CITY OF SLIDELL

MASTER DRAINAGE PLAN

ADDENDUM TO Task Order No. 9

Plan of Action

Prepared by:

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October, 1994

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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

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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.

III. Project Hydraulics

The St. Tammany Parish Master Drainage Plan (Burk & Associates, Inc., 1983) provides 10 Year-24 Hour Design Storm Event flows and water surface elevations computed at various roadway crossings along the W-14 Canal. The design flows are compared with the existing canal capacity at each point. These tabulations are reproduced in Appendix A. Typically, the canal section at each point is inadequate and as a result high-velocity flows are being pushed through the canal. It may be expected that the fast-flowing water is eroding away the natural earthen side slopes of the canal banks, and the slope stability may be in question. Side slope erosion is particularly dangerous in the northern reaches of the W-14, where properties adjacent to the canal may be lost during landslides. Hydraulic analysis (see Appendix B) at each design point shows the required W-14 channel sections capable of handling the 10 Year-24 Hour design flows predicted by the 1983 Master Drainage Plan. The entire length of the W-14 Canal with proposed design values is mapped in Figures 3 through 5.

IV. Proposed Improvements

The first step in improving drainage in the W-14 Canal basin is installing flood prevention measures at the W-14 Canal outfall, as described in Section V of the *Plan of Action*. Recalling this discussion, the first stage of improvements involves levee construction, automatic drainage gate installation, and addition of the first-phase pump station designed to pump 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 timeconsuming 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
	4970									
Robert Rd.		1172	45.0	10.0	2:1	0.013	8.0	2.6	12.36	11.51
	780									
Independ. Dr.		1688	50.0	15.0	2:1	0.013	8.0	1.3	13.42	10.33
	2200									
Gause Blvd.		2016	55.0	20.0	2:1	0.013	8.0	1.0	11.66	9.66
	750									
Florida Ave.		2030	55.0	20.0	2:1	0.013	8.0	0.0	11.58	9.35
	2000									
Fremaux Ave.		2054	55.0	20.0	2:1	0.013	8.0	-0.3	8.68	8.73
	750									
Cousin St.		2078	55.0	20.0	2:1	0.013	8.0	-0.5	8.43	8.66
	2040									
Daney St.		2102	75.0	30.0	2:1	0.013	8.0	-2.0	7.98	8.57
Earth Canal										
	6310									
I-10		3420	110.0	25.0	3:1	0.033	3.0	-6.7	9.67	7.93
	2300									
Kingspt. Blvd.		3686	115.0	35.0	3:1	0.033	3.0	-6.9	10.81	7.28
	790									
Voters Rd.		3925	125.0	50.0	3:1	0.033	3.0	-7.0	7.30	7.03
	13110									
LA 433		4000	125.0	65.0	3:1	0.033	3.0	-8.0	1.65	2.00

