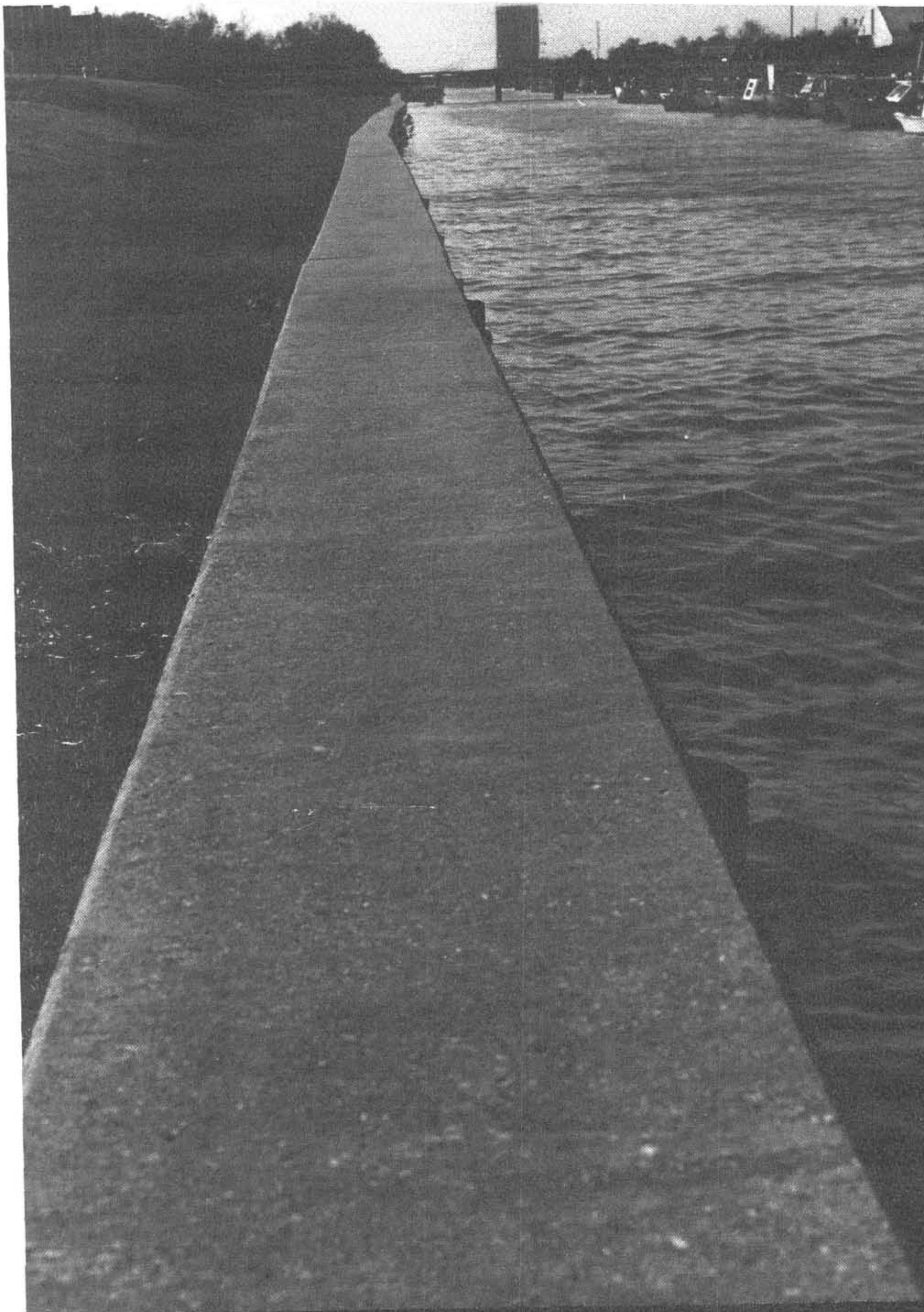


SEVENTEENTH STREET CANAL DRAINAGE BASIN STUDY

Linfield, Hunter and Gibbons, Inc.

New Orleans Houston

Seventeenth Street Canal Drainage Basin Study

Funded by the Louisiana State Legislature

prepared
under the direction of

the Sewerage & Water Board of New Orleans

and

the Jefferson Parish Council

Linfield, Hunter & Gibbons, Inc.

January 1983

LINFIELD, HUNTER AND GIBBONS, INC.
CONSULTING ENGINEERS
651 RICHARD STREET
NEW ORLEANS, LOUISIANA 70130
(504) 581-1407

DAVID A. HUNTER, JR.
RALPH W. JUNIUS, JR.

R. P. LINFIELD, 1957-1979
ASHBY T. GIBBONS, JR., 1974-1979

December 23, 1982

The Honorable Ernest N. Morial
President
Sewerage & Water Board of New Orleans
City Hall - Civic Center
New Orleans, Louisiana 70112

Mr. Robert B. Evans, Jr.
Chairman
Jefferson Parish Council
3330 North Causeway Boulevard
Metairie, Louisiana 70002

Gentlemen:

Transmitted herewith is our report entitled "Seventeenth Street Canal Drainage Basin Study" submitted in accordance with our contract dated March 29, 1982, between the Parish of Jefferson and the Sewerage & Water Board of New Orleans.

This report provides the first in-depth study of the Seventeenth Street Canal Drainage Basin comprising 7,860 acres of Orleans Parish and 2,550 acres of Jefferson Parish. The report recommends improvements and presents alternate solutions to some of the more difficult problems in the basin.

We wish to acknowledge the very strong support provided by the Sewerage & Water Board of New Orleans and the Jefferson Parish Department of Utilities. The technical representatives of these two bodies helped to develop the technical data upon which this report is based. We believe that their joint support means that the report and the conclusions presented represented a true consensus.

We have enjoyed working on this project and hope that it presents a comprehensive basis for improved drainage in both Orleans and Jefferson Parishes.

Very truly yours,

LINFIELD, HUNTER AND GIBBONS, INC.

David A. Hunter, Jr.

DAHJR/pbh

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

Title **Page No.**

STATEMENT OF INTENT

It is the intent of this study to review and verify the storm drainage requirements in the basin tributary to the 17th Street Canal and the Sewerage & Water Board of New Orleans' Pumping Station No. 6 and to investigate alternative measures of providing adequate facilities for these requirements.

BACKGROUND

Substantial flooding has occurred in recent years in both the Orleans and Jefferson Parish sections of this drainage basin. The former includes most of the Uptown area of New Orleans and the latter the Seventh Ward and Hoey's Basin in Jefferson. Since considerable damage has occurred during flooding, there has been great pressure on local governing bodies to provide drainage improvements to relieve these conditions.

As part of its capital improvement program, the Sewerage & Water Board has proposed a project that would increase the capacity of both Pumping Station No. 6 and the Outfall Canal to the Board's stated requirements for proper drainage of the area. The Corps of Engineers has delayed the issuance of a dredging permit pending further information concerning flow requirements, levee stability and alternative measures. In addition, members of the Jefferson Parish Council have also raised questions concerning the necessity of the project. In response to these concerns, the Louisiana State Legislature has funded this independent study in order to establish whether or not the project is necessary and to clear the way for improved drainage in the area.

SYNOPSIS

The following is a brief summary of the findings of this study:

A Design Storm producing 5 inches of rainfall in 5 hours was agreed upon as the basis for this study.

A total discharge of approximately 10,400 cubic feet per second is produced by the Design Storm. The Orleans contribution is 7700 cfs and the Jefferson contribution is 2700 cfs.

The capacities of Pumping Station No. 6 and the Outfall Canal to Lake Pontchartrain must be each increased by approximately fifty percent to accommodate the 10,400 cfs discharge.

This increase to the Outfall Canal can be most economically achieved by widening and deepening the existing canal for its entire length.

The increased discharge and velocities under improved conditions presents a health and safety hazard to the Bucktown area. If no improvements are made within this reach of the canal, many of the existing stuctures will wash away with time. An alternative is presented in which the Bucktown structures could be rebuilt in a protected area at approximately their same locations.

The capacity of the Seventeenth Street Canal between Station 6 and Jefferson Highway must be increased in order to maintain water levels low enough to permit Jefferson and Uptown New Orleans to drain by gravity.

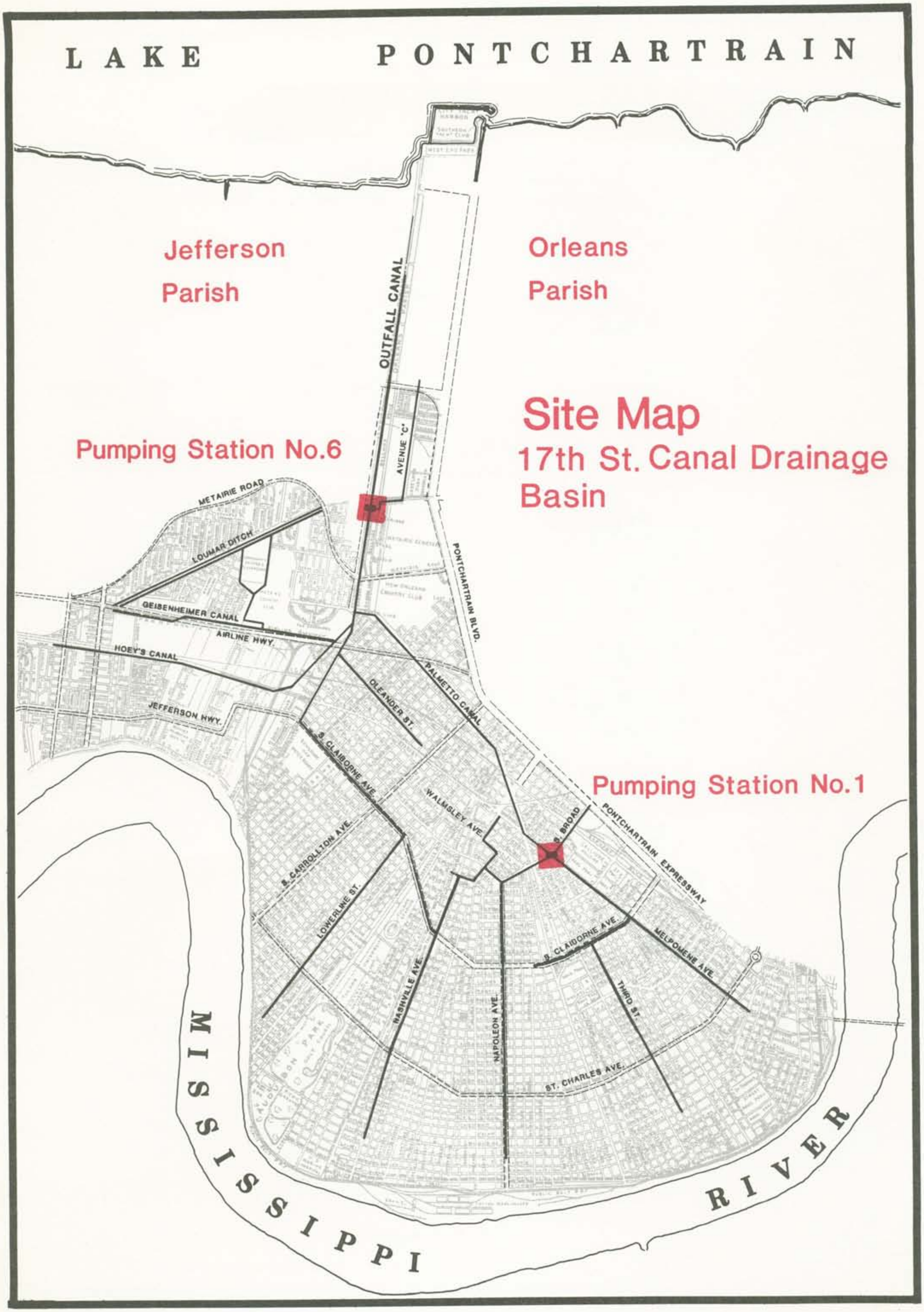
The capacity of Pumping Station No. 1 (Broadmoor) must be increased to a minimum of 6000 cfs. The Palmetto Canal (linking Pumping Station No. 1 to the Seventeenth Street Canal) capacity must be increased to handle this additional discharge.

Major improvements must be made in Hoey's Canal and the Geisenheimer Canal to deliver the required 2550 cfs flows to the Seventeenth Street Canal.

The capacities of the existing gravity systems in both Orleans and Jefferson parishes must be doubled to meet the design flow requirements.

All of the above requirements must be met if the general flooding in the basin is to be relieved. Construction of these impovements must be started at the most downsteam point in the system, Lake Pontchartrain, and continued progressively upstream. Only after improvements to the major components of the drainage system are in place should extensive improvements be made to local street drainage.

Many of the recommended improvements are beneficial to both Orleans and Jefferson parishes and, in fact, are necessary for proper drainage. It is strongly recommended that the two governing bodies work together to obtain funding for thse improvements.


LAKE PONTCHARTRAIN
Jefferson Parish
Orleans Parish
Site Map
17th St. Canal Drainage Basin
Pumping Station No.6
Pumping Station No.1
OUTFALL CANAL
AVENUE "C"
METAIRIE ROAD
LOUMAR DITCH
GEISENHEIMER CANAL
AIRLINE HWY.
HOEY'S CANAL
JEFFERSON HWY.
PONTCHARTRAIN BLVD.
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PONTCHARTRAIN EXPRESSWAY
S. CARROLLTON AVE.
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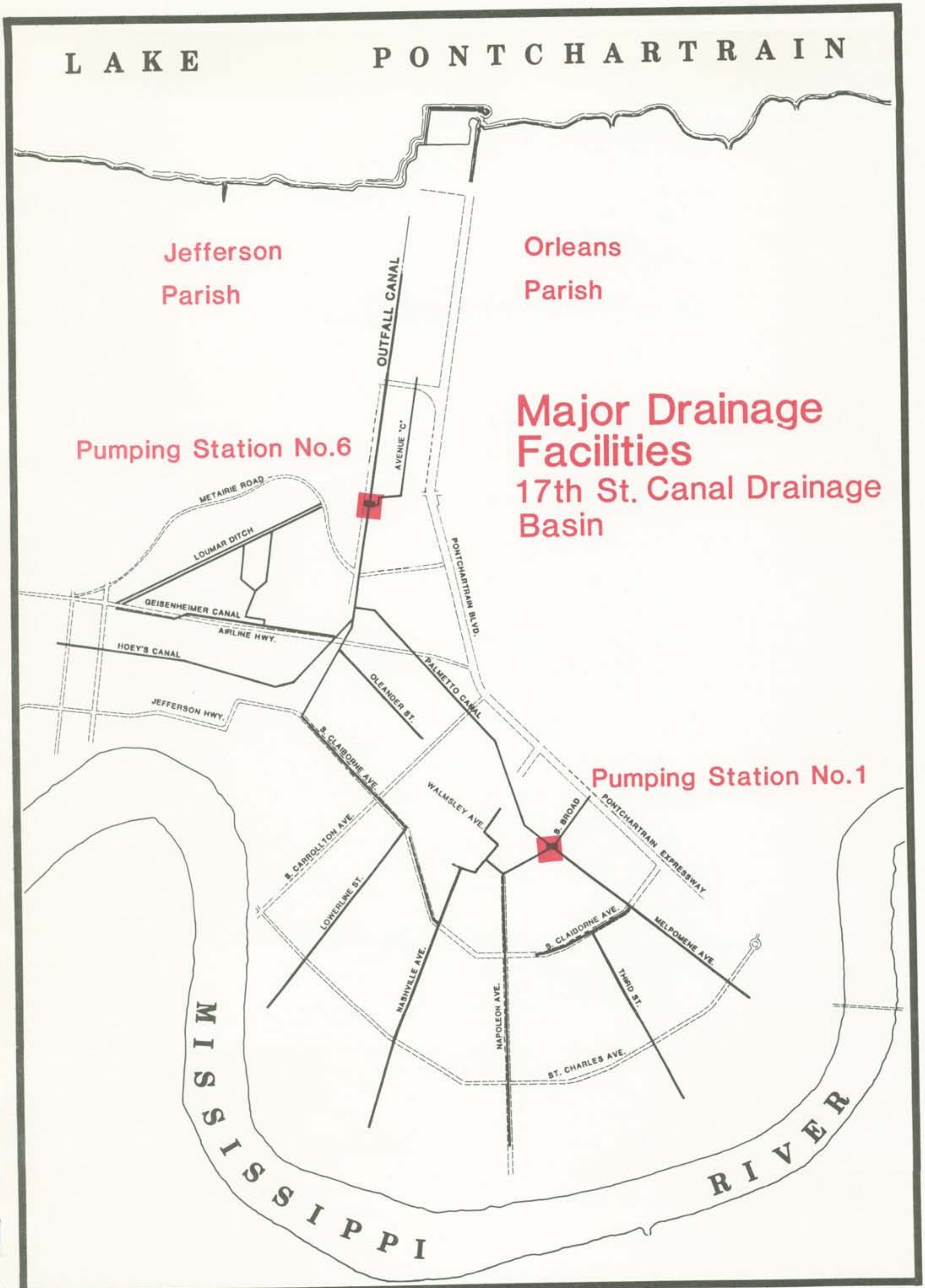
MELPOMENE AVE.
ST. CHARLES AVE.
MISSISSIPPI RIVER


LAKE PONTCHARTRAIN
Jefferson Parish
Orleans Parish
Major Drainage Facilities
17th St. Canal Drainage Basin
OUTFALL CANAL
AVENUE "C"
Pumping Station No.6
METAIRIE ROAD
LOUMAR DITCH
PONTCHARTRAIN BLVD.
GEISENHEIMER CANAL
AIRLINE HWY.
HOEY'S CANAL
OLEANDER ST.
PALMETTO CANAL
JEFFERSON HWY.
S. CLAIBORNE AVE.
WALMSLEY AVE.
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S. BROAD
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THIRD ST.
ST. CHARLES AVE.
MISSISSIPPI RIVER

SUMMARY OF FINDINGS

A Technical Committee, consisting of members of the Sewerage & Water Board of New Orleans and the Jefferson Parish Department of Public Utilities, was established for the duration of the study. Membership of this committee included G. Joseph Sullivan, James Parker and Wesley L. Busby of the Sewerage & Water Board of New Orleans and Peter J. Russo, Ross Ketchum, James A. Lawrence and P. Prat Reddy from the Jefferson Parish Department of Public Utilities. Chairmanship of the committee was rotated between Messrs. Sullivan and Russo. Ten meetings were held over the eight month duration of this study. The purpose of the committee was to involve the separate governmental units in the study process, principally to discuss and agree on study criteria, methods and results at each step of the investigation. This committee proved very helpful in providing a sounding board for the study findings.

1 One of the most important criteria established was the particular design storm for which the drainage system should be designed. The storm agreed upon was an event with a precipitation of approximately 5 inches in 5 hours over the entire study area. This storm is similar to the design storms used in each parish.

2 Another important design consideration was the extent to which improvements within each system should be considered "in place" for the purpose of calculating the design discharges. It was agreed that all major elements, that is, main line collector pipes, canals, and pumping stations, should be considered improved so that they have the capacity to carry, without flooding, all discharges delivered by the local street systems.

3 Based on these assumptions, the peak discharge requirement of the entire system was determined to be approximately 10,400 cfs. It was found that the peaks from each segment of the system will occur simultaneously at the pumping station suction basin and will be additive. The split of peak discharges between Orleans and Jefferson parishes will be as follows:

Orleans — 7700 cfs (74%)(0.98 cfs/acre)
Jefferson — 2700 cfs (26%)(0.94 cfs/acre)

4 As a comparison, the existing system total capacity is approximately 5500 cfs. The split of peak discharges under the existing system is as follows:

Orleans — 4300 cfs (78%)
Jefferson — 1200 cfs (22%)

It is readily seen at this point that considerable improvements are necessary to meet the above requirements.

5 It is strongly recommended that as improvements are made, they be made to satisfy the total recommended design discharge established herein and that they not be phased. This will avoid subsequent additional projects which would surely increase the total cost. Also, improvements should be made starting at the most downstream point in the system, Lake Pontchartrain, and be continued upstream. Upon completion of the upgrading of the main canal system, it will then be possible to improve any deficiencies in the local street drainage lines.

6 The single factor most detrimental to good drainage in the study area is the

general lack of sufficient ground slope. In addition, the general ground subsidence, particularly in the Broadmoor area near Pumping Station No. 1, has reduced the existing pumping capacity and has further limited the available hydraulic slope in the canal systems. The entire drainage basin has seen tremendous development, with the resulting increased runoff, since the basic system was constructed some 60 years ago. For these reasons, many of the existing facilities need to be enlarged and upgraded.

7 The Engineers generally agree with the statement made in a S&WB report prepared in October of 1981, "it would be cost-prohibitive to design and construct a system for New Orleans that would completely eliminate the probability of flooding." With this in mind, the peak discharges as developed under the proposed system allow for some flooding or storage, to occur at the local street level during design storm conditions; however, this flooding will be of short duration and should not cause extensive damage. This flooding (storage) tends to lessen the peak flow at the pump stations and is deemed to be the most reasonable and cost effective approach to the design.

8 Each of the major gravity drainage basins was reviewed to determine the adequacy of the system under design storm conditions and to determine the adequacy of the interior mainline collector pipes, box culverts and ditches. A brief commentary follows:

Hoey's Basin

Under present conditions, the maximum flow from the entire Hoey's Basin is approximately 1100 cfs. The proposed improvements will increase this to approximately 2550 cfs, more than doubling the system capacity. These improvements will allow the entire basin to be drained by gravity as long as the water surface in the 17th Street Canal can be held at or below elevation 16.43 C.D. (−4.00 M.S.L.) at its juncture with Hoey's Canal. Proposed improvements do not contemplate the addition of new canal locations but foresee an increased capacity in most of the major drainage canals including Hoey's, Geisenheimer, Loumar Outfall and many of the other smaller drains, particularly between Jefferson Highway and Hoey's Canal.

Pumping Station No. 1 Basin

At the present time, this gravity system has, at the ground slopes available, the capacity to deliver flows of approximately 3100 cfs to Pumping Station No. 1. The proposed improvements to the system will increase this to 6000 cfs, thus doubling the system capacity. Calculations indicate that portions of all the major canals, except Melpomene, are undersized for present flow requirements. In order to increase the capacity of the existing gravity system, several new covered drainage canals are proposed to be added to the system. These include new lines down Jefferson Avenue, Louisiana Avenue and Washington Avenue. The locations where additional capacity will be required under the improved system include portions of the Walmsley Avenue system and the entire South Broad Street system. Several locations along the smaller feeder canals should also be enlarged. A manifold system is proposed for South Broad Street and South Claiborne Avenue which will equalize flows through the system.

Lowerline & Oleander Basins

Proposed improvements in the 17th Street Canal which will cause lowering of the water surface at the outfall of the canals draining these basins will add capacity to the existing system; however, improvements will be necessary. The Lowerline system is undersized in several locations. At these locations the capacity must be increased from 900 cfs to 1300 cfs. The existing Oleander system is pumped under storm conditions. The pumping capacity is 100 cfs while the canal capacity is approximately 200 cfs. The existing Dublin Canal can be improved to add 50 cfs into the Palmetto Canal.

9 The improvements listed above will increase flows to Pumping Station No. 1 located in the Broadmoor area of New Orleans. These increased flows will require an increase in the capacity of this station from approximately 5400 cfs to a minimum of 6000 cfs.

10 The present capacity of the Palmetto Canal (linking Pumping Station No. 1 to the 17th Street Canal) is limited to approximately 3500 cfs due to the many obstructions, primarily bridges, crossing the canal. The capacity must be increased to approximately 6000 cfs to carry the proposed discharge. Combined with improvements downstream, this can be most economically accomplished by eliminating the obstructions which become submerged in channel flows.

11 The capacity of the 17th Street Canal between Jefferson Highway and Pumping Station No. 6 must be increased in order to meet water surface elevations necessary for proper gravity drainage of Hoey's Basin in Jefferson and the Lowerline Basin in Orleans. It is proposed that this be done by constructing open top concrete box culverts in the present canal location.

12 The present capacity of Pumping Station No. 6 is approximately 6650 cfs. Due to the improvements upstream, this must be increased to a nominal capacity of 10,000 cfs.

13 Due to the increased discharge requirement of 10,000 cfs and the limited head losses allowable between Lake Pontchartrain and Station 6, the capacity of the 17th Street Outfall Canal must be increased. The S&WB has proposed that the entire length of the canal, except under the existing bridges, be excavated and enlarged. This project includes the removal of all structures in the Bucktown area and their replacement with a sheet pile bulkhead and approximately fifty timber pile slips.

This study concludes that the work proposed in the S&WB project will not meet the water surface elevation requirements and that additional work is necessary. In addition to the original proposals for the S&WB project, excavation under all existing bridges, with all required incidental work, and the rebuilding of the Southern Railroad bridge to reduce losses will be required to provide the necessary capacity.

14 Special attention was given to the possibility of leaving the Bucktown area intact. The existing structures, however, would be below the projected water level and, given the high velocities associated with the 10,000 cfs design discharge, would be subject, at best, to an unhealthy and unsafe environment and, at worst, to eventual destruction.

Many alternatives for leaving Bucktown intact were reviewed, and it was concluded that the most feasible method of accomplishing this would be to constrict the proposed channel in the Bucktown area, line the channel with concrete revetment to prevent scour, and construct a protective bulkhead in a location which would provide sufficient land for relocation of the structures in approximately their same location. The necessary channel constriction for this alternative would produce additional headlosses that must be offset by additional widening of the channel upstream. The estimated cost of the required additional work under this plan is approximately $3,000,000.

15 In summary, it was found that the existing drainage system must be upgraded from a capacity of approximately 5500 cfs to 10,000 cfs, an increase of over eighty percent. In order to accomplish this, all of the improvements proposed must be effected.

SECTION 1

INTRODUCTION

CHAPTER I	BACKGROUND
CHAPTER II	EXISTING DRAINAGE SYSTEM
CHAPTER III	PRELIMINARY STUDIES

CHAPTER I
BACKGROUND

ORLEANS

The creation of a modern drainage system began in 1899 with the establishment and funding of the Sewerage and Water Board of New Orleans. At that time the city was sparsely populated with residents and industries located mainly on the high ground along the Mississippi River. The city was then drained by a series of canals and drainage pumping machines which pushed the storm runoff with large paddle wheels from the city into Bayou Bienvenue which then drained into Lake Borgne.

The population densities and the submitted drainage system as originally proposed in 1896 are shown on Figure 1. It can be seen that most of the canals now in use were designed and built at the turn of the century. Heavy rains during the period between 1926 and 1929 prompted a major increase in the pumping capacity of the system; however, by the early 1930's much of the drainage system now being utilized to drain the city had been constructed. In the area now the subject of this study, most of the original system is still in place and still provides its main drainage.

There has been considerable general areal subsidence in the entire drainage area since the system was designed in 1896. This subsidence ranges from 2-4 feet, and nowhere has it been more apparent than in the Broadmoor section of New Orleans. Figure 2, a profile of the Orleans Basin, shows this dramatically. This subsidence not only causes the low areas to flood but also reduces the system pumping capacity.

The City of New Orleans and the East Bank of Jefferson Parish are unique with regard to drainage problems, as they form one of few metropolitan areas surrounded on all sides by protective levees which must pump storm water runoff out over these levees. Most of the pumps used to pump the stormwater in New Orleans were designed by A. Baldwin Wood of the S&WB and were constructed by 1929. These pumps are still in operation and still provide excellent service. Pumping Station No. 6, which pumps the flow from the entire 10,000 acres in the drainage basin out into Lake Pontchartrain, is still one of the largest stations of its kind in the world.

JEFFERSON

The entire population on the Jefferson Parish East Bank in 1950 was 50,000. It is now over 300,000. This population explosion and resulting development have caused many drainage problems for residents of the parish. The area under consideration in this study was developed earlier than most of the parish because of its proximity to the river and to the metairie ridge, the highest areas of the parish.

Hoey's Canal was constructed as a ditch along the Illinois Central Railroad at the turn of the century. It was the main drainage for the area bounded by the river and metairie ridge and drained into the 17th St. Canal, then known as the Metairie Relief Canal. The S&WB took over the 17th St. Canal and in

1913 completed the construction of Pumping Station No. 6, which in addition to draining a large portion of New Orleans, also drained the area described above; Hoey's Basin. Hoey's Canal was improved over the years, and in 1925 a box culvert was constructed at its entrance to the 17th St. Canal. In 1949 this culvert was increased in size and in 1966, in recognition of its increased contribution to the flow of the S&WB's Pumping Station No. 6, the parish paid for an additional 1000 cfs pumping capacity to be added to the station.

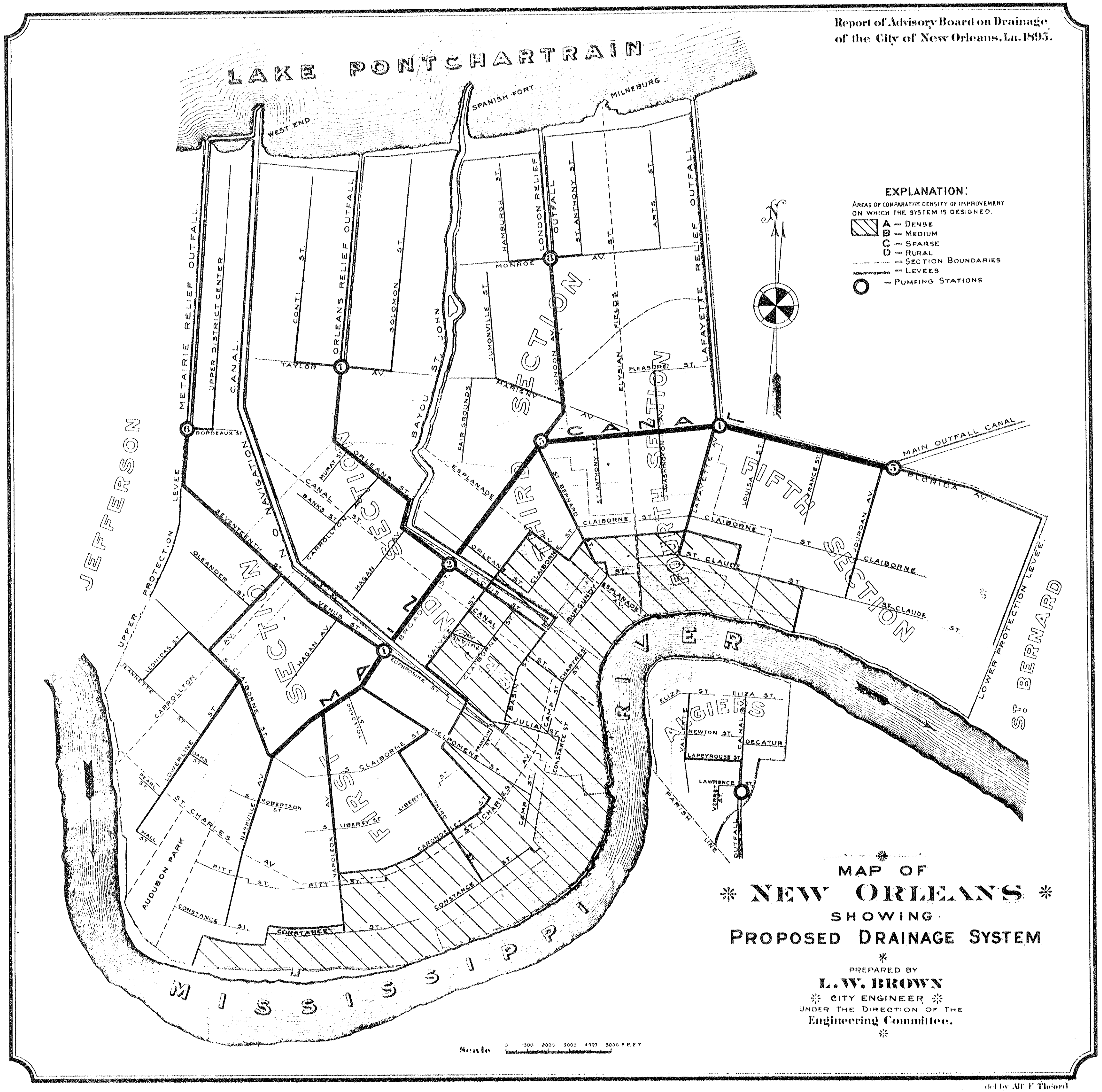
RECENT FLOODING CONCERNS

There has always been a certain amount of flooding in many parts of the study area. However, since with the very heavy rains of May 3, 1978, which caused extensive flooding and millions of dollars in property damage, and the rains of April, 1980 and June, 1981 which also caused considerable flooding and property damage, there has been a concerted effort by officials in both Orleans and Jefferson parishes to improve the existing drainage facilities.

The main concern of Jefferson Parish officials has been that the S&WB might not be able to maintain water levels in the 17th Street Canal that would permit gravity drainage in Hoey's Basin. If this were to be the case, it would then not be productive for the parish to improve the drainage facilities in this area. Therefore, Jefferson officials must be reasonably assured that this can be done before money is spent on drainage improvements in these areas. The 1980 Jefferson East Bank Master Drainage Plan stated that if the water levels in the 17th Street Canal could not be kept to an acceptable elevation, a new pumping station would be necessary. This eventuality would cause a complete reevaluation of drainage facilities.

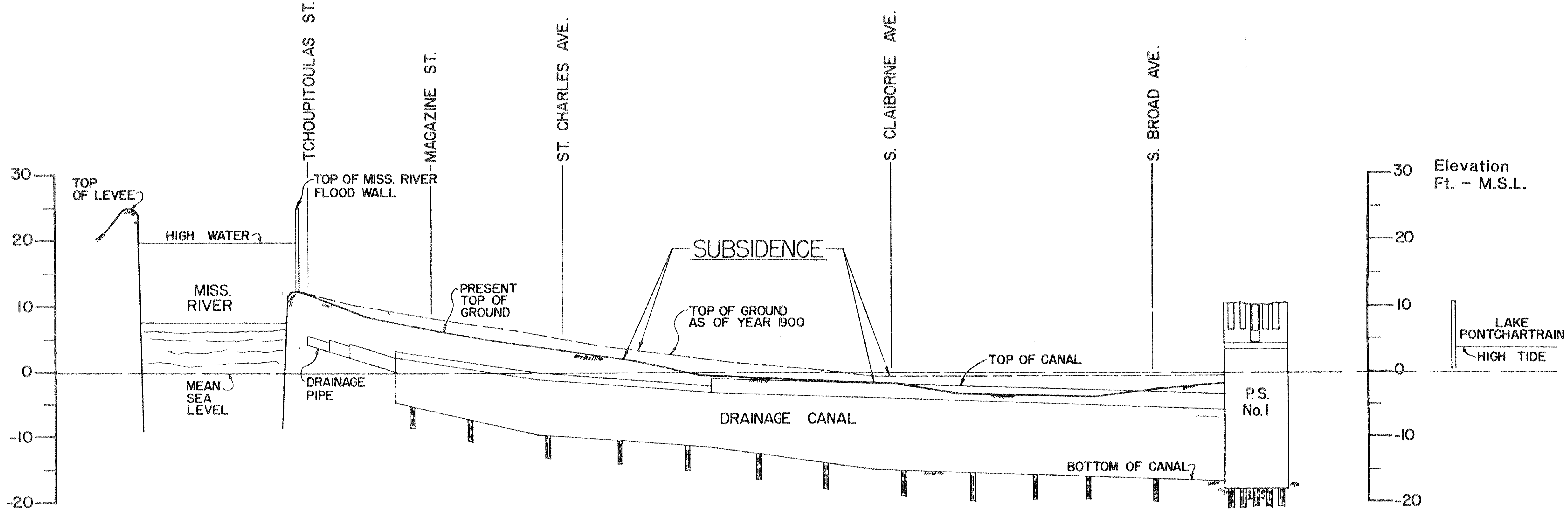
The S&WB officials recognize the need for major improvements to their drainage system and have proposed increasing the capacity of Pumping Station No. 6 and enlarging the 17th Street Outfall Canal. These improvements are intended to be the first steps in upgrading a system that was constructed over a half century ago. The main concern of the Orleans Parish officials at this point is that if these improvements are delayed or prevented, very little further improvement can be made upstream.

PLATE IV



TAKEN FROM "A REPORT ON STORM WATER DRAINAGE WITH SPECIAL EMPHASIS ON THE PERIOD MAY 1978 TO JUNE 1981 AND PROBLEMS ENCOUNTERED" PREPARED BY THE SEWERAGE & WATER BOARD OF NEW ORLEANS, THIS DRAWING INDICATES THE POPULATION DENSITY AND PROPOSED DRAINAGE SYSTEM FOR NEW ORLEANS AT THE TURN OF THE CENTURY.

Figure 1



scale
horizontal : 1" = 1000'
vertical : 1" = 10'

source : "A Report on Storm Water Drainage with Special Emphasis on the Period May 1978 to June 1981 and Problems Encountered" prepared by the Sewerage & Water Board of New Orleans

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Ground Subsidence
in Vicinity of
Louisiana Ave. and Napoleon Ave.

Figure 2

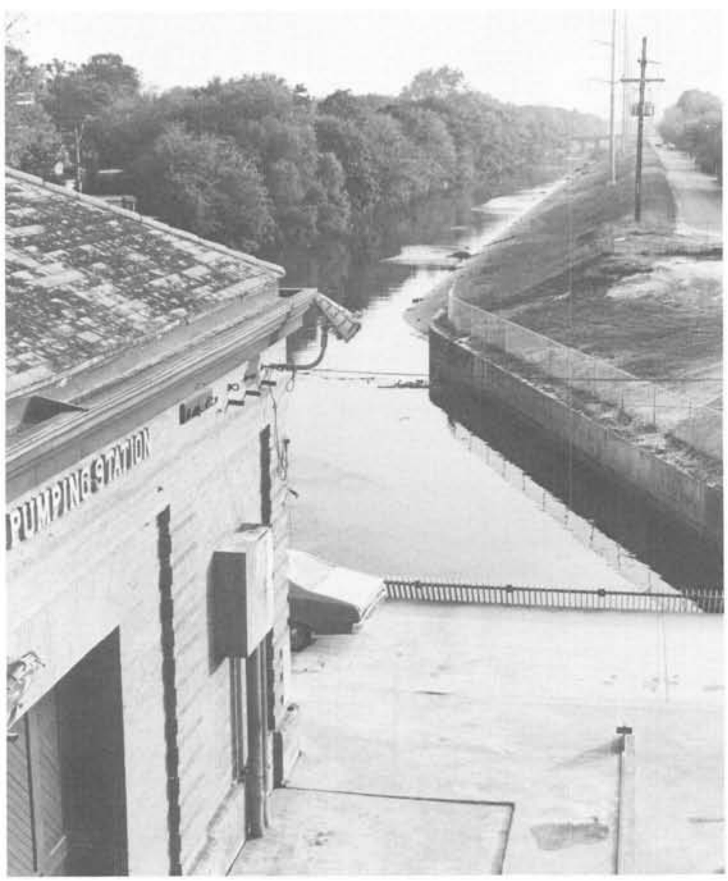
CHAPTER II

EXISTING DRAINAGE SYSTEM

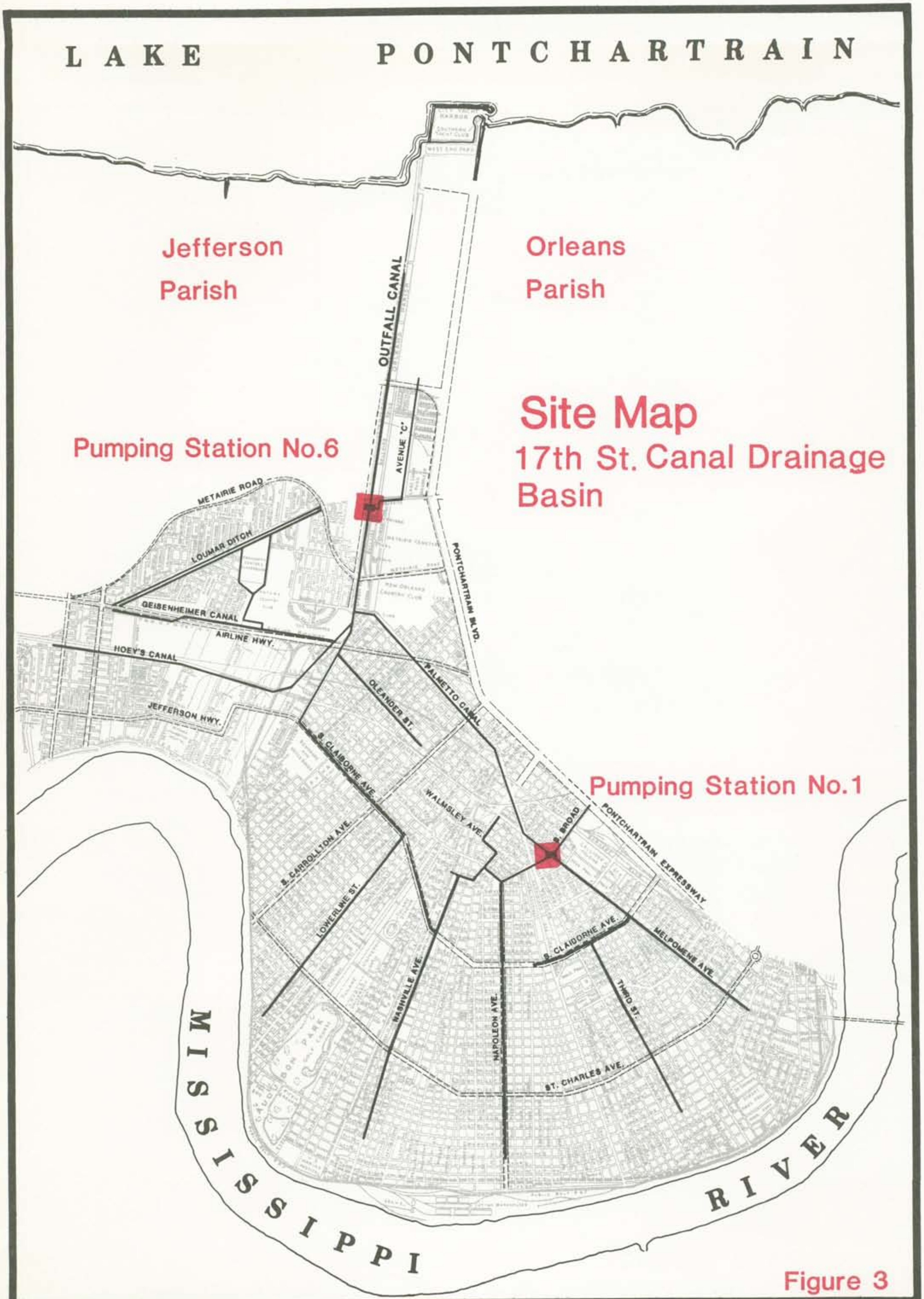
GENERAL DESCRIPTION

The drainage basin under study consists of approximately 2550 acres in Jefferson Parish and 7850 acres in Orleans Parish and is shown on the site map (Figure 3). A general map of the drainage area showing existing ground contour lines is shown on Figure 4. Figure 5 shows the major drainage facilities of the basin as discussed below. These facilities are shown in more detail in Sections 2 and 3 of this report.

The drainage area is subdivided into several major basins, as shown on Figure 6. The largest, consisting of approximately 5500 acres in Uptown New Orleans, drains into Pumping Station No. 1. Station 1 lifts the storm water into the Palmetto Canal which then flows by gravity into the 17th Street Canal and on to Pumping Station No. 6. The remainder of the Orleans portion of the basin, approximately 2350 acres, and the entire Hoey's Basin in Jefferson Parish, approximately 2550 acres, drain by gravity directly into the 17th Street Canal and then to Station 6. Station 6 lifts the water into the Outfall Canal which flows directly into Lake Pontchartrain. A more detailed description of the different portions of the system follows.



Drainage Pumping Station No. 6



LAKE PONTCHARTRAIN
Jefferson Parish
Orleans Parish
OUTFALL CANAL
AVENUE "C"
Site Map
17th St. Canal Drainage Basin
Pumping Station No.6
METAIRIE ROAD
LOUMAR DITCH
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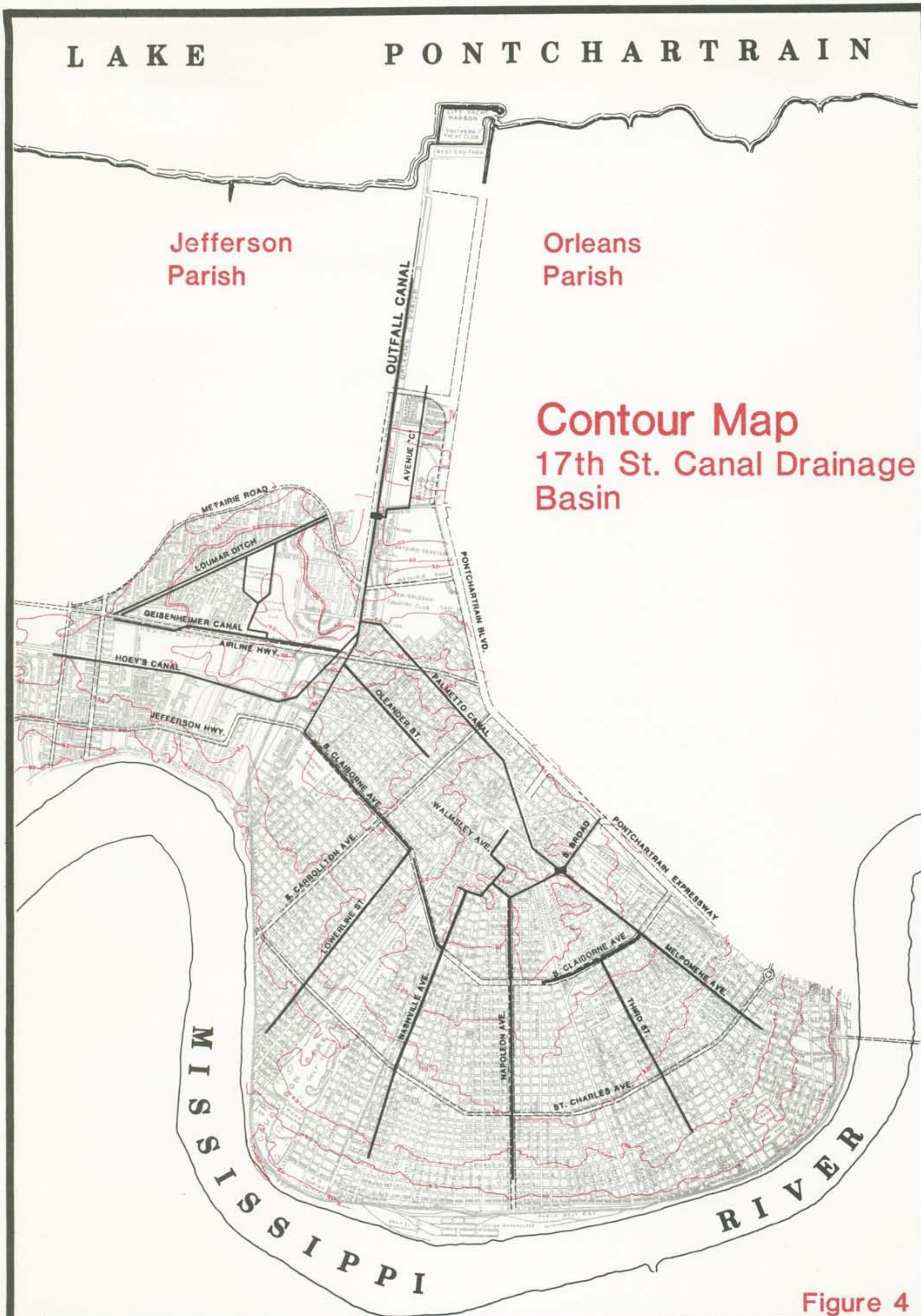
MELPOMENE AVE.
THIRD ST.
ST. CHARLES AVE.
MISSISSIPPI RIVER

Figure 3



LAKE PONTCHARTRAIN
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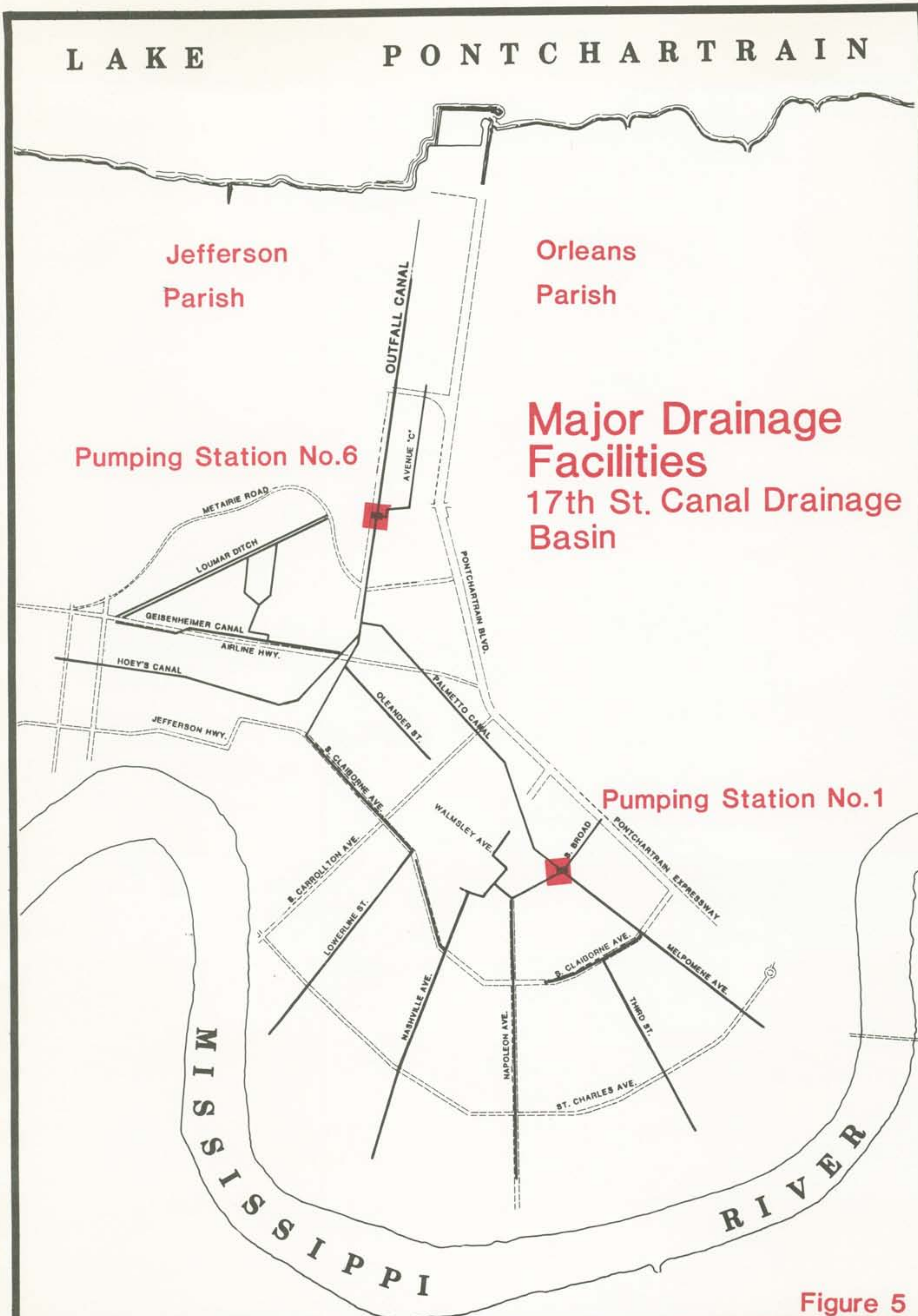
Figure 4



LAKE PONTCHARTRAIN
Jefferson Parish
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Major Drainage Facilities
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Pumping Station No.1
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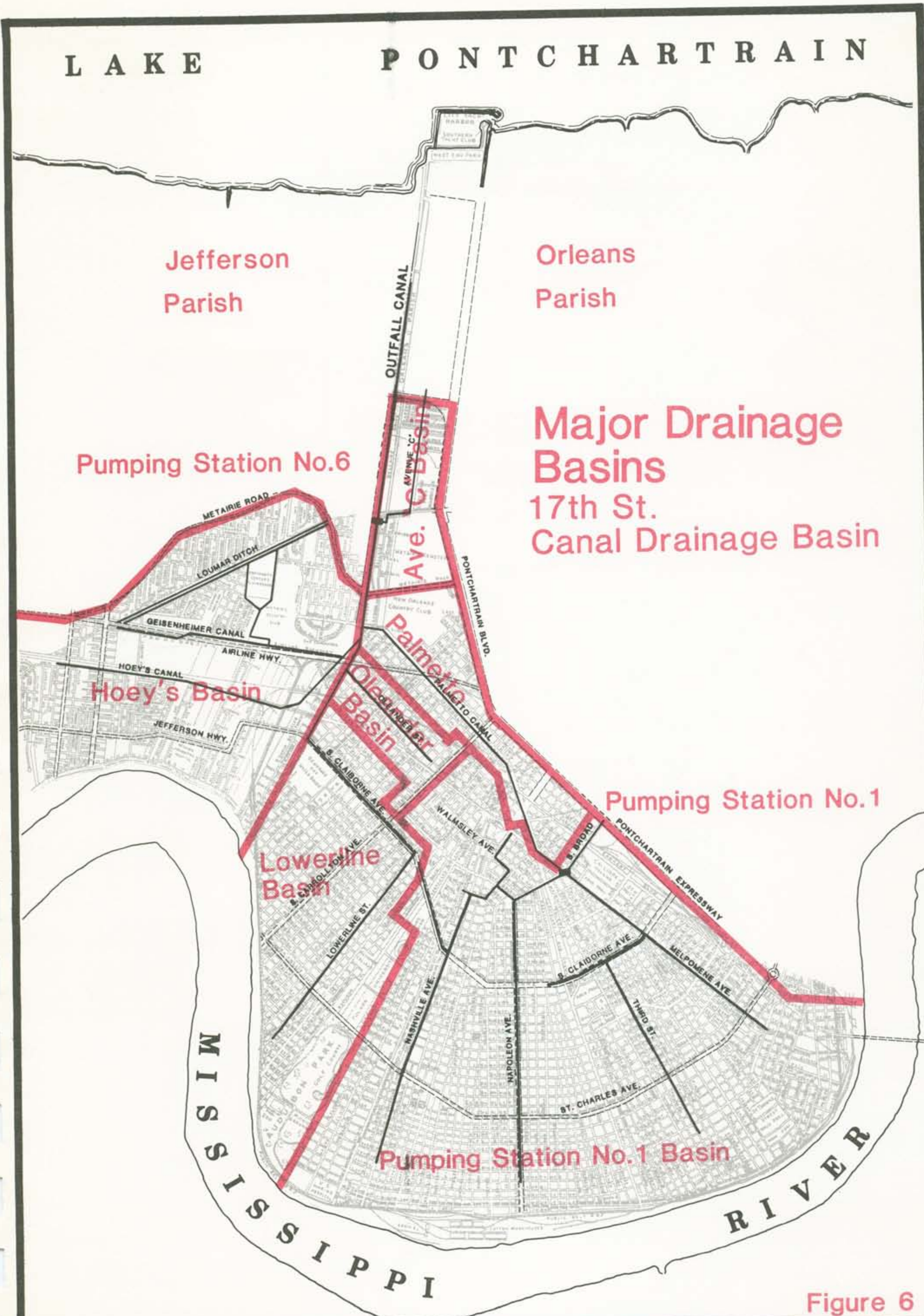
MISSISSIPPI RIVER

Figure 5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NAPOLEON AVE.
S. CLAIBORNE AVE.
MELPOMENE AVE.
THALIA ST.
ST. CHARLES AVE.
Pumping Station No.1 Basin
MISSISSIPPI RIVER

Figure 6

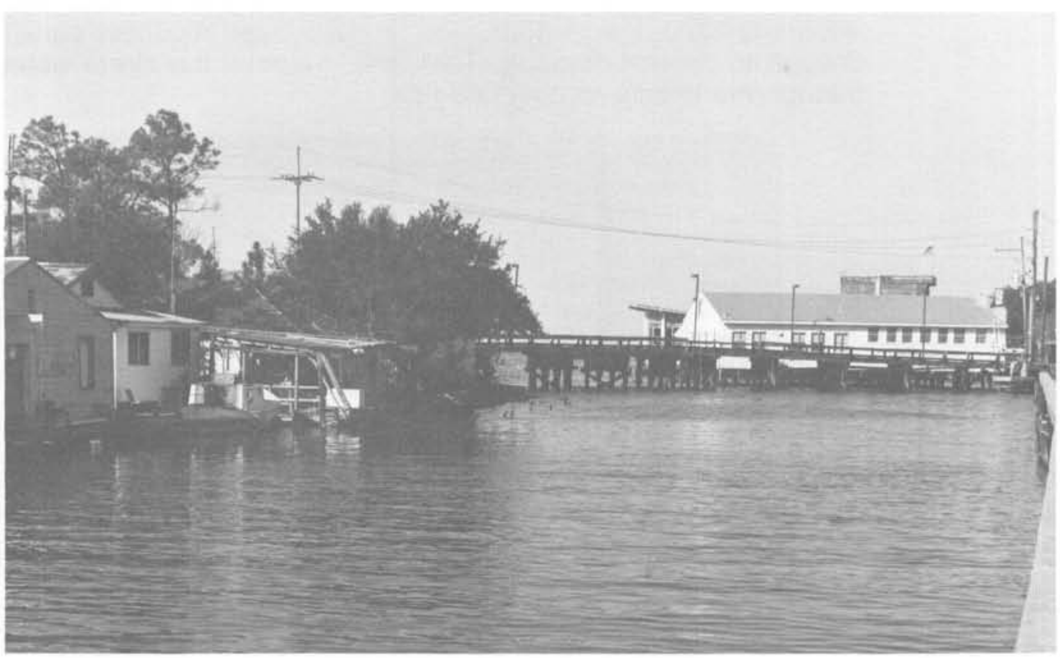
OUTFALL CANAL

The Outfall Canal is an earthen canal, approximately 13,000 feet long, situated on the border of Jefferson and Orleans parishes. It is owned and maintained by the Sewerage and Water Board of New Orleans. It is crossed by a number of bridges, each causing some constriction in the canal. The last 1300 feet of canal before entering the lake has been constricted by fill placed along the Jefferson side of the canal in the area commonly known as Bucktown. The land adjacent to the canal is protected by levees or floodwalls on both sides of the canal, except for the Bucktown area, which is unprotected.

INTERIOR TRANSPORT SYSTEM

The Interior Transport System includes Pumping Station Nos. 1 and 6, the Palmetto Canal and 17th Street Canal above Station 6. Station 1, which is located in the Broadmoor area of New Orleans, has a nominal capacity of 5400 cfs. Station 6, which is one of the world's largest drainage pumping stations, has a nominal capacity of 6650 cfs. This station is now the subject of model studies by the S&WB to determine the best means of increasing its capacity.

The Palmetto Canal, approximately 14,000 feet long, is a concrete-lined open canal which is constructed on piling. This canal presently has multiple crossings, many of which restrict the canal discharge. The S&WB now has plans to remove all but the major street, railroad, and major pipe crossings. It was found, however, that these constrictions alone limit the capacity of the canal to approximately 3500 cfs,



Outfall Canal at Lake Pontchartrain

The 17th Street Canal, extending from Station 6 back to Jefferson Highway has three distinct sections. First, the section between Station 6 and the Palmetto Canal, 3700 feet in length, is a concrete lined open canal with a vertical floodwall on the Orleans side and a levee on the Jefferson side. The second section between the Palmetto Canal and the junction with Hoey's Canal, 430 feet in length, has considerably less cross section and is partially lined with concrete. The third reach of canal, even smaller in section and 4000 ft. in length, is also a partially lined canal. The lined portions of the last two sections are in a poor state of repair and are in need of upgrading.

PUMPING STATION NO. 1 GRAVITY BASIN

The Pumping Station No. 1 gravity basin is the largest gravity basin in the study, containing approximately 5500 acres, and encompassing most of the Uptown New Orleans area. It consists of a pie-shaped area with Pumping Station No. 1 at its point and fans out to the Mississippi River. It is bordered by Carrollton on the west and the Pontchartrain Expressway on the east. Drainage for the area is provided by a series of spokelike underground canals, referred to herein as the Napoleon Avenue System, the Nashville-Broad System and the Third Street-Melpomene System, each extending from Station 1 to Constance Street along the river. The Walmsley System drains the smaller area north of Station 1 and is bounded by South Carrollton Avenue, South Claiborne Avenue, the Pontchartrain Expressway and South Broad Street.

These existing covered canals, except for the newly constructed Melpomene Canal, appear to be inadequately sized to carry runoff from the design storm. Also, in most cases, the existing local street laterals are undersized and design storm runoff is not drained into these canals rapidly enough to prevent flooding. Therefore, much of the storm water travels through the streets as overland flow.

Drainage Pumping Station No. 1

The pie-shaped configuration magnifies the flooding in the low areas near the Pumping Station by funneling runoff from a very large area into this constriction. St. Charles Avenue and South Claiborne Avenue form natural barriers to the overland flow. These barriers tend to reduce flooding at the low point of the system, but localized flooding is caused along these barriers due to ponding. While the existing drainage system in this area is capable of handling the smaller, more frequently occurring storms, extensive flooding occurs during the larger storms, largely because of this overland flow phenomena. It should be noted here that the capacity of the existing Pumping Station No. 1 is adequate to handle the maximum outflows from this existing system and does not appear to be the weak link in the drainage chain.

LOWERLINE GRAVITY BASIN

The Lowerline Basin, containing approximately 1300 acres, bounded generally by Broadway, South Claiborne, the river and the parish line, drains by gravity through an underground canal down Lowerline Street and South Claiborne Avenue into the upper end of the 17th Street Canal. The capacity of this gravity system is critically affected by the water surface elevation in the 17th Street Canal. Under present conditions, mainly because of the general lack of ground slope, portions of the system are undersized for the design storm.

