CITY OF SLIDELL

MASTER DRAINAGE PLAN

Task Order No. 9

Plan of Action

Prepared by:

BURK-KLEINPETER, INC.
HARTMAN ENGINEERING, INC.

May, 1994

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PREFACE

EXECUTIVE SUMMARY

CITY OF SLIDELL
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EXECUTIVE SUMMARY

Measures to alleviate residential and street flooding in the City of Slidell have been proposed in the Plan of Action hereby submitted to the City Engineer. These include the following items:

1.) Widening and clearing the West Diversion Canal;
2.) Installing additional pumping capacity at the City Barn Pumping Station;
3.) Installing additional pumping capacity at the Delwood Pumping Station;
4.) Installing a trunk line at Southpark Drive in the Delwood Subdivision;
5.) Installing pumping capacity at the W-14 Canal outfall;
6.) Evaluating previous plans for W-14 Canal crossings at Gause Boulevard and Robert Road.

PROJECT SUMMARIES

WEST DIVERSION CANAL

The West Diversion Canal is designed to divert flows from the overburdened W-14 Canal to Bayou Vincent. The West Canal's capacity has been greatly reduced through the years due to brush overgrowth and accumulation of debris. Proposed canal improvements include clearing the right-of-way, excavating the canal bottom, grading the side slopes, seeding the banks, and modifying the roadway crossing at Carnation Road. Proposed modifications will increase the West Canal's current capacity of 255 cubic-feet-per-second (cfs) to 500 cfs.

If surveys show an inadequate right-of-way to maintain the above-mentioned improvements, an alternate was developed using a limestone-filled geo-cell material on the canal banks. This design allows for steeper canal side slopes in order to fit within the existing right-of-way.

As a result of the West Canal flow increase, downstream improvements are required on small sections of Bayou Vincent and Bayou Bonfouca, the receiving waterbodies of the West Canal, in order to meet the new capacity demands. Further downstream, the existing Bayou Bonfouca section is sufficient to render these increased flows negligible. The net result is the transfer of potential flood waters from the W-14 Canal to the much larger Bayou Bonfouca.

CITY BARN PUMPING STATION

Increased urbanization and land development in the City of Slidell have produced increasing volumes of storm water runoff, and the present pumping capacity of 267 cfs at the City Barn Station is now inadequate. The installation of an additional 134-cfs pump will help alleviate flooding in the area by increasing the total pumping capacity to 400 cfs. In addition, capacity improvements are required in Bayou Pattasat, the canal serving the pumping station, including clearing the canal right-of-way, dredging the canal bottom, and lining the crossing at the New Orleans & Northeastern Railroad. Downstream, the section of Bayou Bonfouca which will receive the pumping station discharge is adequate to handle the increased flow without any improvements necessary. Increasing the capacity of the pumping station, followed by increasing the capacity of Bayou Pattasat, will allow more storm water to be pumped out of the area at a faster rate, resulting in alleviation of flooding problems in the area.

DELWOOD PUMPING STATION

For the reasons mentioned above, the existing pumping capacity of 156 cfs at the Delwood Station is inadequate. The installation of an additional 56-cfs pump will help alleviate flooding in the area by increasing the total pumping capacity to 212 cfs. Again, no improvements are required downstream in Bayou Bonfouca. Installation of the Southpark Trunk Line, described below, will assist flood control in conjunction with this project.

SOUTHPARK TRUNK LINE

An inadequate drainage system in the northern section of the Delwood Subdivision produces residential and street flooding. Proposed improvements include installation of a reinforced concrete pipe (RCP) trunk line along portions of Nassau Drive and Brookwood Drive, and along the length of Southwood Drive and Southpark Drive, as well as improving the capacity of the existing Front Street collector ditch. These improvements should be carried out following implementation of the Delwood Pumping Station addition, as described above. The sum of these two projects will alleviate storm water flooding in the Delwood, Salmen, and Lakeshore Village subdivisions, while not inducing any further downstream flooding in the much larger Bayou Bonfouca basin.

W-14 CANAL OUTFALL

High water stages in Lake Pontchartrain are currently creating a backflow in the W-14, combining with inadequate canal sections to cause water to overflow the canal banks. Proposed modifications include installing a pumping station in the W-14 Canal south of the King's Point Subdivision just past Voters Road, and constructing a ring levee around it equipped with automatic drainage gate structures to prevent intrusion of lake waters into the canal. Because of the large flows through the W-14 Canal, the project should be constructed in stages: the first stage includes construction of the pumping station to handle the existing W-14 Canal capacity of 1650 cfs; and future stages (not included in the scope of this project) would involve increasing the pumping station capacity to 4000 cfs, then modifying the capacity of the W-14 Canal accordingly.

W-14 CANAL at GAUSE BOULEVARD and ROBERT ROAD

W-14 Canal crossings at Gause Boulevard and Robert Road are currently "bottleneck" areas which encourage sedimentation and erosion, restricting the free flow of water through the canal. Proposals for each section were previously submitted by Cortech, Inc., and those proposals were reviewed and analyzed. Cortech, Inc. proposed full canal enclosure at each section, since the right-of-way widths at each location are inadequate to maintain an earthen canal section. While we agree with this assessment of the existing conditions, we note some differences with Cortech's proposed solution.

We propose slope paving the inlet and outlet of the culverts at each location to stabilize the banks, as well as installing side discharge control structures on the upstream end of each crossing to provide a transition to the box culverts. If additional right-of-way can not be obtained in the area, we propose instead a full canal enclosure at each crossing with Con / Span culverts. Each of the above alternates is based on supplying the full W-14 Canal capacity demand from a 10 Year-24 Hour Design Storm Event.

PROJECT COSTS

Project	**Total Cost**
WEST DIVERSION CANAL	
Alternate A: Natural canal embankments	$337,700
Alternate B: Geo-Cell material embankments	$614,700
CITY BARN PUMPING STATION	$1,388,600
DELWOOD PUMPING STATION	$653,700
SOUTHPARK TRUNK LINE	
Alternate A: Front Street open ditch	$658,700
Alternate B: Front Street RCP enclosure	$1,098,000
W-14 CANAL OUTFALL	$14,819,100
W-14 CANAL at GAUSE BLVD. and ROBERT RD.	
Alternate A: Slope paving and weir installation	$624,800
Alternate B: Con / Span Culverts	$1,403,500

SECTION I

WEST DIVERSION CANAL

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I. Introduction

Extensive street flooding and limited residential flooding associated with storm water runoff overflow caused by inadequate capacity of the W-14 Canal is a major concern for residents of the north Slidell area and the City of Slidell. In order to address the inadequacy of the W-14 Canal we are proposing to increase the flow capacity of the West Diversion Canal. We conclude that an increase in flows through the West Diversion Canal will reduce flows in the W-14 Canal and thereby reduce residence and street flooding in the vicinity of the W-14 Canal.

The existing West Diversion Canal at Highway 11 and North Village is a diversion canal in north Slidell which currently diverts a portion of the flow from the W-14 Canal into Bayou Vincent. Flows through this diversion canal travel through approximately 5000 linear feet of open channel with three drainage structures at roadway crossings along the way and two railroad trestle bridges.

II. Project Description

This project consists of the evaluation of the West Diversion Canal's existing capacity and condition as well as development of feasible alternatives to improve the diversion canal's capacity and address maintenance-related difficulties.

After feasible alternatives were developed, a conceptual design of the project was prepared along with an estimated design and construction cost and a project schedule.

In addition, this report addresses any potential environmental considerations which are anticipated to be encountered during the implementation of the proposed improvements.

III. Existing Canal Condition

The West Diversion Canal is a natural stream through mainly wooded areas of the North Slidell area. According to the Slidell Engineering Department Records, the existing stream was scheduled to be enlarged and straightened in 1985. The planned improvements included reshaping the existing stream to have a ten foot (10') bottom width and 2 (horizontal) to 1 (vertical) side slopes. The planned improvements indicated that the approximate canal depth was to be five feet and the canal longitudinal slope was to be approximately 0.001 feet/foot. Additionally, the 1985 plans called for replacement of a 72" diameter corrugated metal pipe (CMP) at Carnation Road with a 12' X 6' Reinforced Concrete Box Culvert (RCB).

Through interviews with City of Slidell personnel and site investigations, it has been determined that partial implementation of the 1985 plan has occurred. At the time of the site visit, we estimated that the canal had an average cross sectional shape of a ten foot (10') bottom width with approximately 1 (horizontal) to 1 (vertical) side slopes. In addition, the side slopes of the canal are overgrown with trees and brush which obviously restrict flows during high discharge periods. Also, limbs and debris restrict the box culvert road crossings and railroad trestle bridges during these high flow periods.

The topography of the canal area restricted access to much of the canal. Therefore, significant restrictions to the actual capacity which could not be detected may also be present along the canal flow line.

IV. Proposed Improvements

The proposed improvements to the West Diversion Canal include clearing the canal right-of-way of all trees and brush which may restrict flows, excavation of the canal bottom to a twelve foot (12') bottom width, grading the canal side slopes to 2 (horizontal) to 1 (vertical), and seeding the canal banks to stabilize the banks from erosion and allow for maintenance mowing of the canal right-of-way. The proposed improvements also include replacing the 72" CMP at Carnation Road with a 3-cell 4' X 6' RCB crossing. In addition, during the design phase of implementation of the plan, the canal invert elevation should be reviewed to determine if acquisition of additional right-of-way is necessary to provide adequate access for maintenance of the canal.

As an alternate to the earthen canal section, a low maintenance, stabilized bank open canal could be provided if necessary to stay within the existing canal right-of-way. This stabilized bank open canal would be constructed using a limestone filled geo-cell material placed on canal banks at up to a 1 horizontal to 1 vertical slope. The bottom width of the canal would be adjusted to provide an equivalent cross section and conveyance capacity to the earthen canal section while requiring less right-of-way and less maintenance of the canal banks.

V. Conceptual Design

The conceptual design of the proposed improvements would be for the canal to have a twelve foot bottom width and 2-to-1 side slopes with a longitudinal slope of 0.001 feet/foot. The cross section would be maintained throughout the length of the diversion canal without substantial deviation at any reach of the canal.

The cross section would be maintained at the two trestle bridge railroad crossings as much as is practical without disturbing, undermining, or otherwise affecting the bridges.

The 3-cell 4' X 6' RCB crossing at Carnation Road could be either precast or poured in place. We anticipate the required length of this RCB crossing to be approximately 40 feet in length.

For the conceptual design of the proposed improvements the headwater and tailwater canal stages have been interpolated from values given in the 1983

Slidell Master Drainage Plan. These values, along with other design criteria, are listed in Table 1.

A flow analysis of the existing canal was performed using the "Advanced ICPR" computer program developed by Streamline Technologies. The design parameters shown in Table 1 were used for all the analyses. For the computer runs the headwater and tailwater stages shown in Table 1 were set to remain constant and the maximum canal capacity for each of the alternative improvements evaluated was determined. The results of these determinations are shown in Table 2.

Table 2 indicates that by implementing the proposed improvements the capacity of the West Diversion Canal could be approximately doubled from the existing estimated capacity of 255 cubic feet per second (cfs) to approximately 500 cfs. This analysis is based on the water surface profile developed by the headwater and tailwater elevations shown in Table 1. Additional analysis performed indicated that, by implementing the proposed improvements to the West Diversion Canal, the improved canal would have the capacity to convey the existing flow of 255 cfs with a decrease in the water surface profile of approximately three feet. This decrease in water surface profile corresponds to lowering the headwater at the start of the diversion canal from the existing 12.41' National Geodetic Vertical Datum (N.G.V.D.) to 9.41' N.G.V.D.

Bayou Vincent is the receiving waterbody of the West Diversion Canal, near Cypress Street and its intersection with Bayou Bonfouca. According to the

1983 MDP, the proposed increase in flow through the West Canal will overburden a small stretch of the drainage network from Bayou Vincent at the West Canal Outfall to Bayou Bonfouca past the Highway 190 crossing. Calculations (see Appendix A) show that the section of Bayou Vincent downstream of the West Canal is required to have a capacity of 700 cfs, and the section of Bayou Bonfouca near Highway 190 is required to have a capacity of over 1100 cfs. Figures 5 and 6 show the proposed improvements to these sections. The hydraulic grade line was kept as close as possible to existing conditions throughout hydraulic analysis.

Thus, the removal of approximately 250 cfs of runoff from the W-14 Canal basin will significantly reduce residential and street flooding in that basin while the addition of this runoff to the improved Bayou Vincent/Bayou Bonfouca basin should have no additional impact to this much larger basin.

VI. Environmental Consideration

In the design phase of this project, there will be several environmental considerations to be addressed which go beyond the scope of this preliminary Plan of Action. A listing of several of these considerations follows:

> Initially the area affected by this project should be reviewed by field investigation to assure that this project will not encroach upon areas classified as wetlands. If any areas affected by this project are classified as wetlands, an agreement with the Corps of Engineers

should be reached on the course of action to be taken prior to the start of construction.

Any existing storm water discharge permits should be reviewed to ensure that this project will not violate these permits.

The City of Slidell should consider contacting the Environmental Protection Agency to determine if this redirection of flows will require any additional storm water discharge permitting.

The City of Slidell should consider testing the material to be excavated from within the existing canal to ensure that this material has not been contaminated by previous discharges.

In addition to the listed concerns, the design phase of this project may uncover additional environmental liabilities which could delay the design or construction of this project.

VII. Estimated Design / Construction Cost

The estimated construction cost for the West Diversion Canal Improvement Project using an earthen section without stabilization is approximately $275,200. A breakdown of this cost estimate is outlined in the construction cost estimate presented in Table 3. The estimate is based on the unit cost given in the table and does not include design, construction administration, or resident inspection fees. For the purposes of this report, the design cost has been estimated to be 10% of construction cost and construction administration to be 4% of the construction cost. Adding $27,500 for design fees, $11,000 for construction administration, and $24,000 for resident inspection would bring the total estimated project cost to approximately $337,700.

An alternate construction cost estimate has been prepared using the low maintenance, stabilized bank open canal constructed using a limestone filled geo-cell material placed on canal banks. This cost estimate is approximately $511,100. A breakdown of this cost estimate is outlined in the construction cost estimate presented in Table 4. The estimate is based on the unit cost given in the table and does not include design, construction administration, or resident inspection fees. For the purposes of this report, the design cost has been estimated to be 10% of construction cost and construction administration to be 4% of the construction cost. Adding $51,100 for design fees, $20,500 for construction administration, and $32,000 for resident inspection would bring the total estimated project cost to approximately $614,700.

It should be noted that these cost estimates do not include the acquisition of additional property for enlarging the right-of-way nor do they include any utility relocation which may be encountered during the design phase of this project. In addition, final design may dictate that the 3,000-foot length of W-14 upstream of North Boulevard be improved up to the I-12 crossing. These improvements may require a combination of slope paving and excavation.

VIII. Estimated Design / Construction Time Schedule

The estimated design time schedule for this project is three months for the project design, which includes preparation of plans, specifications, and contract documents, two months for bidding and award of the contract, and an additional three months for construction of the base project with one additional month for the construction of the geo-cell bank stabilized project. This estimated time schedule assumes the required funding for the project to be available and does not include possible delays caused by appropriation of money to fund this project; it also does not include the acquisition of additional property for enlarging the right-of-way which may be encountered during the design phase of this project.

TABLE 1

DESIGN PARAMETERS

Parameter	Value	Source
Headwater at Diversion Canal	12.41' N.G.V.D.	Interpolated from M.D.P.
Tailwater at Diversion Canal	5.35' N.G.V.D.	Interpolated from M.D.P.
Longitudinal Slope of Canal	0.001 ft/ft	1985 planned improvements
Upstream Invert Elevation	5.06' N.G.V.D.	U.S.G.S. Maps
Downstream Invert Elevation	0.0' N.G.V.D.	U.S.G.S. Maps
Manning's "N" (existing canal)	0.050	Design Manual
Manning's "N" (improved canal)	0.032	Design Manual
Manning's "N" (concrete)	0.013	Design Manual
Restriction @ Rail Crossings	Negligible	Assumed

TABLE 2

ALTERNATE IMPROVEMENTS EVALUATED

Alternative	Description of Work / Canal Description	Canal Capacity
1.	Existing Diversion Canal with maintenance cleaning only (10' canal bottom width with 1(H) to 1(V) side slopes and no restrictions caused by debris in canal)	255 cfs
2.	Existing Diversion Canal with crossing improvement at Carnation Road and maintenance cleaning of canal (10' canal bottom width with 1(H) to 1(V) side slopes and no restrictions caused by debris in canal)	290 cfs
3.	Improved Diversion Canal cross section, clearing & grubbing entire right-of-way, and maintenance cleaning of road crossings only (12' canal bottom width with 2(H) to 1(V) side slopes and no restrictions caused by debris in canal)	345 cfs
*4.	Improved Diversion Canal cross section, clearing & grubbing entire right-of-way, crossing improvement at Carnation Road and maintenance cleaning of the canal (12' canal bottom width with 2(H) to 1(V) side slopes and no restrictions caused by debris in canal)	500 cfs

* Recommended Alternative

TABLE 3

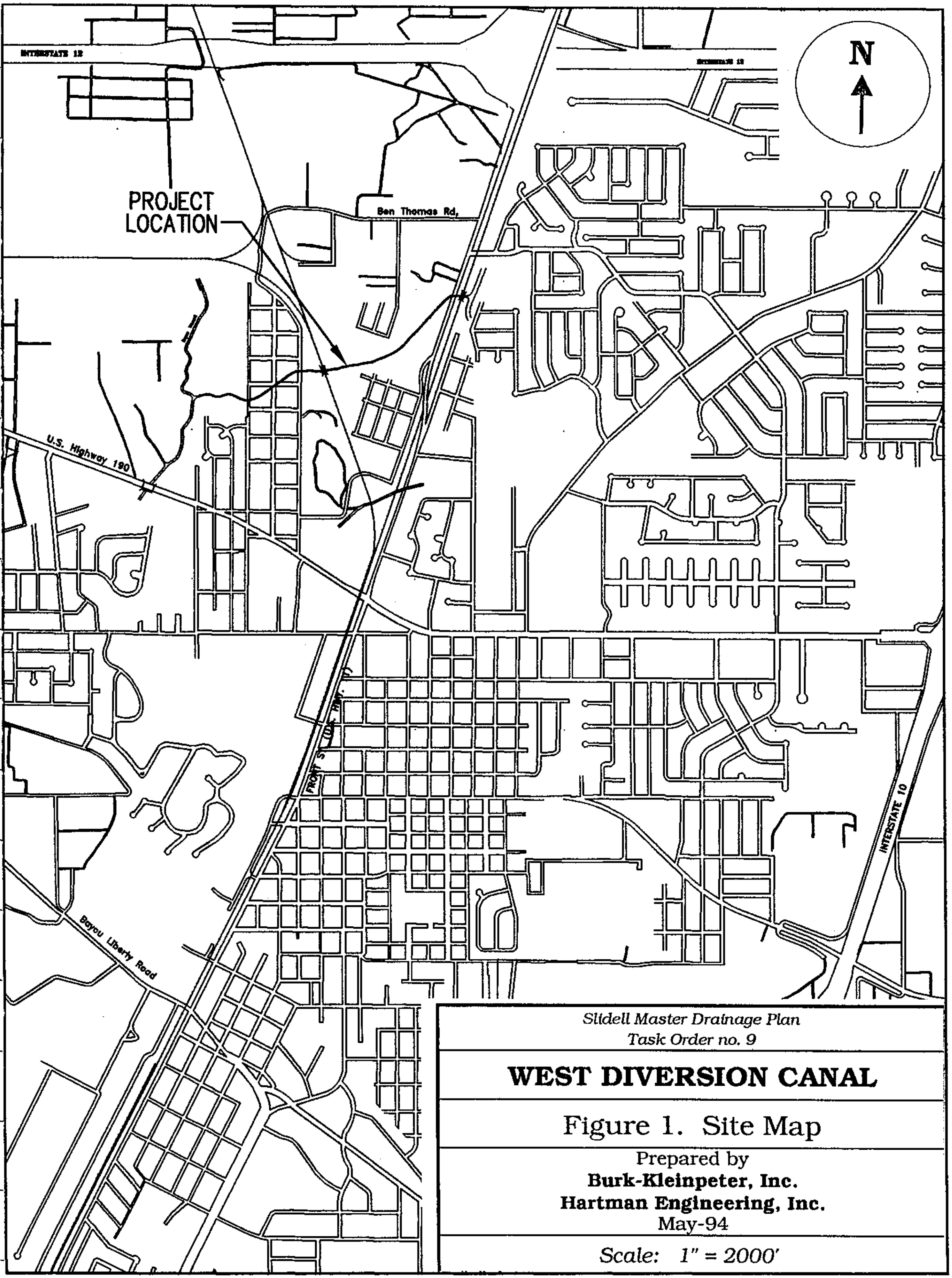
ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump Sum	1	Lump Sum	$13,000
Clearing and Grubbing	Acre	14	$2,500	$35,000
Excavation	Cubic Yard	11550	$10	$115,500
3-cell 4' X 6' RCB	Linear Ft	40	$1,250	$50,000
Roadway Removal & Replacement	Sq. Yard	180	$35	$6,300
Crossing Guardrails	Linear Ft	50	$50	$2,500
Seeding	Acre	14	$500	$7,000
SUBTOTAL				$229,300
CONTINGENCY @ 20%				$45,900
TOTAL				$275,200

TABLE 4

ESTIMATED DESIGN / CONSTRUCTION COST
GEO-CELL STABILIZED CANAL BANK ALTERNATE

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump Sum	1	Lump Sum	$24,100
Clearing and Grubbing	Acre	14	$2,500	$35,000
Excavation	Cubic Yard	11550	$10	$115,500
Geo-Cell Fabric	Sq. Yard	13500	$10	$135,000
Geo-Cell Fill (Limestone)	Cubic Yard	1500	$35	$52,500
3-cell 4' X 6' RCB	Linear Ft	40	$1,250	$50,000
Roadway Removal & Replacement	Sq. Yard	180	$35	$6,300
Crossing Guardrails	Linear Ft	50	$50	$2,500
Seeding	Acre	10	$500	$5,000
SUBTOTAL				$425,900
CONTINGENCY @ 20%				$85,200
TOTAL				$511,100



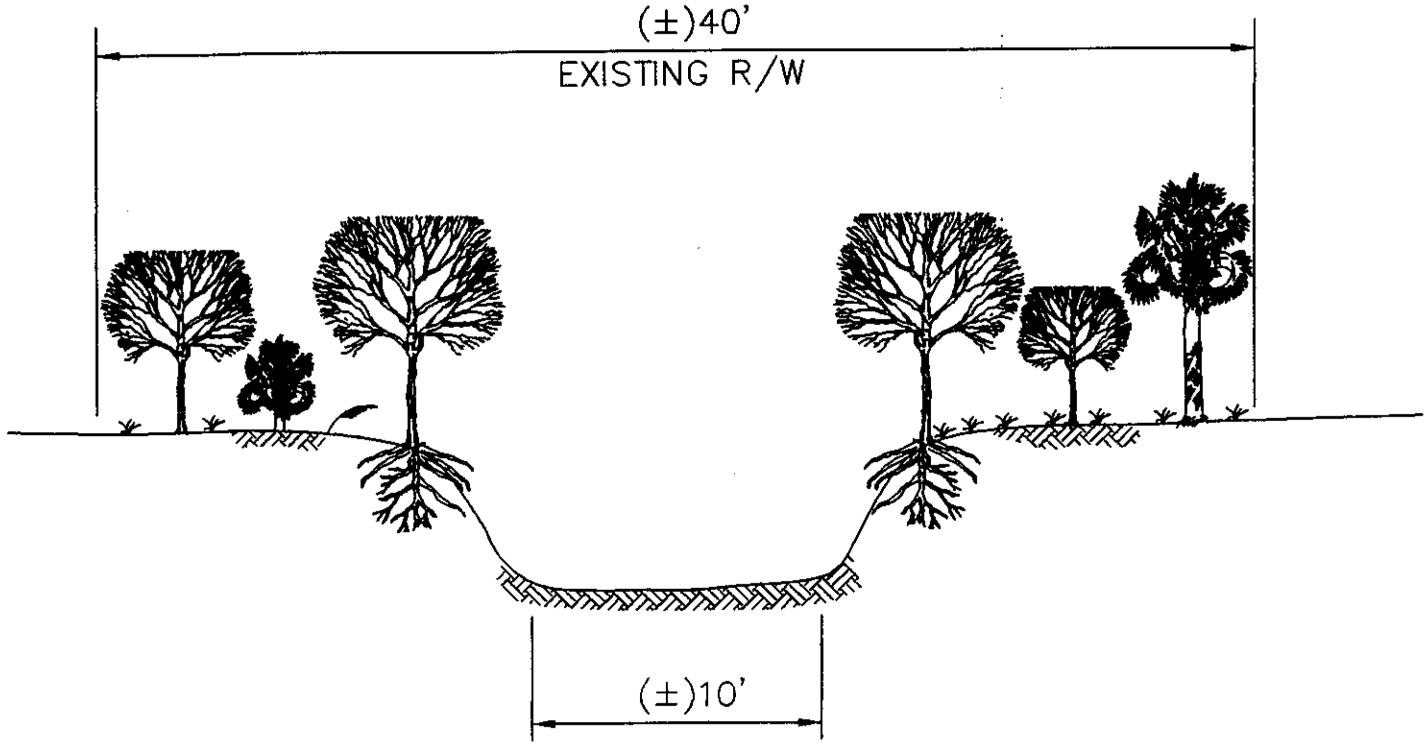
Slidell Master Drainage Plan
Task Order no. 9

WEST DIVERSION CANAL

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



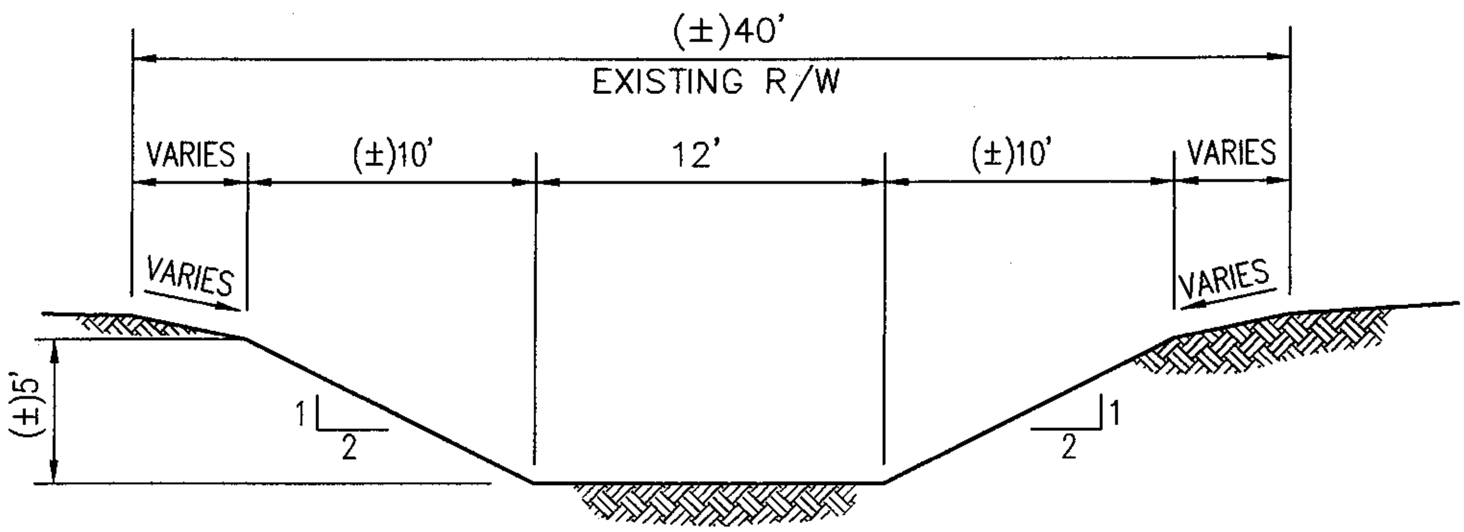
Slidell Master Drainage Plan
Task Order no. 9

WEST DIVERSION CANAL

Figure 2. Existing Canal Cross-Section

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Not to Scale



Slidell Master Drainage Plan
Task Order no. 9

WEST DIVERSION CANAL

Figure 3. Proposed Canal Cross Section

Prepared by
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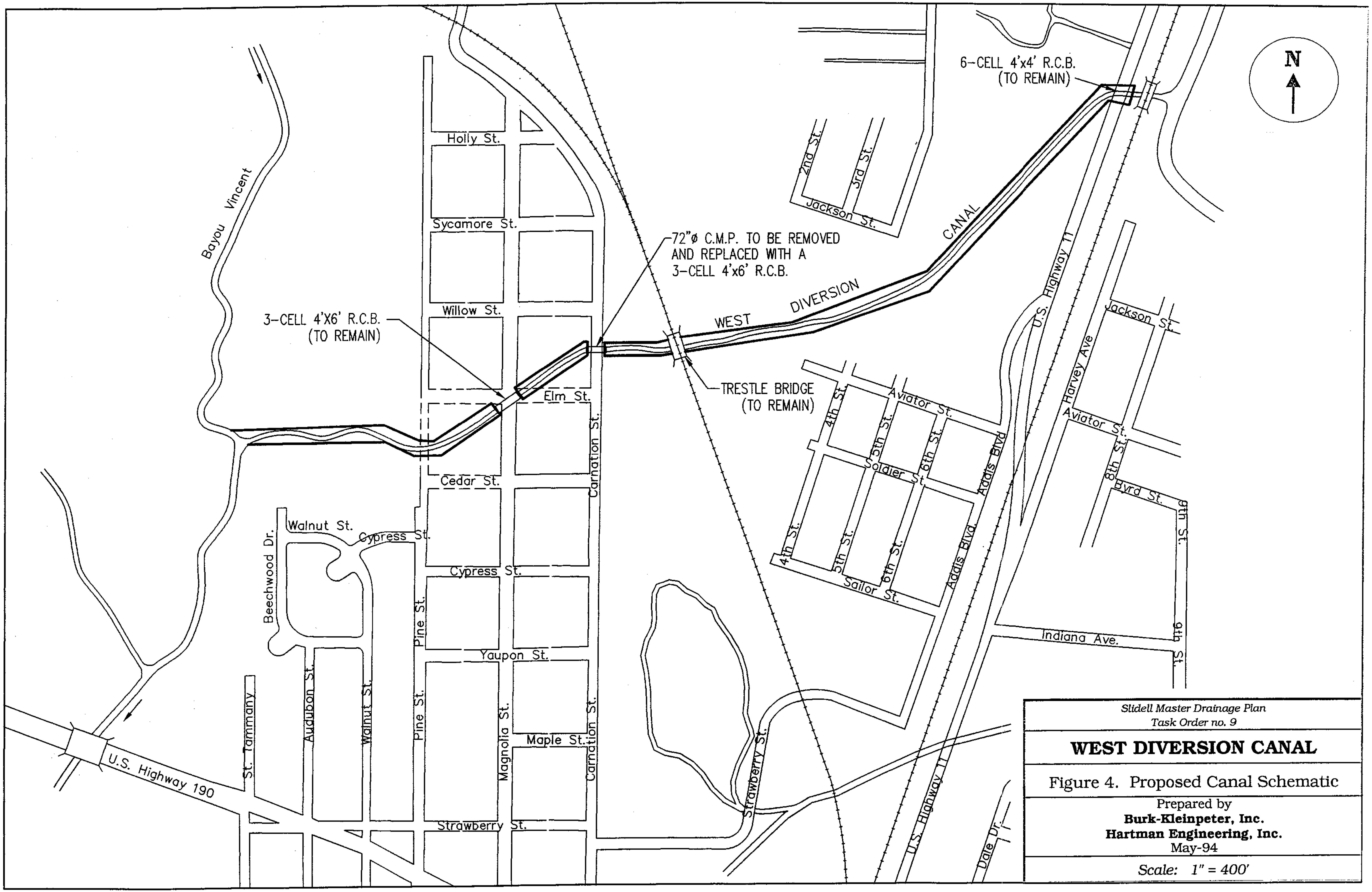
May-94

Not to Scale



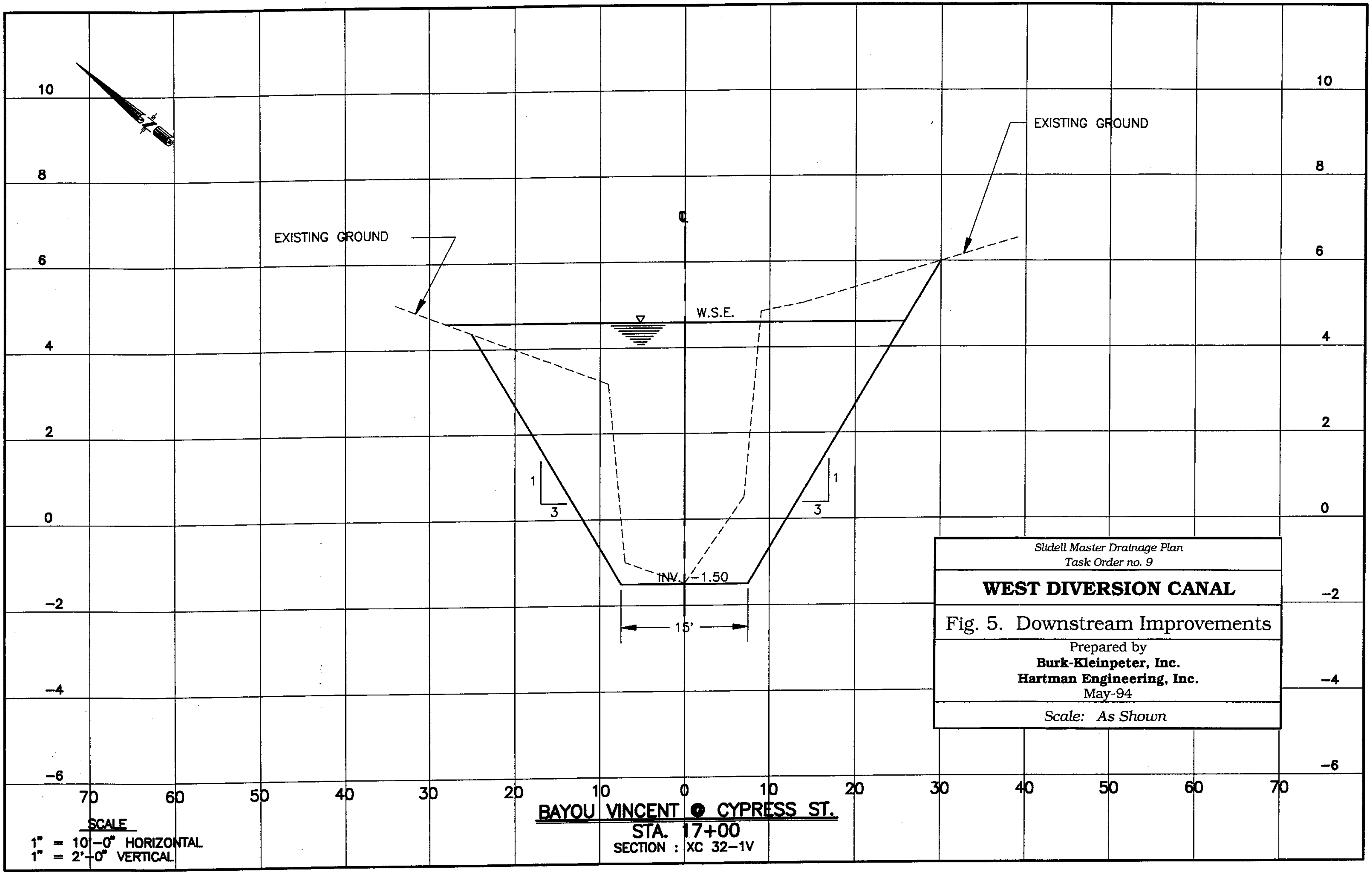
Slidell Master Drainage Plan
Task Order no. 9

WEST DIVERSION CANAL

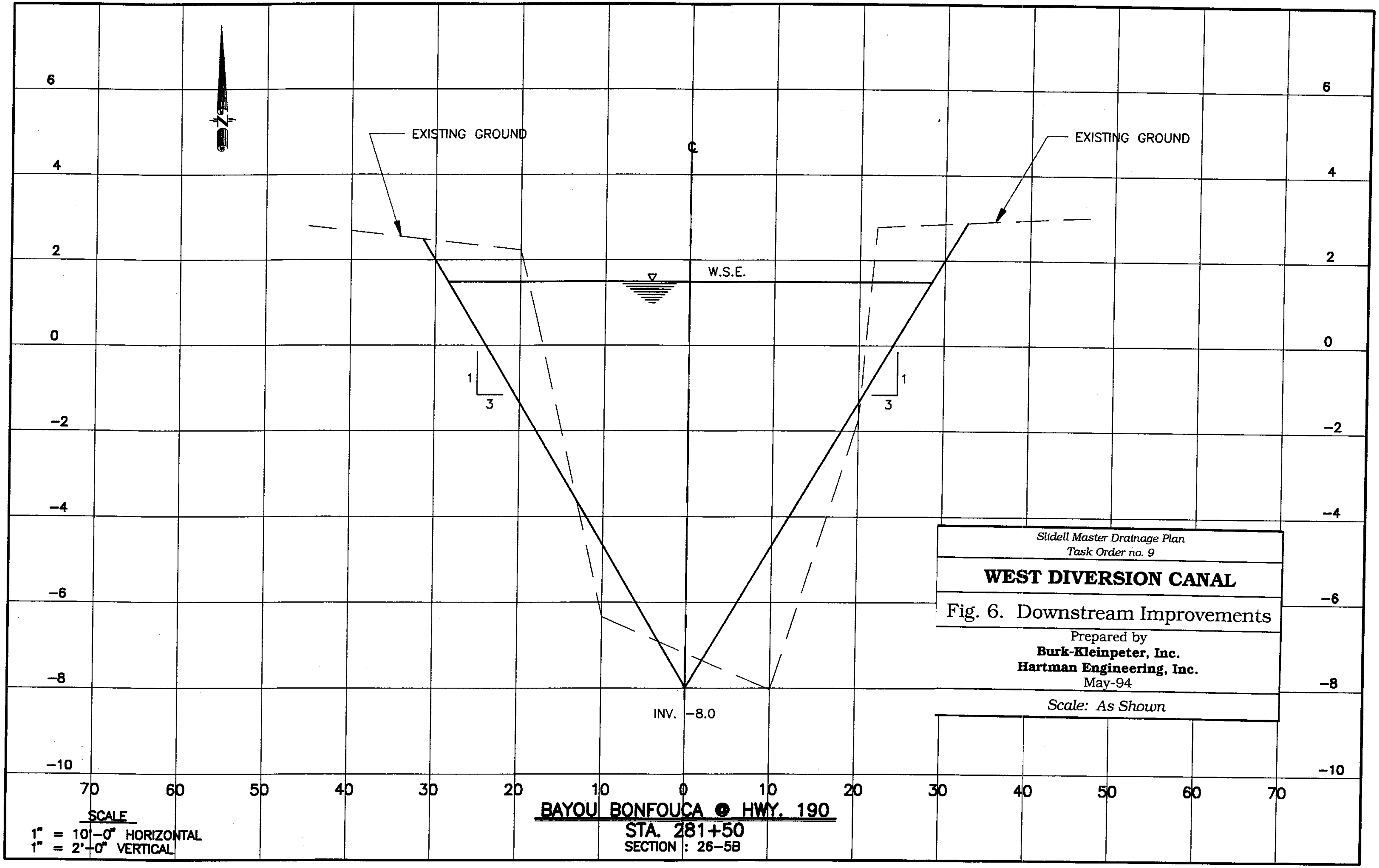
Figure 4. Proposed Canal Schematic

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 400'



EXISTING GROUND
EXISTING GROUND
W.S.E.
INV. −1.50
15'
1
3
1
3
10
8
6
4
2
0
−2
−4
−6
70
60
50
40
30
20
10
0
10
20
30
40
50
60
70
BAYOU VINCENT @ CYPRESS ST.
STA. 17+00
SECTION : XC 32−1V
SCALE
1" = 10'−0" HORIZONTAL
1" = 2'−0" VERTICAL
Slidell Master Drainage Plan
Task Order no. 9
WEST DIVERSION CANAL
Fig. 5. Downstream Improvements
Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94
Scale: As Shown


EXISTING GROUND
EXISTING GROUND
℄
W.S.E.
1
3
1
3
INV. −8.0
6
4
2
0
−2
−4
−6
−8
−10
70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
SCALE
1" = 10'−0" HORIZONTAL
1" = 2'−0" VERTICAL
BAYOU BONFOUCA @ HWY. 190
STA. 281+50
SECTION : 26−5B
Slidell Master Drainage Plan
Task Order no. 9
WEST DIVERSION CANAL
Fig. 6. Downstream Improvements
Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94
Scale: As Shown

APPENDIX A

HYDRAULIC COMPUTATIONS

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: WEST DIVERSION CANAL

Description: BAYOU VINCENT @ CYPRESS ST.

