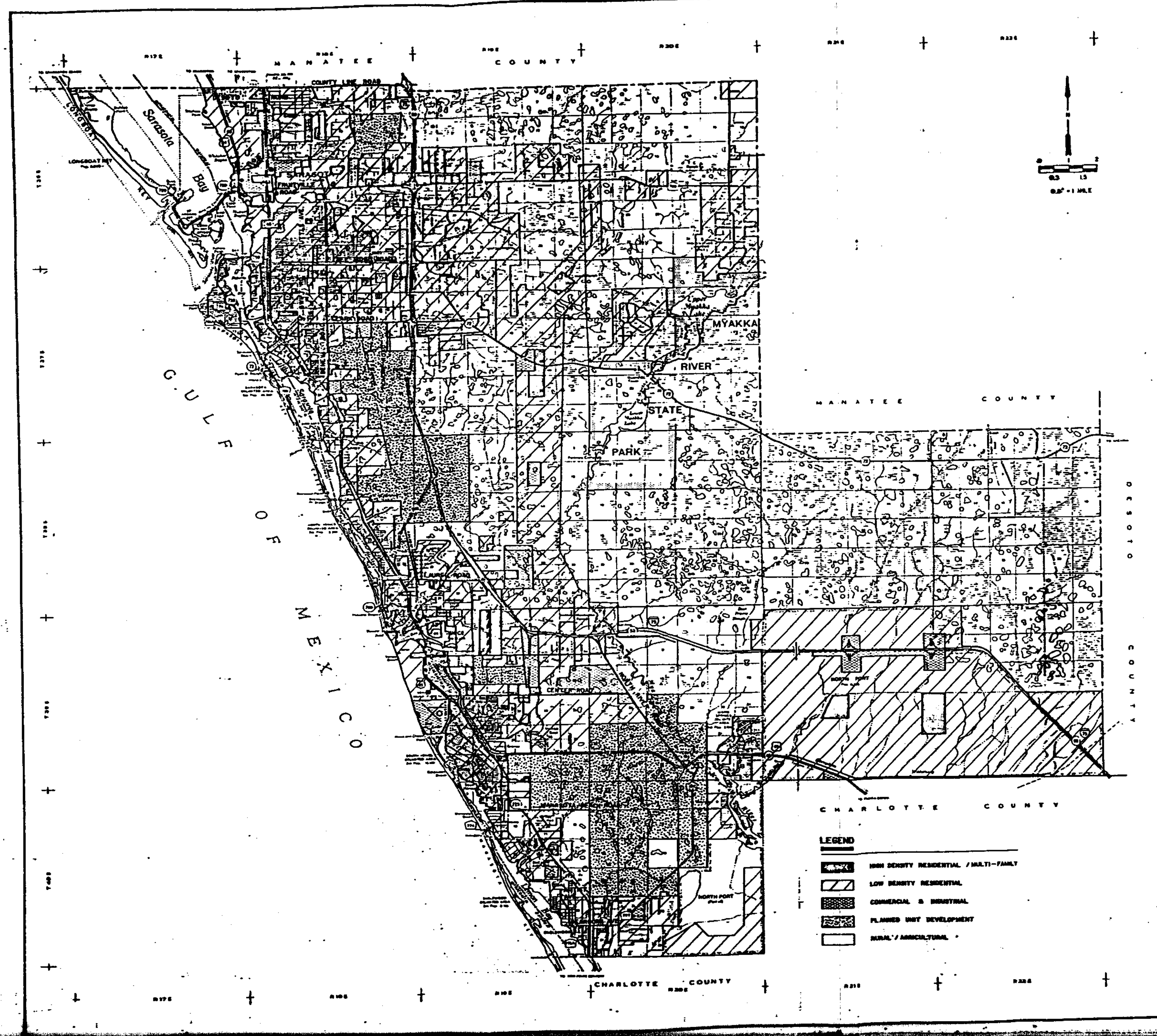
## 3.2 FUTURE LAND USE

The future projected land use is shown in Figure 3-2 (Plate 3-2). This map was created using the existing land use data as shown previously in Plate 3-1, the County Apoxsee - Future Land Use Plan section, and information on the extent of developments which will have a significant influence on the future land use patterns.

The Apoxsee's future land use map, as well as the directives therein, provide only the basis for the management of growth within the County and are not meant to dictate the land use of any particular parcel. They do, however, establish guidelines for development through the use of development zones. Basically, these zones form concentric rings around the major cities within the County with the highest density of development at the center. The density decreases as you move away from the hub, until it reaches areas that are proposed to remain rural.

There are currently two new large developments within the County which will have a major impact on the land use within the areas they control. The largest, Palmer Ranch, is centrally located within the study area and involves approximately 10,000 acres. The other is Berry Properties, about half the size of Palmer Ranch, encompassing approximately 5,000 acres. Together, these two planned developments account for about 23.5 square miles of future growth. Although both projects are still in the initial development stage, they will both follow the Planned Unit Development (PUD) Concept, in which many different types of development will occur. Preliminary reports stress that both single and multi-family residential units are planned, as well as commercial and some light industrial development.

The completion of Interstate 75 through the County, as well as the soon to be completed links at Tampa and Miami, will undoubtedly cause greater development along that corridor. In just the few short years since its opening, a marked change in land use along this road is evident. It is expected that a commercial/industrial corridor will form along the interstate with associated support developments occurring to the east.




BASED ON SARASOTA COUNTY GENERAL HIGHWAY MAP

CAMP DRESSER & McKEE INC.

CDM

MAPPING STUDY

POTENTIAL FUTURE LAND USE MAP

SARASOTA COUNTY, FLORIDA

PROJECT No. 9250-6

SHEET No. 2

Another probable primary development corridor is located along a strip approximately one mile wide, running north and east from Interstate 75 at Jacaranda Boulevard, near Venice, to Fruitville Road. Although virtually undeveloped at present, this land seems uniquely suitable for future development given its location along a natural ridge line, its proximity to public services, and the availability of adequate feeder road systems at both its northern and southern boundaries. It is likely that these initial feeder roads will be connected forming a new north-south corridor. Discussion with developers, land owners, and planners suggest that this area will be developed at ultimate buildout.

# 4.0 DESIGN HYDROLOGY/METHODOLOGY

# 4.0 DESIGN HYDROLOGY/METHODOLOGY

## 4.1 DESIGN RAINFALL

The development of a stormwater management program for Sarasota County must be based on specific objectives or levels of service in controlling flooding. Implicit in these objectives is the design storm, which is the basis for hydrologic evaluation of drainage systems.

### 4.1.1 GENERAL

An analysis of the various past rainfall studies in the vicinity of Sarasota County by the National Oceanic and Atmospheric Administration (formerly the U.S. Weather Bureau) and various private firms, for the purpose of establishing a design storm, is presented herein. The four facets which define a particular design storm are: (1) the frequency of occurrence, (2) the storm duration, (3) the total amount of rainfall for the particular storm frequency and duration, and (4) the temporal distribution of the rainfall amount over the storm duration. The basis for definition of each facet is reviewed and specific criteria are proposed.

Four reports are reviewed in detail to define the design storm including: (1) U.S. Weather Bureau Technical Paper No. 40 (published in 1961), (2) the August 1979 report by Camp Dresser and McKee Inc., titled "Rainfall Analysis," which was prepared as part of the Pinellas County Storm Drain Study, (3) the July 1981 report by Howard, Needles, Tammen and Bergendoff, Inc. (HNTB), titled "Emergency Spillway, Lake Manatee Dam," and (4) the September 1982, report by Reynolds, Smith and Hills Inc. (RS&H), titled "Hydrologic Investigation and Stormwater Management Plan for the Curiosity Creek Basin." Reference is also made to the October 1983 NURP Report by Metcalf and Eddy titled "Tampa Nationwide Urban Runoff Program, Phase III, Final Report."

### 4.1.2 RAINFALL FREQUENCY

Based on benefit/cost studies made over the last 35 years, protection against a rainfall event equivalent to the 25-year frequency is thought to be economically justified in Sarasota County. Based on these studies, the 25-year frequency was previously adopted for purposes of establishing a design rainfall by Sarasota County and incorporated into the requirements of the County land development and drainage regulations. This same frequency is generally used throughout the State of Florida for stormwater management studies and is adopted herein for the Sarasota County Stormwater Management Study.

### 4.1.3 RAINFALL DURATION

From various studies of past major rainfall events that have occurred in the southeastern portion of the United States, and in the western portion of Florida in particular, it is apparent that a large portion of the total rainfall of most major storms occurs within a 24-hour period. This was graphically demonstrated in the Curiosity Creek study (RS&H, 1982) which lists the maximum annual rainfall volumes for one day, and consecutive two- and three-day periods for each year. These volumes were determined by an analysis of 49 years of daily rainfall records at the Tampa International Airport (Tampa) rain gage. From an examination of these storm volumes, reproduced herein as Tables 4-1 through 4-3, it is evident that 80 to 90 percent of the total volume of most major rainfalls in west central Florida occurs within a 24-hour period. In the Curiosity Creek study, no continuous 48- or 72-hour rainfall events were found at the Tampa gage. The Lake Manatee Dam Report (HNTB, 1981) noted that a great portion of most past major rainfalls fell within a 24-hour period. Sarasota County has previously adopted the 24-hour storm as the basis for analysis of drainage systems.

Therefore, based on these past studies, a duration of 24 hours was adopted for the design rainfall. The 24-hour duration design rainfall is in accordance with the "rule-of-thumb" for hydrologic studies which mandates that the duration of the design rainfall should be approximately equal to

TABLE 4-1

ANNUAL PEAK RAINFALL VOLUME
24-HOUR DURATION

| Storm Date | Rainfall Amount (inches) | Storm Date | Rainfall Amount (inches) |
|---|---|---|---|
| 9-07-33 | 5.75 | 2-28-58 | 2.69 |
| 6-15-34 | 6.54 | 6-21-59 | 2.82 |
| 9-06-35 | 6.30 | 6-30-60 | 9.07 |
| 7-06-36 | 3.53 | 2-05-61 | 2.23 |
| 2-12-37 | 5.06 | 9-22-62 | 4.06 |
| 6-24-38 | 3.04 | 11-12-63 | 3.81 |
| 8-14-39 | 3.28 | 7-28-64 | 4.04 |
| 4-10-40 | 2.26 | 8-02-65 | 2.32 |
| 4-05-41 | 4.00 | 6-10-66 | 2.92 |
| 7-17-42 | 2.17 | 8-14-67 | 3.25 |
| 6-27-43 | 3.99 | 10-20-68 | 2.49 |
| 10-21-44 | 3.61 | 5-18-69 | 3.05 |
| 6-25-45 | 9.88 | 8-10-70 | 2.55 |
| 7-21-46 | 4.66 | 5-17-71 | 3.45 |
| 9-20-47 | 3.57 | 8-21-72 | 3.44 |
| 1-26-48 | 2.16 | 3-27-73 | 3.17 |
| 8-14-49 | 4.92 | 6-28-74 | 5.47 |
| 8-30-50 | 3.66 | 6-20-75 | 3.13 |
| 4-09-51 | 3.25 | 5-17-76 | 3.86 |
| 3-28-52 | 2.23 | 8-22-77 | 1.32 |
| 3-23-53 | 3.30 | 2-18-78 | 1.86 |
| 11-15-54 | 3.49 | 5-08-79 | 11.45 |
| 7-03-55 | 2.84 | 4-14-80 | 2.35 |
| 5-10-56 | 1.89 | 2-08-81 | 3.11 |
| 7-27-57 | 5.38 | | |

TABLE 4-2

ANNUAL PEAK RAINFALL VOLUME
48-HOUR DURATION

| Storm Date | Rainfall Amount (inches) | Storm Date | Rainfall Amount (inches) |
|---|---|---|---|
| 9-07-33 | 8.14 | 2-28-58 | 2.85 |
| 6-15-34 | 7.91 | 6-20-59 | 3.90 |
| 9-06-35 | 7.78 | 7-30-60 | 13.96 |
| 7-06-36 | 3.76 | 12-21-61 | 2.93 |
| 2-12-37 | 5.06 | 9-22-62 | 4.44 |
| 10-16-38 | 4.84 | 11-13-63 | 4.29 |
| 7-08-39 | 4.02 | 7-28-64 | 5.44 |
| 8-04-40 | 2.52 | 8-02-65 | 4.11 |
| 4-06-41 | 4.74 | 7-24-66 | 3.81 |
| 8-26-42 | 2.38 | 8-15-67 | 5.78 |
| 6-28-43 | 4.47 | 10-20-68 | 3.07 |
| 10-21-44 | 5.49 | 12-16-69 | 3.48 |
| 6-26-45 | 10.51 | 8-11-70 | 3.23 |
| 7-21-46 | 4.66 | 9-12-71 | 4.51 |
| 9-20-47 | 4.08 | 8-22-72 | 3.56 |
| 7-31-48 | 3.39 | 3-27-73 | 3.17 |
| 8-15-49 | 5.63 | 6-28-74 | 6.41 |
| 8-30-50 | 6.52 | 6-20-75 | 3.20 |
| 9-17-51 | 4.46 | 5-18-76 | 4.20 |
| 3-28-52 | 3.88 | 8-22-77 | 2.08 |
| 9-12-53 | 4.46 | 10-29-78 | 2.15 |
| 11-15-54 | 3.56 | 5-08-79 | 12.30 |
| 9-02-55 | 3.79 | 7-25-80 | 2.43 |
| 6-10-56 | 2.18 | 8-21-81 | 4.12 |
| 7-28-57 | 6.14 | | |

TABLE 4-3

ANNUAL PEAK RAINFALL VOLUME
72-HOUR DURATION

| Storm Date | Rainfall Amount (inches) | Storm Date | Rainfall Amount (inches) |
|---|---|---|---|
| 9-07-33 | 8.98 | 3-01-58 | 2.98 |
| 6-15-34 | 9.15 | 3-19-59 | 4.48 |
| 9-06-35 | 8.12 | 7-31-60 | 14.57 |
| 7-07-36 | 3.81 | 12-21-61 | 2.93 |
| 3-14-37 | 5.64 | 8-25-62 | 4.77 |
| 10-17-38 | 5.45 | 11-13-63 | 4.70 |
| 7-09-39 | 5.06 | 7-28-64 | 5.77 |
| 8-05-40 | 2.82 | 8-02-65 | 4.85 |
| 4-06-41 | 4.74 | 6-10-66 | 4.37 |
| 6-11-42 | 2.50 | 8-16-67 | 6.33 |
| 6-29-43 | 6.55 | 7-11-68 | 3.95 |
| 10-21-44 | 5.49 | 7-21-69 | 4.36 |
| 6-26-45 | 10.89 | 8-12-70 | 3.25 |
| 7-21-46 | 4.66 | 9-12-71 | 6.01 |
| 9-21-47 | 4.21 | 8-21-72 | 3.95 |
| 8-13-48 | 4.24 | 9-06-73 | 3.42 |
| 8-15-49 | 5.92 | 6-28-74 | 9.08 |
| 8-31-50 | 7.38 | 6-20-75 | 3.20 |
| 9-17-51 | 4.48 | 5-20-76 | 4.72 |
| 3-29-52 | 4.27 | 8-22-77 | 2.10 |
| 9-13-53 | 4.61 | 2-18-78 | 2.91 |
| 11-17-54 | 3.59 | 5-08-79 | 12.98 |
| 9-03-55 | 4.22 | 7-25-80 | 2.43 |
| 10-07-56 | 2.21 | 9-18-81 | 4.78 |
| 7-28-57 | 6.14 | | |

or greater than the time of concentration of the basin. The time of concentration of the Phillippi Creek Basin (the largest in the County) is believed to be in the range of 15 to 20 hours.

### 4.1.4 RAINFALL VOLUME

The total volume of rainfall for a 25-year frequency, 24-hour duration storm for Sarasota County can be determined from the isopleth maps in Technical Paper No. 40 (U.S. Weather Bureau, 1961), and varies from 9.0 inches on the eastern border of Sarasota County to just under 10 inches on the coast line, with an average of approximately 9.5 inches. This pattern of variation across the County also occurs in the Pinellas/Hillsborough County area. An average volume of 9.4 inches was previously determined from Technical Paper No. 40 for the Curiosity Creek Basin in Hillsborough County (RS&H, 1982).

Technical Paper No. 40 was published in 1961 and hence does not reflect the last 24 years of record. More recent analyses of rainfall volume to establish annual duration-frequency relationships have been prepared by CDM (1979) and RS&H (1982). Both of these analyses used a Log Pearson Type III distribution and procedure to estimate storm volume—RS&H on the Tampa data, and CDM on the Tampa, St. Petersburg, St. Leo, and Parrish rain gage data. The results of these studies are summarized in Table 4-4.

The studies by CDM (1979) and RS&H (1982) analyzed rainfall volume based on a normal 24-hour "clock" day; i.e., if a storm began before twelve o'clock midnight and extended into the next day, the storm volume was split into two 24-hour periods, even though the total duration of the storm was less than 24 hours. Statistical analyses have shown that storm volume estimates based on a clock day can be related to estimates based on an actual continuous 24-hour period by a multiplication factor of 1.13. Making the necessary corrections to the previous estimates by RS&H (1982) and CDM (1979), the adjusted 24-hour storm volumes were computed and are shown in Table 4-5.

TABLE 4-4

25 YEAR FREQUENCY - 24 HOUR DURATION RAINFALL
RESULTS OF LOG PEARSON ANALYSIS

| Location | Company Which Performed the Analysis | Amount |
|---|---|---|
| Tampa | (RSH) | 8.07 |
| Tampa | (CDM) | 8.00 |
| St. Petersburg | (CDM) | 8.07 |
| St. Leo | (CDM) | 8.73 |
| Parrish | (CDM) | 8.60 |

TABLE 4-5

25 YEAR FREQUENCY - 24 HOUR DURATION RAINFALL
(Corrected for Clock Hour Difference)

| Location | Company Which Performed the Analysis | Amount |
|---|---|---|
| Tampa | (RSH) | 9.12 |
| Tampa | (CDM) | 9.04 |
| St. Petersburg | (CDM) | 9.12 |
| St. Leo | (CDM) | 9.86 |
| Parrish | (CDM) | 9.72 |

Based on the new analysis, the 25-year frequency, 24-hour duration rainfall varies from approximately 9.0 at the coast line of the County to approximately 10.0 on its eastern boundary, which is opposite the pattern of variation that is indicated by Technical Paper No. 40. However, an average volume of approximately 9.5 inches for the County is consistent with previous studies and is adopted herein as the 25-year frequency, 24-hour duration design rainfall.

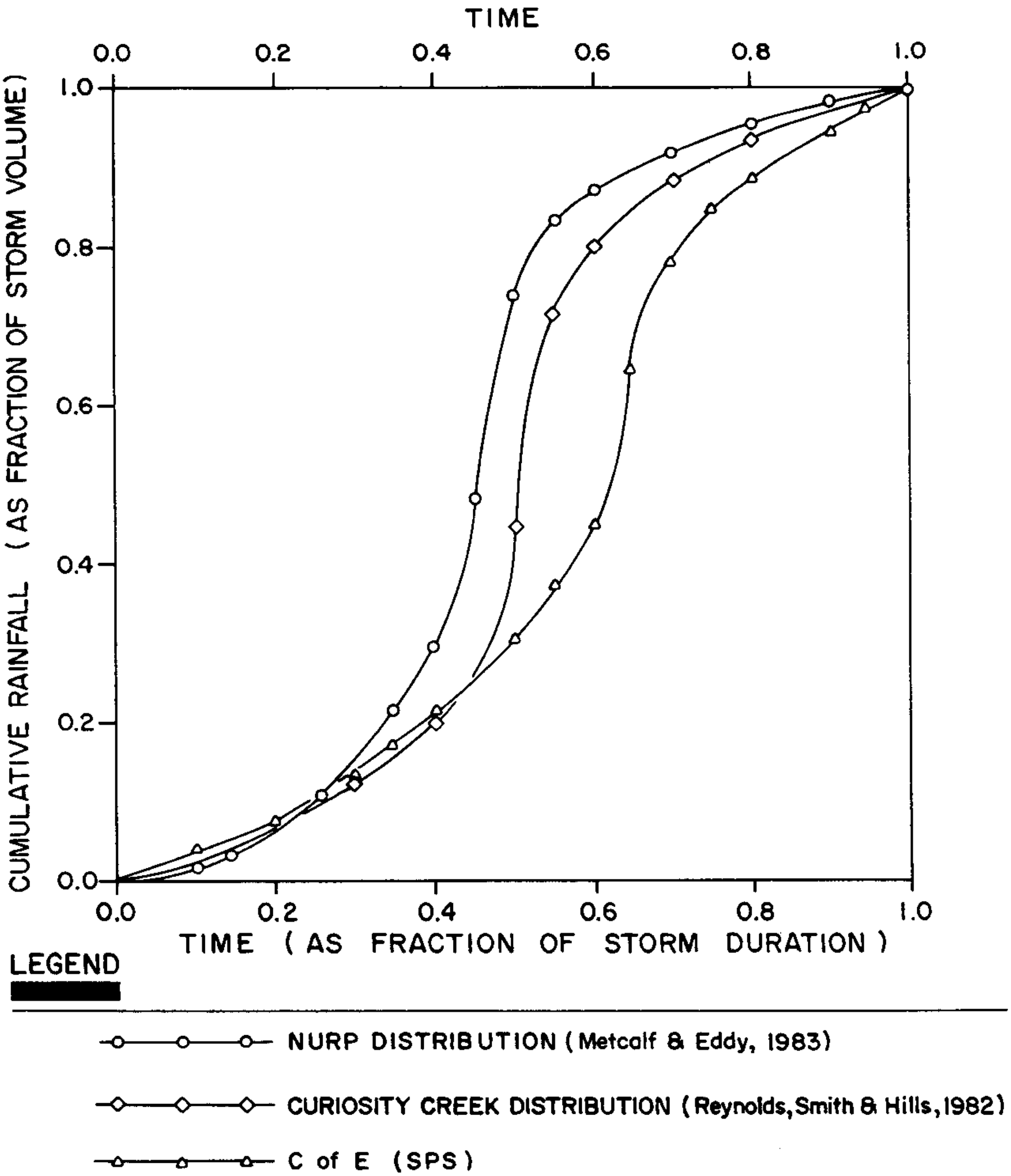
4.1.5 RAINFALL DISTRIBUTION

Four rainfall distributions have recently been defined or computed which apply in the Sarasota/Manatee/Hillsborough area.

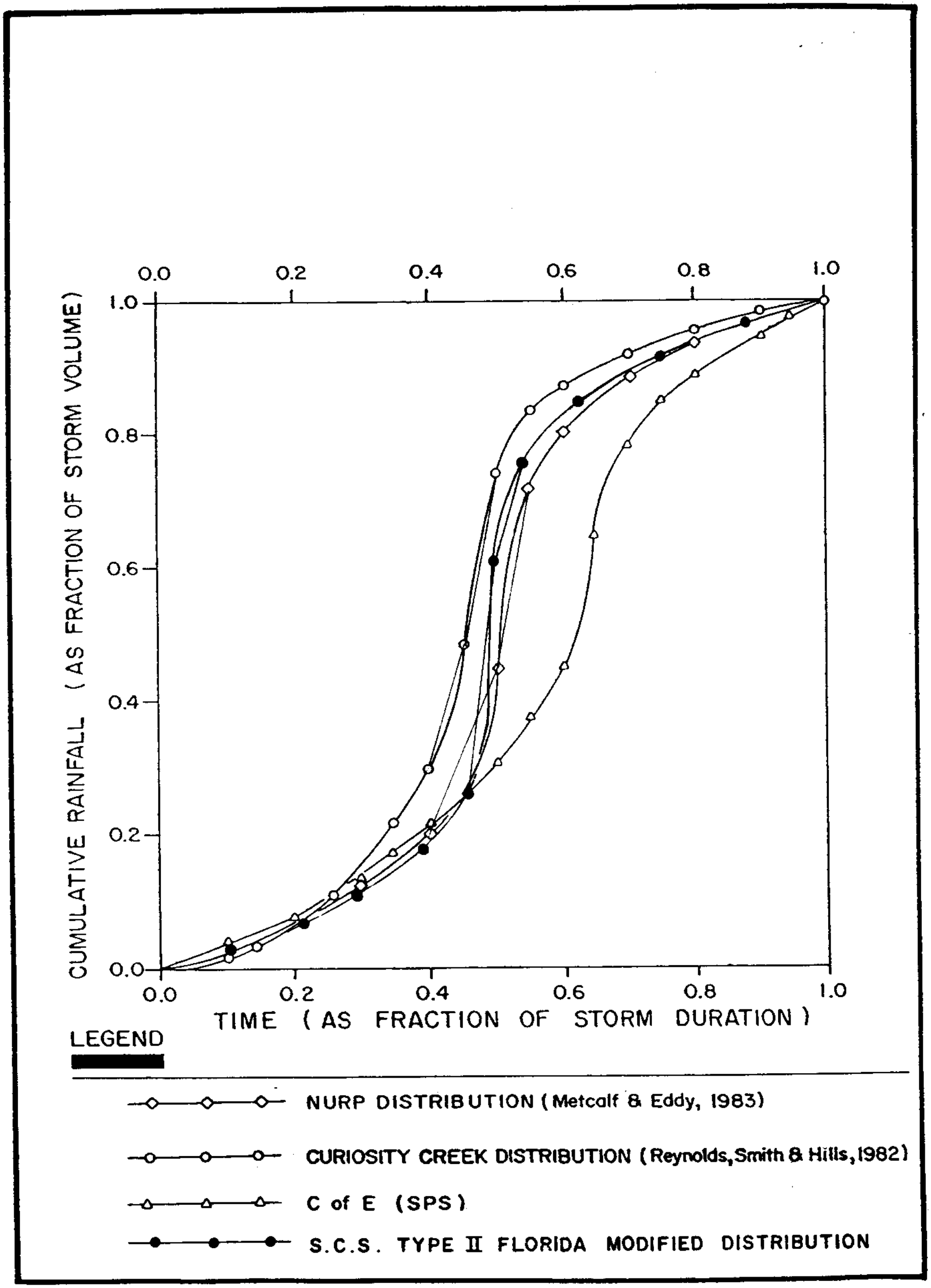
The most recent distribution was prepared by RS&H (1983) as part of the Curiosity Creek Study. As part of their work, RS&H applied the method of Pilgrim and Cordery to 12 of the largest 24-hour duration Tampa rainfalls for the periods 1948 to 1952 and 1958 to 1979. The mass curves which typify these storms are shown in Figure 4-1. The early peaking storm curve was recommended as the 24-hour rainfall distribution for the Curiosity Creek Study (RS&H, 1982), as it produced higher runoff peaks than the late peaking or combined storm curves.

As part of hydrologic analyses for the Lake Manatee Dam and new emergency spillway, HNTB (1981) selected the distribution of the Corps of Engineers Standard Project Storm as being most representative of heavy rainfalls. This same rainfall distribution was also used by CDM for the 25-year frequency, 24-hour duration design rainfall in the South Venice Gardens Stormwater Management Study (1983).

As part of their work on the NURP study in the City of Tampa, Metcalf and Eddy (1983) also established a dimensionless rainfall distribution. A fourth distribution is the United States Soil Conservation Service Type II (Modified for Florida) - SCS Type II Modified. All four distributions are plotted for comparative purposes as dimensionless mass curves in Figure 4-2.




Non-Dimensional Mass Curves of Rainfall Volume **FIGURE 4-1**




Non-Dimensional Mass Curves of Rainfall Volume **FIGURE 4-2**

As can be observed from Figure 4-2, all four mass curves are very similar. However, the NURP and RS&H curves are somewhat steeper than the Corps of Engineers curve such that their use would result in slightly higher hydrograph peaks. However, because of the many variables involved in hydrograph computations, the use of any of the mass curves would produce similar results.

### 4.1.6 CONCLUSIONS

Based on the above discussions, the 25-year frequency, 24-hour duration design rainfall is defined by 9.5 inches of total rainfall volume and is to be distributed over 24 hours in accordance with the Corps of Engineers mass curve which will be used as the standard for the Sarasota County Stormwater Management Study.

As stated in Section 2.2.2, the currently accepted SWFWMD design storm is the SCS Type II Modified for Florida. The above discussion indicates that the Corps of Engineers Standard Project Storm (SPS) is more realistic for the Sarasota County coastal area. Discussions were held between SWFWMD, Sarasota County, and CDM at the beginning of the modeling effort, culminating in the agreement that the Corps of Engineers' SPS would be satisfactory for this investigation.

Successive 30-minute rainfall increments for this design-storm and a summation of these rainfall increments are listed in Table 4-6. A hyetograph of ~~these rainfall increments is plotted in~~ Figure 4-3. This design storm is the same as previously used in the South Venice Gardens Stormwater Management Study by CDM (1983).

## 4.2 ANTECEDENT CONDITIONS

Chapter 40-D4 of the Rules of the Southwest Florida Water Management District - Management and Storage of Surface Waters - Section 40D-4.301 specifies that Soil Conservation Services (SCS) antecedent moisture condition II (AMC II) be used for both pre- and post-development conditions.

TABLE 4-6

25 YEAR FRQUENCY - 24 HOUR RAINFALL
(One Half Hour Increments)

| Time Hours | P Inches | P Inches | Time Hours | P Inches | P Inches |
|---|---|---|---|---|---|
| 0.5 | 0.07 | 0.07 | 12.5 | 3.18 | 0.23 |
| 1.0 | 0.14 | 0.07 | 13.0 | 3.42 | 0.24 |
| 1.5 | 0.21 | 0.07 | 13.5 | 3.77 | 0.35 |
| 2.0 | 0.28 | 0.07 | 14.0 | 4.13 | 0.36 |
| 2.5 | 0.35 | 0.07 | 14.5 | 4.58 | 0.55 |
| 3.0 | 0.42 | 0.07 | 15.0 | 5.23 | 0.55 |
| 3.5 | 0.49 | 0.07 | 15.5 | 6.11 | 0.88 |
| 4.0 | 0.56 | 0.07 | 16.0 | 6.98 | 0.97 |
| 4.5 | 0.66 | 0.10 | 16.5 | 7.39 | 0.41 |
| 5.0 | 0.76 | 0.10 | 17.0 | 7.79 | 0.40 |
| 5.5 | 0.88 | 0.12 | 17.5 | 8.01 | 0.22 |
| 6.0 | 1.00 | 0.12 | 18.0 | 8.22 | 0.21 |
| 6.5 | 1.12 | 0.12 | 18.5 | 8.36 | 0.14 |
| 7.0 | 1.24 | 0.12 | 19.0 | 8.50 | 0.14 |
| 7.5 | 1.40 | 0.16 | 19.5 | 8.62 | 0.12 |
| 8.0 | 1.56 | 0.16 | 20.0 | 8.74 | 0.12 |
| 8.5 | 1.72 | 0.16 | 20.5 | 8.84 | 0.10 |
| 9.0 | 1.89 | 0.17 | 21.0 | 8.94 | 0.10 |
| 9.5 | 2.06 | 0.17 | 21.5 | 9.04 | 0.10 |
| 10.0 | 2.23 | 0.17 | 22.0 | 9.14 | 0.10 |
| 10.5 | 2.40 | 0.17 | 22.5 | 9.23 | 0.09 |
| 11.0 | 2.57 | 0.17 | 23.0 | 9.32 | 0.09 |
| 11.5 | 2.75 | 0.19 | 23.5 | 9.41 | 0.09 |
| 12.0 | 2.95 | 0.19 | 24.0 | 9.50 | 0.09 |

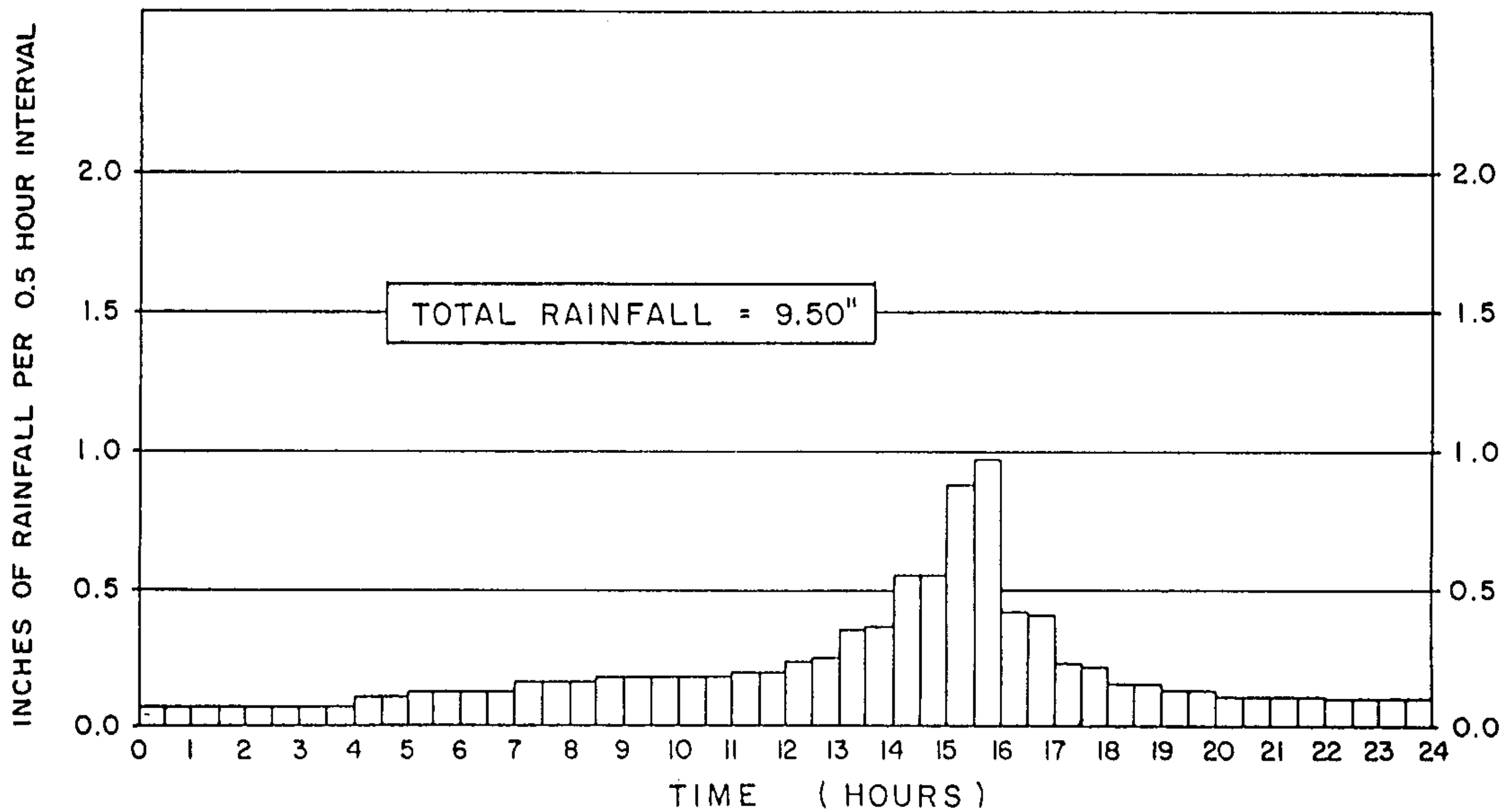



FIGURE 4-3

25 Year - 24 Hour Duration Rainfall - 30 Minute Increments

For the purposes of this planning study, average antecedent conditions (AMC II) have been used. The adoption of AMC II means that between 0.5 inches and 2.1 inches of rain had fallen over the previous 5 days. This would be representative of the rainfall that would be expected during the rainy season (May through September) during which the 25-year design storm would most likely occur.

## 4.3 TIDAL BOUNDARY

For the purposes of this planning study, the tidal conditions are assumed to be at mean-high tide. Statistical correlation studies done by the United States Geological Survey and the United States Army Corps of Engineers have shown that high tides do not necessarily occur during periods of higher rainfall. Additionally, the 25-year, 24-hour design storm will span the full diurnal tidal fluctuations. Therefore, the County has agreed that the mean-high tide, while representing a conservative scenario, can be considered a probable worst case. The value of the mean-high tide for the basin with in-depth hydraulic analyses will be given later in this report.

## 4.4 MODELING APPROACH

The master planning effort will rely on the use of sophisticated stormwater hydraulic and hydrologic modeling to define the flooding problem and determine the workable flood mitigation measures. To accomplish this task, two models are to be used. The first model, the Microcomputer Stormwater Simulation Model (MSSM) uses the basin hydrology as well as the channel hydraulics to simulate the runoff from a given rainfall event. In this way, rainfall hyetographs are converted to runoff hydrographs. A discussion of the model methodologies as well as the input requirements is given in Appendix A of this report.

The MSSM model input data for the two major basins is discussed in detail in the section dealing specifically with that basin. Assumptions are made at the outset which are then verified during the calibration process. The calibration process is the process whereby known rainfall is used as input to the model with the resulting hydrographs compared to known runoff hydrographs. The calibration process for the Sarasota County Stormwater Master Plan involves the use of rainfall and gaged streamflow data from the Phillippi Creek watershed. The results of the calibration efforts are also given in Appendix A of this report.

The MSSM model results provide outflow hydrographs for the basin based on existing and future design conditions. The peak discharges at the various locations along the stream is modeled to determine the actual flood heights and aerial extent. The HEC-2 Surface Water Profile model is used to model the backwater conditions that occur along the two creeks. This model, developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center, is generally accepted as state-of-the-art in backwater simulation. The HEC-2 model application is also discussed in Appendix A.

# 5.0 ALLIGATOR CREEK

# 5.0 ALLIGATOR CREEK

## 5.1 BASIN CHARACTERISTICS

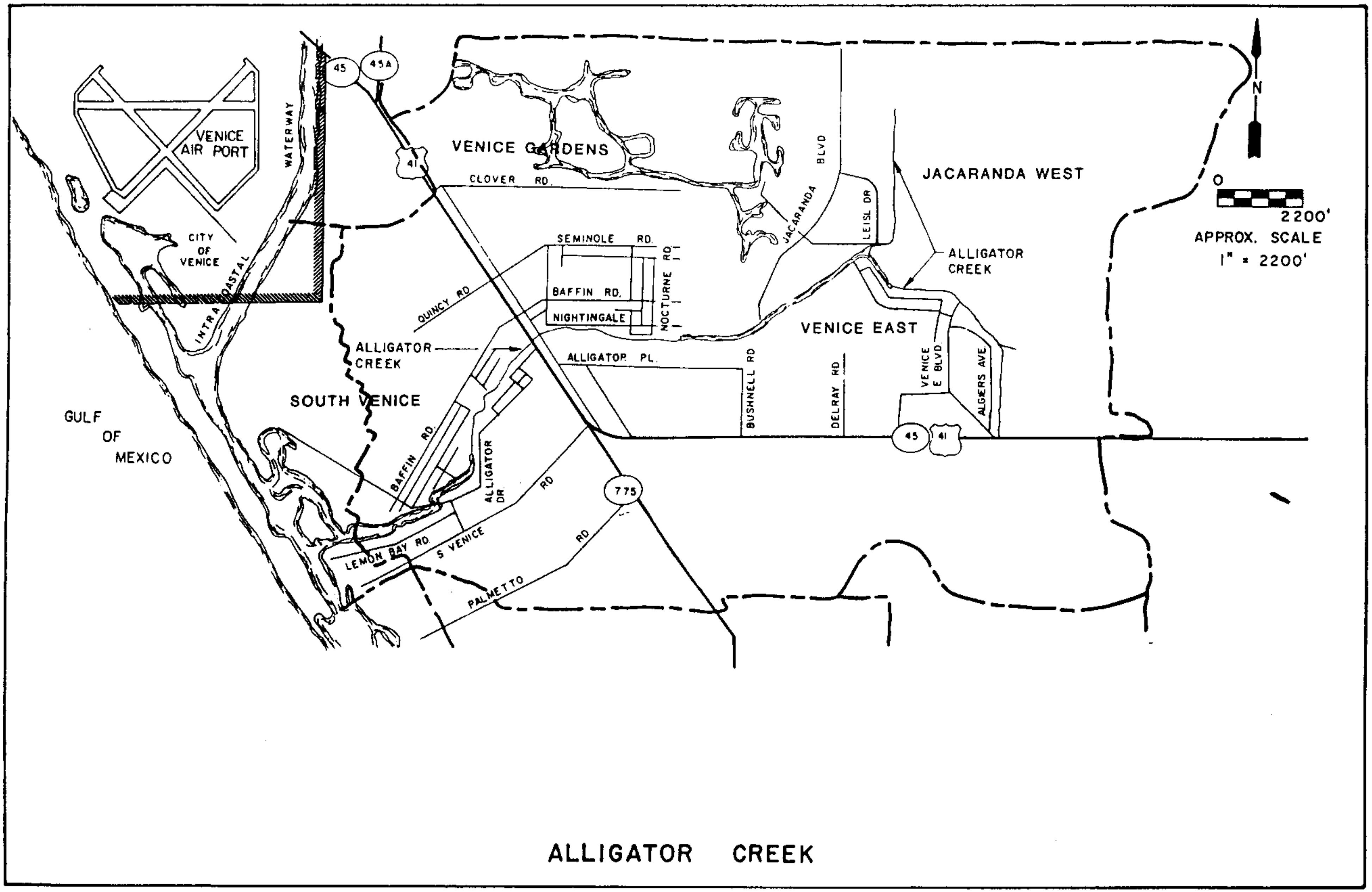
The Alligator Creek basin is located southeast of the City of Venice and covers an area of approximately 10 square miles (Figure 5-1). The basin consists mainly of natural low ridges which form the boundaries for the subbasins. The basin's northern boundary is along a man-made ridge caused by the construction of Center Road. Alligator Creek is bounded on the north by the Hatchett Creek basin and on the south by the Forked Creek basin. The eastern boundary of Alligator Creek is the Myakka River basin. The western boundary of the Creek is Lemon Bay.

Alligator Creek, in its natural state, is generally composed of gentle sloping topography. The typically low vertical slope gradients found at the outer reaches of the basin increase near the main Creek channel. Areas where natural drainage still exist exhibit poorly defined channels with the majority of runoff as sheet flow. This is particularly true of the eastern and southeastern areas with the exception of Venice East.

Elevations within the basin range from approximately 16 feet at the northeastern boundary to sea level closer to Lemon Bay. Much of the undeveloped area is dotted with small shallow depression areas with virtually no outlet. These areas act as small surface reservoirs in the form of marshes and ponds.

A portion of the Alligator Creek basin is still undeveloped. However, 70 percent of the basin is at some stage of development. Current development plans call for about 90 percent urbanization by the year 2000.

FIGURE 5-1




Alligator Creek Study Area

FIGURE 5-1

## 5.2 EXISTING DRAINAGE SYSTEMS

### 5.2.1 HISTORICAL DEVELOPMENT

#### Alligator Creek

The Alligator Creek basin, like most of the County, has been developed from west to east. As early as the late 1940s, land in what is now known as South Venice and Venice Gardens was being developed. Current developments include Jacaranda and Woodmere. A diversity in the ages of the various developments indicates that the developments occurred with differing stormwater drainage requirements. This diversity causes problems when determining the alternative stormwater management plans that will be studied.

#### South Venice

South Venice lies primarily in an area west of U.S. 41 and Englewood Road within the Alligator Creek basin, with an additional area east of U.S. 41. Lots within this subdivision were developed as early as the 1950s, with development continuing today. Many of the older homes, though constructed under the prevailing requirements at the time, would not meet current elevation and building code requirements. Thus, in many of the neighborhoods, you can find several older homes set at an elevation much lower than adjacent homes currently under construction.

The drainage from the South Venice area is carried via swales and channels to the various outfalls along the Creek. One of the major problems with this type of drainage conveyance is that the swales are converted to culverts and pipes to allow flow under driveways and, on occasion, the entire yard. In theory, this method is satisfactory for flow transfer, but in practice, these pipes and culverts become clogged with dirt and debris. This clogging causes the swales in front of the houses to act as mini-reservoirs, delaying both the peak and the timing of the runoff.

Venice East

Venice East is located south of Alligator Creek adjacent to Jacaranda West. This area has been developed in phases beginning 20 to 30 years ago. The final phase of construction occurred very recently. Generally, the runoff is directed by swales and inlets to a series of canals and a large detention pond. This detention pond was designed with two weir-type outflow structures, one at each end. Additionally, the Florida Department of Transportation has routed some of the U.S. 41 stormwater discharge through and around the Venice East area.

Venice Gardens

The Venice Gardens subdivision is located east of U.S. 41 and just north of South Venice. Stormwater management is accomplished through the use of an extensive lake system which serves the entire subdivision. The lake system has an outfall structure located at its southeastern boundary at what was once a connection to the Jacaranda West lake system. Drainage now occurs through a relief canal that runs southward between Jacaranda West and South Venice, which serves the dual purpose of an emergency discharge canal for the sewage treatment plant located just east of the canal.

Jacaranda West

The most recently developed subdivision within the Alligator Creek basin is Jacaranda West, developed under the Planned Unit Development (PUD) concept. The stormwater system was designed in accordance with the requirements that the post-development runoff be less than or equal to the pre-development runoff for the 25-year rainfall. This is accomplished through the use of curbs/gutters and inlets draining to an extensive system of lakes and water control structures.

New Developments

There are several new developments in the area which have either begun construction or are in the design phases. Immediately east of the Jacaranda West PUD are Heron Lakes and Willow Spring. These subdivisions will border Alligator Creek and use curbs and inlets to a lake system for drainage. This lake system will outfall to the wetland environment which surrounds the Creek via overflow berms. Additionally, a new subdivision to the south of the Creek, opposite the South Venice subdivision, has been planned. This subdivision's drainage is being developed by the firm responsible for the Jacaranda West PUD, and it is envisioned that its drainage system will be similar in nature.

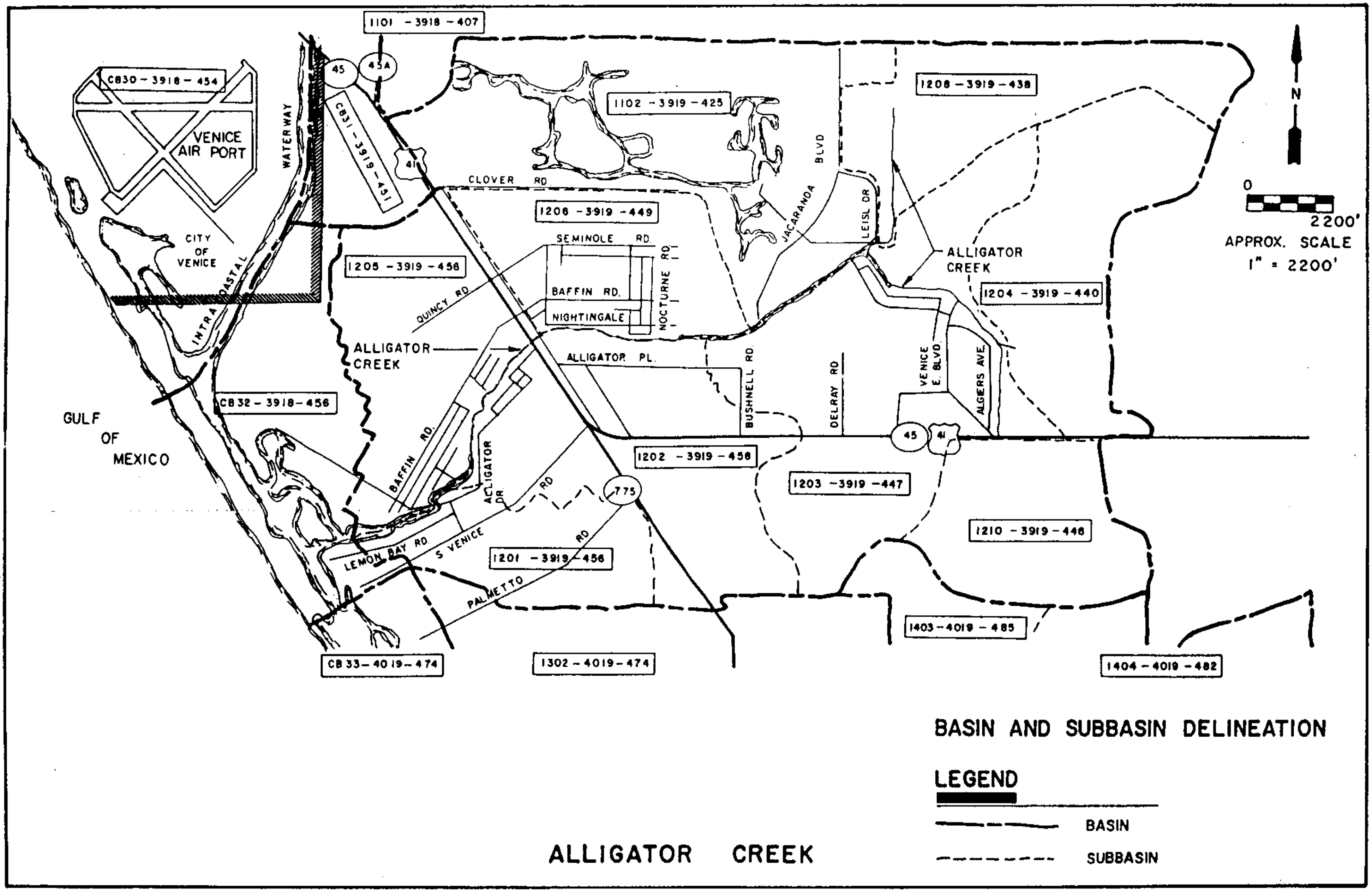
5.2.2 SUBBASIN DRAINAGE

Runoff from the various subbasins within the Alligator Creek basin (Figure 5-2) flow through various natural and man-made conveyance systems to the main channel of the Creek. A brief description of the model flow paths within each subbasin is given in Table 5-1 and shown in Figure 5-3. The subbasin numbering system between Figures 5-2 and 5-3, vary; as those in Figure 5-3 are those actually input into the model, while those given in Figure 5-2 reflect the previous basin and subbasin delineation report numbering system. Thus, subbasin number 12100 in Figure 5-3 equates to that subbasin number 1201 in Figure 5-2.

5.2.3 SYSTEM/FACILITY INVENTORY

A survey of the main Alligator Creek channel, starting west of the Shamrock Road bridge and running upstream to Jacaranda (area of in-depth study), was conducted as part of this study. The primary focus was to obtain stream channel cross-section data to be used during the modeling effort, which is discussed in a later section. Additionally, the main stormwater control and conveyance structures throughout the basin were identified. Subsequent field and historical record investigations have resulted in the compilation of a major facility inventory (Table 5-2) and a related structure location map (Figure 5-4).

FIGURE 5-2




Alligator Creek Subbasin Delineation

FIGURE 5-2

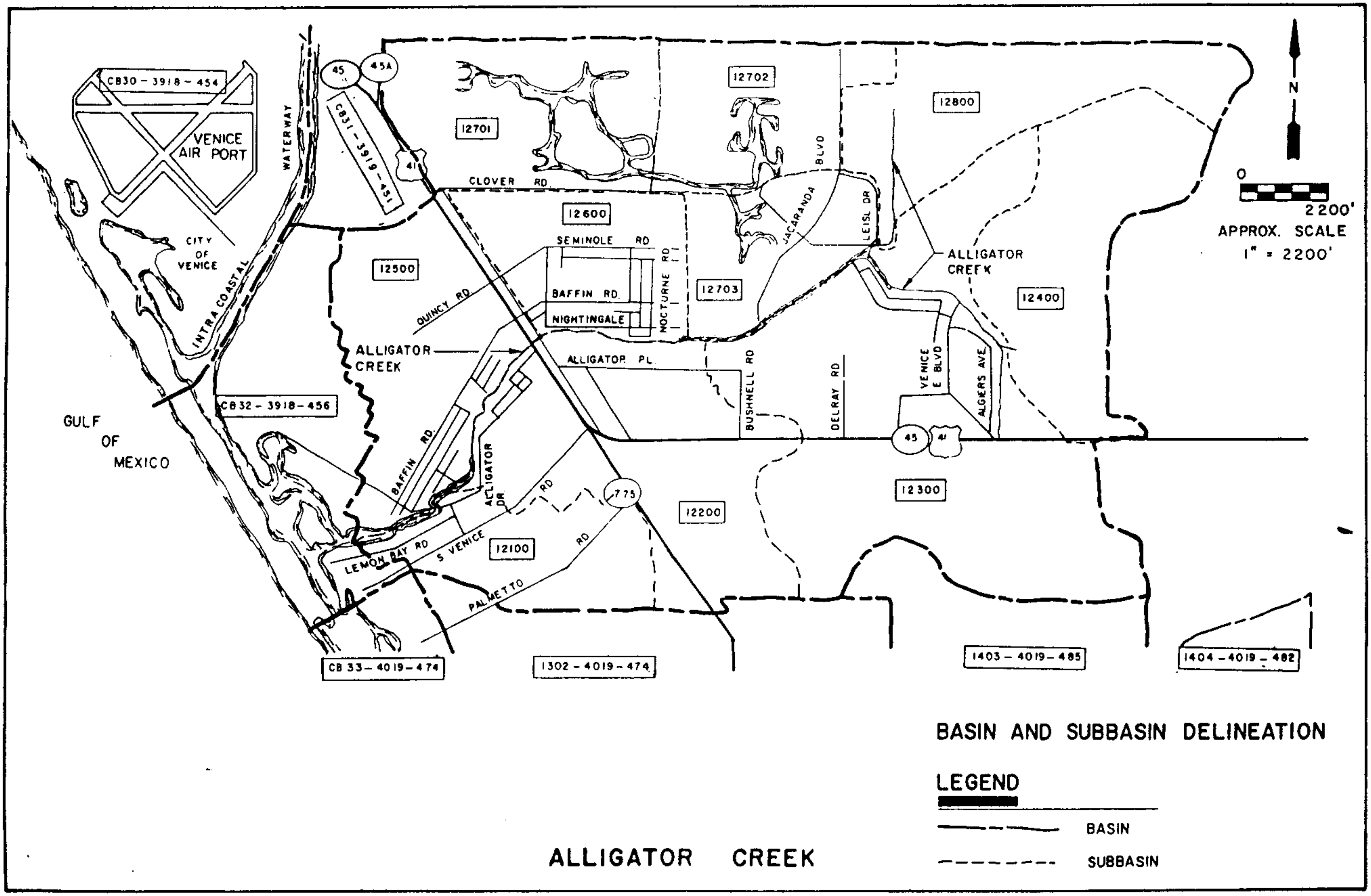
TABLE 5-1

ALLIGATOR CREEK SUBBASIN DRAINAGE DESCRIPTION

| Subbasin No. | Description |
|---|---|
| 12100 | This basin generally drains north and west first by man-made channels connecting two lakes located along Marlin Road and then along a natural meandering flow path to an outfall just east of the Shamrock Drive Bridge. |
| 12200 | This subbasin is split by U.S. 41 and S.R. 775. The runoff from the portion west of U.S. 41 generally flows along man-made channels from the lake located at Fayson Road and South Venice Blvd., along Fayson Road to an outfall west of U.S 41. The area east of U.S 41 flows through various man-made ditches north to its outfall. |
| 12300 | Subbasin 12300 is largely undeveloped with Venice East being its major subdivision. Flow is generally northward through man-made ditches and channels. Venice East and the area just south drains through a river of canals to a retention pond located to the south of Alligator Creek. |
| 12400 | This subbasin is primarily undeveloped and lies between Plantation and Venice East. Flow is primarily through natural drainage ways and sheet flow to the head of the creek. |
| 12500 | This subbasin generally encompases the area north of the creek and west of U.S. 41. The primary drainage path is through a man-made channel southward along Siesta Drive. |

TABLE 5-1
(Continued)

| Subbasin No. | Description |
|---|---|
| 12600 | This subbasin drains the area that is known as South Venice Gardens. Stormwater generally flows eastward through a service of lakes or southward to the creek. The eastward outfall is into a man-made canal on the eastern side of the property. |
| 12700 | For the purposes of modeling the extensive lake system, this subbasin has been further subdivided into subbasins 12701, 12702, and 12703. |
| 12701 | This subbasin encompasses Venice Gardens and drains through a series of interconnected lakes to a weir outfall structure and then to the canal to the east of South Venice Gardens. |
| 12702 | This subbasin covers the area known as Vivienca and is drained by a series of lakes to the McNeary Water Way control structure. Flow over this weir flows through Subbasin 12703. |
| 12703 | Subbasin 12703 is comprised of the area known as Jacaranda West. Again, a series of lakes route stormwater flow to an outfall in Woodlake directly to the creek. |
| 12800 | This subbasin is in the process of being developed as Heron Lakes and other subdivisions. Plans call for a series of interconnected lakes which outflow by overbank spillways to Alligator Creek. |




Alligator Creek Model Flow Path

FIGURE 5-3

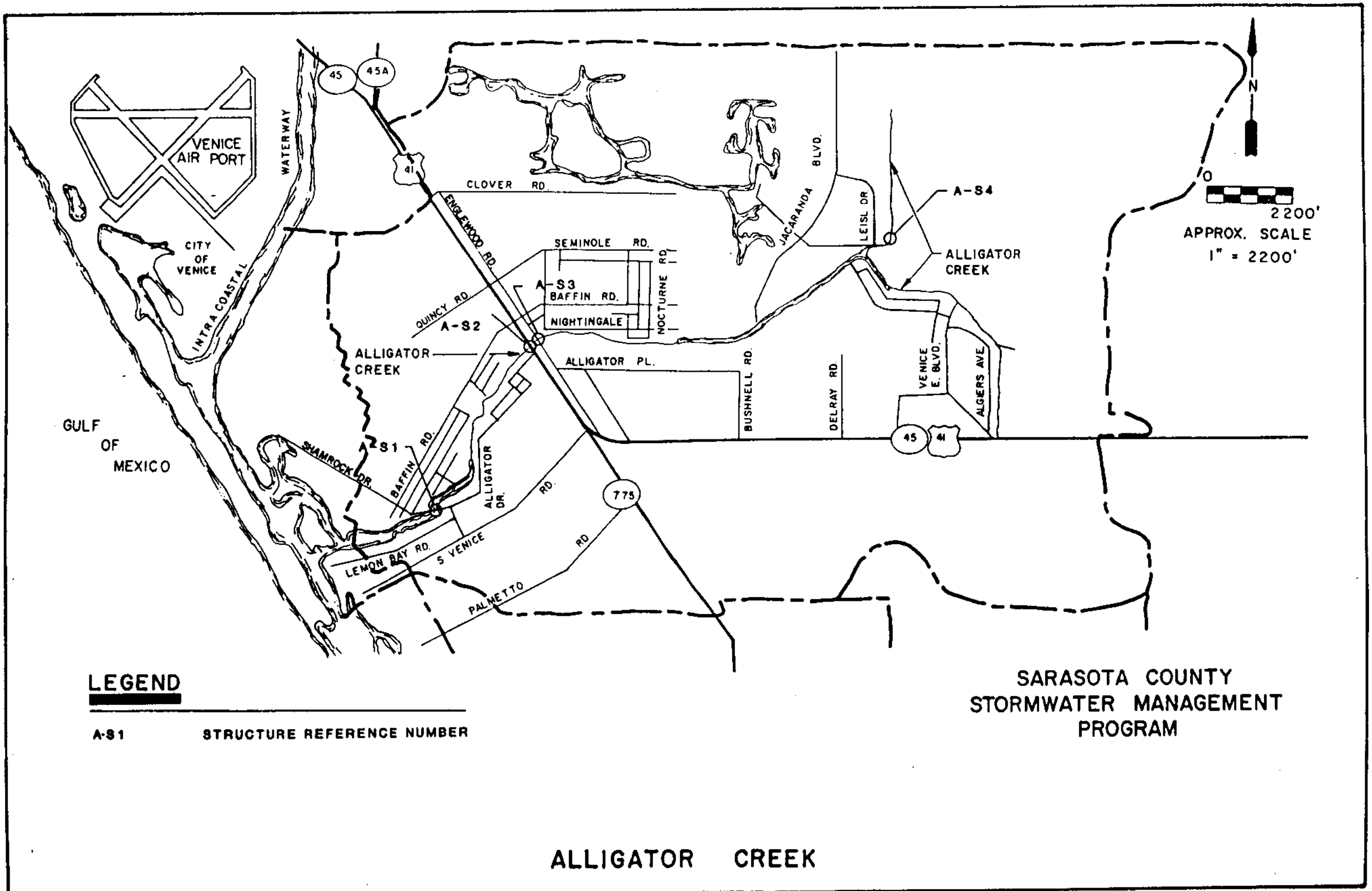
FIGURE 5-3

TABLE 5-2

ALLIGATOR CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1201 | A-1 | Shamrock Drive and W.N.W. of Lemon Bay Drive Concrete Drive - 45' x 25' - 9"; ± 11' - 4 1/2"* Vertical Clearance |
| 1202/1205 | A-2 | U.S. 41 and ± 600' northwest of Lemon Bay Drive 2 - Concrete Bridge - 10' x 36' each, ± 13'* Vertical Clearance |
| 1202/1206 | A-3 | Just south of Geneva Road crossing Alligator Creek Pipeline Crossing - 6" and 12" D.I.P.; ± 13' Vertical Clearance |
| 1203/1208 | A-4 | Just north of the east end of Skylar Drive South Gate Weir Structure |

*Vertical Clearance - from botton of girder to stream floor.




Alligator Creek Structure Location Map

FIGURE 5-4

FIGURE 5-4

## 5.3 STORMWATER RUNOFF MODEL - MSSM

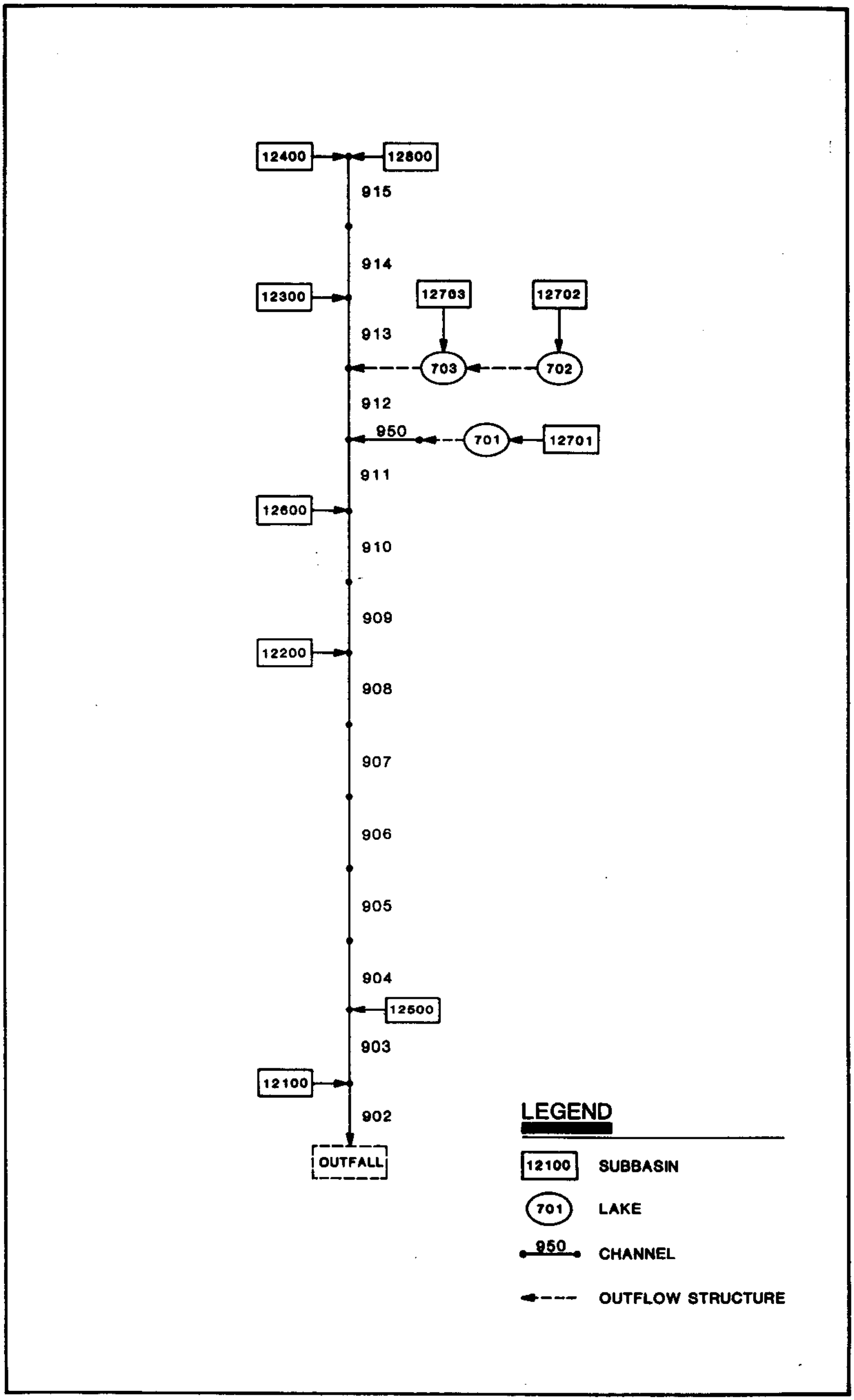
The MSSM model used to quantify the stormwater runoff within the basin is discussed in Appendix A of this report. The following section details the model set-up and input selection process.

### 5.3.1 MODEL NETWORK

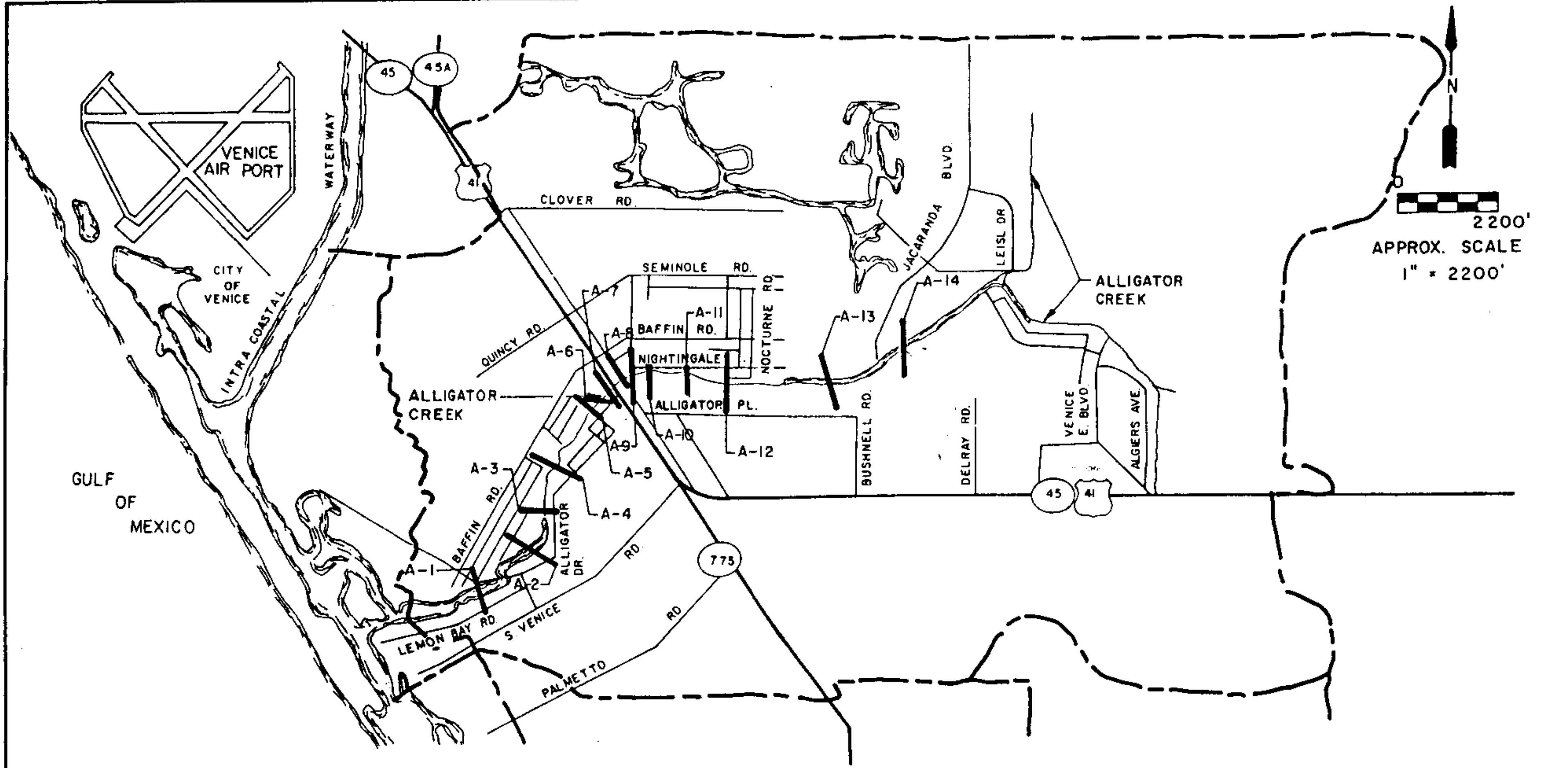
The Alligator Creek basin is represented by a system of subbasins, lakes, control structures, and channels. These are represented in the model by defining their stormwater control/conveyance properties. A model representation is constructed which ties the facilities together in a logical manner. This model representation is depicted in Figure 5-5 and shows the subbasins interconnectivity with the lakes and channels. The subbasins were delineated during the previous Sarasota County Basin and Subbasin Delineation project (CDM, 1983) and modified based on recent aerial photography and survey data. Subbasin 12700 was subdivided into three subbasins, 701, 702, and 703, for the purposes of modeling, to account for the three separate and distinct lake systems which serve them.

Fifteen channel cross-sections resulting from the survey were used to model the Alligator Creek conveyance system. These cross-sections are located in Figure 5-6 and graphically depicted in Appendix B.

Channel reaches 12902 through 12914 are idealized as double-trapezoidal channels based on the downstream cross-section. Channel reach 12915 represents the existing large flat wetland area at the head of the Creek, an area identified as waters of the State. Reach 12950 represents the canal to the east of Venice Gardens west of Jacaranda which conveys stormwater flow from the lake system in Subbasin 12701 to the main conveyance channel.




**Alligator Creek MSSM Model Network FIGURE 5-5**




## CROSS SECTIONS LOCATION MAP

**LEGEND**

A-1 CROSS SECTION REFERENCE NUMBER

## ALLIGATOR CREEK

Alligator Creek Cross-Section Location Map

The MSSM model RUNOFF Program computes the volume of stormwater runoff based on the input parameters for overland flow and the 25-year, 24-hour design rainfall event. The peak flows obtained by this simulation are used as input to the HEC-2 surface profile model. HEC-2 computes the surface profiles along the stream using the outfall condition of mean high tide plus 2.50 feet NGVD (National Geodetic Vertical Datum).

### 5.3.2 PARAMETER SELECTION

The use of the MSSM model requires initial model input parameter selection. Basically, this first attempt at determining parameter values creates a starting point from which adjustments can be made during the calibration process. The calibration procedure is discussed in Appendix A of this report. The remainder of this section is devoted to the parameter selection for the Alligator Creek simulation.

#### Subbasin Discretization

Table 5-3 lists the subbasins in Alligator Creek and the parameters that describe their existing condition hydrology. MSSM represents each subbasin as a set of three rectangular planes describing the pervious area, directly connected impervious areas with detention storage, and directly connected impervious areas without detention storage (Figure 5-7). Each of these planes has an identical slope and width, resulting in the same surface conveyance rate throughout the subbasin.

#### Infiltration

MSSM uses a modified Horton's equation to simulate infiltration within the pervious areas of each subbasin. Analysis of soils in the previous Basin and Subbasin Delineation Study found most of the basins to be composed of hydrologic Class C soils. However, some of the basins do exhibit Class A and B soils.

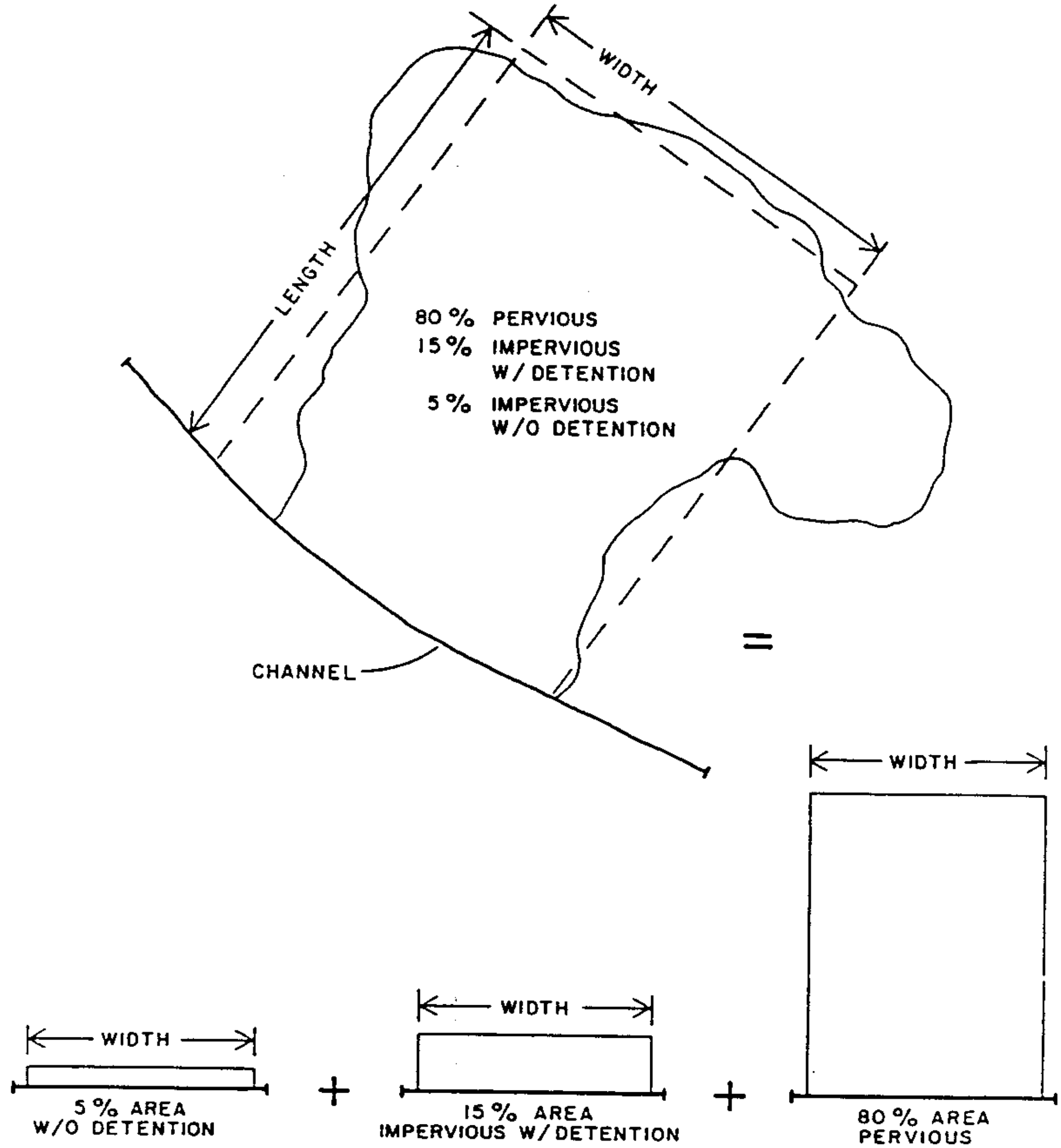
TABLE 5-3
ALLIGATOR CREEK
EXISTING SUBBASIN INPUT DATA

*****ENGLISH UNITS*****

| INT NUM | SUBAREA NUMBER | CHANNEL NUMBER | LENGTH (FT) | AREA (ACRE) | SLOPE (FT/FT) | PCT IMP | MANNING N IMP | MANNING N PERV | DEP STOR (IN) IMP | DEP STOR (IN) PERV | INFILTRATION RATE (IN/HR) MAX | INFILTRATION RATE (IN/HR) MIN | MAXIMUM INFL(IN) | INFIL DECAY (1/HR) | RETENTION STORAGE (AC-FT) | HYET NO |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 12100 | 12902 | 6900. | 383. | .00145 | 12. | .017 | .320 | .500 | .100 | 2.0 | .67 | 3.8 | 2.0 | 0. | 3000 |
| 2 | 12200 | 12908 | 5000. | 844. | .00180 | 20. | .017 | .320 | .400 | .300 | 2.1 | .18 | 2.2 | 2.0 | 1. | 3000 |
| 3 | 12300 | 12913 | 8500. | 1703. | .00143 | 11. | .017 | .320 | .200 | .500 | 2.4 | .20 | 2.2 | 2.0 | 1. | 3000 |
| 4 | 12400 | 12915 | 4000. | 586. | .00200 | 1. | .017 | .320 | .700 | .200 | 2.3 | .25 | 2.2 | 2.0 | 1. | 3000 |
| 5 | 12500 | 12903 | 7500. | 693. | .00133 | 12. | .017 | .320 | .500 | .100 | 2.0 | .33 | 3.0 | 2.0 | 0. | 3000 |
| 6 | 12600 | 12910 | 4500. | 566. | .00220 | 13. | .017 | .320 | .500 | .100 | 2.1 | .17 | 2.2 | 2.0 | 0. | 3000 |
| 7 | 12701 | 12701 | 3500. | 350. | .00029 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 8 | 12702 | 12702 | 5000. | 350. | .00100 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 9 | 12703 | 12703 | 6000. | 270. | .00070 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 10 | 12800 | 12915 | 2500. | 570. | .00125 | 1. | .017 | .320 | .700 | .200 | 2.0 | .25 | 2.2 | 2.0 | 1. | 3000 |

OTOTAL NUMBER OF SUBCATCHMENTS, 10

OTOTAL TRIBUTARY AREA (AC), 6315.00




Typical Watershed Subarea

**FIGURE 5-7**

Figure 5-8 shows the change in infiltration rates over time, as described by Horton's equation. In addition, it shows that the soil has a set maximum amount of moisture which can infiltrate before all voids in the soil are filled. Additionally, part of the infiltration capacity of the soil contains antecedent soil moisture. Table 5-4 lists infiltration parameters for each soil type and various antecedent soil conditions as used and verified for this report during calibration procedures.

It was assumed that the design rainfall event would occur when the average antecedent moisture condition existed. To account for moisture already in the soil, the initial infiltration rate, $f_o$, was reduced, according to Horton's equation, to the value f on Figure 5-8. MSSM also permits the entry of a total maximum infiltration volume. This was set to the available soil storage capacity less the antecedent soil moisture.

A further reduction in infiltration was performed to account for impervious areas which drain onto pervious areas, e.g., roof drains onto lawns. RUNOFF uses an effective pervious area which lumps the not directly connected impervious area with the pervious area. Rainfall subject to infiltration falls over the entire effective pervious area, but can only infiltrate into the truly pervious area. In order to limit predicted infiltration to truly pervious areas, the infiltration rates were reduced by the ratio of not directly connected impervious area to effective pervious area.

Depression Storage

Typically, 0.1 to 0.3 inches of runoff can collect in depression storage within watersheds, such as those in the Alligator basin. During simulation, this storage is filled before any runoff occurs. Additionally, several subbasins within these basins contain roadside swales. Water ponds in those swales since they have little infiltration capacity and blocked driveway drains. To simulate the water trapped in these swales, depression storage on impervious areas was increased, in some cases as high as 0.7 inches. Specific values for the various subbasins are determined during model calibration (Appendix A).

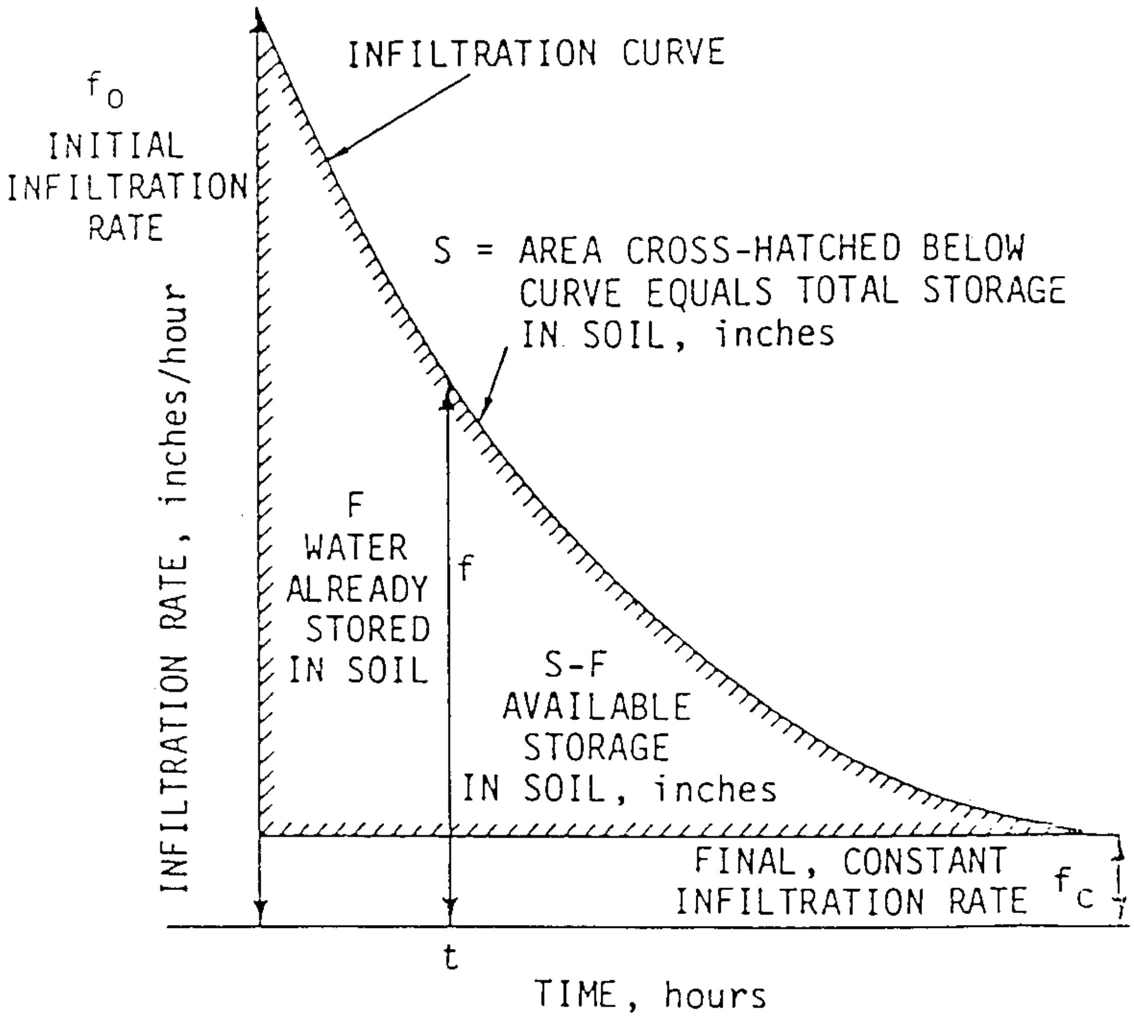
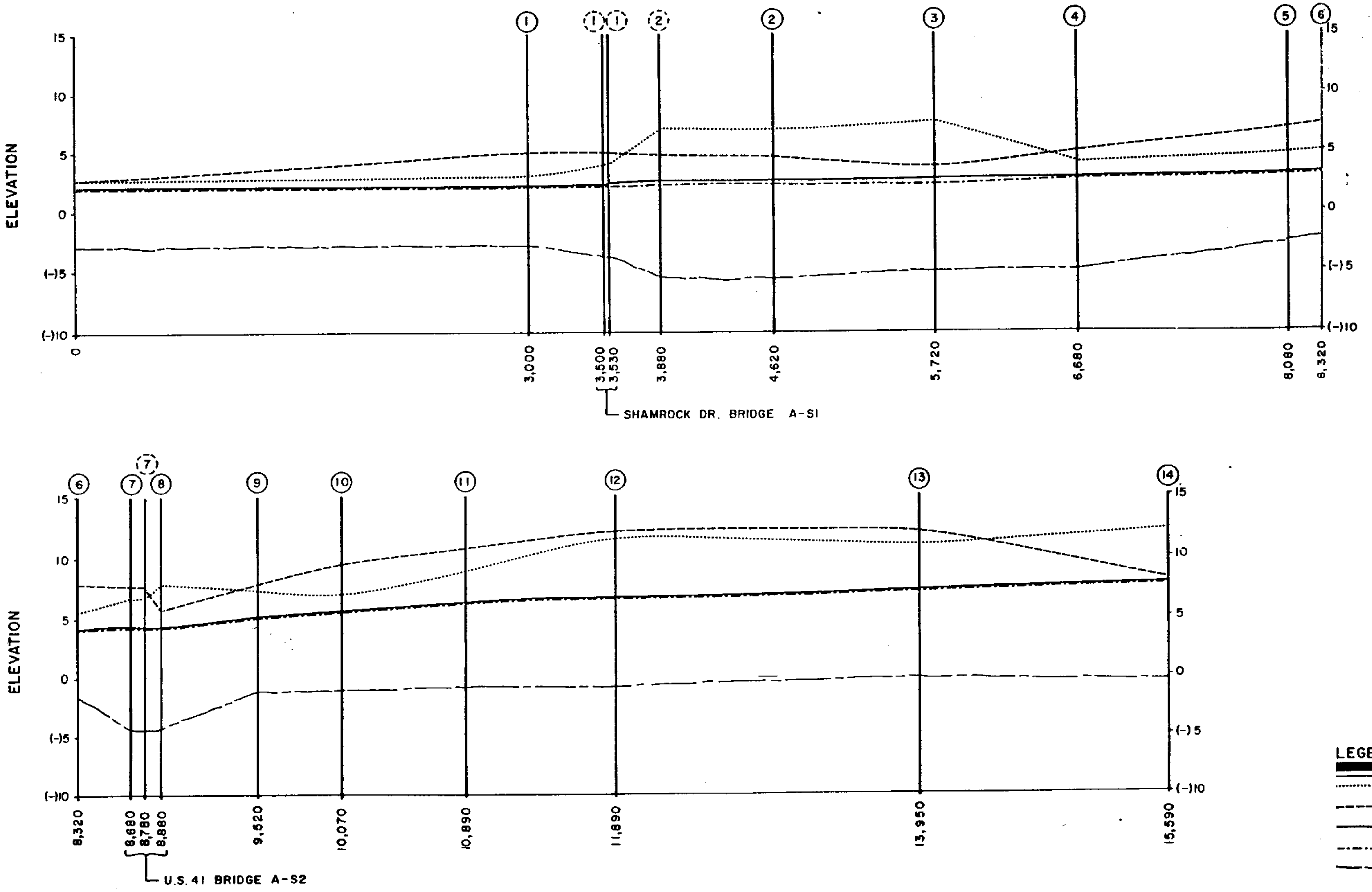



Diagram of Infiltration Curve and Infiltration Rates
as Related to Storage in Soil (CDM, 1983)

**FIGURE 5-8**




## LEGEND

- ··········· RIGHT BANK
- – – – – LEFT BANK
- ——— FUTURE LAND USE WITH 25 YEAR FLOOD
- —·—·— FUTURE 25 YEAR FLOOD WITH PROJECTED IMPROVEMENTS
- —— – —— FLOW LINE (CHANNEL BOTTOM)

## NOTE

1. ALL ELEVATIONS ARE REFERRED TO NGVD 1929 DATUM OF THE NATIONAL OCEAN SURVEY.

# ALLIGATOR CREEK HEC-2 MODEL

0 400'

SCALE:
HORIZ.: 1" = 400'
VERT.: 1" = 5'

FIGURE 5-11

Alligator Creek HEC-2 Results - Future And Future With Alternatives

TABLE 5-4

SELECTED INFILTRATION PARAMETERS

AVERAGE DRY CONDITIONS

| Soil Classification | Maximum Rate (in/hr) | Minimum Rate (in/hr) | Total | Decay Rate |
|---|---|---|---|---|
| A | 3.9 - 4.1 | 0.96 | 3.8 | 2 |
| B | 3.1 - 3.3 | 0.48 | 3.0 | 2 |
| C | 2.0 - 2.1 | 0.23 | 2.2 | 2 |
| D | 1.10 | 0.14 | 1.6 | 2 |

AVERAGE WET CONDITIONS

| Soil Classification | Maximum Rate (in/hr) | Minimum Rate (in/hr) | Total | Decay Rate |
|---|---|---|---|---|
| A | 2.7 - 2.9 | 0.96 | 1.7 | 2 |
| B | 1.9 - 2.1 | 0.48 | 1.3 | 2 |
| C | 1.2 - 1.35 | 0.23 | 1.0 | 2 |
| D | 0.6 - 0.75 | 0.14 | 0.7 | 2 |

## Surface Roughness

Most impervious areas within the Alligator Creek basin are asphalt pavement, with an appropriate Manning's roughness coefficient of 0.017. Pervious areas are usually lawns and fields whose usual Manning's roughness coefficients are 0.32.

## Channel Geometry

RUNOFF simulates channels using a double-trapezoidal cross-section, with the lower trapezoid representing the main channel and the upper trapezoid representing any overbank channel or flood plain. These trapezoidal cross-sections were set to best represent each natural channel cross-section, preserving the channel cross-sectional area and wetted perimeter. Table 5-5 lists each channel and its parameters, while Table 5-6 shows the overbank channels used for Alligator Creek.

## Channel Roughness

Field inspection of Alligator Creek found most sections of the channel and overbank to be heavily overgrown with weeds, trees, etc. The main channel is well developed but small and meandering. A Manning's roughness coefficient of 0.03 was assigned to these areas. The overbank area is not well defined and was assigned a Manning's roughness coefficient of 0.075 downstream of the Route 41 bridge, and 0.09 upstream where the overgrowth is denser. The two upstream channel sections, 12914 and 12915, are essentially wetlands areas which convey flow very slowly. To simulate this, Manning's roughness was set at 0.1.

## Lakes

Three major lake systems within Alligator Creek were simulated. Table 5-7 shows the major characteristics of each lake. The outlet of each lake is a rectangular weir. MSSM computes an elevation-discharge curve for each lake based on the lake topography and outlet control.