TABLE 2

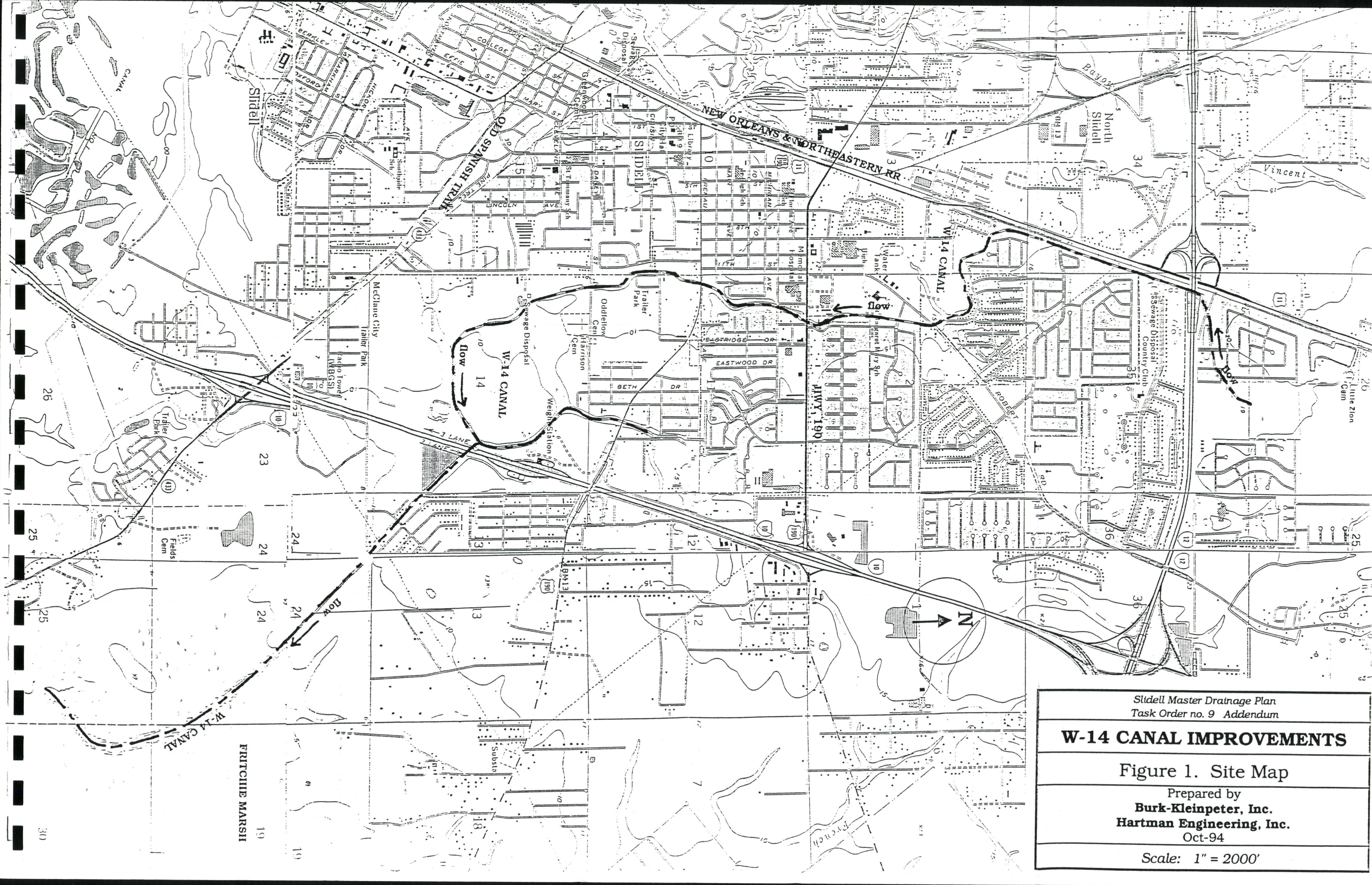
DESIGN PARAMETERS

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



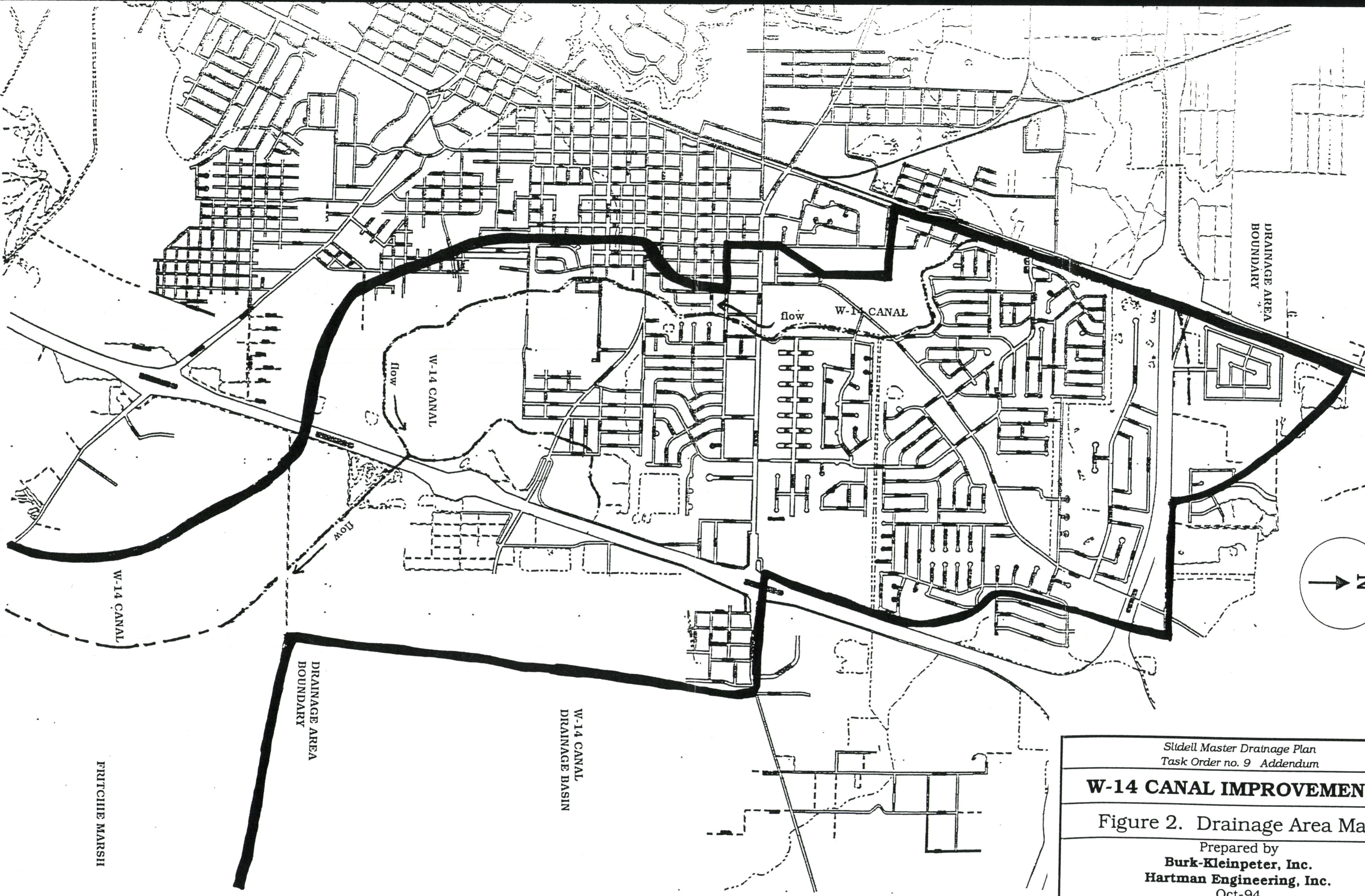
Slidell Master Drainage Plan
Task Order no. 9 Addendum