Solve For Depth

Given Constant Data;

Bottom Width....... 15.00
Z-Left............. 3.00
Z-Right............ 3.00
Mannings 'n'....... 0.033
Channel Slope...... 0.0010
Channel Discharge.. 700.00

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	COMPUTED Channel Depth ft	Channel Discharge cfs	COMPUTED Velocity fps
15.00	3.00	3.00	0.033	0.0010	6.09	700.00	3.46

Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: WEST DIVERSION CANAL

Description: BAYOU BONFOUCA @ HIGHWAY 190

Solve For Depth

Given Constant Data;

Z-Left............. 3.00
Z-Right............ 3.00
Mannings 'n'....... 0.033
Channel Slope...... 0.0011
Channel Discharge.. 1100.00

Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	COMPUTED Channel Depth ft	COMPUTED Channel Discharge cfs	Velocity (fps)
3.00	3.00	0.033	0.0011	9.49	1100.00	4.07

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

EXISTING DIVERSION CANAL
4/22/94

CONTROL PARAMETERS

START TIME: .00
END TIME: 2.00

TO TIME (hours)	SIMULATION INC (secs)	PRINT INC (mins)
1.50	1.00	45.00
2.00	1.00	5.00

RUNOFF HYDROGRAPH FILE: DEFAULT
OFFSITE HYDROGRAPH FILE: DEFAULT
BOUNDARY DATABASE FILE: NONE

NOTE:

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EXISTING DIVERSION CANAL
4/22/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
A	TIME	12.410	.000	.000	.000	12.410	.000
						12.410	10.000
B	AREA	12.130	.000	.000	.000	7.860	.000
						8.860	.010
						12.860	.100
C	AREA	12.020	.000	.000	.000	7.780	.000
						8.780	.010
						12.780	.100
D	AREA	7.970	.000	.000	.000	4.880	.000
						5.880	.010
						9.880	.100
E	AREA	7.930	.000	.000	.000	4.850	.000
						5.850	.010
						9.850	.100
F	AREA	7.370	.000	.000	.000	4.450	.000
						5.450	.010
						9.450	.100
G	AREA	7.230	.000	.000	.000	4.350	.000
						5.350	.010
						9.350	.100
H	TIME	5.350	.000	.000	.000	5.350	.000
						5.350	10.000

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EXISTING DIVERSION CANAL
4/22/94

```
>>REACH NAME       : 2
  FROM NODE        : B
  TO NODE          : C
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
        SPAN (in):  48.000        RISE (in):  48.000     LENGTH (ft):  80.000
   U/S INVERT (ft):   4.860 D/S INVERT (ft):   4.780       MANNING N:    .013
      ENTRNC LOSS:     .500   # OF CULVERTS:   6.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

           NOTE:

>>REACH NAME       : 4
  FROM NODE        : D
  TO NODE          : E
  REACH TYPE       : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
        SPAN (in):  72.000        RISE (in):  72.000     LENGTH (ft):  30.000
   U/S INVERT (ft):   1.880 D/S INVERT (ft):   1.850       MANNING N:    .022
      ENTRNC LOSS:     .500   # OF CULVERTS:   1.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

           NOTE:
```

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EXISTING DIVERSION CANAL
4/22/94

```
>>REACH NAME       : 6
  FROM NODE        : F
  TO NODE          : G
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
         SPAN (in):   66.000      RISE (in):   42.000     LENGTH (ft): 100.000
   U/S INVERT (ft):    1.450 D/S INVERT (ft):   1.350       MANNING N:    .013
      ENTRNC LOSS:      .500  # OF CULVERTS:    3.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

               NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

EXISTING DIVERSION CANAL
4/22/94

```
>>REACH NAME        : 1
  FROM NODE         : A
  TO NODE           : B
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft): 200.000 U/S INVERT (ft):    5.060 D/S INVERT (ft):    4.860
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

               NOTE:

>>REACH NAME        : 3
  FROM NODE         : C
  TO NODE           : D
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft):2900.000 U/S INVERT (ft):    4.780 D/S INVERT (ft):    1.880
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

               NOTE:

>>REACH NAME        : 5
  FROM NODE         : E
  TO NODE           : F
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft): 400.000 U/S INVERT (ft):    1.850 D/S INVERT (ft):    1.450
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

               NOTE:

>>REACH NAME        : 7
  FROM NODE         : G
  TO NODE           : H
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft):1350.000 U/S INVERT (ft):    1.350 D/S INVERT (ft):     .000
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

               NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

EXISTING DIVERSION CANAL
4/22/94

REACH SUMMARY

INDEX	RCHNAME	FRMNODE	TONODE	REACH TYPE
1	2	B	C	CULVERT, RECTANGULAR w/ ROADWAY
2	4	D	E	CULVERT, CIRCULAR w/ ROADWAY
3	6	F	G	CULVERT, RECTANGULAR w/ ROADWAY
4	1	A	B	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
5	3	C	D	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
6	5	E	F	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
7	7	G	H	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

EXISTING DIVERSION CANAL
4/22/94

NODAL MAXIMUM CONDITIONS REPORT

NODE ID	STAGE (ft)	VOLUME (af)	INFLOW RUNOFF (cfs)	INFLOW OFFSITE (cfs)	INFLOW OTHER (cfs)	OUTFLOW (cfs)
A	12.41	.00	.00	.00	.00	256.12
B	12.28	.59	.00	.00	256.12	256.15
C	12.07	5.18	.00	.00	256.15	255.93
D	10.67	5.02	.00	.00	255.93	255.23
E	8.46	.69	.00	.00	255.23	255.24
F	8.05	.74	.00	.00	255.24	255.25
G	7.60	1.97	.00	.00	255.25	255.23
H	5.35	38.85	.00	.00	255.23	.00

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
4/25/94

CONTROL PARAMETERS

START TIME: .00
END TIME: 2.00

TO TIME (hours)	SIMULATION INC (secs)	PRINT INC (mins)
1.50	1.00	45.00
2.00	1.00	5.00

RUNOFF HYDROGRAPH FILE: DEFAULT
OFFSITE HYDROGRAPH FILE: DEFAULT
BOUNDARY DATABASE FILE: NONE

NOTE:

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
4/25/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
A	TIME	12.410	.000	.000	.000	12.410	.000
						12.410	10.000
B	AREA	12.130	.000	.000	.000	7.860	.000
						8.860	.010
						12.860	.100
C	AREA	12.020	.000	.000	.000	7.780	.000
						8.780	.010
						12.780	.100
D	AREA	7.970	.000	.000	.000	4.880	.000
						5.880	.010
						9.880	.100
E	AREA	7.930	.000	.000	.000	4.850	.000
						5.850	.010
						9.850	.100
F	AREA	7.370	.000	.000	.000	4.450	.000
						5.450	.010
						9.450	.100
G	AREA	7.230	.000	.000	.000	4.350	.000
						5.350	.010
						9.350	.100
H	TIME	5.350	.000	.000	.000	5.350	.000
						5.350	10.000

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        DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
        4/25/94

>>REACH NAME         : 2
  FROM NODE          : B
  TO NODE            : C
  REACH TYPE         : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH       : OFF

  CULVERT DATA       :
         SPAN (in):   48.000       RISE (in):   48.000     LENGTH (ft):   80.000
   U/S INVERT (ft):    4.860 D/S INVERT (ft):    4.780      MANNING N:     .013
       ENTRNC LOSS:     .500   # OF CULVERTS:    6.000

  POSITION A         : NOT USED

  POSITION B         : NOT USED

             NOTE:

>>REACH NAME         : 4
  FROM NODE          : D
  TO NODE            : E
  REACH TYPE         : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH       : OFF

  CULVERT DATA       :
         SPAN (in):   66.000       RISE (in):   48.000     LENGTH (ft):   40.000
   U/S INVERT (ft):    1.880 D/S INVERT (ft):    1.850      MANNING N:     .013
       ENTRNC LOSS:     .500   # OF CULVERTS:    3.000

  POSITION A         : NOT USED

  POSITION B         : NOT USED

             NOTE:
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Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
4/25/94

```
>>REACH NAME       : 6
  FROM NODE        : F
  TO NODE          : G
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
         SPAN (in):  66.000       RISE (in):  48.000     LENGTH (ft): 100.000
   U/S INVERT (ft):   1.450 D/S INVERT (ft):   1.350       MANNING N:    .013
       ENTRNC LOSS:    .500   # OF CULVERTS:   3.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

            NOTE:
```

```
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           DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
           4/25/94

>>REACH NAME        : 1
  FROM NODE         : A
  TO NODE           : B
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft): 200.000 U/S INVERT (ft):    5.060 D/S INVERT (ft):    4.860
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 3
  FROM NODE         : C
  TO NODE           : D
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft):2900.000 U/S INVERT (ft):    4.780 D/S INVERT (ft):    1.880
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 5
  FROM NODE         : E
  TO NODE           : F
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft): 400.000 U/S INVERT (ft):    1.850 D/S INVERT (ft):    1.450
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 7
  FROM NODE         : G
  TO NODE           : H
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):   10.000   LEFT SS (h/v):    1.000   RGHT SS (h/v):    1.000
       LENGTH (ft):1350.000 U/S INVERT (ft):    1.350 D/S INVERT (ft):     .000
         MANNING N:     .050 MAX. DEPTH (ft):  15.000

              NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
4/25/94

REACH SUMMARY

INDEX	RCHNAME	FRMNODE	TONODE	REACH TYPE
1	2	B	C	CULVERT, RECTANGULAR w/ ROADWAY
2	4	D	E	CULVERT, RECTANGULAR w/ ROADWAY
3	6	F	G	CULVERT, RECTANGULAR w/ ROADWAY
4	1	A	B	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
5	3	C	D	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
6	5	E	F	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
7	7	G	H	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL W/ CROSSING IMPROVEMENTS ONLY
4/25/94

NODAL MAXIMUM CONDITIONS REPORT

NODE ID	STAGE (ft)	VOLUME (af)	INFLOW RUNOFF (cfs)	INFLOW OFFSITE (cfs)	INFLOW OTHER (cfs)	OUTFLOW (cfs)
A	12.41	.00	.00	.00	.00	290.67
B	12.23	.59	.00	.00	290.67	290.68
C	12.02	5.14	.00	.00	290.68	290.59
D	9.44	4.13	.00	.00	290.59	290.27
E	8.94	.79	.00	.00	290.27	290.27
F	8.53	.82	.00	.00	290.27	290.27
G	7.96	2.11	.00	.00	290.27	290.26
H	5.35	44.23	.00	.00	290.26	.00

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

CONTROL PARAMETERS

START TIME: .00
END TIME: 2.00

TO TIME (hours)	SIMULATION INC (secs)	PRINT INC (mins)
1.50	1.00	45.00
2.00	1.00	5.00

RUNOFF HYDROGRAPH FILE: DEFAULT
OFFSITE HYDROGRAPH FILE: DEFAULT
BOUNDARY DATABASE FILE: NONE

NOTE:

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
A	TIME	12.410	.000	.000	.000	12.410	.000
						12.410	10.000
B	AREA	12.130	.000	.000	.000	7.860	.000
						8.860	.010
						12.860	.100
C	AREA	12.020	.000	.000	.000	7.780	.000
						8.780	.010
						12.780	.100
D	AREA	7.970	.000	.000	.000	4.880	.000
						5.880	.010
						9.880	.100
E	AREA	7.930	.000	.000	.000	4.850	.000
						5.850	.010
						9.850	.100
F	AREA	7.370	.000	.000	.000	4.450	.000
						5.450	.010
						9.450	.100
G	AREA	7.230	.000	.000	.000	4.350	.000
						5.350	.010
						9.350	.100
H	TIME	5.350	.000	.000	.000	5.350	.000
						5.350	10.000

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

```
>>REACH NAME        : 2
  FROM NODE         : B
  TO NODE           : C
  REACH TYPE        : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :
         SPAN (in):  48.000        RISE (in):  48.000     LENGTH (ft):  80.000
   U/S INVERT (ft):   4.860 D/S INVERT (ft):   4.780       MANNING N:     .013
       ENTRNC LOSS:    .500   # OF CULVERTS:   6.000

  POSITION A        : NOT USED

  POSITION B        : NOT USED

             NOTE:

>>REACH NAME        : 4
  FROM NODE         : D
  TO NODE           : E
  REACH TYPE        : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :
         SPAN (in):  72.000        RISE (in):  72.000     LENGTH (ft):  30.000
   U/S INVERT (ft):   1.880 D/S INVERT (ft):   1.850       MANNING N:     .022
       ENTRNC LOSS:    .500   # OF CULVERTS:   1.000

  POSITION A        : NOT USED

  POSITION B        : NOT USED

             NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

```
>>REACH NAME      : 6
  FROM NODE       : F
  TO NODE         : G
  REACH TYPE      : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION  : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH    : OFF

  CULVERT DATA    :
        SPAN (in):   66.000       RISE (in):  48.000     LENGTH (ft): 100.000
   U/S INVERT (ft):   1.450 D/S INVERT (ft):   1.350       MANNING N:    .013
      ENTRNC LOSS:     .500   # OF CULVERTS:   3.000

  POSITION A      : NOT USED

  POSITION B      : NOT USED

            NOTE:
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```
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DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

>>REACH NAME        : 1
  FROM NODE         : A
  TO NODE           : B
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft): 200.000 U/S INVERT (ft):    5.060 D/S INVERT (ft):    4.860
         MANNING N:    .032 MAX. DEPTH (ft):   15.000

              NOTE:

>>REACH NAME        : 3
  FROM NODE         : C
  TO NODE           : D
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft):2900.000 U/S INVERT (ft):    4.780 D/S INVERT (ft):    1.880
         MANNING N:    .032 MAX. DEPTH (ft):   15.000

              NOTE:

>>REACH NAME        : 5
  FROM NODE         : E
  TO NODE           : F
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft): 400.000 U/S INVERT (ft):    1.850 D/S INVERT (ft):    1.450
         MANNING N:    .032 MAX. DEPTH (ft):   15.000

              NOTE:

>>REACH NAME        : 7
  FROM NODE         : G
  TO NODE           : H
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft):1350.000 U/S INVERT (ft):    1.350 D/S INVERT (ft):     .000
         MANNING N:    .035 MAX. DEPTH (ft):   15.000

              NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

REACH SUMMARY

INDEX	RCHNAME	FRMNODE	TONODE	REACH TYPE
1	2	B	C	CULVERT, RECTANGULAR w/ ROADWAY
2	4	D	E	CULVERT, CIRCULAR w/ ROADWAY
3	6	F	G	CULVERT, RECTANGULAR w/ ROADWAY
4	1	A	B	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
5	3	C	D	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
6	5	E	F	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
7	7	G	H	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL IMPROVEMENTS ONLY
4/25/94

NODAL MAXIMUM CONDITIONS REPORT

NODE ID	STAGE (ft)	VOLUME (af)	INFLOW RUNOFF (cfs)	INFLOW OFFSITE (cfs)	INFLOW OTHER (cfs)	OUTFLOW (cfs)
A	12.41	.00	.00	.00	.00	344.56
B	12.37	.77	.00	.00	344.56	344.57
C	12.02	7.89	.00	.00	344.57	344.52
D	11.68	8.79	.00	.00	344.52	344.51
E	7.93	.86	.00	.00	344.51	344.50
F	7.40	.87	.00	.00	344.50	344.50
G	7.23	2.67	.00	.00	344.50	344.50
H	5.35	53.66	.00	.00	344.50	.00

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

CONTROL PARAMETERS

START TIME: .00
END TIME: 2.00

TO TIME (hours)	SIMULATION INC (secs)	PRINT INC (mins)
1.50	1.00	45.00
2.00	1.00	5.00

RUNOFF HYDROGRAPH FILE: DEFAULT
OFFSITE HYDROGRAPH FILE: DEFAULT
BOUNDARY DATABASE FILE: NONE

NOTE:

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DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
A	TIME	12.410	.000	.000	.000	12.410	.000
						12.410	10.000
B	AREA	12.130	.000	.000	.000	7.860	.000
						8.860	.010
						12.860	.100
C	AREA	12.020	.000	.000	.000	7.780	.000
						8.780	.010
						12.780	.100
D	AREA	7.970	.000	.000	.000	4.880	.000
						5.880	.010
						9.880	.100
E	AREA	7.930	.000	.000	.000	4.850	.000
						5.850	.010
						9.850	.100
F	AREA	7.370	.000	.000	.000	4.450	.000
						5.450	.010
						9.450	.100
G	AREA	7.230	.000	.000	.000	4.350	.000
						5.350	.010
						9.350	.100
H	TIME	5.350	.000	.000	.000	5.350	.000
						5.350	10.000

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DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

```
>>REACH NAME       : 2
  FROM NODE        : B
  TO NODE          : C
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
        SPAN (in):   48.000      RISE (in):   48.000     LENGTH (ft):   80.000
   U/S INVERT (ft):   4.860 D/S INVERT (ft):   4.780      MANNING N:      .013
      ENTRNC LOSS:     .500   # OF CULVERTS:   6.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

            NOTE:

>>REACH NAME       : 4
  FROM NODE        : D
  TO NODE          : E
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
        SPAN (in):   66.000      RISE (in):   48.000     LENGTH (ft):   40.000
   U/S INVERT (ft):   1.880 D/S INVERT (ft):   1.850      MANNING N:      .013
      ENTRNC LOSS:     .500   # OF CULVERTS:   3.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

            NOTE:
```

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DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

```
>>REACH NAME       : 6
  FROM NODE        : F
  TO NODE          : G
  REACH TYPE       : CULVERT, RECTANGULAR w/ ROADWAY
  FLOW DIRECTION   : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH     : OFF

  CULVERT DATA     :
       SPAN (in):   66.000      RISE (in):   48.000     LENGTH (ft): 100.000
  U/S INVERT (ft):   1.450 D/S INVERT (ft):   1.350      MANNING N:     .013
      ENTRNC LOSS:    .500   # OF CULVERTS:   3.000

  POSITION A       : NOT USED

  POSITION B       : NOT USED

               NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)
Copyright 1989, Streamline Technologies, Inc.

DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

```
>>REACH NAME        : 1
  FROM NODE         : A
  TO NODE           : B
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft): 200.000 U/S INVERT (ft):    5.060 D/S INVERT (ft):    4.860
         MANNING N:     .032 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 3
  FROM NODE         : C
  TO NODE           : D
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft):2900.000 U/S INVERT (ft):    4.780 D/S INVERT (ft):    1.880
         MANNING N:     .032 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 5
  FROM NODE         : E
  TO NODE           : F
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft): 400.000 U/S INVERT (ft):    1.850 D/S INVERT (ft):    1.450
         MANNING N:     .032 MAX. DEPTH (ft):  15.000

              NOTE:

>>REACH NAME        : 7
  FROM NODE         : G
  TO NODE           : H
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : CRITICAL DEPTH
   BOT. WIDTH (ft):  12.000   LEFT SS (h/v):    2.000   RGHT SS (h/v):    2.000
       LENGTH (ft):1350.000 U/S INVERT (ft):    1.350 D/S INVERT (ft):     .000
         MANNING N:     .032 MAX. DEPTH (ft):  15.000

              NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

REACH SUMMARY

NDEX	RCHNAME	FRMNODE	TONODE	REACH TYPE
1	2	B	C	CULVERT, RECTANGULAR w/ ROADWAY
2	4	D	E	CULVERT, RECTANGULAR w/ ROADWAY
3	6	F	G	CULVERT, RECTANGULAR w/ ROADWAY
4	1	A	B	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
5	3	C	D	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
6	5	E	F	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
7	7	G	H	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

DIVERSION CANAL AND CROSSING IMPROVEMENTS
4/25/94

NODAL MAXIMUM CONDITIONS REPORT

NODE ID	STAGE (ft)	VOLUME (af)	INFLOW RUNOFF (cfs)	INFLOW OFFSITE (cfs)	INFLOW OTHER (cfs)	OUTFLOW (cfs)
A	12.41	.00	.00	.00	.00	502.25
B	12.33	.76	.00	.00	502.25	502.23
C	12.02	7.89	.00	.00	502.23	501.83
D	10.51	7.46	.00	.00	501.83	501.21
E	9.02	1.12	.00	.00	501.21	501.21
F	8.85	1.19	.00	.00	501.21	501.21
G	7.23	2.67	.00	.00	501.21	501.22
H	5.35	77.13	.00	.00	501.22	.00

<u>**SECTION II**</u>

CITY BARN PUMPING STATION

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I. Introduction

Proposed modifications to the City Barn Pump Station include additional pumping capacity at the station in order to reduce street flooding in neighboring areas. Presently, the system of three (3) pumps is able to discharge a maximum of 120,000 gallons-per-minute (gpm) or about 267 cubic-feet-per-second (cfs) of storm water out of the surrounding residential area into Bayou Bonfouca. This pumping capacity, while once probably adequate for the undeveloped area, is now insufficient to handle the increased flows of storm water runoff. Because of increasing urbanization in the area, once-pervious soils are being built-on and covered with impervious concrete and asphalt, creating a larger volume of runoff. Urbanization produces a higher value of "C" in the rational equation[1] which in turn decreases the allowable peak rainfall intensity over the area since the pump capacity remains constant. For the City Barn Drainage Basin, the current allowable peak rainfall intensity, based on a maximum outflow of 267 cfs, is roughly one-half inch per hour (0.56 in/hr), significantly lower than even the 2-year storm event for the area. A rain event of this magnitude, accompanied by residential street flooding, can be expected to occur several times per year.

II. Project Description

The scope of this phase of the project is to analyze increased pumping capacity at the City Barn Pump Station and its hydraulic effects on the

[1]Rational equation: Peak flow (Qp) = Coefficient (C)*Rainfall intensity (I)*Drainage area (A)

drainage network. Hydraulic analysis includes checking both increased water surface elevations downstream of the station in Bayou Bonfouca and increased flow velocities upstream of the station in Bayou Pattasat up to the railroad crossing near Highway 11. Based on this study, schematics and cost estimates were developed for the pump station and for revisions to Bayou Pattasat west of the railroad crossing; and a project schedule for the proposed modifications is also included.

III. Existing Conditions

The City Barn Pumping Station serves the Bayou Vincent / Bayou Bonfouca Drainage Basin, lying immediately to the west of the W-14 Canal Drainage Basin and to the east of the Bayou Liberty Drainage Basin (see Site Map, Figure 1). The area contributory to the City Barn Pump Station lies at the extreme east of the Bayou Vincent / Bayou Bonfouca Drainage Basin, bounded on the west by the New Orleans and Northeastern Railroad, on the north by U.S. Highway 190 (Gause Blvd.), on the south by La. Highway 433 (Old Spanish Trail Rd.), and the western edge of the W-14 Canal Drainage Basin. The area is drained by Bayou Pattasat, a forked canal leading west to the pump station, which then discharges the storm water into Bayou Bonfouca. The City Barn Drainage Area is mapped in Figure 2.

Two visits to the project site were conducted, and visual inspections were compared with drawing sets titled <u>Bayou Vincent Pump Station</u> (Roessle & Cartier, Inc., 1974) and <u>Bayou Vincent Pump Station Addition</u> (Cartier &

Associates, Inc., 1982), both provided by the City of Slidell. The 1974 drawing set included a revision (Change Order No. 1) which proposed the typical Bayou Pattasat cross-section shown in Figure 3. However, the 1982 drawing set made no reference to this cross-section and indicated a winding natural channel. Thus, either this revision was never built or eight years of urbanization in the area produced flows large enough to completely destroy the modified channel.

Present-day conditions of Bayou Pattasat based on site inspection include the following:

a.) The channel is overgrown with weeds and has trees and brush low on its banks.

b.) Some side-bank erosion has taken place, perhaps as a result of previous clearing efforts and a subsequent failure to adequately stabilize the banks. It can thus be expected that significant sedimentation has occurred and possibly as much as 12 inches of silt has been deposited and built up on the channel bottom.

c.) The bayou narrows significantly at certain points, particularly under the railroad crossing.

IV. Project Hydraulics

Project hydraulics were based on the St. Tammany Parish Master Drainage Plan, prepared by Burk & Associates, Inc. (1983), and the drawing sets mentioned above. As described above, visual inspection of the site location indicated that significant bank overgrowth, erosion, and sedimentation have occurred in recent years, yielding a hydraulically less-efficient channel cross-section. Thus, hydraulic analysis was carried out in two parts: first, an analysis of the 1974 cross-section; and second, an analysis with certain hydraulic parameters adjusted accordingly to fit visually-estimated conditions.

Hydrologic computations (see Appendix A) reveal that a 10-year storm produces a flow of approximately 1000 cfs over the area. Hydraulic analysis (see Appendix B) shows that the channel section appearing in the 1974 drawings is sufficient to carry a flow of over 1800 cfs, about 180% of the peak flow expected for the 10-year design storm. However, with the following cross-sectional properties adjusted, the cross-section was proven to be insufficient:

a.) Bank erosion: the side slopes were reduced from 2.5 : 1 to 1.5 : 1.

b.) Bank overgrowth: the Manning's "n" value was increased from 0.030 to 0.060.

c.) Sedimentation and siltation: the bayou bottom depth was reduced from 10 ft. to 9 ft.

d.) Cross-section variance: the channel bottom width was reduced from 20 ft. to 10 ft. to simulate restrictions, bends, or crossings under streets or railroad tracks.

Results show that if all of the above reductions occurred, the bayou would be able to handle a flow of only 375 cfs, or about 38% of the peak 10-year flow.

Further analysis reveals that the greatest contributor to flow restriction is the increase in roughness of the channel (the parameter "n"): a doubling of Manning's "n" cuts the allowable flow quantity in half.

V. Proposed Improvements

The current pumping system is the limiting factor in the design; however, the cost of upgrading the station from 267 cfs to 1000 cfs would be prohibitive. Instead, it is proposed to install one (1) additional 60,000 gpm pump, which would increase the pumping capacity of the station to 180,000 gpm or approximately 400 cfs. The section of Bayou Pattasat downstream of the railroad crossing can then be improved to create a temporary storage reservoir for the additional flow until it is ultimately discharged through the pumping station. Improvements to Bayou Pattasat include the following:

a.) Clearing, mowing, and cutting weeds and brush from the channel sides and bottom in order to keep it as hydraulically "smooth" as possible. The canal may also be lined, if it is determined that this will provide a more reliable and cost-effective solution.

b.) Dredging the channel bottom to elevation (-)6.00' NGVD.

c.) Lining the section underneath the New Orleans & Northeastern Railway and Highway 11. The optimum solution would be to widen the channel at these crossings, but limitations occur due to the utility crossings on the south side of the channel located a few feet from the top of bank. The lining will also serve to stabilize the banks.

Currently, the water surface elevation in Bayou Pattasat ranges from a low value of (-)1.00 to a high of (+)3.50, and the average water surface elevation in Bayou Bonfouca is about (+)1.00. Typically, Bayou Bonfouca is about 1 foot higher than Bayou Pattasat. Thus, it is apparent that the pumps may be run quite frequently. During periods of high flow through Bayou Pattasat or low flow in Bayou Bonfouca, drainage will be accomplished through the existing gravity outfall structure and system of three sluice gates to the southwest of the pump station.

According to the 1983 Slidell MDP, the existing section of Bayou Bonfouca at the junction with the City Barn Pumping Station is capable of transporting about 4000 cfs, corresponding to a 10-year peak water surface elevation of about 2.85 feet. Typical daily flows are much less, with an average water surface elevation of about 1.00 feet. The bayou at this section is very wide and has good hydraulic properties. Thus, the increase in maximum discharge from 267 cfs to 400 cfs at the pump station will raise the water surface elevation in Bayou Bonfouca slightly, yet sufficient channel width and side-bank height exist to contain this additional flow.

It should be noted that the water surface elevation will also rise upstream due to improvements previously described in the West Diversion Canal (Section I) and downstream due to those to be discussed later at the Delwood Pumping Station (Sections III and IV). However, Bayou Bonfouca is estimated to have adequate properties to restrain the sum of these increased flows from overflowing its banks.

VI. Conceptual Design

Conceptually, Bayou Pattasat should convey 400 cfs of water at a maximum velocity of 3 fps to prevent channel bottom scour and side-slope erosion. In addition, the banks should be sufficiently high to provide necessary storage for runoff from a 10-year storm event. The 1974 cross-section meets these criteria; again, it is shown in Figure 3. Modifications necessary to the existing channel (clearing and grubbing, excavation, and lining at the crossings) have been described previously.

Additionally, the pump station and sump area will have to be modified. The existing 8-inch-thick pump and sump slabs will be extended an additional 13'-9" to accommodate the new pump. This assumes that the top elevations of (+)15.33 for the pump slab and (-)12.00 for the sump slab are adequate for the new pump selected for this design. If this is not true, the new pump slab may have to be constructed at a higher elevation and provided with stair access similar to the construction for the slab under the existing 60,000 gpm pump. Also required as part of this extension are additional precast concrete piles under the slab, additional sheet piling, fencing, and mesh-screening around the slab, and additional fill material, all complying with the original materials and design.

As shown in the hydraulic analysis in Appendix B, the new pump is required to have a 54" discharge pipe. The pipe will be steel and will be installed in sections from the station down underneath the J-rap protective levee system along Bayou Bonfouca. The elevation of the discharge end of the pipe will be such that its crown rests below the 1.00-foot average water surface elevation in the bayou. Modifications to the levee system will be necessary to allow passage of this pipe. A schematic drawing of the proposed pump station additions is provided in Figure 4.

Finally, the existing 48"-diameter fuel tank on the site must be moved from its present pile-supported platform to solid ground and fully enclosed to comply with current regulations. It is proposed to erect an 8-foot-high metal shed around the 13' x 8'-6" tank support slab and relocate the assembly to the northeast.

The structural details of the above modifications are beyond the scope of this report, but will for the most part remain true to the original design criteria specified for each phase of the project. However, approximate costs have been developed for each of the above items and will be discussed in the following section.

VII. Estimated Design / Construction Cost

Table 2 shows a preliminary list of each item included in this project and its associated cost. The total construction cost is estimated at $1,091,000; adding design costs ($110,000), construction administration ($43,600), and resident inspection ($144,000) brings the total project cost to an estimated $1,388,600.

It should be noted that because this cost estimate is preliminary, it may be lacking some required cost items. Regarding the bayou improvements, for example, the survey may show that additional right-of-way must be acquired or utilities must be relocated, particularly at the crossing widenings. These additional items would raise the total project cost slightly.

VIII. Estimated Design / Construction Time Schedule.

The estimated design time schedule for this project is three months for the project design, which includes preparation of plans, specifications, and contract documents; two months for bidding and award of the contract; and an additional 18 months for construction. This estimated time schedule assumes the required funding for the project is available and does not include possible delays caused by appropriation of money to fund the project.

TABLE 1

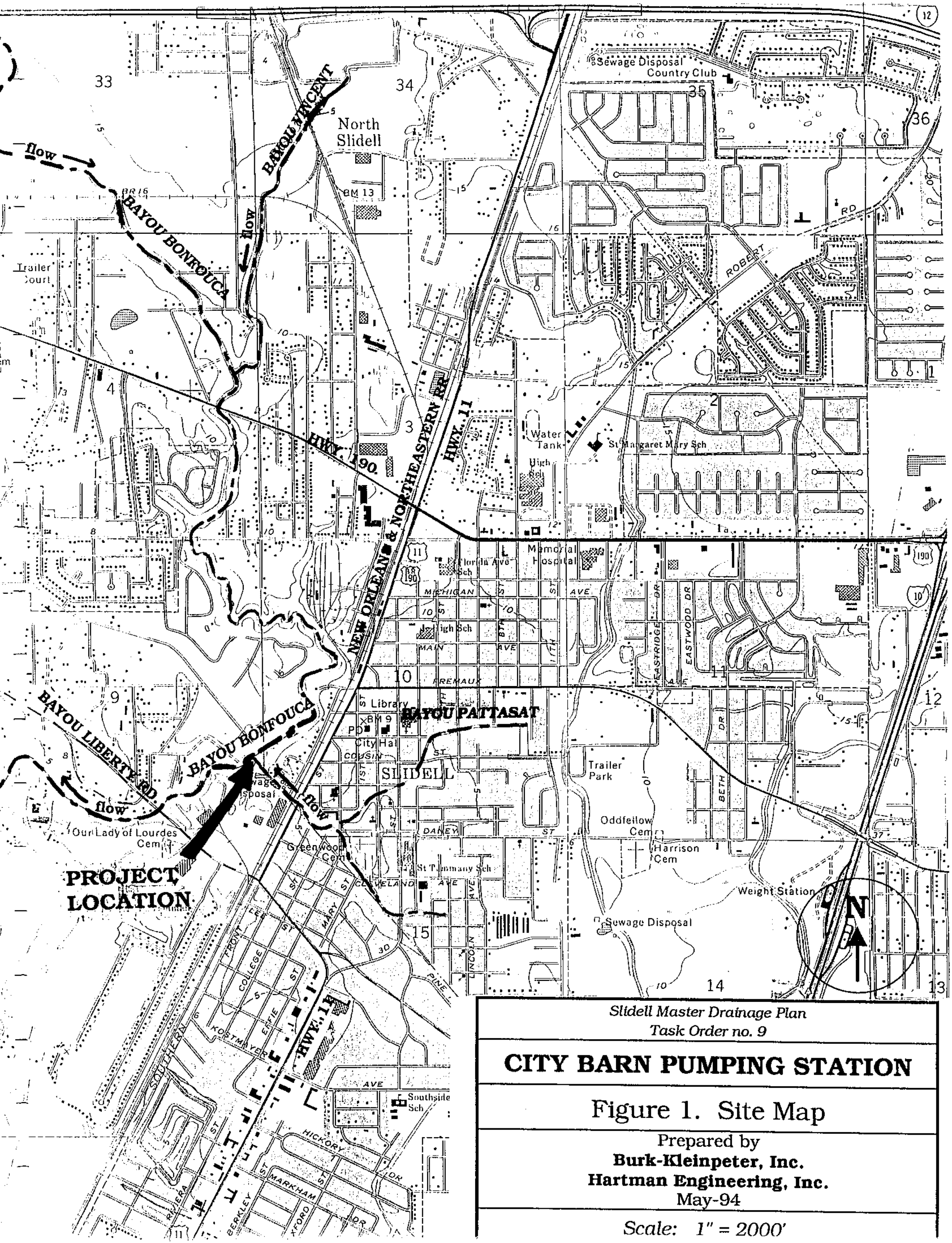
DESIGN PARAMETERS

Parameter	Value	Source
Drainage basin area	1120 acres	U.S.G.S. Maps
Ground slope	0.000625 ft/ft	U.S.G.S. Maps
Hydraulic length of watershed	8000 ft.	U.S.G.S. Maps
Runoff coefficient "C"	0.42	Design Manual (Single family residential)
Time of concentration	110 min.	Appendix A
Design rainfall intensity	2.11 in/hr	Appendix B (Based on 110 min.)
Peak flow of runoff	1000 cfs	Rational Method
Pump capacity	120,000 gpm 267 cfs	City of Slidell
W.S.E. at pumps	-1.00 (low) +3.50 (high)	Cartier & Assoc. (1982)
Manning's "N" of canal	0.030	Design Manual (Natural earth channel)
Bed slope of canal	0.000625 ft/ft	U.S.G.S. Maps
Bottom width of canal	20 ft.	Roessle & Cartier (1974)
Side slopes of canal	2.5:1	Roessle & Cartier (1974)
Total depth of canal	10 ft.	Roessle & Cartier (1974)

TABLE 2

ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$45,000
Clearing and Grubbing	Acre	2	$2,500	$5,000
Excavation	Cubic Yard	2,000	$8	$16,000
Lining @ Canal Crossing	SF	400	$1	$400
60,000 gpm Pump	Each	1	$802,000	$802,000
Pump Slab	Cubic Yard	13	$300	$3,900
Sump Slab	Cubic Yard	13	$300	$3,900
Precast Concrete Piles	Each	4	$1,200	$4,800
Steel Sheet Piling (MZ-27)	Square Ft	140	$12	$1,700
54" Steel Pipe	Linear Ft	75	$270	$20,250
Relocating & Enclosing Fuel Tank	Lump	1	Lump Sum	$6,000
SUBTOTAL				$ 908,950
CONTINGENCY @ 20%				$181,800
TOTAL				$1,091,000



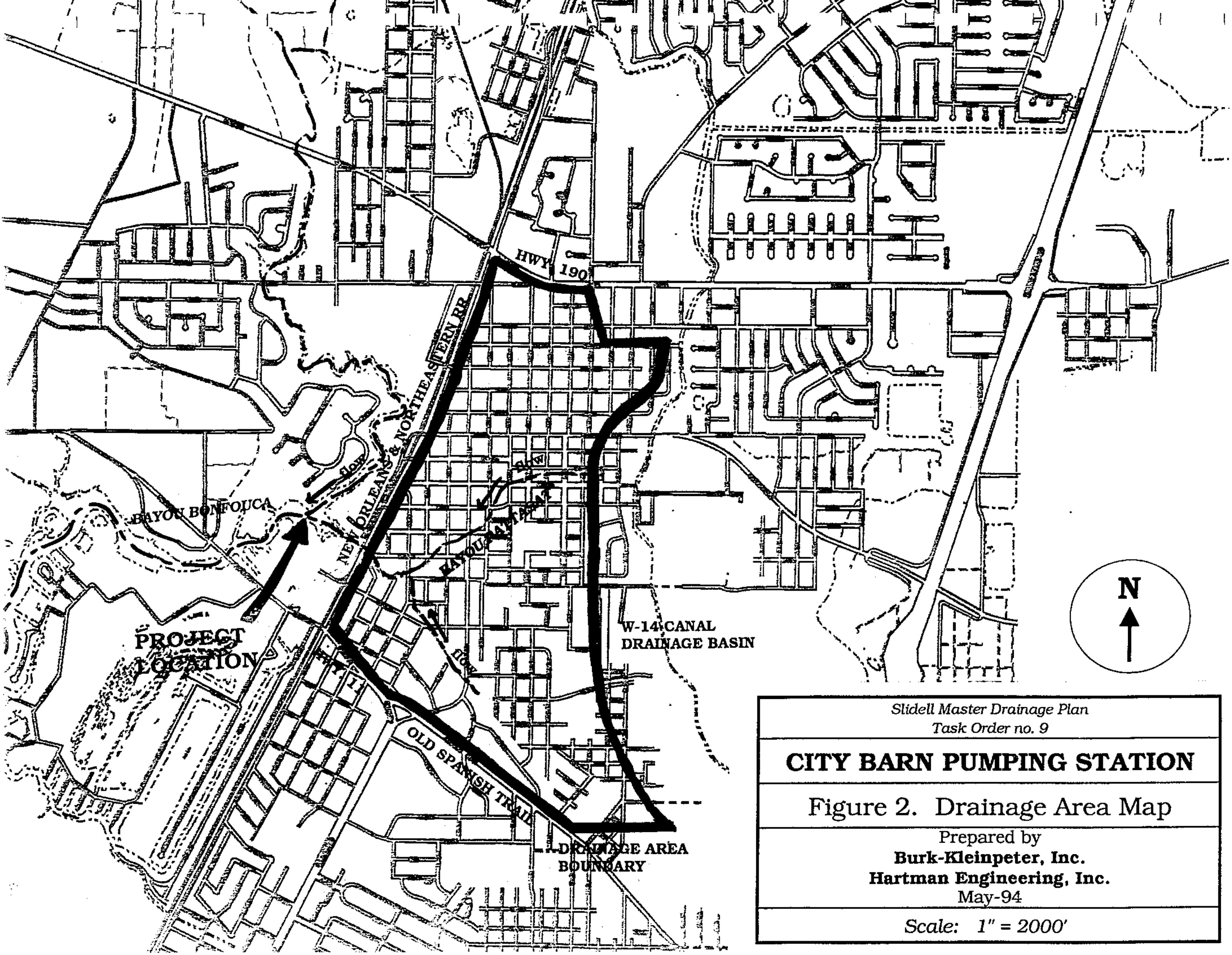
Slidell Master Drainage Plan
Task Order no. 9

CITY BARN PUMPING STATION

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



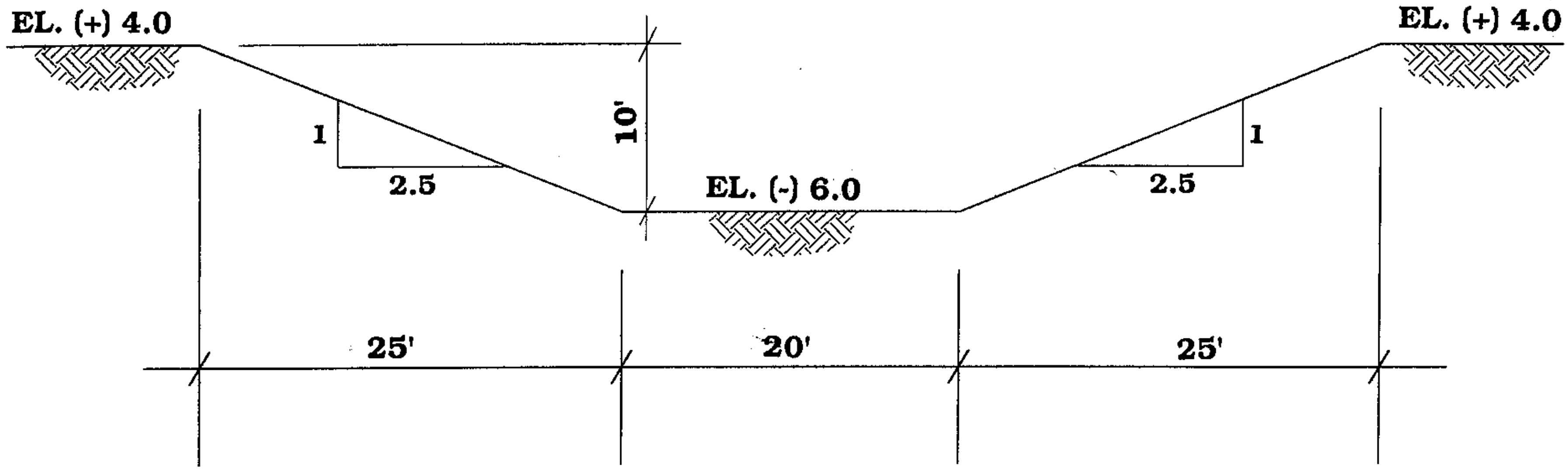
Slidell Master Drainage Plan
Task Order no. 9

CITY BARN PUMPING STATION

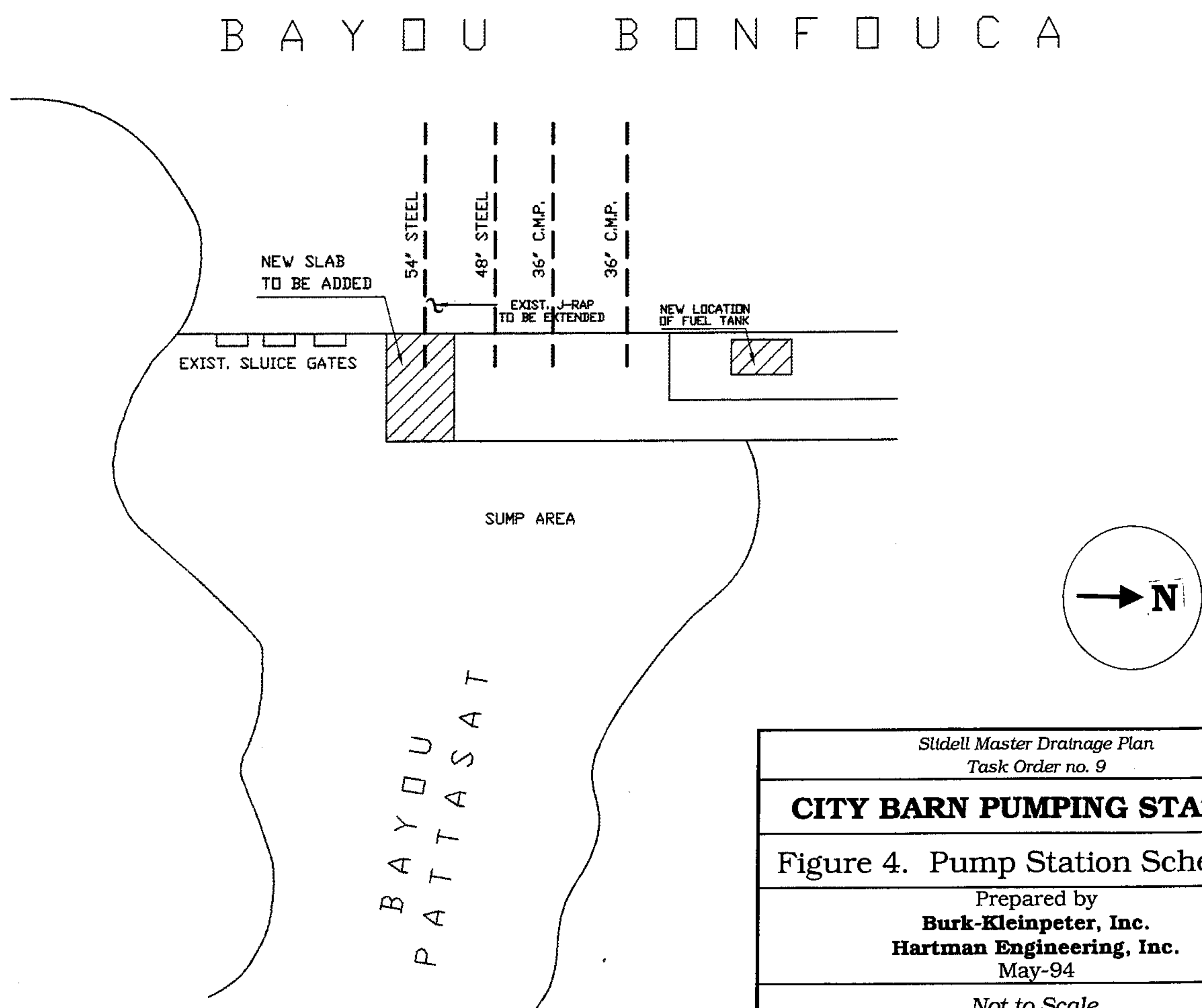
Figure 2. Drainage Area Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



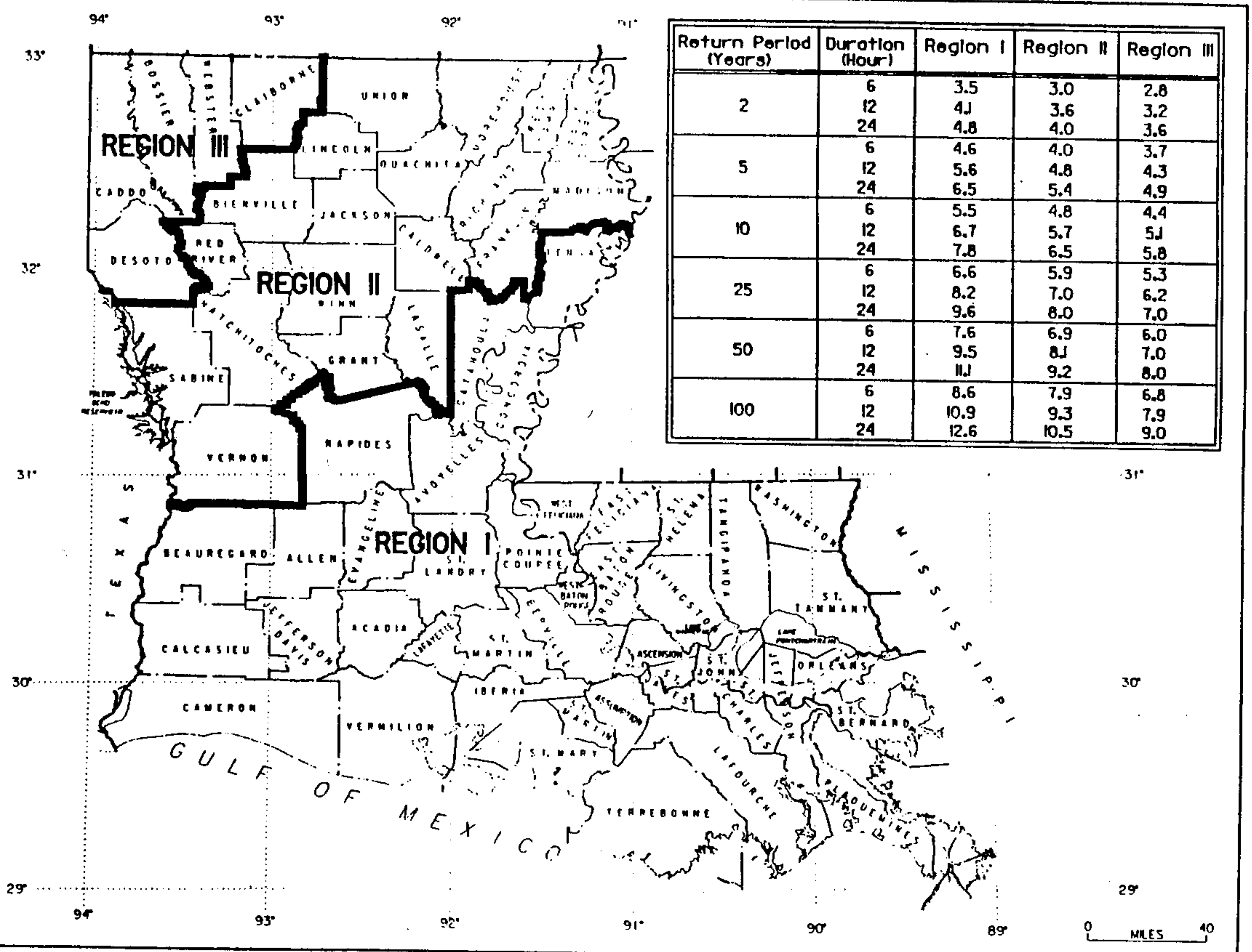
Slidell Master Drainage Plan *Task Order no. 9*
CITY BARN PUMPING STATION
Figure 3. Proposed Canal Cross-Section
Prepared by **Burk-Kleinpeter, Inc.** **Hartman Engineering, Inc.** May-94
Scale: 1" = 10'



Slidell Master Drainage Plan *Task Order no. 9*
CITY BARN PUMPING STATION
Figure 4. Pump Station Schematic
Prepared by **Burk-Kleinpeter, Inc.** **Hartman Engineering, Inc.** May-94
Not to Scale

APPENDIX A

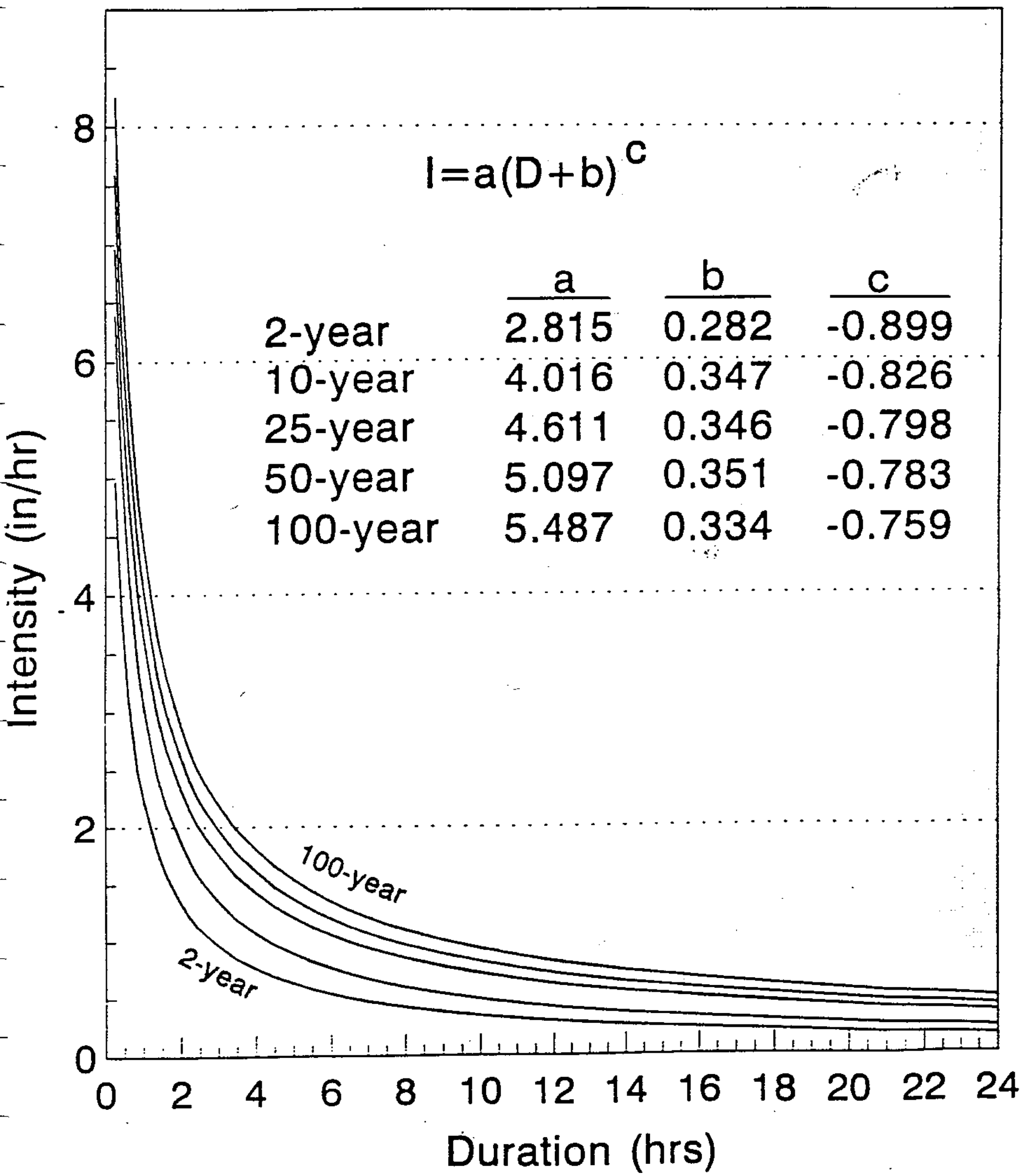
HYDROLOGIC COMPUTATIONS



Return Period (Years)	Duration (Hour)	Region I	Region II	Region III
2	6	3.5	3.0	2.8
	12	4.1	3.6	3.2
	24	4.8	4.0	3.6
5	6	4.6	4.0	3.7
	12	5.6	4.8	4.3
	24	6.5	5.4	4.9
10	6	5.5	4.8	4.4
	12	6.7	5.7	5.1
	24	7.8	6.5	5.8
25	6	6.6	5.9	5.3
	12	8.2	7.0	6.2
	24	9.6	8.0	7.0
50	6	7.6	6.9	6.0
	12	9.5	8.1	7.0
	24	11.1	9.2	8.0
100	6	8.6	7.9	6.8
	12	10.9	9.3	7.9
	24	12.6	10.5	9.0

RAINFALL DEPTH (INCHES) FOR VARIOUS PRECIPITATION REGIONS

Region I



I=a(D+b)^c
a b c
2-year 2.815 0.282 -0.899
10-year 4.016 0.347 -0.826
25-year 4.611 0.346 -0.798
50-year 5.097 0.351 -0.783
100-year 5.487 0.334 -0.759
100-year
2-year
Intensity (in/hr)
Duration (hrs)
0
2
4
6
8
0 2 4 6 8 10 12 14 16 18 20 22 24

APPENDIX B

HYDRAULIC ANALYSIS

RUNOFF CALCULATIONS (RATIONAL METHOD)

$Qp = CIA$

where: Qp = Peak Runoff Flow, [cfs]
C = Runoff Coefficient
I = Rainfall Intensity, [in/hr]
A = Area of Basin, [acres]

<u>Rainfall Intensity</u>

I is based on a duration equal to the Time of Concentration, Tc:
where: $Tc = 0.7039\ (L^{0.3917})\ (C^{-1.1309})\ (S^{-0.1985})$, [min]
L = Overland Flow Length, [ft]
S = Ground Slope, [%]

$Tc = 0.7039\ (8000^{0.3917})\ (0.42^{-1.1309})\ (0.0625^{-0.1985})$
= 110 min., or 1.83 hrs.

Then, $I = a\ (D+b)^{c}$
where: a,b,c = *f* (region, return period) - see previous pages
D = Duration of Storm = Tc, [hrs]

$I = 4.016\ (1.83+0.347)^{-0.826}$
= 2.11 in/hr

$Qp = 0.42\ (2.11\ \text{in/hr})\ (1120\ \text{acres})$
= 992 cfs

<u>Results</u>

Qp = 1000 cfs from a 10-year storm

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: CITYBARN

Description: MAX. FLOW THROUGH BAYOU PATTASAT.

Solve For Discharge

Given Constant Data;

Channel Slope...... 0.0006

Variable Input Data	Minimum	Maximum	Increment By
Bottom Width	10.00	20.00	10.00
Z-Left	1.50	2.50	1.00
Z-Right	1.50	2.50	1.00
Mannings 'n'	0.030	0.060	0.030
Channel Depth	9.00	10.00	1.00

VARIABLE Bottom Width ft	VARIABLE Z-Left (H:V)	VARIABLE Z-Right (H:V)	VARIABLE Mannings 'n'	Channel Slope ft/ft	VARIABLE Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
10.00	1.50	1.50	0.030	0.0006	9.00	748.58	3.54
20.00	**1.50**	**1.50**	**0.030**	**0.0006**	**9.00**	**1173.88**	**3.89**
10.00	2.50	1.50	0.030	0.0006	9.00	893.35	3.55
20.00	**2.50**	**1.50**	**0.030**	**0.0006**	**9.00**	**1317.39**	**3.85**
10.00	1.50	2.50	0.030	0.0006	9.00	893.35	3.55
20.00	**1.50**	**2.50**	**0.030**	**0.0006**	**9.00**	**1317.39**	**3.85**
10.00	**2.50**	**2.50**	**0.030**	**0.0006**	**9.00**	**1038.11**	**3.55**
20.00	**2.50**	**2.50**	**0.030**	**0.0006**	**9.00**	**1461.19**	**3.82**
10.00	1.50	1.50	0.060	0.0006	9.00	374.29	1.77
20.00	1.50	1.50	0.060	0.0006	9.00	586.94	1.95
10.00	2.50	1.50	0.060	0.0006	9.00	446.67	1.77
20.00	2.50	1.50	0.060	0.0006	9.00	658.69	1.93
10.00	1.50	2.50	0.060	0.0006	9.00	446.67	1.77
20.00	1.50	2.50	0.060	0.0006	9.00	658.69	1.93
10.00	2.50	2.50	0.060	0.0006	9.00	519.06	1.77
20.00	2.50	2.50	0.060	0.0006	9.00	730.60	1.91
10.00	1.50	1.50	0.030	0.0006	10.00	936.88	3.75
20.00	**1.50**	**1.50**	**0.030**	**0.0006**	**10.00**	**1439.93**	**4.11**
10.00	**2.50**	**1.50**	**0.030**	**0.0006**	**10.00**	**1128.53**	**3.76**
20.00	**2.50**	**1.50**	**0.030**	**0.0006**	**10.00**	**1630.56**	**4.08**
10.00	**1.50**	**2.50**	**0.030**	**0.0006**	**10.00**	**1128.53**	**3.76**
20.00	**1.50**	**2.50**	**0.030**	**0.0006**	**10.00**	**1630.56**	**4.08**
10.00	**2.50**	**2.50**	**0.030**	**0.0006**	**10.00**	**1320.20**	**3.77**
20.00	**2.50**	**2.50**	**0.030**	**0.0006**	**10.00**	**1821.46**	**4.05**
10.00	1.50	1.50	0.060	0.0006	10.00	468.44	1.87
20.00	1.50	1.50	0.060	0.0006	10.00	719.96	2.06
10.00	2.50	1.50	0.060	0.0006	10.00	564.26	1.88
20.00	2.50	1.50	0.060	0.0006	10.00	815.28	2.04
10.00	1.50	2.50	0.060	0.0006	10.00	564.26	1.88
20.00	1.50	2.50	0.060	0.0006	10.00	815.28	2.04
10.00	2.50	2.50	0.060	0.0006	10.00	660.10	1.89
20.00	2.50	2.50	0.060	0.0006	10.00	910.73	2.02

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: CITYBARN

Description: CHECK MAXIMUM VELOCITY THROUGH BAYOU PATTASAT.

Solve For Depth

Given Constant Data;

Bottom Width....... 20.00
Z-Left............. 2.50
Z-Right............ 2.50
Mannings 'n'....... 0.030
Channel Slope...... 0.0006
Channel Discharge.. 400.00

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	COMPUTED Channel Depth ft	Channel Discharge cfs	COMPUTED Velocity fps
20.00	2.50	2.50	0.030	0.0006	4.70	400.00	2.68

Circular Channel Analysis & Design

Open Channel - Uniform flow

Worksheet Name: CITY BARN PUMP STATION

Description: Pump Discharge Pipe

Solve for Diameter

Given Constant Data;

Discharge.................	60,000 gpm 134 cfs
Discharge Velocity........	10 fps (typical)

Required Area = Q/V
= (134 cfs)/(10 fps)
= **13.4 sq. ft.**

Required Diameter = **50 in.**

use 54 in. pipe

<u>SECTION III</u>

DELWOOD PUMPING STATION

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I. Introduction

Proposed modifications to the Delwood Pump Station include additional pumping capacity at the station in order to reduce street flooding in the Delwood, Salmen, and Lakeshore Village subdivisions. Presently, the system of three (3) pumps is able to pump a maximum of 70,000 gallons-per-minute (gpm) or about 156 cubic-feet-per-second (cfs) of water out of the residential subdivision into Bayou Bonfouca. This pumping capacity, while once probably adequate for the undeveloped area, is now insufficient to handle the increased flows of storm water runoff. Because of increasing urbanization in the area, once-pervious soils are being built-on and covered with impervious concrete and asphalt, creating a larger volume of runoff. Urbanization produces a higher value of "C" in the rational equation, which in turn decreases the allowable peak rainfall intensity over the area since the pumping capacity remains constant. For the Delwood Drainage Basin, the current allowable peak rainfall, based on a maximum outflow of 156 cfs, is roughly 1.35 inches per hour (in/hr), approximately equal to the 2-year storm event for the area; in other words, a rain event of greater or equal magnitude, which can be expected to occur once every two years, will result in street flooding in the area.

II. Project Description

The scope of this phase of the project is to analyze increased pumping capacity at the Delwood Pump Station and its hydraulic effects on the water surface elevation in Bayou Bonfouca. In addition, the existing drainage system must be checked to ensure that it can handle the runoff flows currently encountered in the area, keeping in mind that additional drainage relief will be provided by the proposed Southpark Trunk Line (covered in Section IV of this report). The scope includes developing schematics and cost estimates for the pump station and any other affected areas of the project, as well as a design/construction time schedule for the proposed modifications to the area.

III. Existing Conditions

The Delwood Pumping Station serves the Bayou Bonfouca Drainage Basin, lying immediately to the east of the Bayou Liberty Drainage Basin and to the west of the W-14 Canal and Schneider Canal Drainage Basins (see Site Map, Figure 1). The contributory area to the Delwood Pump Station lies at the extreme southeast of the Bayou Bonfouca Drainage Basin, bounded on the west by the New Orleans and Northeastern Railroad, on the south by Lake Pontchartrain, and on the north and east by U.S. Highway 11 (Pontchartrain Rd.). The Delwood Drainage Area is mapped in Figure 2. The sump area is served by three pipes which drain excess rainfall from the subdivisions: two (2) 62" X 102" reinforced concrete arch pipes (R.C.A.P.) running at the far west end of the basin along Front Street, and one (1) 40" X 65" R.C.A.P. trunk line running east to west between Sun Valley Drive and Bermuda Drive (see schematic, Figure 3). The pumped water is transported from the station through three (3) steel discharge tubes, which converge into two larger-diameter steel pipes to form the railroad crossing. These pipes discharge into Bayou Bonfouca.

Two visits to the project site were conducted and a visual inspection was compared with drawing sets titled Delwood Pump Station Addition (Cartier & Associates, Inc., 1982) and Delwood Drainage Improvements - Phase A (City of Slidell Engineering Dept., 1985), both provided by the City of Slidell.

IV. Project Hydraulics

Project hydraulics were based on the St. Tammany Parish Master Drainage Plan, prepared by Burk & Associates, Inc. (1983), and the drawing sets mentioned above. Computations reveal that a 10-year storm produces a flow of approximately 211 cfs over the area (see Appendix A); hydraulic analysis shows that the existing pipe drainage system can easily handle this flow:

Existing Pipe	Quantity	Full-flow Capacity	Total Capacity
62" X 102" RCAP (@ 0.15%)	2	224 cfs	448 cfs
40" X 65" RCAP (@ 0.15%)	1	70 cfs	70 cfs
TOTAL CAPACITY			**518 cfs**

Therefore, the water is getting to the pump station; but the existing pumping capacity is not sufficient to pump the entire quantity out.

V. Proposed Improvements

The current pumping system, which can handle 156 cfs, is deficient by 55 cfs (24,684 gpm) compared with the peak runoff volume of 211 cfs expected from a 10-year storm. An additional 25,000 gpm pump would increase the capacity of the station to around 212 cfs, providing the necessary drainage for a 10-year storm over the area.

Currently, the water surface elevation in the sump ranges from a low value of (-)3.50' NGVD to a high of (+)1.50, with an average elevation of (-)0.75. The sump area is very small; the water depth is typically ranges from about 3 to 4 feet. The small overall volume of the area coupled with the fact that there is no means of gravity outflow from the sump across the railroad tracks into the bayou requires the pumps to be run fairly often to drain the area. Because the new pump configuration is designed to allow the 10-year storm to be fully drained, no additional storage capacity is required in the sump area; and so no modifications are necessary.

According to the 1983 St. Tammany Parish MDP, the section of Bayou Bonfouca at the discharge point of the Delwood Pump Station is very wide and has a capacity of around 4500 cfs. The existing 10-year water surface elevation in the Bayou is in the order of only 2.50 feet, well below the top of its banks. Typically, the water surface elevation is much lower, usually less than 1.00' NGVD. The addition of 56 cfs into the bayou from the Delwood Pumping Station is not sufficient to cause flooding of Bayou Bonfouca's banks, even considering the additional flows previously discussed from the West Diversion Canal improvements (Section I) and the City Barn Pump Station addition (Section II); the existing channel section is adequate.

VI. Conceptual Design