# TABLE 5-5

## ALLIGATOR CREEK CHANNEL INPUT DATA

* * * * * E N G L I S H   U N I T S * * * * *

| | INT NUM | CHAN NUM | CHAN CONN | WIDTH (FT) | LENGTH (FT) | SLOPE (FT/FT) | SIDE SLOPES LEFT | SIDE SLOPES RIGHT | MANNING N | DEPTH (FT) | V MAX. (FPS) | Q MAX. (CFS) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 12902 | 12901 | 60.00 | 1075. | .00046 | 5.1 | 4.3 | .030 | 8.00 | 3.39 | 2647.172 |
| + | 2 | 12903 + | 12902 | 58.00 | 960. | .00010 | 8.5 | 4.5 | .030 | 6.70 | 1.38 | 939.052 |
| + | 3 | 12904 + | 12903 | 56.00 | 1400. | .00143 | 1.7 | 7.7 | .030 | 7.00 | 5.49 | 3415.167 |
| + | 4 | 12905 + | 12904 | 72.00 | 230. | .00380 | 1.6 | 4.3 | .040 | 6.75 | 7.06 | 4384.807 |
| + | 5 | 12906 + | 12905 | 90.00 | 350. | .00029 | 2.4 | 6.3 | .040 | 6.25 | 1.85 | 1356.034 |
| + | 6 | 12907 + | 12906 | 40.00 | 600. | .00026 | 1.0 | .3 | .040 | 5.75 | 1.67 | 419.434 |
| + | 7 | 12908 + | 12907 | 37.50 | 680. | .00074 | 2.8 | 1.8 | .040 | 6.00 | 2.77 | 855.975 |
| + | 8 | 12909 + | 12908 | 24.00 | 550. | .00091 | 3.2 | 2.0 | .040 | 7.50 | 3.25 | 1061.876 |
| + | 9 | 12910 + | 12909 | 28.00 | 820. | .00037 | 2.2 | 12.5 | .040 | 4.50 | 1.45 | 398.164 |
| + | 10 | 12911 + | 12910 | 20.00 | 970. | .00103 | 2.2 | 2.7 | .060 | 8.25 | 2.39 | 788.764 |
| + | 11 | 12912 + | 12911 | 23.00 | 2050. | .00034 | 1.9 | 2.0 | .060 | 8.00 | 1.39 | 428.873 |
| + | 12 | 12913 + | 12912 | 30.00 | 1650. | .00042 | 2.1 | 1.6 | .060 | 7.25 | 1.52 | 478.101 |
| + | 13 | 12914 + | 12913 | 30.00 | 2500. | .00040 | 20.0 | 20.0 | .100 | 7.50 | .76 | 1025.496 |
| | 14 | 12915 | 12914 | 400.00 | 9000. | .00030 | 267.0 | 267.0 | .100 | 3.00 | .38 | 1372.068 |
| | 15 | 12950 | 12911 | 12.00 | 4000. | .00040 | 1.9 | 1.9 | .040 | 4.50 | 1.53 | 140.014 |

OTOTAL NUMBER OF CHANNELS, 15

0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS

0*** NOTE *** DATA FOR DOUBLE TRAPEZOIDAL CHANNELS ARE FOR THE LOWER TRAPEZOID -
THE DATA FOR THE UPPER TRAPEZOID ARE GIVEN IN THE FOLLOWING OVERBANK TABLE

TABLE 5-6

ALLIGATOR CREEK OVERBANK CHANNEL DATA

1

0SUMMARY OF OVERBANK CHANNEL DATA FOR BASIN 12

| INT NUM | CHAN NUM | WIDTH (FT) | SIDE SLOPES LEFT | SIDE SLOPES RIGHT | MANNING N | DEPTH (FT) | V MAX. (FPS) | Q MAX. (CFS) |
|---|---|---|---|---|---|---|---|---|
| 1 | 12902 | 232.00 | 23.5 | 7.5 | .075 | 3.00 | 3.48 | 5630.57 |
| 2 | 12903 | 175.00 | 33.6 | 51.5 | .075 | 3.00 | 1.43 | 2272.28 |
| 3 | 12904 | 158.00 | 4.8 | 40.0 | .075 | 1.25 | 5.82 | 4973.43 |
| 4 | 12905 | 112.00 | 2.0 | 25.0 | .075 | 1.00 | 7.76 | 5790.73 |
| 5 | 12906 | 145.00 | 2.7 | 16.7 | .075 | 1.50 | 2.16 | 2094.05 |
| 6 | 12907 | 70.00 | 2.0 | 2.0 | .075 | 7.25 | 2.33 | 2012.22 |
| 7 | 12908 | 81.00 | 2.5 | 78.1 | .075 | 4.00 | 2.35 | 3003.92 |
| 8 | 12909 | 81.00 | 45.5 | 23.5 | .090 | 4.00 | 2.72 | 3269.47 |
| 9 | 12910 | 177.00 | 5.5 | 2.3 | .090 | 4.75 | 1.97 | 2374.97 |
| 10 | 12911 | 78.00 | 2.7 | 2.7 | .090 | 4.50 | 3.11 | 2288.33 |
| 11 | 12912 | 75.00 | 4.0 | 2.2 | .090 | 2.75 | 1.61 | 866.64 |
| 12 | 12913 | 57.00 | 14.1 | 2.0 | .100 | 2.75 | 1.80 | 958.45 |

0TOTAL NUMBER OF OVERBANK CHANNELS, 12
0***NOTE*** Q MAX. AND V MAX. ARE FOR OVERBANK + MATCHING LOWER CHANNEL
1

# TABLE 5-7

## ALLIGATOR CREEK LAKE INPUT DATA

1

OSUMMARY OF LAKE DATA FOR BASIN NO.12

| INT NUM | LAKE NUM | CHAN CONN | LAKE AREA (ACRE) | OUTLET CONTROL | LENGTH (FT) | INIT. DEPTH (FT) | INIT. STORAGE (ACRE-FT) | HYET |
|---|---|---|---|---|---|---|---|---|
| 16 | 12701 | 12950 | 89.00 | REC WEIR | 3500.0 | 11.0 | .0 | 3000 |
| 17 | 12702 | 12703 | 86.00 | REC WEIR | 3600.0 | 10.5 | .0 | 3000 |
| 18 | 12703 | 12912 | 128.0 | REC WEIR | 1800.0 | 7.9 | .0 | 3000 |

OTOTAL NUMBER OF LAKES, 3

1 WATERSHED DATA FOR BASIN NO. 12, BASIN NAME - ALLEGATOR CREEK

Calibration

The preferred sequence of events in the modeling effort would be to follow the initial parameter selection with the calibration of the basin. There is no gaged flow data currently available with which Alligator Creek can be calibrated directly. Therefore, for the purposes of accuracy, the calibration of Phillippi Creek, which will be discussed in Appendix A, has been used as a basis for refinement of the initial Alligator Creek parameters.

5.3.3 SIMULATION RESULTS

Existing Subbasin Runoff

Table 5-8 lists the peak runoff along the main channel within the Alligator Creek basin for the 25-year, 24-hour rainfall event. Peak runoff averages nearly 200 cfs/sq. mi. over the entire Alligator Creek basin. Typically, peak runoff has been found to range from 400 to 800 cfs/sq. mi. from small suburban areas near Tampa. The subbasins simulated in the current study are rather large compared with those in Tampa and include significant routing through local drainage systems. Thus, the predicted subbasin runoff in the Alligator Creek basin should exhibit greater peak attenuation and are reasonable (USGS - Water Resource Investigation 82-42, 1983). Individual subbasin outlfows hydrographs have been predicted and provided to the County under separate cover.

Future Subbasin Runoff

Table 5-9 lists the peak runoff from the various Alligator Creek subbasins for the 25-year, 24-hour rainfall event for the future land use conditions. The estimated percentage of imperviousness, based on the projected future land use, has been used as input. The subbasin input file for the basins, showing the changes made for the future conditions, is shown in Table 5-10.

TABLE 5-8

# ALLIGATOR CREEK EXISTING PEAK OUTFLOW

1

TIME IN HOURS
SUMMARY STATISTICS FOR CHANNELS/PIPES
===================================

0

| CHAN NUMBER | MAX. FLOW (CFS) | MAX. VELOCITY (FPS) | MAX. DEPTH (FT) | MAXIMUM COMPUTED FLOW (CFS) | MAXIMUM COMPUTED VELOCITY (FPS) | MAXIMUM COMPUTED DEPTH (FT) | TIME OF OCCURENCE HR. | MIN. | LENGTH OF SURCHARGE (MIN) | MAXIMUM SURCHARGE VOLUME (AC-FT) | RATIO OF MAX. TO DESIGN FLOW | RATIO OF MAX. DEPTH TO DESIGN DEPTH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12902 | 5630.6 | 3.5 | 11.00 | 1665.0 | 2.96 | 6.24 | 17 | 30 | .0 | .00000E+00 | .30 | .57 |
| + 12903 + | 2272.3 | 1.4 | 9.70 | 1572.3 | .63 | 8.28 | 17 | 30 | .0 | .00000E+00 | .69 | .85 |
| + 12904 + | 4973.4 | 5.8 | 8.25 | 1390.9 | 4.20 | 4.32 | 17 | 0 | .0 | .00000E+00 | .28 | .52 |
| + 12905 + | 5790.7 | 7.8 | 7.75 | 1388.8 | 4.81 | 3.48 | 17 | 0 | .0 | .00000E+00 | .24 | .45 |
| + 12906 + | 2094.1 | 2.2 | 7.75 | 1388.4 | 1.83 | 6.30 | 17 | 0 | .0 | .00000E+00 | .66 | .81 |
| + 12907 + | 2012.2 | 2.3 | 13.00 | 1373.0 | 1.49 | 10.59 | 17 | 0 | .0 | .00000E+00 | .68 | .81 |
| + 12908 + | 3003.9 | 2.4 | 10.00 | 1368.2 | 1.20 | 7.39 | 17 | 0 | .0 | .00000E+00 | .46 | .74 |
| + 12909 + | 3269.5 | 2.7 | 11.50 | 1041.5 | 3.24 | 7.42 | 24 | 30 | .0 | .00000E+00 | .32 | .65 |
| + 12910 + | 2375.0 | 2.0 | 9.25 | 1040.8 | .97 | 6.52 | 24 | 0 | .0 | .00000E+00 | .44 | .70 |
| + 12911 + | 2288.3 | 3.1 | 12.75 | 881.7 | 1.88 | 8.63 | 24 | 30 | .0 | .00000E+00 | .39 | .68 |
| + 12912 + | 866.6 | 1.6 | 10.75 | 792.9 | 1.17 | 10.35 | 24 | 30 | .0 | .00000E+00 | .91 | .96 |
| + 12913 + | 958.4 | 1.8 | 10.00 | 648.5 | 1.26 | 8.37 | 24 | 0 | .0 | .00000E+00 | .68 | .84 |
| + 12914 + | 1025.5 | .8 | 7.50 | 294.5 | .55 | 4.46 | 26 | 30 | .0 | .00000E+00 | .29 | .59 |
| 12915 | 1372.1 | .4 | 3.00 | 297.3 | .25 | 1.47 | 25 | 30 | .0 | .00000E+00 | .22 | .49 |
| 12950 | 140.0 | 1.5 | 4.50 | 89.7 | 1.35 | 3.57 | 25 | 0 | .0 | .00000E+00 | .64 | .79 |

0 TOTAL NUMBER OF CHANNELS/PIPES, 15

0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS

0*** NOTE *** SUMMARY DATA FOR DOUBLE TRAPEZOIDAL CHANNELS IS FOR THE ENTIRE CHANNEL - (IE. LOWER+UPPER CHANNELS)

0*** NOTE *** THE MAXIMUM DEPTHS ARE THE DEPTHS CALCULATED AT THE END OF THE TIME INTERVAL -
THE MAXIMUM FLOWS ARE THE AVERAGE FLOWS OVER THE TIME INTERVAL

TABLE 5-9

ALLIGATOR CREEK FUTURE PEAK OUTFLOW

1

SUMMARY STATISTICS FOR CHANNELS/PIPES

0

| CHAN NUMBER | MAX. FLOW (CFS) | MAX. VELOCITY (FPS) | MAX. DEPTH (FT) | MAXIMUM COMPUTED FLOW (CFS) | MAXIMUM COMPUTED VELOCITY (FPS) | MAXIMUM COMPUTED DEPTH (FT) | TIME OF OCCURENCE HR. | MIN. | LENGTH OF SURCHARGE (MIN) | MAXIMUM SURCHARGE VOLUME (AC-FT) | RATIO OF MAX. TO DESIGN FLOW | RATIO OF MAX. DEPTH TO DESIGN DEPTH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12902 | 5630.6 | 3.5 | 11.00 | 1669.2 | 2.96 | 6.25 | 17 | 30 | .0 | .00000E+00 | .30 | .57 |
| + 12903 + | 2272.3 | 1.4 | 9.70 | 1577.0 | .63 | 8.29 | 17 | 30 | .0 | .00000E+00 | .69 | .86 |
| + 12904 + | 4973.4 | 5.8 | 8.25 | 1392.8 | 4.20 | 4.32 | 17 | 0 | .0 | .00000E+00 | .28 | .52 |
| + 12905 + | 5790.7 | 7.8 | 7.75 | 1391.0 | 4.82 | 3.48 | 17 | 0 | .0 | .00000E+00 | .24 | .45 |
| + 12906 + | 2094.1 | 2.2 | 7.75 | 1390.7 | 1.83 | 6.31 | 17 | 0 | .0 | .00000E+00 | .66 | .81 |
| + 12907 + | 2012.2 | 2.3 | 13.00 | 1378.2 | 1.50 | 10.62 | 17 | 0 | .0 | .00000E+00 | .68 | .82 |
| + 12908 + | 3003.9 | 2.4 | 10.00 | 1374.0 | 1.20 | 7.41 | 17 | 0 | .0 | .00000E+00 | .46 | .74 |
| + 12909 + | 3218.2 | 2.7 | 11.50 | 1075.2 | 2.00 | 7.54 | 24 | 0 | .0 | .00000E+00 | .33 | .66 |
| + 12910 + | 2336.7 | 1.9 | 9.25 | 1075.4 | .92 | 6.62 | 24 | 0 | .0 | .00000E+00 | .46 | .72 |
| + 12911 + | 2273.1 | 3.1 | 12.75 | 912.2 | 1.83 | 8.74 | 24 | 30 | .0 | .00000E+00 | .40 | .69 |
| + 12912 + | 862.6 | 1.6 | 10.75 | 822.8 | 1.12 | 10.53 | 24 | 30 | .0 | .00000E+00 | .95 | .98 |
| + 12913 + | 762.1 | 1.7 | 9.00 | 679.8 | 1.24 | 8.54 | 24 | 0 | .0 | .00000E+00 | .89 | .95 |
| + 12914 + | 1025.5 | .8 | 7.50 | 311.6 | .56 | 4.56 | 25 | 30 | .0 | .00000E+00 | .30 | .61 |
| 12915 | 1372.1 | .4 | 3.00 | 314.8 | .26 | 1.51 | 24 | 30 | .0 | .00000E+00 | .23 | .50 |
| 12950 | 140.0 | 1.5 | 4.50 | 89.7 | 1.35 | 3.57 | 25 | 0 | .0 | .00000E+00 | .64 | .79 |

TOTAL NUMBER OF CHANNELS/PIPES, 15

0

0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS

0*** NOTE *** SUMMARY DATA FOR DOUBLE TRAPEZOIDAL CHANNELS IS FOR THE ENTIRE CHANNEL - (IE. LOWER+UPPER CHANNELS)

0*** NOTE *** THE MAXIMUM DEPTHS ARE THE DEPTHS CALCULATED AT THE END OF THE TIME INTERVAL -
THE MAXIMUM FLOWS ARE THE AVERAGE FLOWS OVER THE TIME INTERVAL

TABLE 5-10
ALLIGATOR CREEK
FUTURE SUBBASIN INPUT DATA

*****ENGLISH UNITS*****

| INT NUM | SUBAREA NUMBER | CHANNEL NUMBER | LENGTH (FT) | AREA (ACRE) | SLOPE (FT/FT) | PCT IMP | MANNING N IMP | MANNING N PERV | DEP STOR (IN) IMP | DEP STOR (IN) PERV | INFILTRATION RATE (IN/HR) MAX | INFILTRATION RATE (IN/HR) MIN | MAXIMUM INFL(IN) | INFIL DECAY (1/HR) | RETENTION STORAGE (AC-FT) | HYET NO |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 12100 | 12902 | 6900. | 383. | .00145 | 12. | .017 | .320 | .500 | .100 | 2.0 | .67 | 3.8 | 2.0 | 0. | 3000 |
| 2 | 12200 | 12908 | 5000. | 844. | .00180 | 20. | .017 | .320 | .400 | .300 | 2.1 | .18 | 2.2 | 2.0 | 30. | 3000 |
| 3 | 12300 | 12913 | 8500. | 1703. | .00143 | 11. | .017 | .320 | .200 | .500 | 2.4 | .20 | 2.2 | 2.0 | 35. | 3000 |
| 4 | 12400 | 12915 | 4000. | 586. | .00200 | 15. | .017 | .320 | .700 | .200 | 2.3 | .25 | 2.2 | 2.0 | 25. | 3000 |
| 5 | 12500 | 12903 | 7500. | 693. | .00133 | 12. | .017 | .320 | .500 | .100 | 2.0 | .33 | 3.0 | 2.0 | 0. | 3000 |
| 6 | 12600 | 12910 | 4500. | 566. | .00220 | 13. | .017 | .320 | .500 | .100 | 2.1 | .17 | 2.2 | 2.0 | 0. | 3000 |
| 7 | 12701 | 12701 | 3500. | 350. | .00029 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 8 | 12702 | 12702 | 5000. | 350. | .00100 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 9 | 12703 | 12703 | 6000. | 270. | .00070 | 12. | .017 | .320 | .100 | .100 | 2.2 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 10 | 12800 | 12915 | 2500. | 570. | .00125 | 15. | .017 | .320 | .700 | .200 | 2.0 | .25 | 2.2 | 2.0 | 24. | 3000 |

0TOTAL NUMBER OF SUBCATCHMENTS, 10

0TOTAL TRIBUTARY AREA (AC), 6315.00

The lake systems that have been envisioned for the ongoing and future development within the creek have been accounted for by using a basin storage factor.

The basin storage factor selected for use within the basins was derived through investigations of two subdivisions that currently function in accordance with the FDER regulations. This calls for the storage and filtration of the runoff from the first inch of rainfall or, the first one-half inch of runoff. Additionally, the County regulations are interpreted to require the detention of the difference between the pre- and post-development peak runoff for the design storm Using outflow knowledge of both the Jacaranda subdivision in Alligator Creek and the Meadows subdivision in Phillippi Creek, correlation between peak discharge and storage within the basin was found. The data indicates that generally, the post-development basin runoff for a properly designed system can be approximated by removing the first inch of rainfall from the watershed. Therefore, for future development, it was assumed that the areas developed would store approximately the initial one-inch of rainfall. The basin storage factors for both Alligator and Phillippi Creeks have been selected to account for this storage.

Interestingly, the peak runoff relating to future development is almost equal to the runoff for existing conditions individual subbasin runoff hydrographs were provided to the County under separate cover. The peak runoff amounts at the mouth of the creek are virtually identical. This can be explained by the FDER storage requirement which removes the first portion of rainfall and results in an attenuation of the peak downstream. Generally, the peak outflows average about 200 cfs/sq. mi. over the entire basin. While the peak flows are generally equal, the total volume of flow out of the basin has increased. This increase in volume produces no further flooding problems within the basin. Lemon Bay has recently been reclassified to an outstanding water of the State, an increase in total volume may not be permitted. The County will have to require and ensure that peak and total flows are consistent with existing conditions to the extent that they will cause no degradation of water quality.

## 5.4 BACKWATER ANALYSIS

In order to evaluate the potential level of flooding in Alligator Creek during the design storm event, a hydraulic (backwater) model was developed. The peak flows generated in the hydrologic analysis were input to the model to simulate the surface water profiles. The backwater profiles were then compared to the carrying capacity of the creek and were used to analyze the flooding consequences within the basin. The data, methodologies, and assumptions used to develop the hydraulic model, and the results of the hydraulic analysis are described in the following sections.

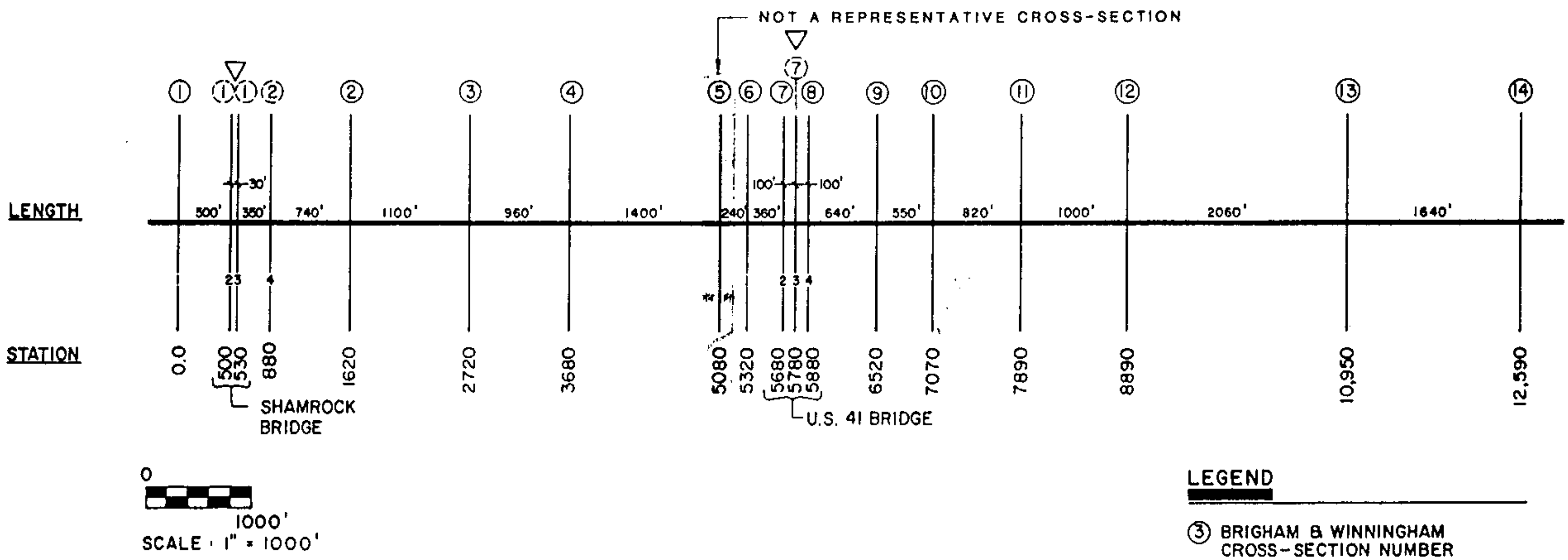
### 5.4.1 MODEL DEVELOPMENT

The HEC-2 model of Alligator Creek consisted of an approximate 4.5 mile stretch from below cross-section #1 of the Brigham and Winningham field survey (approximately the middle of Lemon Bay) upstream through cross-section #14 (Venice East subdivision) to the head of the stream. A schematic diagram of the modeled system is given in Figure 5-9.

#### Cross-Sections

The basic input data required to develop the Alligator Creek system model includes the cross-section data for the channels, and other more detailed data which describe the two bridge crossings. The primary source of data was the 1985 Brigham and Winningham field survey completed as part of this project (Figure 5-6). The survey produced stations and elevations for 14 cross-sections along Alligator Creek (Appendix B). Detailed drawings of the Shamrock Drive and Tamiami Trail (U.S. 41) bridge crossings were developed by CDM. The 14 cross-sections were used to define the channel in the HEC-2 model with the following exceptions. First, cross-section #5 was omitted from the model because the cross-section transect was not oriented perpendicular to flow conveyance contours. Second, the northern side of cross-section #11 represents a drainage ditch and is not representative of the average geometry between cross-sections #10 and #12. The northern bank of cross-section #11 was modified to better represent the channel by

FIGURE 5-9




# ALLIGATOR CREEK HEC-2
# MODEL CROSS-SECTION LOCATIONS

LEGEND

(3) BRIGHAM & WINNINGHAM CROSS-SECTION NUMBER

(5) REPEATED SECTION FOR NEEDED DETAILED DEFINITION OF SYSTEM

BRIDGE CARD NUMBER

△ STRUCTURE DETAIL PLAN & PROFILE

Alligator Creek HEC-2 Model Representation assuming a transition between cross-sections #10 and #12. Third, the left and right overbanks of all surveyed cross-sections were extended to accommodate expected water surface elevations. The extensions are based on ground elevation contours from the 1976 flood study of Alligator Creek (Smalley, Welford, and Nalven, 1976). Cross-sections downstream of surveyed cross-section #4 were extended to an elevation of 12 feet NGVD. The remaining cross-sections were extended to 14 feet NGVD. Fourth, additional cross-sections were necessary to adequately represent the bridge crossings. The bridge crossings are discussed in detail in the next section.

## Structures

The Shamrock Drive and U.S. 41 bridges were simulated with the model. In the HEC-2 model, the energy losses through structures can be computed by either the normal bridge or special bridge methods. The normal bridge method handles the cross-section at a bridge just as it would at any section except that the area of the bridge obstruction below the water surface is subtracted from the total area of flow. The special bridge method is used where there is the possibility of not only low flow, but also pressure and weir flow. The special bridge method was used to represent the Shamrock Drive and Rt. 41 bridge structures.

The data required to simulate these conditions for each crossing included upstream and downstream sections without the structure geometry. Two cross-sections were required to simulate the structure itself. One section portrays the bridge opening for the low flow case (no pressure or weir flow) as a trapezoidal opening which was derived from the shape and area of the individual structures. The other section included the total area of the opening which is used for pressure flow, and the top of the roadway which would be used to compute weir flow over the roadway in those cases where such a condition exists. The four sections required for each bridge are labeled "1" through "4" in Figure 5-9.

Ground elevations at the Shamrock Drive bridge were included in the Brigham and Winningham survey. The geometry of cross-section #1 was repeated at the location of the Shamrock Drive bridge to define the ground profile of the two sections representing the structure. The channel invert elevations of the repeated sections were modified to equal computed thalwags which were determined by subtracting known bridge dimensions from road surface bench mark elevations. Cross-section #2 was repeated immediately upstream of Shamrock Drive to define the expansion back to the natural channel. The four bridge sections are shown in Figure 5-9 with the sections labeled "2" and "3" being used to define the actual structure.

Cross-sections 6, 7, and 8 were used to describe the Rt. 41 bridge crossing. Sections 6 and 8 defined the natural channel downstream and upstream of the bridge, respectively. Section 7 was used and repeated as the two cross-sections required to define the structure. The four bridge sections are shown in Figure 5-9 with the sections labeled "2" and "3" used to define the structure.

A pier shape coefficient used in the energy equation, a loss coefficient for the orifice flow equation, and a coefficient of discharge for the weir flow equation were provided in the HEC-2 model to account for head losses through bridges. The pier shape coefficient was set to 1.25 to represent square nosed piers as suggested in the HEC-2 Users Manual. The loss coefficient for the orifice flow equation was set at 1.56, and was based on a typical value suggested by the Bureau of Public Roads, as stated in the HEC-2 Users Manual. A value of 2.6 was used for the weir flow equation. This value was based on the recommendation in the HEC-2 Users Manual assuming a rectangular weir for flow over the bridge deck. The same coefficients were used at both bridges.

## Roughness Coefficients, and Contraction and Expansion Coefficients

In addition to structure loss coefficients, the HEC-2 model uses the Manning equation for friction loss, and contraction and expansion coefficients to evaluate transition losses. A Manning's "n" roughness coefficient of between 0.03 and 0.06 was used for the creek channel.

Roughness coefficients for overbank sections were set equal to 0.075 for cross-sections west of Rt. 41, and 0.09 for the less urbanized sections east of Rt. 41.

Contraction and expansion coefficients were used to model energy losses where contraction and expansion of flow exists due to changes in channel cross-sections. For Alligator Creek, channel section changes are relatively small. Typical values suggested by the HEC-2 Users Manual for small changes were used for all sections. They were 0.1 for the contraction coefficient and 0.3 for the expansion coefficient. Contraction and expansion coefficients of 0.3 and 0.5, respectively, were used for the bridge sections as suggested in the HEC-2 Users Manual.

### 5.4.2 RESULTS OF DESIGN STORM SIMULATION

#### Existing Conditions

The 25-year design storm was analyzed by simulating the spoil banks as a levee and the downstream water surface elevation based on mean high tide (2.5 ft NGVD). The results of the simulation are given in Table 5-11 which include the section number (SECNO), discharge (Q), average channel velocity (VCH), depth, water surface elevation (CWSEL), minimum ground elevation of the section (ELMIN), elevations of the left and right top of bank for each modeled cross-section (XLBCL & RBEL), elevation of the top of road (ELTRO), and the energy grade line (EG).

The 25-year storm analysis resulted in water surface elevations ranging from 2.47 feet NGVD at Shamrock Bridge to 7.40 feet NGVD at cross-section #4. Water depths ranged from 4.00 feet at Shamrock Bridge to 8.65 feet at cross-section #2. In general, water surface profiles did not exceed bank full conditions, and except for the upstream reaches, the maximum water surface elevations were below the top of bank. Neither the Shamrock Drive nor U.S. 41 top of road elevations were exceeded by the flood. The U.S. 41 bridge had little effect on water surface elevations and flow velocities in the channel. The Shamrock Drive bridge had a significant effect, however, resulting in restricted flow, and decreased

TABLE 5-11

## ALLIGATOR CREEK HEC-2 OUTPUT - EXISTING CONDITIONS

1.0 * 25-YEAR DESIGN STO

SUMMARY PRINTOUT

| SECNO | XLCH | Q | VCH | DEPTH | CWSEL | ELMIN | XLBEL | RBEL | ELTRD | EG |
|---|---|---|---|---|---|---|---|---|---|---|
| .000 | .00 | 1665.00 | .11 | 5.00 | 2.50 | -2.50 | 3.00 | 3.00 | .00 | 2.50 |
| 3000.000 | 3000.00 | 1665.00 | 1.62 | 4.78 | 2.48 | -2.30 | 5.40 | 3.30 | .00 | 2.52 |
| 3500.000 | 500.00 | 1665.00 | 1.44 | 6.09 | 2.59 | -3.50 | 5.40 | 4.33 | .00 | 2.63 |
| 3530.000 | 30.00 | 1665.00 | 1.44 | 6.10 | 2.60 | -3.50 | 5.40 | 4.33 | 12.70 | 2.63 |
| 3880.000 | 350.00 | 1665.00 | 1.33 | 7.85 | 2.65 | -5.20 | 5.10 | 7.30 | .00 | 2.68 |
| 4620.000 | 740.00 | 1665.00 | 1.31 | 7.93 | 2.73 | -5.20 | 5.10 | 7.30 | .00 | 2.76 |
| 5720.000 | 1100.00 | 1572.00 | 2.20 | 7.48 | 2.88 | -4.60 | 3.80 | 7.90 | .00 | 2.96 |
| 6680.000 | 960.00 | 1391.00 | 2.03 | 7.59 | 3.19 | -4.40 | 5.60 | 4.60 | .00 | 3.26 |
| 8320.000 | 1640.00 | 1389.00 | 3.38 | 5.53 | 3.83 | -1.70 | 7.80 | 5.40 | .00 | 4.00 |
| 8680.000 | 360.00 | 1389.00 | 2.23 | 8.61 | 4.11 | -4.50 | 7.40 | 6.50 | .00 | 4.19 |
| 8780.000 | 100.00 | 1389.00 | 2.22 | 8.62 | 4.12 | -4.50 | 7.40 | 6.50 | 11.90 | 4.20 |
| 8880.000 | 100.00 | 1368.00 | 5.31 | 8.49 | 3.99 | -4.50 | 5.40 | 7.60 | .00 | 4.43 |
| 9520.000 | 640.00 | 1041.00 | 3.62 | 6.32 | 4.92 | -1.40 | 7.70 | 7.00 | .00 | 5.13 |
| 10070.000 | 550.00 | 1041.00 | 4.44 | 6.65 | 5.35 | -1.30 | 9.40 | 6.40 | .00 | 5.66 |
| 10890.000 | 820.00 | 882.00 | 2.53 | 7.15 | 6.05 | -1.10 | 10.60 | 8.50 | .00 | 6.15 |
| 11890.000 | 1000.00 | 792.00 | 2.83 | 7.30 | 6.38 | -.92 | 12.20 | 11.80 | .00 | 6.50 |
| 13950.000 | 2060.00 | 649.00 | 2.43 | 7.42 | 7.12 | -.30 | 12.20 | 10.80 | .00 | 7.22 |

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| SECNO | XLCH | Q | VCH | DEPTH | CWSEL | ELMIN | XLBEL | RBEL | ELTRD | EG |
|---|---|---|---|---|---|---|---|---|---|---|
| 15590.000 | 1640.00 | 297.00 | 1.05 | 7.09 | 7.45 | .36 | 8.10 | 12.20 | .00 | 7.46 |

1

velocities 3,200 feet upstream to cross-section #4. Average channel velocities were reduced from about two feet per second to under one foot per second, and water depths were increased by approximately four feet in this reach in spite of increased bed slopes from approximately 0.0008 ft/ft to about 0.003 ft/ft. Through the bridge, velocities increased to approximately six feet per second. The design storm current condition stream profile is shown in Figure 5-10.

Future Conditions

The results of the MSSM simulation of Alligator Creek were then incorporated into the HEC-2 model input. The results of the simulation are given in Table 5-12 which include the section number, discharge, average channel velocity, water depth, water surface elevation, minimum ground elevation of the section, and elevation of the left and right top of bank for each cross section.

The 25-year 24-hour design storm analysis using the future conditions projected to exist in the basin resulted in water surface elevations and depth very similar to the existing condition results. This is justified in that the majority of the basin is currently developed. The new developments that are proposed or under construction within the basin will be designed to retain the required flow based on Chapter 17-25 FAC quality regulations, and the difference between the pre- and post-development flows due to the design storm. Generally, construction, while maintaining peak flows to the pre-development, causes an increase in the total volume of water that is conveyed off of the site. For basins with development occurring a long a distance downstream from the head, this "bleed-off" of detained water may cause the peak volume to increase. However, as is the case in Alligator Creek, if the development is on the upstream portion of the basin, the maintenance of peak flows from the site and the subsequent bleed-off will not affect the peak flows along the Creek significantly.

The profile of the creek as generated by the HEC-2 model is also shown in Figure 5-11.

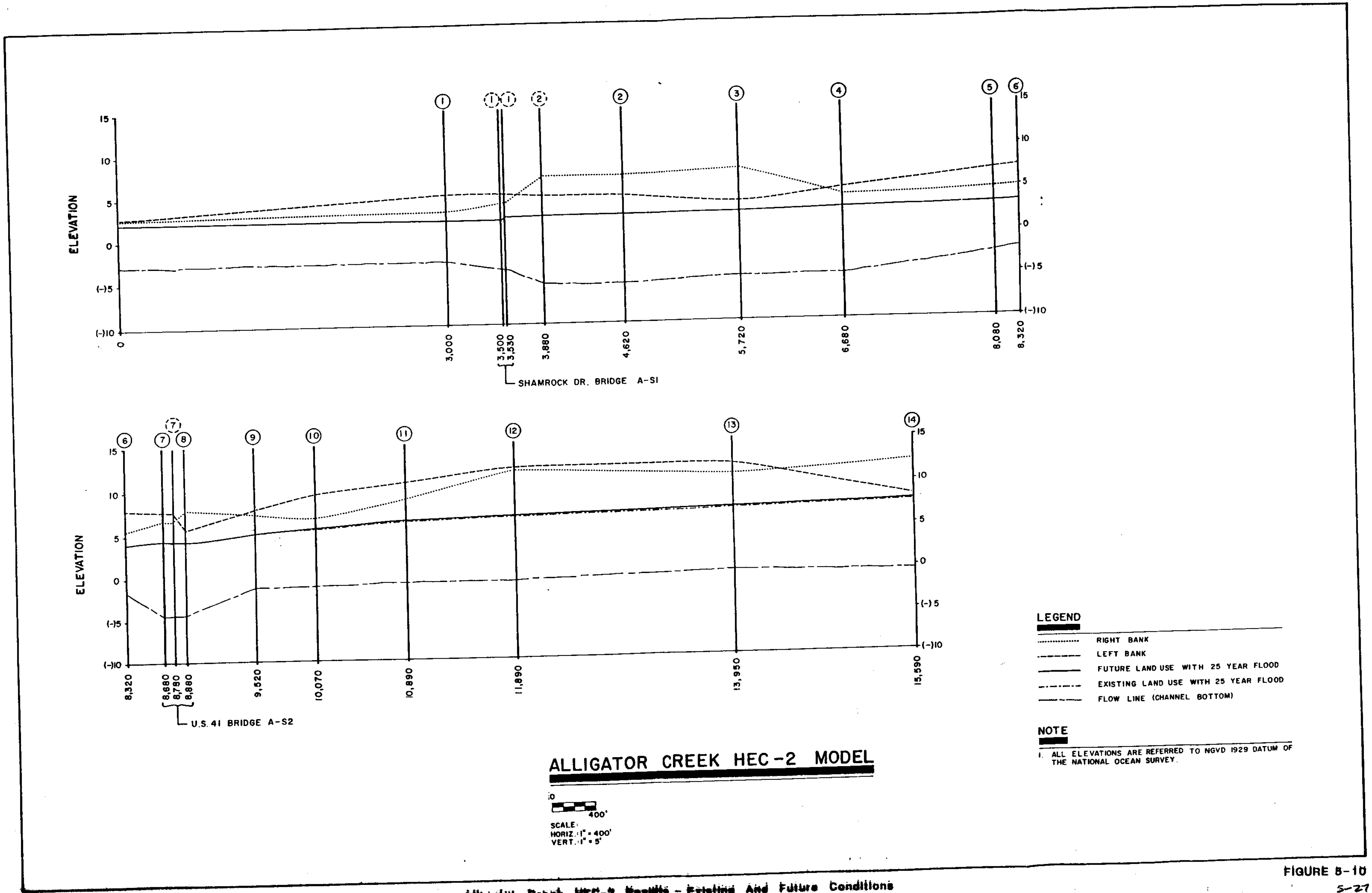


FIGURE 5-10

Alligator Creek HEC-2 Results - Existing And Future Conditions

TABLE 5-12

ALLIGATOR CREEK HEC-2 OUTPUT - FUTURE CONDITIONS

1.0 * 25-YEAR DESIGN STO

SUMMARY PRINTOUT

| SECNO | XLCH | Q | VCH | DEPTH | CWSEL | ELMIN | XLBEL | RBEL | ELTRD | EG |
|---|---|---|---|---|---|---|---|---|---|---|
| .000 | .00 | 1669.00 | .11 | 5.00 | 2.50 | -2.50 | 3.00 | 3.00 | .00 | 2.50 |
| 3000.000 | 3000.00 | 1669.00 | 1.63 | 4.78 | 2.48 | -2.30 | 5.40 | 3.30 | .00 | 2.52 |
| 3500.000 | 500.00 | 1669.00 | 1.44 | 6.09 | 2.59 | -3.50 | 5.40 | 4.33 | .00 | 2.63 |
| 3530.000 | 30.00 | 1669.00 | 1.44 | 6.10 | 2.60 | -3.50 | 5.40 | 4.33 | 12.70 | 2.63 |
| 3880.000 | 350.00 | 1669.00 | 1.33 | 7.85 | 2.65 | -5.20 | 5.10 | 7.30 | .00 | 2.68 |
| 4620.000 | 740.00 | 1669.00 | 1.31 | 7.93 | 2.73 | -5.20 | 5.10 | 7.30 | .00 | 2.76 |
| 5720.000 | 1100.00 | 1577.00 | 2.21 | 7.48 | 2.88 | -4.60 | 3.80 | 7.90 | .00 | 2.96 |
| 6680.000 | 960.00 | 1393.00 | 2.03 | 7.59 | 3.19 | -4.40 | 5.60 | 4.60 | .00 | 3.26 |
| 8320.000 | 1640.00 | 1391.00 | 3.38 | 5.53 | 3.83 | -1.70 | 7.80 | 5.40 | .00 | 4.01 |
| 8680.000 | 360.00 | 1391.00 | 2.23 | 8.61 | 4.11 | -4.50 | 7.40 | 6.50 | .00 | 4.19 |
| 8780.000 | 100.00 | 1391.00 | 2.22 | 8.62 | 4.12 | -4.50 | 7.40 | 6.50 | 11.90 | 4.20 |
| 8880.000 | 100.00 | 1374.00 | 5.33 | 8.49 | 3.99 | -4.50 | 5.40 | 7.60 | .00 | 4.43 |
| 9520.000 | 640.00 | 1075.00 | 3.73 | 6.34 | 4.94 | -1.40 | 7.70 | 7.00 | .00 | 5.15 |
| 10070.000 | 550.00 | 1075.00 | 4.55 | 6.69 | 5.39 | -1.30 | 9.40 | 6.40 | .00 | 5.71 |
| 10890.000 | 820.00 | 912.00 | 2.59 | 7.21 | 6.11 | -1.10 | 10.60 | 8.50 | .00 | 6.21 |
| 11890.000 | 1000.00 | 823.00 | 2.90 | 7.37 | 6.45 | -.92 | 12.20 | 11.80 | .00 | 6.58 |
| 13950.000 | 2060.00 | 680.00 | 2.50 | 7.53 | 7.23 | -.30 | 12.20 | 10.80 | .00 | 7.32 |

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| SECNO | XLCH | Q | VCH | DEPTH | CWSEL | ELMIN | XLBEL | RBEL | ELTRD | EG |
|---|---|---|---|---|---|---|---|---|---|---|
| 15590.000 | 1640.00 | 315.00 | 1.09 | 7.20 | 7.56 | .36 | 8.10 | 12.20 | .00 | 7.58 |

## 5.5 IDENTIFICATION OF FLOOD PRONE AREAS

The HEC-2 backwater model was used to simulate the elevation of the flood waters resulting from the 25-year 24-hour storm from both the existing and future conditions. Generally, the model simulations showed that only minor flooding along the main channel of the Creek occurs due to the design storm. However, the backwater effects on the main tributaries cause some localized flooding within the tributaries' drainage basin.

### 5.5.1 MAINSTEM FLOODING

As stated, the flooding along the main channel of Alligator Creek from the bay to cross-section 14 is relatively minor with only those areas below the top of banks being flooded. The area above cross-section 14 however shows some flooding particularly in the area of Venice East. Additionally, there is a small head loss through the Shamrock Road Bridge. The model was run with either the U.S. 41 or the Shamrock Road bridge redesigned to determine if the bridge had a significant effect on the backwater in the upper reaches of the Creek. The removal of Shamrock Road was found to lower the backwater upstream and thus lower the flood elevations. Additionally, a structure with increased capacity at Shamrock Road will virtually eliminate the head loss and the peak velocities through the structure. This lowering of velocities would then minimize scour and erosion at the bottom of the channel.

### 5.5.2 SUBBASIN FLOODING

As stated previously, this study was to focus primarily on the main channel of the Creek with analysis of the major tributaries only as they related to flooding along the main channel. Although none of the major tributaries in the Alligator Creek basin caused or added to flooding along the main channel, three tributaries have been identified as areas where flooding occurs.

The following outlines the various problem areas and their probable causes:

- Area 1 - The area south of the creek and west of U.S. 41 has some localized flooding problems caused by insufficient capacity within the natural channel draining the lake located at Palmetto Drive and Alligator Drive.

- Area 2 - The area north of Alligator Creek and west of U.S. 41 experiences some minor flooding due again to insufficient channel capacity. The outfall of this channel is located along Siesta Drive.

- Area 3 - The area known as Venice Gardens East still experiences flooding during the 25-year design storm event. This flooding is relatively minor in nature, but, due to the necessity of traffic passability as outlined in Section 4.0, it must be controlled.

## 5.6 IDENTIFICATION OF ALTERNATIVES

The flooding identified in Section 5.5 must be controlled in order to provide the service level flooding as identified in Section 2.5. Generally, flood control measures fall into three major categories. Either additional capacity for stormwater outflow, storage, or flood proofing must be provided.

### Additional Channel Capacity

Additional channel capacity can be obtained through the several means including channelization of existing channels, creating new channels, diversion, or cleaning out existing channels. Channelization of existing channels and the creation of new channels, though possible, present some difficult problems. New channels and channelization usually require additional right-of-way purchases of sufficient size to allow for creation and subsequent maintenance of the channel. Typical of most counties in

Florida, Sarasota has developed along the banks of many of the natural stream channels. Development with its increased runoff, at times causes severe flooding problems for those developments which are within the flood plane.

Diversion of flow can be accomplished by the use of structures or piping which transfers the flow burden to another area. Piping can be a viable alternative for channel capacity improvement in areas where insufficient rights-of-way exist.

Cleaning out existing channels can provide for increased channel capacity in areas where either natural or man-made channels have been allowed to clog due to natural growth. Cleaning (cleaning and snagging), generally not considered a capital improvement (structural), can be so classified for the initial cleanout of a channel. Usually, the initial cleanout has substantial cost, whereas the yearly cost of maintaining these cleaned channels is minimal.

Storage

Additional storage within a basin is obtained by creating detention ponds or other detention areas where stormwater can be stored prior to outflow. The primary advantage to storage in the basin is quality control. A detention pond, when properly designed, provides for the trapping of the first flush of runoff. Storage can also be obtained within the existing channels by constructing control devices which keep the water at a level that is higher than it would naturally be. This again provides some quality benefits and like ponds provides for a general raising of the local surficial aquifer (water table) and some groundwater recharge.

The above methods of providing for either additional storage or channel capacity will be investigated. Additionally, floodproofing, whereby individual structures on a case by case basis are prevented from flooding, will also be investigated. Floodproofing can be accomplished by either controlling the level of flood around the area or by physically raising the structure out of the flood plain.

5.6.1 MAINSTEM FLOODING

Problem area 1 is the Shamrock Road bridge. Generally, the capacity of this bridge is insufficient to allow the necessary flow to pass during the 25-year design flood. The solutions to the flow problems can be accomplished by increasing the under bridge flow capacity or by providing an alternative means of flow through the area. The following are two alternative reviews to eliminate the Shamrock bridge restriction:

1. Increasing the flow capacity of the bridge involves constructing a new bridge with sufficient capacity to allow for the necessary flow. From the computer simulation it was determined that a bridge with about a third larger flow area would be sufficient to carry the required capacity. However, due to the sensitive nature of the surrounding areas, this bridge should be constructed such that at the time of normal flow the flow channel is essentially equal to the existing flow area. This would provide for the maintenance of the existing estuary type habitats that surround the bridge. Construction can be accomplished by providing an overflow weir-type structure across one of the spans of the bridge. Thus, when the water rises sufficiently due to the storm event, added capacity for flow would be available. The costs associated with this alternative are presented in Table 5-13.

2. Providing an alternative route for the flow to reach its downstream destination is another method of providing additional flow capacity. This can be accomplished by constructing another flow channel through the earthen dike which is under Shamrock Drive. The Alternative channel should be located approximately 50 feet north of existing bridge opening. Box culverts and head work structures can be

TABLE 5-13

ALLIGATOR CREEK

PROBLEM AREA SHAMROCK ROAD BRIDGE

SOLUTION: REPLACE SHAMROCK ROAD BRIDGE

ESTIMATED COST

| CAPITAL PROJECT | COST |
|---|---|
| NEW BRIDGE STRUCTURE<br>30' X 60' ROAD SURFACE = 1800 SQ.FT.<br>@ $100.00 SQ.FT. | $180,000 |
| CONTINGENCIES @ 20% | $36,000 |
| ESTIMATE OF CONSTRUCTION COSTS...... | $216,000 |
| LEGAL, ADMINISTRATIVE, ENGINEERING, AND SURVEY @ 15% | $32,400 |
| TOTAL PROJECT COSTS----------------- | $248,400 |

BENEFITS:

A) REPLACEMENT OF EXISTING BRIDGE STRUCTURE
B) MAINTENANCE OF EXISTING FLOW PATTERNS
C) CONTROL OF THE 25-YEAR DESIGN STORM

constructed to provide for additional flow without altering the current Shamrock Road bridge. The costs associated with the construction of this alternative are presented in Table 5-14.

### 5.6.2 SUBBASIN FLOODING

The tributary channels that experience flooding were identified in Section 5.5.2. While flood problems were identified, specific modeling was not done at the subbasin level. Therefore, the flood mitigation alternatives for each problem listed below have not been specifically engineered, but are given as aids to future analysis.

Area 1

Area 1 is an area south of the main channel of Alligator Creek in an area known as South Venice. This area has historically had problems with flooding on a local level. This flooding is the primary result of insufficient channel capacity between the lake located at Palmetto Drive and Alligator Drive, and the outfall just east of the Shamrock Road bridge. The natural channel meanders generally northwesterly from the lake through a poorly defined channel. This channel is extremely overgrown with a correspondingly high Manning's roughness coefficient. Additionally, various culverts have been installed over the years which allow flow to cross under streets and in some cases under entire properties. The alternatives that are suggested for analysis include: channelization, storage, and flow diversion which are discussed below:

1. Channelization involves the regrading and shaping of the natural channel to the extent that sufficient stormwater flow is obtained for sufficient stormwater runoff. To accomplish this, sufficient rights-of-way would have to be purchased to allow for the creation and subsequent maintenance of the channel.

TABLE 5-14

ALLIGATOR CREEK

PROBLEM AREA SHAMROCK ROAD BRIDGE

SOLUTION: SUPPLEMENT SHAMROCK ROAD BRIDGE

ESTIMATED COST

| CAPITAL PROJECT | | | | COST |
|---|---|---|---|---|
| TWO 5' X 8' BOX CULVERTS | | | | |
| 50 LF....@ $550.00 LF | | | | $55,000 |
| ROAD SURFACE REPAIR | 1350 SQ-YD | @ | $10.00 SQ-YD | $13,500 |
| EARTHWORK | 1000 CU-YD | @ | $4.00 CU-YD | $4,000 |
| RESODDING OF SURFACE | 250 SQ-FT | @ | $4.00 SQ-FT | $1,000 |
| CONCRETE ENTRANCE AND OUTFLOW STRUCTURES | | | | $18,000 |
| SUB-TOTAL.......... | | | | $91,500 |
| CONTINGENCIES @ 20 % | | | | $18,300 |
| ESTIMATE OF CONSTRUCTION COSTS...... | | | | $109,800 |
| LEGAL, ADMINISTRATIVE, ENGINEERING, AND SURVEY @ 15% | | | | $16,470 |
| TOTAL PROJECT COSTS----------------- | | | | $126,270 |

BENEFITS:

A) MAINTENANCE OF EXISTING FLOW PATTERNS
B) CONTROL OF THE 25-YEAR DESIGN STORM

2. Storage within the area would provide for an attenuation of flow out of the area to the extent that a maintained natural outflow channel would be sufficient. This is accomplished through the purchase of land within the basin at a location near the head of the outflow channel, and the creation of a storage pond. One of the major problems associated with the construction of a retention basin in this area is the high, tidally controlled shallow water table. Generally, during the dry season you can only expect to get a maximum of 2.0 feet of storage within the pond, and the wet season storage is considerably less.

   Additionally, tidal fluctuations change the water levels daily. Thus, a detention pond would have to have a large surface area to provide a small amount of storage.

3. Flow diversion would be accomplished by providing an alternative flow path from the lake at Palmetto Drive to an outflow. It is envisioned that this flow diversion alternative would consist of a flow splitter device constructed so that normal low volume flows are directed through the natural channel, and only larger flows through the diversion. The overflow or diversion channel would consist of a 42-inch pipe leading from the lake along Mangrove Road to an outfall at Sarasota Bay. Additionally, the natural channel would have to be cleaned and a storage area provided for treatment of the first flush runoff.

In addition to the three alternatives presented herein, it has been determined that regardless of the alternatives selected, outlet control structures on each of the existing lakes should be constructed. The benefits associated with the control structures are that they have the ability to retain the first flush runoff from the areas draining to the lakes, thus providing a measure of water quality treatment that does not currently exist.

Area 2

Area 2 is the area north of the main channel of the creek and west of U.S. 41. Generally, the outlet channel that parallels Siesta Drive is not of sufficient capacity to provide for the required flow volume. The areas outlet channel south of Baffin Road is severely constricted and overgrown causing the outflow from the channel to be restricted. This restriction then causes some minor flooding within the basin. The alternatives suggested for this problem area are additional storage and channel improvements. These alternatives are discussed below:

1. Additional storage within the area would provide for attenuation of peak and runoff quality treatment. As stated above, the pond would require a relatively large surface area for the amount of storage obtained due to the high surficial aquifer table.

2. Channel improvements in this area would consist of expanding the size of the existing channel north of Baffin Road, and constructing a flow control device which would maintain a higher than normal water level within the channel banks and provide for first flush runoff capture. The Baffin Road crossing would not be altered as there is currently sufficient capacity for the design flow.

Area 3

Area 3 is the area known as Venice Gardens East. Generally, this area is drained by a series of small canals along property boundaries. These canals are typically undersized and poorly maintained. However, there is no need for capital improvements to this area because the additional capacity required will be obtained by lowering the design storm water level in the creek through the Shamrock Road bridge opening. Two weir structures (recommended by the Stormwater Management Division) and the channel maintenance program that is recommended for the entire basin will also help alleviate this problem.

## 5.7 RECOMMENDATIONS

The Alligator Creek in-depth study has identified one mainstem and three off-channel problem areas where flooding occurs during the 25-year 24-hour design storm. Alternative solutions for these problems were presented in the preceding section. The purpose of this section is to provide an aid in the final selection process by ranking the alternatives in accordance with the engineering and cost analysis criteria described in detail in Section 2.4 of this report.

It must be noted that the alternatives for mainstem improvements described in the preceding section have already passed through the preliminary screening process. This preliminary screening process was accomplished through the aid of the hydrologic and hydraulic models employed for the study. Thus, the alternatives described are those alternatives which were found to provide some measure of flood relief and have a reasonable expectancy for approval and construction. Alternatives that dealt with cross-basin flow routing and individual storage were dismissed during this preliminary phase of alternative selection, due to the reasons mentioned above.

### 5.7.1 RANKING

The ranking process described in Section 2.4 was employed to position the stormwater management alternatives such that a final selection can be made. The alternative ranking processes are described in the following paragraphs:

#### Reliability

The reliability of the system is concerned with the system's ability to provide flood protection for flows expected from the 25-year 24-hour design storm. Additionally, the alternative is considered reliable if, for storms greater than the design storm, the alternative continues to function as designed. Generally, for the purposes of this ranking methodology, the higher rankings will go to those alternatives which in themselves have the capacity to alleviate the total problem associated with a particular problem area. The majority of the alternatives discussed previously provide for the necessary flood control. Therefore, the reliability factor is used in many cases to rate the reliability of the alternative for flows greater than the 25-year 24-hour design storm.

Environmental Impact

The environmental impacts of the various stormwater alternatives are minimized in that all of the alternatives must meet the requirements of the applicable local, state, and federal regulations. However, some of the alternatives discussed provide for a measure of improvement in the quality of stormwater runoff. Notably, those alternatives that provide for retention and detention with filtration have a positive effect on the quality of stormwater. Additionally, the alternative methodologies which do not disrupt the local ecosystem would be deemed most acceptable. For these reasons, the detention/retention storage and the improved or new channels with filtration on the existing ponds were rated better than those alternatives that had no measurable environmental impact.

Public Awareness/Acceptance

The public's impression of a project can mean the difference between whether the project is accepted or rejected. Generally, the public is perceived to accept the alternatives that will prevent flooding of their specific areas while causing no alteration in their lifestyle. Typically, those alternatives that involve the purchase of land and private homesites are not readily accepted. Thus, as can be seen in Table 5-15, the alternatives that involve large purchases of developed land are not thought to be high in public acceptance.

TABLE 5-15

ALLIGATOR CREEK
ENGINEERING ANALYSIS
25-YEAR STORMWATER CONTROL ALTERNATIVES

| Stormwater Control Options | Evaluation Factors (1) | (2) | (3) | (4) | (5) | (Total) | Final Problem Area Ranking |
|---|---|---|---|---|---|---|---|
| BASIN WIDE | | | | | | | |
| 1. Institute Basin Wide Maintenance Program | 5 | 3 | 4 | 5 | 5 | 22 | 1 |
| PROBLEM AREA - 1 SHAMROCK ROAD BRIDGE | | | | | | | |
| 1. Replace Shamrock Bridge | 3 | 4 | 4 | 3 | 4 | 18 | 1 |
| 2. Supplement Shamrock Bridge | 3 | 4 | 3 | 3 | 4 | 17 | 2 |
| PROBLEM AREA - 2 SOUTH VENICE (SOUTH) | | | | | | | |
| 1. Channelize Existing Channel | 2 | 2 | 1 | 3 | 3 | 11 | 2 |
| 2. Create Retention/Detention Ponds | 3 | 2 | 2 | 2 | 2 | 11 | 2 |
| 3. Create By-Pass Channel | 4 | 3 | 3 | 4 | 4 | 18 | 1 |
| PROBLEM AREA - 3 SOUTH VENICE (NORTH) | | | | | | | |
| 1. Create Retention/Detention Ponds | 2 | 2 | 2 | 2 | 2 | 10 | 2 |
| 2. Improve Existing Channel | 2 | 3 | 3 | 3 | 3 | 14 | 1 |

(1) = Reliability Factor
(2) = Environmental Factor
(3) = Public Awareness/Acceptance Factor
(4) = Master Plan Agreement Factor
(5) = Implementability Factor

Master Plan Agreement

This engineering criterion is concerned with the integration and functioning of the entire stormwater management system within the Alligator Creek basin. Generally, the alternatives reviewed were ranked from having slight agreement to total agreement with the Master Plan. The basin-wide maintenance program for the Creek, and the increase in capacity for the Shamrock Road bridge were found to be in total agreement with the proposed Master Plan objectives. The other subbasin alternatives were found to be in general agreement due to their positive water quality impacts, but because of the localized nature of their flood prevention, they were deemed lower in agreement with the Master Plan concepts.

Implementability

The implementability of an alternative is a measure of the ease in which the alternative can be constructed, permitted, funded, and politically accepted. Typically, those alternatives which may involve the condemnation and purchase of private houses are considered to be difficult to implement. Alternatives which exhibit this characteristic include new ponds, new outfalls, and the improvement of existing channels. Environmental considerations, especially the permitting process, make alternatives that do not provide significant water quality benefit unattractive. Thus, as can be seen in Table 5-15, those alternatives that involve the technical features discussed are ranked lower than some of the other alternatives.

5.7.2 CONCLUSION

As far as engineer considerations are concerned, the implementation of a comprehensive basin-wide maintenance program is recommended for selection. The mainstem problems are best controlled by the supplementation of Shamrock Bridge. A new bridge is scheduled for construction in the near future. Its design should conform to the precepts expounded in this report. Area 1 flooding would be best solved by the channelization of the existing channel, along with the necessary retention and filtering structure near the channel outfall. Area 2 can be best served by the improvement of the existing channel. Area 3 is best served by the construction of two water level control structures and channel maintenance.

## 5.8 CAPITAL IMPROVEMENT PLAN

To facilitate the implementation of the recommended stormwater management facilities, it is necessary to devise an implementation strategy which will allow the orderly design, acquisition, and construction of the needed facilities. This can best be accomplished through the use of a five year Capital Improvement Plan (CIP). The CIP has been used extensively as a means of prioritizing the needs and expenditures for various public facility projects. Annually, this program is updated and an additional years projects are included. An example would be the five-year plan currently used by the Transportation Department to identify and fund future road improvement projects.

A CIP for the Alligator Creek basin was developed whereby the proposed improvements were prioritized and apportioned over a five-year plan. That is, the projects and their associated costs were spread over a five-year time period in a manner that facilitates their planning and construction. The plan is shown in Table 5-16.

Project 2 is specifically recommended by the results of the modeling of the main channel of the creek. Thus, the costs have been estimated with some degree of accuracy. Projects 3 and 4 are the perceived solutions to the off-channel flooding problems identified, but not specifically modeled within the report while Projects 5 and 6 were those requested by the Sarasota County Stormwater Management Division. Thus, the costs associated with them are given as a starting point for comparison purposes. Final costs of these projects may vary substantially from the estimate, depending on the study of the individual problem areas. Project 1 is the contingency fund whereby additional projects, if warranted, can be adequately financed.

TABLE 5-16

SARASOTA COUNTY STORMWATER MANAGEMENT PLAN

ALLIGATOR CREEK - FIVE YEAR CAPITAL IMPROVEMENT PROJECTS

| PROJECT DESCRIPTION | COSTS (/year) (A) 1987 | 1988 | 1989 | 1990 | 1991 |
|---|---|---|---|---|---|
| 1. CONTINGENCY CIP (B) | $120,000 | $120,000 | $120,000 | $120,000 | $120,000 |
| 2. SUPPLEMENT SHAMROCK ROAD BRIDGE | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $16,500 | | | | |
| B. CONSTRUCTION | | | | $130,774 | |
| 3. SOUTH VENICE (SOUTH) (C) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $75,560 | | | | |
| B. LAND AQUISITION | | $254,400 | | | |
| C. CONSTRUCTION | | | $168,540 | | |
| 4. SOUTN VENICE (NORTH) (C) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | | | $53,371 | | |
| B. LAND AQUISITION | | | | | |
| C. CONSTRUCTION | | | | | $157,810 |
| 5. VENICE GARDENS/JACARANDA OUTLET (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $34,370 | | | | |
| B. CONSTRUCTION | | | | | $124,000 |
| 6. EAST VENICE WEIRS (2) (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $30,000 | | | | |
| B. CONSTRUCTION | | | | $104,809 | |
| | $276,430 | $374,400 | $341,911 | $355,583 | $401,810 |

NOTE....

(A) All costs are expressed as present worth in 1986 dollars utilizing 6% as the rate of cost increase per year.
(B) Projects not specifically described.
(C) The costs associated with these projects have been estimated without the benefit of a complete hydrologic/hydraulic analysis and thus may vary.
(D) These improvements recommended by the Sarasota County Stormwater Management Division

# 6.0 PHILLIPPI CREEK

# 6.0 PHILLIPPI CREEK

## 6.1 BASIN CHARACTERISTICS

The Phillippi Creek basin is the largest basin within the study area (Figure 6-1). The creek drains approximately 57 square miles of northwest Sarasota County and a small portion of lower Manatee County. The main channel of the creek flowing through the City of Sarasota is approximately 5.2 miles long and is fed at its upstream by two major canals, Main A and Main B. Main A and Main B canals are in turn fed by smaller branch canals and tributaries. The entire watershed consists of gently sloping terrain which rises from sea level at the outflow in Roberts Bay to just over 40 feet (NGVD) at the headwaters. The surface of the basin has an average slope of just over 0.2 percent or a 1-foot drop for every 500 feet of distance. The slope of the stream bed varies from about 4 feet per mile at its upper reaches to less than 1-foot per mile at its outfall.

Flow begins over 12 miles from the outlet of the creek where the channel is only a few feet wide, and ends at the mouth of the creek at Roberts Bay with a channel width of ± 350 feet wide. Typical of Florida coastal streams, the base flow in the creek is shallow and slow with a velocity approaching 1-foot per second.

There are a number of bridges crossing the creek throughout its reach. The largest bridge across the main channel is at the intersection of the creek and Tamiami Trail (U.S. 41), approximately one-half mile above the mouth of the creek. There is one major flow control structure within the creek—a spillway located on the main channel approximately 3.4 miles above the outlet. This weir-type structure was once used as a salinity barrier to aid local citrus growers in obtaining suitable irrigation water. It is no longer maintained and tidal induced salt water movement has been noted some 8 miles above the mouth of the creek.

The channel below the confluence of Main A and Main B lies almost entirely within the boundaries of the City of Sarasota. This highly urbanized area

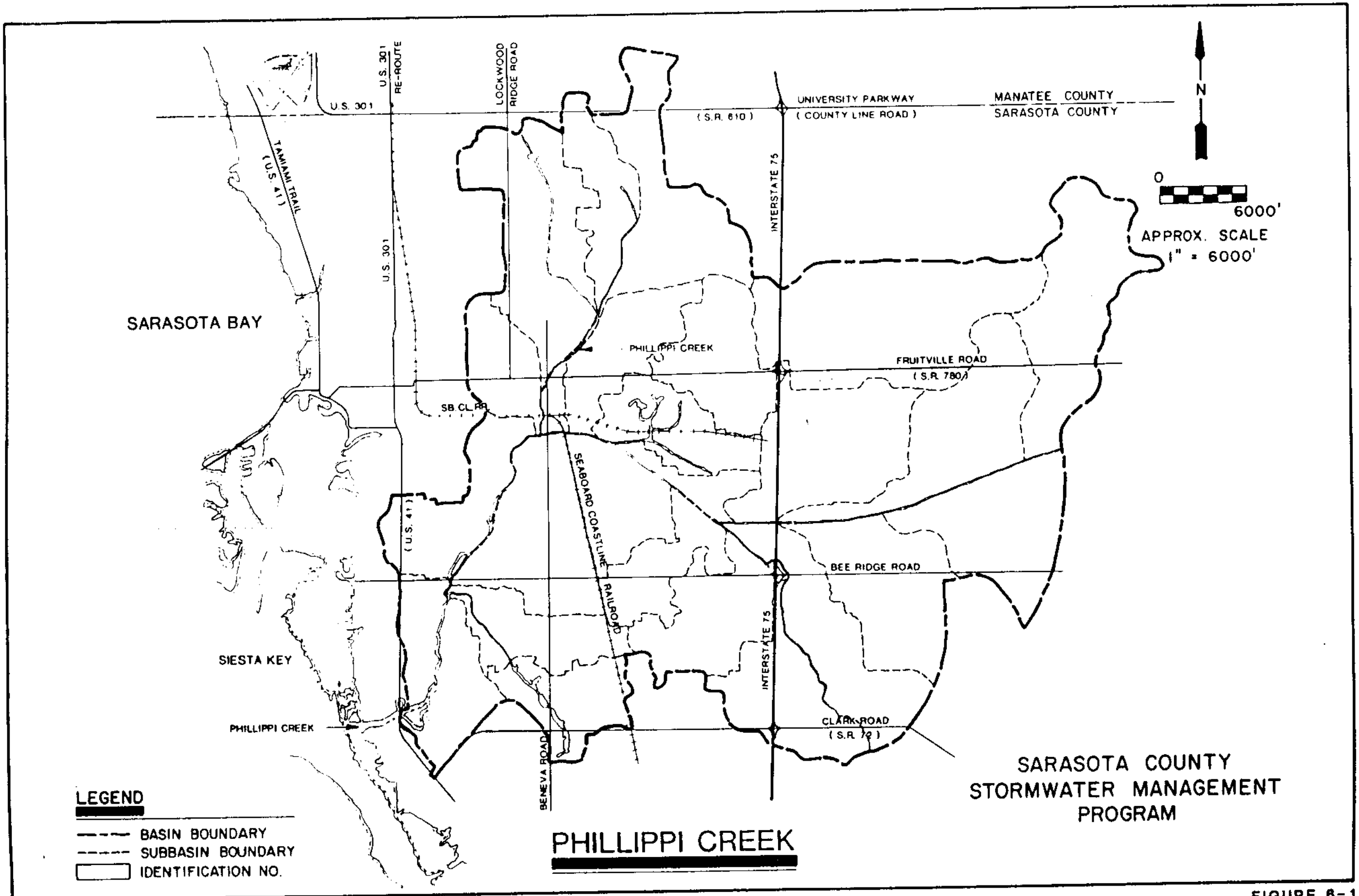



FIGURE 6-1

Phillippi Creek Study Area

is laced with small tributary channels flowing into the creek. Historical flooding of this area has at time been rather severe. The flood of September 1962 in which about 15 inches of rain fell over a 48-hour period was the most damaging storm on record. The highest stages and discharges ever recorded for the creek were due to this storm. This greater than 100-year return frequency storm caused widespread and disastrous flooding all along the creek, with most adjacent streets and drainageways under water from one to three days. Damage from this storm was estimated to be over $2,300,000.

The 1962 storm, although severe, would have even greater effects if it were to occur today. The percentage of urbanization within the watershed has almost doubled. This increased urbanization has changed the runoff characteristics such that the peak and timing of the runoff has been significantly altered.

## 6.2 PREVIOUS STUDIES

There have been several stormwater studies of the Phillippi Creek basin dating as far back as April 1961 with a report from Smalley Welford and Nalven (SWN). In their report, SWN used the rational method to determine the peak flows resulting from the 25-year design storm over the entire basin. Using the rational method and a coefficient of 0.35, SWN obtained a peak flow value of over 10,000 cfs at the outfall. SWN survey work of the entire creek was extensive and included the major tributaries. Their conclusion was that the creek was extremely undersized, being able to pass only about half of the required flow during their design event. The majority of their recommended improvements to the creek were never constructed.

The J.E. Greiner Company completed a report in August 1961 entitled "The Flood Relief Study of Phillippi Creek at the Tamiami Trail, Sarasota County, Florida." Greiner, using the method of Dr. R.W. Pride of the USGS, obtained a discharge for the 25-year design event of 7,600 cfs. Their report outlined the measures they deemed necessary to control flooding along the creek.

The U.S. Army Corps of Engineers conducted a survey report of Phillippi Creek concluding in October 1963. This report again discusses possible flooding resulting from various design storms. The total discharge from the Phillippi Creek is estimated to be about 5,000 cfs for the 30-year design storm.

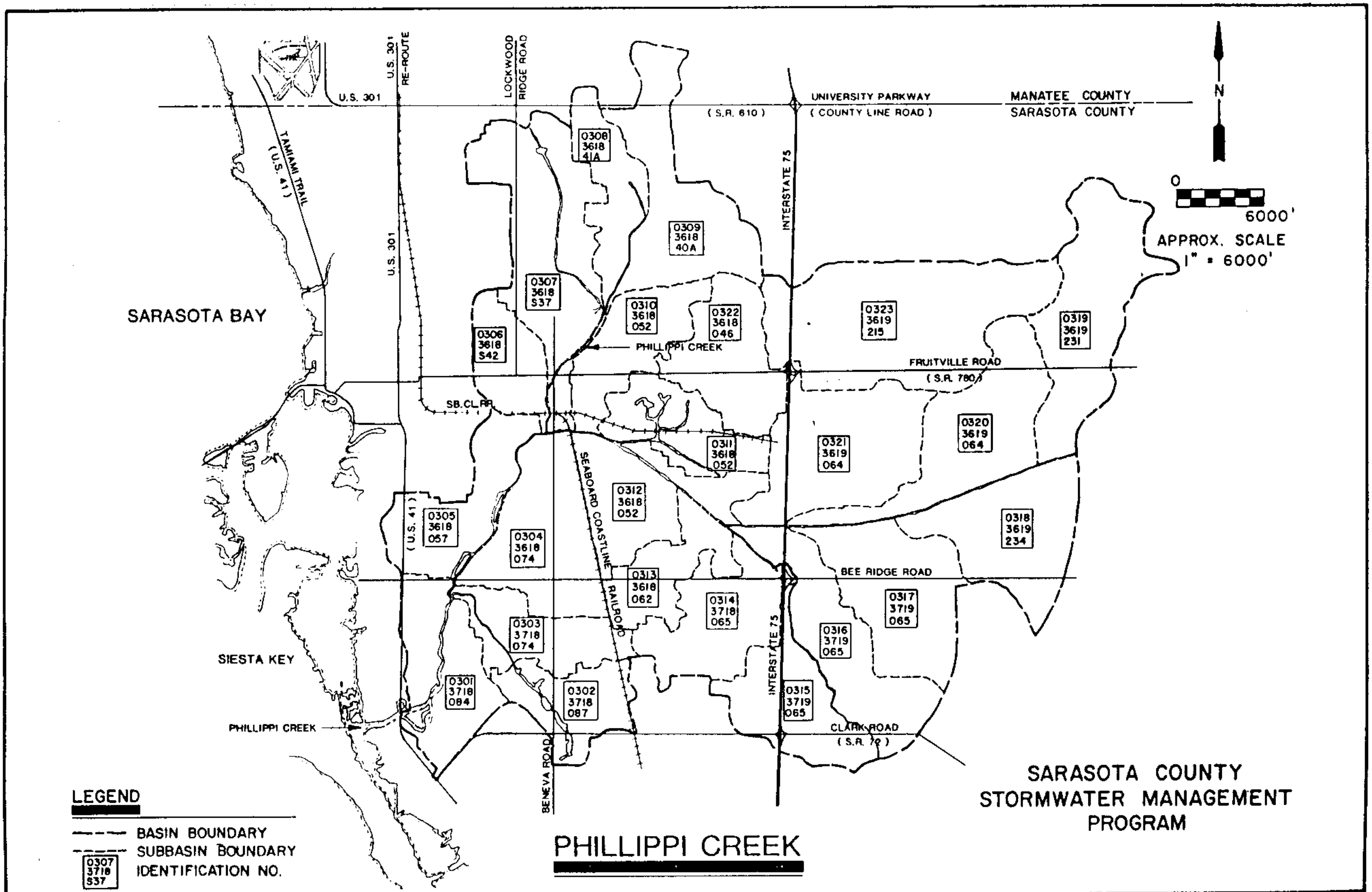
Mote Marine Laboratory, in their Phillippi Creek Water Quality Study completed in March 1982, did not specifically address the peak flow due to a specific design event. Instead, they studied the quality parameters associated with many actual rainfall events.

In association with the proposed development of Wyndwood by the Lee Bradley Corporation, Tri-County Engineering did a study of Phillippi Creek. Unfortunately, the complete results of the study are not available for public dissemination. However, their report notes an inflow to the reach located at the salinity barrier of about 5,500 cfs.

## 6.3 EXISTING DRAINAGE SYSTEMS

The Phillippi Creek basin was delineated into 23 subbasins during the previous Sarasota County Basin and Subbasin Delineation and Refinement projects (Figure 6-2). For ease of modeling and to preserve the homogeneity of the subbasins, several of these subbasins have been further subdivided into smaller hydrologic units. As typical of the region, the predominant development has occurred from the coastline on the west coast inward. Much of the lower reaches of the creek run through the City of Sarasota. Generally, this area is highly developed with the most dense development surrounding the Gulf of Mexico. No attempt has been made to discuss each and every subdivision's drainage patterns, rather a subbasin-wide approach was taken. The results of this subbasin drainage investigation are presented in Table 6-1.

The land surveying firm of Lemonde/Biscayne was contracted to survey the main reaches of the creek. They were tasked with providing cross-sectional data for 32 cross-sections located as shown in Figure 6-3. The 32 cross-sections have been plotted and can be found in Appendix C.




Phillippi Creek Basin & Subbasin Delineation

FIGURE 6-2

FIGURE 6-2

TABLE 6-1

SUBBASIN DRAINAGE DESCRIPTIONS

| Subbasin No. | Description |
|---|---|
| 03011 | This subbasin is the portion of basin 0301 which lies west of Phillippi Creek. Generally, flow is eastward through natural and man-made channels outfalling directly into the creek. Primary developments served by this subbasin are: River Forest, Los Lomas, and Tamiami Terrace. |
| 03012 | This eastern portion of subbasin 0301 drains westward via natural and man-made channels to the creek. Additionally, a canal system draining to Phillippi Bayou provides gulf access and drainage for Phillippi Gardens. In addition to Phillippi Gardens, the primary subdivisions are Hope Acres, Phillippi Cove, and Forest Hills. |
| 0302 | This subbasin drains the area known as Clark Lakes northwesterly to subbasin 0303 via the Clark Lakes Branch. Primary subdivisions include: Clark Lakes, Clark Meadows, Beneva Woods, and Country Place. |
| 0303 | This subbasin drains through Clark Lakes Branch northwest to the creek. Both subbasin 0302 and 0303 have significant flooding problems due primarily to the lack of capacity in the branch. |
| 0304 | This subbasin drains westward via natural and man-made channels to the creek. Many of the channels have weir overflow structures at their outfall to keep the groundwater artificially high. Primary subdivisions include: Pinecraft, Forest Lakes, Forest Pines, South Gate Manor, and Village Green. |
| 0305 | This subbasin on the west side of the creek drains primarily through man-made channels eastward to the creek. Again, as in 0304, outfall structures tend to keep the channel water surface high. Generally, channels are in need of maintenance as some bypass channels are clogged. Major subdivisions include: South Gate, Bellvue Terrace, and Wildwood Gardens. |
| 0306 | This subbasin drains southeasterly through a channel system to an outfall near Beneva Road. Conveyance is through natural and man-made channels and several lakes in the northwest section. Major developments are Carolina Estates and Oakwood Manor. |

TABLE 6-1
(Continued)

| Subbasin No. | Description |
|---|---|
| 0307 | Subbasin 0307 has been further divided into basin 03071 and 03072. |
| 03071 | This subbasin is composed of the southern, highly developed portion of 0307. Drainage is primarily via channel BA to the outfall at the southeastern edge of the basin. Leisure Lakes, Glen Oaks, and Kensington Park are the major subdivisions. |
| 03072 | This subbasin is largely sparsely developed drainage portions of DeSoto Lakes, Beverly Terrace, and DeSoto Acres. Primary drainage is along a channel known as BA which is centrally located within the basin. Outflow is south to the channel in basin 03071. |
| 0308 | This subbasin is drained primarily by channel BC, a man-made channel connecting several low lying wetland areas. Portions of both Ravenwood and DeSoto Lakes are within this subbasin. |
| 0309 | This basin was further subdivided for modeling purposes into 03091 and 03092. |
| 03091 | This subbasin covers the area known as the Meadows subdivision. Drainage is through a series of interconnected lakes and control structures. Primary outfall is westward to branch B. |
| 03092 | The 03092 subbasin is primarily undeveloped and drains via natural drainage ways to main channel B. |
| 0310 | The majority of this basin is the site of the Bobby Jones Golf Course. Drainage from the basin is southwest through a man-made channel. |
| 0311 | This subbasin contains the developments known as Sherwood Lake Estates, Sherwood Forest, and Hidden Oaks. Drainage is northwesterly to the creek via a man-made channel and several lakes. |
| 0312 | This subdivision has been subdivided into subbasin numbers 03121, 03122, and 03123. |
| 03121 | Subbasin 03121 is located north of the main channel of the creek. This predominantly undeveloped area drains westerly through natural channels to the creek. |

TABLE 6-1
(Continued)

| Subbasin No. | Description |
|---|---|
| 03122 | This subbasin covers the northwest portion of basin 0312. The major development is Tamarron which drains through various lakes and control structures to the creek. |
| 03123 | The primary development within this subbasin is Sarasota Springs. Drainage is via man-made channels including swales and gutter, through lakes, to an outfall just south of The Lakes subdivision. |
| 0313 | The major subdivisions within this basin include Sarasota Springs, Spring Lake, South Gate Ridge, Sunnyland, Ridgewood Estates, and Colonial Gables. This relatively highly developed area drains via various channels northeasterly to the creek. |
| 0314 | This subbasin has as its principal developments Center Gate and Winding Woods. Drainage is primarily northwesterly beginning with natural drainage channels and flowing via man-made canal to an outfall east of Colonial Gables. |
| 0315 | This subbasin located south of subbasin 0314 has as its principal development Camelot. Flow is generally toward the channel which is located on the eastern side of the basin between 0315 and 0316. |
| 0316 | The primary subdivisions within the basin are Lake Sarasota and Foxfire. Runoff flows through the lakes of Foxfire to the creek channel on the eastern side of the subbasin. |
| 0317 | There are two primary subdivisions within this basin: Bent Tree Village and Shirley Oaks. Bent Tree Village drains through a system of lakes and channels to an outflow weir located just south of Bee Ridge Road. North of Bee Ridge, the flow is northwestward to the creek. |
| 0318 | This totally agricultural basin utilizes natural drainage and a system of canals to carry water northward to the creek. |

TABLE 6-1
(Continued)

| Subbasin No. | Description |
|---|---|
| 0319 | Subbasin 0319 is primarily agricultural with runoff flowing basically southward through a channel to the creek. |
| 0320 | This subbasin can be classified as rural in that only a portion of the basin is developed. The eastern part of the Sarasota Golf Colony is within the basin. Channels and overland flow direct the runoff southward to the creek. |
| 0321 | This predominantly rural basin is bisected by two man-made channels which drain south and west along a channel known as channel C. |
| 0322 | This basin is approximately two-thirds developed, having Georgetown as its major development. Generally, drainage is by a drainage ditch to the northeast corner of the basin. Runoff then joins with that from basin 0323 flowing southward along channel C. |
| 0323 | This basin is comprised of both agricultural and low density residential areas. Lot sizes are in the range of one acre or greater. A man-made channel traverses the subbasin from east to west to a large wetland area near the western edge. Outflow is south out of this wetland via channel C. |

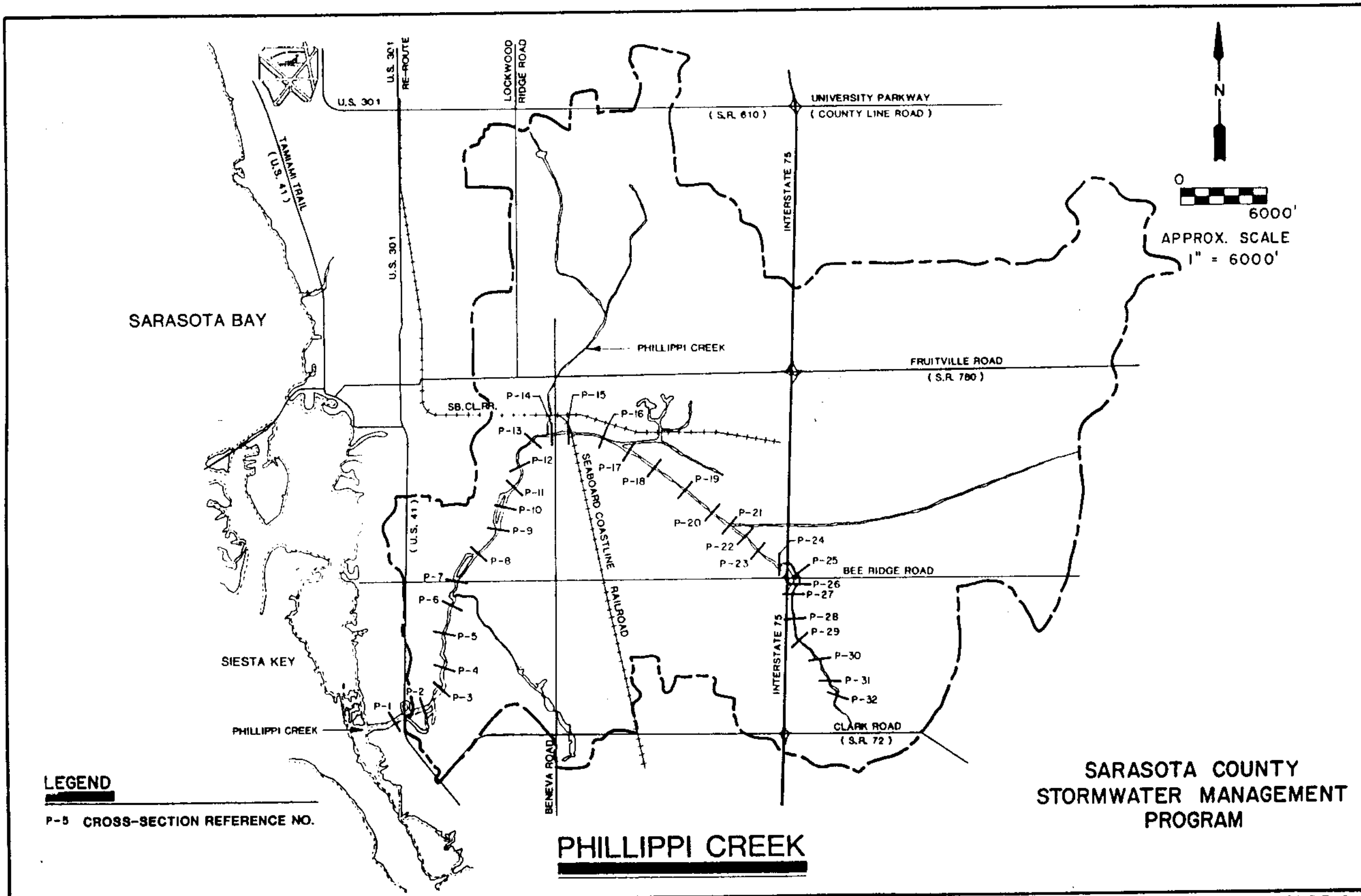



FIGURE 6-3

Phillippi Creek Cross-Section Location Map

FIGURE 6-3

## 6.4 STORMWATER MANAGEMENT MODEL - MSSM

The MSSM model that has been used to quantify the stormwater runoff in Phillippi Creek is discussed in Appendix A of this report. The following section details the model set-up and input selection process.

### 6.4.1 MODEL NETWORK

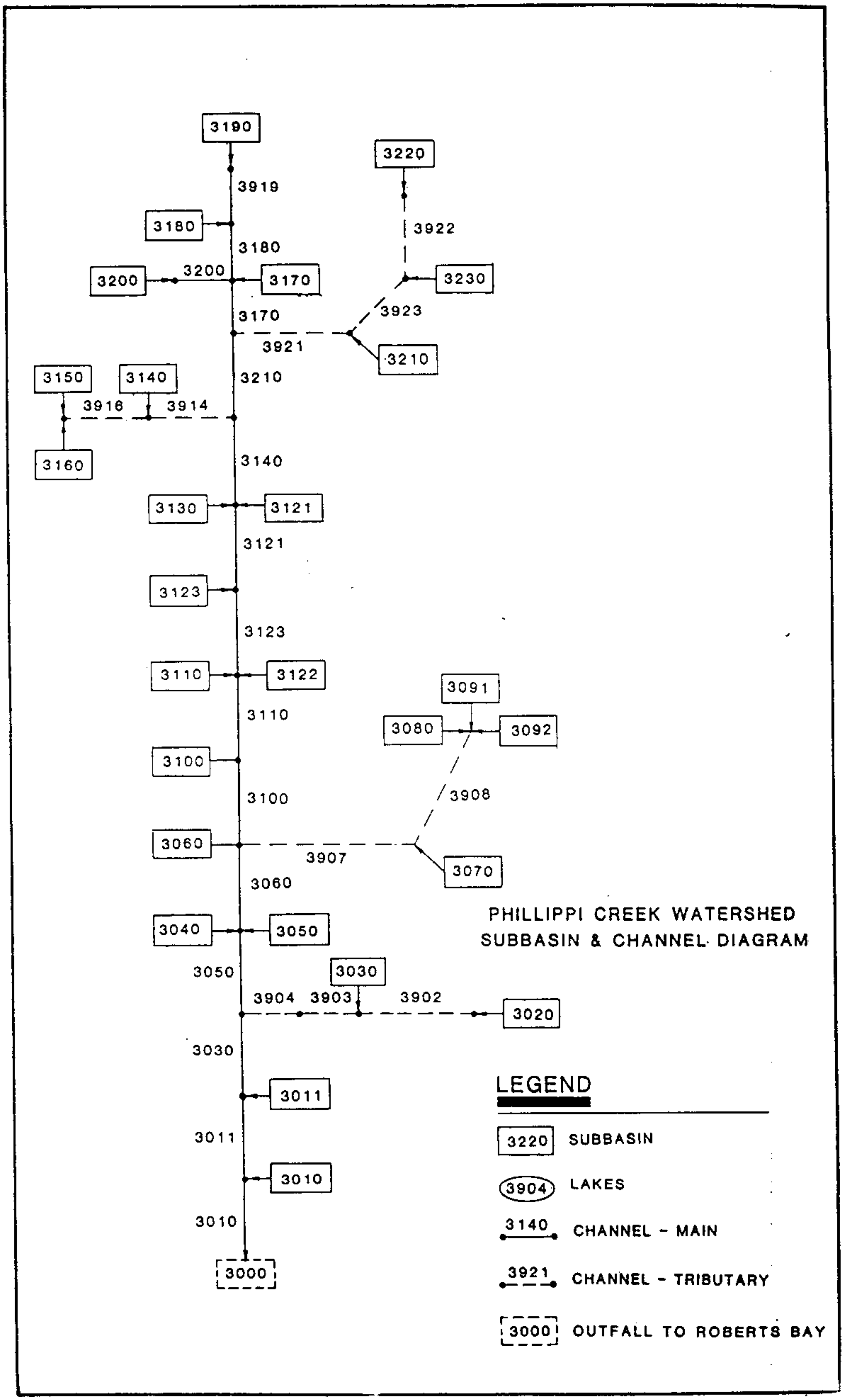
The Phillippi Creek basin is represented by a system of subbasins, control structures, and channels (Figure 6-4). These are represented in the model by defining their stormwater control/conveyance properties, similar to the process involved with Alligator Creek and discussed in Section 5.3.1 of this report. A listing of the major structures in Phillippi Creek is given in Table 6-2 wth their location shown in Figure 6-5.

The model representation of the stormwater management structures is depicted in Figure 6-4 and shows the interconnectivity of the subbasins and channels. The subbasins were defined previously during the Sarasota County Basin and Subbasin Delineation and Refinement Projects (CDM, 1983) and have been modified based on recent aerial photography and survey data. For the purposes of modeling, subbasins 03010, 03090, and 03120 have been subdivided into homogeneous hydrologic units due to outlet location, hydrologic properties, and interconnectivity.

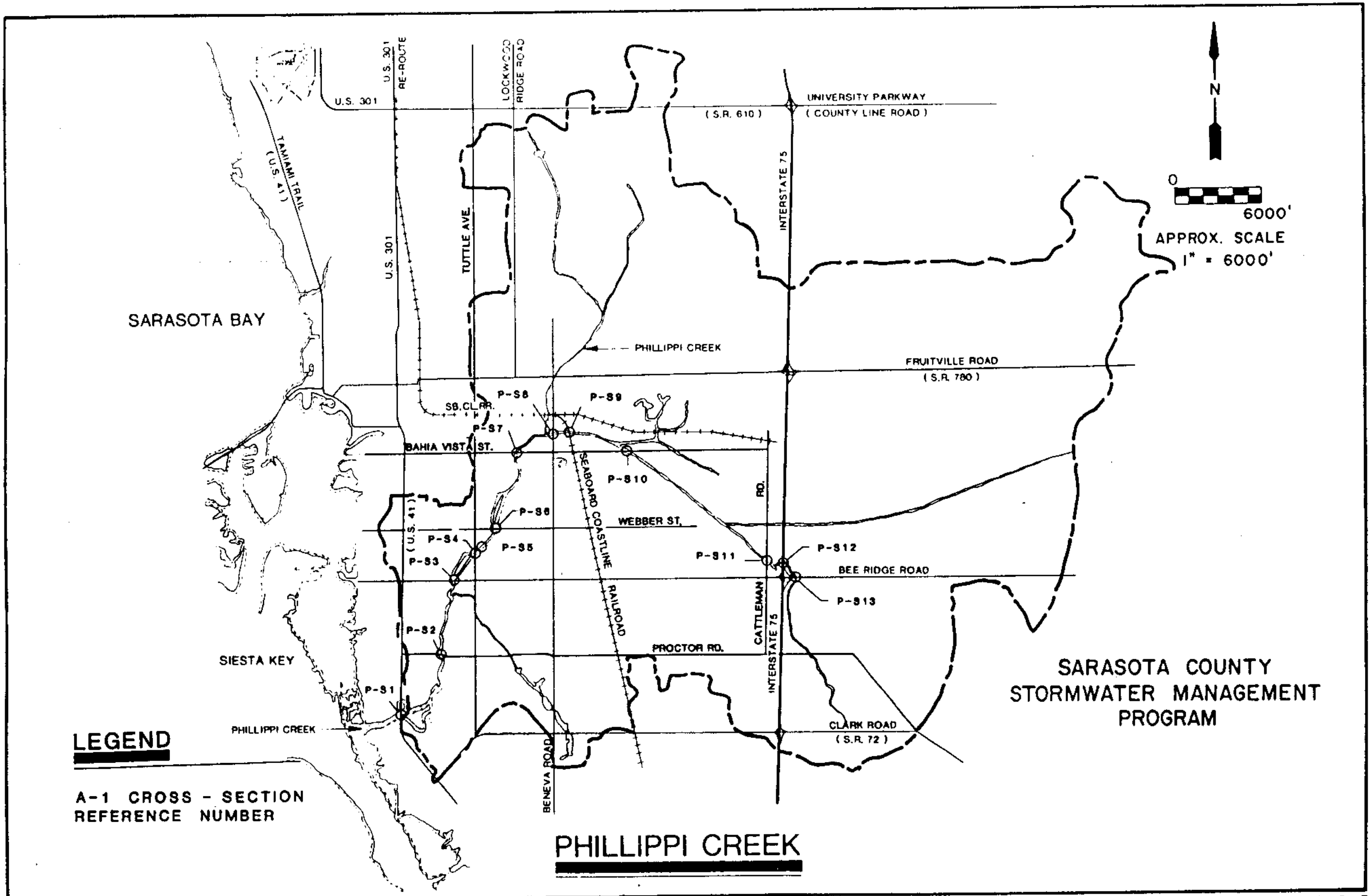
Thirty-two channel cross-sections resulting from the Lemonde/Biscayne survey were used to simulate the Phillippi Creek conveyance system.

All of the channel reaches have been idealized for the simulation as double-trapezoidal channels based on the downstream cross-section. Channel reaches not on the main channel (not explicitly surveyed as part of this report) have been idealized based on previous study survey data.