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'

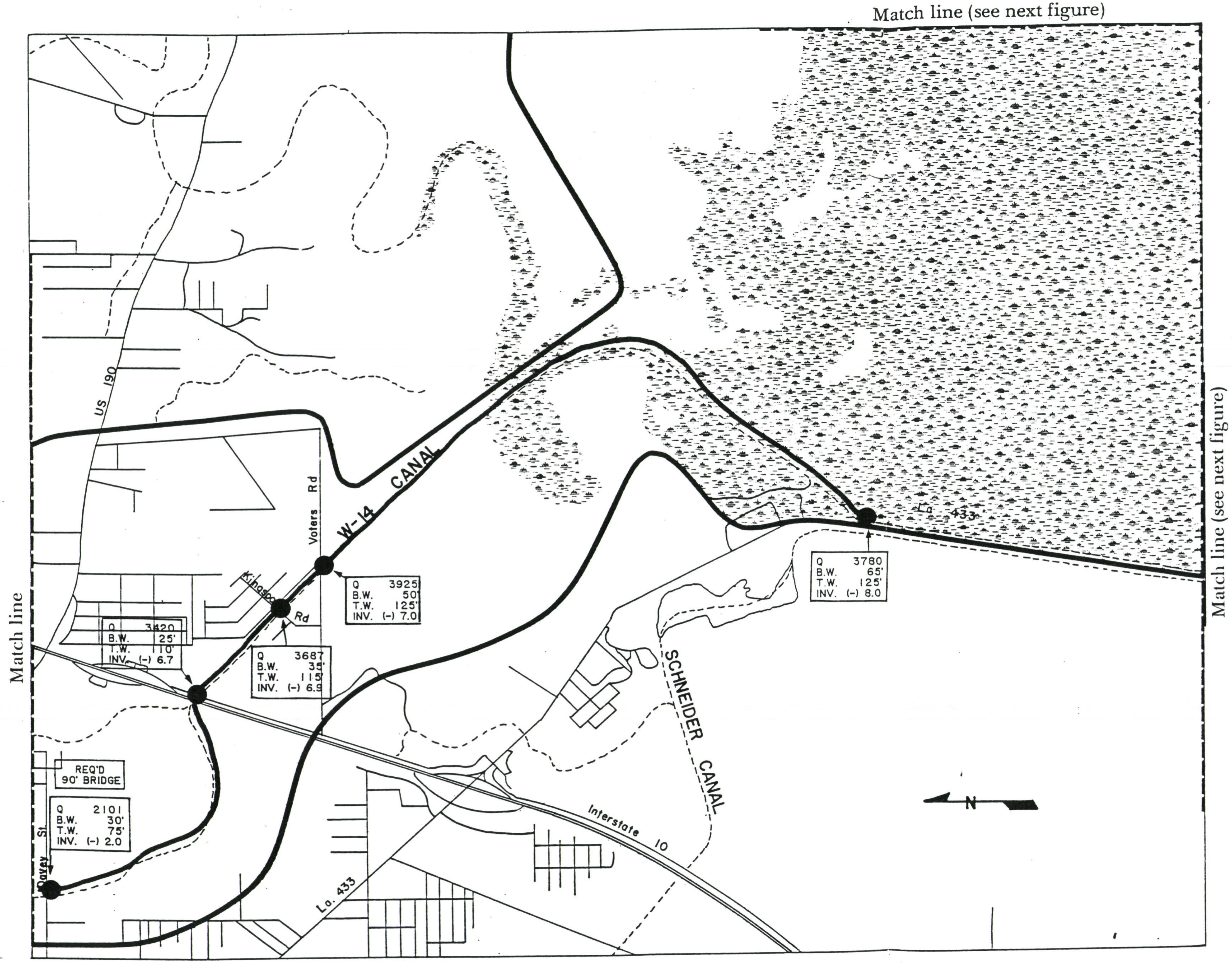
Slidell Master Drainage Plan
Task Order no. 9 Addendum