Palmetto Canal

OLEANDER BASIN

The existing system is drained by gravity until the water surface in the 17th Street Canal rises to approximately elevation 6.0 M.S.L. At this point the existing pumping station is activated and the system is pumped. The existing pump capacity is approximately 100 cfs and presently drains approximately 150 acres. The capacity of the existing canal is approximately 200 cfs with the pump in operation. The Dublin Canal, which connects to the Oleander Canal, is not presently part of the system but drains directly into the Palmetto Canal.

PALMETTO BASIN

The areas adjacent to the Palmetto Canal, approximately 500 acres, now drain directly into the canal through a series of small pipes. Because much of the land is below water surface elevations in the canal during heavy rains, the S&WB has installed flap gates on the outlet drains in this area to keep water in the canal from flowing back into the streets. This situation, however, prevents proper drainage of these streets during design storm conditions, as the water must be stored in the streets until peak discharges from Station 1 have subsided.

AVENUE C BASIN

The Avenue C Basin includes approximately 300 acres of land adjacent to the 17th Street Canal between the Palmetto Canal and Veterans Highway. Approximately 200 acres drain into the underground Avenue C Canal which drains directly into the Pumping Station No. 6 suction basin. The remainder, which includes the Metairie Cemetery and the New Orleans Country Club, drains directly into the 17th Street Canal or the Palmetto Canal via several small pipes.

Palmetto Canal

HOEY'S BASIN

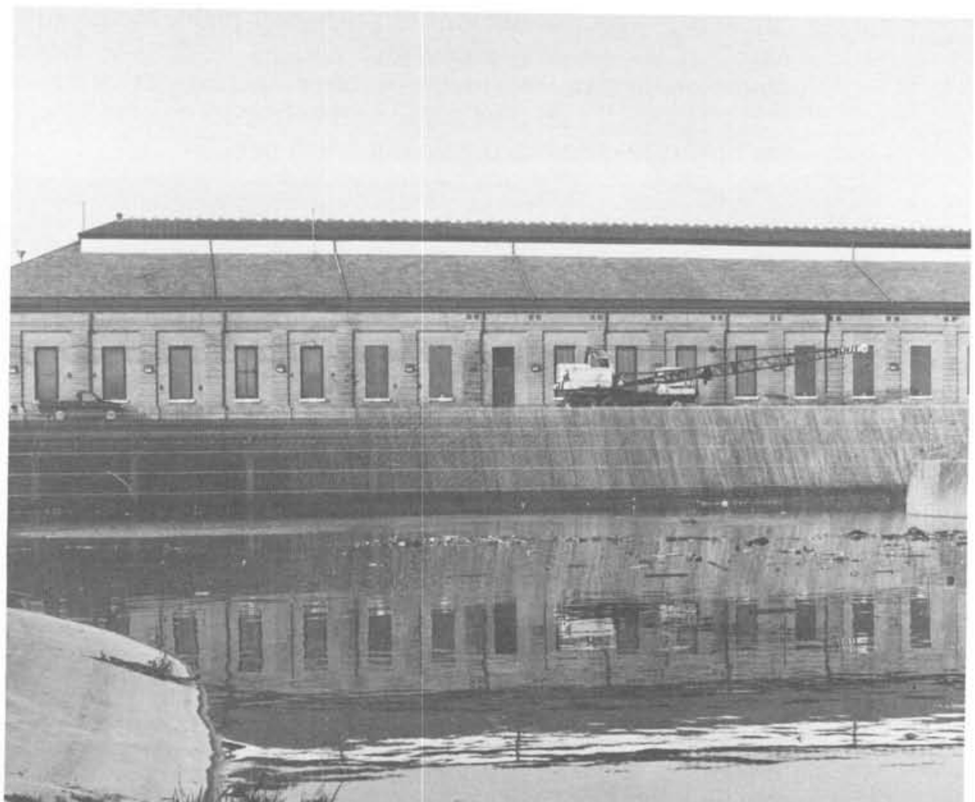
Hoey's Basin includes approximately 2550 acres of Jefferson Parish which drain by gravity directly into the 17th Street Canal. The basin is bordered on the north by a ridge along Metairie Road, on the south by the Mississippi River, on the east by Orleans Parish and on the west by the Arnault Ditch. There are two main collection canals in this basin, Hoey's Canal and the Geisenheimer Canal. Once again because of the lack of ground slope, water surface elevations in the 17th Street Canal are critical to proper drainage of the area.

Hoey's Canal is an open ditch between the Southern Railroad tracks midway between Airline Highway and Jefferson Highway. It drains all of the land between Airline Highway and the river, approximately 1400 acres, before joining with the Geisenheimer Canal and flowing into the 17th St. Canal. Jefferson Highway now forms a major barrier to drainage from the river due to the inadequacy of existing cross drains. At the present time the State Highway Department is preparing plans to remedy this situation. Many of the existing pipes and ditches leading to Hoey's Canal from Jefferson Highway are undersized and must be enlarged to carry increased flows. Hoey's Canal itself is also undersized for most of its length and should be boxed to obtain a consistent cross section.

Box Culvert at Hoey's Canal Entrance to Seventeenth Street Canal

The Geisenheimer Canal is a closed underground box culvert running just north of Airline Highway. It drains approximately 1000 acres between Airline Highway and Metairie Road. The main collection point for this area is the Loumar Ditch, which is an open ditch paralleling the Illinois Central Railroad. This ditch collects runoff from approximately 300 acres between it and Metairie Road. The streets contributory to this ditch have very little subsurface drainage; consequently, runoff is by overland flow down these streets directly into Loumar Ditch. Loumar Road, however, forms a barrier to this overland flow and causes major flooding in this area. Most of the drainage from Loumar Ditch is transported by the Loumar Outfall Canal box culvert into Geisenheimer Canal. The flooding at Loumar Ditch attenuates the peak flows thus limiting peaks in the Loumar Outfall Canal and the Geisenheimer Canal.

The remainder of Hoey's Basin, approximately 150 acres, drains directly into the 17th Street Canal through several small pipes.



Drainage Pumping Station No. 6

CHAPTER III

PRELIMINARY STUDIES

GENERAL

Prior to commencing the detailed hydrologic and hydraulic evaluation of the drainage system considerable preliminary work was necessary. This included the collection of data regarding the existing facilities, ground survey work, establishment of study criteria and selection of methods to be used in the storm water drainage analysis.

COOPERATION WITH RESPONSIBLE GOVERNING BODIES

Before the beginning of the study a Technical Committee was established to meet with the Engineers to discuss and agree upon basic design criteria and study methodology. This committee was made up of technical personnel from the Sewerage and Water Board of New Orleans and the Jefferson Parish Department of Public Utilities.

The committee was instrumental in furnishing the Engineers much of the required data concerning existing facilities, such as drainage network plans, pump data, bench mark information, precipitation data, ground elevations and contours, proposed plans for the area, drainage reports and other pertinent information.

The Engineers and the Technical Committee also met with the Corps of Engineers to coordinate with them and to inform their representatives of the progress of the study and to answer any questions.

SURVEYS

Field surveys were made to obtain additional survey information that was not available through the above mentioned agencies. This included a check on existing bench marks in both Orleans and Jefferson parishes to determine if they were based on the same datum. The S&WB bench mark No. 111, located at Pumping Station No. 6, was used as the datum. Elevations determined for the bench marks checked indicated that they were on the same datum and that there was very little difference between published elevations and those measured. Table 1 gives the results of these surveys.

Other surveys made included a profile and cross sections of the Palmetto Canal, Hoey's Canal, and several spot checks of the Outfall Canal and of the subsurface drainage canals in Orleans and Jefferson parishes. Cross sections of the 17th Street Canal were furnished by the S&WB.

Existing ground contour lines were obtained from the Corps of Engineers aerial maps and are presented in feet, mean sea level (M.S.L.). Canal cross sections and lake elevations were obtained as Cairo Datum (C.D.) elevations. (Elev. 20.43 C.D. = Elev. 0.00 M.S.L.) Since the data was obtained differently for different segments of the system, water surface profiles in the open canals are presented in the Cairo Datum and water surface elevations in the subsurface collection systems are presented in elevation mean sea level.

TABLE 1
BENCH MARK SURVEY
(Surveyed May 1982 — BFM Corp.)

LOOP No. 1 — New Orleans

Bench Mark No.	S&WB Elev.	Elev. (Surveyed)	(Difference)
1. #111	22.43 CD		
2. #117	26.26	26.28	(+0.02)
3. #122	19.23	19.32	(+0.09)
4. #282	19.81	20.01	(+0.20)
5. #316	22.18	22.26	(+0.08)
6. #411	17.96	17.99	(+0.03)
7. #404	22.65	22.73	(+0.08)
8. #334	24.66	24.56	(−0.10)
9. #319	22.52	22.70	(+0.18)

Bench Mark loop run using BM #111 (Pump Sta. No. 6) as datum. Error of closure (approx. 10 mi. run) was 0.03.

LOOP No. 2 — Jefferson Parish

Bench Mark No.	Jeff. Elev.	Elev. (Surveyed)	(Difference)
1. #277 (LGS)	6.926 MSL	6.92 MSL	(−0.01)
2. U-147	3.999	4.13	(+0.13)
3. A-276	7.103	7.29	(+0.19)
4. Y-188	2.651	2.09	(−0.56)
5. M-188	4.554	4.77	(+0.22)
6. Q-147	3.668	3.89	(+0.22)
7. L-147	1.499	1.67	(+0.17)
8. K-147	1.781	1.94	(+0.16)
9. P-147	0.981	1.08	(+0.10)
10. A-3188 (LGS)	3.173	3.27	(+0.10)
11. A-3185 (LGS)	3.783	3.37	(−0.41)
12. A-3184 (LGS)	1.716	2.05	(+0.33)
13. N-147	−1.204	−1.01	(+0.19)
14. 276 (LGS)	8.37	8.16	(−0.21)

LOOP No. 2 — New Orleans

Bench Mark No.	S&WB Elev.	Elev. (Surveyed)	(Difference)
1. #5	28.85 (8.42)	8.21 MSL	(−0.29)
2. #132	30.20 (9.77)	9.48	(−0.21)

Bench Mark loop run from BM #277 (S&WB). Closure error (in approx. 10 mi.) is given as +0.04'.

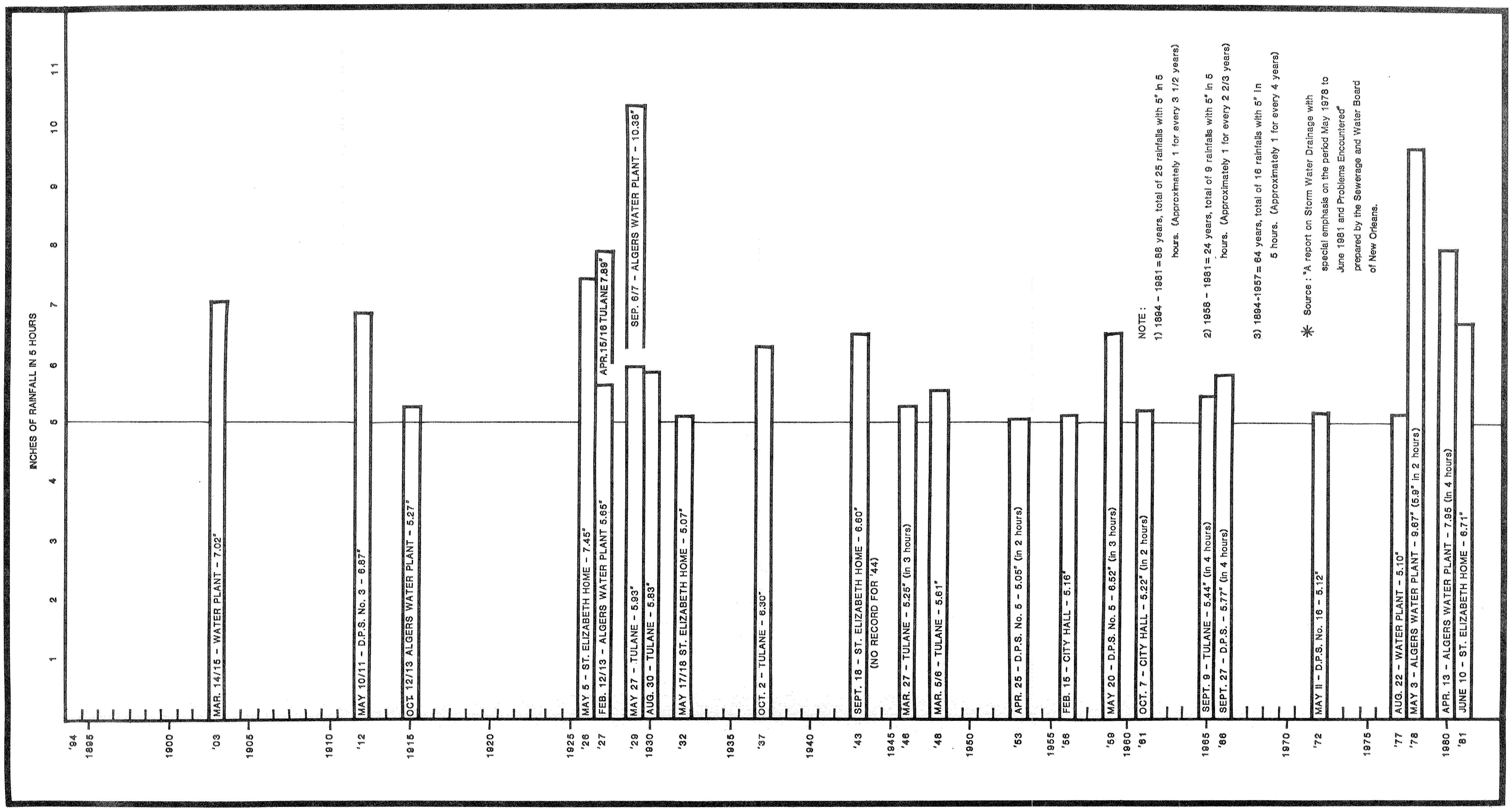
Datum for all BM's is S&WB BM #111 (Pump Sta. No. 6) — Elev. 22.43 C.D. (2.00 MSL).

DESIGN STORM

Before the existing drainage system can be properly evaluated there must be some design criteria against which the capacity of the system can be measured. The best criterion for drainage studies of this sort is a design storm. This design storm should be of sufficient size and frequency to reasonably reflect the considerable damage incurred by drainage areas not properly protected. It should not, though, be so large as to make preventive measures prohibitively expensive.

The Technical Committee was in full agreement on the particular design storm to be used for the study. The storm selected produces approximately 5 inches of rain in a 5 hour period. The recurrence interval of the design storm, depicted on Figure 7, was obtained from S&WB records. It indicates that rainfall of this magnitude occurs relatively frequently in the Greater New Orleans Area. The rainfall intensity-duration curve for this selected storm is shown in Figure 8. This curve is approximately the same as that selected by Jefferson Parish in its 1980 East Bank Major Drainage Plan and is the same as that used by the S&WB for the design of its major facilities.

A rainfall hyetograph was developed from the intensity-duration curve selected and is shown on Figure 9. The method used in the development of the hyetograph is based upon a method used by the Corps of Engineers. This method is explained in Appendix A. The hyetograph developed will give the maximum peak runoff for the design storm. The storm begins with lesser rainfall intensities in the beginning of the storm. This tends to fill the available depression storage and to saturate the ground before the peak intensities occur. The peak intensities occur in the second and third quarter of the storm. These peak intensities will result in the highest possible runoff, since most possible abstractions will have been satisfied.

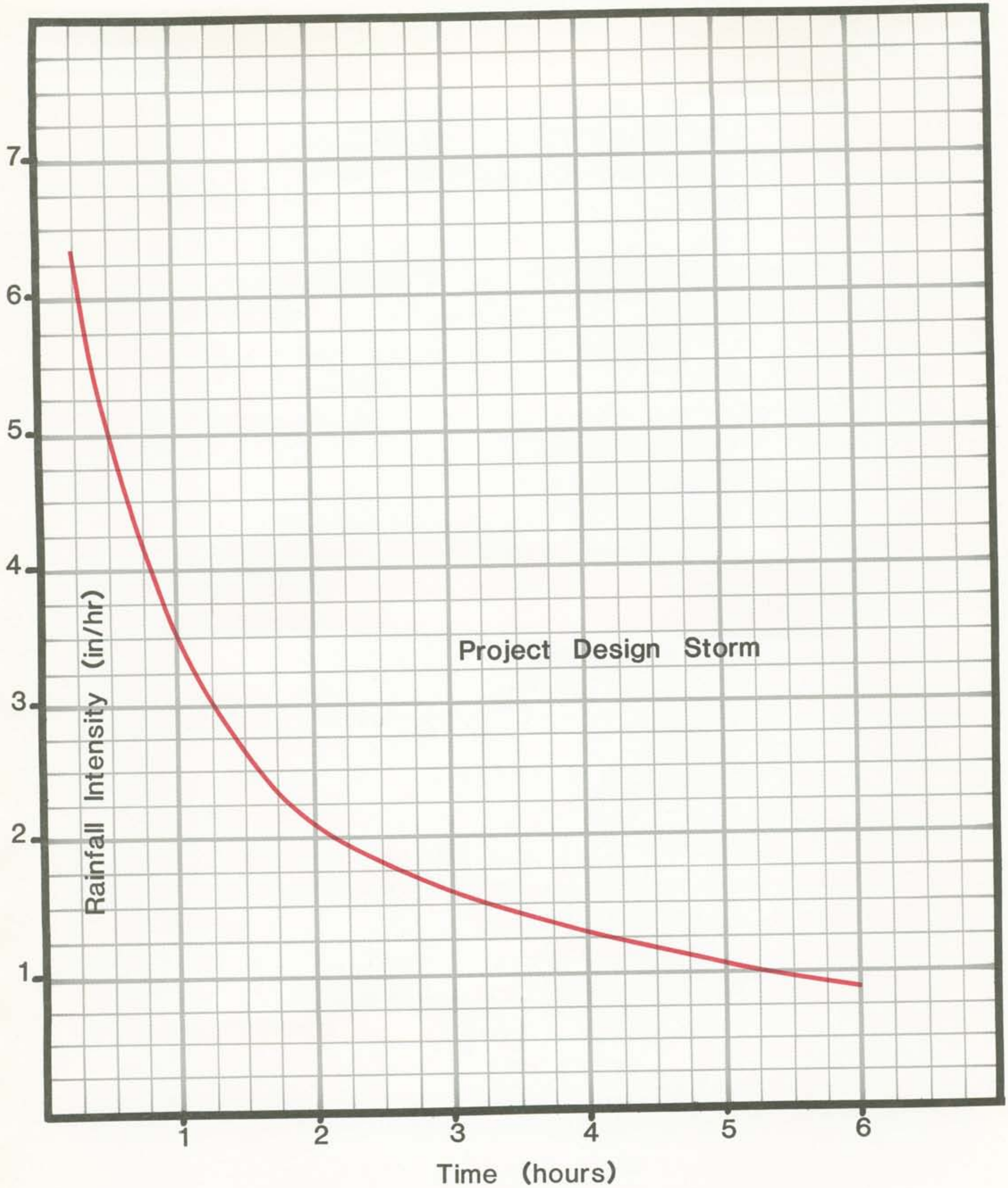


Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

1897 - 1981
N. O. RAINFALL
5 INCHES IN 5 HOURS

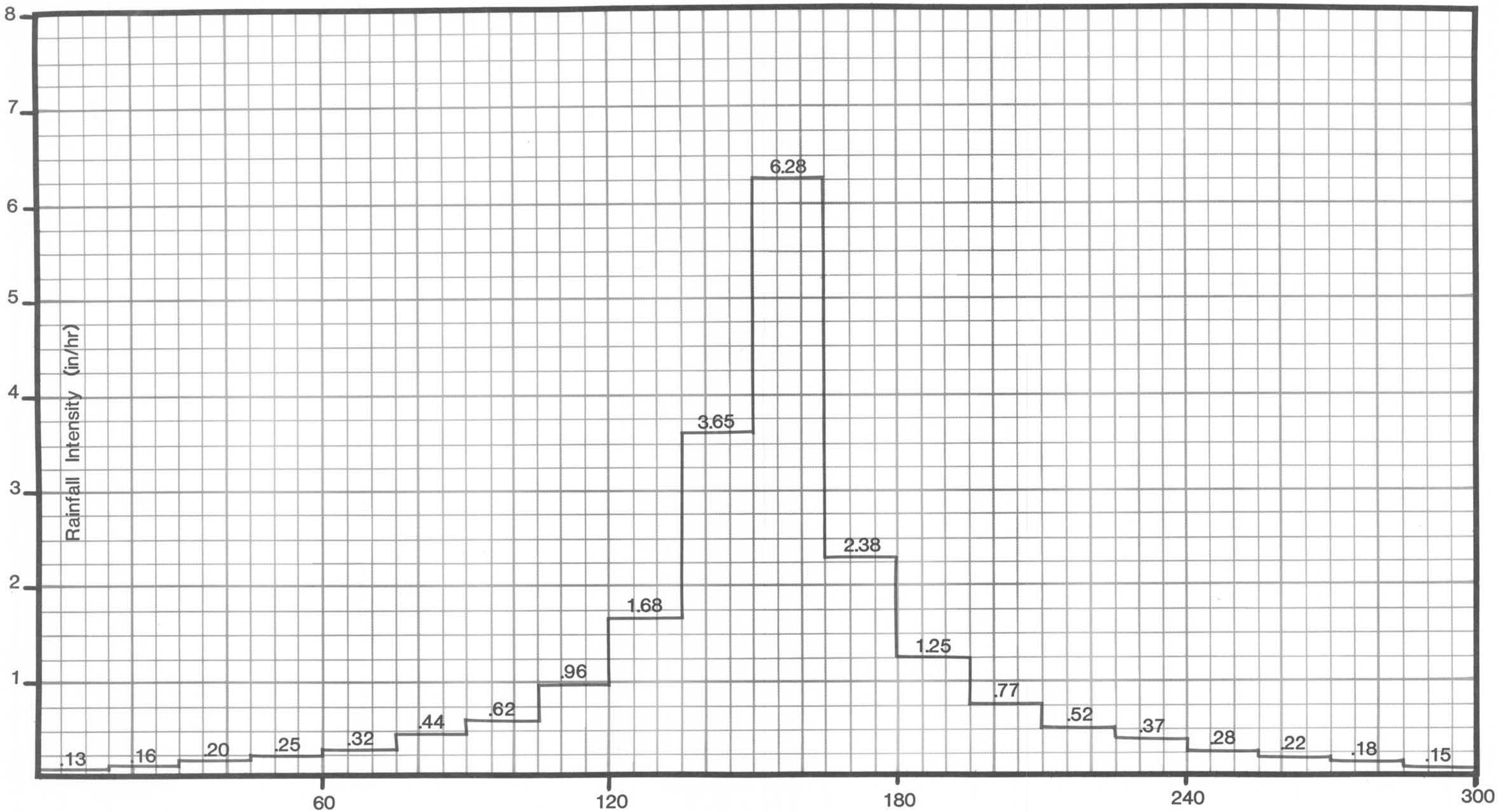
Figure 7



Design Rainfall Intensity vs. Duration

Seventeenth Street Canal Drainage Basin Study

Figure 8



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hyetograph
Design Storm

Figure 9

OUTFALL CANAL HYDRAULIC CRITERIA

In the review of the Outfall Canal hydraulics, it was of primary importance to select the Lake Pontchartrain elevation to be used during the design storm. Several elevations taken from the Lake Pontchartrain West End Gage were considered, and their appropriateness evaluated. These elevations are listed in Table 2 in Cairo Datum.

TABLE 2
LAKE PONTCHARTRAIN WATER ELEVATIONS

Mean Low Water	21.33 C.D.
Mean High Water	21.92 C.D.
Normal Lake Level (per S&WB)	22.50 C.D.
Avg. Yearly High Water (20 year record)	24.70 C.D.
High Water — Hurricane (1965 — Betsy)	25.80 C.D.

It was agreed by the committee that the average of the yearly high water elevations (Elev. 24.7 C.D.) should be used in the study to determine design storm water levels in the canal system. The "Normal Lake Level" as defined by the S&WB was used to check for maximum allowable velocities occurring under these conditions.

Prevention of damaging scour in an earthen canal is a major consideration in design. In reviewing alternative designs for the Outfall Canal, the following maximum velocities were agreed upon by the Technical Committee.

Desirable Velocity (Earthen Canal) — 3.0 fps.
Maximum Velocity (Earthen Canal) — 3.5 fps.
Maximum Velocity (Concrete Lined Canal) — 10.0 fps.

PROPOSED IMPROVEMENTS

In order to calculate the peak discharge from the design storm it was necessary to determine the extent of improvements to be assumed in place. It was agreed upon by all members of the Technical Committee that, for any particular major drainage component in the drainage basin, its proposed design capacity should consider that all major drainage facilities upstream of, and contributary to, that component would be improved to the point where no surcharging existed. It was also agreed that all improvements proposed in either the Sewerage and Water Board of New Orleans or the Jefferson Parish Capital Budget for the next 5 years would be assumed to be in place.

It is noted here that the local street drainage system has been simulated by a set of typical Standard Streets in which certain size drain lines are assumed. These Standard Streets provide all of the runoff to the major components in the drainage system. The pipe sizes assumed are the sizes that would generally be installed today by both Orleans and Jefferson Parishes, although this may not match the existing drains. This assumption will allow some flooding in these Standard Streets since runoff is limited to the capacity of the assumed pipe sizes on the assumed slope. This Standard Street method as agreed upon by the Technical Committee and later validated by the "calibration" process, is described later in Section 2.

STORM WATER ANALYSIS

As part of the preliminary segment of the study, it was necessary to establish the methods to be used in the analysis of storm water runoff. This could best be described in the form of a hydrograph, which is a plot of runoff versus time. Because of the large size and complexity of the drainage system under study, a computer oriented analysis was warranted. In addition, there were two distinct and separate hydraulic problems to be analyzed in the study. First was the analysis of the collection system, which consists of streets, gutters, closed subsurface conduits and open ditches; and second was the analysis of the transport system, consisting of large open canals and pumping stations.

Analytic methods for urban drainage systems range from simple calculation techniques for individual conduits to sophisticated continuous simulation techniques. Procedures have been developed which permit hydrographs to be generated from even the simplest methods, such as the Rational Method, but the value of such a hydrograph is questionable since it bears no relation to an actual storm pattern. A representative sampling of analytic techniques is presented below, with individual discussion of each method.

The Rational Method is one of the simplest techniques available. It is considered suitable for small areas (up to 1 sq. mi.) provided no storage exists in the system. Data requirements are minimal, rainfall is introduced in the form of an average intensity-duration curve, and all abstractions from rainfall are contained within the runoff coefficient (c) which is established based upon gross physical characteristics of the drainage area (slope, type of development, etc.). The procedure calculates the discharge which would result at each point in the system from a rainfall corresponding to the average intensity during a period equal to the time of concentration for the area tributary to that point. The analysis proceeds stepwise from upstream to downstream yielding a peak flow at each point. While it is possible to synthesize a hydrograph from the Rational Method, this is not usually done.

The SCS Method was originally developed for rural areas but has been modified for application to urban and urbanizing regions. Flow is calculated based upon the duration of excess rainfall (rainfall in excess of abstractions), rainfall depth, area (and areal adjustment of rainfall depth), and frequency of design storm. The major data requirements include the curve number (CN), hydraulic length of each subcatchment, and the average slope of the area. The curve number is a runoff coefficient of sorts (although it has other functions as well). The curve number includes effects of infiltration and detention storage and may be modified to account for antecedent moisture conditions. In addition to influencing the peak runoff, the curve number also determines the duration of excess rainfall. The curve number is determined from tabulated values based upon the soil and the degree of urbanization. Detailed procedures are presented in SCS Technical Release No. 55.

The design rainfall is based upon return period and duration. From the selected value, the direct runoff can be determined from prepared graphs and tables. A design hydrograph is then synthesized, with the general form being a function of curve number and rainfall. The information needed to define the hydrograph includes the time of concentration (overland flow plus channel flow) which is determined from the curve number and the average slope. The time can be adjusted for various degrees of urbanization using a pair of graphs. The SCS technique yields a hydrograph of standard shape which permits determination of both peak flow and total volume of runoff.

The standard SCS procedure is not useful for design of small drainage conduits but can be used to advantage in analyzing the discharge of large areas. Data requirements are quite small. The SCS has developed a computer program (published in Technical Release 20) which can route the hydrographs from individual subareas through multiple structures and stream or channel reaches. The program is more suitable than the basic SCS method when the area exceeds 2000 acres, the area is hydrologically complex, and it is desired to analyze historical events.

HEC-1 is one of a family of hydrologic/hydraulic computer models which have been developed by the U.S. Army Corps of Engineers. The HEC-1 model has undergone considerable modification since it was first developed in 1967. The most recent edition, published in 1981, has the capability of modeling the components of urban drainage systems.

HEC-1 offers a number of internal options for generating hydrographs and for routing the flow. The basic input data include a hyetograph, definition of an infiltration/detention function, and sufficient hydraulic detail to permit routing by the selected technique. The program includes a number of storage routing techniques and can model pumping from one basin to another.

The hyetograph can be an historical record or may be synthesized from a duration-intensity curve. Interception-infiltration-detention can be modeled by any of four techniques: initial abstraction plus uniform loss rate; exponential loss rate; the SCS curve number; or the Holtan loss rate. Only the Holtan option permits consideration of multiple storms. Surface flow may be determined by either a unit hydrograph technique or a kinematic wave approximation.

Three unit hydrograph techniques are available: the Clark Unit Hydrograph, requiring three input parameters; a modification of the Snyder Unit Hydrograph, requiring two parameters; and the SCS dimensionless unit hydrograph, requiring a single parameter (which may not be readily obtainable). All the hydrograph techniques require historical data for calibration of the model.

The kinematic wave approximation requires definition of the hydraulic characteristics of all system elements: overland flow, collector channels, and main channel. Values for roughness, hydraulic radius, area, and slope must be introduced into the program. The calculation procedure involves a solution of the finite difference approximation to the kinematic wave equation. The solution is not necessarily correct since the kinematic wave approach does not permit modeling of backwater effects. The solution proceeds downstream. The subbasin may be considerably simplified by neglecting minor collectors. In such a case, the model must be adjusted by comparing historical data to predicted discharges.

The urban modeling capability of HEC-1 is incidental to its major functions. The model does not have the resolution capability of SWMM or the Illinois Storm Runoff Method.

HEC-2 is the second in the series of computer models developed by the Corps of Engineers. HEC-2 is not a storm-water model, but rather a computerized technique for determining water surface profiles for conditions of gradually varied steady flow. HEC-2 is not useful in developing actual runoff data but may have application in analysis of the major canals downstream of pumping stations. HEC-2 calculates water surface profiles using the standard step method but is also able to incorporate the effects of bridges, culverts, and other obstructions to flow. The storm-water models discussed herein do not have such a capability.

HEC-2 calculates friction losses based upon one of five options. The fifth option directs the program to use whichever of the four methods is best suited to the particular reach in question. Losses through bridges and other obstructions can be determined using either the "normal" bridge or "special" bridge method. The special method is most applicable to crossings of the sort found in the larger drainage canals.

HEC-2 can be used to determine multiple water surface profiles for various flows. These profiles can then be used in conjunction with HEC-1 to route actual hydrographs through the canals using a modified Puls storage routing technique. In the case of the 17th Street Drainage Basin, this technique was not found to be necessary. The flatness and length of the peak of storm-water hydrographs from this large drainage system makes routing of the flow in the canals relatively pointless. Since the peak is so broad, no significant attenuation will occur and the only difference between the peak at the upper ends of the major canals will be the time of occurence. For this reason HEC-2 can be used alone to determine the water surface profile resulting from peak flow.

STORM is a Corps of Engineers model which was developed originally to assist in estimating pollutant loadings resulting from urban runoff. STORM is a continuous simulation model which permits analysis of multiple storm events. The hydraulic routing procedure is far simpler than HEC-1 and therefore requires less data and yields less definitive results.

Runoff may be computed in STORM by three methods: a coefficient method similar to the Rational Method with an adjustment for depression storage; the SCS method, including adjustment for antecedent moisture; and a combination of two using the coefficient method for impervious areas and the SCS method for porous areas.

Flows are routed through each sub-basin using the SCS triangular unit hydrograph. In order to use this procedure the time of concentration and the ratio of recession time to peak time must be entered in the program.

The STORM program does not permit detailed modeling of hydraulic features of the drainage system. Its major intended use is planning level analysis of storm water pollution. The 1977 version does not permit channel routing nor combination of sub-basins.

ILLUDAS is the U.S. Version of the RRL method developed in Great Britain. The model requires that the basin be broken down into sub-areas tributary to collection system inlets. Each sub-area is subdivided into directly connected paved areas, contributing grassed areas, and supplemental paved areas.

The program calculates inlet hydrographs from a hyetograph and descriptive data for the basin. Losses are calculated as an initial abstraction plus infiltration based on Horton's equation. Overland flow times are based upon Manning's equation for paved areas and Izzard's overland flow hydrograph for grassed areas. Required data include length and slope of the sub-basin, coefficients for Horton's equation, and percentages of paved, grassed, and supplemental paved area. In general, the required data is available from physical inspection of the area.

Routing within the collection system may be effected by three different procedures: lagging without storage; storage routing with explicit solution of the continuity equation based upon an assumption which is not justifiable in a system such as the 17th Street Canal area; and storage routing with implicit solution of the continuity equation in a fashion analogous to the modified Puls method.

The hydraulic routing procedure is the weakest feature of ILLUDAS. The program has a design capability which will size required conduits. In the analysis mode, however, there is no carry-over procedure for water which is unable to enter the system. Flooding is thus not definable. The program will indicate a conduit is inadequate but will not give any measure of the extent of flooding. Water which is "lost" at the inlets is lost from the simulation and thus non-conservative errors can be introduced into subsequent computations.

The Inlet Method employs a triangular synthetic hydrograph which is developed based upon an assumed hyetograph. The technique presumes that the time of concentration of the subarea is less than the duration of the "intense" period of the design storm. The triangular hydrograph is the basis of a number of similar techniques.

As the method was applied in the East Bank Drainage Study, peak runoff was adjusted to reflect a perceived variation in the rational method runoff coefficient with intensity, and hydrographs were routed in pipes using a theoretical relationship which does not reflect conduit storage capacity. The resulting routed hydrographs were then lagged to account for travel time in the major canals. The effect of storage capacity of the canals upon the final hydrograph appears to have been neglected.

The inlet method requires little data beyond that required by the rational method. The hydrographs which are obtained are different from, but not inherently superior to, those which are obtained in some modifications of the Rational Method. The routing techniques used in the East Bank Study are not fundamental to the inlet method.

The major disadvantages of the inlet method include its inability to model historical storms and its inability to account for surface detention and flooding during storms exceeding design level. Its major advantage is the minimal data requirement.

SWMM (Storm Water Management Model) has been developed over an extended period by Metcalf and Eddy, Water Resources Engineers (CDM), and the University of Florida. SWMM, like HEC-1, has substantial capabilities beyond simple storm water modeling.

Rainfall is introduced in the form of hyetographs — which can be varied for different sub-areas. Abstractions are based upon detention storage and infiltration calculated from Horton's equation. When rainfall exceeds losses, overland flow is calculated from Manning's equation. The calculation requires specification of a "characteristic width" for each subcatchment as well as roughness, slope and length. Overland flow is routed using the continuity equation in storage form, which implies that depth changes uniformly over the subcatchment. SWMM is the first of the methods discussed here which considers storage in routing overland flow.

The overland flow may be routed to gutters, or these may be neglected, with flow routed directly to the sewers. The gutter routing, if used, employs a quasi-steady-state method based on Manning's equation and the continuity equation.

The output from the Runoff block of the program is then routed through major collectors using a non-linear kinematic wave procedure employing a four-point implicit difference scheme and a dynamic wave approximation. The hydraulic calculations are superior to those used in HEC-1 but overall backwater effects are not modeled.

The program incorporates storage and pumping simulation capabilities similar to those in HEC-1 and can operate in either design or evaluation mode. A significant feature of SWMM is its ability to keep track of flows in excess of conduit capacity. Excess flows are "stored" at the next upstream manhole until the system can accommodate them. The program will print out messages at each time step for each location at which surcharging occurs. These messages report the volume of water stored and can be used to estimate the extent of flooding.

SWMM has capabilities which exceed those of the other techniques discussed above. Data requirements can be minimized by the use of default values and by simplification of the Runoff module. The model can be "tuned" or calibrated by use of historical rainfall-runoff data. The method was used in the West Bank Master Drainage Study recently prepared for Jefferson Parish.

IUSR (Illinois Urban Storm Runoff Method) is one of the most sophisticated models available. Like SWMM and HEC-1, the model has capabilities beyond those of interest in this study.

Input hyetographs supply water to subcatchments consisting of linear strips, and to gutters. All surface flows are modeled by the continuity equation and the kinematic wave approximation of the momentum equation. The difference equations are solved numerically in a manner similar to that used for major conduits in SWMM and HEC-1. The technique, while sophisticated, is not necessarily superior to use of the Manning or Izzard equations. No technique used for surface flow in these models will simulate backwater effects.

IUSR models inlets explicity as weirs, orifices, or a combination thereof. Other models do not include inlet effects. Flow in excess of inlet or conduit capacity is either stored at the inlet or bypassed to the next inlet downstream.

The model requires substantial data including geometric characteristics of subcatchments, gutters, and inlets. The model, like SWMM, will accommodate different hyetographs for different subareas.

The sewer routing subroutine, like that in SWMM, may be operated in either design or analysis mode. In either case the hydraulic calculations are conducted in a fashion which employs the St. Venant equations and a method of successive approximations. This procedure permits modeling of backwater effects to a degree, but does not yield to a precise solution. The solution may, however, be better than that provided by SWMM or HEC-1.

IUSR requires substantially more data and more computer time than SWMM, HEC-1, or ILLUDAS. The data required is unlikely to be available, and assuming values for the necessary parameters will tend to offset the potentially greater precision of the calculations.

Other Methods include the Dorsch Hydrograph-Volume Method, a proprietary model intermediate in complexity between SWMM and IUSR; the Chicago Hydrograph Method, one of the first computerized analytic methods; the Cincinnati Urban Runoff model, which is conceptually similar to the Chicago Method and considerably less useful than SWMM or IUSR; and various unit hydrograph techniques, usually tailored to particular communities.

The review of available methods presented above permits elimination of certain models. The Rational Method, Inlet Method, and STORM are not adequate for a system as large and complex as that of the study area; the Chicago and Cincinnati methods do not consider storage capacity of major conduits; the unit hydrograph method requires substantial local data, would need to be tailored to the study area, and cannot readily model the effects of proposed improvements; the Dorsch model is proprietary; IUSR requires data far in excess of that which is readily available, and uses substantially more computer time than HEC-1, SWMM, ILLUDAS, or the SCS method.

The SCS method is based upon a standard storm rather than a specified hyetograph. Although the routing method in Technical Release 20 can be used to route complex hydrographs, the input data from the SCS method will not, in itself, permit modeling of historical rainfall-runoff events. The SCS method is most useful in estimating development effects for urbanizing areas. Since the study area is substantially developed, it is less useful than techniques specifically tailored to urban areas.

ILLUDAS, HEC-1 and SWMM are all capable of modeling urban areas. Of the three, SWMM is the most sophisticated and has the best hydraulic and hydrologic capability. ILLUDAS requires less data but is unable to model flooding. HEC-1 has somewhat less modeling ability than SWMM, but requires more or less equivalent data. SWMM appears to be the best technique available for modeling of the gravity drainage system.

HEC-2 is the only technique among those reviewed here which will permit ready modeling of the major canals. HEC-2 not only will model backwater effects, but also includes the effects of bridges and other obstructions. None of these features are available in the other models. It has been concluded, based upon the review presented above, that the best simulation of this complex drainage system will be achieved by the joint use of HEC-2 and SWMM. SWMM will be used to develop hydrographs for the gravity drainage system and HEC-2 will be used to simulate the major canals.

SECTION 2

COLLECTION SYSTEM

SECTION 2

CHAPTER IV
DESCRIPTION OF SWMM MODEL

INTRODUCTION

The Storm Water Management Model (SWMM) version III, updated in November, 1981 was used to model storm water flow in the subsurface gravity drainage systems of Orleans and Jefferson Parishes. The model is organized into computational blocks which may be operated independently of each other. In the simulation conducted for this study only the Runoff Block and the Transport Block were used.

RUNOFF BLOCK

The Runoff Block uses a defined physical system and an established rainfall pattern (hyetograph) to calculate the storm water discharge from subareas (subcatchments) of the drainage basin. The definition of the physical system requires establishment of values for specific parameters. These parameters include the following:

a) Subcatchment Area — the area in acres of each subcatchment. Discharge varies directly with area.

b) Subcatchment Width — the width of each subcatchment perpendicular to the direction of overland flow. This distance defines, indirectly, the depth of overland flow resulting from a particular runoff event.

c) Percent Imperviousness — the percent of the area which is impervious to infiltration.

d) Slope — the slope of the overland flow profile.

e) Depression Storage — the average depth across the subcatchment of low-lying areas in which water will pond, plus the depth required to initiate flow across the surface. Depression storage constitutes a loss of water and is different for areas of different character.

f) Infiltration — a measure of the rate at which water is lost into the ground. This rate varies with time and with antecedent conditions. In SWMM this rate is defined by Horton's equation which requires establishment of three constants.

g) Roughness — a measure of the resistance to flow afforded by different surfaces. The value of the roughness coefficient defines, in large part, the capacity of different components of the system and the velocity of overland flow.

h) Gutter Parameters — description of the physical characteristics of the gutters which affect their carrying capacity. These characteristics include the dimensions and hydraulic roughness.

Once the drainage area has been described by establishing the parameters above, any hyetograph may be used; therefore, the effect of any storm can be predicted.

The Runoff Block proceeds in a series of time-step calculations in which rainfall is added, depression storage and infiltration are subtracted, and the remaining water (if any) is routed overland to the gutters. The routing procedure used is based upon Manning's equation and the Kinematic Wave theory. The resulting hydrographs are then routed down the gutters, using a similar procedure, until the flow reaches an inlet to the collection system. The routing in the collection system is performed by the Transport Block.

TRANSPORT BLOCK

The Transport Block uses a storage-routing procedure in which the hydrographs produced by the Runoff Block are combined and routed through the major conduits of the drainage system. Necessary data include the dimensions of the channel, the hydraulic slope, the length, the roughness, and definition of the conduits. Calculations proceed in a downstream direction, hence SWMM is unable to model the effect of downstream controls. Calculations, like those in the Runoff Block, proceed in a series of time-steps, producing hydrographs at each junction in the drainage system. The final result is a hydrograph at the point furthest downstream which represents the discharge resulting from the storm event described by the original hyetograph.

The Transport Block will predict flooding resulting from inadequacies in the system by maintaining a continuous record of flows delivered to each pipe junction which are in excess of the conduit capacity. The Transport Block may also be operated in a design mode which will calculate the conduit size necessary to pass the flows delivered to each point without producing flooding.

CHAPTER V
MODELING THE EXISTING DRAINAGE SYSTEM

GENERAL

As noted earlier in this report, the total area of the drainage basin is approximately 10,000 acres. Within this area are thousands of city blocks, thousands of inlets with many pipes and hundreds of major drainage structures 30 inches in diameter and larger. To separately describe each element of such a large and complex system would not be possible within the scope of this study; nevertheless, the similarities among different areas permitted simulation of each individual city block through the use of Standard Streets. All major drainage structures within the basin, however, were individually modeled. The concept of the Standard Street was originally developed for the City of Chicago and has been demonstrated to be a satisfactory technique for simulation of metropolitan areas which are substantially developed.

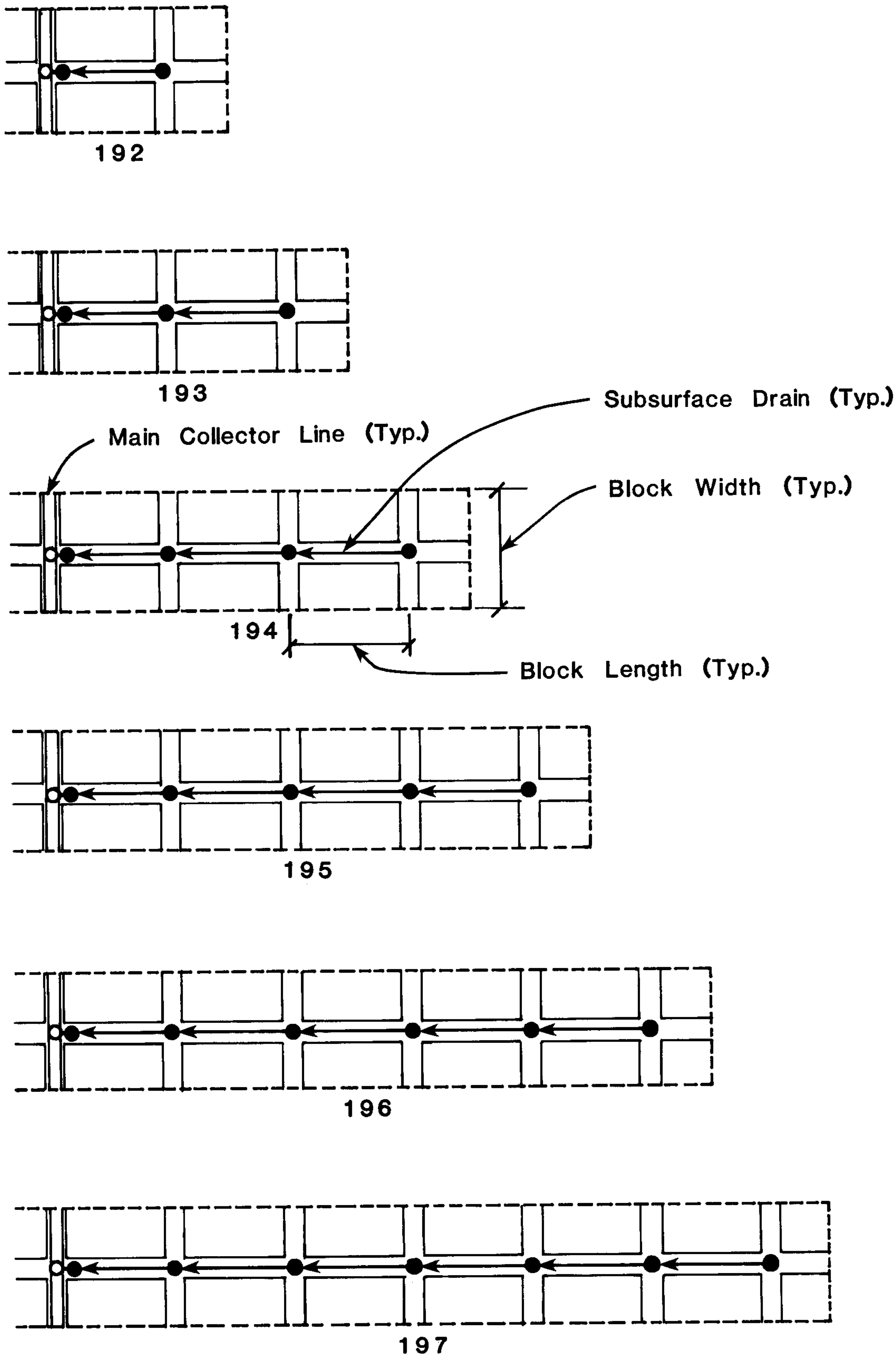
STANDARD STREETS

The Orleans Parish portion of the drainage basin was described using six different Standard Streets as shown in Figure 10. The Jefferson Parish basin was described by seven Standard Streets as shown in Figure 11. The various streets vary in length, area, size of the terminal pipe and hydraulic slope. Pipe sizes for the Standard Streets were chosen based upon criteria used by the Sewerage & Water Board of New Orleans and the Jefferson Parish Department of Public Utilities. Hydraulic slopes were calculated by assuming water surface elevations to be at street level at the most remote inlet and six inches below the top of the conduit receiving street flow.

The Standard Streets were developed to match as closely as possible the existing drainage patterns and tributary areas for streets within the drainage basin. As a result, the Standard Streets model very closely, but not exactly, the number of actual blocks represented by each. The actual contributory drainage areas for each subbasin were matched by the total simulated from the Standard Streets. Detailed plan-profile drawings of each Standard Street along with a description of the numbering system used for coding each element are included in Appendix B.

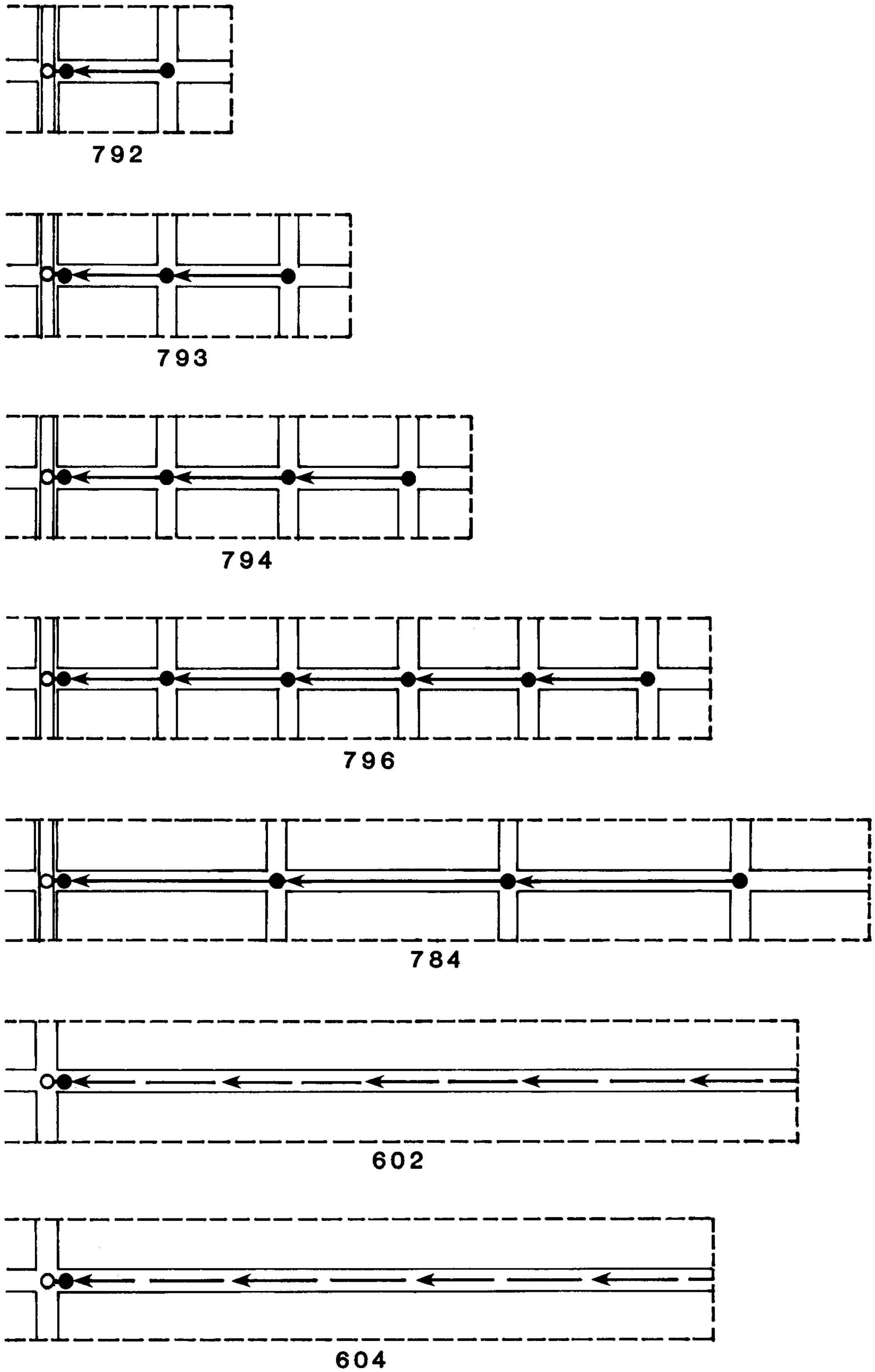
SWMM DRAINAGE MODEL

The drainage model consists of two basic elements, the Standard Streets coded as described above and the main drainage conduits. These main drainage conduits are coded into the SWMM Transport Block using parameters as previously described. Manholes are coded at each corner, and Standard Streets are input into each of these corner manholes. Conduits between manholes are coded using the appropriate Transport Block parameters. Schematic diagrams of the entire drainage basin, showing each manhole by number with its Standard Street input, are included in Appendix B. The entire system is divided into several subareas to accommodate SWMM limitations in capacity.



Standard Streets
Orleans Parish

Figure 10



Standard Streets
Jefferson Parish

Figure 11

One of the most important parameters necessary in the coding of the main drainage conduits is the hydraulic slope. This slope was estimated based upon expected water surface elevations in the receiving waters of each system, and the existing ground slopes. The entire system of water surface elevations must be compatible with assumptions made throughout, including the assumed Standard Street hydraulic slopes.

To operate the model, the hyetograph for the storm being simulated is first routed through the Standard Streets using the Runoff and Transport Blocks. This results in output hydrographs for each street. The output hydrographs are then stored on magnetic tape and used as input into the main drainage system. The hydrographs are then routed through the main drainage system, using the Transport Block, to their points of outfall. Outfall hydrographs can be obtained at all manholes in the system as desired. In addition to outflow hydrographs, the SWMM model also calculates the total volume of runoff for each system. This value was used in the calibration process as described below.

CALIBRATION OF THE MODEL

General

In order to verify the Standard Street method and to insure that the values selected for the various parameters in the Runoff and Transport Blocks were representative of actual physical conditions in the drainage basin, calibration and verification tests were conducted using actual storm hyetographs and actual pumping records at Pumping Station No. 1.

The Calibration Storm selected was the November 26, 1980 storm. The Verification Storm was that of April 22, 1979. These storms had precipitation of about two inches in a 5 to 6 hour period of time. They were selected because rainfall was fairly uniform over the city and because the intensity was generally not so severe as to cause surcharging. Hyetographs for these storms are shown in Figures 12 & 13.

The pumped hydrograph at Pumping Station No. 1 for each storm was developed from stage-discharge relationships for the individual pumps, operational records and stage recordings at the station. This procedure is described in more detail in Section 3 of this report.

Procedure

The calibration procedure was to run the calibration storm through the model, then adjust the subcatchment parameters until a good match was obtained between the SWMM output and the pumped hydrograph for the calibration storm. These values were then checked by running the verification storm through the adjusted model using the same procedure.

The total volume of discharge at Pumping Station No. 1 was the only quantitative check that could be made. It was agreed by the Technical Committee that if the total volume calculated by SWMM from the calibration and verification storms matched that calculated from actual pump records to within 5%, and if the general shape of the SWMM hydrographs compared reasonably well with the pump output hydrographs, then the SWMM model could be considered calibrated.

Exact congruence is not expected between pumped and gravity systems. Since pumps are either on or off, the pumped hydrograph will vary in a step-wise fashion, while the gravity flow hydrograph will vary continually. Operation sequence can also vary the shape of the pumped hydrograph. Initially, the gravity flow hydrograph calculated by SWMM should lead the pumped hydrograph in time, delivering flow in excess of that pumped. This causes the water level in the pumping station forebay to rise, at which time the operator starts additional pumps. At times the pumped rate will exceed the supply rate causing the water level in the forebay to fall and the operator to reduce the pumping rate. Therefore, the SWMM hydrograph should lead the pumped hydrograph in time.

Results

The calibration run was conducted with the following results:

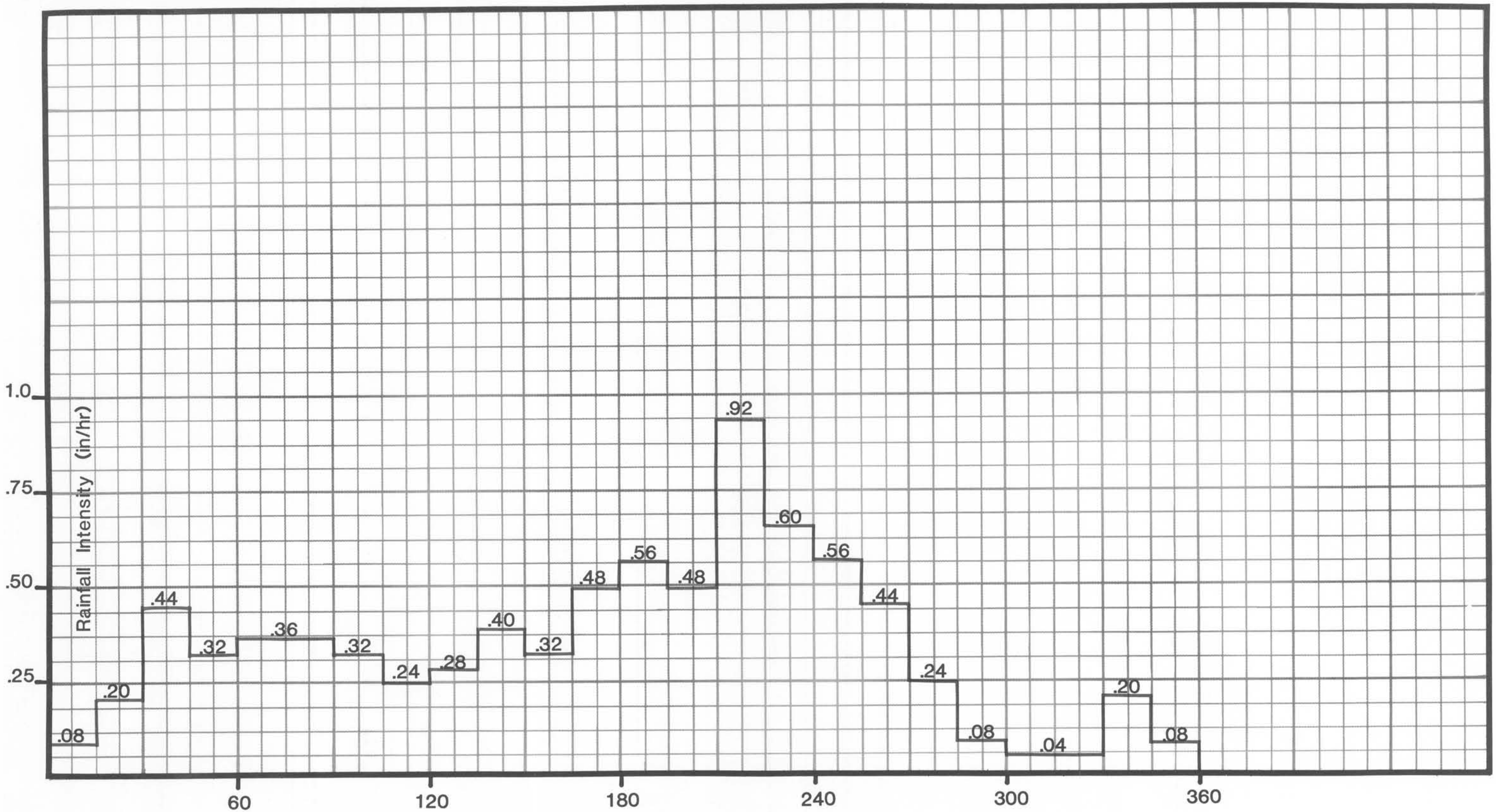
In the initial run the volume of storm runoff calculated by SWMM was 25.7 million cubic feet vs. 33.4 million cubic feet calculated from pump records. The flow patterns indicated by the general shape of the hydrographs matched the pumped hydrograph well. From preliminary experiments it was known that the total volume could most easily be increased by increasing the percent imperviousness of the subcatchment areas in the Standard Streets. This one parameter was increased from 70% to 76% and the calibration run conducted again. The total volume of the second run calculated by the calibrated model was 33.4 million cu. ft. vs. 31.6 million cu. ft. or 2.8% error. The resulting hydrograph is shown in Figure 14 along with the pumped hydrograph. Note that the SWMM hydrograph leads the pumped hydrograph as expected.

The verification run was conducted using the storm of April 22, 1979. Total volumes calculated were 43.7 million cu. ft. from SWMM vs. 43.5 million cu. ft. from the pump records or a difference of 1.0%. The resulting hydrograph is shown in Figure 15, along with the pumped hydrograph. Again, the SWMM hydrograph leads the pumped hydrograph, as one would expect.

On the basis of these results the system was considered calibrated, and the parameters obtained herein were used throughout the drainage basin. The values for the subcatchment parameters are shown below in Table 3.