Conceptually, the new pump station is re-designed to handle the total runoff volume expected from a 10-year flood over the Delwood, Salmen, and Lakeshore Village Subdivisions. The drain pipes currently serving the Delwood Pump Station are sufficient to handle flows of this magnitude, so no modifications are necessary to the existing drainage system or the sump.

The new pump can not be installed in the present 36' x 16' steel frame metal pump station building. The building should be extended approximately 15 to 18 feet to accommodate the new pump; construction of this new portion of building will be identical to the existing structure and follow the original design specifications. The new building extension will be along the north-south axis because of restrictions in the east-west direction due to the sump and the railroad right-of-way. If the new pump will require additional suction height than that currently provided by the 8" pump platform at elevation (+)12.25 and the 8" sump slab at elevation (-)9.25, the pump platform should be raised and access stairs provided inside the building. Also required as part of the extension are additional sheet piling and chain link fencing surrounding the building.

The discharge pipe from the new 25,000 gpm pump is required to be 36" in diameter (see Hydraulic Analysis, Appendix B), and is to be constructed of steel (a Manning's roughness of approximately 0.012). This new pipe would be laid on a slope of 0.2% from the pump station, jacked and bored underneath the New Orleans & Northeastern Railroad crossing, and then discharge into Bayou Bonfouca. Since the pipe crossing nearest to the new discharge pipe is also 36 inches, the two can not be manifolded together underneath the tracks. Thus, a separate railroad crossing is required for the new 25,000 pump. To accommodate this new 36" crossing, the existing concrete headwalls on either side of the tracks will have to be extended to the south. The new pipe will be laid on a horizontal angle adequate to cross under the railroad tracks and remain within the banks of the bayou on the opposite side, and a on a vertical angle such that the invert rests at approximately elevation (-)1.00, near the bottom of the bayou.

Finally, the existing fuel tanks are required to be relocated to land away from the pump station building. The new location of the fuel tanks will be to the south of the new building extension. New construction is required to house these tanks, matching the design and specifications of the pump station building.

While details of the above-mentioned additional items go beyond the scope of this report, approximate costs for each have been developed and will be discussed in the following section.

VII. Estimated Design / Construction Cost

Table 2 presents a preliminary tabulation of each construction item covered in this project and its associated cost. The total construction cost is estimated at $510,300; adding design costs ($51,000), construction administration ($20,400), and resident inspection ($72,000) brings the total project cost to an estimated $653,700.

It should be noted that because this cost estimate is preliminary, it may be lacking some required cost items which will be discovered during final design and survey.

VIII. Estimated Design / Construction Time Schedule.

The estimated design time schedule for this project is three months for the project design, which includes preparation of plans, specifications, and contract documents; two months for bidding and award of the contract; and an additional nine months for construction. This estimated time schedule assumes the required funding for the project is available and does not include possible delays caused by appropriation of money to fund the project.

TABLE 1

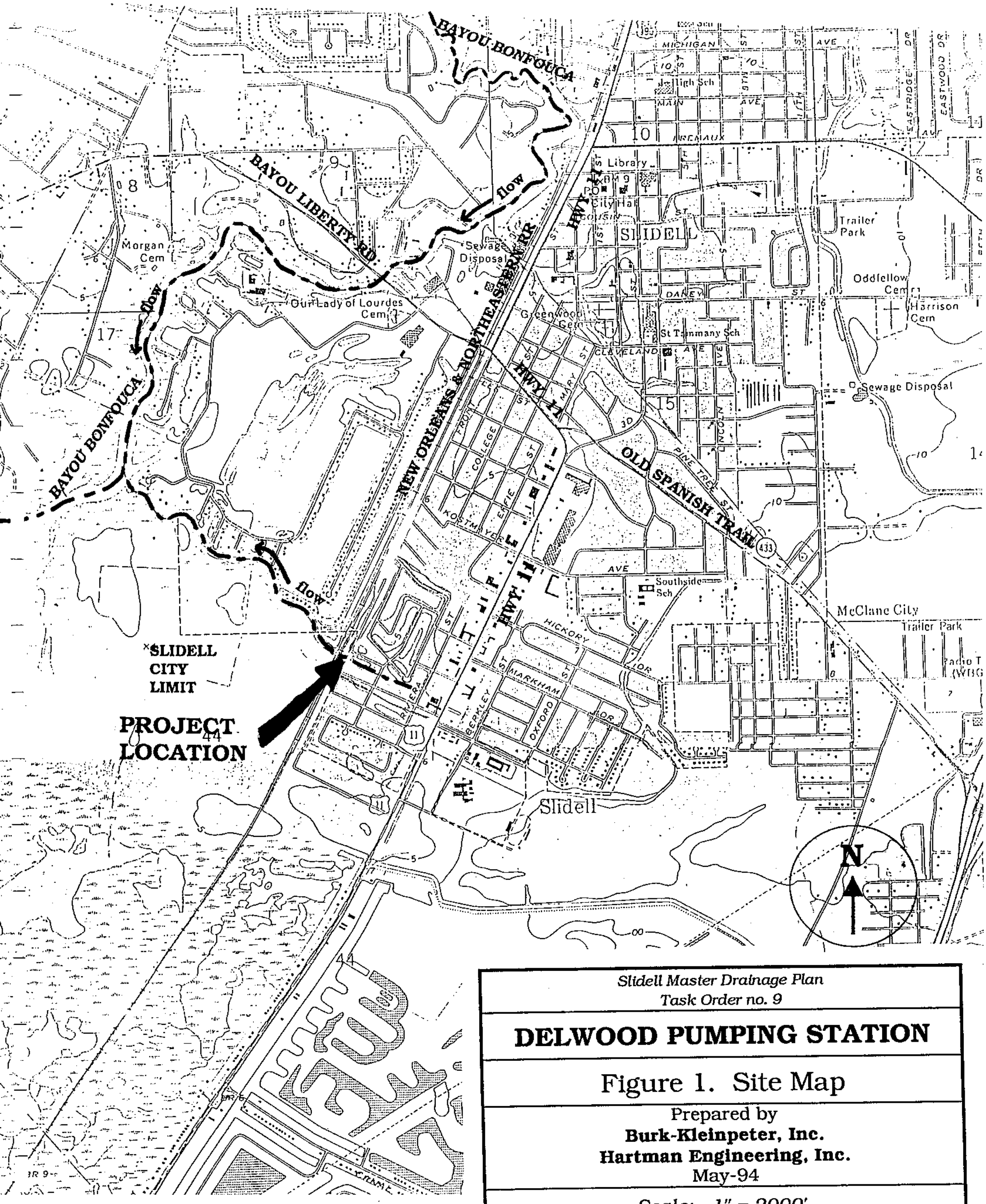
DESIGN PARAMETERS

Parameter	Value	Source
Drainage basin area	350 acres	U.S.G.S. Maps
Ground slope	0.0005 ft/ft	U.S.G.S. Maps
Hydraulic length of watershed	6000 ft.	U.S.G.S. Maps
Runoff coefficient "C"	0.33	Design Manual (Single Family Residential with some unimproved area)
Time of concentration	135 min.	Appendix A
Design rainfall intensity	1.83 in/hr	Appendix B (Based on 135 min.)
Peak flow of runoff	211 cfs	Rational Method
Pump capacity	70,000 gpm 156 cfs	City of Slidell
W.S.E. at pumps	-3.50 (low) +1.50 (high)	Cartier & Assoc. (1982)
Manning's "N" of RCP	0.012	Design Manual
Manning's "N" of steel pipe	0.012	Design Manual

TABLE 2

ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$18,700
Relocating & Enclosing Fuel Tank	Lump	1	Lump Sum	$6,000
25,000 gpm Pump with Enclosure	Each	1	$350,000	$350,000
36" Steel Pipe	Linear Ft	103	$250	$25,750
Jack & Bore New 36" Pipe	Linear Ft	55	$450	$24,750
SUBTOTAL				$425,200
CONTINGENCY @ 20%				$85,100
TOTAL				$510,300



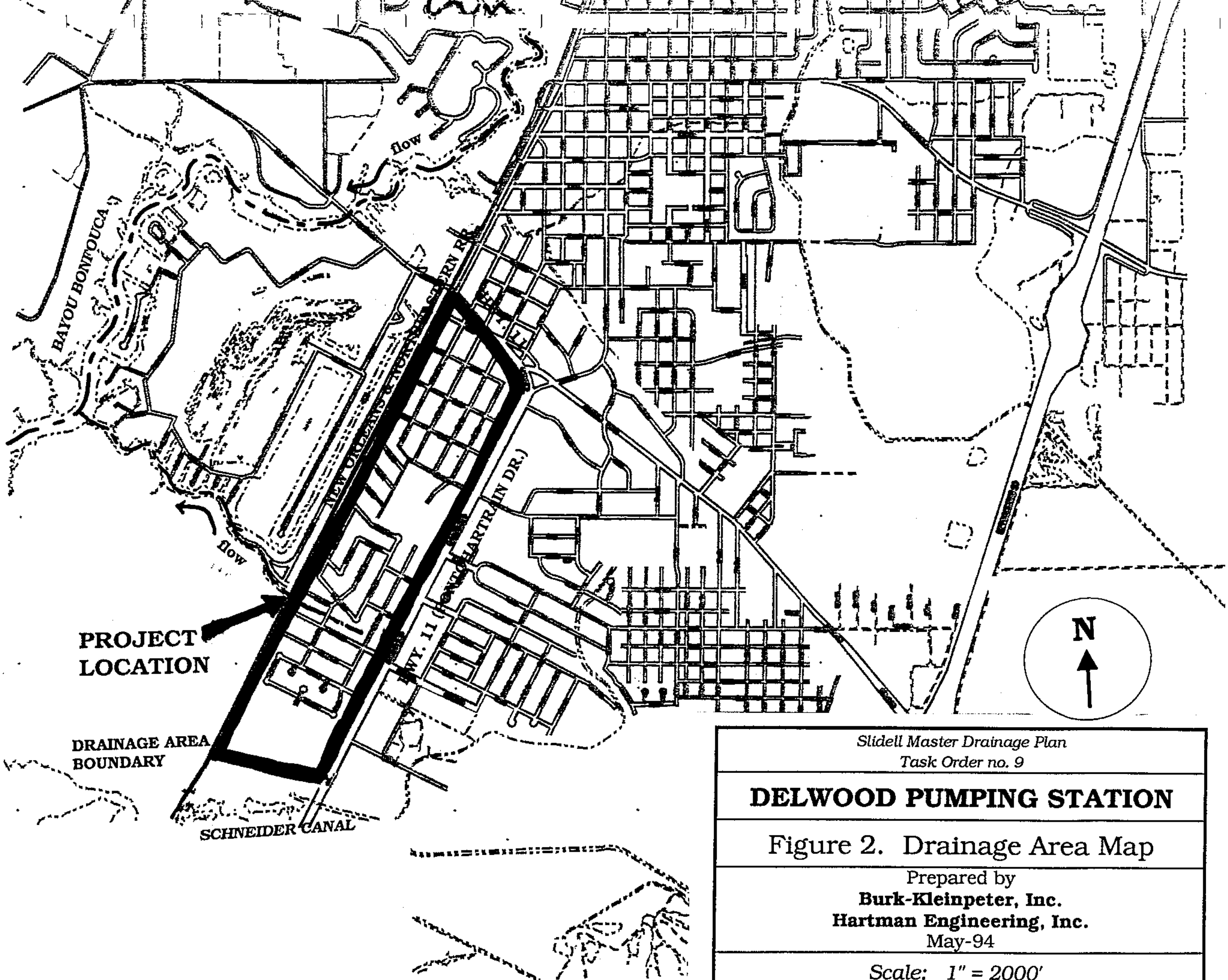
Slidell Master Drainage Plan
Task Order no. 9

DELWOOD PUMPING STATION

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



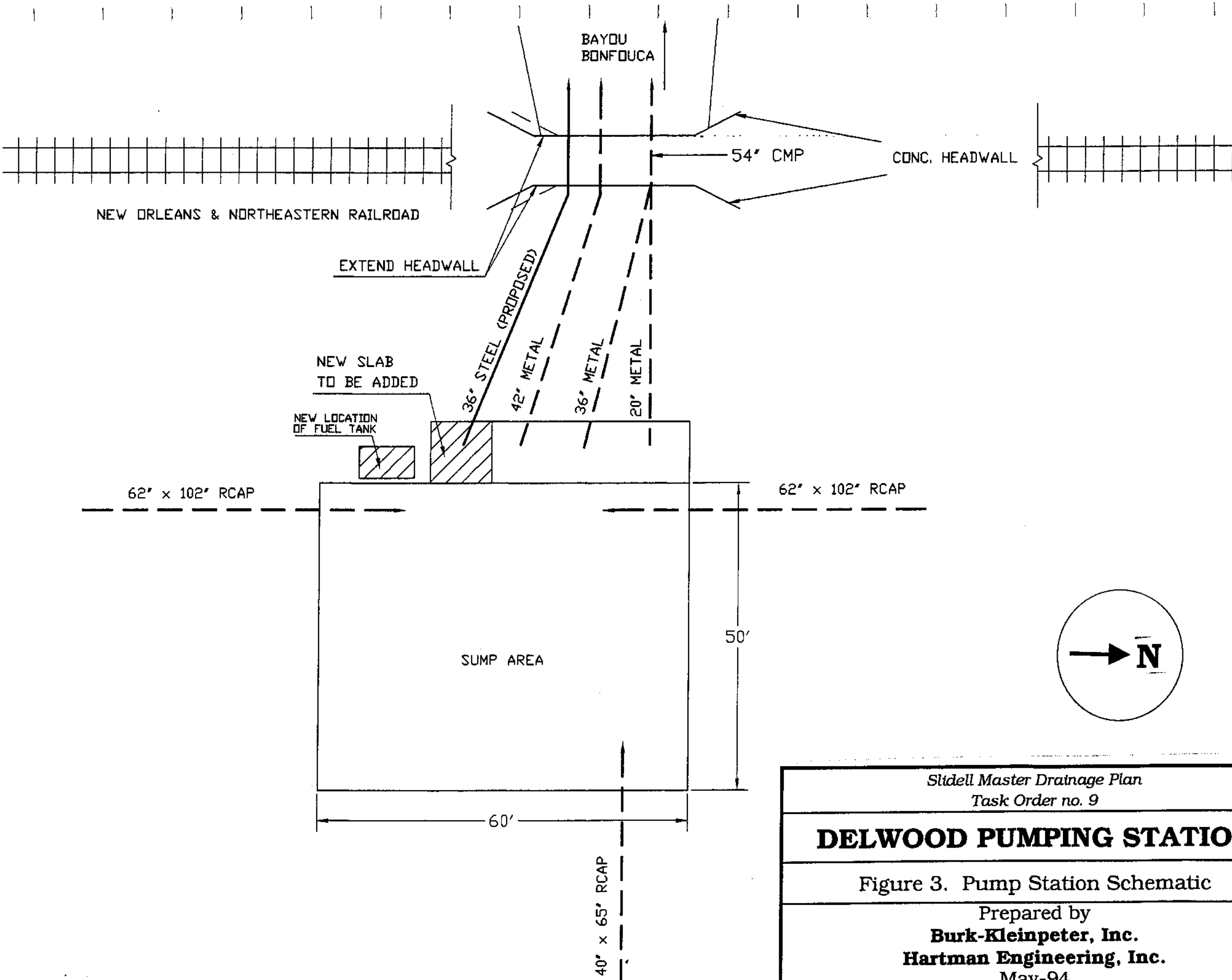
Slidell Master Drainage Plan
Task Order no. 9

DELWOOD PUMPING STATION

Figure 2. Drainage Area Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



Slidell Master Drainage Plan
Task Order no. 9

DELWOOD PUMPING STATION

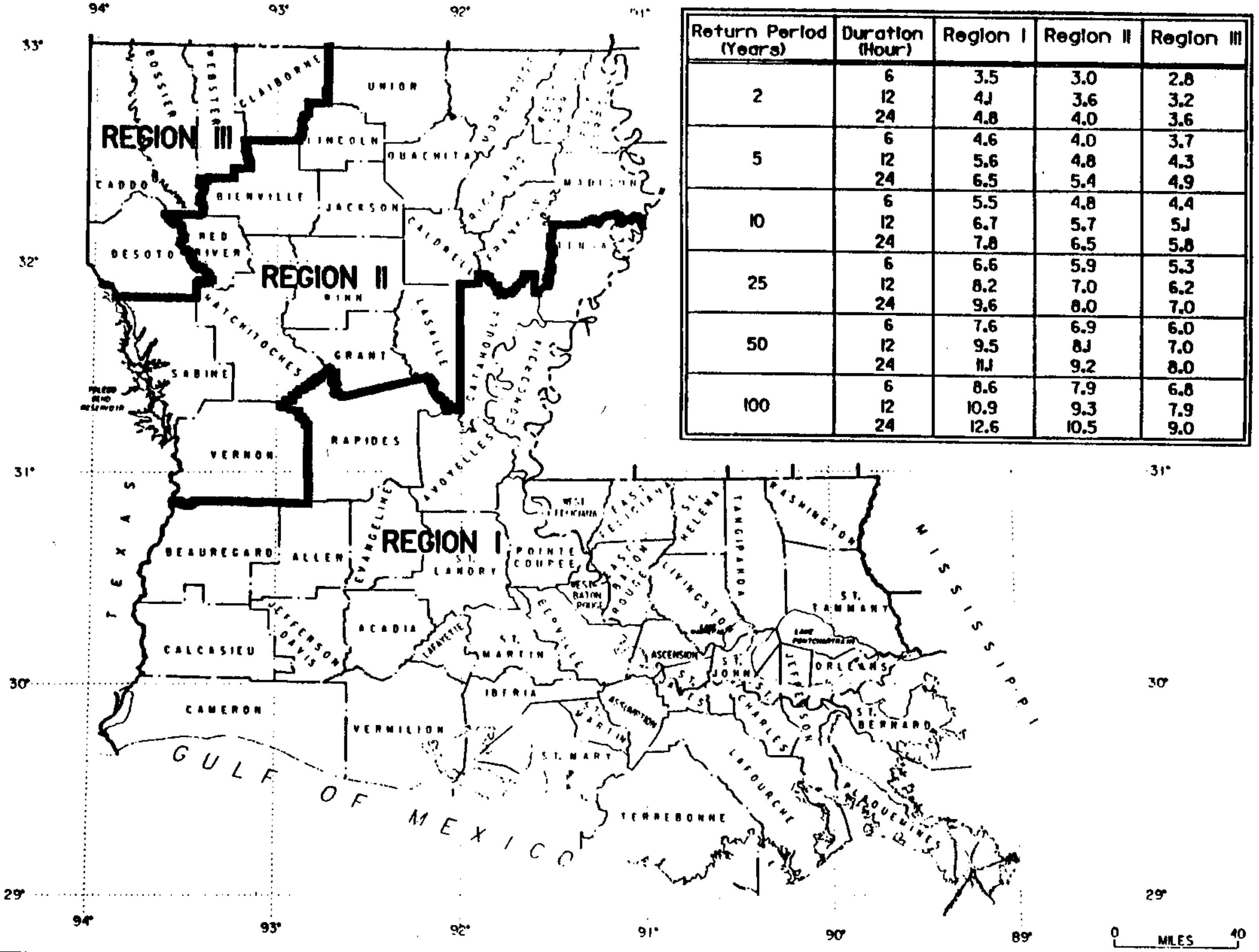
Figure 3. Pump Station Schematic

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Not to Scale

APPENDIX A

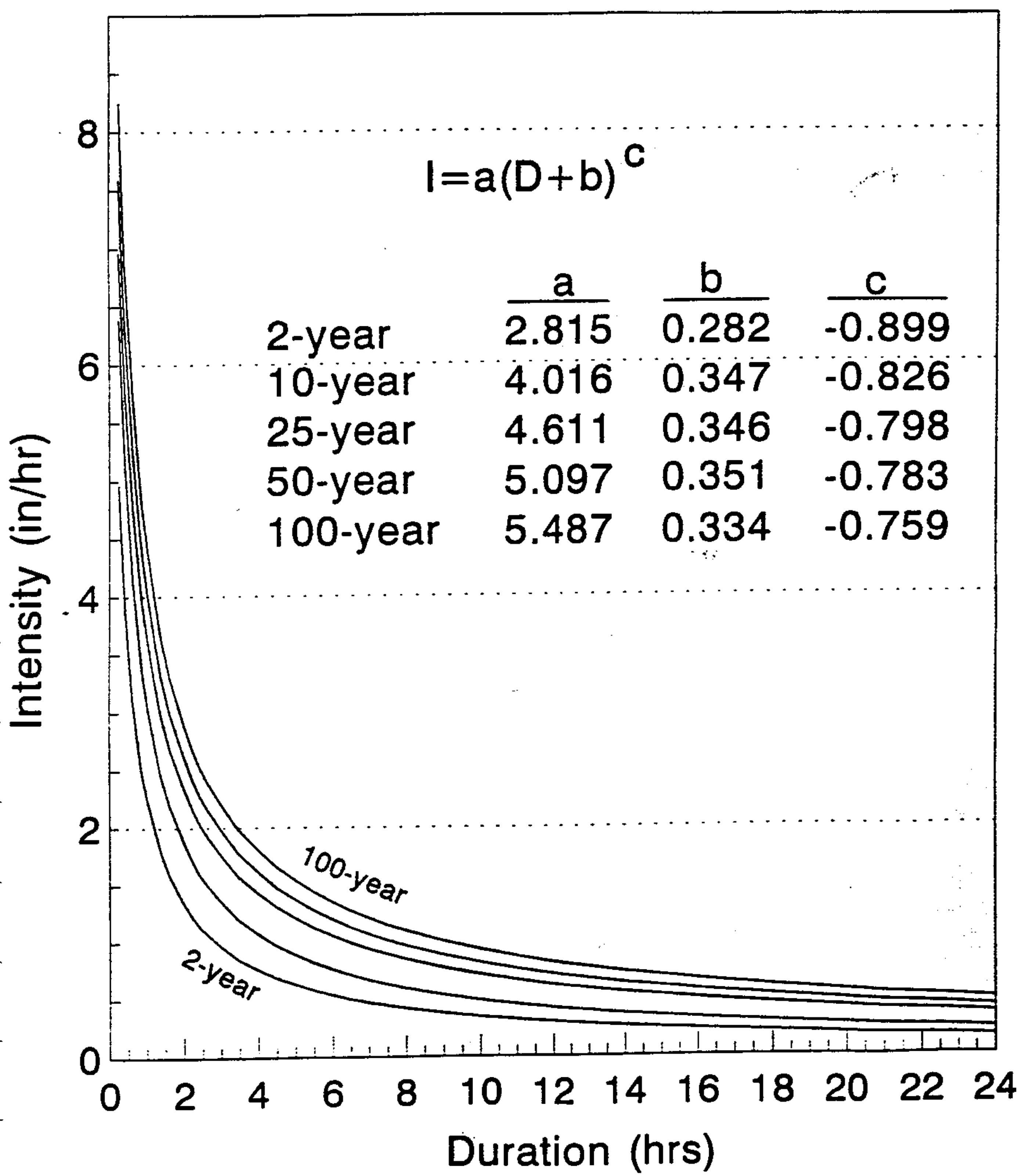
HYDROLOGIC COMPUTATIONS



Return Period (Years)	Duration (Hour)	Region I	Region II	Region III
2	6	3.5	3.0	2.8
	12	4.1	3.6	3.2
	24	4.8	4.0	3.6
5	6	4.6	4.0	3.7
	12	5.6	4.8	4.3
	24	6.5	5.4	4.9
10	6	5.5	4.8	4.4
	12	6.7	5.7	5.1
	24	7.8	6.5	5.8
25	6	6.6	5.9	5.3
	12	8.2	7.0	6.2
	24	9.6	8.0	7.0
50	6	7.6	6.9	6.0
	12	9.5	8.1	7.0
	24	11.1	9.2	8.0
100	6	8.6	7.9	6.8
	12	10.9	9.3	7.9
	24	12.6	10.5	9.0

RAINFALL DEPTH (INCHES) FOR VARIOUS PRECIPITATION REGIONS

Region I


I=a(D+b)^c
a b c
2-year 2.815 0.282 -0.899
10-year 4.016 0.347 -0.826
25-year 4.611 0.346 -0.798
50-year 5.097 0.351 -0.783
100-year 5.487 0.334 -0.759
100-year
2-year
Intensity (in/hr)
0
2
4
6
8
Duration (hrs)
0 2 4 6 8 10 12 14 16 18 20 22 24

RUNOFF CALCULATIONS (RATIONAL METHOD)

$Qp = CIA$

where: Qp = Peak Runoff Flow, [cfs]
C = Runoff Coefficient
I = Rainfall Intensity, [in/hr]
A = Area of Basin, [acres]

Rainfall Intensity

I is based on a duration equal to the Time of Concentration, Tc:
where: $Tc = 0.7039 (L^{0.3917}) (C^{-1.1309}) (S^{-0.1985})$, [min]
L = Overland Flow Length, [ft]
S = Ground Slope, [%]

$Tc = 0.7039 (6000^{0.3917}) (0.33^{-1.1309}) (0.05^{-0.1985})$
= 135 min., or 2.25 hrs.

Then, $I = a (D+b)^c$
where: a,b,c = *f* (region, return period) - see previous pages
D = Duration of Storm = Tc, [hrs]

$I = 4.016 (2.25+0.347)^{-0.826}$
= 1.83 in/hr

$Qp = 0.33$ (1.83 in/hr) (350 acres)
= 211 cfs

Results

Qp = 211 cfs from a 10-year storm

APPENDIX B

HYDRAULIC ANALYSIS

Circular Channel Analysis & Design

Open Channel - Uniform flow

Worksheet Name: DELWOOD PUMP STATION

Description: Pump Discharge Pipe

Solve for Diameter

Given Constant Data;

Discharge.................	20,000 gpm 45 cfs
Discharge Velocity........	10 fps (typical)

Required Area = Q/V
= (45 cfs)/(10 fps)
= **4.5 sq. ft.**

Required Diameter = **30 in.**

use 36 in. pipe

Circular Channel Analysis & Design
Solved with Kutter's Equation

Open Channel - Uniform flow

Worksheet Name: DELWOOD PUMP STATION

Description: 60" RCP Discharge into drainage canal

Solve For Full Flow Capacity

Given Constant Data;

Diameter........... 60.00
Slope.............. 0.0020
Mannings n......... 0.012
Discharge.......... 58430.25

Diameter in	Channel Slope ft/ft	Mannings 'n'	COMPUTED Discharge gpm	COMPUTED Depth in	COMPUTED Velocity fps	COMPUTED Capacity Full gpm
60.00	0.0020	0.012	58430.25	60.00	6.63	58430.25

SECTION IV

SOUTHPARK TRUNK LINE

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I. Introduction

As shown in Figure 1, the Delwood Subdivision is located in the southern Slidell area. The northern section of the Delwood Subdivision is plagued by street flooding caused by an inadequate drainage system within this section of the subdivision. This inadequacy of the drainage system has caused a considerable amount of concern among the residents of this area as well as the City of Slidell. In order to address this issue, we are proposing to install a trunk line, including a series of open grate drainage structures, to collect the storm water runoff from the northern end of the subdivision and divert the collected runoff to an existing drainage ditch (the Front Street collector ditch) which is located along the west side of Front Street.

The Front Street collector ditch will then route the flows approximately 1350 feet to a 66" X 102" Reinforced Concrete Arch Pipe that conveys the runoff to the Delwood Pumping Station sump area.

The existing drainage system within the subdivision includes two trunk lines and the existing Front Street collector ditch which delivers runoff to the Delwood Pumping Station. The two existing trunk lines within the subdivision are located near the center and the southern end of the subdivision. In order to delineate drainage basins within the subdivision, for the scope of this report, we have anticipated that runoff from within the subdivision will be routed through the nearest trunk line.

II. Project Description

This project consists of the evaluation of the existing drainage system within the northern section of the Delwood Subdivision and development of a feasible method to relieve the street flooding experienced in the area. This project is limited to providing a trunk line to remove storm related runoff from the immediate area surrounding the trunk line and does not include the routing of runoff to the location of the trunk line.

III. Existing Drainage Structures

The northern section of the Delwood Subdivision is currently drained by shallow open ditches along both sides of the residential streets within the subdivision. The open ditches are subjected to numerous restrictions including slab span driveways which cross the ditches, restrictions caused by varying inverts of the ditches, and restrictions caused by small diameter pipes, apparently installed by homeowners, replacing the open ditch at some lots.

As shown in Figure 3, the open ditches within the subdivision are drained by two existing trunk lines, (one near the center of the subdivision and one in the southern half of the subdivision) and a larger collector open ditch along the west edge of the Front Street right of way. Runoff from within the northern area of the subdivision generally must flow through the roadside open ditches toward either the trunk line near the center of the subdivision or else westward across the Front Street right of way and into the Front Street collector ditch.

According to the Slidell Engineering Department personnel and observations made during a visit of the site, the existing drainage system in the northern section of the subdivision is not adequate to prevent street flooding within this area. Slidell Engineering Department Records indicate that the approximately 400 feet of 66" X 102" RCPA, which runs from the Delwood Pumping Station northward within the Front Street right of way, was installed and approximately 400 feet of ditch enlarged in 1985. These improvements would be capable of conveying the runoff associated with a 10 year reoccurrence interval storm of 24 hours duration. Therefore, this project consist of providing a method of conveying the runoff from the northern section of the Delwood Subdivision to the Front Street collector ditch.

IV. Proposed Improvements

The proposed improvements to the Delwood Subdivision include improving the Front Street collector ditch capacity from Southpark Drive to the Delwood Pumping Station, installing a reinforced concrete pipe trunk line along the entire length of Southpark Drive and along Brookwood Drive from Southpark Drive to Nassau Drive, and installing drop inlets and catch basins along the trunk line.

The proposed improvements to the Front Street collector ditch will include enlargement of approximately 1000 feet of ditch to have a minimum of one foot bottom width and side slopes of approximately 3 (horizontal) to 1 (vertical) to allow for maintenance mowing of the ditch and roadway right of way. The improvements to the ditch are shown in Figure 4 and will also include stabilization of the ditch banks to prevent erosion.

Additional improvements include the installation of two drainage laterals within the area drained by the trunk line. The first lateral will require installation of approximately 600 feet of 24 inch RCP (reinforced concrete pipe) which will follow Southwood Drive from Whitehall Drive to Brookwood Drive where this lateral will tie into the trunk line. The second lateral will require installation of approximately 300 feet of 30 inch RCP which will follow Nassau Drive from Riviera Drive to Brookwood Drive where this lateral will tie into the trunk line.

As shown in Figure 5, the proposed alignment of the trunk line is on the south side of Southpark Drive and the east side of Brookwood Drive. The proposed line will require installation of a total of 1600 feet of 36, 42, and 48 inch diameter RCP. Along the south side of Southpark Drive approximately 200 feet of 48 inch diameter RCP is proposed to be installed between the Front Street collector ditch and Meadowdale Drive. From the intersection of Southpark Drive and Meadowdale Drive eastward to Brookwood Drive and then southward to Southwood Drive proposed improvements include approximately 1200 feet of 42 inch diameter RCP. From the intersection of

Southwood Drive and Brookwood Drive southward to Nassau Drive proposed improvements include approximately 200 feet of 36 inch diameter RCP.

The installation of the trunk line and two laterals will require the installation of approximately twelve drainage catch basins and eight drop inlets. The twenty proposed drainage catch basins and drop inlets will allow for the efficient and quick interception of storm water runoff in the areas immediately adjacent to the trunk line. However, until the existing open ditches are adjusted to the proper invert elevations and slopes, some ditch and street flooding may still occur in the Delwood Subdivision despite the addition of the trunk line.

Although maintenance of the existing ditches and installation of drainage inlets across the street from the trunk line and lateral crossings at several locations along Southpark Drive and at the intersection of Southwood Drive and Brookwood Drive would further provide additional protection against street flooding, these improvements are not within the scope of this project. However, the proposed layout of drainage catch basins along the trunk line allows for the installation and connection of additional drainage laterals at a later time.

V. Conceptual Design

The conceptual design of the proposed trunk line is for the Front Street collector ditch to be as shallow as possible yet still be able to convey the anticipated quantity of runoff without any significant restriction to the flow in the trunk line. The trunk line, as proposed, is to be installed at a slope that will provide a self cleansing flow velocity of three feet per second when the pipe is flowing half full.

The conceptual design of the proposed trunk line pipe and ditch sizes were based on a ten year recurrence interval storm with a duration of 24 hours. The rainfall intensity for this storm of 7.8 inches was obtained from the revised DOTD Rainfall Depth Tables.

The Front Street ditch and culvert invert elevations were obtained from project "as-built" drawings obtained from the Slidell Engineering Department Records. For the conceptual design of these improvements the design tailwater at the Delwood Pumping Station was assumed to be 2.0 feet N.G.V.D. (National Geodetic Vertical Datum). These values, along with other design criteria, are listed in Table 1.

A flow analysis of the proposed trunk line and Front Street ditch was performed using the "Advanced ICPR" computer program developed by Streamline Technologies. The design parameters shown in Table 1 were used for the analyses. For the computer model, the maximum runoff stage and peak volume of runoff were determined for each of the individual drainage subbasins shown in Figure 2 based on the design peak tailwater stage of 2.0' at the Delwood Pumping Station. The results of these determinations are shown in Table 2.

Table 2 indicates that, by constructing the proposed trunk line, the street flooding within the northern section of the Delwood Subdivision could be relieved during the design storm. As stated previously, this analysis relies on overland flow from some areas not adjacent to the trunk line and the installation of laterals between isolated areas of the subdivision and the trunk line to relieve some isolated ponding areas.

For this conceptual design the possibility of conflicts with existing utilities does exist and will require additional investigation during the design phase of this project. An additional cost was added for this item.

VI. Environmental Consideration

In the design phase of this project there will be several environmental considerations to be addressed which go beyond the scope of this preliminary Plan of Action. A listing of several of these considerations are as follows:

> Any existing storm water discharge permits should be reviewed to ensure that this project will not violate these permits.
>
> The City of Slidell should consider contacting the Environmental Protection Agency to determine if this redirection of flows will require any additional storm water discharge permitting.

The existing Delwood Pumping Station and the remainder of the drainage basin should be investigated to ensure that the runoff from this site does not adversely affect the operation of the pumping station or cause flooding within other areas of the drainage basin.

In addition to the listed concerns, the design phase of this project may uncover additional environmental liabilities which could delay the design or construction of this project.

VII. Estimated Design / Construction Cost

The estimated construction cost for the Delwood Subdivision Trunk Line Project is $551,280. A breakdown of this cost estimate is outlined in the construction cost estimate presented in Table 3. The estimate is based on the unit costs given in the table and does not include design, construction administration, or resident inspection fees. For the purposes of this report, $55,100 was estimated for design fees, while $22,100 and $30,200 were estimated for construction administration and resident inspection, respectively. The addition of these fees would bring the total estimated project cost to approximately $658,700.

As an alternate to the Front Street open ditch, a cost estimate which provides for enclosing the ditch with 54 and 60 inch diameter RCP has been provided as Table 4. These improvements would increase the project cost by about 67% and would bring the total project cost to approximately $1,098,000.

The 54 and 60 inch diameter RCP is adequate to convey runoff from within the area serviced by the trunk line. However, these pipe sizes may not be adequate to handle additional runoff which is presently conveyed by the open ditch but is generated from areas not serviced by the trunk line. Therefore this alternative may contribute to some localized ditch and street flooding in areas not serviced by the proposed trunk line.

These cost estimates do not include the installation of additional trunk line laterals which could alleviate isolated ponding areas. As mentioned previously, these laterals may be necessary in the future after the trunk line is installed; but they are beyond the scope of this report.

VIII. Estimated Design / Construction Time Schedule

The estimated design time schedule for this project is three months for the project design which includes preparation of plans, specifications, and contract documents, two months for bidding and award of the contract, and an additional three months for construction. This estimated time schedule assumes the required funding for the project is available and does not include possible delays caused by appropriation of money to fund this project.

If extensive utility conflicts are encountered which require the relocation of a complete utility service within the project limits, the construction cost estimate and time schedule should be revised to reflect these changes.

TABLE 1

DESIGN PARAMETERS

Parameter	Value	Source
10 Year - 24 Hour Storm	7.8 in.	D.O.T.D. Design Manual
Runoff Curve Number	85	Site Visit Observations
Tailwater at Delwood P.S.	2.0' N.G.V.D.	Slidell Engineering Dept.
Self Cleansing Velocity	3.0 ft/sec	Slidell Engineering Dept.
Culvert Invert Slope (36")	0.0010 ft/ft	Self Cleansing Velocity
Culvert Invert Slope (42")	0.00081 ft/ft	Self Cleansing Velocity
Culvert Invert Slope (48")	0.00070 ft/ft	Self Cleansing Velocity
Culvert Invert Slope (60")	0.00052 ft/ft	Self Cleansing Velocity
Culvert Invert Slope (72")	0.00041 ft/ft	Self Cleansing Velocity
Manning's "N" (culvert sections)	0.013	Design Manual
Manning's "N" (ditch sections)	0.030	Design Manual
Average Surface Elevation	5.0' N.G.V.D.	U.S.G.S. Maps

TABLE 2

IMPROVED CONDITIONS

BASIN MAXIMUMS REPORT

BASIN NUMBER	STAGE (ft)	AREA (acres)	RUNOFF (cfs)	OUTFLOW (cfs)
1	5.39	4.73	12.1	4.7
2	5.35	4.73	12.7	9.3
3	5.25	2.25	8.4	3.3
4	5.34	4.36	11.7	4.5
5	5.31	3.76	10.1	8.5
6	5.25	4.00	13.6	25.3
7	4.95	4.15	10.6	7.6
8	5.01	4.51	16.8	28.1
9	4.66	3.98	14.9	36.6
10	3.94	1.90	7.4	42.1
11	3.44	1.36	5.7	46.9
12	3.11	1.27	5.4	51.1

TABLE 3

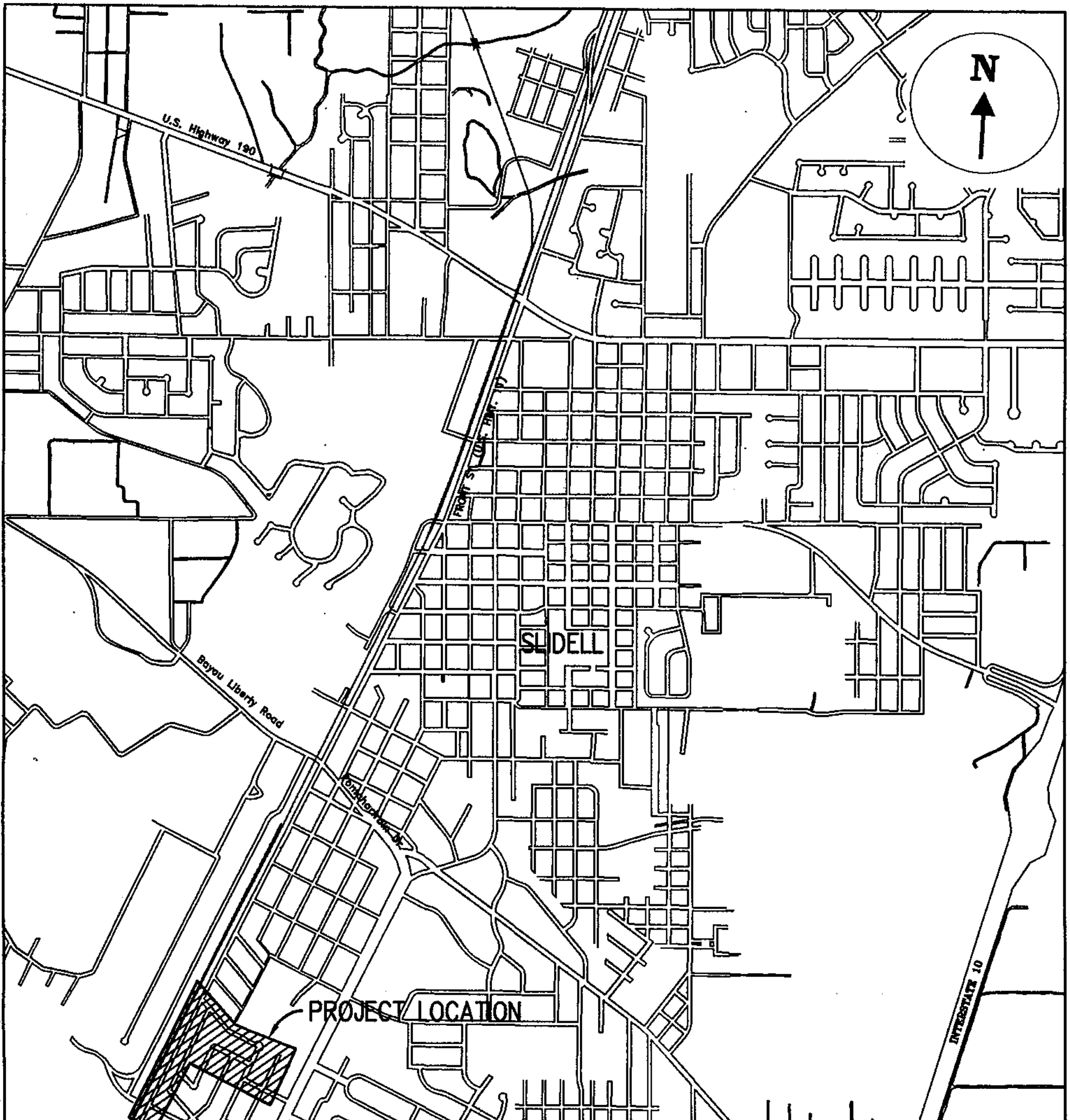
ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump Sum	1	5 %	$23,000
Ditch Excavation	Cubic Yard	2900	$8	$23,200
24" RCP Lateral (Installed)	Linear Ft.	600	$65	$39,000
30" RCP Lateral (Installed)	Linear Ft.	300	$80	$24,000
36" RCP (Installed)	Linear Ft.	200	$100	$20,000
42" RCP (Installed)	Linear Ft.	1200	$120	$144,000
48" RCP (Installed)	Linear Ft.	200	$160	$32,000
Drainage Catch Basin	Each	12	$4,500	$54,000
Drop Inlet	Each	8	$3,500	$28,000
Driveway Removal & Replacement	Sq. Yard	500	$35	$17,500
Roadway Removal & Replacement	Sq. Yard	234	$50	$11,700
Utility Relocation	Lump Sum	1	$40,000	$40,000
Seeding	Acre	2	$1,500	$3,000
SUBTOTAL				$459,400
CONTINGENCY @ 20%				$91,880
TOTAL				$551,280

TABLE 4

ESTIMATED DESIGN / CONSTRUCTION COST
ENCLOSED FRONT STREET DITCH ALTERNATIVE

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump	$38,300
Ditch Excavation	Cubic Yd.	2900	$8	$23,200
24" RCP Lateral (Installed)	Linear Ft.	600	$65	$39,000
30" RCP Lateral (Installed)	Linear Ft.	300	$80	$24,000
36" RCP (Installed)	Linear Ft.	200	$100	$20,000
42" RCP (Installed)	Linear Ft.	1200	$120	$144,000
48" RCP (Installed)	Linear Ft.	200	$160	$32,000
54" RCP (Installed)	Linear Ft.	700	$175	$122,500
60" RCP (Installed)	Linear Ft.	650	$210	$136,500
Drainage Catch Basin	Each	16	$4,500	$72,000
Drop Inlet	Each	12	$3,500	$42,000
Drive Removal & Replacement	Sq. Yard	500	$35	$17,500
Road Removal & Replacement	Sq. Yard	234	$50	$11,700
Utility Relocation	Lump	1	$40,000	$40,000
Seeding	Acre	2	$1,500	$3,000
SUBTOTAL				$765,700
CONTINGENCY @ 20%				$153,140
TOTAL				$918,840



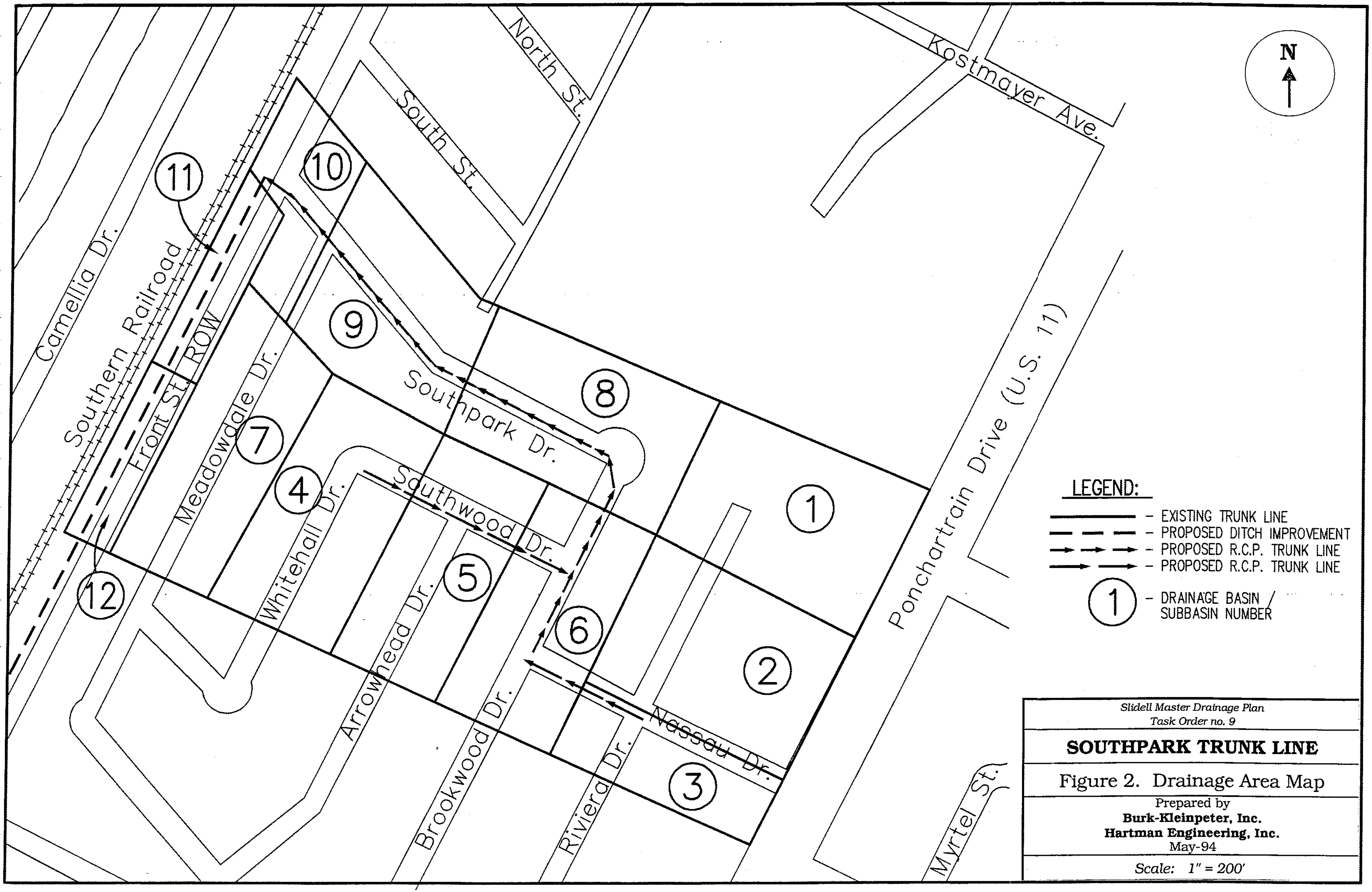
Slidell Master Drainage Plan
Task Order no. 9

SOUTHPARK TRUNK LINE

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



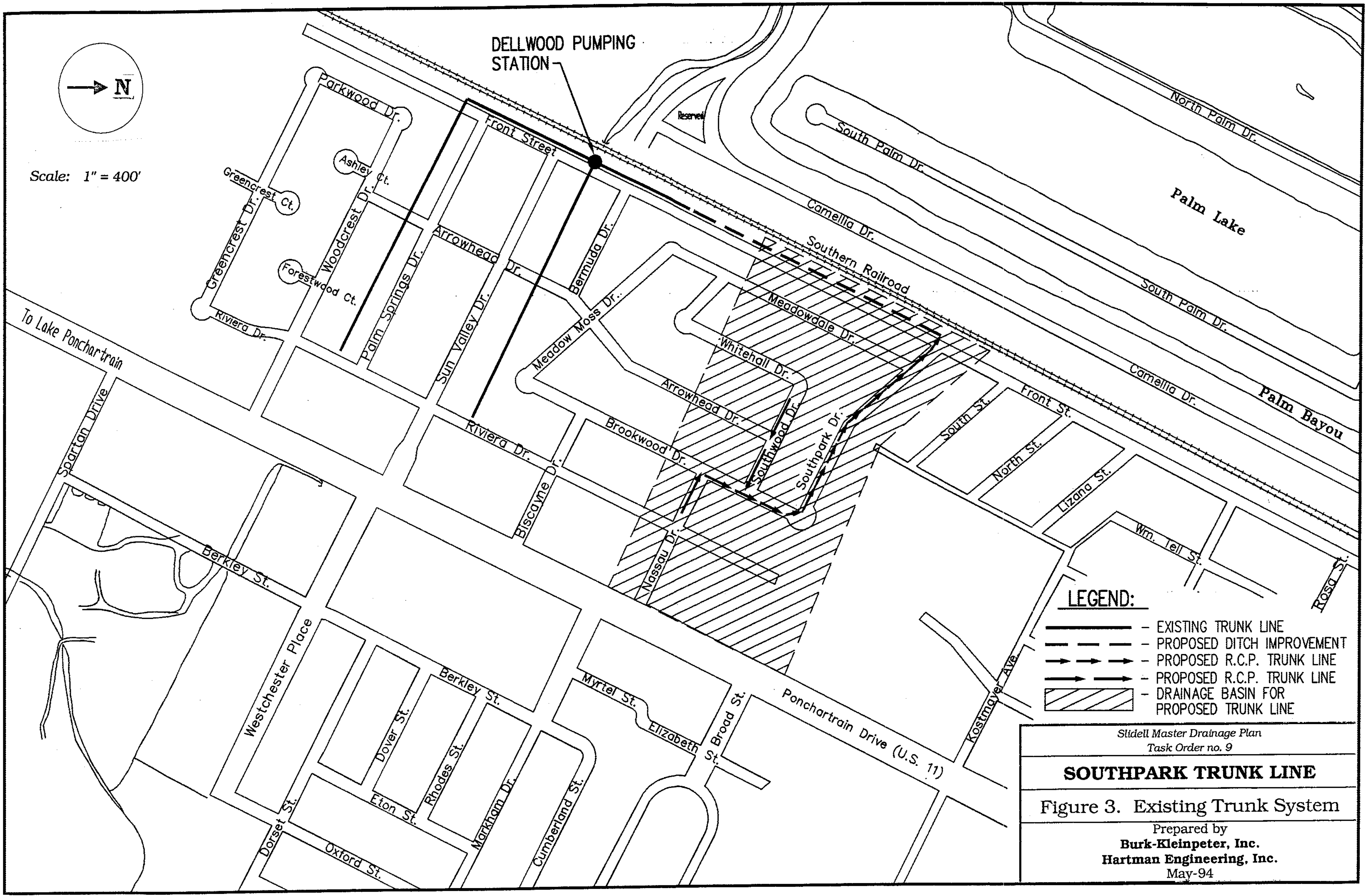
Slidell Master Drainage Plan
Task Order no. 9

SOUTHPARK TRUNK LINE

Figure 2. Drainage Area Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 200'

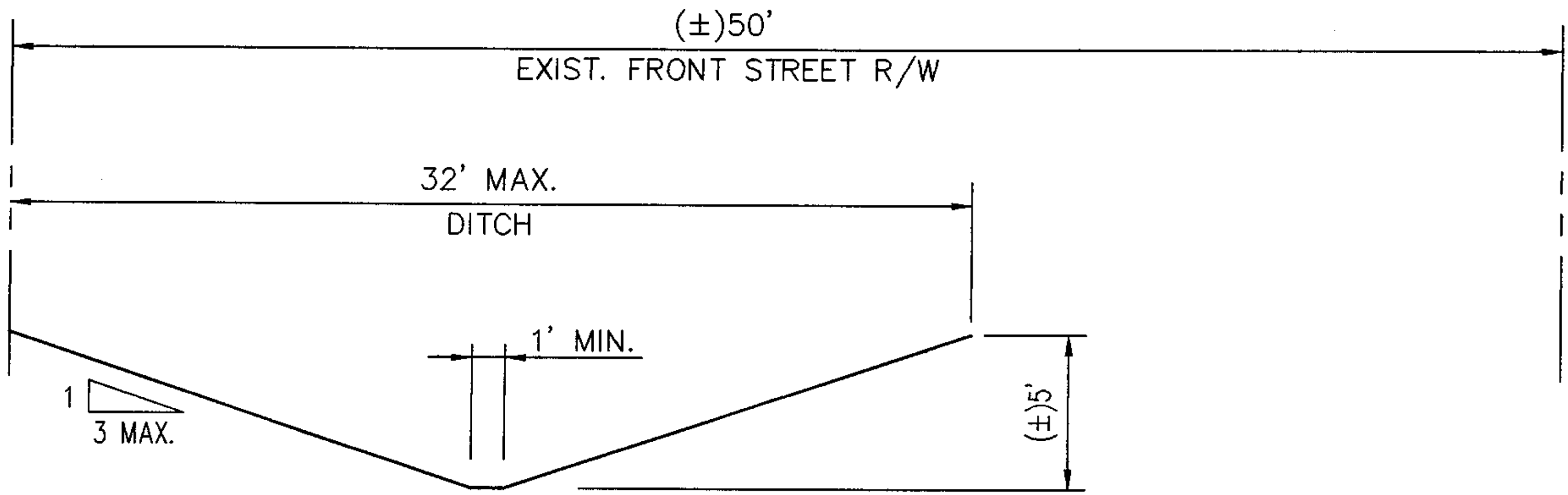


Slidell Master Drainage Plan
Task Order no. 9

SOUTHPARK TRUNK LINE

Figure 3. Existing Trunk System

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94



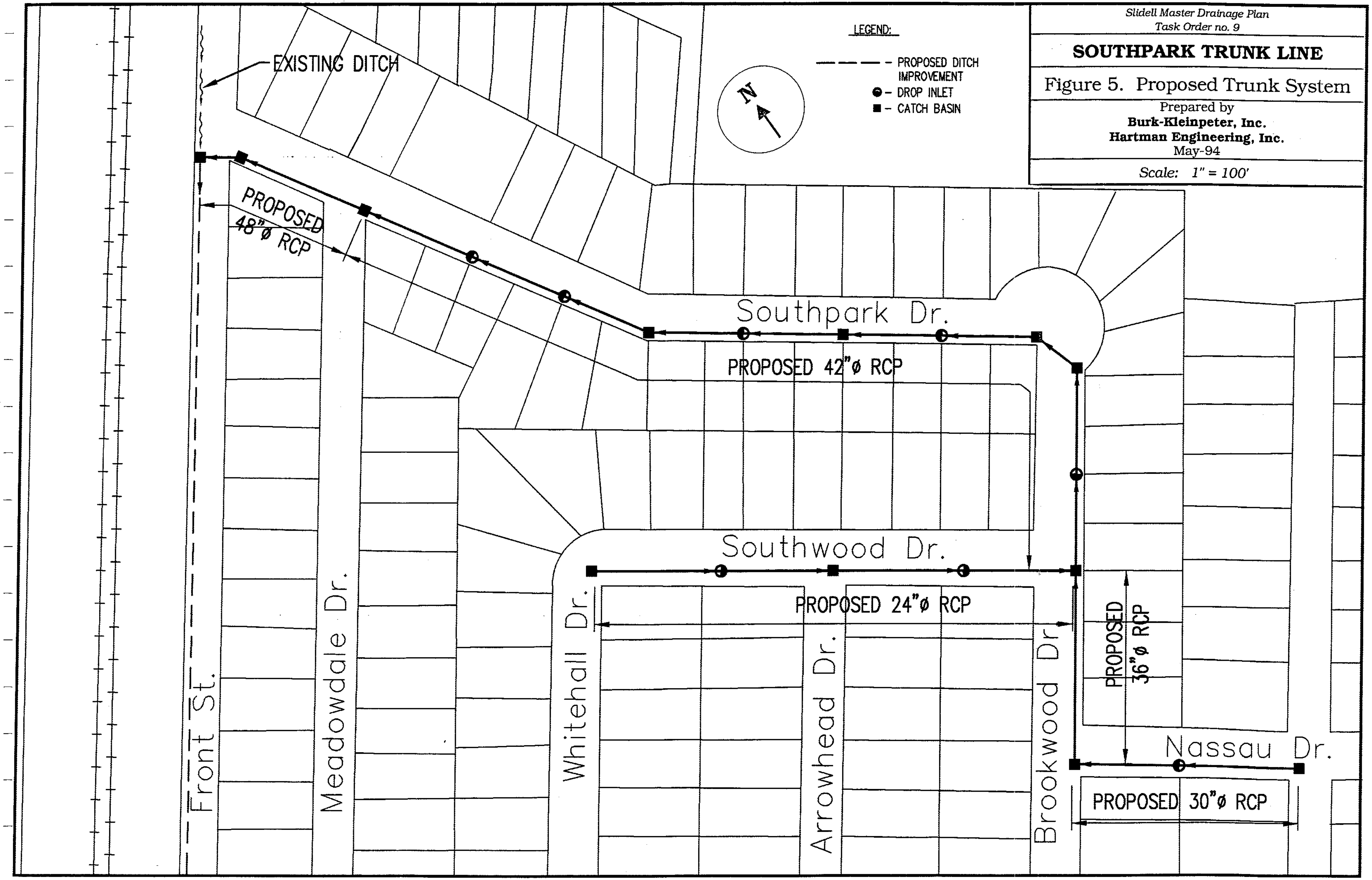
Slidell Master Drainage Plan
Task Order no. 9

SOUTHPARK TRUNK LINE

Figure 4. Proposed Canal Cross-Section

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Not to Scale


Slidell Master Drainage Plan
Task Order no. 9
SOUTHPARK TRUNK LINE
Figure 5. Proposed Trunk System
Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94
Scale: 1" = 100'
LEGEND:
PROPOSED DITCH IMPROVEMENT
DROP INLET
CATCH BASIN
N
EXISTING DITCH
PROPOSED 48"ø RCP
Southpark Dr.
PROPOSED 42"ø RCP
Southwood Dr.
PROPOSED 24"ø RCP
PROPOSED 36"ø RCP
Nassau Dr.
PROPOSED 30"ø RCP
Front St.
Meadowdale Dr.
Whitehall Dr.
Arrowhead Dr.
Brookwood Dr.

APPENDIX A

HYDRAULIC COMPUTATIONS

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

CONTROL PARAMETERS
=================

START TIME: 9.00
END TIME: 16.00

TO TIME (hours)	SIMULATION INC (secs)	PRINT INC (mins)
10.00	.50	15.00
14.00	.30	10.00
16.00	.50	15.00

RUNOFF HYDROGRAPH FILE: DEFAULT
OFFSITE HYDROGRAPH FILE: DEFAULT
BOUNDARY DATABASE FILE: NONE

NOTE:

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

BASIN NAME	1	2	3	4	5
NODE NAME	1	2	3	4	5
UNIT HYDROGRAPH	UH323	UH323	UH323	UH323	UH323
PEAKING FACTOR	323.	323.	323.	323.	323.
RAINFALL FILE	SCSIII	SCSIII	SCSIII	SCSIII	SCSIII
RAIN AMOUNT (in)	7.80	7.80	7.80	7.80	7.80
STORM DURATION (hrs)	24.00	24.00	24.00	24.00	24.00
AREA (ac)	4.73	4.73	2.25	4.36	3.76
CURVE NUMBER	85.00	85.00	85.00	85.00	85.00
DCIA (%)	.00	.00	.00	.00	.00
TC (mins)	40.00	35.90	18.20	35.90	35.90
LAG TIME (hrs)	.00	.00	.00	.00	.00
BASIN STATUS	ONSITE	ONSITE	ONSITE	ONSITE	ONSITE

BASIN	QMX (cfs)	TMX (hrs)	VOL (in)	NOTES
1	12.08	12.53	6.02	
2	12.93	12.45	6.02	
3	8.47	12.29	6.02	
4	11.92	12.45	6.02	
5	10.28	12.45	6.02	

BASIN NAME	6	7	8	9	10
NODE NAME	6	7	8	9	10
UNIT HYDROGRAPH	UH323	UH323	UH323	UH323	UH323
PEAKING FACTOR	323.	323.	323.	323.	323.
RAINFALL FILE	SCSIII	SCSIII	SCSIII	SCSIII	SCSIII
RAIN AMOUNT (in)	7.80	7.80	7.80	7.80	7.80
STORM DURATION (hrs)	24.00	24.00	24.00	24.00	24.00
AREA (ac)	4.00	4.15	4.51	3.98	1.90
CURVE NUMBER	85.00	85.00	85.00	85.00	85.00
DCIA (%)	.00	.00	.00	.00	.00
TC (mins)	23.00	40.00	18.20	18.20	15.30
LAG TIME (hrs)	.00	.00	.00	.00	.00
BASIN STATUS	ONSITE	ONSITE	ONSITE	ONSITE	ONSITE

BASIN	QMX (cfs)	TMX (hrs)	VOL (in)	NOTES
6	13.63	12.32	6.02	
7	10.60	12.53	6.02	
8	16.98	12.29	6.02	
9	14.98	12.29	6.02	
10	7.64	12.27	6.02	

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

BASIN NAME	11	12
NODE NAME	11	12
UNIT HYDROGRAPH	UH323	UH323
PEAKING FACTOR	323.	323.
RAINFALL FILE	SCSIII	SCSIII
RAIN AMOUNT (in)	7.80	7.80
STORM DURATION (hrs)	24.00	24.00
AREA (ac)	1.36	1.27
CURVE NUMBER	85.00	85.00
DCIA (%)	.00	.00
TC (mins)	10.50	10.50
LAG TIME (hrs)	.00	.00
BASIN STATUS	ONSITE	ONSITE

BASIN	QMX (cfs)	TMX (hrs)	VOL (in)	NOTES
11	6.07	12.25	6.02	
12	5.67	12.25	6.02	

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5/3/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
1	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.240
						5.300	2.400
2	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.240
						5.300	2.400
3	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.120
						5.300	1.200
4	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.220
						5.300	2.200
5	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.190
						5.300	1.900
6	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.200
						5.300	2.000
7	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.210
						5.300	2.100
8	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.230
						5.300	2.300
9	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.200
						5.300	2.000

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

NODE NAME	NODE TYPE	INI STAGE (ft)	X-COOR (ft)	Y-COOR (ft)	LENGTH (ft)	STAGE (ft)	AR/TM/STR (ac/hr/af)
10	AREA	3.500	.000	.000	.000	3.500	.000
						4.500	.000
						4.800	.100
						5.300	1.000
B	AREA	2.000	.000	.000	.000	2.000	.000
						4.500	.000
						4.800	.050
						5.300	.100
11	AREA	2.000	.000	.000	.000	2.000	.000
						4.500	.000
						4.800	.070
						5.300	.700
12	AREA	2.000	.000	.000	.000	2.000	.000
						4.500	.000
						4.800	.070
						5.300	.700
PS	TIME	2.000	.000	.000	.000	2.000	.000
						2.000	1.000
						2.000	30.000

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FIRST RUN
5/3/94