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

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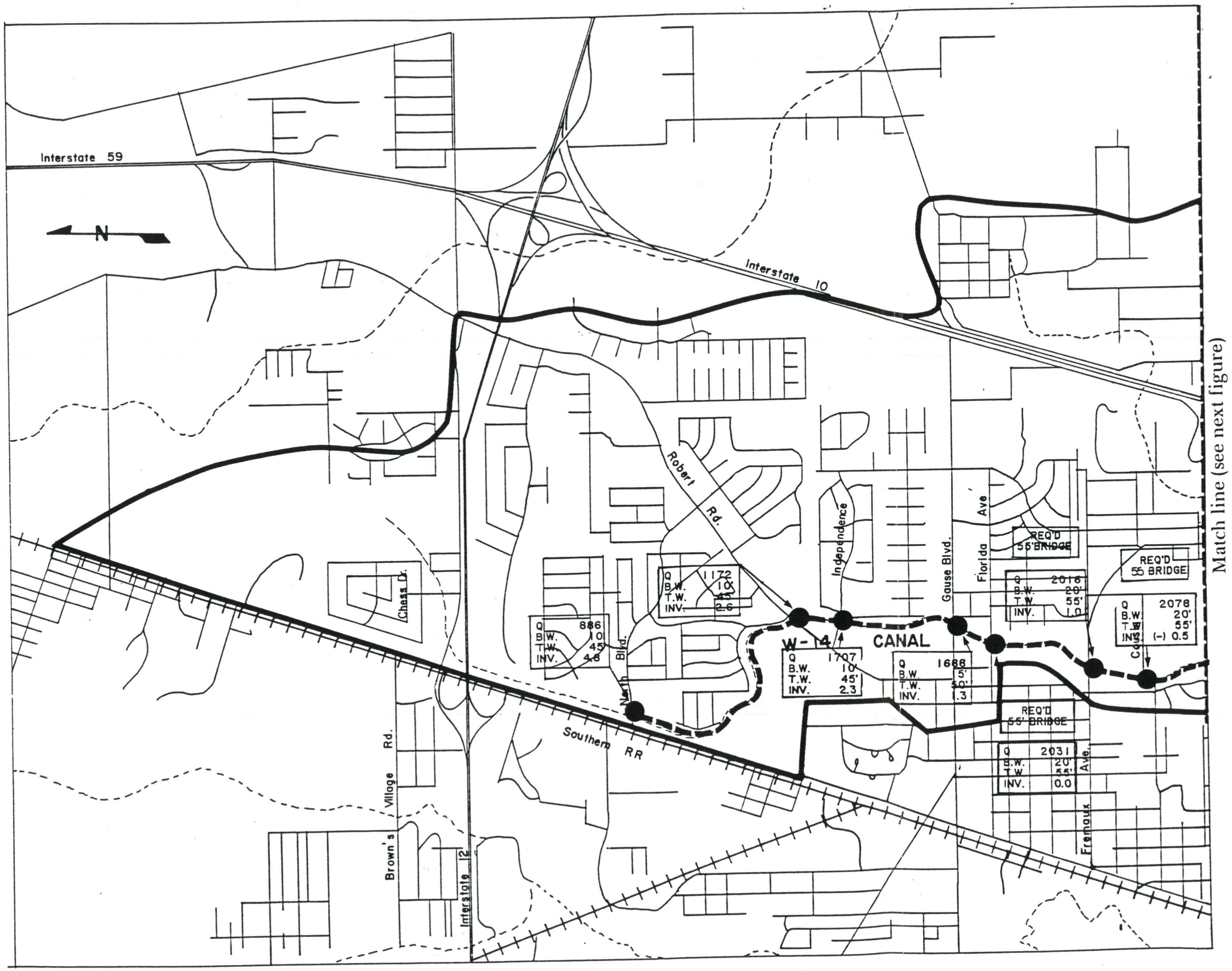