TABLE 3
CALIBRATED SUBCATCHMENT PARAMETERS
17th STREET CANAL DRAINAGE BASIN

Parameter	Calibrated Value
Percent imperviousness	76.0%
Ground Slope	0.015 feet/feet
Depression Storage:	
Impervious Area	0.01 inches
Pervious Area	0.10 inches
Infiltration Rate:	
Maximum	1.50 inches/Hour
Minimum	0.25 inches/Hour
Decay Constant	0.0011
Ground Roughness:	
Impervious Area	0.018
Pervious Area	0.200



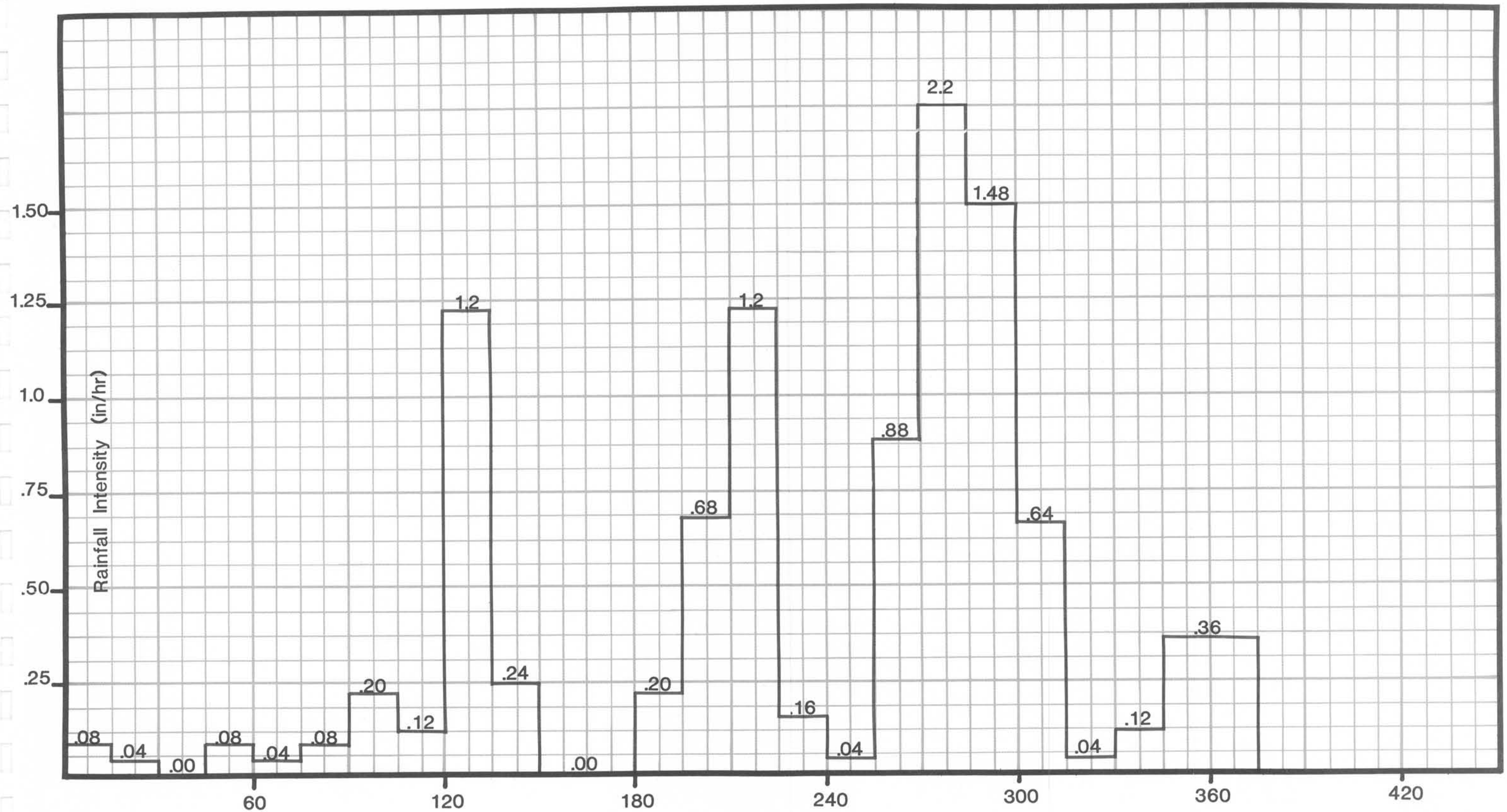
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hyetograph

26 November 1980
Calibration Storm

Figure 12



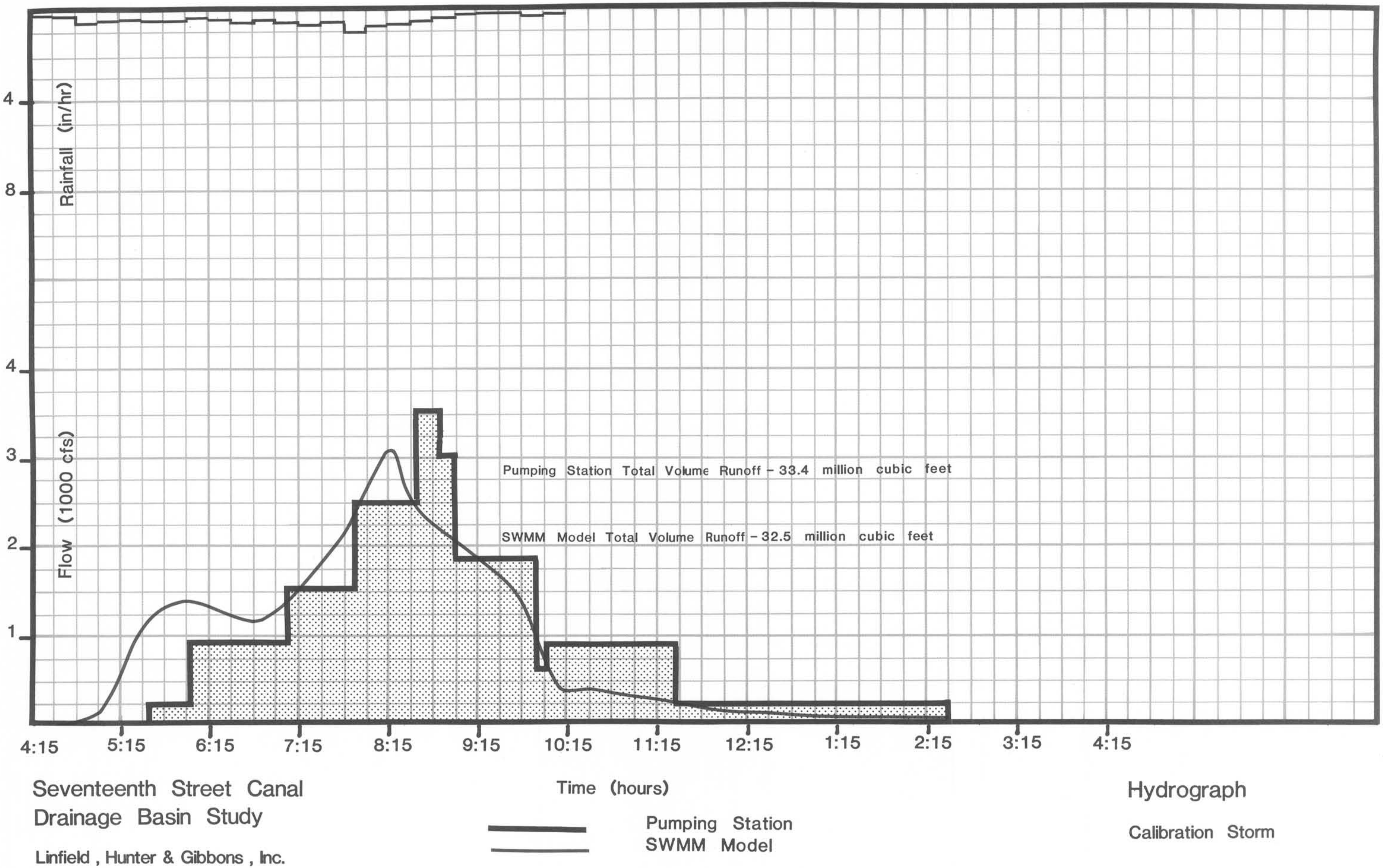
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hyetograph

22 April 1979
Verification Storm

Figure 13



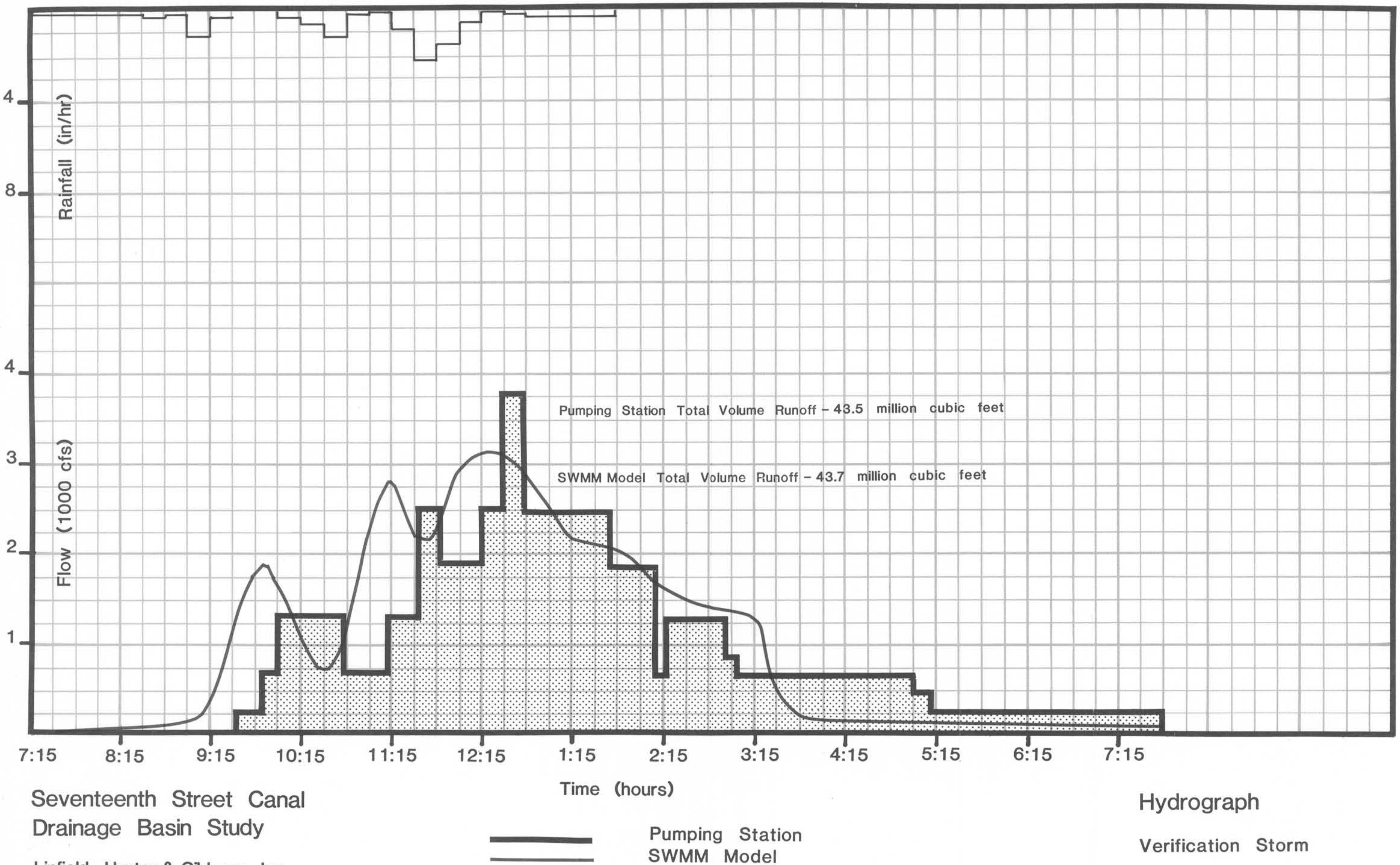
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrograph

Calibration Storm

Figure 14



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrograph

Verification Storm

Figure 15

CHAPTER VI

ANALYSIS OF DRAINAGE COLLECTION SYSTEM

GENERAL

As previously described, the existing drainage system will be analyzed based on its capacity to drain the project design storm as represented by the hyetograph shown on Figure 9. The calibration process described above validated the fact that the Standard Street method can reproduce a given storm runoff. Using the Standard Street drainage model, the design storm was routed through the entire subsurface drainage system tributary to Pumping Station No. 6.

EXISTING DRAINAGE SYSTEM

Computer printouts of the SWMM output for the design storm routing through the existing system were carefully reviewed. Each major conduit was checked to determine its ability to transport design flows. The indicated surcharging was checked against known flooding patterns to verify model output. It turned out that the surcharging in the model reasonably matched the actual flooding. A brief discussion of the findings follows.

The lower half of the Nashville, Napoleon and Third St. canals draining to Pumping Station No. 1, were of insufficient capacity to drain the input from the Standard Streets. The Fontainbleau-Broad Street Canal was deficient throughout. The Walmsley system was largely deficient. The maximum flow into Station 1 was shown to be 3100 cfs, which is considerably less than the present capacity (5400 cfs) of Station 1.

Of all of the original canals in the Orleans system, the Lowerline canal was closest to having the required capacity. It was approximately 20 percent below required capacity. The Oleander Canal is pumped by a new 100 cfs pumping station. The existing canal has a 200 cfs capacity. The maximum flows delivered at the outfall of the Lowerline Canal and the Oleander Canal were 900 cfs and 100 cfs, respectively.

Most of the major conduits in Hoey's Basin are inadequate to carry design flows under present conditions. At present, however, because of major flooding upstream at Loumar Ditch and along Jefferson Highway, the major canals, i.e. Hoey's, Geisenheimer, Loumar Outfall, are not overloaded. The maximum flow that can be delivered into the 17th Street Canal is 1100 cfs.

IMPROVEMENTS

After reviewing the results of modeling the design storm through the existing subsurface collection system, it was apparent that substantial improvements were necessary if the outflows from the Standard Streets were to be transported without flooding. To determine the conduit sizes required the SWMM model, except for the Standard Streets, was put in the "design mode." In this mode, the SWMM program calculates the conduit size necessary to carry flows without surcharging.

In addition to the redesign of existing canals using the SWMM design mode, several new canals were coded into the New Orleans system. These included the Jefferson Avenue and Louisiana Avenue canals which connect the Broad Street collector with Constance Street near the river; the "Washington Interceptor Canal" which will run parallel to the Palmetto Canal and collect drainage from the adjacent areas and will drain directly into Station 1; and the Claiborne Avenue connection between the Nashville and Lowerline Canals. In Jefferson only one new canal was added; this was the Shrewsbury Canal extending from Jefferson Highway to Hoey's Canal. Each of these new drainage canals is either presently funded or is in the 5 year capital budget. Coding of the existing system was altered to match the new drainage patterns required by these new drainage canals. Schematic diagrams of the entire improved system are included in Appendix B.

Hydraulic slopes used for the existing system were adjusted to match design water surface elevations in the receiving waters as discussed in Section 3 following. It is important to note here that, unless the receiving waters, specifically the 17th Street Canal and the Palmetto Canal, are improved as required to obtain these elevations, the system will not function as proposed. Results of these improved condition routings are discussed briefly in the next paragraph and are shown in the plates following.

PLATES — EXISTING AND IMPROVED

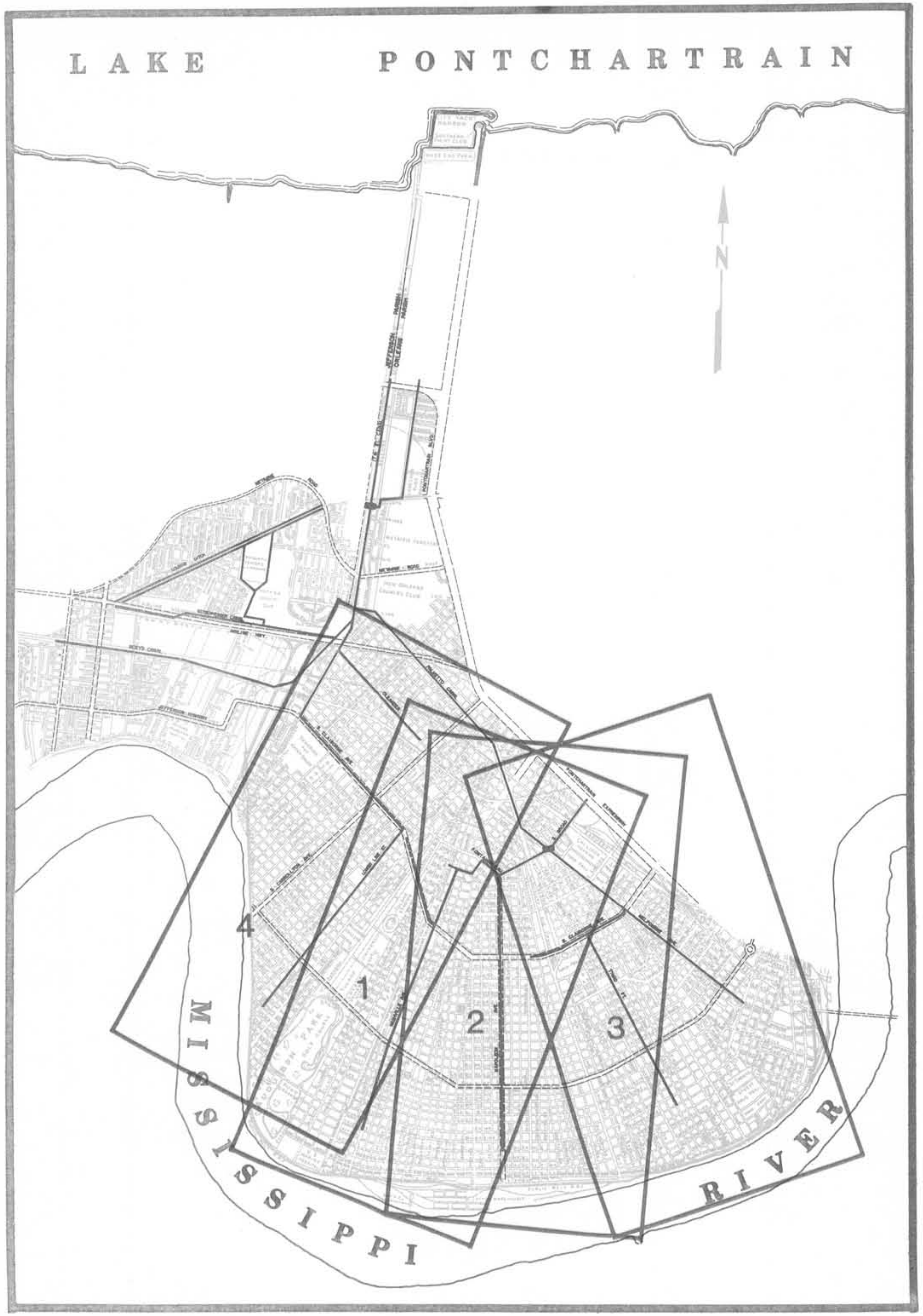
The main canals and conduits making up the existing and improved drainage collection system are shown on Plates 1 through 6. Comments herein refer to these Plates.

The Plates showing "Existing Facilities" indicate the general location, shape and size of all major drainage conduits in the collection system. The sizes shown generally indicate the largest size in the particular segment being defined. Actual sizes for each block were coded into the model, however, they were not shown here, as this information can be obtained from the printouts. Plates also show ground contour lines which were used to calculate the hydraulic slope for each segment of conduit.

The Plates showing "Improved Facilities" indicate the actual conduits that need to be enlarged and the proposed new collection canals as discussed above. Required sizes are given for the major canals where enlargement is recommended. Required improvements for the smaller collector canals are not shown. It should be noted here that the conduit sizes for each block of the main collection system were designed in the SWMM routings and are available in the SWMM printouts.

Design flows in cubic feet per second (cfs) are also shown at various intersections within the system. These are maximum flows that will occur during the design storm if all improvements indicated have been made.

Water surface elevations shown were calculated using the hydraulic slopes coded in the SWMM model. These slopes are also shown for the major canals. Water surface elevations in the receiving waters used as beginning elevations were obtained from calculations described in Section 3.



DRAINAGE SYSTEM	PLATE NO.
Nashville and Broad (Existing)	1
Nashville and Broad (Proposed)	1
Napoleon (Existing)	2
Napoleon (Proposed)	2
Third and Melpomene (Existing)	3
Third and Melpomene (Proposed)	3
Lower Line and Oleander (Existing)	4
Lower Line and Oleander (Proposed)	4

Orleans Drainage System

Plate Index

Seventeenth Street Canal Drainage Basin Study

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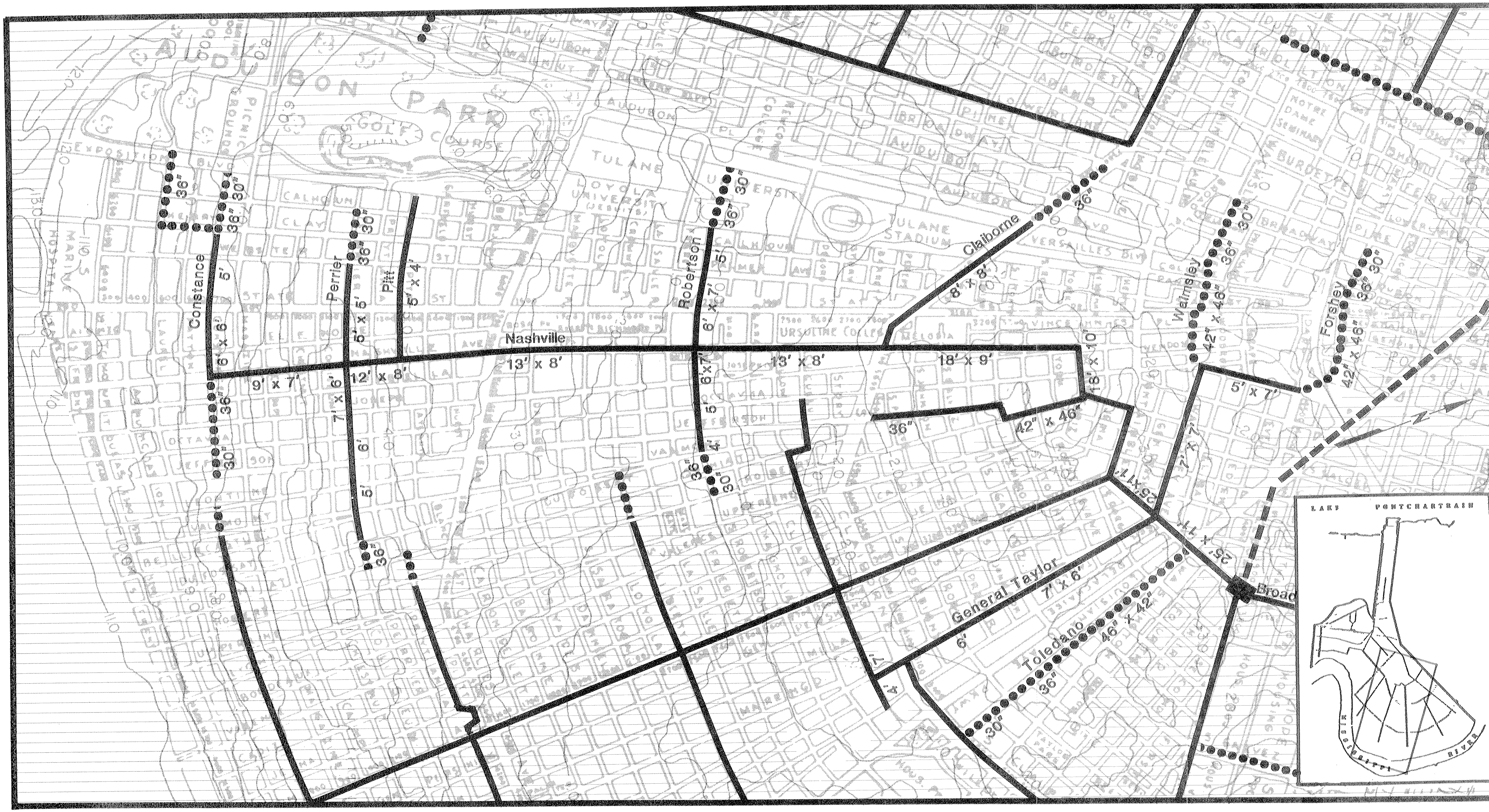


Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0' 825' 1,650' 2,475' 3,300'

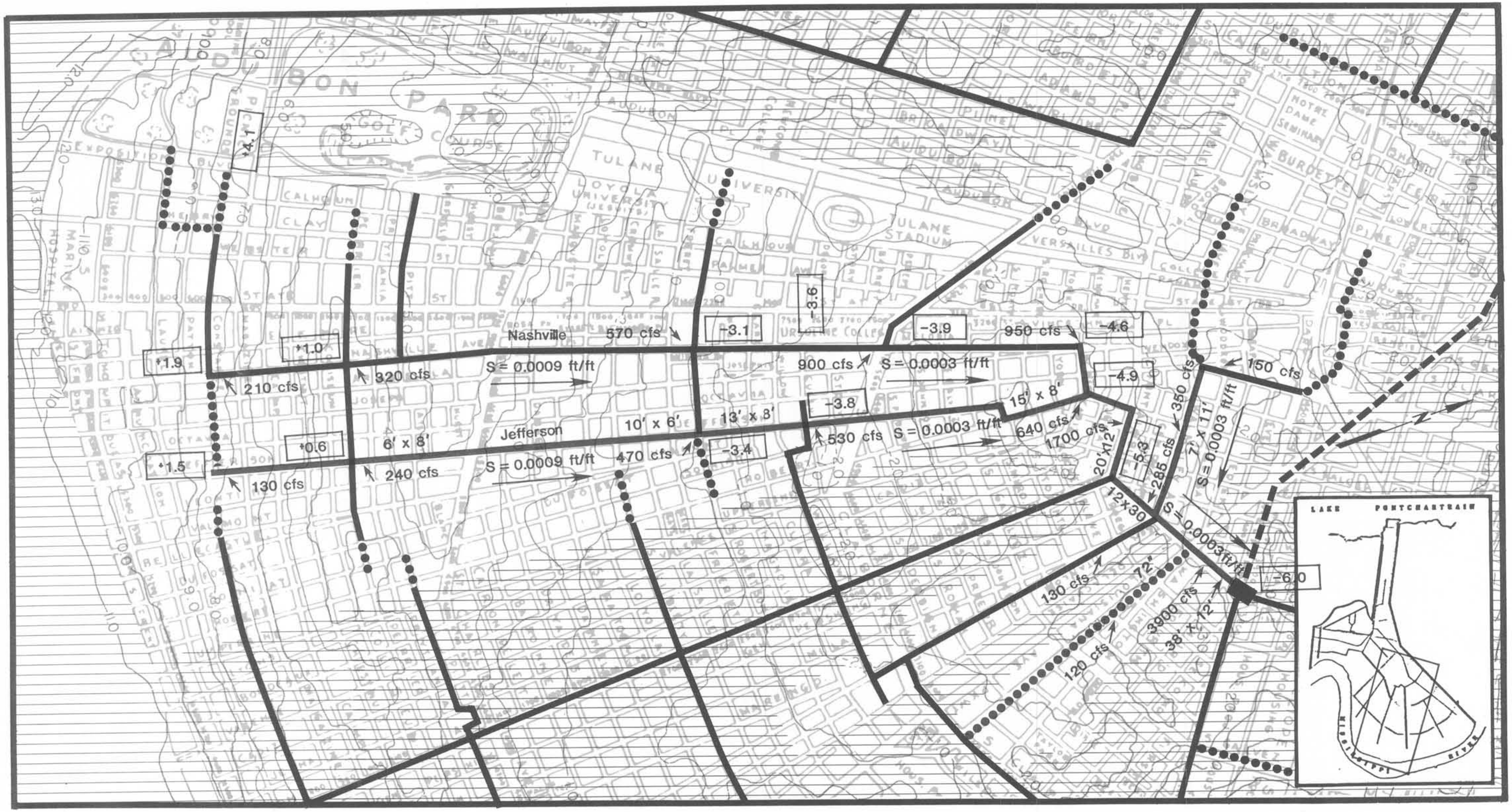
Approximate Scale

Orleans Drainage Basin

Nashville & Broad System

Plate No. 1

Existing Facilities



Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬	▬ ▬ ▬	Open Conc. Canal
··········	··········	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

☐ Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Seventeenth Street Canal Drainage Basin Study

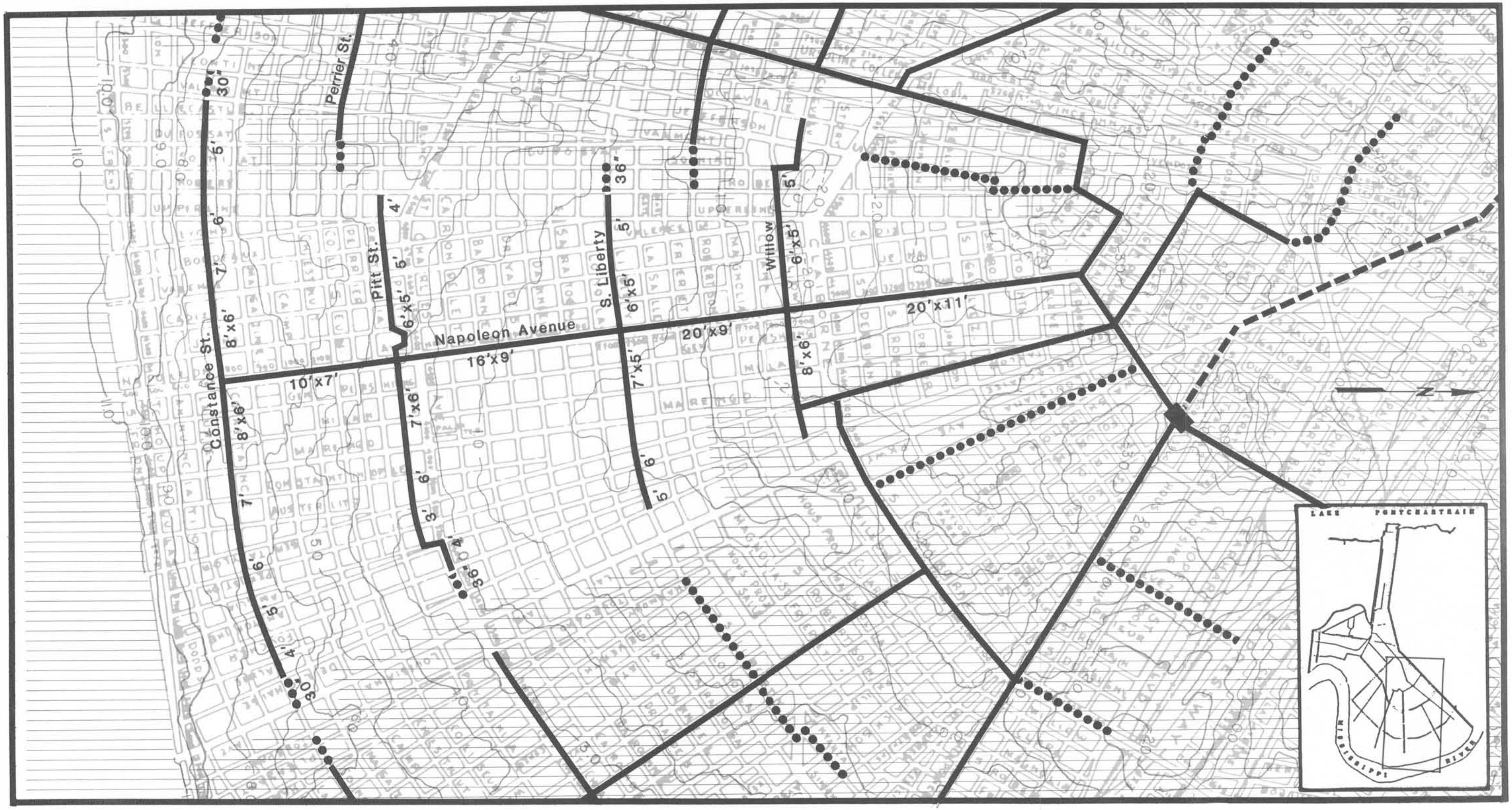
Linfield, Hunter & Gibbons, Inc.

Orleans Drainage Basin

Nashville & Broad System

Plate No. 1

Proposed Improvement



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

- Box Culvert
- Open Conc. Canal
- Open Canal / Ditch
- Pipe

0' 825' 1,650' 2,475' 3,300'

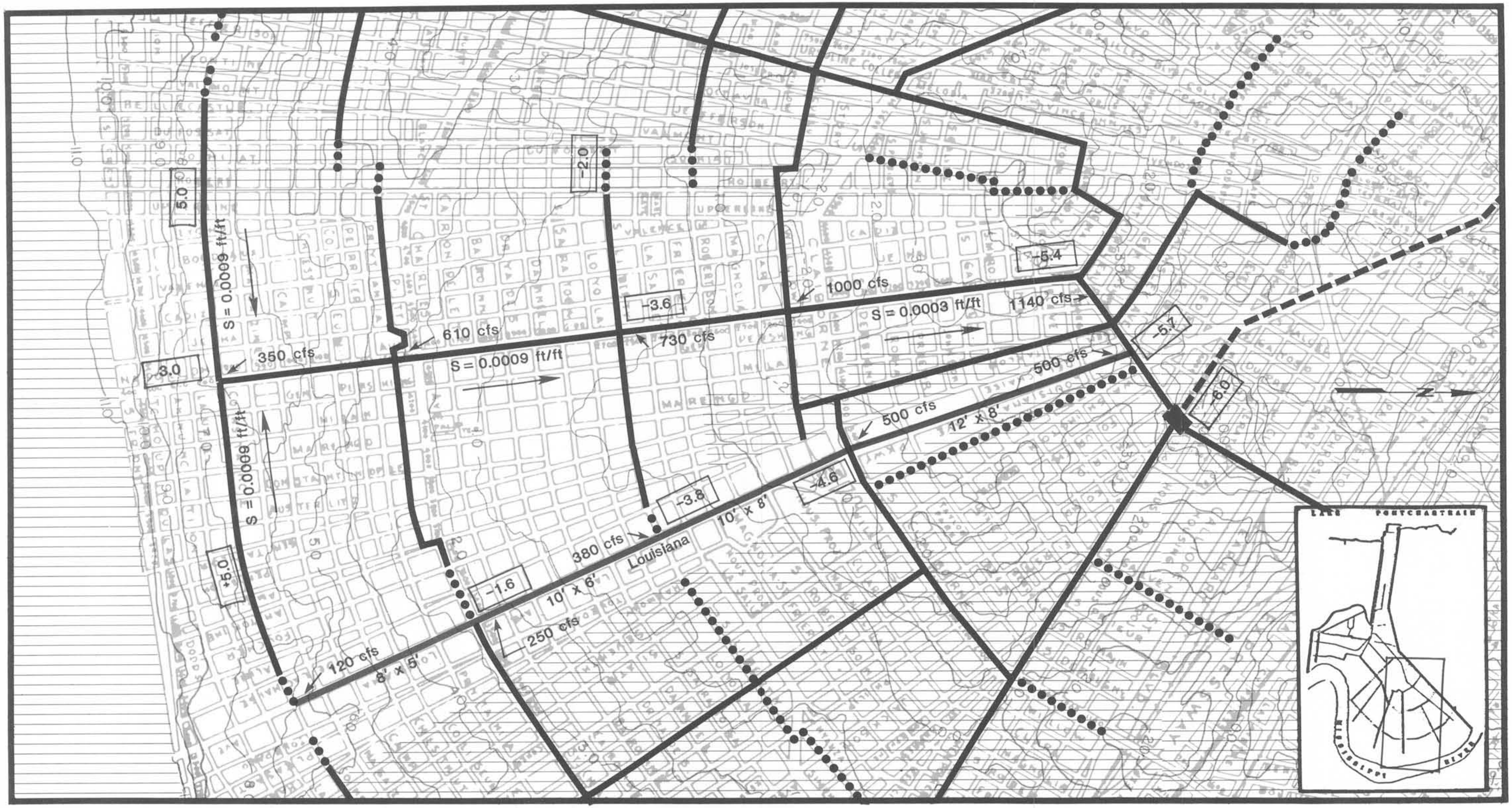
Approximate Scale

Orleans Drainage Basin

Napoleon

Plate No. 2

Existing Facilities



Existing	Proposed	
		Box Culvert
		Open Conc. Canal
		Open Canal / Ditch
		Pipe

Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Seventeenth Street Canal Drainage Basin Study

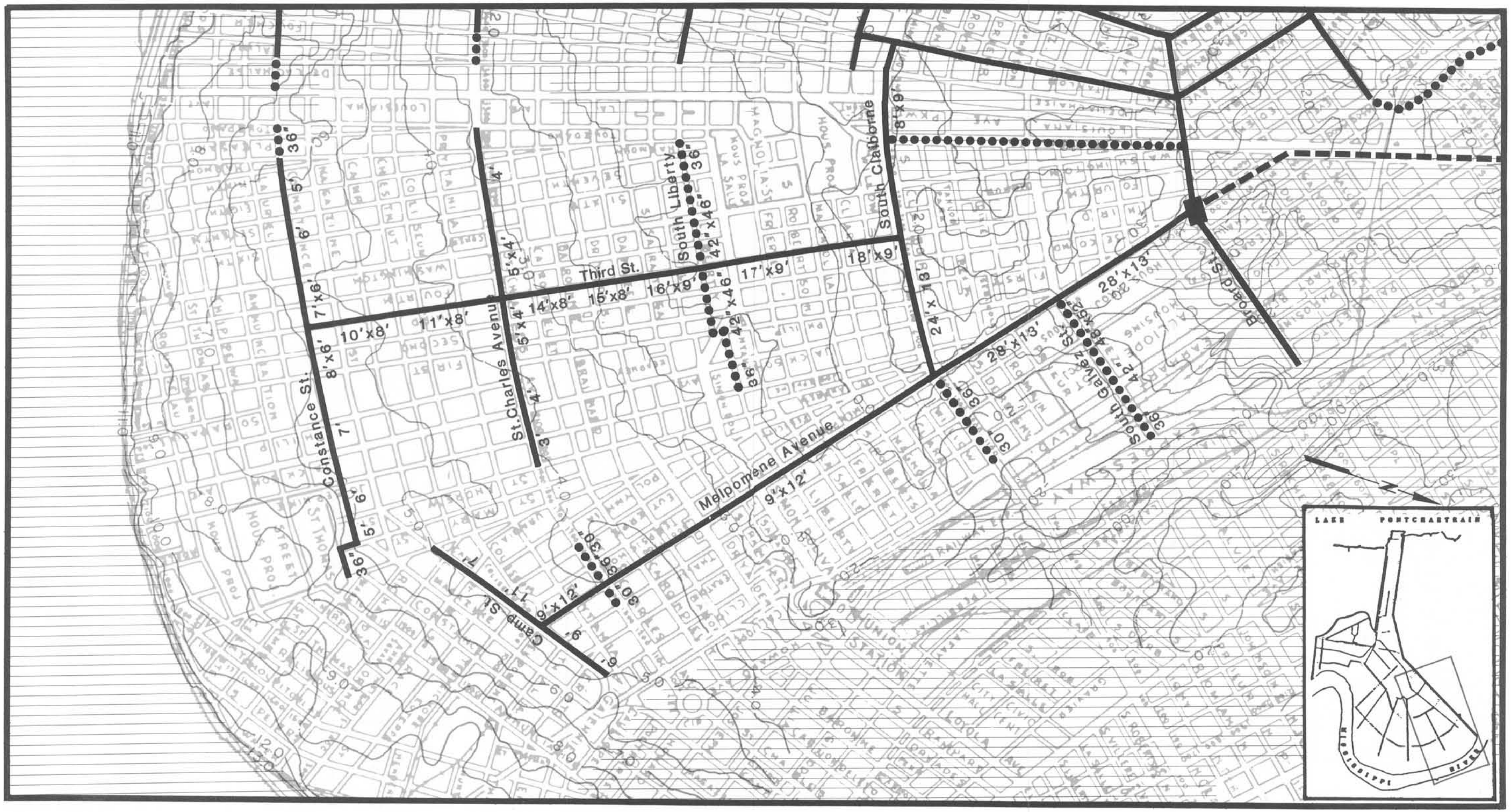
Linfield, Hunter & Gibbons, Inc.

Orleans Drainage Basin

Napoleon

Plate No. 2

Proposed Improvement



Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Seventeenth Street Canal
Drainage Basin Study

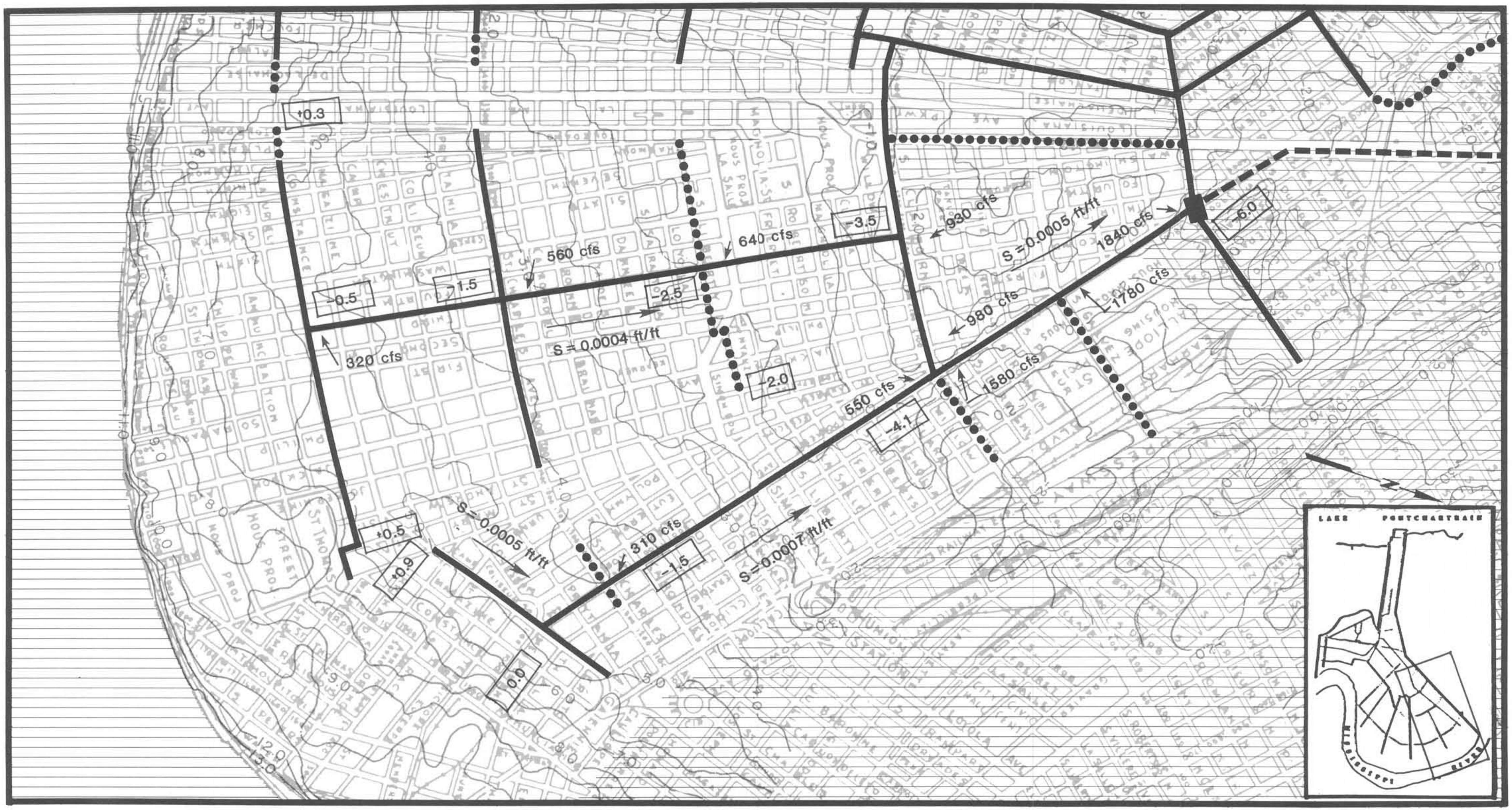
Linfield, Hunter & Gibbons, Inc.

Orleans Drainage
Basin

Third & Melpomene

Plate No. 3

Existing Facilities



Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬ ▬	▬ ▬ ▬ ▬	Open Conc. Canal
············	············	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

☐ Denotes Water Surface Elev.

0′ 825′ 1,650′ 2,475′ 3,300′

Approximate Scale

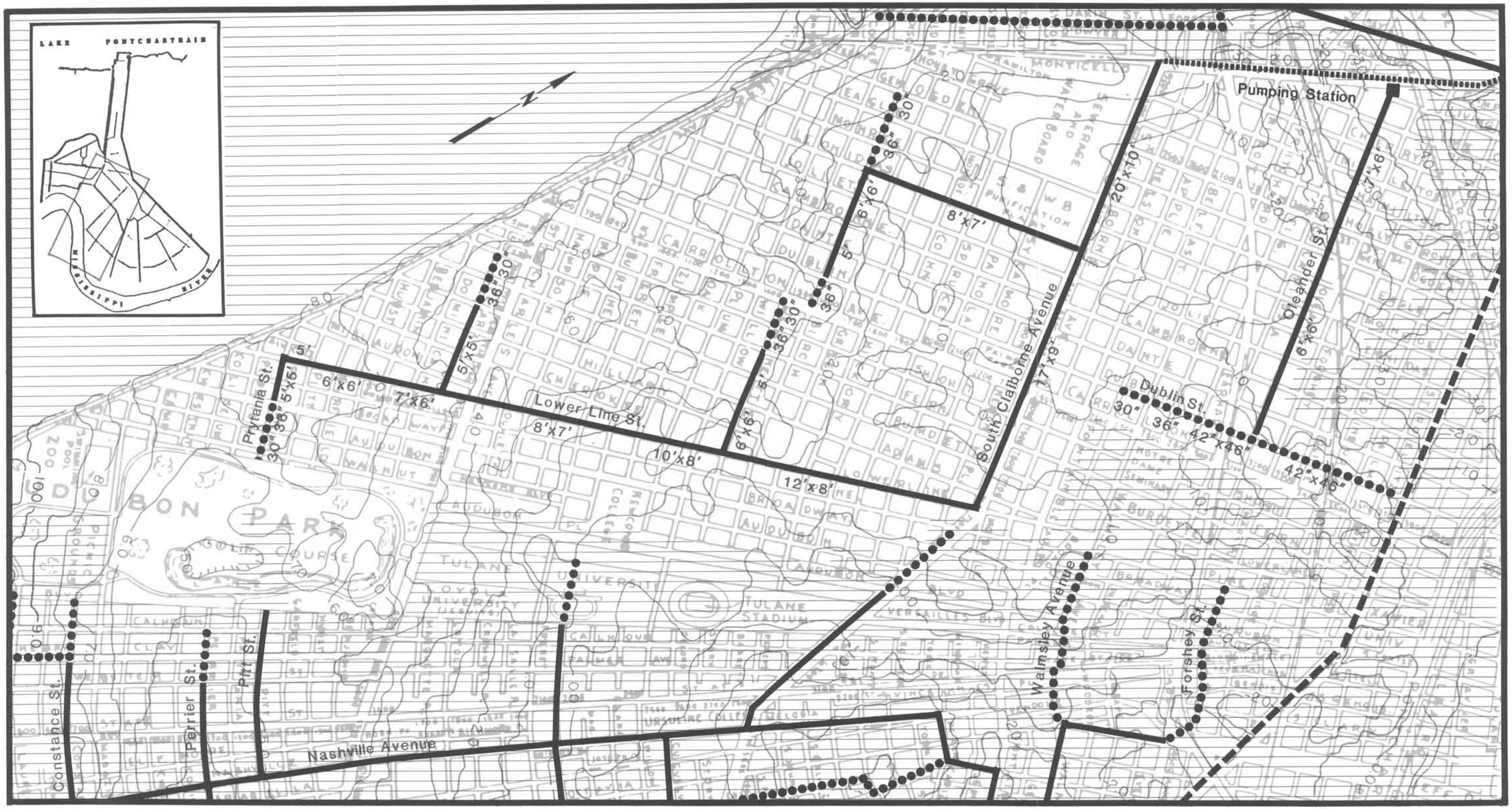
Note: All Elevations Are In Mean Sea Level

Orleans Drainage Basin

Third & Melpomene

Plate No. 3

Proposed Improvement



Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

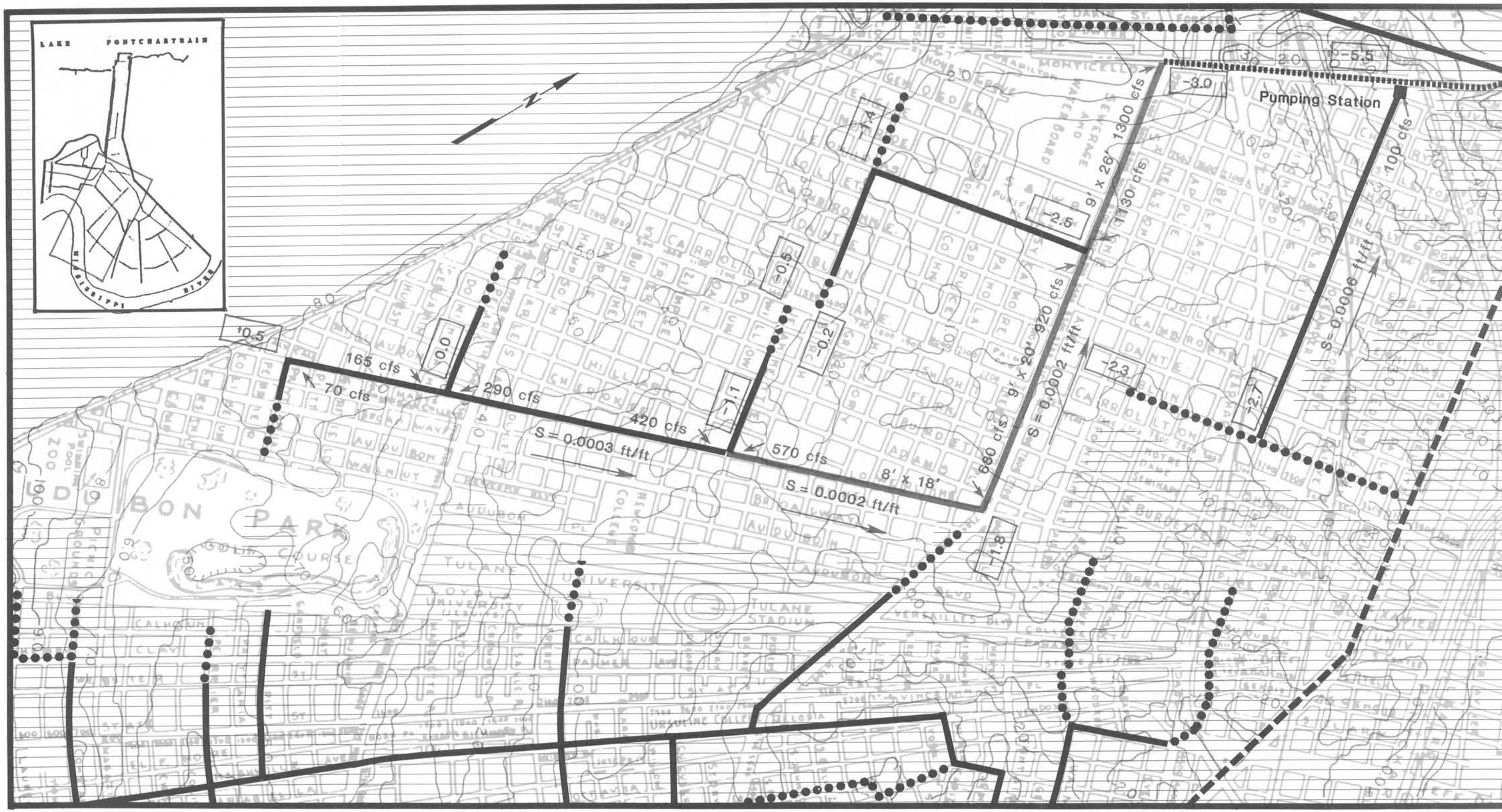
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Orleans Drainage Basin

Oleander & Lower Line

Plate No. 4
Existing Facilities



Existing Proposed

Box Culvert

Open Conc. Canal

Open Canal / Ditch

Pipe

Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Seventeenth Street Canal Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Orleans Drainage Basin

Oleander & Lower Line

Plate No. 4

Proposed Improvement

DRAINAGE SYSTEM	PLATE NO.
Hoey's (Existing)	5
Hoey's (Proposed)	5
Geisenheimer (Existing)	6
Geisenheimer(Proposed)	6

Jefferson Drainage System

Plate Index

Seventeenth Street Canal Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

- Box Culvert
- Open Conc. Canal
- Open Canal / Ditch
- Pipe

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Jefferson Drainage Basin

Hoey's Canal

Plate No. 5
Existing Facilities

Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬	▬ ▬ ▬	Open Conc. Canal
┊┊┊┊┊┊	┊┊┊┊┊┊	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

▭ Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Seventeenth Street Canal Drainage Basin Study

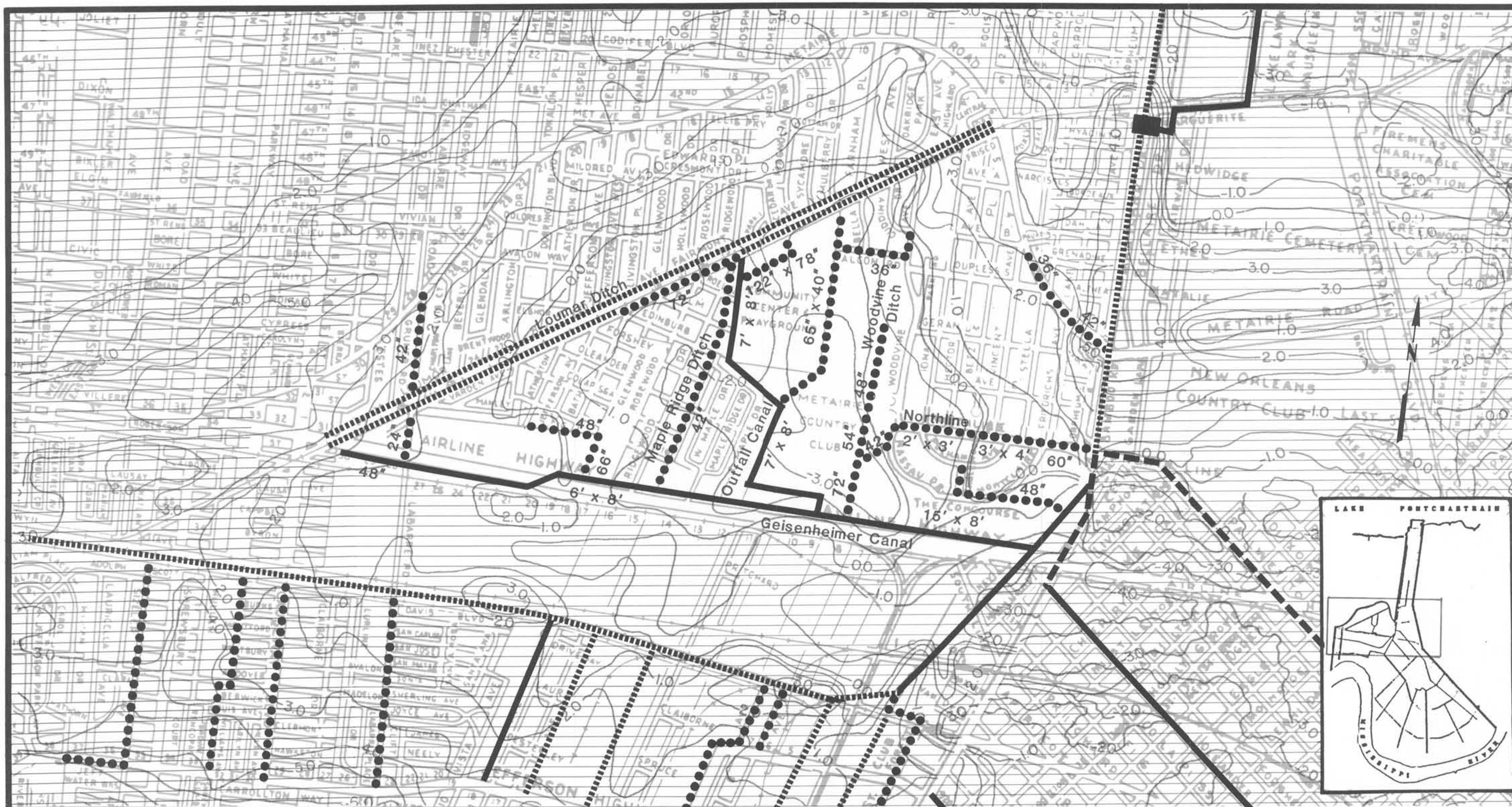
Linfield , Hunter & Gibbons , Inc.

Jefferson Drainage Basin

Hoey's Canal

Plate No. 5

Proposed Improvement



Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

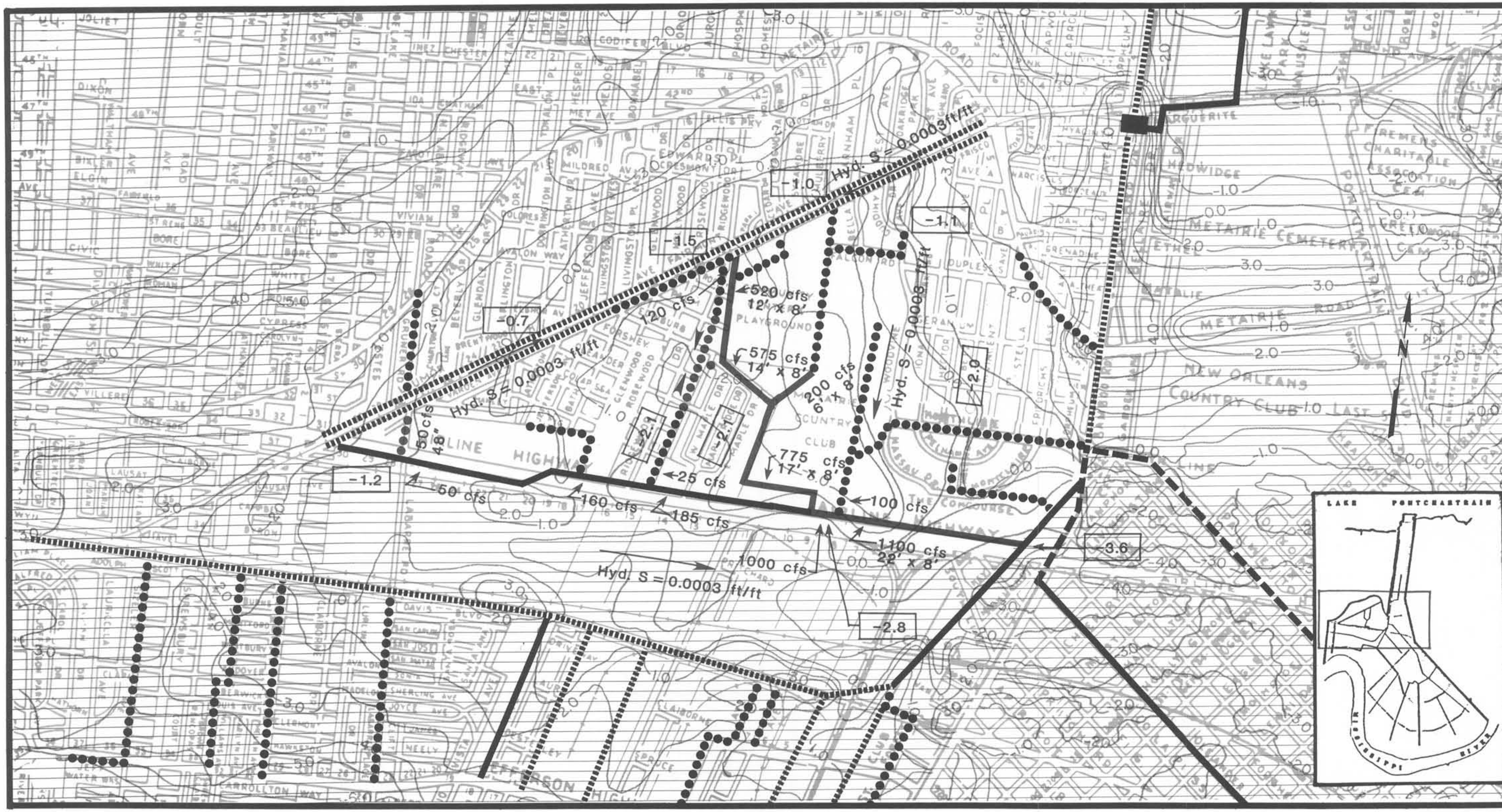
Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Jefferson Drainage
Basin

Geisenheimer Canal

Plate No. 6
Existing Facilities



Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬	▬ ▬ ▬	Open Conc. Canal
┃┃┃┃┃┃	┃┃┃┃┃┃	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

□ Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Seventeenth Street Canal Drainage Basin Study

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Jefferson Drainage Basin

Geisenheimer Canal

Plate No. 6

Proposed Improvement

OUTFALL HYDROGRAPHS

Input hydrographs from the Standard Streets are shown in Figures 16 and 17; and outflow hydrographs for the main collection systems are shown in Figures 18 through 23.

The input hydrographs all have a long, flat peak, resulting from the outflow constricting effect of the pipe sizes used in the Standard Streets. As stated earlier, these pipes will surcharge and some flooding is expected in these streets. This flat peak of the Standard Street hydrographs directly affects the outflow hydrographs of the main collection system. It can be seen that the hydrographs for all existing systems have very long flat peaks and lengthy runouts, indicating that the system has been flooding for over 12 hours. The hydrographs for the improved system have flat peaks, generally lasting 2-3 hours; however, the systems are dry after approximately 7 hours.

Table 4 shows a listing of the peak outflows from each contributary area for both existing and improved conditions. These peak flows are used as input into the Transport System described in Section 3.

Jefferson

Some special comments are appropriate concerning improvements shown on Plates 5 and 6 in Hoey's Basin. Improvements shown on Plate 5 indicate the particular main collector drain lines that were modeled between Jefferson Highway and Hoey's canal. Design flows and recommended sizes shown are based on this assumed drainage pattern. If these collector lines are revised in number, the flows and sizes must be altered accordingly. Equivalent pipe sizes for the proposed new crossings of Jefferson Highway were also modeled into the system and flows shown have been coordinated with these new crossings. In most instances these collector lines should be enlarged to carry the increased flows resulting from the new crossings. It is noted here that all streets between Jefferson Highway and the river were modeled as Typical Streets.