```
>>REACH NAME         : A
  FROM NODE          : 1
  TO NODE            : 2
  REACH TYPE         : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL     : FREE
   BOT. WIDTH (ft):    1.000   LEFT SS (h/v):    3.000   RGHT SS (h/v):    3.000
       LENGTH (ft):  400.000 U/S INVERT (ft):    3.700 D/S INVERT (ft):    3.500
        MANNING N:     .030 MAX. DEPTH (ft):    6.000

          NOTE:

>>REACH NAME         : B
  FROM NODE          : 2
  TO NODE            : 6
  REACH TYPE         : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL     : FREE
   BOT. WIDTH (ft):    1.000   LEFT SS (h/v):    3.000   RGHT SS (h/v):    3.000
       LENGTH (ft):  400.000 U/S INVERT (ft):    3.500 D/S INVERT (ft):    3.500
        MANNING N:     .030 MAX. DEPTH (ft):    6.000

          NOTE:

>>REACH NAME         : C
  FROM NODE          : 3
  TO NODE            : 6
  REACH TYPE         : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL     : FREE
   BOT. WIDTH (ft):    1.000   LEFT SS (h/v):    3.000   RGHT SS (h/v):    3.000
       LENGTH (ft):  500.000 U/S INVERT (ft):    3.700 D/S INVERT (ft):    3.500
        MANNING N:     .030 MAX. DEPTH (ft):    6.000

          NOTE:

>>REACH NAME         : D
  FROM NODE          : 4
  TO NODE            : 5
  REACH TYPE         : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION     : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL     : FREE
   BOT. WIDTH (ft):    1.000   LEFT SS (h/v):    3.000   RGHT SS (h/v):    3.000
       LENGTH (ft):  350.000 U/S INVERT (ft):    3.700 D/S INVERT (ft):    3.500
        MANNING N:     .030 MAX. DEPTH (ft):    6.000

          NOTE:
```

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FIRST RUN
5/3/94

```
>>REACH NAME        : E
  FROM NODE         : 5
  TO NODE           : 6
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : FREE
   BOT. WIDTH (ft):   1.000   LEFT SS (h/v):   3.000   RGHT SS (h/v):   3.000
       LENGTH (ft): 250.000 U/S INVERT (ft):   3.500 D/S INVERT (ft):   3.500
         MANNING N:    .030 MAX. DEPTH (ft):   6.000

              NOTE:

>>REACH NAME        : G
  FROM NODE         : 7
  TO NODE           : 9
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : FREE
   BOT. WIDTH (ft):   1.000   LEFT SS (h/v):   3.000   RGHT SS (h/v):   3.000
       LENGTH (ft): 300.000 U/S INVERT (ft):   3.600 D/S INVERT (ft):   3.500
         MANNING N:    .030 MAX. DEPTH (ft):   5.000

              NOTE:

>>REACH NAME        : K
  FROM NODE         : B
  TO NODE           : 11
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : FREE
   BOT. WIDTH (ft):   2.000   LEFT SS (h/v):   3.000   RGHT SS (h/v):   3.000
       LENGTH (ft): 250.000 U/S INVERT (ft):   1.000 D/S INVERT (ft):    .700
         MANNING N:    .030 MAX. DEPTH (ft):   5.000

              NOTE:

>>REACH NAME        : L
  FROM NODE         : 11
  TO NODE           : 12
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : FREE
   BOT. WIDTH (ft):   2.000   LEFT SS (h/v):   3.000   RGHT SS (h/v):   3.000
       LENGTH (ft): 450.000 U/S INVERT (ft):    .700 D/S INVERT (ft):    .400
         MANNING N:    .030 MAX. DEPTH (ft):   5.000

              NOTE:
```

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FIRST RUN
5/3/94

```
>>REACH NAME        : F
  FROM NODE         : 6
  TO NODE           : 8
  REACH TYPE        : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :                                          LENGTH (ft): 500.000
        SPAN (in):  42.000        RISE (in):   42.000           MANNING N:     .013
  U/S INVERT (ft):    -.400 D/S INVERT (ft):    -.700
      ENTRNC LOSS:     .500   # OF CULVERTS:    1.000

  POSITION A        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):    5.100  CREST LN. (ft):  20.000         WEIR COEF.:    2.800
         RESERVED:********         RESERVED:********            RESERVED:********

  POSITION B        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):********  CREST LN. (ft):********          WEIR COEF.:********
         RESERVED:********         RESERVED:********            RESERVED:********

              NOTE:

>>REACH NAME        : H
  FROM NODE         : 8
  TO NODE           : 9
  REACH TYPE        : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :                                          LENGTH (ft): 650.000
        SPAN (in):  42.000        RISE (in):   42.000           MANNING N:     .013
  U/S INVERT (ft):    -.700 D/S INVERT (ft):   -1.100
      ENTRNC LOSS:     .500   # OF CULVERTS:    1.000

  POSITION A        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):    5.100  CREST LN. (ft):  20.000         WEIR COEF.:    2.800
         RESERVED:********         RESERVED:********            RESERVED:********

  POSITION B        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):********  CREST LN. (ft):********          WEIR COEF.:********
         RESERVED:********         RESERVED:********            RESERVED:********

              NOTE:
```

```
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        FIRST RUN
        5/3/94

>>REACH NAME        : I
  FROM NODE         : 9
  TO NODE           : 10
  REACH TYPE        : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :
        SPAN (in):  42.000       RISE (in):  42.000     LENGTH (ft): 350.000
   U/S INVERT (ft): -1.100 D/S INVERT (ft):  -1.300      MANNING N:     .013
      ENTRNC LOSS:    .500   # OF CULVERTS:   1.000

  POSITION A        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):   5.100  CREST LN. (ft):  20.000    WEIR COEF.:   2.800
         RESERVED:********        RESERVED:********       RESERVED:********

  POSITION B        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):********  CREST LN. (ft):********    WEIR COEF.:********
         RESERVED:********        RESERVED:********       RESERVED:********

            NOTE:

>>REACH NAME        : J
  FROM NODE         : 10
  TO NODE           : B
  REACH TYPE        : CULVERT, CIRCULAR w/ ROADWAY
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  TURBO SWITCH      : OFF

  CULVERT DATA      :
        SPAN (in):  48.000       RISE (in):  48.000     LENGTH (ft): 100.000
   U/S INVERT (ft): -1.300 D/S INVERT (ft):  -1.400      MANNING N:     .013
      ENTRNC LOSS:    .500   # OF CULVERTS:   1.000

  POSITION A        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):   5.100  CREST LN. (ft):  20.000    WEIR COEF.:   2.800
         RESERVED:********        RESERVED:********       RESERVED:********

  POSITION B        : RECTANGULAR ROADWAY/BERM WEIR
    CREST EL. (ft):********  CREST LN. (ft):********    WEIR COEF.:********
         RESERVED:********        RESERVED:********       RESERVED:********

            NOTE:
```

```
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        FIRST RUN
        5/3/94

>>REACH NAME        : M
  FROM NODE         : 12
  TO NODE           : PS
  REACH TYPE        : TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
  FLOW DIRECTION    : POSITIVE AND NEGATIVE FLOWS ALLOWED
  OUTLET CONTROL    : FREE
   BOT. WIDTH (ft):    2.000   LEFT SS (h/v):    3.000   RGHT SS (h/v):    3.000
       LENGTH (ft): 650.000 U/S INVERT (ft):     .400 D/S INVERT (ft):     .000
         MANNING N:    .030 MAX. DEPTH (ft):    5.000