The MSSM model RUNOFF program computes the volume of stormwater runoff from each subbasin based on the input parameters for overland flow and the 25-year, 24-hour design rainfall event. The computed flows are then routed




**Phillippi Creek MSSM Model Network** **FIGURE 6-4**




Phillippi Creek Structures - Location Map

FIGURE 6-5

FIGURE 6-5

TABLE 6-2

PHILLIPPI CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| CB24/0301 | P-1 | U.S. 41 and approximately 5400 block Concrete Bridge Structure two spans - 192.0′ x 40.0′ |
| CB24 | P-1A | U.S. 41 and approximately 5300 block Concrete Bridge Structure - 436.0′ x 67.0′ |
| 0301 | P-1B | America Drive and just north of Constitution Blvd. - Concrete Bridge Structure |
| 0301 | P-2 | Proctor Road and approximately 800′ east of Riverwood Avenue - Concrete Bridge Structure |
| 0304/0305 | P-3 | Bee Ridge Road and approximately 1000′ east of Jaffa Drive - New Concrete Structure Bridge |
| 0304/0305 | P-4 | Tuttle Avenue and just south of Valencia Drive Concrete Bridge Structure |
| 0304/0305 | P-5 | NW of Tuttle Avenue and SW of Webber Street Saltwater Intrusion Barrier |
| 0304/0305 | P-6 | Webber Street and just east of Brink Avenue Concrete Bridge Structure - 145.8′ x 56.4′ |
| 0304/0305 | P-7 | Bahia Vista Street and just east of Lockwood Ridge Road - Conrete Bridge Structure - 100.0′ x 33.8′ |
| 0304/0309 | P-8 | Beneva Road and just south of Teate Drive Concrete Bridge Structure - 189.4′ x 78.0′ |
| 0304/0309 | P-9 | Seaboard Coast Line R.R. and approximately 1500′ east of Beneva Road - Wooden Railroad Trestle - 138.0′ x 9.0′ |
| 0312 | P-10 | Bahia Vista and just east of McIntosh Road Concrete Bridge Structure - 100.0′ x 29.0′ |
| 0314/0316 | P-11 | Cattlemen Road and approximately 1200′ north of Bee Ridge Road - Concrete Bridge Structure - 32.5′ x 29.2′ |

TABLE 6-2
(Continued)

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0314/0316 | P-12 | Interstate 75 and approximately 750' north of Bee Ridge - Concrete Bridge Structure - Two Spans |
| 0316 | P-13 | Bee Ridge Road and just east of I-75 overpass Concrete Bridge Structure - Arch Culvert 13.3' x 6.6' |
| 0303 | P-A1 | Brookside Drive and south of Forest Lane Structure - Bridge |
| 0303 | P-A2 | Swift Avenue and just north of Wilkinson Road Bridge Structure - 4-36" R.C.P. |
| 0303 | P-A3 | Wilkinson Road and just east of Swift Avenue Bridge Structure - Arch Culvert 60 sq. ft. |
| 0303 | P-A4 | Proctor Road and approximately 1180' east of Swift Road - Structure - Bridge |
| 0302 | P-A5 | Beneva Road and just north of Fox Run Road Concrete Bridge Structure - 2-28" x 42" CMP |
| 0302 | P-A6 | Clark Road and approximately 1000' east of Beneva Road - Concrete Bridge Structure 2-48" R.C.P. |
| 0306/0309 | P-B1 | Seaboard Coastline R.R. and approximately 200' west of Beneva Road - Railroad Trestle |
| 0306/0309 | P-B2 | Oakwood Blvd. and just east of Oakwood Blvd. East - Structure - Bridge |
| 0309 | P-B3 | Beneva Road and just south of Fruitville Road Structure - Bridge |
| 0309 | P-B4 | Fruitville Road and just east of Beneva Road Structure - Bridge |
| 0309 | P-B5 | Calliandra Drive and just south of Circus Blvd. Concrete Bridge Structure |

TABLE 6-2
(Continued)

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0311 | P-C1 | McIntosh Road and approximately 500' sout of S.C.L.R.R. - Bridge Structure |
| 0316/0321 | P-D1 | Cattlemen Road and just north of Webber Street Structure - Bridge |
| 0316/0321 | P-D2 | Interstate 75 and approximately 3600' north of Bee Ridge Road - Concrete Bridge Structure |
| 0320 | P-D3 | S. Leewynn Drive and approximately 200' south of Shadow Wood Lane - Structure - Bridge |

downstream using the Kinematic Wave Approximation which yields the peak flows along the reach for a given time increment.

The peak flows obtained from this simulation are used as input to the HEC-2 steady-state surface profile model. HEC-2 then computes the surface profiles along the stream using the outfall conditions of mean-high tide.

### 6.4.2 PARAMETER SELECTION

The parameter selection regime for Phillippi Creek is identical to that used for the Alligator Creek basin and discussed in Section 5.0 of this report.

Table 6-3 lists the subbasins in Phillippi Creek and the parameters that describe them. These parameters are based on the survey data, the results of the Basin Delineation Study, and the calibration procedure discussed in the following section. The infiltration parameters are the general soil group related parameters as discussed in the previous chapter for Alligator Creek. The depression storage parameters are similar to those used in the Alligator Creek simulation, as is the surface roughness.

The channels within the Phillippi Creek basin are idealized as double-trapezoidal channels with the lower trapezoid representing the main channel and the upper trapezoid representing the overbank channel or flood plain. These trapezoids were set to best represent the natural channel cross-sections preserving, as much as possible, the cross-sections area and wetted perimeter. Table 6-4 shows the parameters used for the main channel trapezoids, and Table 6-5 depicts the overbank channels for Phillippi Creek.

The channel roughness parameters selected for Phillippi Creek are again similar to those in the previous section. Manning's "n" values for roughness range from 0.3 at the lower and better defined reaches to 0.10 in the more poorly defined reaches.

TABLE 6-3
PHILLIPPI CREEK
EXISTING SUBBASIN DATA

| INT NUM | SUBAREA NUMBER | CHANNEL NUMBER | LENGTH (FT) | AREA (ACRE) | SLOPE (FT/FT) | PCT IMP | MANNING N IMP | MANNING N PERV | DEP STOR (IN) IMP | DEP STOR (IN) PERV | INFILTRATION RATE (IN/HR) MAX | INFILTRATION RATE (IN/HR) MIN | MAXIMUM INFL(IN) | INFIL DECAY (1/HR) | RETENTION STORAGE (AC-FT) | HYET NO |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 3010 | 3010 | 4000. | 1061. | .00203 | 20. | .018 | .320 | .200 | .100 | 1.7 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 2 | 3011 | 3011 | 2800. | 460. | .00200 | 20. | .018 | .320 | .200 | .100 | 1.7 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 3 | 3020 | 3902 | 8200. | 959. | .00085 | 11. | .018 | .320 | .250 | .100 | 4.1 | .75 | 3.8 | 2.0 | 9. | 3000 |
| 4 | 3030 | 3903 | 7000. | 1054. | .00314 | 11. | .018 | .320 | .200 | .100 | 4.1 | .96 | 3.8 | 2.0 | 2. | 3000 |
| 5 | 3040 | 3050 | 18000. | 1777. | .00094 | 8. | .018 | .320 | .100 | .100 | 4.1 | .96 | 3.8 | 2.0 | 0. | 3000 |
| 6 | 3050 | 3050 | 7000. | 1309. | .00242 | 12. | .018 | .320 | .200 | .100 | 1.8 | .17 | 2.2 | 2.0 | 0. | 3000 |
| 7 | 3060 | 3060 | 12800. | 1008. | .00063 | 12. | .025 | .400 | .200 | .200 | 1.8 | .18 | 2.2 | 2.0 | 9. | 3000 |
| 8 | 3070 | 3907 | 15200. | 2155. | .00111 | 11. | .018 | .320 | .200 | .100 | 3.7 | .78 | 3.8 | 2.0 | 0. | 3000 |
| 9 | 3080 | 3908 | 10500. | 1058. | .00040 | 5. | .018 | .320 | .150 | .100 | 2.0 | .24 | 2.2 | 2.0 | 0. | 3000 |
| 10 | 3091 | 3908 | 6900. | 800. | .00029 | 3. | .018 | .320 | .100 | .100 | 2.1 | .19 | 2.2 | 2.0 | 45. | 3000 |
| 11 | 3092 | 3908 | 5800. | 1000. | .00020 | 1. | .018 | .320 | .100 | .100 | 2.1 | .23 | 2.2 | 2.0 | 0. | 3000 |
| 12 | 3100 | 3100 | 15000. | 1174. | .00013 | 5. | .018 | .320 | .150 | .100 | 4.0 | .96 | 3.8 | 2.0 | 0. | 3000 |
| 13 | 3110 | 3110 | 8700. | 1091. | .00138 | 6. | .018 | .320 | .200 | .100 | 2.0 | .24 | 2.2 | 2.0 | 5. | 3000 |
| 14 | 3121 | 3121 | 2200. | 444. | .00318 | 1. | .018 | .320 | .150 | .100 | 2.0 | .23 | 2.2 | 2.0 | 0. | 3000 |
| 15 | 3122 | 3110 | 3000. | 412. | .00150 | 21. | .018 | .320 | .150 | .100 | 2.0 | .17 | 2.2 | 2.0 | 19. | 3000 |
| 16 | 3123 | 3123 | 11600. | 657. | .00112 | 8. | .018 | .320 | .300 | .100 | 2.0 | .17 | 2.2 | 2.0 | 0. | 3000 |
| 17 | 3130 | 3121 | 10500. | 948. | .00029 | 12. | .018 | .320 | .350 | .100 | 1.9 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 18 | 3140 | 3914 | 10800. | 1379. | .00074 | 1. | .018 | .320 | .150 | .100 | 2.1 | .23 | 2.2 | 2.0 | 0. | 3000 |
| 19 | 3150 | 3916 | 12800. | 1462. | .00094 | 10. | .018 | .320 | .150 | .100 | 2.0 | .24 | 2.2 | 2.0 | 10. | 3000 |
| 20 | 3160 | 3916 | 17250. | 1874. | .00087 | 1. | .018 | .320 | .150 | .100 | 2.1 | .24 | 2.2 | 2.0 | 0. | 3000 |
| 21 | 3170 | 3170 | 13500. | 1665. | .00089 | 1. | .060 | .400 | .250 | .200 | 2.1 | .23 | 2.2 | 2.0 | 60. | 3000 |
| 22 | 3180 | 3180 | 12500. | 2013. | .00056 | 1. | .018 | .320 | .100 | .100 | 1.1 | .14 | 1.6 | 2.0 | 0. | 3000 |
| 23 | 3190 | 3919 | 18700. | 2071. | .00096 | 1. | .018 | .320 | .100 | .100 | 2.1 | .24 | 2.2 | 2.0 | 0. | 3000 |
| 24 | 3200 | 3200 | 9500. | 1714. | .00105 | 1. | .018 | .320 | .100 | .100 | 2.1 | .24 | 2.2 | 2.0 | 0. | 3000 |
| 25 | 3210 | 3921 | 15000. | 2098. | .00020 | 1. | .018 | .320 | .100 | .100 | 1.1 | .14 | 2.2 | 2.0 | 0. | 3000 |
| 26 | 3220 | 3922 | 13000. | 1290. | .00077 | 12. | .060 | .400 | .100 | .150 | 2.0 | .18 | 2.2 | 2.0 | 15. | 3000 |
| 27 | 3230 | 3923 | 16800. | 3135. | .00119 | 1. | .060 | .400 | .200 | .250 | 2.1 | .24 | 2.2 | 2.0 | 60. | 3000 |

OTOTAL NUMBER OF SUBCATCHMENTS, 27
OTOTAL TRIBUTARY AREA (AC), 36068.00

TABLE 6-4
PHILLIPPI CREEK
CHANNEL INPUT DATA

*****E N G L I S H  U N I T S*****

| INT NUM | CHAN NUM | CHAN CONN | WIDTH (FT) | LENGTH (FT) | SLOPE (FT/FT) | SIDE SLOPES LEFT | SIDE SLOPES RIGHT | MANNING N | DEPTH (FT) | V MAX. (FPS) | Q MAX. (CFS) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 3010 | 3000 | 10.00 | 6000. | .00100 | 32.7 | 30.3 | .030 | 3.50 | 2.34 | 984.538 |
| + | + | | | | | | | | | | |
| 2 | 3011 | 3010 | 1.00 | 3000. | .00111 | 12.5 | 16.7 | .030 | 5.50 | 3.25 | 1450.638 |
| + | + | | | | | | | | | | |
| 3 | 3030 | 3011 | 42.00 | 4500. | .00044 | 5.9 | 7.1 | .030 | 5.30 | 2.45 | 994.323 |
| + | + | | | | | | | | | | |
| 4 | 3050 | 3030 | 117.00 | 8000. | .00028 | 3.3 | 3.3 | .030 | 6.80 | 2.67 | 2534.905 |
| + | + | | | | | | | | | | |
| 5 | 3060 | 3050 | 38.00 | 9000. | .00089 | 1.2 | 1.5 | .040 | 6.80 | 3.36 | 1073.499 |
| + | + | | | | | | | | | | |
| 6 | 3100 | 3060 | 29.00 | 2500. | .00056 | 1.9 | 1.8 | .040 | 6.20 | 2.42 | 606.537 |
| + | + | | | | | | | | | | |
| 7 | 3110 | 3100 | 30.00 | 3500. | .00045 | .8 | 1.4 | .040 | 8.00 | 2.53 | 779.052 |
| + | + | | | | | | | | | | |
| 8 | 3121 | 3123 | 23.00 | 5000. | .00029 | 1.2 | 1.1 | .040 | 10.00 | 2.19 | 756.298 |
| + | + | | | | | | | | | | |
| 9 | 3123 | 3110 | 21.00 | 3000. | .00057 | 1.2 | 1.3 | .040 | 8.30 | 2.75 | 720.097 |
| + | + | | | | | | | | | | |
| 10 | 3140 | 3121 | 1.00 | 3000. | .00245 | 5.0 | 2.0 | .040 | 4.00 | 2.90 | 173.826 |
| + | + | | | | | | | | | | |
| 11 | 3170 | 3210 | 20.00 | 5000. | .00089 | 2.5 | 2.5 | .075 | 10.00 | 1.97 | 887.360 |
| + | + | | | | | | | | | | |
| 12 | 3180 | 3170 | 20.00 | 800. | .00086 | 2.8 | 2.8 | .075 | 8.40 | 1.75 | 644.049 |
| 13 | 3200 | 3170 | 15.00 | 7000. | .00039 | 4.5 | 4.4 | .065 | 7.50 | 1.20 | 435.536 |
| 14 | 3210 | 3140 | 20.00 | 5000. | .00052 | 4.2 | 4.2 | .040 | 9.50 | 2.66 | 1518.263 |
| + | + | | | | | | | | | | |
| 15 | 3902 | 3903 | 5.00 | 3500. | .00057 | 2.8 | 2.8 | .040 | 6.00 | 1.93 | 252.735 |
| 16 | 3903 | 3904 | 5.00 | 3500. | .00260 | 2.8 | 2.8 | .040 | 6.00 | 4.13 | 539.777 |
| + | + | | | | | | | | | | |
| 17 | 3904 | 3030 | 11.00 | 3000. | .00230 | 1.8 | 1.8 | .100 | 2.50 | 1.06 | 41.160 |
| + | + | | | | | | | | | | |
| 18 | 3907 | 3060 | 20.00 | 8000. | .00155 | 1.6 | 1.6 | .040 | 11.20 | 5.26 | 2208.254 |
| 19 | 3908 | 3907 | 20.00 | 6000. | .00083 | 2.3 | 2.3 | .040 | 6.90 | 2.93 | 726.669 |
| 20 | 3914 | 3140 | 1.00 | 3000. | .00050 | 2.0 | 5.0 | .040 | 4.50 | 1.41 | 106.440 |
| + | + | | | | | | | | | | |
| 21 | 3916 | 3914 | 27.00 | 4500. | .00022 | 1.5 | 1.5 | .065 | 5.50 | .87 | 169.719 |
| + | + | | | | | | | | | | |
| 22 | 3919 | 3180 | 20.00 | 2000. | .00070 | 2.4 | 2.4 | .065 | 7.20 | 1.69 | 456.074 |
| 23 | 3921 | 3210 | 10.00 | 3500. | .00200 | 2.6 | 2.6 | .070 | 10.40 | 3.02 | 1175.729 |
| 24 | 3922 | 3923 | 23.00 | 2000. | .00100 | 2.1 | 2.1 | .075 | 7.30 | 1.81 | 508.277 |
| 25 | 3923 | 3921 | 13.00 | 9500. | .00084 | 2.4 | 2.4 | .075 | 7.80 | 1.59 | 391.351 |
| + | + | | | | | | | | | | |

OTOTAL NUMBER OF CHANNELS, 25
0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS
0*** NOTE *** DATA FOR DOUBLE TRAPEZOIDAL CHANNELS ARE FOR THE LOWER TRAPEZOID -
THE DATA FOR THE UPPER TRAPEZOID ARE GIVEN IN THE FOLLOWING OVERBANK TABLE

TABLE 6-5
PHILLIPPI CREEK
OVERBANK INPUT DATA

1

0SUMMARY OF OVERBANK CHANNEL DATA FOR BASIN 3

| INT NUM | CHAN NUM | WIDTH (FT) | SIDE SLOPES LEFT | RIGHT | MANNING N | DEPTH (FT) | V MAX. (FPS) | Q MAX. (CFS) |
|---|---|---|---|---|---|---|---|---|
| 1 | 3010 | 406.00 | 1.7 | .2 | .035 | 10.50 | 7.39 | 35396.89 |
| 2 | 3011 | 165.00 | 3.5 | 7.1 | .035 | 9.20 | 7.69 | 18555.19 |
| 3 | 3030 | 115.00 | 1.3 | 8.3 | .045 | 9.50 | 4.83 | 9332.42 |
| 4 | 3050 | 165.00 | 5.6 | 6.7 | .045 | 10.70 | 4.56 | 15580.00 |
| 5 | 3060 | 60.00 | 2.1 | 1.7 | .060 | 9.20 | 5.53 | 5723.32 |
| 6 | 3100 | 60.00 | 4.1 | 4.1 | .060 | 10.10 | 4.04 | 5367.68 |
| 7 | 3110 | 63.00 | 9.8 | 1.1 | .060 | 9.50 | 3.21 | 4485.64 |
| 8 | 3121 | 60.00 | 1.0 | 1.0 | .060 | 12.00 | 3.45 | 4176.74 |
| 9 | 3123 | 59.00 | 2.3 | .9 | .060 | 10.00 | 4.14 | 4199.95 |
| 10 | 3140 | 30.00 | 4.9 | 6.0 | .060 | 8.50 | 5.54 | 3920.18 |
| 11 | 3170 | 200.00 | 5.0 | 5.0 | .080 | 2.00 | 1.87 | 1628.51 |
| 12 | 3210 | 120.00 | 2.5 | 2.5 | .060 | 2.00 | 3.10 | 2545.37 |
| 13 | 3903 | 200.00 | 5.0 | 5.0 | .060 | 1.00 | 2.64 | 791.45 |
| 14 | 3904 | 22.00 | 4.7 | 4.7 | .060 | 6.50 | 2.72 | 1032.98 |
| 15 | 3914 | 33.00 | 4.9 | 6.0 | .060 | 6.50 | 2.44 | 1267.49 |
| 16 | 3916 | 48.00 | 1.9 | 1.9 | .100 | 9.00 | 1.50 | 1162.35 |
| 17 | 3923 | 200.00 | 5.0 | 5.0 | .060 | 10.00 | 2.99 | 8202.71 |
| 18 | 3919 | 200.00 | 5.0 | 5.0 | .060 | 10.00 | 2.88 | 7971.33 |
| 19 | 3180 | 200.00 | 5.0 | 5.0 | .060 | 10.00 | 3.08 | 8782.66 |

0TOTAL NUMBER OF OVERBANK CHANNELS, 19

0***NOTE*** Q MAX. AND V MAX. ARE FOR OVERBANK + MATCHING LOWER CHANNEL

### 6.4.3 RESULTS OF SUBBASIN RUNOFF

Following the calibration procedures, the 25-year, 24-hour design rainfall event was applied over the basin. Table 6-6 gives the peak runoff from each subbasin within Phillippi Creek. Peak runoff averages 200 cfs/sq. mi. within the basins. Typically, peak runoff has been found to be about 600 cfs/sq. mi. from small highly urbanized basins in Tampa as determined by Lopez and Woodham, 1983 (USGS WRF 82-42). The subbasins studied as part of the current modeling effort are somewhat larger compared to those in the USGS study, and include significant routing through local drainage systems. Thus, the predicted peak subbasin runoff should exhibit greater attenuation and the lower values are thought to be acceptable. Individual existing condition subbasin hydrographs have been provided to the County under separate cover.

Peak channel flows in each section of the creek are given in Table 6-7 and 6-8 for existing and future conditions respectively. It should be noted that the timing of the peak within the channels is lagged significantly if lakes and wetlands are present. Future development (input shown in Table 6-9) scenarios must include retention of regulated runoff, and thus must not increase downstream peak flows. Individual future condition hydrographs have been provided to the County under separate cover.

As a general check on the accuracy of the modeling for the entire creek, the values for peak runoff obtained from the MSSM model were compared to previous Phillippi Creek values and values from other studies. The MSSM peak outflow compares favorably with the previous Phillippi Creek studies, but in all cases is somewhat lower. This can be attributed to the apparent degree of accuracy of the model technique. Previous studies have relied primarily on the rational method with a lag on the rainfall. The model simulates the existing conditions throughout the basin routing the rainfall from the farthest point to the outfall, accounting for the basin lag and attenuation of the peak using the most accurate equations available.

TABLE 6-6

PHILLIPPI CREEK PEAK SUBBASIN FLOWS

| Basin Number | Peak Flow (cfs) |
|---|---|
| 301 | 411 |
| 302 | 307 |
| 303 | 487 |
| 304 | 532 |
| 305 | 498 |
| 306 | 301 |
| 307 | 574 |
| 308 | 243 |
| 309 | 369 |
| 310 | 130 |
| 311 | 609 |
| 312 | 489 |
| 313 | 429 |
| 314 | 269 |
| 315 | 322 |
| 316 | 320 |
| 317 | 296 |
| 318 | 320 |
| 319 | 299 |
| 320 | 417 |
| 321 | 215 |
| 322 | 399 |
| 323 | 528 |

TA 6-7

# PHILLIPPI CREEK EXISTING PEAK OUTFLOW

TIME IN HOURS
SUMMARY STATISTICS FOR CHANNELS/PIPES

1

| CHAN NUMBER | MAX. FLOW (CFS) | MAX. VELOCITY (FPS) | MAX. DEPTH (FT) | MAXIMUM COMPUTED FLOW (CFS) | MAXIMUM COMPUTED VELOCITY (FPS) | MAXIMUM COMPUTED DEPTH (FT) | TIME OF OCCURENCE HR. | MIN. | LENGTH OF SURCHARGE (MIN) | MAXIMUM SURCHARGE VOLUME (AC-FT) | RATIO OF MAX. TO DESIGN FLOW | RATIO OF MAX. DEPTH TO DESIGN DEPTH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3010 | 35396.9 | 7.4 | 14.00 | 5190.1 | 3.34 | 5.98 | 23 | 0 | .0 | .00000E+00 | .15 | .43 |
| 3011 | 18555.2 | 7.7 | 14.70 | 4858.3 | 4.63 | 8.38 | 23 | 0 | .0 | .00000E+00 | .26 | .57 |
| 3030 | 9332.4 | 4.8 | 14.80 | 4718.3 | 3.21 | 10.68 | 24 | 0 | .0 | .00000E+00 | .51 | .72 |
| 3050 | 15580.0 | 4.6 | 17.50 | 4337.8 | 2.79 | 9.00 | 24 | 0 | .0 | .00000E+00 | .28 | .51 |
| 3060 | 5044.1 | 5.4 | 15.00 | 3827.0 | 3.91 | 13.02 | 24 | 30 | .0 | .00000E+00 | .76 | .87 |
| 3100 | 3119.3 | 3.5 | 13.30 | 3037.4 | 2.55 | 13.14 | 24 | 30 | .0 | .00000E+00 | .97 | .99 |
| 3110 | 3364.7 | 3.1 | 15.50 | 2983.7 | 2.13 | 14.72 | 24 | 30 | .0 | .00000E+00 | .89 | .95 |
| 3121 | 3078.1 | 3.2 | 19.00 | 2518.5 | 2.37 | 17.26 | 25 | 0 | .0 | .00000E+00 | .82 | .91 |
| 3123 | 3860.6 | 4.1 | 17.60 | 2632.1 | 2.85 | 14.77 | 24 | 30 | .0 | .00000E+00 | .68 | .84 |
| 3140 | 2491.2 | 5.1 | 10.50 | 2287.9 | 3.74 | 10.16 | 25 | 30 | .0 | .00000E+00 | .92 | .97 |
| 3170 | ******* | 3.3 | 25.00 | 1013.6 | .32 | 10.21 | 26 | 0 | .0 | .00000E+00 | .01 | .41 |
| 3180 | 644.0 | 1.7 | 8.40 | 536.1 | 1.67 | 7.70 | 24 | 0 | .0 | .00000E+00 | .83 | .92 |
| 3200 | 435.5 | 1.2 | 7.50 | 337.3 | 1.13 | 6.70 | 23 | 30 | .0 | .00000E+00 | .77 | .89 |
| 3210 | 3820.2 | 3.5 | 13.50 | 1628.2 | 2.23 | 9.74 | 26 | 30 | .0 | .00000E+00 | .43 | .72 |
| 3902 | 252.7 | 1.9 | 6.00 | 215.5 | 1.86 | 5.65 | 16 | 30 | .0 | .00000E+00 | .85 | .94 |
| 3903 | 5600.0 | 4.5 | 11.00 | 532.8 | 4.12 | 5.97 | 16 | 30 | .0 | .00000E+00 | .10 | .54 |
| 3904 | 1033.0 | 2.7 | 9.00 | 525.2 | 2.26 | 7.01 | 17 | 0 | .0 | .00000E+00 | .51 | .78 |
| 3907 | 2208.3 | 5.3 | 11.20 | 665.9 | 3.79 | 6.00 | 24 | 0 | .0 | .00000E+00 | .30 | .54 |
| 3908 | 726.7 | 2.9 | 6.90 | 427.7 | 2.53 | 5.26 | 24 | 0 | .0 | .00000E+00 | .59 | .76 |
| 3914 | 748.8 | 2.2 | 9.00 | 689.2 | 1.58 | 8.73 | 24 | 0 | .0 | .00000E+00 | .92 | .97 |
| 3916 | 1189.8 | 1.5 | 14.50 | 483.0 | .95 | 9.24 | 17 | 30 | .0 | .00000E+00 | .41 | .64 |
| 3919 | 456.1 | 1.7 | 7.20 | 243.8 | 1.42 | 5.25 | 24 | 0 | .0 | .00000E+00 | .53 | .73 |
| 3921 | 1175.7 | 3.0 | 10.40 | 615.9 | 2.56 | 7.84 | 26 | 30 | .0 | .00000E+00 | .52 | .75 |
| 3922 | 508.3 | 1.8 | 7.30 | 292.5 | 1.54 | 5.38 | 17 | 0 | .0 | .00000E+00 | .58 | .74 |
| 3923 | ******* | 4.3 | 22.80 | 470.9 | .30 | 7.95 | 27 | 0 | .0 | .00000E+00 | .00 | .35 |

TOTAL NUMBER OF CHANNELS/PIPES. 25

0

0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS

0*** NOTE *** SUMMARY DATA FOR DOUBLE TRAPEZOIDAL CHANNELS IS FOR THE ENTIRE CHANNEL - (IE. LOWER+UPPER CHANNELS)

0*** NOTE *** THE MAXIMUM DEPTHS ARE THE DEPTHS CALCULATED AT THE END OF THE TIME INTERVAL -
THE MAXIMUM FLOWS ARE THE AVERAGE FLOWS OVER THE TIME INTERVAL

TABLE 8

# PHILLIPPI CREEK FUTURE PEAK OUTFLOW

1

## SUMMARY STATISTICS FOR CHANNELS/PIPES

| CHAN NUMBER | MAX. FLOW (CFS) | MAX. VELOCITY (FPS) | MAX. DEPTH (FT) | MAXIMUM COMPUTED FLOW (CFS) | MAXIMUM COMPUTED VELOCITY (FPS) | MAXIMUM COMPUTED DEPTH (FT) | TIME OF OCCURENCE HR. | MIN. | LENGTH OF SURCHARGE (MIN) | MAXIMUM SURCHARGE VOLUME (AC-FT) | RATIO OF MAX. TO DESIGN FLOW | RATIO OF MAX. DEPTH TO DESIGN DEPTH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3010 | 35396.9 | 7.4 | 14.00 | 6935.3 | 3.76 | 6.69 | 20 | 30 | .0 | .00000E+00 | .20 | .48 |
| 3011 | 18555.2 | 7.7 | 14.70 | 6530.0 | 5.06 | 9.43 | 20 | 0 | .0 | .00000E+00 | .35 | .64 |
| 3030 | 9332.4 | 4.8 | 14.80 | 6336.1 | 3.40 | 12.30 | 20 | 0 | .0 | .00000E+00 | .68 | .83 |
| 3050 | 15580.0 | 4.6 | 17.50 | 5927.6 | 2.91 | 10.59 | 20 | 0 | .0 | .00000E+00 | .38 | .61 |
| 3060 | 5723.3 | 5.5 | 16.00 | 5392.4 | 4.17 | 15.49 | 19 | 30 | .0 | .00000E+00 | .94 | .97 |
| 3100 | 5367.7 | 4.0 | 17.30 | 4523.5 | 2.82 | 16.03 | 19 | 0 | .0 | .00000E+00 | .84 | .93 |
| 3110 | 4485.6 | 3.2 | 17.50 | 4451.9 | 2.31 | 17.42 | 19 | 0 | .0 | .00000E+00 | .99 | 1.00 |
| 3121 | 4176.7 | 3.5 | 22.00 | 3861.3 | 2.62 | 21.18 | 18 | 30 | .0 | .00000E+00 | .92 | .96 |
| 3123 | 4100.0 | 4.1 | 18.30 | 3979.9 | 3.16 | 17.90 | 18 | 30 | .0 | .00000E+00 | .95 | .98 |
| 3140 | 3920.2 | 5.5 | 12.50 | 3432.7 | 4.09 | 11.85 | 18 | 30 | .0 | .00000E+00 | .88 | .95 |
| 3170 | 1628.5 | 1.9 | 12.00 | 1516.7 | 1.35 | 11.74 | 18 | 0 | .0 | .00000E+00 | .93 | .98 |
| 3180 | 8782.7 | 3.1 | 18.40 | 957.6 | 1.31 | 9.54 | 16 | 30 | .0 | .00000E+00 | .11 | .52 |
| 3200 | 435.5 | 1.2 | 7.50 | 412.6 | 1.19 | 7.32 | 18 | 30 | .0 | .00000E+00 | .95 | .98 |
| 3210 | 2545.4 | 3.1 | 11.50 | 2515.9 | 2.54 | 11.44 | 18 | 30 | .0 | .00000E+00 | .99 | .99 |
| 3902 | 252.7 | 1.9 | 6.00 | 215.5 | 1.86 | 5.65 | 16 | 30 | .0 | .00000E+00 | .85 | .94 |
| 3903 | 791.4 | 2.6 | 6.00 | 499.1 | 1.23 | 5.43 | 17 | 0 | .0 | .00000E+00 | .63 | .90 |
| 3904 | 1033.0 | 2.7 | 9.00 | 492.2 | 2.21 | 6.83 | 17 | 30 | .0 | .00000E+00 | .48 | .76 |
| 3907 | 2208.3 | 5.3 | 11.20 | 890.9 | 4.10 | 6.98 | 17 | 30 | .0 | .00000E+00 | .40 | .62 |
| 3908 | 726.7 | 2.9 | 6.90 | 561.8 | 2.72 | 6.01 | 17 | 30 | .0 | .00000E+00 | .77 | .87 |
| 3914 | 1267.5 | 2.4 | 11.00 | 1048.6 | 1.71 | 10.22 | 17 | 0 | .0 | .00000E+00 | .83 | .93 |
| 3916 | 1162.4 | 1.5 | 14.30 | 705.4 | 1.01 | 11.07 | 17 | 0 | .0 | .00000E+00 | .61 | .77 |
| 3919 | 7971.3 | 2.9 | 17.20 | 458.8 | .78 | 7.22 | 16 | 30 | .0 | .00000E+00 | .06 | .42 |
| 3921 | 1175.7 | 3.0 | 10.40 | 1030.2 | 2.92 | 9.83 | 18 | 30 | .0 | .00000E+00 | .88 | .94 |
| 3922 | 508.3 | 1.8 | 7.30 | 292.5 | 1.54 | 5.38 | 17 | 0 | .0 | .00000E+00 | .58 | .74 |
| 3923 | 8202.7 | 3.0 | 17.80 | 766.1 | 1.22 | 9.15 | 19 | 0 | .0 | .00000E+00 | .09 | .51 |

TOTAL NUMBER OF CHANNELS/PIPES, 25

0+ DENOTES DOUBLE TRAPEZOIDAL CHANNELS

0*** NOTE *** SUMMARY DATA FOR DOUBLE TRAPEZOIDAL CHANNELS IS FOR THE ENTIRE CHANNEL - (IE. LOWER+UPPER CHANNELS)

0*** NOTE *** THE MAXIMUM DEPTHS ARE THE DEPTHS CALCULATED AT THE END OF THE TIME INTERVAL

THE MAXIMUM FLOWS ARE THE AVERAGE FLOWS OVER THE TIME INTERVAL

TABLE 6-9
PHILLIPPI CREEK
FUTURE SUBBASIN INPUT DATA

1 WATERSHED DATA FOR BASIN NO. 3, BASIN NAME -

*****E N G L I S H   U N I T S*****

| INT NUM | SUBAREA NUMBER | CHANNEL NUMBER | LENGTH (FT) | AREA (ACRE) | SLOPE (FT/FT) | PCT IMP | MANNING N IMP | MANNING N PERV | DEP STOR (IN) IMP | DEP STOR (IN) PERV | INFILTRATION RATE (IN/HR) MAX | INFILTRATION RATE (IN/HR) MIN | MAXIMUM INFL(IN) | INFIL DECAY (1/HR) | RETENTION STORAGE (AC-FT) | HYET NO |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 3010 | 3010 | 4000. | 1061. | .00203 | 20. | .018 | .320 | .200 | .100 | 1.7 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 2 | 3011 | 3011 | 2800. | 460. | .00200 | 20. | .018 | .320 | .200 | .100 | 1.7 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 3 | 3020 | 3902 | 8200. | 959. | .00085 | 11. | .018 | .320 | .250 | .100 | 4.1 | .75 | 3.8 | 2.0 | 9. | 3000 |
| 4 | 3030 | 3903 | 7000. | 1054. | .00314 | 11. | .018 | .320 | .200 | .100 | 4.1 | .96 | 3.8 | 2.0 | 2. | 3000 |
| 5 | 3040 | 3050 | 18000. | 1777. | .00094 | 8. | .018 | .320 | .100 | .100 | 4.1 | .96 | 3.8 | 2.0 | 0. | 3000 |
| 6 | 3050 | 3050 | 7000. | 1309. | .00242 | 12. | .018 | .320 | .200 | .100 | 1.8 | .17 | 2.2 | 2.0 | 0. | 3000 |
| 7 | 3060 | 3060 | 12800. | 1008. | .00063 | 12. | .025 | .400 | .200 | .200 | 1.8 | .18 | 2.2 | 2.0 | 9. | 3000 |
| 8 | 3070 | 3907 | 15200. | 2155. | .00111 | 11. | .018 | .320 | .200 | .100 | 3.7 | .78 | 3.8 | 2.0 | 10. | 3000 |
| 9 | 3080 | 3908 | 10500. | 1058. | .00040 | 10. | .018 | .320 | .150 | .100 | 2.0 | .24 | 2.2 | 2.0 | 22. | 3000 |
| 10 | 3091 | 3908 | 6900. | 800. | .00029 | 8. | .018 | .320 | .100 | .100 | 2.1 | .19 | 2.2 | 2.0 | 45. | 3000 |
| 11 | 3092 | 3908 | 5800. | 1000. | .00020 | 10. | .018 | .320 | .100 | .100 | 2.1 | .23 | 2.2 | 2.0 | 42. | 3000 |
| 12 | 3100 | 3100 | 15000. | 1174. | .00013 | 8. | .018 | .320 | .150 | .100 | 4.0 | .96 | 3.8 | 2.0 | 27. | 3000 |
| 13 | 3110 | 3110 | 8700. | 1091. | .00138 | 9. | .018 | .320 | .200 | .100 | 2.0 | .24 | 2.2 | 2.0 | 22. | 3000 |
| 14 | 3121 | 3121 | 2200. | 444. | .00318 | 8. | .018 | .320 | .150 | .100 | 2.0 | .23 | 2.2 | 2.0 | 19. | 3000 |
| 15 | 3122 | 3110 | 3000. | 412. | .00150 | 21. | .018 | .320 | .150 | .100 | 2.0 | .17 | 2.2 | 2.0 | 19. | 3000 |
| 16 | 3123 | 3123 | 11600. | 657. | .00112 | 10. | .018 | .320 | .300 | .100 | 2.0 | .17 | 2.2 | 2.0 | 6. | 3000 |
| 17 | 3130 | 3121 | 10500. | 948. | .00029 | 12. | .018 | .320 | .350 | .100 | 1.9 | .18 | 2.2 | 2.0 | 0. | 3000 |
| 18 | 3140 | 3914 | 10800. | 1379. | .00074 | 12. | .018 | .320 | .150 | .100 | 2.1 | .23 | 2.2 | 2.0 | 32. | 3000 |
| 19 | 3150 | 3916 | 12800. | 1462. | .00094 | 10. | .018 | .320 | .150 | .100 | 2.0 | .24 | 2.2 | 2.0 | 10. | 3000 |
| 20 | 3160 | 3916 | 17250. | 1874. | .00087 | 10. | .018 | .320 | .150 | .100 | 2.1 | .24 | 2.2 | 2.0 | 0. | 3000 |
| 21 | 3170 | 3170 | 13500. | 1665. | .00089 | 12. | .060 | .400 | .250 | .200 | 2.1 | .23 | 2.2 | 2.0 | 70. | 3000 |
| 22 | 3180 | 3180 | 12500. | 2013. | .00056 | 10. | .018 | .320 | .100 | .100 | 1.1 | .14 | 1.6 | 2.0 | 75. | 3000 |
| 23 | 3190 | 3919 | 18700. | 2071. | .00096 | 10. | .018 | .320 | .100 | .100 | 2.1 | .24 | 2.2 | 2.0 | 58. | 3000 |
| 24 | 3200 | 3200 | 9500. | 1714. | .00105 | 12. | .018 | .320 | .100 | .100 | 2.1 | .24 | 2.2 | 2.0 | 79. | 3000 |
| 25 | 3210 | 3921 | 15000. | 2098. | .00020 | 12. | .018 | .320 | .100 | .100 | 1.1 | .14 | 2.2 | 2.0 | 96. | 3000 |
| 26 | 3220 | 3922 | 13000. | 1290. | .00077 | 12. | .060 | .400 | .100 | .150 | 2.0 | .18 | 2.2 | 2.0 | 15. | 3000 |
| 27 | 3230 | 3923 | 16800. | 3135. | .00119 | 13. | .060 | .400 | .200 | .250 | 2.1 | .24 | 2.2 | 2.0 | 130. | 3000 |

0TOTAL NUMBER OF SUBCATCHMENTS, 27

0TOTAL TRIBUTARY AREA (AC), 36068.00

The USGS has completed several studies on the runoff and flood volumes that can be attributed to natural streams. Generally, these values tend to be somewhat lower than the values found during the modeling effort. This can be attributed to the general nature of the equations and the degree of urbanization through most of the lower portions of the creek.

### 6.4.4 SUMMARY

The peak flows within Phillippi Creek as simulated are slightly less than the previous Smalley, Welford, and Nalven determination. This may be due to the methodology employed in the SWN report, wherein peak flows were calculated via the rational method. The MSSM computer simulation method deals with each basin as a separate entity and then routes the flow through the channel using the Kinematic Wave Approximation.

## 6.5 BACKWATER ANALYSIS

In order to evaluate the potential level of flooding in Phillippi Creek during the design storm even, the hydraulic (backwater) model was employed. The peak flows generated in the hydraulic analysis were input to the HEC-2 model to simulate the water surface profile along the channel. The model compares the backwater profiles with the channel carrying capacity, enabling an analysis of the flooding within the basin. The methodologies and assumptions used in the HEC-2 modeling has been discussed in Section 5.4.1.

### 6.5.1 MODEL DEVELOPMENT

The HEC-2 modeling effort for Phillippi Creek consisted of an approximate 12 mile stretch from below cross-section P-1 of the Lemonde-Biscayne field survey (approximately the middle of Roberts Bay) upstream through cross-section P-32 (west of Interstate 75 and north of Clark Road) to the extreme upstream end of the creek. A schematic diagram of the modeled system is presented in Figure 6-6.

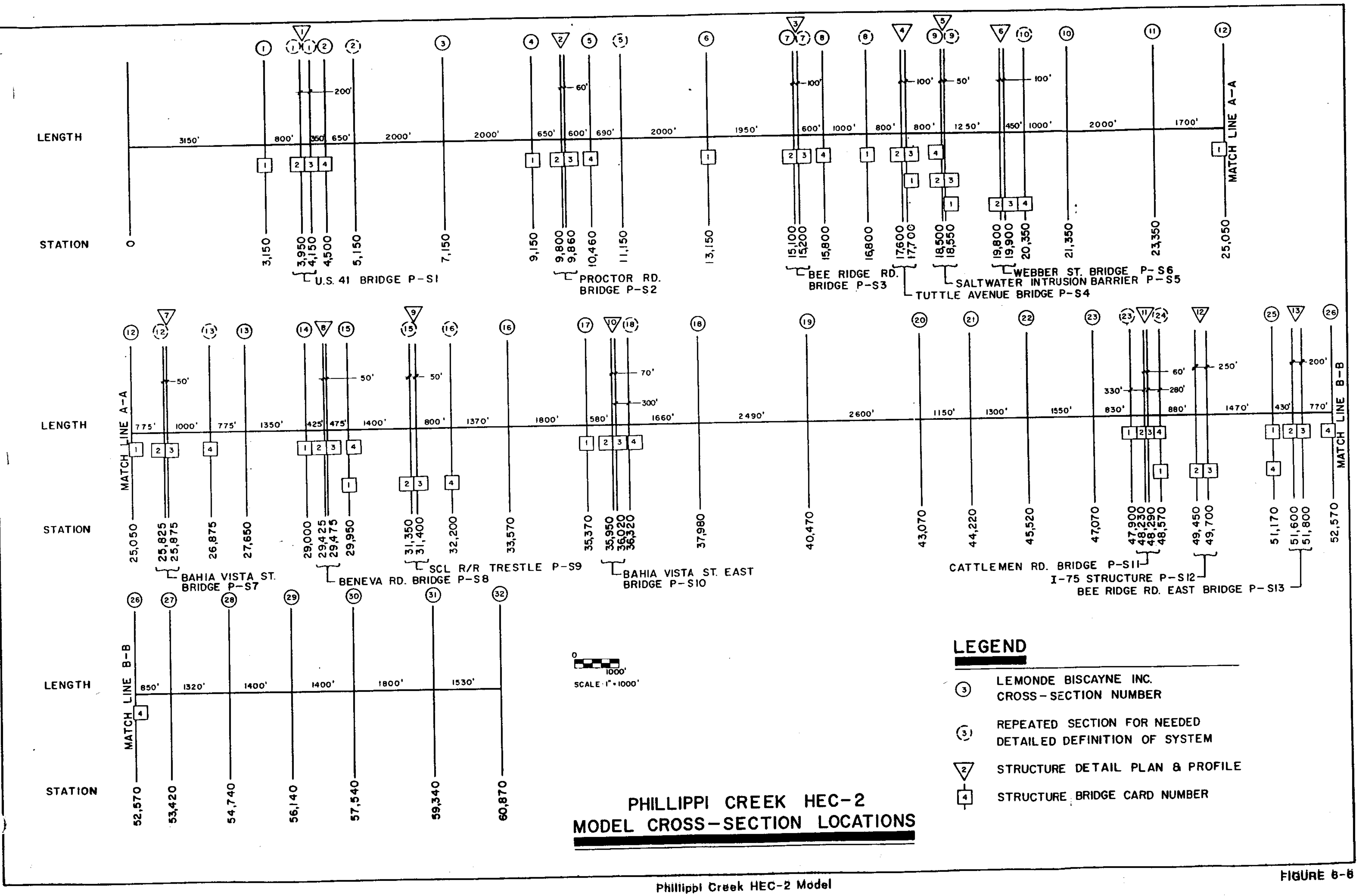



FIGURE 6-6

Phillippi Creek HEC-2 Model

Cross-Sections

The basic input data required for the backwater analysis model for Phillippi Creek includes the cross-sectional data for the main channels surveyed and detailed data on the 13 structures that cross the creek. The primary source of cross-sectional information, as well as the structure location and physical characteristics, was the Lemonde-Biscayne field survey completed as part of this project. The survey produced cross-section stations and elevations at 32 locations along Phillippi Creek (Appendix C). Detailed drawings of the major structures along the creek were developed by CDM from the field notes provided in the survey. The 32 cross-sections were used to develop the 65 cross-sections and bridge data points needed for model representation. The model requires that four cross-sectional data cards be supplied for each structure crossing being modeled; cross- sections some distance up and down stream of the crossing, and cross-sections on either side of the structure. Additionally, in some cases the cross-sections as provided by the surveyor were extended on either side of the creek to adequately reflect the flood plane and provide the necessary surface to accommodate the expected water surface elevations.

Structures

As stated previously, 13 structures were simulated during the modeling effort, beginning downstream with the U.S. 41 bridge and ending at the box culvert under Bee Ridge Road west of Interstate 75. In the HEC-2 model, energy losses through structures can be computed by either the normal or special bridge routines. The normal bridge method handles the cross-section at the bridge just as it should any other section except that the area of the bridge obstruction below the water level is subtracted from the total flow area. On the other hand, the special bridge routine is used where there is a possibility of not only low flow but also pressure or weir flow. Pressure flow occurs when the water level is above the low cord of the bridge but below the roadway surface. The head difference causes the flow through the bridge to be under pressure. Weir flow through a bridge occurs when the water level rises above the roadway elevation and the road is over-topped. The special bridge routine was used to represent all of the structures modeled along the creek.

The data required to simulate the structure crossings include upstream and downstream sections without the structure geometry. As mentioned above, two cross-sections are required to model the case of weir flow and pressure flow. The section used for the low flow case (or pressure of weir flow) describes the structure as a trapezoidal opening which is derived from the shape and area of the individual structures. The other section includes the total area of the opening (used for pressure flow), and the top of roadway (used for weir flow). The sections required for each structure crossing are labeled 1 through 32 in Figure 6-6.

Coefficients describing the pier shape (used in the energy equation) and head loss (used in the orifice flow equation) are input into the HEC-2 model to account for the head losses through structures. The pier shape coefficient was set to 1.25 to present the square nosed piers, typical of the structure crossings in Phillippi Creek, as suggested by the HEC-2 users manual. The loss coefficient for the orifice flow equation was set at 1.56 and was based on a typical value suggested by the Bureau of Public Roads, as stated in the HEC-2 Users Manual. A value of 2.6 was used for the weir flow equation. This value is typical of rectangular weir flow which is assumed to occur over the decks of bridges. The same values of coefficients were used at all structure crossings throughout the creek.

Roughness, Contraction, and Expansion Coefficients

The roughness coefficients used in the modeling of Phillippi Creek have been discussed in the section dealing with the calibration parameters for the MSSM model. The expansion and contraction coefficients, however, are used specifically in the HEC-2 model to account for energy losses where changes in the channel cross-sections occur. Generally, the changes occurring in the Phillippi Creek channel are relatively small. Typical values suggested by the HEC-2 Users Manual for small changes are 0.1 and

0.3 for the contraction and expansion coefficients respectively. Contraction and expansion coefficients for the structure sections were set to 0.3 and 0.5 respectively as suggested in the model Users Manual.

### 6.5.2 RESULTS OF DESIGN STORM SIMULATION

#### Existing Conditions

The 25-year design storm was analyzed using an outflow condition based on mean-high tide value of 2.5 ft NGVD. The results of the simulation are given in Table 6-10 (Figure 6-7) which includes the section number (SECNO), discharge (Q), average channel velocity (VCM), depth, water surface elevation (CWSEL), minimum ground elevation of the section (ELMIN), elevations of the left and right top of bank for each modeled cross-section (XLBEL & REBEL) and elevation of the roadway (ELTRO).

The 25-year storm analysis resulted in water surface elevations ranging from 2.74 at the U.S. 41 bridge to 29.16 at cross-section P-32. Water depths range from 4.00 feet at the bay to 19.88 at cross-section P-9. In general, water surface profiles did not exceed bank full in the lower reaches of the creek.

#### Future Conditions

The results of the 25-year design storm simulation for the future conditions using the MSSM models were used as input for the HEC-2 model. The results of this simulation are given in Table 6-11 and shown in Figure 6-7.

The future conditions create flooding in essentially the same areas as identified for the existing condition except they are to a greater extent.

LE 6-10

# PHILLIPPI CREEK HEC-2 OUTPUT EXISTING CONDITIONS

THIS RUN EXECUTED 01/08/86 07:05:15

```
**************************************************
  HEC2 RELEASE DATED NOV 76 UPDATED MAY  1984
  ERROR CORR -  01,02,03,04,05,06
  MODIFICATION -  50,51,52,53,54,55,56
  IBM-PC-XT VERSION APRIL 1985
**************************************************
```

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

EAR 24-HOUR STORM EVENT

SUMMARY PRINTOUT

| SECNO | XLCH | Q | VCH | DEPTH | CWSEL | ELMIN | XLBEL | RBEL | ELTRD |
|---|---|---|---|---|---|---|---|---|---|
| .000 | .00 | 5190.00 | .43 | 4.00 | 2.50 | -1.50 | 3.00 | 3.00 | .00 |
| 3150.000 | 3150.00 | 5190.00 | 2.58 | 7.54 | 2.54 | -5.00 | 3.50 | 3.00 | .00 |
| 3950.000 | 800.00 | 5190.00 | 2.63 | 9.73 | 2.73 | -7.00 | 13.50 | 13.50 | .00 |
| 4150.000 | 200.00 | 5190.00 | 2.63 | 9.74 | 2.74 | -7.00 | 13.50 | 13.50 | 13.50 |
| 4500.000 | 350.00 | 5190.00 | 5.04 | 6.59 | 2.74 | -3.85 | 9.05 | 5.00 | .00 |
| 5150.000 | 650.00 | 5190.00 | 4.33 | 7.36 | 3.51 | -3.85 | 9.05 | 5.00 | .00 |
| 7150.000 | 2000.00 | 4858.00 | 4.95 | 13.01 | 4.91 | -8.10 | 4.90 | 3.40 | .00 |
| 9150.000 | 2000.00 | 4858.00 | 3.38 | 10.75 | 6.05 | -4.70 | 4.70 | 3.70 | .00 |
| 9800.000 | 650.00 | 4718.00 | 1.76 | 13.32 | 6.32 | -7.00 | 14.15 | 14.15 | .00 |
| 9860.000 | 60.00 | 4718.00 | 1.76 | 13.33 | 6.33 | -7.00 | 14.15 | 14.15 | 14.15 |
| 10460.000 | 600.00 | 4718.00 | 2.86 | 13.27 | 6.37 | -6.90 | 4.40 | 5.60 | .00 |
| 11150.000 | 690.00 | 4718.00 | 2.77 | 13.41 | 6.51 | -6.90 | 4.40 | 5.60 | .00 |
| 13150.000 | 2000.00 | 4718.00 | 3.98 | 11.39 | 6.89 | -4.50 | 4.50 | 4.00 | .00 |
| 15100.000 | 1950.00 | 4338.00 | 2.87 | 12.64 | 7.64 | -5.00 | 11.17 | 9.80 | .00 |
| 15200.000 | 100.00 | 4338.00 | 2.86 | 12.65 | 7.65 | -5.00 | 11.17 | 9.80 | 16.10 |
| 15800.000 | 600.00 | 4338.00 | 3.61 | 13.38 | 7.78 | -5.60 | 5.20 | 9.90 | .00 |
| 16800.000 | 1000.00 | 4338.00 | 3.49 | 13.64 | 8.04 | -5.60 | 5.20 | 9.90 | .00 |

## 6.6 IDENTIFICATION OF FLOOD PRONE AREAS

The HEC-2 backwater model was used to simulate the elevation of the flood waters resulting from the 25-year 24-hour design storm for both the existing and future land use conditions. Generally, the modeling effort along the mainstem of Phillippi Creek from the outfall to cross-section 32 identified one major area of flooding. However, the backwater effects on one of the major tributaries also causes some localized flooding within the tributaries' drainage basins.

### 6.6.1 MAINSTEM FLOODING

As stated, the flooding along the mainstem of Phillippi Creek occurs upstream of the Bahia Vista bridge. The head loss experienced through the bridge and the general over-bank condition along the main channel upstream of the bridge must be relieved. The bridge can currently pass only about two-thirds of its design flow due in part to approximately one-third of its flow area being restricted because one of its three spans has been filled in with earth.

Simulations of the model were used wherein the bridge was removed from the model to allow the Creek to flow without constriction. It was found that removal of the bridge, while relieving most of the main channel flooding upstream, caused significant flooding below the bridge. Thus, even if the bridge is replaced with a structure with greater flow capacity, flooding will still occur, except that now it will be in areas downstream of the bridge. Clearly, another method of flood relief for the main channel would have to be provided--either increased channel capacity downstream of the structure or some measure of storage upstream.

### 6.6.2 SUBBASIN FLOODING

Investigation of the main channel of Phillippi Creek, while identifying one area of mainstem flooding, also identified a subbasin area where flooding occurs. The subbasins identified as having flooding problems are in the Clark Lakes area. Though not specifically modeled, this area has a history of flooding caused primarily by insufficient channel capacity to drain the over 2,000 acres. Channel capacity is estimated to be less than half of what is needed to provide the required flow.

As can be seen, the peak flow elevations for the future conditions within the basin are greatly increased (Figure 6-7). In contrast, elevations for future conditions within Alligator Creek display very little difference from the elevations for existing conditions. This is explained by reviewing how the regulated system (17-25 FAC and 40D-4 SWFWMD) is designed, and the location of development. New developments are required to capture the difference between the pre- and post-development peak runoff to maintain the pre-development peak flows. This is accomplished by removing the top of the post-development hydrograph through the use of storage, and then "bleeding off" the storage areas to their normal levels within a specified time increment. Thus, the peak flow is maintained. However, the volume of flow from the developed site is increased. This increase in volume occurs during what is referred to as the recession limb of the runoff hydrograph. Where the pre-development hydrograph shows a rapid recession to a low flow condition, the post-development hydrographs show a slow recession limb with increased flows from the site. If the development occurs at the upstream area of the basin, as in Alligator Creek, the bleed-off has minimal effect on the system. If the bleed-off is occurring downstream of a large portion of the basin as in Phillippi Creek, the subsequent peak flows may be greater.

In addition to the two major problem areas identified, the basin suffers from a general lack of maintenance resulting in restrictive flow in many of the basin's main channels and tributaries. As part of the master plan, a basin wide maintenance plan will be developed.

## 6.7 IDENTIFICATION OF STORMWATER MANAGEMENT ALTERNATIVES

The flooding of areas identified in Section 6.5 must be controlled to ensure that the public welfare is maintained and meets the requirement of the service levels as identified in Section 2.5 of this report. As stated previously, flood control measures usually fall into three major categories: additional channel capacity, storage, or flood proofing. A discussion of these three alternative measures is given in Section 5.6.

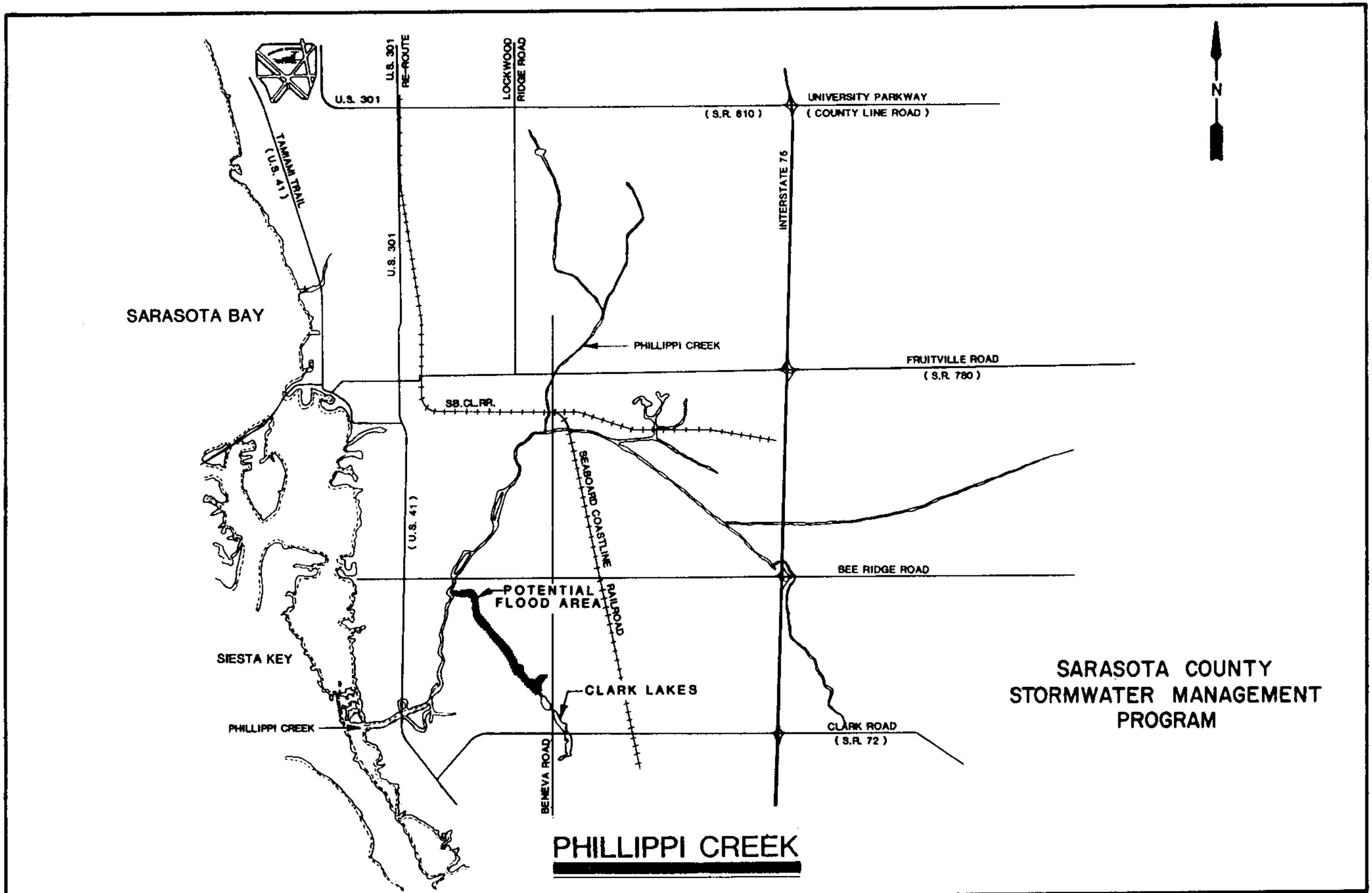
### 6.7.1 SUBBASIN FLOODING

Subbasin flooding mainly occurs in the area identified and generally referred to as Clark Lakes (Figure 6-8). As discussed, flooding in this area is generally due to insufficient channel capacity north of the lakes along the outflow channel. The area is developed to such an extent that there is virtually no land available for increasing the outflow channel size without the condemnation of many private homes to acquire the necesary rights-of-way. In addition, there has been a significant development recently within the flood plane of the channel, leaving no room for lake expansion. Three methods of flood control are suggested for the Clark Lakes area including: increasing the channel capacity, providing for increased storage, and diverting the flow. These methods of flood control are discussed below:

1. Increasing the channel capacity out of the Clark Lakes area can only be accomplished by condemnation of the private properties and residences which line the channel. Sufficient rights-of-way for the construction and maintenance of an outflow channel with the capacity required is an expensive proposition.

2. Increased storage within the area, for the purpose of flood reduction, has the advantage of providing not only flood control but also quality enhancement. However, there is insufficient land area available to provide all of the needed storage without some condemnation of private property.

3. Flow diversion is another method of flood control that was investigated. For the Clark Lakes area, there are two possible flow diversion alternatives. These two alternatives are discussed as follows:




Potential Flooding of Clark Lakes

FIGURE 6-8

FIGURE 6-8

a. Divert the flow from the southern most lake to the Matheny Creek basin that is located just below the lake. Runoff, to a small extent, naturally flows over the bank of the lake and runs by sheet flow to the channel. However, the Matheny Creek basin is a separate and distinct basin which itself has problems. Allowing the natural flow to occur is relatively insignificant to the basin outflow, but a major diversion of flow to Matheny may cause significant deleterious effects; such as increased flood elevations, increased flows and even habitat disruption.

b. Divert flow from the northern most lake to the main channel of the creek by using a right-of-way along Ashton Road. This could be accomplished through the use of a pump station, and the construction of a channel or pipe network that would allow the flow to be taken westward to the creek.

The subbasin flood mitigation alternatives are diverse in nature. The costs associated with the alternatives range from an estimated $2.0 million for the by-pass channel alternative to a high of $6.5 million for the creation of new storage areas alternative. It must again be emphasized that the alternatives presented for the subbasin problems have not been analyzed in-depth, but are merely presented as possible suggested mitigation methods.

6.7.2 MAINSTEM FLOODING

The flooding problems encountered on the mainstem are caused by the lack of flow capacity under the Bahia Vista bridge, and insufficient channel capacity between Interstate 75 and Beneva Road. Additionally, the replacement of the Bahia Vista Road bridge (currently under design) was considered to occur as scheduled, with sufficient capacity to pass the flow required.

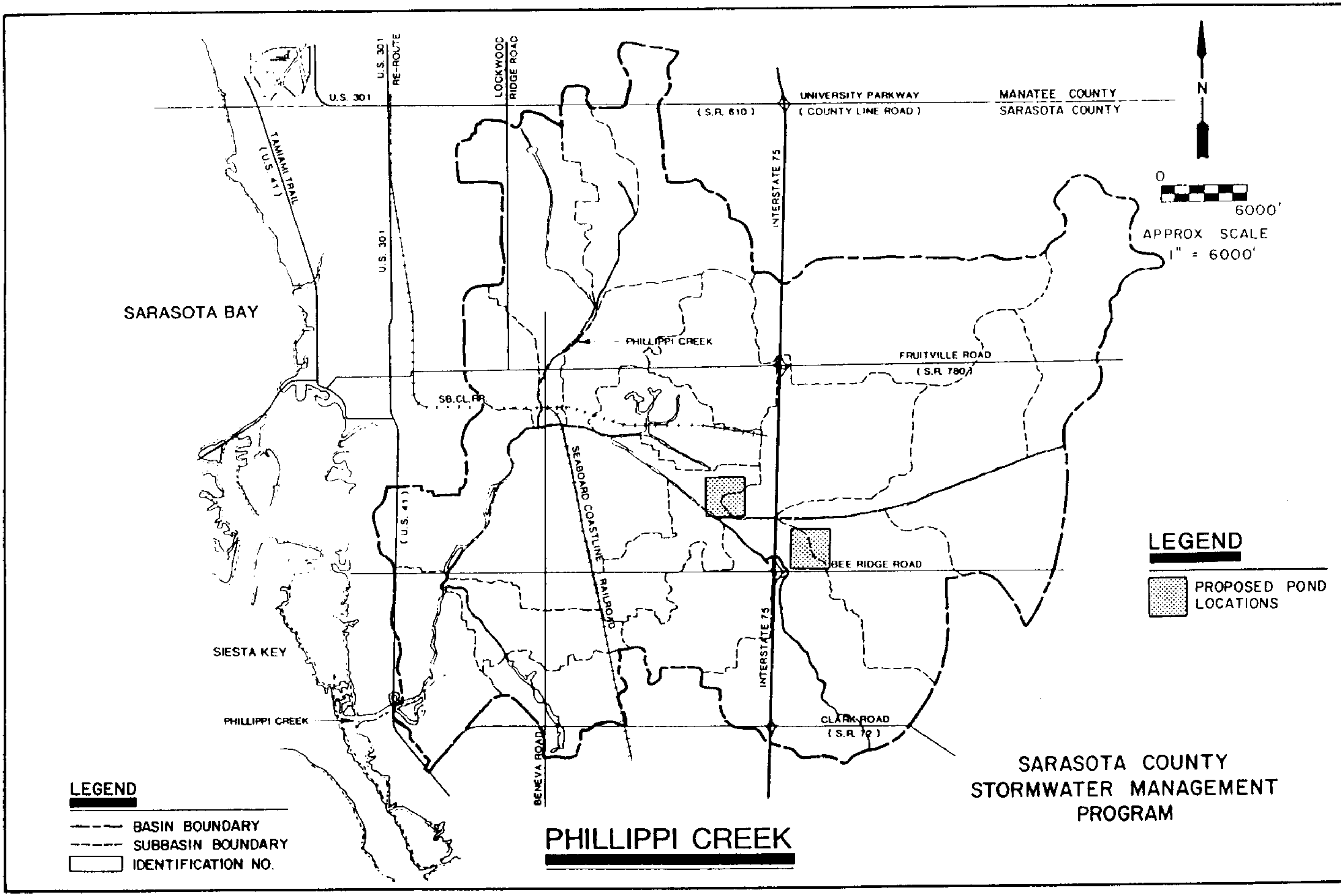
Two alternatives to solve this problem have been identified and investigated. The results of these investigations are given below:

1. The first alternative is the placement of a pond(s) upstream of Bahia Vista to provide the amount of storage required for the flood flows in this area of Phillippi Creek (1,700 acre-ft). It is recommended that two smaller ponds be constructed rather than one large pond. The ponds, located in areas near those shown in Figure 6-9, would be wet bottom types, designed to retain first flushes of runoff from the upstream areas, and detain a larger amount.

   The area upstream is generally agricultural land which typically means high nutrients and pesticides loadings within the stormwater runoff stream. The future conditions will see more development and a change in the characteristics of the stormwater flow. It is desireable for the County to develop a regional detention system to make future stormwater management more cost effective.

   It is recommended that the sites for the ponds be much larger than the minimum required so that recreation facilities or parks can be created, thus giving the county dual use facilities. These dual use facilities should be designed such that the facility is fully functional as a recreation area until such time as the large storm event occurs, and then returns to its desired use as rapidly as possible. The cost of these ponds is outlined in Table 6-12.

2. The second alternative involves the replacement of the Bahia Vista bridge, and the increase of the channel capacity downstream. Channel capacity in this reach would have to be almost doubled to allow for the required flow. This alternative would require the purchase of a significant




**Proposed Location Of Detention Ponds**

FIGURE 6-9

TABLE 6-12

PHILLIPPI CREEK

PROBLEM AREA MAIN CHANNEL CARRYING CAPACITY

SOLUTION: CREATE NEW DETENTION/RETENTION PONDS

ESTIMATED COST

| CAPITAL PROJECT | | | | | COST |
|---|---|---|---|---|---|
| LAND PURCHASE | 250 | ACRES | @ | $15,000 | $3,750,000 (1) |
| EARTHWORK | 560000 | CU-YD | @ | $4.00 | $2,240,000 (2) |
| OUTFLOW AND INFLOW STRUCTURES | | | | | $70,000 |
| RESODDING | 18000 | SQ-YD | @ | $4.00 | $72,000 |
| | | SUB-TOTAL........ | | | $6,132,000 |
| CONTINGENCIES @ 20 % | | | | | $1,226,400 |
| ESTIMATE OF CONSTRUCTION COSTS...... | | | | | $7,358,400 |
| ENGINEERING AND DESIGN @ 15% | | | | | $1,103,760 |
| TOTAL COSTS------------------------ | | | | | $8,462,160 |

BENEFITS:

(A) PROVIDE FOR RETENTION/DETENTION FOR QUANTITY AND QUALITY IMPROVEMENT, RELIEVING CLARK LAKES FLOODING.

(B) PROVIDE TWO DUAL USE FACILITIES IN WHICH STORMWATER MANAGEMENT FACILITIES EXIST WITH RECREATIONAL FACILITIES.

(C) PROVIDES FOR THE CURRENT FLOW PATTERNS TO EXIST DURING PERIODS OF LOW FLOW.

NOTE:

(1) LAND PURCHASE PRICE IS AN ESTIMATE OF THE COST AND DOES NOT REFLECT PROPERTY APPRAISERS VALUES. ALSO, PARKS AND RECREATION DEPARTMENT MAY BE WILLING TO CONTRIBUTE.

(2) SOIL REMOVED FROM THE SITE MAY BE OF USE AS CONSTRUCTION FILL MATERIAL. THUS, THE COST OF EXCAVATION MAY BE ELIMINATED.

amount of land along the channel by either outright purchase or condemnation of existing homesites. The costs associated with this alternative are shown in Tables 6-13 and 6-14.

## 6.8 RECOMMENDATIONS

The Phillippi Creek in-depth study of the mainstem has identified one mainstem and one subbasin area where flooding is caused by the 25-year 24-hour design storm. Alternative solutions for these problem areas were presented in the preceding section. The purpose of this section is to aid in the final selection process to allow for its inclusion in the five-year Capital Improvement Program (CIP). The ranking system used herein ranks the mainstem alternatives using the results of the previously discussed simulations. The subbasin alternatives are ranked solely on the basis of perceived benefits and costs--no simulation analysis was performed.

It must be noted that the alternatives for mainstem improvements described in the preceding section have already passed through the preliminary screening process. This preliminary screening process was accomplished through the aid of the hydrologic and hydraulic models employed for the study. Thus, the alternatives described are those alternatives which were found to provide some measure of flood relief and have a reasonable expectancy for approval and construction. Alternatives that dealt with cross-basin flow routing and individual storage were dismissed during this preliminary phase of alternative selection due to the reasons mentioned previously.

### 6.8.1 RANKING

The ranking process described in Section 2.4 was employed to position the stormwater management alternatives so that a final selection can be made. The alternative ranking processes are described in the following paragraphs.

TABLE 6-13

PHILLIPPI CREEK

PROBLEM AREA BAHIA VISTA BRIDGE

SOLUTION: REPLACE BAHIA VISTA BRIDGE

ESTIMATED COST

| CAPITAL PROJECT | COST |
|---|---|
| NEW BRIDGE STRUCTURE CURRENTLY UNDER DESIGN CONTRACT<br>32' X 120' ROAD SURFACE 3840 SQ.FT.<br>@ $125.00 SQ.FT. | $480,000 * |
| CONTINGENCIES @ 20% | $96,000 |
| BEST ESTIMATE OF CONSTRUCTION COSTS | $576,000 |
| DESIGN AND ENGINEERING @ 15% | $86,400 |
| TOTAL COSTS | $662,400 |

BENEFITS:

A) Replacement of existing bridge structure
B) Maintenance of existing flow patterns
C) Pedestrian passage lane
D) Control of the 25-year design storm

* ESTIMATE OF BRIDGE COST IS PURELY FOR COMPARISON PURPOSES. CONSULT WITH DESIGN ENGINEERS (DSA) FOR ACTUAL ESTIMATE COST

TABLE 6-14

PHILLIPPI CREEK

PROBLEM AREA MAIN CHANNEL CARRYING CAPACITY

SOLUTION: INCREASE CHANNEL CAPACITY

ESTIMATED COST

| CAPITAL PROJECT | | | | | COST | |
|---|---|---|---|---|---|---|
| LAND PURCHASE | 65 | ACRES | @ | $75,000 | $4,875,000 | (1) |
| EARTHWORK | 300000 | CU-YD | @ | $4.00 | $1,200,000 | (2) |
| FLOW CONTROL STRUCTURES | | | | | $48,000 | |
| RESODDING | 19000 | SQ-YD | @ | $4.00 | $76,000 | |
| NEW BRIDGE STRUCTURES | | | | | $182,000 | |
| | | SUB-TOTAL........ | | | $6,381,000 | |
| CONTINGENCIES @ 20 % | | | | | $1,276,200 | |
| ESTIMATE OF CONSTRUCTION COSTS...... | | | | | $7,657,200 | |
| ENGINEERING AND DESIGN @ 15% | | | | | $1,148,580 | |
| TOTAL COSTS------------------------- | | | | | $8,805,780 | |

NOTE:

(1) LAND PURCHASE PRICE IS AN ESTIMATE OF THE COST AND DOES NOT REFLECT PROPERTY APPRAISERS VALUES. ALSO, PARKS AND RECREATION DEPARTMENT MAY BE WILLING TO CONTIRBUTE.

(2) SOIL REMOVED FROM THE SITE MAY BE OF USE AS CONSTRUCTION FILL MATERIAL. THUS, THE COST OF EXCAVATION MAY BE ELIMINATED.

## Reliability

The reliability of the system is concerned with the system's ability to provide flood protection for flows expected from the 25-year 24-hour design storm. Additionally, the alternative is considered reliable if, for storms greater than the design storm, the alternative continues to function as designed. Generally, for the purposes of this ranking methodology, the higher rankings will go to those alternatives which in themselves have the capacity to alleviate the total problem associated with a particular problem area. The majority of the alternatives discussed previously provide for the necessary flood control. Therefore, the reliability factor is used in many cases to rate the reliability of the alternative for flows greater than the 25-year 24-hour design storm.

## Environmental Impact

The environmental impacts of the various stormwater alternatives are minimized in that all of the alternatives must meet the requirements of the applicable local, state, and federal regulations. However, some of the alternatives discussed provide for a measure of improvement in the quality of stormwater runoff. Notably, those alternatives that provide for retention and detention with filtration have a positive effect on the quality of stormwater. Additionally, alternative methodologies which not disrupt the local ecosystem would be deemed most acceptable. For these reasons, the detention/retention storage and the improved or new channels with filtration on the existing ponds was rated better than those alternatives that had no measurable environmental impact.

## Public Awareness/Acceptance

The public's impression of a project can mean the difference between whether the project is accepted or rejected. Generally, the public is perceived to accept the alternatives that will prevent flooding of their specific areas, while causing no alteration in their lifestyle. Typically, those alternatives that involve the purchase of existing homesites are not readily accepted. Thus, as can be seen in Table 6-15, the alternatives that involve large purchases of developed land are not thought to be high in public acceptance.

Master Plan Agreement

This engineering criteria is concerned with the integration and functioning of the entire stormwater management system within the Phillippi Creek basin. The mainstem alternative and the perceived subbasin alternative were ranked with regard to their agreement with the Master Plan. The basin-wide maintenance program was found to be in total agreement with the proposed Master Plan. The mainstem and subbasin alternatives were ranked as to their general agreement, with those offering increased quality of the runoff having the higher ranking.

Implementability

The implementability of an alternative is a measure of the ease in which the alternative can be constructed, permitted, funded, and politically accepted. Typically, those alternatives which may involve the condemnation and purchase of private houses are considered to be difficult to implement. Alternatives which exhibit this characteristic include new ponds, new outfalls, and the improvement of existing channels. Environmental considerations, especially the permitting process, make alternatives that do not provide a significant water quality benefit unattractive. Thus, as can be seen in Table 6-15, those alternatives that involve the technical features discussed are ranked lower than some of the other alternatives.

6.8.2 CONCLUSION

As far as the final ranking of the alternatives for both the mainstem and the subbasin problem areas indicate, the basin-wide maintenance program should be implemented. The mainstem problem will be best served through the construction of a wet-bottomed detention pond(s) located near Interstate 75 and Bee Ridge Road East. The pond(s) have been selected over channel improvement due to the improved quality of the runoff from the

TABLE 6-15

PHILLIPPI CREEK
ENGINEERING ANALYSIS
25-YEAR STORMWATER CONTROL ALTERNATIVES

| Stormwater Control Options | Evaluation Factors (1) | (2) | (3) | (4) | (5) | (Total) | Final Problem Area Ranking |
|---|---|---|---|---|---|---|---|
| BASIN WIDE | | | | | | | |
| 1. Institute Basin Wide Maintenance Program | 5 | 3 | 4 | 5 | 5 | 22 | 1 |
| PROBLEM AREA - 1 CLARK LAKES | | | | | | | |
| 1. Increase Outlet Channel Capacity | 2 | 1 | 0 | 3 | 1 | 7 | 4 |
| 2. Create Retention/Detention Ponds | 3 | 3 | 3 | 3 | 4 | 16 | 1 |
| 3. Flow Diversion | | | | | | | |
| A. Souther Lake Division | 1 | 3 | 2 | 2 | 3 | 11 | 2 |
| B. Pump From Northern Lake | 2 | 2 | 1 | 2 | 2 | 9 | 3 |
| PROBLEM AREA - 2 BAHIA VISTA BRIDGE | | | | | | | |
| 1. Replacement of Bahia Vista Bridge | 3 | 4 | 3 | 4 | 5 | 19 | 1 |
| 2. Increase Channel Capacity | 2 | 2 | 2 | 3 | 3 | 12 | 2 |
| 3. Create Retention/Detention Ponds | 4 | 3 | 4 | 4 | 4 | 19 | 1 |

(1) = Reliability Factor
(2) = Environmental Factor
(3) = Public Awareness/Acceptance Factor
(4) = Master Plan Agreement Factor
(5) = Implementability Factor

agricultural lands and the pond(s) dual-use facility nature. The subbasin improvement perceived ranking indicates that the detention pond alternative should be selected. The cost of this alternative will be used in the calculation of the five-year Capital Improvement. The flood profile with the recommended alternative is shown in Figure 6-10.

## 6.9 CAPITAL IMPROVEMENT PLAN

To facilitate the implementation of the recommended stormwater management facilities, it is necessary to devise an implementation strategy which will allow the orderly design, acquisition, and construction of the needed facilities. This, as with the Alligator Creek improvements, can best be accomplished through the use of a five year Capital Improvement Plan (CIP). The CIP has been used extensively as a means of prioritizing the needs and expenditures for various public facility projects.

A CIP for the Phillippi Creek basin was developed whereby the proposed improvements were prioritized and discretized to a five-year plan. That is, the projects and their associated costs were spread over a five-year time period in a manner that facilitates their planning and construction. The plan is shown in Table 6-16.

Project 2 is specifically recommended by the results of the modeling of the main channel of the creek. Thus, the costs have been estimated with some degree of accuracy. Project 3 is the perceived solution to the off-channel flooding problems identified, but not specifically modeled within the report. Thus, the costs associated with them are given as a starting point for comparison purposes. Final costs of these projects may vary substantially from the estimate depending on the study of the individual problem areas. Projects 4 through 7 are those projects identified by County staff as needed in the near future, and are presented with the costs provided.

Project 1 is the program. This program, discussed previously, is a result of conversations with the county to apportion the maintenance costs equitably throughout the basin.

TABLE 6-16

SARASOTA COUNTY STORMWATER MANAGEMENT PLAN

PHILLIPPI CREEK - FIVE YEAR CAPITAL IMPROVEMENT PROJECTS

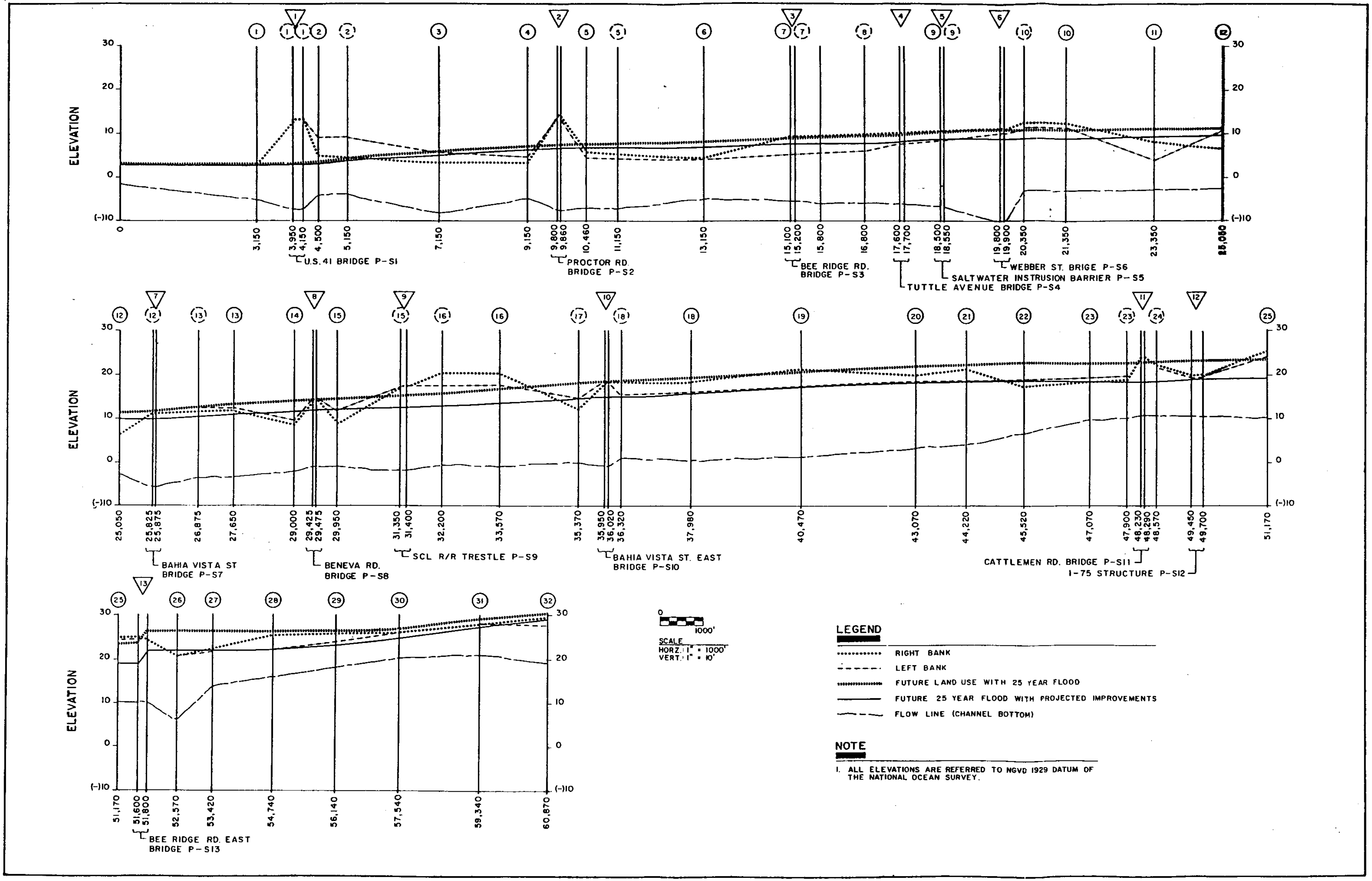
| | COSTS (/year) (A) | | | | |
|---|---|---|---|---|---|
| PROJECT DESCRIPTION | 1987 | 1988 | 1989 | 1990 | 1991 |
| 1. CONTINGENCY CIP (B) | $700,000 | $700,000 | $700,000 | $700,000 | $700,000 |
| 2. CONSTRUCT DETENTION PONDS (C) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $736,000 | $390,080 | | | |
| B. LAND AQUISITION | $937,500 | $1,987,500 | | | |
| C. CONSTRUCTION | | $835,617 | $1,576,636 | $995,233 | $1,256,459 |
| 3. CLARK LAKES DRAIN (C) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $568,000 | $301,040 | | | |
| B. LAND AQUISITION | | | $1,685,400 | $1,786,524 | $1,893,715 |
| C. CONSTRUCTION | | | | | $277,745 |
| 4. 17th AND OLD 31 (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $46,200 | | | | |
| B. CONSTRUCTION | $132,000 | | | | |
| 5. FRUITVILLE ROAD AND OLD 31 (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $38,500 | | | | |
| B. CONSTRUCTION | $110,000 | | | | |
| 6. 52 CANAL NORTH OF MAIN (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $33,600 | | | | |
| B. CONSTRUCTION | $96,000 | | | | |
| 7. MAIN EAST OF 52 CANAL (D) | | | | | |
| A. DESIGN, ADMIN, ENG, LEGAL, ETC. | $41,300 | | | | |
| B. CONSTRUCTION | $118,000 | | | | |
| | $3,557,100 | $4,214,237 | $3,962,036 | $3,481,757 | $4,127,919 |

NOTE....

(A) All costs are expressed as present worth in 1986 dollars utilizing 6% as the rate of cost increase per year.

(B) The costs associated with these projects have been estimated without the benefit of a complete hydrologic/hydraulic analysis and thus may vary.

(C) These improvements recommended by the Sarasota County Stormwater Management Division




Phillippi Creek HEC-2 Results - Future And Future With Improvements

FIGURE 6-10

# 7.0 ASSESSMENT OF REMAINING BASINS

# 7.0 ASSESSMENT OF REMAINING BASINS

As stated previously, the primary objective of this report is to provide Sarasota County with a useful tool to aid in the development of a comprehensive stormwater management utility system. Both Alligator and Phillippi Creeks were specifically identified as basins requiring in-depth investigation into stormwater control and flooding problems. Additionally, the remaining 14 non-coastal basins are to be generally studied to allow the extrapolation of stormwater control and flooding problems from the Alligator and Phillippi Creek studies.

The extrapolation of results is accomplished by a preliminary stormwater modeling effort being undertaken for each of these basins. Basically, the MSSM model, previously identified, is used to project the amount of peak runoff that can be expected from both the existing and the projected future land use conditions. The modeling effort used in this portion of the study is discussed in detail in Section 7.2.

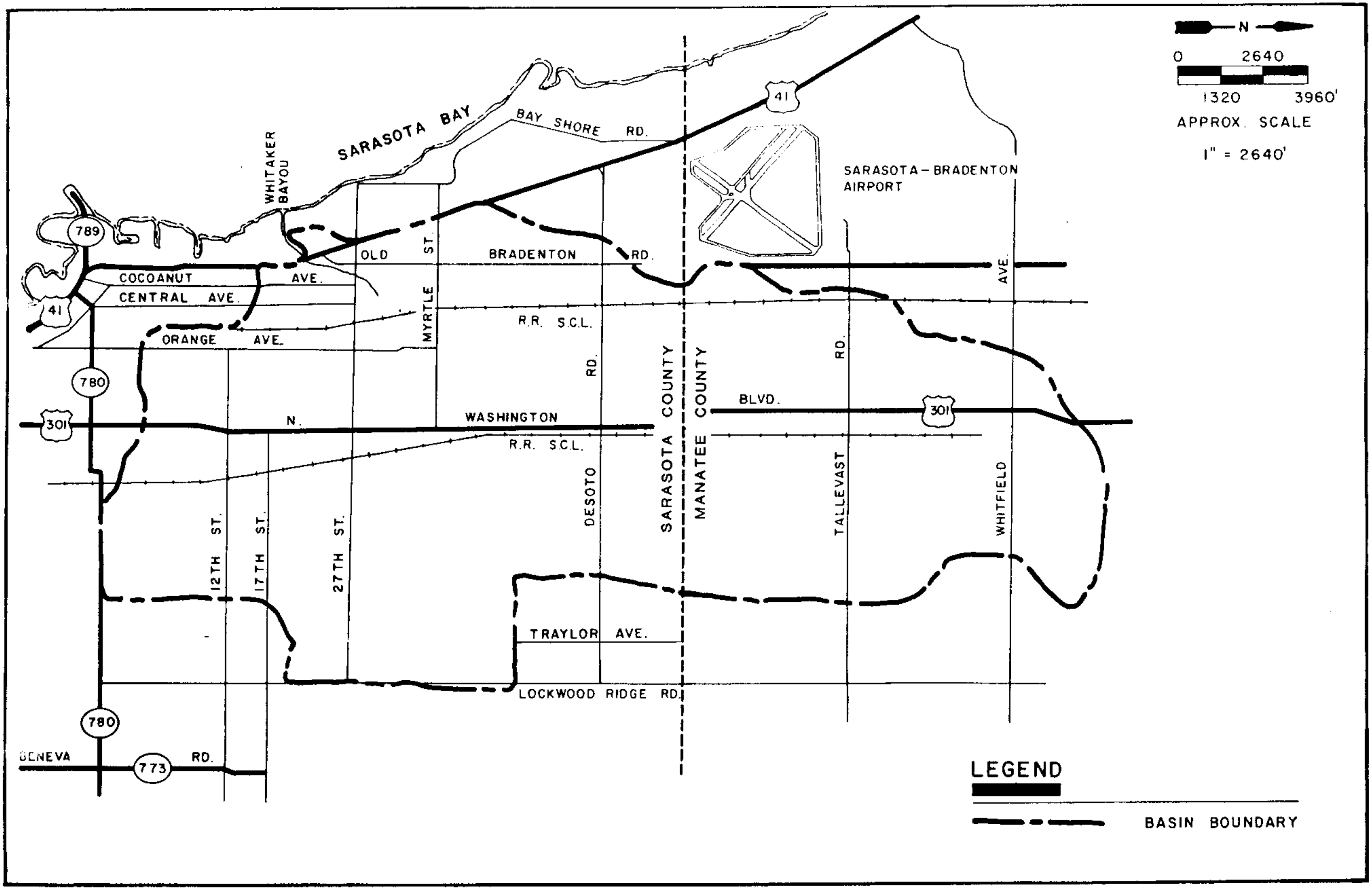
## 7.1 SUBBASIN CHARACTERISTICS

The general basin characteristics and drainage way descriptions for the remaining 14 non-coastal subbasins are given as follows.

### Basin 1 - Whitaker Bayou

Whitaker Bayou (Figure 7-1) is located in the northwest section of the county and drains approximately 8,200 acres. This basin's outfall, located in Sarasota County, drains a portion of southwestern Manatee County. The area of Manatee County is composed of undeveloped commercial/industrial zoned areas. Generally, the basin's primary drainage channel is via a natural, albeit improved, channel. The area drained within Sarasota County is highly urbanized. Many of the area's developments occurred some time ago, and thus provided for only minimal stormwater retention and detention. A major stormwater facility inventory is given in Table 7-1.

FIGURE 7-1




Whitaker Bayou Study Area

FIGURE 7-1

TABLE 7-1

WHITAKER BAYOU STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0101 | W-1 | U.S. 41 and approximately 2000 block<br>Concrete Bridge |
| 0101 | W-2 | 27th Street and just west of Cocoanut Avenue<br>30' x 12' Bridge |
| 0101 | W-3 | Riverside Drive and approximately 2900 block<br>Concrete Bridge |
| 0101 | W-4 | 32nd Street and just east of Clark Drive<br>Concrete Bridge |
| 0101 | W-5 | Myrtle Road and east of Cocoanut Avenue<br>Concrete Bridge |

## Basin 2 - Hudson Bayou

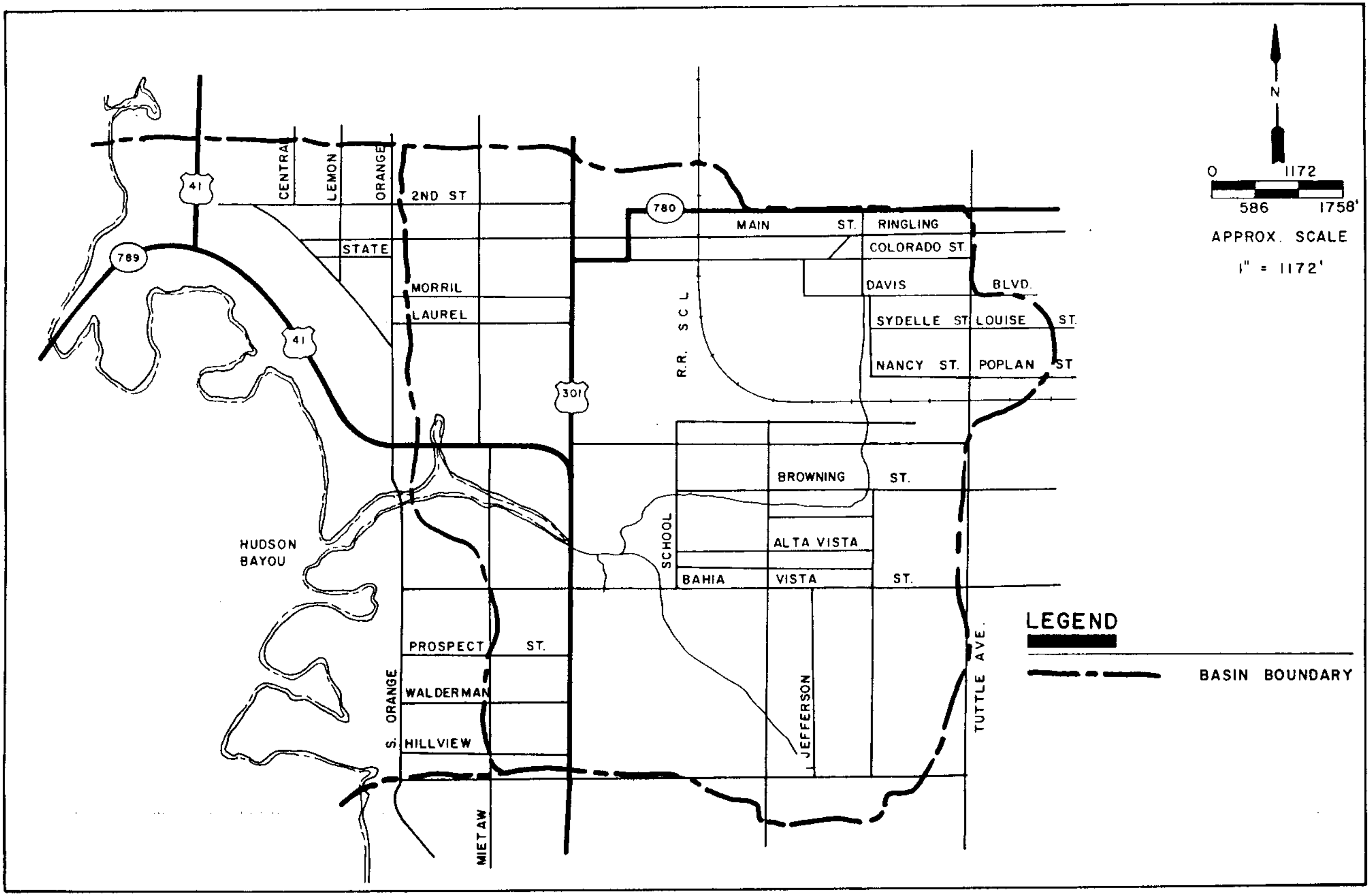
The Hudson Bayou basin (Figure 7-2) is located just south of basin 1 and encompass some 1,400 acres. This completely urbanized basin lies almost totally within the City of Sarasota corporate limits and includes some of the most highly developed land within the county. Stormwater is collected utilizing storm sewers and natural channels. Primary outflow is to Sarasota Bay near Bay Point. The primary developments within the basin include: local government offices, the downtown business district, and the La Linda Terrace subdivision. Major stormwater facilities are given in Table 7-2.