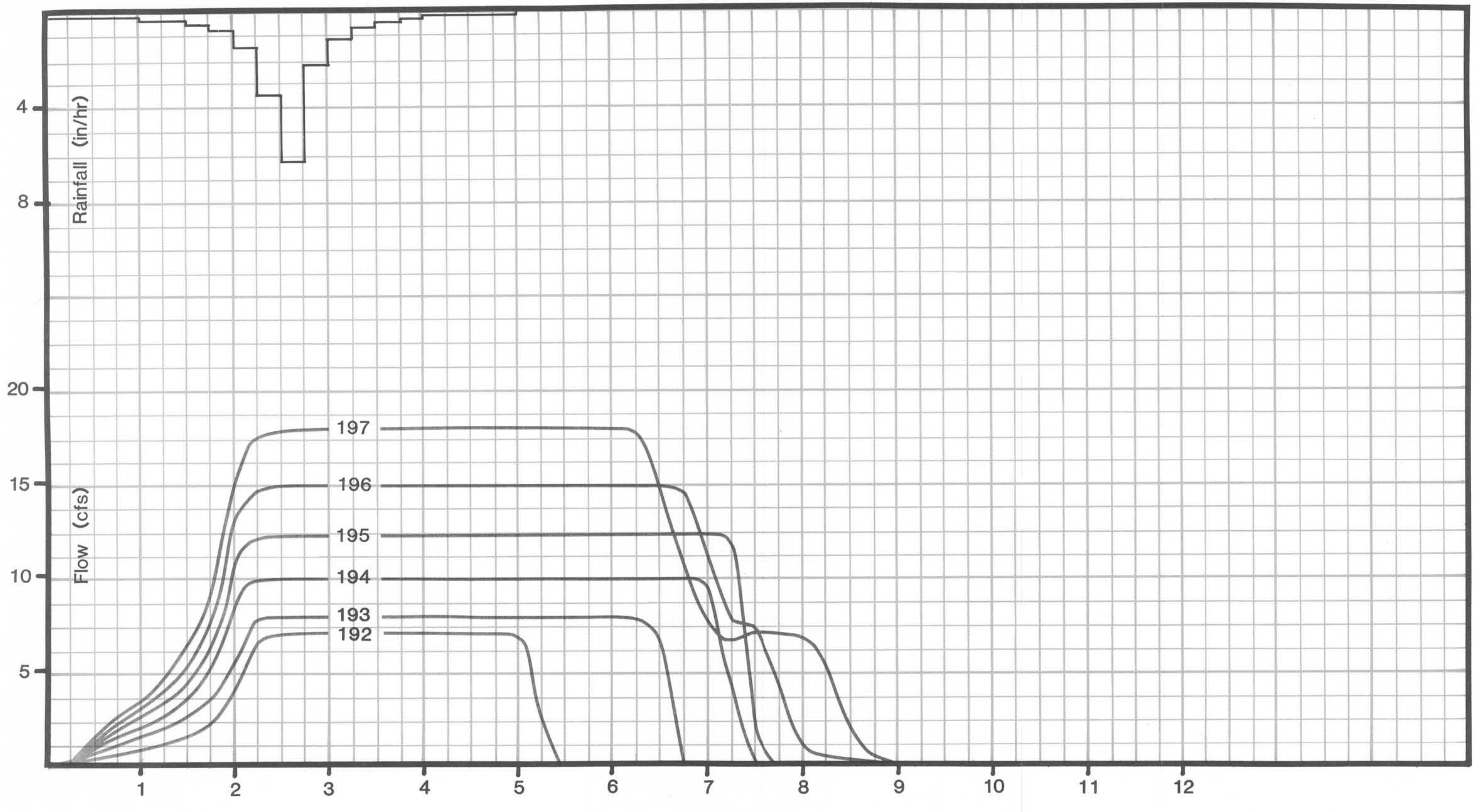
Regarding Plate 6, special note is made concerning the recommendation of a major change in the drainage pattern of Maple Ridge Ditch (now a subsurface pipe) and the Loumar Outfall Canal. Since the Loumar Outfall Canal must be enlarged in any case, it is proposed that all drainage now flowing into the Maple Ridge Ditch be diverted into the Loumar Outfall Canal as indicated. This relieves the necessity for major improvements to the Maple Ridge Ditch and the Geisenheimer Canal above its junction with the Loumar Outfall Canal. The design flows shown are based on the assumption that drainage contributary to Loumar Ditch will drain into the ditch as proposed on the Standard Street plans and that Loumar Ditch can discharge flows into the Outfall Canal without surcharging. Other major improvements recommended are the enlargement of the Geisenheimer Canal below the Loumar Outfall Canal and the change in Hoey's Canal from an earthen canal to an open concrete box.

Orleans

Regarding improvements shown for the Orleans Parish portion of the system, shown on Plates 1 through 4, only proposed improvements to the main drainage canals are shown. Several of the lateral lines also require improvement. It is strongly recommended that a manifold canal be constructed along S. Claiborne Avenue in order to distribute flows into the Broad St. System. If this is done, it is possible that most of the required increased capacity in this area can be built into the new canals, thus reducing the necessity of major improvements in the existing canals.

TABLE 4
PEAK OUTFALL FLOWS — DESIGN STORM
17th STREET CANAL DRAINAGE AREA

BASIN	AREA (ACRES)	FLOWS (cfs) EXISTING CONDITION	FLOWS (cfs) IMPROVED CONDITION
Orleans Parish			
Pumping Station No. 1 (existing)	5500	3100	—
Pumping Station No. 1 (improved)	(5700)	—	6000
Lowerline	1300	900	1300
Oleander	150	100	100
Avenue C	600	100	150
Palmetto (existing)	200	100	—
Palmetto (improved)	300	—	150
Totals	7850	4300	7700
Jefferson Parish			
Hoey's Basin	2400	1100	2550
Metairie Rd. & Northline	150	100	150
Totals	2550	1200	2700
Basin Total	10400	5500	10400



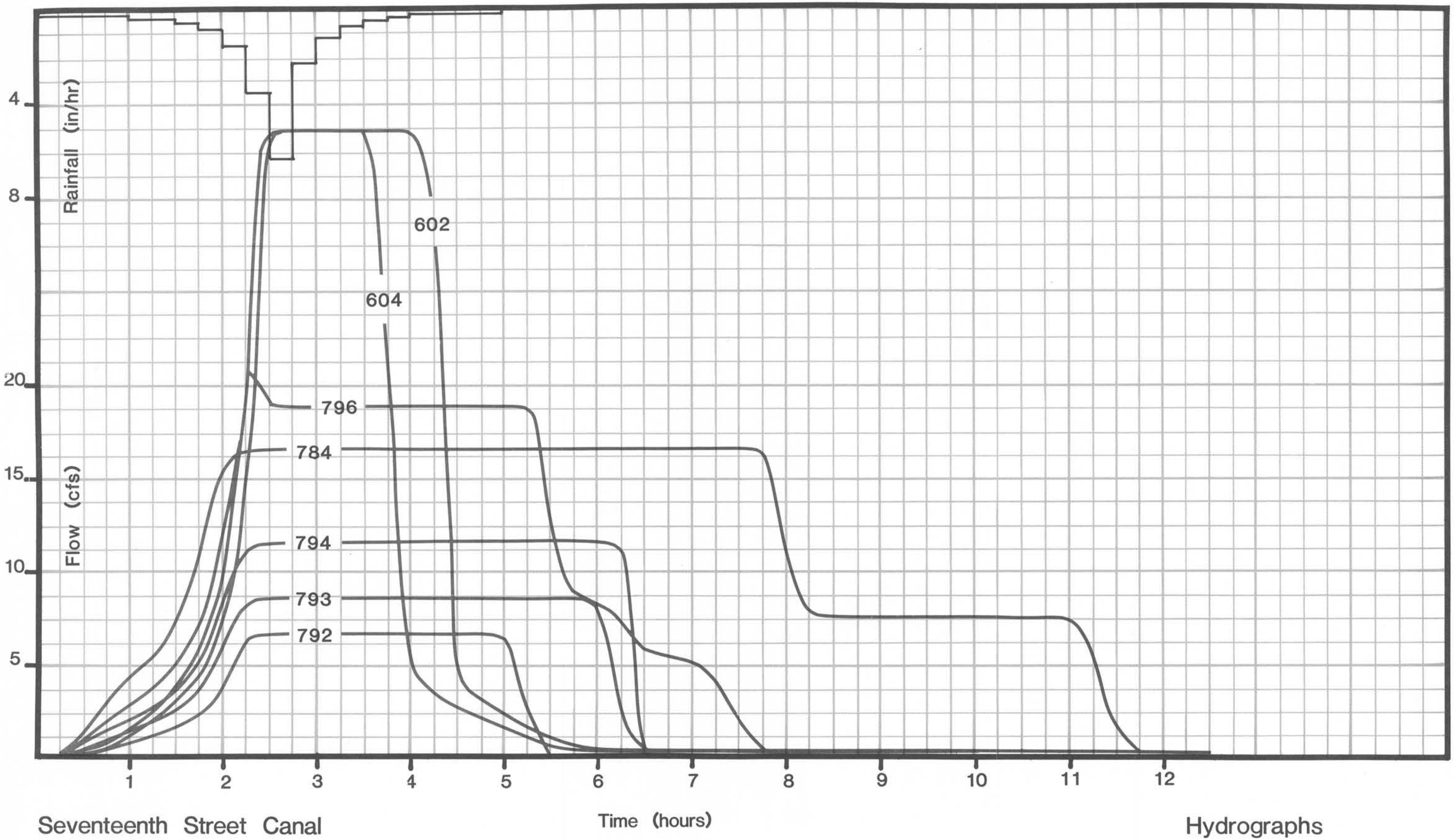
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrographs

Standard Streets
Orleans Parish

Figure 16



Seventeenth Street Canal
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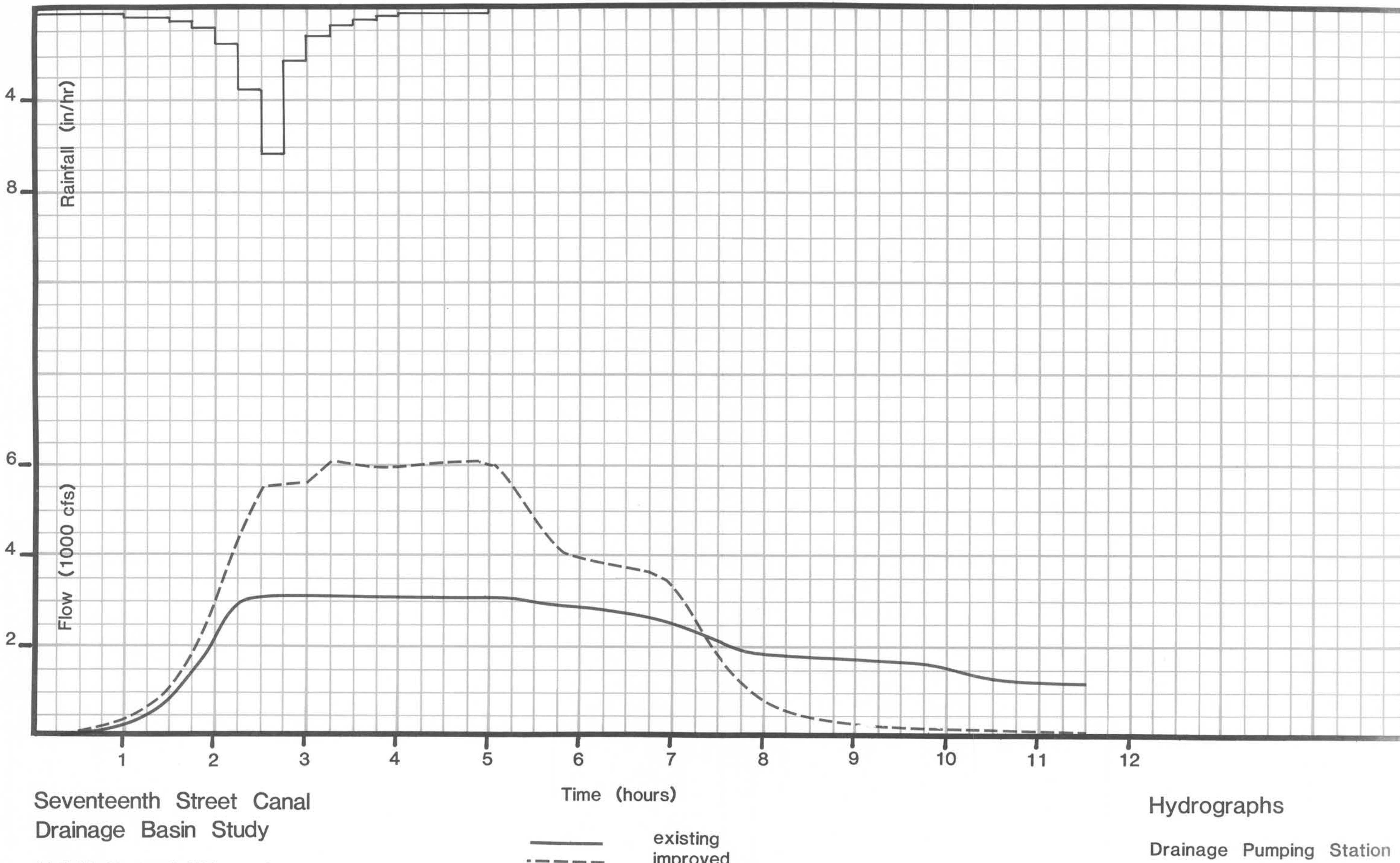
Hydrographs

Standard Streets
Jefferson Parish

Figure 17



Seventeenth Street Canal
Drainage Basin Study

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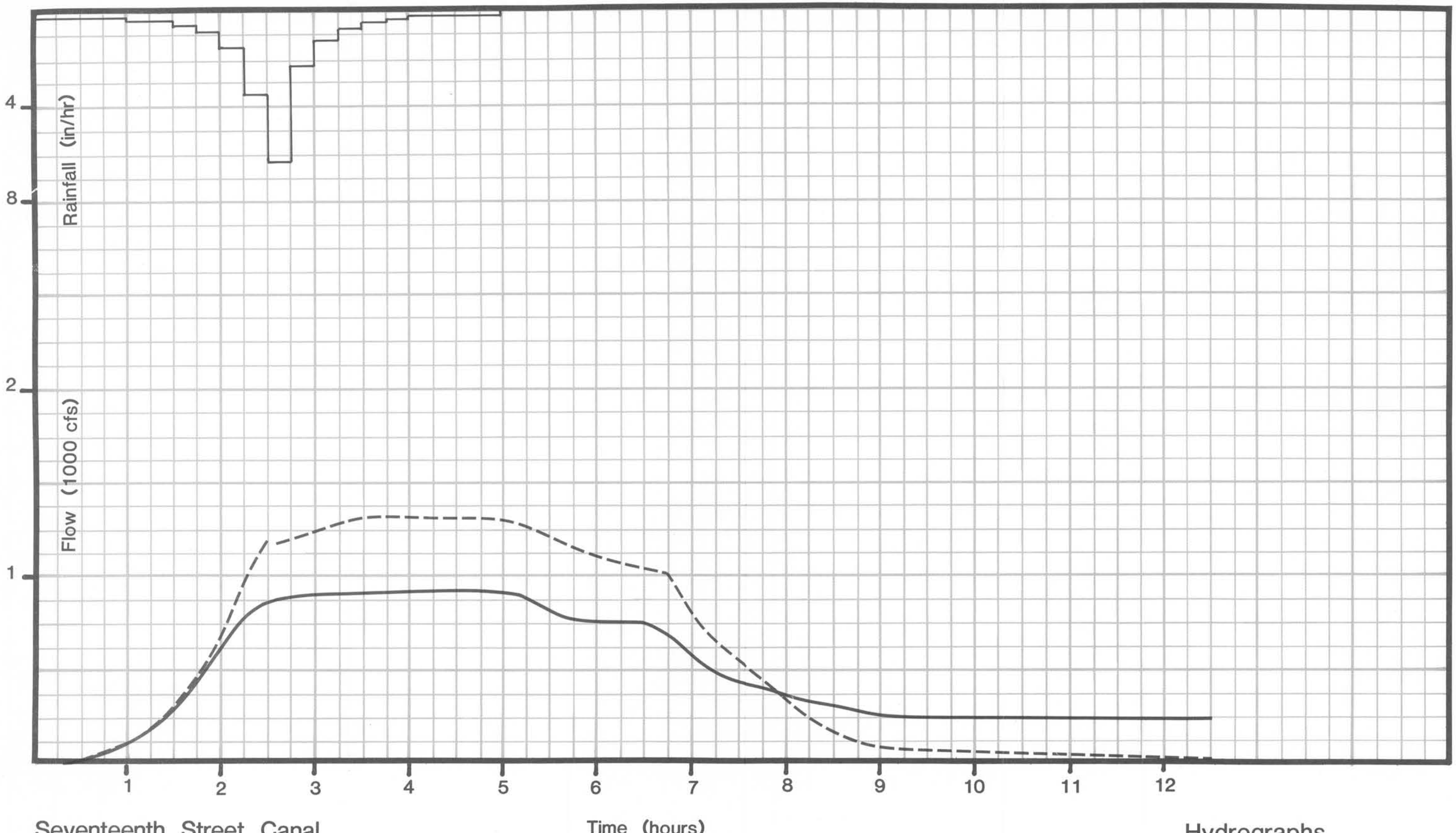
Hydrographs

Drainage Pumping Station No. 1

Figure 18



Seventeenth Street Canal
Drainage Basin Study

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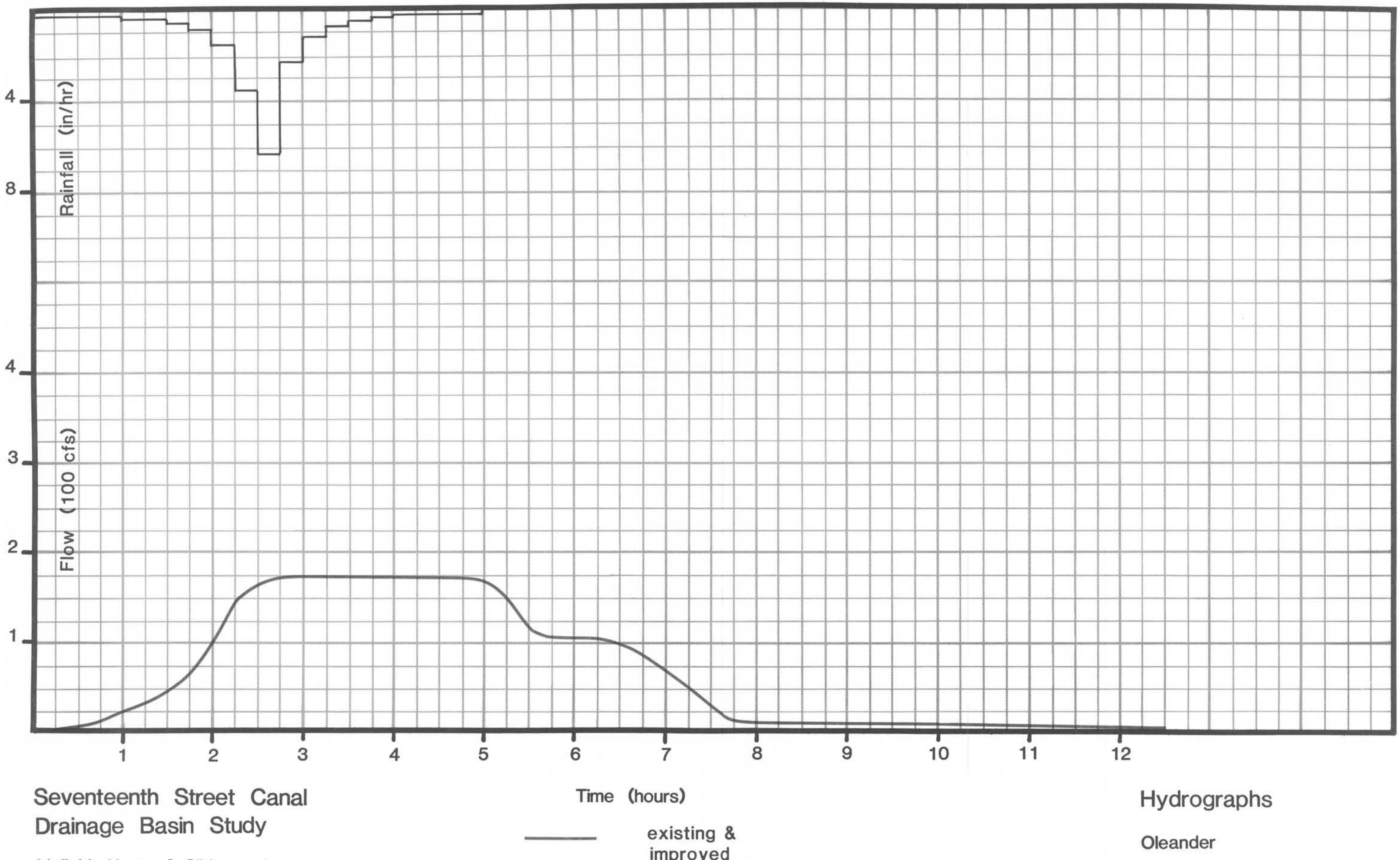
Hydrographs

Lowerline

Figure 19



Seventeenth Street Canal
Drainage Basin Study

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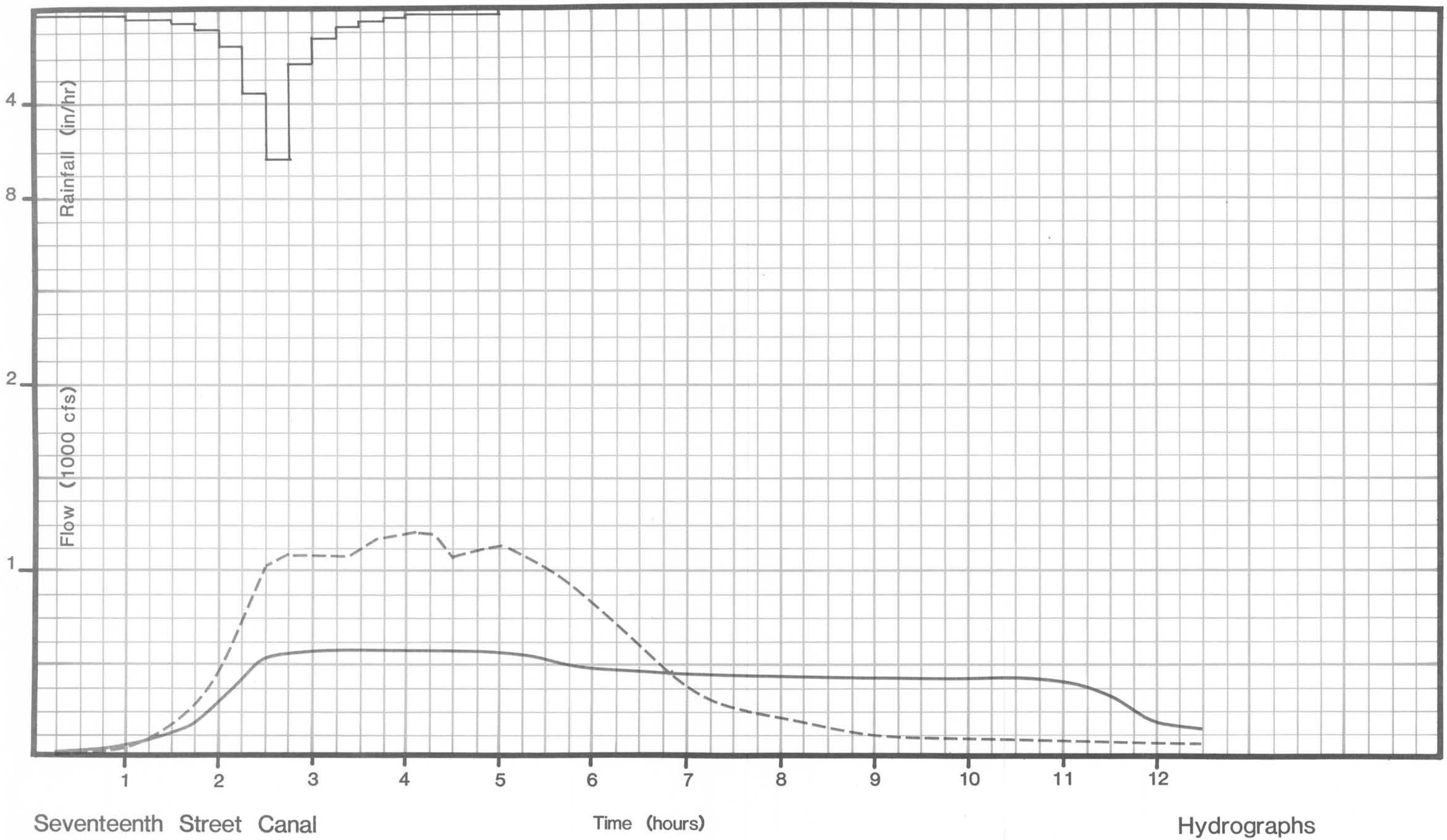
Hydrographs

Oleander

Figure 20



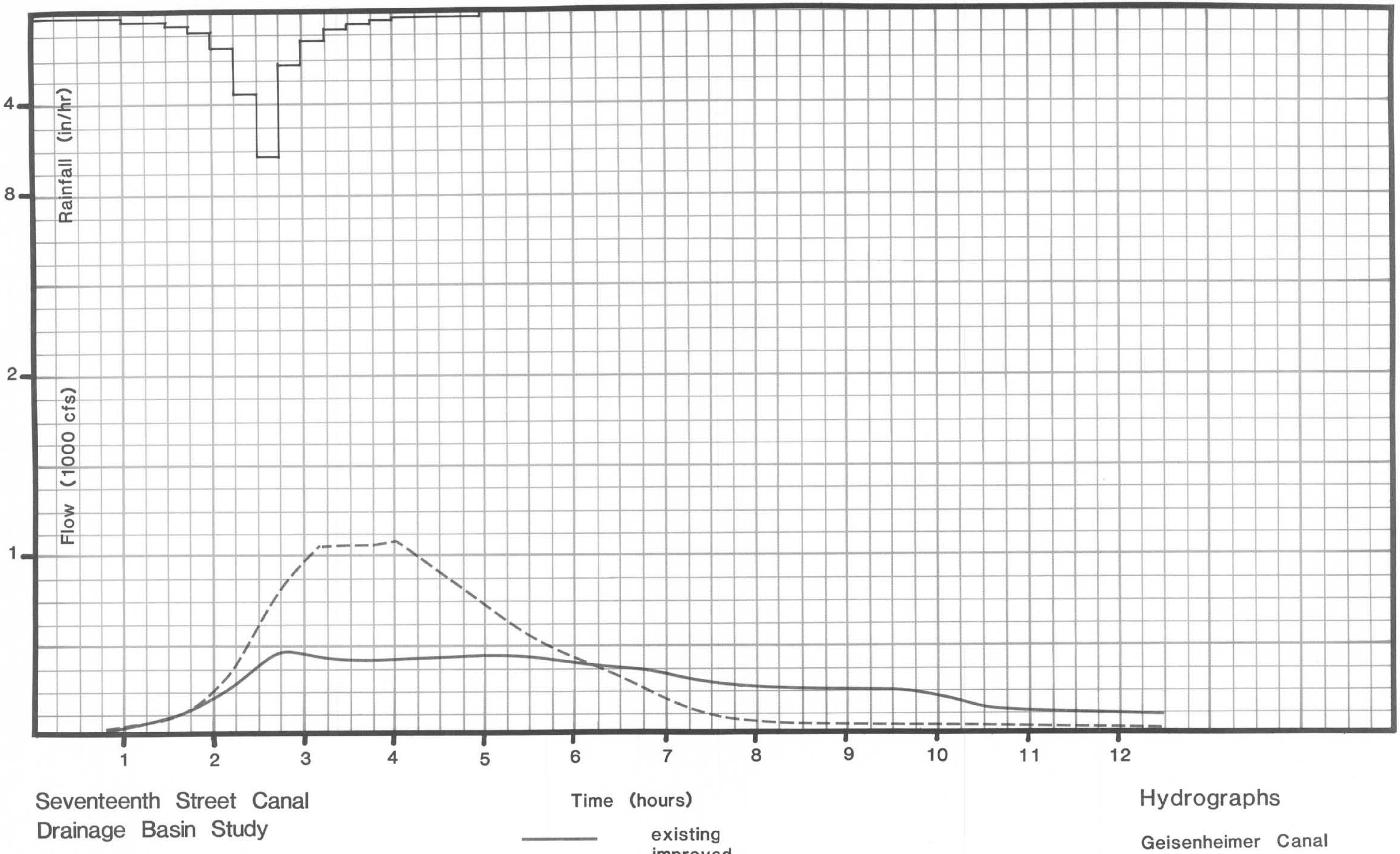
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrographs

Hoey's Canal
before Geisenheimer

Figure 21



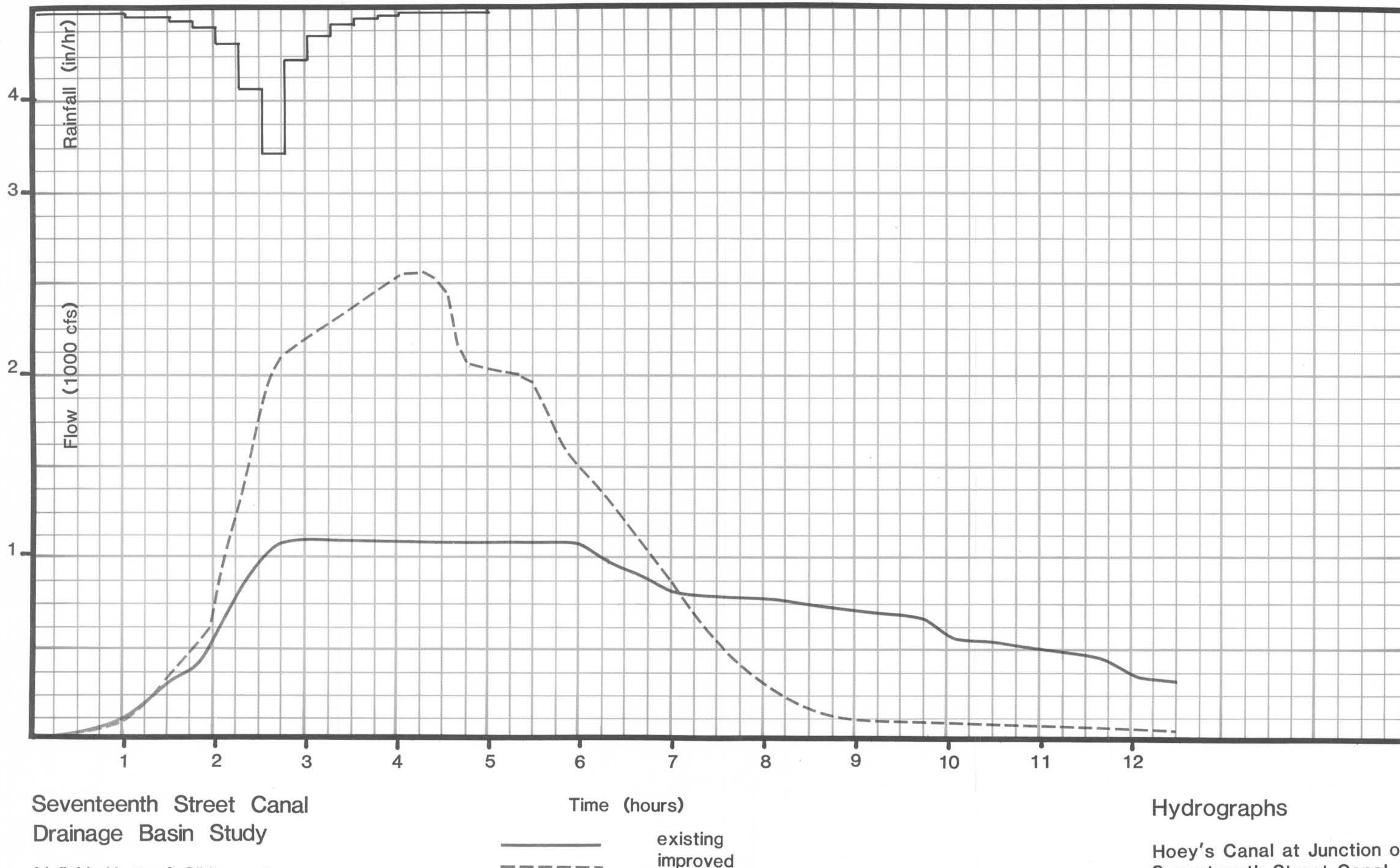
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrographs

Geisenheimer Canal

Figure 22



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrographs

Hoey's Canal at Junction of Seventeenth Street Canal

Figure 23

SECTION 3

TRANSPORT SYSTEM

CHAPTER VII
INTERIOR SYSTEM

Runoff from the Orleans Parish Drainage Basin, and from the 7th and 8th Wards of Jefferson Parish flows through a system of interior canals and pumping stations to Lake Pontchartrain. This system consists of the Palmetto and 17th Street Canals, and Drainage Pumping Station Nos. 1 and 6.

INTERIOR CANALS

The Palmetto Canal receives flow from Pumping Station No. 1 as well as minor gravity flow along its length. This gravity flow enters through a series of small pipes, many equipped with flap gates. The area drained by the Palmetto Canal is approximately bounded by the Mississippi River to the south, the Pontchartrain Expressway to the east, Audubon Place to the west, South Broad to the North, plus a narrow strip on either side of the canal. The Palmetto Canal flows into the 17th Street Canal at the junction of Bamboo Road and Northline Street.

The 17th Street Canal, located at the boundary of Orleans and Jefferson parishes, flows as an open channel from Claiborne Avenue northward to Lake Pontchartrain. Throughout this report, the term "17th Street Canal" is used to refer to the segment of the canal from Claiborne Avenue to Pumping Station No. 6. The term "Outfall Canal" refers to the segment from Pumping Station 6 to Lake Pontchartrain.

The 17th Street Canal receives seven major flows. The Lowerline flow enters at Claiborne Avenue, marking the beginning of the open canal. Prior to this point, the canal is located underground. Flow from the Oleander Basin is pumped or flows by gravity into the 17th Street Canal just south of Airline Highway. The third flow is gravity flow brought in by Hoey's Canal, draining parts of the 7th and 8th Wards of Jefferson Parish. As previously stated, the Palmetto Canal flows into the 17th Street Canal approximately at the junction of Northline Street and Bamboo Road. The fifth and sixth inflows occur at Northline Street and Metairie Road, and drain small areas of Jefferson Parish. The last flow is carried by the Avenue C Canal. This canal leads from Pumping Station No. 6 to 12. The flow in the Avenue C Canal is largely influenced by the water surface elevation at the two pumping stations, and is therefore at times, capable of flowing in either direction. During medium to low flow conditions in the 17th Street Canal, the Avenue C Canal will carry flow towards it. During the high flow conditions in the 17th Street Canal, the Avenue C Canal probably flows towards Pumping Station No. 12.

DRAINAGE PUMPING STATIONS

The study basin is drained by two major pumping stations. Pumping Station No. 1 receives gravity flow from the Orleans basin and pumps into the Palmetto Canal. Pumping Station No. 6, on the other hand, receives flow from the 17th Street Canal and pumps into the Outfall Canal, which leads to Lake Pontchartrain.

PUMPING STATION NO. 1

General

Station 1 is located on South Broad at Melpomene. Its pump inventory consists of two twelve-foot diameter Wood screw pumps (Pumps A & B), three fourteen-foot diameter Wood screw pumps (Pumps C, D and E), two vertical pumps (Pumps V1 & V2), and one constant duty pump. Pump curves for the 12' Wood screw, 14' Wood screw and vertical pumps are shown in Appendix C.

Station 1 is capable of discharging into either of two canals. The Broad Street Canal leads from Station 1 to Station 2 and is used only for dry weather flows. During normal operations storm water is discharged into the Palmetto Canal. For these reasons, the constant duty pumping capacity (dry weather flows) is not figured into the station capacity, and all storm flow is assumed to be discharged only into the Palmetto Canal.

Mode of Operation

Operation of Station 1 begins when a report of rain in the Orleans basin is received, by first free wheeling the two twelve-foot Wood screw pumps (Pumps A & B). As storm water reaches the station and the water elevation in the suction basin begins to rise, either one or both of the vertical pumps are started, and vacuum priming of Pumps A and/or B begins. The B pump is usually the first of the Wood screw pumps started (loaded). Loading occurs when the operator feels that the incoming storm flow is sufficient to support the pump operation. Loading the pump too soon will result in too great a drawdown of the water surface elevation on the suction side of the pump, and the pump will lose prime. Should this occur, the pump will again have to be evacuated before pumping can begin. The evacuation and loading process takes between five and ten minutes, depending on the pump and the number of pumps being loaded. Additional pumps are loaded as required to maintain a predetermined elevation in the suction side of the station. The two vertical pumps are used largely to recover pumping capacity when a screw pump loses prime, and between loading cycles. The number of pumps in operation is maintained until the suction side water elevation drops and the pumps lose prime. A total capacity curve for Station 1 is shown in Figure 24. This illustration assumes that all pumps are operating at a given time and represents the maximum existing capacity.

All pumping activity is recorded by the station operator in the daily log reports. These reports contain suction and discharge water surface elevations, all pump records of times on, loaded, lost prime, and off, as well as any pertinent comments which the operator makes. Suction and discharge water surface elevations are read from continuous recorder gages located within the station.

Pump Station Calibration

Use of the Storm Water Management Model (SWMM) requires a calibration and a check in order to evaluate the reproducibility and the adequacy of the constants used within the model. Rainfall and pumping data collected for the storm event of November 26, 1980, were used to calibrate the model, and data for the storm event of April 22, 1979, was used to verify the model. To perform the calibration and verification, the total quantity of water pumped and the shape of the hydrographs are compared for the pumped values and those predicted by the model. The pumped hydrograph and the total flow pumped are determined from the pump station log records and suction and discharge gage recorder charts. The results of these calculations are shown in Table 5 and Figure 14 for the calibration storm, and in Table 6 and Figure 15 for the verification storm. The calculations were performed as follows:

Pumping times for each discrete pumping sequence were determined from the station pump log records. For each time period, average suction and discharge elevations were determined, and a differential head calculated as the difference between the two, plus one foot. The additional foot accounts for the water surface drawdown between the location of the suction side recorder gage, and the actual pump location; in some instances a distance in excess of one hundred feet. This additional foot of head loss was verified during the storm event of September 20, 1982, as the difference between the height of a calibrated water column at a tap in the vertical Pump No. 1 and the gage reading, with the B pump on.

The individual pumping capacities were determined at the differential head for each time period and the station pumping capacity was calculated as the sum of the individual capacities. The quantity pumped during any time period is the product of the station capacity during the period, and the length of the period. The total quantity pumped is the sum of the quantities pumped for all the discrete time periods.

PUMPING STATION NO. 6

General

Station 6 is located on Orpheum Street at Hyacinth Street. Its pump inventory consists of two twelve-foot diameter Wood screw pumps (Pumps A & B), four fourteen-foot diameter Wood screw pumps (Pumps C-F), one twelve-foot diameter screw pump (G pump), and two constant duty pumps. Pump curves for the twelve-foot Wood screw, fourteen-foot Wood screw, and twelve-foot screw pumps are shown in Appendix C. As in Station 1, the constant duty pumps are only used for dry weather flows and are thus not included in the total station capacity. Total station capacity is shown in Figure 25. Station 6 receives flow from the Orleans Basin and areas of Jefferson Parish via the 17th Street Canal and discharges into the Outfall Canal.

Mode of Operation

Normal operation of Station 6 is similar to that of Station 1. It should be noted, however, that unlike Station 1, Station 6 receives both pumped flow and gravity flow. Suction side elevations at Station 6 need to be maintained below certain levels if the Lowerline Basin, and Jefferson Parish are to be drained by gravity. Additionally, any disruption in operation of Station 6 directly reflects as a necessary shutdown of Station 1, to prevent overflowing of the Palmetto Canal.

TABLE 5
PUMPING STATION NO. 1 — CALIBRATION CALCULATIONS
(STORM DATED NOV. 26, 1980)

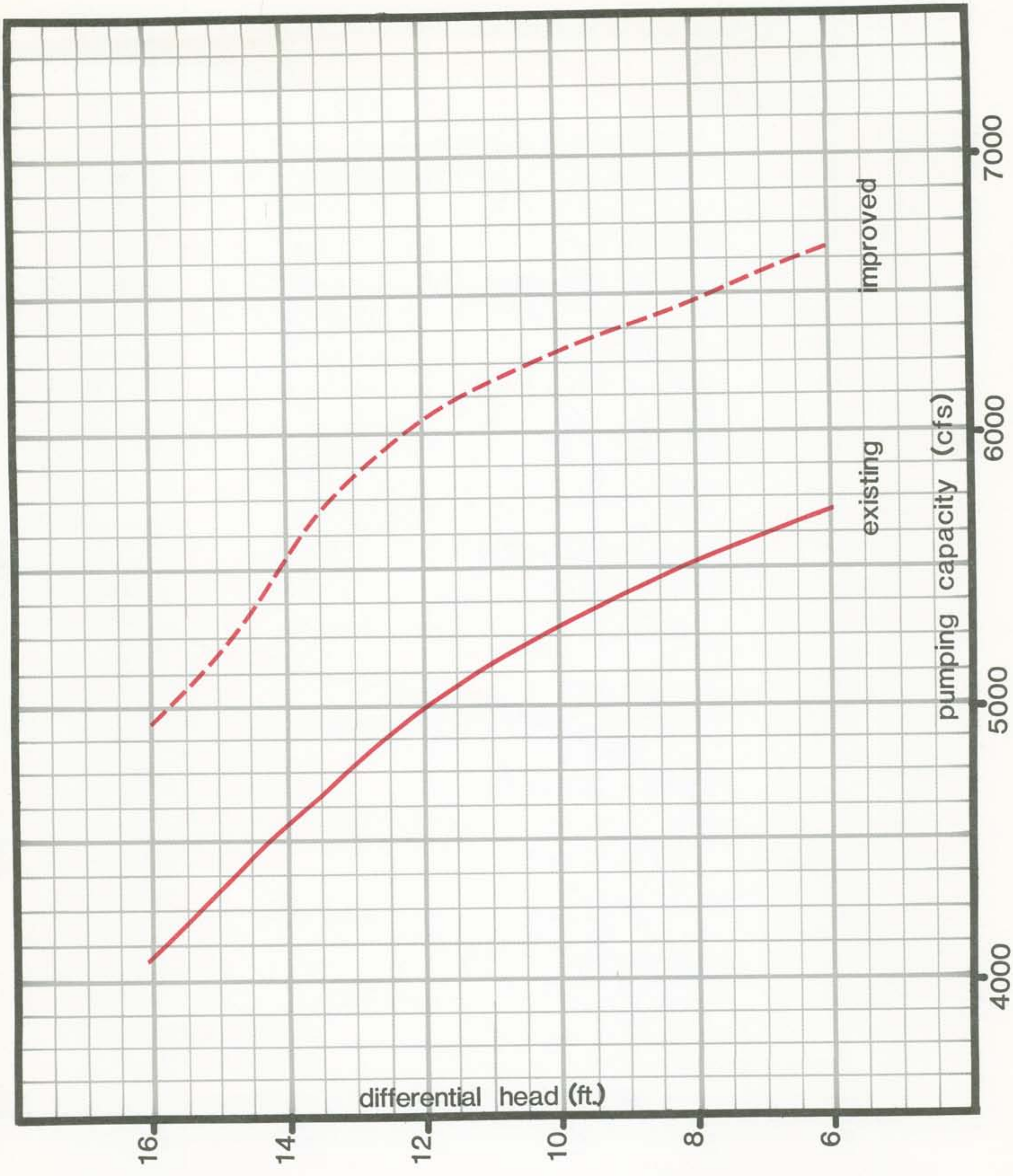
TIME	TIME PERIOD (MIN.)	AVERAGE ELEVATIONS (C.D.) SUCTION	DISCHARGE	HEAD (FT.)	PUMPING SEQUENCE (PUMPS ON)	PUMPING CAPACITIES (C.F.S.) VERT.	12' DIA.	14' DIA.	STATION PUMPING CAPACITY (C.F.S.)	VOLUME PUMPED (CU. FT.)
5:34 am										
	26	9.0	13.6	5.6	V_2	245			245	3.82×10^5
6:00										
	68	10.7	16.2	6.5	V_2B	242	690		932	3.80×10^6
7:08										
	42	10.4	17.4	8.0	V_2BA	238	660		1558	3.93×10^6
7:50										
	1	11.2	18.4	8.2	V_2BAE	236	658	1220	2772	1.66×10^5
7:51										
	42	11.4	19.5	9.1	BAE		630	1225	2485	6.26×10^6
8:33										
	19	11.2	20.6	10.4	BAED		590	1190	3560	4.06×10^6
8:52										
	6	11.0	20.0	10.0	BED		610	1205	3020	1.09×10^6
8:58										
	57	11.4	19.0	8.6	BE		647	1230	1877	6.40×10^6
9:55										
	3	10.4	17.8	8.4	B		653		653	1.18×10^5
9:58										
	82	9.8	16.2	7.4	BV_2	239	670		909	4.47×10^6
11:28										
	190	8.0	14.2	7.2	V_2	240			240	2.74×10^6
2:30 pm									TOTAL	3.34×10^7

KEY: Vertical Pumps — V_1&V_2
12' Dia. Screw Pumps — A & B
14' Dia. Screw Pumps — C, D, & E

TABLE 6
PUMPING STATION NO. 1 — CALIBRATION CALCULATIONS (STORM DATED APRIL 22, 1979)

TIME	TIME PERIOD (MIN.)	AVERAGE ELEVATIONS (C.D.) SUCTION	DISCHARGE	HEAD (FT.)	PUMPING SEQUENCE (PUMPS ON)	PUMPING CAPACITIES (C.F.S.) VERT.	12' DIA.	14' DIA.	STATION PUMPING CAPACITY (C.F.S.)	VOLUME PUMPED (CU. FT.)
9:30 am	19	8.7	13.0	5.3	V_1	245			245	2.79×10^5
9:49	7	9.3	15.2	6.9	B		680		680	2.85×10^5
9:56	46	10.8	17.0	7.2	BA		675		1350	3.72×10^6
10:42	28	10.0	16.2	7.2	BV_1	240	675		675	1.13×10^6
11:10	18	11.6	18.0	7.4	BAV_1	239	670		1340	1.44×10^6
11:28	5	11.8	19.2	8.4	$BAEV_1$	236	650	1235	2535	7.60×10^5
11:33	15	11.6	19.0	8.4	BAE		650	1235	2535	2.28×10^6
11:48	27	11.0	18.6	8.6	BE		648	1233	1880	3.04×10^6
12:15 pm	5	11.6	19.0	8.4	BAE		650	1235	2535	7.60×10^5
12:20	4	12.6	20.0	8.4	$BAEV_1V_2$	236	650	1235	3007	7.21×10^5
12:24	7	12.7	20.4	8.7	BAED		645	1230	3750	1.57×10^6
12:31	9	12.6	21.4	9.8	$BAEDV_1$	232	615	1217	3896	2.10×10^6
12:40	2	12.2	21.0	9.8	$BAEV_1$	232	615	1217	2679	3.22×10^5
12:42	48	12.8	20.7	8.9	$BAEV_1V_2$	234	637	1225	2733	7.87×10^6
1:30	3	10.9	20.0	10.1	$BAEV_1$	230	605	1200	2640	4.75×10^5
1:33	5	10.7	19.8	10.1	BAE		605	1200	2410	7.23×10^5
1:38	32	11.0	18.8	8.8	BE		640	1227	1867	3.59×10^6
2:10	5	10.7	17.7	8.0	B		660		660	1.98×10^5
2:15	35	9.8	17.4	8.6	BA		647		1294	2.71×10^6
2:50	1	9.4	17.0	8.6	B		647		647	3.88×10^4
2:51	12	9.4	16.7	8.3	BV_1	237	653		890	6.40×10^5
3:03	115	9.2	15.8	7.6	B		668		668	4.60×10^6
4:58	1				⊖					
4:59	11	8.3	15.4	8.1	V_1V_2	238			476	3.14×10^5
5:10	210	8.2	14.4	7.2	V_1	240			240	3.02×10^6
8:40	55				⊖					
9:35	57	7.4	14.0	7.6	V	239			239	8.17×10^5
10:32										
									TOTAL	4.35×10^7

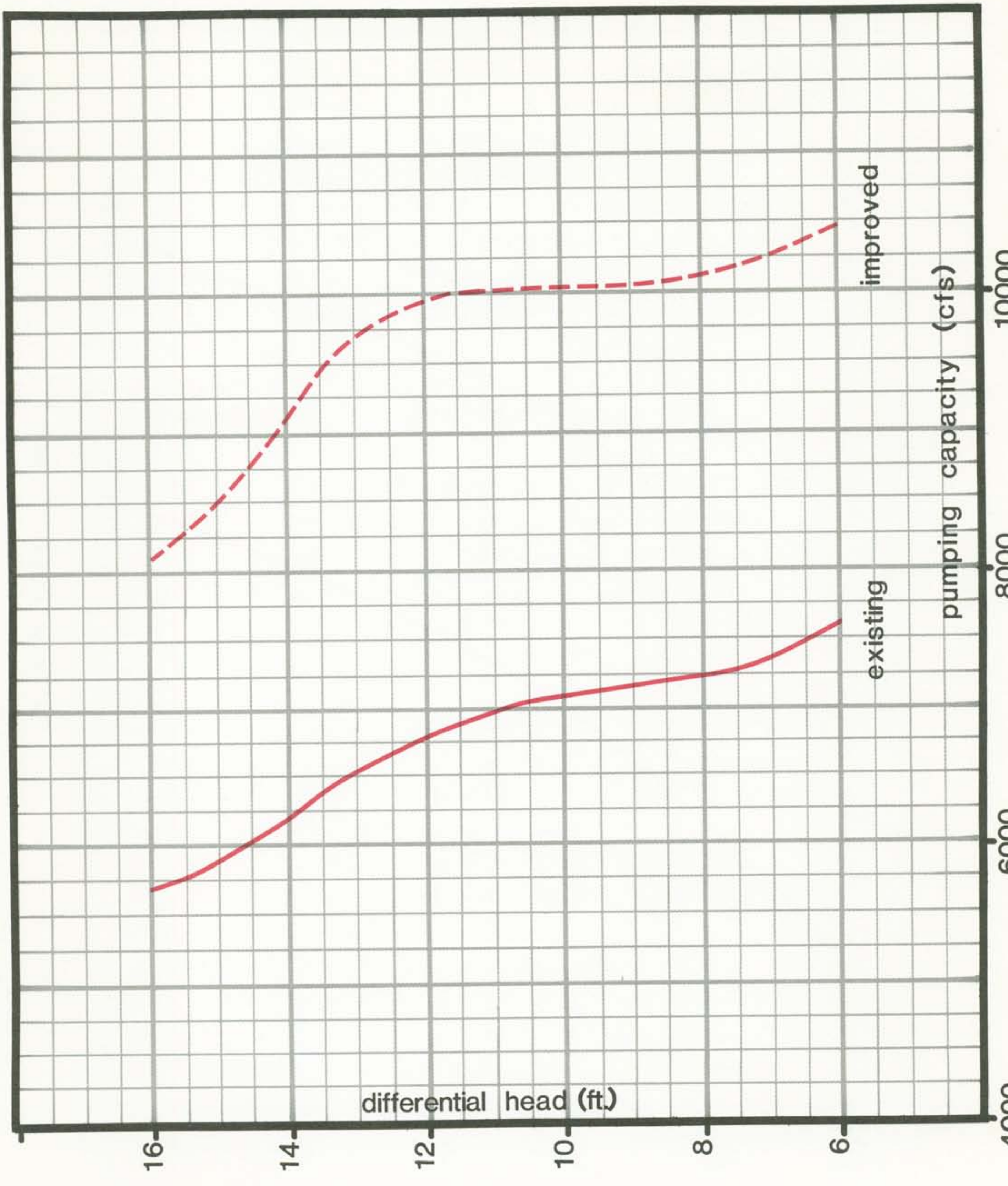
KEY: Vertical Pumps — V_1&V_2
12' Dia. Screw Pumps — A & B
14' Dia. Screw Pumps — C, D, & E



System Pump Curve
Drainage Pumping Station No. 1
Existing and Improved

Seventeenth Street Canal Drainage Basin Study

Figure 24



System Pump Curve
Drainage Pumping Station No. 6
Existing and Improved

Seventeenth Street Canal Drainage Basin Study

Figure 2

CHAPTER VIII

ANALYSIS OF INTERIOR SYSTEM

Analysis of the interior canals was made using the January, 1981 version of HEC-2 Water Surface Profiles Program of the Hydrologic Engineering Center.

PROGRAM DESCRIPTION

The HEC-2 program is used for calculating water surface profiles for gradually varied, steady flow in channels. The effects of various obstructions such as bridges, culverts, and structures in the flood plain are modeled. The computational procedure is based on solving the one-dimensional energy equation with energy loss due to friction evaluated with the Manning equation. The computational procedure used is the Standard Step Method. Energy losses due to bridges or culverts are calculated in two parts. The first part considers losses that occur in the reaches immediately upstream and downstream of the bridge. The second part consists of losses due to the structure itself and is calculated using one routine for culverts and bridges without piers, and another using the Yarnell equation for bridges with piers. The program has capabilities for designing channel improvements known as the CHIMP option, as well as capabilities for modeling man-made levees. Data required by the program include: flow regime, starting elevation, discharge, loss coefficients, cross section geometry, and reach lengths.

Several energy loss coefficients are used by the program to evaluate head losses. These include Manning's 'n' values for friction losses, contraction and expansion coefficients to evaluate transition losses, and bridge shape coefficients used to evaluate losses related to weir flow, pressure flow and pier configuration. Table 7 below indicates Manning's 'n' values used.

TABLE 7

Conditions	'n'
Existing Outfall Canal	0.04
Improved earthen canals	0.024
Earthen canal overbanks	0.035
Concrete lined canals	0.012
Lake Pontchartrain	0.02

Values used for other coefficients in the program were chosen in accordance with the HEC-2 manual's recommendations. Modeling of interior canals and all improved canals assumes clean sides and inverts. These conditions must be maintained to realize maximum flow.

PALMETTO CANAL

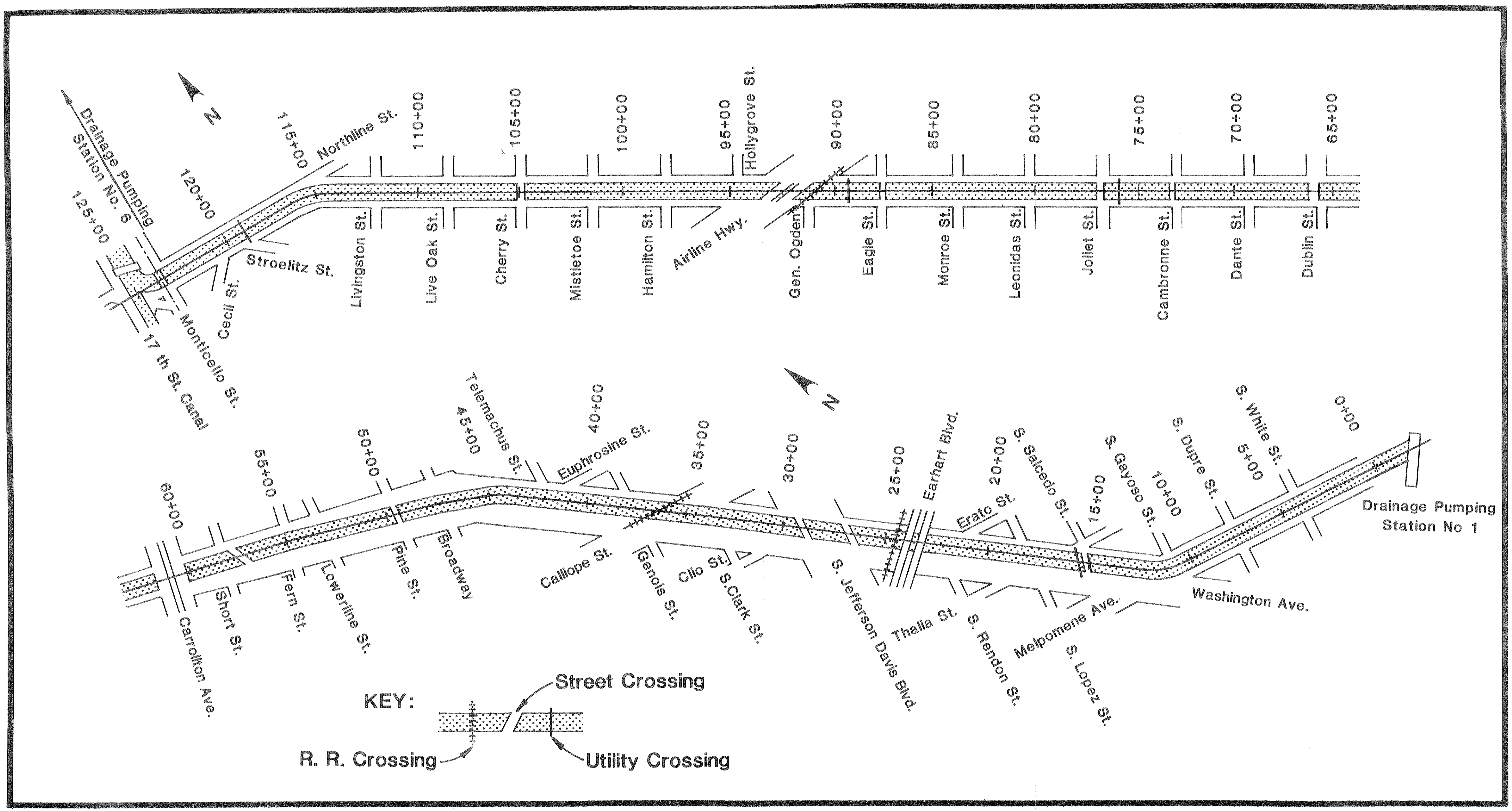
The Palmetto Canal is a concrete lined channel flowing from Pumping Station No. 1 to the 17th Street Canal. The channel cross section is very uniform throughout; the main variation is the position of the dry weather subchannel within the canal. A plan of the canal with the stationing used is shown in Figure 26. Typical cross sections are shown in Figure 27. Offset stationing was coded with zero being on the left descending bank for all cross sections.

At present there are forty-four crossings of the canal. The Sewerage and Water Board of New Orleans is in the process of relocating many of the present crossings as clear span bridges over the canal. Crossings planned for relocation in the near future are not included in the study. These include all pedestrian crossings, the White Street bridge, and most utility crossings. Crossings included in the study are shown in Figure 26. Existing crossings to remain are modeled with the Special Bridge Option to account for structure losses due to obstructions in the line of flow.

17TH STREET CANAL

The 17th Street Canal flows as an open channel from Jefferson Highway, north to Pumping Station No. 6. The channel cross section is uniform only within specific reaches. From Station 6, to an area in the vicinity of the junction of Hoey's Canal, the section is concrete lined. Southward from this junction the cross section lining varies from concrete to concrete with wooden side planks to largely earthen. Several areas within this reach are in need of repair. A plan of the canal is shown in Figure 28. Typical cross sections for the 17th Street Canal are shown in Figures 29 and 30.

Crossings in the 17th Street Canal were modeled using the normal bridge option for culverts, and the special bridge option for crossings with piers in the line of flow. Several obstructions which were largely outside the line of flow were not considered to contribute energy losses.