             NOTE:
```

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

NODAL MAXIMUM CONDITIONS REPORT

NODE ID	STAGE (ft)	VOLUME (af)	INFLOW RUNOFF (cfs)	INFLOW OFFSITE (cfs)	INFLOW OTHER (cfs)	OUTFLOW (cfs)
1	5.39	.86	12.07	.00	.00	4.70
2	5.35	.88	12.71	.00	4.70	9.25
3	5.25	.37	8.40	.00	.00	3.26
4	5.34	.73	11.72	.00	.00	4.53
5	5.31	.64	10.10	.00	4.53	8.48
6	5.25	.73	13.61	.00	20.06	25.29
7	4.95	.23	10.59	.00	.00	7.55
8	5.01	.46	16.84	.00	25.29	28.06
9	4.66	.16	14.87	.00	32.97	36.56
10	3.94	.06	7.41	.00	36.56	42.11
B	3.60	.18	.10	.00	42.11	41.96
11	3.44	.51	5.73	.00	41.96	46.88
12	3.11	.85	5.35	.00	46.88	51.13
PS	2.00	13.50	.00	.00	51.13	3.39

Advanced Interconnected Channel & Pond Routing (adICPR Ver 1.40)

FIRST RUN
5/3/94

REACH SUMMARY
============

INDEX	RCHNAME	FRMNODE	TONODE	REACH TYPE
1	F	6	8	CULVERT, CIRCULAR w/ ROADWAY
2	H	8	9	CULVERT, CIRCULAR w/ ROADWAY
3	I	9	10	CULVERT, CIRCULAR w/ ROADWAY
4	J	10	B	CULVERT, CIRCULAR w/ ROADWAY
5	A	1	2	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
6	B	2	6	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
7	C	3	6	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
8	D	4	5	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
9	E	5	6	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
10	G	7	9	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
11	K	B	11	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
12	L	11	12	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.
13	M	12	PS	TRAPEZOIDAL CHANNEL, MOMENTUM EQ.

SECTION V

W-14 CANAL OUTFALL

TABLE OF CONTENTS

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I. Introduction

One of the studies to be undertaken as part of Task Order no. 9 of the Slidell Master Drainage Plan is an analysis of pumping capacity at the outfall of the W-14 Main Diversion Canal. The W-14 is the major canal draining the center of the City of Slidell, covering a 5-mile stretch from its source just north of Interstate Highway 12 near Brownswitch Road to its crossing with Interstate Highway 10 (see Site Map, Figure 1). From there it continues southeasterly, draining into the Fritchie Marsh and ultimately into Lake Pontchartrain. The W-14 drains approximately 1800 acres of residential subdivisions from Whisperwood Estates and Country Club Estates at the north end to Broadmoor, Fountainbleau, and Lakewood at the south. The entire drainage basin covers over 5500 acres; its boundaries are described in the Drainage Area Map, Figure 2.

Increasing suburban and commercial development in the area have caused higher volumes of runoff and quicker times to peak runoff during storm events. Higher demands are thus placed on the W-14 Canal; and its capacity is often exceeded during heavy rains, causing street flooding and property damage. Because of the length of the canal, the population it serves, and the area it drains, the 1983 St. Tammany Master Drainage Plan (prepared by Burk & Associates, Inc.) found that the lower reaches of the W-14 may experience flows as high as 4000 cfs from a 10-year design storm. The existing capacity of the canal in this area is not adequate to handle flows of this magnitude.

Flooding in the area of the W-14 Canal is due not only to the inadequate capacity of the channel itself, but also to backwater flooding from high water levels in Lake Pontchartrain. At its lower reaches, just south of Interstate 10, the W-14 drains into the Fritchie Marsh. Because the W-14 is gravity-drained, and because the marshland is relatively flat and low-lying, tidal flows in the lake will impede the flow of the W-14 and force the flow direction to reverse as the system attempts to alleviate pressure built up by the high water levels in the lake. Incoming high water from the lake will flow upstream through the W-14 and create a damming effect within the canal, causing overflow. This phenomenon will continue to occur as long as the system is drained by gravity only.

II. Project Description

The scope of this phase of the project is to analyze the feasibility of adding pumping capacity at the outfall of the W-14 Canal where it empties into the Fritchie Marsh and Lake Pontchartrain, and the possible effects it will have on the rest of the drainage system and the surrounding areas. The pumps would prevent Lake Pontchartrain flows from backing up into the canal and allow the W-14 to be drained more rapidly. The proposed pumping station would be located east of the Interstate 10 crossing and south of Voter's Road, past the Kingspoint Subdivision and any residential areas. Based on this study, schematics and cost estimates for the pump station were developed along with a project schedule for the proposed construction.

III. Project Hydraulics

Project hydraulics were based on the St. Tammany Parish Master Drainage Plan, prepared by Burk & Associates, Inc. (1983), and U.S.G.S. Quadrangle Maps of the areas covering Slidell, the Rigolets, Northshore, and Haaswood. These maps were used for obtaining ground slopes and distances.

Design flows and water surface elevations for a 10-year design storm were computed for various points along the W-14 in the 1983 Master Drainage Plan, and these are reproduced in Appendix A. Hydraulic analysis (see Appendix B) shows that a typical W-14 Canal section having the properties listed in Table 1 can handle a discharge of about 1650 cfs. Thus, the canal is deficient by approximately 2350 cfs, based on the 4000 cfs of runoff predicted for the basin by the Master Drainage Plan.

IV. Proposed Improvements

It is proposed that the pump station be designed in stages; the scope of this project is the first stage, designed to pump the entire existing W-14 capacity of 1650 cfs (approximately 740,500 gpm) into the marsh. A large sump area will also be excavated immediately upstream of the station to provide brief retention capacity and prevent surges to the pump station. Also included in the first phase is the construction of a high ring levee at the southern tip of the drainage basin and the addition of low-head flap gates for gravity drainage during periods of low tide in Lake Pontchartrain. The flap gates would be operational when the W-14 level is above that of the lake. When the lake level rises above the W-14, the higher water pressure on the lake side would cause the gates to close, preventing lake water from backing up through the system. The W-14 water will then pond in the sump area behind the levee until the water reached a height sufficient for the pumps to turn on. This water will then be pumped out of the sump area over the levee into Lake Pontchartrain.

Future improvements, beyond the scope of this report, will include providing the additional pumping capacity of 2350 cfs required to handle the entire design flow of 4000 cfs through the southernmost improved canal section. The entire length of the W-14 would then be improved to handle the full runoff volume predicted from a 10-year storm over each subarea. This end may be achieved by improving the hydraulic radius of the canal through widening or excavation and/ or by improving the "roughness" of the natural earthen canal by lining or paving it. The pump station designed here in the first stage should be constructed larger than needed to provide additional space to accommodate future pump addition.

V. Conceptual Design

Because of the inclusion of flap gate valves, the plant will be not be operated continuously at its peak capacity. Thus, it is desired to have several small pumps rather than one large one. Conceptually, the 1650 cfs should be divided into two (2) smaller 300-cfs pumps which will be operated continuously and one (1) large 1050-cfs pump which will be turned on during storm events or tidal surges. In this way, only when the lake water level rises above that of the W-14 would the large pump be utilized to lift the canal water above the levee and into the surrounding marsh. As seen in Appendix B, the 1050-cfs pump will require an 11-ft.-diameter discharge pipe, and the two 300-cfs pumps will each require 72-inch-diameter (6-ft.) discharge pipes. Because the water depth in the lake is typically shallow, the pipes would be laid in vertically-angled sections through the levee so that their inverts would rest very near the lake bottom.

Since normal water surface elevations in the lake range from +4.0' National Geodetic Vertical Datum (N.G.V.D.) to +6.0' N.G.V.D., depending on tidal or seasonal effects, it is proposed to construct the levee so that its crown will reach an elevation of about 12 feet. This will create a high barrier around the W-14 which will prevent lake water from entering. The lower-elevation flap gates will allow the water from the W-14 to pass through this barrier if the water level differential permits.

It should be noted that the Fritchie Marsh, in which the pump station is proposed to be constructed, provides a very unstable building foundation for the pump and sump slabs. Thus deep pile supports are required, and construction costs will be higher than those for a similar pump station on stable ground. Again, the pump station building is required to be quite large in order to accommodate the three pumps proposed in this design stage as well as those pumps to be added in future stages.

VI. Estimated Design / Construction Cost

Table 2 presents a preliminary tabulation of each construction item covered in this project and its associated cost. The total construction cost is estimated at $12,827,000; adding design costs ($1,287,000), construction administration ($513,080), and resident inspection ($192,000) brings the estimated total project cost to $14,819,100. It should be noted that this represents only the cost of Phase 1 of the W-14 pump station; further improvements as explained previously may need to be implemented in the future.

VII. Estimated Design / Construction Time Schedule.

The estimated design time schedule for this phase of the project is three months for the project design, which includes preparation of plans, specifications, and contract documents, two months for bidding and award of the contract, and an additional 24 months for construction of the project. This estimated time schedule assumes the required funding for the project to be available and does not include any possible delays caused by appropriation of money to fund this project.

TABLE 1

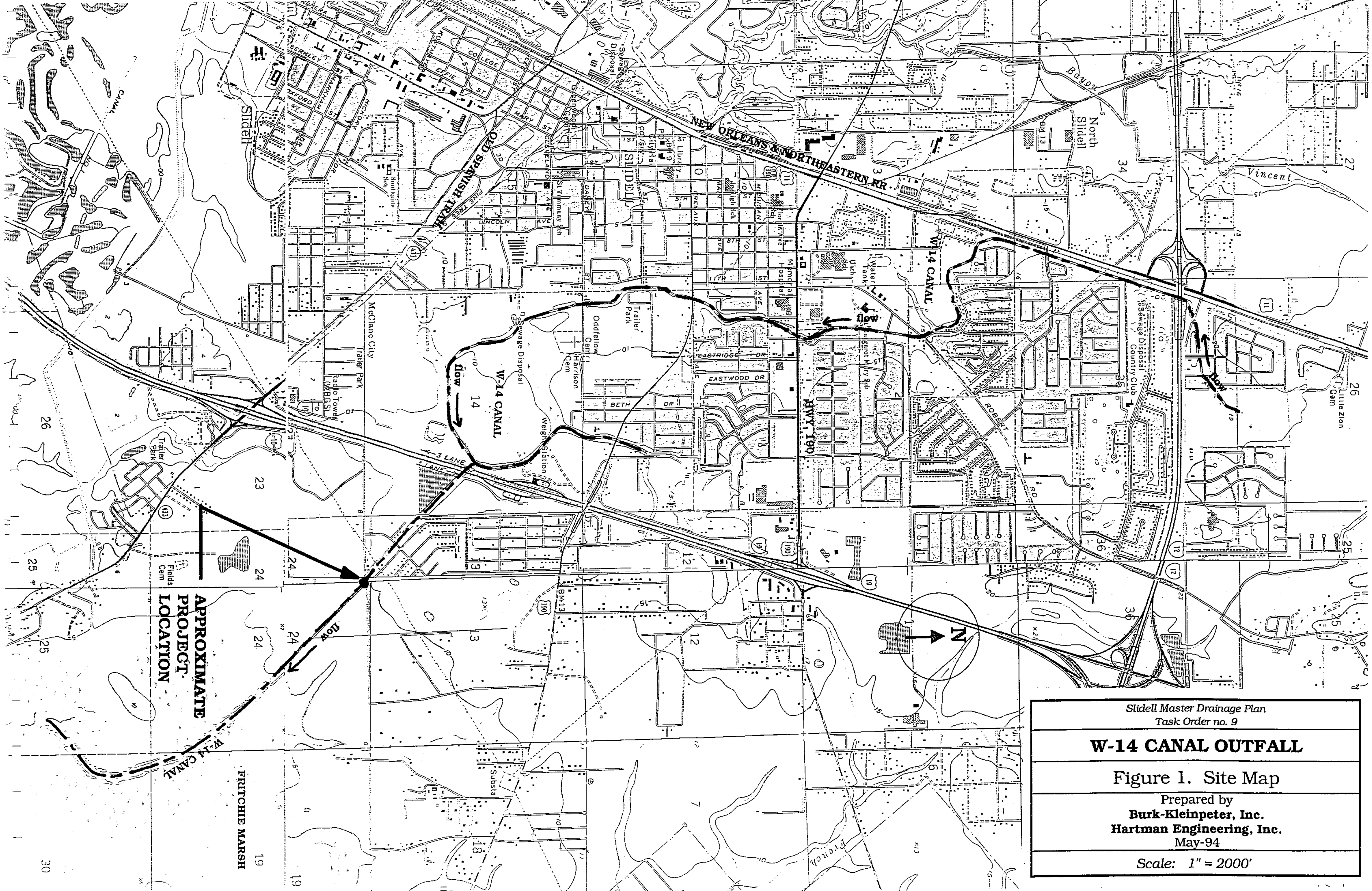
DESIGN PARAMETERS

Parameter	Value	Source
Ground slope	0.00044 ft/ft	U.S.G.S. Maps
Peak flow of runoff	4000 cfs	St. Tammany MDP
Typical W-14 bottom width	15 ft	Borgen Engineering (1981)
Typical W-14 side slopes	2:1	Borgen Engineering (1981)
Typical W-14 depth	12 ft	St. Tammany MDP
Manning's "N" of canal	0.030	Design Manual

TABLE 2

ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$509,000
Excavation	Cubic Yard	1,100	$5	$5,600
Sheet Pile Wall	Square Ft	5,850	$12	$70,200
1050 cfs Pump	Each	1	$6,300,000	$6,300,000
300 cfs Pump	Each	2	$1,800,000	$3,600,000
72" Steel Pipe	Linear Ft	60	$400	$24,000
11' Steel Pipe	Linear Ft	60	$700	$42,000
Constructing Levee	Linear Ft	6,000	$13	$78,000
Flap gates	Each	3	$20,000	$60,000
SUBTOTAL				$10,688,800
CONTINGENCY @ 20%				$2,138,200
TOTAL				$12,827,000



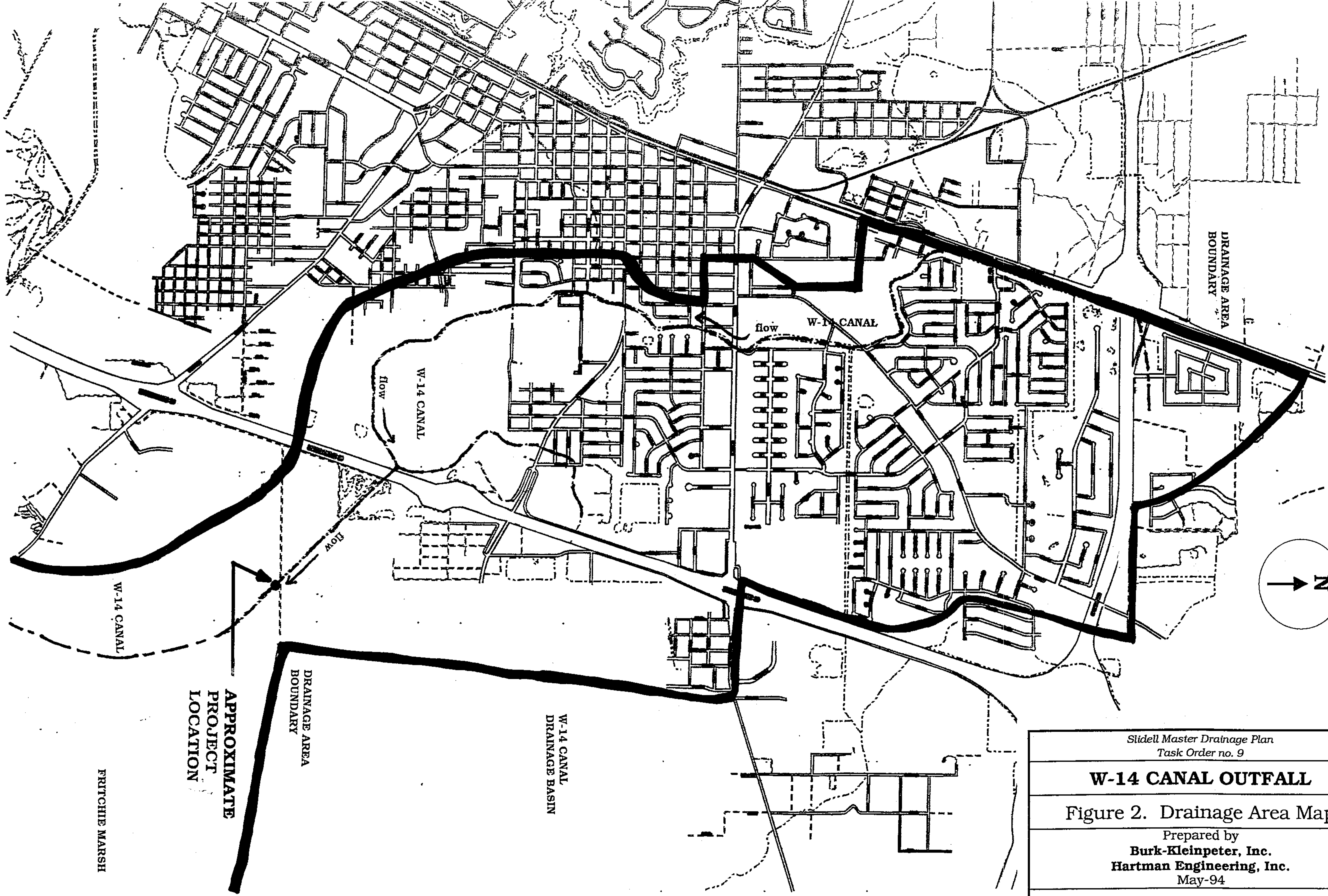
Slidell Master Drainage Plan
Task Order no. 9

W-14 CANAL OUTFALL

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



Slidell Master Drainage Plan
Task Order no. 9

W-14 CANAL OUTFALL

Figure 2. Drainage Area Map

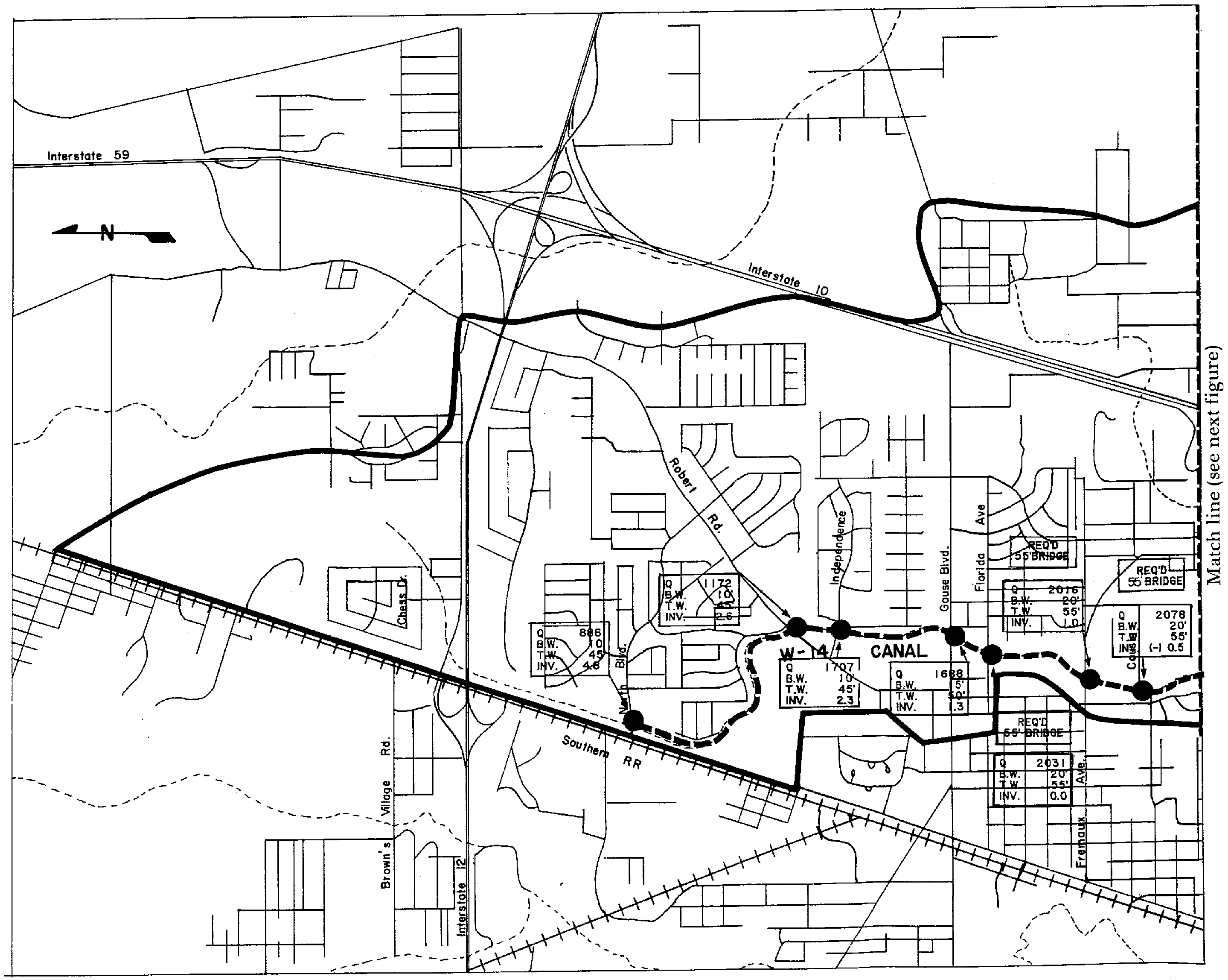
Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'

APPENDIX A

HYDROLOGIC COMPUTATIONS

Figure 38
Proposed Improvements
W-14 Main Diversion Canal



Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Concrete Slope Paving

Earthen Section

Burk & Associates
Incorporated

1000 0 1000 2000 3000 FEET

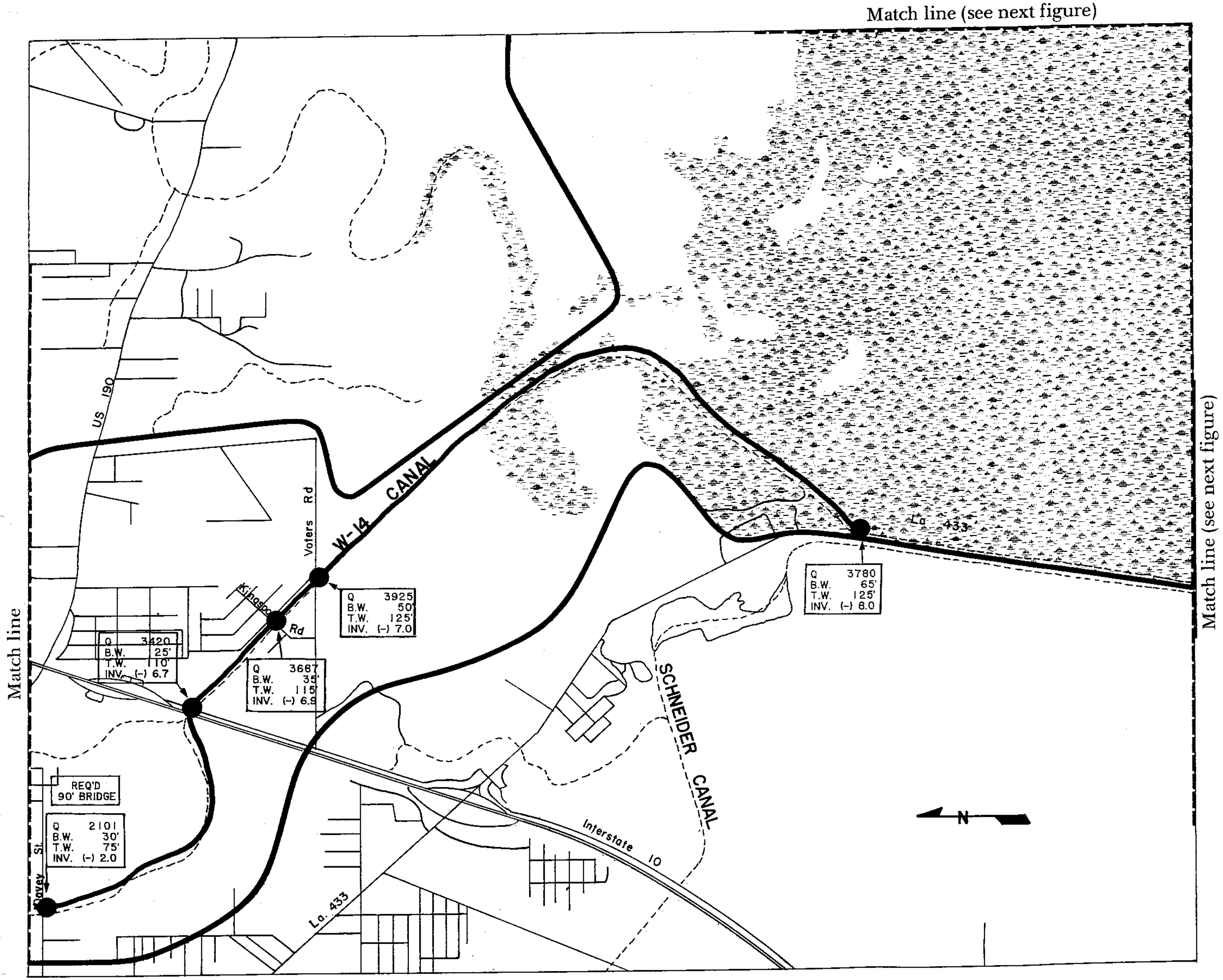


Figure 39
Proposed Improvements
W-14 Main Diversion Canal
Continued

Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Earthen Section

Burk & Associates Incorporated

1000 0 1000 2000 3000 FEET

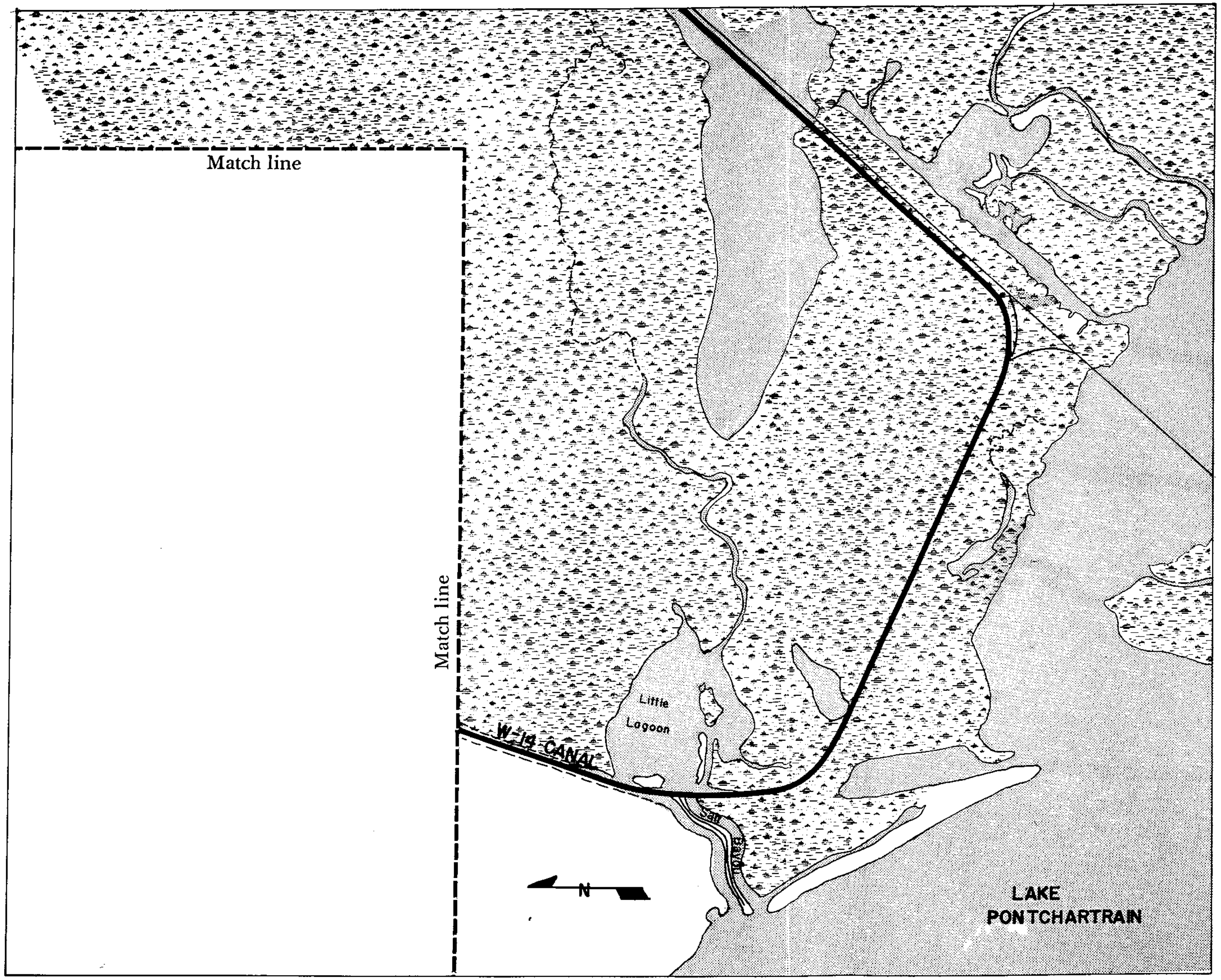


Figure 40
Proposed Improvements
W-14 Main Diversion Canal
Continued

Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Earthen Section

Burk & Associates Incorporated

1000 0 1000 2000 3000 FEET

APPENDIX B

HYDRAULIC ANALYSIS

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 Canal

Description: Analysis of existing canal capacity

Solve For Discharge

Given Constant Data;

Bottom Width....... 15.00
Z-Left............. 2.00
Z-Right............ 2.00
Mannings 'n'....... 0.030
Channel Slope...... 0.0004
Channel Depth...... 12.00

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
15.00	2.00	2.00	0.030	0.0004	12.00	1666.65	3.56

Circular Channel Analysis & Design

Open Channel - Uniform flow

Worksheet Name: W-14 PUMP STATION

Description: Pump Discharge Pipes

Solve for Diameter

Given Constant Data;

Discharge.................	741,000 gpm 1050 cfs
Discharge Velocity........	10 fps (typical)

Required Area = Q/V
= (1050 cfs)/(10 fps)
= **105 sq. ft.**

Required Diameter = **11 ft.**

use 11 ft.-diameter pipe

Given Constant Data;

Discharge.................	135,000 gpm 300 cfs
Discharge Velocity........	10 fps (typical)

Required Area = Q/V
= (300 cfs)/(10 fps)
= **30 sq. ft.**

Required Diameter = **74.3 in.**

use 72 in.-diameter pipe (O.K.)

SECTION VI

W-14 CANAL at GAUSE BOULEVARD and ROBERT ROAD

TABLE OF CONTENTS

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LIST OF APPENDICES

I. Project Description

In accordance with Task Order No. 9 of the Slidell Master Drainage Plan, contained herein is a review of the W-14 Canal Engineering Report prepared by Cortech, Inc. and submitted to the City of Slidell in November, 1992 (revised December, 1992). The report proposes recommendations to relieve deficient canal sections or "bottleneck" areas at both the Gause Boulevard and the Robert Road crossings. These bottlenecks effectively restrict the free flow of storm water through the W-14 Canal. The report prepared by Cortech, Inc. is reproduced in its entirety in Appendix C of this section.

II. Existing Conditions

a.) Gause Boulevard Crossing

The present average top width of the canal in this section is approximately 45 feet. It has been estimated that an adequate canal section with stable banks would have at least a 60-foot top width, requiring a minimum right-of-way width of 110 feet for proper maintenance; however, the present right-of-way is only 50 to 60 feet wide. The crossing at Gause Boulevard consists of a four celled 11' x 8.5' reinforced concrete box (RCB) culverts with flared headwalls. There are earthen transition sections from the natural channel shape to the box culvert section. Sediments are being deposited upstream and downstream of the crossing, reducing the canal's hydraulic properties and thus also its flow capacity. Also, residential and roadway development up to the canal's right-of-way have eliminated clearings for maintenance equipment and machinery.

b.) Robert Road Crossing

Three (3) 90" corrugated metal pipe (CMP) culverts convey the W-14 Canal under Robert Road. Sediments are being deposited just downstream of the crossing, reducing the canal's hydraulic properties and thus also its flow capacity. In addition, the existing canal banks in this section are close enough to residential backyards to pose a threat to private property and safety. Finally, there are several locations where the canal right-of-way does not appear to be legally defined, making accessibility for maintenance difficult.

III. Proposed Improvements - Cortech, Inc.

a.) Gause Boulevard Crossing

Obtaining additional right-of-way in this area would be time-consuming and cost-prohibitive due to the degree of land development in the area. Thus, Cortech, Inc. proposes full enclosure of the canal section by a 172" x 107" medium-gauge galvanized steel corrugated metal pipe arch (CMPA) with asphaltic cement paved bottom. The enclosure would lie totally within the existing right-of-way and cover a distance of approximately 330 feet on either side of the existing 4-cell box culvert under Gause Boulevard (to remain). Two of the four cells of the upstream end of the existing box culvert would be blocked to maintain a minimum of three feet per second (fps) velocity in the channel. The total construction cost of this design is about $191,000.

b.) Robert Road Crossing

Cortech, Inc. proposes that additional right-of-way (to a total of 100 feet in width) be resurrected or acquired to provide for maintenance operations. Then, based on the availability of right-of-way, the area available will dictate which improvements are required and can be implemented. Generally, the new widened canal will have a top width of about 60 feet, allowing for 20 feet of maintenance area and shoulder on either side of the canal.

IV. Review of Cortech Inc.'s Proposed Improvements

We have reviewed the report submitted by Cortech, Inc. and note a few differences with its recommendations. While we do agree with the evaluation that the right-of-way widths along the W-14 canal are inadequate for the City of Slidell to maintain an earthen section, and that the canal should be improved in the Gause Boulevard and Robert Road areas, we disagree with the improvements suggested by Cortech, Inc.

The recommended improvements for Gause Boulevard and Robert Road included in the report prepared by Cortech, Inc. are technically correct for the assumptions they used. Their assumptions were based upon the existing capacity of the W-14 Canal sections as they are today. This includes the debris in the canal and the overgrowth on the banks of the canal. The proposed culvert would convey the same amount of storm water as the existing channel in its existing condition. We disagree with this concept since it is not compatible with the predicted 10 Year - 24 Hour Storm Event runoff flows in the 1983 Master Drainage Plan nor allows for any improvement in storm water flow created by cleaning of the existing channel. Further, any restriction in the channel will increase the water surface elevations north of each crossing which could cause and/or increase flooding and property damage.

The corrugated metal pipe arch (CMPA) section with paved invert that Cortech, Inc. has selected has a smaller capacity relative to the canal section in which it is to be laid. The transition from trapezoidal earthen canal flow to a lower-capacity arch pipe flow is a restriction and will cause a significant increase in the water surface elevations upstream from both Robert Road and Gause Boulevard. In addition, from our previous observations of corrugated metal culverts with paved linings, the linings deteriorate rapidly, creating additional roughness, thereby reducing the hydraulic capacity of the pipe.

a.) Gause Boulevard Crossing

Cortech Inc. proposes that a paved invert 172" x 107" CMPA (144-inch equivalent diameter) be placed in the W-14 Canal on either side of the Gause Boulevard crossing as described previously. Cross-sections of the W-14 taken from a 1992 survey by Albert A. Lovell & Associates, Inc. show that the average canal bed slope is approximately 0.001 feet/foot. From our evaluation of the pipe's hydraulic capacity using this value, the CMPA is able to convey storm water at a rate of about 442 cubic feet per second (cfs). Hydraulic computations have been included in Appendix B.

From our review of the St. Tammany Master Drainage Plan, prepared by Burk & Associates, Inc. (1983), the existing capacity as determined in 1983 of the W-14 Canal section at Gause Boulevard was 770 cfs (Excerpts of the 1983 Master Drainage Plan have been included in Appendix A). This flow value assumes that the banks and channel have been cleared of debris and obstructions. The pipe that Cortech proposes is thus deficient in hydraulic capacity by about 43% or 328 cfs, based on the existing canal conveyance capability.

b.) Robert Road Crossing

The St. Tammany Master Drainage Plan shows the 1983 existing capacity of the W-14 at Robert Road to be 375 cfs (see Appendix A), which is lower than the capacity of Cortech's proposed CMPA. However, calculations based on the Lovell & Associates cross-sections show that in 1992 the canal at this section had a capacity of about 861 cfs (see Appendix B). Comparing this value with the one provided in the 1983 Master Drainage Plan, it is apparent that the increasing flow and velocities through the canal over years of land development in the area have scoured out the canal section. Based on the 1992 survey, the proposed 172" X 107" CMPA section is deficient in flow capacity by about 50%.

Further, Cortech, Inc. proposes that the new W-14 section at Robert Road have a minimum 60-foot top width. A newly-excavated natural channel of this top width, 2.5 :1 side slopes, a 0.001 feet/foot bed slope, and a 9-foot depth (as proposed in their report, Appendix C) would have a capacity of 1610 cfs, about 3.5 times as great as that of the proposed CMPA. It is apparent that during high W-14 Canal discharge events, the water will back up behind the proposed CMPA at each location, further encouraging upstream flooding.

V. Proposed Improvements - BKI

The 1983 MDP determined that the 10 Year - 24 Hour Storm Event design storm discharges for the W-14 Canal at Robert Road and Gause Boulevard are 1172 cfs and 2016 cfs, respectively (see Appendix A). The existing canal section is unable to handle these flows and as a result significant erosion and sedimentation is occurring as high-velocity flows are being pushed through the canal. In conjunction with the previous discussion from Section V, W-14 Canal Outfall, it is recommended to modify the W-14 Canal to handle the 10 Year - 24 Hour Storm Event design flows (Proposed recommendations for these modifications are beyond the scope of this report; refer to recommendations of the 1983 Master Drainage Plan in Appendix A). One critical design criteria is to allow a maximum flow velocity of 3 feet per second through the canal to prevent scour.

We feel the most economical solution to the erosion and sedimentation problems at the Robert Road and Gause Boulevard crossings is a combination of slope paving the inlet and outlet of the culverts and a side discharge control structure on the upstream end of each crossing. The slope paving will solve the stream bed erosion along the approach to and the exit from the box culvert. Slope paving the channel will also stabilize the bank and prevent encroachment toward adjacent roadways and private property. The proposed slope paving section would consist of a trapezoidal channel with a 10 foot bottom width and 2H:1V bank slopes for the Robert Road crossing and a trapezoidal channel with a 15 foot bottom width and 2H:1V bank slopes for the Gause Boulevard crossing. A side discharge control structure at each crossing will provide the necessary transition from the slope paving section to the existing box culvert. The side discharge control structure would consist of multiple weirs (two at Robert Road and three at Gause Boulevard) set at elevations to provide a minimum of 3.5 fps in the first culvert cell prior to discharging into the adjacent culvert cell. During periods of low rate of runoff, one or two cells of the culvert may be utilized and for periods of high rate of runoff three to four cells will be utilized.

For the crossings at Robert Road and Gause Boulevard if additional right-of-way can not be obtained, we propose that the W-14 Canal be enclosed with an 9' x 20' Con/Span Culvert at Robert Road and a 9' x 32' Con / Span Culvert at Gause Boulevard. The Con / Span culverts are pre-cast inverted U-shaped culvert sections that are placed on a concrete slab. These culverts are advantageous in tight right-of-way limits since their use drastically shortens the construction time.

The capacity of each of these culverts is rated at 1,260 cfs and 2,158 cfs respectively, exceeding the 10 Year - 24 Hour runoff requirements. The culverts would thus offer little resistance to flow and prevent the headwater from backing up upstream of the highway crossings.

VI. Estimated Design / Construction Cost

Table 2 presents a preliminary tabulation of each construction item covered in this project and its associated cost for the slope paving alternative. The total construction cost is approximately $506,000; adding design costs ($50,600), construction administration ($20,200), and resident inspection ($48,000) brings the estimated total project cost to about $624,800.

Table 3 presents a preliminary tabulation of each construction item covered in this project and its associated cost for the Con/Span culvert alternative. The total construction cost is approximately $1,189,000; adding design costs ($118,900), construction administration ($47,600), and resident inspection ($48,000) brings the estimated total project cost to about $1,403,500.

It should be noted that because this cost estimate is preliminary, it may be lacking some required cost items which will be discovered during final design and survey. Required right-of-way costs have not been included in either cost estimate.

VII. Estimated Design / Construction Time Schedule

The estimated design time schedule for this project is three months for the project design, which includes preparation of plans, specifications, and contract documents; two months for bidding and award of the contract; and an additional six months for construction. This estimated time schedule assumes the required funding for the project is available and does not include possible delays caused by appropriation of money to fund the project.

TABLE 1

DESIGN PARAMETERS

Parameter	Value	Source
Existing peak flow of runoff -		St. Tammany MDP
W-14 @ Robert Road	375 cfs	
W-14 @ Gause Blvd.	770 cfs	
10-yr. peak flow of runoff -		St. Tammany MDP
W-14 @ Robert Road	1172 cfs	
W-14 @ Gause Blvd.	2016 cfs	