Drainage Basin Boundary

Tributaries

Q Peak Flow, cfs

B.W. Bottom Width

T.W. Top Width

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

Design Points

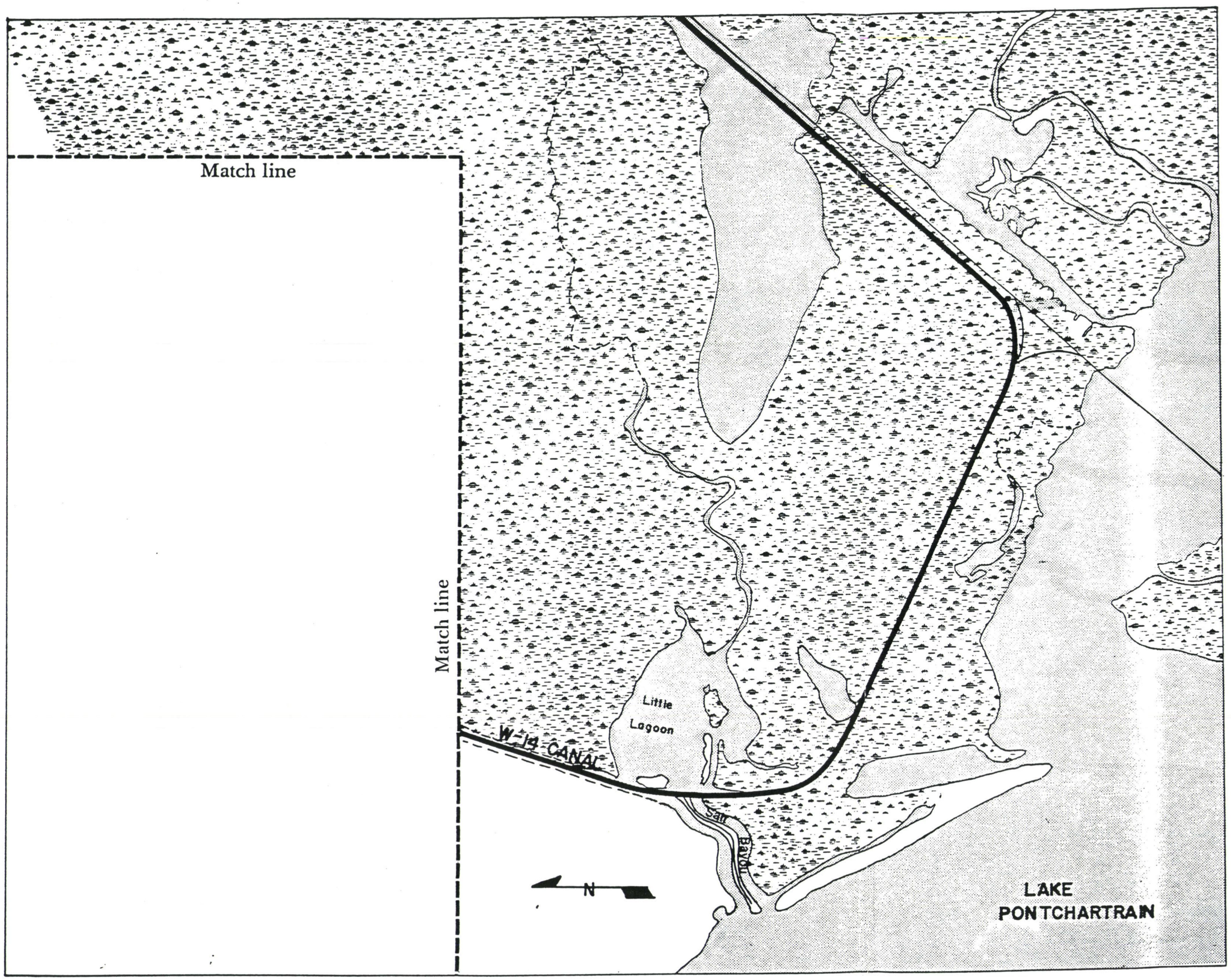
Concrete Slope Paving

Earthen Section

Burk & Associates
Incorporated

1000 0 1000 2000 3000 FEET

96m



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W-14 CANAL IMPROVEMENTS

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Q Peak Flow, cfs

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

Design Points

Earthen Section

Burk & Associates Incorporated

1000 0 1000 2000 3000 FEET

96o

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

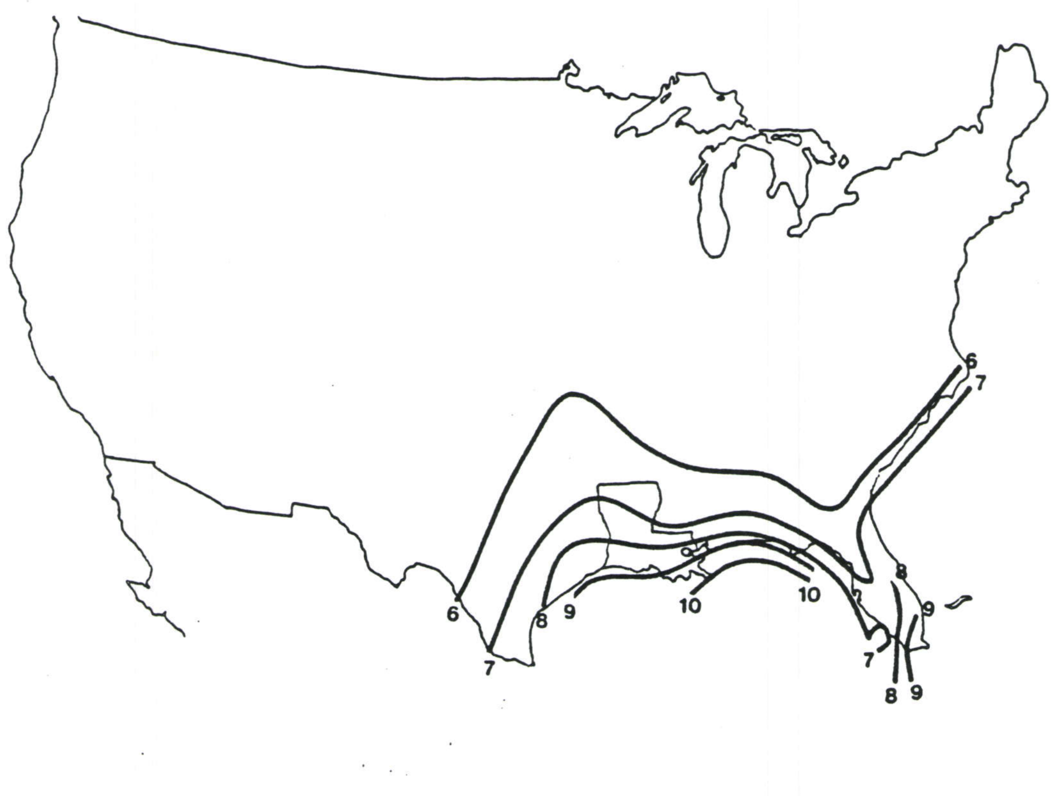
Runoffs for the 10 year-24 hour design storm were computed using the Soil Conservation Service (SCS) Runoff Curve Method. This technique is described in the SCS Handbook entitled, "National Engineering Handbook, Section 4, Hydrology" (NEH-4).