## Basin 4 - Matheny Creek

The Matheny Creek basin (Figure 7-3) is highly developed and covers about 1,500 acres. The basin, located just south of Phillippi Creek and Clark Lakes is composed primarily of single family home sites, condominiums, and trailer parks. The primary developments include: Colonial Terrace, Gulf Gate, Shadow Lakes, and Gulf Gate Gardens. Flow out of the basin is to Sarasota Bay primarily via natural channels. Several bridge crossings along with other structures occur throughout the basin. The facility inventory for this basin is given in Table 7-3.

## Basin 5 - Gulf Gate Canal

The Gulf Gate Canal basin (Figure 7-4) is located just south of the Matheny Creek basin. The area west of Beneva Road is highly developed, primarily with single- and multi-family residences. Major developments include Siesta Heights, Gulf Gate Woods, Pinehurst Park, Pine Tree, and Park East. Outflow from this basin is through three small channels: Elligraw Bayou, Holiday Bayou, and Clower Creek. An inventory of the main stormwater facilities is given in Table 7-4.



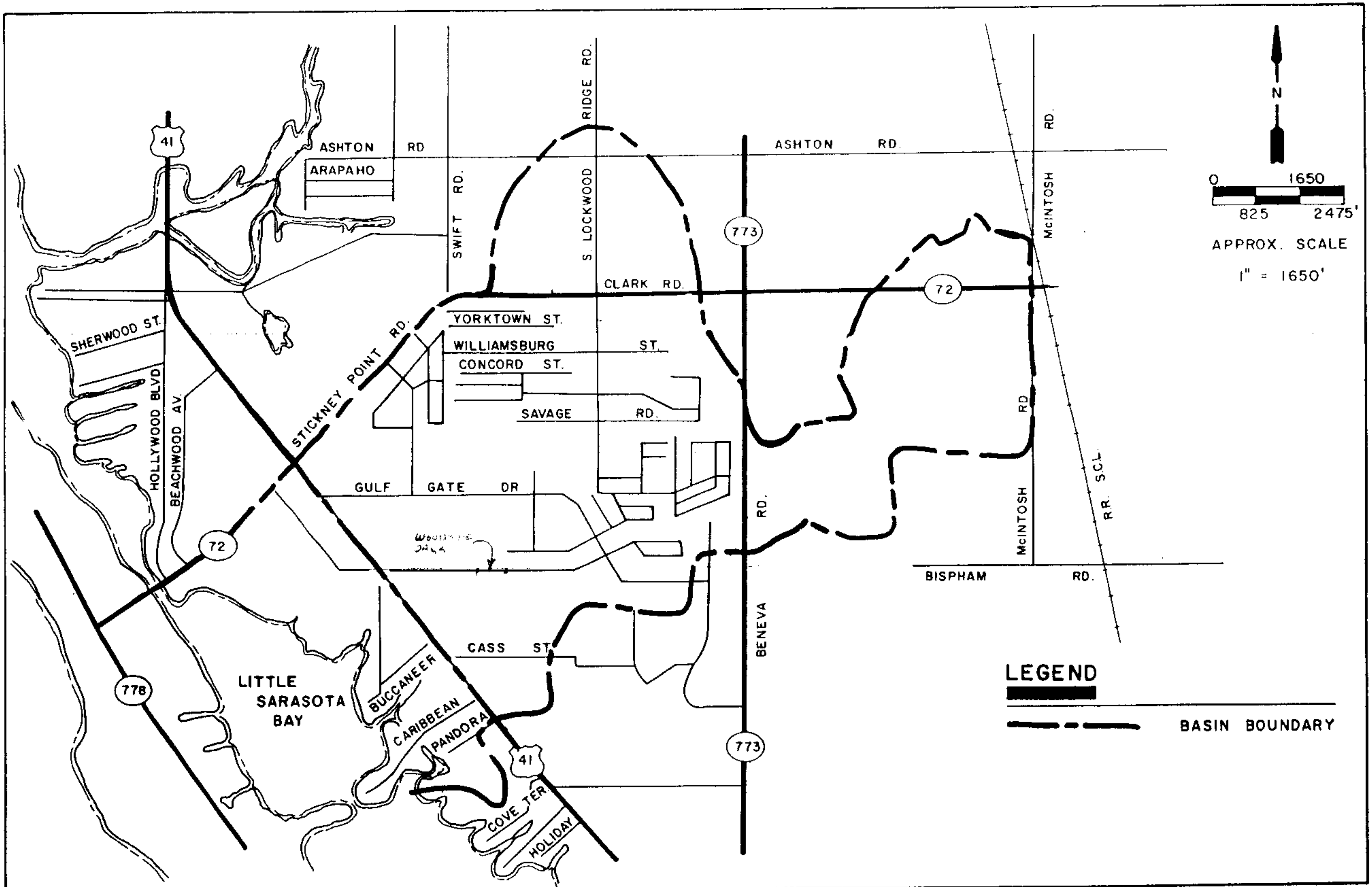

Hudson Bayou Study Area

FIGURE 7-2

TABLE 7-2

HUDSON BAYOU STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| CB23 | H-1 | Orange Ave. and approximately 2 blocks south of U.S. 41 - Concrete Bridge |
| 0201/0202 | H-2 | South Osprey Avenue and approximately 2 blocks south of U.S. 41 - Concrete Bridge |
| 0201/0202 | H-3 | U.S. 41 and approximately 900 block Concrete Bridge |
| 0202 | H-A1 | U.S. 41 and approximately 1700 block Concrete Bridge |



Matheny Creek Study Area

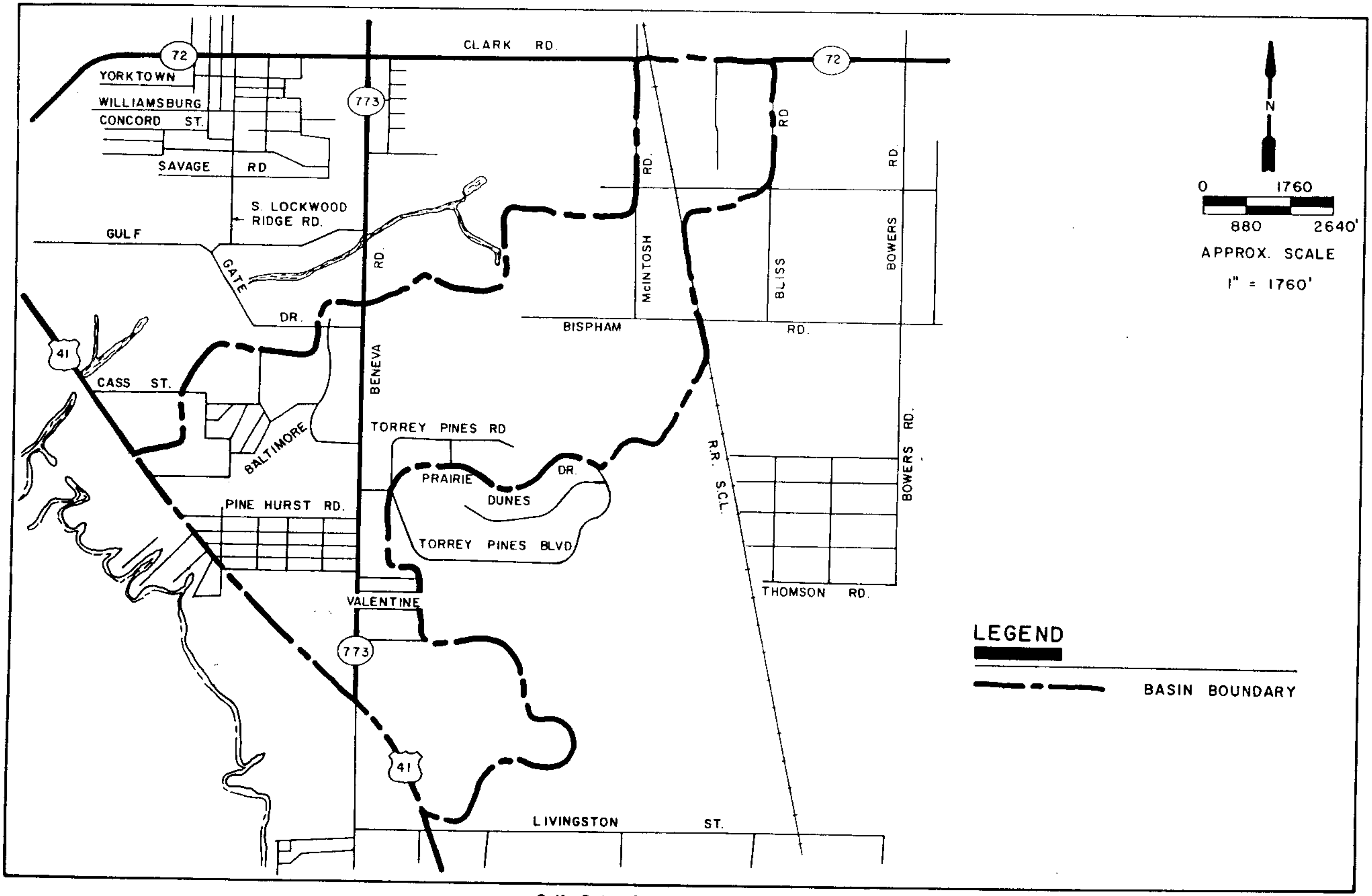
FIGURE 7-3

FIGURE 7-3

TABLE 7-3

MATHENY CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0401 | M-2 | U.S. 41 and approximately 7200 block Concrete Bridge - 2-12' x 10' Box |
| 0401 | M-4 | Bispham Road and near grey Squirrel Street Bridge - 48" CMP/Prop. 12' x 8' Br. SWN 9/61 |
| 0401 | M-5 | Gulf Gate Drive and just southeast of Regatta Drive - Bridge |
| 0401 | M-8 | Beneva Road and just NE of Westford Lane Bridge - 2-6' x 3' CMP |




Gulf Gate Canal Study Area

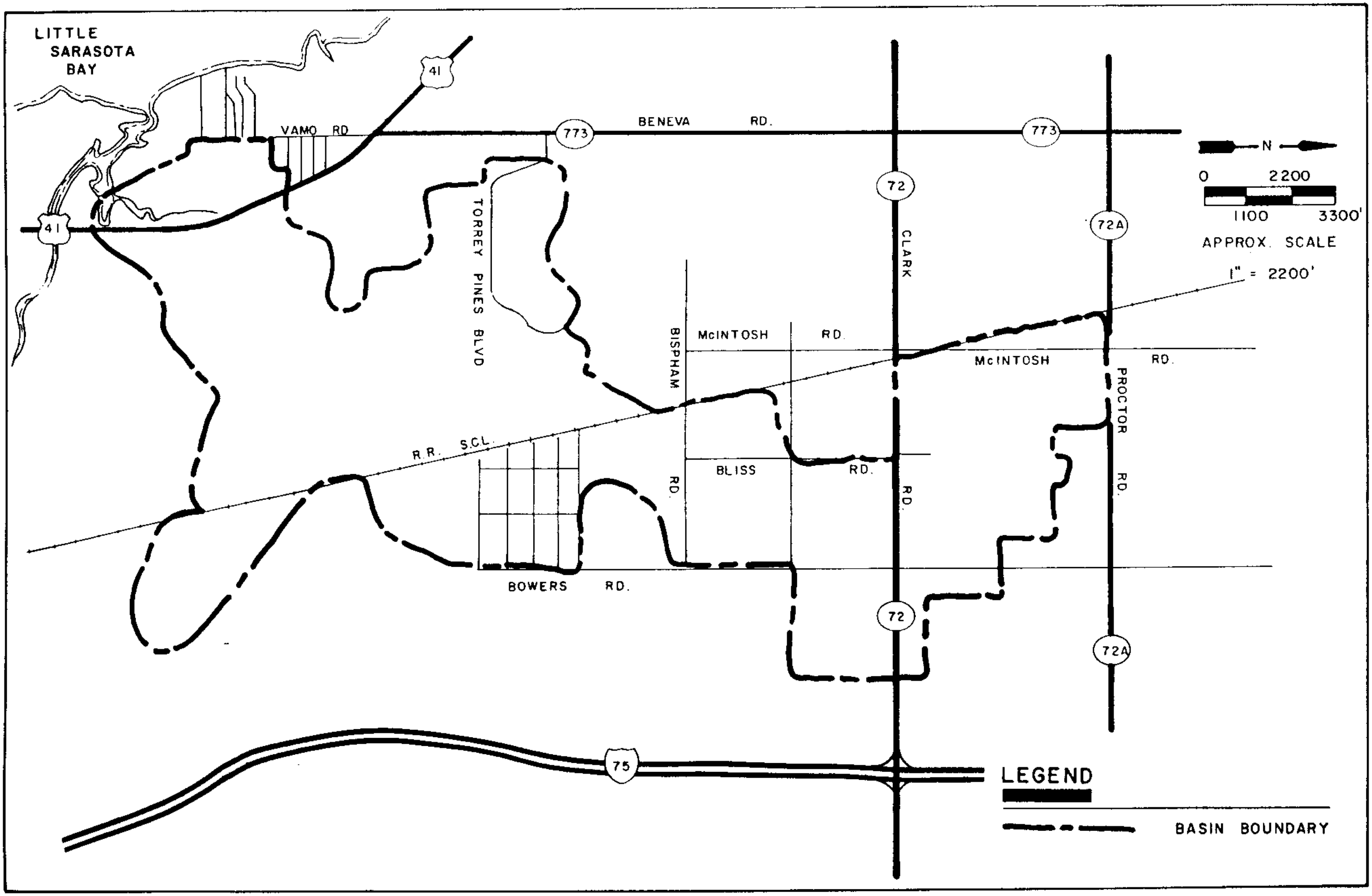
FIGURE 7-4

FIGURE 7-4

TABLE 7-4

GULF GATE CANAL STRUCTURE INVENTORY

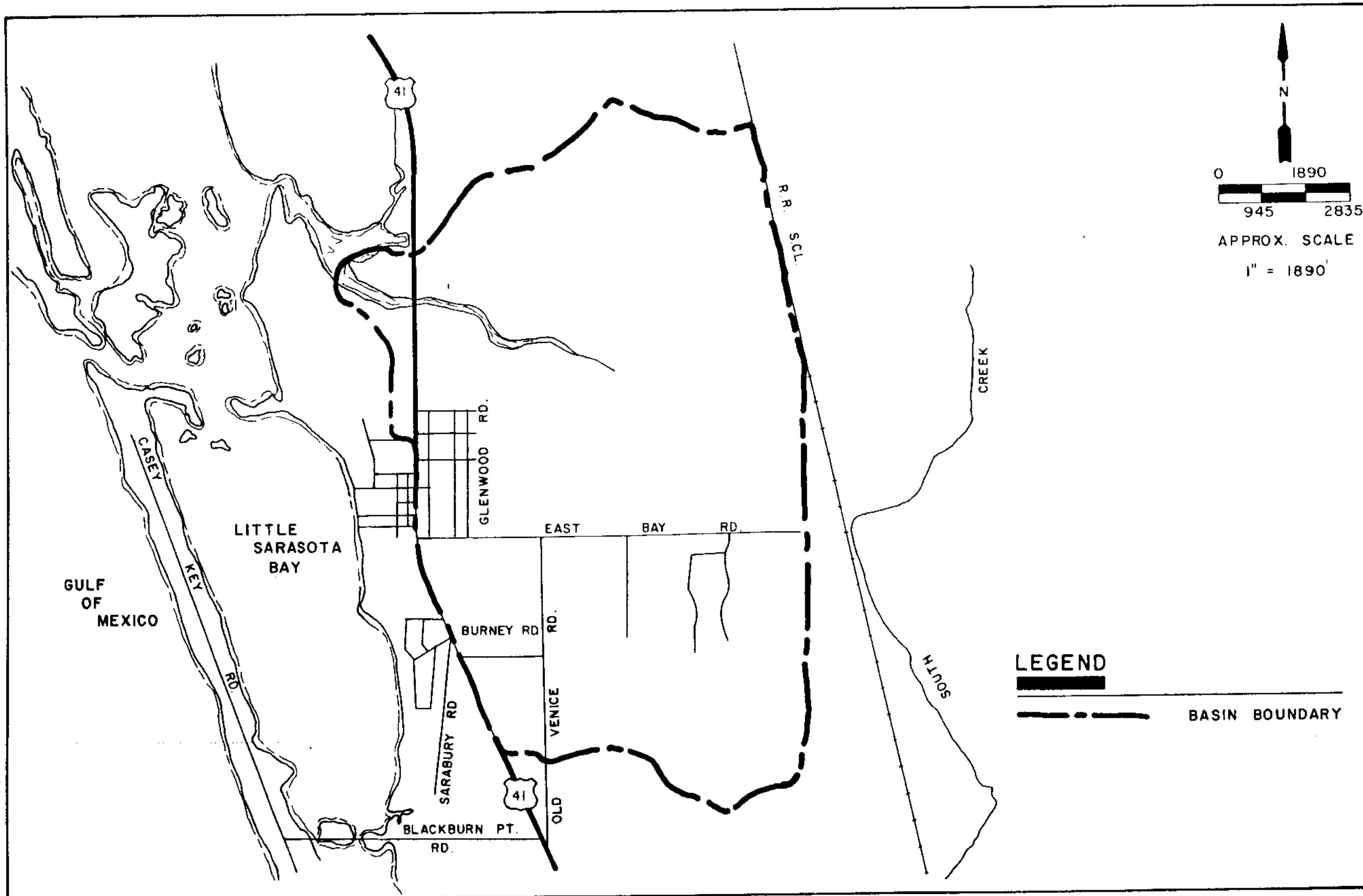
| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| Elligraw Bayou: | | |
| 0502 | G-1 | U.S. 41 and approximately 7600 block<br>2 - 12' x 8' Box Culverts |
| 0502 | G-2 | Pinehurst Street and approximately ± 600' west of U.S. 41<br>Drainage Structure |
| 0502 | G-3 | Biltmore Drive and north of Tuckerstown Drive<br>Drainage Structure |
| 0502 | G-4 | Beneva Road and south of Curtiss Avenue<br>3 - 8' x 8' Box Culverts |
| Holliday Bayou: | | |
| 0502 | G-A1 | U.S. 41 and approximately 7700 block<br>2 - 12' x 8' Box Culverts |
| Clower Creek: | | |
| CB26 | G-B1 | Vamo Road and approximately 8200 block<br>5' x 2.5' Box/Proposed 15' x 6' Bridge - SWN 9/61 |
| CB26 | G-B1 | U.S. 41 and approximately 8200 block<br>2 - 6' x 6' Box Culverts |




Catfish Creek Study Area

FIGURE 7-5

FIGURE 7-5




**North Creek Study Area**

FIGURE 7-6

FIGURE 7-6

Basin 6 - Catfish Creek

The Catfish Creek basin (Figure 7-5) is a largely undeveloped area which forms the western boundaries of Phillippi Creek, Matheny Creek, and Gulf Gate Canal. The basin is bisected by the Seaboard Coastline Railroad which forms a man-made flow diversion structure. The natural channel is largely ill-defined and meanders from wetland to wetland on its southwesterly journey to Little Sarasota Bay. This basin, because of the proposed Palmer Ranch development, is expected to see radical changes to its current largely undeveloped nature over the next few decades. The Palmer Ranch developers are proposing the alteration of the flow patterns within the basin as part of their development-wide master drainage plan. The major existing facilities are given in Table 7-5.

Basin 7 - North Creek

The North Creek basin (Figure 7-6) is a sparsely developed basin of approximately 2,400 acres. Generally, flow to the main channel is by sheet flow across the ground surface, and flow through a largely man-made lake system through the development known as The Oaks II. Like Catfish Creek to the north, this basin will be affected by the Palmer Ranch development. The main channel flows northwesterly to an outfall in Little Sarasota Bay. Major facilities are listed in Table 7-6.

Basin 8 - South Creek

The South Creek basin (Figure 7-7) covers an almost totally undeveloped area of approximately 13,000 acres. This basin is bisected by the Seaboard Coastline Railroad, Interstate 75, and the Venice By-pass. This extremely flat lowland basin drains to the southwest primarily by sheet flow to the natural channels. Typical of major structures, both the railroad and the interstate construction has altered natural flow paths. The Palmer Ranch development largely affects this basin also. Their initial stormwater proposal would divert significant quantities of flow from the other basin

TABLE 7-5

CATFISH CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0601 | C-1 | Vamo Way and approximately 500' west of U.S. 41 18.5' x 7.5 Arch/Proposed 49' x 10' Bridge - SWN 9/61 |
| 0601 | C-3 | U.S. 41 and ± 300' south of Wharf Road 1 - 32' x 9' Concrete Bridge and 1 - 36' x 9' Concrete Bridge |

TABLE 7-6

NORTH CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0701/0702 | N-1 | On U.S. 41, just south of Cordes Street<br>1 - 40' x 9' Concrete Bridge<br>1 - 44' x 9' Concrete Bridge |
| 0701/0702 | N-4 | In the northeast corner of Section 2<br>Twp 38S Rng 18E, just east of East Bay Road -<br>32' x 5' Wood Bridge |

into South Creek. The effects of this diversion of flow through the downstream development, the Oscar Sherer State Park, and Blackburn Bay must be determined. The existing major structures on South Creek are listed in Table 7-7.

Basin 9 - Shakett Creek

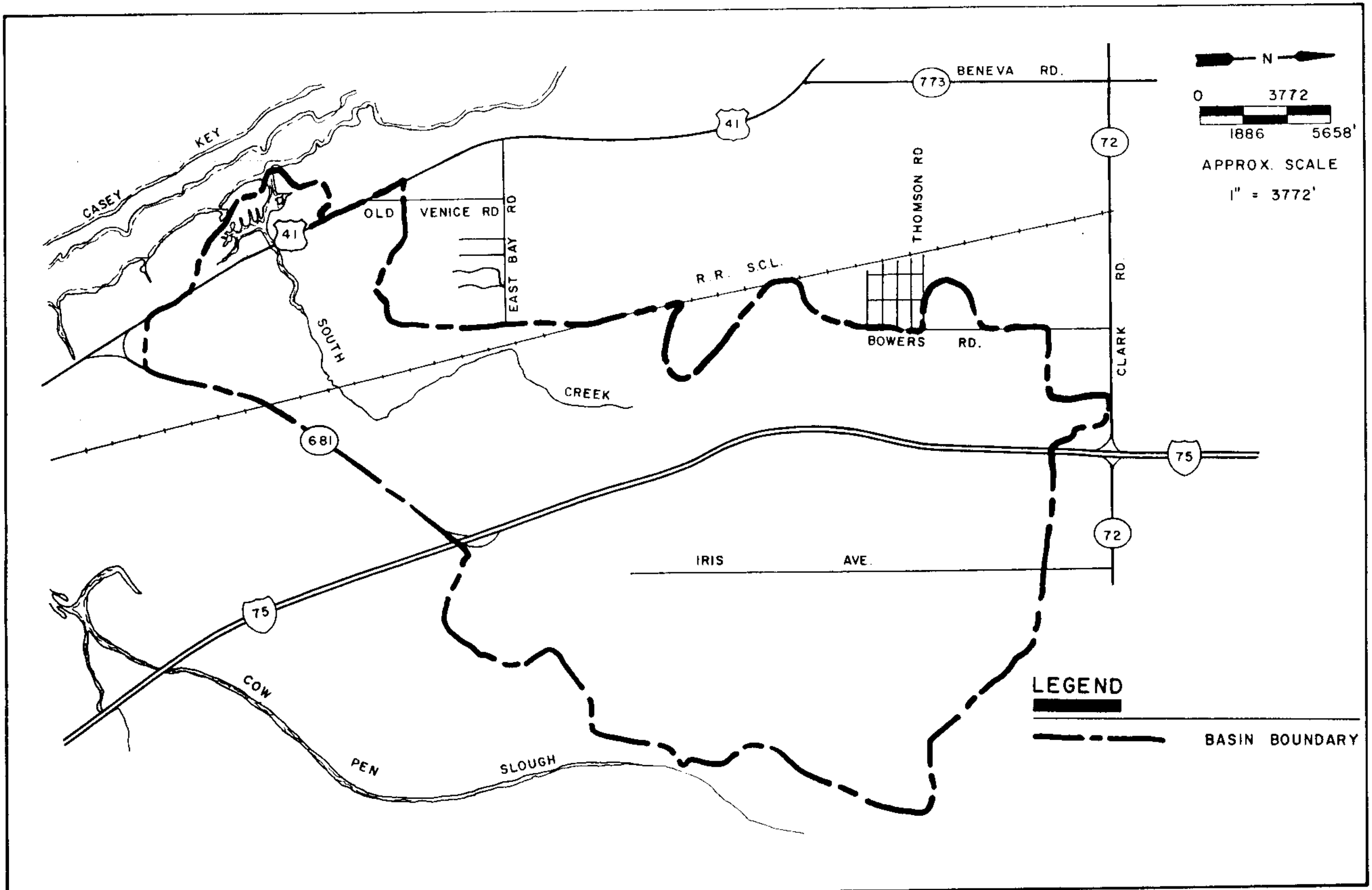
Shakett Creek (Figure 7-8) is bordered on the northwest by the Venice by-pass which forms a man-made basin boundary. This basin, approximately 8,700 acres in size, drains southward to Dona Bay in Nokomis. Draining into Shakett Creek at its northern headwaters are Fox Creek, Salt Creek, and Cow Pen Slough. The main channel, downstream from its intersection with the slough, has been extremely channelized. A listing of the major structures within this basin can be found in Table 7-8. It should be noted that although Cow Pen Slough drains through the creek noted, it has not been modeled at the County's request.

Basin 10 - Curry Creek

The Curry Creek basin (Figure 7-9) lies immediately south of Shakett Creek, and drains the northeastern part of the City of Venice. Development within this basin, typical for Sarasota County, is dense near the Gulf of Mexico and becomes rather sparse as you go upstream. Major developments within this basin include Bay Indies, Colonial Manor, East Gate Terrace, Pinebrook South, and Capri Isles. Drainage is primarily westerly along the main natural channel to an outfall in Roberts Bay. Runoff from four of the basins and subbasins intersect at the eastern boundary of Capri Isles. The major stormwater structures existing within the basin are listed in Table 7-9.

Basin 11 - Hatchett Creek

The Hatchett Creek basin (Figure 7-10), covering an area of some 3,500 acres, is sandwiched between the Curry and Alligator Creek basins. This basin is generally highly developed at its downstream end with almost no




South Creek Study Area

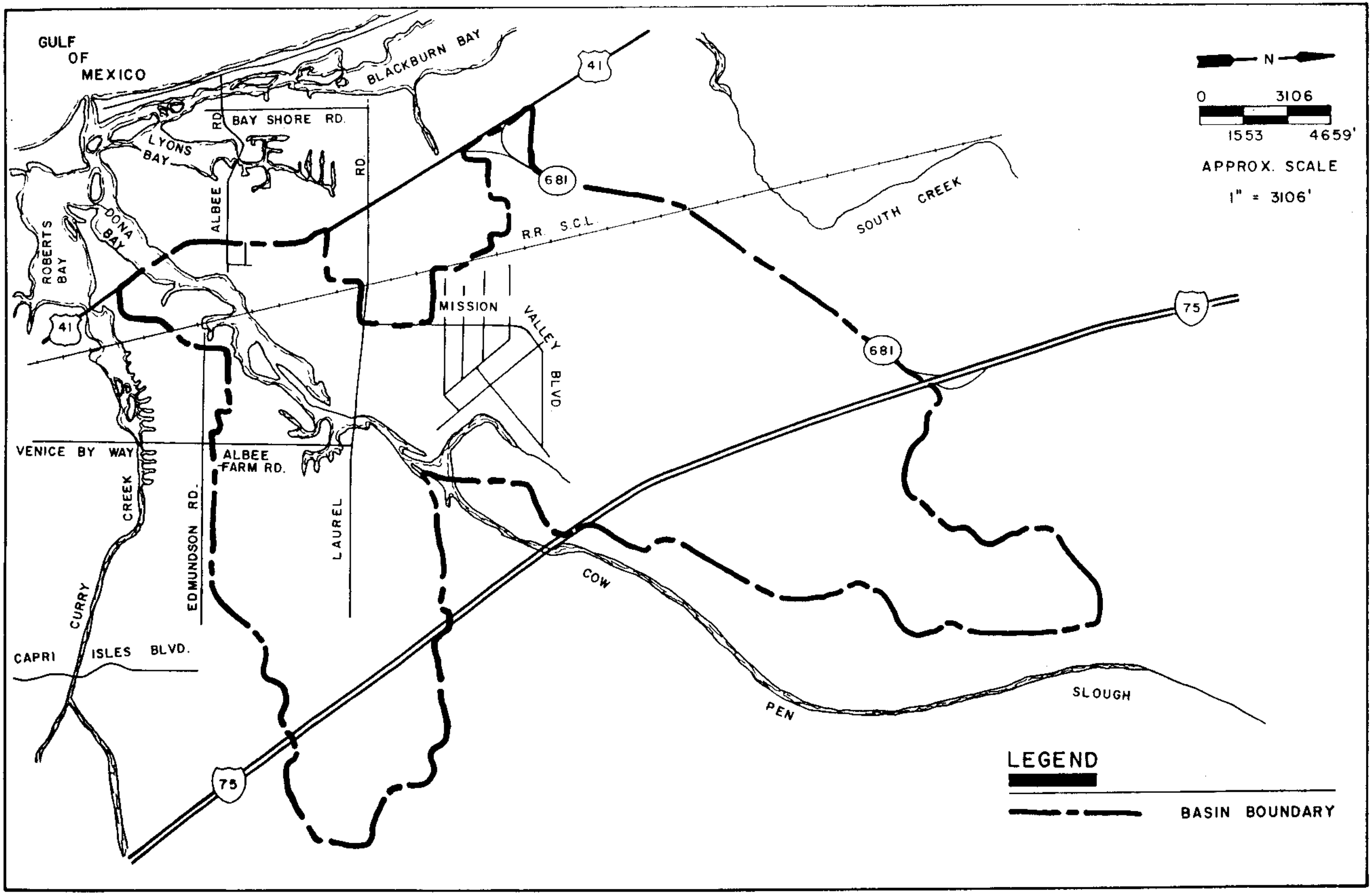
FIGURE 7-7

FIGURE 7-7

TABLE 7-7

SOUTH CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0801/0802 | S-1 | On U.S. 41 between Shoreland Drive and Rubens Drive<br>50' x 10' Bridge |
| 0801/0801 | S-3 | On S.A.L.R.R.<br>168' x 8' Wood Bridge |




Shakett Creek Study Area

FIGURE 7-8

FIGURE 7-8

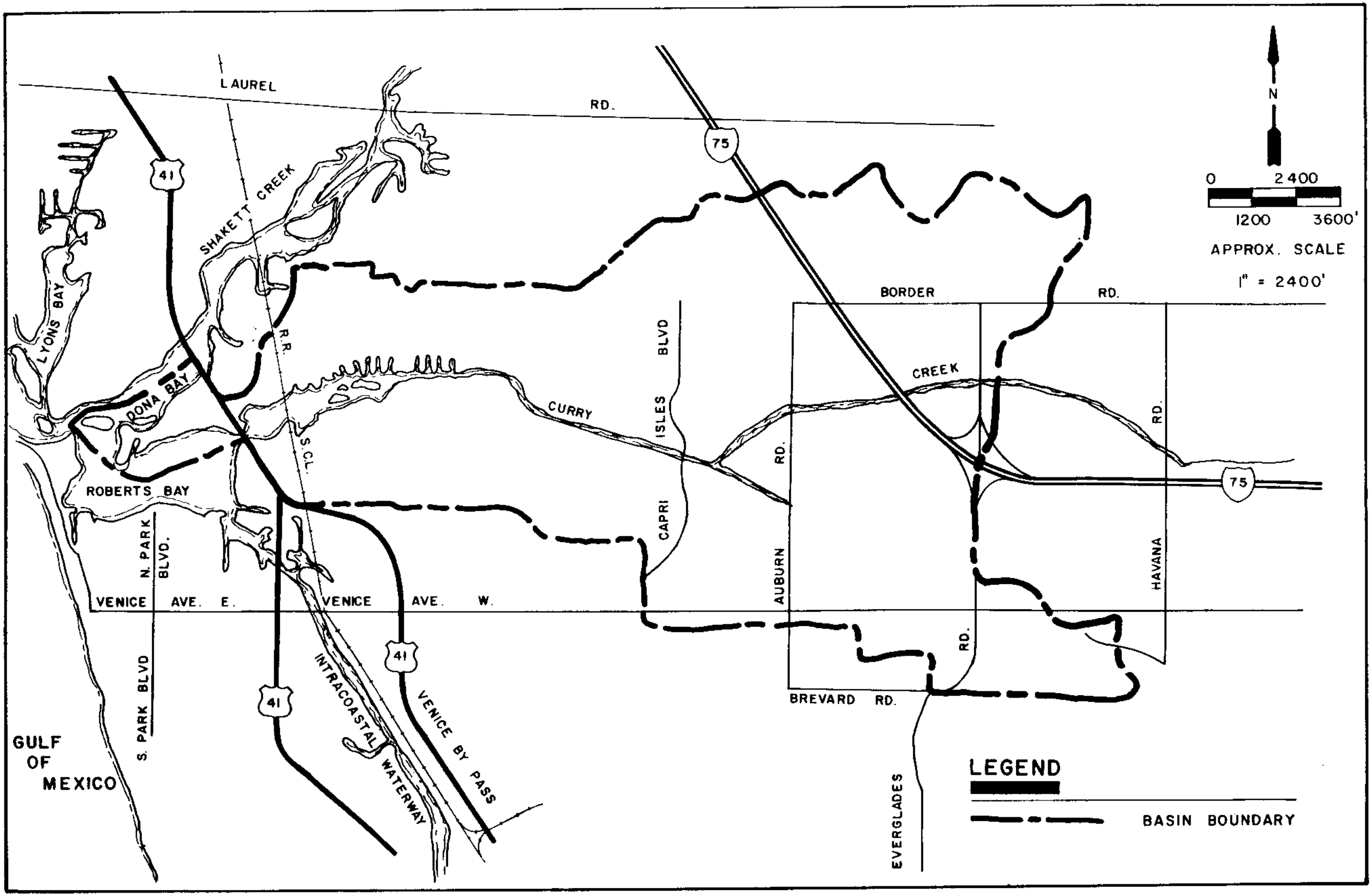
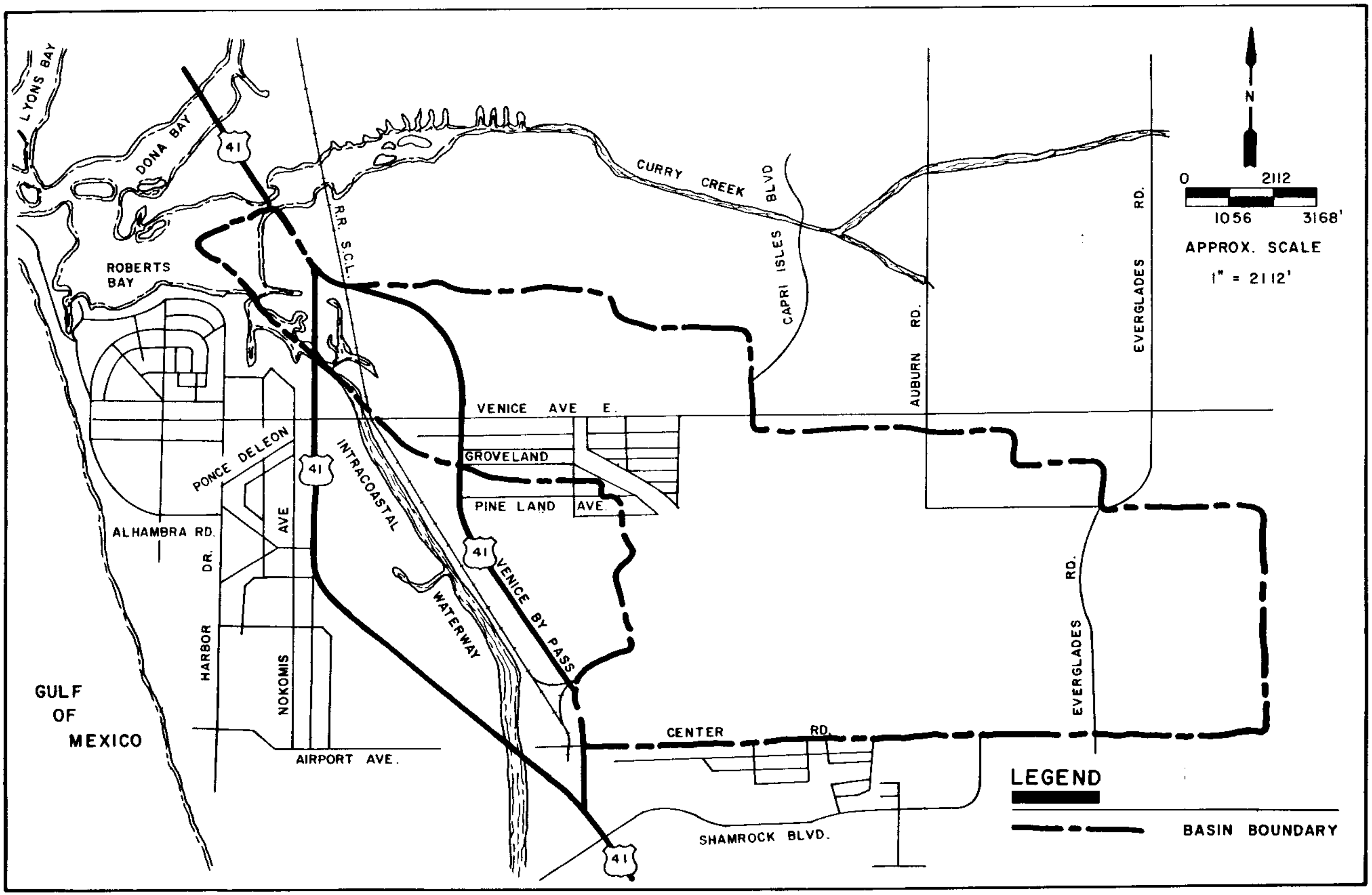



FIGURE 7-9

Curry Creek Study Area

FIGURE 7-9




Hatchett Creek Study Area

FIGURE 7-10

FIGURE 7-10

TABLE 7-8

SHAKETT CREET STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 0901 | SH-1 | On U.S. 41 ± 500' south of Bay-View Parkway Bridge |
| 0901 | SH-2 | S.C.L.R.R. and approximately 200 block Railroad Bridge |
| 0901 | SH-3 | Laurel Road and ± 700' west of Albee Farm Road Bridge |

TABLE 7-9

CURRY CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1001/1005 | CR-1 | On U.S. 41, south of Sunrise Drive Bridge |
| 1001/1005 | CR-2 | On Seaboard Coastline Railroad ± 1500' south of Colonia Avenue - Bridge |
| 1001/1005 | CR-3 | On Venice-By-Way, south of Colonia Avenue 60' x 12' Bridge |

existing development near its upper reaches. However, developments north of Center Avenue surrounding Jacaranda Boulevard are currently planned. The major structures within this basin are listed in Table 7-10.

Basin 13 - Woodmere

The Woodmere basin (Figure 7-11) is a 3,000 acre basin located between the cities of Venice and Englewood. Generally, the stormwater flow is westward to the Intercoastal Waterway through a series of man-made canals and lakes. The area's development is comprised primarily of single-family home sites, with the exception of Circlewood and Japanese Gardens. A list of some of the stormwater management facilities are given in Table 7-11.

Basin 14 - Forked Creek

The Forked Creek basin (Figure 7-12) is located just north of the City of Englewood. This basin, covering about 5,600 acres, is predominantly undeveloped except along the coastline and Highway 775. Generally, the flow within the basin is westward along the main, improved channel to its outflow in Lemon Bay. The major structures within this basin are listed in Table 7-12.

Basin 15 - Gottfried Creek

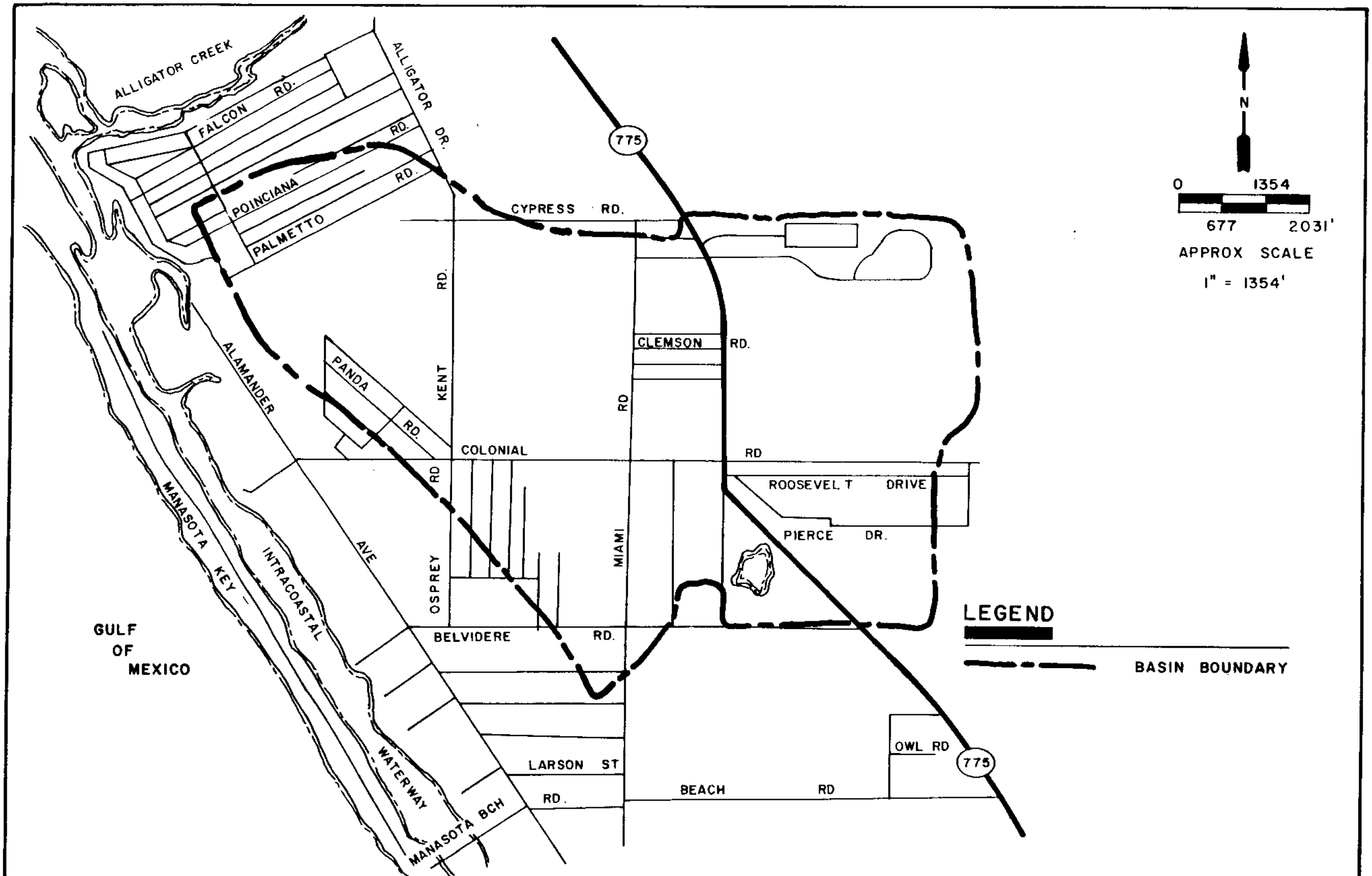
Gottfried Creek (Figure 7-13) flows primarily southward along the eastern boundary of Englewood, with an outflow in Charlotte County. This 6,900-acre basin is primarily undeveloped with the vast majority of its area currently being used as pasture and farm land. The largest part of this undeveloped area is currently planned to be developed under the Planned Unit Development (PUD) concept by Berry Properties. The PUD allows varied types and densities of housing and commercial/industrial areas to be integrated into a community type setting. Structures found within this basin are listed in Table 7-13.

TABLE 7-10

HATCHETT CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1101/1105 | HT-1 | On S.R. 45 (U.S. 41), north of Tampa Avenue<br>Highway Bridge 104' x 5' Concrete |
| 1104/1107 | HT-2 | On S.C.L., to the east of U.S. 41 and to the north of Venice Avenue East<br>R.R. Bridge 75' x 8' Wood |

FIGURE 7-11




Woodmere Study Area

FIGURE 7-11

TABLE 7-11

WOODMERE CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1301 | WM-1 | Heron Road and just northeast of Sceneca Road Structure |
| 1301 | WM-2 | Colonial Road and just east of Orchis Road Structure |
| 1302 | WM-A1 | Heron Road and between Ponderosa and Duquesne Structure |
| 1302 | WM-A2 | Corner of Duquesne and Missouri Road Structure |

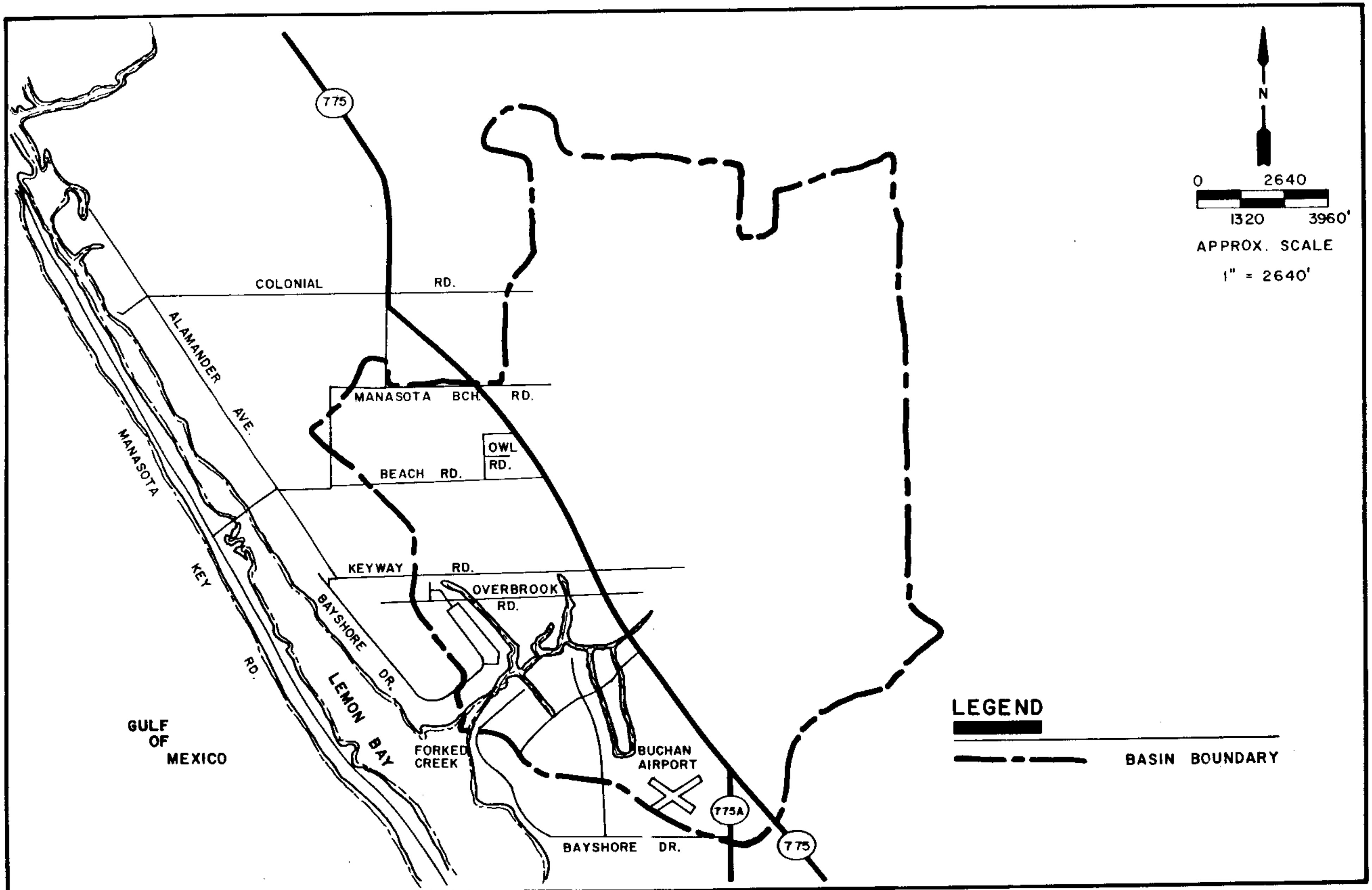



FIGURE 7-12

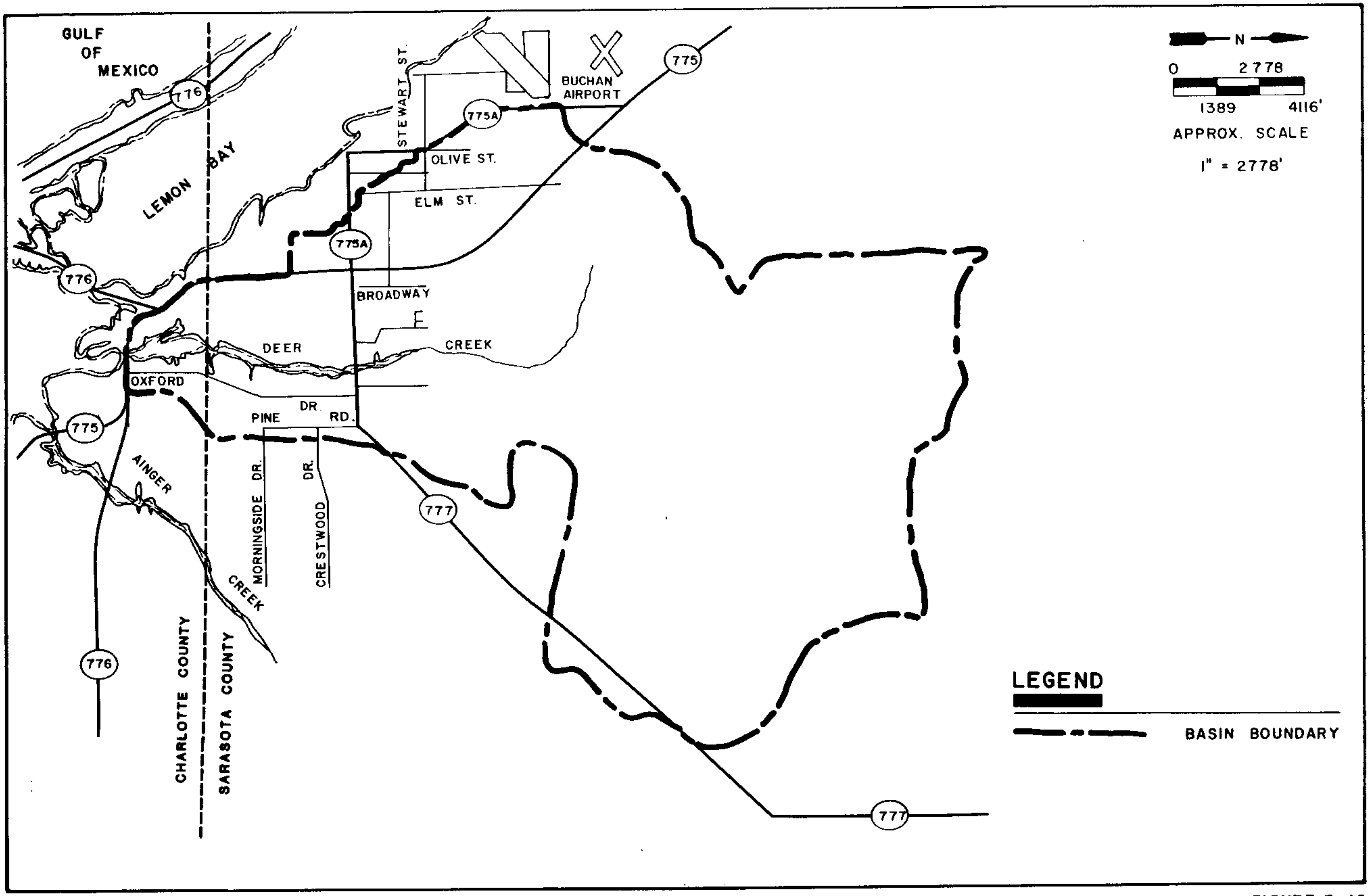
Forked Creek Study Area

FIGURE 7-12

TABLE 7-12

FORKED CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1401/1403 | F-1 | On S.R. 775, between Freedann Street and Northbrook Drive<br>42' x 10' Bridge |




Godfrey Creek Study Area

FIGURE 7-13

FIGURE 7-13

TABLE 7-13

GOTTFRIED CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1501/1502 | GF-1 | On S.R. 777, just west of Oxford Drive<br>90' x 9' Bridge |
| 1501/1502 | GF-1 | On State Road 775, just west of Oxford Drive<br>Bridge |

Basin 16 - Ainger Creek

Ainger Creek (Figure 7-14) is located on the extreme southern edge of Sarasota County, with its outflow well into Charlotte County. Generally, the portions of basin 16 lying in Sarasota County are undeveloped. However, the portion within Charlotte County is densely populated. The basin covers a total area of 10,000 acres, with about 90 percent in Sarasota County. Principal structures within the basin are listed in Table 7-14.

## 7.2 HYDROLOGIC ANALYSIS

A hydrologic analysis of the remaining 14 basins must be done to establish design discharge for both existing and ultimate land use must be done. Once again, the MSSM model will be used to predict peak flows and the general hydrograph shape and timing. The required input variables have been extrapolated from either the Phillippi or Alligator Creek studies. It is assumed for the purposes of this report that any secondary channel modifications, detention, storage, or enclosed storm drain system will be the responsibility of the developer. Thus, only outfall conditions on the main channel are of primary significance.

The basin and subbasin characteristics such as acreage, impervious percentage, infiltration, etc. will form the core of the input data. Much of this data is taken from the previously completed Sarasota County Basin Delineation and Refinement projects. However, recently available information such as subdivision plans and contour aerials, has necessitated the review of and, in many cases, modifications to this information.

Constraints of the scope of this project require that, where possible, information derived from the two in-depth studies be used in this modeling effort. To accomplish this, the subbasins to be modeled were compared to those in Phillippi and Alligator Creeks and, where appropriate, input information such as: overland flow Manning's "n" values, depression storage, initial abstraction, etc. was used as input to the model. For a discussion of input parameters.

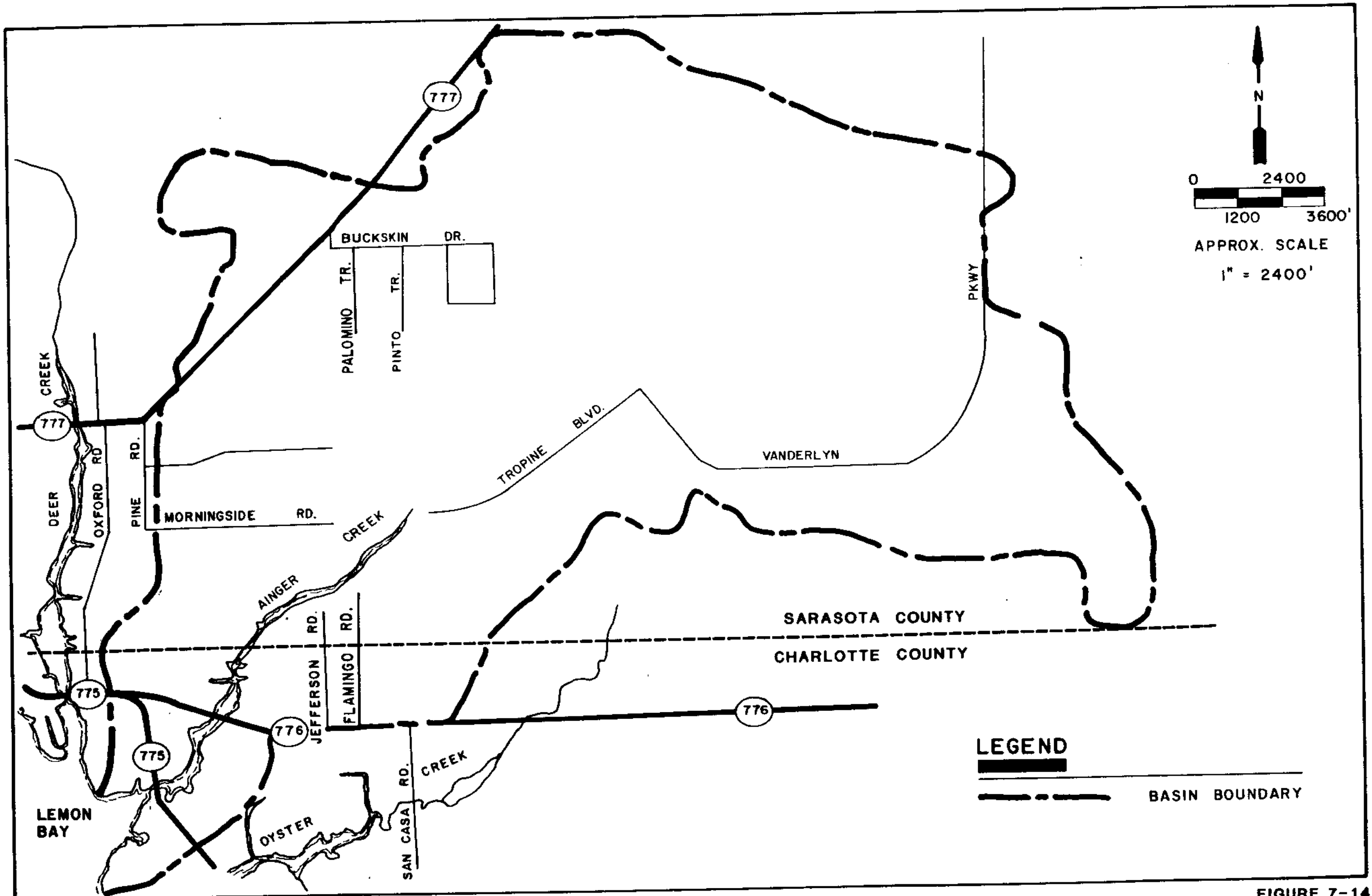



FIGURE 7-14

Ainger Creek Study Area

FIGURE 7-14

TABLE 7-14

AINGER CREEK STRUCTURE INVENTORY

| Subbasin Number | Structure Number | Location and Description of Structures |
|---|---|---|
| 1601/1603 | AG-1 | State Highway 775 and $\pm$ 400' south of Manor Road Concrete Bridge |
| 1601/1603 | AG-2 | State Highway 776 and $\pm$ 800' E.S.E. of Manor Road Concrete Bridge |

## 7.3 IDENTIFICATION OF RUNOFF POTENTIAL

The potential flood-prone areas within the 14 non-coastal basins were determined by the use of the MSSM model. Ideally, the flood-prone areas would be identified through the use of aerial photography with contours such as those by SWFWMD. Although the scope of work for their project assumed that SWFWMD aerials would be available for use, they are not. Thus, it has been necessary to use information available on the basin topography from other sources.

### 7.3.1 BASIN RUNOFF

The projected volume of runoff from each of the 14 non-coastal basins is given in Table 7-15. Individual outputs including hydrographs were provided under separate cover. It is assumed that all development within these basins will be required to meet the county's criteria of pre- and post-development being essentially equal. Therefore, it was agreed that for the purpose of this study the peak flow would be determined for the existing conditions only. As was shown in the Alligator Creek in depth study the county's criteria generally controls peak changes for the smaller sized basins.

Generally, the peak flow from these basins ranges between 103 and 252 cfs per square mile. This range is consistent with the findings of previous investigations in these areas. Generally speaking, the smaller the basin size the larger the peak runoff. This is due primarily to the fact that small basins, typically highly urbanized, provide little or no storage or attenuation potential.

### 7.3.2 FLOOD-PRONE AREAS AND MANAGEMENT ALTERNATIVES

The flood prone areas identified followed the format outlined above and are outlined by basin below.

TABLE 7-15

PREDICTED RUNOFF RATES
(25-Year Storm)

| | Basin | Area (square miles) | Unit Peak Flow (cfs/mi²) | Peak Flow, cfs |
|---|---|---|---|---|
| 01 | Whitaker Bayou | 12.32 | 213 | 2,735 |
| 02 | Hudson Bayou | 2.15 | 246 | 543 |
| 03 | Phillippi Creek | 56.4 | 123 | 6,935 |
| 04 | Matheny Creek | 2.36 | 252 | 607 |
| 05 | Gulf Gate Canal | 1.67 | 167 | 168 |
| 06 | Catfish Creek | 6.44 | 165 | 1,064 |
| 07 | North Creek | 3.81 | 134 | 513 |
| 08 | South Creek | 20.13 | 103 | 2,082 |
| 09 | Shakett Creek | 10.76 | 103 | 1,118 |
| 10 | Curry Creek | 9.10 | 186 | 1,818 |
| 11 | Hatchett Creek | 5.42 | 338 | 1,818 |
| 12 | Alligator Creek | 9.87 | 169 | 1,669 |
| 13 | Woodmere | 3.90 | 156 | 720 |
| 14 | Forked Creek | 8.76 | 67 | 587 |
| 15 | Godfrey Creek | 10.74 | 137 | 1,464 |
| 16 | Ainger Creek | 16.44 | 170 | 2,614 |
| | | 179.26 | | |

Basin 1 - Whitaker Bayou

Whitaker Bayou experiences minimal flooding in its current condition. A portion of the main channel west of the SCLRR between Desoto Road and 27th Street is undersized and in need of maintenance. Sufficient area exists within the portion of the basin inside the County to provide storage. Some of these potential storage areas include: (1) expansion of an existing pond located south of 27th Street between Euclid and Mango Avenues; (2) an area west of the railroad right-of-way west of Independence Boulevard; and (3) an area east of Myrtle Street and north of Bannaker Way.

A major hurdle that needs to be addressed in the Whitaker Bayou basin is the runoff being contributed by the area of the basin that lies within Manatee County. This area, largely undeveloped, is zoned commercial and light industrial. Sarasota County should request Manatee County to provide them with development plans for review and comment.

Basin 2 - Hudson Bayou

The Hudson Bayou basin currently experiences very minimal flooding during the 25-year design storm. The basin's channels are generally of adequate capacity and are reasonably well maintained. However, the Stormwater Management Division recommends that a weir structure be placed at the intersection of the North Branch with Shade Avenue. This would provide a control point for flows from the area north of Browning Street.

Basin 4 - Matheny Creek

Matheny Creek currently experiences flooding from the 25-year design storm only at its uppermost reaches. However, it has been recommended by the Stormwater Management Division that stream flow control devices at the following locations be considered.

- Matheny Creek and Gulf Gate Drive
- Denhan Branch and Gulf Gate Drive
- Lake Wright

These structures will in the control of stormwater levels through the watershed.

Basin 5 - Gulf Gate Canal

The Gulf Gate Canal basin is relatively small and experiences flooding primarily in the area of its headwaters north of the Sarasota Country Club. This area is composed primarily of low-lying wetland type areas that are covered with water during parts of the rainy season. Development of this area is going to occur within the next few years as part of the Palmer Ranch development. Future flooding due to the 25-year design storm as modeled does not pose a threat to the welfare of the public and can generally be classified as nuisance flooding.

Basin 6 - Catfish Creek

The Catfish Creek basin currently experiences flooding along its interior canals and in parts of the Sarasota Country Club which empties into it. Generally, the flooding of the interior lowlands poses no current threat to public welfare. The flooding within the Country Club area has at times been rather severe. A general cleaning out of the existing subbasin tributaries and a channelization of the main outlet channel is recommended. However, the Palmer Ranch development, which is to occupy a large majority of this basin, has plans for diverting flow from this basin to the South Creek basin. Thus, a complete review of the Palmer Ranch development's final stormwater design plans should be completed prior to all major channel work.

Basin 7 - North Creek

The North Creek basin currently experiences flooding at its extreme upstream end around the SCLRR. This is primarily caused by inadequate drainage through the railroad right-of-way and the existence of low-lying wetlands. In its existing condition, the creek does not pose a threat to public welfare, with flooding occurring in outlying areas immediately along the stream channel. Additionally, the connecting canal that joins this basin with that of Catfish Creek experiences some flooding. Localized flooding in the community occurs but does not effect the entire basin. We recommend that the entire length of the main channel be cleaned and that the crossings within The Oaks II should be checked to determine if proper maintenance has been done.

Basin 8 - South Creek

The South Creek basin currently experiences flooding of its upper reach lowlands. Generally, this flooding is of open land and wetlands, and causes no major problems. Additionally, the recently completed interstate system has altered the flow patterns within the upper reaches. This basin comprises a great portion of the Palmer Ranch development and will be significantly altered by the proposed changes. Recommendations for this basin are: cleaning out the main channel, where possible, in the area near U.S. 41, and conducting a comprehensive review of the flow generated by the Palmer Ranch development in light of their proposal to augment the existing flow with flow from other basins. Additionally, a flow control structure is needed in the Bay Road area.

Basin 9 - Shakett Creek

Shakett Creek currently experiences flooding along its primary tributary channels in subbasins 902, 903, and 904. Flooding also occurs along the Interstate 75 right-of-way. Generally, the wetland area flooding that occurs in the upland reaches will not be of concern for existing conditions. However, flooding of the area around Mission Valley Estates and the Kings Gate Club may become severe if large amount of development occur in the upland areas of this basin. The creek has been channelized and fairly well maintained due to the construction of Cow Pen Slough. Recommendations for this creek include the maintenance of all major and minor tributary channels, along with the replacement of bridge and culvert structures that are deteriorating.

Basin 10 - Curry Creek

Curry Creek currently experiences lowland and wetland flooding throughout its reach. Some private property flooding occurs in the outer reaches of the tributary channels and along the main channels. Areas north and south of the main channel of the creek exist where flood control storage areas can be located. Particularly, the area south of Settlement Road and north of Pinebrook South should be investigated for this purpose.

Basin 11 - Hatchett Creek

The Hatchett Creek basin currently experiences some flooding along the main channel of the creek and in the upstream areas. Minor flooding has been reported in the Venice Ranch area. Development is projected to occur in the near future in much of the area that is now open, especially in the area near Jacaranda Boulevard. Peak flow and volume must be kept at a level near the existing level if flooding of Venice Isles and East Gate is to be prevented. Recommendations for the basin include the general maintenance and cleaning program outlined for each basin, as well as the purchase of a right-of-way for either channel improvements or pond location in the area mentioned above. Additionally, the Stormwater Maintenance Division has recommended the placement of a stormwater control structure at the outlet from U.S. 41 of Jolonda Circle.

Basin 13 - Woodmere

The Woodmere Basin currently experiences flooding in the low lying areas at its upstream end, particularly south of Circlewood. The drainage channels from both of the subbasins need to be improved to allow for increased flows. Aerial photographs reveal sufficient right-of-way exists to allow the two main drainage ways to be improved.

Basin 14 - Forked Creek

The Forked Creek basin currently experiences minimal flooding along the main channels and tributaries, with heavy flooding confined to the wetland areas north and east of the urbanized portion of the basin. Recommendations include general clearing and cleaning of the existing channel, and the procurement of sufficient right-of-way to allow for the main channel to be improved west of State Road 775. A stipulation that the developer of this area dedicate and improve the existing channel would be appropriate.

Basin 15 - Godfrey Creek

The Godfrey Creek basin currently experiences flooding of its upper reaches. This is due to the nature of the low wetland areas that exist. The main channel above the County Road 777 experiences flooding caused by the lack of channel capacity. It is recommended that the channel north of the county line and upstream approximately one mile from Dearborn Street be cleared and improved to allow sufficient flow to pass.

Basin 16 - Ainger Creek

The Ainger Creek basin currently experiences the wetland flooding typical of these lowland areas. Generally, this flooding is confined to the area within Sarasota County. Because of this creek's outfall within Charlotte County, it is necessary that any flood control projects be coordinated with the proper Charlotte County authorities. All developments proposed within this basin should be reviewed by both counties involved to ensure a consensus of opinion on the types of stormwater controls that are acceptable. The Stormwater Management Division has identified the stream channel near Crestwood as an appropriate site for a flood control weir. The structure could then be used to moderate water levels in the portion of the Ainger Creek basin within Sarasota County.

## 7.4 CAPITAL IMPROVEMENT PLAN

To provide the County with cost projections for the 14 non-coastal basins, it is necessary that the costs of the two primary basins studied be extrapolated in some fashion. There are basically two methods which can be used to extrapolate costs based on the two in-depth studies.

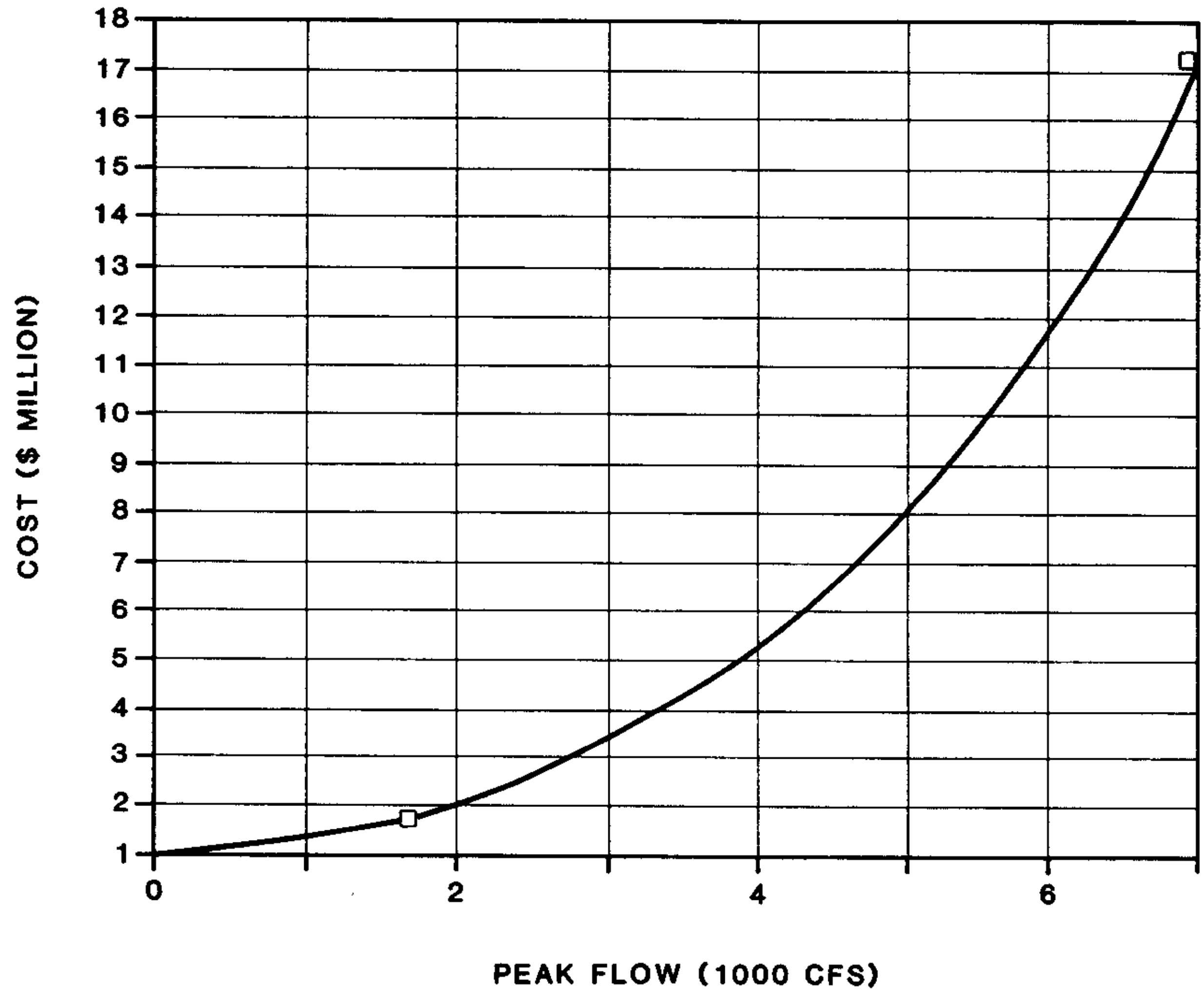
The first would be to extrapolate the costs based on the peak flow projected within the basins. Utilizing this method, the larger the peak flow the greater the capital costs. This method proves equitable if the basins are all primarily the same size and are comprised of comparable land use.

The second method is one in which the costs are apportioned based on total basin area. This method proves most reliable when dealing with basins whose characteristics are somewhat divergent. Generally, this method is more appropriate for basins of the diverse nature typical of Sarasota County.

## 7.5 PEAK FLOW

To apportion capital costs for the 14 non-coastal basins utilizing peak flow within Alligator and Phillippi Creek requires the use of a graphical representation of peak flow versus costs. The graph derived for this method is shown in Figure 7-15. It would, of course, not be appropriate to graph just two numbers and derive a viable cost/flow relationship. Therefore, flow and cost numbers from similar studies in Manatee and Sarasota Counties were used to supplement the Alligator and Phillippi Creek data.

The accompanying graph shows the peak flow from the various studies versus costs. The peak flows from the 14 basins have also been plotted. Table 7-16 gives the costs associated with the peak flows for the 14 non-coastal basins.




Peak Methodology Cost

FIGURE 7-15

TABLE 7-16

PEAK FLOW METHOD COSTS

| Basin Number | Name | Capital Costs($) |
|---|---|---|
| 1 | Whitaker Bayou | 3,400,000 |
| 2 | Hudson Bayou | 200,000 |
| 3 | Phillippi Creek | 17,252,041 |
| 4 | Matheny Creek | 650,000 |
| 5 | Gulf Gate Canal | 200,000 |
| 6 | Catfish Creek | 1,000,000 |
| 7 | South Creek | 500,000 |
| 8 | South Creek | 2,350,000 |
| 9 | Shakett Creek | 1,100,000 |
| 10 | Curry Creek | 2,000,000 |
| 11 | Hatchett Creek | 2,000,000 |
| 12 | Alligator Creek | 1,725,186 |
| 13 | Woodmere | 750,000 |
| 14 | Forked Creek | 575,000 |
| 15 | Godfrey Creek | 1,600,000 |
| 16 | Ainger Creek | 3,400,000 |

## 7.6 AREA METHOD

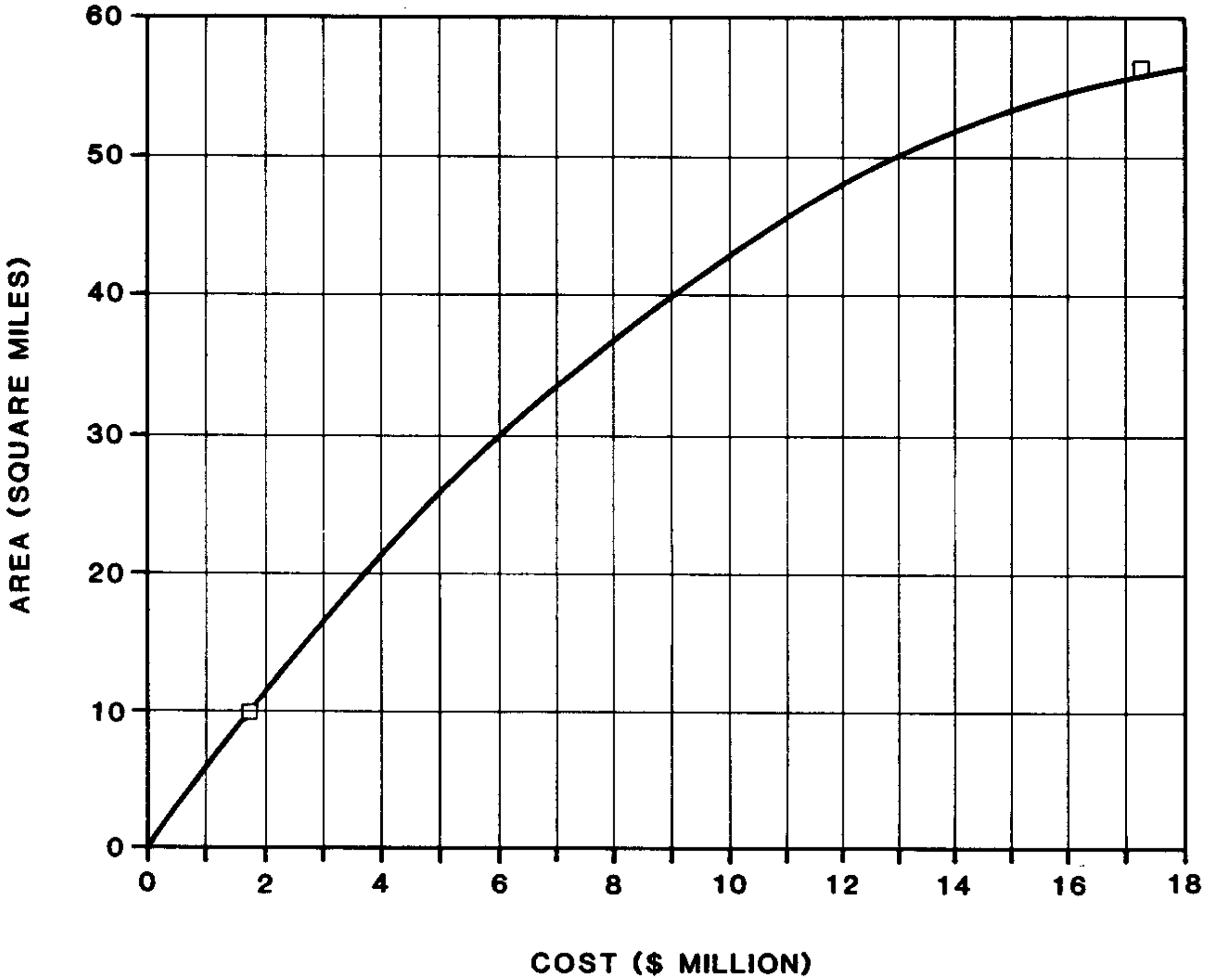
To apportion the capital costs to the 14 non-coastal basins using the basin area as the decisive factor the area of the individual basins are utilized. A graph of the relationship between area and capital cost is shown in Figure 7-16. To arrive at this graph, the costs of both the Alligator and Phillippi Creek studies were plotted versus area. Additional relationships derived in previous studies were also utilized. Thus, in this method the larger the basin the greater the projected capital costs.

The costs associated with each of the basins as derived from the graph are given in Table 7-17. This method was chosen for use in this study.

## 7.7 OPERATIONS AND MAINTENANCE

The costs associated with the operations and maintenance of stormwater facilities throughout the various 14 non-coastal basins have been extrapolated utilizing the projected costs of the Stormwater Maintenance Division of the County's Transportation Department. The procedure used to apportion costs involved the estimation by the Division of the total costs that would be expected if the Division had to begin a comprehensive maintenance program for all of the basins immediately.

Table 7-18 shows the projected additional labor costs associated with the Divisions new O&M responsibilities. These costs include labor (with fringe benefits), equipment costs, and administrative costs. As is shown, the total projected costs associated with operations and maintenance for the entire basin is $1,788,200 per year. This total includes the 1986/87 budget for the Division of $1,111,919.00. It was projected by the County that a full 80 percent of this budget number was expended to do maintenance within the study area. Using this 80 percent figure, the total O&M budget for the study area is approximately $1,565,800.00




Area Methodology Cost FIGURE 7-16

TABLE 7-17

AREA METHOD COSTS

| Basin Number | Name | Capital Costs($) |
|---|---|---|
| 1 | Whitaker Bayou | 2,200,000 |
| 2 | Hudson Bayou | 400,000 |
| 3 | Phillippi Creek | 17,252,041 |
| 4 | Matheny Creek | 480,000 |
| 5 | Gulf Gate Canal | 200,000 |
| 6 | Catfish Creek | 1,200,000 |
| 7 | North Creek | 600,000 |
| 8 | South Creek | 4,000,000 |
| 9 | Shakett Creek | 1,850,000 |
| 10 | Curry Creek | 1,600,000 |
| 11 | Hatchett Creek | 800,000 |
| 12 | Alligator Creek | 1,725,186 |
| 13 | Woodmere | 700,000 |
| 14 | Forked Creek | 1,500,000 |
| 15 | Godfrey Creek | 1,900,000 |
| 16 | Ainger Creek | 3,100,000 |

TABLE 7-18

ESTIMATED OPERATIONS AND MAINTENANCE COSTS

| CATEGORY | NUMBER | | COST PER YEAR |
|---|---|---|---|
| LABOR (1) | | | |
| Equipment Operators | 7 | @ $20,040 /Year | $ 140,280 |
| Foreman | 1 | @ $22,116 /Year | 22,116 |
| Laborers | 2 | @ $16,700 /Year | 33,400 |
| Herbicide Personnel | 4 | @ $18,871 /Year | 75,484 |
| | | | $ 271,280 |
| SUPPLIES | | | |
| Chemicals | | | $ 60,000 |
| EQUIPMENT (2) | | | |
| Dump Trucks | 3 | @ $11,000 /Year | $ 33,000 |
| Gradall | 1 | @ $25,400 /Year | 25,000 |
| Spyder | 1 | @ $24,000 /Year | 24,000 |
| Tractors | 2 | @ $7,000 /Year | 14,000 |
| Side Arm Mowers | 2 | @ $6,000 /Year | 12,000 |
| Bush Hog | 1 | @ $1,100 /Year | 1,100 |
| Spray Trucks | 2 | @ $3,000 /Year | 6,000 |
| Pick-Ups | 7 | @ $2,800 /Year | 19,600 |
| Mowers | 2 | @ $200 /Year | 400 |
| | | | $ 138,000 |
| EQUIPMENT MAINTENANCE (@ 30 PERCENT OF TOTAL EQUIPMENT COSTS) | | | $ 207,000 |
| TOTAL OPERATIONS AND MAINTENANCE BUDGET FOR STUDY AREA | | | |
| 1986/1987 Year Budget | | | $1,111,919 |
| Equipment Costs | | | 138,000 |
| Equipment Maintenance | | | 207,000 |
| Supplies | | | 60,000 |
| Labor | | | 271,280 |
| | | | $1,788,199 |
| (LESS 20% OF 1986/1987 BUDGET) | | | ($222,384) |
| | TOTAL | | $1,565,815 |

NOTE: (1) All labor costs include fringe benefits @ 67 percent.
(2) All equipment costs have been amortized over five years.

This total project wide O&M cost number is then apportioned over the 16 in accordance with their areas. Table 7-19 shows the approximate annual maintenance costs per basin. As would be expected, Phillippi Creek which is by far the largest basin requires the greatest outlay for operations and maintenance.

TABLE 7-19

PROJECTED BASIN
OPERATIONS AND MAINTENANCE COSTS

| BASIN NUMBER | AREA (SQ. MI.) | COST |
|---|---|---|
| 1 | 12.32 | $107,644 |
| 2 | 2.15 | 18,785 |
| 3 | 56.35 | 492,348 |
| 4 | 2.36 | 20,620 |
| 5 | 1.67 | 14,591 |
| 6 | 6.44 | 56,268 |
| 7 | 3.81 | 33,289 |
| 8 | 20.13 | 175,882 |
| 9 | 10.76 | 94,014 |
| 10 | 9.10 | 79,510 |
| 11 | 5.42 | 47,356 |
| 12 | 9.86 | 86,150 |
| 13 | 3.90 | 34,076 |
| 14 | 8.76 | 76,539 |
| 15 | 10.74 | 93,839 |
| 16 | 15.44 | 134,904 |
| | | $1,565,815 |

# 8.0 FINANCING DRAINAGE IMPROVEMENTS

# 8.0 FINANCING DRAINAGE IMPROVEMENTS

## 8.1 GENERAL

This section will present a methodology for cost recovery and the assessment of annual costs. A discussion on the legal basis of the assessments, the general financing requirements, and the necessary steps for implementation of the county-wide program is also included.

## 8.2 COST RECOVERY - GENERAL

The primary consideration in the development of recovery alternatives is to establish an easily measurable basis for cost allocations that reasonably represents the purpose(s) for which the cost(s) were incurred. Once such a basis is established, the assessment of costs can be equitably allocated to the parties receiving the benefit of the cost incursion(s).

Sarasota County will incur both capital and operations and maintenance costs for drainage facilities and activities whose main purpose is to eliminate or mitigate damage to life and property and improve the water quality of stormwater runoff. The benefits thus derived accrue to the land holders and citizens of the County.

An important requirement affecting cost recovery is covered in the County ordinances which established their special assessment (or drainage) districts, (ordinance numbers 85-08 through 85-38, enacted 1984) in each of the 16 non-coastal basins within the County's boundaries. The ordinances were explicit regarding cost recovery, stating that each basin would bear the cost burden for all benefits or improvements within the basin's boundaries. (Thus the analyses in the previous sections of this report concentrated on determining the drainage improvements required on a basin by basin basis.)

## 8.3 BASIS OF COST ALLOCATIONS

Runoff from property with specific land use (i.e., residential, commercial industrial, open space, rural) for a given amount of rainfall can generally be attributed to the percentage of the property's impervious area.

The contribution of a given amount of runoff to the peak value (the value for which drainage improvements are designed to control) is also a function of the type of land use, especially the percentage of impervious area. The greater the percentage of impervious area on an individual piece of property, the greater its contribution to the peak and total runoff.

Impervious area was the unit selected as the basis of allocating costs. The selection was based upon:

- o the importance of this parameter in contributing to the peak storm water runoff conditions;

- o the ease in which this parameter could be measured (i.e., assessors' maps of properties on the existing tax roles); and

- o the credibility of incorporating this parameter based upon its use, in one manner or another, in almost every storm drainage rate structure in the United States.

The analysis developed herein consists of applying an adjustment factor to the impervious area value of a particular land use classification. The factor acts as a weighting mechanism which adjusts the numerical value of land area to an equivalent area reflecting the runoff contribution. This land adjustment factor, Ip, represents the percentage, expressed as the decimal equivalent, of impervious area of a specific land use type or particular parcel.

For Sarasota County, the factors adopted were as follows:

| Land Use | Ip |
|---|---|
| Residential | 0.35 |
| Commercial/Industrial | 0.95 |
| Rural | 0.04 |

Note: The commercial/industrial land use category includes institutional, governmental, and utility land use.

If residential land use is used as the primary unit for assessing charges, then in terms of the residential value of Ip, the values for other land uses are as follows:

| | | | | |
|---|---|---|---|---|
| Ip (Residential) | = | 1.00 | = | (0.35/0.35) |
| Ip (Commercial/Industrial) | = | 2.71 | = | (0.95/0.35) |
| Ip (Rural) | = | 0.11 | = | (0.04/0.35) |

In effect, the Ip factors indicate that for equal sizes of land area, commercial/industrial contributions to peak runoff conditions (on the average) will be less than three times that of residential areas. Residential areas would contribute about nine times that of rural areas.

The percentage of imperviousness used in the above calculations were selected to represent the average case. However, when the final user charge analysis is completed the actual percent impervious for each parcel may be compared to the average residential unit to determine the actual utility user charge.

## 8.4 APPLICATION TO ALLIGATOR AND PHILLIPPI CREEK BASINS

Application of the land use adjustment factor (Ip) to the Alligator and Phillippi Creek basins is shown in Table 8-1. The computation in Table 8-1 considers the percentages of land use constant throughout the basin. However, the calculation could be made for each subbasin within the major basins, using existing tax assessor roles.

TABLE 8-1

SARASOTA COUNTY STORMWATER MASTER PLAN
ADJUSTED LAND USE FOR COST ALLOCATION

| Basin | Land Use Type | Number of Actual ( Acres | x | Ip) | = | Adjusted Acreage | % Adjusted Acreage |
|---|---|---|---|---|---|---|---|
| Alligator Creek | Residential | 3,867 | | 1.0 | | 3,867 | 69.7 |
| | C/I | 543 | | 2.71 | | 1,452 | 26.5 |
| | Rural | 1,904 | | 0.11 | | 209 | 3.8 |
| TOTALS | | 6,314 | | | | 5,528 | 100.0 |
| Phillippi Creek | Residential | 13,209 | | 1.0 | | 13,209 | 65.2 |
| | C/I | 1,736 | | 2.71 | | 4,705 | 23.2 |
| | Rural | 21,368 | | 0.11 | | 2,350 | 11.6 |
| TOTALS | | 36,313 | | | | 20,264 | 100.0 |

The important concept inherent in the methodology is that the percentage shown for adjusted acreage is equal to percentage of the total costs incurred in the particular basin allocated for the specific land use or parcel. The cost implications of this methodology for the Alligator and Phillippi Creek basins can be better understood by examining the following comparisons:

| Land Use Type | Alligator Creek Basin | | | | Phillippi Creek Basin | | | |
|---|---|---|---|---|---|---|---|---|
| | Actual Acreage | % | Adjusted Acreage | % | Actual Acreage | % | Adjusted Acreage | % |
| Residential | 3,867 | 61.2 | 3,867 | 69.7 | 13,209 | 36.4 | 13,209 | 65.2 |
| C/I | 543 | 8.6 | 1,471 | 26.5 | 1,736 | 4.8 | 4,705 | 23.2 |
| Rural | 1,904 | 30.2 | 209 | 3.8 | 21,368 | 58.8 | 2,350 | 11.6 |
| | | 100.0 | | 100.0 | | 100.0 | | 100.0 |

The comparison between the actual and adjusted acreages clearly indicates that the total acreage for commercial and industrial land use represents a relatively small percentage of the total acreage, but contributes significantly to peak runoff. This land use type will therefore be allocated a much higher percentage of the cost responsibility than indicated by actual acreage values. The converse is true for the rural land use areas, while the cost responsibility of the residential areas will be about the same percentage, as indicated by the actual land use values.

In summary, the keys to the level of cost allocations to various types of land use are:

- the basin size and land use mix;
- the type of improvements required and their total cost; and
- the number of acres and type of property sharing in the cost.

## 8.5 COST REQUIREMENTS

### 8.5.1 FACILITIES COSTS

To estimate the cost to each parcel of property, it is necessary to use not only the costs associated with the main channel improvements, but also those subbasin improvements that are likely to have an impact on the five-year Capital Improvement Program (CIP). The facilities costs associated with these various main channel improvements have been discussed in Sections 5.8, 6.9 and 7.4. The estimated cost for subbasin improvements were also included.