Manning's "N" of earth channel	0.030	Design Manual
Manning's "N" of RCB	0.012	Design Manual
Manning's "N" of CAP (25% paved)	0.025	Design Manual
Max. velocity through canal	3 fps	Design Manual
W-14 bed slope	0.001 ft/ft	Lovell & Assoc. (1992)
Side slopes of existing canal	1.5 : 1	Lovell & Assoc. (1992)
Bottom width of existing canal	8 ft	Lovell & Assoc. (1992)
Depth of existing canal	9 ft	Lovell & Assoc. (1992)

TABLE 2

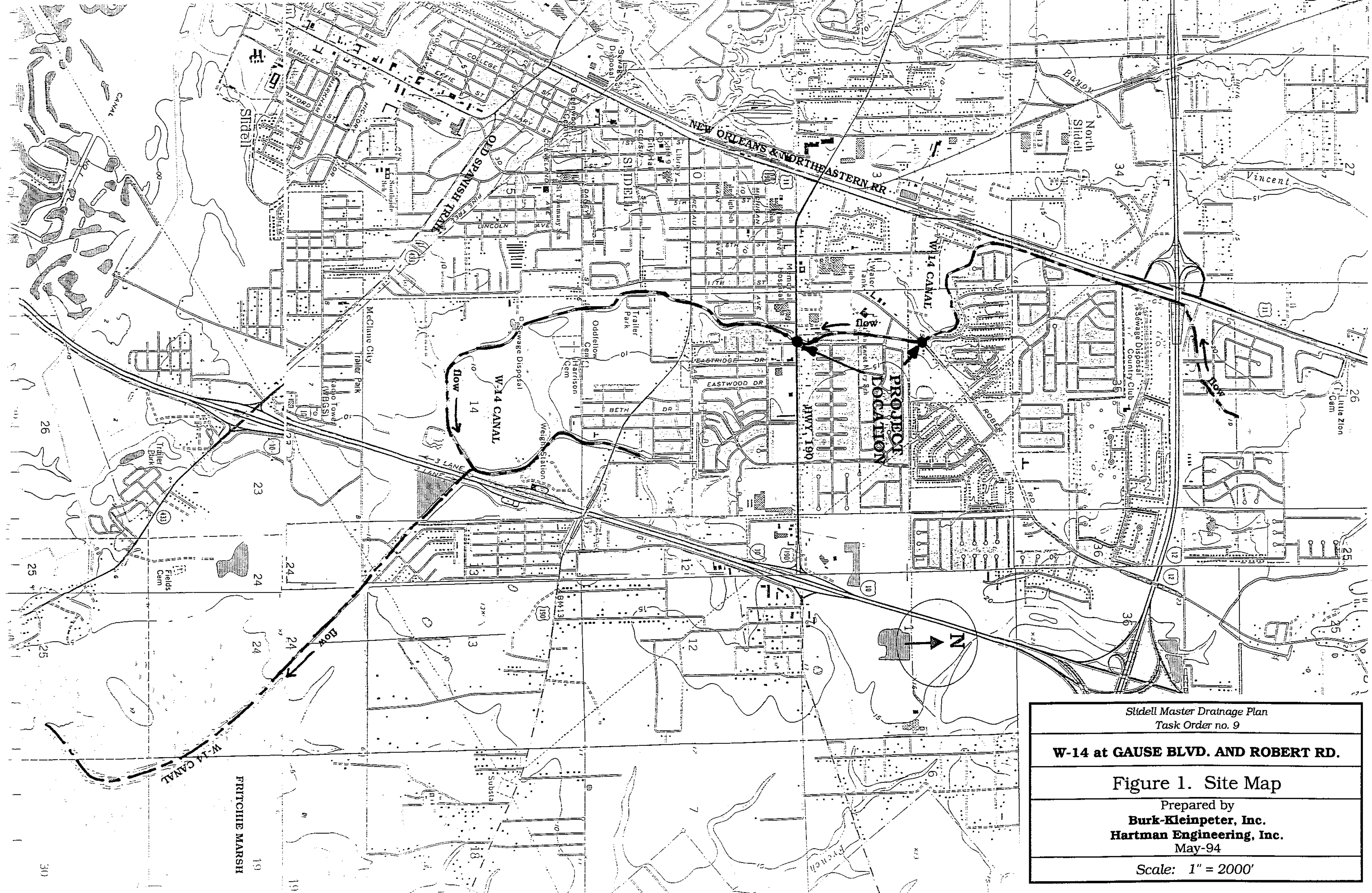
ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$21,000
Slope Paving - 10' BW, 2:1 SS	Lin. Ft.	280	$500.00	$140,000
Transition Structure	Lin. Ft.	50	$1,000.00	$50,000
Slope Paving - 15' BW, 2:1 SS	Lin. Ft.	280	$575.00	$161,000
Transition Structure	Lin. Ft.	50	$1,00.00	$50,000
SUBTOTAL				$422,000
CONTINGENCY @ 20%				$84,000
TOTAL				$506,000

TABLE 3

ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$47,200
Con / Span Culvert 9' x 20'	Lin. Ft.	330	$1,200.00	$396,000
Con / Span Culvert 9' x 32'	Lin. Ft.	330	$1,660.00	$547,800
SUBTOTAL				$991,000
CONTINGENCY @ 20%				$156,400
TOTAL				$1,189,000



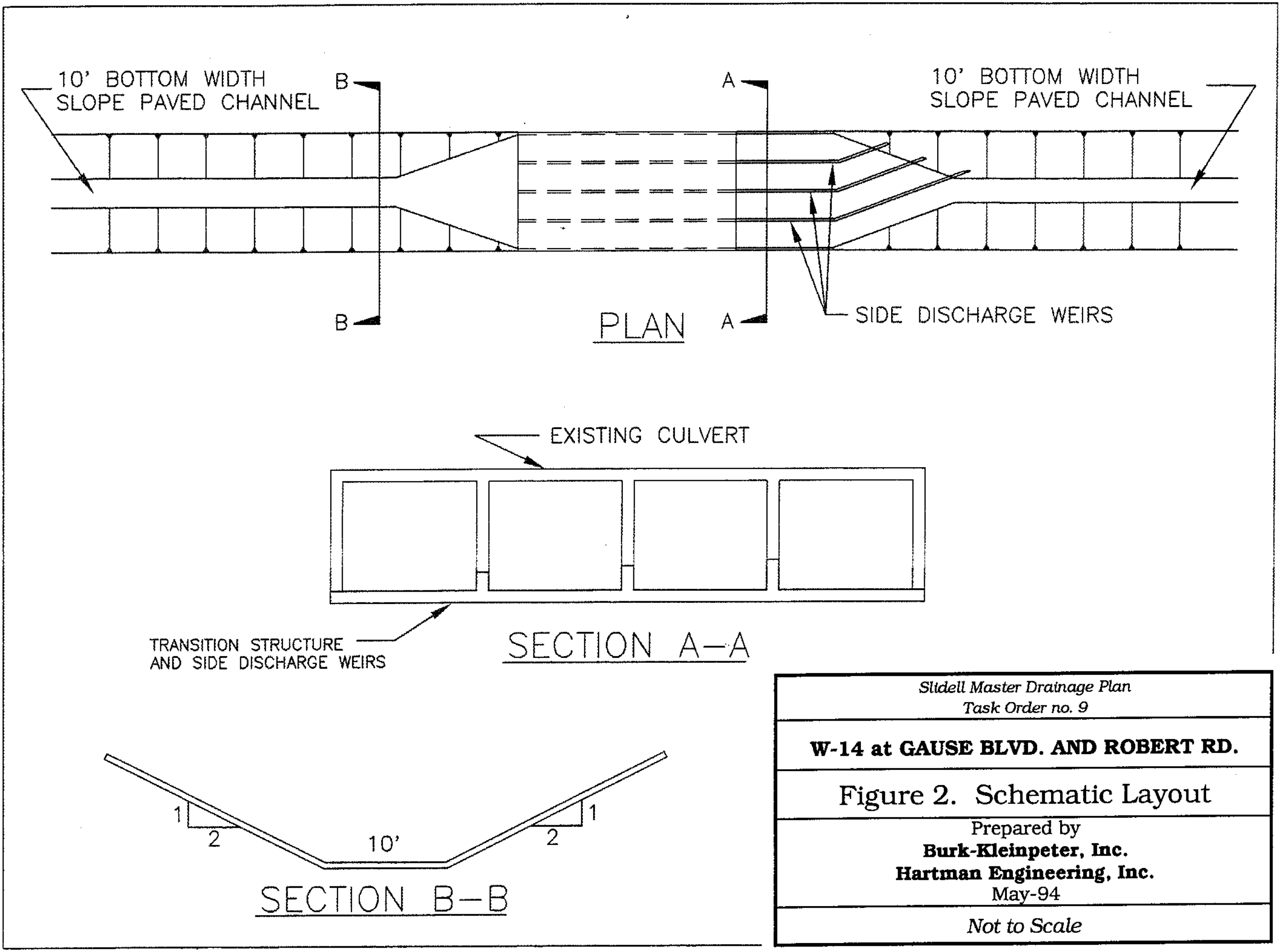
Slidell Master Drainage Plan
Task Order no. 9

W-14 at GAUSE BLVD. AND ROBERT RD.

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Scale: 1" = 2000'



Slidell Master Drainage Plan
Task Order no. 9

W-14 at GAUSE BLVD. AND ROBERT RD.

Figure 2. Schematic Layout

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
May-94

Not to Scale

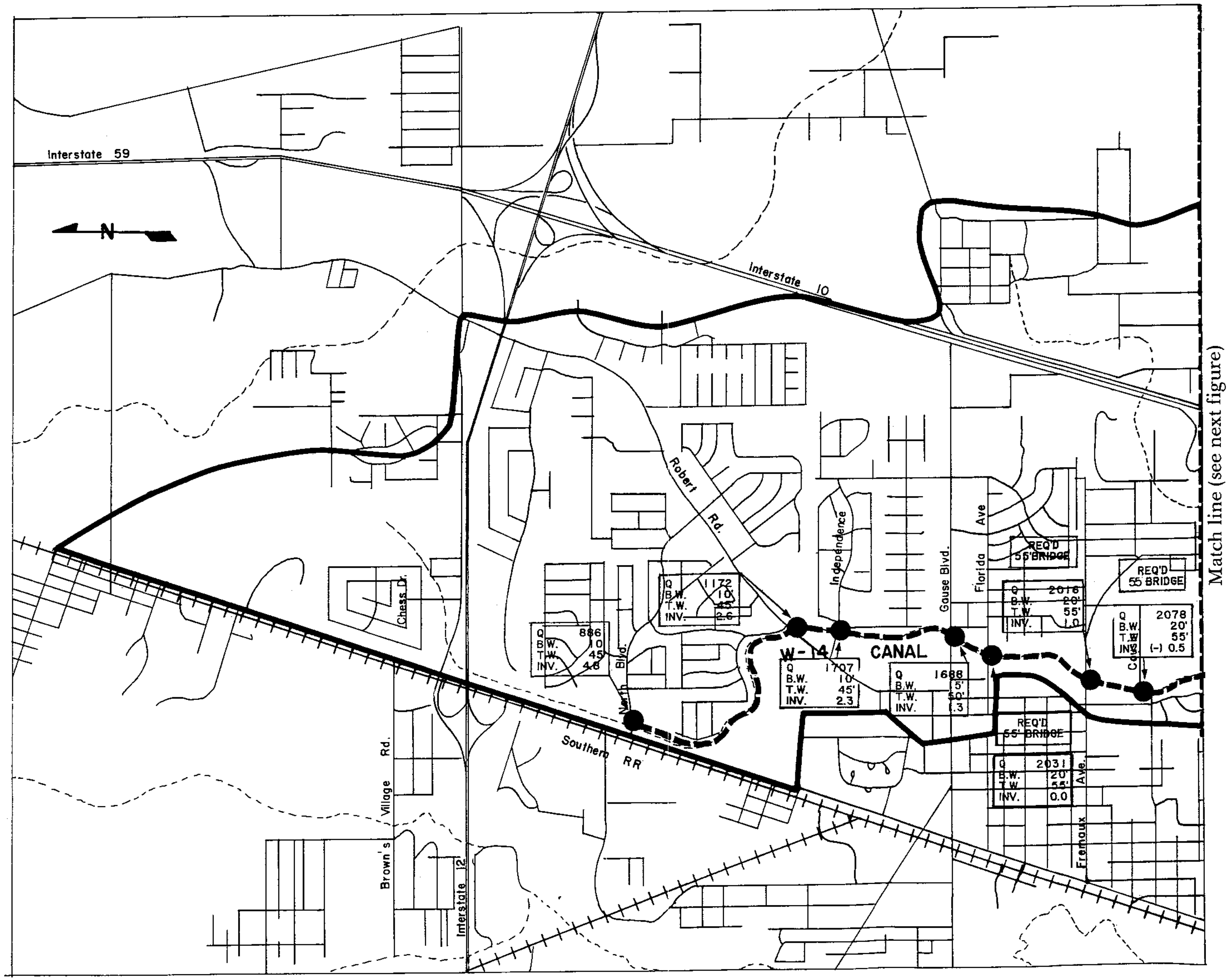
APPENDIX A

HYDROLOGIC COMPUTATIONS

Table 9
continued

Channel	Design Point	Drainage Area (Mi^2)	Present Capacity (cfs)	Required Capacity (cfs)	Design Water Surface Elev.
Schneider Canal Cont.	West of I-10	0.06	254	80	3.85
	East of I-10	0.06	45	80	3.96
	LA 433	0.86	86	573	1.80
	Lake Pontchartrain	1.40	1089	705	1.00
W-14	North Blvd.	1.77	259	886	12.59
	Robert Rd.	2.32	375	1172	11.51
	Independence Drive	3.15	117	1688	10.33
	Gause Blvd.	3.64	770	2016	9.66
	Florida Ave.	3.72	590	2030	9.35
	Fremaux Ave.	3.83	570	2054	8.73
	Cousin St.	3.93	8.10	2078	8.66
	Daney St.	4.04	1473	2102	8.57
	I-10	6.21	2838	3420	7.93
	Kingspoint Boulevard	6.76	3291	3686	7.28
	Voters Rd.	7.20	4340	3925	7.03
	LA 433	8.61	590	3780	2.00
French Branch	Southern R.R.	1.08	88	244	24.46
	Hasswood Rd.	1.65	326	514	23.78

Figure 38
Proposed Improvements
W-14 Main Diversion Canal



Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Concrete Slope Paving

Earthen Section

Burk & Associates
Incorporated

1000 0 1000 2000 3000 FEET

APPENDIX B

HYDRAULIC ANALYSIS

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: W-14 CANAL

Description: EVALUATION OF 172" X 107" CAP

Solve For Full Flow Capacity

Given Constant Data;

Diameter........... 12.00
Slope.............. 0.0010
Mannings n......... 0.025
Discharge.......... 442.19

Diameter ft	Channel Slope ft/ft	Mannings 'n'	COMPUTED Discharge cfs	COMPUTED Depth ft	COMPUTED Velocity fps	COMPUTED Capacity Full cfs
12.00	0.0010	0.025	442.19	12.00	3.91	442.19

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 CANAL

Description: ROBERT ROAD CROSSING CAPACITY (1992 DATA)

Solve For Discharge

Given Constant Data;

Bottom Width....... 8.00
Z-Left............. 1.50
Z-Right............ 1.50
Mannings 'n'....... 0.030
Channel Slope...... 0.0010
Channel Depth...... 9.00

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
8.00	1.50	1.50	0.030	0.0010	9.00	860.51	4.45

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 CANAL

Description: EVALUATION OF CORTECH'S PROPOSED CANAL SECTION

Solve For Discharge

Given Constant Data;

Bottom Width.......	15.00
Z-Left............	2.50
Z-Right...........	2.50
Mannings 'n'.......	0.030
Channel Slope......	0.0010
Channel Depth......	9.00

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
15.00	2.50	2.50	0.030	0.0010	9.00	1610.61	4.77

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 @ Gause Blvd

Description: Paved Channel Section

Solve For Discharge

Given Constant Data;

Bottom Width.......	15.00
Z-Left.............	2.00
Z-Right............	2.00
Mannings 'n'.......	0.015
Channel Slope......	0.0010

Variable Input Data	Minimum	Maximum	Increment By
Channel Depth	0.25	9.00	0.25

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	VARIABLE Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
15.00	2.00	2.00	0.015	0.0010	0.25	4.69	1.21
15.00	2.00	2.00	0.015	0.0010	0.50	15.02	1.88
15.00	2.00	2.00	0.015	0.0010	0.75	29.81	2.41
15.00	2.00	2.00	0.015	0.0010	1.00	48.65	2.86
15.00	2.00	2.00	0.015	0.0010	1.25	71.35	3.26
15.00	2.00	2.00	0.015	0.0010	1.50	97.83	3.62
15.00	2.00	2.00	0.015	0.0010	1.75	128.03	3.95
15.00	2.00	2.00	0.015	0.0010	2.00	161.97	4.26
15.00	2.00	2.00	0.015	0.0010	2.25	199.65	4.55
15.00	2.00	2.00	0.015	0.0010	2.50	241.12	4.82
15.00	2.00	2.00	0.015	0.0010	2.75	286.41	5.08
15.00	2.00	2.00	0.015	0.0010	3.00	335.57	5.33
15.00	2.00	2.00	0.015	0.0010	3.25	388.67	5.56
15.00	2.00	2.00	0.015	0.0010	3.50	445.76	5.79
15.00	2.00	2.00	0.015	0.0010	3.75	506.91	6.01
15.00	2.00	2.00	0.015	0.0010	4.00	572.20	6.22
15.00	2.00	2.00	0.015	0.0010	4.25	641.67	6.42
15.00	2.00	2.00	0.015	0.0010	4.50	715.42	6.62
15.00	2.00	2.00	0.015	0.0010	4.75	793.50	6.82
15.00	2.00	2.00	0.015	0.0010	5.00	875.99	7.01
15.00	2.00	2.00	0.015	0.0010	5.25	962.97	7.19
15.00	2.00	2.00	0.015	0.0010	5.50	1054.50	7.37
15.00	2.00	2.00	0.015	0.0010	5.75	1150.67	7.55
15.00	2.00	2.00	0.015	0.0010	6.00	1251.53	7.73
15.00	2.00	2.00	0.015	0.0010	6.25	1357.17	7.90
15.00	2.00	2.00	0.015	0.0010	6.50	1467.66	8.06
15.00	2.00	2.00	0.015	0.0010	6.75	1583.07	8.23
15.00	2.00	2.00	0.015	0.0010	7.00	1703.48	8.39
15.00	2.00	2.00	0.015	0.0010	7.25	1828.95	8.55
15.00	2.00	2.00	0.015	0.0010	7.50	1959.56	8.71
15.00	2.00	2.00	0.015	0.0010	7.75	2095.39	8.86
15.00	2.00	2.00	0.015	0.0010	8.00	2236.50	9.02
15.00	2.00	2.00	0.015	0.0010	8.25	2382.96	9.17
15.00	2.00	2.00	0.015	0.0010	8.50	2534.86	9.32
15.00	2.00	2.00	0.015	0.0010	8.75	2692.25	9.47
15.00	2.00	2.00	0.015	0.0010	9.00	2855.21	9.61

Rectangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 @ Gause Blvd

Description: Single Cell Computations

Solve For Discharge

Given Constant Data;

Bottom Width....... 11.00
Mannings 'n'....... 0.015
Channel Slope...... 0.0010

Variable Input Data	Minimum	Maximum	Increment By
Channel Depth	0.25	9.00	0.25

Bottom Width ft	Mannings 'n'	Channel Slope ft/ft	VARIABLE Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
11.00	0.015	0.0010	0.25	3.32	1.21
11.00	0.015	0.0010	0.50	10.24	1.86
11.00	0.015	0.0010	0.75	19.59	2.37
11.00	0.015	0.0010	1.00	30.83	2.80
11.00	0.015	0.0010	1.25	43.61	3.17
11.00	0.015	0.0010	1.50	57.67	3.50
11.00	0.015	0.0010	1.75	72.85	3.78
11.00	0.015	0.0010	2.00	88.97	4.04
11.00	0.015	0.0010	2.25	105.92	4.28
11.00	0.015	0.0010	2.50	123.61	4.50
11.00	0.015	0.0010	2.75	141.95	4.69
11.00	0.015	0.0010	3.00	160.87	4.87
11.00	0.015	0.0010	3.25	180.31	5.04
11.00	0.015	0.0010	3.50	200.22	5.20
11.00	0.015	0.0010	3.75	220.56	5.35
11.00	0.015	0.0010	4.00	241.28	5.48
11.00	0.015	0.0010	4.25	262.35	5.61
11.00	0.015	0.0010	4.50	283.74	5.73
11.00	0.015	0.0010	4.75	305.42	5.85
11.00	0.015	0.0010	5.00	327.38	5.95
11.00	0.015	0.0010	5.25	349.59	6.05
11.00	0.015	0.0010	5.50	372.02	6.15
11.00	0.015	0.0010	5.75	394.68	6.24
11.00	0.015	0.0010	6.00	417.52	6.33
11.00	0.015	0.0010	6.25	440.56	6.41
11.00	0.015	0.0010	6.50	463.76	6.49
11.00	0.015	0.0010	6.75	487.13	6.56
11.00	0.015	0.0010	7.00	510.64	6.63
11.00	0.015	0.0010	7.25	534.30	6.70
11.00	0.015	0.0010	7.50	558.09	6.76
11.00	0.015	0.0010	7.75	582.00	6.83
11.00	0.015	0.0010	8.00	606.02	6.89
11.00	0.015	0.0010	8.25	630.16	6.94
11.00	0.015	0.0010	8.50	654.40	7.00
11.00	0.015	0.0010	8.75	678.73	7.05
11.00	0.015	0.0010	9.00	703.16	7.10

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: W-14 @ Robert Road

Description: Paved Channel Section

Solve For Discharge

Given Constant Data;

Bottom Width.......	10.00
Z-Left.............	2.00
Z-Right............	2.00
Mannings 'n'.......	0.015
Channel Slope......	0.0010

Variable Input Data	Minimum	Maximum	Increment By
Channel Depth	0.25	9.00	0.25

Bottom Width ft	Z-Left (H:V)	Z-Right (H:V)	Mannings 'n'	Channel Slope ft/ft	VARIABLE Channel Depth ft	COMPUTED Channel Discharge cfs	COMPUTED Velocity fps
10.00	2.00	2.00	0.015	0.0010	0.25	3.14	1.20
10.00	2.00	2.00	0.015	0.0010	0.50	10.11	1.84
10.00	2.00	2.00	0.015	0.0010	0.75	20.19	2.34
10.00	2.00	2.00	0.015	0.0010	1.00	33.18	2.76
10.00	2.00	2.00	0.015	0.0010	1.25	49.02	3.14
10.00	2.00	2.00	0.015	0.0010	1.50	67.72	3.47
10.00	2.00	2.00	0.015	0.0010	1.75	89.30	3.78
10.00	2.00	2.00	0.015	0.0010	2.00	113.82	4.06
10.00	2.00	2.00	0.015	0.0010	2.25	141.34	4.33
10.00	2.00	2.00	0.015	0.0010	2.50	171.93	4.58
10.00	2.00	2.00	0.015	0.0010	2.75	205.68	4.83
10.00	2.00	2.00	0.015	0.0010	3.00	242.65	5.06
10.00	2.00	2.00	0.015	0.0010	3.25	282.94	5.28
10.00	2.00	2.00	0.015	0.0010	3.50	326.62	5.49
10.00	2.00	2.00	0.015	0.0010	3.75	373.77	5.70
10.00	2.00	2.00	0.015	0.0010	4.00	424.49	5.90
10.00	2.00	2.00	0.015	0.0010	4.25	478.85	6.09
10.00	2.00	2.00	0.015	0.0010	4.50	536.94	6.28
10.00	2.00	2.00	0.015	0.0010	4.75	598.84	6.47
10.00	2.00	2.00	0.015	0.0010	5.00	664.63	6.65
10.00	2.00	2.00	0.015	0.0010	5.25	734.41	6.82
10.00	2.00	2.00	0.015	0.0010	5.50	808.24	7.00
10.00	2.00	2.00	0.015	0.0010	5.75	886.22	7.17
10.00	2.00	2.00	0.015	0.0010	6.00	968.41	7.34
10.00	2.00	2.00	0.015	0.0010	6.25	1054.91	7.50
10.00	2.00	2.00	0.015	0.0010	6.50	1145.80	7.66
10.00	2.00	2.00	0.015	0.0010	6.75	1241.15	7.82
10.00	2.00	2.00	0.015	0.0010	7.00	1341.04	7.98
10.00	2.00	2.00	0.015	0.0010	7.25	1445.55	8.14
10.00	2.00	2.00	0.015	0.0010	7.50	1554.76	8.29
10.00	2.00	2.00	0.015	0.0010	7.75	1668.75	8.44
10.00	2.00	2.00	0.015	0.0010	8.00	1787.59	8.59
10.00	2.00	2.00	0.015	0.0010	8.25	1911.36	8.74
10.00	2.00	2.00	0.015	0.0010	8.50	2040.13	8.89
10.00	2.00	2.00	0.015	0.0010	8.75	2173.99	9.03
10.00	2.00	2.00	0.015	0.0010	9.00	2313.00	9.18

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: W-14 @ Robert Road

Description: Single Cell Computations

Solve For Actual Discharge

Given Constant Data;

Diameter...........	7.50
Slope..............	0.0010
Mannings n.........	0.025

Variable Input Data	Minimum	Maximum	Increment By
Depth	0.25	7.50	0.25

Diameter ft	Channel Slope ft/ft	Mannings 'n'	VARIABLE Discharge cfs	COMPUTED Depth ft	COMPUTED Velocity fps	Capacity Full cfs
7.50	0.0010	0.025	0.25	0.25	0.56	126.27
7.50	0.0010	0.025	1.12	0.50	0.88	126.27
7.50	0.0010	0.025	2.64	0.75	1.15	126.27
7.50	0.0010	0.025	4.81	1.00	1.37	126.27
7.50	0.0010	0.025	7.62	1.25	1.58	126.27
7.50	0.0010	0.025	11.06	1.50	1.76	126.27
7.50	0.0010	0.025	15.08	1.75	1.92	126.27
7.50	0.0010	0.025	19.65	2.00	2.08	126.27
7.50	0.0010	0.025	24.73	2.25	2.22	126.27
7.50	0.0010	0.025	30.27	2.50	2.35	126.27
7.50	0.0010	0.025	36.23	2.75	2.47	126.27
7.50	0.0010	0.025	42.55	3.00	2.58	126.27
7.50	0.0010	0.025	49.18	3.25	2.68	126.27
7.50	0.0010	0.025	56.06	3.50	2.77	126.27
7.50	0.0010	0.025	63.13	3.75	2.86	126.27
7.50	0.0010	0.025	70.33	4.00	2.94	126.27
7.50	0.0010	0.025	77.59	4.25	3.00	126.27
7.50	0.0010	0.025	84.83	4.50	3.07	126.27
7.50	0.0010	0.025	91.99	4.75	3.12	126.27
7.50	0.0010	0.025	98.98	5.00	3.16	126.27
7.50	0.0010	0.025	105.72	5.25	3.20	126.27
7.50	0.0010	0.025	112.11	5.50	3.23	126.27
7.50	0.0010	0.025	118.05	5.75	3.25	126.27
7.50	0.0010	0.025	123.42	6.00	3.26	126.27
7.50	0.0010	0.025	128.09	6.25	3.26	126.27
7.50	0.0010	0.025	131.89	6.50	3.24	126.27
7.50	0.0010	0.025	134.58	6.75	3.21	126.27
7.50	0.0010	0.025	135.80	7.00	3.16	126.27
7.50	0.0010	0.025	134.85	7.25	3.08	126.27
7.50	0.0010	0.025	126.27	7.50	2.86	126.27

APPENDIX C

CORTECH, INC.
REPORT

ENGINEERING REPORT

W-14 CANAL

CITY OF SLIDELL, LOUISIANA

Prepared by

CORTECH, INC.
CONSULTING ENGINEERS

November 1992
Revised December 28, 1992

CI JOB NO. 92-10

Arch Properties

$R = 1.25$ @ 80% rise (Hydraulic Radius) A/wp

$A = 63\%$ of total @ 50% rise (Area)

Manning's Equation – Open Channel Flow

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \quad \text{fps.}$$

$$Q = AV = \text{cfs.}$$

For fairly regular section (earthen):

$n = 0.030 - 0.035$ (some grass & weeds, little or no br.)

$= 0.050 - 0.070$ (Some weeds, heavy brush)

(increase 0.01 to 0.02 for trees within channel, submerged branches, etc).

For regular 5×1 corrugation Pipe

$n = 0.020 - 0.022$ (plain)

$n = 0.012 - 0.014$ (Paved bottom)

Conclusion: a culvert can convey more than twice the water of an equivalent size earthen canal (less friction, more veloci[ty]

Water Transporting Silts

Velocity required (minimum):

- Fine Sand ——— 2.5 f.p.s.
- Silt Loam ——— 3.0 f.p.s.
- Ordinary firm Loam — 3.5 f.p.s.
- Stiff clay ——— 5.0 f.p.s.
- Fine Gravel ——— 5.0 f.p.s.

ENGINEERING REPORT

W-14 CANAL

CITY OF SLIDELL, LOUISIANA

STATE OF LOUISIANA
CARLOS A. CORDON
REG. No. 8383
REGISTERED
PROFESSIONAL ENGINEER
IN
CIVIL ENGINEERING

1/6/93

Prepared by

CORTECH, INC.
Consulting Engineers

November 1992
Revised December 28, 1992

CI JOB NO. 92-10

ENGINEERING REPORT

W-14 CANAL

TABLE OF CONTENTS

EXHIBITS

1. <u>Background Information</u>

The W-14 Drainage Canal is the major drainage facility within the Slidell City Limits. From its beginning just north of Brownswitch Road near its intersection with US Highway 11, it flows southward for a distance of approximately five (5) miles to its crossing of Interstate Highway 10.

As the only major conveyor of storm water east of Front Street, it takes care of the runoff from approximately eighteen hundred (1800) acres, including portions of several of Slidell's older subdivisions such as:

o Country Club Estates

o Brookwood Estates

o Forest Manor

o Wimbledon Estates

o Fountain Estates

o Heritage Estates

o Bon Village

o Fountainbleau

o Broadmoor

o Lakewood

Throughout the years, suburban and commercial development have placed increasing capacity demands on the W-14 to be able to remove stormwater from the tributary areas served without causing unacceptable street flooding, residential yard flooding, or property damage.

An aggressive maintenance program undertaken by the City has for the last ten (10) years been able to achieve the W-14 role, or at least minimize its impact on residents along its route.

2. <u>Deficient Sections</u>

Previous studies and engineering reports have correctly identified what has been defined as major "bottleneck" areas. These "bottleneck" areas are deficient canal sections which effectively restrict the free flow of stormwater, are sources of safety concerns, and have created property damage by erosion, subsidence, or a combination of both.

The first major "bottleneck" is located between a point roughly 500 feet north of Gause Boulevard (Highway US 190) to the Fremaux Avenue bridge, a distance of approximately 3,300 linear feet.

The reason why this section of the W-14 has become a source of problems and concern is twofold.

a. Inadequate Drainage Right of Way: The present legally defined right-of-way in this area varies from 50 feet to 60 feet wide. The present average top width of the Canal is approximately 45 feet. It has been estimated that an adequate canal section with stable banks would be at least 60 feet wide at its top of banks. Such a facility would require a minimum right-of-way width of 110 feet for proper maintenance.

b. Inaccessability for Maintenance Work: Development of roadways and residential subdivisions up to the Canal's right-of-way lines make access to the canal with heavy maintenance equipment impossible; and no cost effective way to secure temporary access through private property for maintenance work.

The second major "bottleneck" area is the section north of Robert Boulevard between Robert Boulevard and Darwin Court in Wimbledon Estates.

This section of the W-14 Canal meanders along the back property of residents from Fountain Estates and Wimbledon Estates Subdivisions, and as in the previous deficient section, it lacks adequate accessibility for adequate maintenance work. Additionally, the west right-of-way line of the Canal bordering what is known as the Huey Richardson Property (undeveloped) appears to be not legally defined, so there is no known right-of-way or easement along this section of canal. The existing canal is close enough to the backyards of the residential lots to pose a threat to private property and safety.

The third "bottleneck" area is the canal section south of the Robert Boulevard crossing. This section includes a protion of canal approximately 150 feet in length which although not immediately adjacent to any residential and/or commercial property, traverses a parcel of land known as the Marie H. Olroyd Property. As in the case of the Richardson Property, there appears to be no legally defined right-of-way for the Canal through this property, making accessibility for maintenance and improvements very difficult or impossible.

The main reason this section is considered a "bottleneck" is because sediments deposited just downstream feom the triple 90-inch diameter culverts crossing Robert Boulevard tend to accumulate in that location drastically reducing the Canal's hydraulic section and its ability to convey the stormwater runoff generated by the upstream areas of the W-14 Canal and the Robert Boulevard subsurface drainage.

3. Analysis

The scope of this report will be limited to the study of the first two major deficient sections previously defined.

A. Gause Boulevard Area:

Since obtaining additional right-of-way either by purchase or eminent domain proceedings appears to be very costly and time consuming due to the total development of the area, alternate solutions utilizing the existing available space have been analyzed in the past, such as:

(1) Concrete Paving of the Bottom and Sides of the Canal: This alternative would improve the hydraulic characteristics of the canal; however, the inaccessible nature of the Canal makes this work quite expensive and hard to maintain due to lack accessibility.

(2) Placing of Steel Sheet Piling Walls on Both Sides of the Canal: This alternative would also improve the hydraulic section of the Canal and provide excellent bank stabilization for the protection of adjacent property and structures. Construction cost would be higher than normal because of space limitations, and maintenance would still present the same accessibility problems.

(3) Full Enclosure of the Canal Section: This alternative offers the best features, but also the highest cost of the viable alternatives. A list of the advantages follows:

- A canal enclosure can be accomplished within the existing right-of-way;
- It will reduce canal maintenance to a minimum as vegetation and trees no longer will grow on the canal bottom and sides;
- It provides significantly better hydraulic characteristics requiring reduced canal sections to handle design flows;
- It induces higher flow velocities which reduce the settlement of eroded materials, thus minimizing maintenance costs;
- It will eliminate bank stability and erosion problems;
- It will present a much safer solution than any open canal alternative;

- It will provide a more aesthetically pleasing alternative since parklike grounds would substitute the open canal.

In the past, the merits of the third alternative have been fully addressed and the general consensus has been that the totally enclosed canal section is the alternative that should be implemented. The reason why it has not been started to date has always been the high construction cost normally associated with a concrete structure of the size required to accomplish the task, $1200 to $1400 per linear foot of canal.

New technology and availability of lighter and durable substitute materials to the traditional concrete box culvert now make the canal closure a simpler and more economical solution.

B. Robert Boulevard Area:

All of the restraints and conditions previously outlined for the Gause Boulevard Area are also applicable to this section with two exceptions:

(1) The hydraulic section requirements of this segment of the W-14 Canal are less than those of the Gause Boulevard Area.

(2) The west boundary line of the Canal right-of-way is not legally defined or fronting on developed residential/commercial parcels. Instead, the Canal is adjacent to a large wooded and undeveloped tract of land which makes the securing of necessary right-of-way at least a feasible solution to the problem.

4. <u>Recommendations</u>

A. Gause Boulevard Area:

Considering that the canal enclosure structure will not be subjected to traffic loading and will only have to support the earth fill material necessary to complete the canal closure and grass cutting equipment, medium gauge galvanized steel corrugated arch pipe can be used advantageously. It is recommended that a 144 inch diameter (172 inch span by 107 inch rise equivalent arch section), 8 gauge wall thickness, corrugated galvanized pipe with asphaltic cement paved bottom be used for canal closure structure. This pipe section weighs only 343 pounds per linear foot and can be delivered in thirty to forty foot long sections; installation costs can then be kept to a minimum.

It is recommended that the project be implemented in phases, with the first phase being the narrow upsteam portion north of Gause Boulevard and paralleling Rue Verand, for approximately 250 linear feet, an area that exhibits pronounced narrowing in the canal section, very limited right-of-way and in close proximity to the Rue Verand roadway, and a section south of Gause Boulevard, next to Broadmore Street, also with very limited right-of-way and a history of erosion and sedimentation. Subsequent phases could be implemented in later years under a Capital Improvements Program based upon availability of funds. The recommended improvements for the First Phase Work are shown in the attached drawing.

B. Robert Boulevard Area:

It is recommended that an adequate right-of-way sufficient to enlarge the canal section and provide for maintenance operations be either resurrected or acquired on both sides of the Robert Boulevard crossing. This right-of-way should be one hundred feet (100') in width, similar to the one existing at Heritage Estates.

Based on the availability of right-of-way, improvements can be implemented to fit the requirements of the widened section. A typical canal section arrangement is shown at the end of this Report.

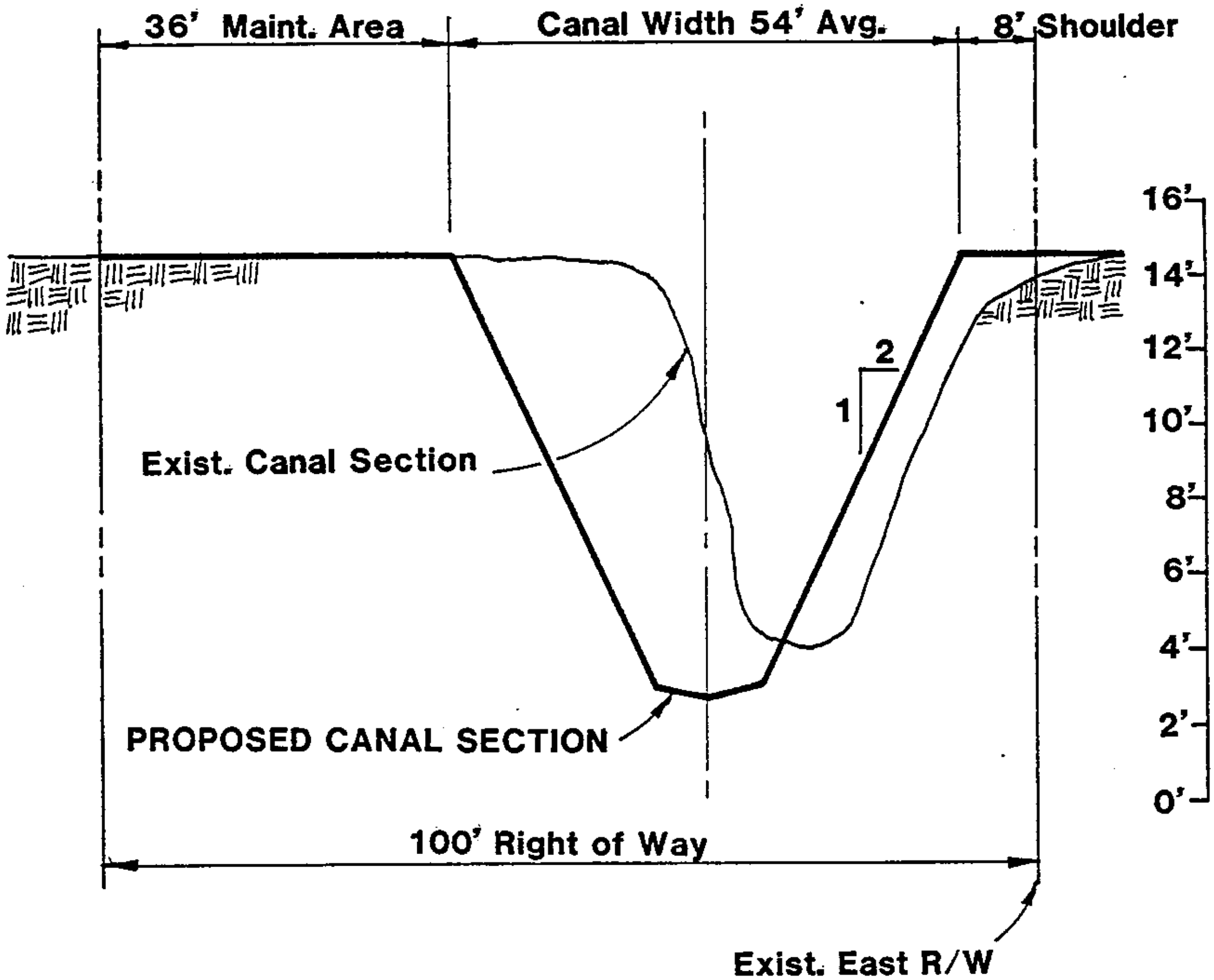
5. <u>Project Cost Estimate</u>

The following is an Opinion of Probable Construction Cost for the Phase One Improvements of the Gause Boulevard Area.

PROPOSED W-14 CANAL CLOSURE
CITY OF SLIDELL

OPINION OF PROBABLE CONSTRUCTION COST

ITEM	DESCRIPTION	AMOUNT
1.	Site Preparation, Clearing, Grubbing, and Excavation, Lump Sum	$ 6,000.00
2.	Installation of 172"x107" Corrugated Metal Arch Pipe, 330 linear feet @ $260.00/LF	85,800.00
3.	Construction of Reinforced Concrete Transition Structure No. 1, including access grating, complete, Lump Sum	12,000.00
4.	Construction of Reinforced Concrete Transition Structure No. 2, including access grating, complete, Lump Sum	20,000.00
5.	Construction of Treated Timber Headwalls with Wing Walls, Pilings, Etc., Lump Sum	9,000.00
6.	Treated Timber Closure Walls at two (2) existing concrete box culverts, approximately 24 linear feet, Lump Sum	4,300.00
7.	Backfill Materials for completion of Canal Section Enclosure, including topsoil, seeding, and fertilizing, Lump Sum	22,000.00
	SUB TOTAL CONSTRUCTION COST	$159,100.00
8.	Contingencies (10%)	15,900.00
9.	Engineering, Topographic Survey, and Soil Testing	16,000.00
	TOTAL CONSTRUCTION COST	$191,000.00

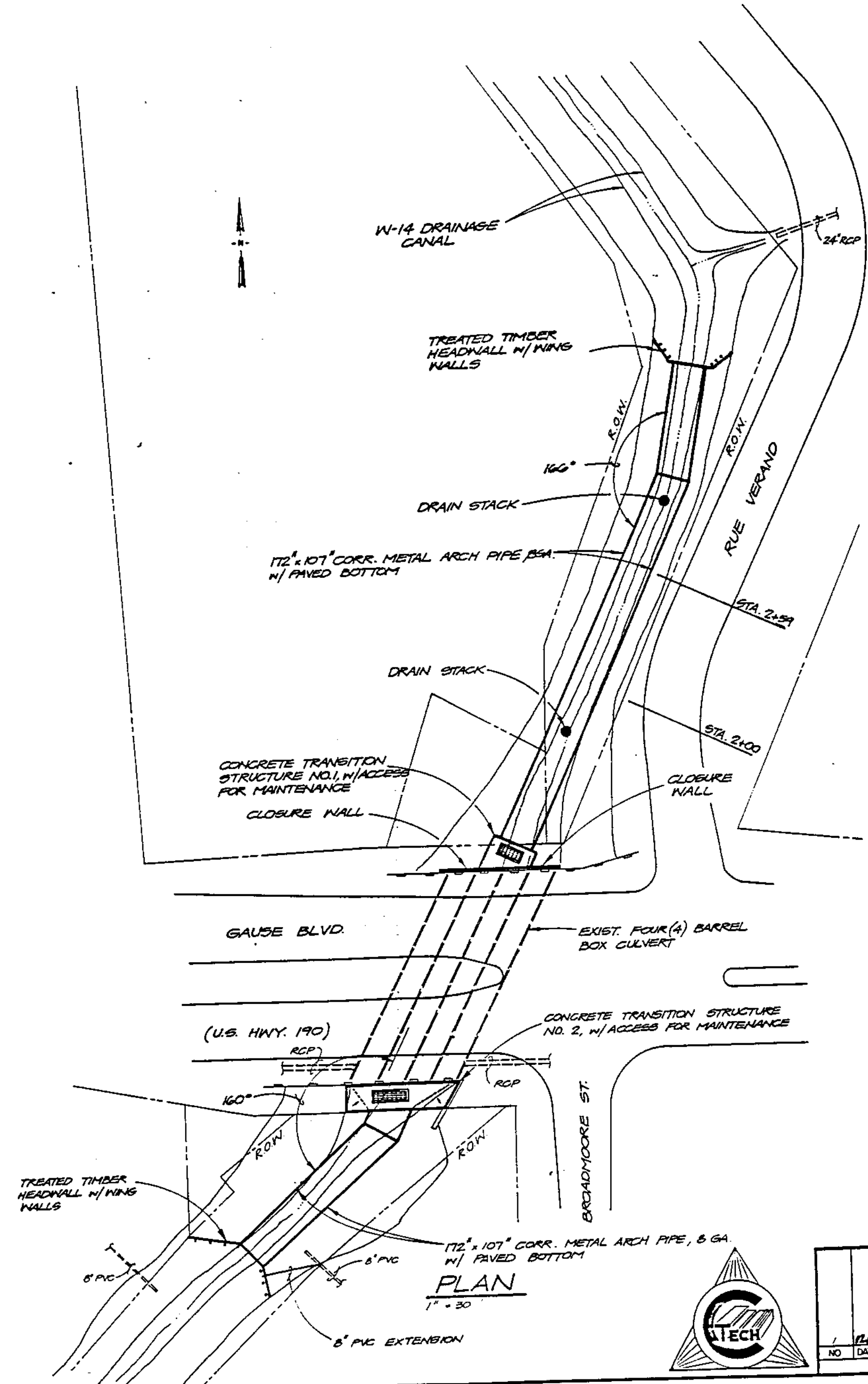


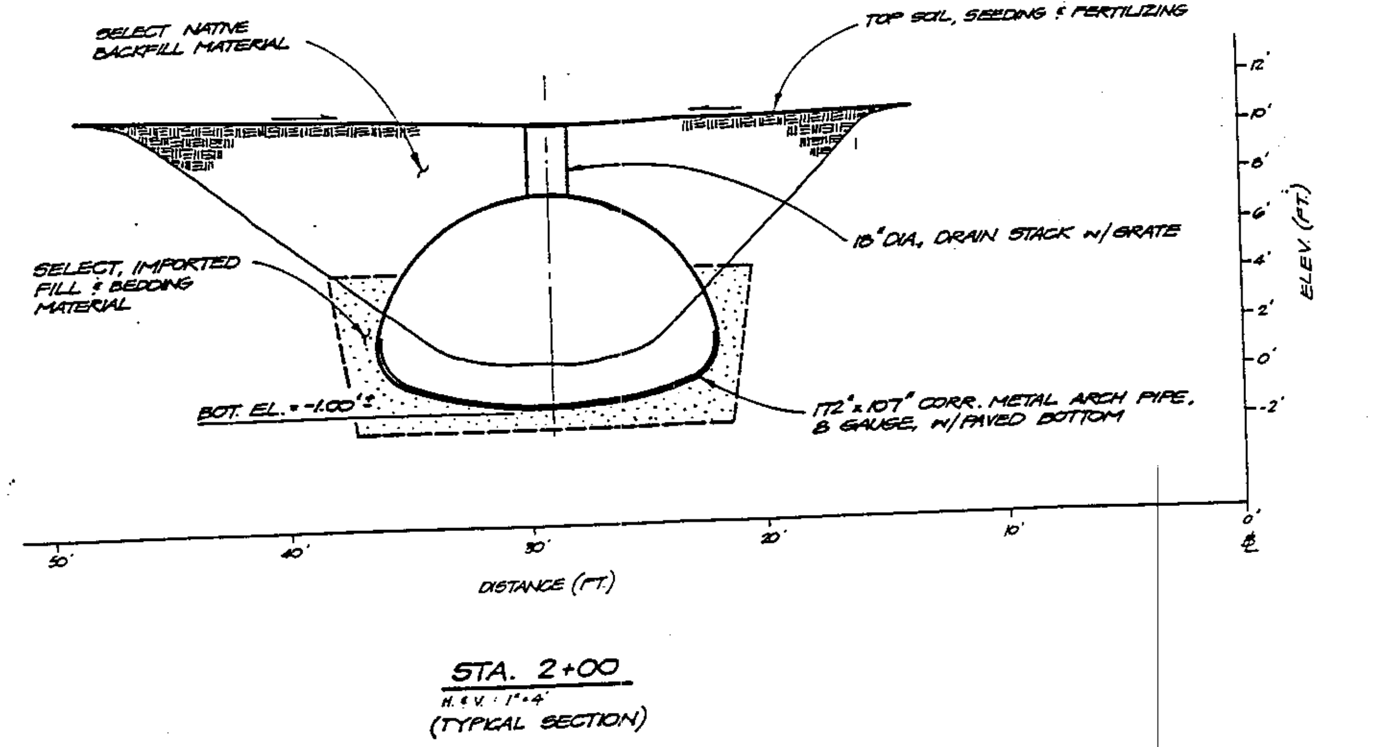
W – 14 Drainage Canal

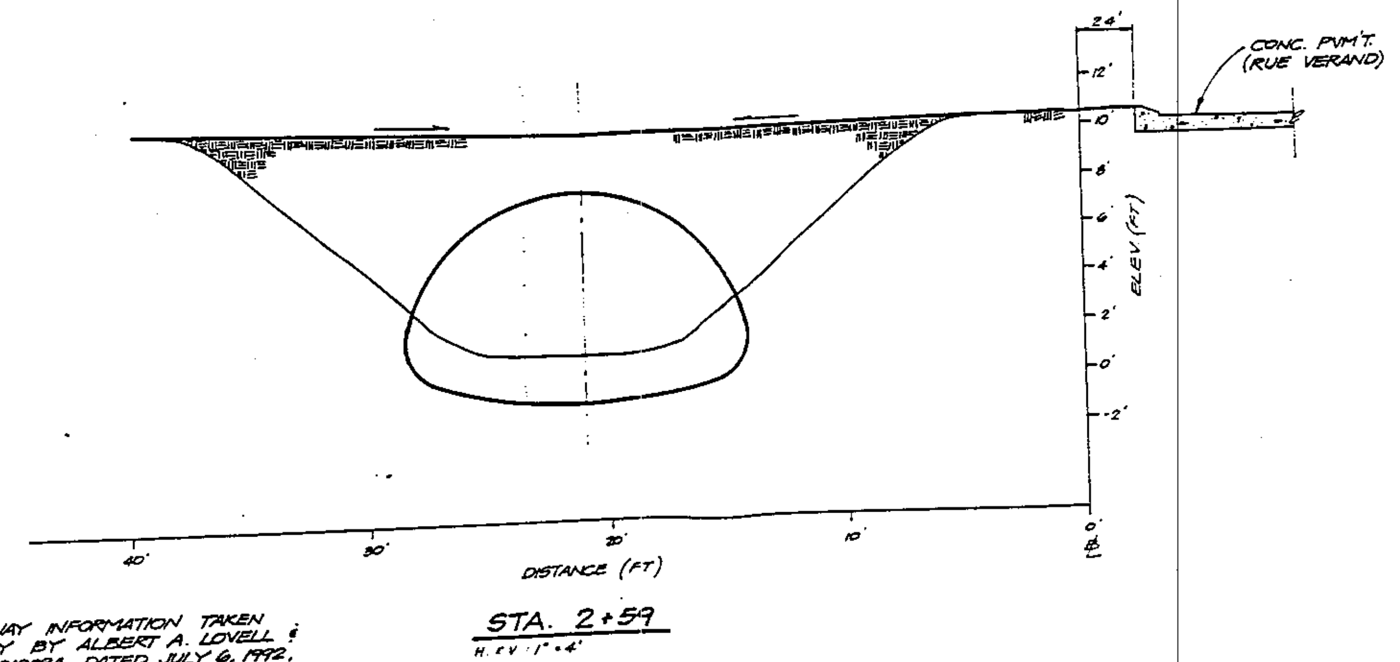
Typical Section North of Robert Blvd.

1" = 20' Horz.
1" = 5' Vert.

CORTECH INC.
Consulting Engineers







NOTE: RIGHT-OF-WAY INFORMATION TAKEN FROM SURVEY BY ALBERT A. LOVELL & ASSOC., NO. 101289A, DATED JULY 6, 1992, PERFORMED FOR CITY OF SLIDELL.

NO	DATE	DESCRIPTION	BY	APP
1	12/92	RELEASED W/ REPORT	RFS	CAC
		REVISION		

DESIGNED: C.A. CORDON
DRAWN: R.F. SIEGERT
CHECKED: C.A. CORDON
APPROVED: C.A. CORDON

Proposed
W-14 Canal Enclosure

PLAN and SECTIONS

STATE OF LOUISIANA
CARLOS A. CORDON
REG. No. 8383
REGISTERED PROFESSIONAL ENGINEER IN CIVIL ENGINEERING

CORTECH, INC.
Consulting Engineers

The City of Slidell

SHEET 1 | DWG —
PROJECT 92-010
DATE DECEMBER 1992

Burk-Kleinpeter, Inc.
Engineers, Architects, Planners
and Environmental Scientists

4176 Canal Street
New Orleans, LA 70119
(504) 486-5901

BKI No. 9368-01
May 1994

CITY OF SLIDELL

MASTER DRAINAGE PLAN

ADDENDUM TO Task Order No. 9

Plan of Action

Prepared by:

BURK-KLEINPETER, INC.
HARTMAN ENGINEERING, INC.

October, 1994

CITY OF SLIDELL
MASTER DRAINAGE PLAN
ADDENDUM TO
Task Order No. 9
Plan of Action

EXECUTIVE SUMMARY

This addendum to the City of Slidell Master Drainage Plan Task Order No. 9 *Plan of Action* proposes recommendations to relieve residential and street flooding in the W-14 Canal drainage basin by combining flood control measures (levees and outlet structures) and pumping capacity at the canal outfall (see Section V of the *Plan of Action*) with channel improvements over the length of the canal. This study was developed based on recommendations and design criteria set forth in the City of Slidell Master Drainage Plan (Burk & Associates, 1983).

The 1983 MDP reveals that the W-14 Canal is largely incapable of handling the design flows from a 10-Year, 24-Hour Storm Event over the basin. This addendum provides the design criteria necessary for upgrading the W-14 to the 10-year flood level capacity. Improvements include excavation and clearing of the entire canal and slope paving some sections through developed neighborhoods where right-of-way restrictions are a concern. The total project cost is estimated at $15,969,300.

SECTION VII

W-14 CANAL IMPROVEMENTS

TABLE OF CONTENTS

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LIST OF APPENDICES