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:

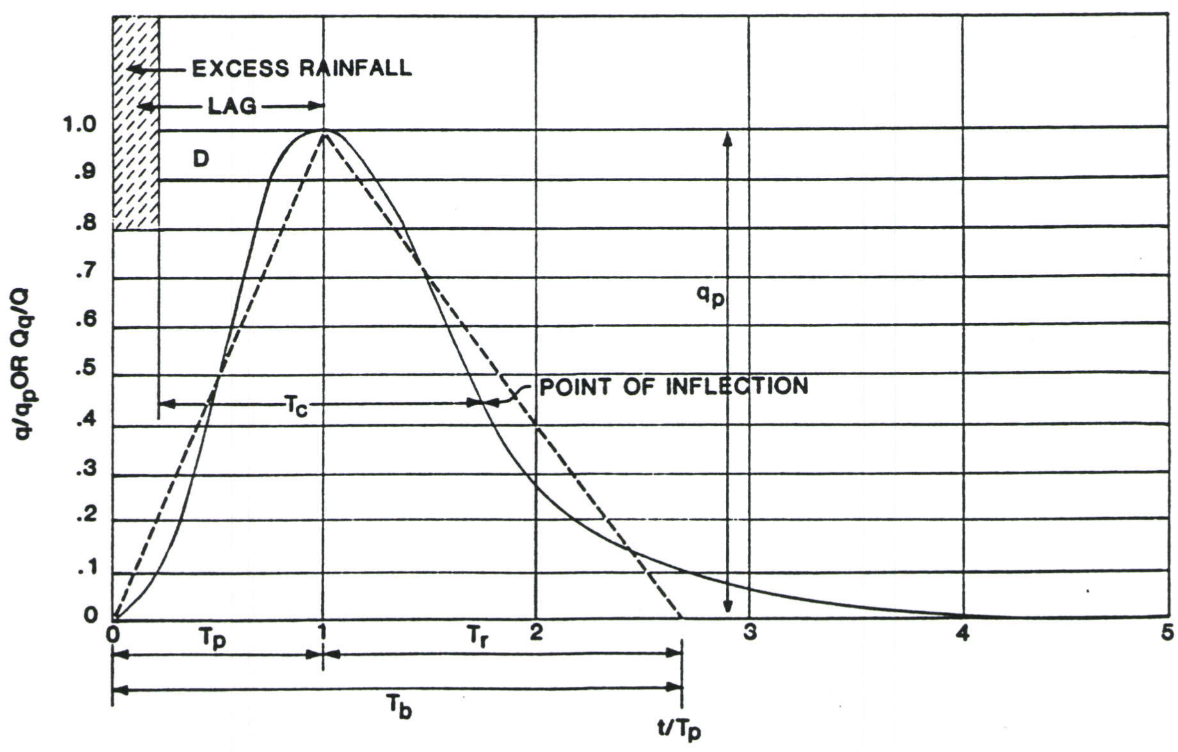
$$CN = \frac{1000}{S + 10}$$

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

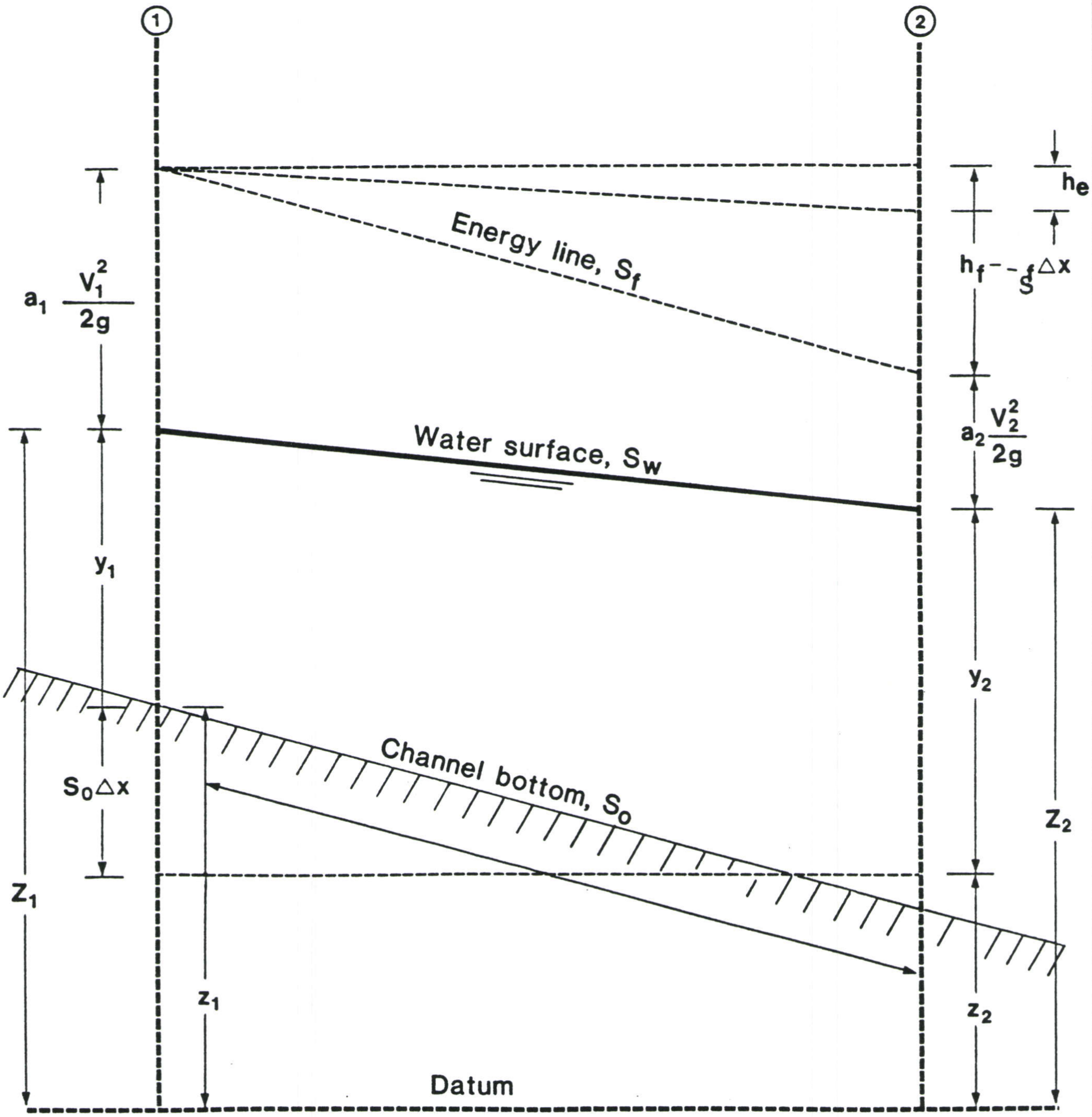
$$h_f = S_f x = 1/2(S_1 + S_2) x \qquad (3)$$

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