### 8.5.2 OPERATION AND MAINTENANCE (O&M) COSTS

The County will provide all O&M service to the various betterment districts. The ordinances which established the betterment districts give total expenditure control to the County.

Thus, the Sarasota County Transportation Department could organize a drainage section and separately account for all operation and maintenance costs it provides to individual districts. Or, the O&M costs can be shared equally among the districts allocated by area in each basin or through some equitable formula. A method which allocates O&M costs based upon basin area and computes a factor for each basin allocating the total O&M costs has been shown previously (see Section 7.4).

### 8.5.3 ADMINISTRATION AND OTHER COSTS

In order to implement and operate the storm drainage assessment programs, additional costs for County administration would be necessary. These costs include review of drainage plans and inspection of drainage works, (say an engineer full-time, and a construction inspector or engineering aid half- to full-time), costs to bill and collect revenues, general expenses for accounting and bank account supervision, and general maintenance of customer accounts (i.e., keeping track of property type and recording changes, additions and deletions to the drainage "customer file," and miscellaneous costs).

Billing and collections plus file maintenance should be paid for by the annual drainage fees. The plan review and inspection should probably be handled by full-time employees in the Transportation Department, and costs could partially be offset by fees for plan filing.

Administrative costs can be roughly estimated as follows:

| | |
|---|---|
| Engineer | $ 35,000/yr (with fringes) |
| Inspector | 22,000/yr |
| Billing/Collection | 18,000/yr |
| Accounting/Administration | 24,000/yr |
| Miscellaneous | 20,000/yr |
| | $119,000/yr |
| | Say $130,000/yr |

An additional one time set-up cost would be required. This would involve classifying <u>all</u> properties by type and establishing the customer master file and billing system. (The billing system could probably be incorporated into the County's property tax system.) The set-up costs could range between $40,000 and $80,000, depending upon the applicability of the County's property tax system.

Other costs refer to major repairs that may occur, on a yearly or every other year basis. These costs would be recovered through direct charges to the properties of the drainage basins affected. However, to make the repairs (actually small construction jobs) with its own forces or through a contractor, the County would be required to have the money in-hand. The initial infusion of funds could be from a portion of the proceeds of an initial bond issue, or a fund could be established by a one time contribution from all property owners. The level of the fund would depend on basin size and development and is estimated to run between $100,000 and $250,000. (If the fund is not used, the interest income could be treated as a contribution to O&M and administrative costs, thus reducing the annual charges necessary.)

## 8.6 UNIT COST ESTIMATES

This section will present sample computations to illustrate the magnitude of the annual costs (per acre) involved from Sections 5.8 and 6.9. The capital and O&M costs for the two drainage areas are as follows:

| Alligator Creek | | Phillippi Creek | |
|---|---|---|---|
| Capital | $1,725,186($\pm$) | Capital | $17,252,041($\pm$) |
| O&M | $ 86,000/yr | O&M | $ 492,000/yr |
| Administration | $ 18,000/yr | Administration | $ 40,000/yr |

The capital costs expressed above are estimates which include the recommended main channel and perceived subbasin improvement costs. Administration costs include the one time set-up cost of $50,000 and $80,000 for Alligator and Phillippi Creeks, respectively.

If the capital improvements and set-up are financed through the issuance of long-term bonds, the annual capital costs, assuming equal annual payments, for 20-year bonds at 8 percent annual interest would cause the annual capital charges to be $94,473 and $922,391 for the Alligator and Phillippi Creek basins, respectively. The unit cost, assuming all properties in each basin contributed towards capital repayment and annual O&M charges, would be as shown in Tables 8-2 and 8-3.

The estimated annual costs for all of the basins is given in Table 8-4. As is shown, the expected annual costs range from a high of $45/acre for Hatchett Creek to a low of $20/acre for Forked Creek.

TABLE 8-2

ALLIGATOR CREEK BASIN
ILLUSTRATIVE COMPUTATIONS: COST ALLOCATIONS AND ANNUAL COST

ANNUAL COSTS ASSUMING ALL PROPERTIES SHARE COSTS

| Land Use | Actual Acres | Percentage of Equivalent Area | Annual Cost Allocation | Annual Cost Per Acre |
|---|---|---|---|---|
| Residential | 3,867 | 69.7 | $138,006 | $ 36 |
| C/I | 543 | 26.5 | 52,470 | 97 |
| Rural | 1,904 | 3.8 | $ 7,524 | 4 |
| TOTALS | 6,314 | 100.0 | $198,000 | $ 31 (Average Basin Wide) |

ANNUAL COSTS ASSUMING ONLY DEVELOPED PROPERTIES SHARE COSTS

| Land Use | Actual Acres | Percentage of Equivalent Area | Annual Cost Allocation | Annual Cost Per Acre |
|---|---|---|---|---|
| Residential | 3,867 | 72.44 | $143,431 | $ 37 |
| C/I | 543 | 27.56 | 54,569 | 100 |
| TOTALS | 4,410 | 100.0 | $198,000 | $ 45 (Average Basin Wide |

TABLE 8-3

PHILLIPPI CREEK BASIN
ILLUSTRATIVE COMPUTATIONS FOR COST ALLOCATIONS

ANNUAL COSTS ASSUMING ALL PROPERTIES SHARE COST

| Land Use | Adjusted Acres | Percentage of Equivalent Area | Annual Cost Allocation | Annual Cost Per Acre |
|---|---|---|---|---|
| Residential | 13,209 | 65.2 | $ 948,263 | $ 72 |
| C/I | 1,736 | 23.2 | 337,419 | 194 |
| Rural | 21,368 | 11.6 | 168,709 | 8 |
| TOTALS | 36,313 | 100.0 | $1,454,391 | $ 40 (Average Basin Wide) |

ANNUAL COSTS ASSUMING ONLY DEVELOPED PROPERTIES SHARE COSTS

| Land Use | Adjusted Acres | Percentage of Equivalent Area | Annual Cost Allocation | Annual Cost Per Acre |
|---|---|---|---|---|
| Residential | 13,209 | 73.7 | $1,072,430 | $ 81 |
| C/I | 1,736 | 26.3 | 381,960 | 220 |
| TOTALS | 14,945 | 100.0 | $1,454,391 | $ 97 (Average Basin Wide) |

TABLE 8-4

COST ALLOCATION BY BASIN

ANNUAL COST PER ACRE[1]

| Basin Number | Name | Capital Costs ($/Yr) | O & M Costs ($/Yr) | Annual Costs[(2)] ($/Acre) |
|---|---|---|---|---|
| 1 | Whitaker Bayou | $128,100 | $108,000 | $30 |
| 2 | Hudson Bayou | 32,025 | 19,000 | 37 |
| 3 | Phillippi Creek | 922,391 | 492,000 | 40 |
| 4 | Matheny Creek | 336,000 | 21,000 | 36 |
| 5 | Gulf Gate Canal | 10,500 | 15,000 | 24 |
| 6 | Catfish Creek | 58,800 | 56,000 | 28 |
| 7 | North Creek | 32,025 | 33,000 | 27 |
| 8 | South Creek | 128,625 | 176,000 | 24 |
| 9 | Shakett Creek | 88,200 | 94,000 | 26 |
| 10 | Curry Creek | 109,725 | 80,000 | 33 |
| 11 | Hatchett Creek | 109,725 | 47,000 | 45 |
| 12 | Alligator Creek | 94,473 | 86,000 | 31 |
| 13 | Woodmere | 43,050 | 34,000 | 31 |
| 14 | Forked Creek | 34,650 | 77,000 | 20 |
| 15 | Godfrey Creek | 82,950 | 94,000 | 26 |
| 16 | Ainger Creek | 190,050 | 135,000 | 33 |

[1] Based on the total capital costs being financed over a 20-year period at an interest rate of 8 percent.

[2] Average annual cost with all properties involved.

If we assume an average density of 4 dwelling units (DU) per acre, the actual cost per year would be calculated for Phillippi Creek as:

$$\frac{40\ \$/\text{acre}}{4\ \text{DU acre}} = \$10.00/\text{year or about } \$1.00/\text{month.}$$

# APPENDIX A

# MODELING APPROACH

# APPENDIX A

# MODELING APPROACH

## A.1 INTRODUCTION

To quantify the stormwater runoff that occurs due to a design rainfall event, it is necessary to consider both the basin hydrology and the channel hydraulics. There are many computer models currently available that convert rainfall hyetographs to runoff hydrographs. These models include TR20, TR55, DABRO, HNV Santa Barbara, and SWMM. All of these models have applications. However, EPA's Stormwater Management Model (SWMM) appears to be the most highly regarded, and most thoroughly applied and documented model. For this reason, an adaptation of the primary hydrology and hydraulics package (RUNOFF) of the SWMM model was chosen for this study. This adaptation, referred to herein as the Microcomputer Stormwater Simulation Model (MSSM), will be discussed in detail in subsequent sections of this chapter.

The MSSM model results yield runoff hydrographs which show the peak flows and duration at various points along the modeled stream channel. A surface water profile model is used to convert the peak flow along the stream channel to the depth of flow, taking into account the channel geometry and flow constrictions. For the purposes of this study, the U.S. Army Corps of Engineers' HEC-2 Water Surface Profile Model is utilized.

## A.2 INTRODUCTION TO MSSM

The Microcomputer Stormwater Simulation Model (MSSM) is composed of two distinct subprograms. RUNOFF, a FORTRAN program, simulates the surface runoff and conveyance system hydrographs for a watershed undergoing a certain rainfall event. PRERUN, a preprocessor running under dBASE III, manages hydrologic data for up to 99 watersheds and creates an input data file to RUNOFF for the hydrologic simulation of runoff within a watershed.

The MSSM subprogram was originally derived from the RUNOFF block of EPA's Stormwater Management Model. In 1977, SWMM-RUNOFF was modified extensively to form the RUNOFF block of the RUNQUAL model developed for use in the Southeast Michigan Council of Governments' 208 planning program. Further modifications were made for stormwater master planning in Pinellas County, Florida, and for the Tennessee Valley Authority. Quality portions of the model were eliminated to streamline the current model for installation on the IBM PC-AT microcomputer for the Sarasota County Stormwater Master Plan project.

### A.2.1 SUBPROGRAM DESCRIPTIONS

MSSM accepts data on the physical characteristics of the watershed being modeled, and the network of channels that drain the area. Rainfall hyetographs can then be applied to the watershed to compute the overland flows and route them through the channel network, storing the outflows on tape. Printed or plotted output may include hydrographs for any channel in the network, and time histories of lake storage.

PRERUN is a menu driven, user-friendly hydrologic data management system for multiple watersheds within one or more stormwater management jurisdictions. Watershed, channel, subarea, lake, rainfall, and analysis information are contained in several data base files constructed via dBASE III. A series of command files transparently access the dBASE III commands required for the maintenance of the hydrologic data bases and creation of input data files suitable for MSSM. Thus, PRERUN is an ideal tool for organizing a variety of hydrologic data and simulations for multiple watersheds.

### A.2.2 CAPABILITIES AND LIMITATIONS

Although MSSM has been developed to be a relatively general program, certain dimensional limitations have been imposed upon it during program development to limit core storage required. The limits are shown in Table A-1.

TABLE A-1

CORE STORAGE REQUIREMENTS

| Limitation | Quality |
|---|---|
| Watersheds | 99 |
| Subareas per watershed | 100 |
| Channels or reaches per watershed | 99 |
| Double trapezoidal channels per watershed | 50 |
| Lakes per watershed | 30 |
| Outlet control structures per lake | 7 |
| Elevation/storage/discharge curve points per lake | 20 |
| Subareas draining into single channel | 10 |
| Rainfall gauges per watershed | 25 |
| Hyetograph points per rainfall event | 100 |

## A.3 FORMULATION OF MSSM

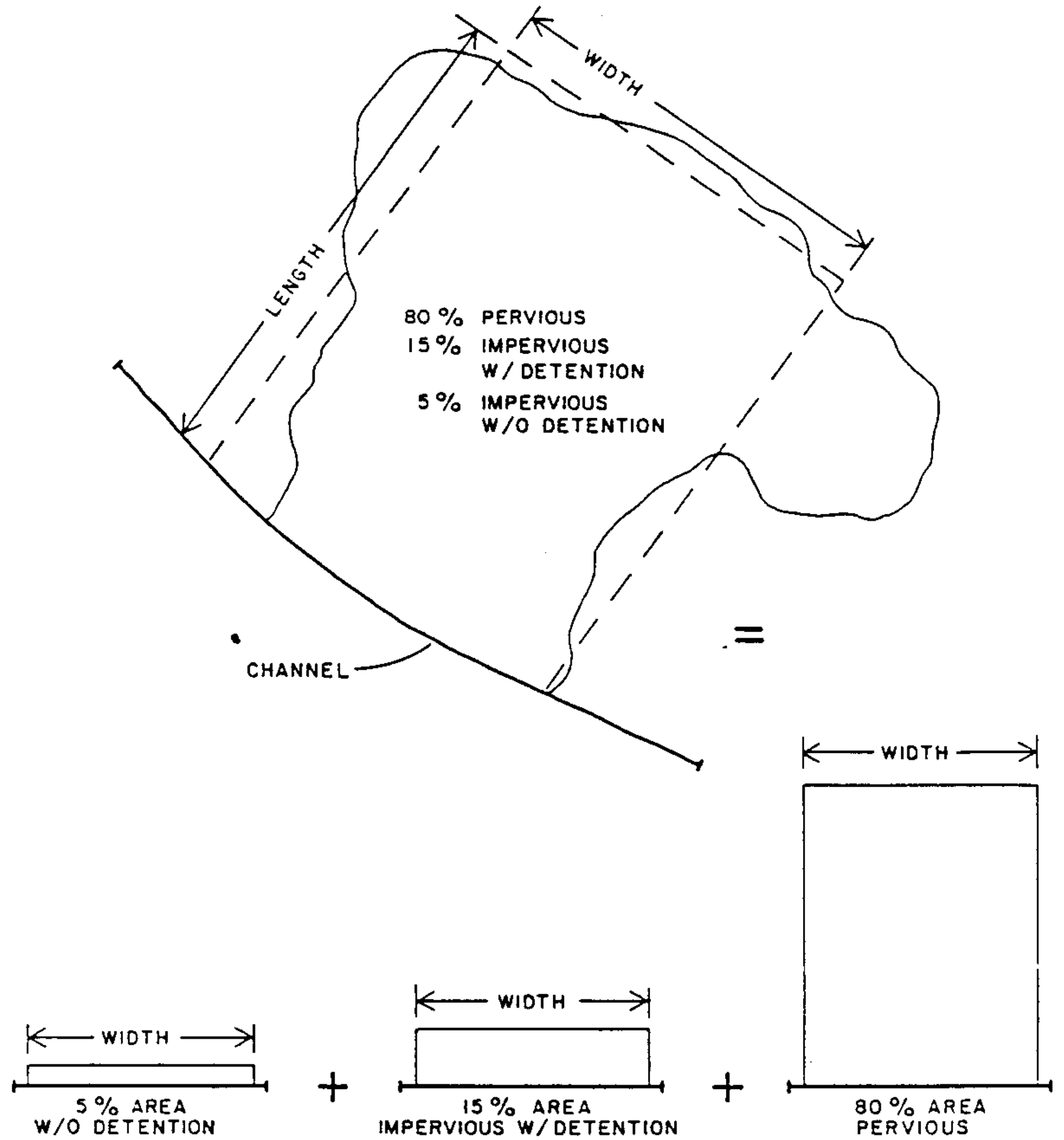
This section describes the formulation and mathematical structure of the surface runoff system represented in MSSM. The discussion is divided into two parts dealing with the major computational routines: surface runoff and channel routing.

### A.3.1 SURFACE RUNOFF

A typical watershed subarea is shown in Figure A-1. Each subarea may contain a complex mixture of land uses, each having a characteristic percentage of its area being impervious. MSSM aggregates all the impervious surfaces directly connected to the stormwater drainage system (having depression storage), regardless of their individual composition, to form a single plane which discharges laterally to a channel. Likewise, all the pervious and impervious area not directly connected to the stormwater drainage system in the subarea is aggregated to form another single plane, having the same width (WIDTH = total subarea AREA/LENGTH). A third plane is defined as that fraction of the directly connected impervious area which has no detention storage at all and produces immediate runoff at the start of a storm. The flow off the subarea is the sum of the flow off the three planes.

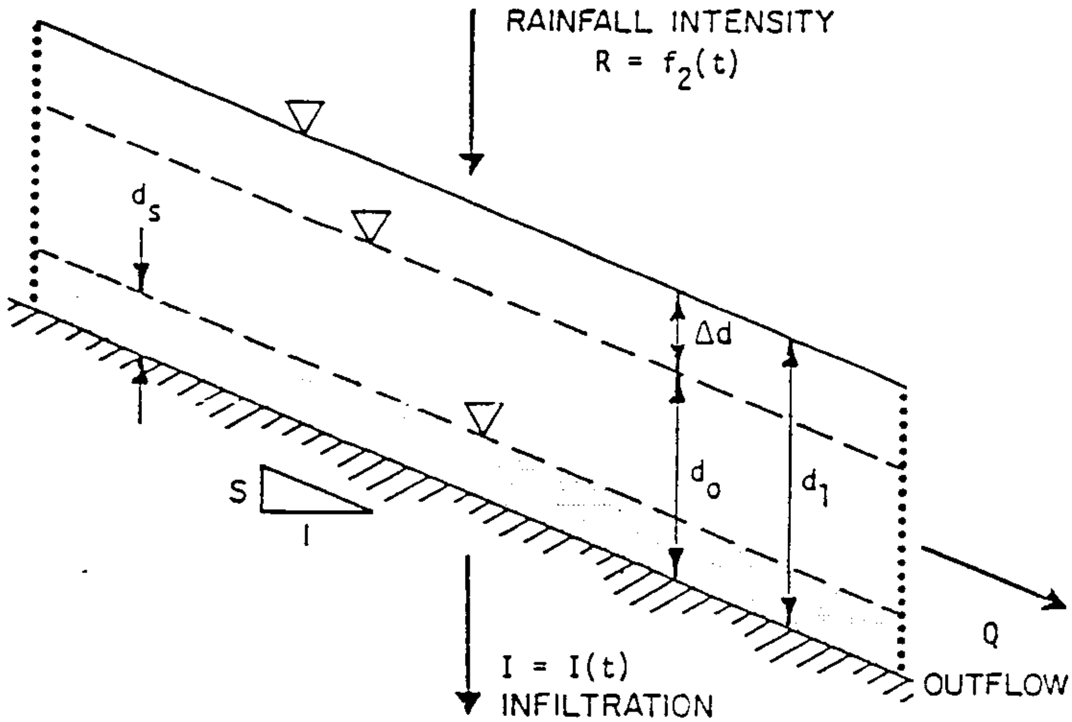
The basic flow routing algorithm in MSSM is the kinetic wave approximation which assumes that the friction slope is equal to the slope of the plane. For this condition, the equations of continuity and uniform flow must be solved simultaneously to define at each time step the depth of flow and the outflow for each of the three flow planes defined above. The flow routing algorithm is applied sequentially to the impervious (with detention), pervious, and impervious (without detention) areas in that order.

The three-plane runoff computation sequence can be generalized for the pervious flow plane shown in Figure A-1. At the end of each time step, $t$, we have two unknowns, $Q$ and $d_1$, and two equations as indicated in Figure A-2. Three flow depths are shown in the figure:




Typical Watershed Subarea

FIGURE A-1




INFILTRATION: $I = k_1 + (k_2 - k_1)e^{-k_3 t}$

FLOW: $Q = \frac{1.49}{n} S^{1/2} w\left(\frac{d_o + d_1}{2} - d_s\right)^{5/3}$

STORAGE: $\frac{\Delta d}{\Delta t} = R - I - \frac{Q}{A_s}$

Basic Flow Calculations for
Typical Watershed Subarea

**FIGURE A-2**

$d_o$ = depth at time t,
$d_1$ = depth at time t + (delta)t, and
$d_s$ = maximum depth of detention storage.

The objective of the calculations which pertain to this element is to find the new depth, $d_1$, determining in the process the outflow, Q, and maintaining mass continuity at all times. To accomplish this, two equations must be solved simultaneously. The first is the continuity or storage equation:

$$\frac{d}{t} = R - I - \frac{Q}{A_s} \tag{A-1}$$

where

d = $d_1 - d_o$,
R = rainfall during t,
I = infiltration to groundwater during t,
Q = outflow from subarea, and
$A_s$ = surface area of plane.

The second is the Manning equation for overland flow with the hydraulic radius set equal to average depth (wide channel assumption):

$$Q = \frac{1.49}{n} s^{1/2} w \quad \frac{d_o + d_1}{2} - d_s^{\quad 5/3} \tag{A-2}$$

where

s = slope of ground surface,
n = Manning coefficient, and
w = width of the plane.

Here we have two equations and two unknowns, Q and $d_1$. Note that the flow computation is based on the average depth during time t, and that surface detention is not included in the effective depth of flow. Rainfall intensity is an input quantity, variable in time, but considered constant during each time interval t. Infiltration is computed by a modified Horton formula written as:

$$I = K_1 + (K_2 - K_1)e^{-K_3 t} \tag{A-3}$$

where

$I$ = infiltration loss rate, inches per hour,
$K_1, K_2$ = minimum and maximum infiltration rates, respectively,
$K_3$ = exponential rate of loss in infiltration capacity, and
$t$ = time in hours.

During periods of light or zero rainfall, the net precipitation value, R-I, could become negative. Traps in the computer program prevent this occurrence and thus modify the Horton approach.

Equations A-1 and A-2 are nonlinear and their simultaneous solution is performed by a Newton-Raphson iterative technique for locating the zero-crossing of the first derivative. First, the equations are combined and rearranged in the form:

$$F = \quad d - \quad t \,(k\bar{d}^{\,5/3} + R_{net}\,) \tag{A-4}$$

where

$F$ = Newton's function whose zero-crossing is to be located

$$k = - \;\frac{1.49}{n} s^{1/2} w \;\; / A_s$$

$$\bar{d} = \frac{d_o + d_1}{2} - d_s = d_o - d_s + \frac{d}{2}$$

$$R_{net} = (R - I)$$

Then, differentiating yields:

$$\frac{dF}{d[(\text{delta})d)} = 1 - \; t\,\frac{5}{6}\,k\,\bar{d}^{\,2/3} \qquad \text{(A-5)}$$

The numerical method is a recursive process for finding the value of d:

$$[(\text{delta})d]_{n+1} = [(\text{delta})d]_n - \frac{F_n}{\dfrac{d\,F_n}{d[(\text{delta})d]}} \qquad \text{(A-6)}$$

where the subscripts refer to the $n^{th}$ and $(n+1)^{th}$ iterations. Repeated application of this expression converges upon F = 0. However, because of the possibility of truncation when subtracting numbers that are very close to each other, F may never converge upon 0 on some computers, although an adequate solution has been reached. For this reason, the convergence check is based on the percentage change in d from the previous iteration reaching some small value. The convergence criterion is:

$$|[(\text{delta})d]_{n+1} - [(\text{delta})d]_n| < |0.01[(\text{delta})d]_n| \qquad \text{(A-7)}$$

The corresponding value of d gives the final depth, $d_1$, and in turn the outflow, Q, which are the desired results. The solution is then repeated for the next flow plane in the sequence, and the entire sequence is repeated for all time steps in the surface runoff simulation period.

The solutions for the impervious with detention and impervious without detention flow planes are similar, the only change being that in the former case infiltration (I) is set to zero and in the latter case infiltration (I) and detention depth ($d_s$) are both set to zero. Of course, each of the three planes has its unique surface area ($A_s$), the sum of the three areas being equal to the total area of the watershed subarea, as input.

## A.3.2 CHANNEL ROUTING

### Basic Flow Computations

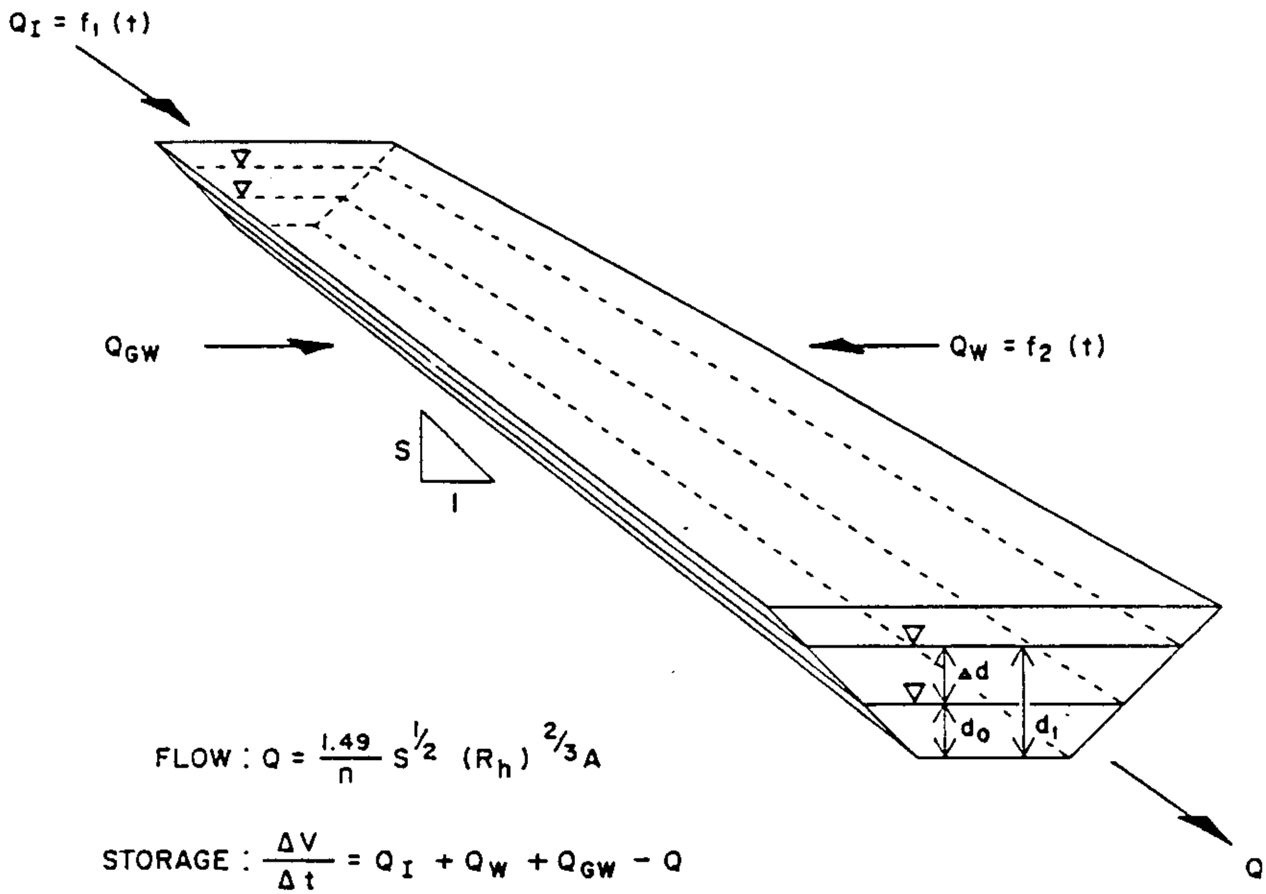
A typical channel is shown in Figure A-3. The channel may be a trapezoidal channel, pipe, or double trapezoid (with overbank channel). Lakes with weir, spillway, riser, orifice, or outlet channel control are treated differently from these channel types and are discussed separately. For each time step, outflow from every channel, Q, is determined beginning with the most upstream channel and working downstream. The outflow from each channel then serves as inflow to the next downstream channel.

As with watershed subareas, the two unknowns at the end of each time step are Q and $d_1$. The known quantities are inflows $Q_I$, $Q_W$, and $Q_{GW}$, and depth $d_0$, where

$d_0$ = depth at time t,
$d_1$ = depth at time t + (delta)t,
$Q_I$ = inflow from upstream channel(s), constrained channels, and inlets,
$Q_W$ = inflow from adjacent watershed subareas,
$Q_{GW}$ = groundwater inflow, and
Q = outflow from channel.

$Q_I$ has three possible sources: outflow from channels immediately upstream, overflows from constrained channels upstream (this is explained later), and inlet inflows read from a tape. $Q_W$ is the sum of the outflows from the three planes in adjacent subareas, as discussed above. $Q_{GW}$ for each channel is constant with time and is computed by establishing a water balance on the system based on input values of baseflows in all channels.

The solution for $d_1$ and Q is similar to that used to compute flow off watershed subareas. Again, the kinematic wave approximation is made and the equations of continuity and uniform flow are solved simultaneously at each time step. The continuity equation is:




$$\text{FLOW}: Q = \frac{1.49}{n} S^{1/2} (R_h)^{2/3} A$$

$$\text{STORAGE}: \frac{\Delta V}{\Delta t} = Q_I + Q_W + Q_{GW} - Q$$

Basic Flow Calculations for Typical Channel

**FIGURE A-3**

$$\frac{V}{t} = Q_I + Q_W + Q_{GW} - Q \tag{A-8}$$

where

V = volume change associated with (delta)d.

The outflow Q is determined from Manning's equation:

$$Q^* = \frac{1.49}{n} s^{1/2} R_h^{2/3} A \tag{A-9}$$

where

s = slope of channel bottom,
n = Manning's coefficient,
$R_h$ = hydraulic radius (= A/wetted perimeter), and
A = cross-sectional area of flow.

Q* is computed for both $d_o$ and $d_1$, and the average taken as Q. The Newton-Raphson iterative technique is employed to solve equations A-8 and A-9. Newton's function is written:

$$F = V + t(Q - Q_I - Q_W - Q_{GW}) \tag{A-10}$$

in which V and Q are expressed in terms of $d_o$ and $d_1$. $\frac{dF}{d[(delta)d]}$ is is found and the recursive process for finding d,

$$[(delta)d]_{n+1} = [(delta)d]_n - \frac{F_n}{\frac{d\,F_n}{d[(delta)d]}} \tag{A-11}$$

where the subscripts refer to the $n^{th}$ and $(n+1)^{th}$ iterations, is employed to reduce the value of F, approaching 0. As in the subarea calculations, a convergence criterion other than F = 0 is required. A convergence criterion based on flow change between iterations has proven to be stable and efficient:

$$|Q_{n+1} - Q_n| < 0.001\ (Q_{n+1}) \qquad \text{(A-12)}$$

After a solution for $d_1$ and Q is reached, the procedure is repeated for the next channel downstream, Q becoming $Q_I$ for that channel. When the new depth, $d_1$, exceeds the full depth of the channel, one of three things may happen, depending on the channel type: surcharge, overbank flow, or constrained channel overflow.

Surcharge

Normal channels (types 1, 2, and 3) will surcharge. The volume of surcharge,

$$V_{SUR} = (Q_I + Q_W + Q_{GW} - Q_{FULL})\ \text{(delta)t} \qquad \text{(A-13)}$$

where

$V_{SUR}$ = surcharge volume for time step, and
$Q_{FULL}$ = outflow from channel at full depth,

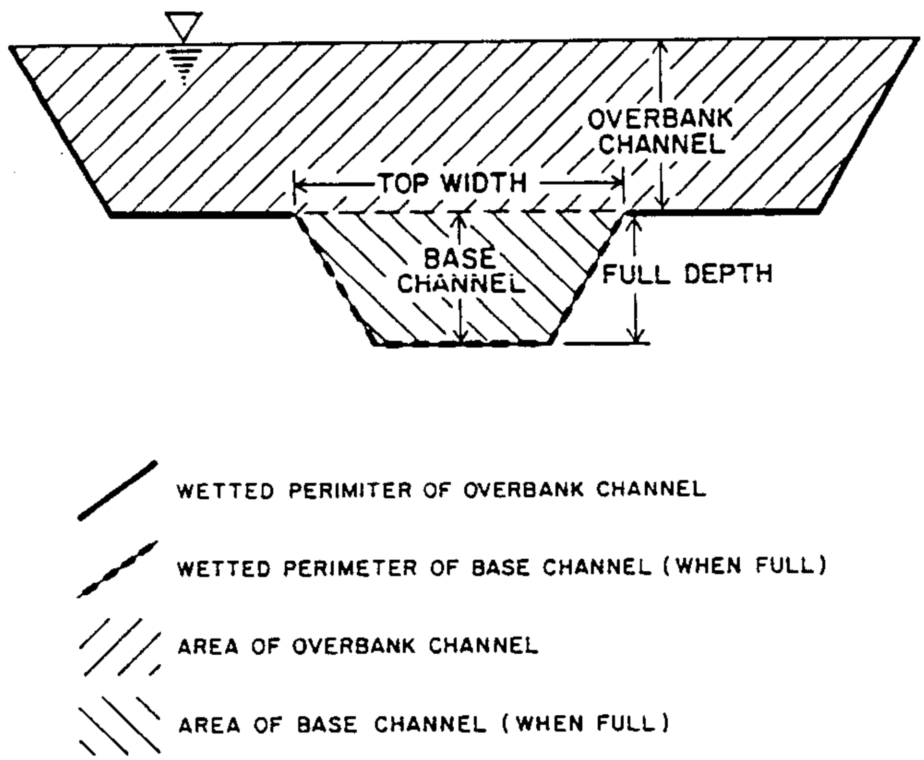
is set aside. Since $V_{SUR}$ may be positive or negative, the total volume in surcharge may increase or decrease with time. Each channel has its own surcharge volume which is handled separately from any other.

Overbank

The flow in double trapezoidal channels, Figure A-4, is handled normally until the depth reaches full channel depth for the base channel. At that point, if inflow exceeds outflow,

$$Q_I + Q_W + Q_{GW} > Q_{FULL} \qquad \text{(A-14)}$$

the excess, $Q_I + Q_W - Q_{FULL}$, is entered into the overbank channel as the sole inflow. The depth and flow from the overbank channel are found by iteration as for standard channels, the only difference being that the




Double Trapezoidal Channel Schematic

FIGURE A-4

wetted perimeter does not include the top width of the base channel when full. The areas, however, are computed in the normal fashion. In addition, length and slope are assumed equal for base and overbank channels.

The outflow from the overbank channel is then added to the outflow from the full base channel to arrive at the total outflow from the double trapezoidal channel. This serves as inflow to the next channel in the system. Thus, both base and overbank channels must flow into the same downstream channel. When the volume in the overbank channel ($V_{OVR}$) is small enough such that the following condition is met:

$$Q_I + Q_W + Q_{GW} + V_{OVR/}(\text{delta})_t < Q_{FULL}, \tag{A-15}$$

the overbank channel is emptied completely and the depth in the base channel falls below full depth. Of course, the overbank channel may flow again if the inflow into the base channel again exceeds $Q_{FULL}$.

## Constrained Channels

The final case, constrained channels (types 6 and 7), is similar to the overbank case. However, the overflow serves as inflow to any downstream channel (any type). This inflow is added to the other inflows into the downstream channel and the outflow is determined. The outflow from the constrained channel when overflowing is, of course, the full flow for that channel. Since there is no accumulated volume of overflow, the overflow is zero in any time step in which the inflow is less than full flow.

## Lakes

Runoff is routed through lakes by performing a mass balance on inflow, outflow, and storage, then using a 20-point elevation/storage/discharge curve to determine the lake outflow. Lake discharge curves may either be entered by the user, or computed based on the operation of up to seven outlet control structures. Lake outlets available include pipe/orifice/riser outlets, trapezoidal and rectangular weirs/spillways, ogee spillways, and trapezoidal channels.

Pipe/orifice outlets are simulated using the equation for a circular orifice:

$$Q_o = C_o' H_o^{1/2} \tag{A-16}$$

where

$Q_o$ = orifice discharge, cfs,
$H_o$ = depth of water above the orifice invert, ft,
$C_o'$ = $C_o A (2g)^{1/2}$,
$C_o$ = orifice coefficient,
$A$ = cross-sectional area of the orifice, $ft^2$, and
$g$ = gravitational acceleration, $ft/sec^2$.

The coefficient $C_o'$ is input by the user.

The operation of a riser depends upon whether the riser entrance is submerged or not. Prior to submergence, the riser rim functions as a weir, with the following discharge equation appropriate:

$$Q_R = C_R L_R H_R^{3/2} \tag{A-17}$$

where

$Q_R$ = riser discharge, cfs,
$C_R$ = weir discharge coefficient for the riser rim,
$L_R$ = length of the riser rim, ft, and
$H_R$ = water depth above the riser rim, ft.

Once submerged, the riser functions as an orifice whose discharge is governed according to equation A-16 above.

Weir/spillway discharge is based upon the standard weir equation:

$$Q_W = C_W \, L_W \, H_W^{3/2} \qquad \text{(A-18)}$$

where

$Q_W$ = weir discharge, cfs,
$C_W$ = weir coefficient,
$L_W$ = weir length, ft, and
$H_W$ = depth of water above weir crest, ft.

For rectangular shapes, the weir coefficient is input. The coefficient for an ogee spillway varies with $H_W$ based on an empirical curve programmed into the model.

The discharge from lakes flowing directly into channels is controlled by Manning's equation (A-9), where the depth in the channel equals the lake elevation minus the invert elevation of the channel at the lake outlet.

## A.4 SET-UP AND INPUT REQUIREMENTS FOR MSSM

### A.4.1 INTRODUCTION

Two types of data are needed for MSSM: physical data representing the prototype which will not change from simulation to simulation, and run-specific data which depends on the rainfall event to be simulated, antecedent conditions, and/or program options to be exercised. The following pages describe both types of data, how to obtain them, and typical values where appropriate.

### A.4.2 DATA PREPARATION FOR MSSM

Physical data required for MSSM describe the watershed subareas and channels represented by the model. The area, average slope, overland flow length, depression storage characteristics, retention storage volume, infiltration characteristics, and hydraulic roughness characteristics of each subarea must be defined. Typically, one channel is used to drain each subarea. However, routing channels which have no drainage area may be included in the channel network. The channel length, bottom width, average side slopes, hydraulic roughness, and depth when full must be determined.

Lakes may be used in lieu of channels. The lake must be described by its surface area, and initial volume and elevation. Either an elevation/storage/discharge curve for the lake is input, or an elevation/storage curve and outlet structure data is used to compute the lake discharge curve. If lake outlet control is a weir or spillway, the length of the weir/spillway must be defined along with a coefficient. Lakes drained with an outlet channel must specify the channel information described above (with the exception of channel length and depth when full). The purpose of this outlet channel is to provide a means of conveying outflow from the lake to the connecting downstream channel. An orifice or riser outfall may also be defined. Input data includes discharge coefficients, the riser crest length, and the riser pipe/orifice diameter.

Run specific data required for MSSM describe the rainfall event in terms of intensity (in/hr) over the duration of the storm. The starting conditions of the channels (baseflow) are specified as well as lake outlet conditions (depth of water over weir or in outlet channel).

Two options are available for MSSM. A resize option enlarges pipe diameters and channel bottom widths if necessary to pass peak flow. Another option provides the capability of modeling combined sewers.

### A.4.3 PHYSICAL DATA

In the following pages, guidelines for setting up watersheds and channels for MSSM simulation are provided.

Method of Discretization

A drainage basin may be represented as a set of watersheds drained by major streams, and further broken down into subareas drained by tributaries. USGS 7-1/2 minute quadrangles and SWFWMD aerials with contours provide sufficient topographic and land use detail for laying out the watersheds and subareas. Since the subareas in the model are representative of "average" conditions, it is advisable to select subareas with homogeneous characteristics. To the extent possible, subareas should be homogeneous with respect to:

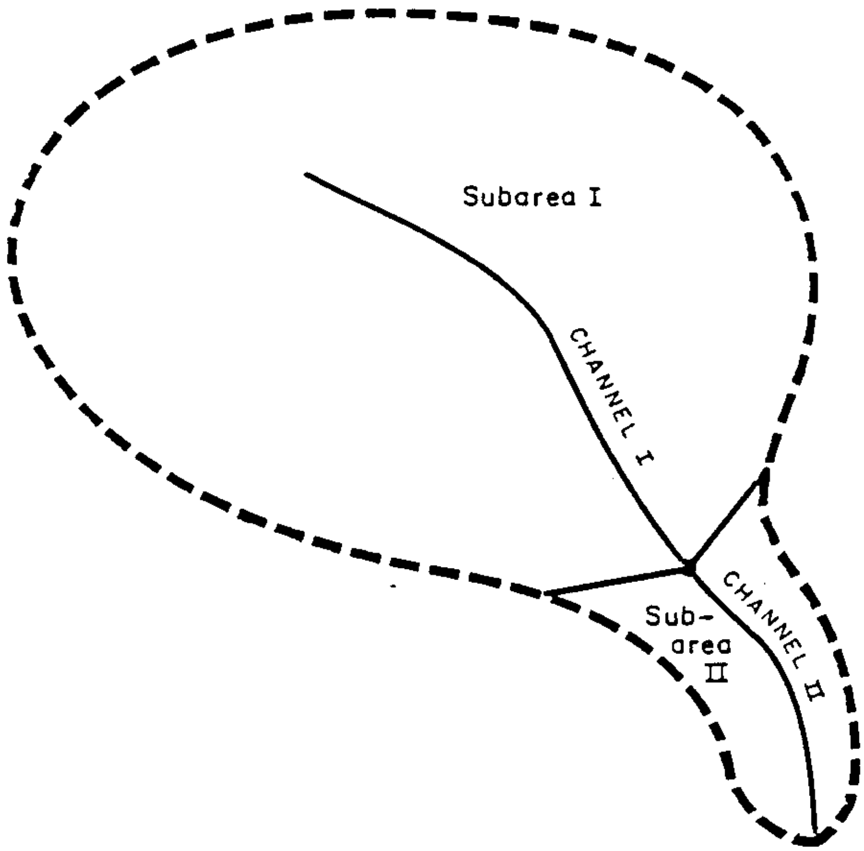
1) slopes,

2) development (type of land use), and

3) overland flow paths to the channel draining the subarea (see Figure A-5).

Subarea acreage may vary considerably depending on the level of detail desired. However, subareas should not exceed 10,000 acres. The simulated channel flow will deviate substantially from observed discharges if subareas are too large.

After the watersheds and subareas are laid out, the channel network should be defined. One channel is used to drain each subarea. Consequently, in many subareas, topographic maps must be reviewed and a decision must be made as to the primary channel. The selected channel will carry all runoff from the subarea it drains, as well as flow from upstream drainage areas. Channels may be included in the network which do not receive drainage but simply route flows to downstream channels. These "routing channels" are often useful in describing short channel reaches with characteristics which are unusual compared to the average conditions of the stream above and below the unusual section. Also, there are cases when the subarea layout is such that the entire channel length does not coincide exactly with one subarea. Small reaches of channels in these cases should be designated as routing channels rather than creating extremely small watersheds to




Watershed Broken into Two Homogeneous Subareas

FIGURE A-5

accommodate "left over" channel sections. Figure A-6 shows a subarea layout and channel network including routing channels. In subareas containing the headwaters of channels, a decision must be made as to where the channel starts and overland flow ends. An inspection of topographic maps will aid in making this decision. Some guidelines for establishing overland flow lengths are provided in the next section describing watershed characterization.

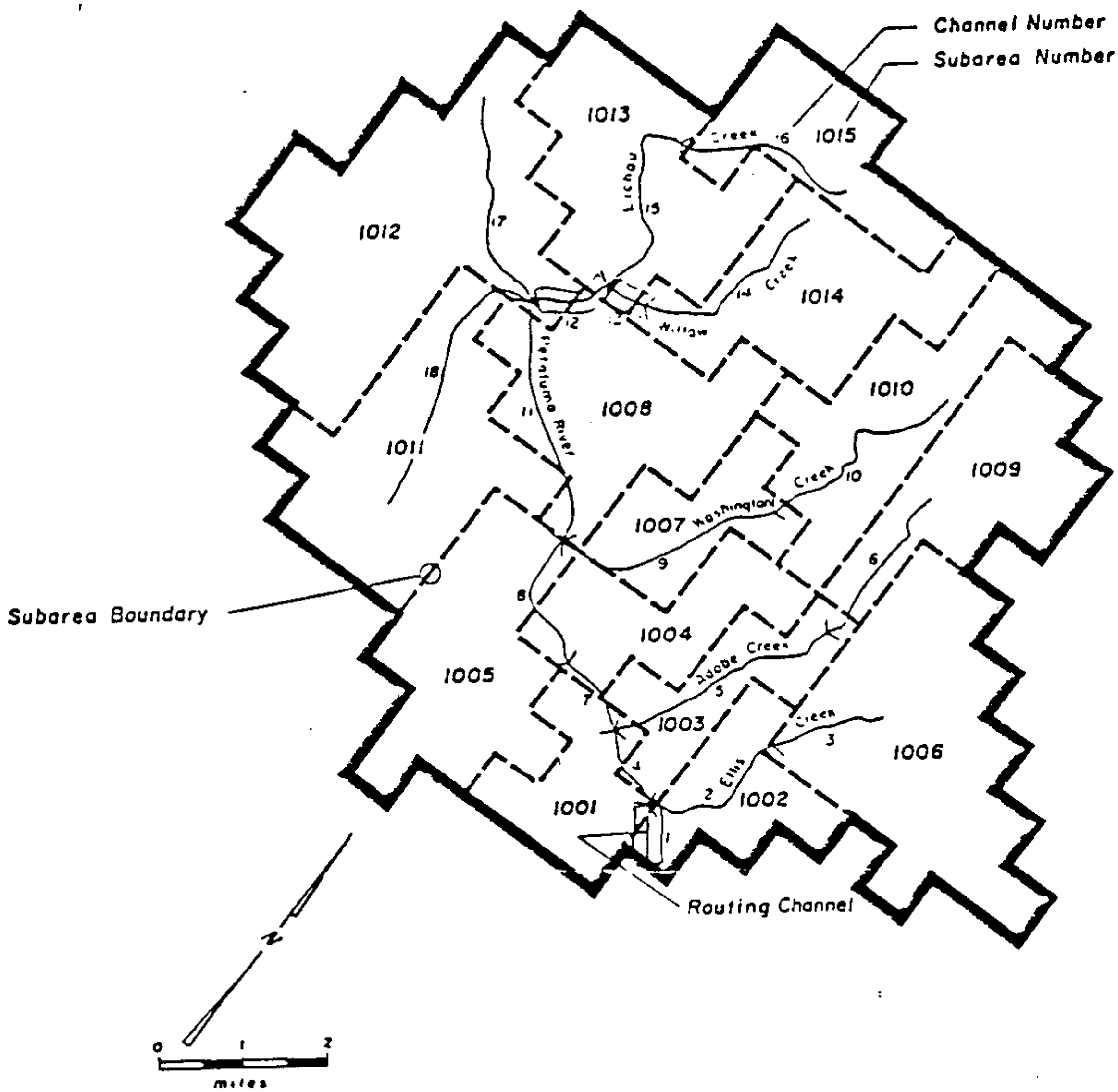
## Watershed Characterization

MSSM assumes that individual subareas within a watershed are rectangular areas having uniform conditions of slope. The area of the subarea, the overland flow length, and the average slope of the subarea should be derived from topography maps.

Determining the area of the subarea is a straightforward process once the boundary is delineated on the topography map or overlay. Determination of the overland flow length and average slope requires judgment. Alternate overland flow paths should be traced out on topographic maps where the subareas and channels are delineated. Averages of the length of alternate flow paths to the drainage channel selected for the subarea should produce a satisfactory overland flow length for the model. The average slope of the subarea may be determined by averaging the slopes of the alternate flow paths. Normally, the slope of the alternate flow path can be determined simply by taking the difference in the elevation of the head of the path and its entry point to the channel, and dividing that difference by the length of the flow path. In those subareas which contain the headwaters of channels, the subarea drainage channel should begin an equal distance to the average overland flow path from the edge of the subarea.

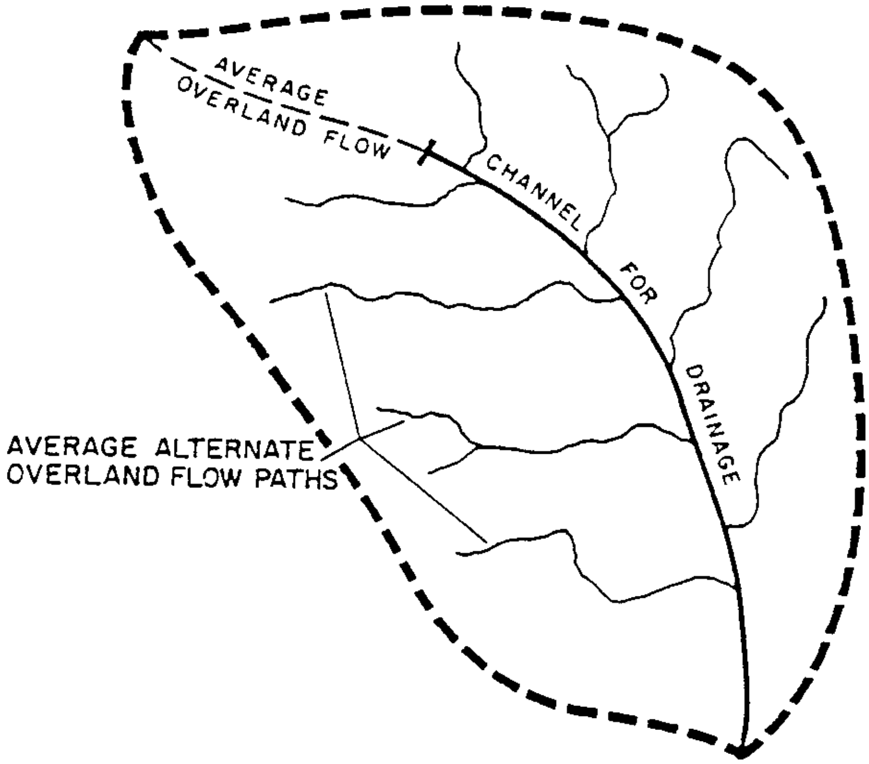
Figure A-7 illustrates how the overland flow path should be determined, and where the drainage channel should begin within the subarea insofar as the model is concerned.

Depression storage must be specified for both impervious and pervious areas in each subarea. Rainfall must be sufficient to fill the depression




Subarea Layout and Channel Network

FIGURE A-6




Overland Flow Paths

FIGURE A-7

storage available in the subarea before runoff can begin. Typical values are 0.05 inches for impervious areas, and 0.2 inches for pervious areas.

Hydraulic roughness coefficients must be specified for both the impervious and pervious areas in each subarea. Typical values are 0.013 and 0.2 for impervious and pervious areas, respectively. The range of Manning's coefficient for various overland flow surfaces is shown by Table A-2.

The percentage of directly connected impervious area (DCIA) associated with each subarea must be specified. Aerial photographs may be helpful in characterizing the percentage of impervious area associated with specific subareas. The user should keep in mind that impervious areas not connected to drainage facilities should not be included in the estimates. For example, in certain low and medium density residential developments, rooftop drains may not be hydraulically connected to street drainage facilities. In these cases, rooftops should not be included in the percentage of impervious area. Table A-3 presents some typical percentages of impervious area associated with various land uses.

Infiltration rates must be specified for the pervious areas in each subarea. Minimum and maximum rates must be specified as well as the total infiltration capacity. The user should determine the rates and total capacity from field test data if available. If data are unavailable, the following information may be used to estimate rates and maximum capacity.

Figure A-8 shows the change in infiltration rates over time as described by Horton's equation. In addition, it shows that the soil has a set maximum amount of moisture which can infiltrate before all voids in the soil are filled, and that part of the infiltration capacity of the soil may contain antecedent soil moisture. Table A-4 lists suggested infiltration parameters for each soil type, and for various antecedent soil conditions.

To account for moisture already in the soil, the initial infiltration rates, $f_o$, may be reduced, according to Horton's equation, to the value f on Figure A-8. The total maximum infiltration volume may be set to the available soil storage capacity less the antecedent soil moisture.

TABLE A-2

ESTIMATE OF MANNING'S ROUGHNESS COEFFICIENTS
Crawford and Linsley (1966)

| Groundcover | Manning's n for Overland Flow |
|---|---|
| Smooth asphalt | 0.012 |
| Asphalt for concrete paving | 0.014 |
| Packed clay | 0.030 |
| Light turf | 0.200 |
| Dense turf | 0.350 |
| Dense shubbery and forest litter | 0.400 |

TABLE A-3

PERCENTAGE OF IMPERVIOUS AREAS

| Land Use Category | Percentage of Impervious Area |
|---|---|
| Low Density Residential | 2-30 |
| Medium Density Residential | 10-65 |
| High Density Residential | 25-80 |
| Commercial | 50-95 |
| Light Industrial/Institutional | 30-90 |
| Heavy Industrial | 80-98 |
| Major Roads | 80-100 |
| Barren/Extractive | 0-5 |
| Grasslands | 0-5 |
| Woodland/Brushland | 0-5 |
| Connected Wetlands | 100 |
| Agriculture | 0-5 |

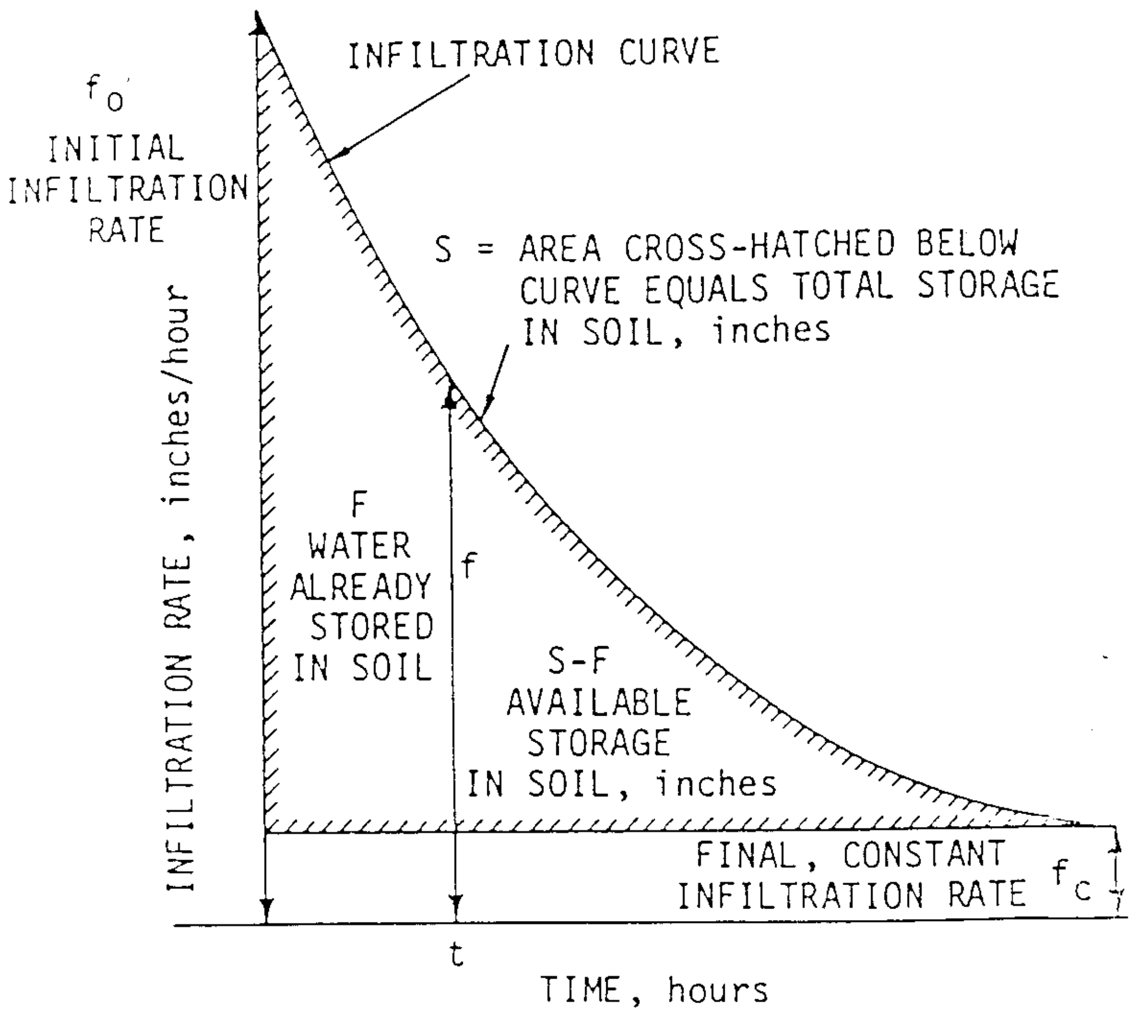



Diagram of Infiltration Curve and Infiltration Rates as Related to Storage in Soil (CDM, 1983)

**FIGURE A-8**

TABLE A-4

STANDARD INFILTRATION PARAMETERS FOR GRASSED AREAS
(CDM, 1983)

FACTORS USED FOR CALCULATING THE STANDARD
INFILTRATION CURVES FOR GRASSED AREAS

| Item | Value | | | |
|---|---|---|---|---|
| Hydrologic soil group USDA designation | A | B | C | D |
| Final constant infiltration rate, f, inches per hour | 1.0 | 0.5 | 0.25 | 0.1 |
| Initial infiltratrion rate, f, inches per hour | 10.0 | 8.0 | 5.0 | 3.0 |
| Shape factor, k, or infiltration curve | 2.0 | 2.0 | 2.0 | 2.0 |
| Available storage capacity, S, in soil mantle, | | | | |
| inches, for four antecedent conditions | | | | |
| Bone dry, Condition 1 | 4.3 | 3.4 | 2.3 | 1.3 |
| Rather dry, Condition 2 | 3.4 | 2.8 | 1.8 | 1.1 |
| Rather wet, Condition 3 | 1.9 | 1.4 | 1.0 | 0.6 |
| Saturated, Condition 4 | 0.0 | 0.0 | 0.0 | 0.0 |
| Infiltration accumulated, F, in soil mantle, | | | | |
| inches, at start of rainfall | | | | |
| Bone dry, Condition 1 | 0.0 | 0.0 | 0.0 | 0.0 |
| Rather dry, Condition 2 | 1.9 | 1.4 | 1.0 | 0.6 |
| Rather wet, Condition 3 | 3.4 | 2.8 | 1.8 | 1.1 |
| Saturated, Condition 4 | 4.3 | 3.4 | 2.3 | 1.3 |

ANTECEDENT MOISTURE CONDITIONS FOR
PERVIOUS AREAS (GRASS)

| Condition | Description | Total Rainfall During 5 Days Preceding Storm (inches) |
|---|---|---|
| 1 | Bone Dry | 0.0 |
| 2 | Rather Dry | 0.0 to 0.5 |
| 3 | Rather Wet | 0.5 to 1.0 |
| 4 | Saturated | Over 1.0 |

A further reduction in infiltration can be performed to account for impervious areas which drain onto pervious areas, e.g., roof drains onto lawns. MSSM uses an effective pervious area which lumps the not directly connected impervious area with the pervious area. Rainfall subject to infiltration falls over the entire effective previous area, but can only infiltrate into the truly pervious area. In order to limit predicted infiltration to truly pervious areas, the infiltration rates can be reduced by the ratio of not directly connected impervious area to effective pervious area.

An alternate method of describing infiltration rates is given by the standard ASCE infiltration capacity curves shown in Figure A-9.

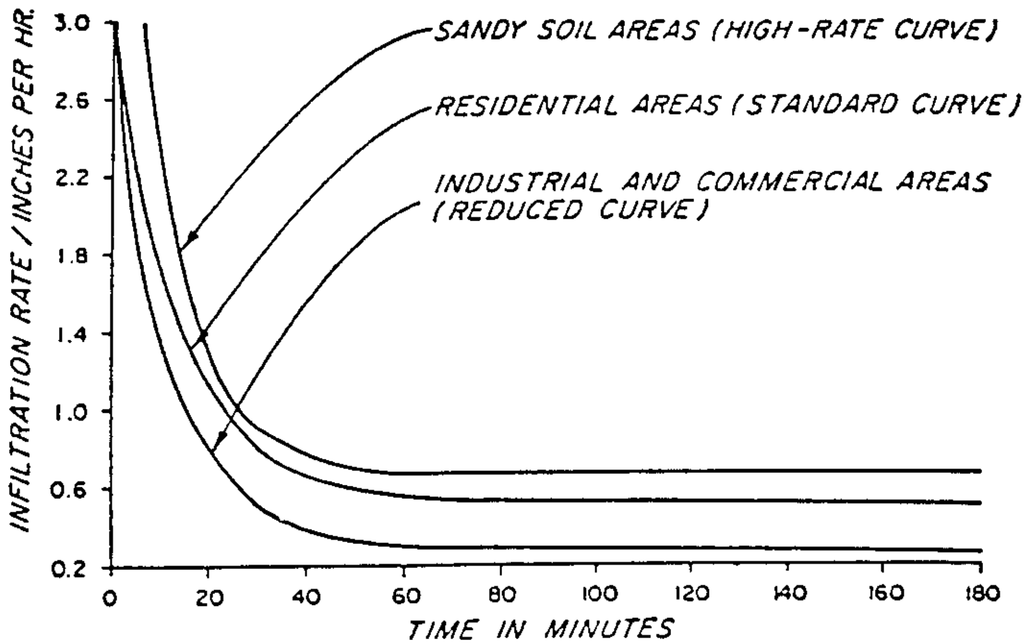
## Channel Characterization

MSSM simulates open channels and pipes flowing partly full. A lake is a special category of a channel, as described below. The length of the channel may be obtained from the topographic map and/or overlays by simply running a map wheel over the length of the channel. Bottom width and average side slopes should be derived from as-built drawings, cross-sections, or field inspection. Trapezoidal channels and double trapezoidal channels require that both bottom widths be specified, and the side slopes of both the lower and upper sections be expressed as the fraction of horizontal run to vertical rise. In the case of pipes, the diameter of the pipe is used as the bottom width of that channel. The depth of the channel when full must be specified. Figure A-10 shows the required channel dimensions.

The hydraulic roughness of the channel must be specified. Typical values for channels with various characteristics are given in Table A-5.

## Lake Characterization

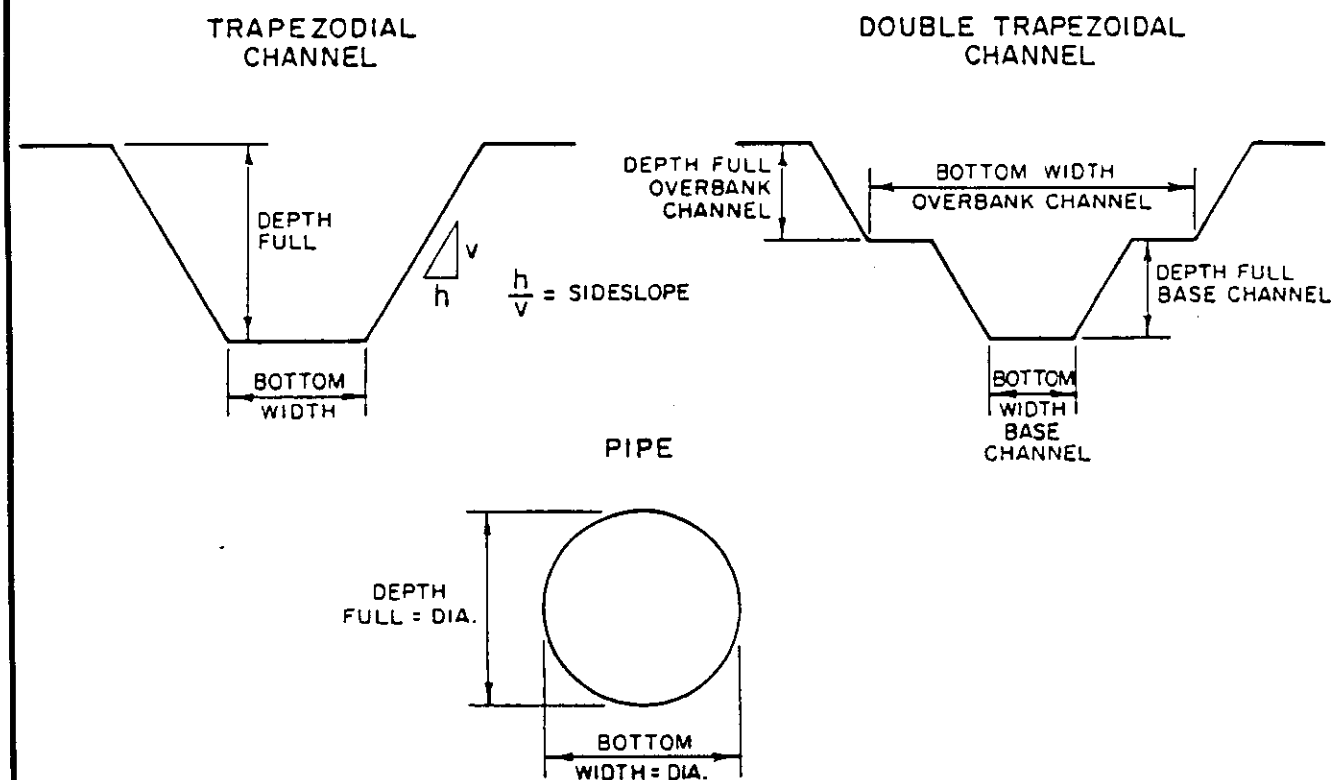
Lakes are treated internally within MSSM as channels. Modifications to the lake routing algorithm have resulted in the need for many additional lake parameters. The following discussion presents the data required.




Source: American Society of Civil Engineers (1960)

Standard Infiltration - Capacity Curves
for Pervious Surfaces

FIGURE A-9




Channel Dimensions

FIGURE A-10

## TABLE A-5

## CHANNEL ROUGHNESS COEFFICIENTS

## Brater and King (1976)

| Surface | Best | Good | Fair | Bad |
|---|---|---|---|---|
| Uncoated cast-iron pipe | 0.012 | 0.013 | 0.014 | 0.015 |
| Coated cast-iron pipe | 0.011 | 0.012* | 0.013* | |
| Commercial wrought-iron pipe, black | 0.012 | 0.013 | 0.014 | 0.015 |
| Commercial wrought-iron pipe, galvanized | 0.013 | 0.014 | 0.015 | 0.017 |
| Smooth brass and glass pipe | 0.009 | 0.010 | 0.011 | 0.013 |
| Smooth lockbar and welded "OD" pipe | 0.010 | 0.011* | 0.013* | |
| Riveted and spiral steel pipe | 0.013 | 0.015* | 0.017* | |
| Vitrified sewer pipe | 0.010<br>0.011 | 0.013* | 0.015 | 0.017 |
| Common clay drainage tile | 0.011 | 0.012* | 0.014* | 0.017 |
| Glazed brickwork | 0.011 | 0.012 | 0.013* | 0.015 |
| Brick in cement mortar; brick sewers | 0.012 | 0.013 | 0.015* | 0.017 |
| Neat cement surfaces | 0.010 | 0.011 | 0.012 | 0.013 |
| Cement mortar surfaces | 0.011 | 0.012 | 0.013* | 0.015 |
| Concrete pipe | 0.012 | 0.013 | 0.015* | 0.016 |
| Wood stave pipe | 0.010 | 0.011 | 0.012 | 0.013 |
| Plank Flumes: | | | | |
| Planed | 0.010 | 0.012* | 0.013 | 0.014 |
| Unplaned | 0.011 | 0.013* | 0.014 | 0.015 |
| With battens | 0.012 | 0.015* | 0.016 | |
| Concrete-lined channels | 0.012 | 0.014* | 0.016* | 0.018 |
| Cement-rubble surface | 0.017 | 0.020 | 0.025 | 0.030 |
| Dry-rubble surface | 0.025 | 0.030 | 0.033 | 0.035 |
| Dressed-ashlar surface | 0.013 | 0.014 | 0.015 | 0.017 |
| Semicircular metal flumes, smooth | 0.011 | 0.012 | 0.013 | 0.015 |
| Semicircular metal flumes, corrugated | 0.0225 | 0.025 | 0.0275 | 0.030 |
| Canals and Ditches: | | | | |
| Earth, straight and uniform | 0.017 | 0.020 | 0.0225* | 0.025 |
| Rock cuts, smooth and uniform | 0.025 | 0.030 | 0.033* | 0.035 |
| Rock cuts, jagged and irregular | 0.035 | 0.040 | 0.045 | |
| Winding sluggish canals | 0.0225 | 0.025* | 0.0275 | 0.030 |
| Dredged earth channels | 0.025 | 0.0275* | 0.030 | 0.033 |
| Canals with rough stony beds, weeds on earth banks | 0.025 | 0.030 | 0.035* | 0.040 |
| Earth bottom, rubble sides | 0.028 | 0.030* | 0.033* | 0.035 |
| Natural Stream Channels: | | | | |
| (1) Clean, straight bank, full stage, no rifts or deep pools | 0.025 | 0.0275 | 0.030 | 0.033 |
| (2) Same as (1), but some weeds and stones | 0.030 | 0.033 | 0.035 | 0.040 |
| (3) Winding, some pools and shoals, clean | 0.033 | 0.035 | 0.040 | 0.045 |
| (4) Same as (3), lower stages, more ineffective slope and sections | 0.040 | 0.045 | 0.050 | 0.055 |
| (5) Same as (3), some weeds and stones | 0.035 | 0.040 | 0.045 | 0.050 |
| (6) Same as (4), stony sections | 0.045 | 0.050 | 0.055 | 0.060 |
| (7) Sluggish river reaches, rather weedy or with very deep pools | 0.050 | 0.060 | 0.070 | 0.080 |
| (8) Very weedy reaches | 0.075 | 0.100 | 0.125 | 0.150 |

* Values commonly used in designing.

First, basic lake parameters are entered. These include the lake surface area, average length from inlet to outlet, attributable rain gauge, channel/lake receiving outflow, and initial elevation/storage. The lake name/description is then entered.

Following this, parameters governing the construction of the elevation/storage/discharge curve are requested. These parameters are the lowest (STE) and highest (ELIMIT) elevations on the routing curve, and the top elevation of the usable storage pool (TSTOR). Also entered are the diameter of any outlet pipe, orifice or riser (DIA), and the invert elevation of this structure (ZHE).

Entry of discharge coefficients for various types of outlet structures follows. The lake is permitted to have up to seven of these structures, providing the lake has no more than one orifice/pipe/riser, one channel, two rectangular weirs, two trapezoidal weirs, two trapezoidal weirs, and two ogee spillways. If more than one outlet of the same type is present, the one with the lower elevation is denoted "1" and the other "2." The crest elevation, type, and dimensions of each lake outlet structure is entered.

The final lake input data is a 20 point or less elevation/storage/discharge relationship. Usually only elevation/storage points need to be entered. In this case, discharge values for each particular elevation on the routing curve are computed based upon the operation of the outlet control structures. Discharges at each elevation may be entered if the lake operation cannot be described with the available set of outlet structures. If any discharge values are entered, RUNOFF assumes that the user has entered his own elevation/discharge curve and will bypass the computation of a discharge curve based on the outlet operation.

### A.4.4 RUN-SPECIFIC DATA FOR MSSM

Certain input data required for MSSM is specific to the simulation to be made. The simulation beginning and ending times and rainfall depend on the storm event to be modeled. Starting conditions in channels and lakes must be specified. Two options are available to:

1) retain runoff from urban areas, and
2) simulate combined sewers which overflow.

Simulation Time and Integration Time Step

The beginning hour and ending hour of the event to be modeled must be specified. The length of runoff simulation should exceed the rainfall period in a sufficient amount of time to allow complete routing of surface runoff to the watershed outlet. The time interval (or step) to be used for numerical integration must be selected and should be within the range of 10 to 30 minutes. Watersheds with great detail require a small time step.

Hyetographs

A hyetograph must be assigned to each subarea. For each, equal increments of rainfall intensity in inches/hour must be specified. Even though there may not be rainfall, the hyetographs must cover the entire duration of the simulation period (which should extend beyond the rainfall period). Zero intensities should be assigned to each interval where no rainfall occurred. If, for example, a storm is to be simulated which started at 10 a.m. and stopped at 1 p.m., the simulation period desired may extend from 9 a.m. to 3 p.m. Assuming a 30 minute time step, the total time steps would be 12. For example, if hourly rainfall data was collected, the RUNOFF hyetograph would be constructed as follows:

| 9 a.m. | 10 | 11 | 12 | 1 p.m. | 2 | 3 |
|---|---|---|---|---|---|---|
| 0.0 | 0.1 | 0.15 | 0.25 | 0.1 | 0.0 | 0.0 |

Baseflows

The user may elect to specify channel flows in the system prior to storm runoff. Baseflow in cfs may be specified for any or all channels and lakes. A balance is performed on the system using the baseflows specified by the user. Any imbalances are adjusted automatically assuming groundwater inflow to make up a shortage, or channel seepage to account for excess flow. If baseflows are specified, they will override the initial depth that may be specified for lakes.

Initial Lake Elevations

Starting conditions for lakes are given as the elevation of water at the weir or outlet channel. This initial elevation may be above or below the outlet elevation. As mentioned previously, the initial flows may be specified by the user in terms of baseflows, in which case the initial depth of lakes is computed and the input value, if any, is ignored.

### A.4.5 RUNOFF OPTIONS

Retention Storage Option

MSSM has the option of storing surface runoff beginning with initial runoff. The user may specify the volume of runoff to be stored for each subarea from the beginning of the storm. No runoff from the subarea will occur until the subarea's retention storage volume is filled. This can be of great use when modeling future conditions, which includes the FDER requirement of capturing the first inch of rainfall.

Combined Sewers Option

MSSM has the capability to simulate overflows from combined sewers. The channel that overflows, or constrained channel, is specified as a type 6 (trapezoidal) or type 7 (pipe) channel. Any channel in the network may be specified as the receiving channel for the overflows, provided that it is downstream from the overflowing channel. (In certain cases, the order in which the channel cards are read may influence the upstream-downstream ordering, so care should be exercised to ensure that the receiving channel is indeed downstream from the constrained channel. The connectivity summary which is printed lists the channels in upstream-downstream order and should help the user in complex cases.)

The overflows from the constrained channel are simply the surcharge flows from these channels, and are entered as inflows to the receiving channels.

## A.5 HEC-2 SURFACE WATER PROFILES

The HEC-2 model was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center in 1976, and is intended to be used to calculate water surface profiles for steady, gradually varied flows in either natural or man-made channels. The model solutions are based solving of one-dimensional energy equation by using the Standard Step method. Energy losses due to friction are evaluated using Manning's Equation. The model has the capability to account for the type of obstructions that are typically found along the stream's reach including: bridges, weirs, and culverts. The model has the ability to simulate both super- and sub-critical flow profiles.

The model requires as input: the flow regime, starting elevations, discharge loss coefficients, detailed cross-section geometry, and reach lengths. Output of the models consists of both tabular and graphic representation of the depths and extent of in-channel and overbank flows.

The flow regime is input to the model by specifying the flow through each cross-section. For the modeling of the Sarasota County basins, these flows have been determined by the MSSM model. Since each reach will ultimately have to be of sufficient size to carry the peak flows that are expected, no accounting of peak timing will be performed. That is, the peak flow expected for the reach, regardless of its timing, has been used as input to the model.

The starting elevation for the HEC-2 simulation will be mean-high-tide at its outflow point. For the purpose of these simulations, the outflow point is considered to be a point where tidal affects begin. In the case of Phillippi and Alligator Creeks, this point is centered in Roberts and Lemon Bays, respectively.

After the volumes of the observed and calculated hydrographs are in agreement, the two hydrographs are checked for timing and general shape. If the study area has been properly defined, the shapes and timing are usually close and any final adjustments in timing are achieved by varying the depression storage for the impervious area (e.g., increasing the depression storage delays at the start of the hydrograph and vice versa).

When the impervious portion of the study area has been calibrated, the next step in calibration is to execute the model on observed storms that were large enough to produce runoff from the pervious portions of the study area. Again, the objectives are to first match the volume of runoff, then match the timing and shape of the observed and calculated hydrographs. The calibration parameter for the impervious areas are held constant based on previous analyses, and the model is run repeatedly varying the maximum and minimum infiltration rates, the maximum infiltration volume, and the pervious area's depression storage. Adjustments to the infiltration rates increase or decrease flows over the period of runoff. Similarly, adjustments to the maximum total infiltration volume shift the timing of the runoff or alter the recession limb of the hydrograph. When the observed and calculated hydrographs for a given set of calibration parameters agree for each storm, the model is considered calibrated and ready for further design storm runs.

### A.6.2 CALIBRATION DATA

As discussed previously, the general calibration procedure involves the comparison of the modeled stream flow with historic stream flow data. For Phillippi Creek, the gaging station is located on the downstream side of the Cattlemen Road (S.R. 785) bridge, as shown in Figure A-11. The areas drained by the creek at this point are predominantly undeveloped. The primary major developments within this drainage area are Sarasota Lakes and Bent Tree Village. The drainage basins discharging through this gaged reach of Phillippi Creek are numbers 317 through 320, and are shown in Figure 6-2. The rainfall used for the calibration effort was supplied by the Sarasota County Pollution Control Division (Cattlemen Road) and SWFWMD

Loss coefficients for the various constrictions within the channel have been determined by using the recommended values in the HEC-2 Users Guide. Historical verification of these values supports their use.

Cross-sectional geometry is determined using field surveys and existing data. Alligator and Phillippi Creeks have been surveyed as part of this project. Cross-sections have been taken at intervals of a maximum of 2,000 feet. Additionally, creek cross-sections have been taken at each major flow control structure, diversion structure, or significant change in direction. Cross-sectional data for the remaining 14 basins within the study area will be taken from existing data.

For a more detailed explanation of the input and output of the HEC-2 model, consult the HEC-2 Water Surface Profiles Programmers and Users Manuals.

## A.6 MODEL CALIBRATION

Calibration of a stormwater model is accomplished by comparing known flows within a stream or reach to model derived flows for a historical event. Generally, each basin or subbasin is calibrated using several storms of varying intensity and duration and various antecedent conditions. As stated previously, no flow data is available for Alligator Creek and only minimal flow data of a timely nature is available for Phillippi Creek. The available Phillippi Creek data will be used in the calibration effort. A discussion of the procedure for calibration and the data limitations follow.

### A.6.1 GENERAL CALIBRATION PROCEDURE

The first step in the calibration procedure is to estimate values for the calibration parameters, and run the model on one or more observed lower volume storms that produce the majority of the runoff solely from the impervious portions of the study area. The volumes of the calculated and observed hydrograph(s) are compared and, if they do not agree, successive runs are made varying the percentage of imperviousness values until the predicted and observed storm volumes agree.

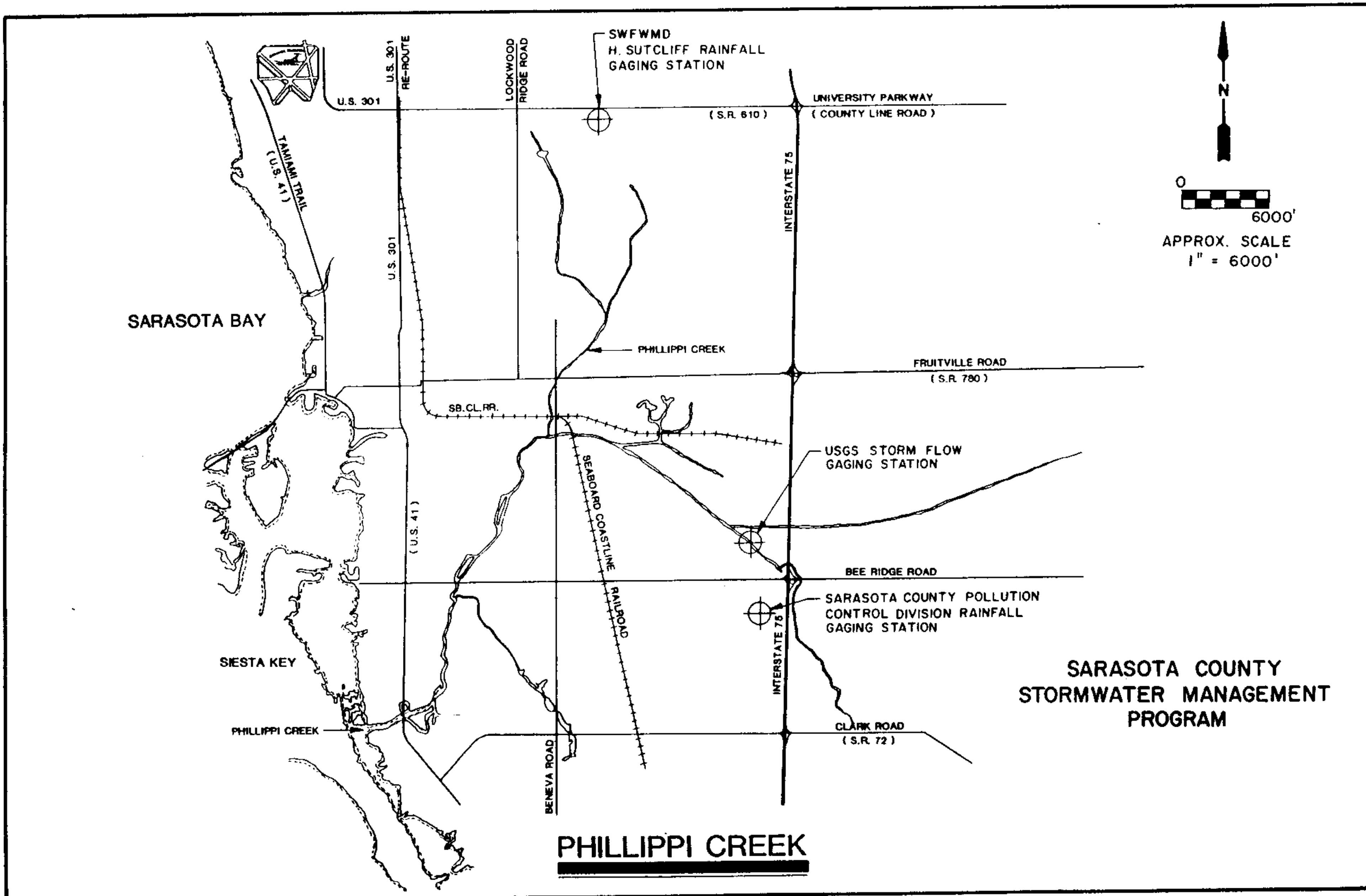



FIGURE A-11

Phillippi Creek Gaging Station Location Map

FIGURE A-11

(H. Sutcliff, Oceanic and Atmospheric Administration). The rainfall at these two stations is recorded as daily total rainfall volume. The volume of rainfall from the H. Sutcliff and Cattlemen Road stations are compared and the average total volume of rainfall obtained. The location of these two gages is given in Figure A-11.

Of concern in this modeling effort is not only the volume of rainfall, but the distribution and timing. Unfortunately, there are no rainfall stations within the basin which record hourly rainfall values from which a distribution can be obtained. For the purposes of this study, the hourly rainfall distributions for Venice, Palmetto, and Tampa were compared and an average regional distribution was obtained.

The previously defined total rainfall volume is distributed using the average distribution, and used as input to the model. Model parameters are then adjusted until the general shape of the outflow is similar to the recorded flow shape.

A.6.3 STORM SELECTION AND STREAM FLOW

For the purposes of calibration, storms of varying rainfall volumes and season of occurrence are chosen. It is important that storms from both the wet and dry seasons be used for calibration in that the infiltration parameters between them will vary substantially.

The hourly stream flow records were obtained from the USGS in Tampa with the assistance of Mr. Roman T. Mycyk. The gages record the stage as measured by a staff gage. This stage is then converted to flow using a stream flow rating curve supplied by USGS. A comparison of the shape of the rainfall hyetograph and discharge hydrographs shows a general agreement. This indicates that the aggregate distribution is acceptable.

The small displacement storms (those occurring on 12/6/79, 12/16/79, 1/26/80, and 2/10/80) are used to calibrate the model for percentage of imperviousness, depression storage, and Mannings "n." Initial computer simulations with these small storms showed a higher peak discharge but a smaller volume. Adjustments of Manning "n," percentage of imperviousness, and depression storage resulted in the predicted hydrographs shown in Figures A-12 through A-15. These simulations result in selection of the following calibration parameters:

| | | |
|---|---|---|
| Depression Storage | Pervious | 0.1 |
| | Impervious | 0.1 |
| Manning "n" | Overland | 0.17 |
| | | 0.32 |
| | Main Channel | 0.03 - 0.10 |

These volumes are within the expected range and thus are expected to accurately simulate existing conditions. Mannings "n" for overland flow are thought to be consistent throughout these subbasins due to the similarity in land use and ground cover.

To calibrate the infiltration parameters, it is necessary to simulate the basin runoff for much larger storms. The storms selected are 4/7/80 and 2/7/81. The maximum and minimum rates of infiltration, as well as the total volume of infiltration, are adjusted until the shape and timing of the predicted hydrographs approach the historical hydrographs. The selected parameters for the historic antecedent conditions compare to those conditions exhibited by soils under the Soil Conservation Services Antecedent Moisture Condition II (S.C.S. AMC-II).

The rainfall, historic flows, and calibrated flows for the two larger storms are shown in Figures A-16 and A-17. Table A-6 lists the infiltration parameters selected for use in this modeling effort. It should be noted that only those parameters for soil groups C and D were obtained from the Phillippi Creek calibration procedure. The values for soil classes A and B, not in the calibration area, were obtained by extrapolation of the C and D infiltration parameters.