Seventeenth Street Canal
Drainage Basin Study

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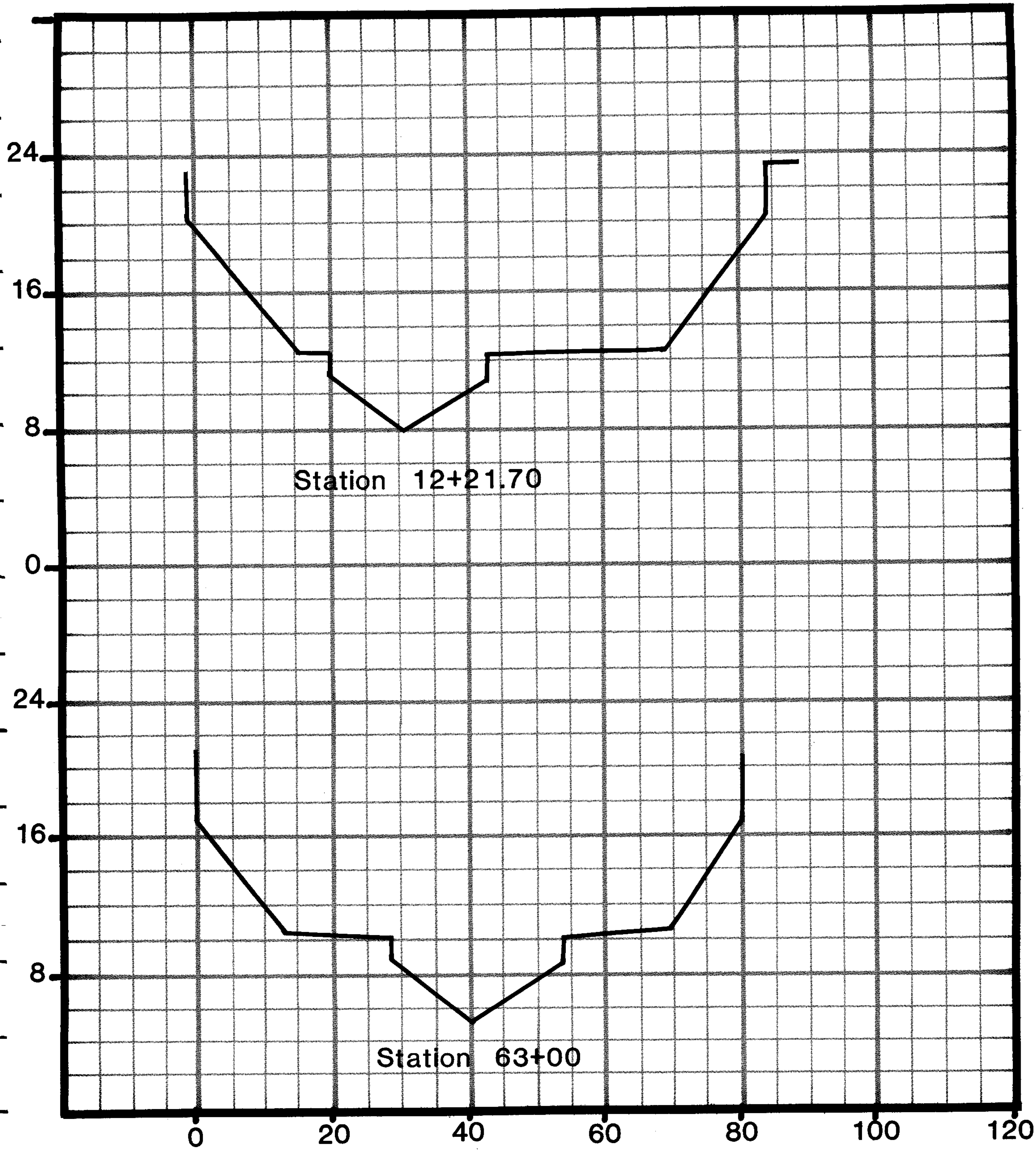
Palmetto Canal
Plan

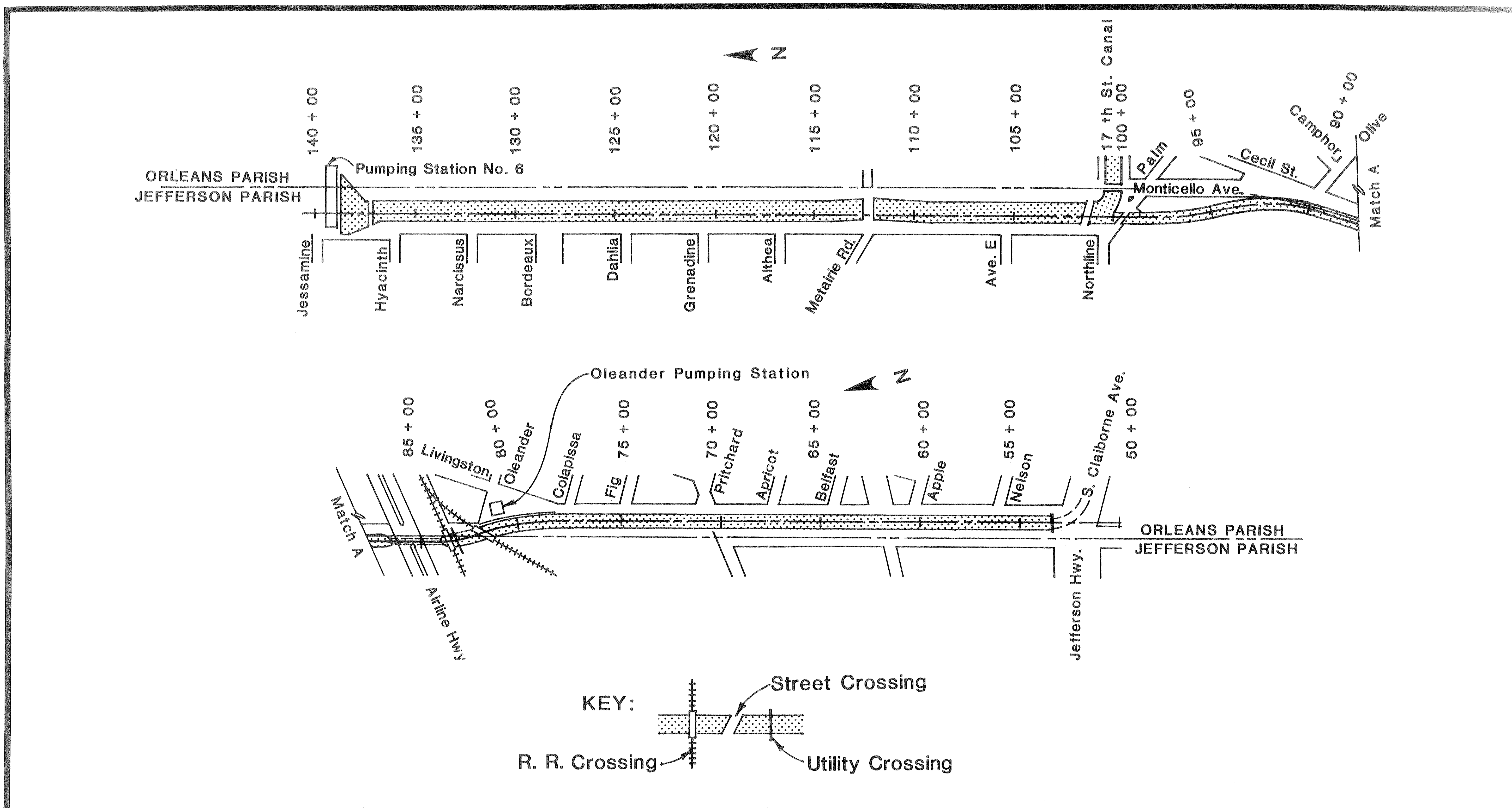
Figure 26



Typical Cross Sections for Palmetto Canal

Seventeenth Street Canal Drainage Basin Study

Figure 27

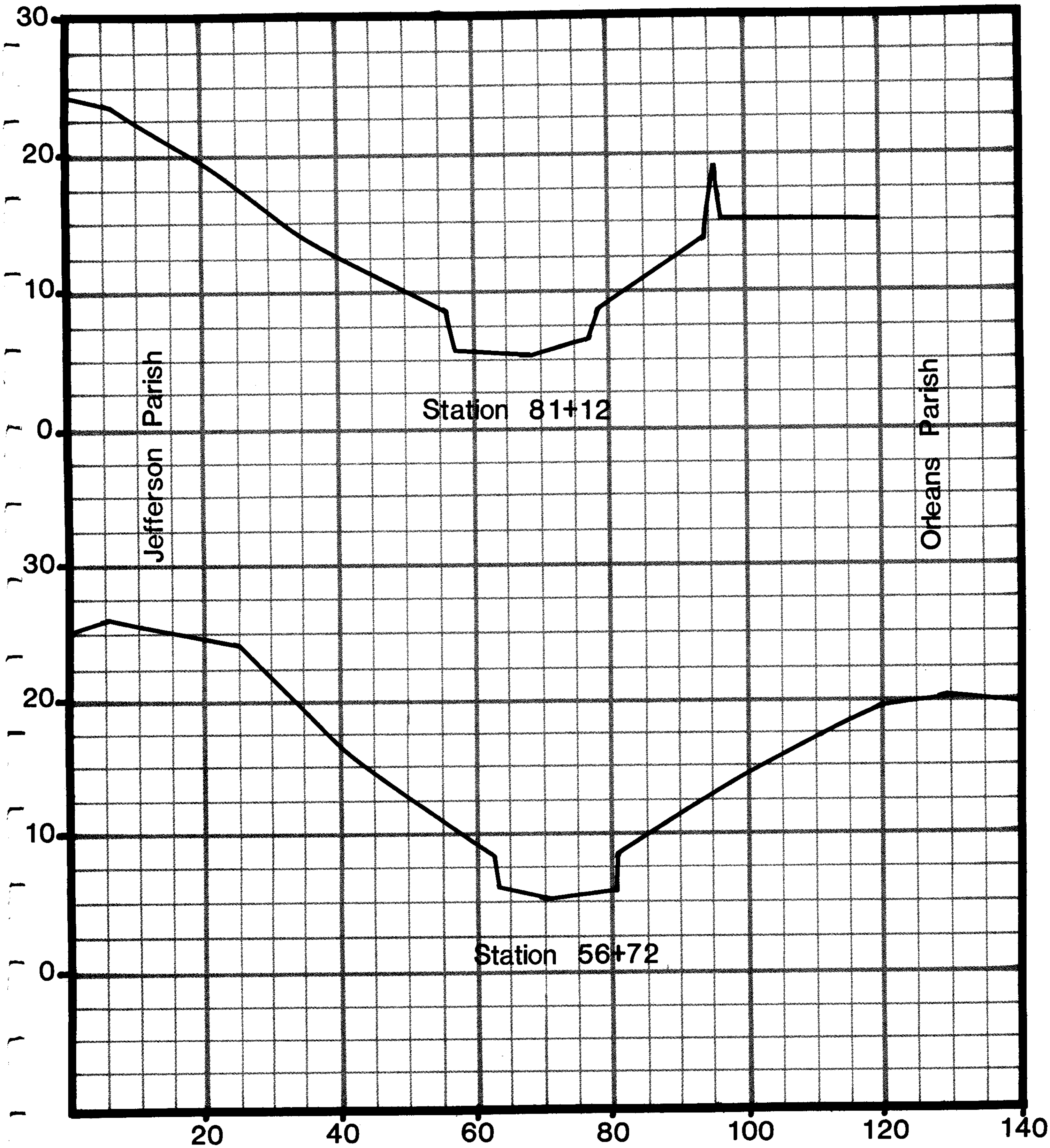


Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Seventeenth Street Canal
Plan

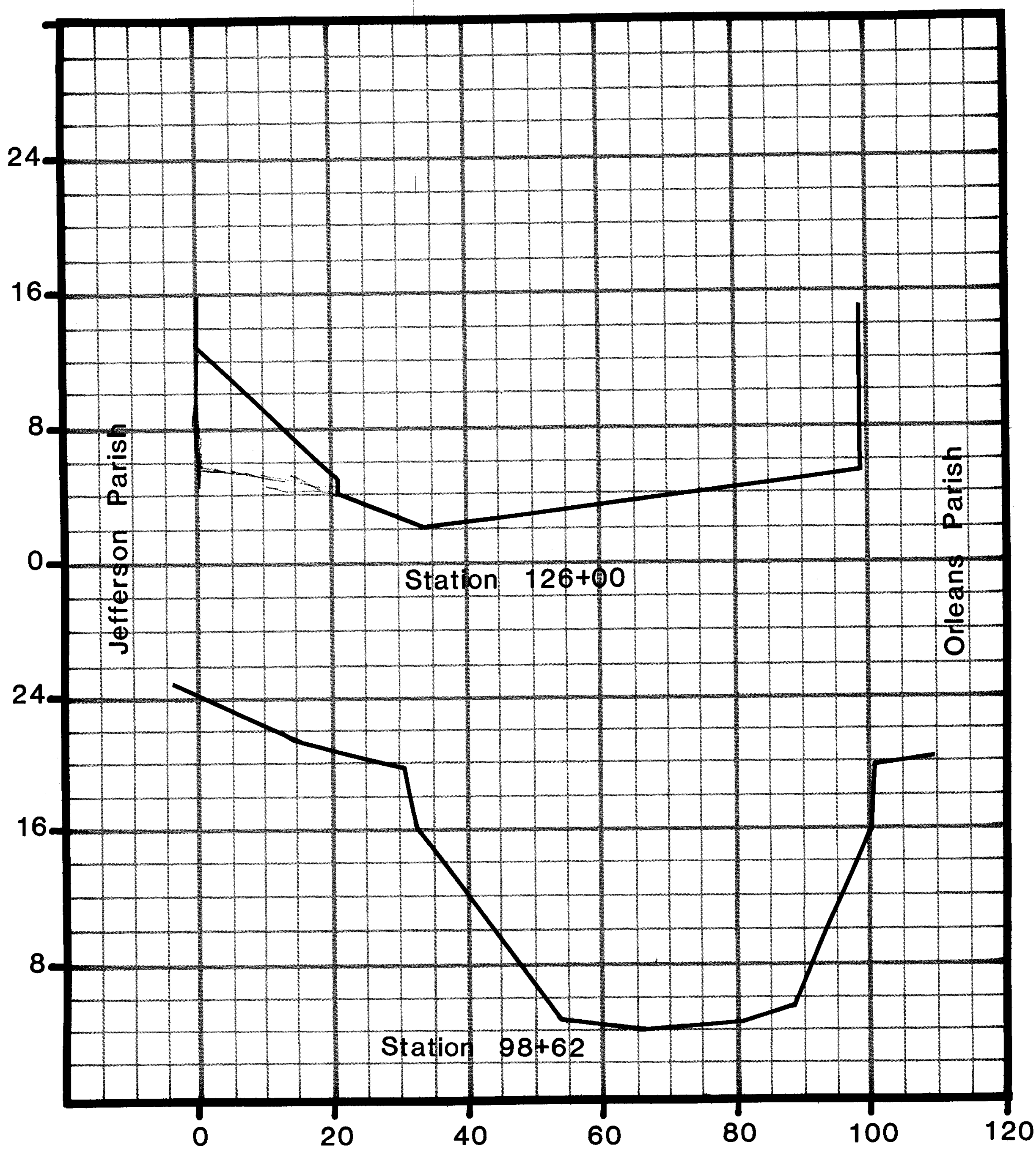
Figure 28



Typical Cross Sections for Seventeenth Street Canal

Seventeenth Street Canal Drainage Basin Study

Figure 29



Typical Cross Sections for Seventeenth Street Canal

Seventeenth Street Canal Drainage Basin Study

Figure 3

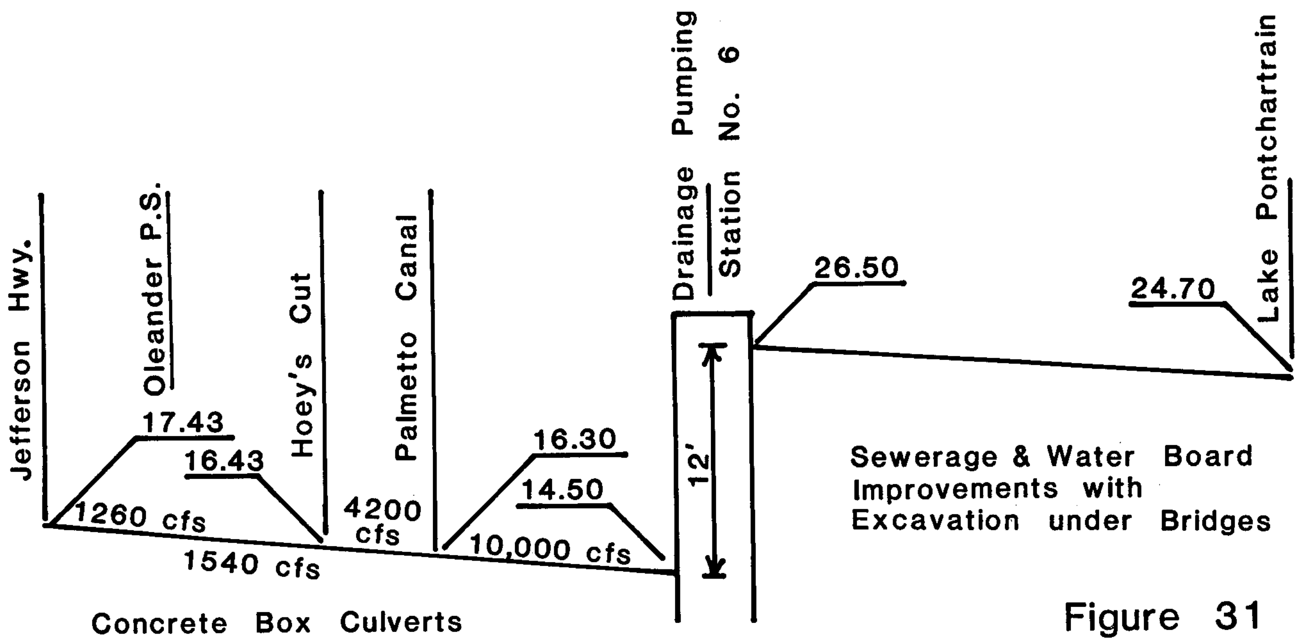


Figure 31

Profile - Maximum Allowable Water Surface Elevations
Design Flow 10,000 cfs

FINDINGS

Various flow profiles were run through the 17th Street and Palmetto Canals. A starting water elevation at Pumping Station No. 6 of 14.0 feet C.D. was used. This value corresponds with field data as the high water elevation on the suction side of Station 6. This forebay elevation allows a discharge side elevation of 26.5 feet C.D. Under existing conditions the carrying capacity of the 17th Street Canal is in the range of 5300 cfs, while that of the Palmetto Canal is approximately 3500 cfs. Higher flows caused localized overflowing of the Palmetto Canal and impeded the gravity drainage into the 17th Street Canal. Since the required capacity of the Palmetto Canal is 6000 cfs and 10,000 cfs in the 17th Street Canal, considerable improvements are required.

IMPROVEMENTS

Analysis of the collection system as described in Section 2 indicates that the flow in the future will be 10,000 cfs at Pumping Station No. 6. Major improvements to the transport system are required to accommodate this improved flow. The present capacity of the 17th Street Canal will have to be increased in a way that will allow gravity drainage from the Lowerline Basin in Orleans, and from Hoey's Basin in Jefferson Parish. In order to maintain gravity drainage, the following elevations must not be exceeded: Elev. 14.5 C.D. at Station 6, Elev. 16.43 C.D. (−4.0 M.S.L.) at the junction of Hoey's Canal, and Elev. 17.43 C.D. (−3.0 M.S.L.) at Jefferson Highway. Figure 31 shows these requirements in profile. Significant departure from these maximum allowable elevations results in a very uneconomical design. Preliminary HEC-2 calculations indicate that an open concrete box can be constructed in the 17th Street Canal that will provide these elevations.

In order to accommodate proposed improvements, the capacity of the Palmetto Canal will have to be increased from the existing 3500 cfs to approximately 6000 cfs. This improvement will require a combination of removal or raising of all the existing structures and possibly an increase in the cross sectional area of the canal.

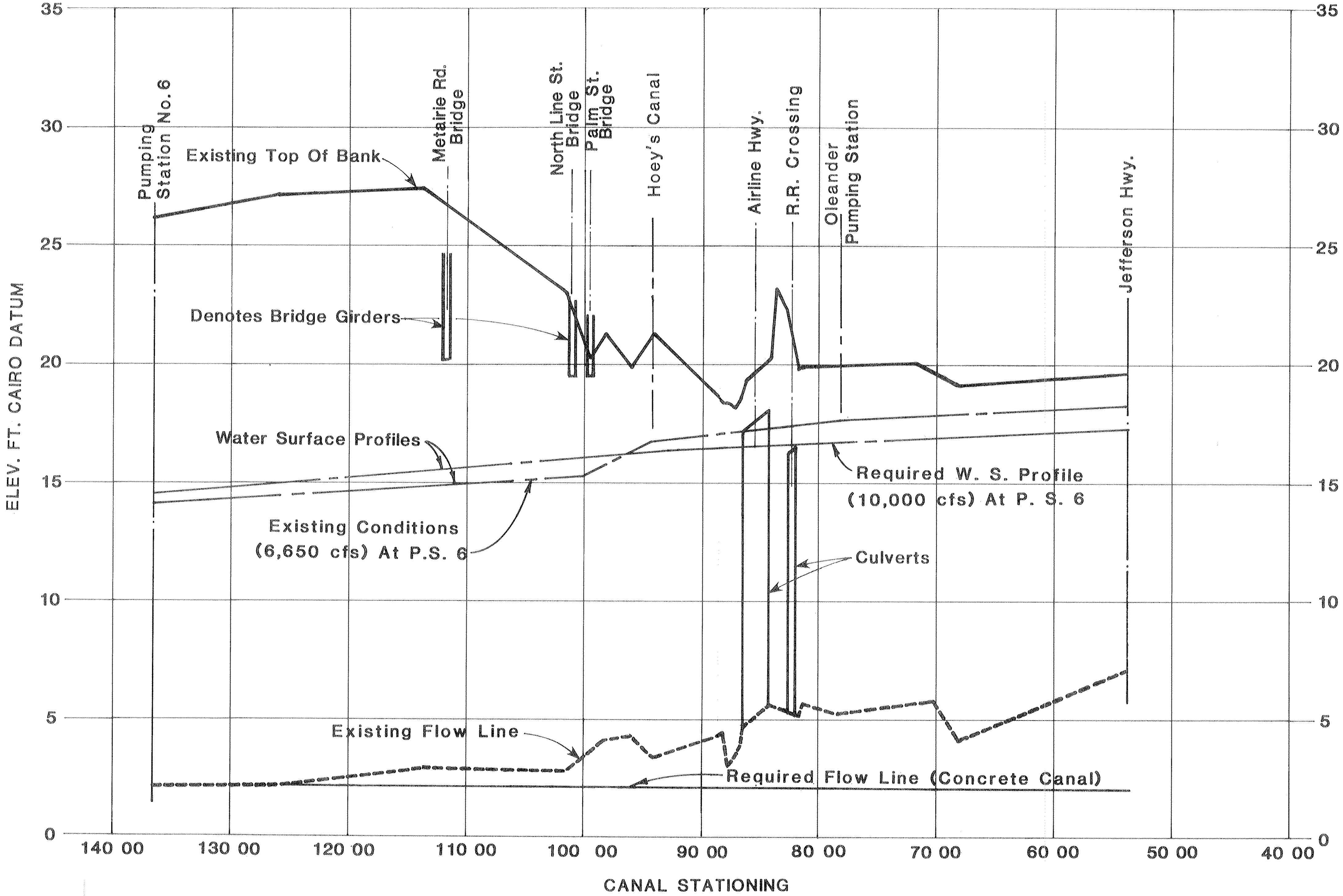
Table 8 shows water surface profile elevations for several flows routed through the 17th Street Canal and the Palmetto Canal for existing and improved conditions. This information is also shown in profile on Figures 32 and 33.

Pumping capacities of the two pumping stations will have to be upgraded to handle improved flows. Pumping Station No. 1 will have to be improved from its nominal capacity of 5400 cfs to 6000 cfs. Pumping Station No. 6 will likewise need to be increased from 6650 cfs to 10,000 cfs. Improved pump curves for Stations 1 and 6 are shown on Figures 24 and 25 respectively.

TABLE 8

INTERIOR CANALS' ELEVATIONS

CONDITION	Q (CFS)	ELEVATION (C.D.) AT:				
		STATION 6	PALMETTO	HOEY	JEFFERSON HWY.	STATION 1
Existing Conditions	5,250	14.0	14.8	15.6	16.7	21.5
	6,650	14.0	15.3	16.5	18.0	Overflows
Improved 17th St. Canal Exist. Palmetto Canal	10,000	14.5	16.1	16.5	17.5	Overflows
Improved 17th St. & Palmetto Canals	10,400	14.5	16.2	16.7	17.8	21.5

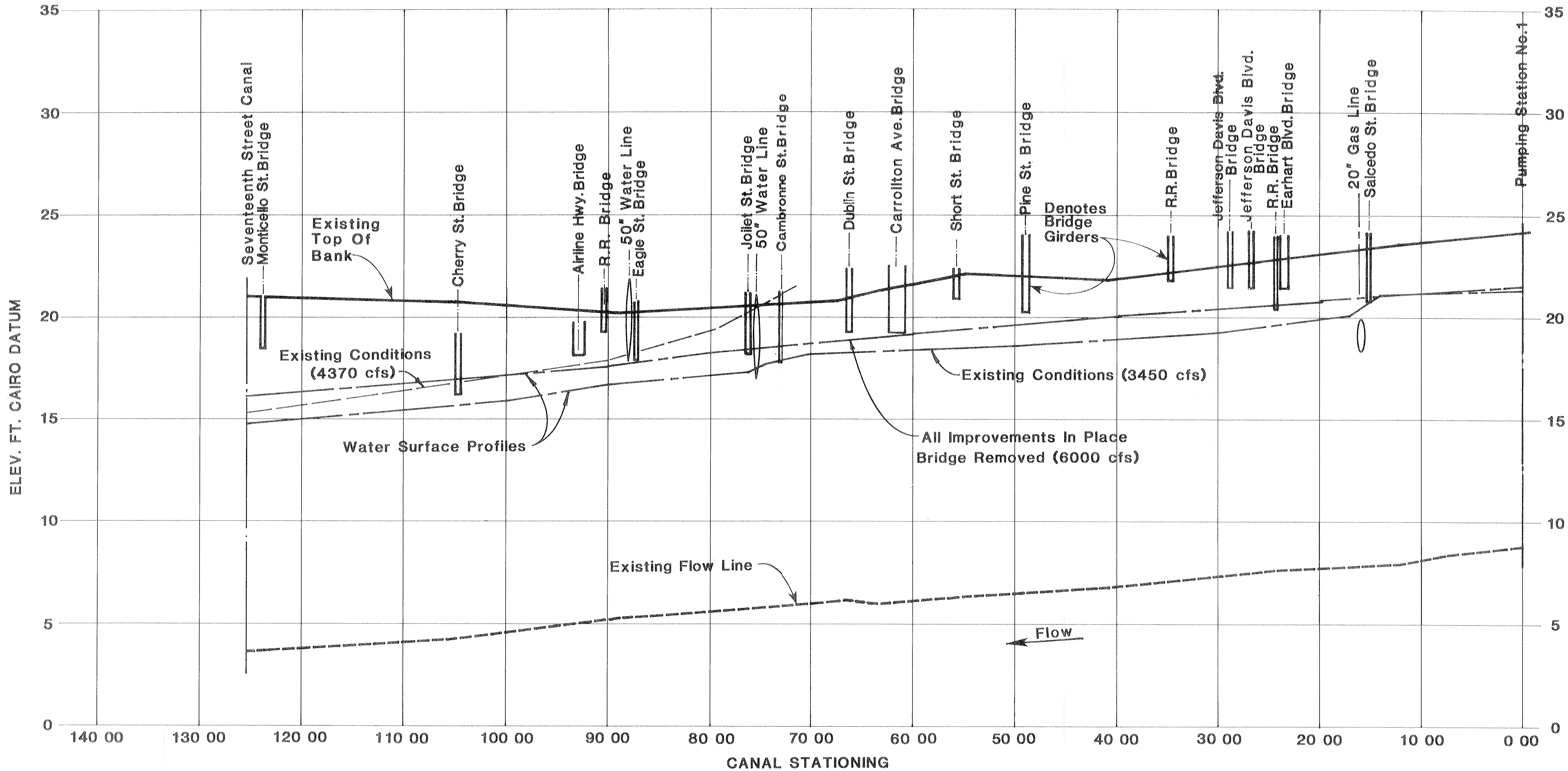


Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons Inc.

Seventeenth Street Canal
Existing Profile

Figure 32



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Palmetto Canal
Profiles

Figure 33

CHAPTER IX
OUTFALL CANAL

DESCRIPTION

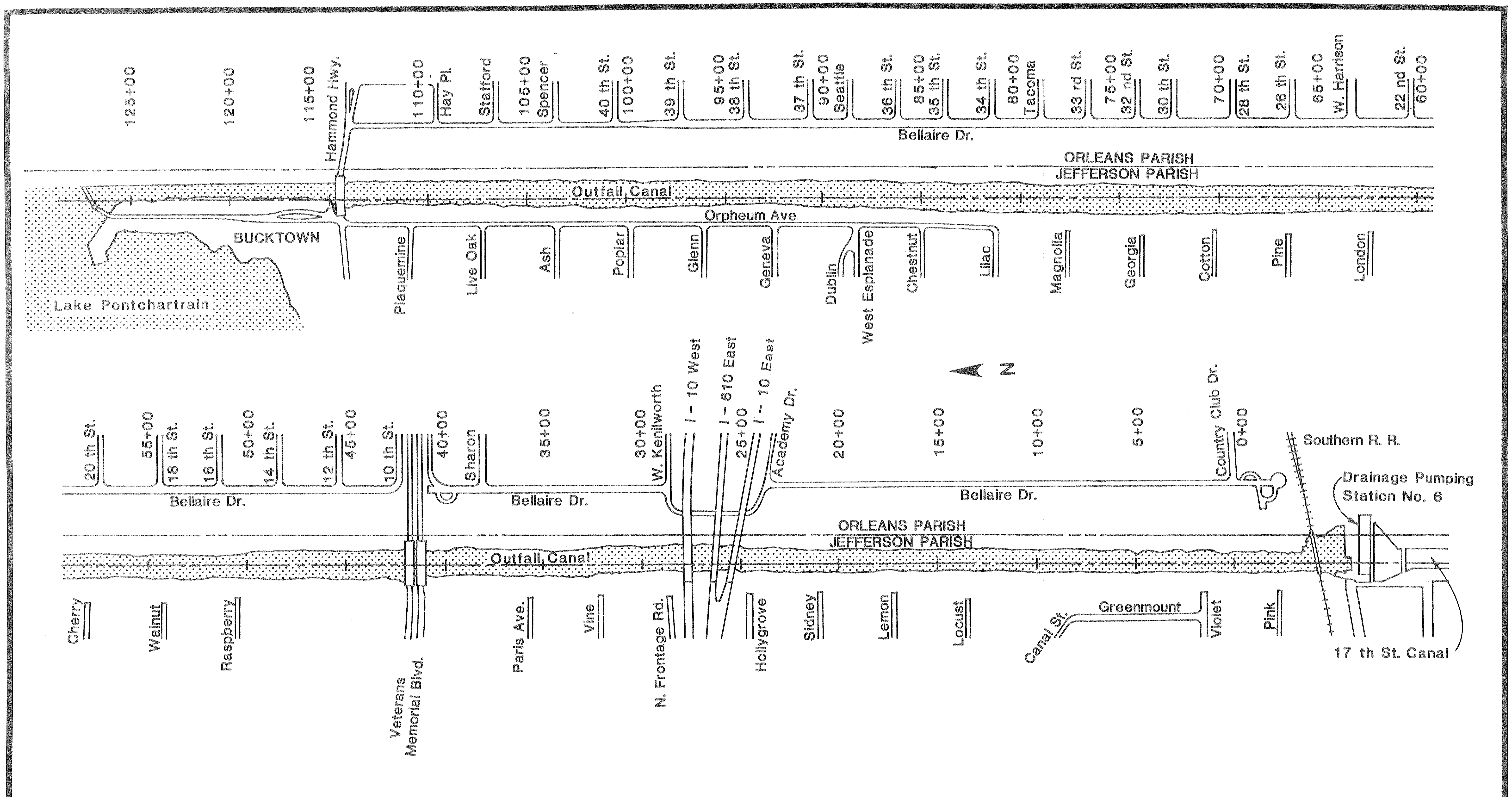
Storm water from the study area is pumped by Pumping Station No. 6 into the Outfall Canal, which then flows north along the Orleans and Jefferson Parish line into Lake Pontchartrain.

The Outfall Canal has five major crossings. The first northward crossing from Pumping Station No. 6 is the Southern Railroad bridge. Second are the three I-10 bridges, followed by the two Veterans Memorial Boulevard bridges. The fourth crossing is the Hammond Highway bridge, followed by the wooden Bucktown bridge. A plan of the Outfall Canal showing bridge locations and stationing for coded cross sections appears on Figure 34. Table 9 shows the elevation of the low cord of each structure in the line of flow.

TABLE 9
BOTTOM CHORD ELEVATIONS OUTFALL CANAL BRIDGES

Bridge	Elevation
Southern Railroad	27.1
I-10	27.8
Veterans Highway	25.9
Hammond Highway	27.2
Bucktown	N/A

The canal is an earthen canal for its entire length with a flow line elevation (thalweg) of approximately 15.0 C.D. at Station 6 and falling to an elevation of approximately 5.0 C.D. just before it enters the lake. As it enters the lake the thalweg rises to elevation 15.0 C.D. Figure 39 shows the existing canal profile. The existing cross section is fairly regular throughout its length, except for the section between the Hammond Highway bridge and the lake adjacent to the area known as Bucktown, where the channel has been constricted by fill and rip-rap placed on the Jefferson bank. Figure 36 shows a typical cross section through the existing canal upstream of Bucktown and a typical section on the Bucktown reach. The constriction in the Bucktown area has caused increased velocities through this reach, which in turn has caused considerable scour in the channel bottom, as is evidenced in the profile. In the past, this scour caused minor failures of the existing levee and sheet pile wall on the Orleans bank.

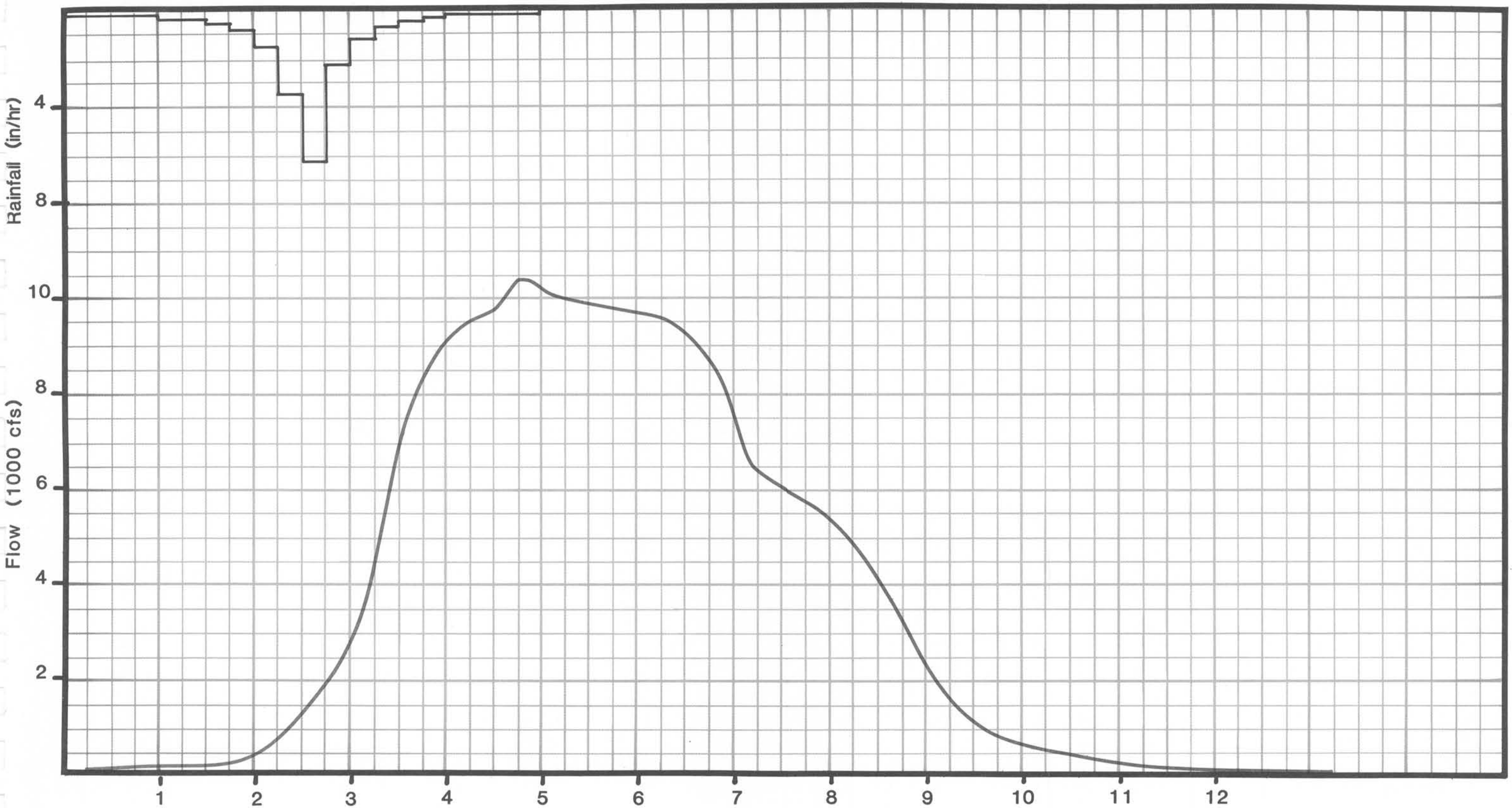


Seventeenth Street Canal Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Outfall Canal Plan

Figure 34



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Hydrograph

Pumping Station No. 6

Figure 35

PREVIOUS STUDIES

Previous hydraulic studies have been made of the Outfall Canal. The Sewerage and Water Board of New Orleans had the first study made by their consultant for their proposed plan of improvement in 1979. The Jefferson Parish Council then commissioned a study completed in 1981. A second review was undertaken for the S&WB by a third consultant later in 1981. A comparison of the findings of these studies is shown in Table 12.

The S&WB has had plans prepared for the improvement of the Outfall Canal from Pumping Station No. 6 to Lake Pontchartrain. This project includes dredging the existing channel to an approximate width of 40-50 feet for the entire length and flow line elevations of 2.0 to 3.0 C.D. at the lower end of the channel and 4.0 C.D. between I-10 and Station 6. No dredging was contemplated under the bridges. Because of an existing sand stratum, there was concern that dredging at the bridges, without driving a sheetpile bulkhead, may cause seepage through the sand and cause roadway failures. Construction in the Bucktown area, between the Hammond Highway and the lake, includes demolition of the existing structures and the construction of a protective bulkhead along Orpheum Road. The S&WB has estimated the cost of this project to be approximately $16,000,000. Figure 37 shows typical sections of the proposed S&WB project.

ANALYSIS

The hydraulic analysis of the Outfall Canal was made using the HEC-2 program previously described. The existing canal was coded using cross sectional data furnished by the S&WB and spot-checked by this Consultant's Surveyor. Sections were coded every 50 feet in the Bucktown area and at 500 foot intervals upstream with additional sections taken at bridges.

Calibration

In order to model the existing channel and to verify the computational method, the channel was calibrated using the following procedure. Staff gages were set at six different locations throughout the canal. During a storm event, the canal water surface was read simultaneously while Pumping Station No. 6 was operating. The flow being pumped during the reading was determined from the station records. The same flow was then routed by the Model, and the "n" values adjusted until the observed surface profile was approximated by the model. An "n" value of 0.04 for the canal section gave the following results:

TABLE 10
OUTFALL CANAL CALIBRATION ELEVATIONS

Gage Stationing	Observed Elevation	HEC-2 Predicted Elev.
Lake	22.39 C.D.	22.39 C.D.
116+00	22.43	22.43
100+00	22.45	22.46
35+00	22.61	22.61
17+40	22.64	22.67
1+42	22.78	22.73

Criteria

The following parameters for the Outfall Canal were agreed upon by the Study Technical Committee.

Design Flow	10,000 cfs
Existing Maximum Flow	6,650 cfs
High Lake Elevation	24.7 Ft. C.D.
Nominal Lake Elevation	22.5 Ft. C.D.
Manning's "n" (Existing)	0.04
Manning's "n" (Improved)	0.024
Desirable Average Velocity (Earthen Channel)	3.0 fps
Maximum Average Velocity (Earthen Channel)	3.5 fps
Maximum Average Velocity (Lined Channel)	10.0 fps
Maximum Water Surface @ Station 6	26.5 Ft. C.D.

The design flow was taken from the generated hydrograph for improved conditions, and the design storm at Pumping Station No. 6. This hydrograph, shown in Figure 35, was generated from the outfall hydrographs predicted by the SWWM model for the following drainage basins: Pumping Station No. 1 (Orleans), Oleander, Lowerline, and Jefferson (Hoey, Metairie Rd., and Northline). The individual hydrographs were added after being lagged by their travel times through the respective canals.

The existing and design flows were routed for each lake elevation using the appropriate "n" values, for the existing channel, the improvements as proposed by the S&WB and several alternative channel configurations. The nominal lake elevation of 22.5 C.D. was used only to determine maximum velocities under low water conditions. Table 11 shows in tabular form the results of the channel routings.

TABLE 11
OUTFALL CANAL HEAD LOSSES

Description of Alternative	FLOW (cfs)	LAKE PONTCHARTRAIN EL. (Ft.-C.D.)	HEAD LOSSES (FEET/REACH) Bucktown	Hammond Hwy Bridge	Veterans Bridges	I-10 Bridges	Southern RR Bridge	Remaining Channel Losses	Pumping Station 6 El. (Ft.-C.D.)
1. Existing Outfall Canal	6,650	22.5	1.7	0.2	0.5	0.3	0.5	3.5	29.2
	6,650	24.7	1.1	0.2	0.5	0.4	0.4	2.6	29.9
	10,000	22.5	3.6	0.4	1.1	1.3	0.7	4.0	33.6
	10,000	24.7	2.5	0.4	1.2	1.3	0.7	3.3	34.1
2. S&WB Improvements	10,000	24.7	0.2	0.1	0.6	0.4	0.3	1.1	27.4
3. S&WB Improvements with Bridges Omitted	10,000	24.7	0.2					1.51	26.4
4. Base Project — S&WB Improvements with Level Flow Line & RR Bridge Raised	10,000	24.7	0.1	—	0.3	0.1		1.4	26.6
5. Alternate 1 — Constricted Channel in Bucktown	10,000	22.5	0.6	0.7	0.1	0.1		1.1	25.1
	10,000	24.7	0.4	0.5	0.3	0.1		0.7	26.7

FINDINGS

Existing Channel

The existing maximum discharge from Station 6 and the design flows routed through the existing channel give water surface elevations at Station 6 of 29.9 C.D. and 34.1 C.D., respectively, using the design high water elevation in the lake of 24.7 C.D. In both instances the water surface partially submerges the existing girders of Veterans Highway, I-10, and Southern Railroad bridges. The 10,000 cfs design flow overflows the bank at the Veterans Highway bridge, while the water surface elevation at Station 6 for the existing 6650 cfs flow is too high to maintain proper gravity flows in the Hoey's and Lowerline Basins. Channel velocities upstream of the Hammond Highway bridge are approximately 3.5 fps for 6650 cfs and 4.2 fps for 10,000 cfs. In the Bucktown area velocities range from 5.4 fps at 6650 cfs to 9.0 fps for 10,000 cfs.

The existing channel does not meet the established study criteria water surface elevations at Station No. 6 or the average channel velocities, for either the maximum existing flow from Station 6 or the design 10,000 cfs flow.

Proposed S&WB Improvements

The design flow routed through the S&WB proposed improvements indicated a water surface elevation of 27.4 C.D. at Station 6. This is 0.9 feet above the desired elevation. Velocities range from approximately 4.5 fps for the nominal low lake elevation to 3.8 fps for the high lake elevation conditions. Velocities increase to approximately 4.5 fps through the bridge opening constrictions.

The proposed S&WB section does not meet the established criteria water surface elevation at Station No. 6 for the design 10,000 cfs flow. The average channel velocity for the 10,000 cfs at low water and under bridges is also somewhat above the maximum velocity requirements.

Alternatives

Several alternatives that would eliminate the excess 0.9 feet of headloss in the proposed S&WB project and reduce the water surface profile elevation at Station 6 to the required 26.5 C.D. were studied. These were reduced to the following two alternates: the Base Project (described below), and widening the proposed channel, except at bridges, throughout its entire length. This second option was determined to be far more costly than the Base Project option as discussed below.

Base Project

The Base Project uses the same channel section proposed by the S&WB. The project, however, would be expanded to include: extending the new dredged flowline under all bridges (eliminating the constrictions); driving a sheetpile bulkhead at the headwall of each bridge (to prevent seepage); rebuilding the Southern Railroad bridge, and continuing the dredged channel to Station No. 6. The design flow routed through the Base Project gives a water surface elevation of 26.6 C.D. at Station No. 6, which was considered acceptable. Velocities are the same as in the proposed S&WB project.

The above stated improvements require the demolition of a portion of the Bucktown area. In order to maintain the integrity of Bucktown, several additional alternatives were considered. Of these, Alternate No. 1 was selected as the most reasonable in terms of cost and feasibility.

Alternate No. 1

The channel bottom through the Bucktown area, between the Hammond Highway Bridge and Lake Pontchartrain, would be reduced to approximately 10 ft. wide. The Bulkhead in the Orleans bank would remain as proposed in the Base Project, however, the bulkhead on the Jefferson bank would be moved approximately 40 feet east of the Base Project location. The existing Bucktown structures would be moved in order to make room for construction of the bulkhead; however, they can be relocated at approximately the same location after construction.

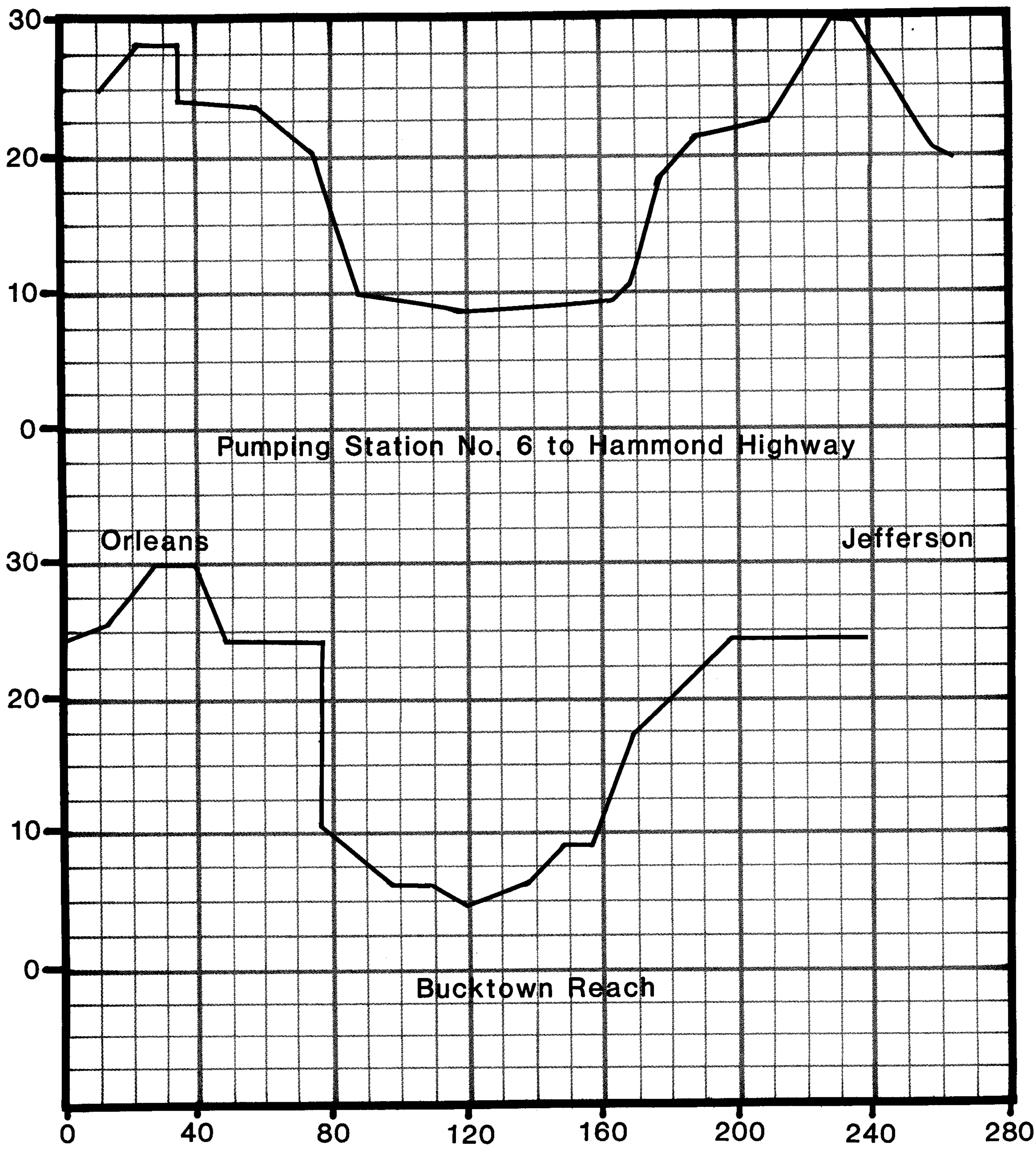
This constriction of the channel would cause two major problems. First, velocities would increase above maximum values for an earthen channel; and second, additional head losses would occur which would raise the water surface profile above the allowable at Station 6. To overcome these problems, considerable costs would be involved. First, probable scouring, which would be caused by the higher velocities would need to be eliminated by lining the channel with a concrete revetment; and second, the additional headlosses must be compensated for by widening the proposed remaining channel bottom by approximately 25 feet.

The design flow routed through this alternate gives a water surface elevation of 26.7 C.D. at Station 6. Velocities in the constricted channel will be approximately 6.0 fps, and, in the enlarged channel 3.0 fps. Typical sections of this alternative are shown on Figure 38. These two items will add approximately $3,000,000 to the project cost.

Alternate No. 2

A second alternative that was closely studied would leave the existing Bucktown structures intact. This alternative was similar in every respect to Alternate No. 1 described above, except that on the Jefferson bank in the Bucktown area no dredging would occur, no bulkhead would be constructed and existing structures would not be moved. The same additional costs due to lining the channel and the additional dredging would be incurred.

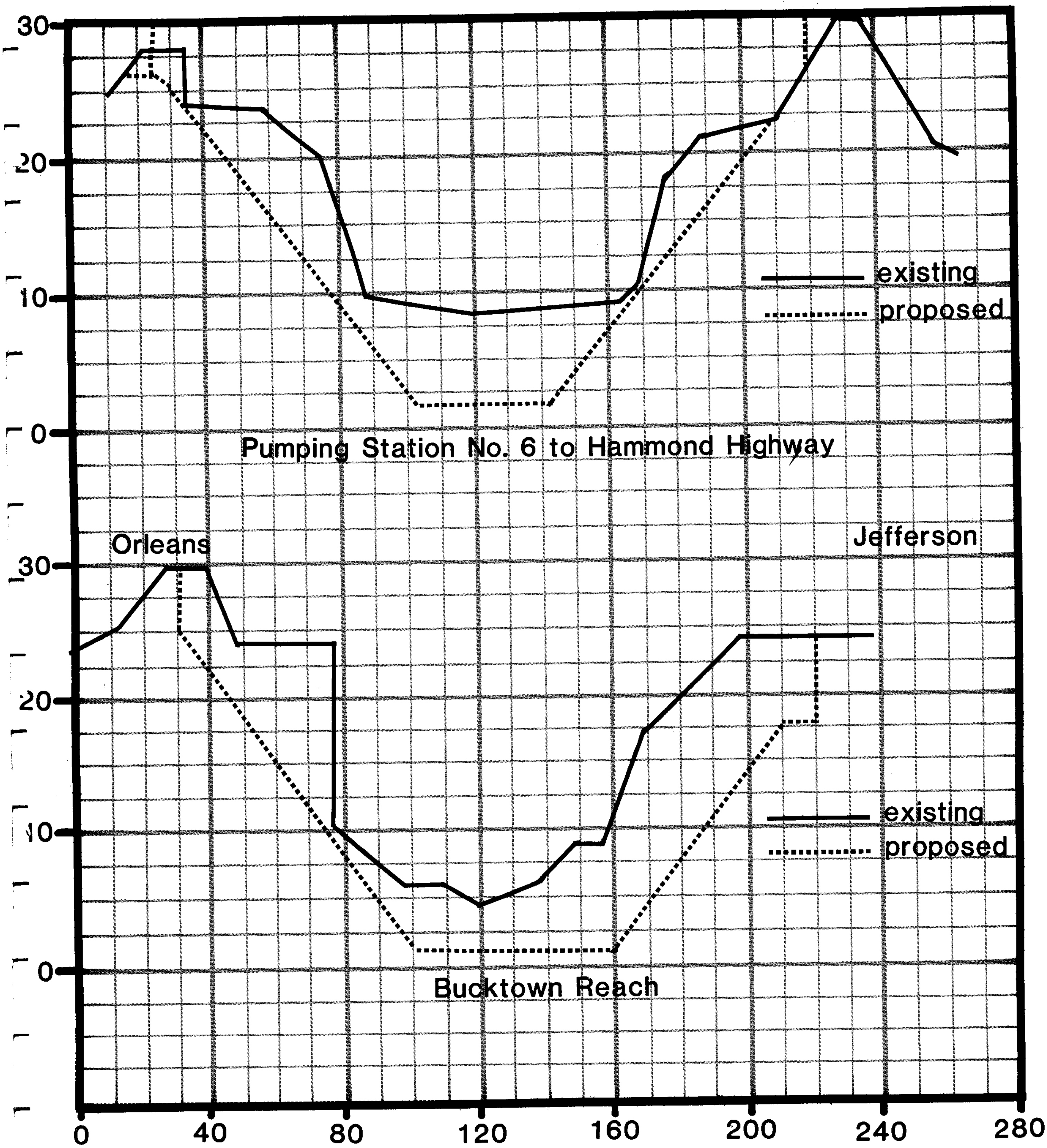
It was determined, however, that since the existing structures would be unprotected, the very high velocities which would occur would endanger the property, health and safety of the residents, and would in time wash away many of the structures. For these reasons this alternative is not recommended.



Typical Cross Sections for Outfall Canal
Existing Conditions

Seventeenth Street Canal Drainage Basin Study

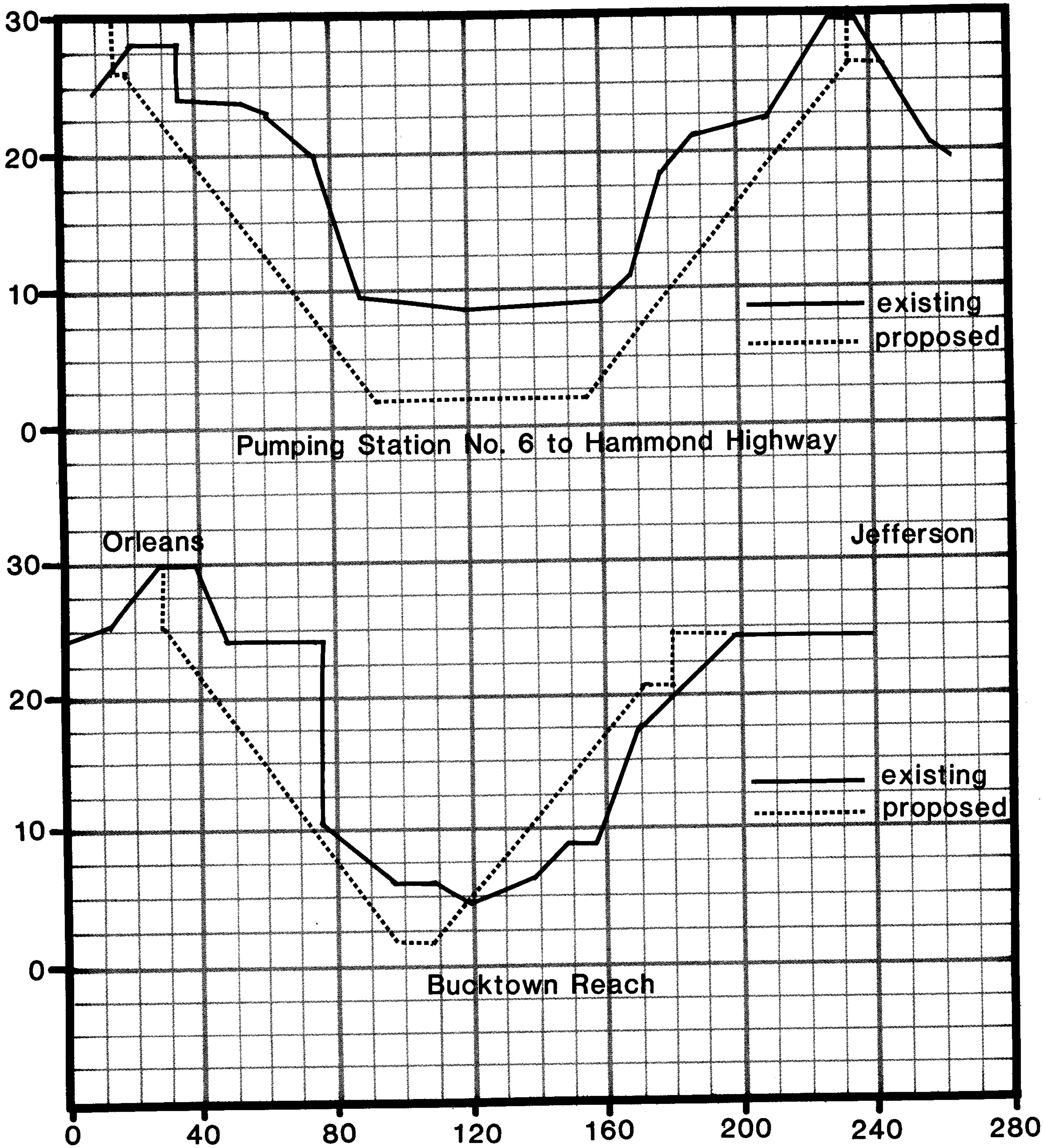
Figure 36



Typical Cross Sections for Outfall Canal
Proposed S & W B Improvements

Seventeenth Street Canal Drainage Basin Study

Figure 37



Typical Cross Sections for Outfall Canal
Alternate No. 1 Improvements

Seventeenth Street Canal Drainage Basin Study

Figure 38

Hydraulic Comparisons

A comparison of the results of this study with the studies made previously is shown on Table 12.

TABLE 12
COMPARISON OF RESULTS WITH PREVIOUS STUDIES

Consultant	Lake	Water Surface Elevation (Cairo Datum) at Pumping Station No. 6 — Existing Condition 6,650 cfs	Existing Condition 10,000 cfs	S&WB Project 6,650 cfs	S&WB Project 10,000 cfs
S&WB Design (1979)	22.5	27.0			26.0
Jefferson Parish (1981)	22.5	26.8	30.0	26.3	29.1
S&WB Review (1981)	22.5	27.7		27.4	
Linfield, Hunter & Gibbons (n=0.024)	22.5	26.6	30.4	24.3	25.9
Linfield, Hunter & Gibbons (n=0.04)	22.5	29.2	33.6		
S&WB Design (1979)	24.7				27.2
Jefferson Parish (1981)	24.7	27.7	30.5	27.3	29.7
S&WB Review (1981)	24.7				
Linfield, Hunter & Gibbons (n=0.024)	24.7	27.8	31.3	25.8	27.4
Linfield, Hunter & Gibbons (n=0.04)	24.7	29.9	34.1		

There appears to be good correlation between the various calculations of the different consultants. The most significant difference occurs in the existing conditions when this Consultant used an "n" value of 0.04. This gave predictably higher results than those of the other Consultants; however, since the calibration done in this study indicated that an "n" value of 0.04 matched the existing condition of the channel, this value was used. Of greater importance is the similarity of values shown in this table which tend to verify the mathematic accuracy of the backwater calculations obtained in the HEC 2 program used in this study.

Profiles

The water surface profiles for several of the conditions discussed above are shown on Figure 39. As can be seen, the existing channel is not large enough to transport either the existing or design peak flows at the selected design high lake elevation. It can also be seen that the proposed Base Project and Alternate No. 1 will lower water surface elevations to acceptable levels. It is noted here that in each of these cases the bottom of the Veterans Highway bridge is partially submerged. The proposed S&WB project water surface profile, not shown, but approximately one foot higher, will also partially submerge the Veterans Highway bridge and will probably also come in contact with the I-10 and Southern Railroad bridges.

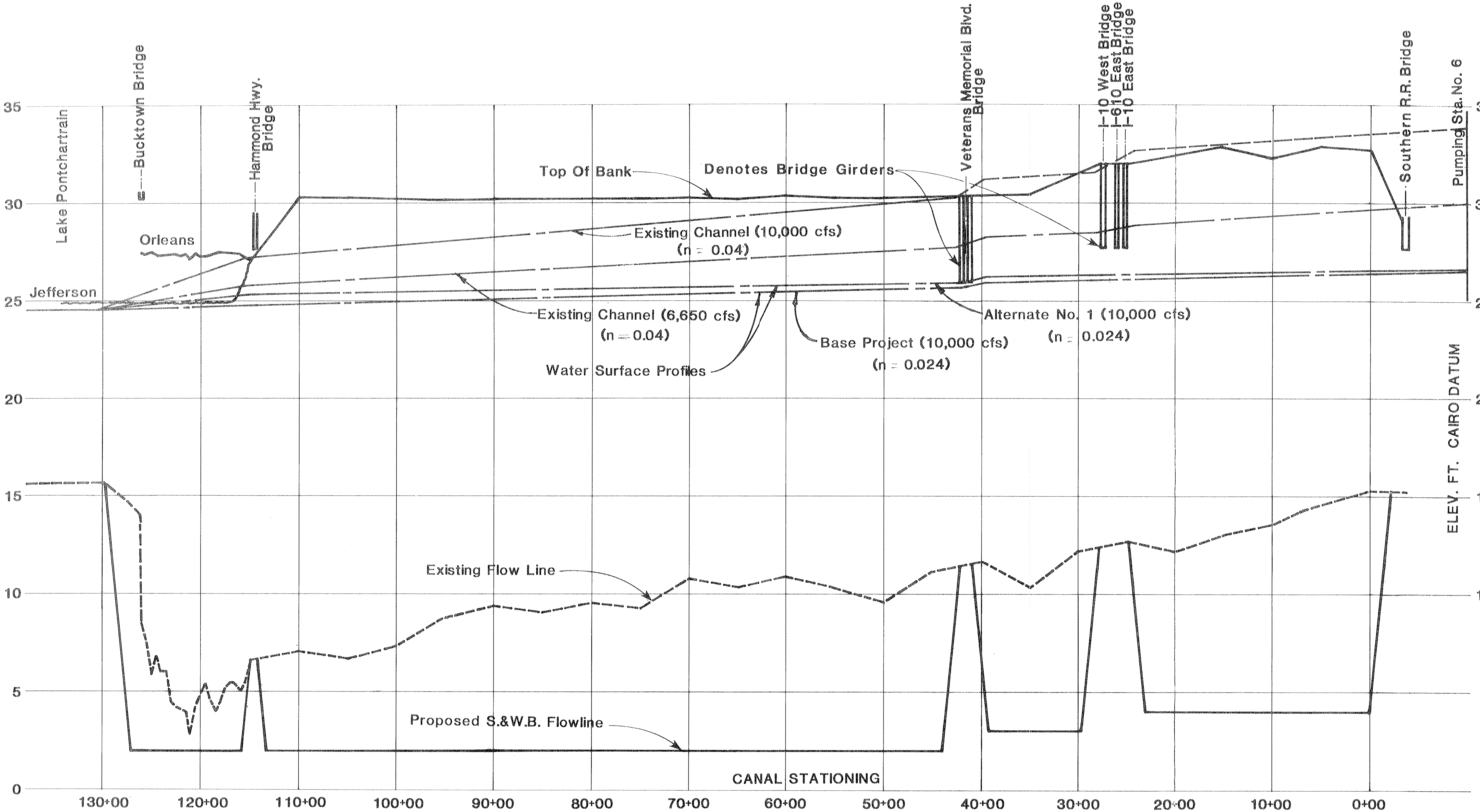
Several additional alternatives that would increase the capacity of the existing Outfall Canal were investigated. Each had severe problems and were not considered further. Among them were:

a Construct Concrete Box Culvert. This was discarded due to cost and the difficulties in construction because the lake waters could not be dammed.

b Widen channel and increase all bridge lengths. The cost of adding width to the proposed channel becomes extremely expensive. Adding length to the bridges is both costly and disruptive to traffic.

c Driving vertical sheetpile walls in the Bucktown area. These walls would have to be approximately 25 feet high. The feasibility of this is questionable due to poor soil conditions and maintenance would be extremely costly.

d Shifting the channel eastward. In addition to extraordinary land acquisition costs due to the existing development, legal problems would probably cause long delays in construction.