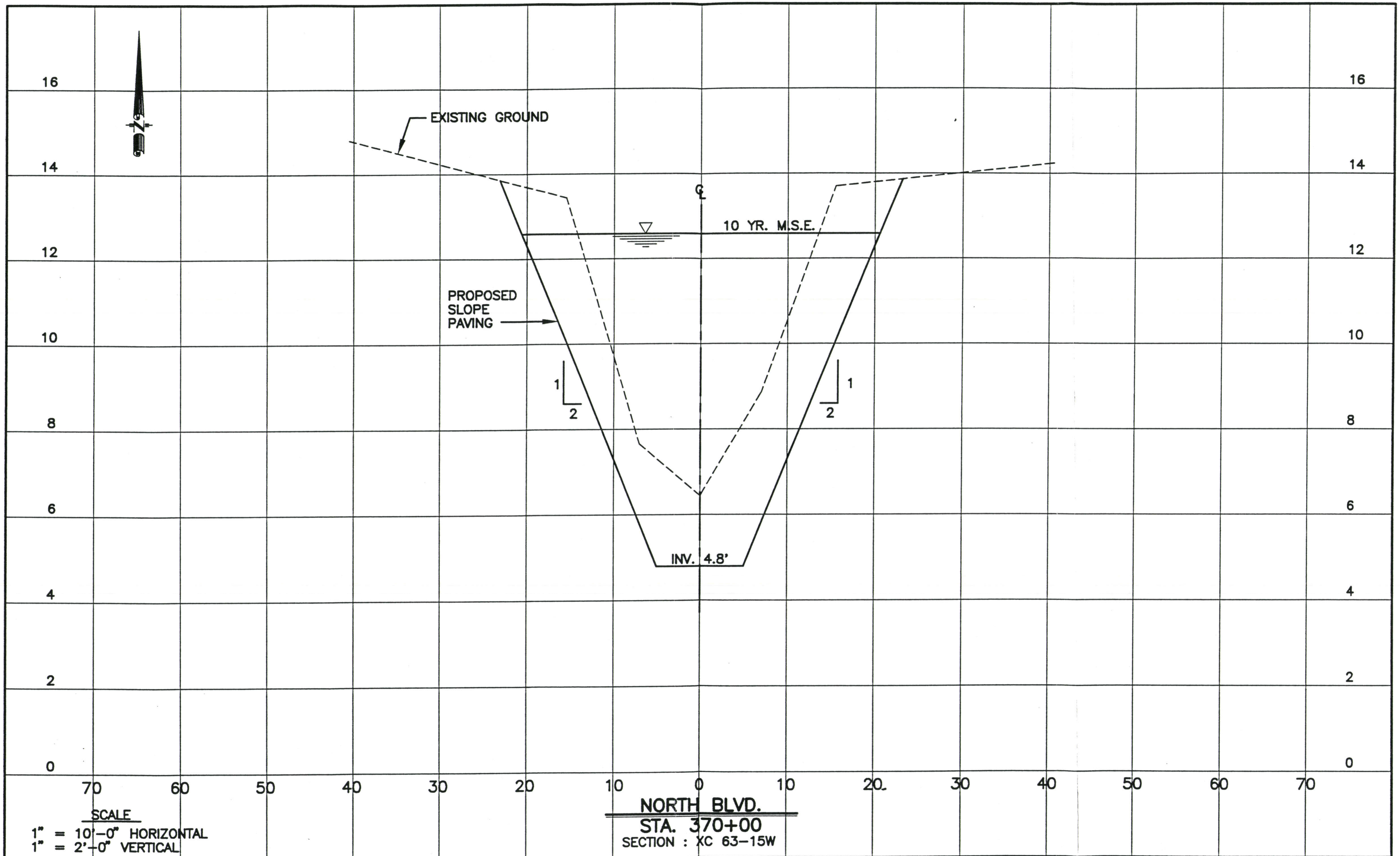
The drainage structures to be used in the Phase I area were designed to conform with proposed water surface elevations while adequately handling anticipated flows. Consequently, a trial and error process was required for this determination.