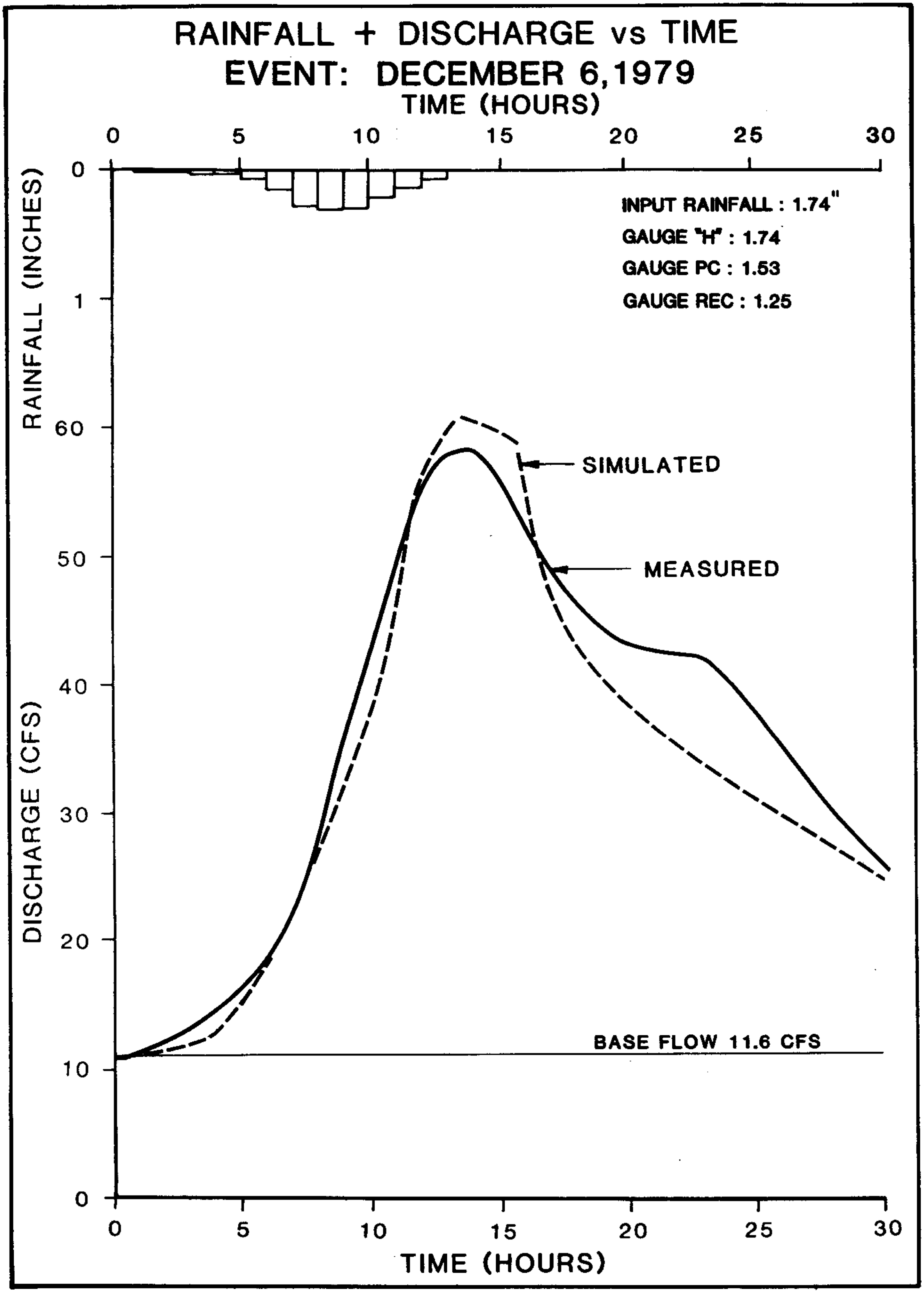

RAINFALL + DISCHARGE vs TIME
EVENT: DECEMBER 6,1979
TIME (HOURS)
RAINFALL (INCHES)
INPUT RAINFALL : 1.74"
GAUGE "H" : 1.74
GAUGE PC : 1.53
GAUGE REC : 1.25
SIMULATED
MEASURED
BASE FLOW 11.6 CFS
DISCHARGE (CFS)
TIME (HOURS)

FIGURE A-12

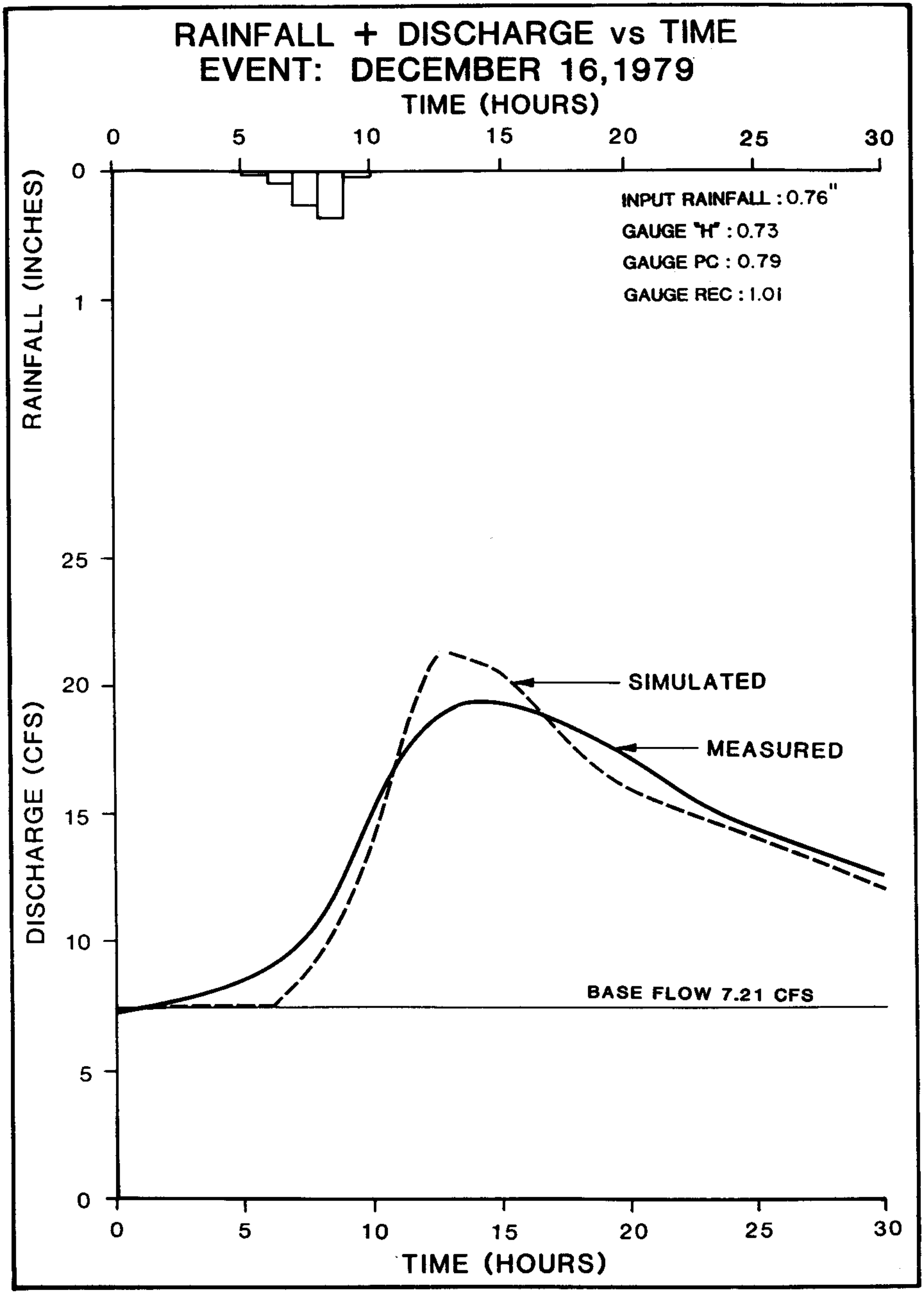

RAINFALL + DISCHARGE vs TIME
EVENT: DECEMBER 16,1979
TIME (HOURS)
0
5
10
15
20
25
30
RAINFALL (INCHES)
0
1
INPUT RAINFALL : 0.76"
GAUGE "H" : 0.73
GAUGE PC : 0.79
GAUGE REC : 1.01
DISCHARGE (CFS)
25
20
15
10
5
0
SIMULATED
MEASURED
BASE FLOW 7.21 CFS
TIME (HOURS)

FIGURE A-13

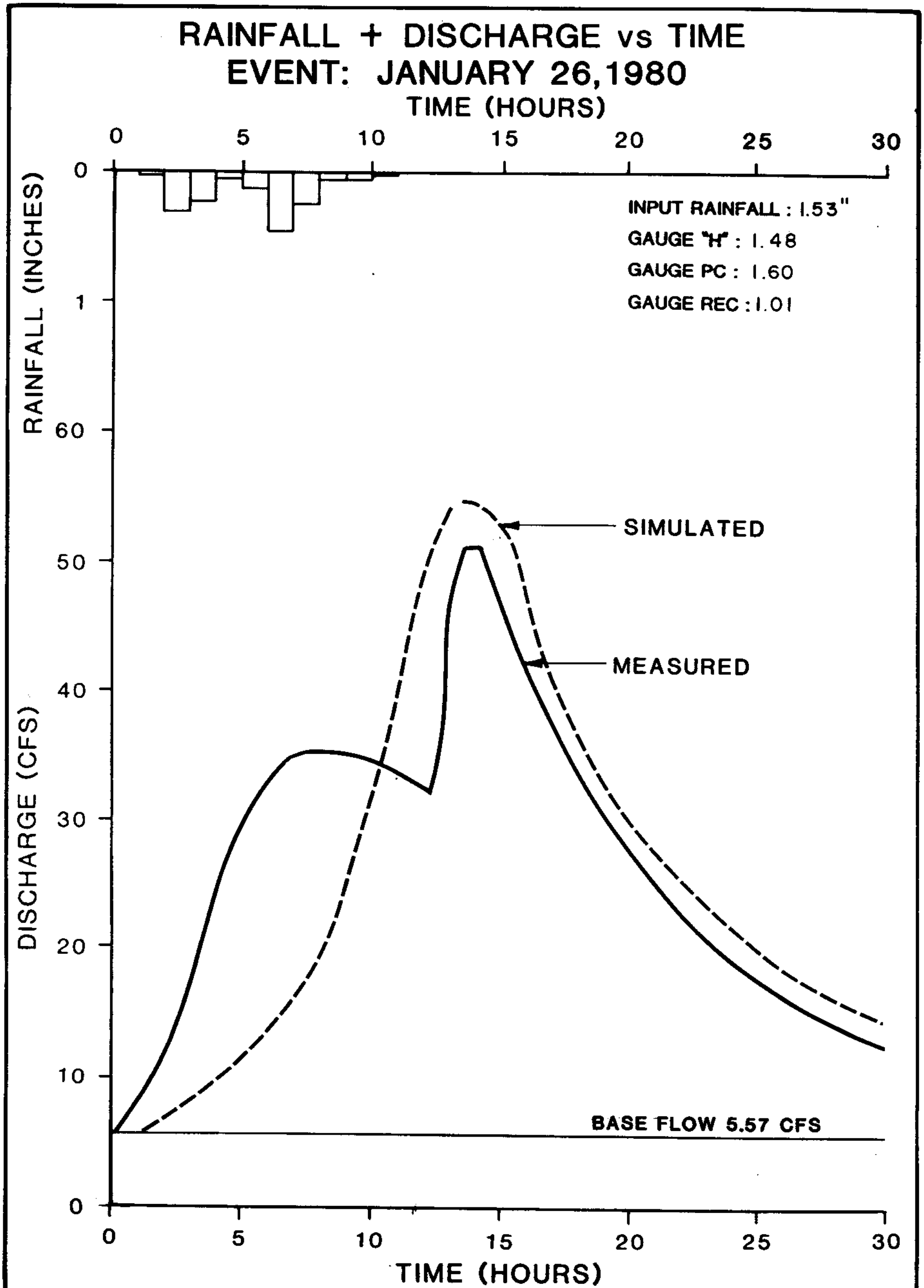

RAINFALL + DISCHARGE vs TIME
EVENT: JANUARY 26,1980
TIME (HOURS)
0
5
10
15
20
25
30
RAINFALL (INCHES)
0
1
INPUT RAINFALL : 1.53"
GAUGE "H" : 1.48
GAUGE PC : 1.60
GAUGE REC : 1.01
DISCHARGE (CFS)
60
50
40
30
20
10
0
SIMULATED
MEASURED
BASE FLOW 5.57 CFS
TIME (HOURS)

FIGURE A-14

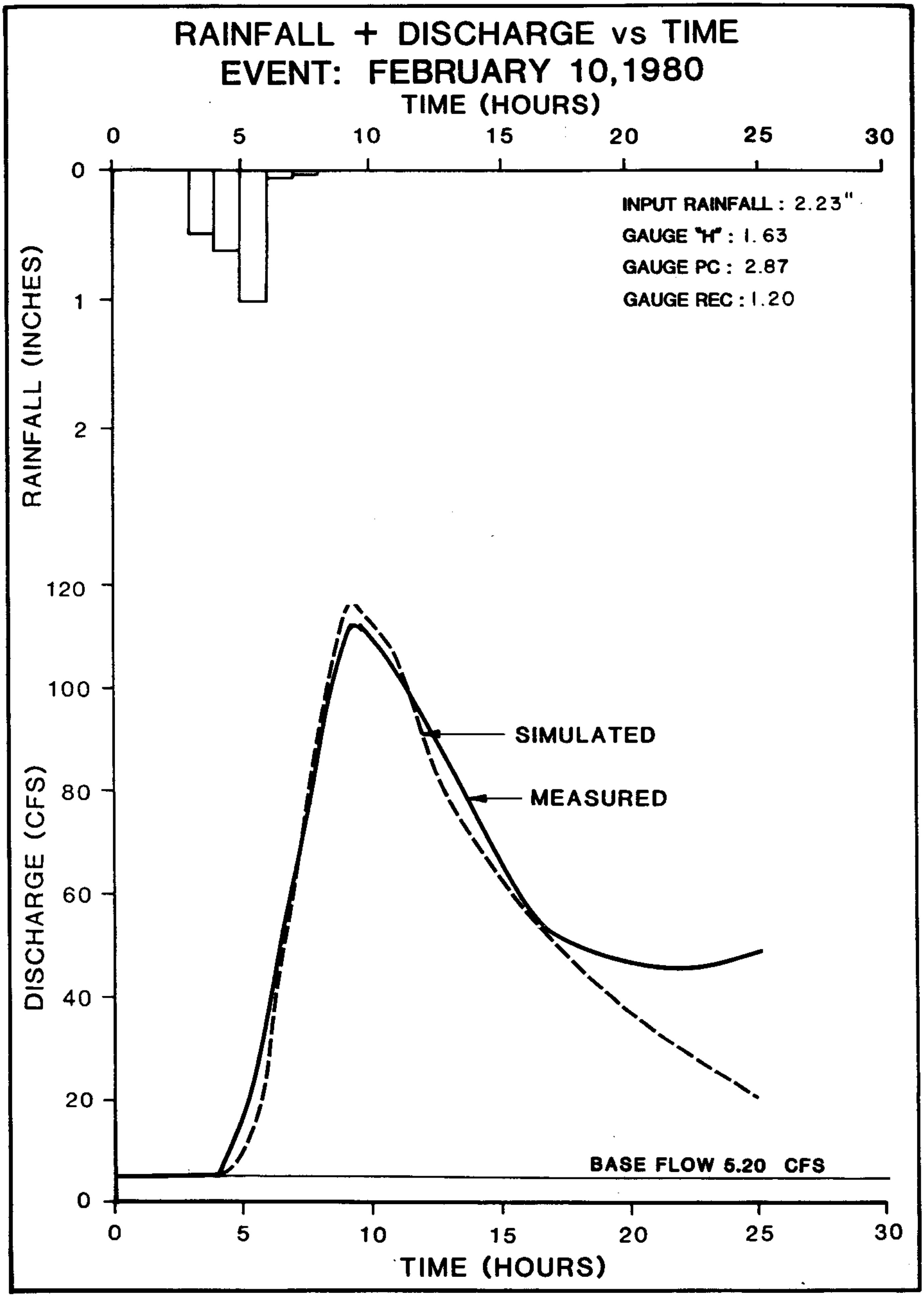

RAINFALL + DISCHARGE vs TIME
EVENT: FEBRUARY 10,1980
TIME (HOURS)
INPUT RAINFALL: 2.23"
GAUGE "H": 1.63
GAUGE PC: 2.87
GAUGE REC: 1.20
RAINFALL (INCHES)
DISCHARGE (CFS)
SIMULATED
MEASURED
BASE FLOW 5.20 CFS
TIME (HOURS)

FIGURE A-15

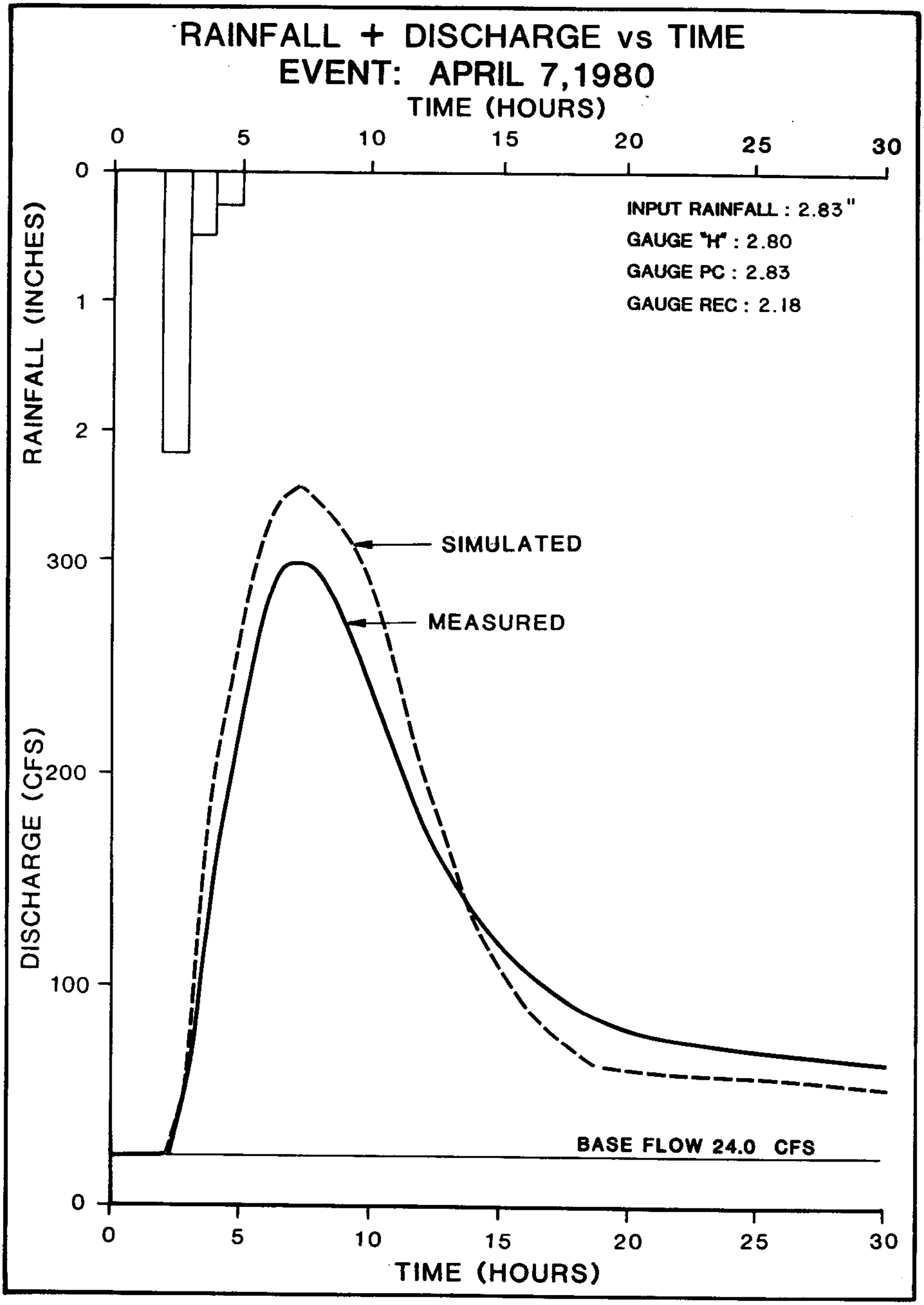

RAINFALL + DISCHARGE vs TIME
EVENT: APRIL 7,1980
TIME (HOURS)
0
5
10
15
20
25
30
RAINFALL (INCHES)
0
1
2
INPUT RAINFALL : 2.83"
GAUGE "H" : 2.80
GAUGE PC : 2.83
GAUGE REC : 2.18
SIMULATED
MEASURED
DISCHARGE (CFS)
300
200
100
0
BASE FLOW 24.0 CFS
TIME (HOURS)

FIGURE A-16

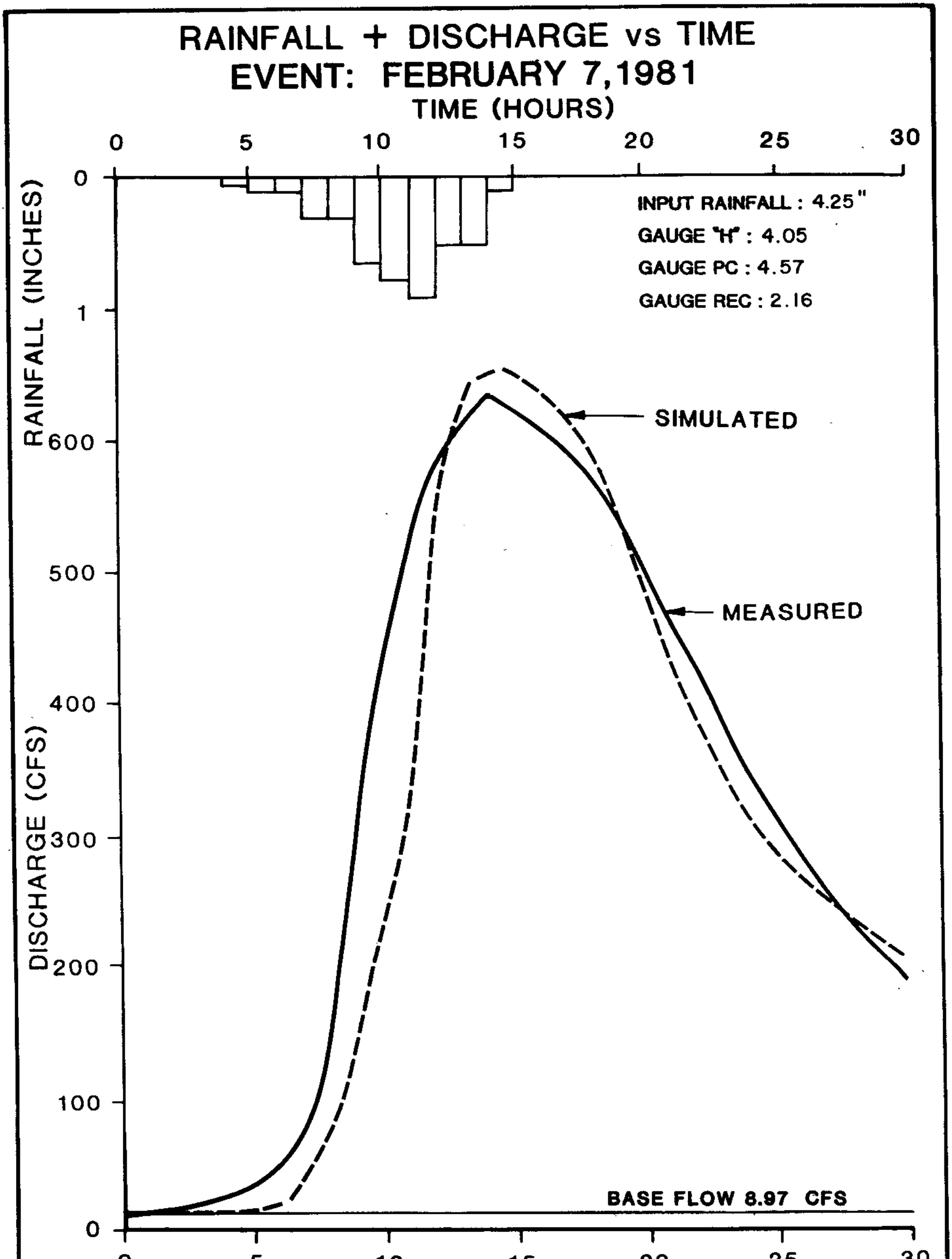

RAINFALL + DISCHARGE vs TIME
EVENT: FEBRUARY 7,1981
TIME (HOURS)
0
5
10
15
20
25
30
RAINFALL (INCHES)
0
1
INPUT RAINFALL : 4.25"
GAUGE "H" : 4.05
GAUGE PC : 4.57
GAUGE REC : 2.16
SIMULATED
MEASURED
DISCHARGE (CFS)
600
500
400
300
200
100
0
BASE FLOW 8.97 CFS
0
5
10
15
20
25
30
TIME (HOURS)

FIGURE A-17

# TABLE A-6

## SELECTED INFILTRATION PARAMETERS

### AVERAGE DRY CONDITIONS

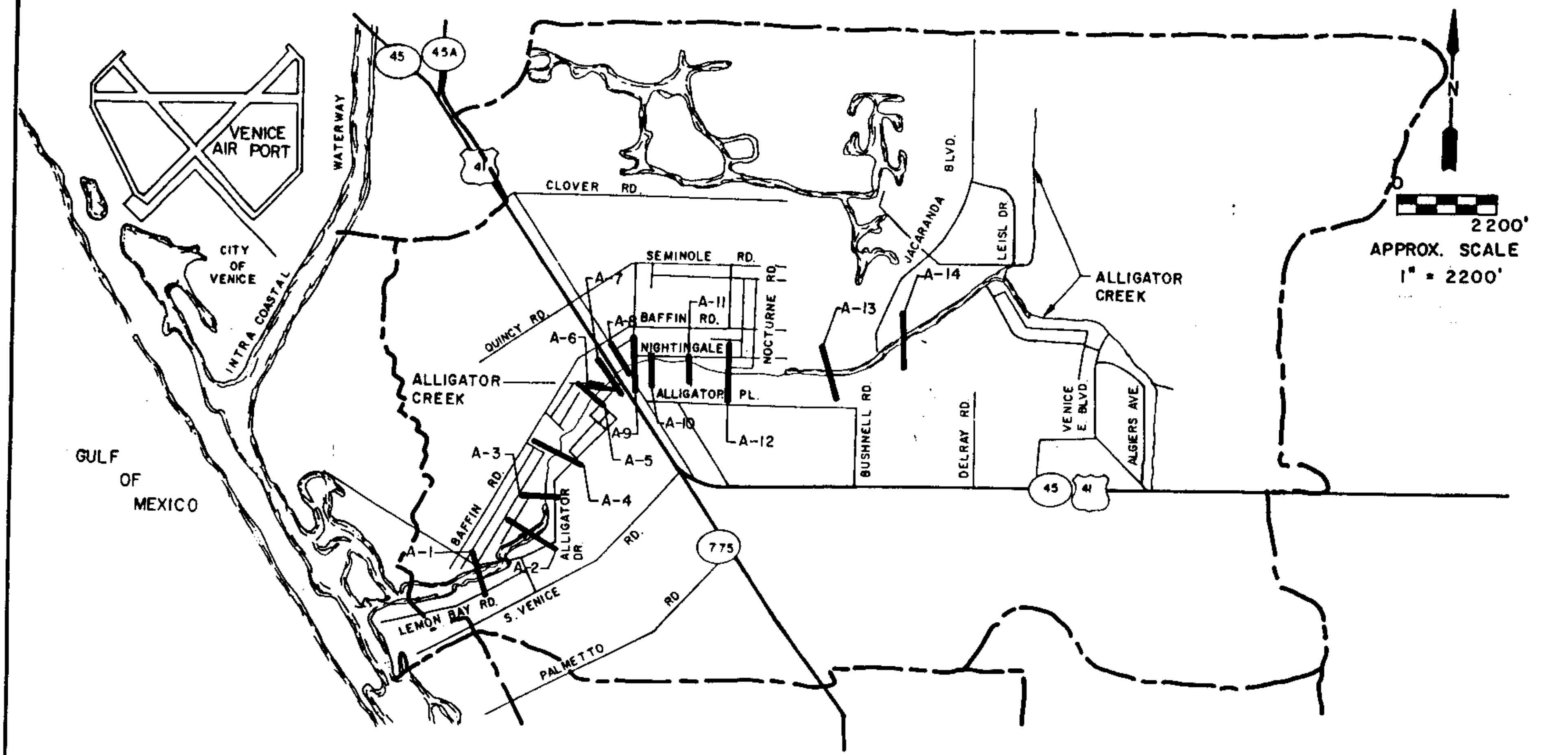
| Soil Classification | Maximum Rate (in/hr) | Minimum Rate (in/hr) | Total | Decay Rate |
|---|---|---|---|---|
| A | 3.9 - 4.1 | 0.96 | 3.8 | 2 |
| B | 3.1 - 3.3 | 0.48 | 2.0 | 2 |
| C | 2.0 - 2.1 | 0.23 | 2.2 | 2 |
| D | 1.10 | 0.14 | 1.6 | 2 |

### AVERAGE WET CONDITIONS

| Soil Classification | Maximum Rate (in/hr) | Minimum Rate (in/hr) | Total | Decay Rate |
|---|---|---|---|---|
| A | 2.7 - 2.9 | 0.96 | 1.7 | 2 |
| B | 1.9 - 2.1 | 0.48 | 1.3 | 2 |
| C | 1.2 - 1.35 | 0.23 | 1.0 | 2 |
| D | 0.6 - 0.75 | 0.14 | 0.7 | 2 |

# APPENDIX B

## ALLIGATOR CREEK
## CROSS-SECTIONS AND STRUCTURES




**LEGEND**

A-1 CROSS SECTION REFERENCE NUMBER

## CROSS SECTIONS LOCATION MAP

## ALLIGATOR CREEK

FIGURE B-1

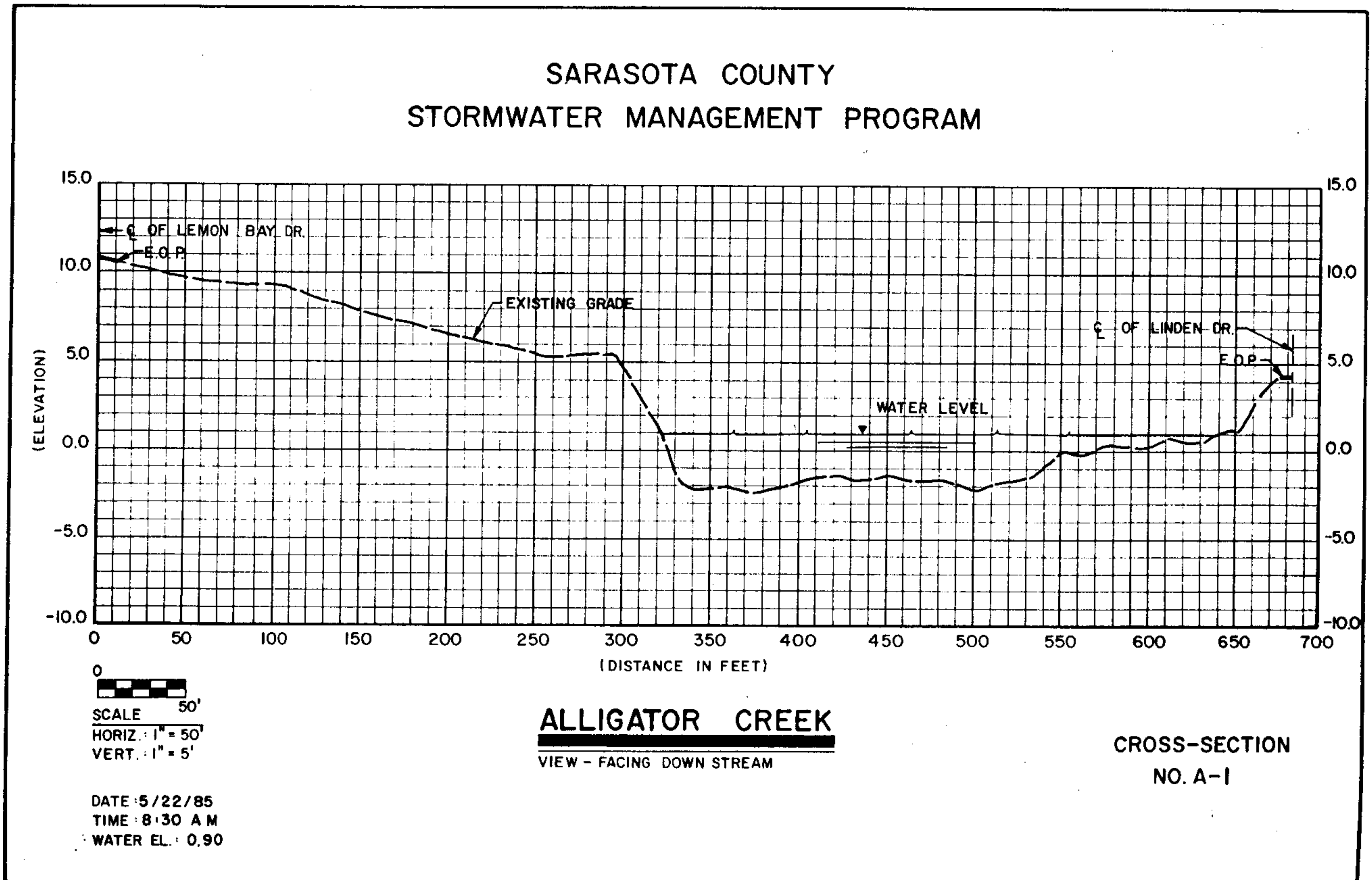
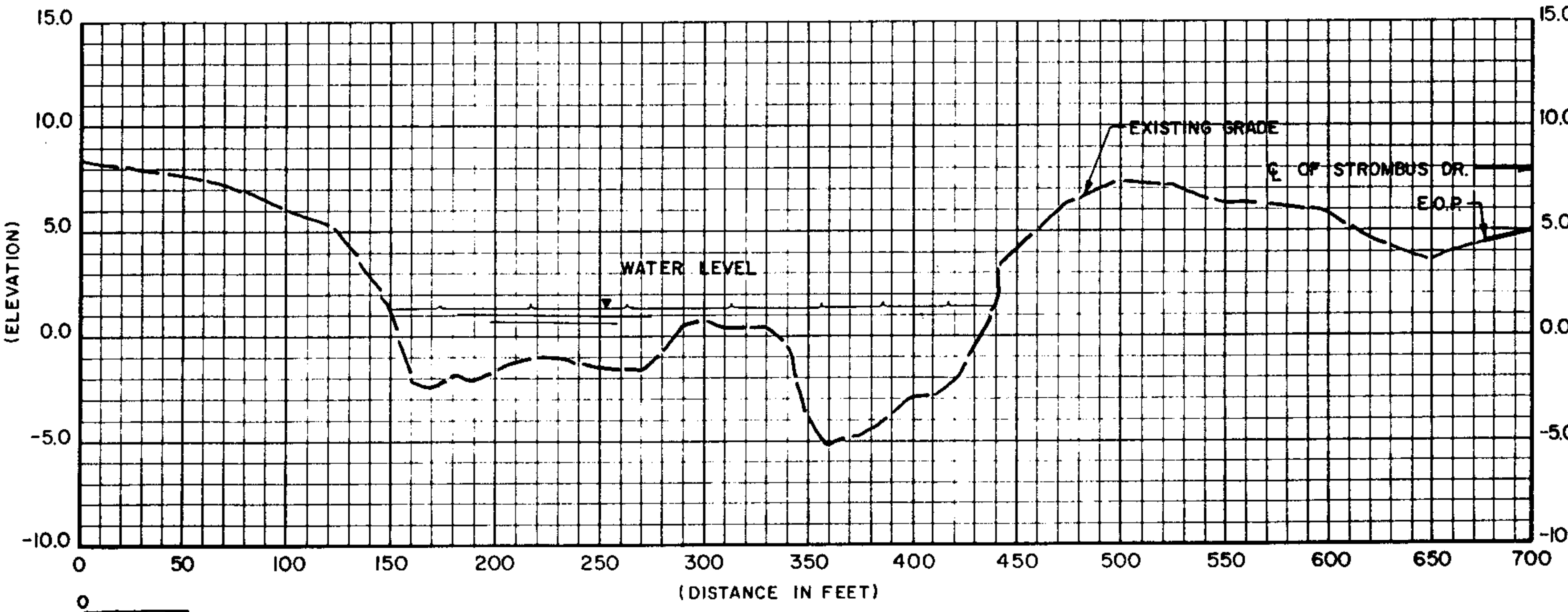



FIGURE AC-1

Station 3000

FIGURE AC-1

# SARASOTA COUNTY
# STORMWATER MANAGEMENT PROGRAM




0
50'

SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/22/85
TIME: 11:25 A M
WATER EL.: 1.36

## ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

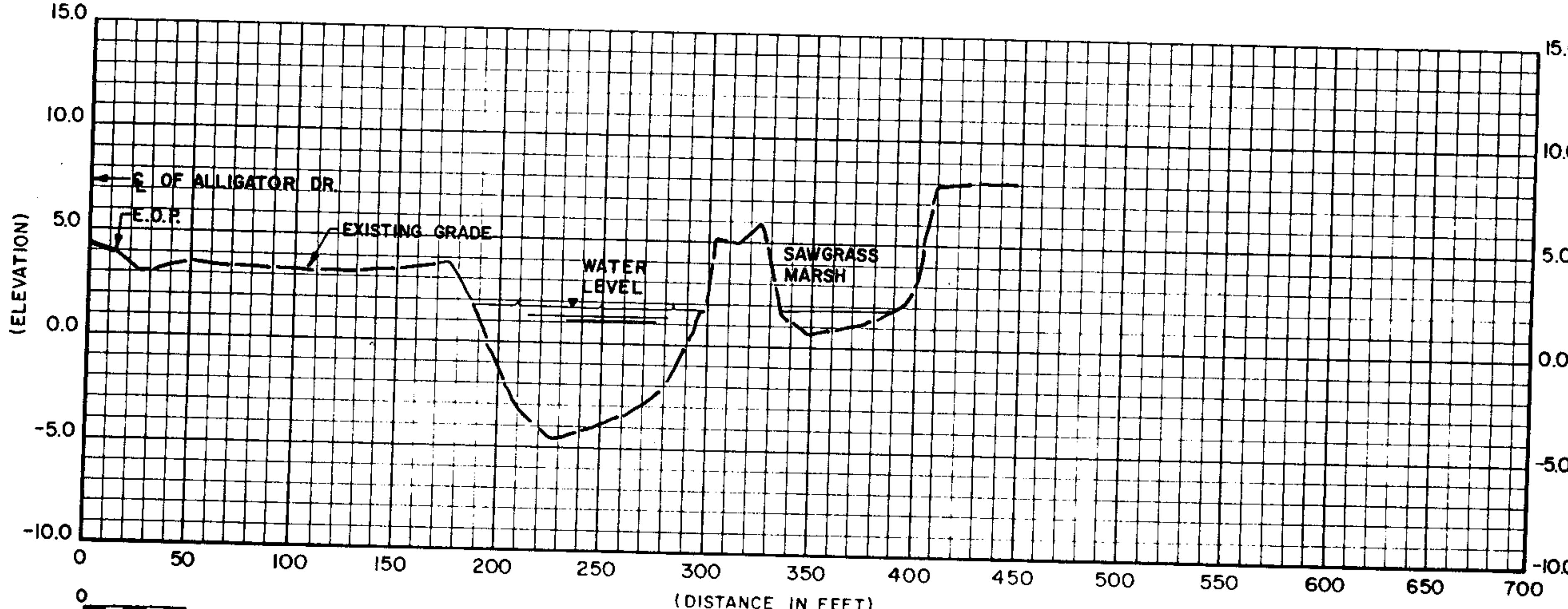
CROSS-SECTION
NO. A-2

FIGURE AC-2

Station 4620

FIGURE AC-2

SARASOTA COUNTY
STORMWATER MANAGEMENT PROGRAM




0 50'

SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/22/85
TIME: 1:30 PM
WATER EL.: 1.80

ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

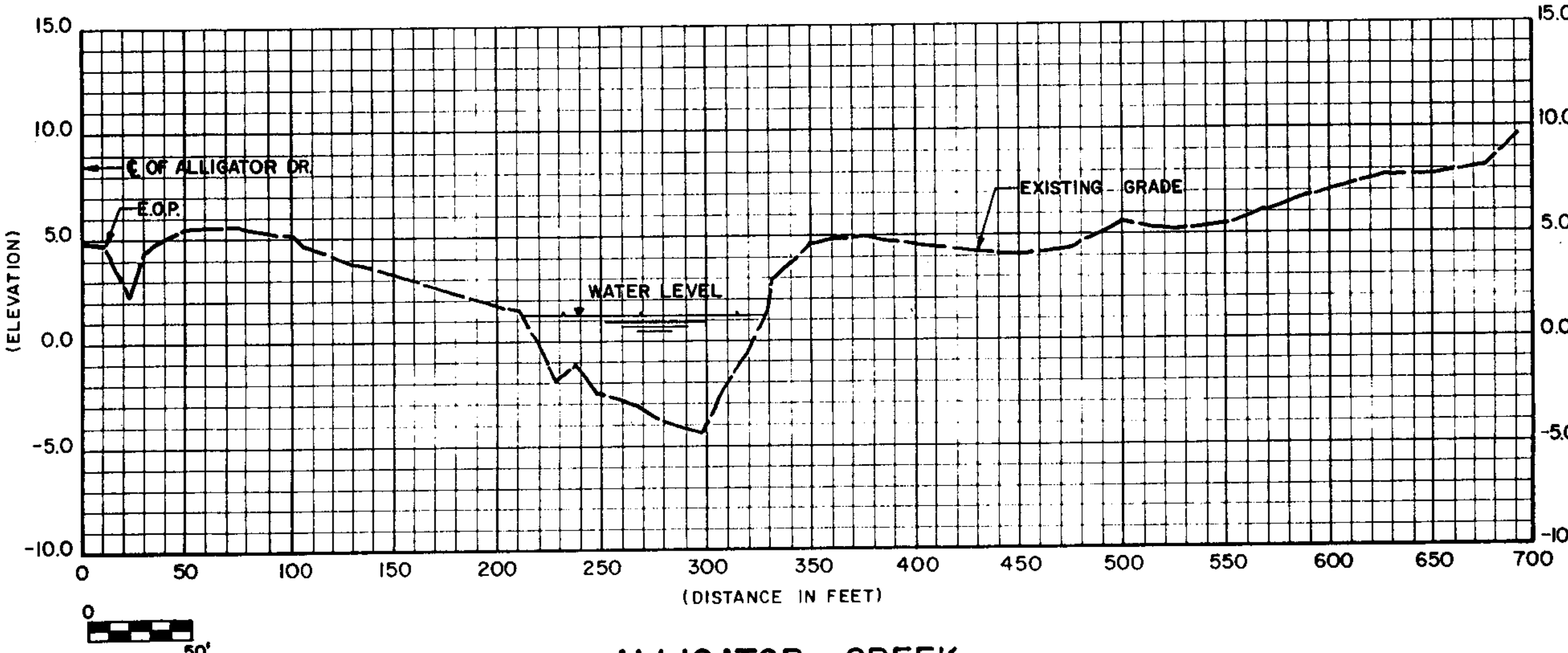
CROSS-SECTION
NO. A-3

FIGURE AC-3

Station 5720

FIGURE AC-3

SARASOTA COUNTY
STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/22/85
TIME: 2:45 PM
WATER EL.: 1.20

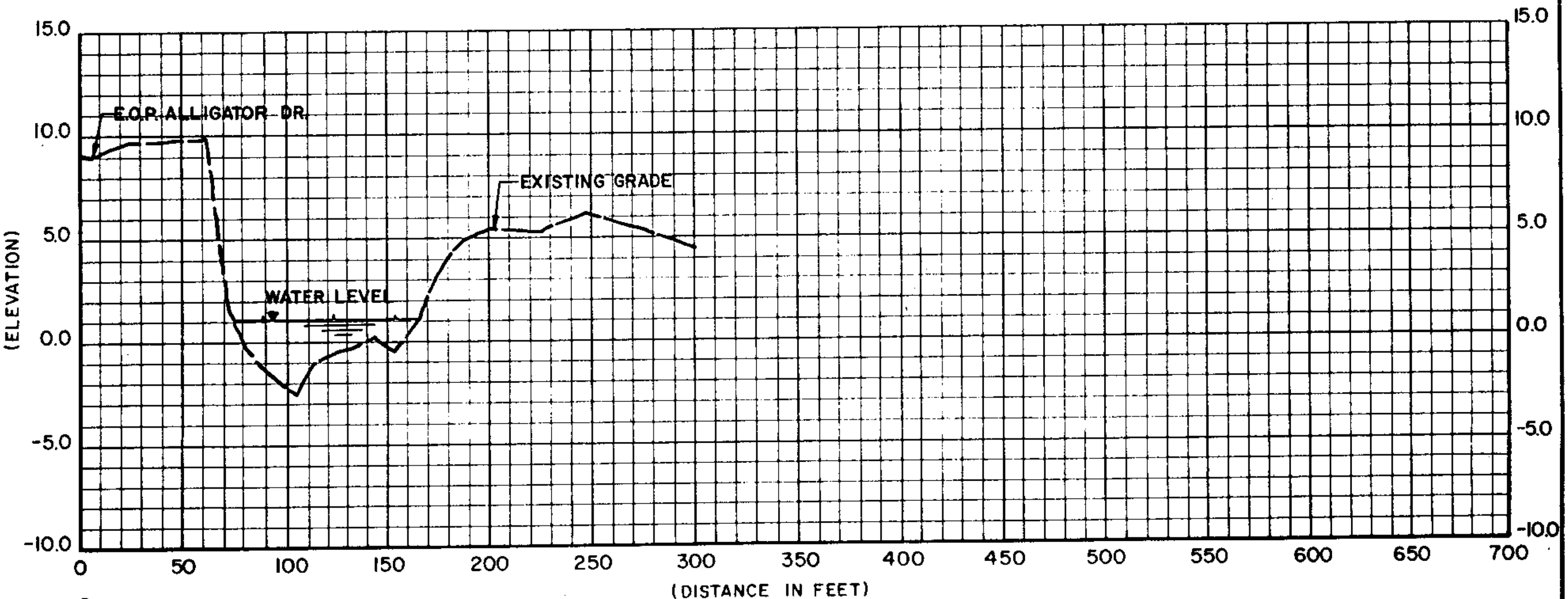
## ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-4

FIGURE AC-4

SARASOTA COUNTY
STORMWATER MANAGEMENT PROGRAM




0
50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

# ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-5

DATE: 5/23/85
TIME: 9:30 AM
WATER EL.: 1.14

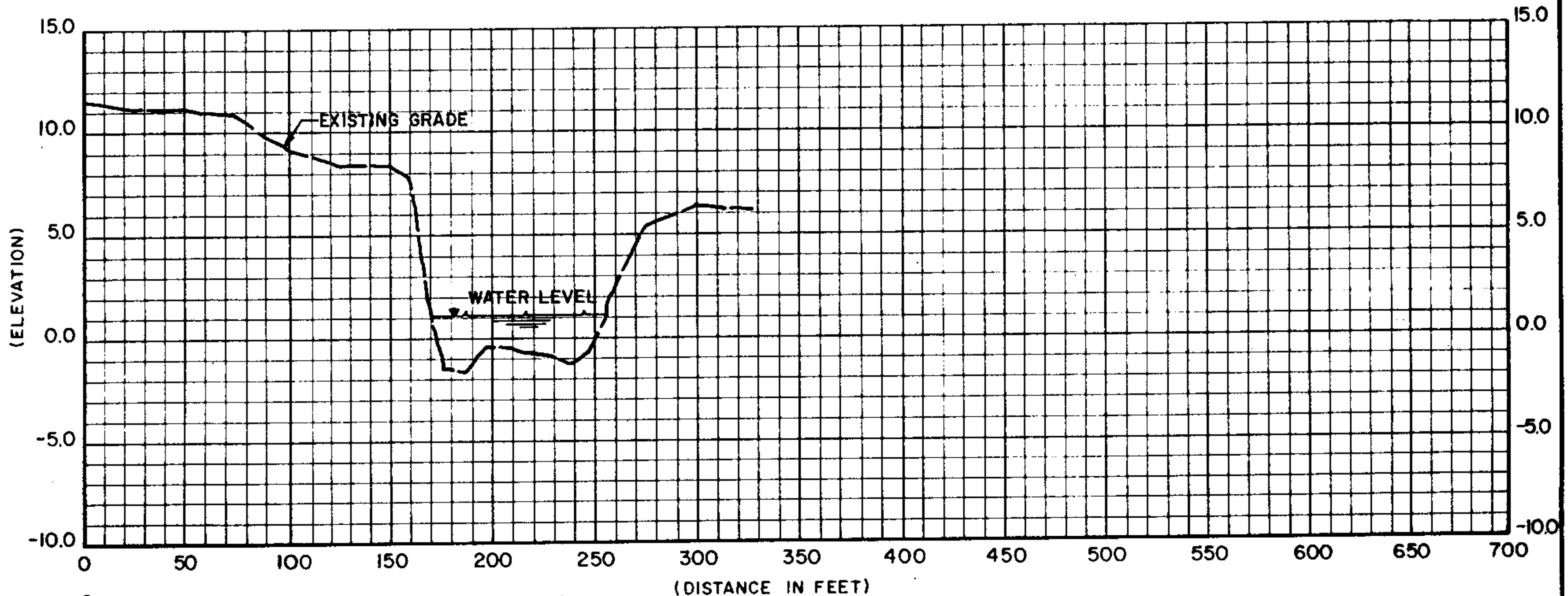
NOT A REPRESENTATIVE CROSS - SECTION

Station 8080

FIGURE AC-5

FIGURE AC-5

# SARASOTA COUNTY
## STORMWATER MANAGEMENT PROGRAM




0
50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 9:55 AM
WATER EL.: 1.10

**ALLIGATOR CREEK**

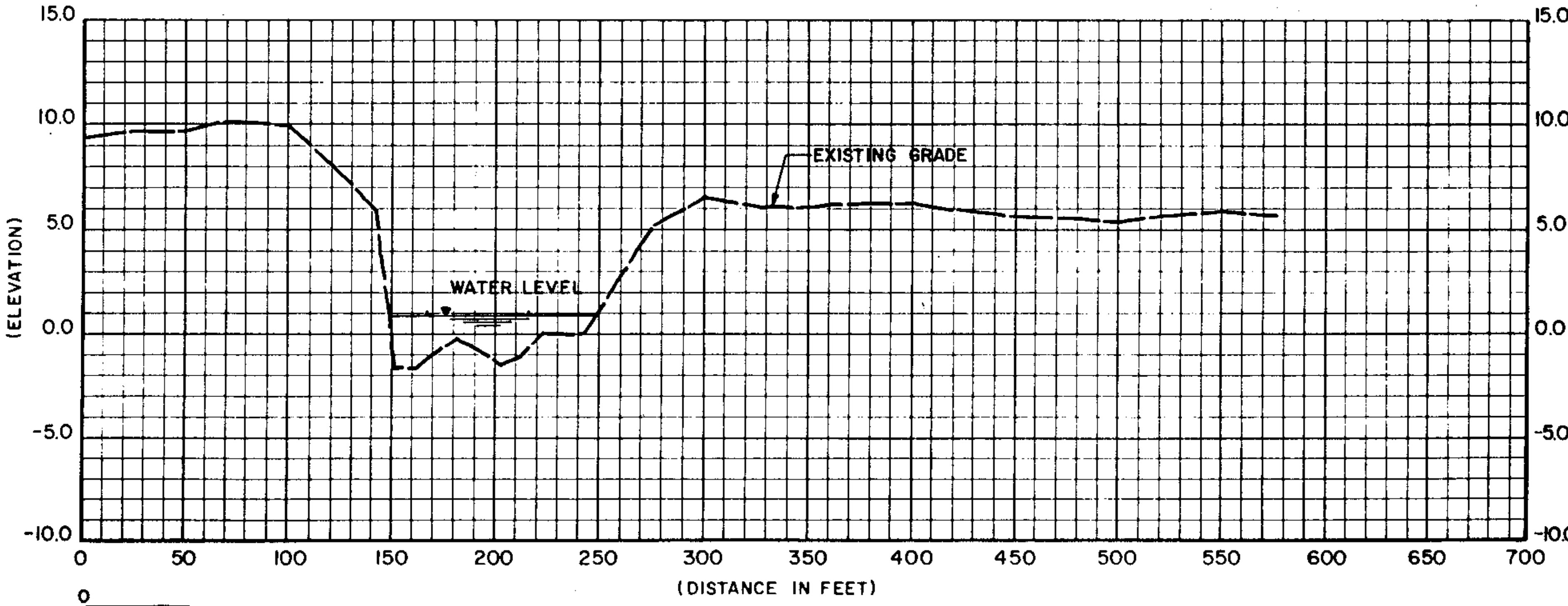
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-6

FIGURE AC-6

Station 8320

# SARASOTA COUNTY
# STORMWATER MANAGEMENT PROGRAM




0
50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 2:25 PM
WATER EL.: 0.85

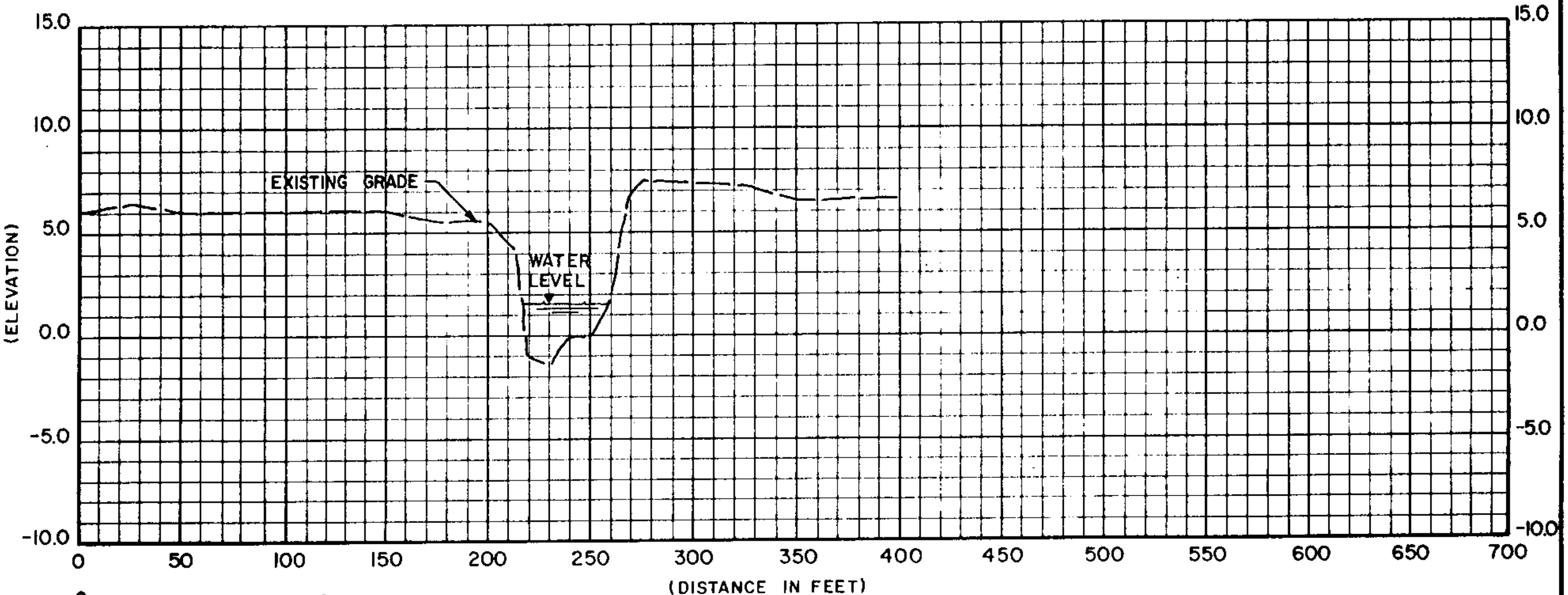
## ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-7

FIGURE AC-7

# SARASOTA COUNTY
## STORMWATER MANAGEMENT PROGRAM




0 50'

SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 11:30 AM
WATER EL.: 1.47

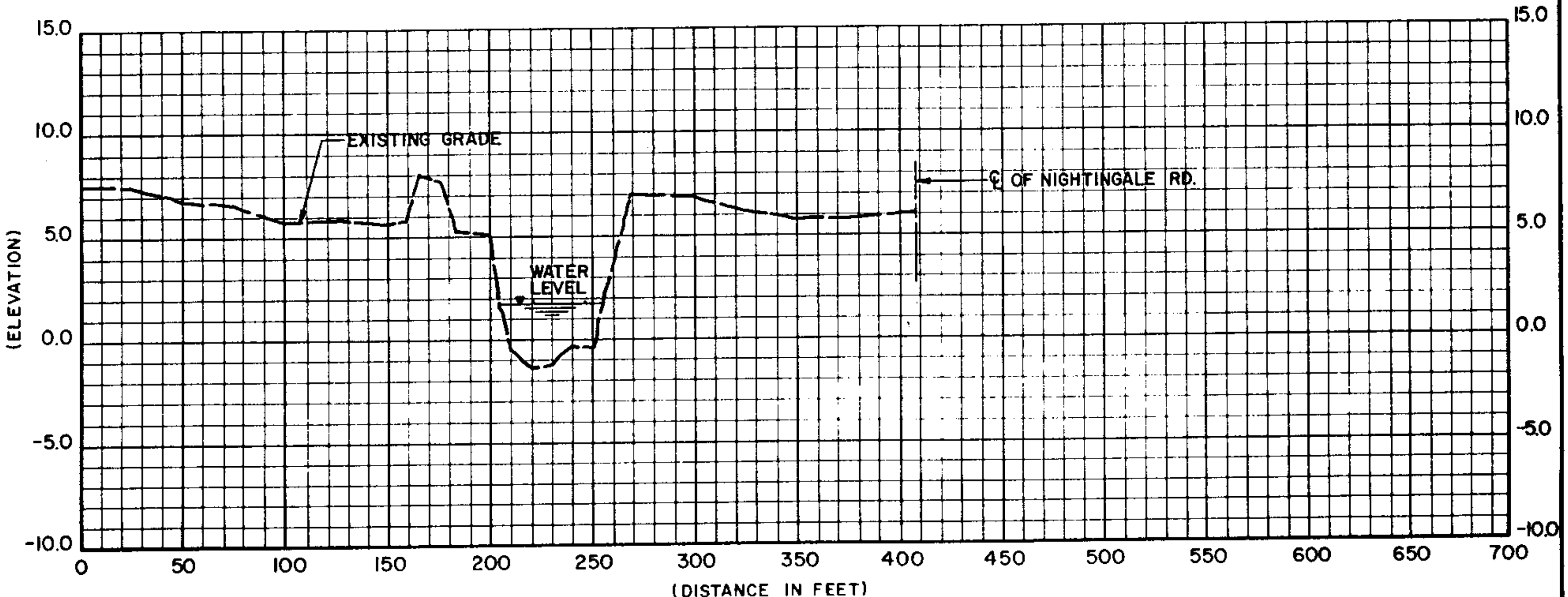
**ALLIGATOR CREEK**

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-8

FIGURE AC-8

# SARASOTA COUNTY
## STORMWATER MANAGEMENT PROGRAM




0

50'

SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 1:02 PM
WATER EL.: 1.80

# ALLIGATOR CREEK

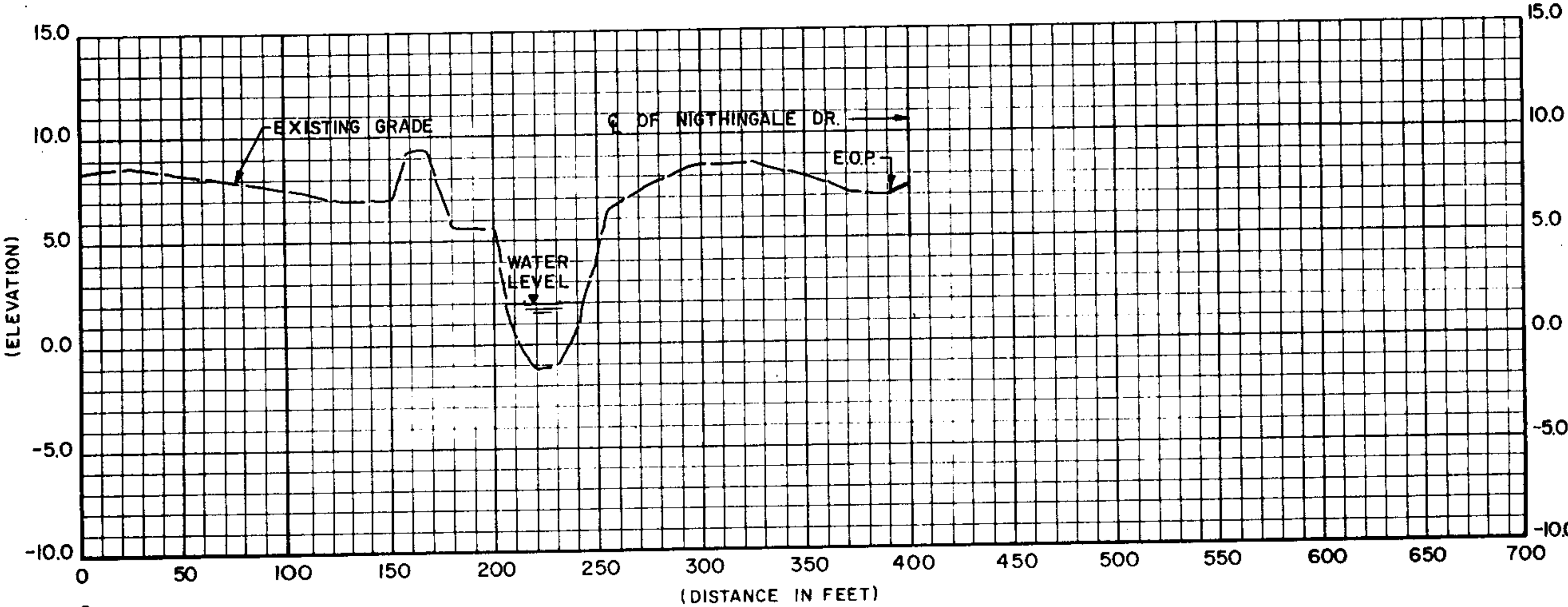
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-9

FIGURE AC-9

SARASOTA COUNTY

STORMWATER MANAGEMENT PROGRAM




0

50'

SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 1:21 PM
WATER EL.: 1.90

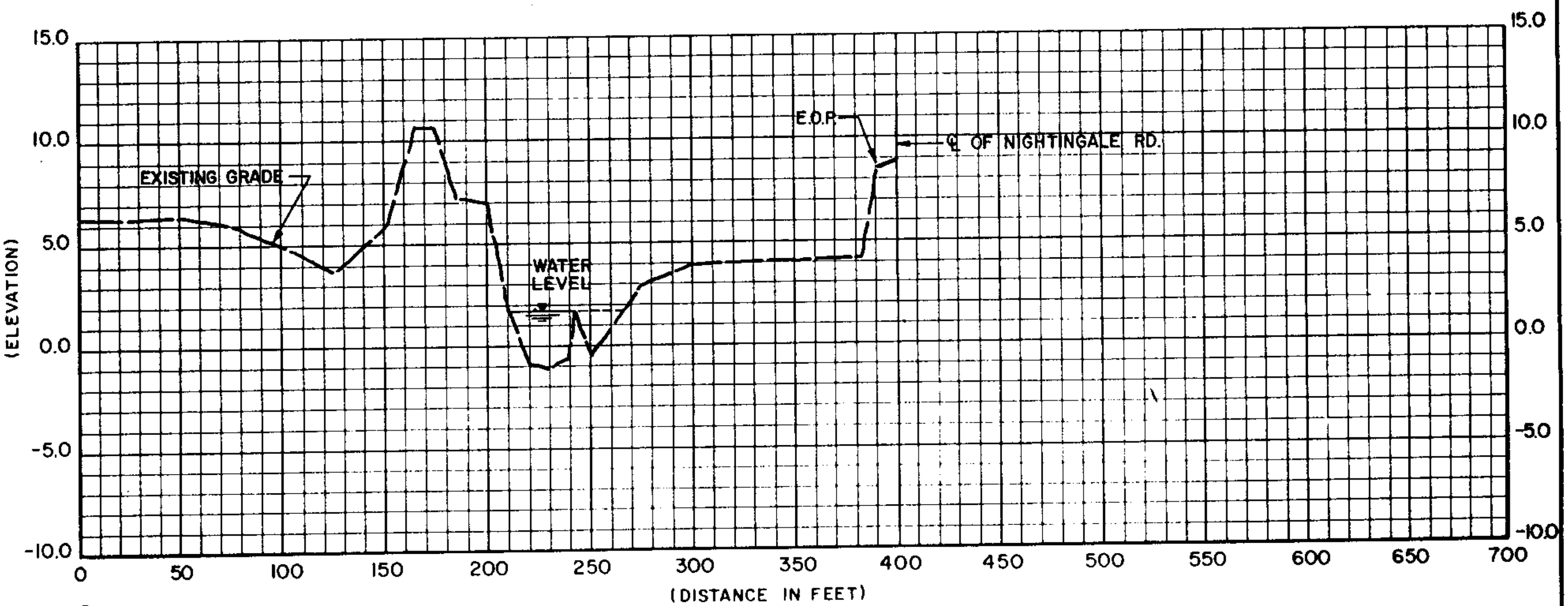
# ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-10

FIGURE AC-10

# SARASOTA COUNTY
# STORMWATER MANAGEMENT PROGRAM




0
50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 1:38 PM
WATER EL.: 1.80

## ALLIGATOR CREEK

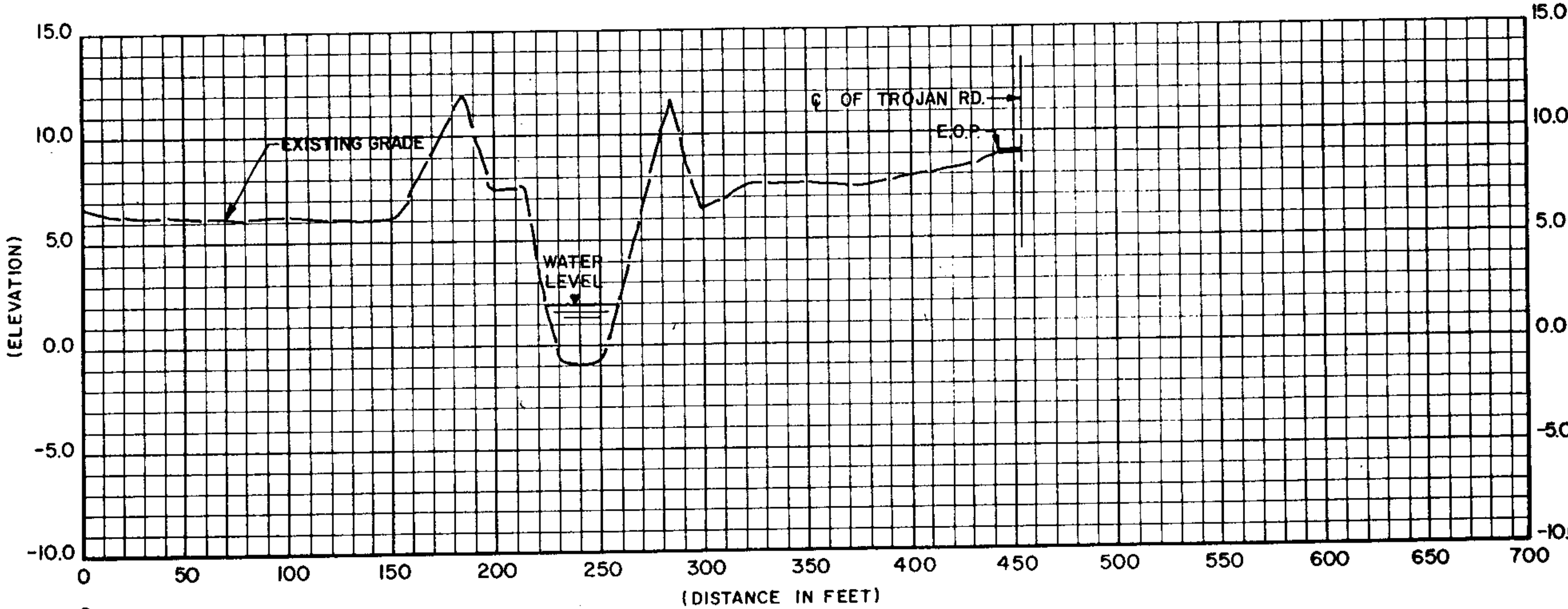
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-11

FIGURE AC-11

Station 10890

# SARASOTA COUNTY
## STORMWATER MANAGEMENT PROGRAM




0 50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

DATE: 5/23/85
TIME: 2:23 AM
WATER EL.: 1.98

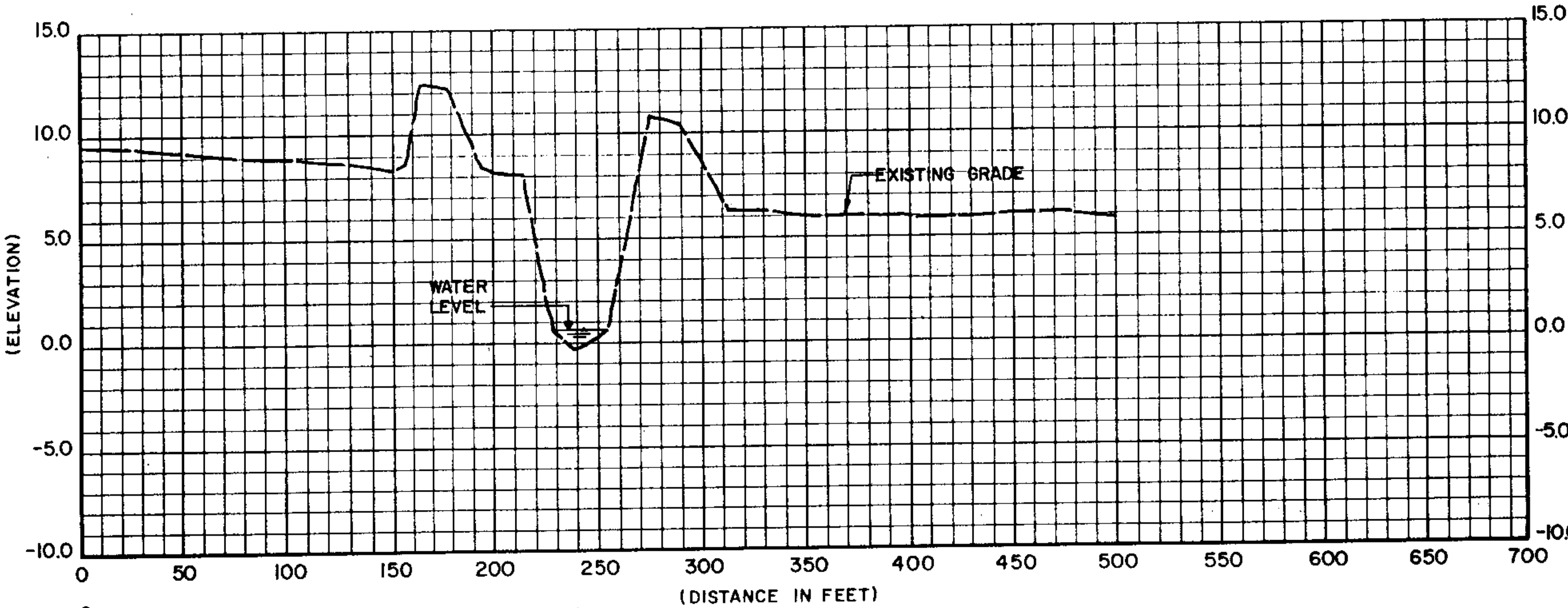
# ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. A-12

FIGURE AC-12

# SARASOTA COUNTY
# STORMWATER MANAGEMENT PROGRAM




0
50'
SCALE
HORIZ.: 1" = 50'
VERT.: 1" = 5'

## ALLIGATOR CREEK

VIEW - FACING DOWN STREAM

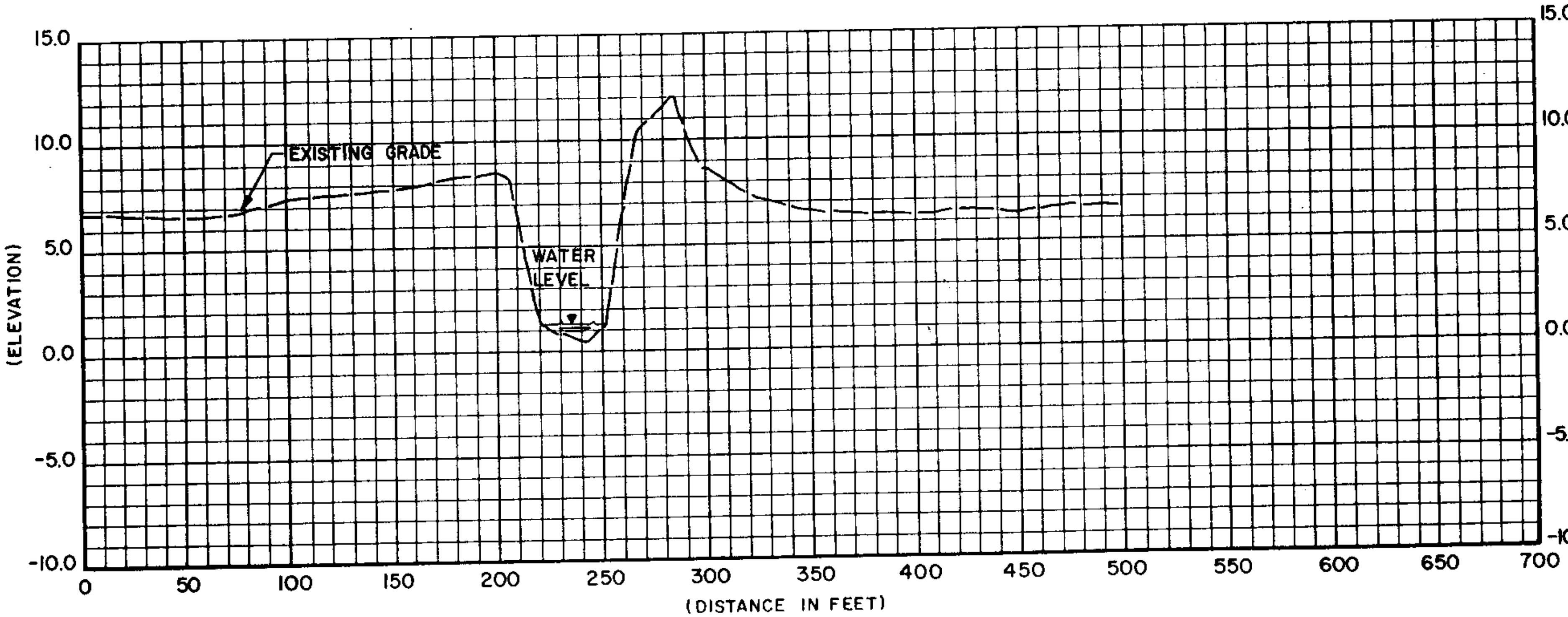
CROSS-SECTION
NO. A-13

DATE: 5/28/85
TIME: 8:30 AM
WATER EL.: 0.60

FIGURE AC-13

SARASOTA COUNTY

STORMWATER MANAGEMENT PROGRAM




0

50'

SCALE

HORIZ.: 1" = 50'

VERT.: 1" = 5'

DATE: 5/28/85

TIME: 11:30 AM

WATER EL.: 1.16

# ALLIGATOR CREEK

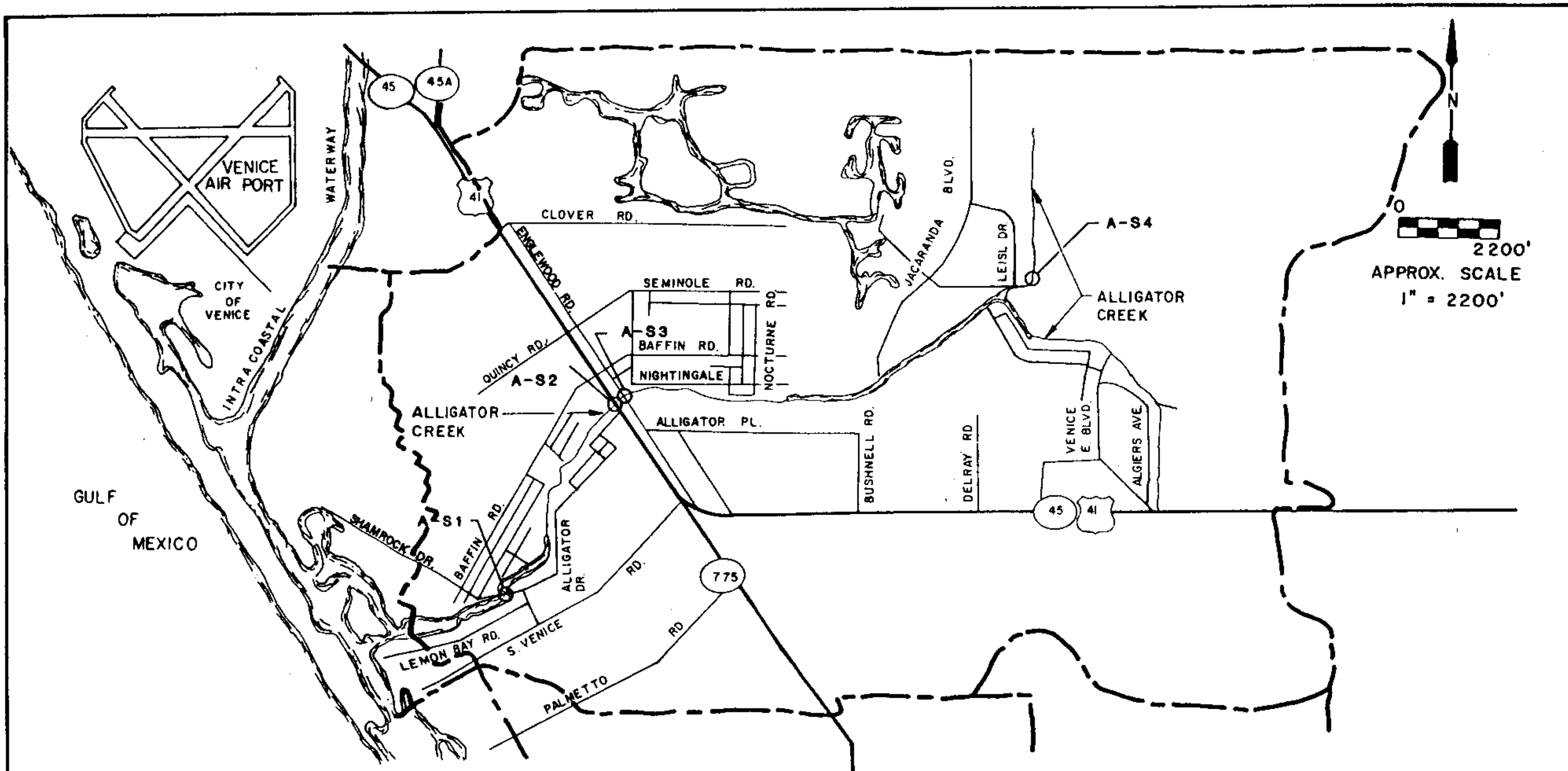
VIEW - FACING DOWN STREAM

CROSS-SECTION

NO. A-14

FIGURE AC-14

Station 15590




## LEGEND

**A-S1** STRUCTURE REFERENCE NUMBER

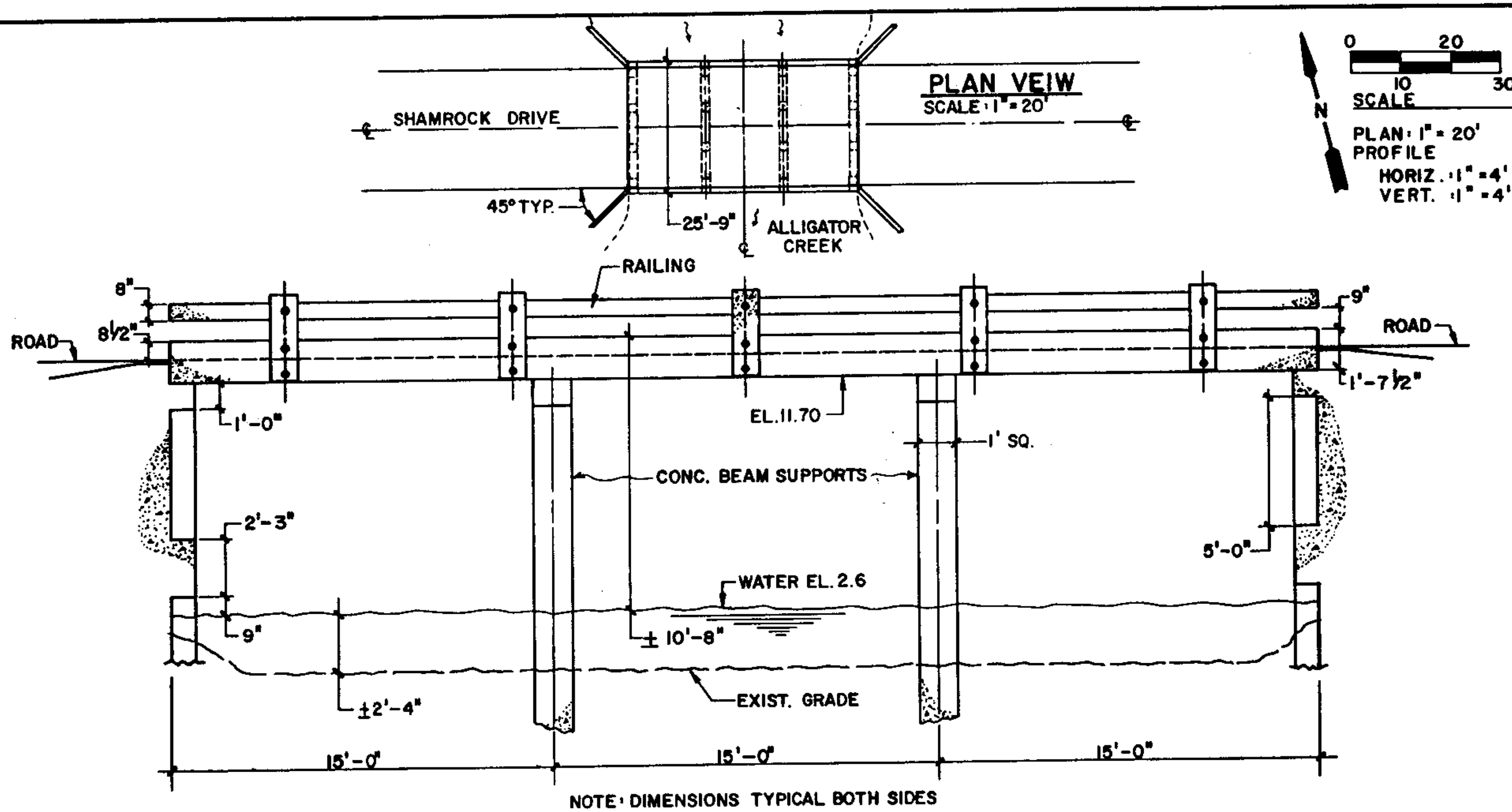
SARASOTA COUNTY
STORMWATER MANAGEMENT
PROGRAM

## ALLIGATOR CREEK

FIGURE B-2

Alligator Creek Structure Location Map

FIGURE B-2

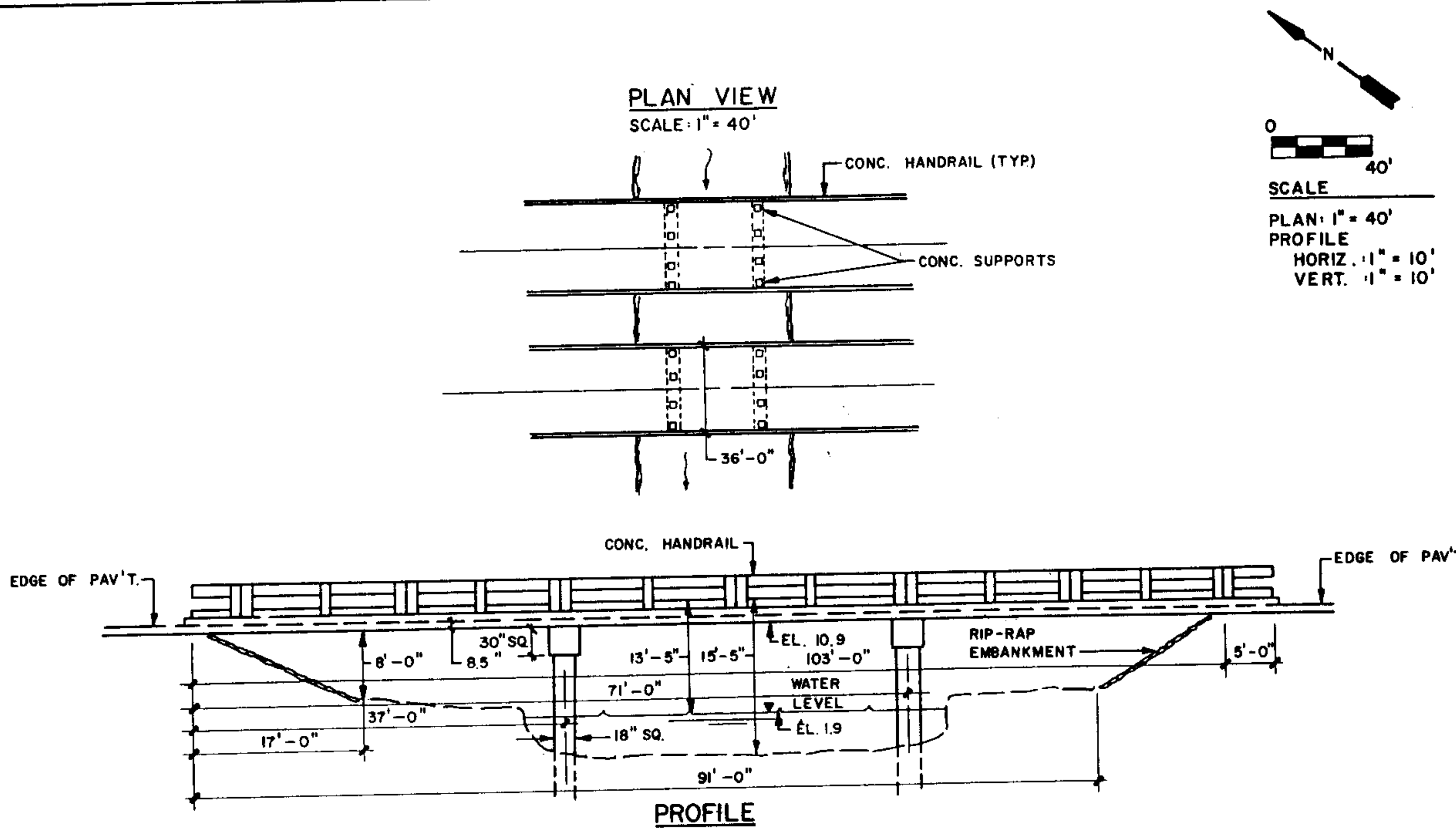



## SHAMROCK DRIVE BRIDGE

SCALE: 1" = 4'

STRUCTURE REFERENCE NO. A-S1

FIGURE AC-S1

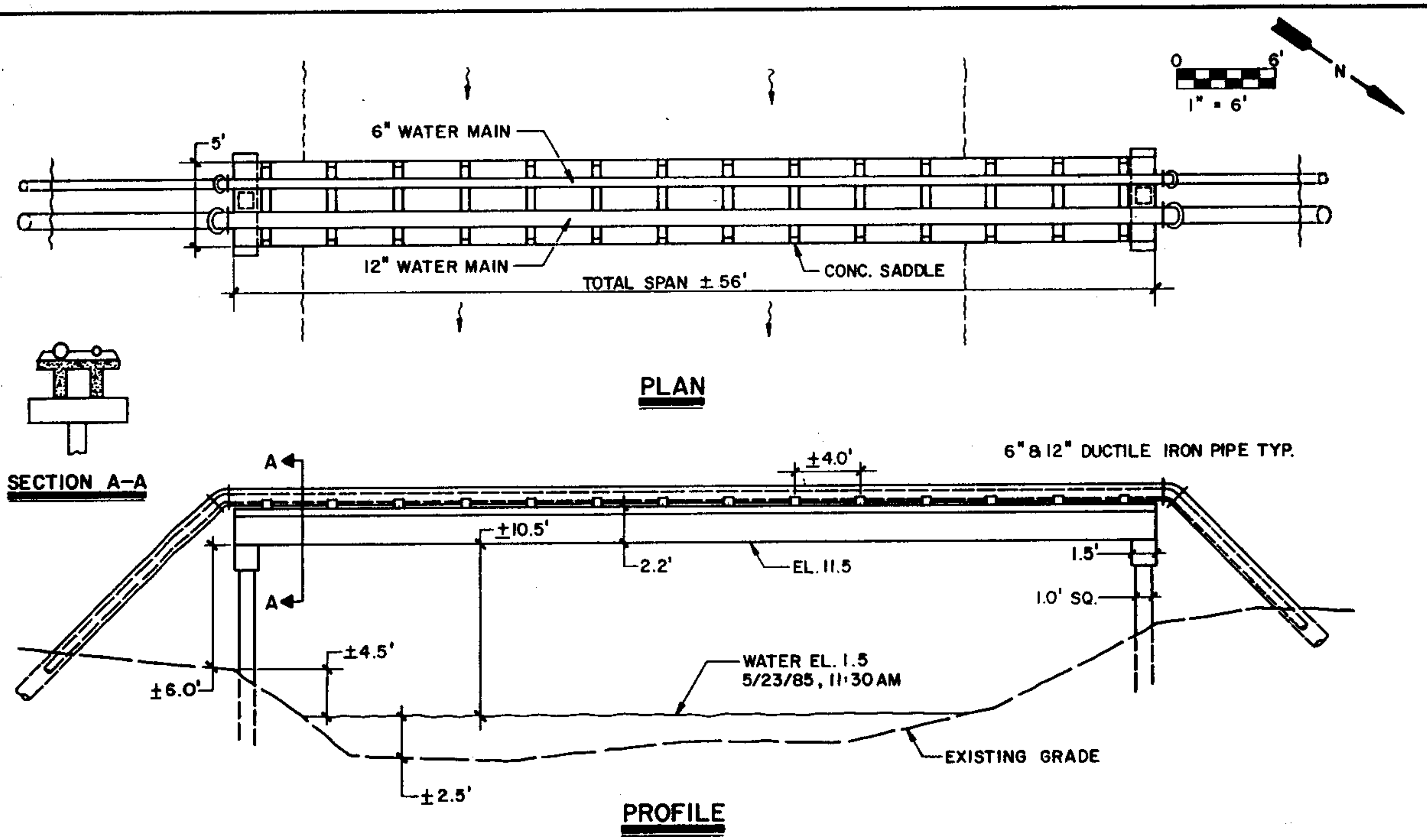



## U.S. 41 & ALLIGATOR CREEK BRIDGE

SCALE : 1" = 10'
NOTE : DIMENSION TPY. BOTH SIDES

STRUCTURE REFERENCE NO. A-S2

FIGURE AC-S2



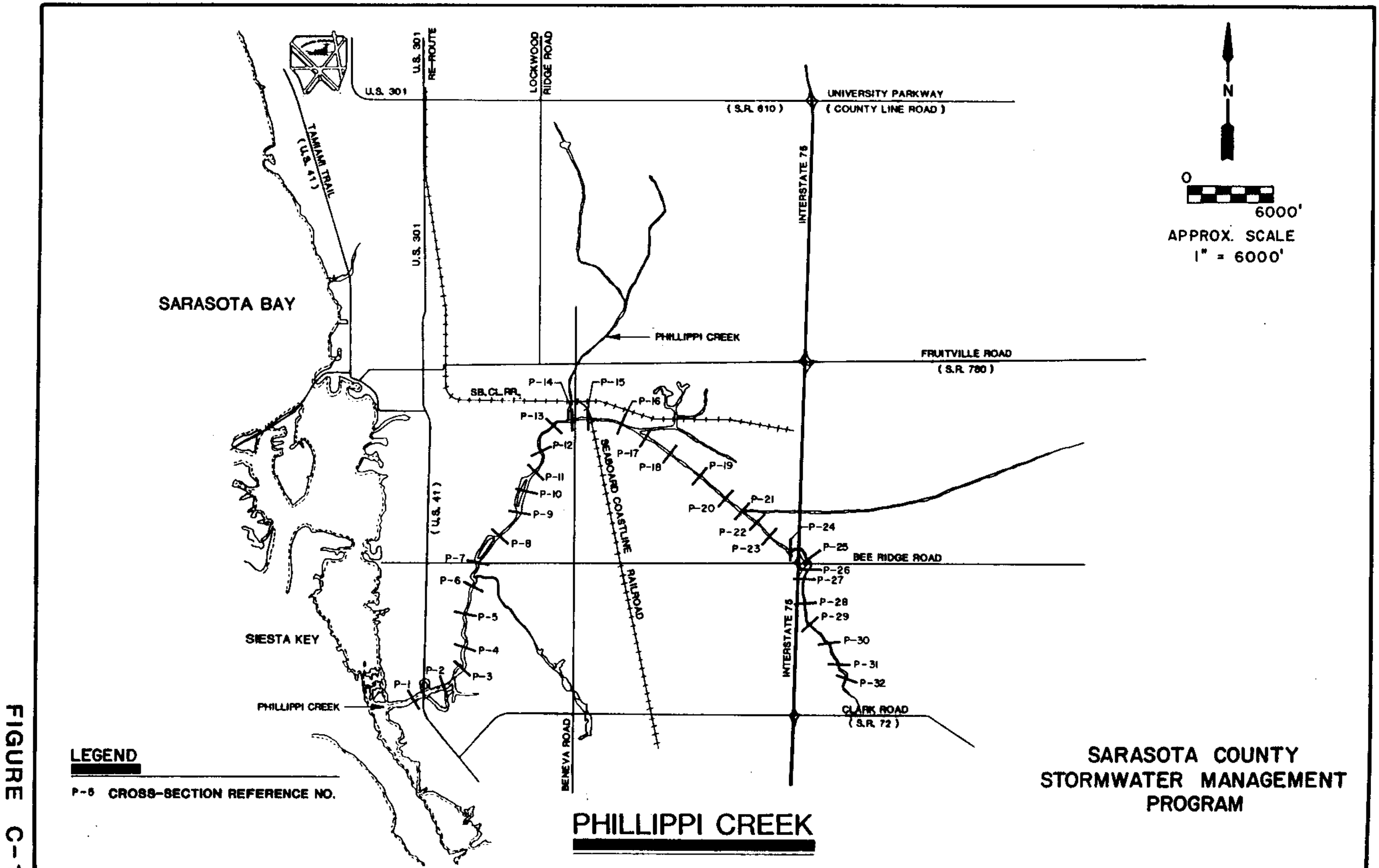

# ALLIGATOR CREEK PIPE CROSSING DETAIL

STRUCTURE REFERENCE NO. A-S3

FIGURE AC-S3

# APPENDIX C

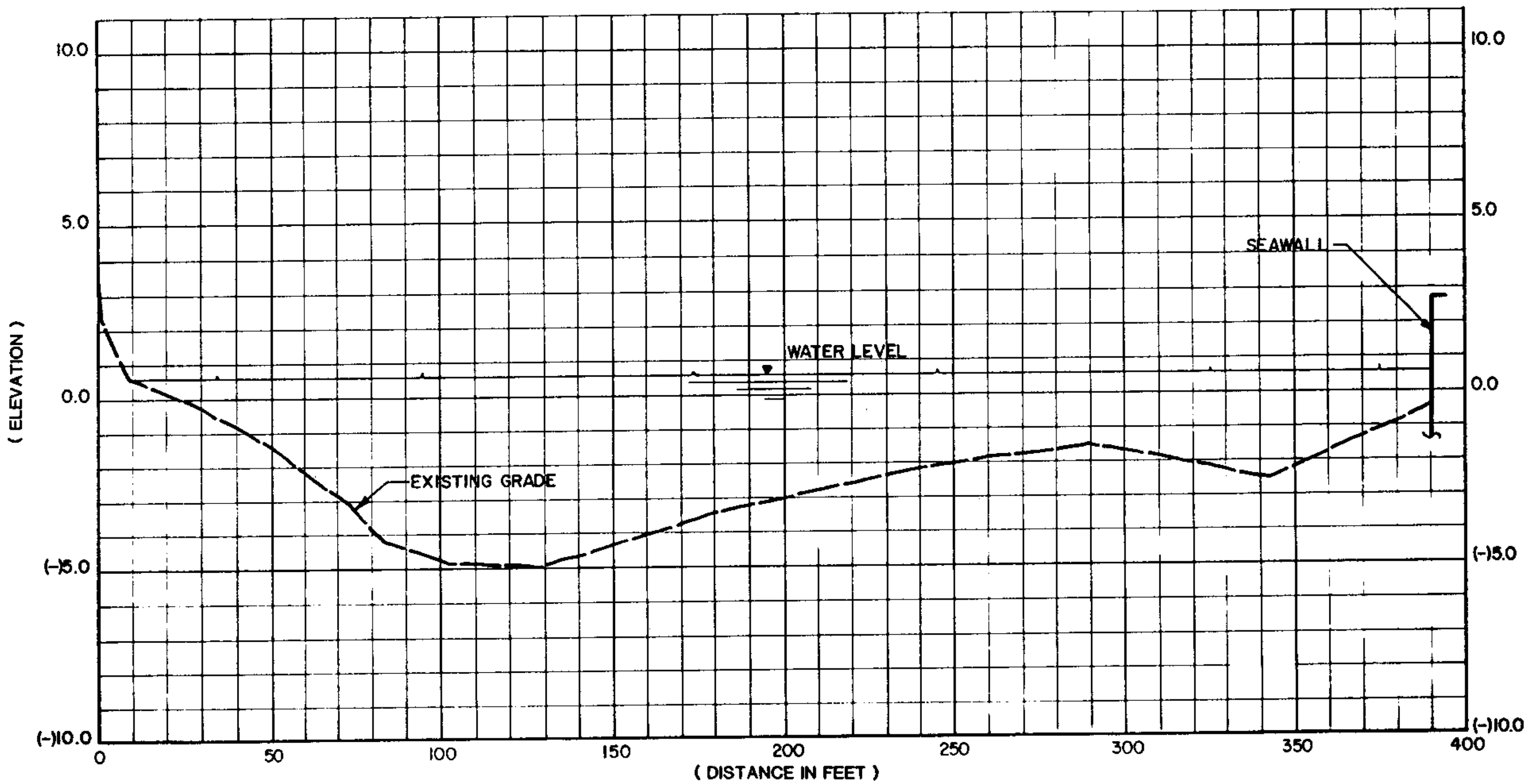
## PHILLIPPI CREEK
## CROSS-SECTIONS AND STRUCTURES




Phillippi Creek Cross-Section Location Map

FIGURE C-1

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'
0
30'
WATER EL. : +0.6

## PHILLIPPI CREEK

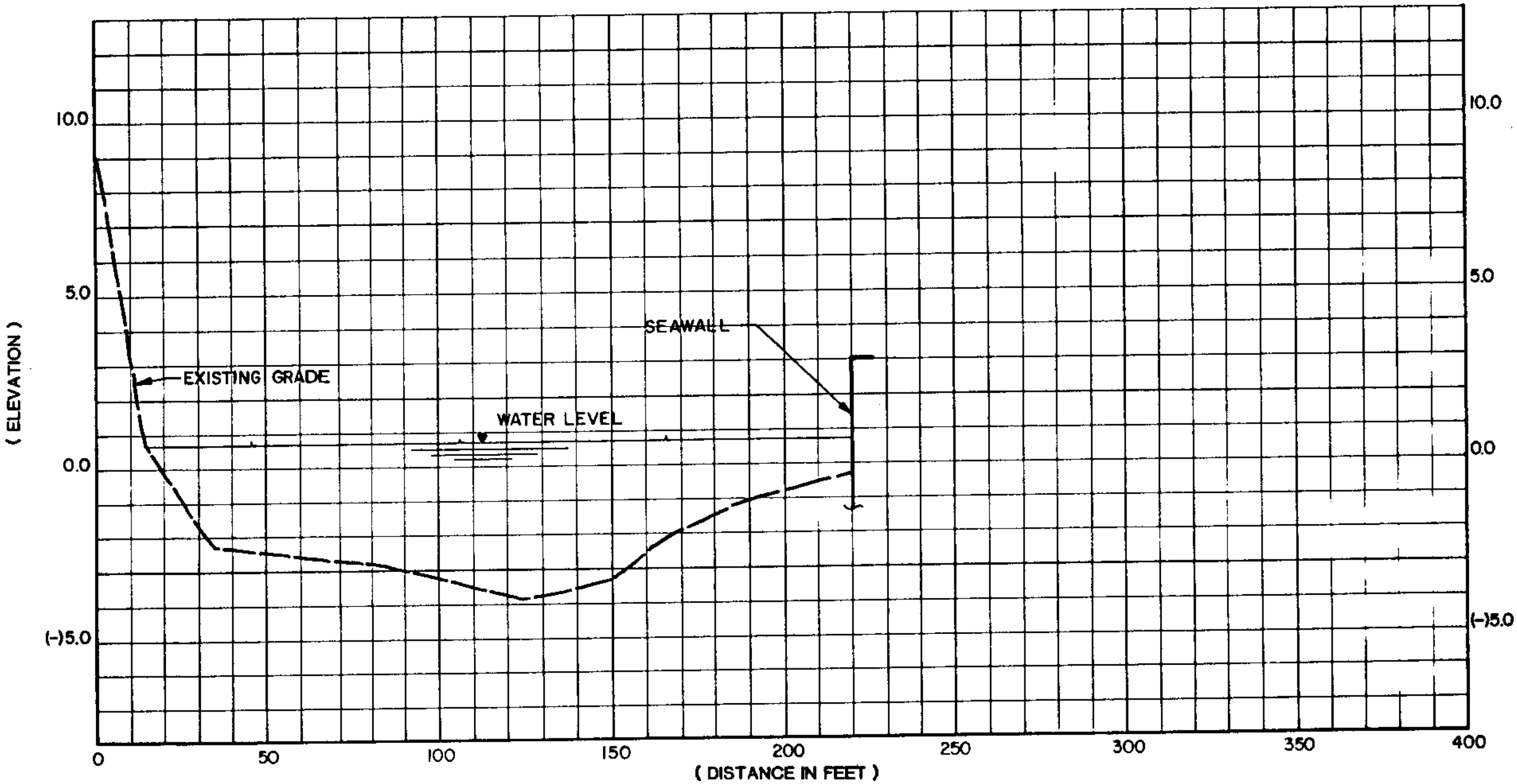
VIEW - FACING DOWNSTREAM

CROSS - SECTION
NO. P-I

FIGURE P-1

Station 3150

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1" = 30'
VERT. : 1" = 3'
0
30'
WATER EL. : +0.7