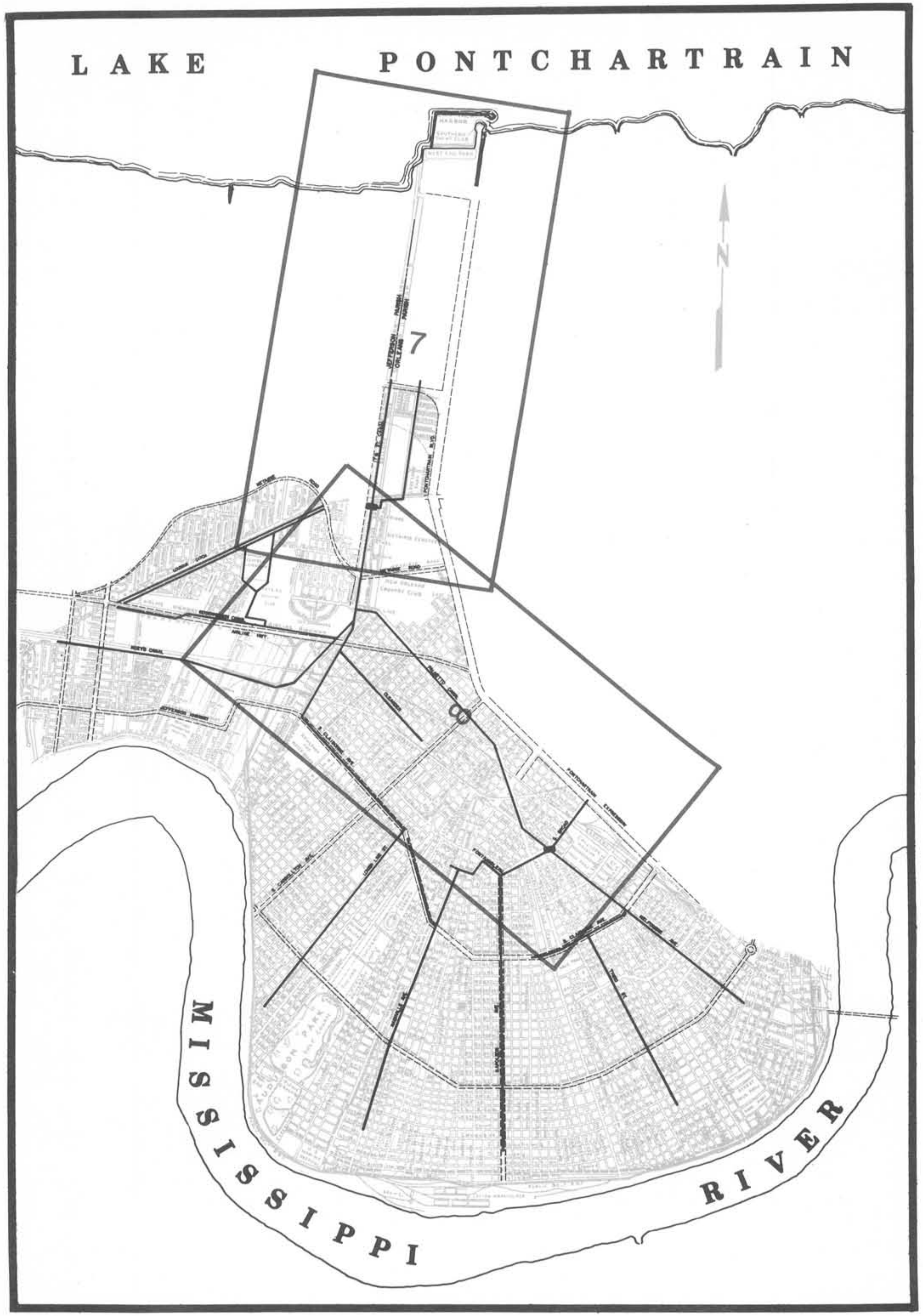


Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Outfall Canal
Profiles

Figure 39



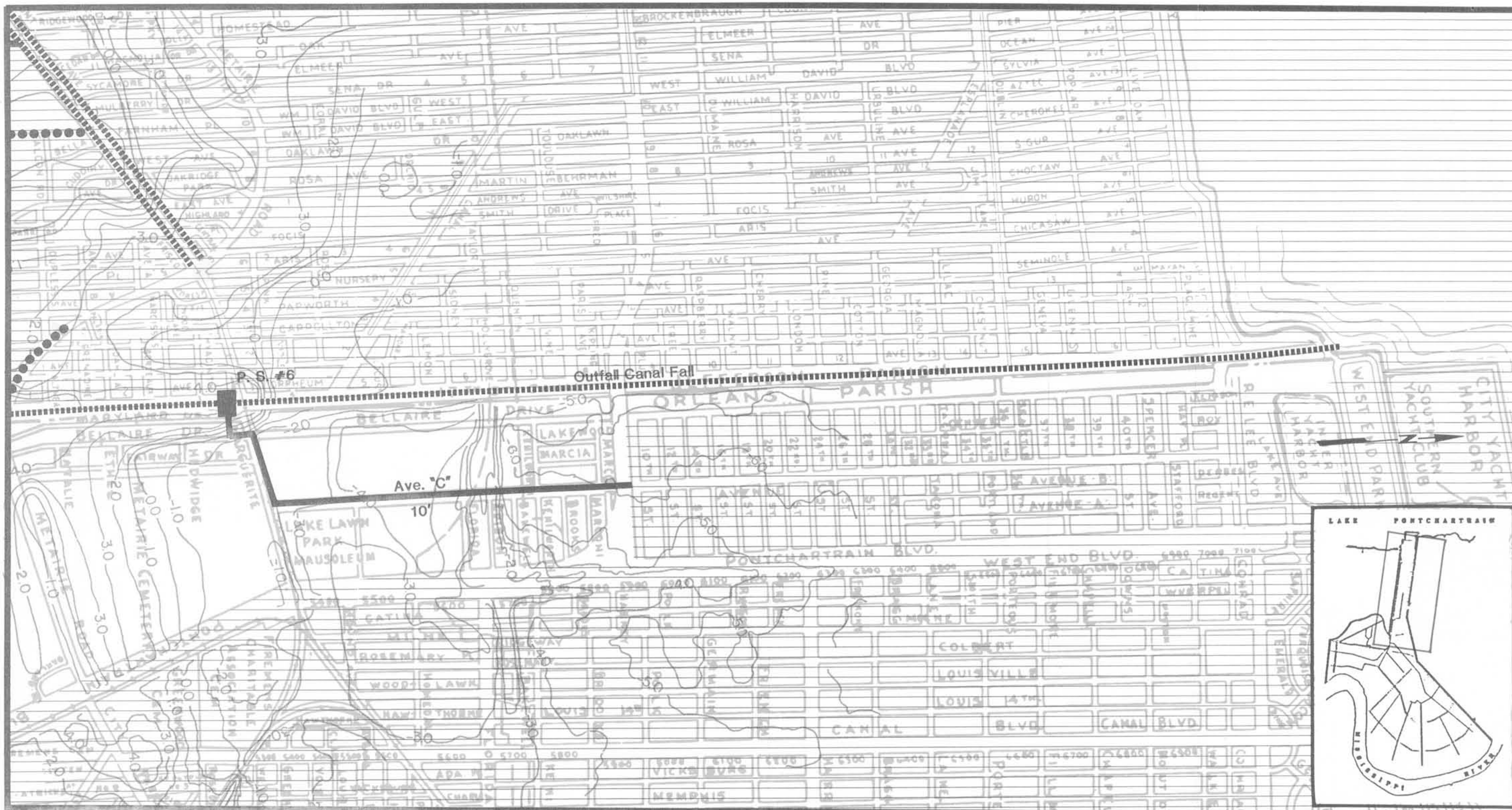
DRAINAGE SYSTEM	PLATE NO.
P.S. No. 6 To Lake (Existing)	7
P.S. No. 6 To Lake (Proposed)	7
P.S. No. 1 To P.S. No. 6 (Existing)	8
P.S. No. 1 To P.S. No. 6 (Proposed)	8

Open Canal Drainage System

Plate Index

Seventeenth Street Canal Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Open Canal System

Pumping Station No. 6 To Lake

Plate No. 7

Existing Facilities

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬ ▬	▬ ▬ ▬ ▬	Open Conc. Canal
┅┅┅┅┅┅	┅┅┅┅┅┅	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

▭ Denotes Water Surface Elev.

0′ 825′ 1,650′ 2,475′ 3,300′

Approximate Scale

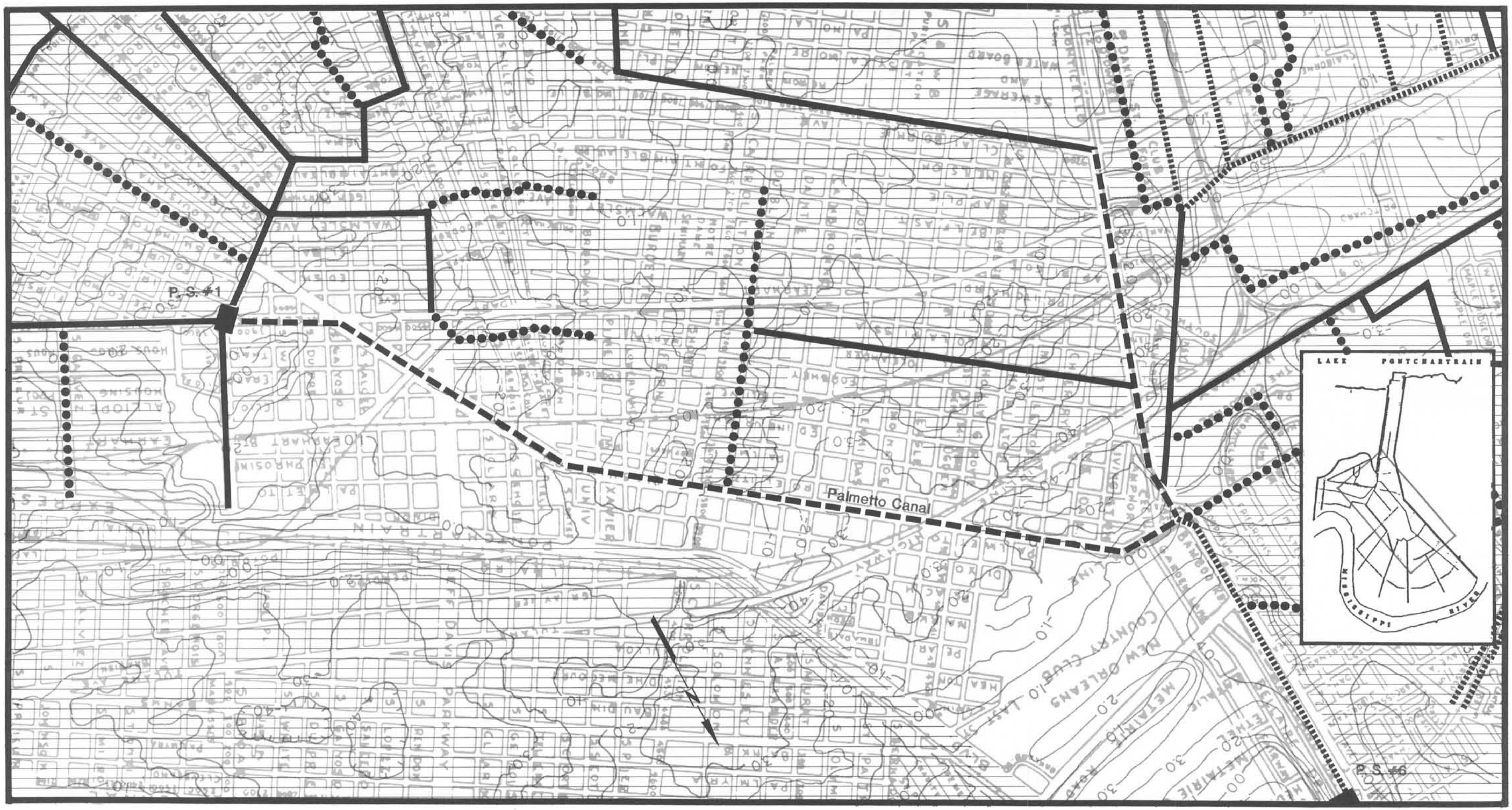
Note: All Elevations Are In Mean Sea Level

Open Canal System

Pumping Station No. 6 To Lake

Plate No. 7

Proposed Improvement



Seventeenth Street Canal Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Box Culvert
Open Conc. Canal
Open Canal / Ditch
Pipe

0′ 825′ 1,650′ 2,475′ 3,300′

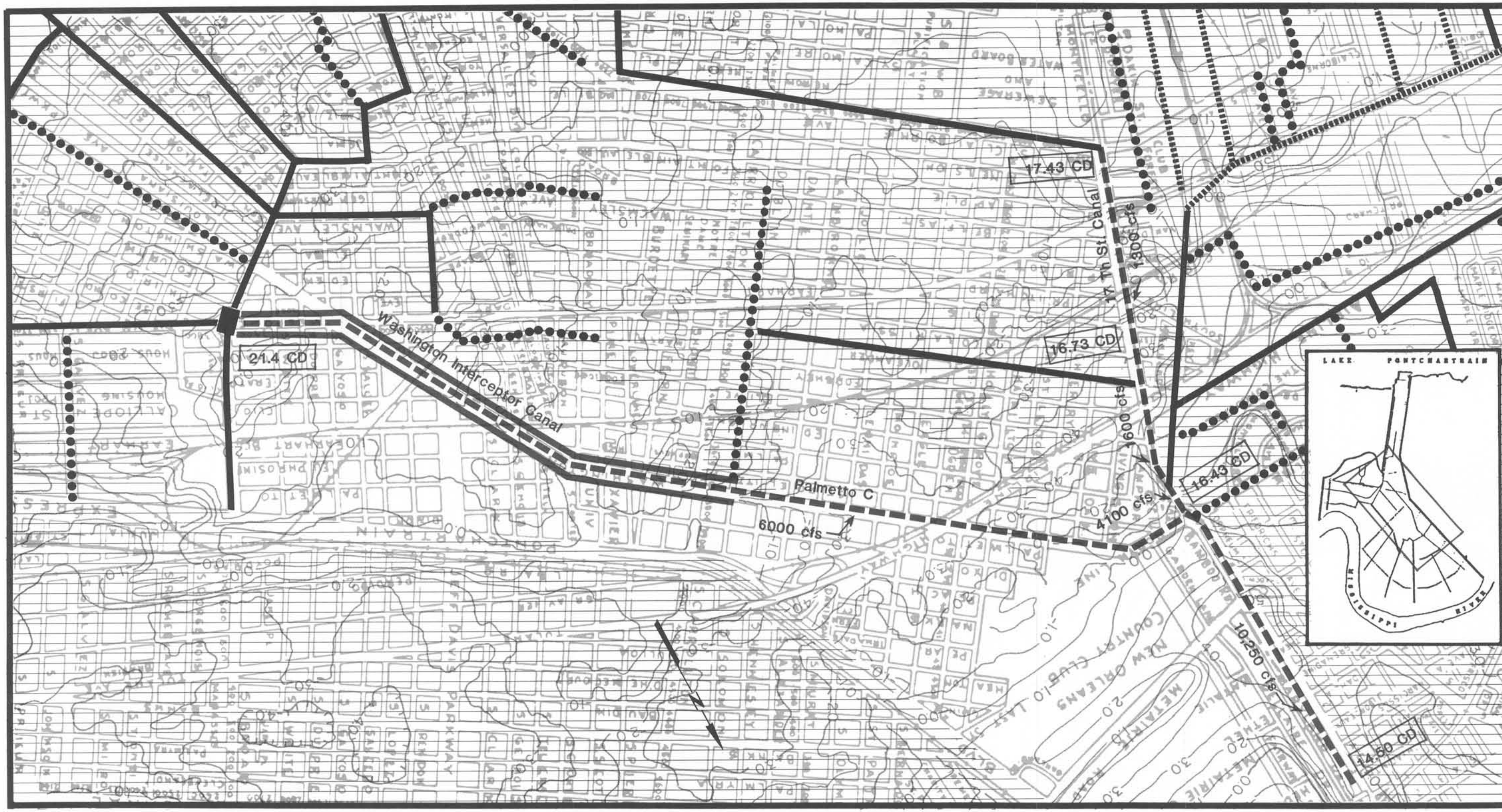
Approximate Scale

Open Canal System

P.S. No.1 To P.S. No.6

Plate No. 8

Existing Facilities



Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Existing	Proposed	
▬▬▬	▬▬▬	Box Culvert
▬ ▬ ▬	▬ ▬ ▬	Open Conc. Canal
┇┇┇┇┇┇	┇┇┇┇┇┇	Open Canal / Ditch
●●●●●●●	●●●●●●●	Pipe

▭ Denotes Water Surface Elev.

0' 825' 1,650' 2,475' 3,300'

Approximate Scale

Note: All Elevations Are In Mean Sea Level

Open Canal System

P.S. No.1 To P.S. No.6

Plate No. 8

Proposed Improvement

SECTION 4

APPENDIX

A DESIGN HYETOGRAPH SYNTHESIS

B STANDARD STREETS

C PUMP CURVES

APPENDIX A
DESIGN HYETOGRAPH SYNTHESIS

The design hyetograph shown on Figure 9 was generated from rainfall data submitted by the S&WB and agreed upon by the Technical Committee. From the design rainfall intensity/duration curve, shown on Figure 8, the following values shown on Column 2 were read:

(1) Time (Min.)	*(2)* Average Rainfall Intensity (In./Hr.)	*(3)* Ordered Rainfall Intensity (In./Hr.)
15	6.28	6.28
30	4.96	3.65
45	4.10	2.38
60	3.49	1.68
75	3.05	1.25
•	•	•
•	•	•
•	•	•
300	1.04	0.13

These values were then ordered for each 15 minute period as first maximum, second maximum, third maximum, etc. for the full five hour storm duration. These values are shown on Column 3 and were obtained as follows:

From the data, the first maximum for a 15 minute period is 6.28 in./hr. To get the second 15 minute maximum, we know that the 30 minute average (4.96) must contain the first and second maximums. The second maximum is obtained by solving the weighted average for X:

$$\frac{6.28(15) + X(15)}{30} = 4.96 \text{ (in./hr.)}$$

X = 3.65 (in./hr.) = second maximum.

Likewise, the 45 minute average must contain the first, second and third maximums. Solving the weighted average, we find:

$$\frac{4.96(30) + Y(15)}{45} = 4.10 \text{ (in./hr.)}$$

Y = 2.38 (in./hr.) = third maximum.

After all of the ordered maximums are determined using the weighted average technique, they are plotted so that the first maximum is placed at the start of the 3rd quarter of the storm. Subsequent maximums are arranged in decreasing order on alternating sides of the first maximum peak.

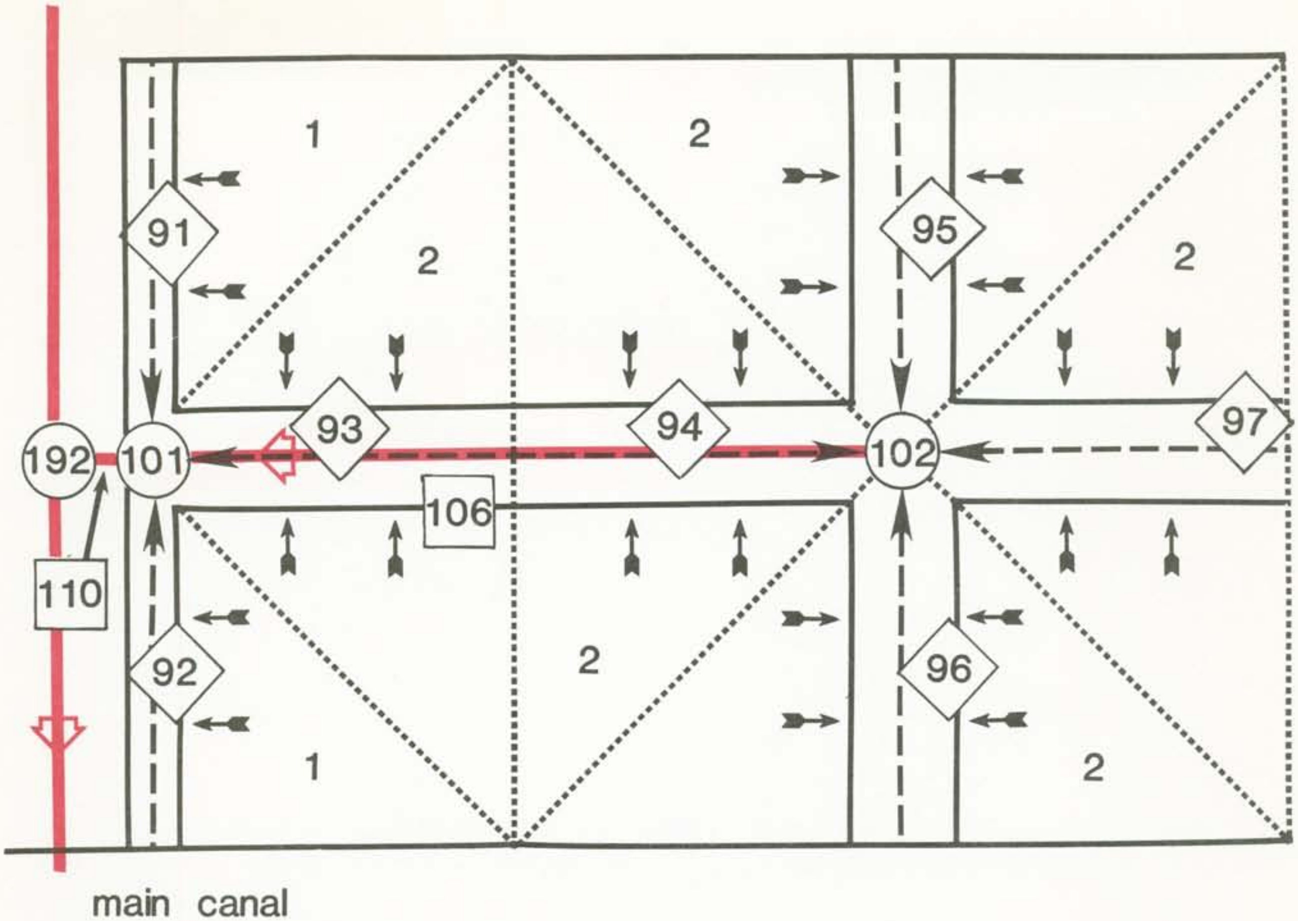
The hyetograph (Figure 9) is thus generated, and as a check, the area under the hyetograph must equal the five hour rainfall intensity average times the storm duration. A hyetograph of this type produces a storm pattern that will generate the maximum runoff from the given rainfall intensity/ duration curve.

APPENDIX B

SWMM NUMBERING SYSTEMS

The numbers 1-199 were used to define Orleans Parish Standard Streets. Figure B1 shows the various street elements and their numbers for Standard Street No. 192. Numbers 600-799 were used to define Jefferson Parish Standard Streets. Figures B2 through B13 show the design of the Standard Streets used in Orleans and Jefferson Parishes.

Numbers 200-399 were used to number sewer elements (manholes and conduits) in the various collection systems in the study. The SWMM Transport Block limited the number of different sewer elements to 159 per Transport Block. For this reason, the entire collection system had to be broken down into smaller parts described with one Transport Block each. Figures B14 through B40 show schematic diagrams of each of these parts. The numbering system for each part is as follows: starting with 301, each manhole was numbered sequentially. Each conduit bears the same number as the manhole it follows, except that the conduit number starts with a 2, rather than a 3. In these figures, manholes are represented graphically with circles, the larger of which indicates manholes at which a hydrograph was printed out. Numbers 901-999 represent system hydrographs. Selected hydrographs are shown elsewhere in this report.



Standard Street Numbering System

Figure B1

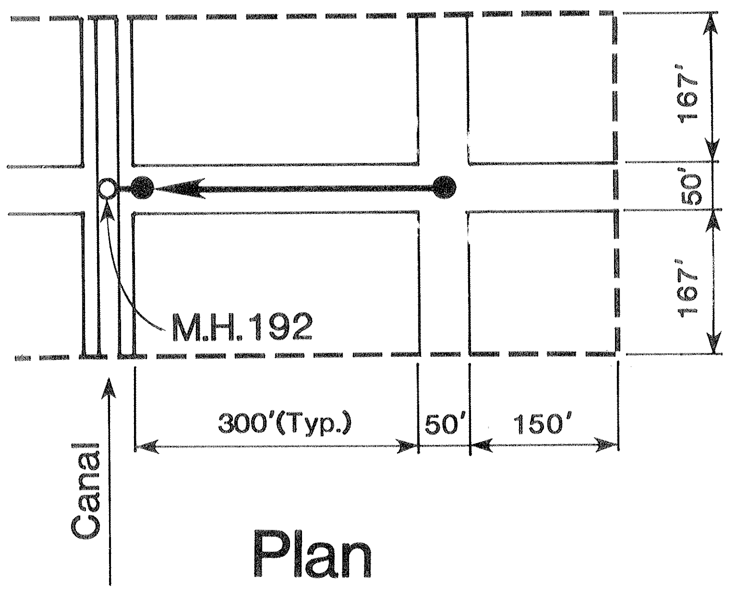
Subcatchments 1 overland flow

Gutters 94 gutter flow

Inlets 102

Pipes 106

Manholes 192 code number for standard street

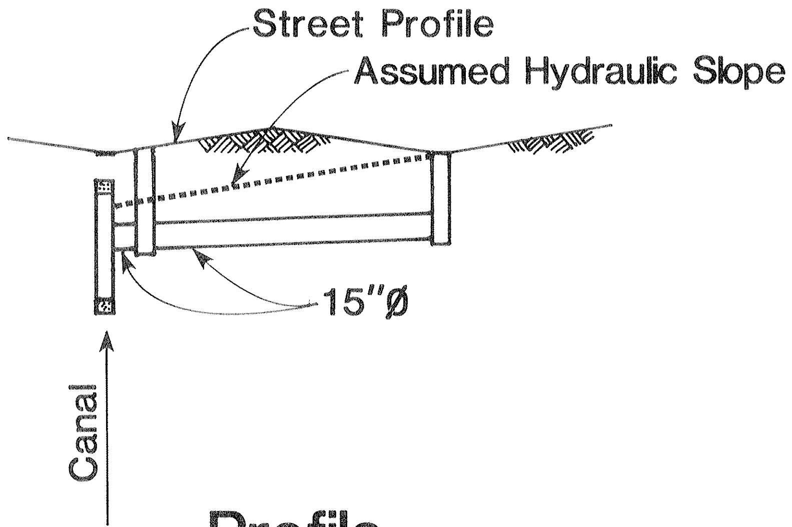


Plan

Design Data

Contributory Area : 4.62 Acres

Hydraulic Slope : 0.0114 ‰



Profile

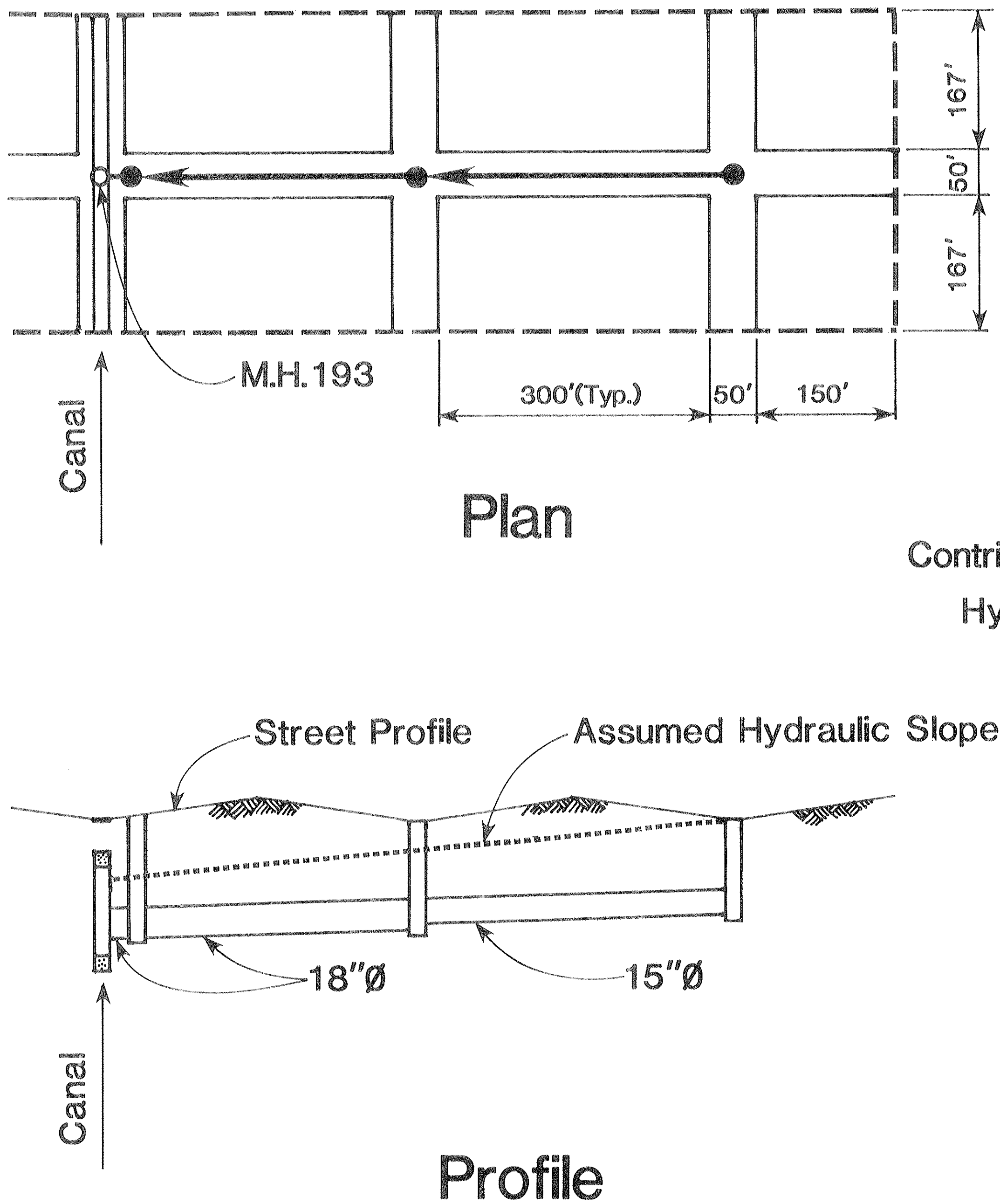
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No. 192

Orleans Parish

Figure B2



Design Data

Contributory Area : 7.70 Acres

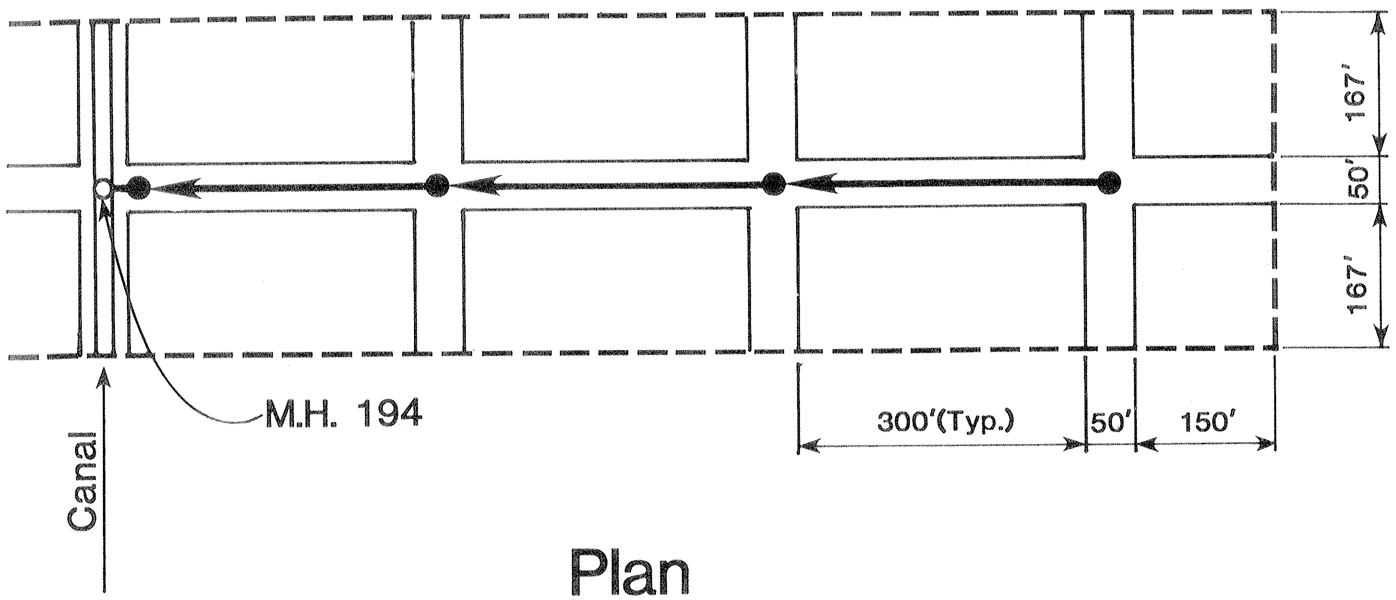
Hydraulic Slope : 0.0057 %

Seventeenth Street Canal
Drainage Basin Study

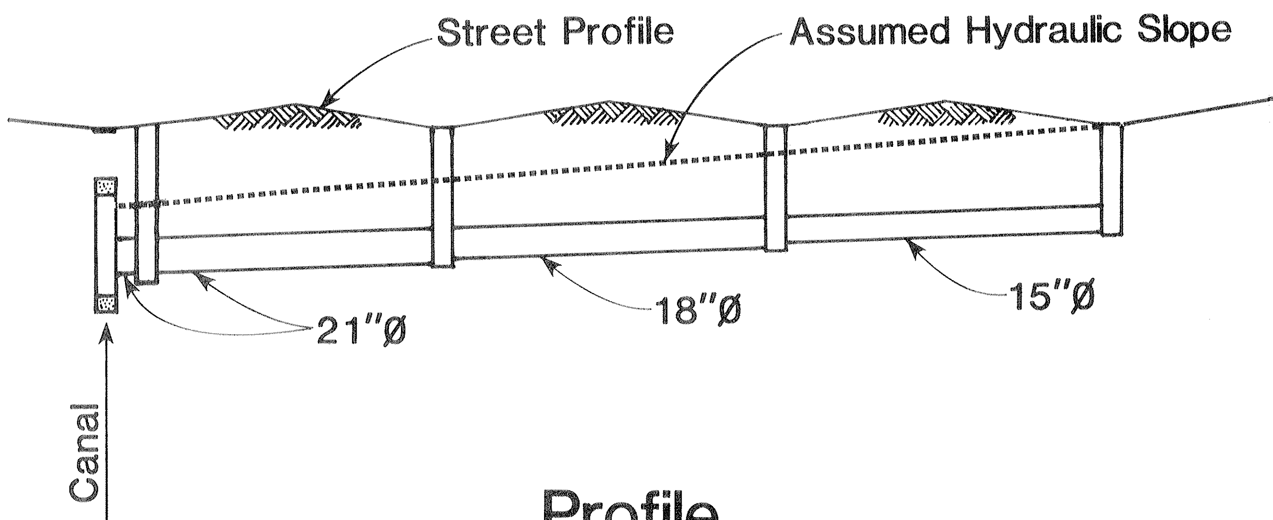
Linfield , Hunter & Gibbons , Inc.

Standard Street No. 193
Orleans Parish

Figure B3



Plan



Profile

Design Data

Contributory Area : 10.78 Acres

Hydraulic Slope : 0.0038 ′/′

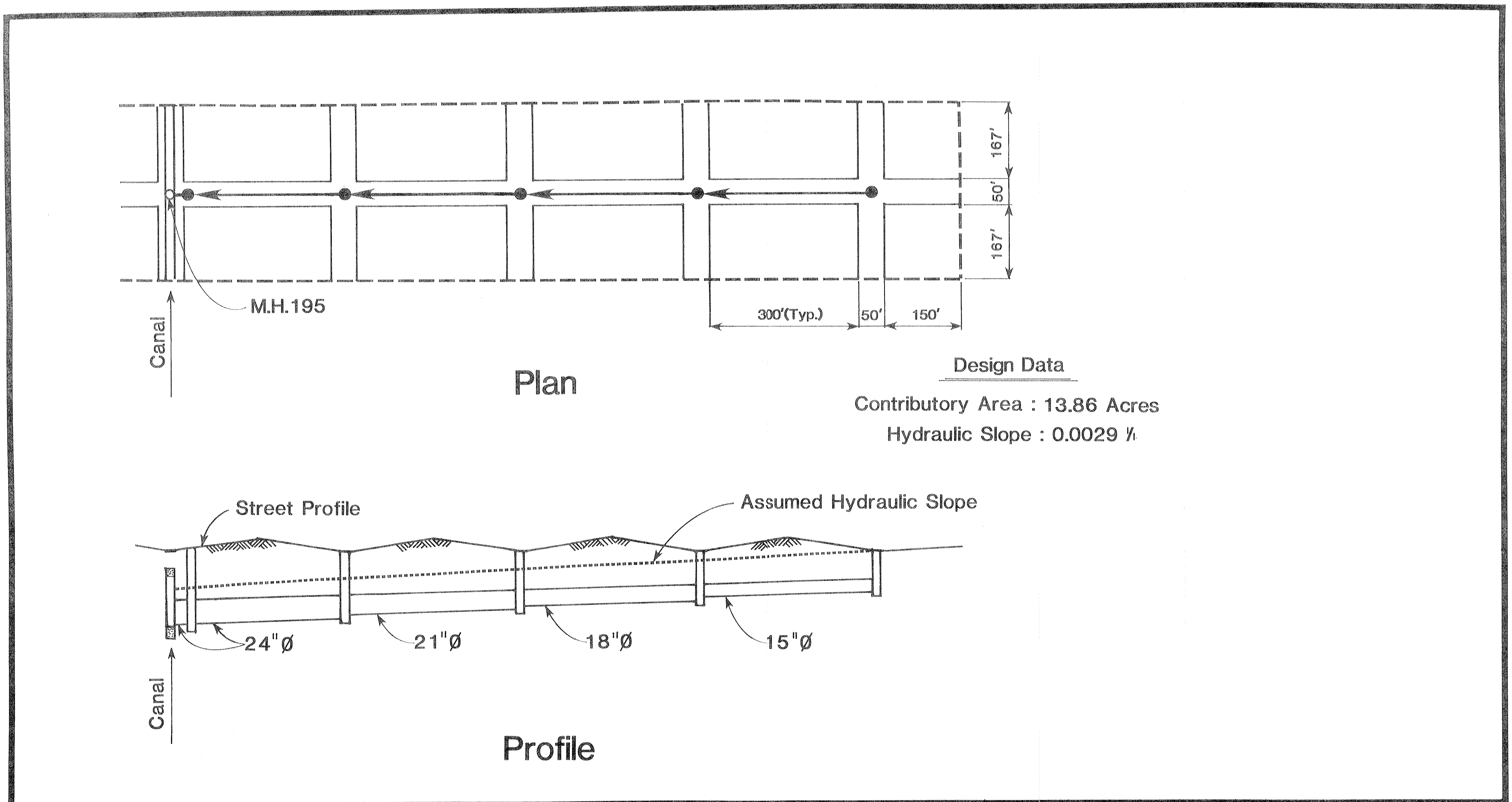
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No. 194

Orleans Parish

Figure B4

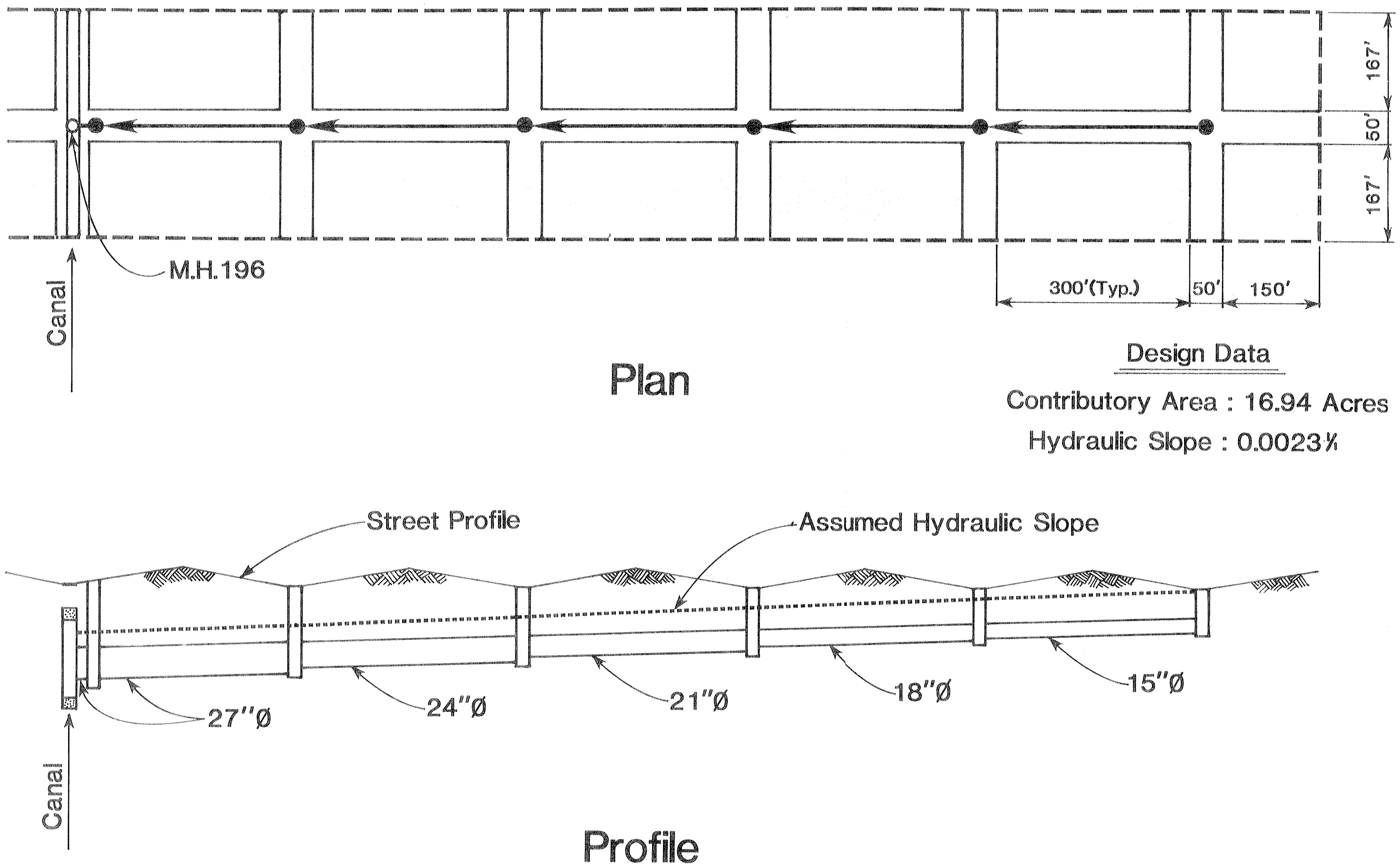


Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No. 195
Orleans Parish

Figure B5



Design Data

Contributory Area : 16.94 Acres

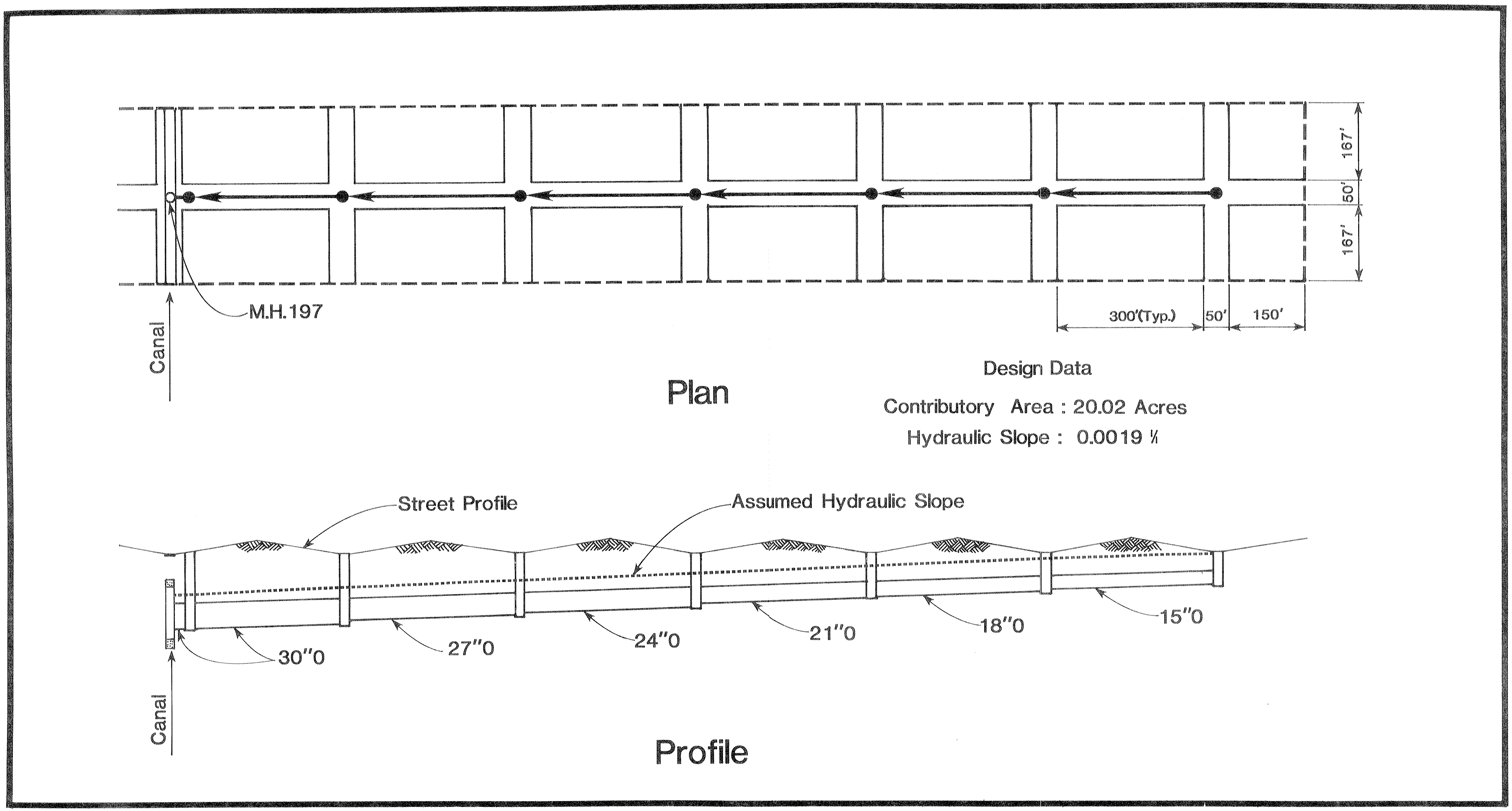
Hydraulic Slope : 0.0023%

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No. 196
Orleans Parish

Figure B6

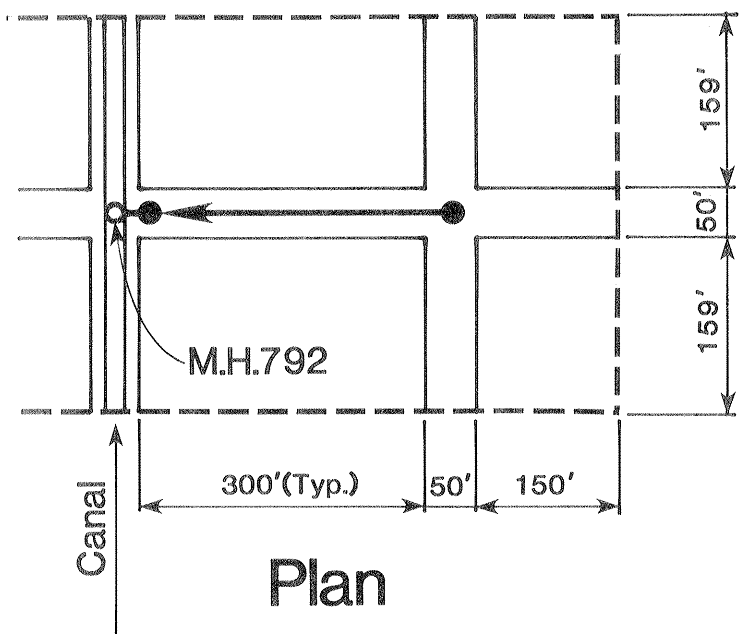


Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No. 197
Orleans Parish

Figure B7

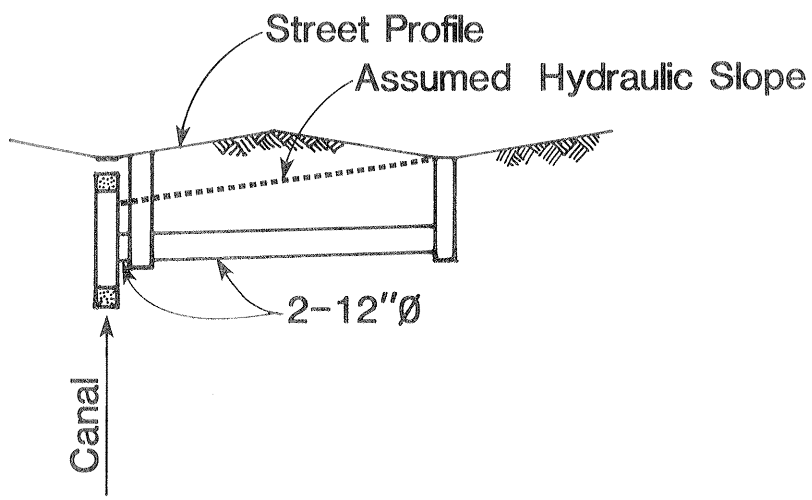


Plan

Design Data

Contributory Area : 4.44 Acres

Hydraulic Slope : 0.0086 '/'



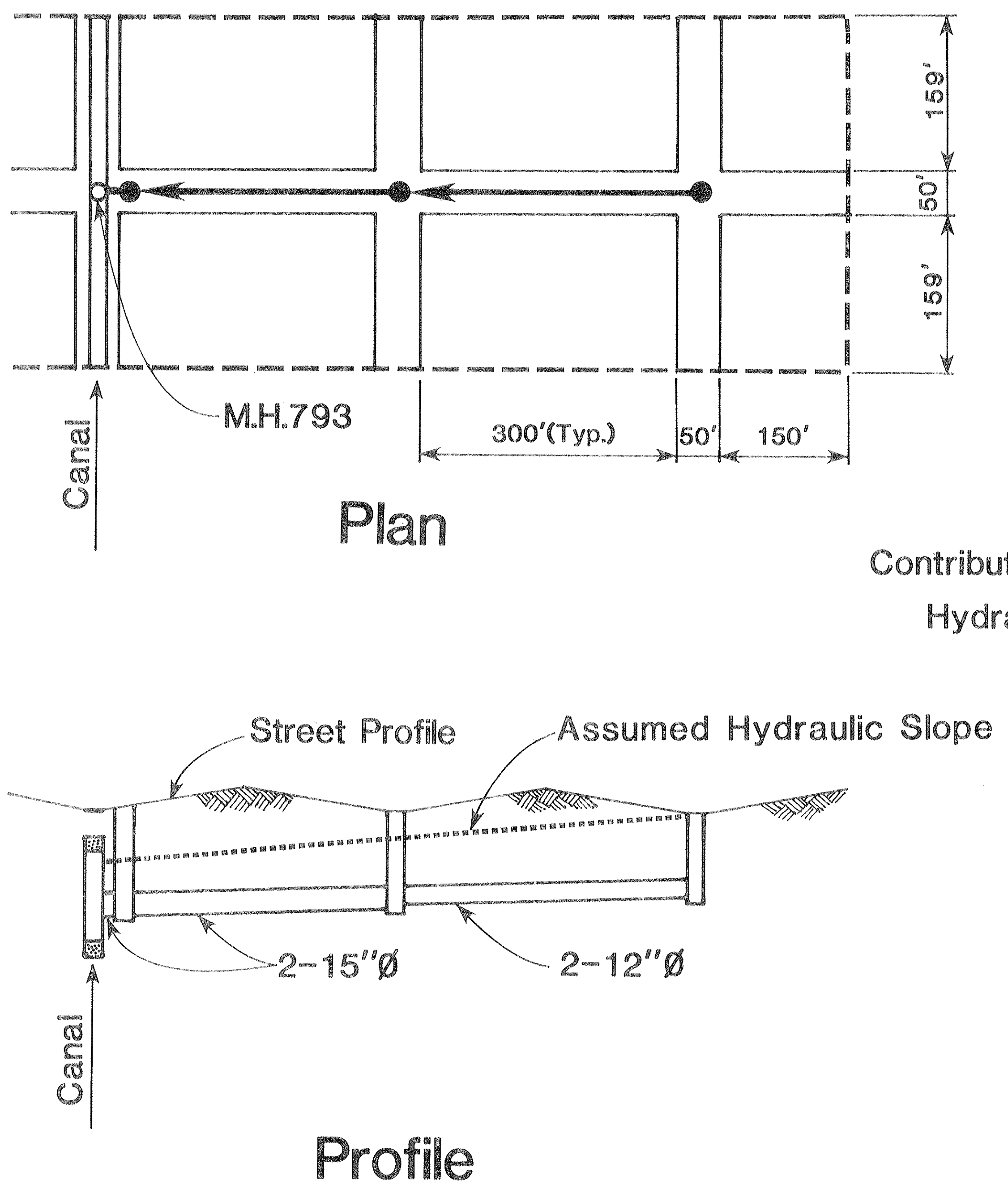
Profile

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No.792
Jefferson Parish

Figure B8



Design Data

Contributory Area : 7.40 Acres

Hydraulic Slope : 0.0043 ft/ft

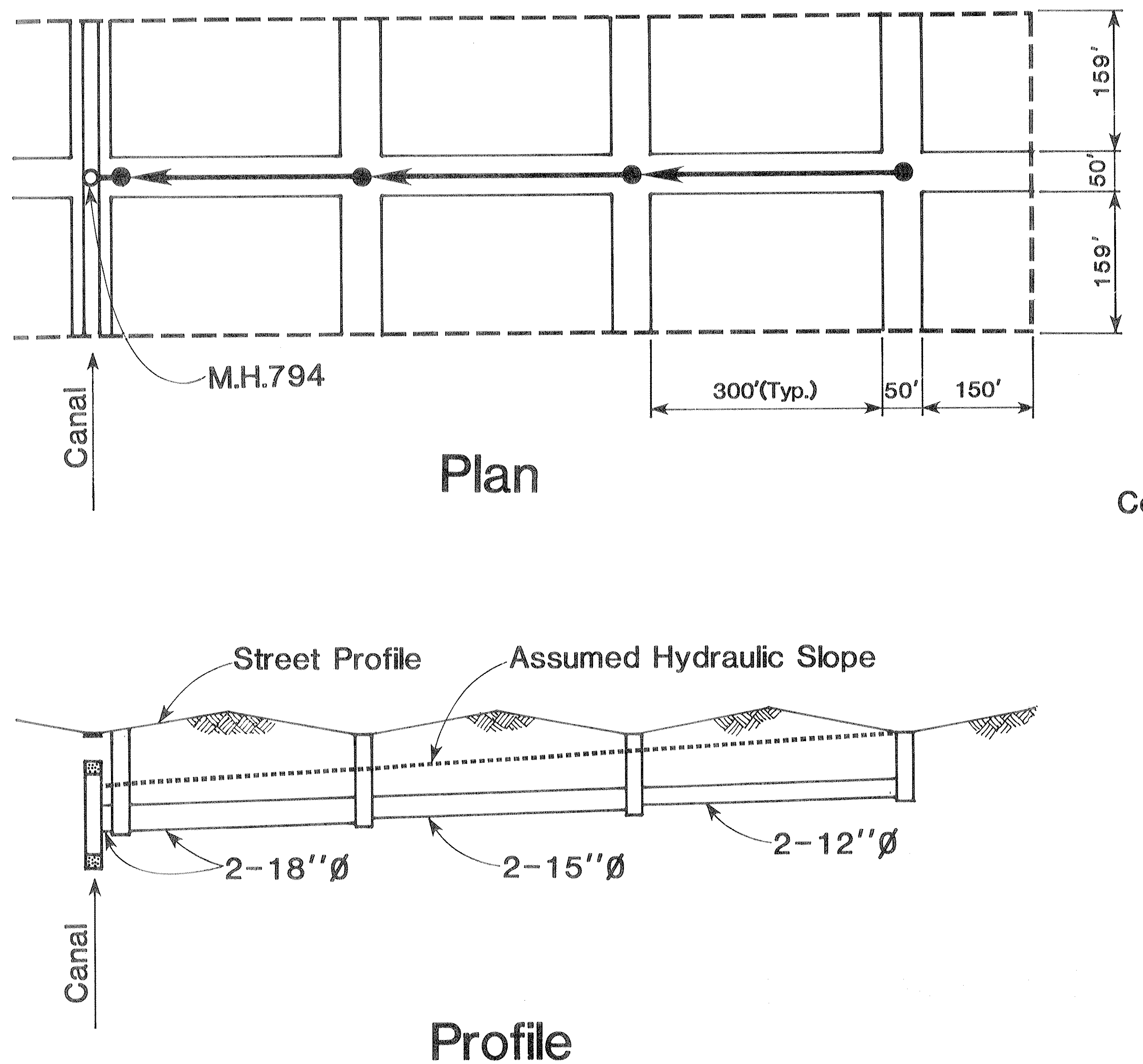
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No.793

Jefferson Parish

Figure B9



Design Data

Contributory Area : 10.36 Acres

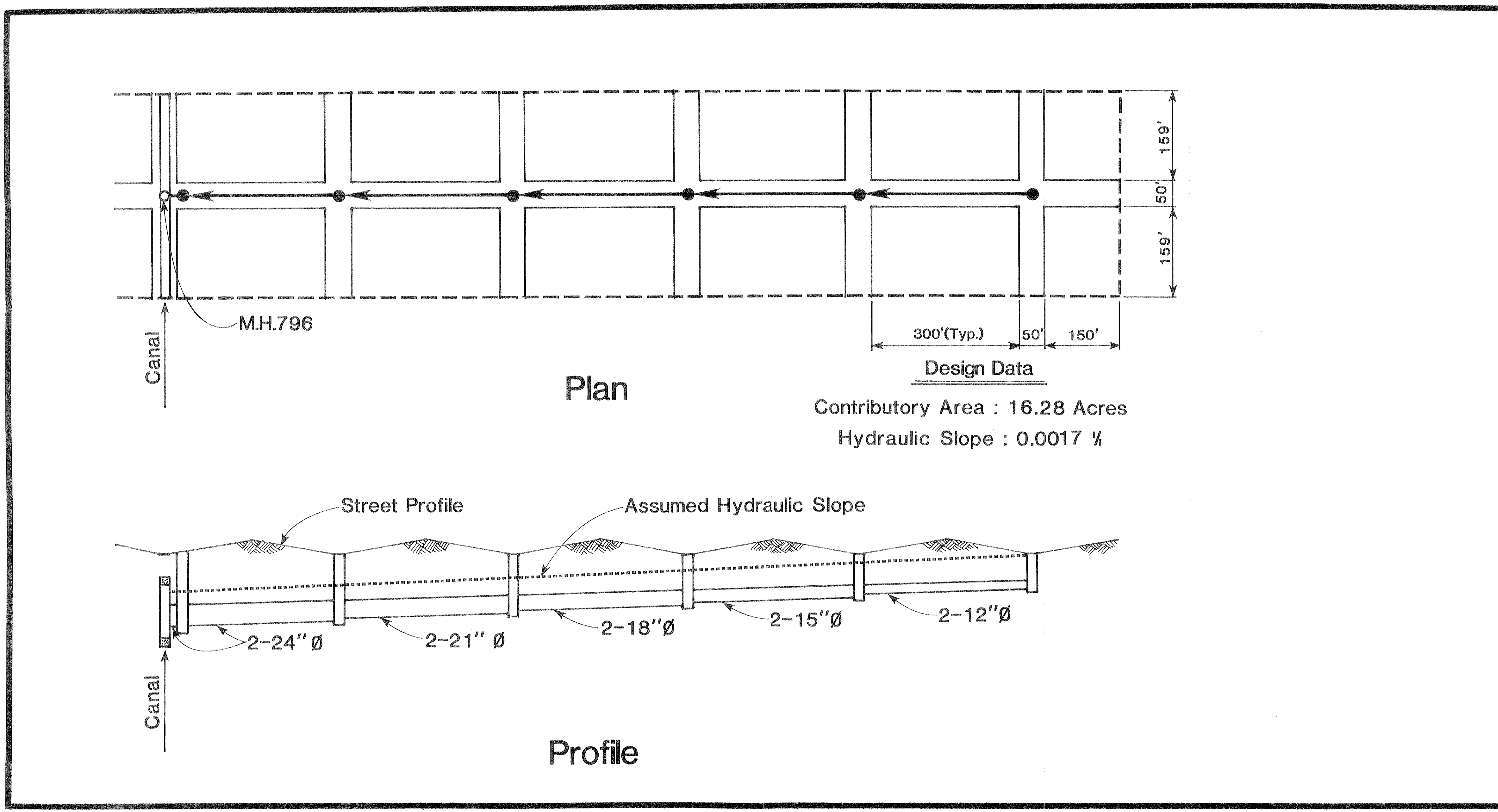
Hydraulic Slope : 0.0029 '/'

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No.794
Jefferson Parish

Figure B10



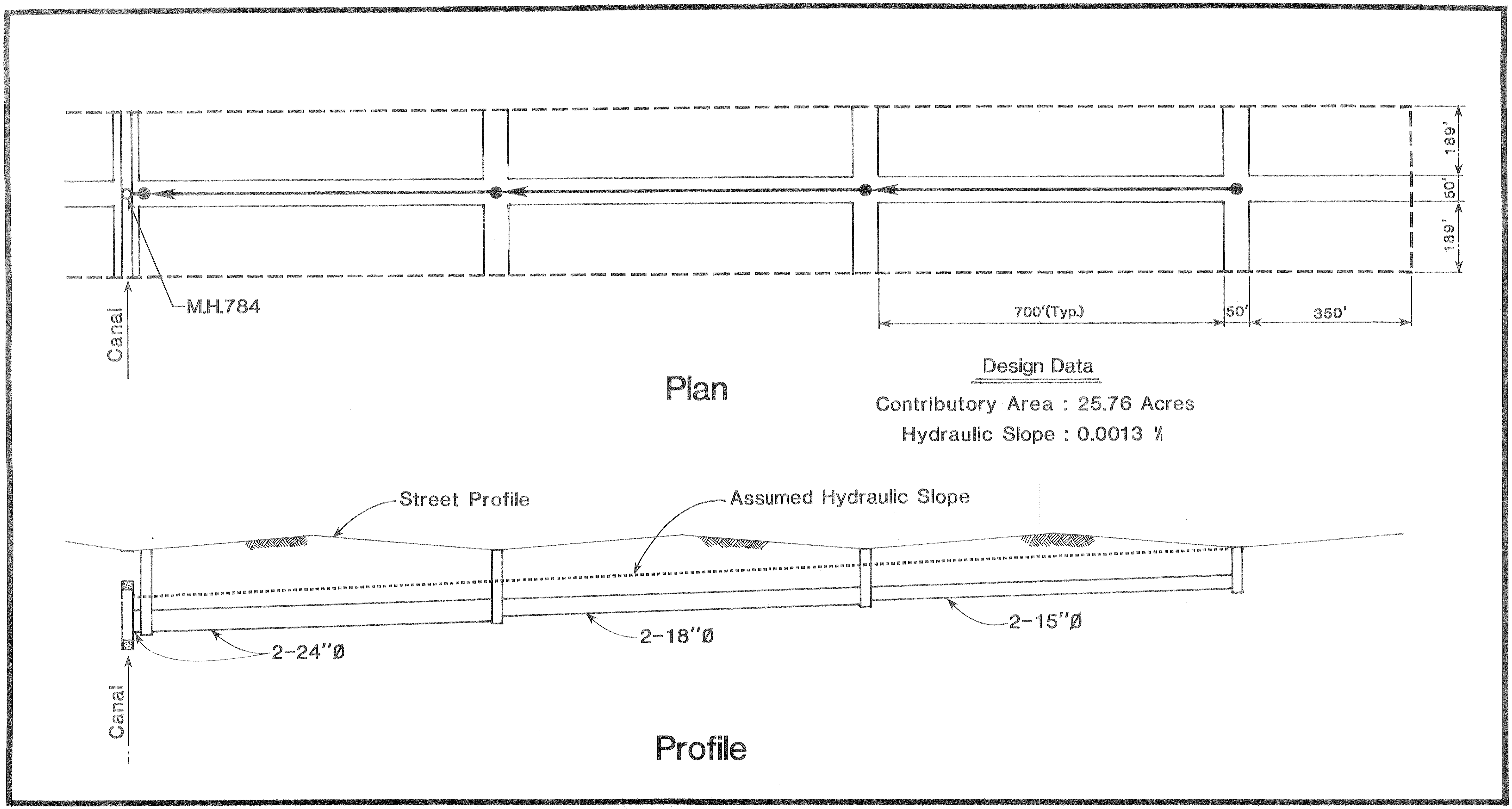
Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

Standard Street No.796

Jefferson Parish

Figure B11

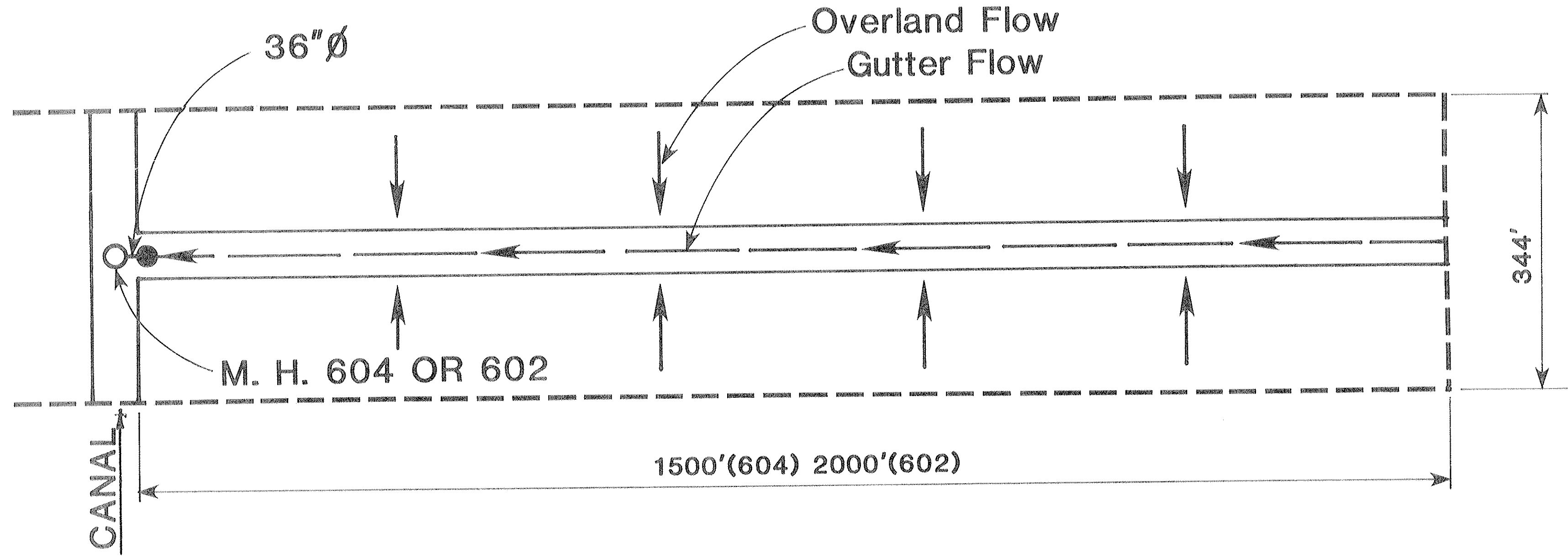


Seventeenth Street Canal
Drainage Basin Study

Linfield , Hunter & Gibbons , Inc.