I. Introduction

The W-14 Main Diversion Canal is the major drainage outlet for the center of the City of Slidell, covering a 5-mile stretch from its source just north of Interstate Highway 12 near Brownswitch Road to its crossing with Interstate Highway 10 (see Site Map, Figure 1). From there it continues southeasterly paralleling Louisiana Highway 433, draining into the Fritchie Marsh and ultimately into Lake Pontchartrain. The W-14 drains approximately 1,800 acres of residential subdivisions from Whisperwood Estates and Country Club Estates at the north to Broadmoor, Fountainbleau, and Lakewood subdivisions at the south. The entire drainage basin covers over 5,500 acres; its boundaries are described in the Drainage Area Map, Figure 2.

Substantial suburban and commercial development in the area over the years has caused increased volumes of runoff and quicker times to peak runoff during storm events. Higher demands are thus placed on the W-14 Canal and its capacity is often exceeded during heavy rains, causing significant street flooding and some property damage. The 1983 St. Tammany Master Drainage Plan (prepared by Burk & Associates, Inc.) found that the existing capacity of the canal is generally not adequate to handle flows of the magnitude experienced from a 10 Year-24 Hour Storm Event.

In addition to flooding due to inadequate channel capacity, the W-14 Canal is also subject to backwater flooding from high water levels in Lake Pontchartrain (refer to Section V of the *Plan of Action*). Because the W-14 Canal is currently gravity-drained, and because the marshland at its outfall is relatively flat and low-lying, tidal flows in the lake will impede the flow of the W-14 and force the flow direction to reverse as the system attempts to alleviate pressure built up by the high water levels in the lake. Incoming high water from the lake will flow upstream through the W-14 Canal and adversely effect the hydraulic slope of the canal, causing overflow.

II. Project Description

The problem of backwater flooding in the W-14 Canal will be handled by the installation of flood protection measures at the outfall of the W-14 Canal where it empties into the Fritchie Marsh and Lake Pontchartrain. As described in Section V of the Task Order No. 9 *Plan of Action* (Burk-Kleinpeter, 1994), these measures include a ring levee with automatic drainage gate structures and a pumping station. When the lake level rises above that of the W-14 Canal, the gates would close under the negative differential head, storm water would fill the sump area behind the levee, and the pumps would begin operating to lift the excess storm water into the lake.

The scope of this addendum to the *Plan of Action* is to propose improvements to the W-14 canal by modifying the channel cross-section in order to meet the changing drainage capacity requirements of its increasingly-urbanized basin. Recommendations are based on the 10 Year-24 Hour Storm Event over the W-14 basin. This project is designed in conjunction with and to be carried out following the construction of the outfall flood protection measures.

the entire *existing* capacity of the W-14 Canal outfall (1650 cfs, or approximately 740,500 gpm) into Lake Pontchartrain. In the future, assuming additional funds can be provided, the pump station would be improved to a capacity equal to the W-14 design flow of 4000 cfs predicted from a 10 Year-24 Hour Storm Event through the southernmost improved canal section. Subsequently, the entire length of the W-14 Canal would be modified to handle the full 10-year runoff volume over each subarea and provide the 4000-cfs capacity at the outfall.

Two alternates are possible for improving the canal section: the hydraulic radius of the canal may be increased through widening or excavation; or the "roughness" of the natural earthen canal may be reduced by lining or paving it with concrete. Paving is an expensive alternative, but it is often required in tight right-of-way situations or where flow velocities are high and may cause scour of the natural earth embankment. In addition, paved channels are more desirable from a maintenance standpoint.

Observing the path of the W-14 Canal (see again Figure 1), the northern section winds through a developed area of the City of Slidell where the right-of-way is typically 50 to 60 feet wide; residential and roadway development up to the canal's right-of-way have eliminated clearings for maintenance equipment and machinery. Based on the 10 Year-24 Hour Storm Event design flows from the 1983 Master Drainage Plan, it is estimated that an adequate earthen canal section with stable banks would have at least a 60-foot top width, requiring a minimum right-of-way width of 110 feet for proper maintenance. Obtaining additional right-of-way in this area would be time-consuming and cost-prohibitive due to the apparent lack of right-of-way definition in some locations as well as the degree of land development in the area. In addition, the existing canal in this section is close enough to residential backyards to pose a threat to private property and safety. For these reasons, it is not feasible to maintain a widened natural earth canal in this stretch of the W-14 Canal; thus, it is proposed to fully concrete-line the canal so that steeper bank slopes may be used, narrower right-of-way widths are permitted, and no maintenance clearings are required.

The degree of land development decreases significantly in the southern reaches of the W-14. For this stretch of canal, the cheaper alternative of widening and/or excavating the canal may be employed, since right-of-way restrictions are not as stringent of a concern.

V. Conceptual Design

It is proposed that the W-14 Canal improvements be implemented over the section from the North Boulevard crossing to the proposed location of the pump station just past Voters Road. The section would be concrete-lined from North Boulevard to Daney Street, and natural earth from Daney Street to the outfall at Lake Pontchartrain.

The design criteria for the paved section is are as follows: side slopes at 2 horizontal to 1 vertical (2H : 1V), a minimum velocity of 4 feet-per-second (fps) to prevent sedimentation, a maximum velocity of 8 fps, and a top width narrow enough to fit in the existing right-of-way. The design criteria for the widened natural earth channel is to maintain 3:1 side slopes and restrict the maximum flow velocity to 3 fps in order to prevent scour. Channel width is not an overriding concern, since additional right-of-way can be purchased in this area, if necessary. The calculated channel data for each design point are given in Table 1.

Currently, the invert of the channel varies from Elevation (+)6.4' National Geodetic Vertical Datum (NGVD) at North Boulevard to Elevation (-)4.0' NGVD at the outfall; the top width of the channel varies from 31 feet to 65 feet at these locations. Preliminary analysis of the channel determines that after excavation of the canal bottom, the invert elevations will range from Elevation (+)4.8' NGVD at North Boulevard to Elevation (-)8.0' NGVD at the outfall, and the corresponding widths will vary from 45 feet to 125 feet. A design water surface elevation of (+)2.0' NGVD at the outfall was selected for the basis of hydraulic analysis so that the water surface in the W-14 Canal would be kept above that in the lake (average water elevation of 1.1' NGVD) as much as possible to utilize gravity drainage and reduce pumping costs. Final canal inverts and water surface elevations will be determined as part of the pumping station design.

Although the proposed canal sections were sized based on the 10 Year-24 Hour Storm Event, it is inevitable that the "roughness" of the canal will increase through the years due to obstructions or aging of the concrete and earthen channels. Increasing roughness will increase the water surface elevation in the canal, and thus some freeboard (typically 2 feet) is required from the design water surface to the top of the canal banks. Thus, some small levee construction and addition of automatic drainage gate structures on existing drainage outfalls will be required along certain sections of the canal where the banks are insufficient to provide this capacity (see Channel Cross-Sections, Appendix C).

VI. Estimated Design / Construction Cost

Table 3 presents a preliminary tabulation of each construction item covered in this project and its associated cost. The total construction cost is estimated at $13,867,800; adding design costs ($1,386,800), construction administration ($554,700), and resident inspection ($160,000) brings the estimated total project cost to $15,969,300.

VII. Estimated Design / Construction Time Schedule.

The estimated design time schedule for this phase of the project is 12 months for the project design, which includes preparation of plans, specifications, and contract documents, 2 months for bidding and award of the contract, and an additional 20 months for construction of the project. This estimated time schedule assumes the required funding for the project to be available and does not include any possible delays caused by appropriation of money to fund this project.

TABLE 1

DESIGN CHANNEL DATA

DESIGN POINT	Reach (ft)	MDP 10-yr. Flow (cfs)	Prop. Top Width (ft)	Prop. Bottom Width (ft)	Side Slope (ft/ft)	"n"	Max V (fps)	Prop. Invert Elev. (MSL)	Exist. Top of Bank (MSL)	Prop. WSE (MSL)
Paved Canal										
North Blvd.		886	45.0	10.0	2:1	0.013	8.0	4.8	13.44	12.59
Robert Rd.	4970	1172	45.0	10.0	2:1	0.013	8.0	2.6	12.36	11.51
Independ. Dr.	780	1688	50.0	15.0	2:1	0.013	8.0	1.3	13.42	10.33
Gause Blvd.	2200	2016	55.0	20.0	2:1	0.013	8.0	1.0	11.66	9.66
Florida Ave.	750	2030	55.0	20.0	2:1	0.013	8.0	0.0	11.58	9.35
Fremaux Ave.	2000	2054	55.0	20.0	2:1	0.013	8.0	-0.3	8.68	8.73
Cousin St.	750	2078	55.0	20.0	2:1	0.013	8.0	-0.5	8.43	8.66
Daney St.	2040	2102	75.0	30.0	2:1	0.013	8.0	-2.0	7.98	8.57
Earth Canal										
I-10	6310	3420	110.0	25.0	3:1	0.033	3.0	-6.7	9.67	7.93
Kingspt. Blvd.	2300	3686	115.0	35.0	3:1	0.033	3.0	-6.9	10.81	7.28
Voters Rd.	790	3925	125.0	50.0	3:1	0.033	3.0	-7.0	7.30	7.03
LA 433	13110	4000	125.0	65.0	3:1	0.033	3.0	-8.0	1.65	2.00

TABLE 2

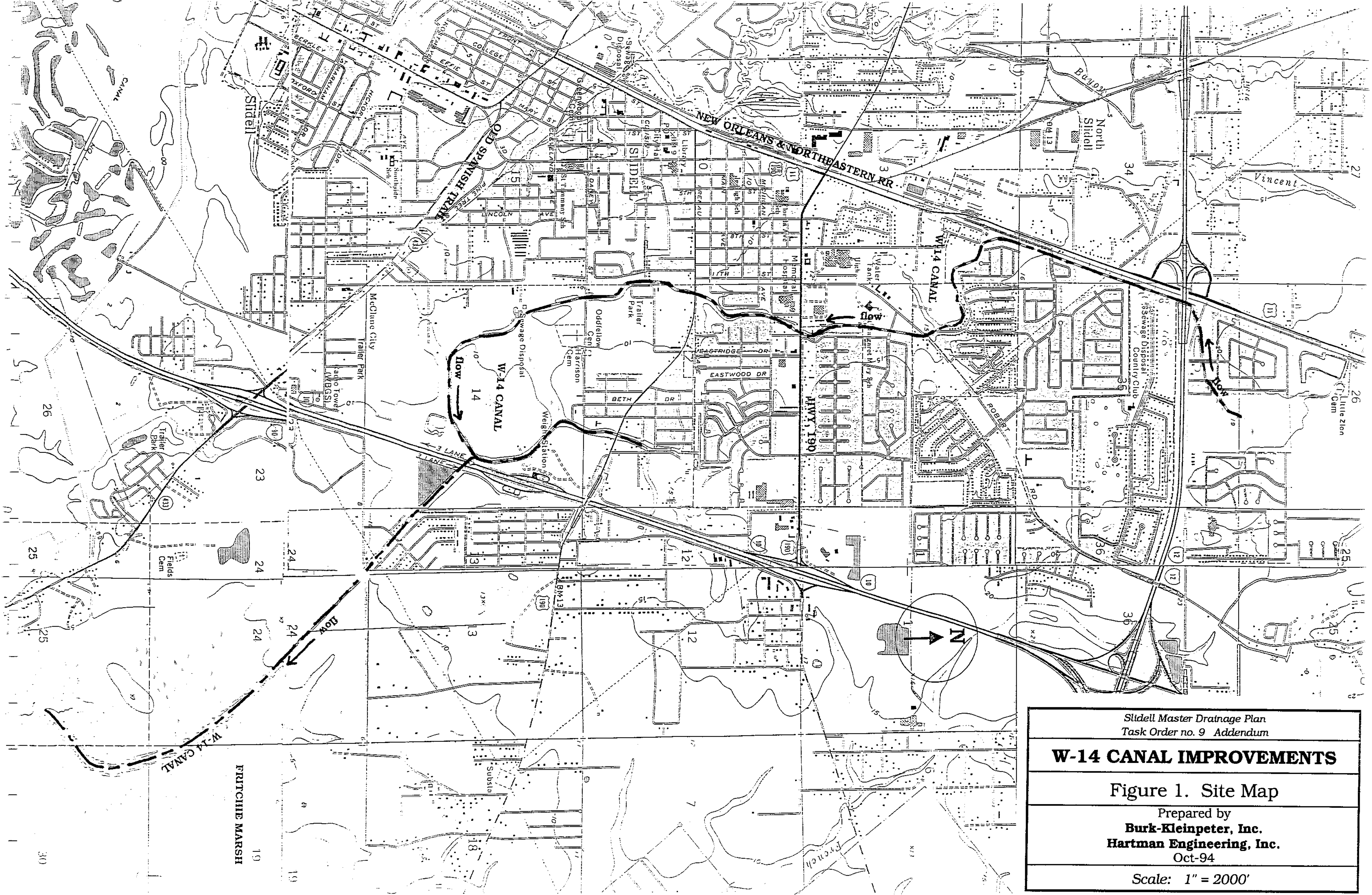
DESIGN PARAMETERS

Parameter	Value	Source
10-yr. design flows	varies	St. Tammany MDP
Right-of-way width	varies	St. Tammany MDP
W-14 side slopes - paved section	2:1	St. Tammany MDP
Manning's "n" - paved section	0.013	Design Manual
Minimum velocity - paved section	4 fps	Design Manual
Maximum velocity - paved section	8 fps	St. Tammany MDP
W-14 side slopes - earth section	3:1	St. Tammany MDP
Manning's "n" - earth section	0.033	Design Manual
Maximum velocity - earth section	3 fps	Design Manual
Design freeboard	2 ft	Design Manual
Lake Pontchartrain levels - Mean	(+)1.1' MSL	Historical Data
Avg. Annual Low	(-)0.5' MSL	
High	(+)4.3' MSL	

TABLE 3

ESTIMATED DESIGN / CONSTRUCTION COST

Description	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump	1	Lump Sum	$550,000
Excavation/Clearing	Cubic Yard	390,000	$10	$3,900,000
Levee construction	Linear Ft	20,500	$13	$266,500
Concrete slope paving	Cubic Yard	67,360	$30	$6,300,000
Crossings at Daney St., Cousin St., Fremaux Ave., and Florida Blvd.	Each	4	$135,000	$540,000
SUBTOTAL				$11,556,500
CONTINGENCY @ 20%				$2,311,300
TOTAL				$13,867,800



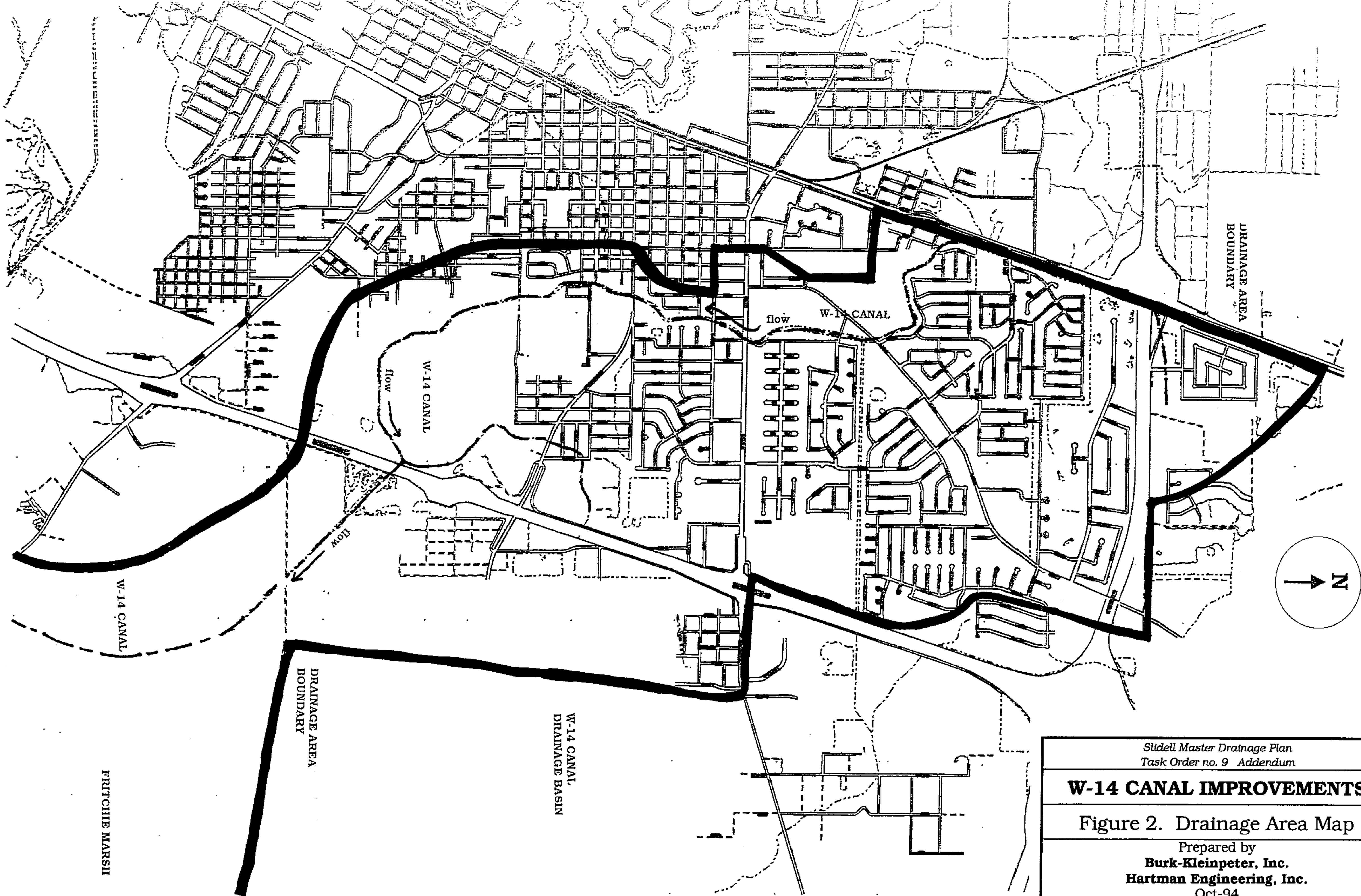
Slidell Master Drainage Plan
Task Order no. 9 Addendum

W-14 CANAL IMPROVEMENTS

Figure 1. Site Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
Oct-94

Scale: 1" = 2000'



Slidell Master Drainage Plan
Task Order no. 9 Addendum

W-14 CANAL IMPROVEMENTS

Figure 2. Drainage Area Map

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
Oct-94

Scale: 1" = 2000'

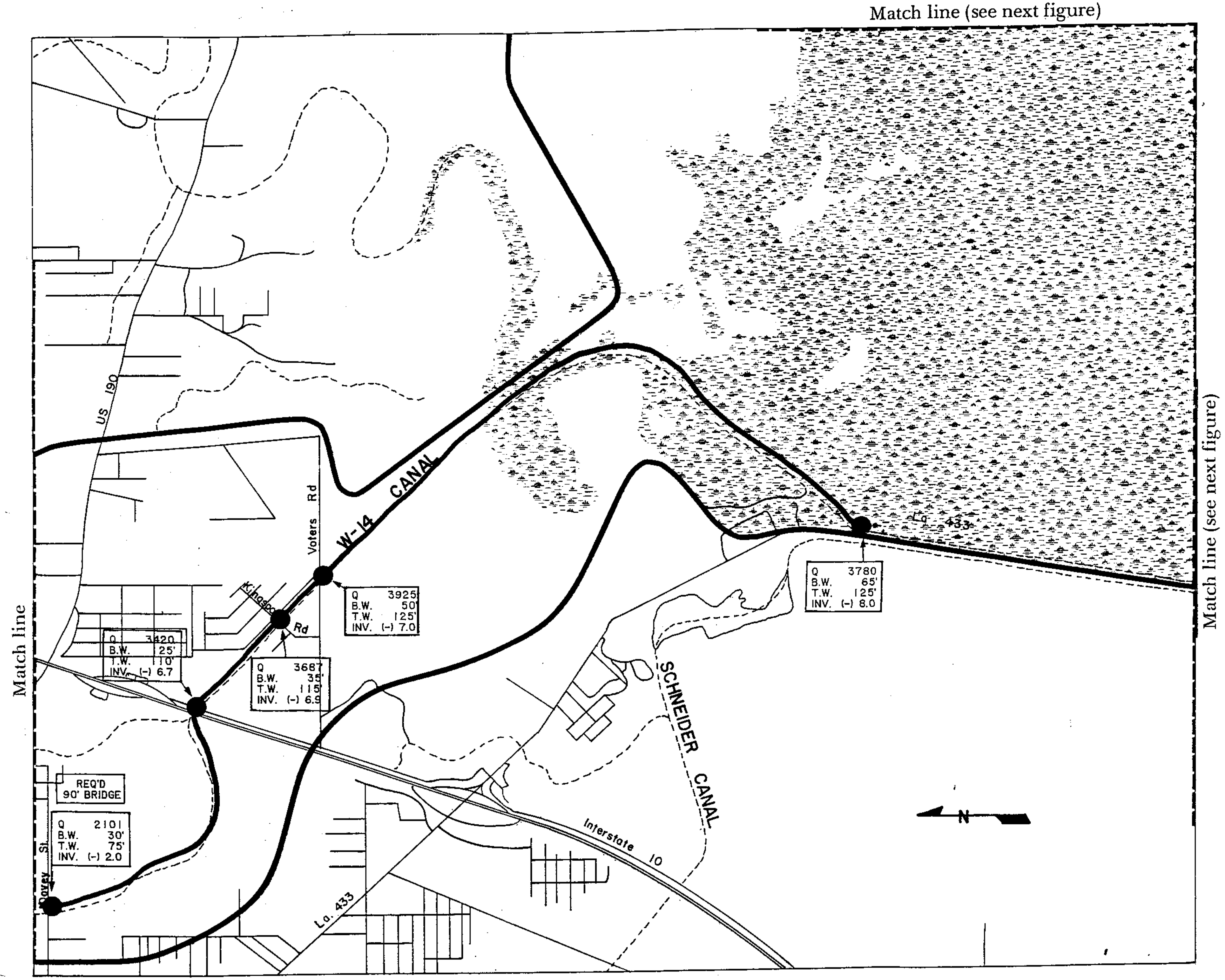
Slidell Master Drainage Plan
Task Order no. 9 Addendum

W-14 CANAL IMPROVEMENTS

Figure 3. Design Canal Layout

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
Oct-94

Scale as noted



Drainage Basin Boundary

----- Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Earthen Section

Burk & Associates Incorporated

1000 0 1000 2000 3000 FEET

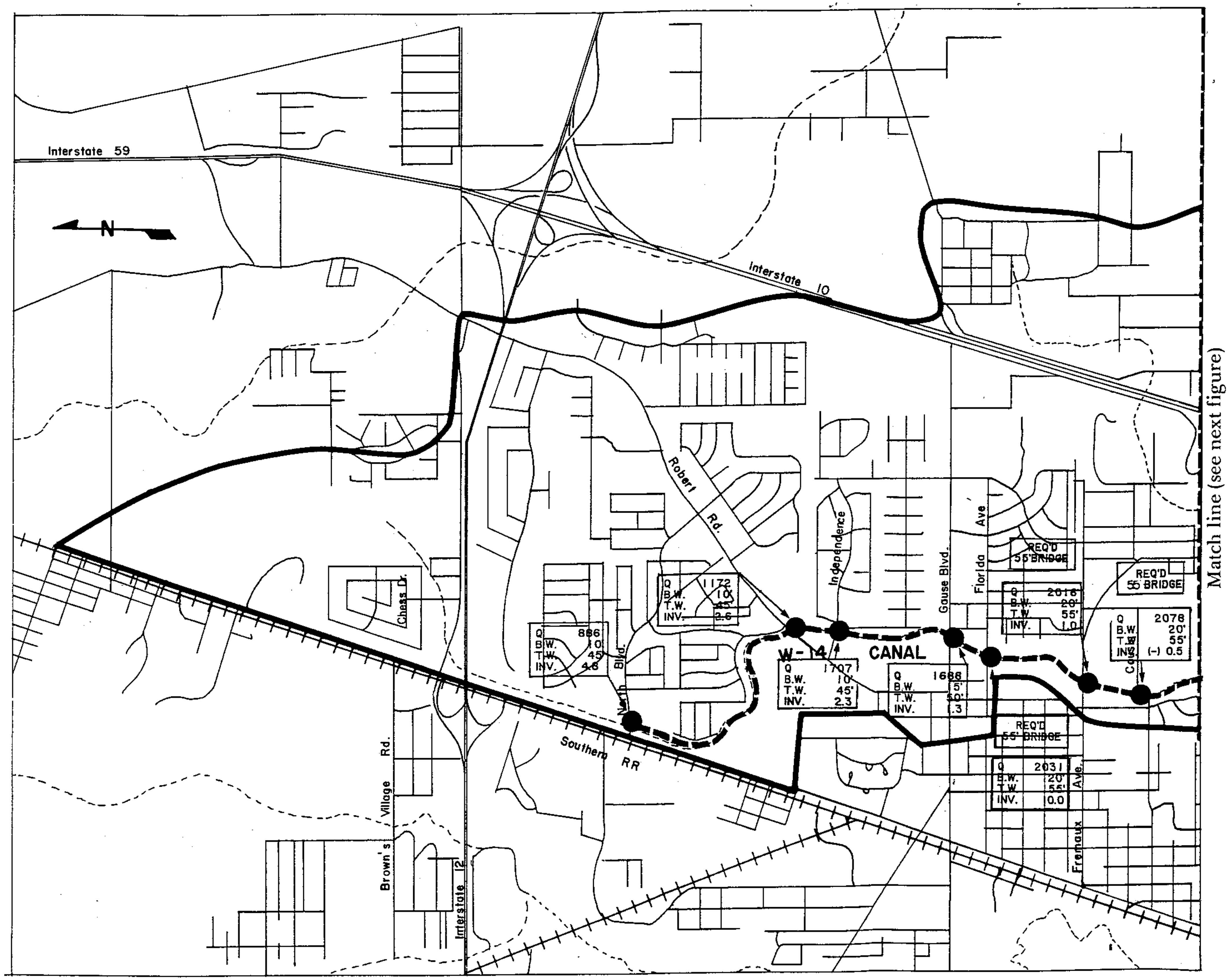
Slidell Master Drainage Plan
Task Order no. 9 Addendum

W-14 CANAL IMPROVEMENTS

Figure 3. Design Canal Layout

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
Oct-94

Scale as noted



Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Concrete Slope Paving

Earthen Section

Burk & Associates
Incorporated

1000 0 1000 2000 3000 FEET

96m

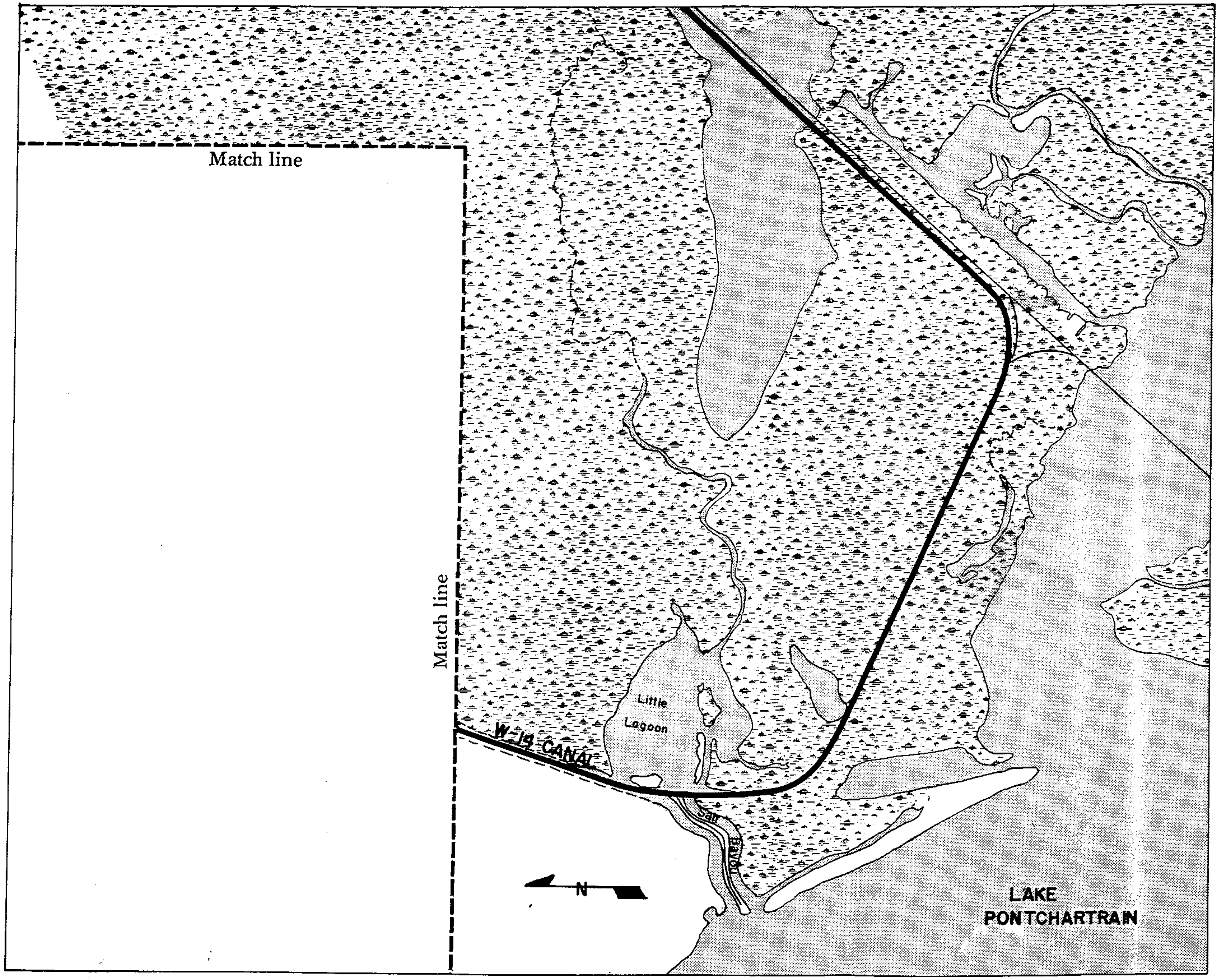
Slidell Master Drainage Plan
Task Order no. 9 Addendum

W-14 CANAL IMPROVEMENTS

Figure 3. Design Canal Layout

Prepared by
Burk-Kleinpeter, Inc.
Hartman Engineering, Inc.
Oct-94

Scale as noted



Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

INV. Proposed Invert Elevation (m.s.l.)

Design Points

Earthen Section

Burk & Associates Incorporated

1000 0 1000 2000 3000 FEET

960

APPENDIX A

HYDROLOGIC COMPUTATIONS

Design Methods

3

General

The foundation of any drainage improvement study is the estimation of the amount of runoff caused by a given rainfall event. Once the amount of runoff has been established this flow must be routed through the stormwater drainage system to establish the flow regime and the water surface profiles. For this study, two widely used techniques developed by the Soil Conservation Service (SCS) and the U.S. Army Corps of Engineers' Hydrologic Engineering Center were utilized to accomplish this. These techniques are outlined in this section.

Design Storm

The first step in the runoff evaluation process is to determine how much precipitation actually falls for a given duration. The relationship between rainfall intensity, duration and return periods vary according to the location and climate of the study area. Ideally, hydrologic studies to determine volume and ratio of runoff should be based on long term stationary streamflow records for the area being analyzed. Such records are not available for every locality, therefore generalized relationships derived statistically from areas where long term data is available are used. Such information is made available by the United States

Weather Bureau in Technical Paper No. 40 entitled "Rainfall Frequency Atlas of the United States".

The choice of a design storm depends upon three major factors: the intensity of a storm, its return frequency, and its duration. The 10 year return frequency design storm is widely used as an acceptable standard for stormwater drainage systems. The 10 year design storm is the standard used for urban drainage system design by the Louisiana Department of Transportation and Development and the Federal Highway Administration. This storm event has a probability of occurring once every ten years on the average. Equivalently, such a storm has a one in ten probability of occurring in any single one year.

Technical Report No. 40 provides isohyetal maps which give the amount of rainfall to be expected from a storm of a given return frequency and duration. Figure 7 below shows the map for the 10 year-24 hour design storm which was used in this study. For St. Tammany Parish, it was determined that the 10 year-24 hour storm would precipitate 8.7 inches of rainfall. This figure was used throughout this study as the basis for all analysis.

Runoff Computations

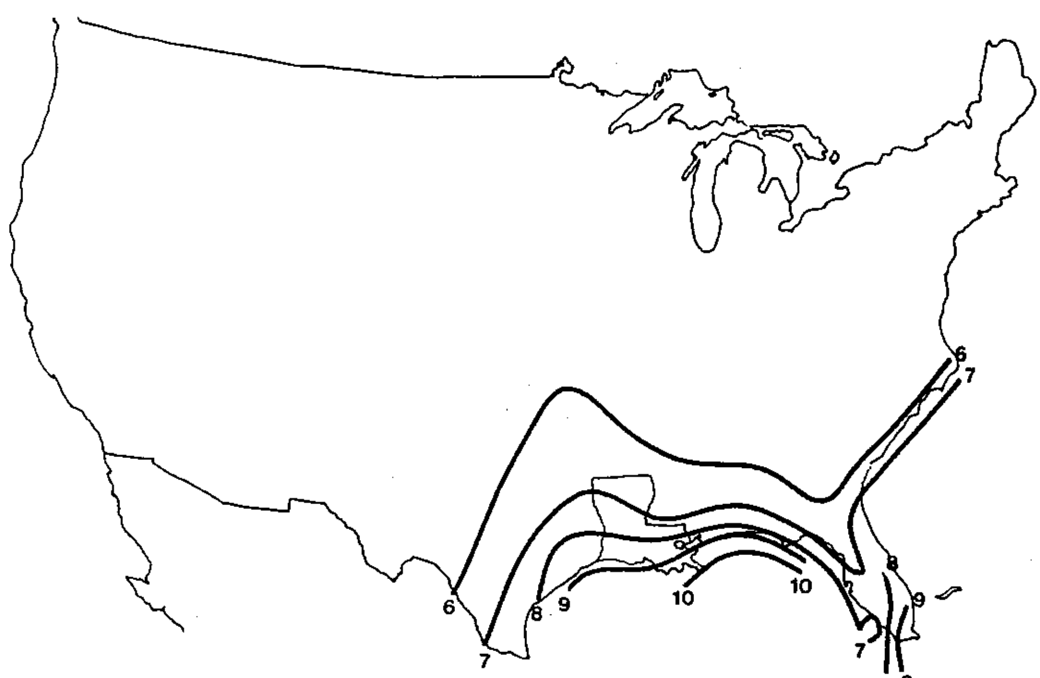
Runoffs for the 10 year-24 hour design storm were computed using the Soil Conservation Service (SCS) Runoff Curve Method. This technique is described in the SCS Handbook entitled, "National Engineering Handbook, Section 4, Hydrology" (NEH-4).

The SCS method of estimating direct runoff from storm rainfall is based on methods developed by SCS hydrologists in the last three decades. The method was made to be usable with rainfall and watershed data that are ordinarily available or easily obtainable for ungauged watersheds (ones not gauged for runoff).

In this method of runoff estimation, the effects

Figure 7
Isohyetal Map
10-Year 24-Hour Rainfall (Inches)

Source: Weather Bureau Technical Paper No. 40



of surface conditions of a watershed are evaluated by means of land use and treatment classes. Over 4000 soils have been classified into four hydrologic soil groups according to their infiltration and transmission rates. The hydrologic soil group of a watershed is used with a description of the prevailing surface culture and vegetative cover to determine a runoff curve number (CN) for the watershed.

The rainfall-runoff relation used in the SCS method of estimating direct runoff from storm rainfall is:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

Where Q = actual runoff
P = potential maximum runoff
$S = S' + I_a$
S' = potential maximum retention
I_a = Initial abstraction (interception, infiltration and surface storage occurring before runoff begins)

Graphs have been developed for the rapid solution of this equation. The parameter CN is a transformation of S, and it is used to make interpolating, averaging and weighting operations more nearly linear. The transformation is:

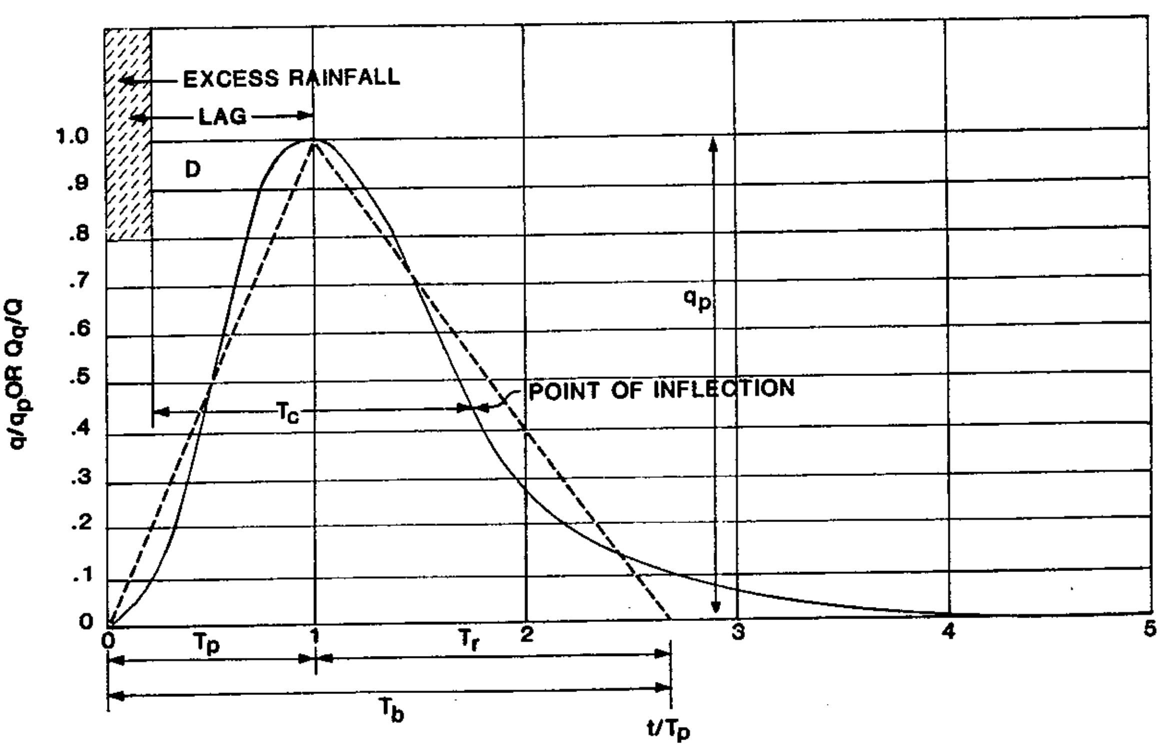
$$CN = \frac{1000}{S + 10}$$

or

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

Hydrographs for the SCS method are based on a dimensionless unit hydrograph. This hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations. This dimensionless curvilinear hydrograph, shown in Figure 8, has its ordinate values expressed in a dimensionless ratio q/q_p or Qa/Q and its abscissa values as t/T.

Figure 8
Dimensionless
Unit Hydrograph



This unit hydrograph has a point of inflection approximately 1.70 times the time-to-peak (T_p) and the time- to-peak 0.2 of the time-of-base (T_b).

The dimensionless curvilinear unit hydrograph has 37.5% of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. This dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle.

$$q_p = \frac{484\ A\ Q}{\frac{D}{2} + 0.6\ Tc}$$

Where q_p = peak discharge
A = drainage area in square miles
Q = total volume of discharge in inches
D = duration of unit excess rainfall
Tc = time of concentration in hours

A lengthy derivation of this equation can be found in the National Engineering Handbook. A computer program, TR-20, has been developed by SCS to compute the surface runoff and route the flow through channels. TR-20 provides for the continuous analysis of nine different storms over a watershed under present conditions and with various combinations of land treatment, floodwater- retarding structures and channel improvements. It can develop and route the runoff from these nine different storm distributions considering an unlimited number of depths and durations for any storm distribution defined in dimensionless units.

Water Surface Profiles

Once runoff quantities have been computed, the drainage channels must be designed to carry this flow with a water surface elevation which will not cause flooding.

DESIGN POINT	Drainage Area (sq. mi.)	DESIGN STORM			
		5 yr (cfs)	10 yr (cfs)	25 yr (cfs)	100 yr (cfs)
North Blvd.	1.77	476	886	1053	1408
Robert Rd.	2.32	918	1172	2022	2692
Independ. Dr.	3.15	917	1688	1999	2658
Gause Blvd.	3.64	1098	2016	2385	3158
Florida Ave.	3.72	1108	2030	2401	3175
Fremaux Ave.	3.83	1122	2054	2427	3201
Cousin St.	3.93	1136	2078	2451	3235
Daney St.	4.04	1150	2102	2481	3266
I-10	6.21	1321	3420	2853	3747
Kingspt. Blvd.	6.76	1870	3686	4036	5306
Voters Rd.	7.20	2131	3925	4637	6101
LA 433	8.61	1998	3780	4498	6003

Notes: 1.) Curve Number (CN) = 86
2.) Assume soil is 75% Type D and 25% Type C.

APPENDIX B

HYDRAULIC ANALYSIS

This unit hydrograph has a point of inflection approximately 1.70 times the time-to-peak (T_p) and the time- to-peak 0.2 of the time-of-base (T_b).

The dimensionless curvilinear unit hydrograph has 37.5% of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. This dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle.

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Water Surface Profiles

Once runoff quantities have been computed, the drainage channels must be designed to carry this flow with a water surface elevation which will not cause flooding.

Water surface profiles along the major drainage conduits within the Phase I study area boundaries were determined using the HEC-2 computer program developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center. The program calculates water surface profiles for gradually varied flow in natural or manmade channels. The effects of obstructions in the channel such as bridges, culverts and other structures may also be considered. The program has the additional capability of assessing the effects of channel improvements and levees on the water surface elevations.

The computational procedure used in HEC-2 is known as the Standard Step Method. This methodology is based on the solution of energy equations in one dimension with frictional losses calculated using Mannings' Equation for Open Channel Flow. In natural channels the hydraulic characteristics are not constant therefore it is generally necessary to conduct field investigations to determine the necessary data at all sections. The computations are then carried on from station to station where the hydraulic element has been determined.

In the Standard Step methodology, it is convenient to reference the water surface elevations to a horizontal datum [in this case all elevations are mean sea level (msl)]. Thus the two end section water surface elevations above the horizontal datum are (See Figure 9)

$$Z_1 = S_o x + y_1 + Z_2 \qquad (1)$$

and

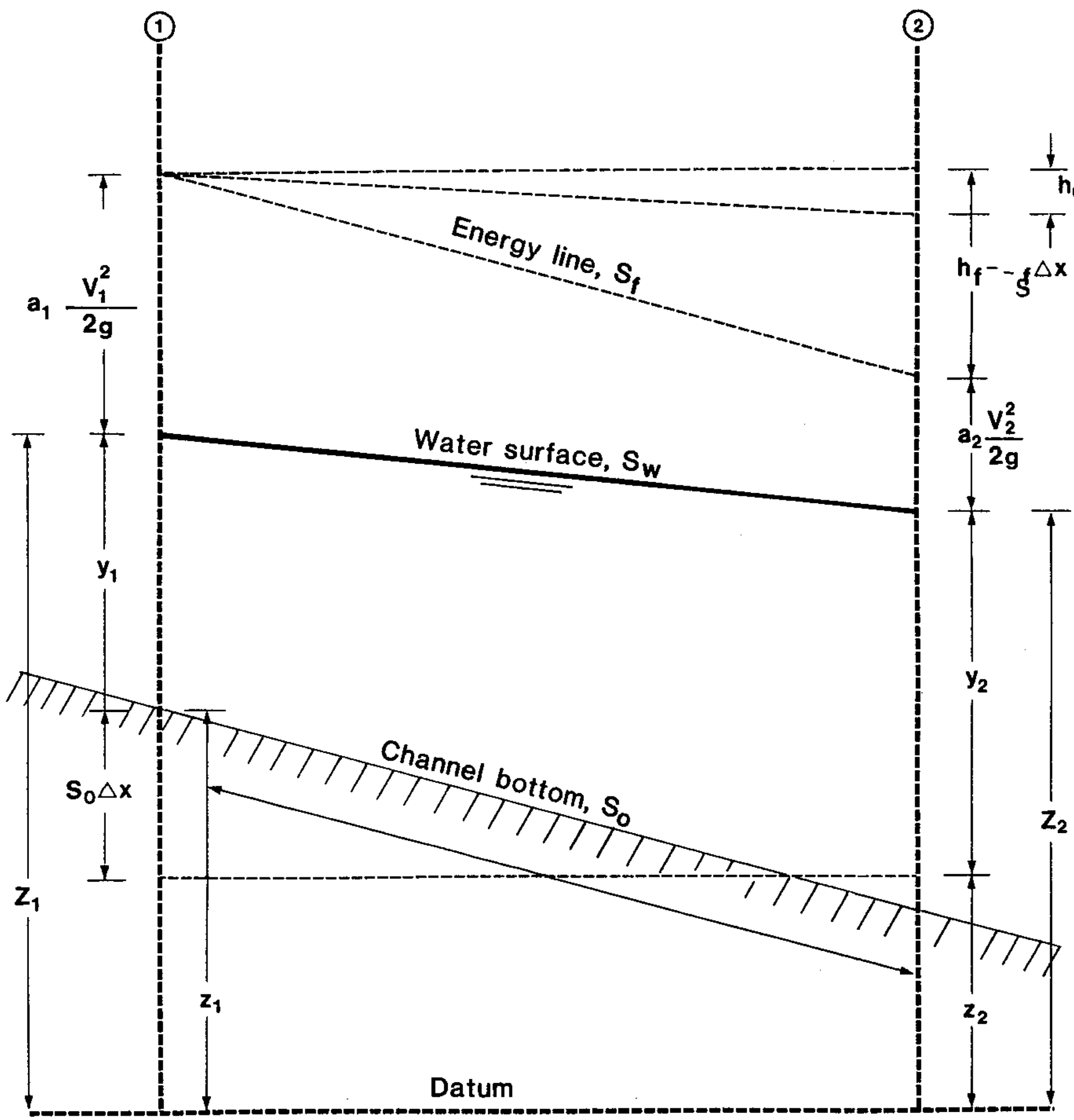
$$Z_2 = Y_2 + Z_2 \qquad (2)$$

The friction loss is

$$h_f = S_f x = 1/2(S_1 + S_2) x \qquad (3)$$

where the friction slope S_f is taken as the average of the slopes at the end sections.

Figure 9
Water Surface Elevation Determination
Standard Step Method: Water Surface Profiles



Equating the total head at the two end sections the following equation may be written:

$$S_o\, x + y_1 + a_1 \frac{V_1^2}{2g} = y_2 + a_2 \frac{V_2^2}{2g} + S_f\, x \quad (4)$$

S_f
Substituting, Eq. (4) becomes

$$Z_1 + a_1 \frac{V_1^2}{2g} = Z_2 = a_2 \frac{V_2^2}{2g} + h_f + h_2 \quad (5)$$

The total heads at the end section are

$$H_1 = Z_1 + a_1 \frac{V_1^2}{2g} \quad (6)$$

$$H_2 = Z_2 + a_2 \frac{V_2}{2g} \quad (7)$$

Thus Eq. (5) can be simplified to

$$H_1 = H_2 + h_f + h_e \quad (8)$$

This is the basic equation used in the Standard Step Method. Beginning with a known water surface elevation and distance between section, equation (8) can be solved for the total head at the new section.

Design of Drainage Structures

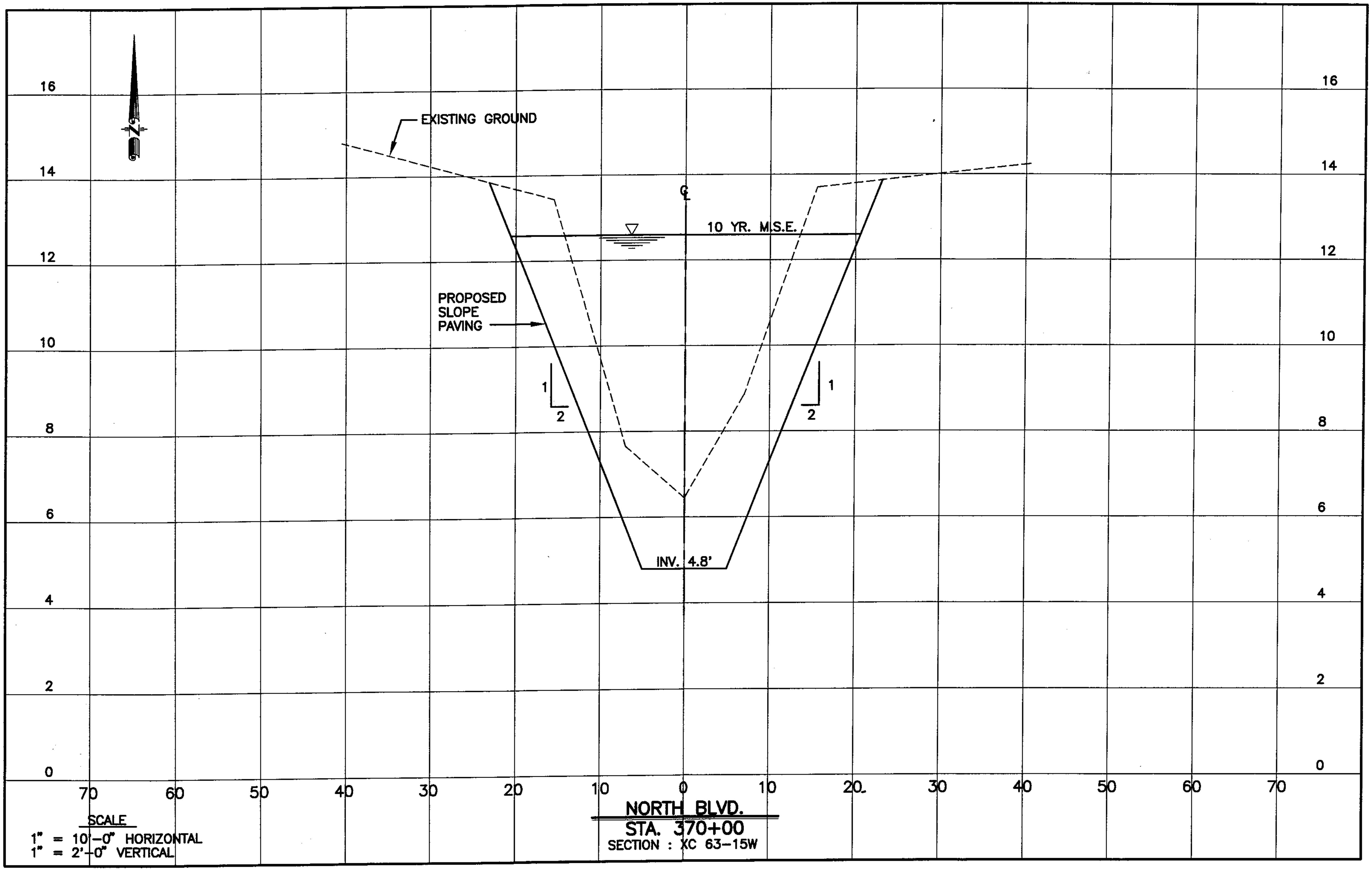
The drainage structures to be used in the Phase I area were designed to conform with proposed water surface elevations while adequately handling anticipated flows. Consequently, a trial and error process was required for this determination.

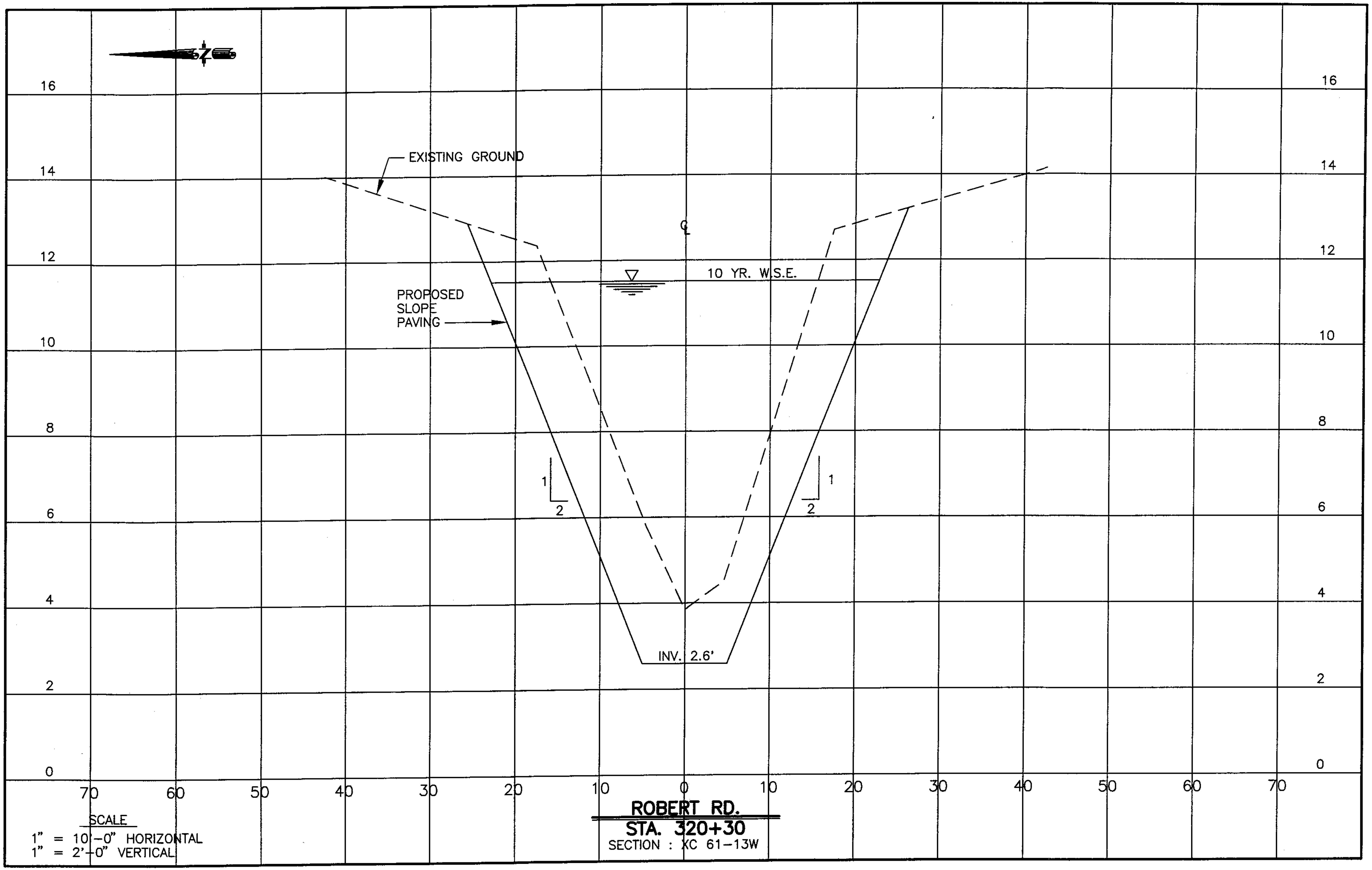
Both concrete slope paved sections and earthen sections are proposed. Allowable velocities differ for each type of section. Naturally, velocities in earthen sections should remain low to prevent erosion and velocities in concrete sections should be high enough to prevent sediment build up.

The following guidelines were established in this

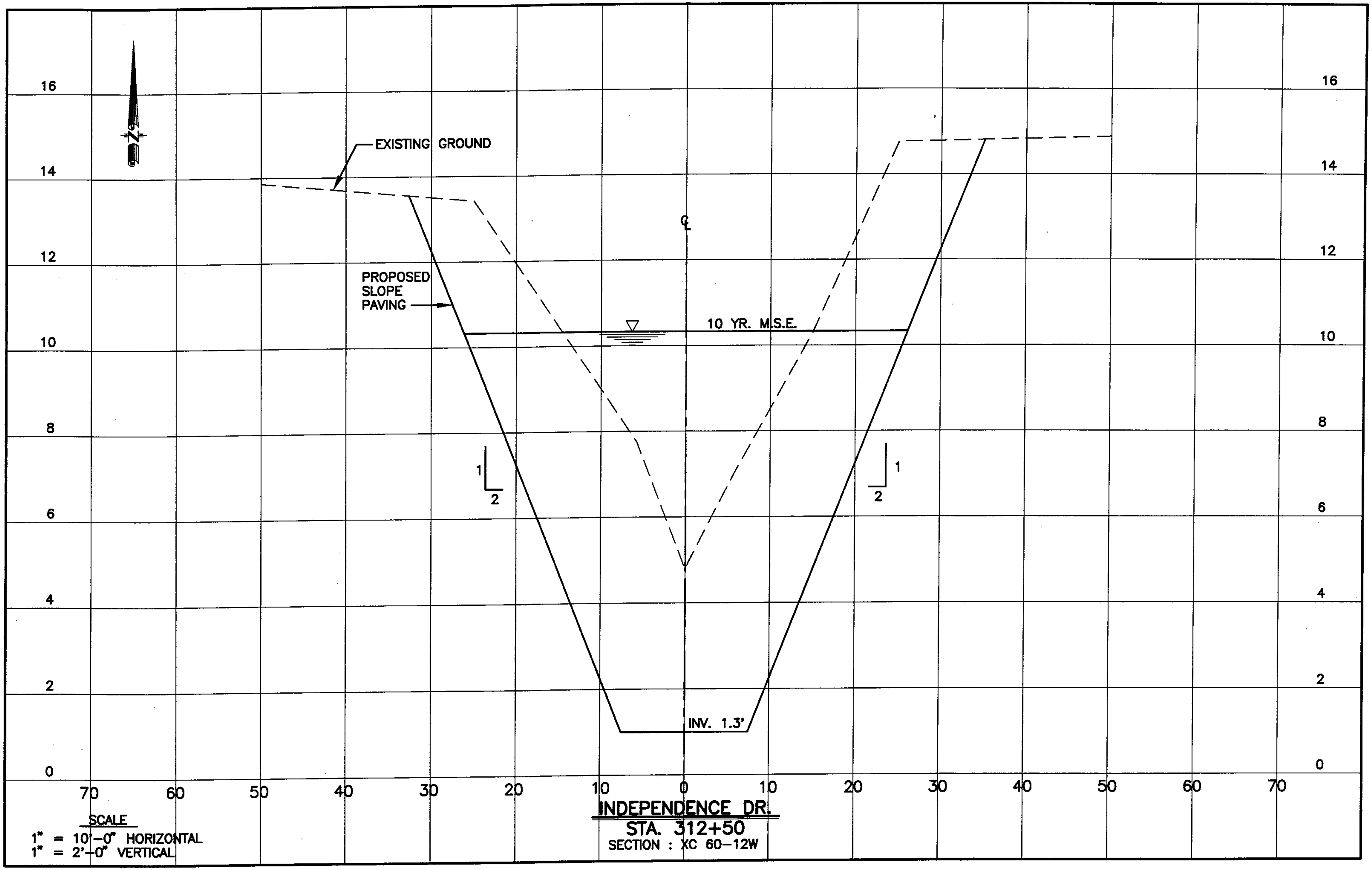
APPENDIX C

CROSS-SECTIONS

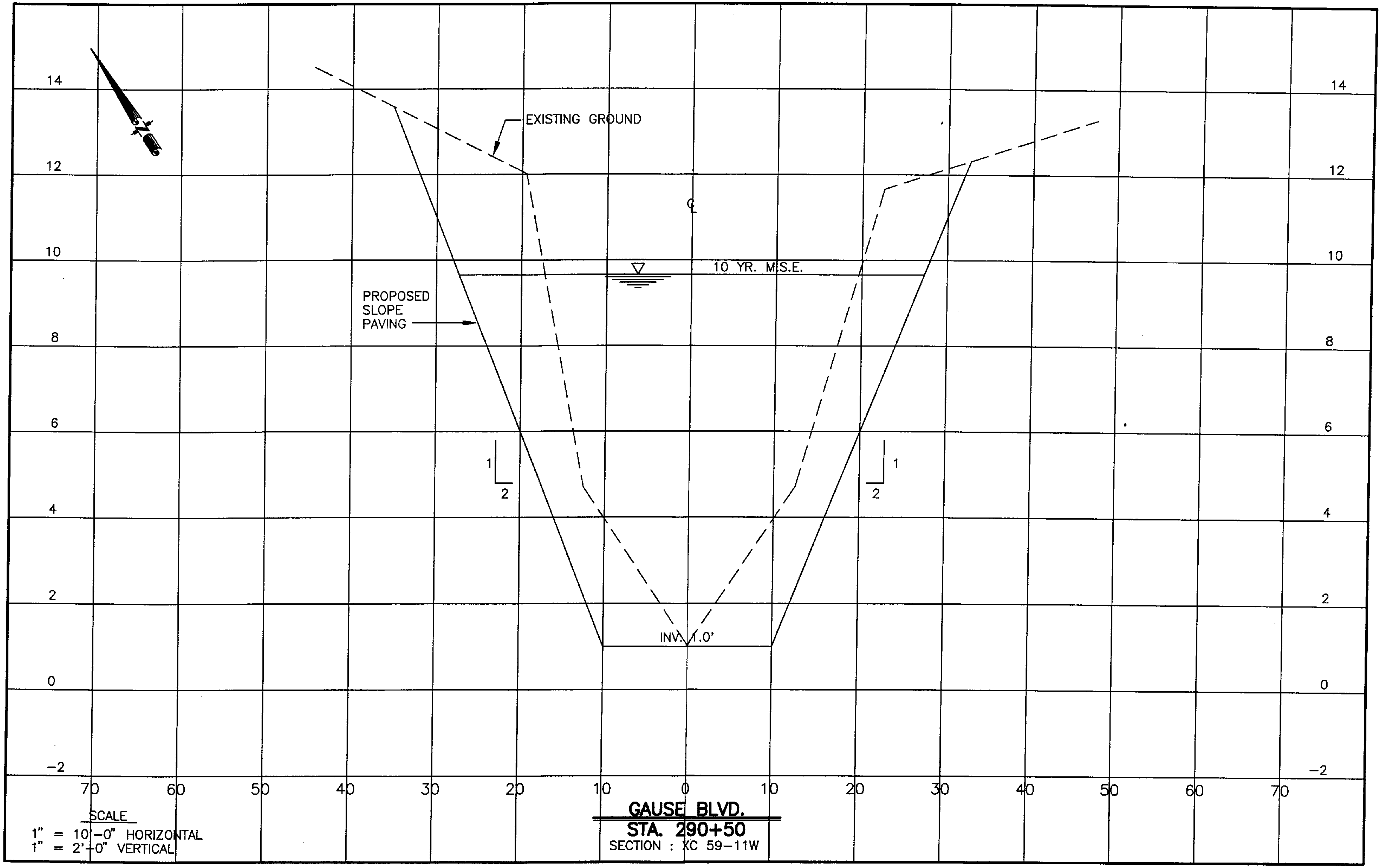

EXISTING GROUND
10 YR. M.S.E.
PROPOSED SLOPE PAVING
1
2
1
2
INV. 4.8'
16
14
12
10
8
6
4
2
0
70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
NORTH BLVD.
STA. 370+00
SECTION : XC 63-15W



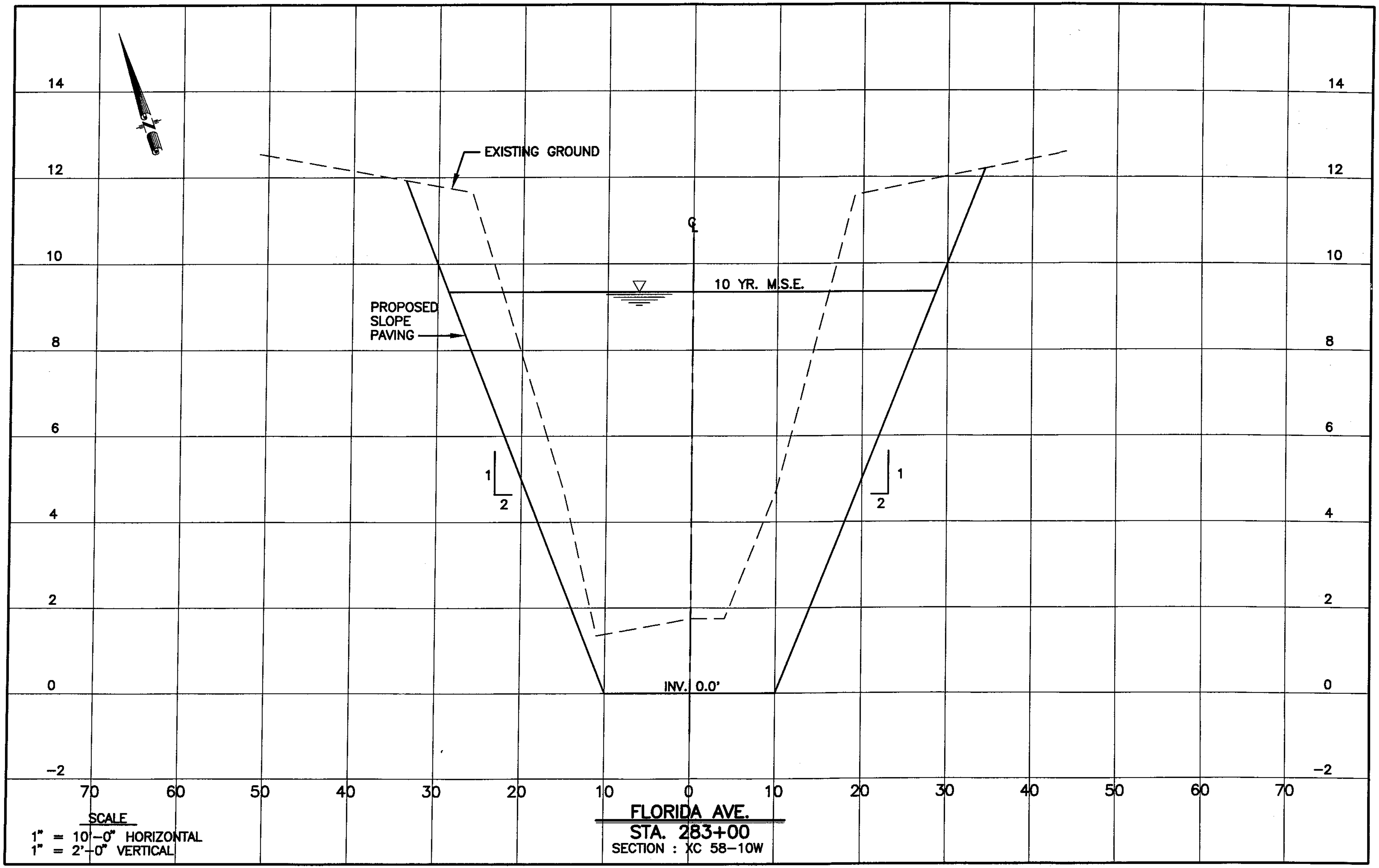
16
14
12
10
8
6
4
2
0
EXISTING GROUND
10 YR. W.S.E.
PROPOSED
SLOPE
PAVING
1
2
INV. 2.6'
70
60
50
40
30
20
10
0
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
ROBERT RD.
STA. 320+30
SECTION : XC 61-13W



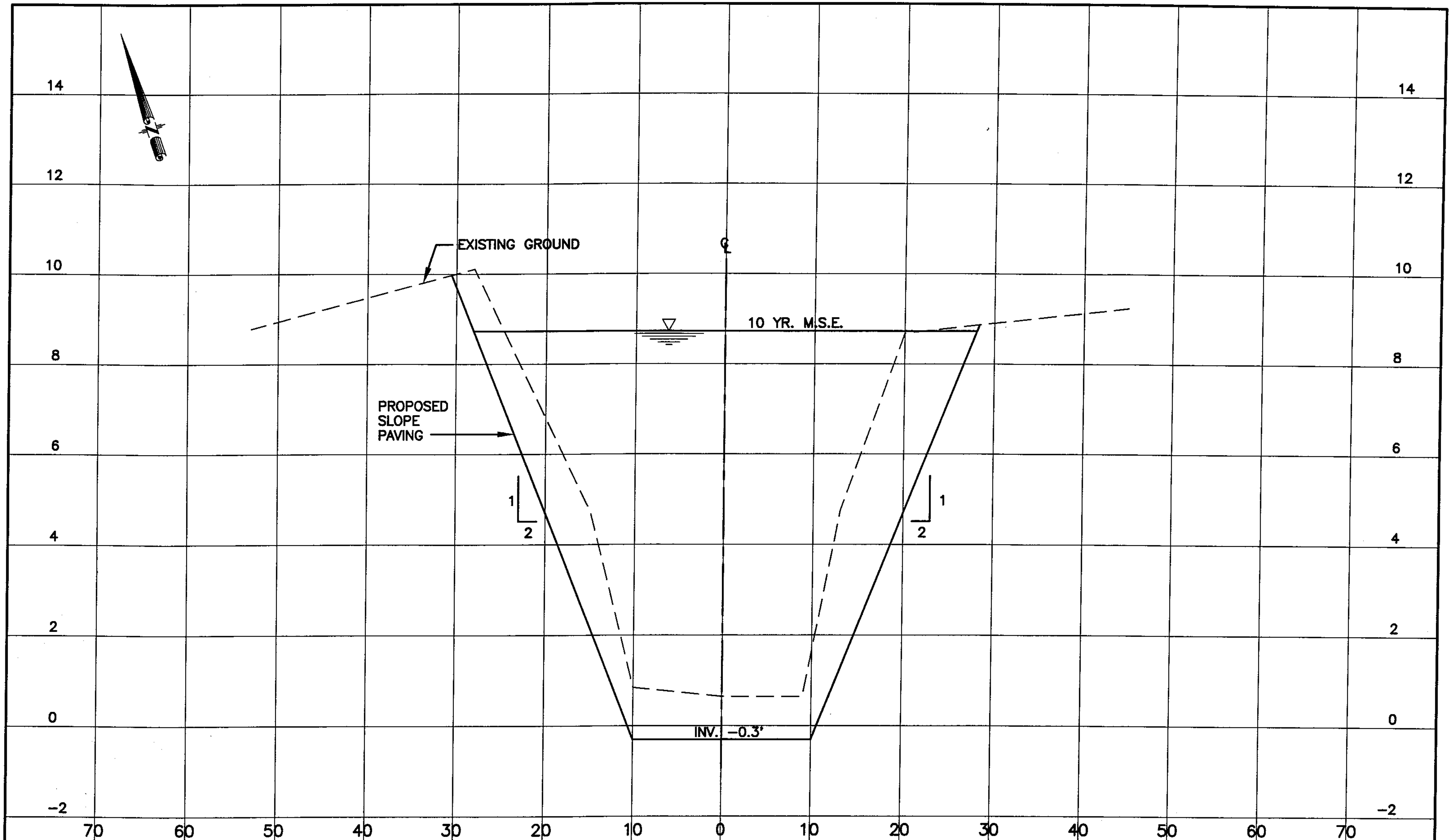
INDEPENDENCE DR.
STA. 312+50
SECTION : XC 60-12W



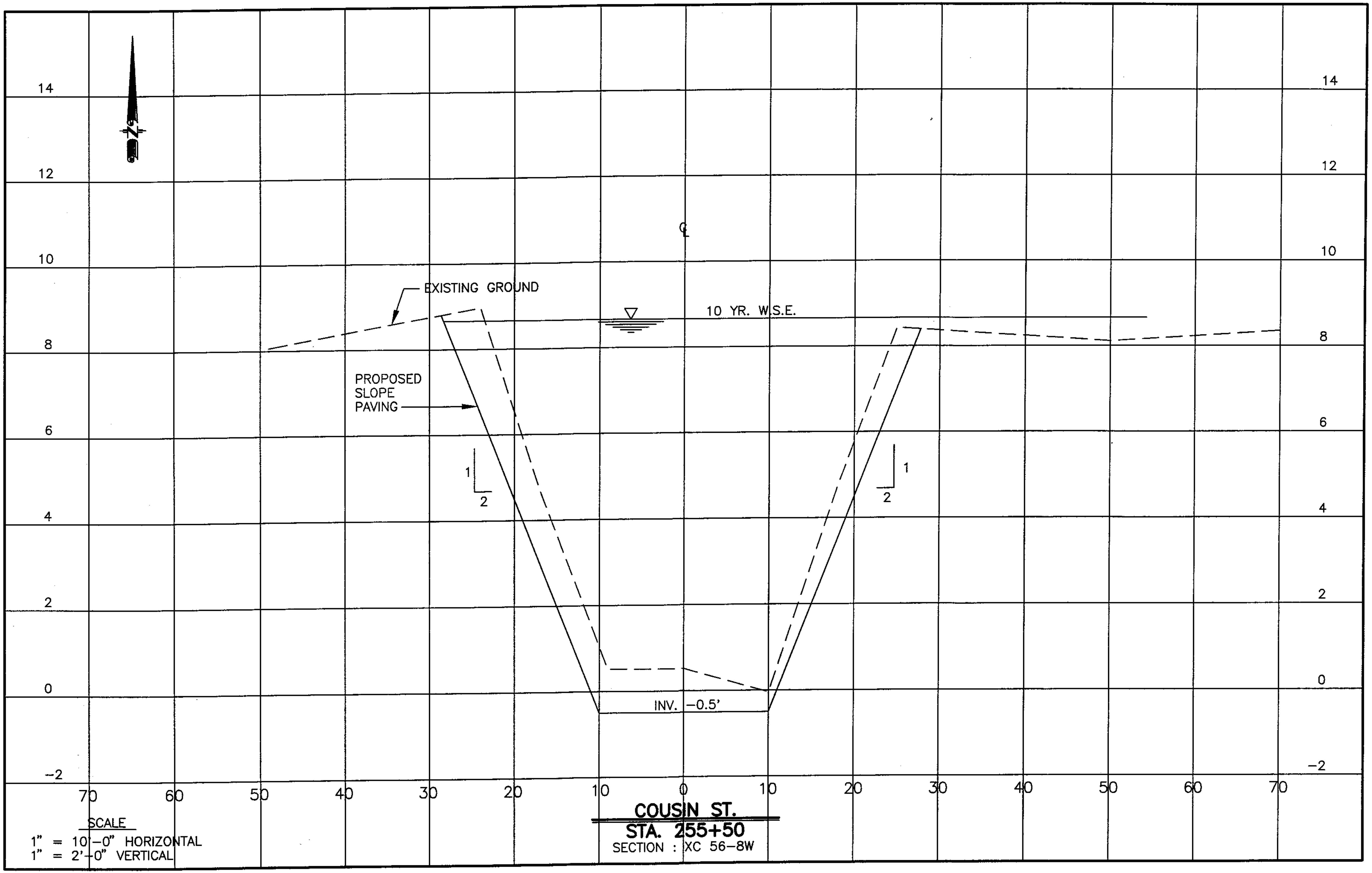
EXISTING GROUND
PROPOSED SLOPE PAVING
10 YR. M.S.E.
INV. 1.0'
1
2
14
12
10
8
6
4
2
0
-2
70
60
50
40
30
20
10
0
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
GAUSE BLVD.
STA. 290+50
SECTION : XC 59-11W



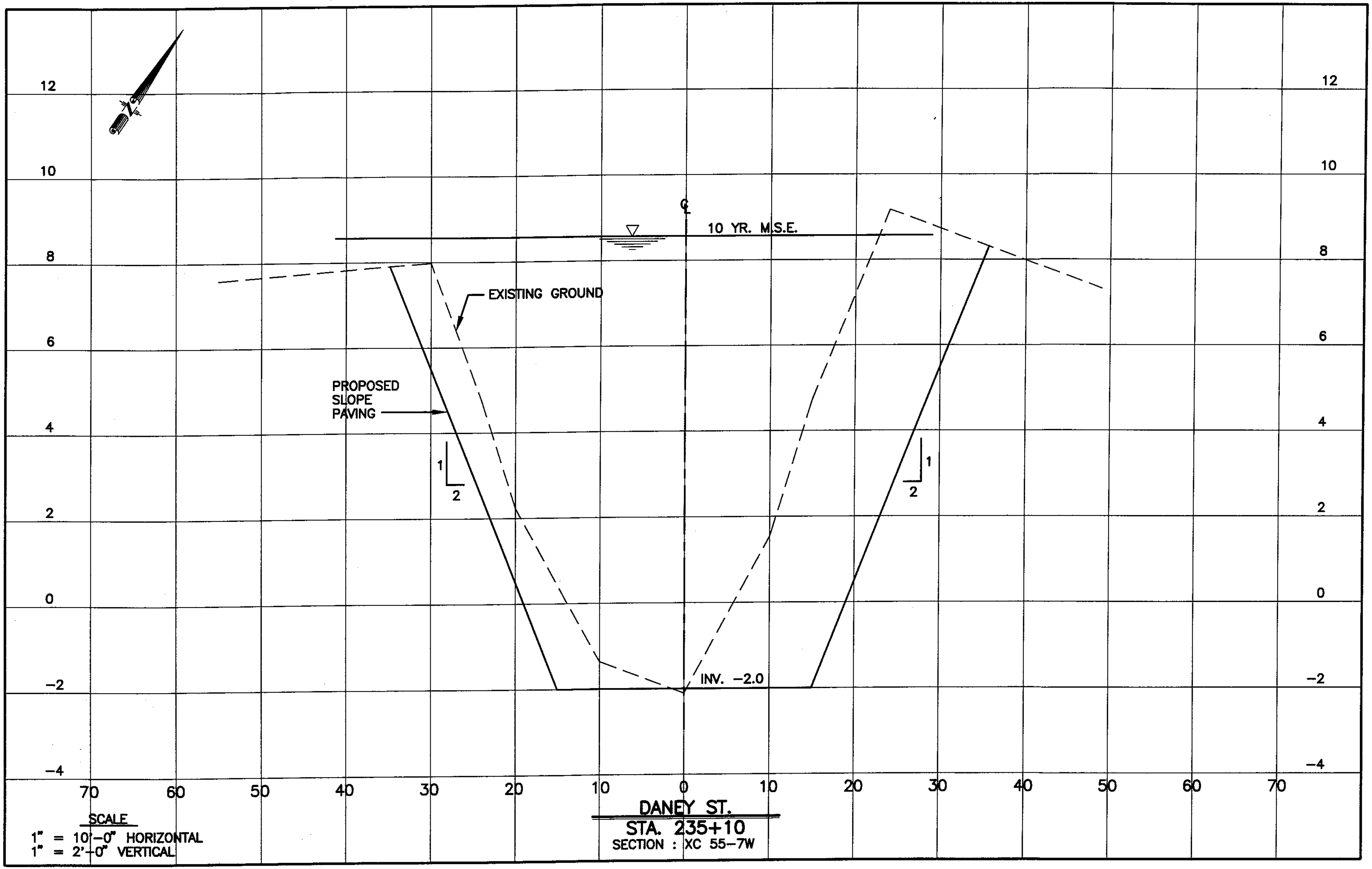
EXISTING GROUND
10 YR. M.S.E.
PROPOSED SLOPE PAVING
INV. 0.0'
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
FLORIDA AVE.
STA. 283+00
SECTION : XC 58-10W



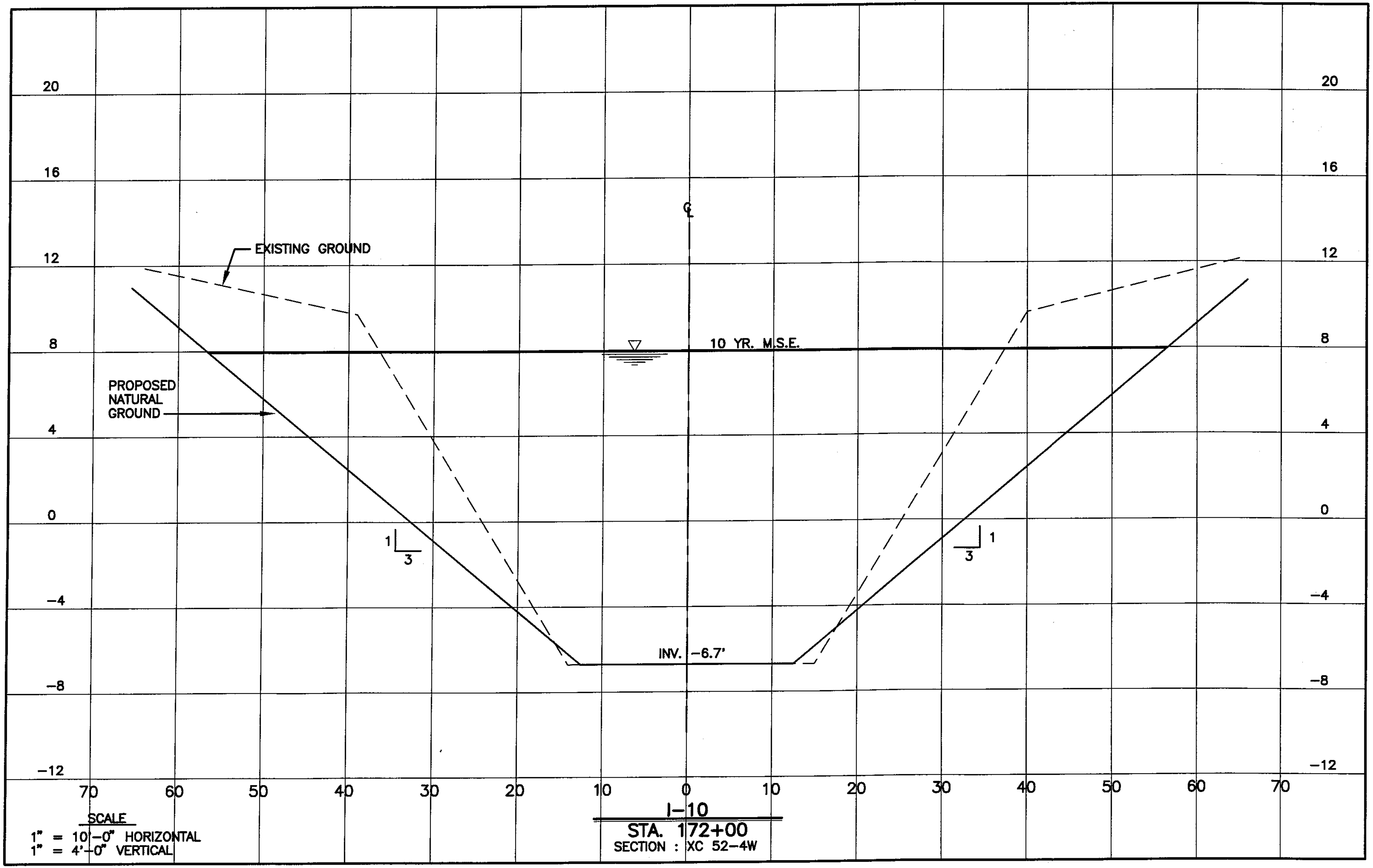
FREMAUX AVE.
STA. 263+00
SECTION : XC 57−9W



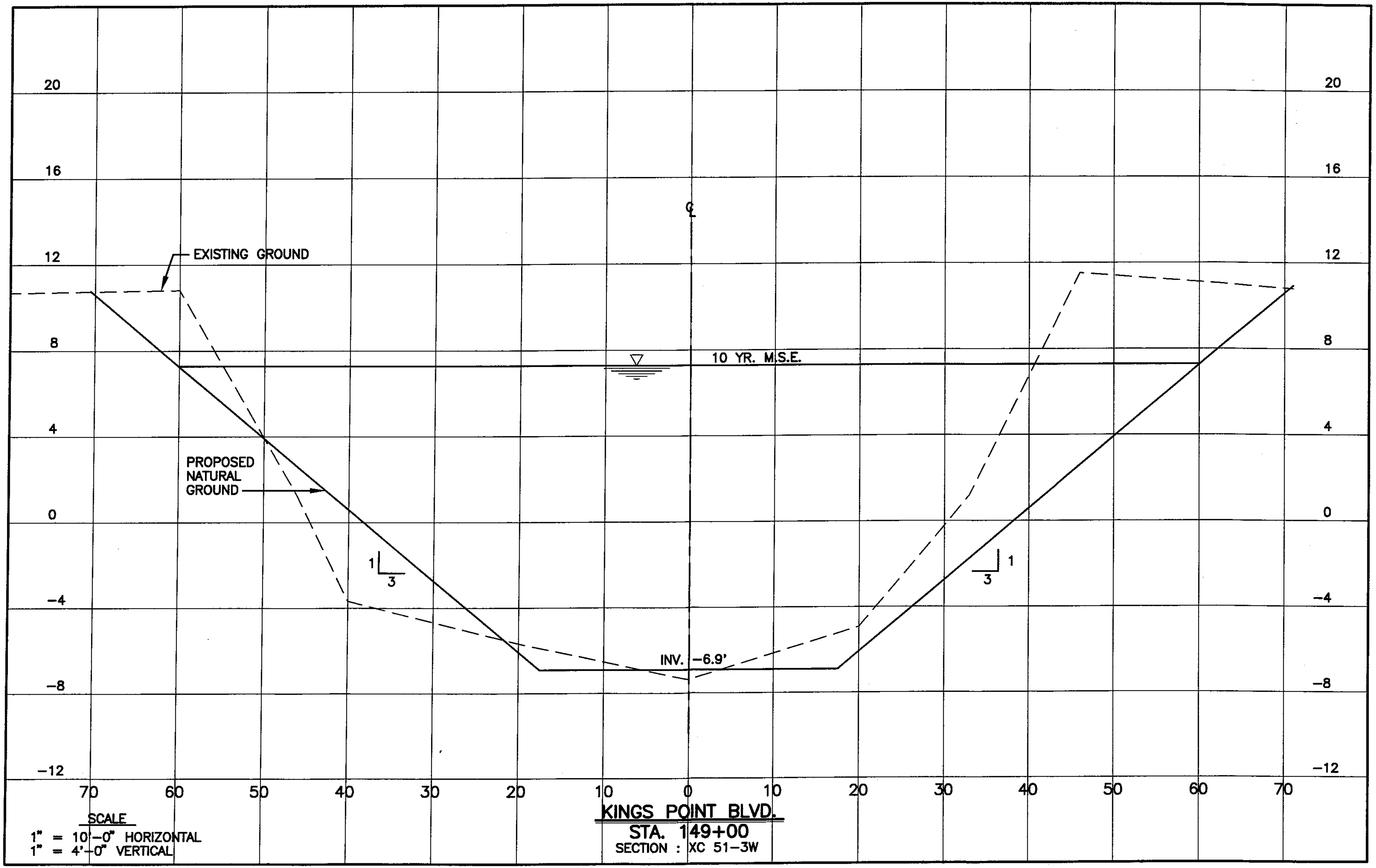
N
14
12
10
8
6
4
2
0
−2
EXISTING GROUND
10 YR. W.S.E.
PROPOSED
SLOPE
PAVING
1
2
INV. −0.5'
70
60
50
40
30
20
10
0
SCALE
1" = 10'−0" HORIZONTAL
1" = 2'−0" VERTICAL
COUSIN ST.
STA. 255+50
SECTION : XC 56−8W



N
12
10
8
6
4
2
0
-2
-4
℄
10 YR. M.S.E.
EXISTING GROUND
PROPOSED
SLOPE
PAVING
1
2
INV. -2.0
70
60
50
40
30
20
10
0
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
DANEY ST.
STA. 235+10
SECTION : XC 55-7W

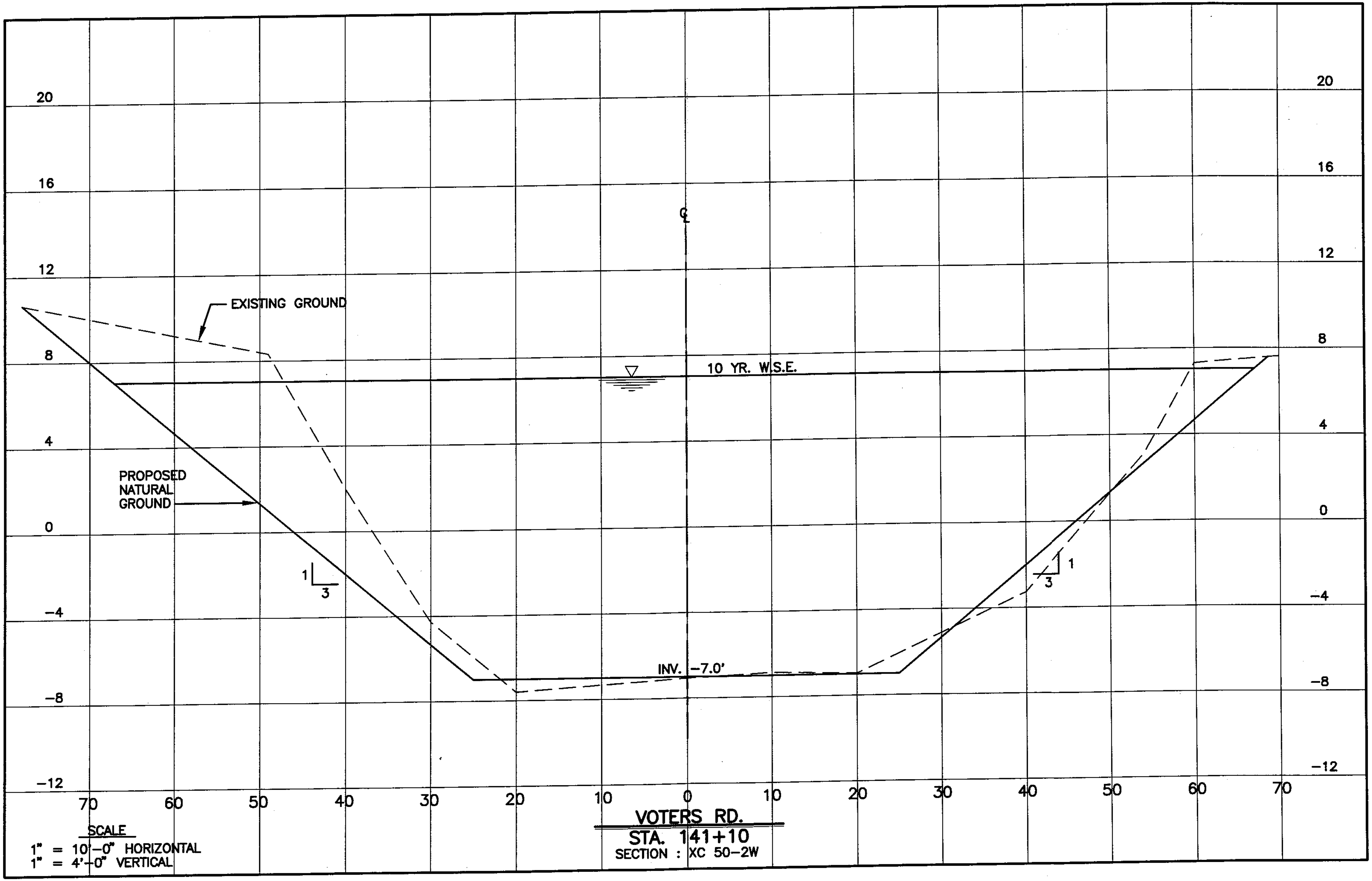


EXISTING GROUND
10 YR. M.S.E.
PROPOSED NATURAL GROUND
1 3
3 1
INV. -6.7'
20
16
12
8
4
0
-4
-8
-12
70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
SCALE
1" = 10'-0" HORIZONTAL
1" = 4'-0" VERTICAL
I-10
STA. 172+00
SECTION : XC 52-4W

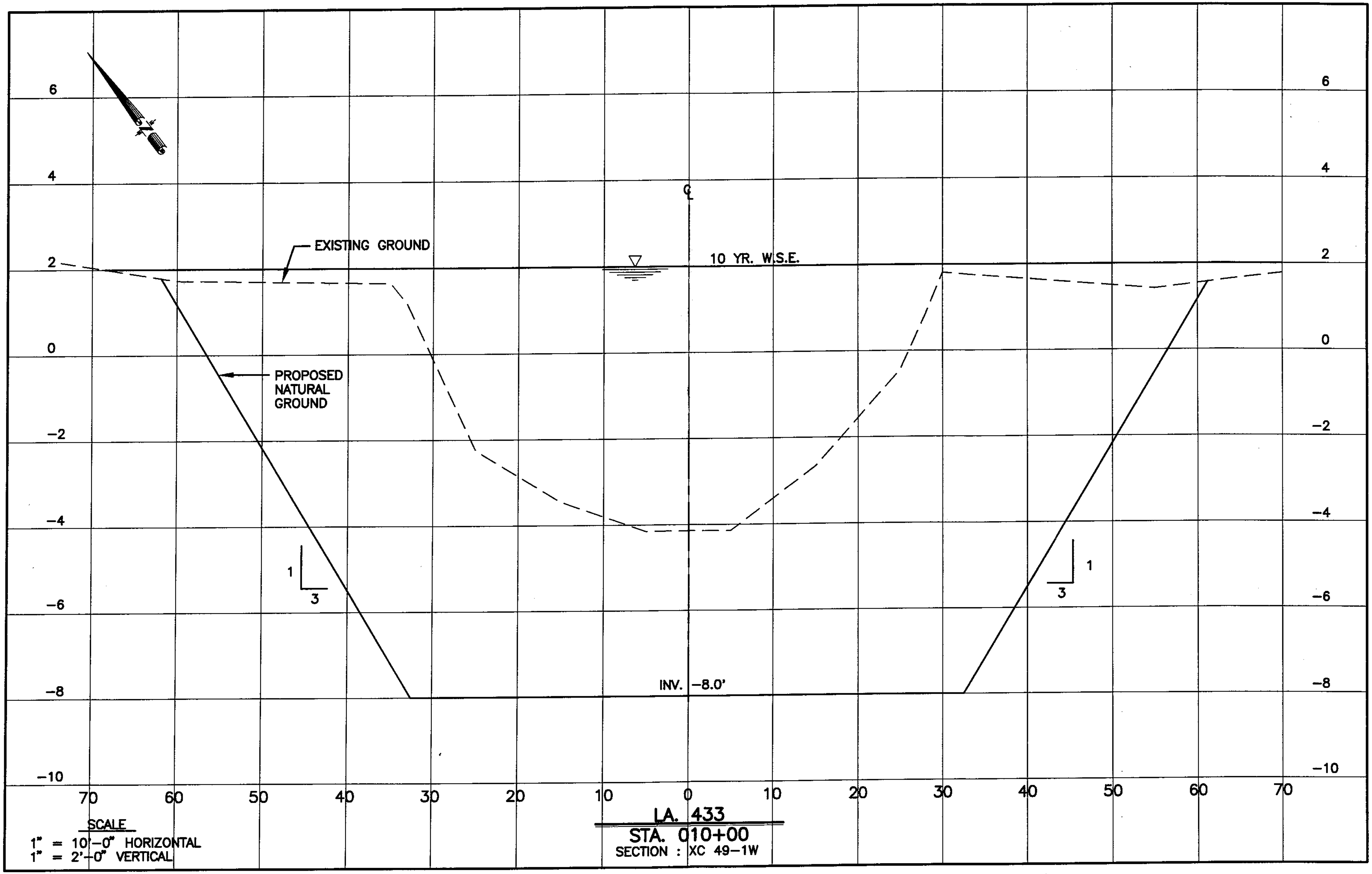


SCALE
1" = 10'−0" HORIZONTAL
1" = 4'−0" VERTICAL

KINGS POINT BLVD.
STA. 149+00
SECTION : XC 51−3W



EXISTING GROUND
10 YR. W.S.E.
PROPOSED NATURAL GROUND
INV. -7.0'
20
16
12
8
4
0
-4
-8
-12
70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
SCALE
1" = 10'-0" HORIZONTAL
1" = 4'-0" VERTICAL
VOTERS RD.
STA. 141+10
SECTION : XC 50-2W



6
4
2
0
-2
-4
-6
-8
-10
EXISTING GROUND
10 YR. W.S.E.
PROPOSED
NATURAL
GROUND
1
3
INV. -8.0'
70
60
50
40
30
20
10
0
10
20
30
40
50
60
70
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
LA. 433
STA. 010+00
SECTION : XC 49-1W

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