PHILLIPPI CREEK

VIEW - FACING DOWNSTREAM

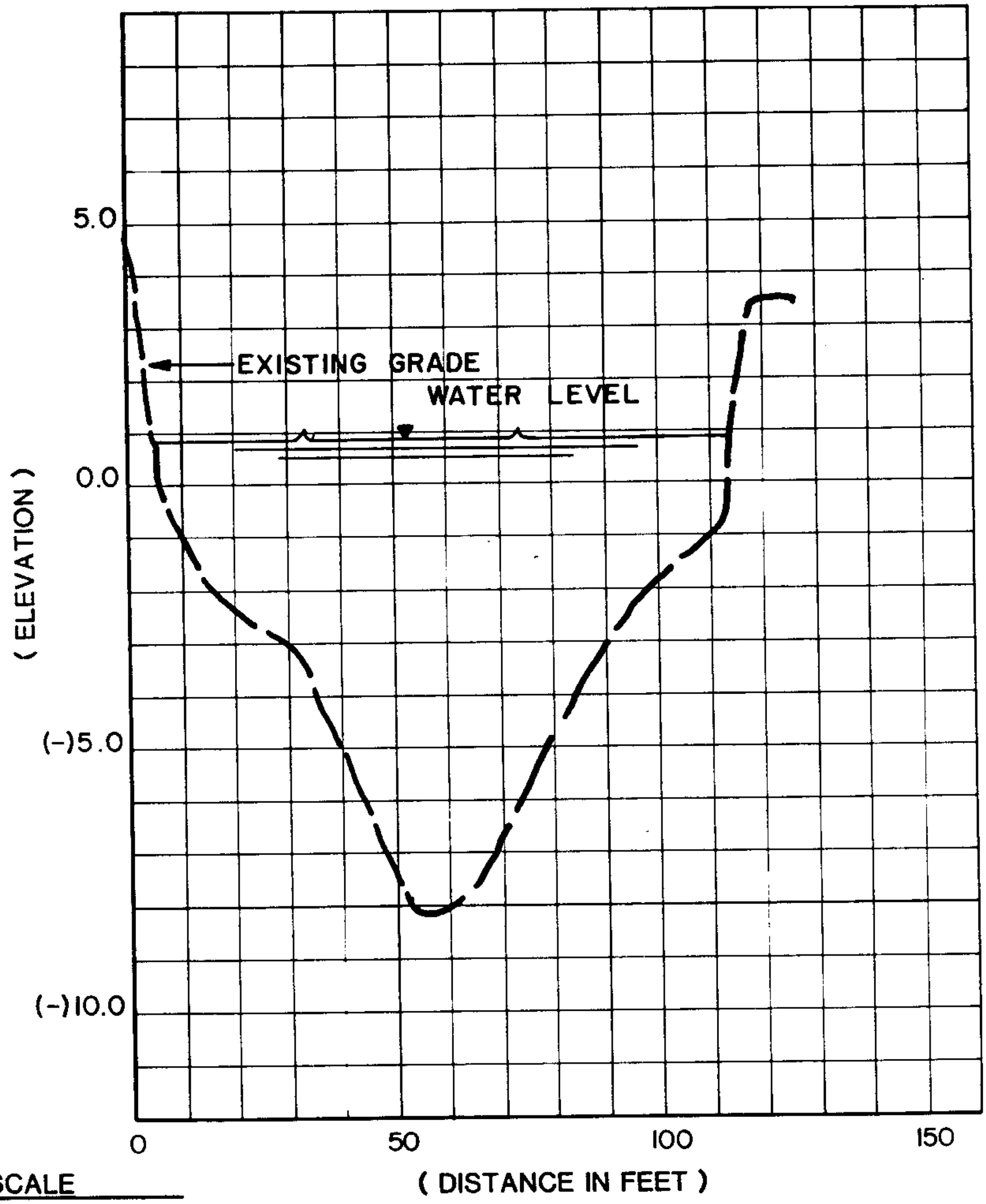
CROSS - SECTION
NO. P-2

FIGURE P-2

Station 4600

FIGURE P-2

## SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : +0.9

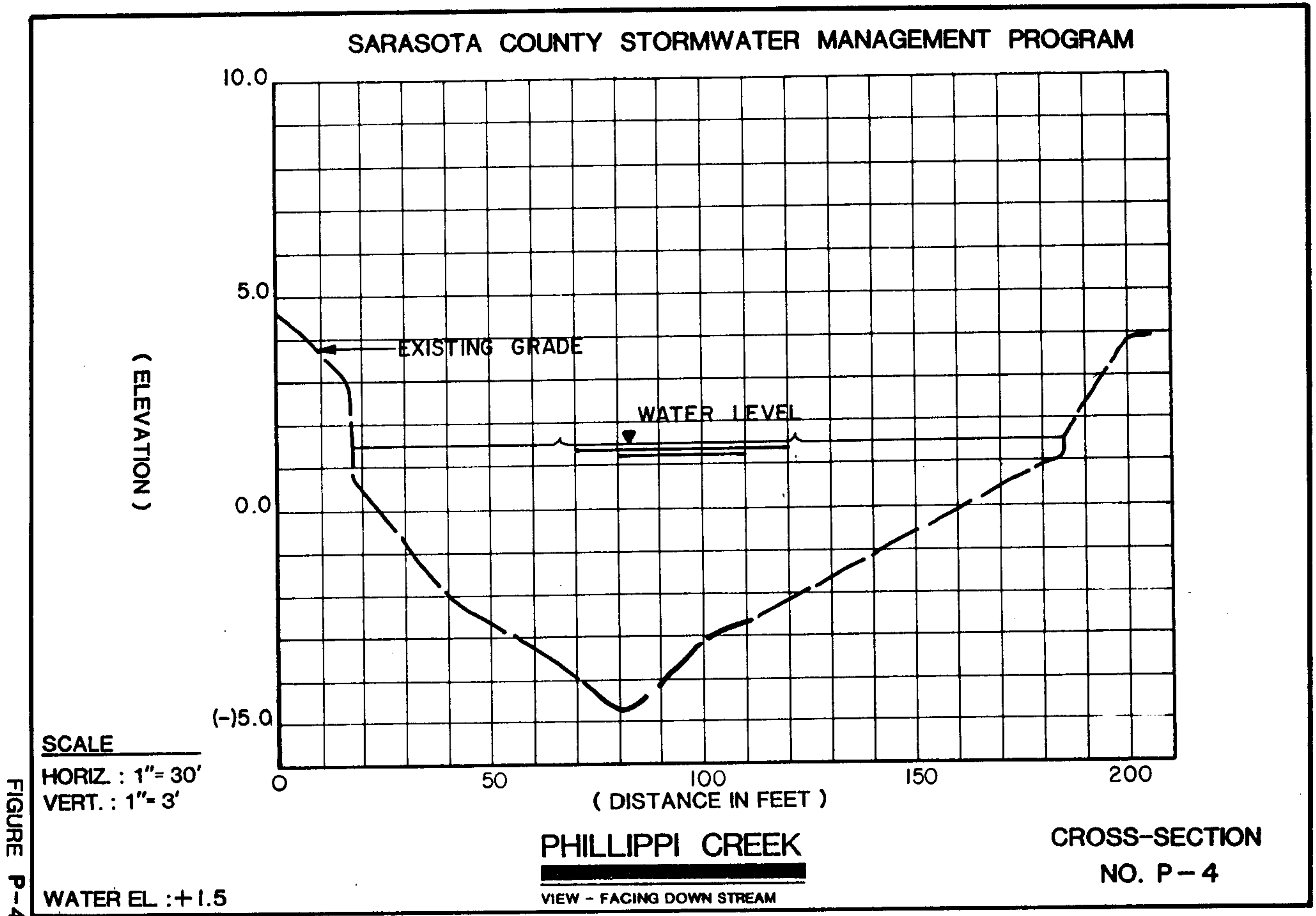
## PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-3

Station 7150

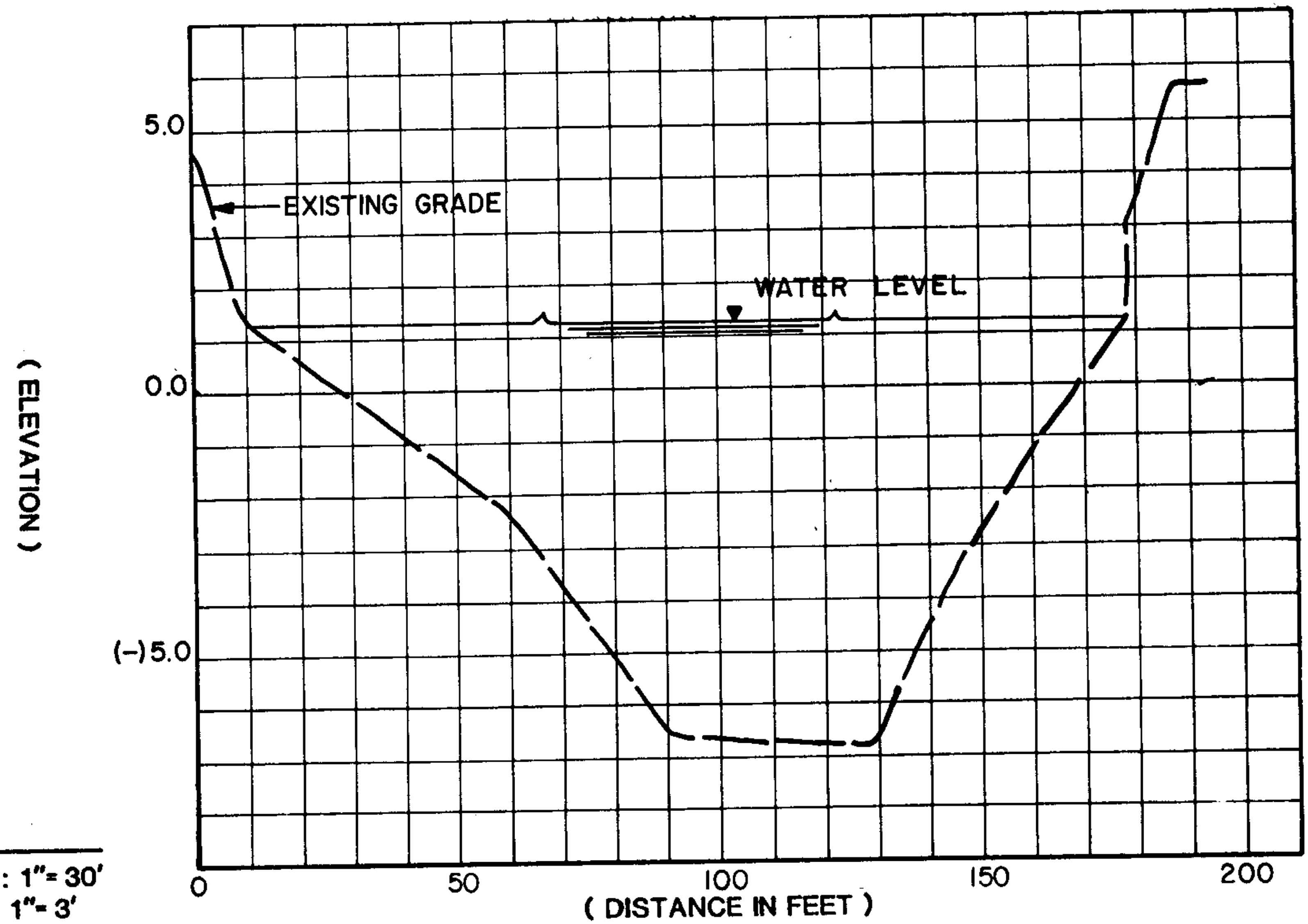
FIGURE P-3




Station 9150

FIGURE P-4

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 1.3

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

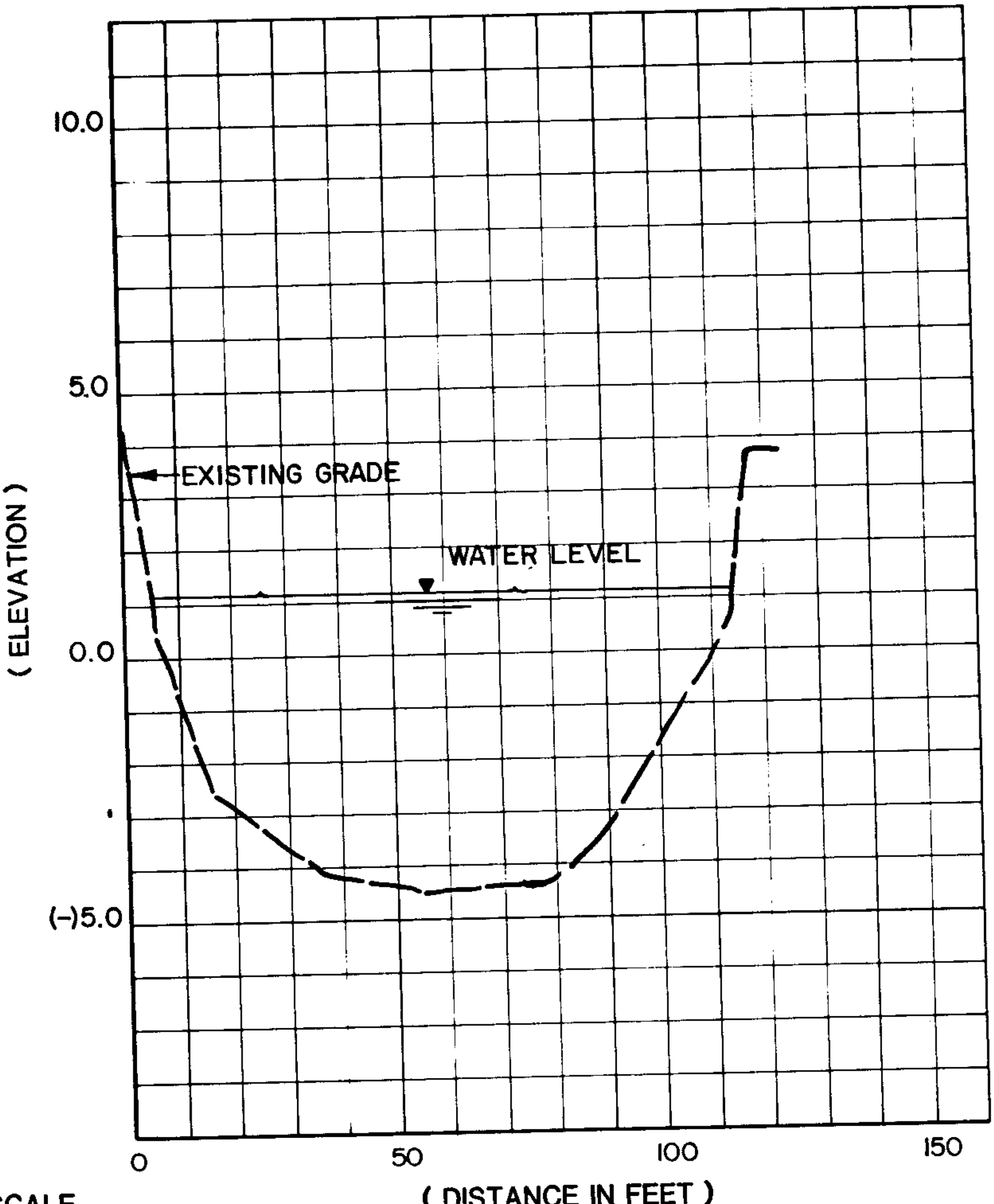
CROSS-SECTION
NO. P-5

FIGURE P-5

Station 10460

FIGURE P-5

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

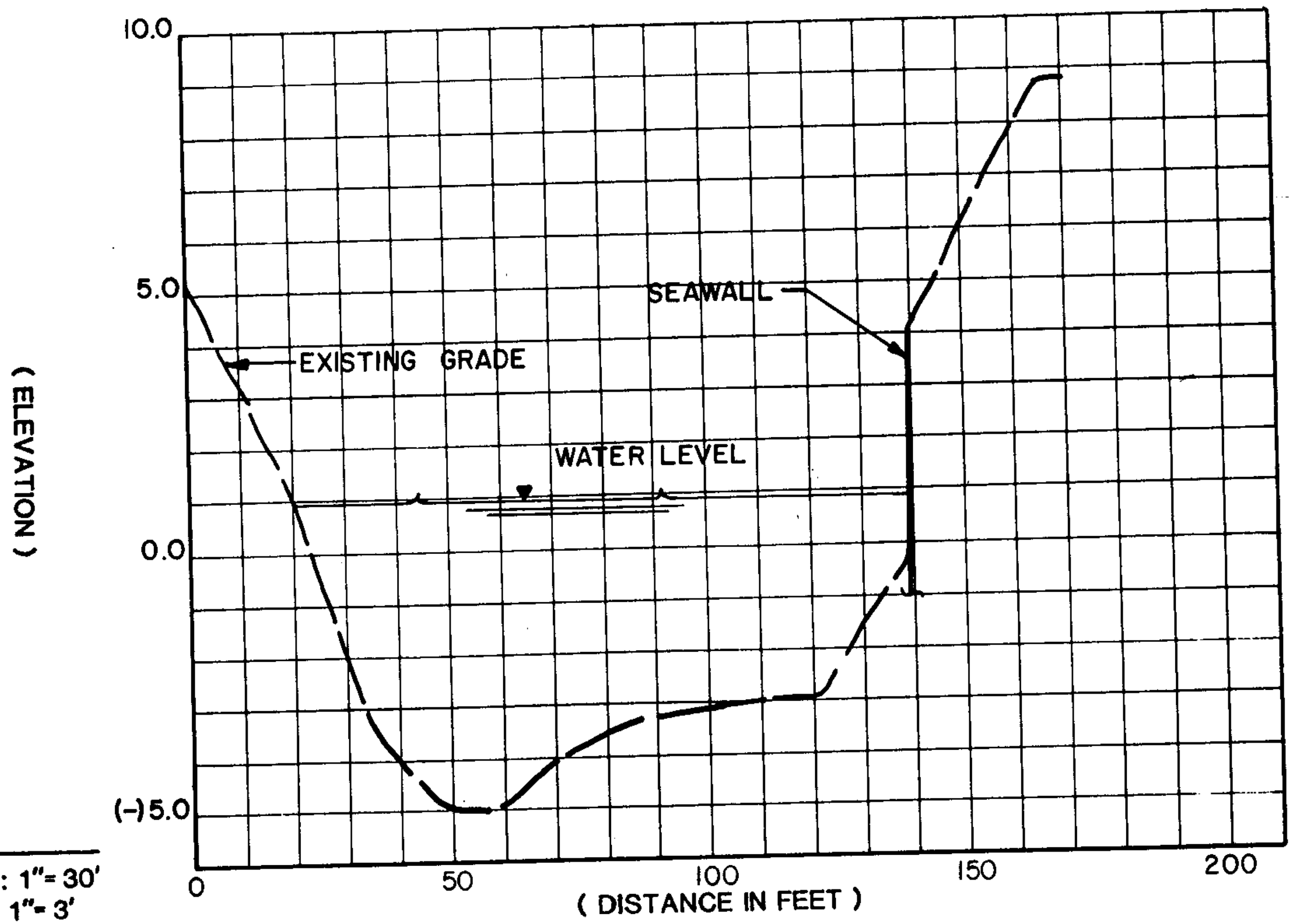
CROSS-SECTION
NO. P-6

WATER EL. : +1.2

Station 13150

FIGURE P-6

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 0.9

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

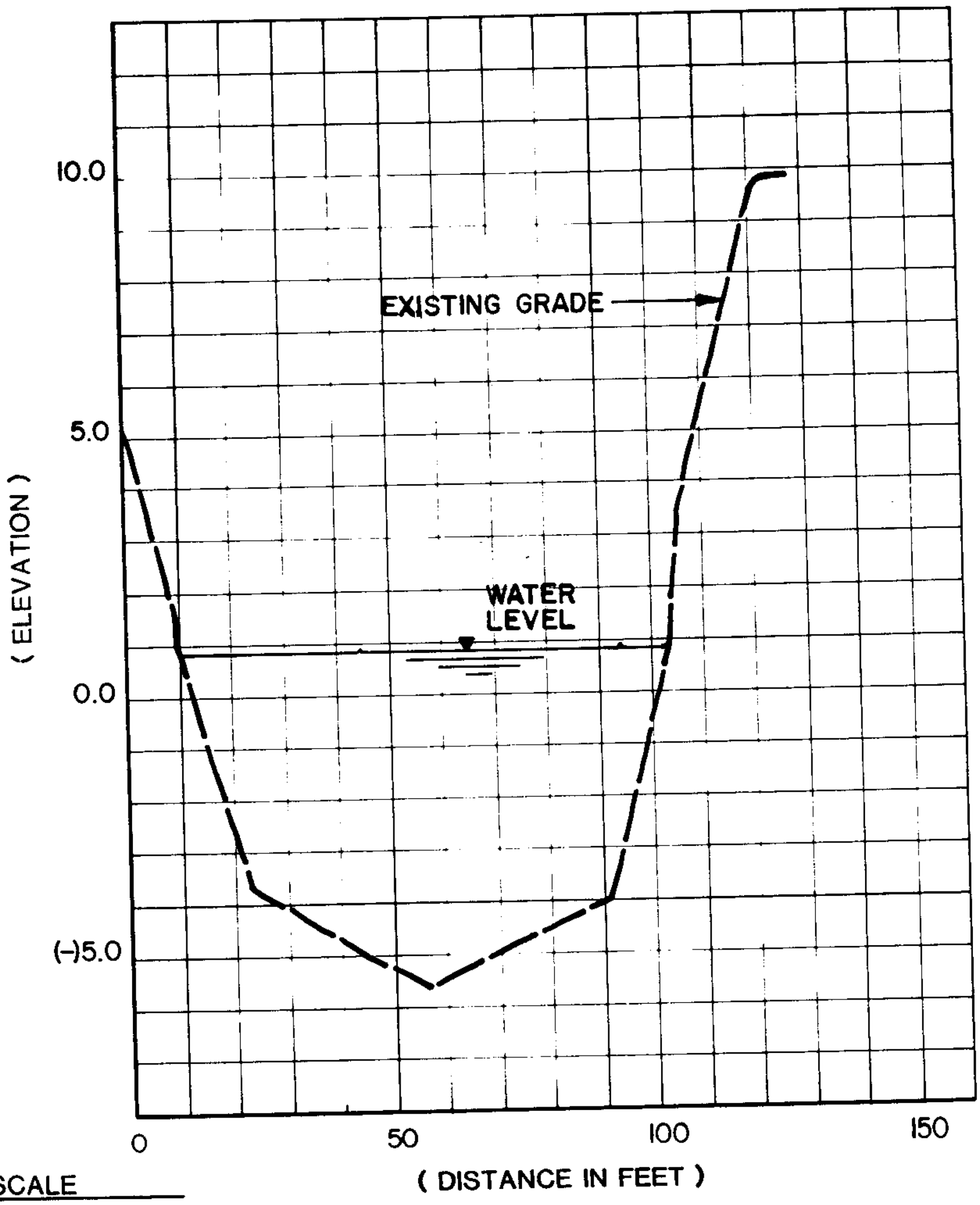
CROSS-SECTION
NO. P-7

FIGURE P-7

Station 15100

FIGURE P-7

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. :+0.8

PHILLIPPI CREEK

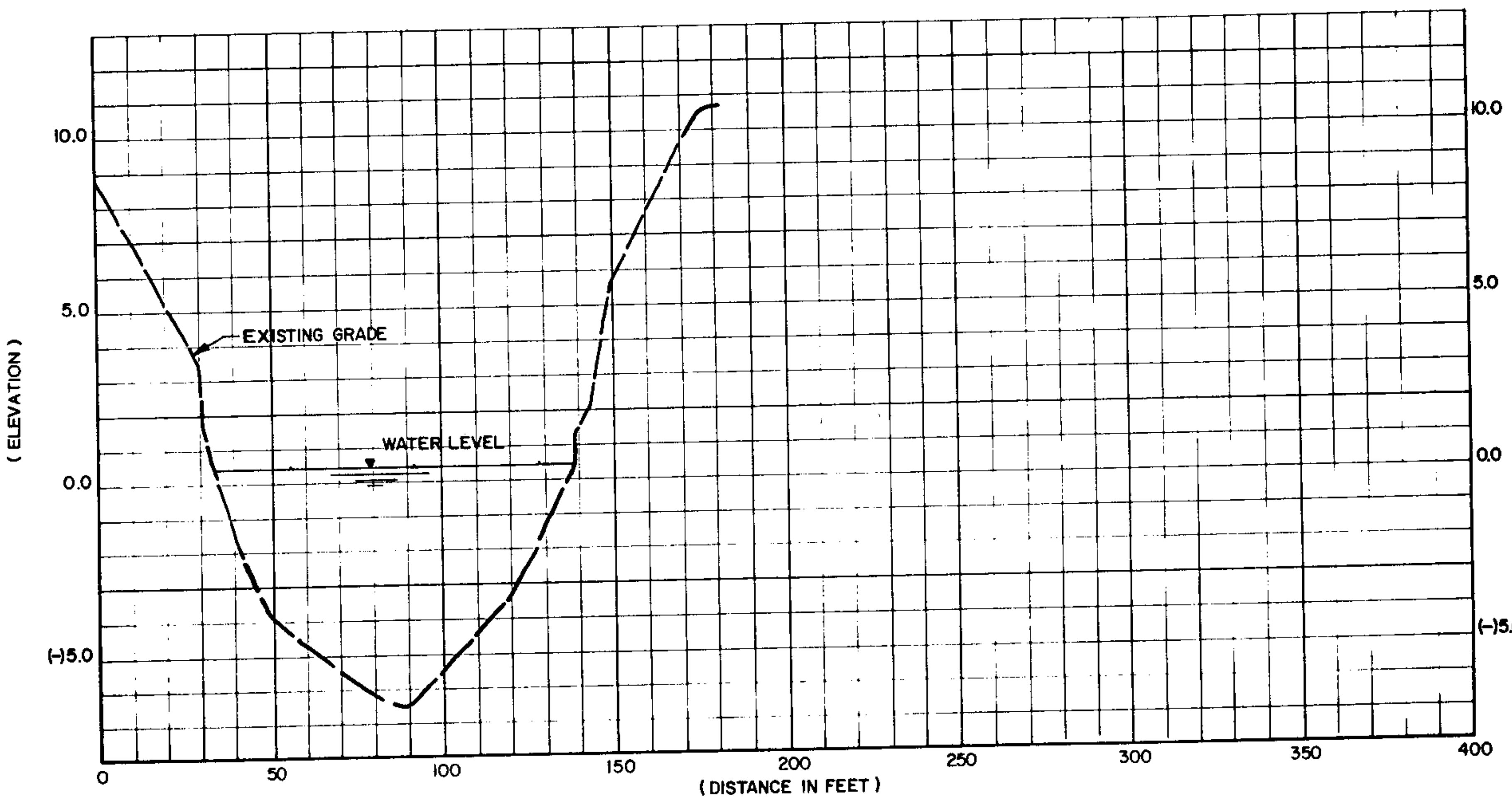
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-8

Station 15800

FIGURE P-8

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'
0 30'
WATER EL. : +0.4

**PHILLIPPI CREEK**

VIEW - FACING DOWNSTREAM

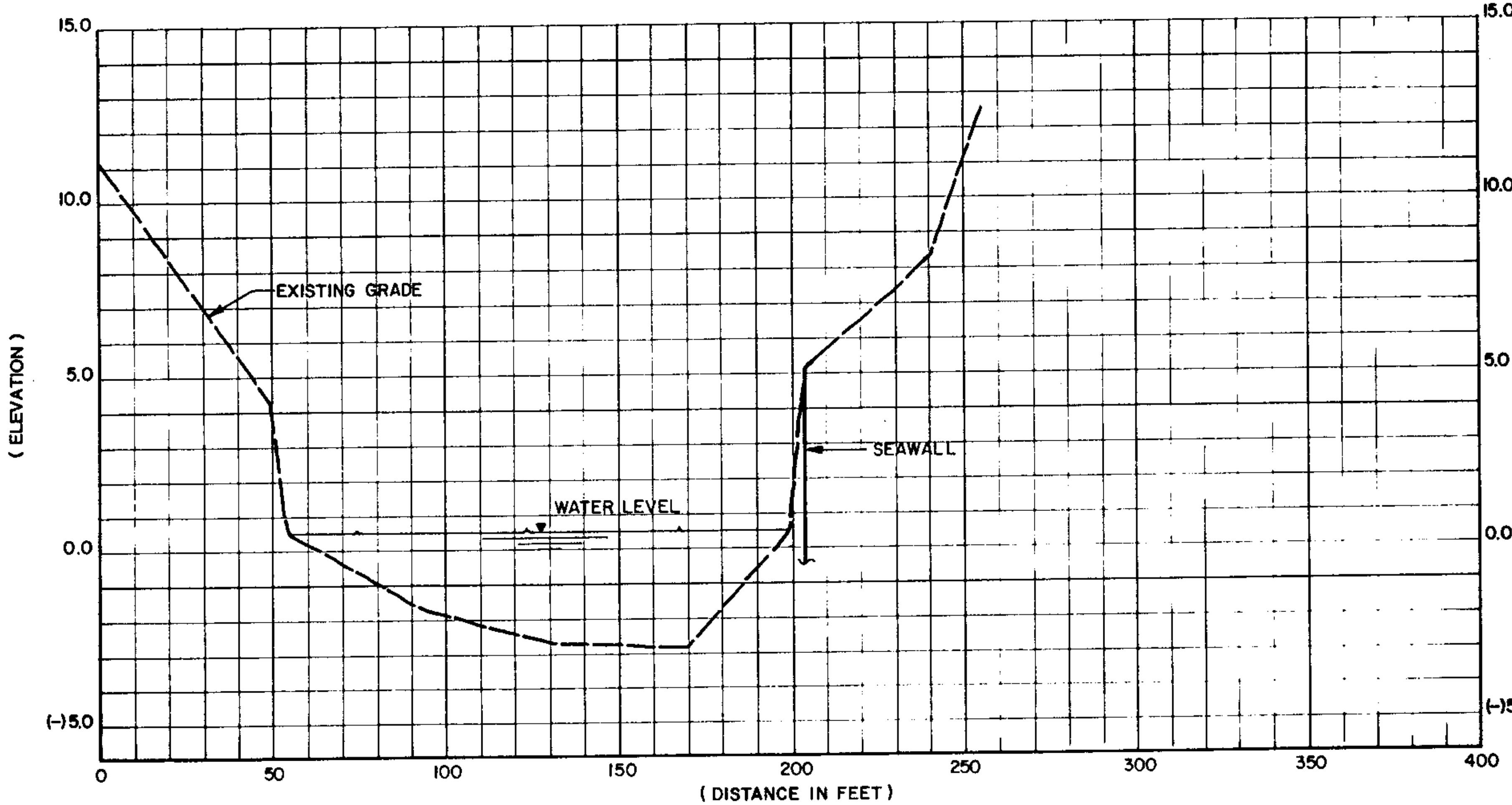
CROSS - SECTION
NO. P-9

Station 18500

FIGURE P-9

FIGURE P-9

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'
0
30'
WATER EL. : +0.5

PHILLIPPI CREEK

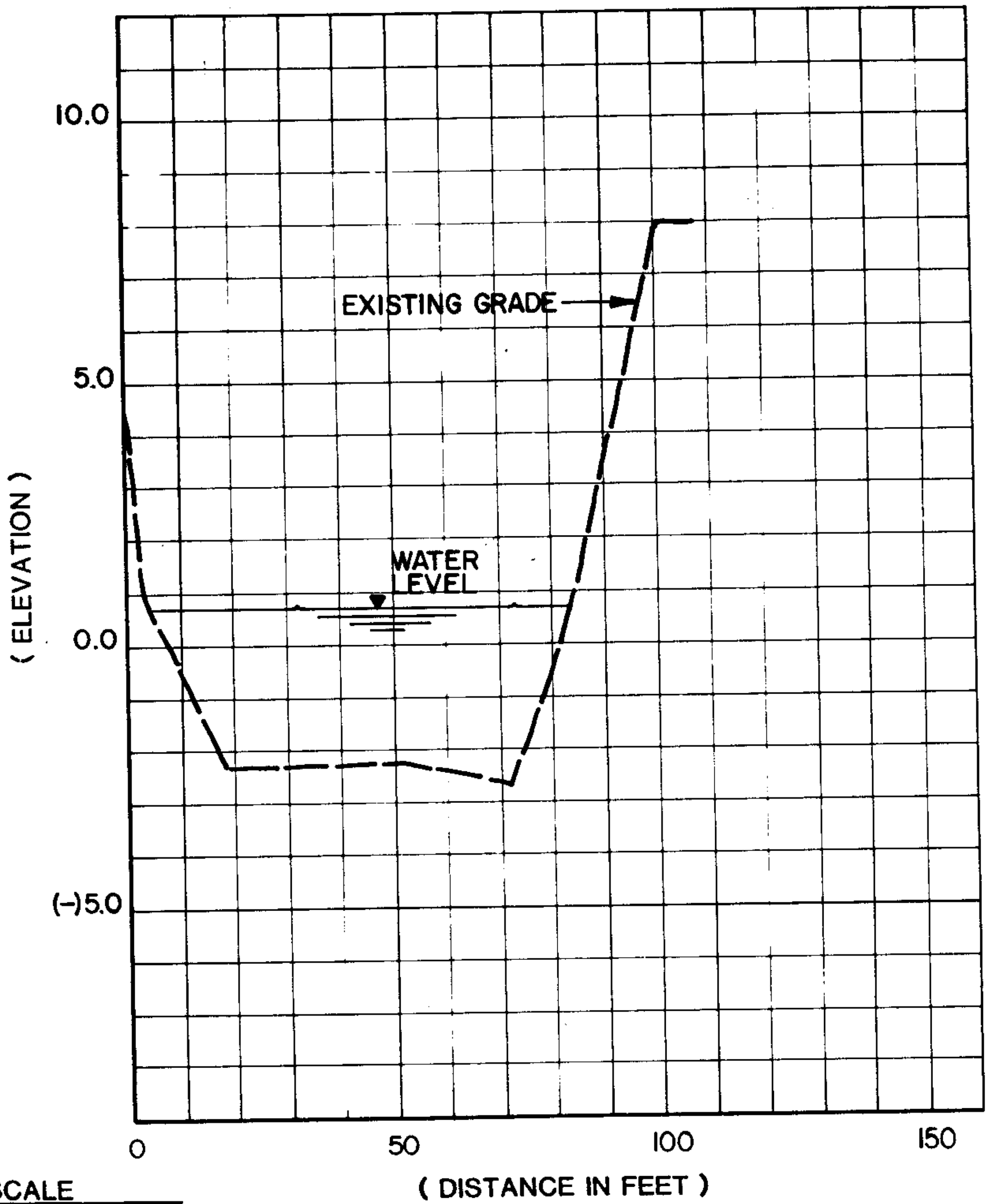
VIEW - FACING DOWNSTREAM

CROSS - SECTION
NO. P-10

FIGURE P-10

Station 21360

## SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

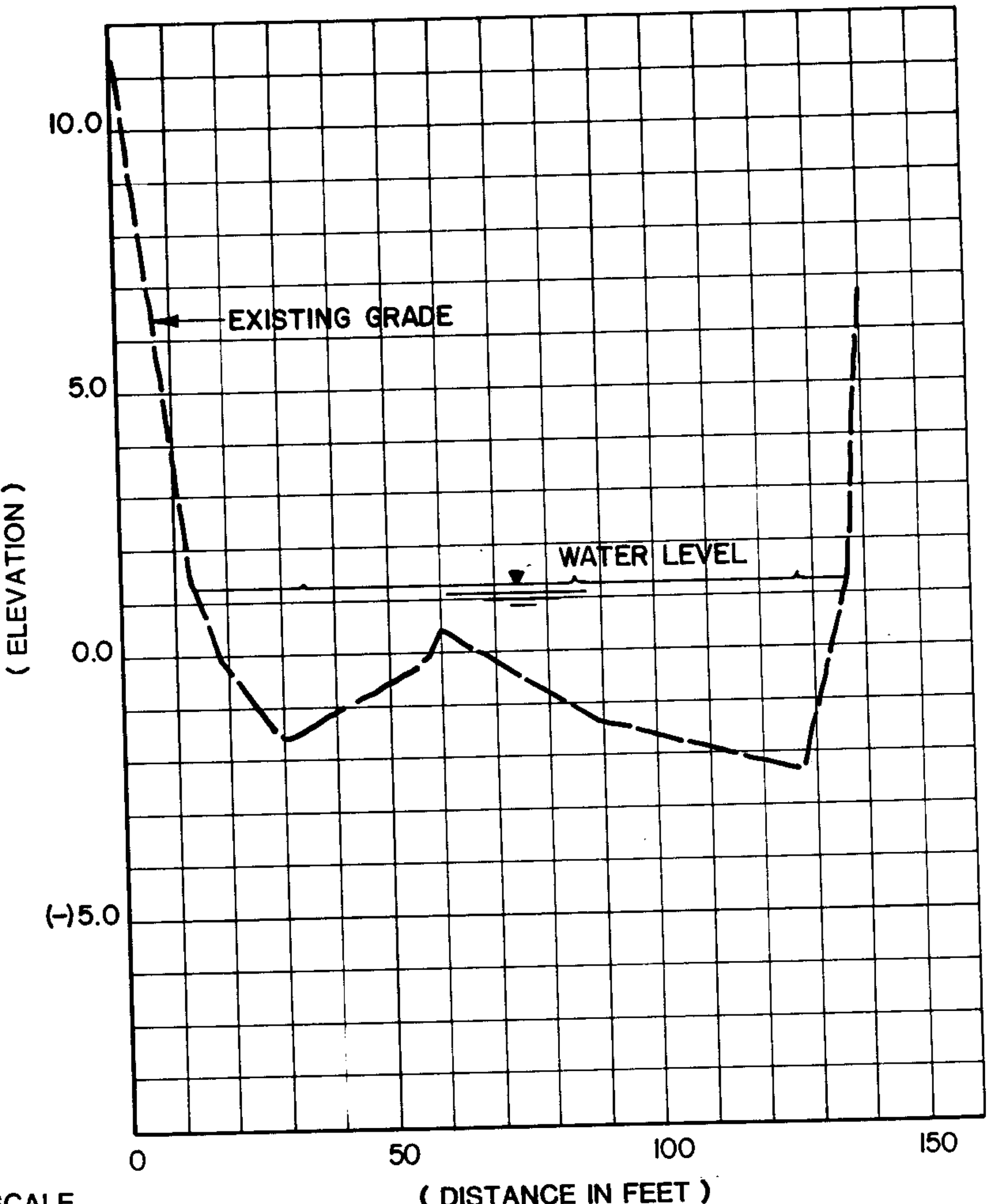
CROSS-SECTION
NO. P-II

WATER EL. :+0.7

Station , 23350

FIGURE P-11

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : +1.3

PHILLIPPI CREEK

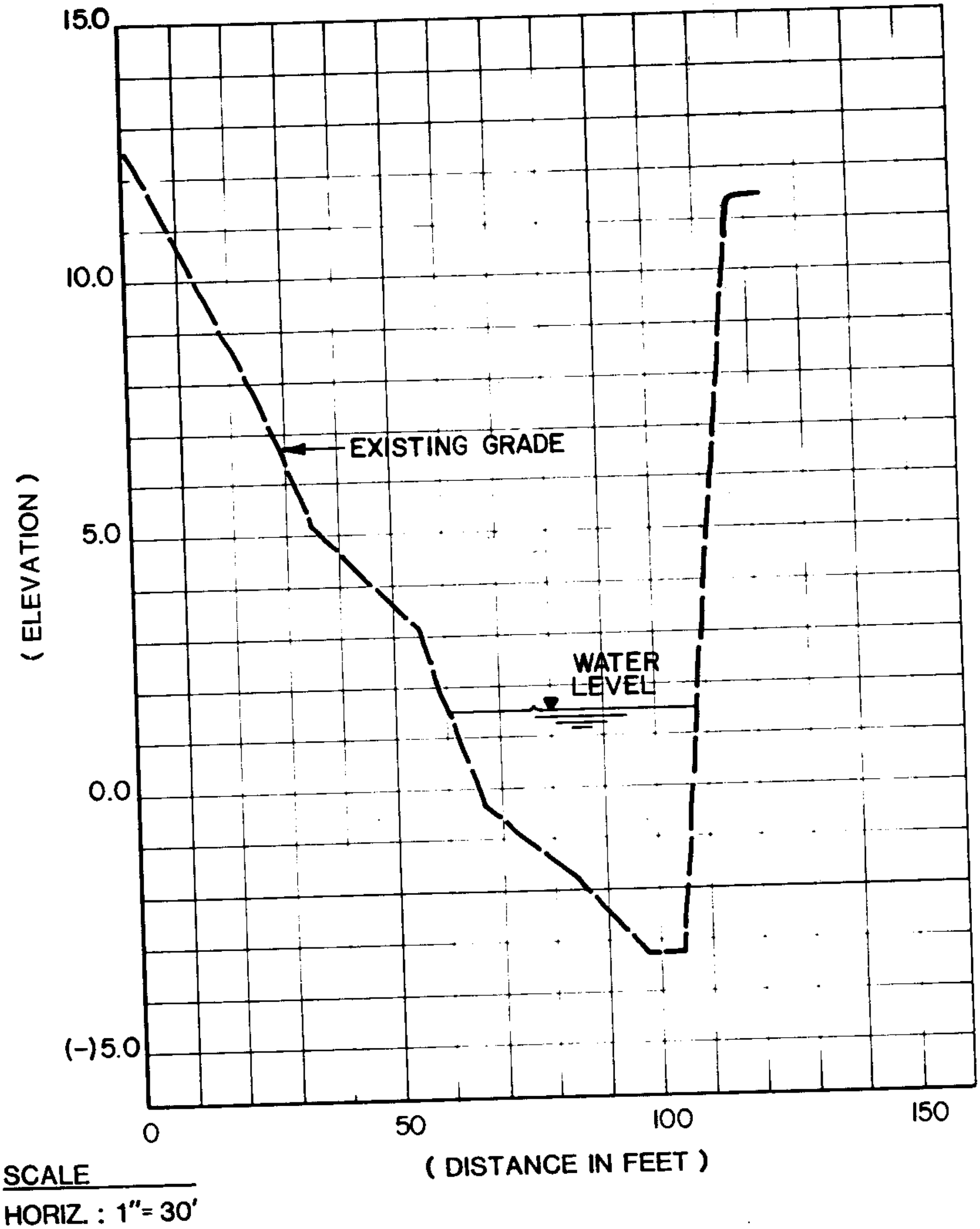
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-12

Station. 25050

FIGURE P-12

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. :+1.4

PHILLIPPI CREEK

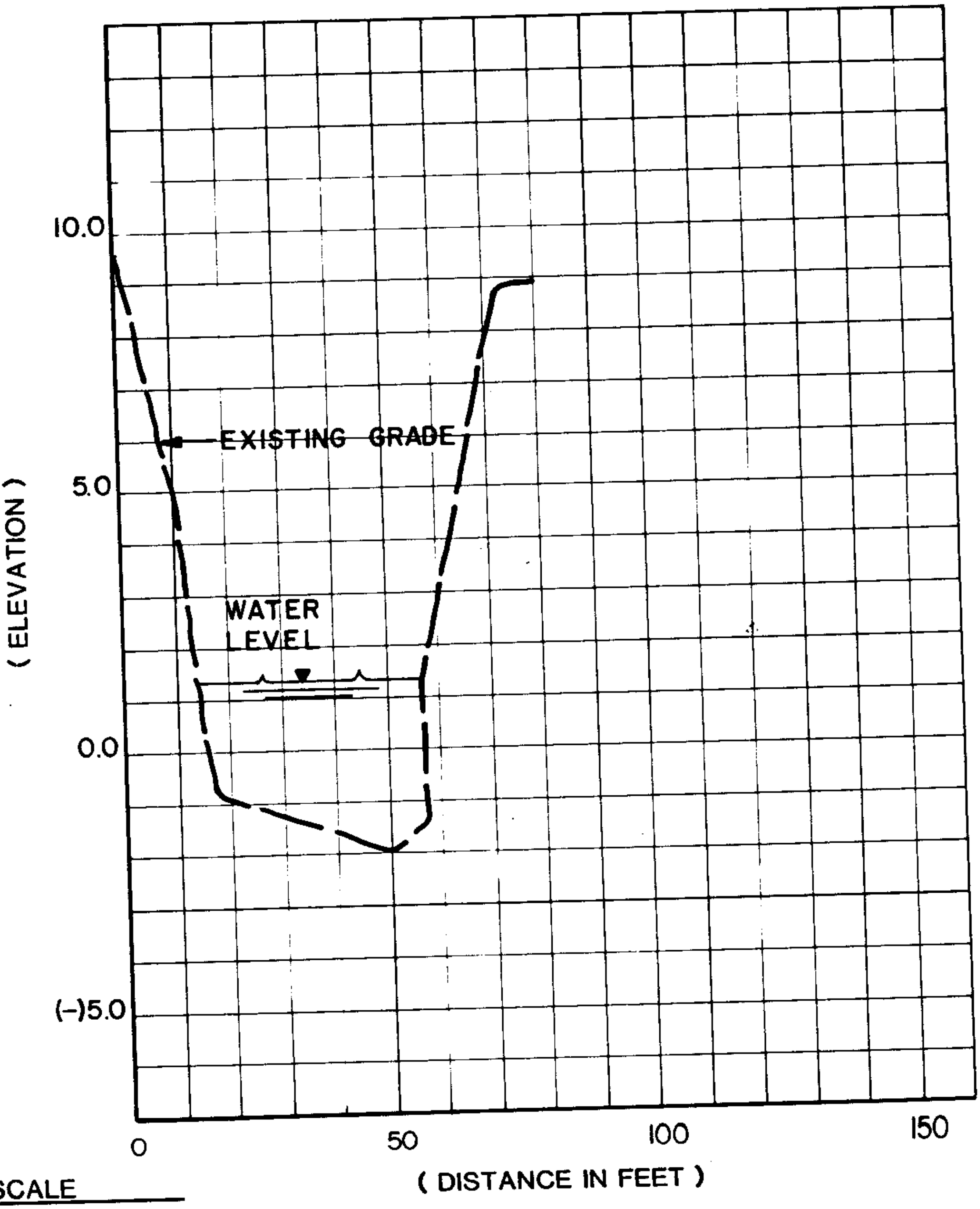
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-13

**Station 27650**

FIGURE P-13

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 1.3

## PHILLIPPI CREEK

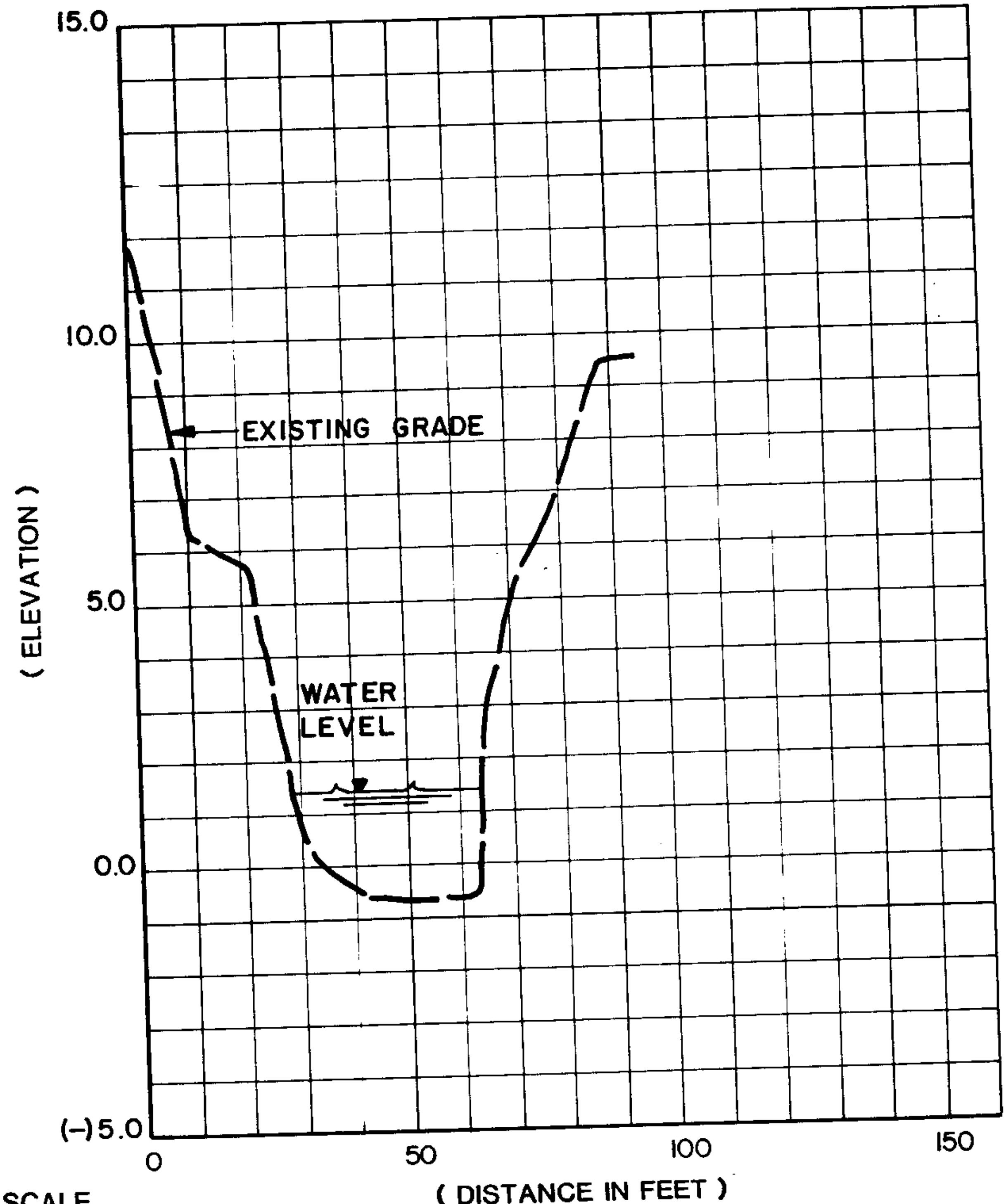
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P - 14

Station 29000

FIGURE P-14

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

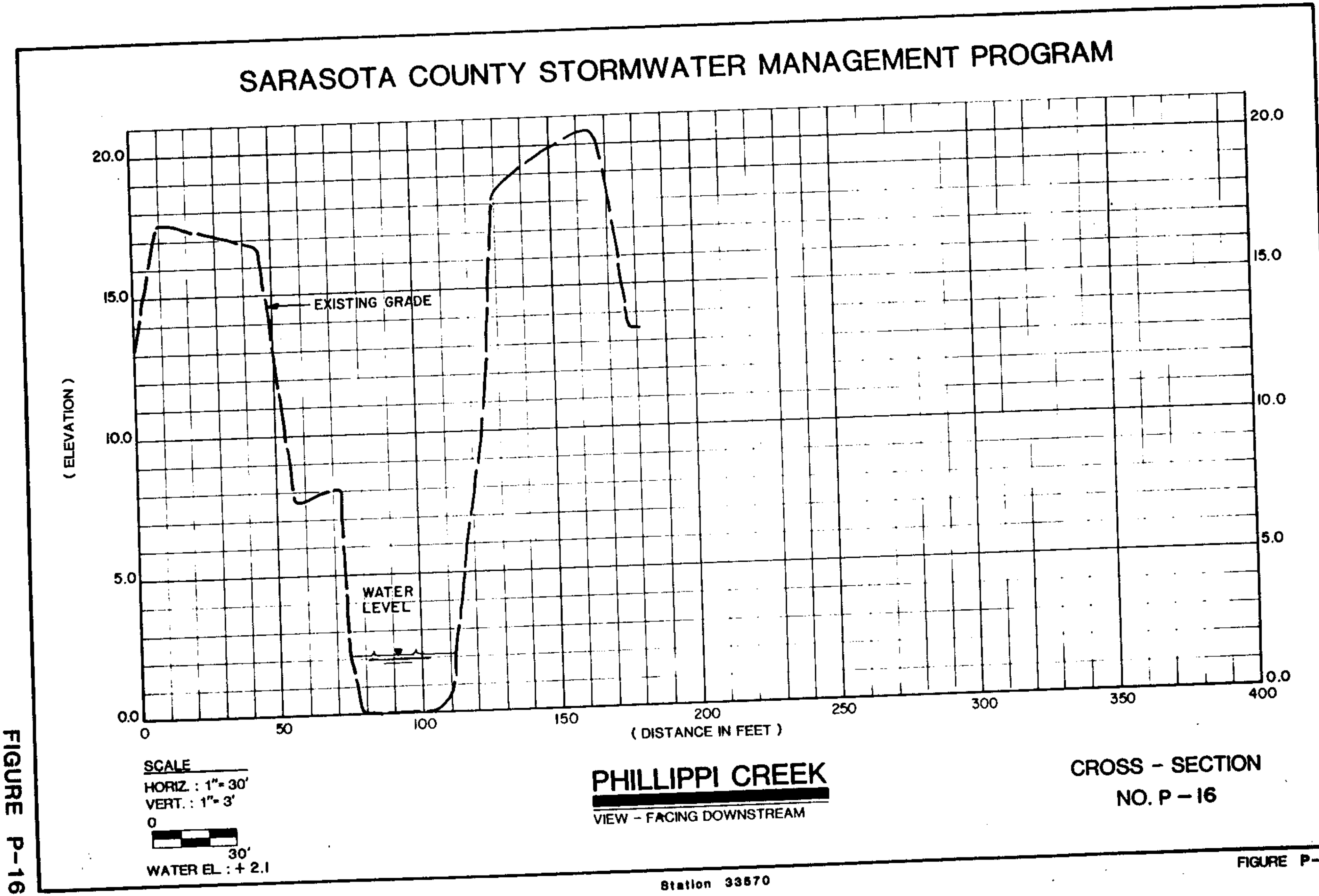
VIEW – FACING DOWN STREAM

CROSS-SECTION
NO. P – 15

WATER EL. : + 1. 4

Station 29950

FIGURE P-15

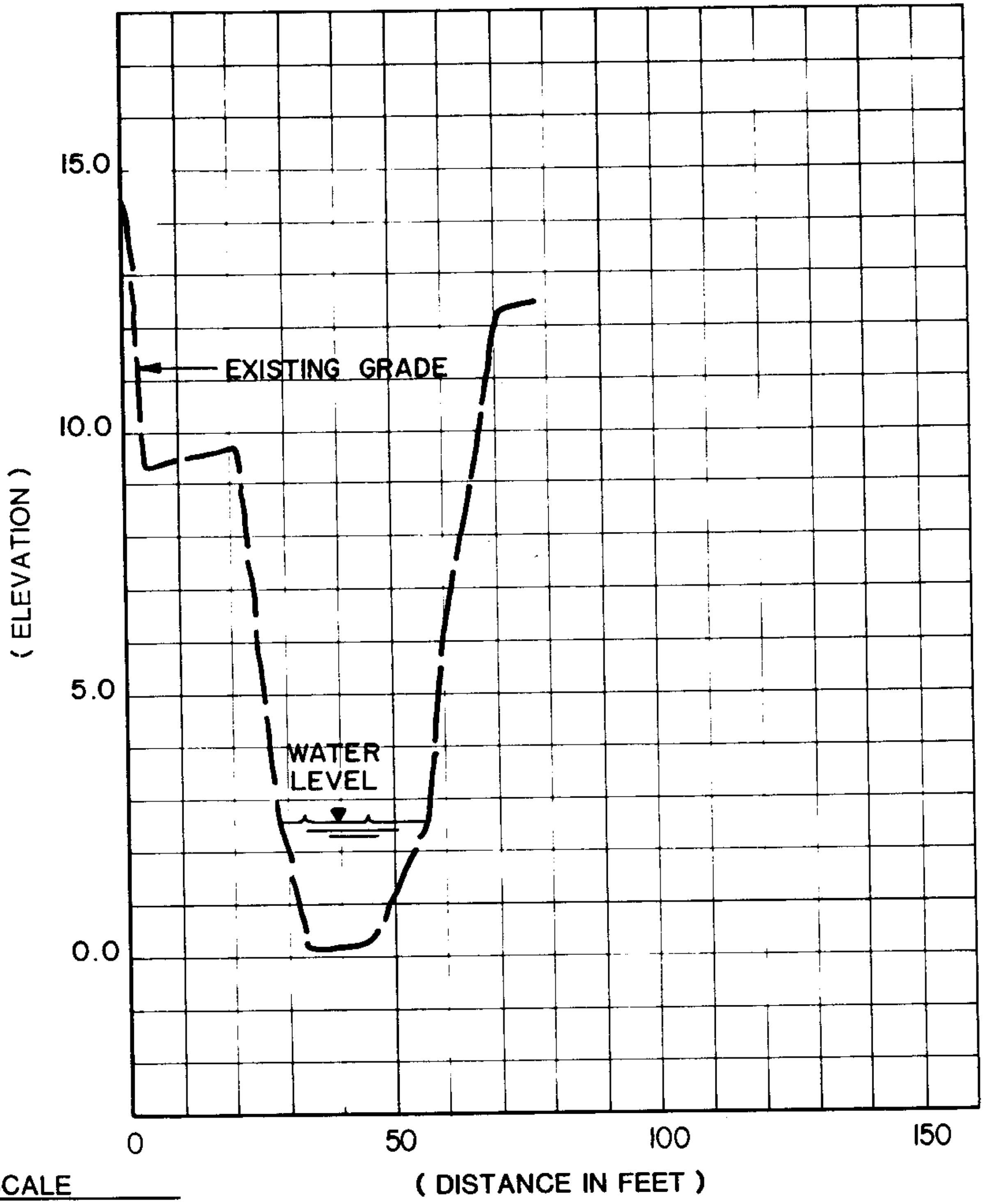



Station 33570

FIGURE P-16

FIGURE P-1

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 2.5

PHILLIPPI CREEK

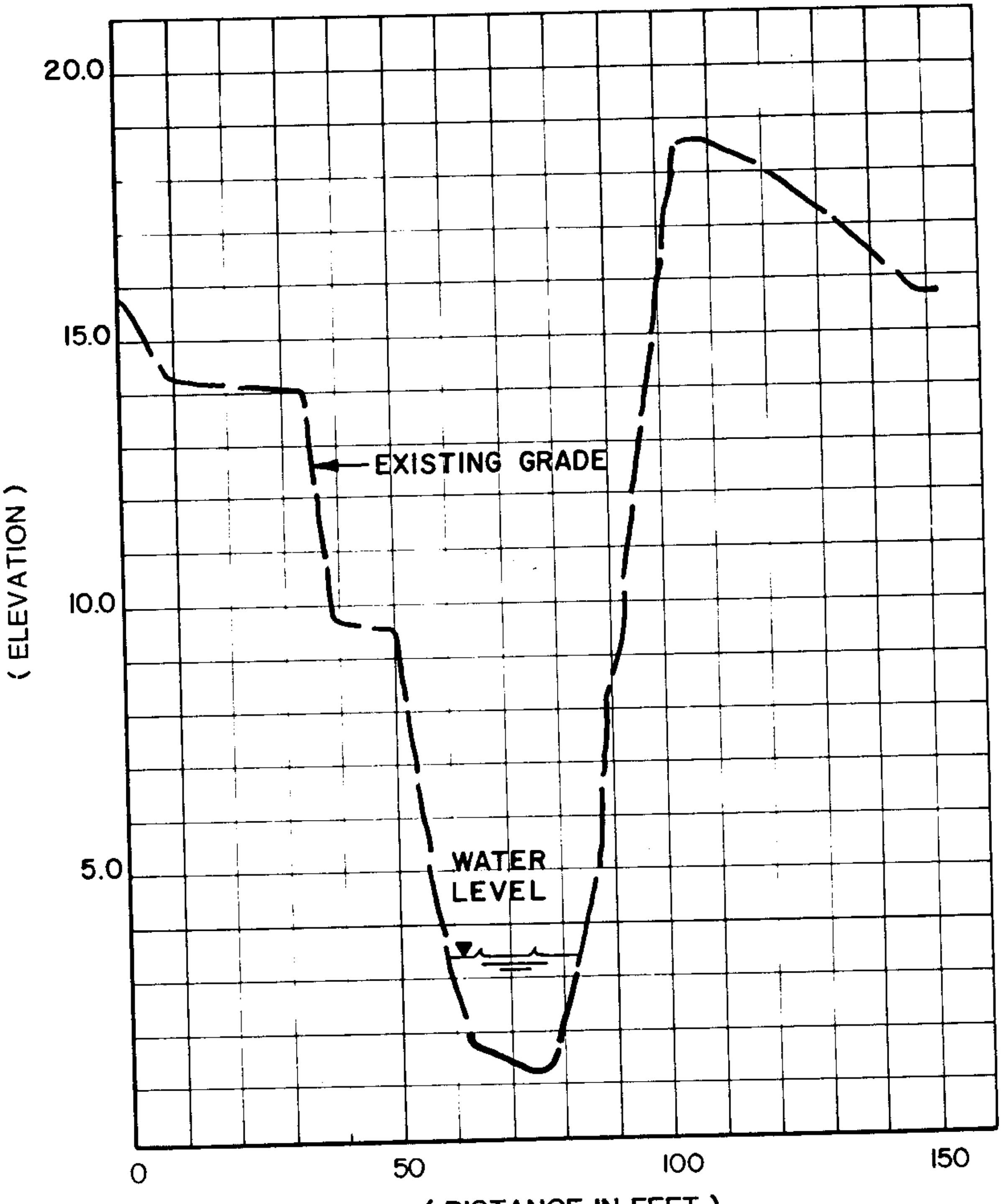
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-17

Station 35370

FIGURE P-17

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 3.4

PHILLIPPI CREEK

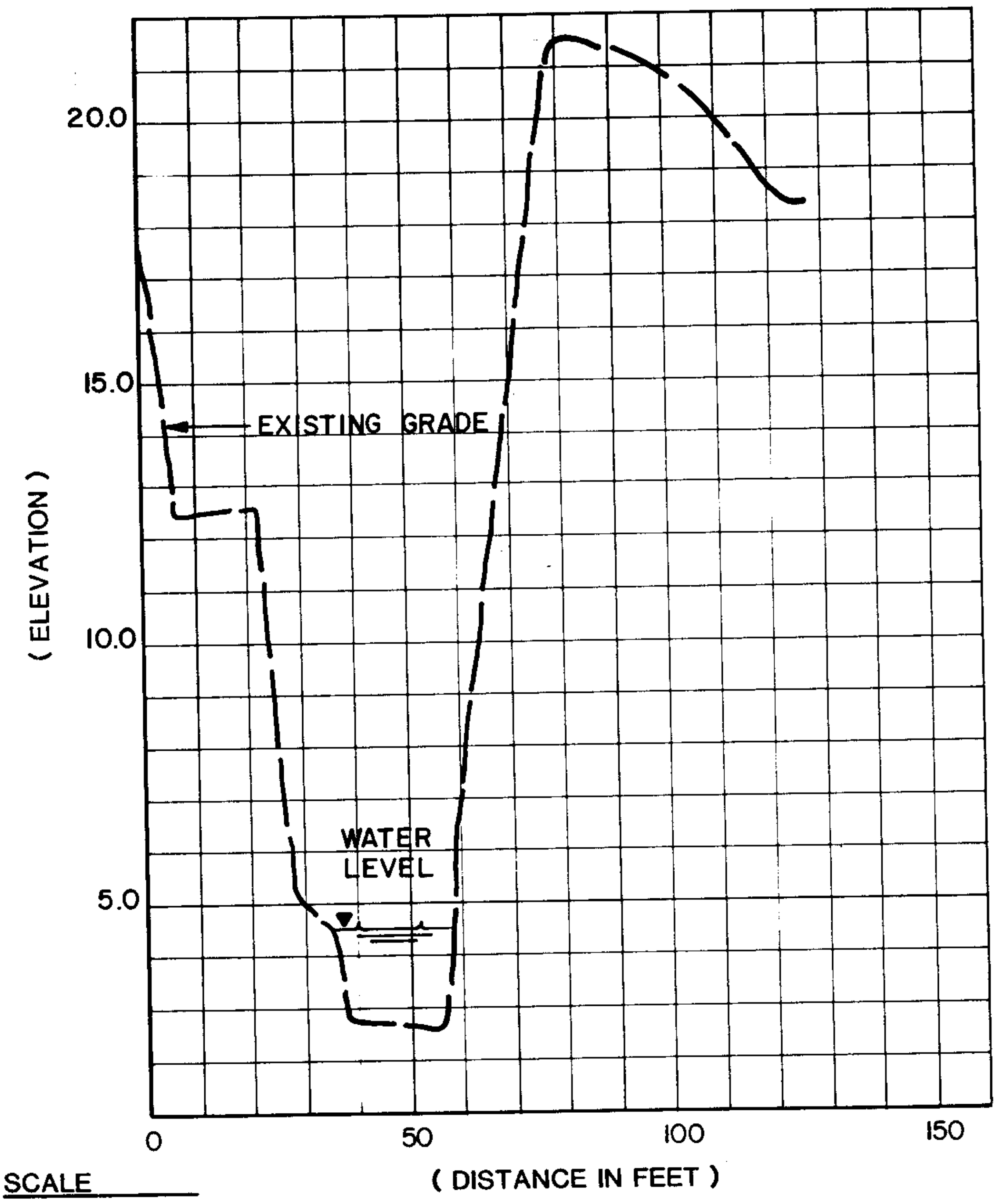
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-18

Station 37980

FIGURE P-18

## SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




**SCALE**

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 4.6

## PHILLIPPI CREEK

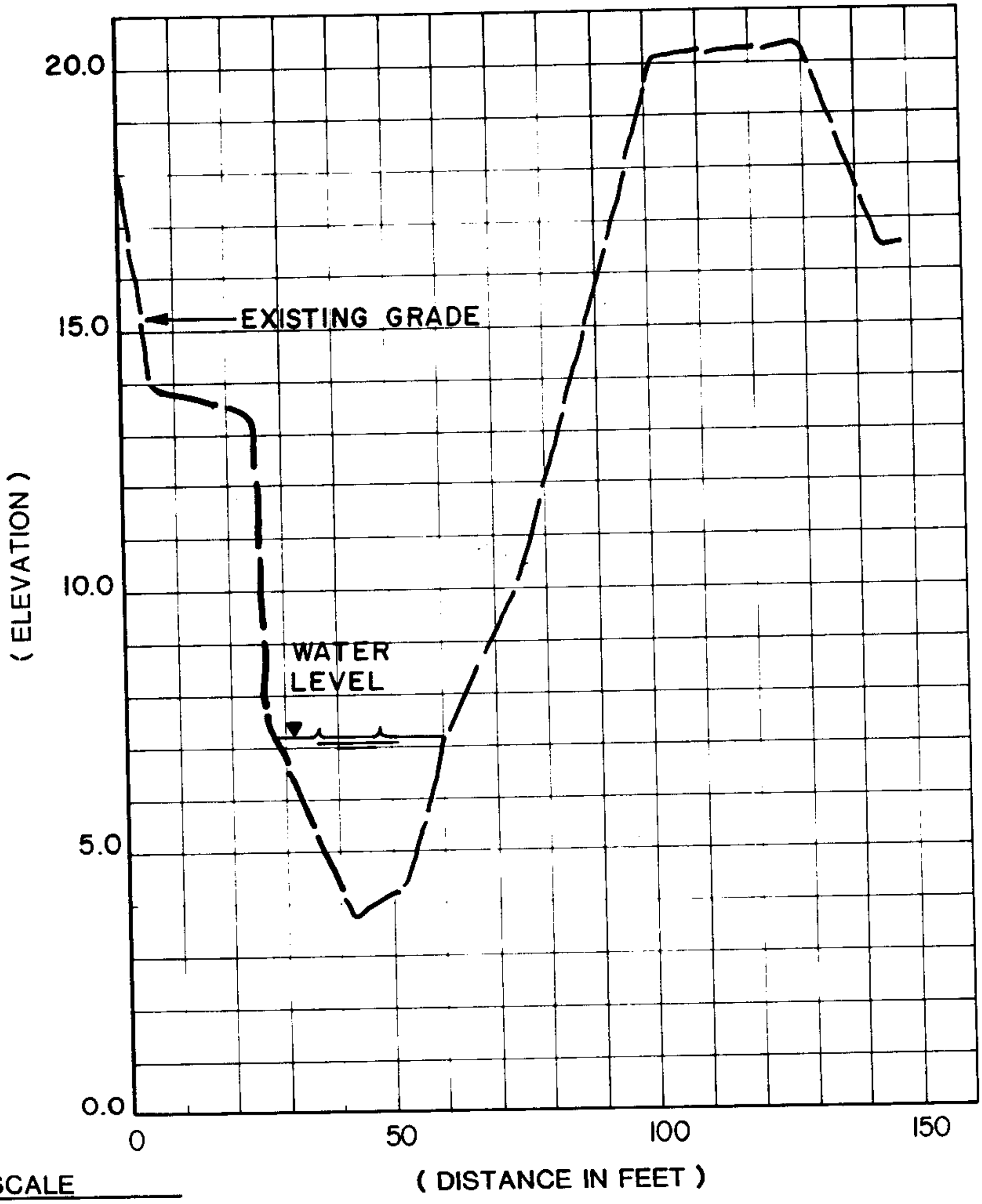
VIEW – FACING DOWN STREAM

CROSS–SECTION
NO. P–19

**Station 40470**

FIGURE P–19

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

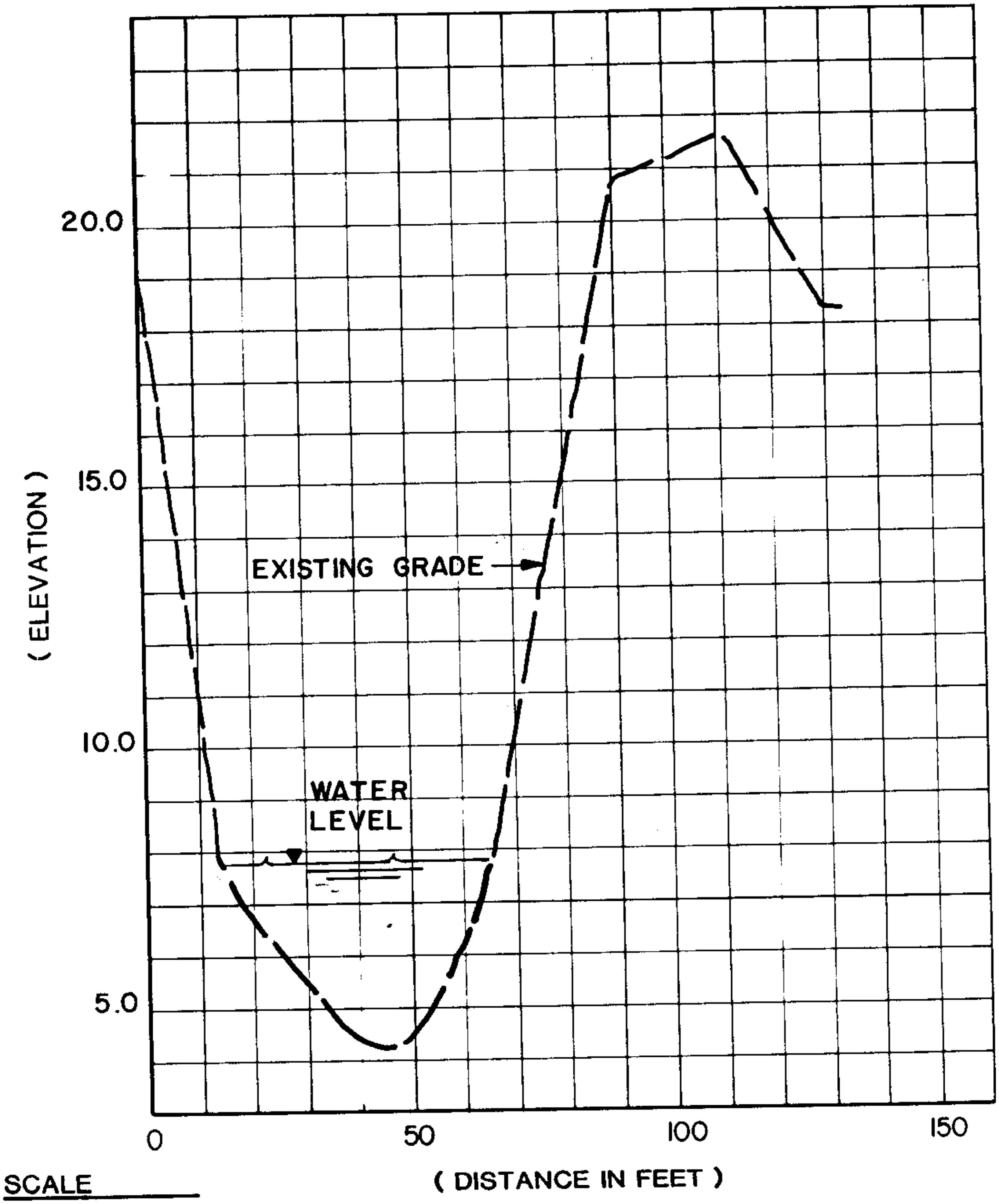
CROSS-SECTION
NO. P-20

WATER EL. : + 7.2

Station 43070

FIGURE P-20

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : +7.8

**PHILLIPPI CREEK**

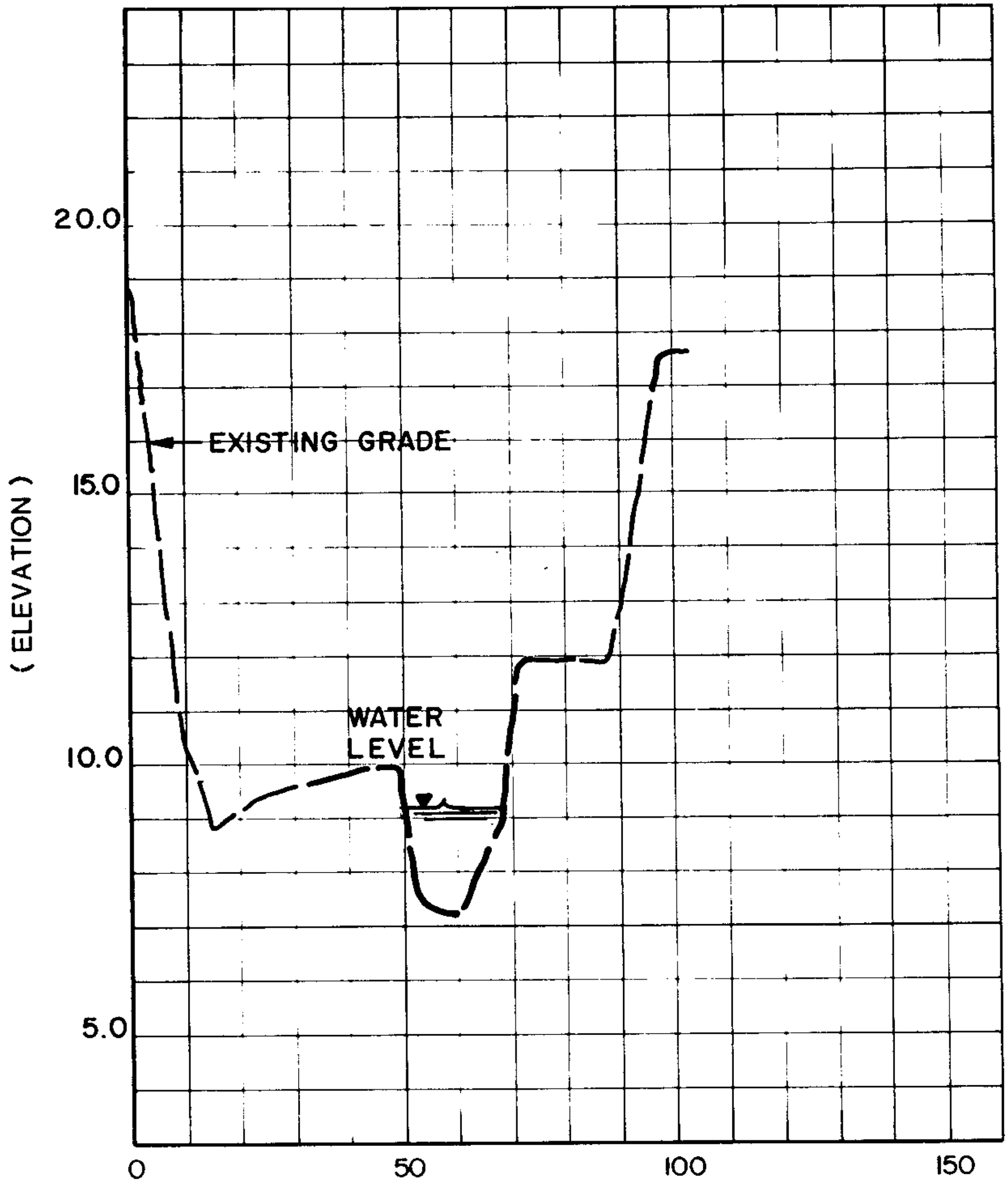
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P - 21

**Station 44220**

FIGURE **P-21**

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

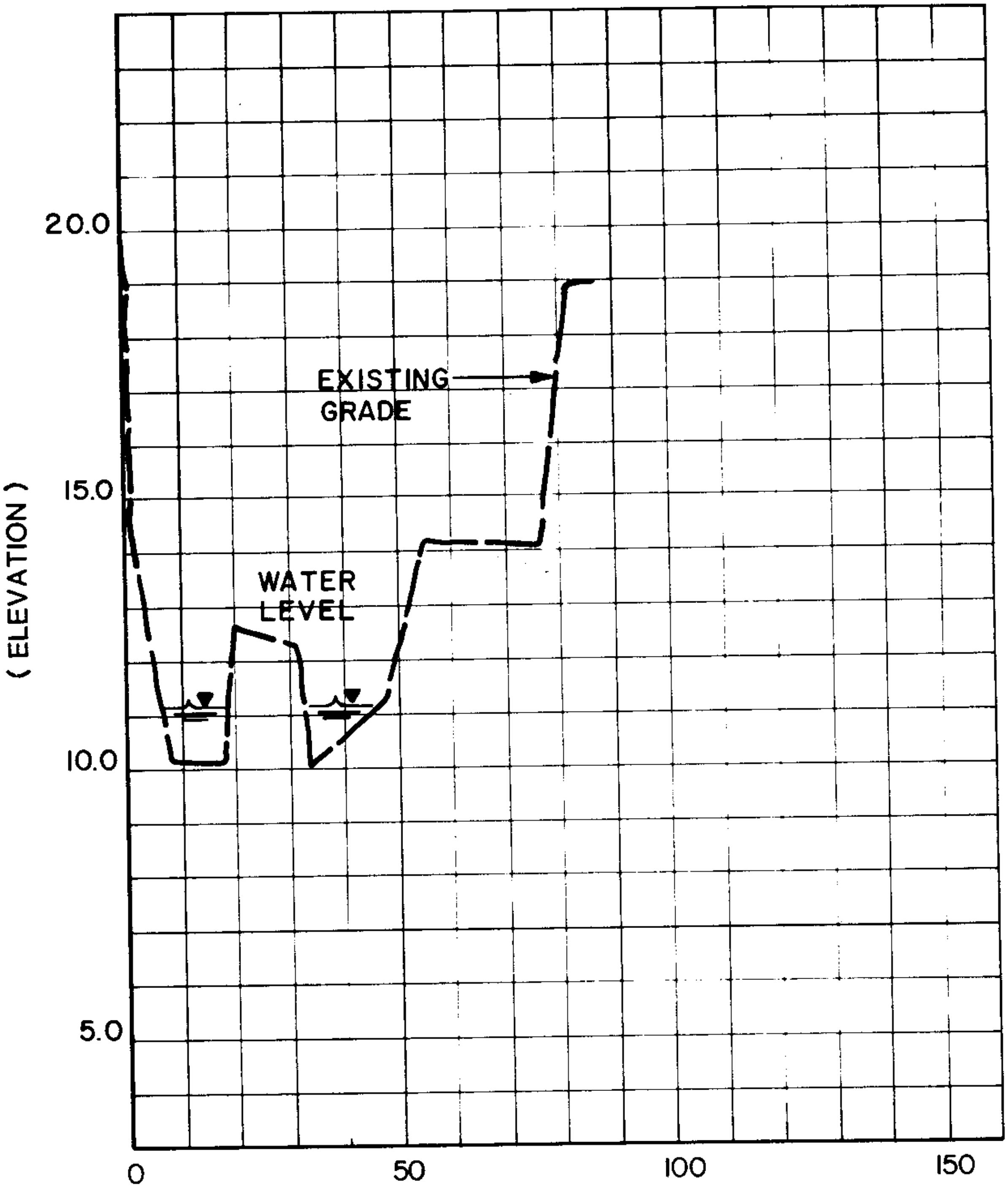
CROSS-SECTION
NO. P-22

WATER EL : + 9.3

Station 45520

FIGURE P-22

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

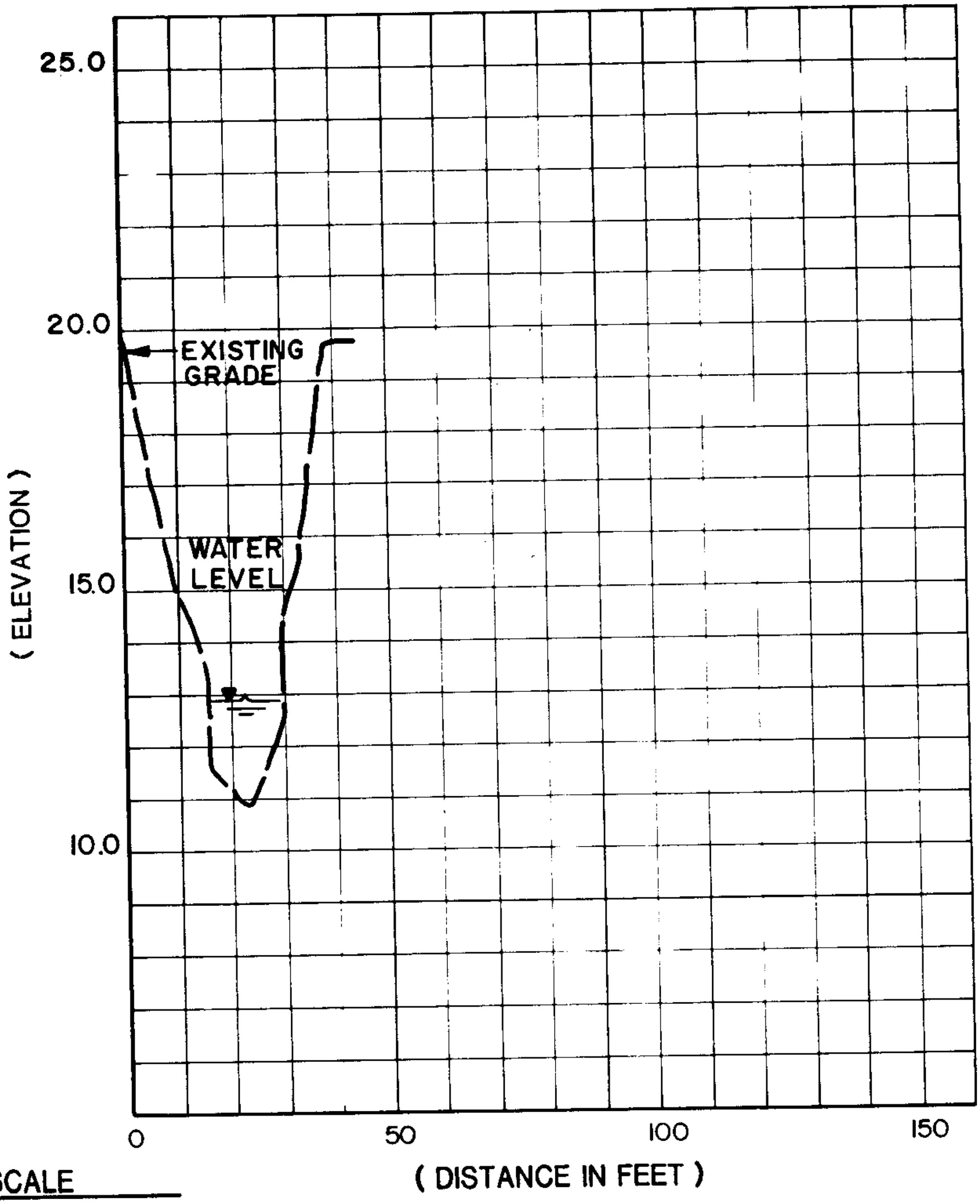
CROSS-SECTION
NO. P-23

WATER EL. :+ 11.2

Station . 47070

FIGURE P-23

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

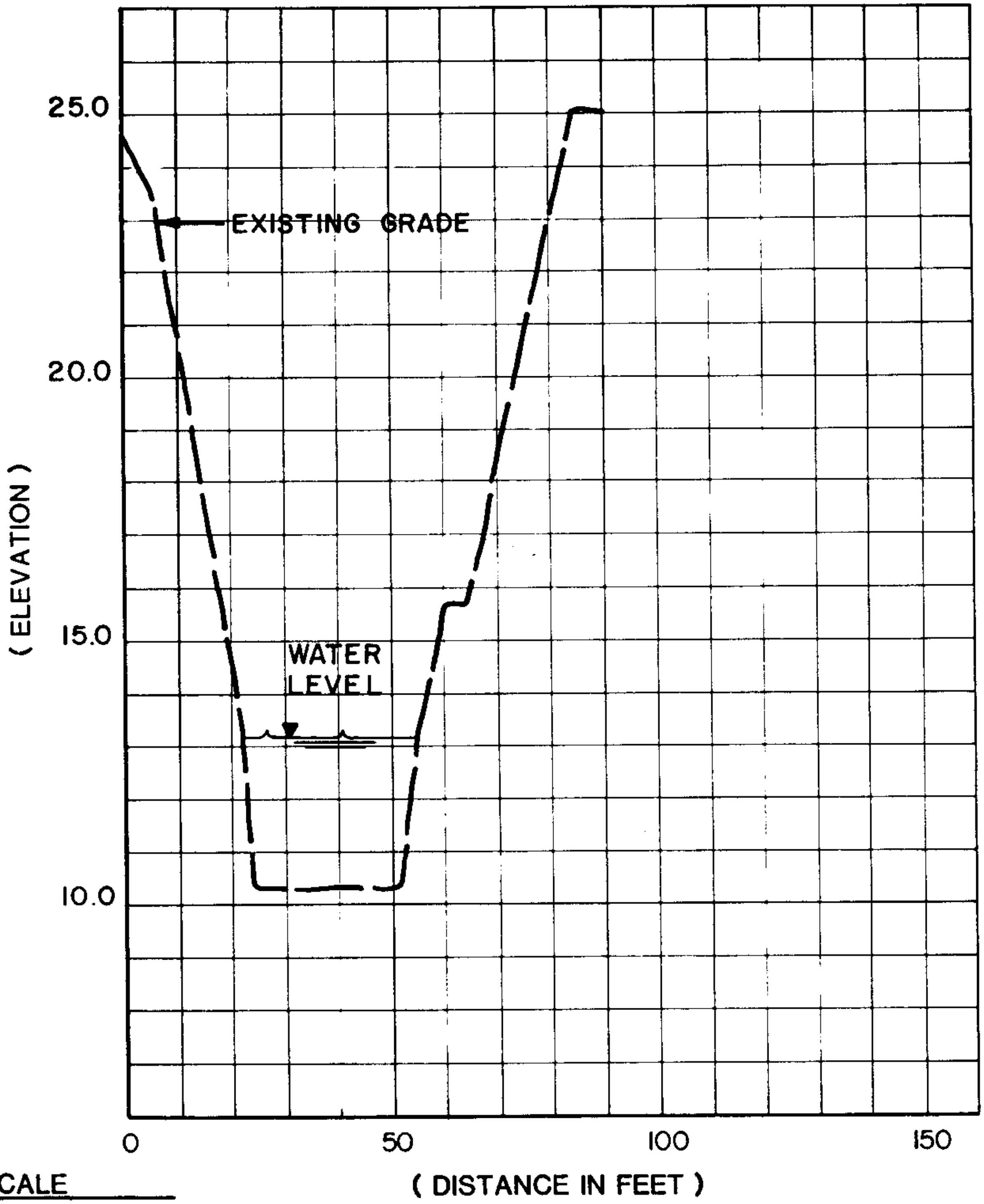
CROSS-SECTION
NO. P-24

WATER EL. : + 12.9

Station 48570

FIGURE P-24

## SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. :+ 13.2

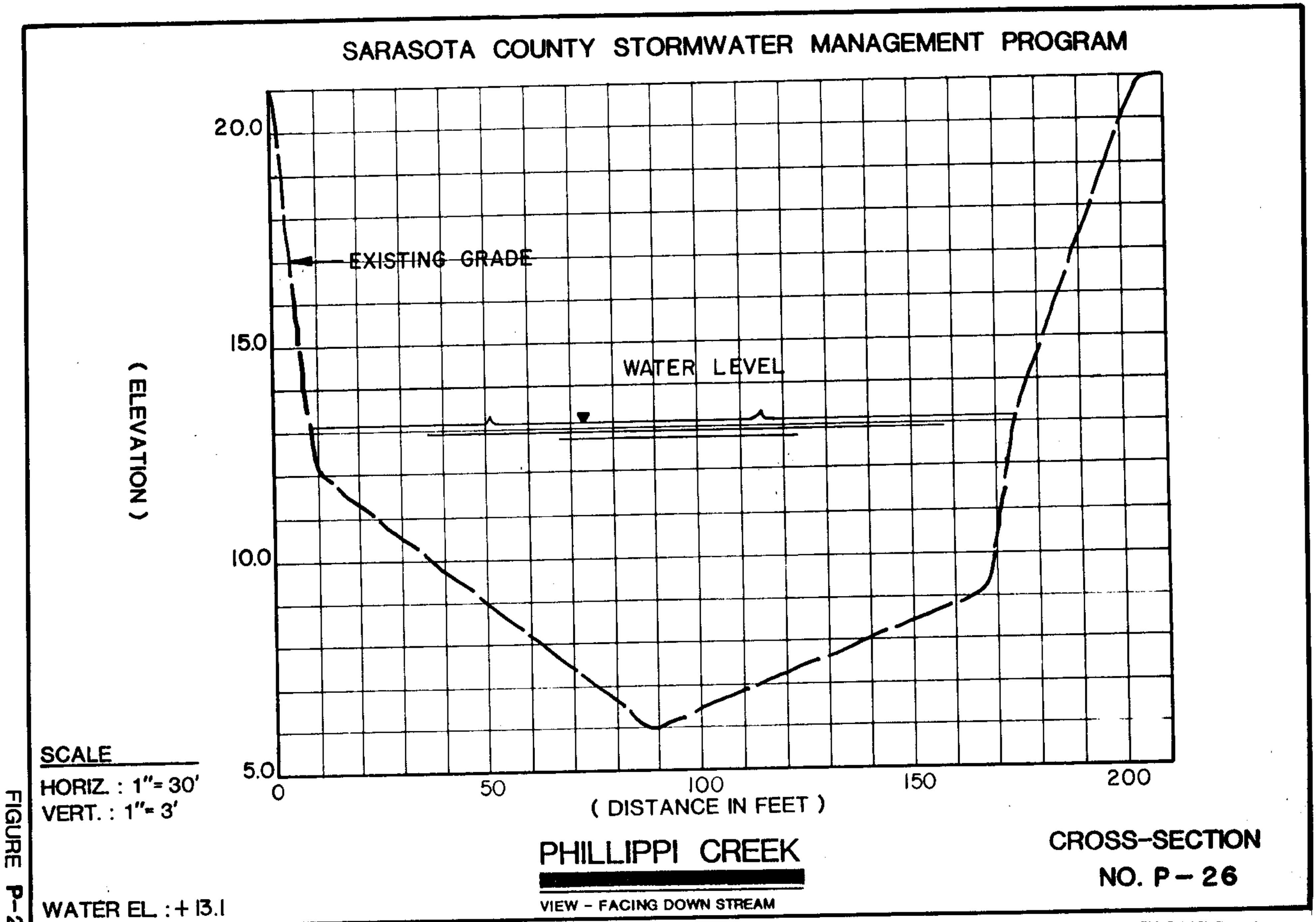
# PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P - 25