Standard Street No. 784
Jefferson Parish

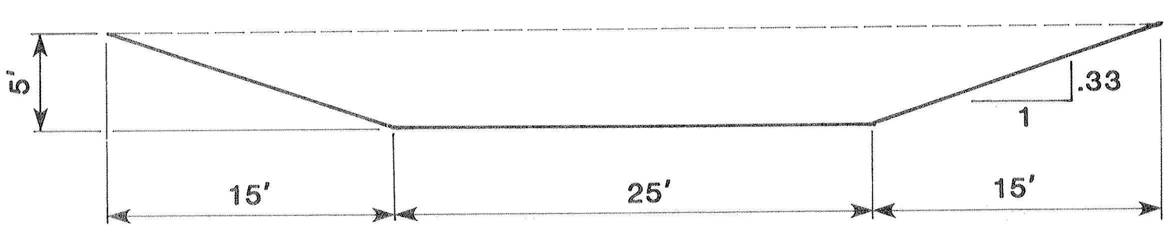
Figure B12



Plan

Design Data

Contributory Area :
11.84 Acres (604) 15.75 Acres (602)
Hydraulic Slope : 0.0025 '/'



Cross Section

Seventeenth Street Canal
Drainage Basin Study

Linfield, Hunter & Gibbons, Inc.

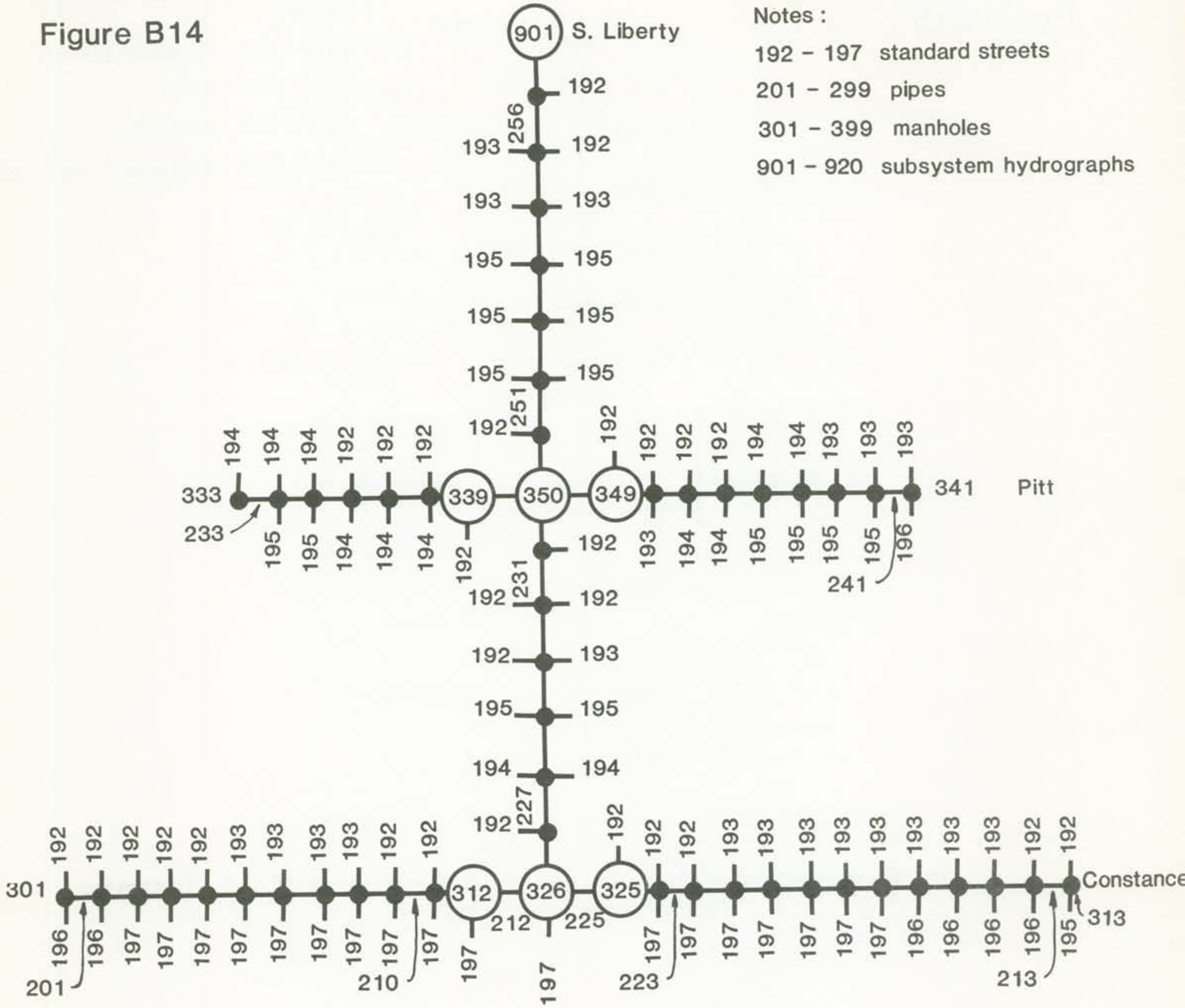
Special Gutter Streets
No.'s 602 & 604
Jefferson Parish
Figure B13

SWMM Simulation Numbering System

Orleans Parish : Napoleon (Part 1)

(Existing System)

Figure B14

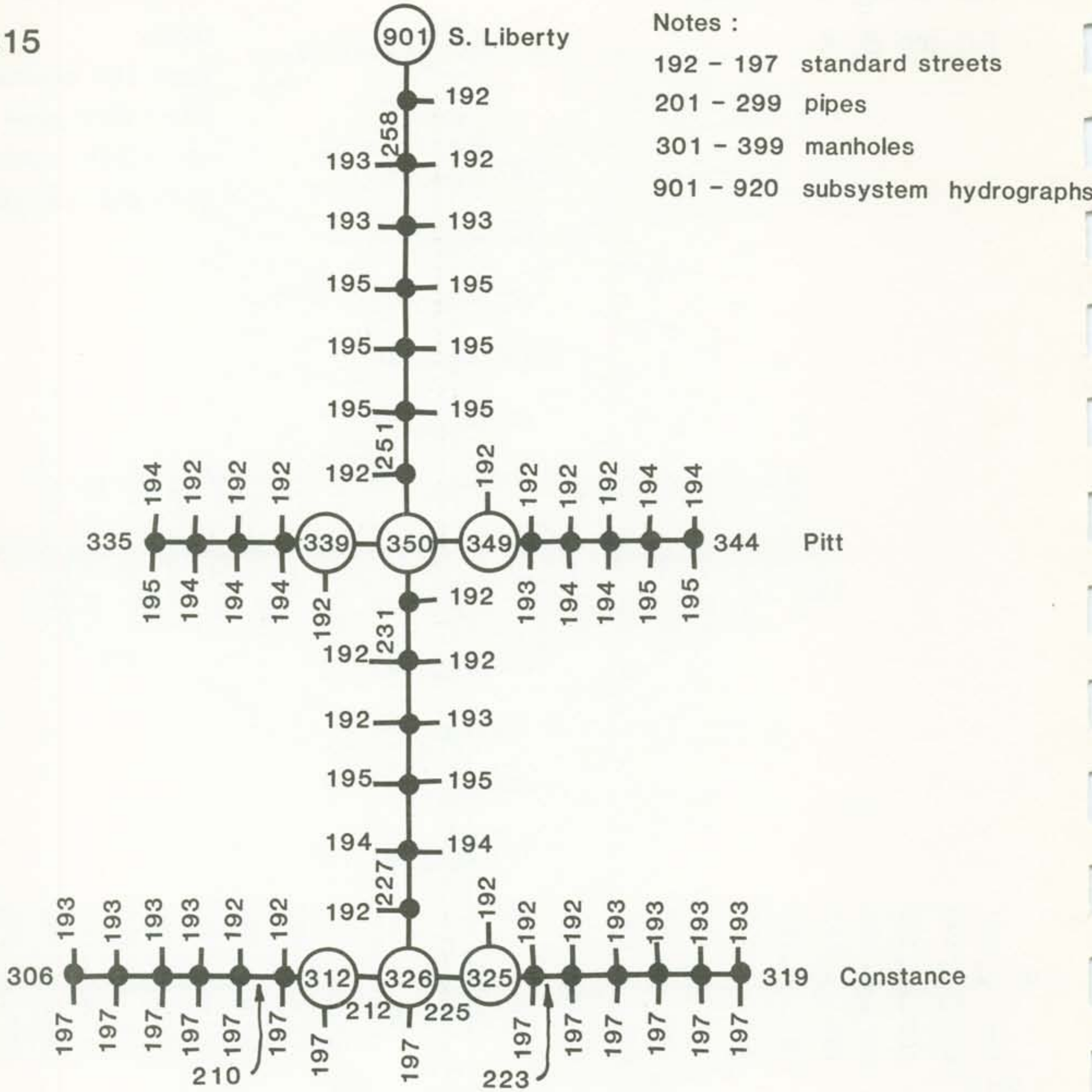


SWMM Simulation Numbering System

Orleans Parish : Napoleon (Part 1)

(Improved System)

Figure B15

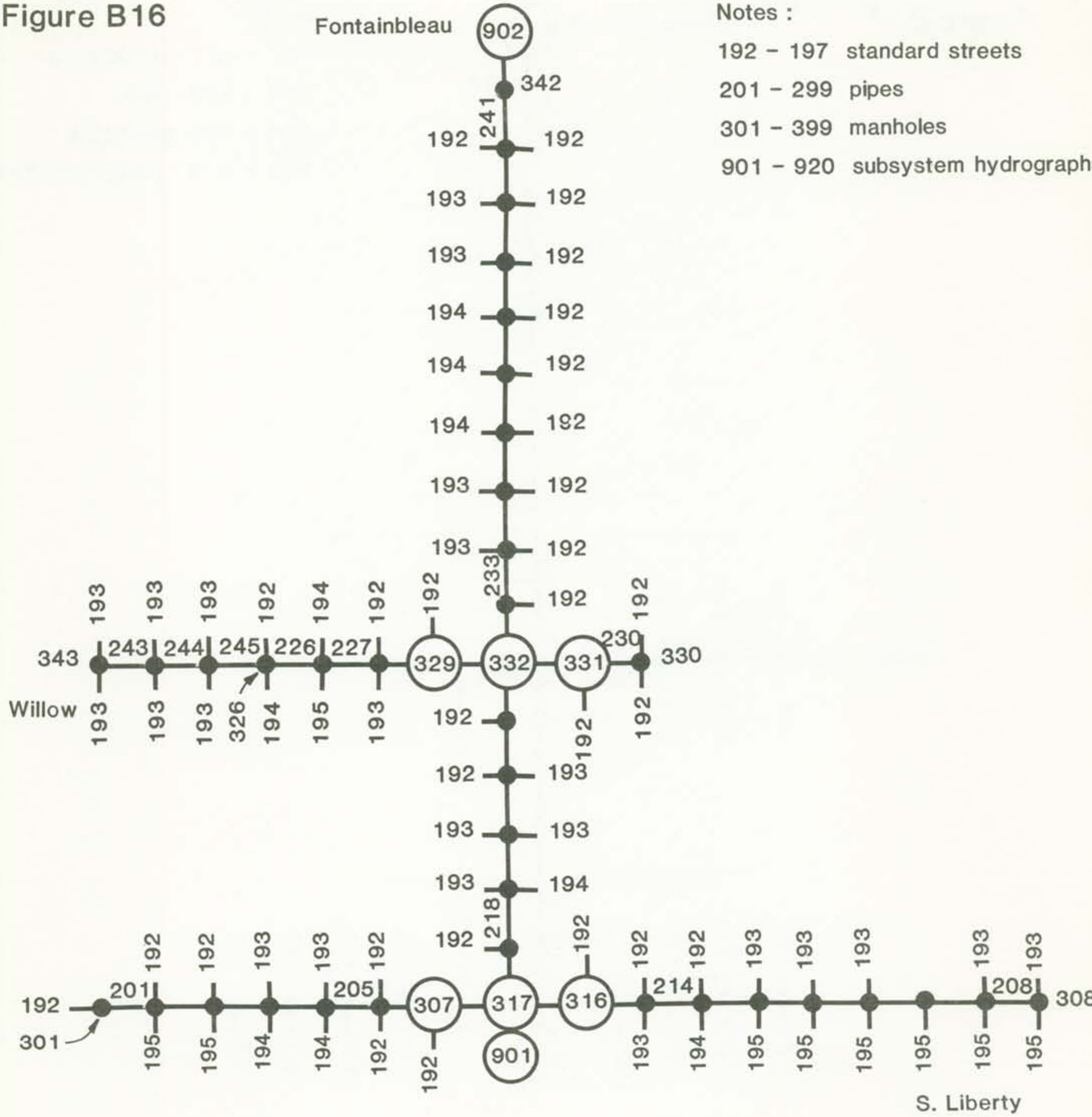


SWMM Simulation Numbering System

Orleans Parish : Napoleon (Part 2)

(Existing System)

Figure B16



Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

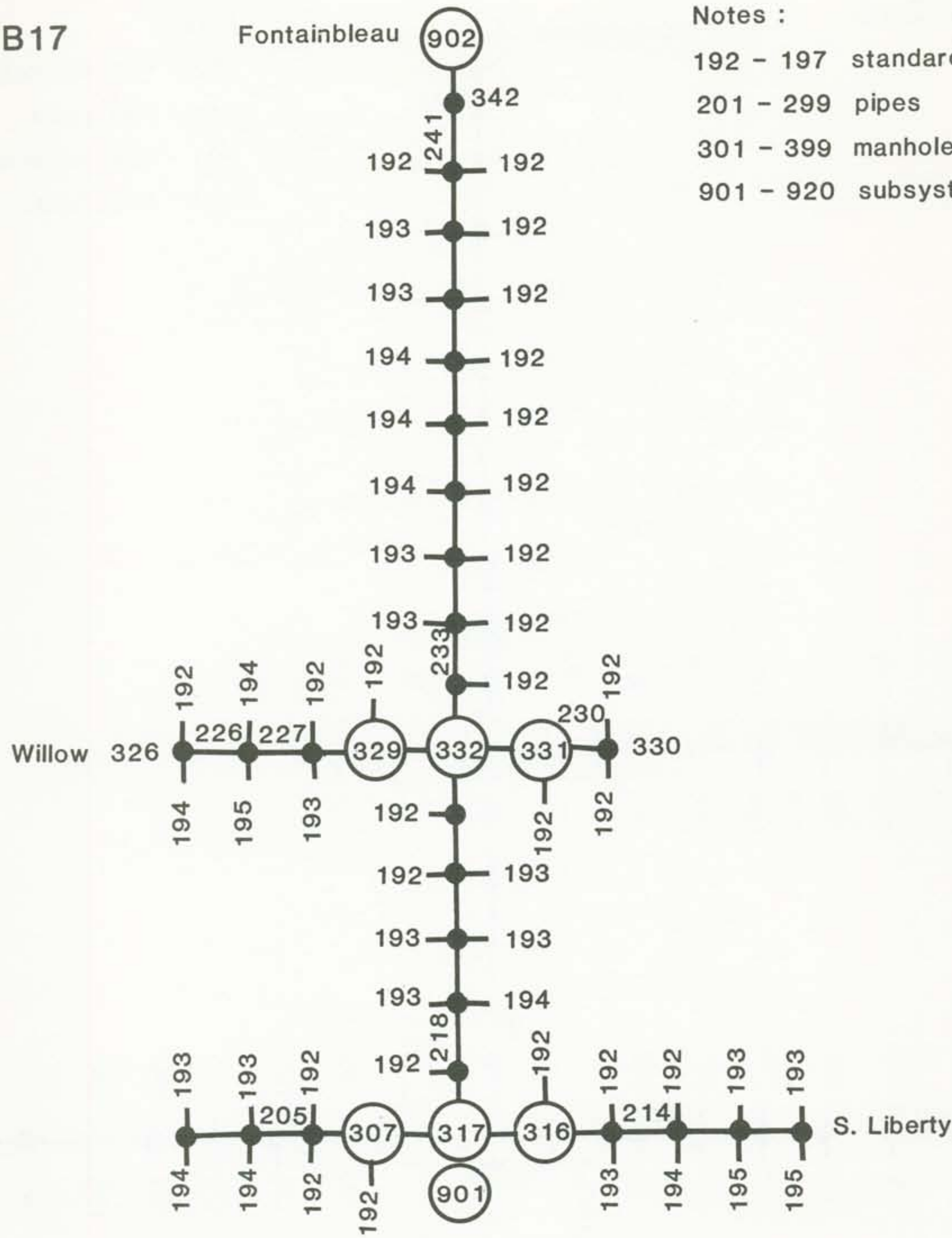
901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Napoleon (Part 2)

(Improved System)

Figure B17



Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Jefferson

(Improved System)

Figure B18

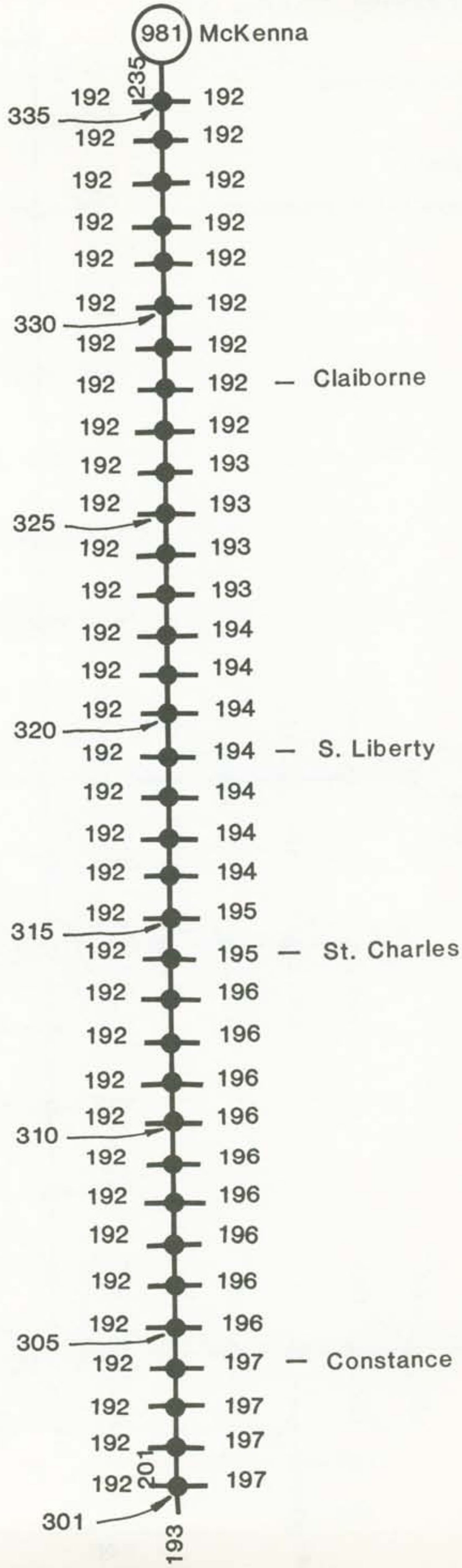
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

981 subsystem hydrograph



SWMM Simulation Numbering System

Orleans Parish: Nashville (Part 1)

(Existing System) Figure B19

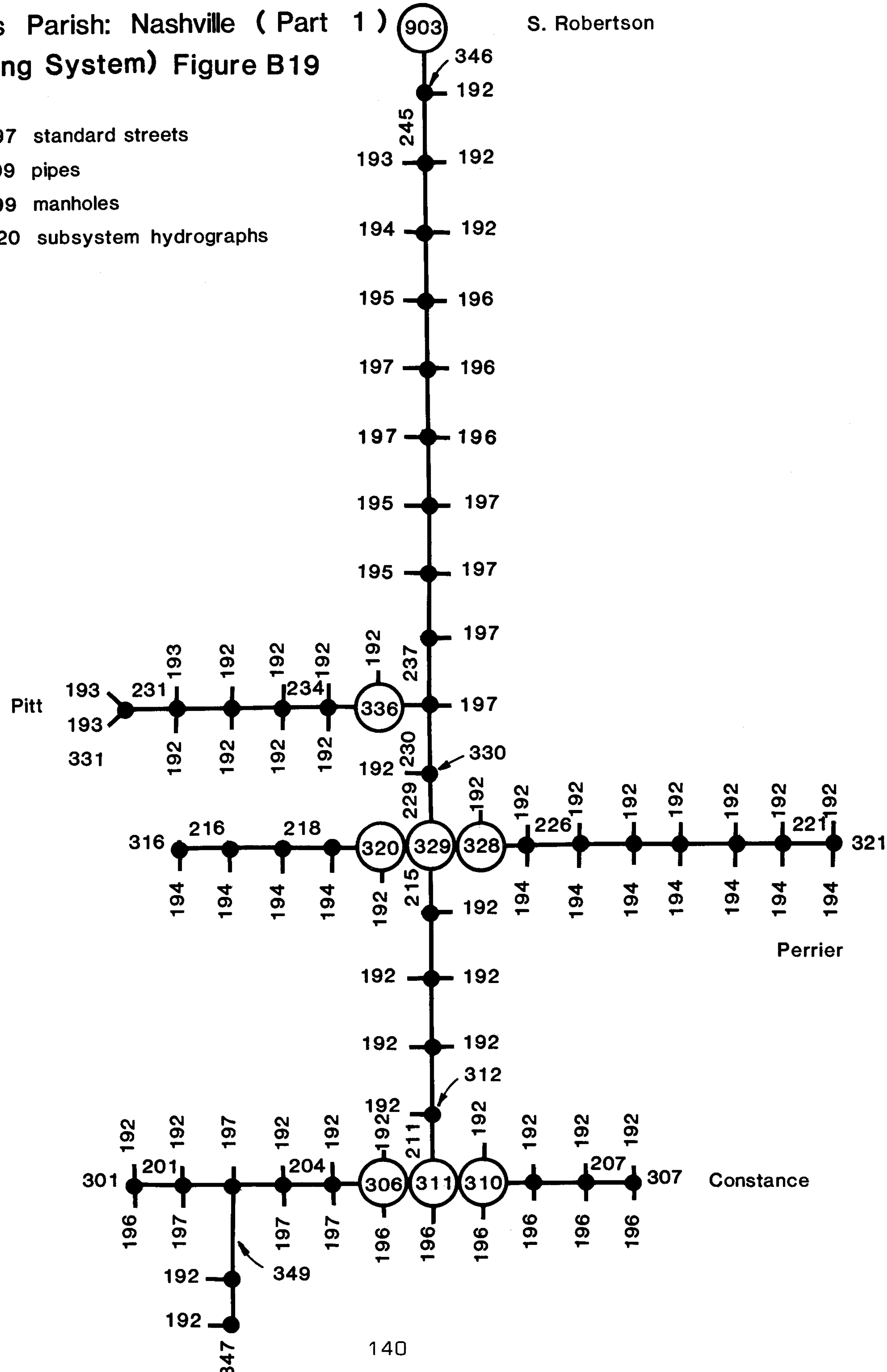
Notes:

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Nashville (Part 1)

(Improved System)

Figure B20

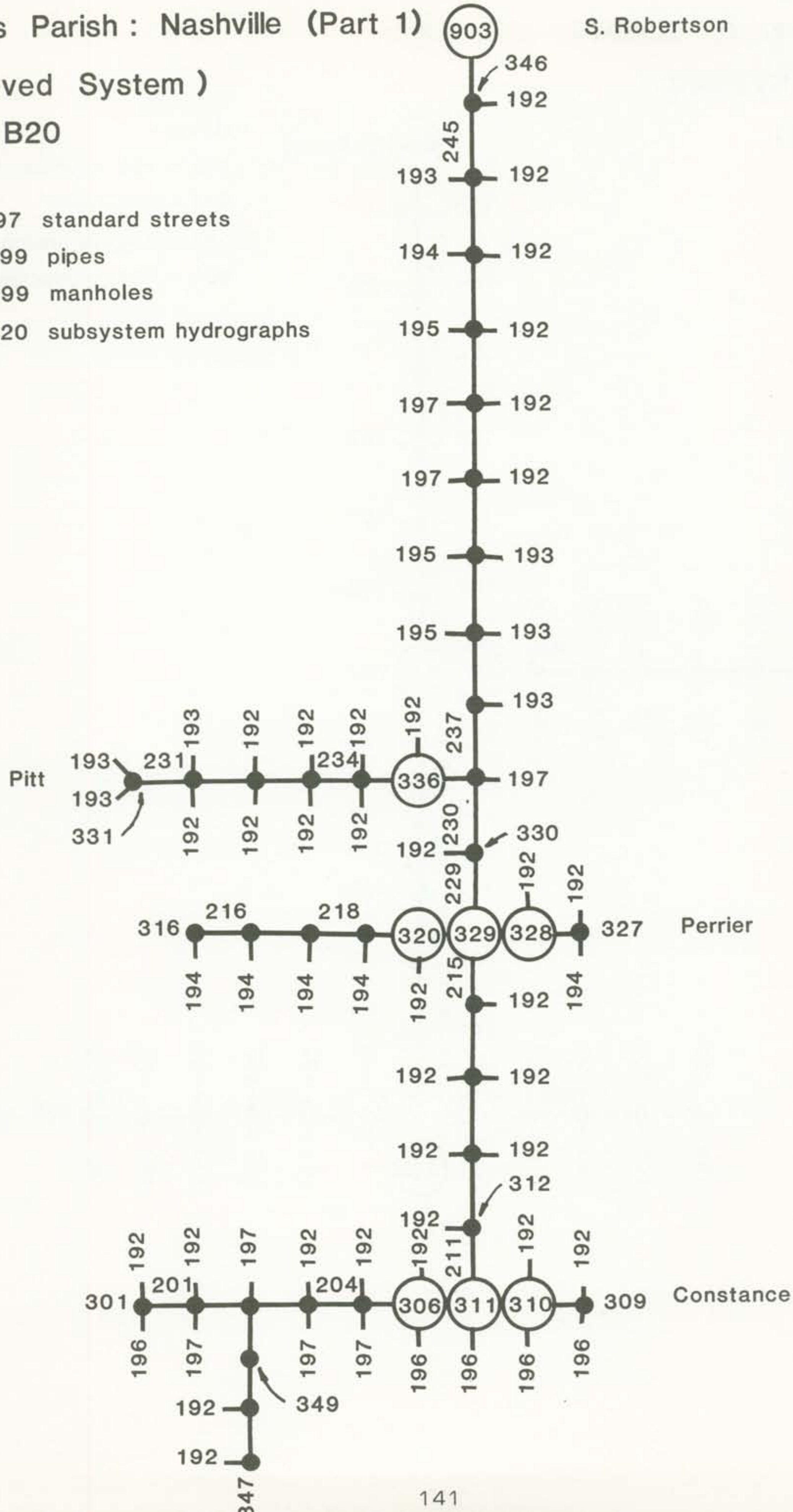
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Nashville (Part 2)

(Existing System)

Figure B21

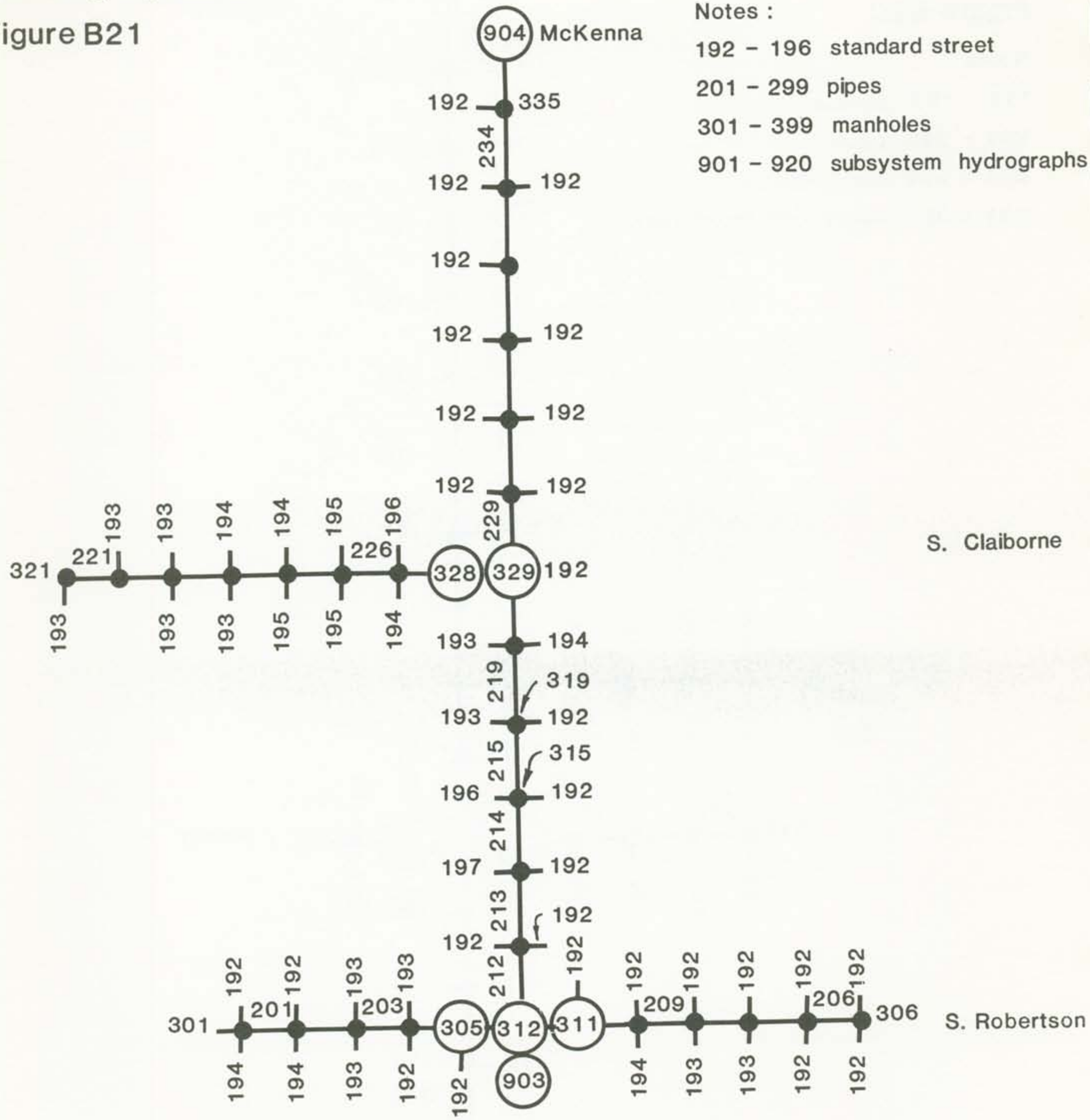
Notes :

192 - 196 standard street

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Nashville (Part 2)

(Improved System)

Figure B22

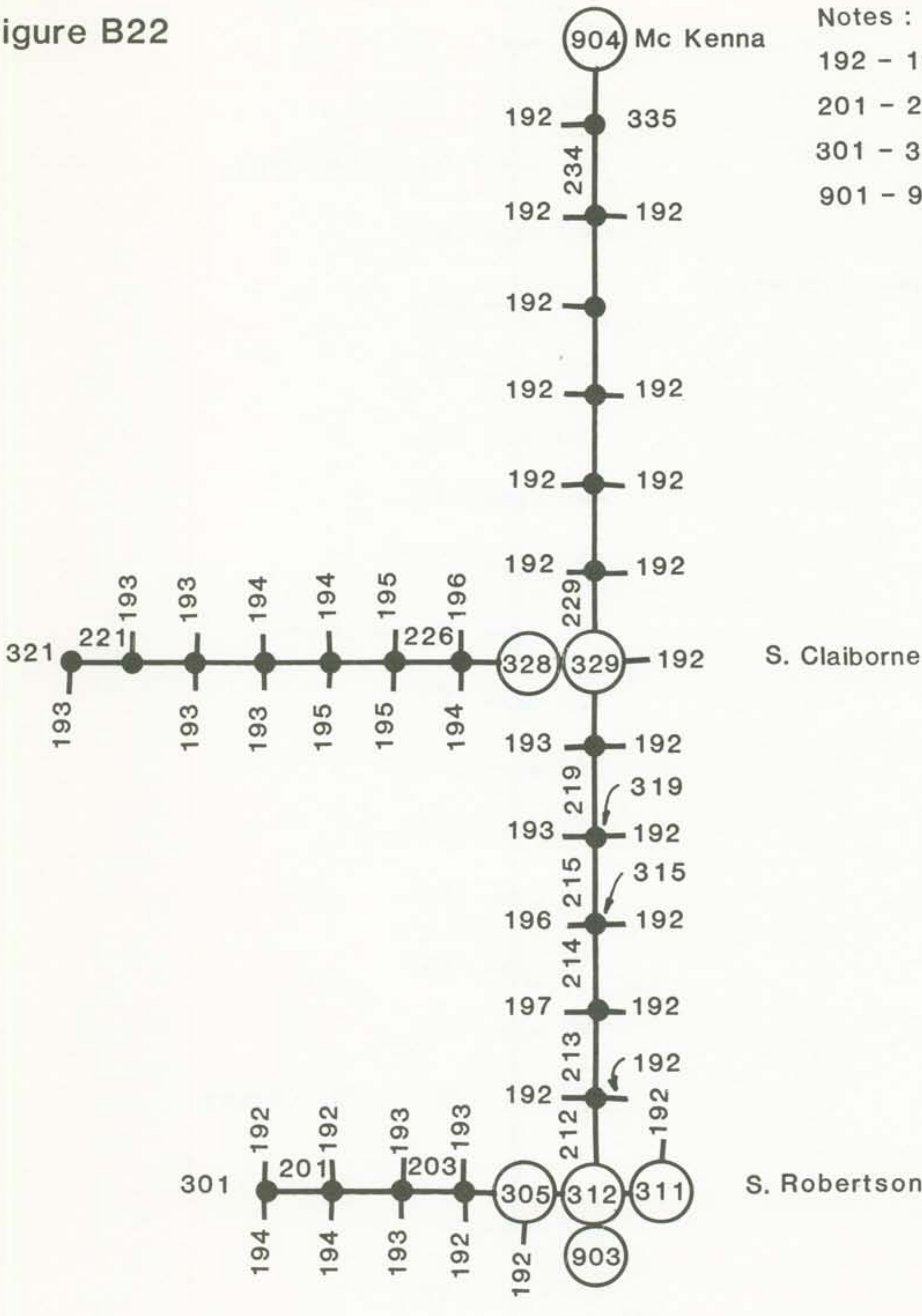
Notes :

192 - 196 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Louisiana
(Improved System)
Figure B23

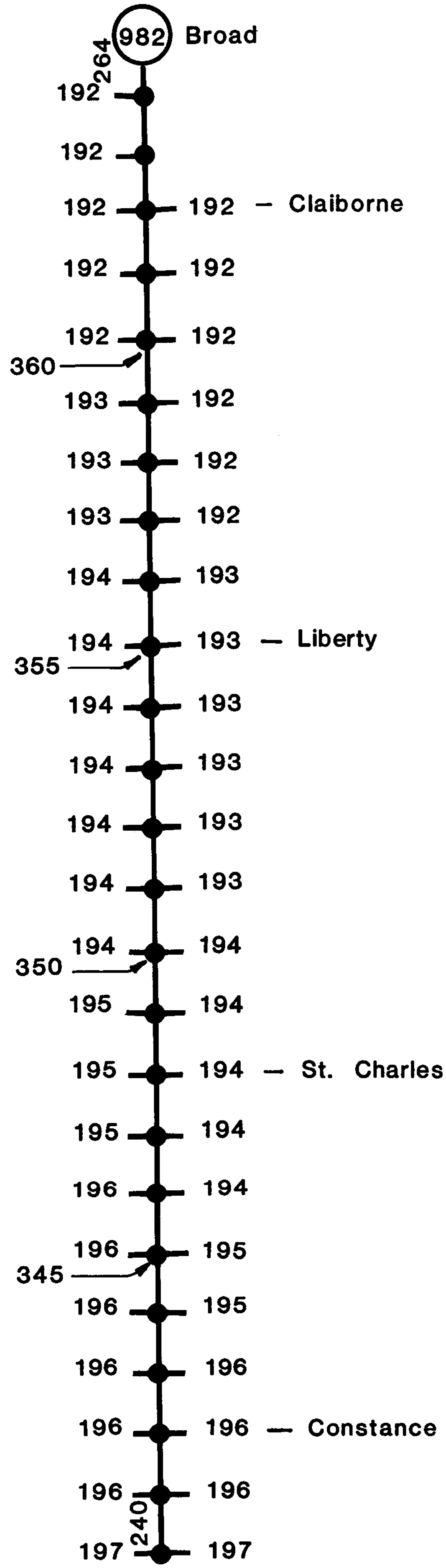
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

982 subsystem hydrograph



SWMM Simulation Numbering System

(Existing And Improved System)

Orleans Parish : Broad

Figure B24

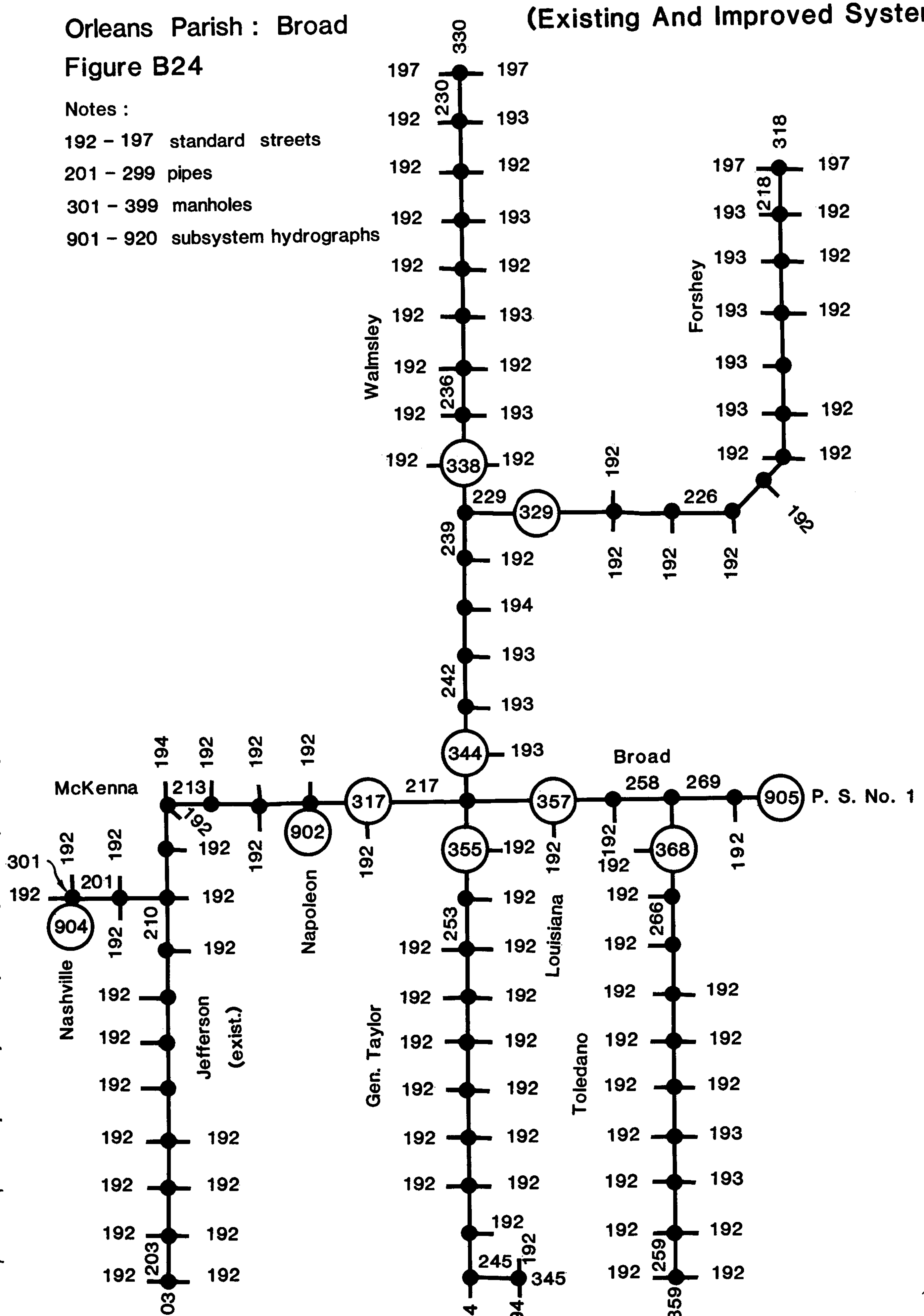
Notes :

192 – 197 standard streets

201 – 299 pipes

301 – 399 manholes

901 – 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Third St. (Part 1)

(Existing System)

Figure B25

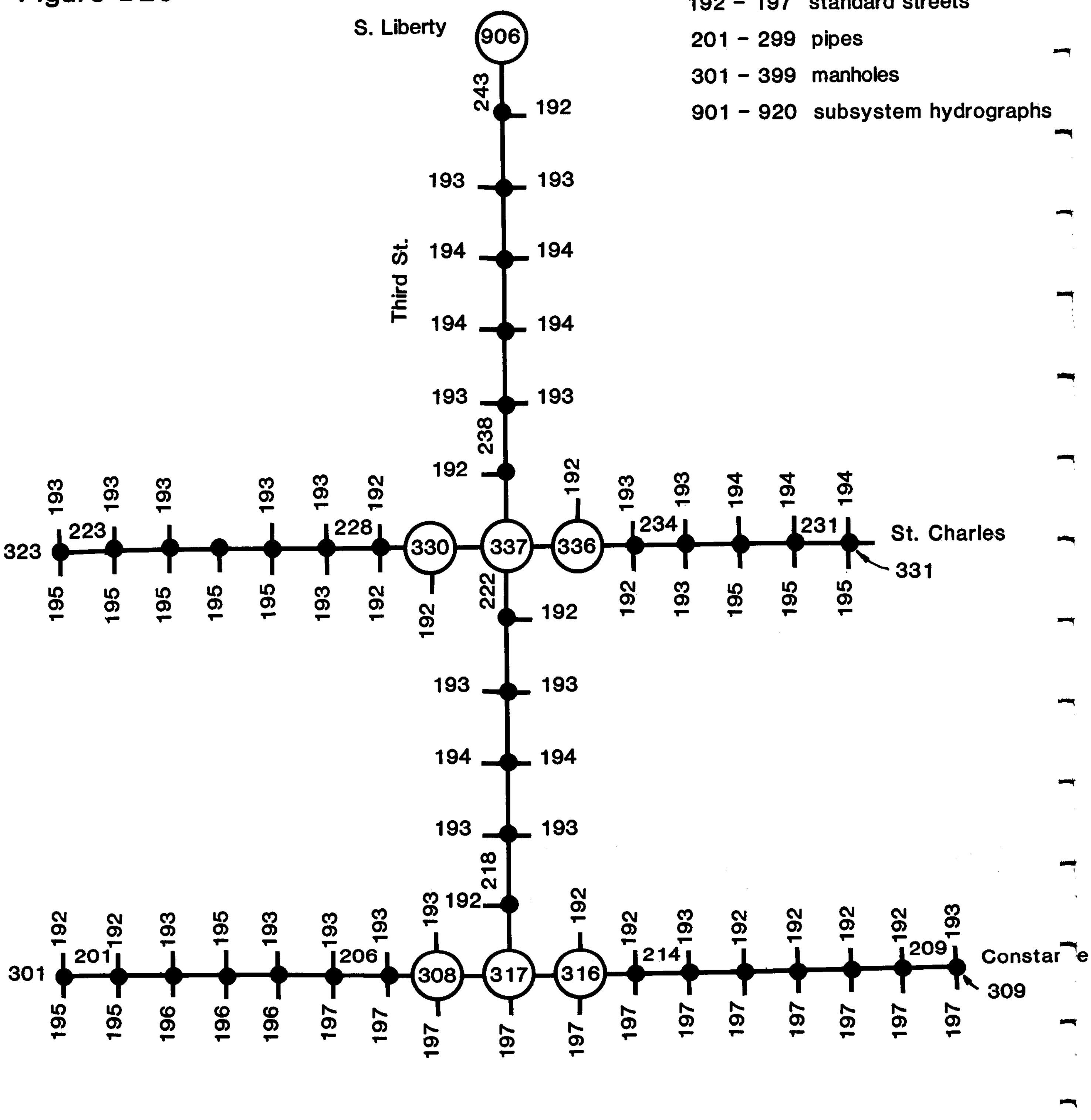
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Third St. (Part 1)

(Improved System)

Figure B26

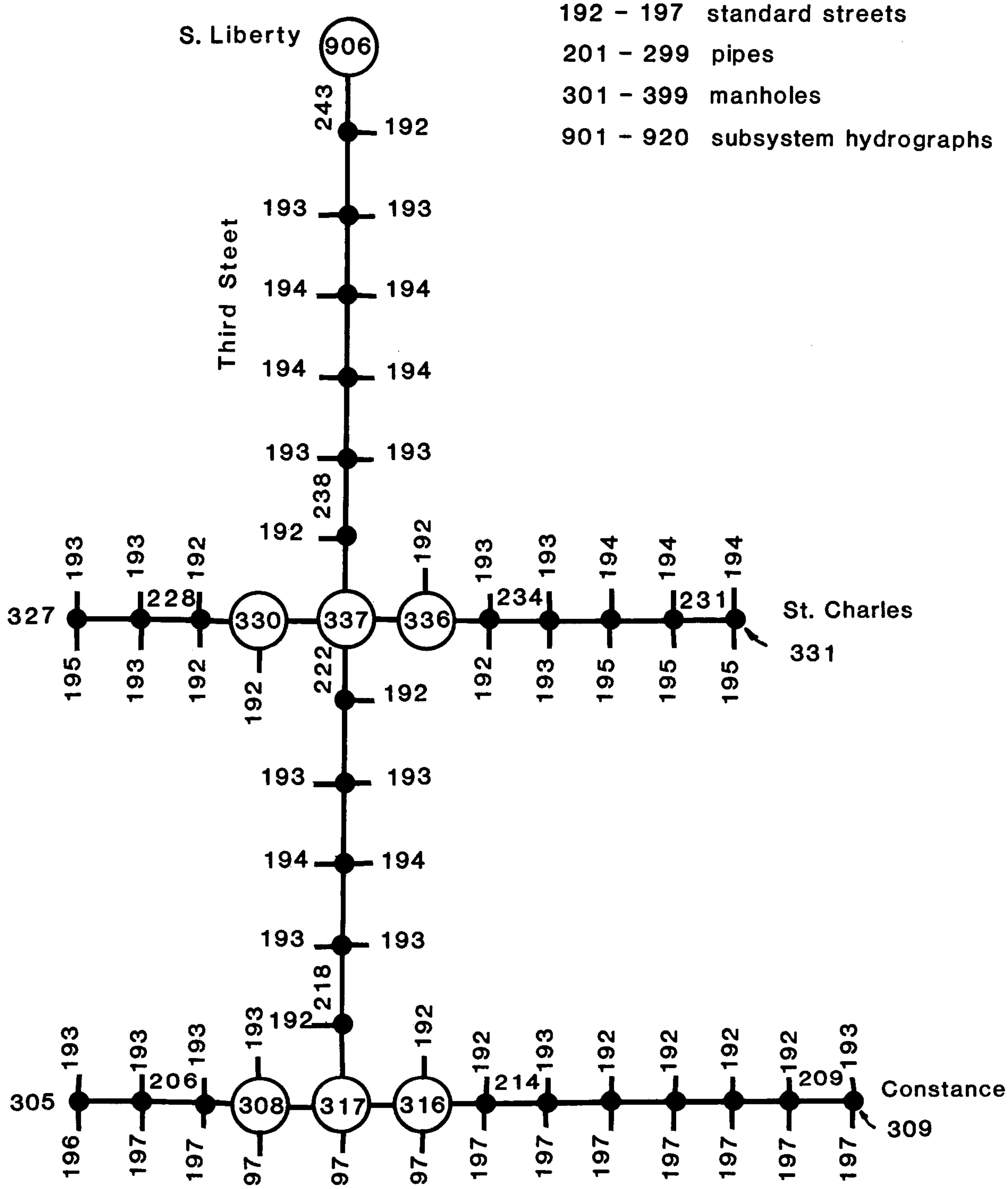
Notes:

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs

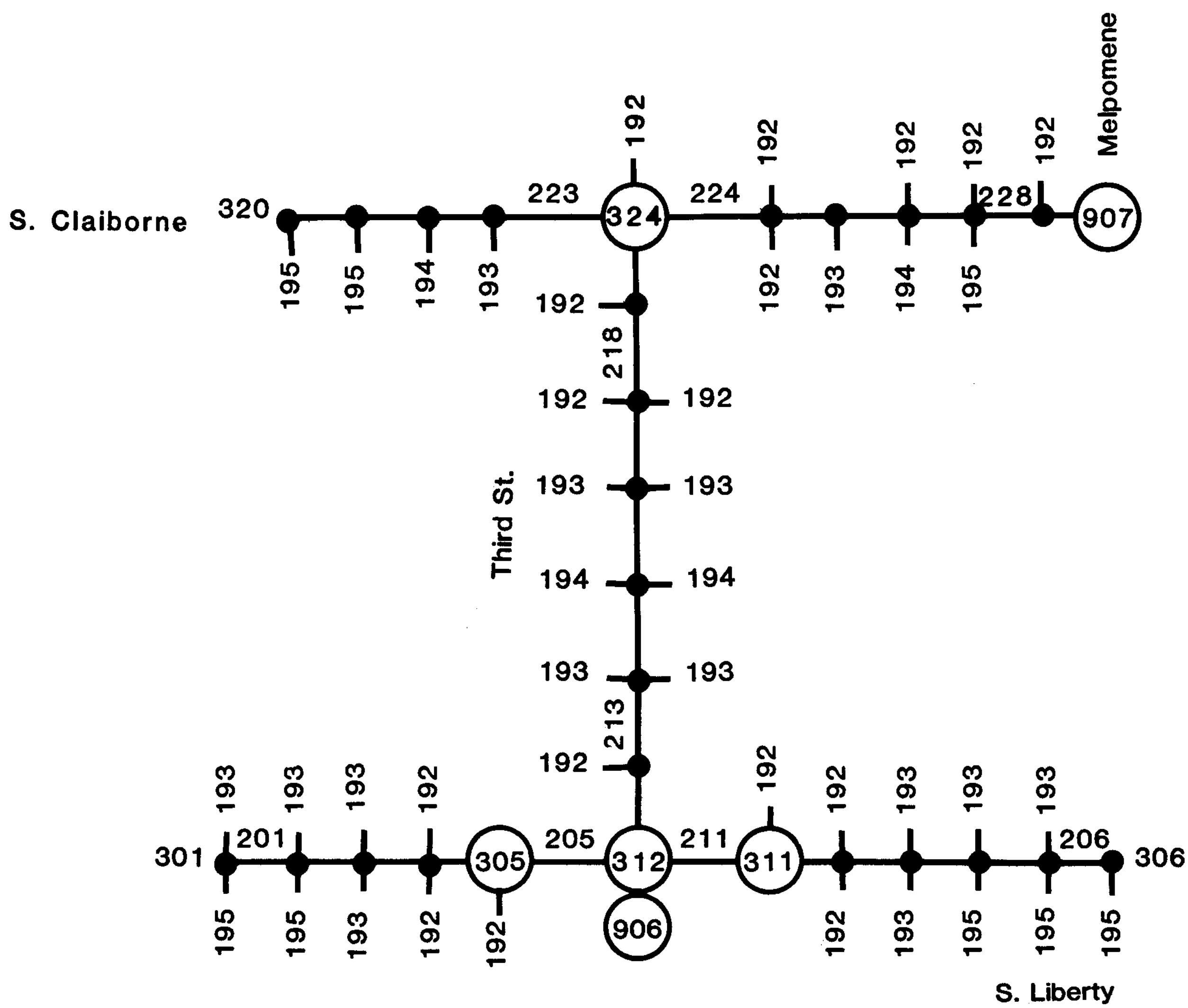


SWMM Simulation Numbering System

Orleans Parish : Third St. (Part 2)

(Existing System)

Figure B27



Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

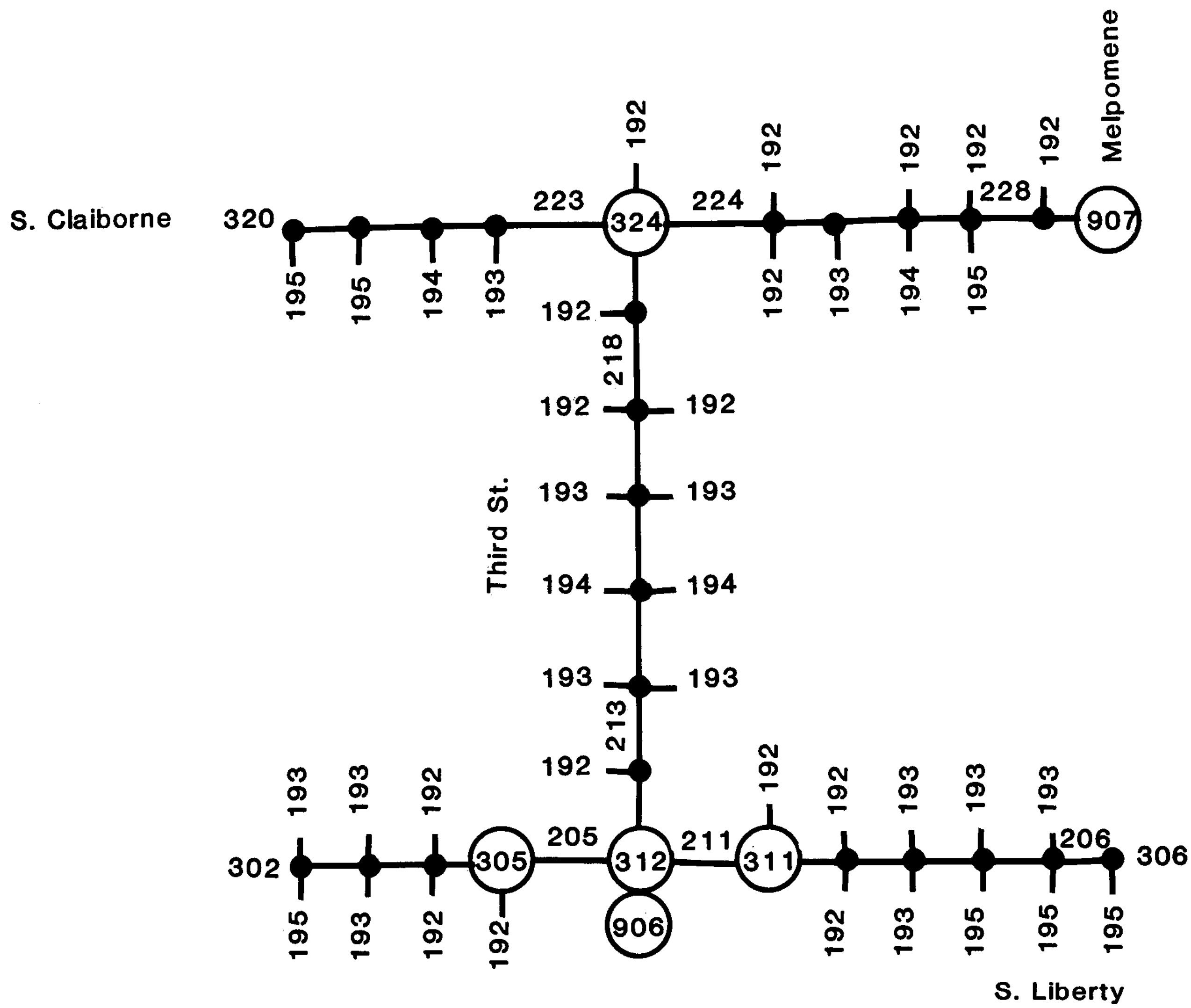
901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Third St. (Part 2)

(Improved System)

Figure B28



Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

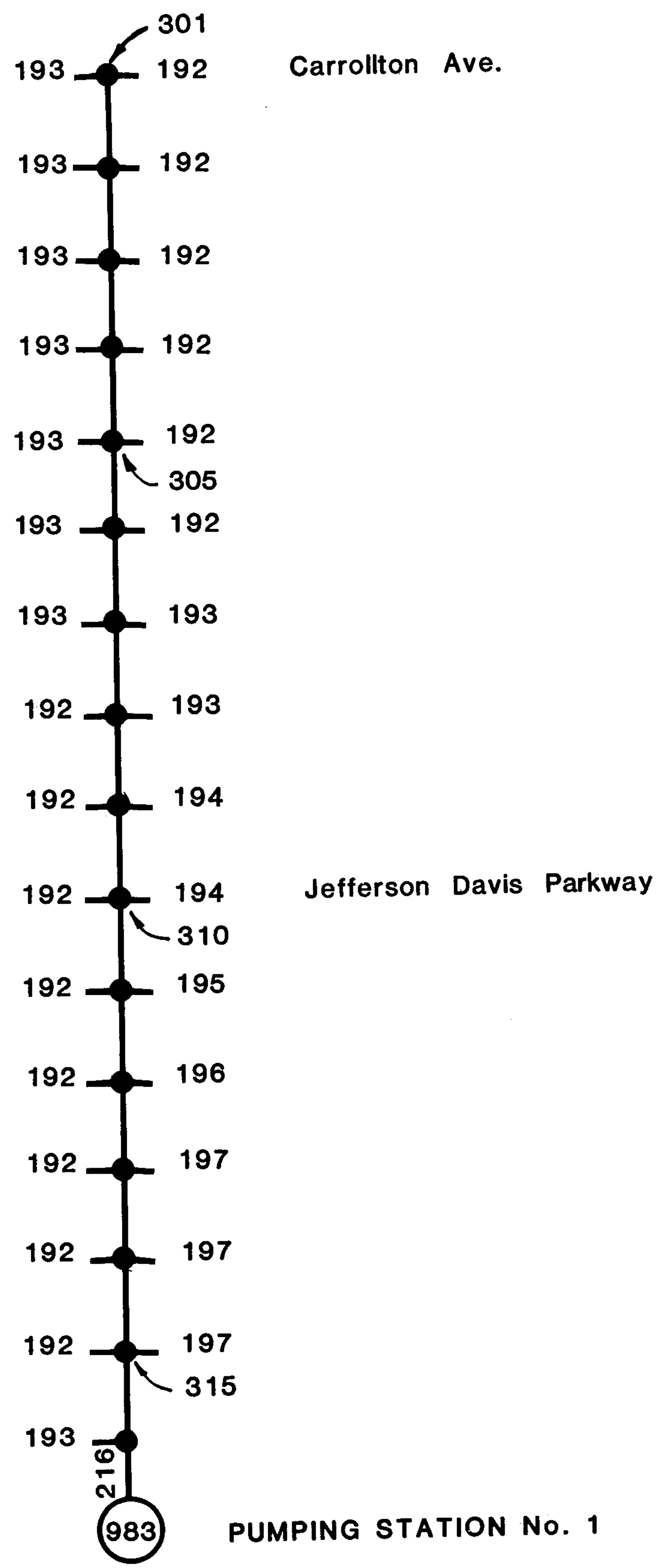
901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Washington Interceptor Canal

(Improved System)

Figure B29



Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

983 subsystem hydrograph

SWMM Simulation Numbering System
Orleans Parish : Melpomene
(Existing And Improved System)

Figure B30

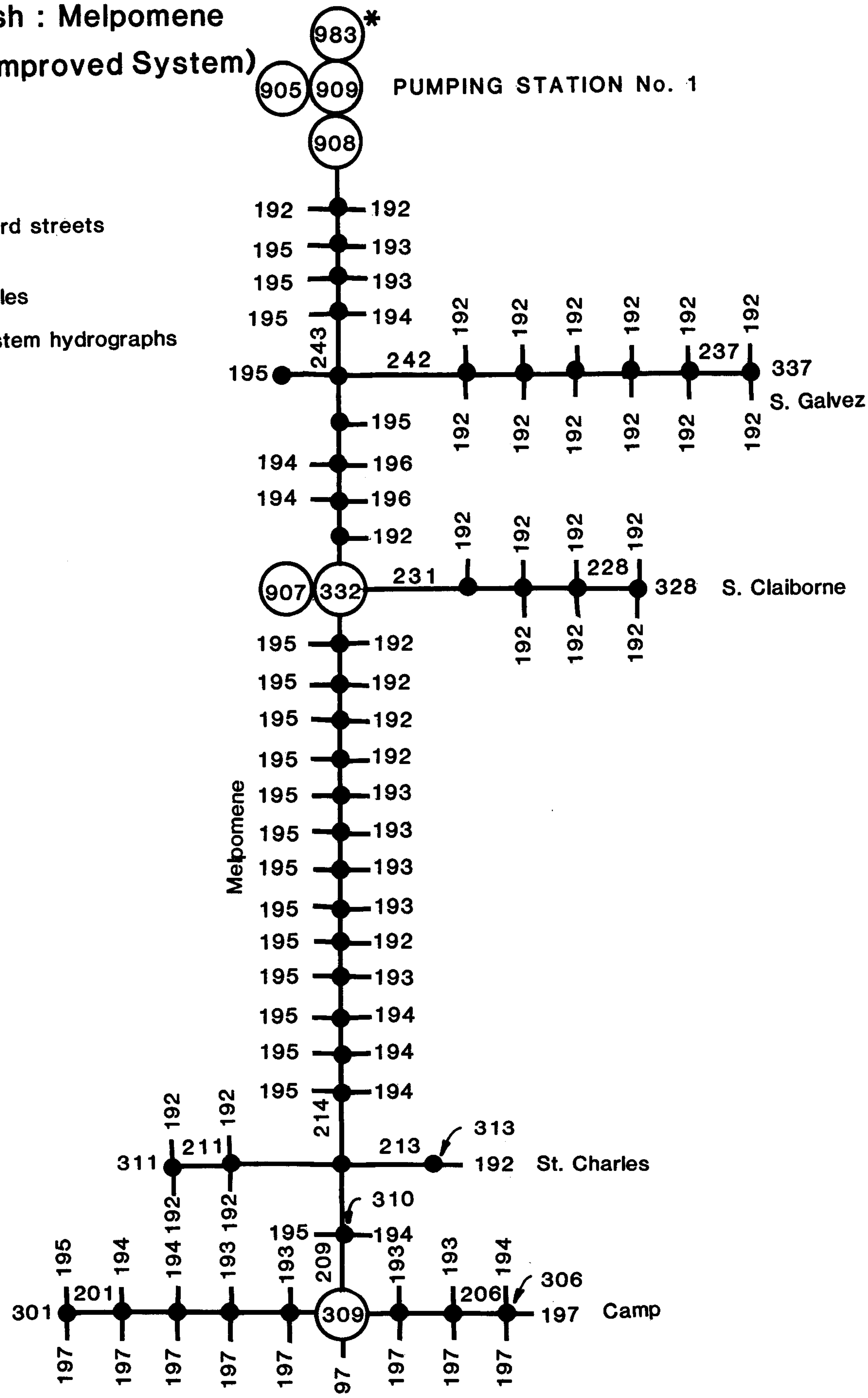
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs



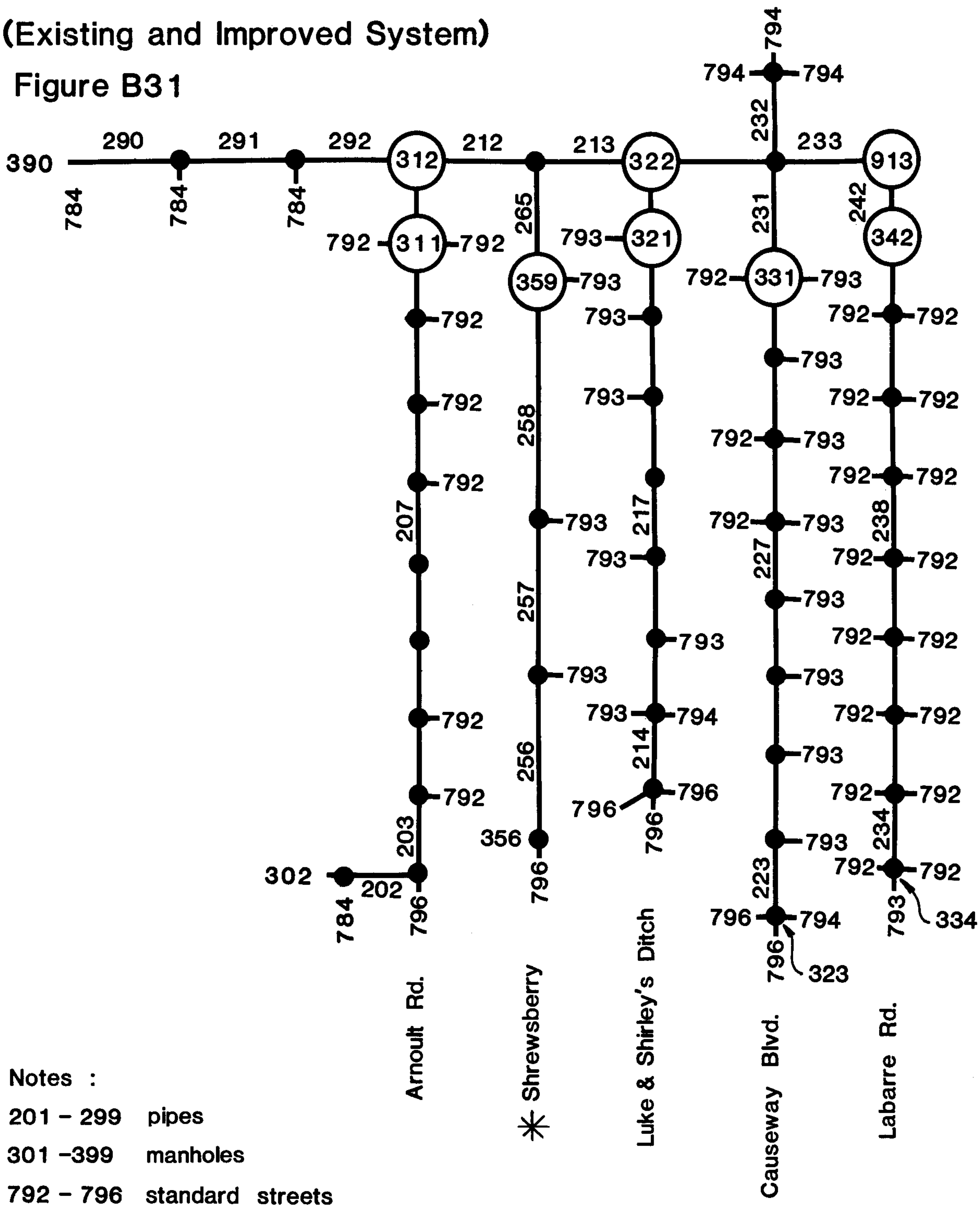
*Indicates improved conditions

SWMM Simulation Numbering System

Jefferson Parish: Hoey's Canal (Part 1)

(Existing and Improved System)

Figure B31



Notes :

201 - 299 pipes

301 -399 manholes

792 - 796 standard streets

901 - 920 subsystem hydrographs

Manholes not shown are numbered consecutively

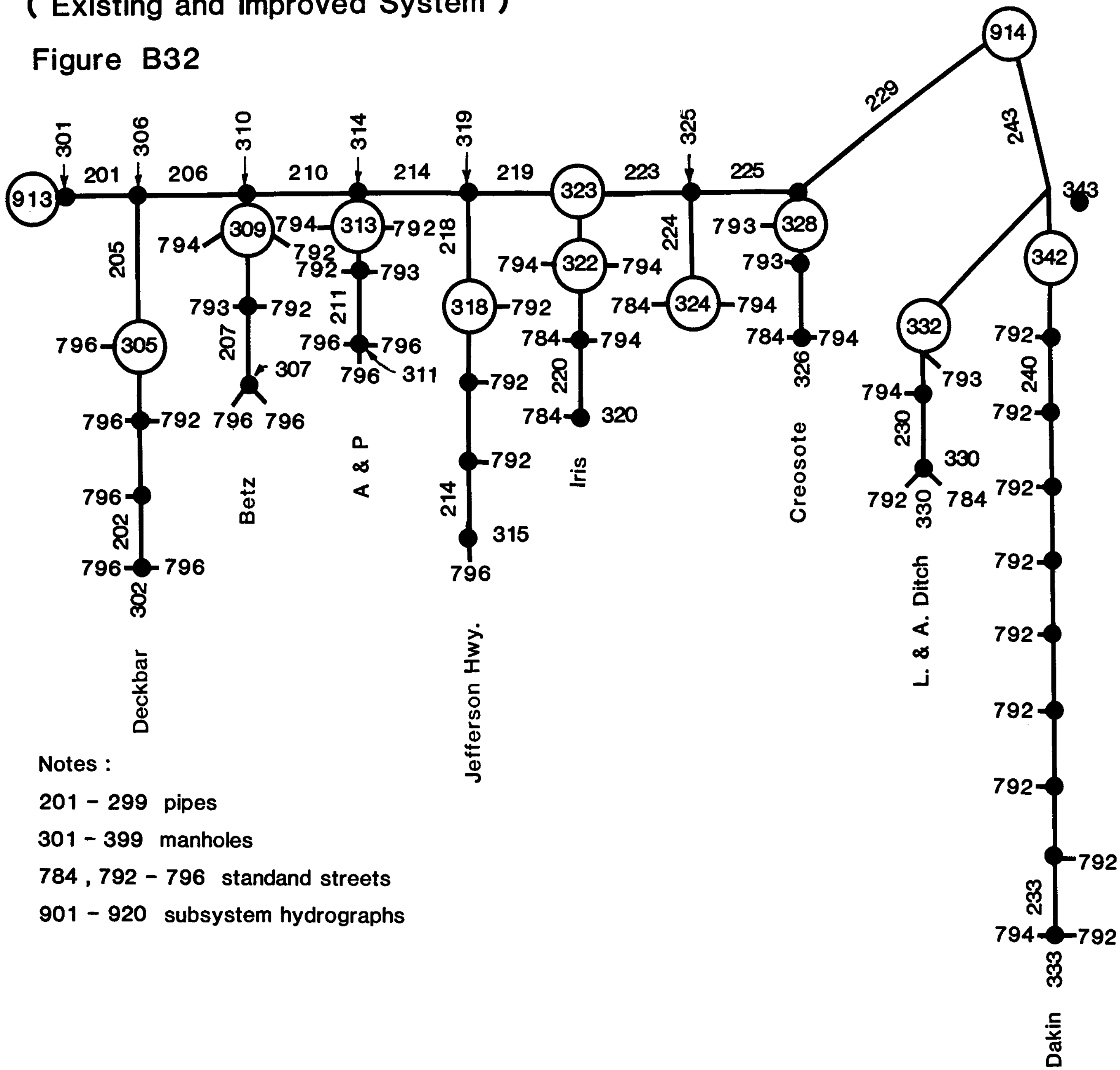
✳ Represents improved system

SWMM Simulation Numbering System

Jefferson Parish : Hoey's Canal (Part 2)

(Existing and Improved System)

Figure B32



Notes :

201 - 299 pipes

301 - 399 manholes

784 , 792 - 796 standand streets

901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Jefferson Parish: Loumar Ditch

(Existing System)

Figure B33

Notes:

201 - 299 pipes

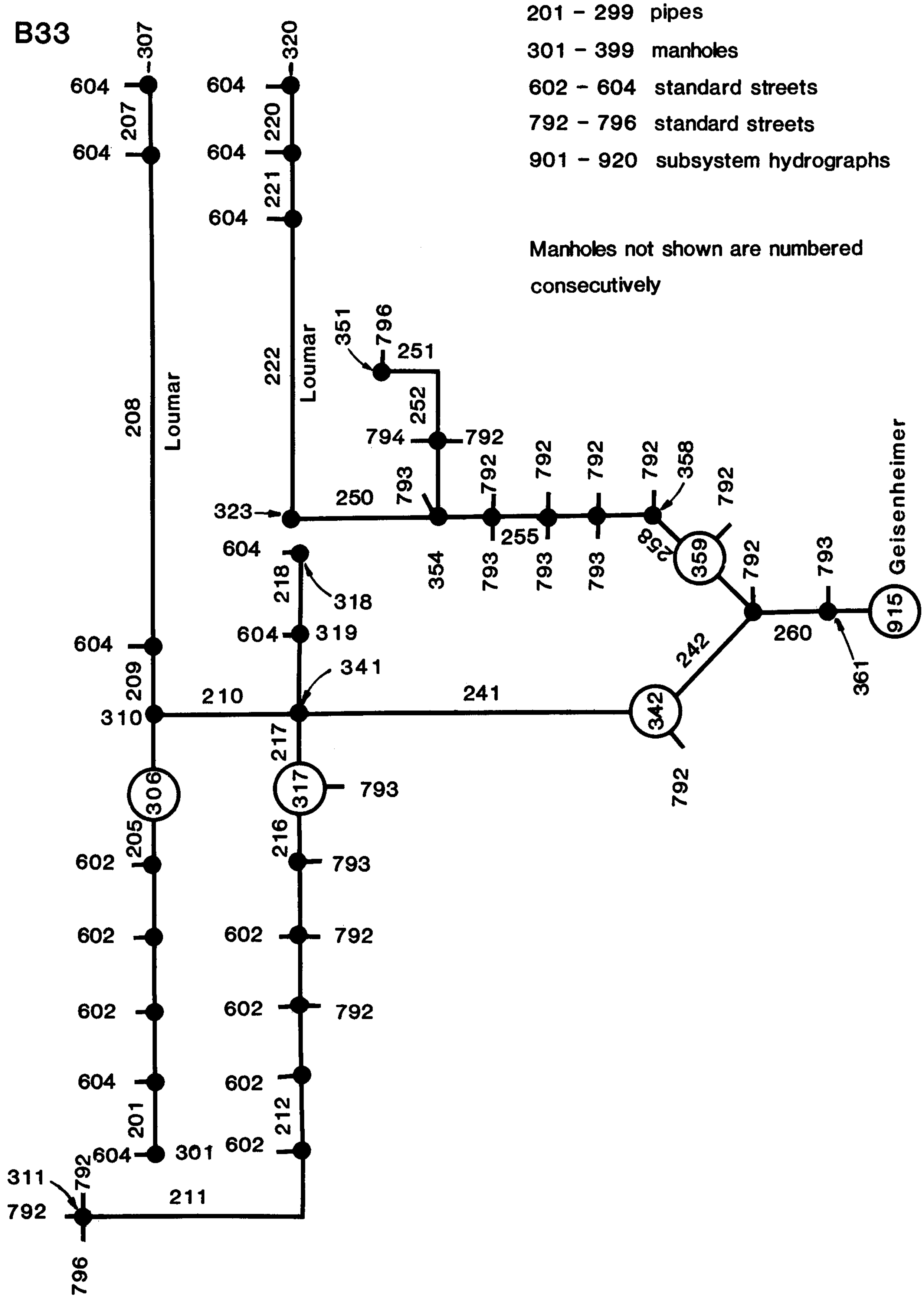
301 - 399 manholes

602 - 604 standard streets

792 - 796 standard streets

901 - 920 subsystem hydrographs

Manholes not shown are numbered consecutively



SWMM Simulation Numbering System

Jefferson Parish : Loumar Ditch (Improved System)

Figure B34

Notes :

201 - 299 pipes

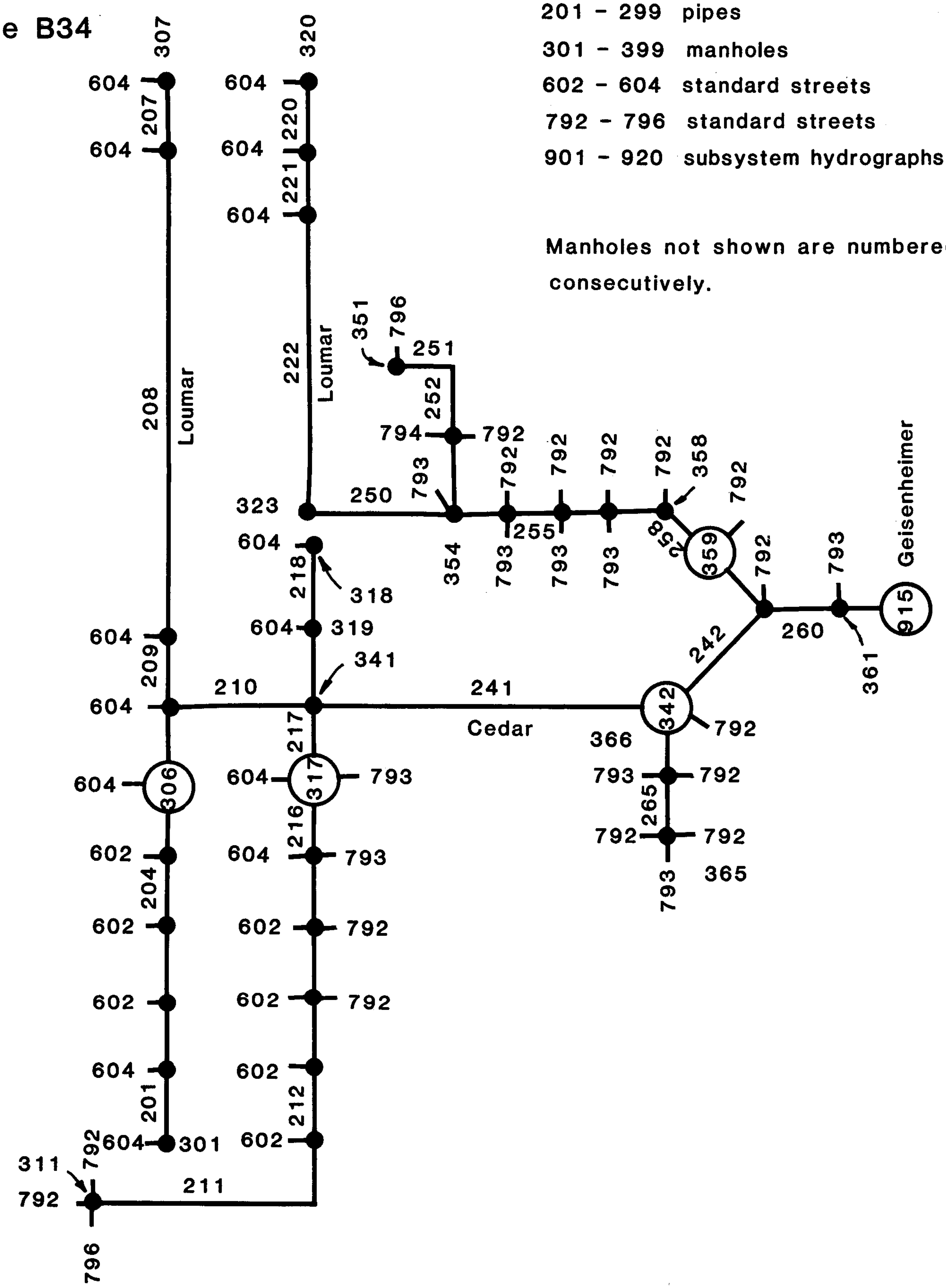
301 - 399 manholes

602 - 604 standard streets

792 - 796 standard streets

901 - 920 subsystem hydrographs

Manholes not shown are numbered consecutively.



SWMM Simulation Numbering System

Jefferson Parish: Geisenheimer Canal (Existing System)

Figure B35

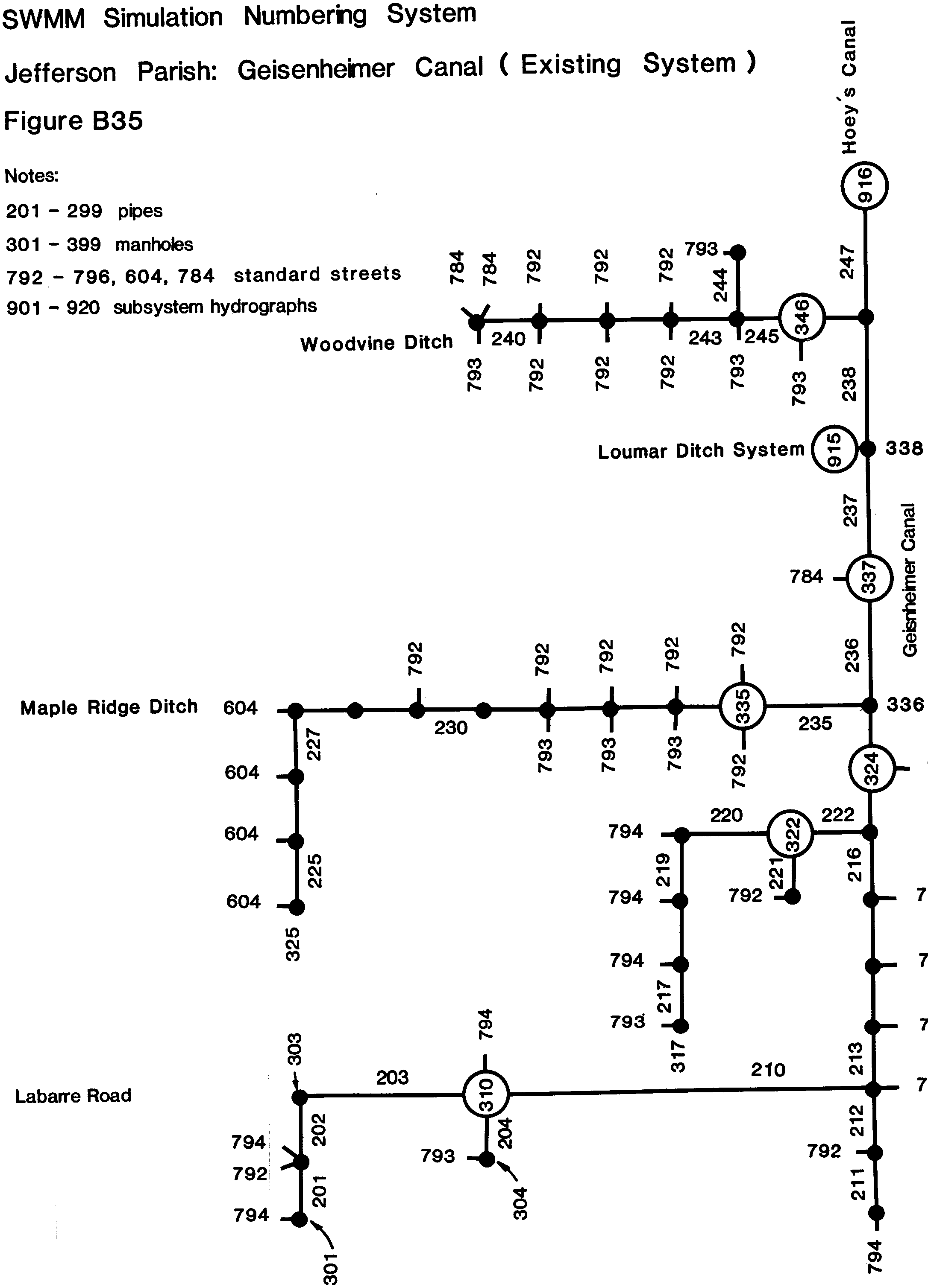
Notes:

201 - 299 pipes

301 - 399 manholes

792 - 796, 604, 784 standard streets

901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Jefferson Parish : Geisenheimer Canal (Improved System)

Figure B36

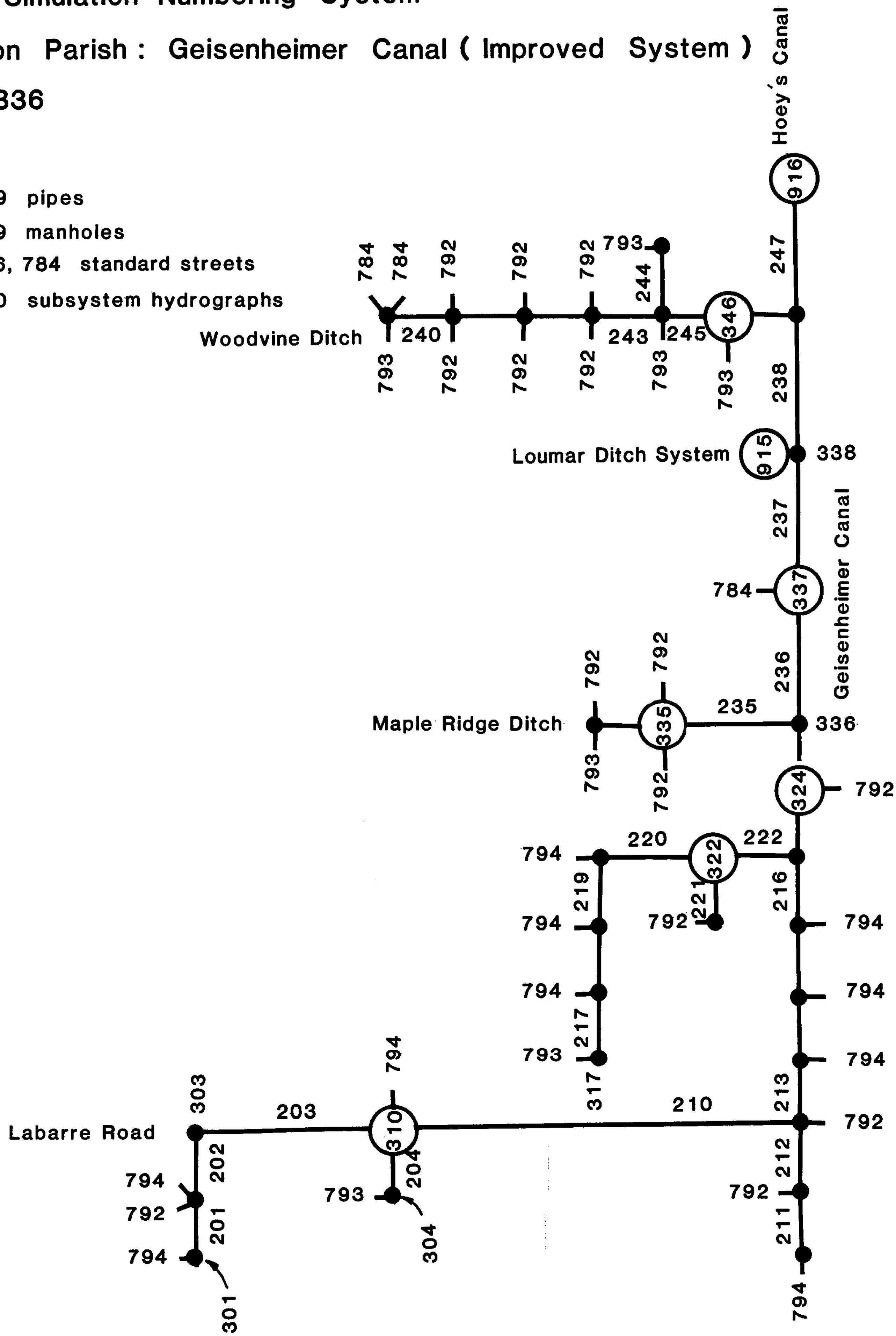
Notes :

201 - 299 pipes

301 - 399 manholes

792 - 796, 784 standard streets

901 - 920 subsystem hydrographs

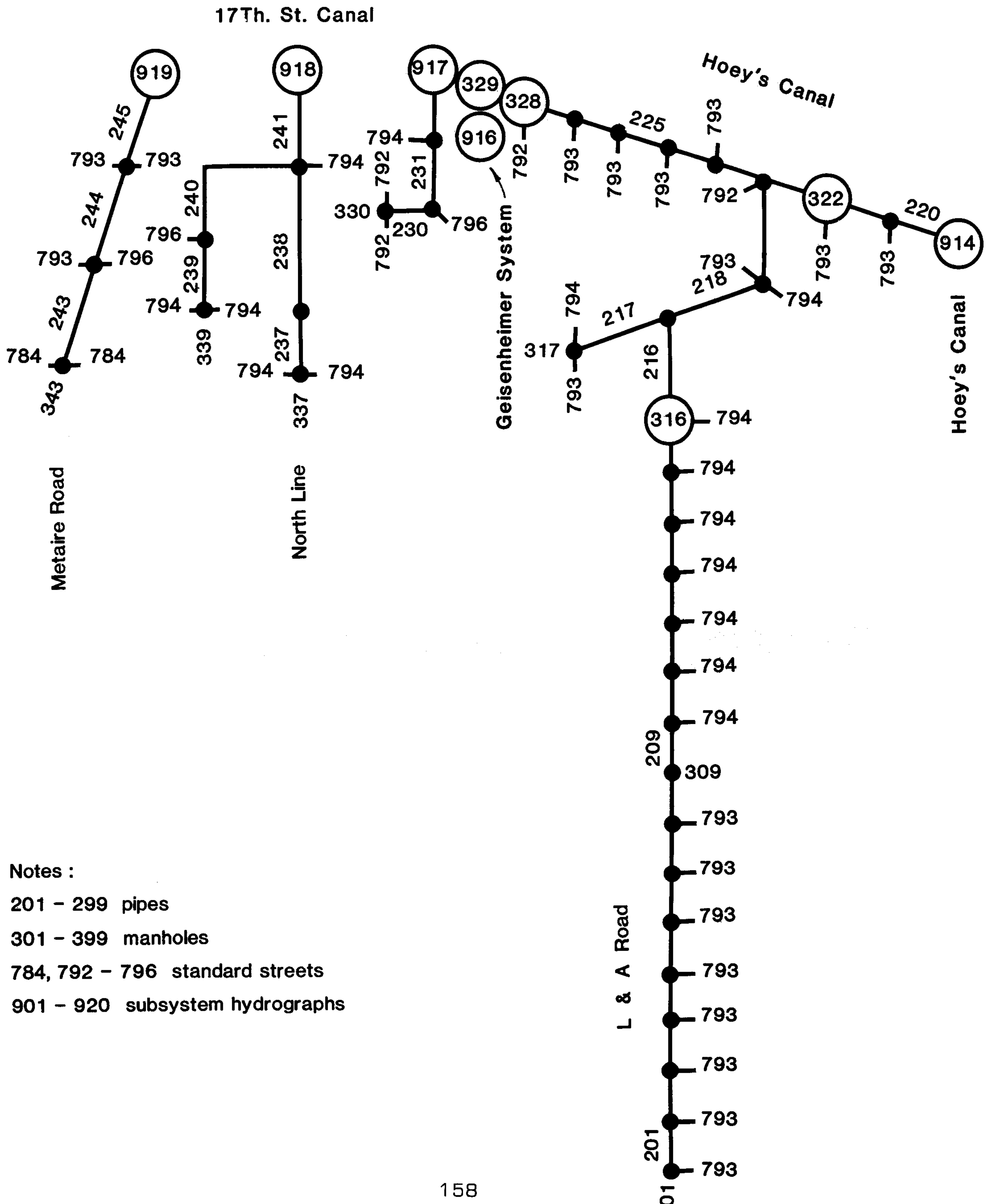


SWMM Simulation Numbering System

Jefferson Parish : Hoey's Canal (Part 3)

(Existing and Improved System)

Figure B37



Notes :

201 - 299 pipes

301 - 399 manholes

784, 792 - 796 standard streets

901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Lowerline (Part 1)

(Existing And Improved System)

Figure B38

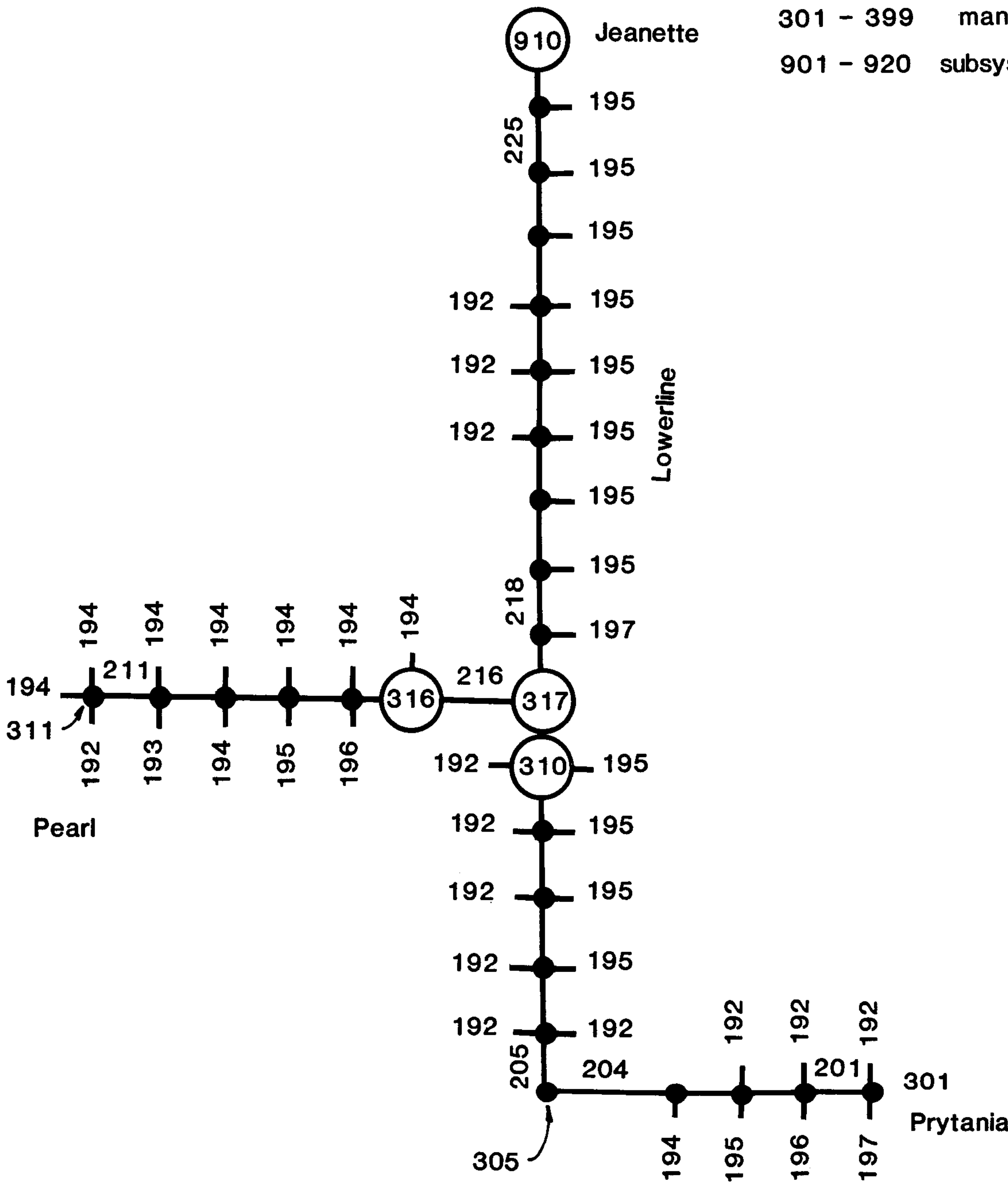
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

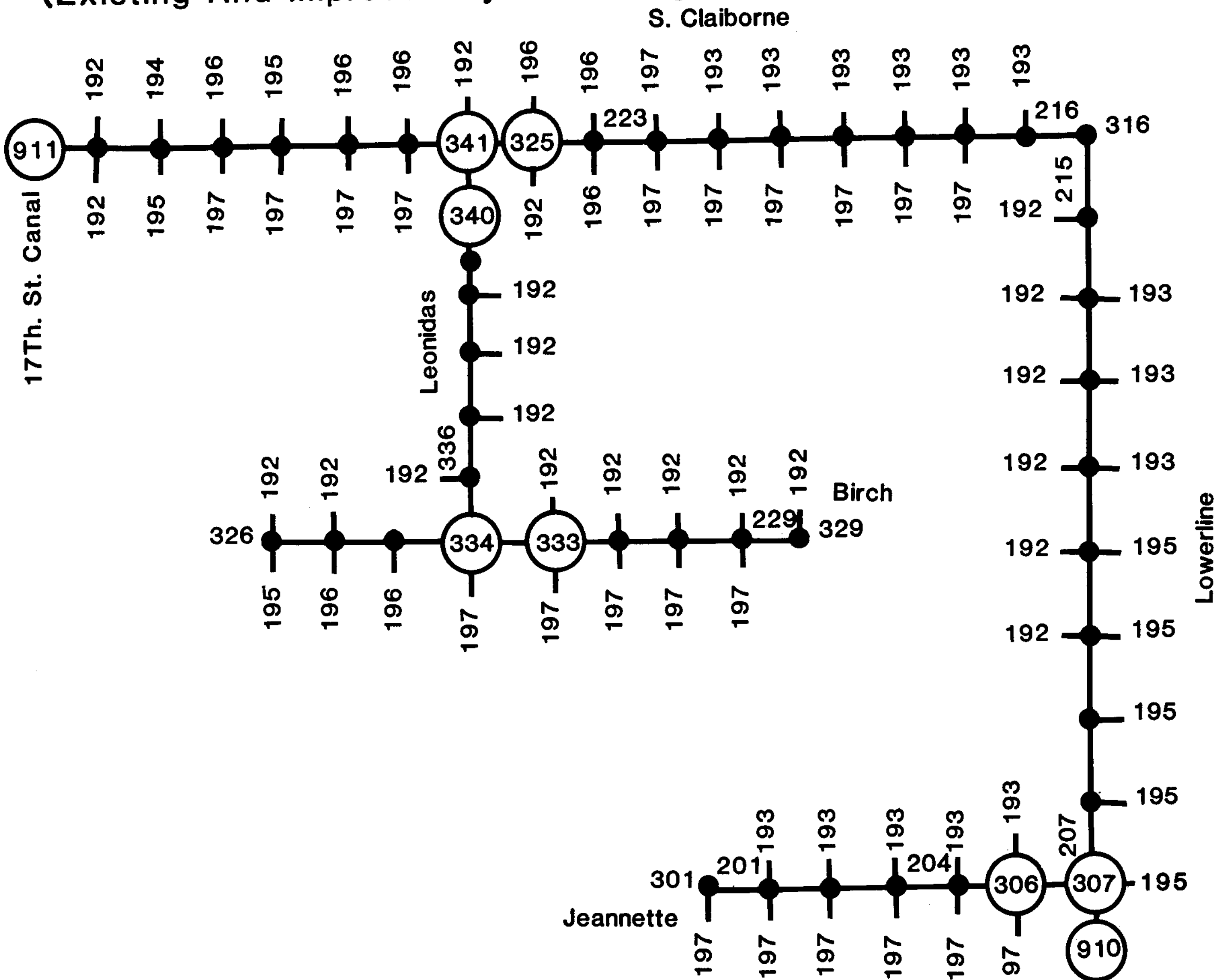
901 - 920 subsystem hydrographs



SWMM Simulation Numbering System

Orleans Parish : Lowerline (Part 2)

(Existing And Improved System) Figure B39



Notes:

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs

SWMM Simulation Numbering System

Orleans Parish : Oleander

(Existing And Improved System) Figure B40

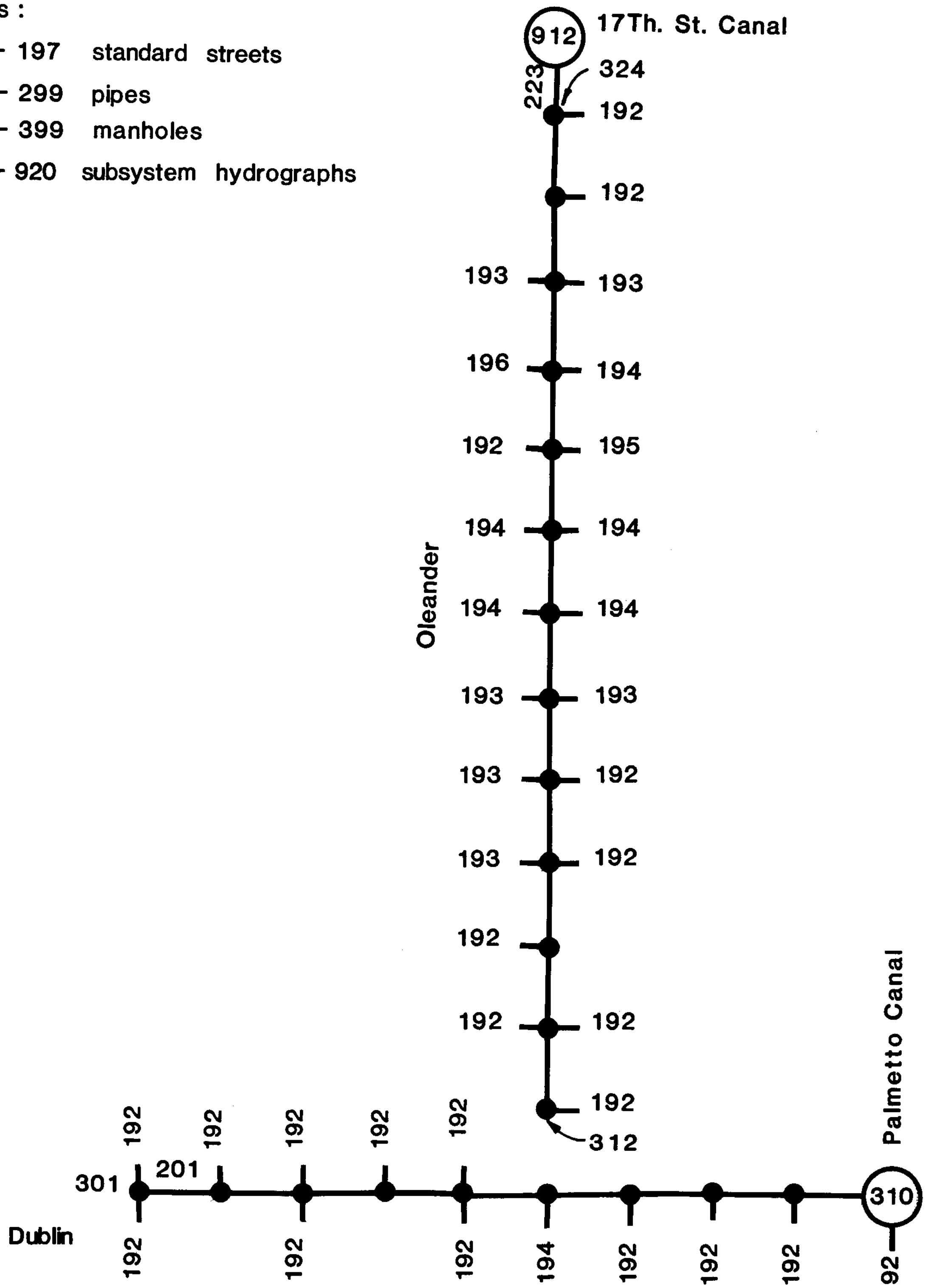
Notes :

192 - 197 standard streets

201 - 299 pipes

301 - 399 manholes

901 - 920 subsystem hydrographs

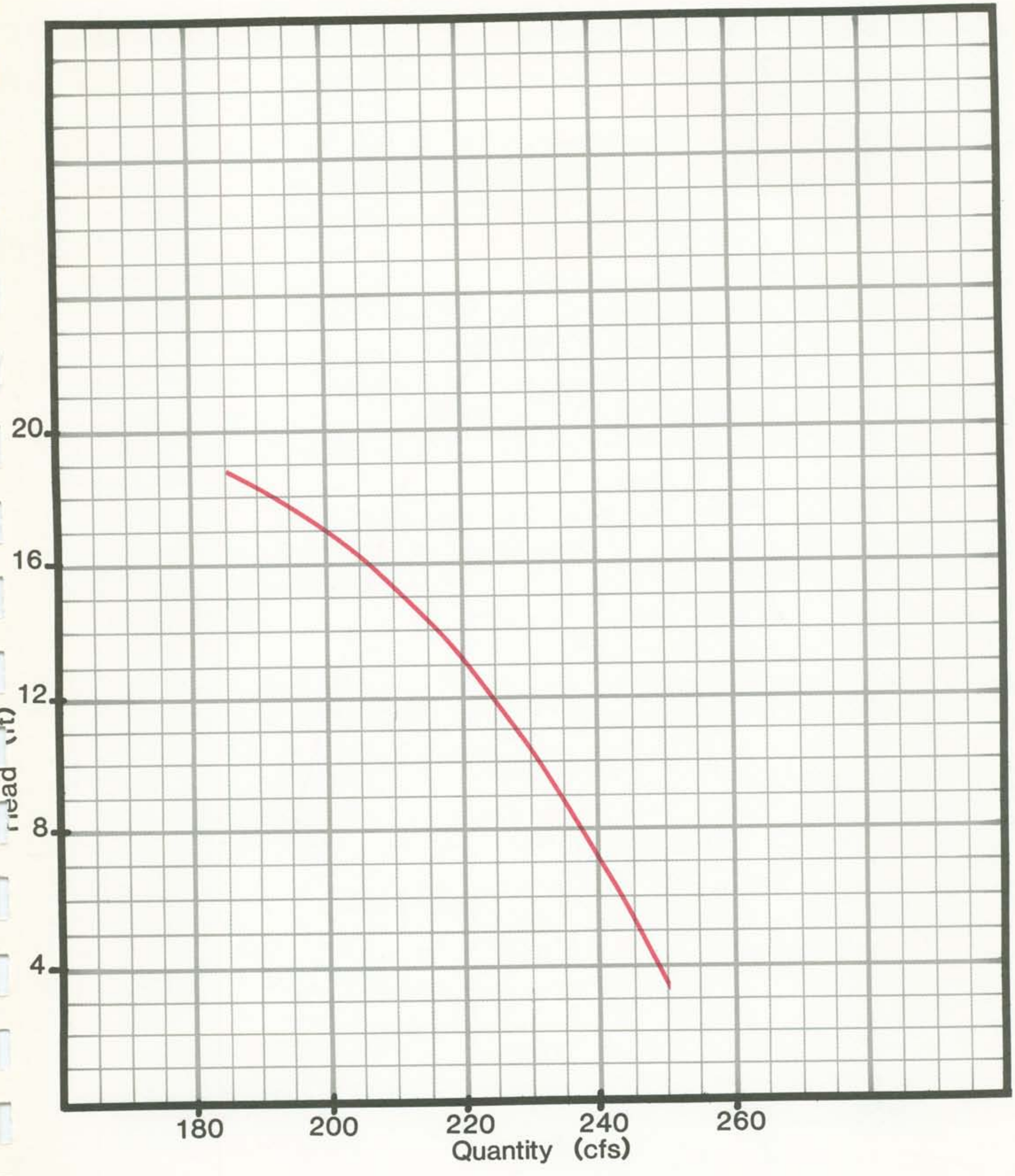


APPENDIX C

PUMP CURVES

Pump curves for the individual pumps located in Pumping Stations Nos. 1 and 6 are included in this appendix. The following curves are shown:

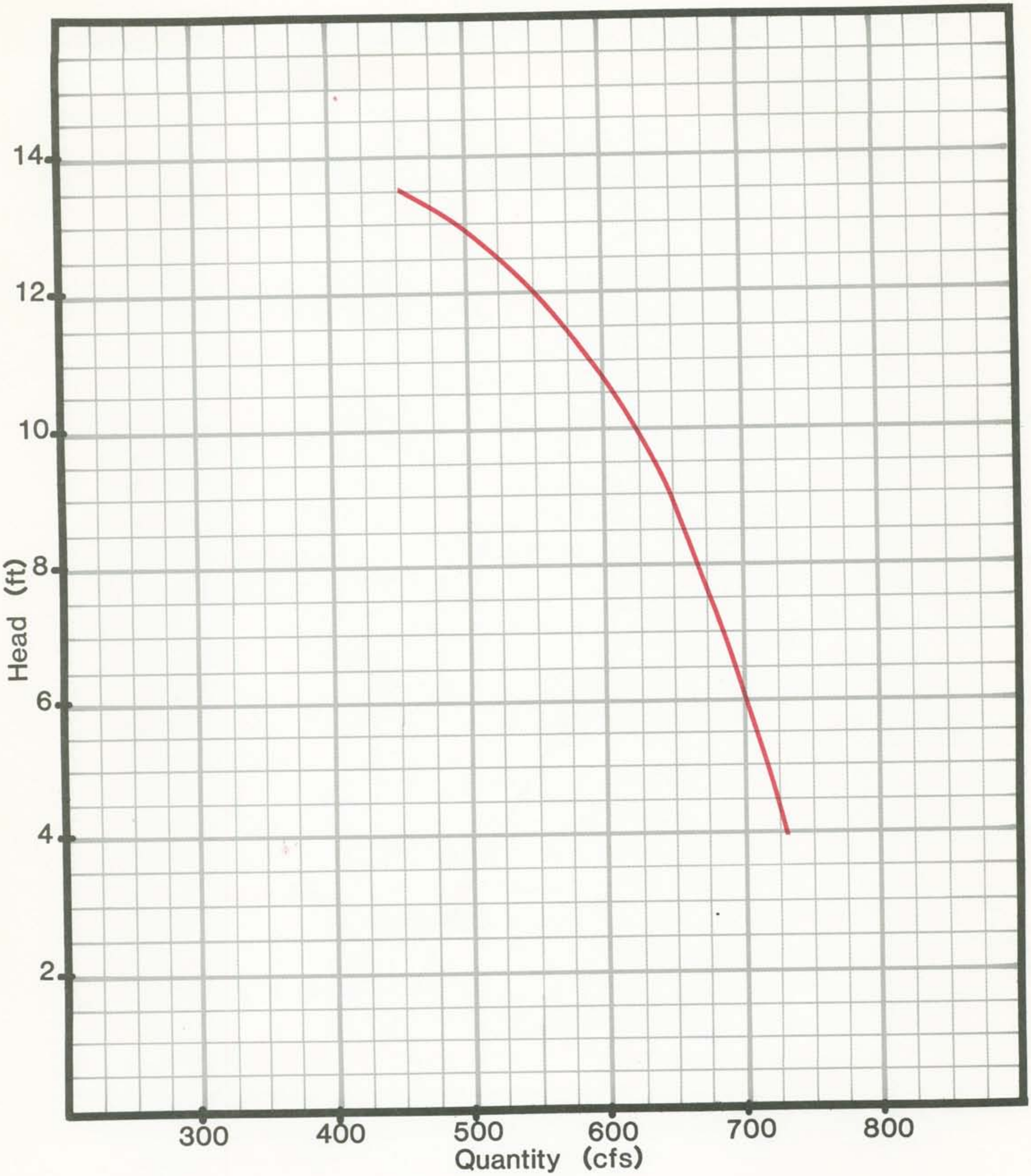
PUMP	*FIGURE*	*INVENTORY*	
		STATION 1	*STATION 6*
Vertical	C-1	V_1 & V_2	
12' Wood Screw	C-2	A & B	A & B
14' Wood Screw	C-3	C, D & E	C, D, E & F
12' Screw	C-4		G



Performance Curve
Vertical Pumps

Figure C1

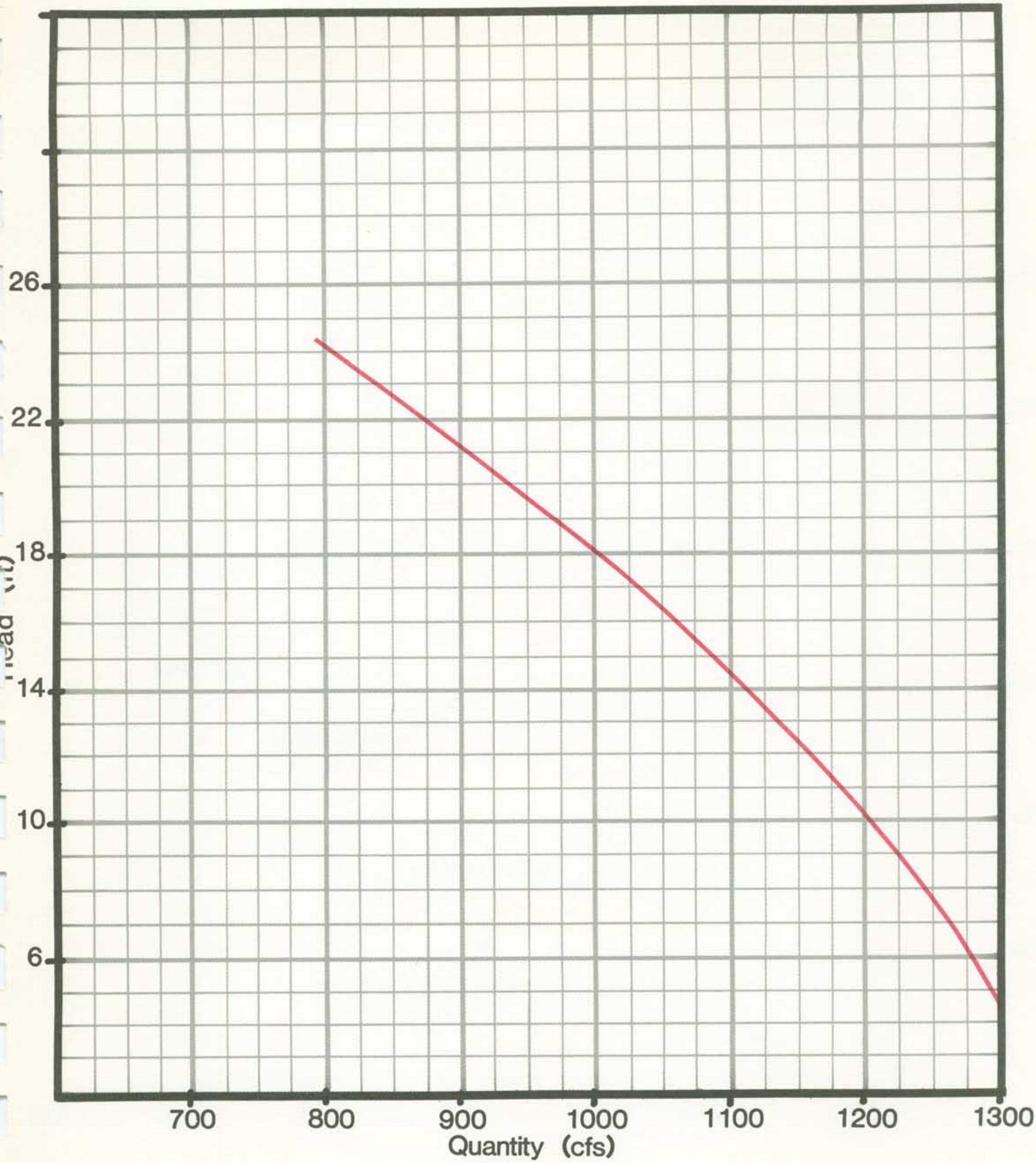
Seventeenth Street Canal Drainage Basin Study



Performance Curve
12 Ft. Wood Screw Pump
83.3 RPM

Figure C2

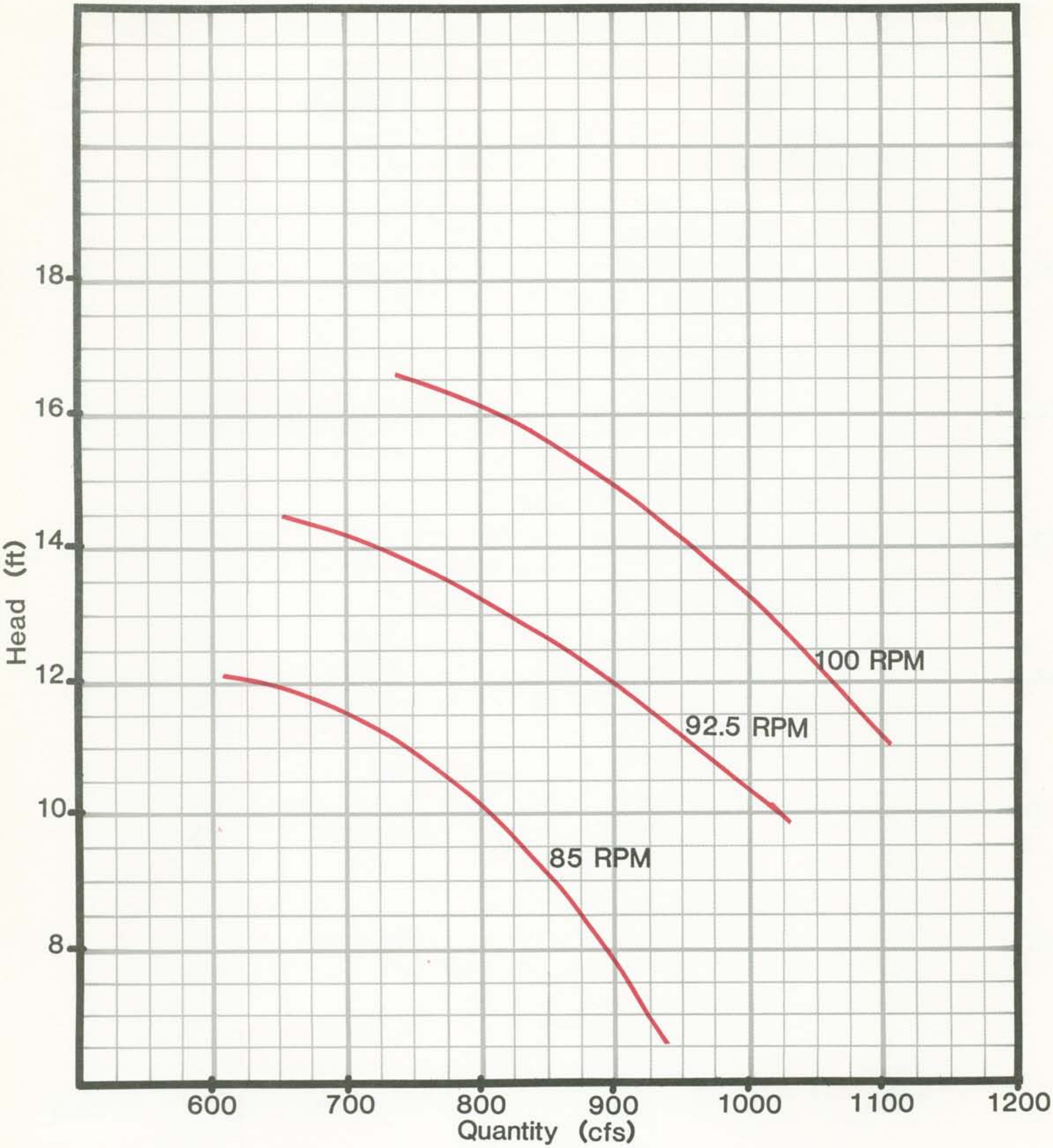
Seventeenth Street Canal Drainage Basin Study



Performance Curve
14 Ft. Wood Screw Pump
93.8 RPM

Figure C3

Seventeenth Street Canal Drainage Basin Study



Performance Curve
Pump G

Figure C4
Seventeenth Street Canal Drainage Basin Study

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