Station. 51170

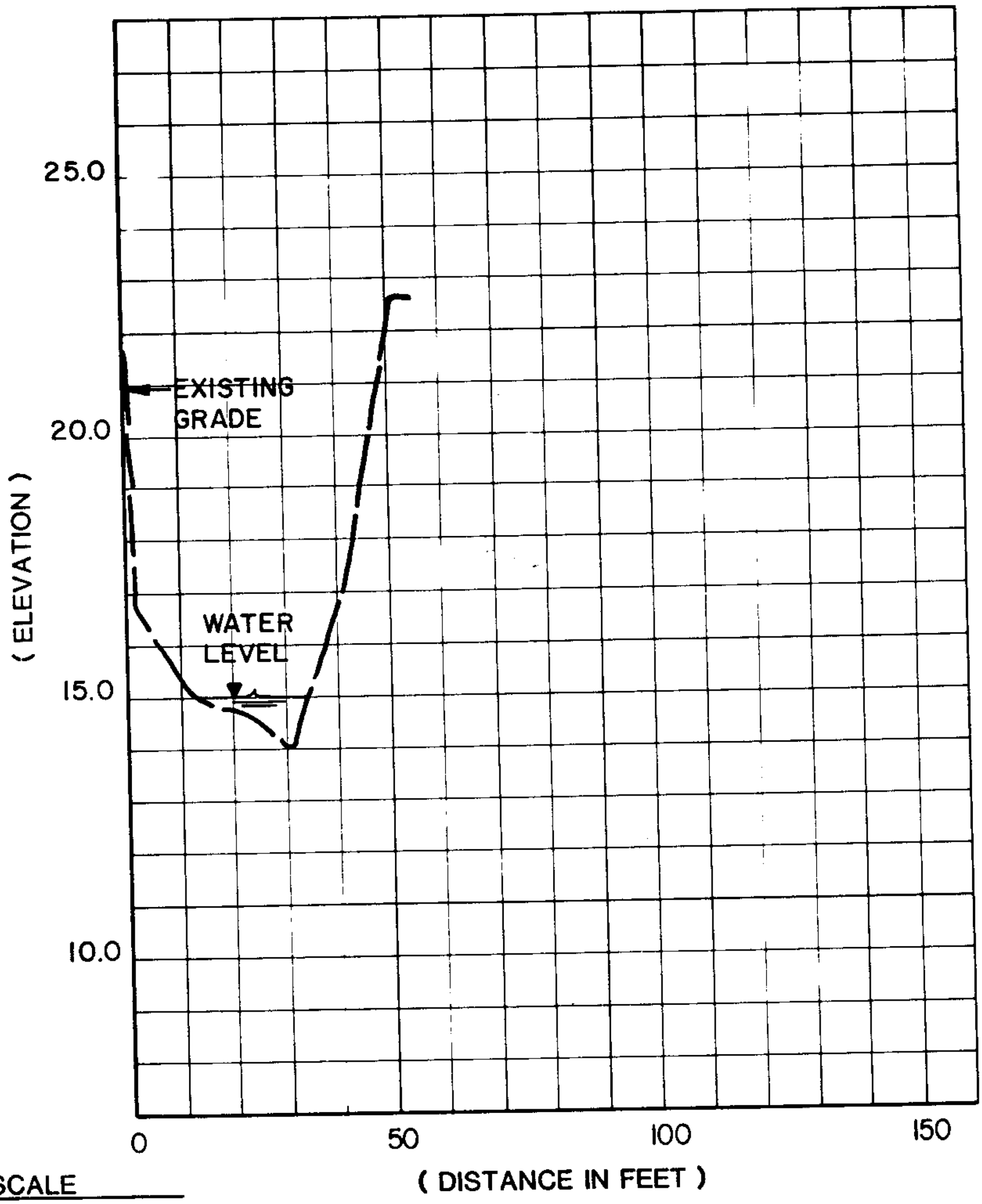
FIGURE P-25




Station 52570

FIGURE P-26

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

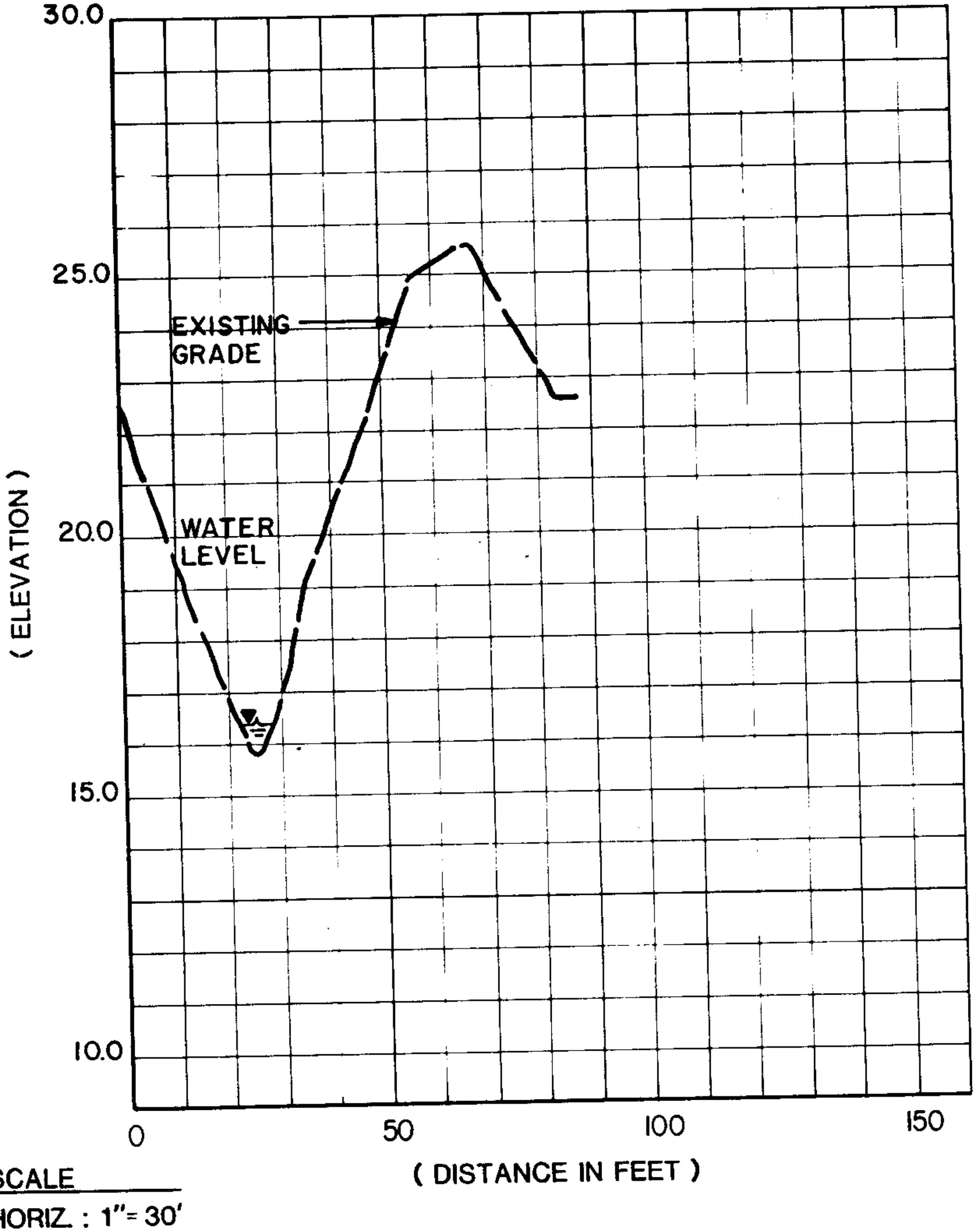
CROSS-SECTION
NO. P-27

WATER EL. : + 15.0

Station 53420

FIGURE P-27

# SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 16.4

PHILLIPPI CREEK

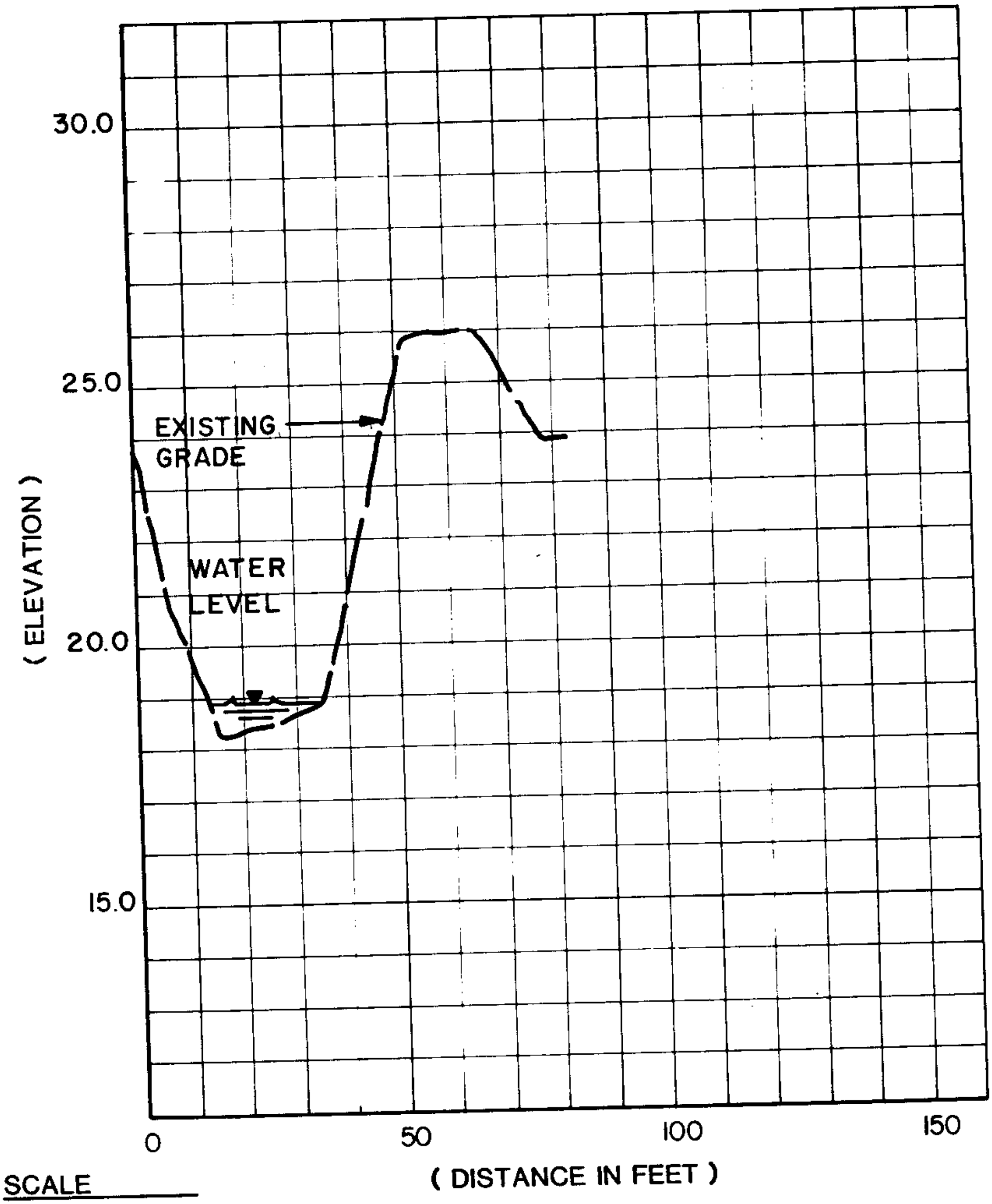
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-28

Station 54740

FIGURE P-28

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

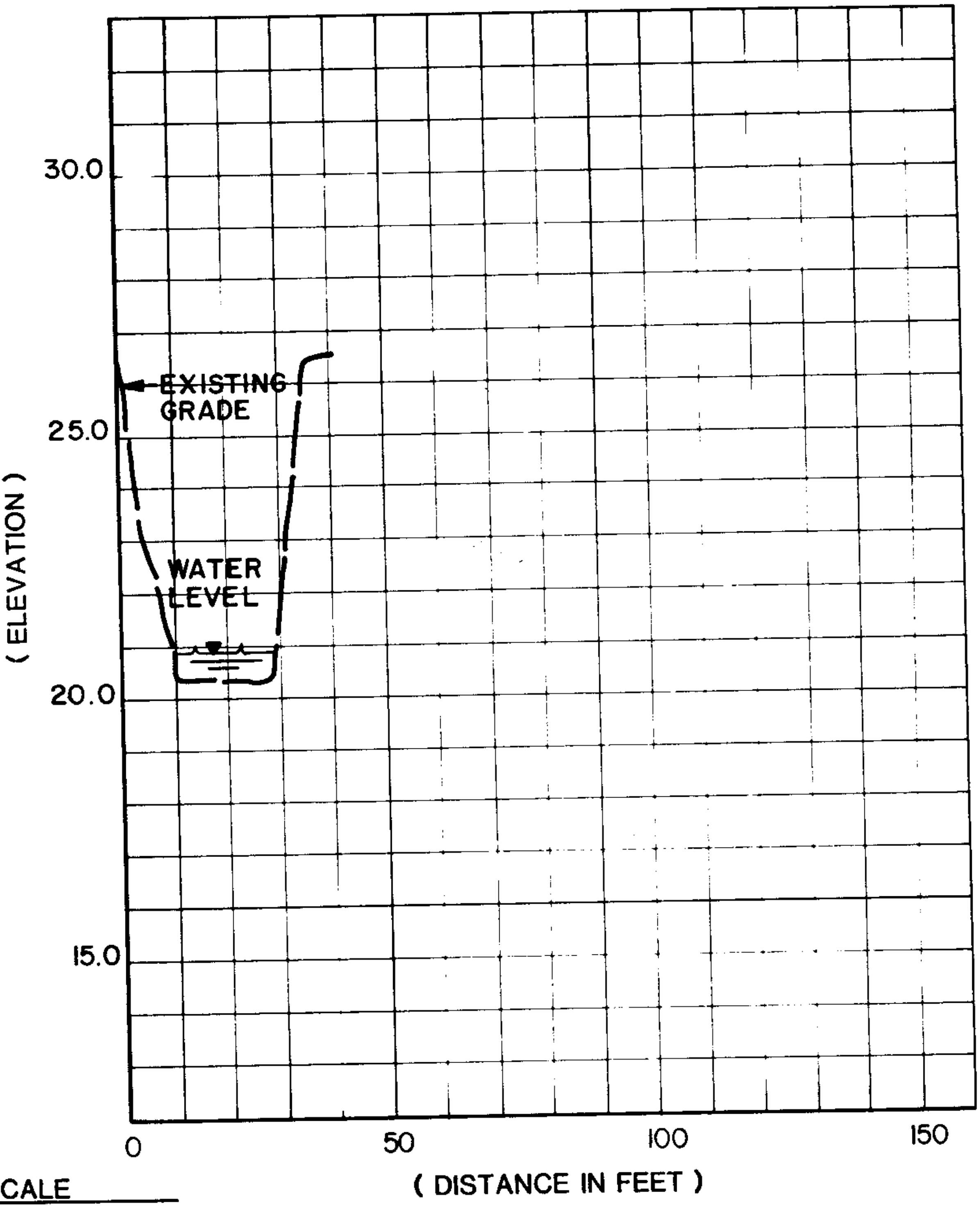
CROSS-SECTION
NO. P-29

WATER EL. : + 18.9

Station 56140

FIGURE P-29

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE
HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : +20.9

PHILLIPPI CREEK

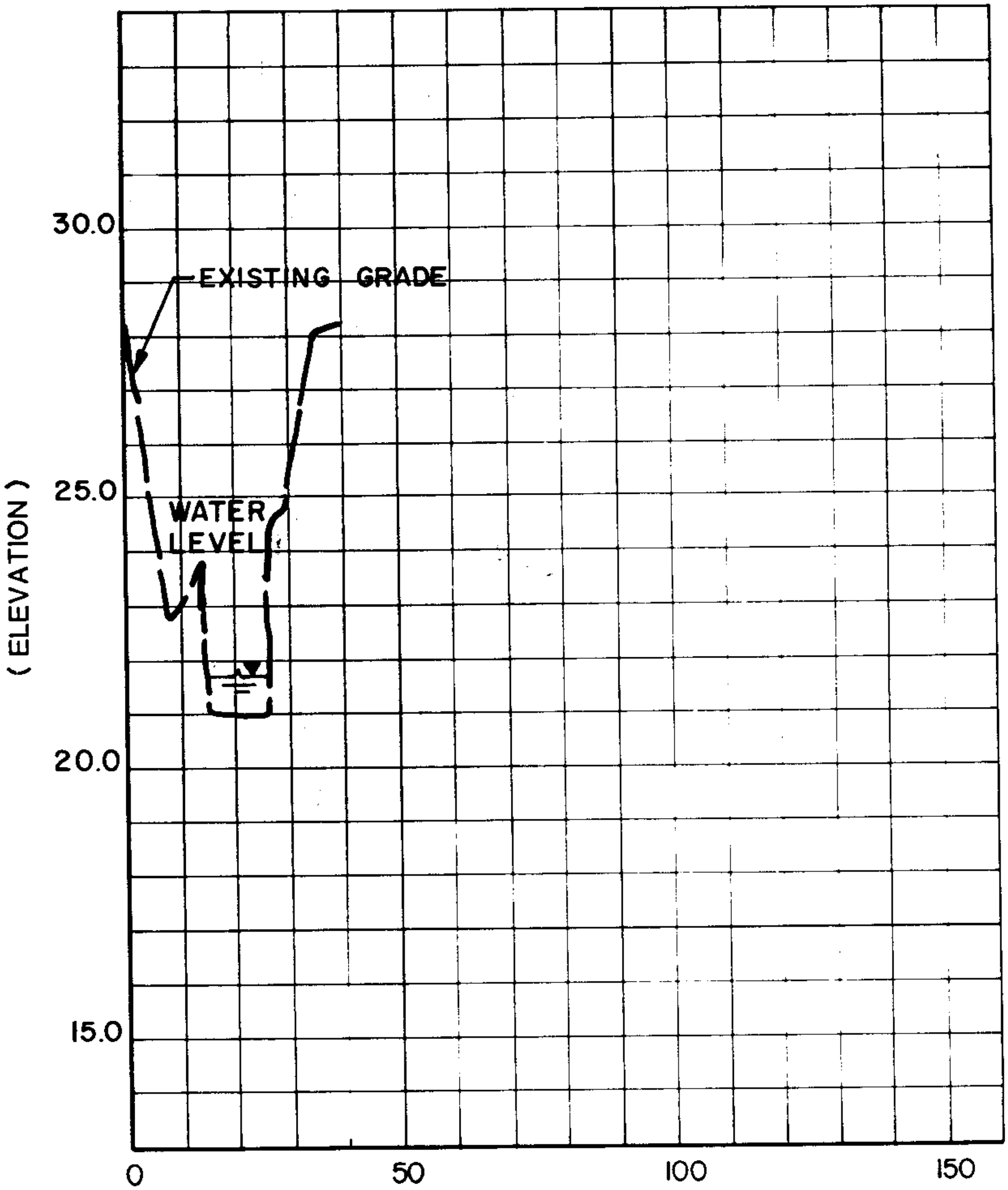
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P - 30

Station 57540

FIGURE P-30

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'
VERT. : 1"= 3'

WATER EL. : + 21.7

PHILLIPPI CREEK

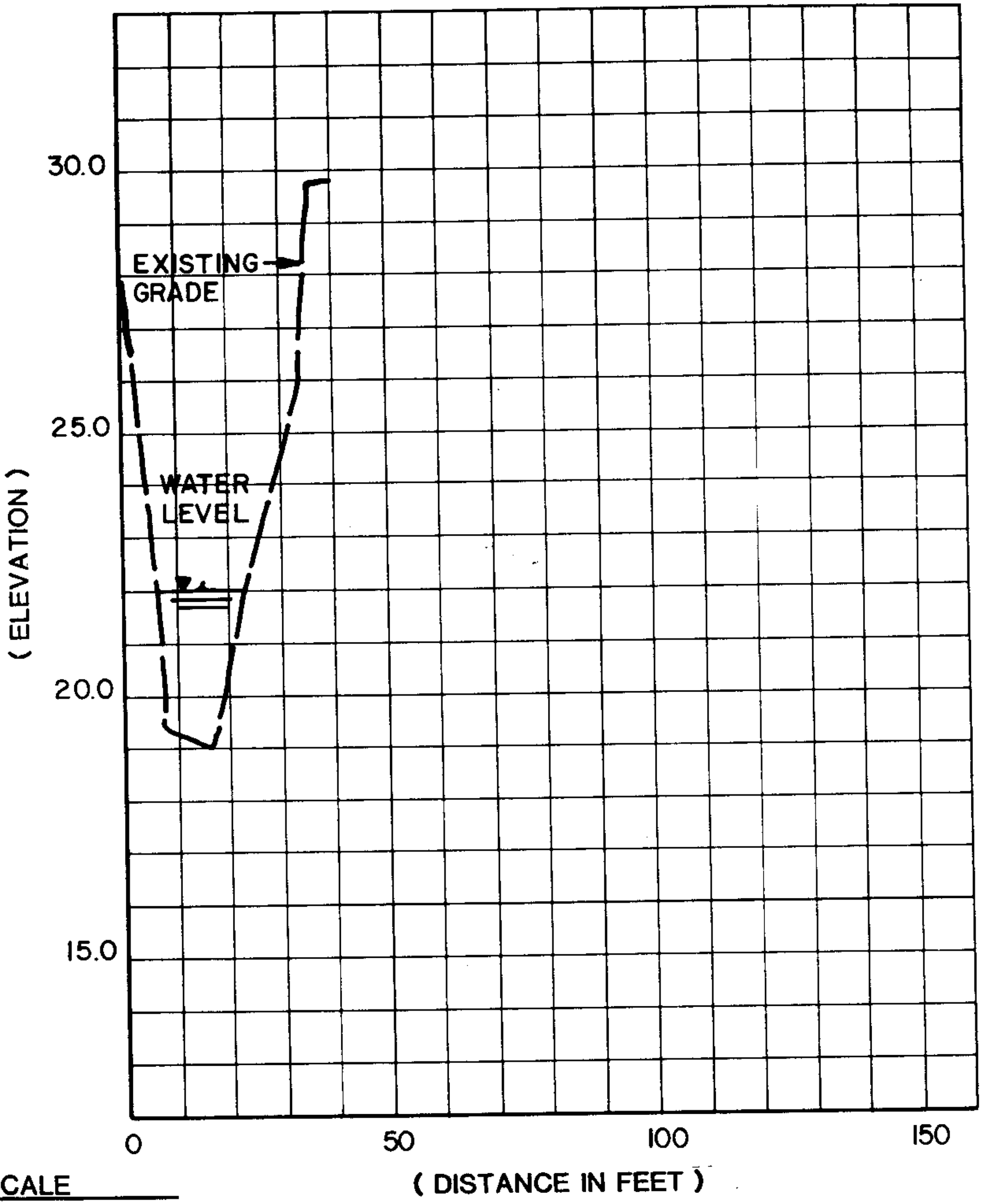
VIEW - FACING DOWN STREAM

CROSS-SECTION
NO. P-31

Station 59340

FIGURE P-31

SARASOTA COUNTY STORMWATER MANAGEMENT PROGRAM




SCALE

HORIZ. : 1"= 30'

~~VERT. : 1"= 3'~~

PHILLIPPI CREEK

VIEW - FACING DOWN STREAM

CROSS-SECTION NO. P-32

WATER EL. : + 22.0

Station 60870

FIGURE P-32

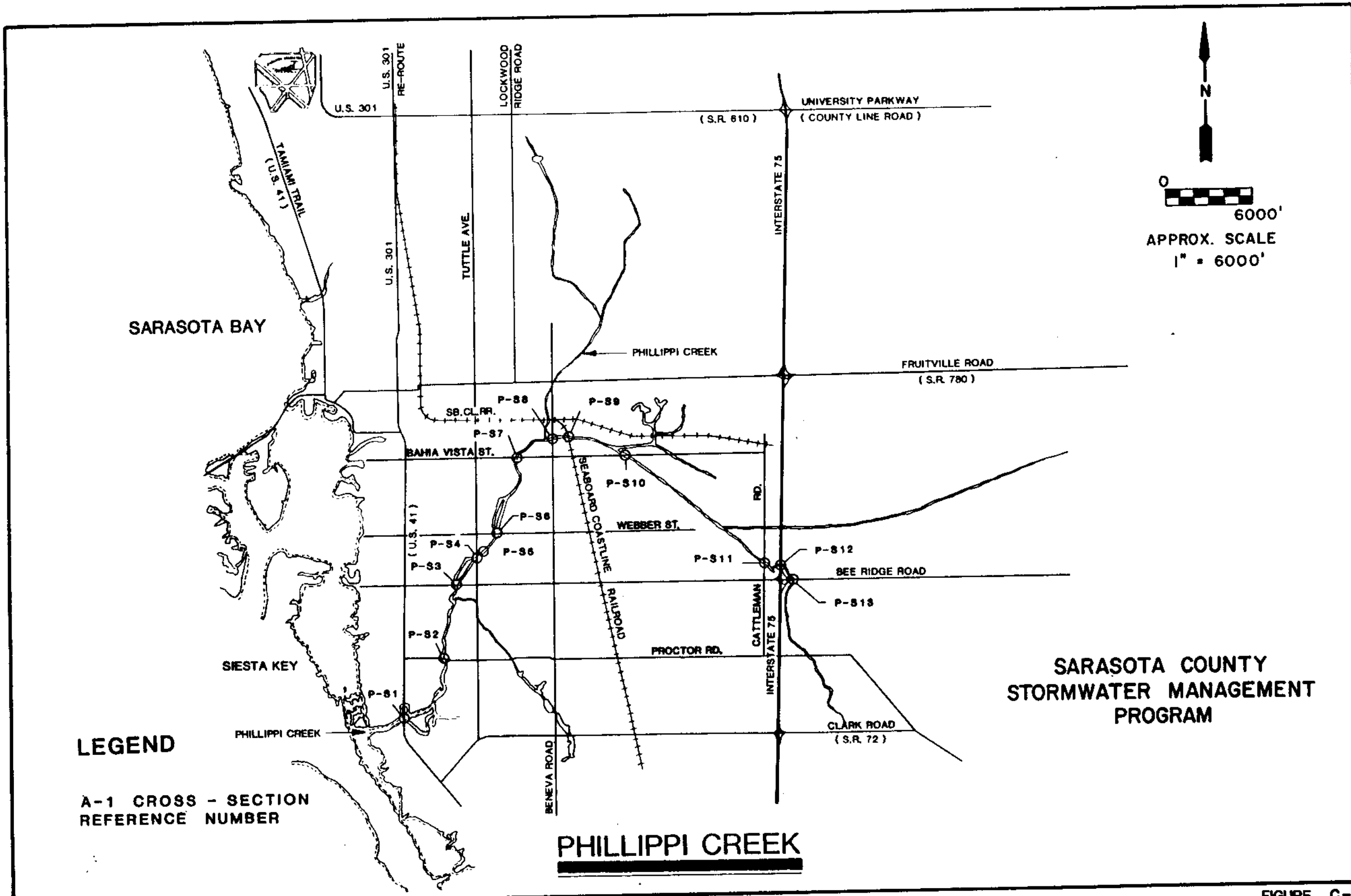
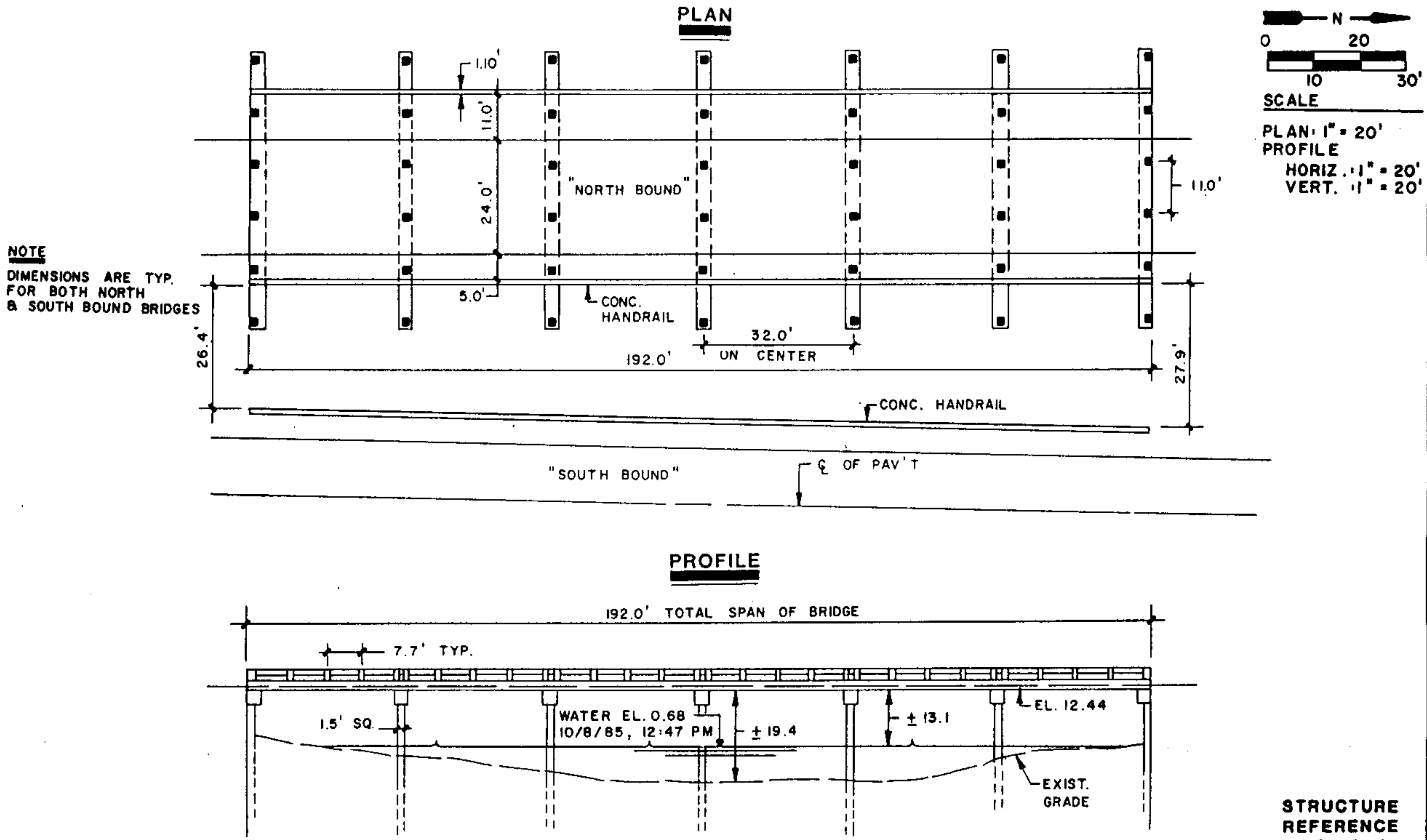



FIGURE C-2

FIGURE C-2

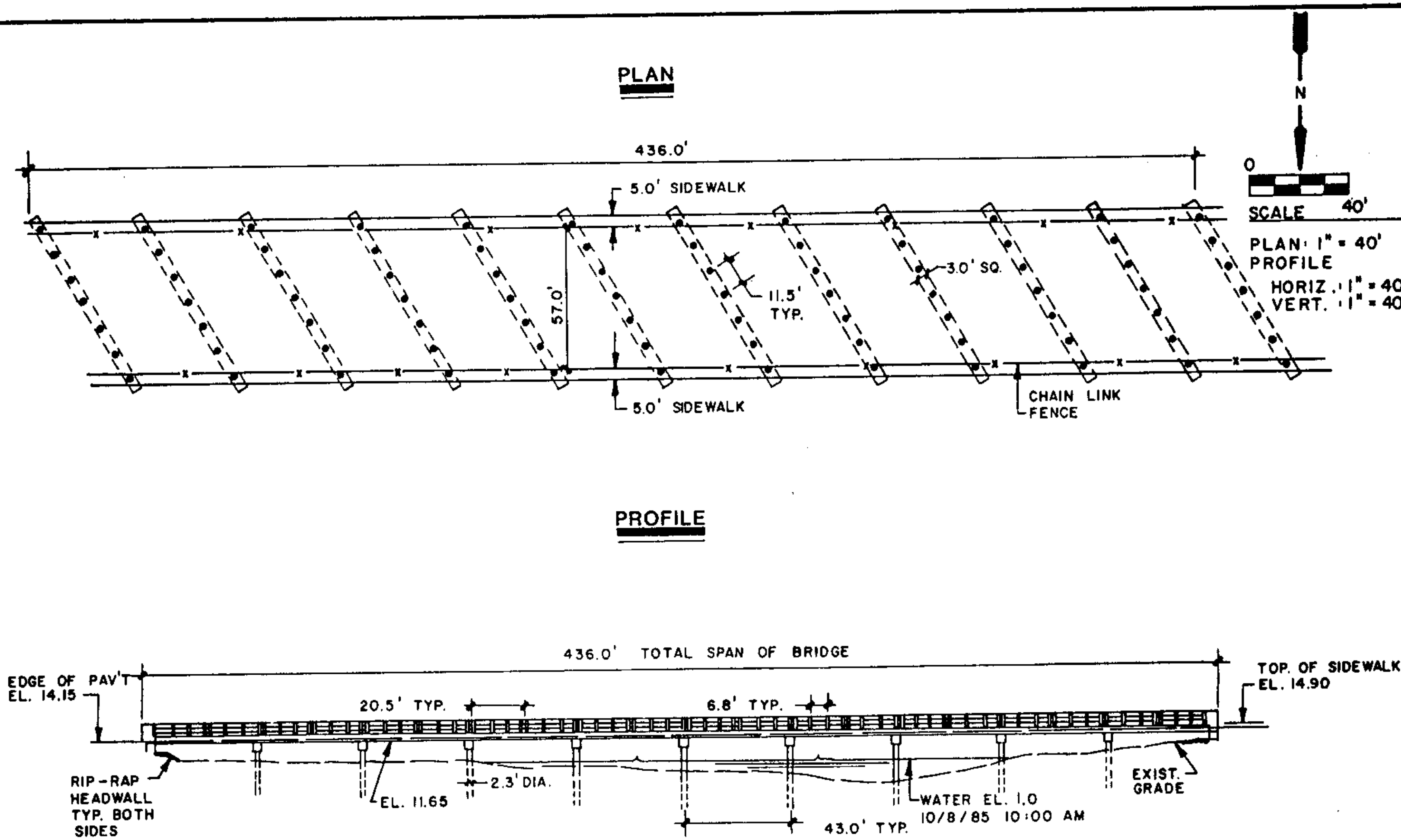
Phillippi Creek Structures - Location Map




STRUCTURE REFERENCE NO. P-S1

## U.S. 41 & PHILLIPPI CREEK BRIDGE

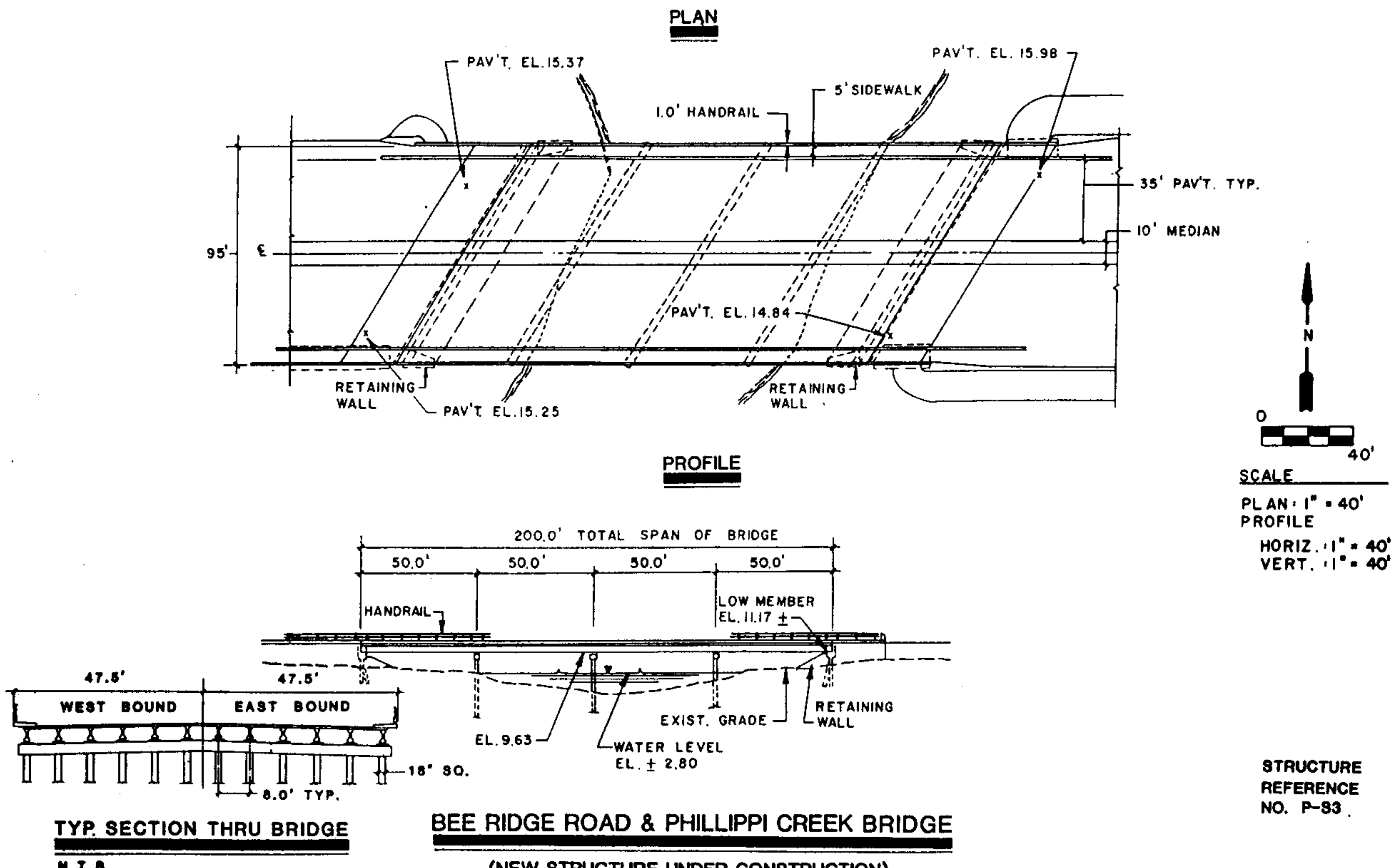
( TWIN STRUCTURES - ONE TYPICAL SHOWN )




STRUCTURE REFERENCE NO. P-S2

PROCTOR ROAD & PHILLIPPI CREEK BRIDGE

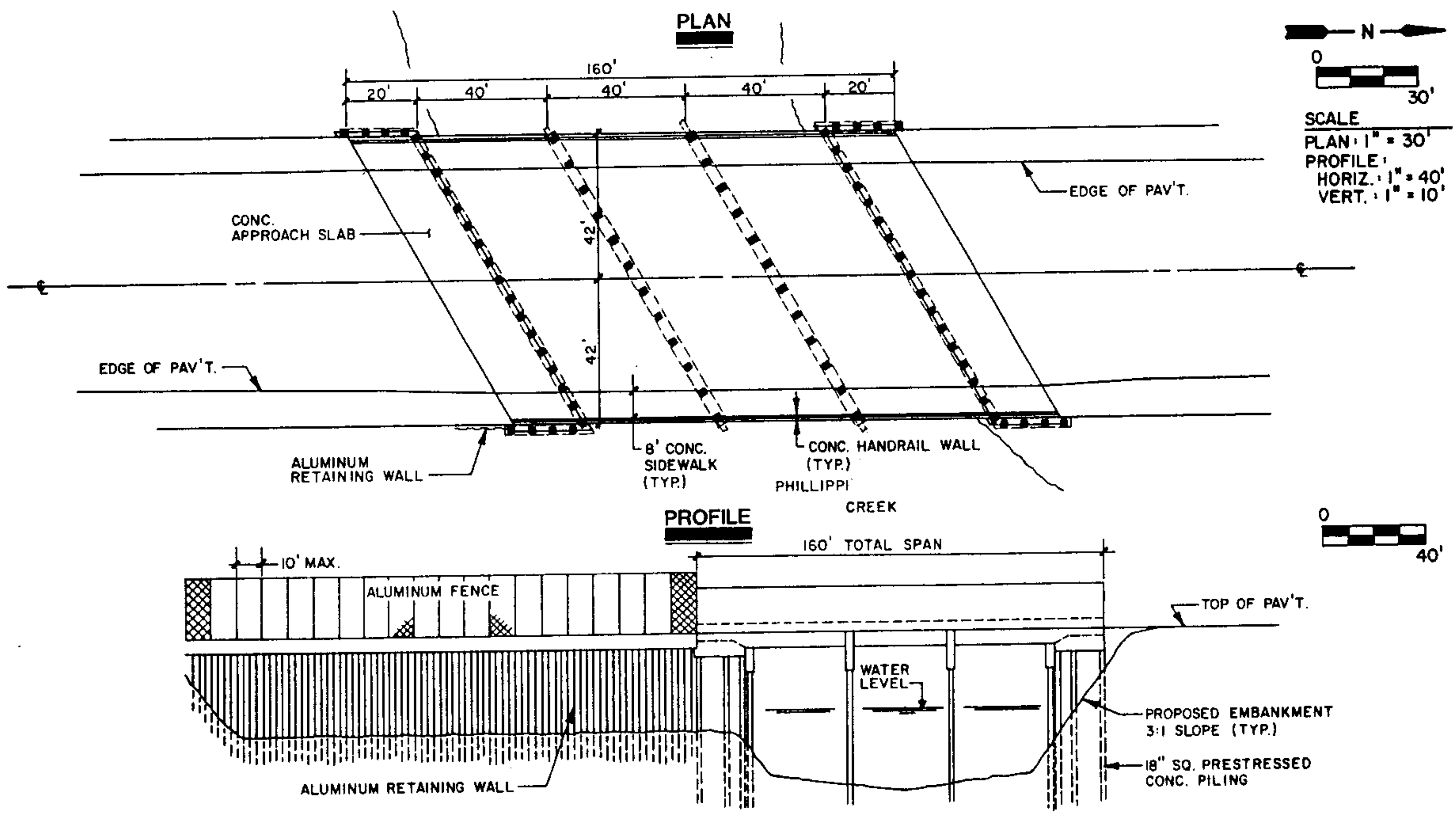
FIGURE P-S2




**BEE RIDGE ROAD & PHILLIPPI CREEK BRIDGE**

(NEW STRUCTURE UNDER CONSTRUCTION)

FIGURE P-S3

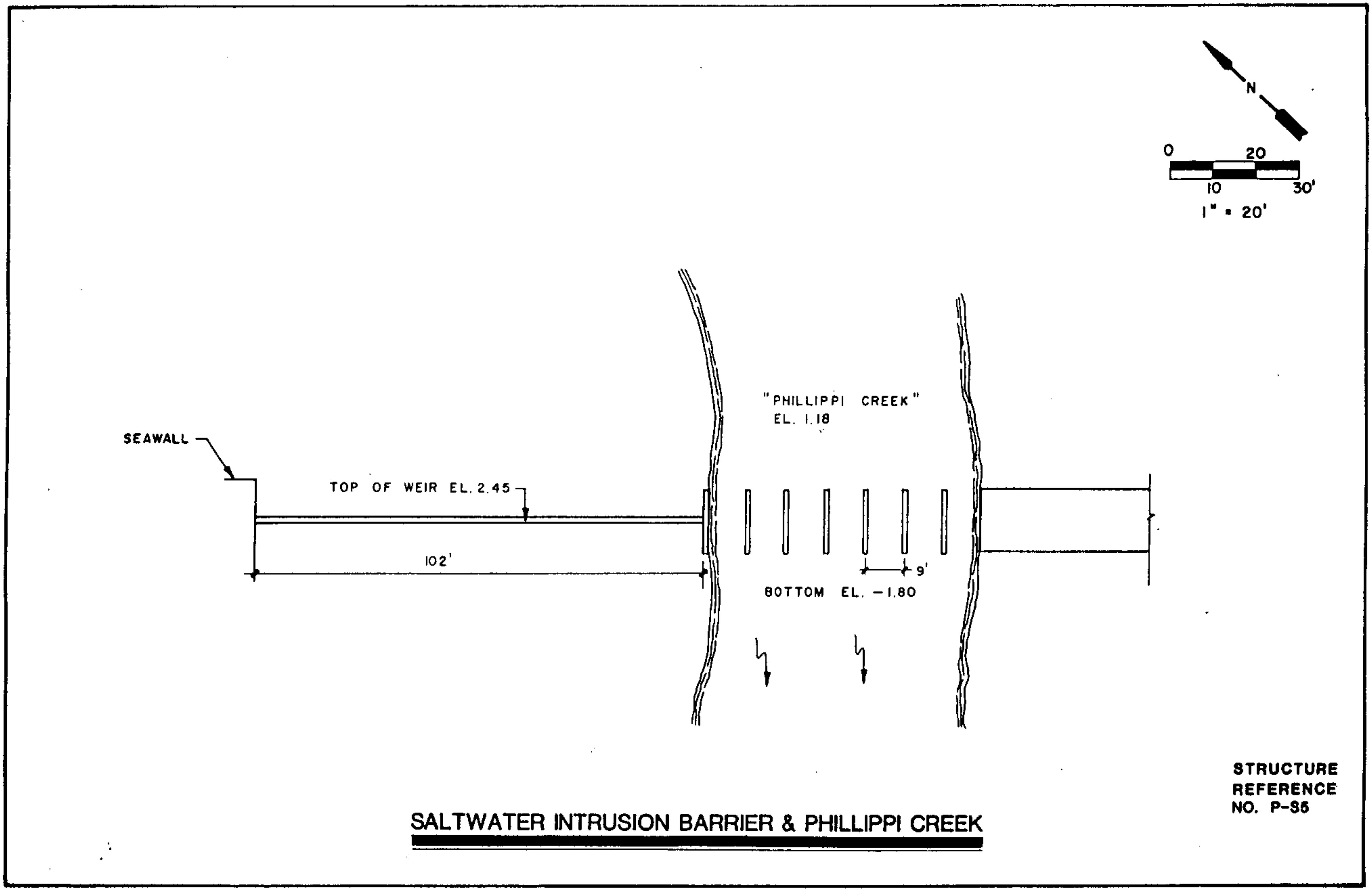



STRUCTURE REFERENCE NO. P-S4

## TUTTLE AVENUE BRIDGE & PHILLIPPI CREEK

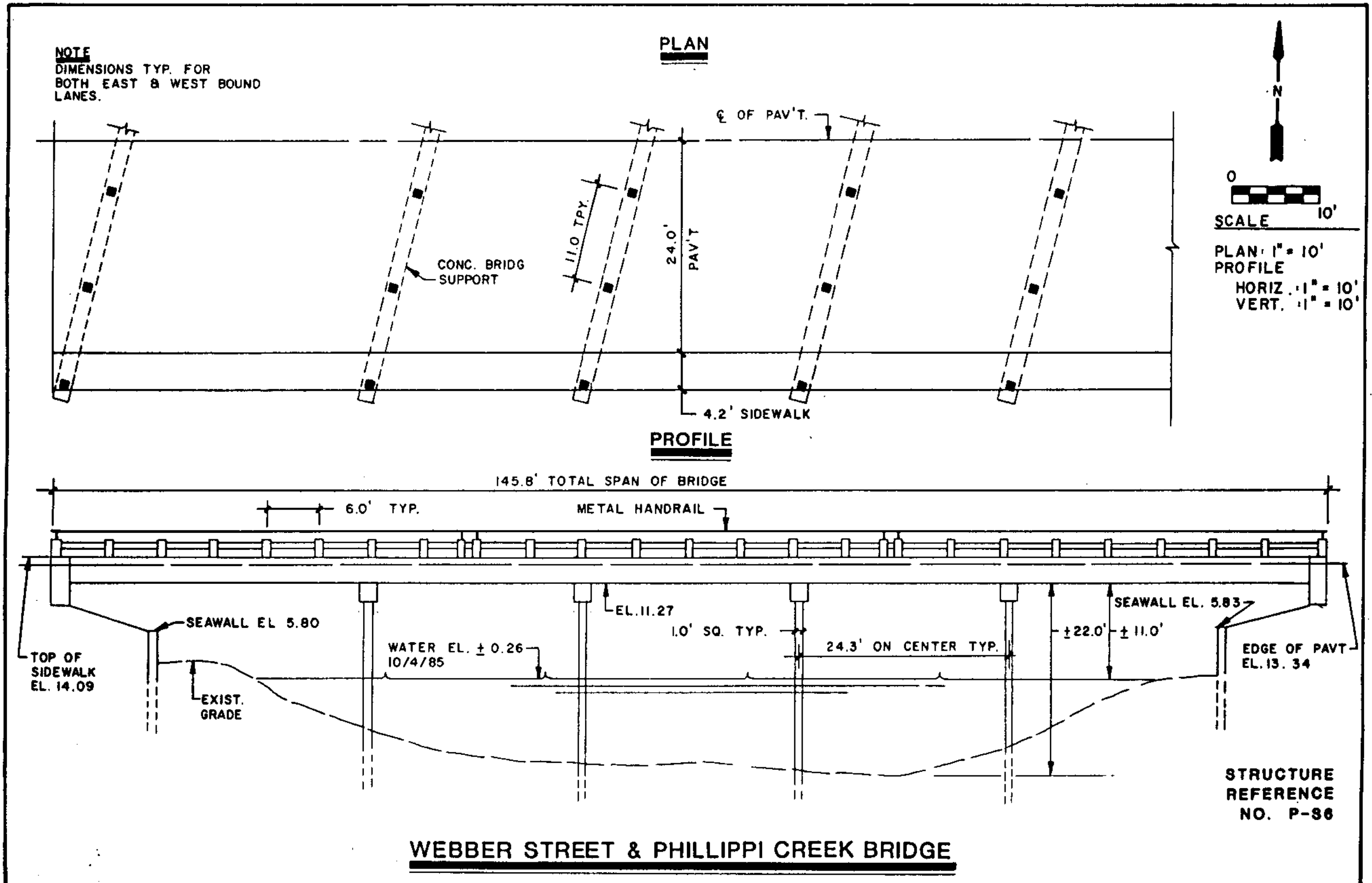
NEW BRIDGE – UNDER CONSTRUCTION

FIGURE P-S4




SALTWATER INTRUSION BARRIER & PHILLIPPI CREEK

FIGURE P-S5

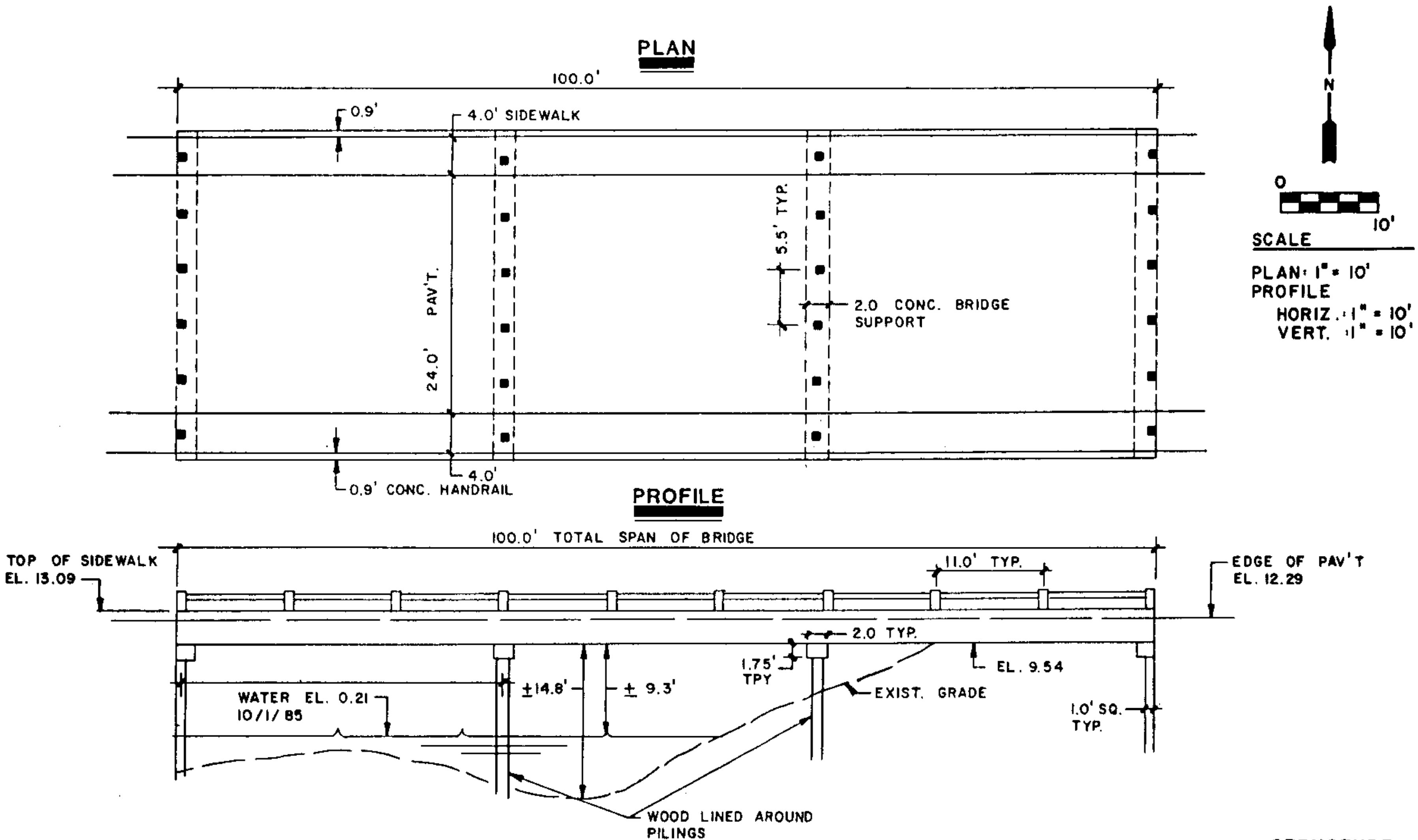



WEBBER STREET & PHILLIPPI CREEK BRIDGE

Webber Street - Structure

FIGURE P-S6

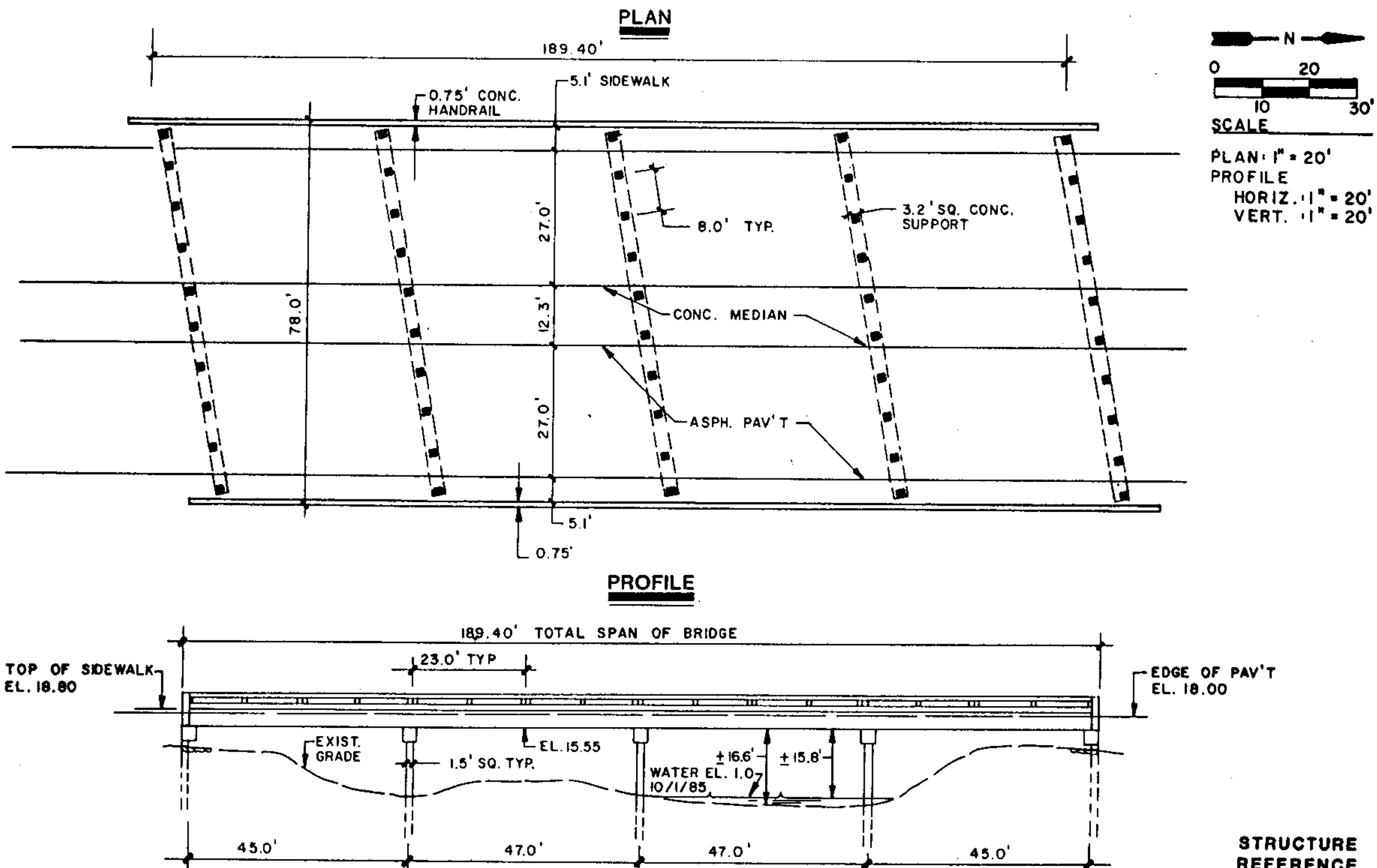
FIGURE P-S6




STRUCTURE REFERENCE NO. P-S7

## BAHIA VISTA & PHILLIPPI CREEK BRIDGE

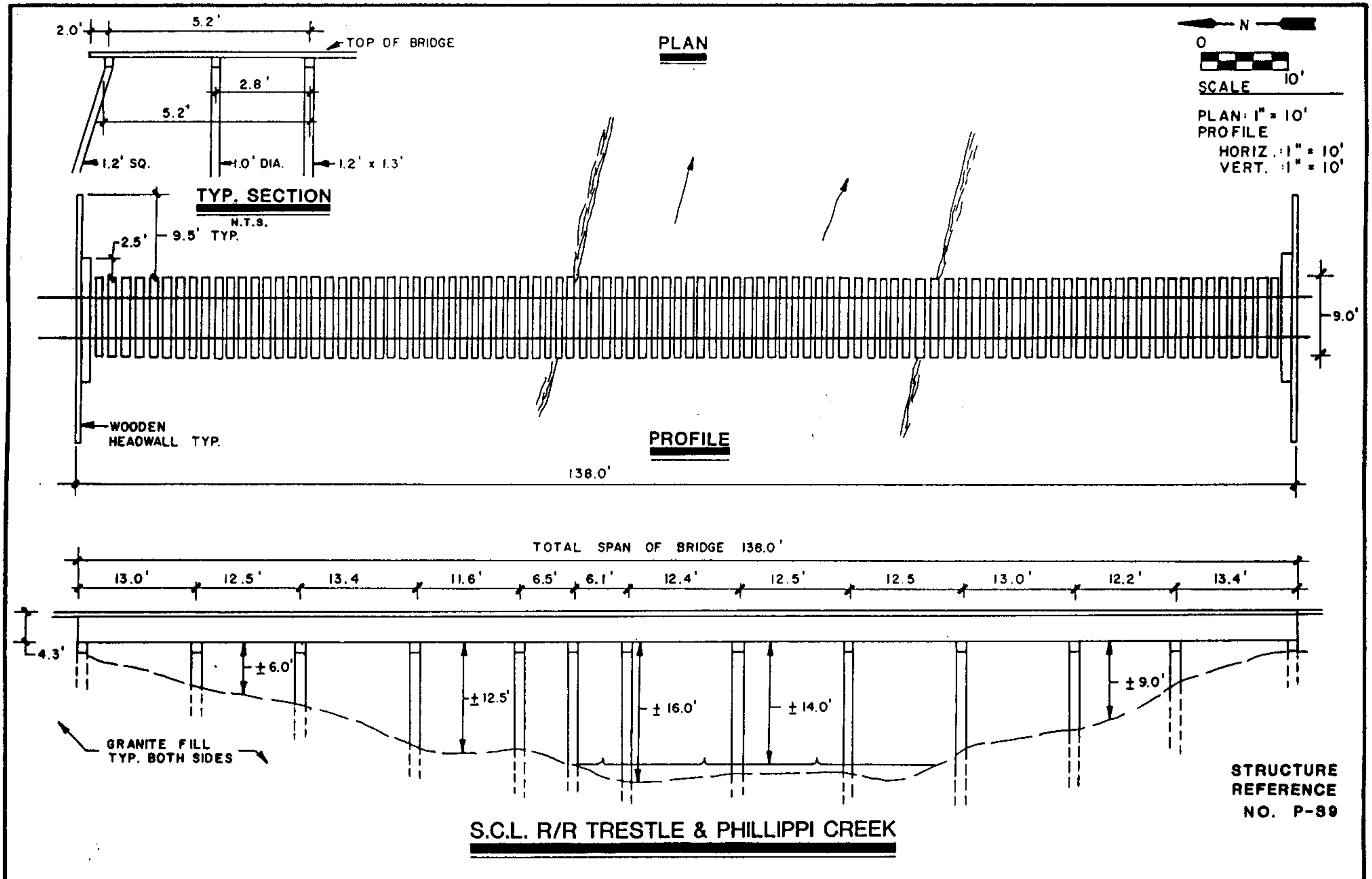
FIGURE P-S7




STRUCTURE REFERENCE NO. P-S8

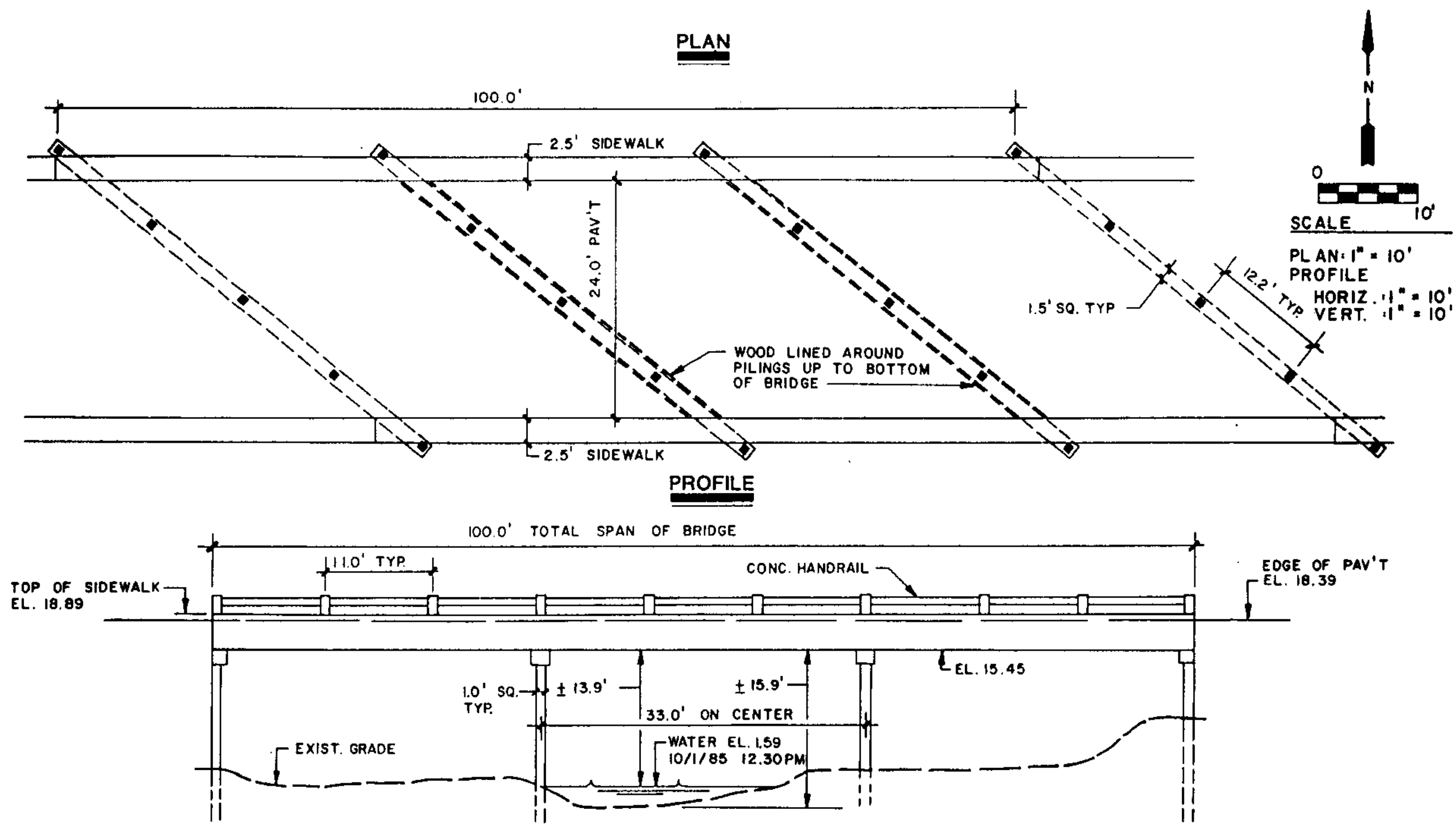
BENEVA ROAD & PHILLIPPI CREEK BRIDGE

FIGURE P-S8



PLAN
TYP. SECTION
N.T.S.
TOP OF BRIDGE
2.0'
5.2'
2.8'
5.2'
1.2' SQ.
1.0' DIA.
1.2' x 1.3'
2.5'
9.5' TYP.
9.0'
WOODEN HEADWALL TYP.
PROFILE
138.0'
N
0
10'
SCALE
PLAN: 1" = 10'
PROFILE
HORIZ.: 1" = 10'
VERT.: 1" = 10'
TOTAL SPAN OF BRIDGE 138.0'
13.0'
12.5'
13.4
11.6'
6.5'
6.1'
12.4'
12.5'
12.5
13.0'
12.2'
13.4'
4.3'
± 6.0'
± 12.5'
± 16.0'
± 14.0'
± 9.0'
GRANITE FILL TYP. BOTH SIDES
STRUCTURE REFERENCE NO. P-S9
S.C.L. R/R TRESTLE & PHILLIPPI CREEK

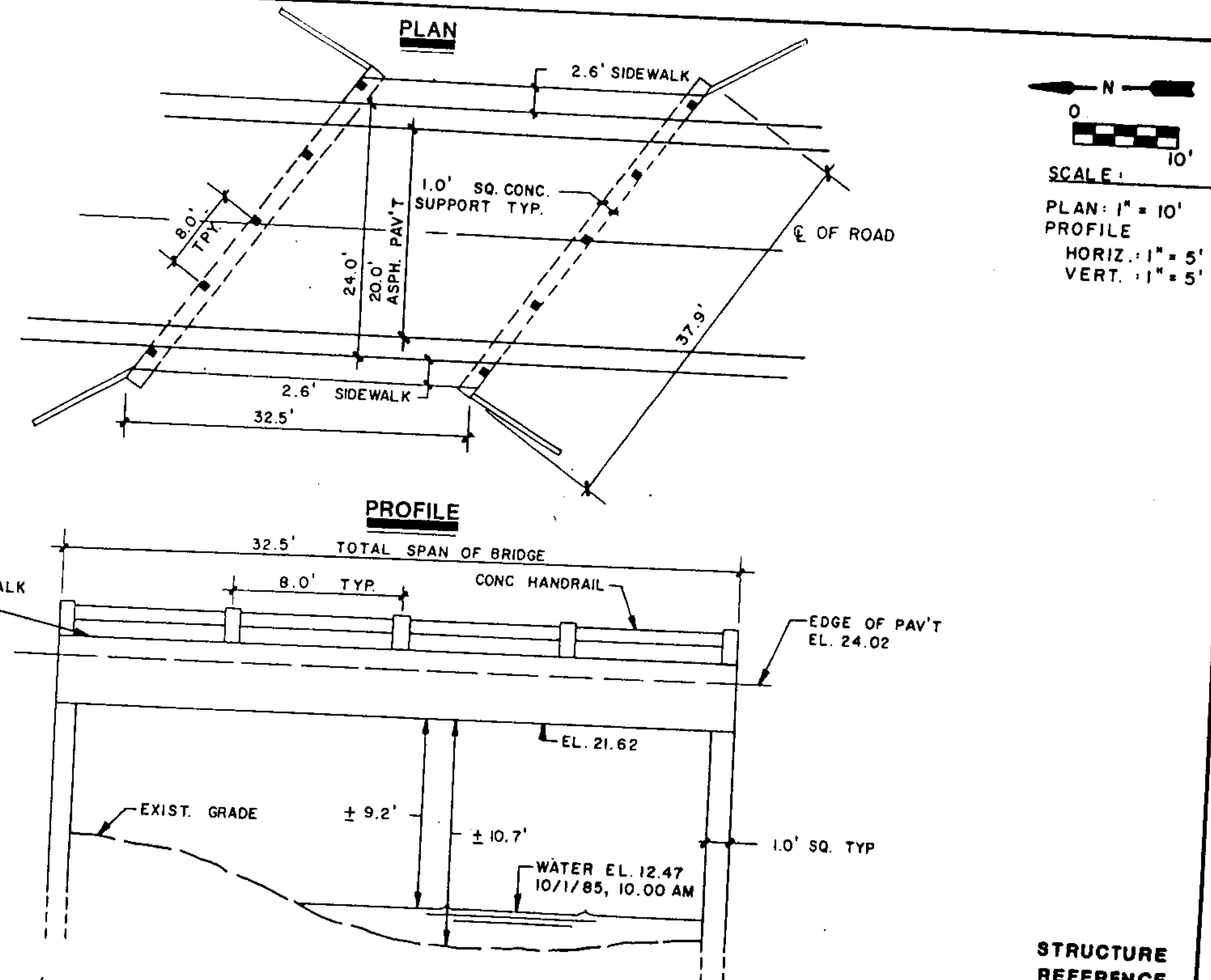

FIGURE P-S9




STRUCTURE REFERENCE NO. P-S10

BAHIA VISTA EAST & PHILLIPPI CREEK BRIDGE

FIGURE P-S10

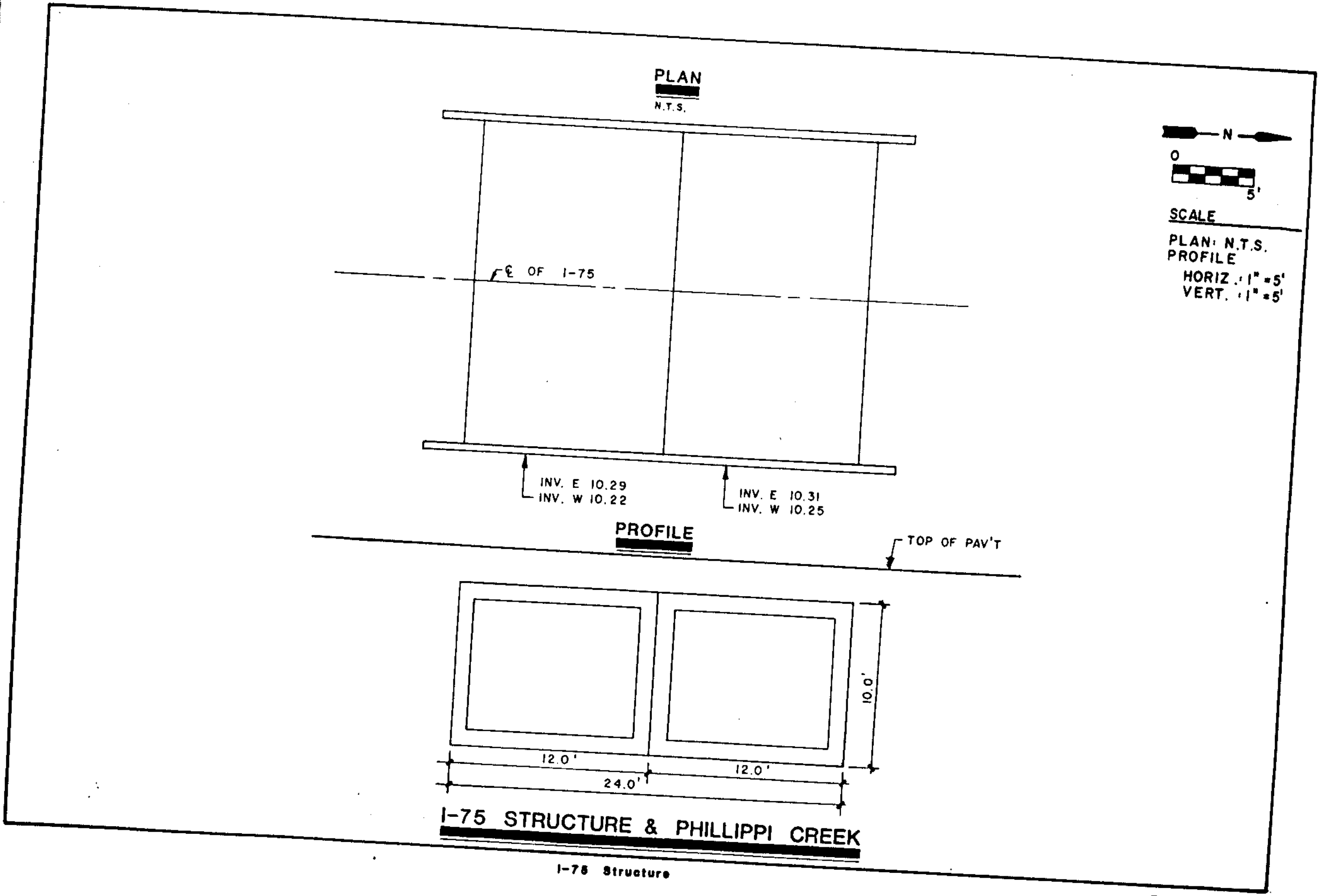



## CATTLEMEN ROAD & PHILLIPPI CREEK BRIDGE

Cattlemen Road - Structure

FIGURE P-S11

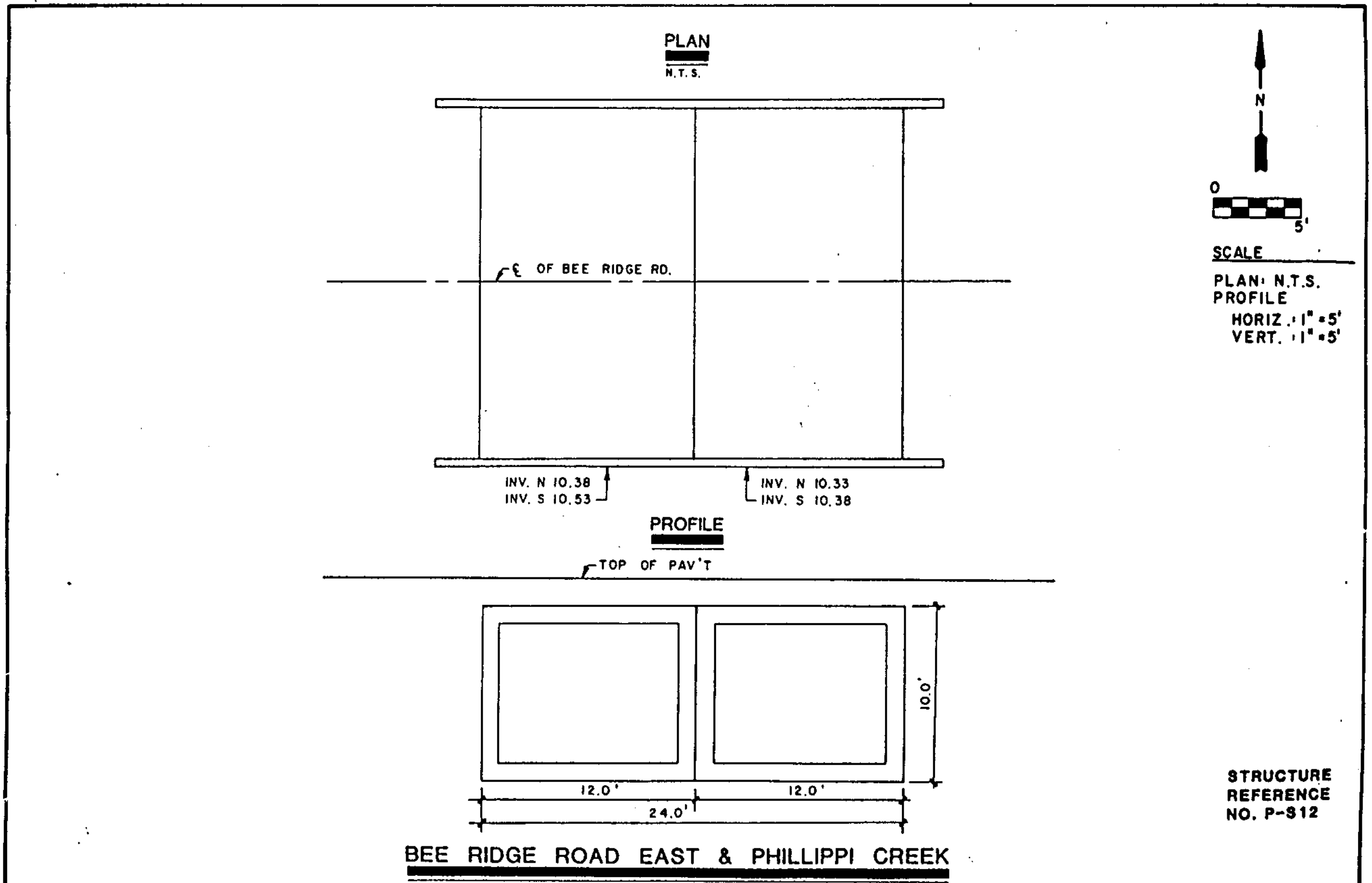
FIGURE P-S11



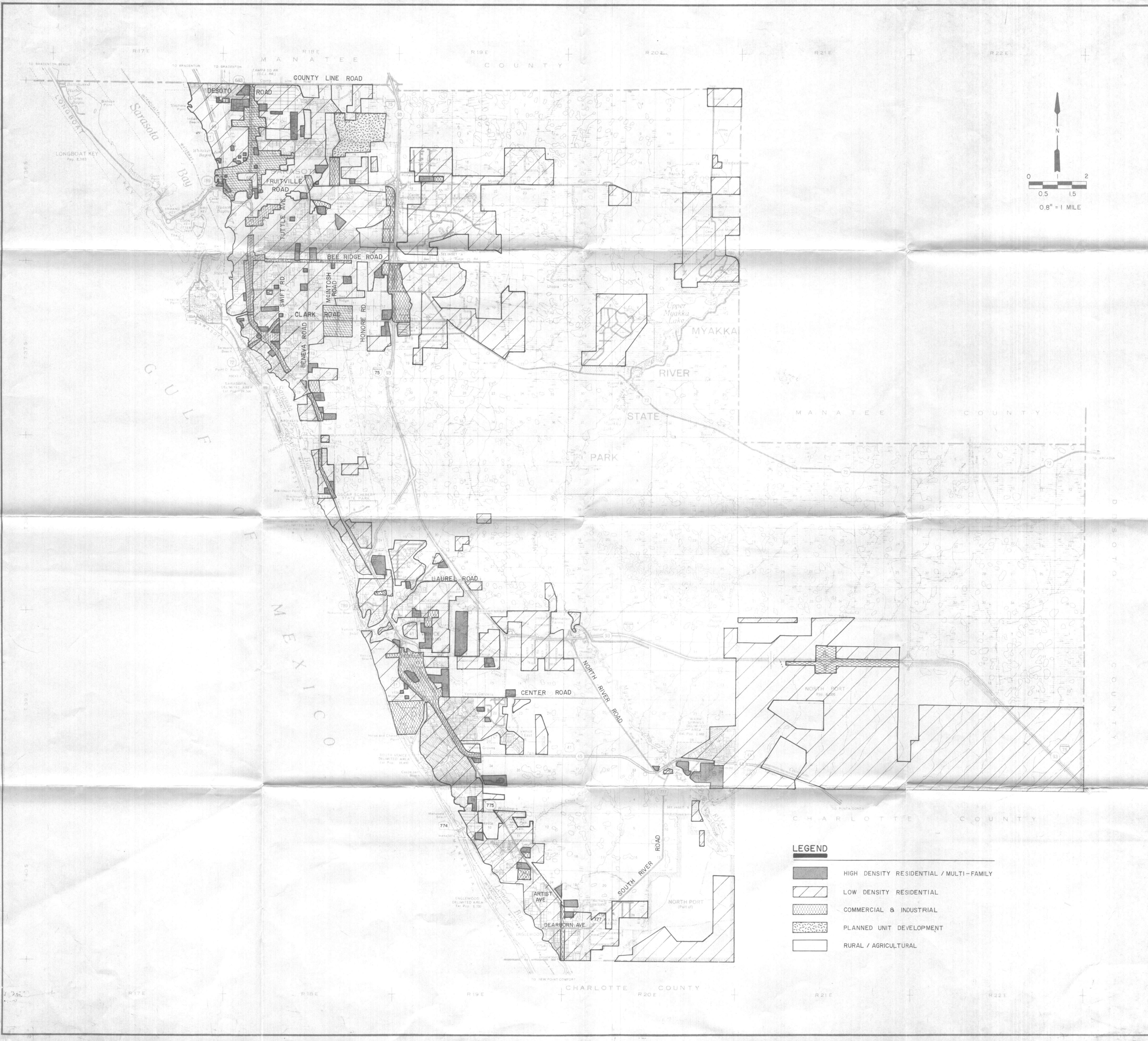

## I-75 STRUCTURE & PHILLIPPI CREEK

I-75 Structure

FIGURE P-812




BEE RIDGE ROAD EAST & PHILLIPPI CREEK




MAPPING STUDY

# EXISTING LAND USE MAP

SARASOTA COUNTY, FLORIDA

CAMP DRESSER & McKEE INC.

CDM

BASED ON SARASOTA COUNTY GENERAL HIGHWAY MAP

PROJECT NO 9250-5

SHEET NO 1

FIGURE 3-1

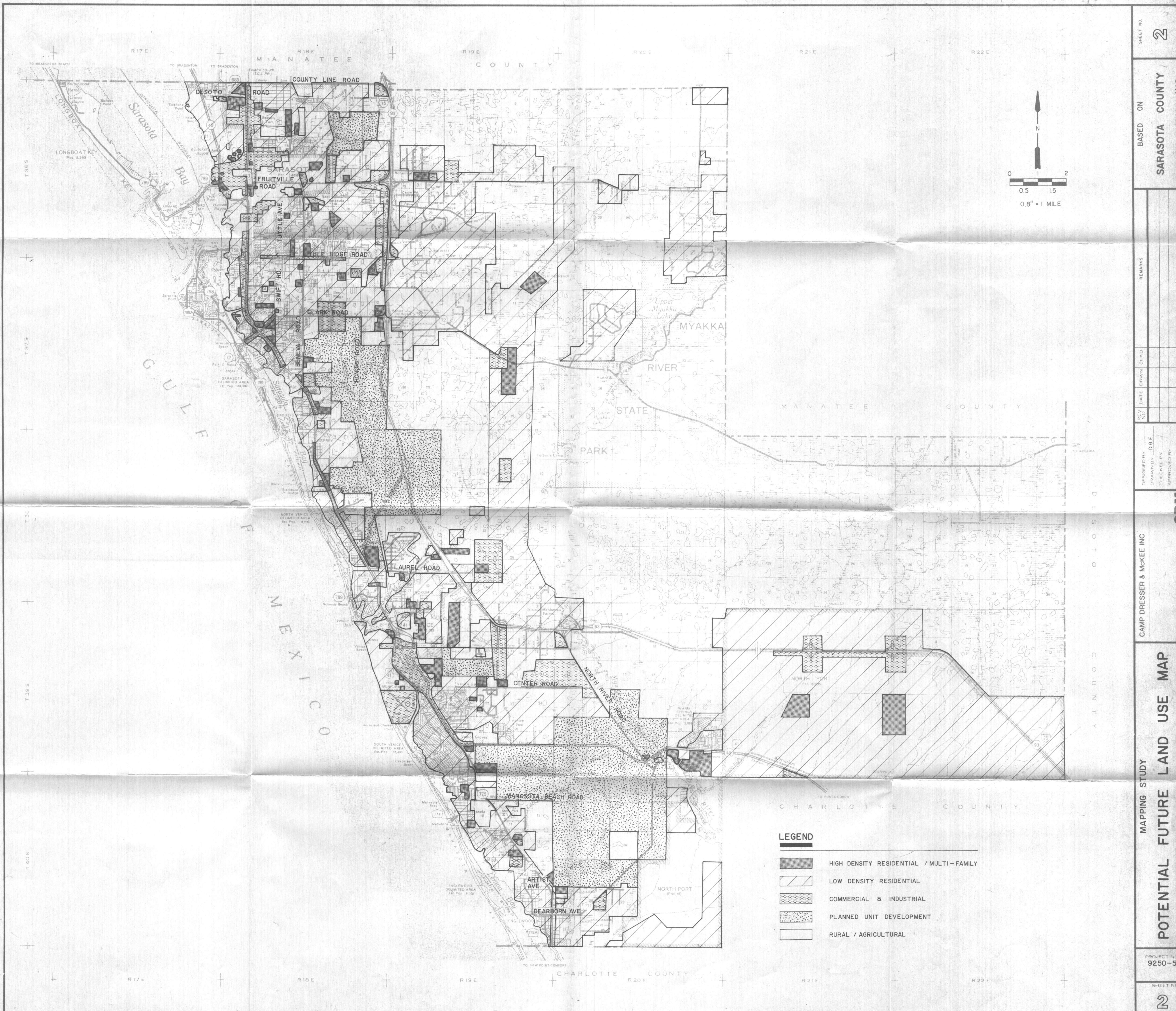

MAPPING STUDY
POTENTIAL FUTURE LAND USE MAP
SARASOTA COUNTY, FLORIDA
CAMP DRESSER & McKEE INC.
CDM
BASED ON SARASOTA COUNTY GENERAL HIGHWAY MAP
SHEET NO. 2
PROJECT NO. 9250-5
DESIGNED BY
DRAWN BY D.G.E.
CHECKED BY
APPROVED BY
DATE
REV NO.
DATE
DRWN
CHKD
REMARKS
LEGEND
HIGH DENSITY RESIDENTIAL / MULTI-FAMILY
LOW DENSITY RESIDENTIAL
COMMERCIAL & INDUSTRIAL
PLANNED UNIT DEVELOPMENT
RURAL / AGRICULTURAL
0.8" = 1 MILE
N
MANATEE COUNTY
CHARLOTTE COUNTY
DESOTO COUNTY
GULF OF MEXICO
MYAKKA RIVER STATE PARK
COUNTY LINE ROAD
DESOTO ROAD
FRUITVILLE ROAD
TUTTLE AVE.
BEE RIDGE ROAD
SWIFT RD.
CLARK ROAD
BENEVA ROAD
HONORE ROAD
LAUREL ROAD
CENTER ROAD
NORTH RIVER ROAD
MANASOTA BEACH ROAD
ARTIST AVE
DEARBORN AVE
Sarasota Bay
LONGBOAT KEY
NORTH PORT
R17E
R18E
R19E
R20E
R21E
R22E

FIGURE 3-2