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.

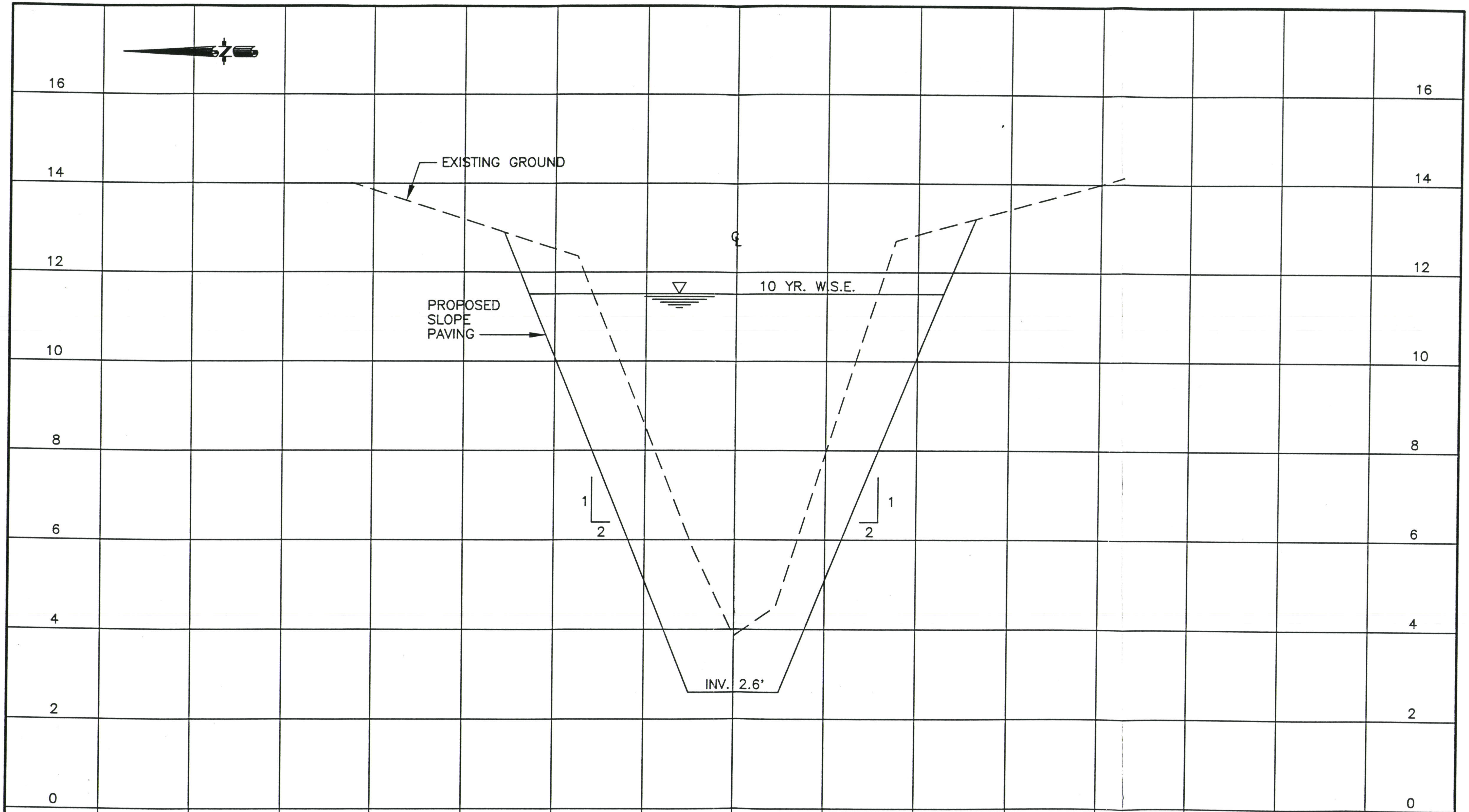
The following guidelines were established in this

APPENDIX C

CROSS-SECTIONS

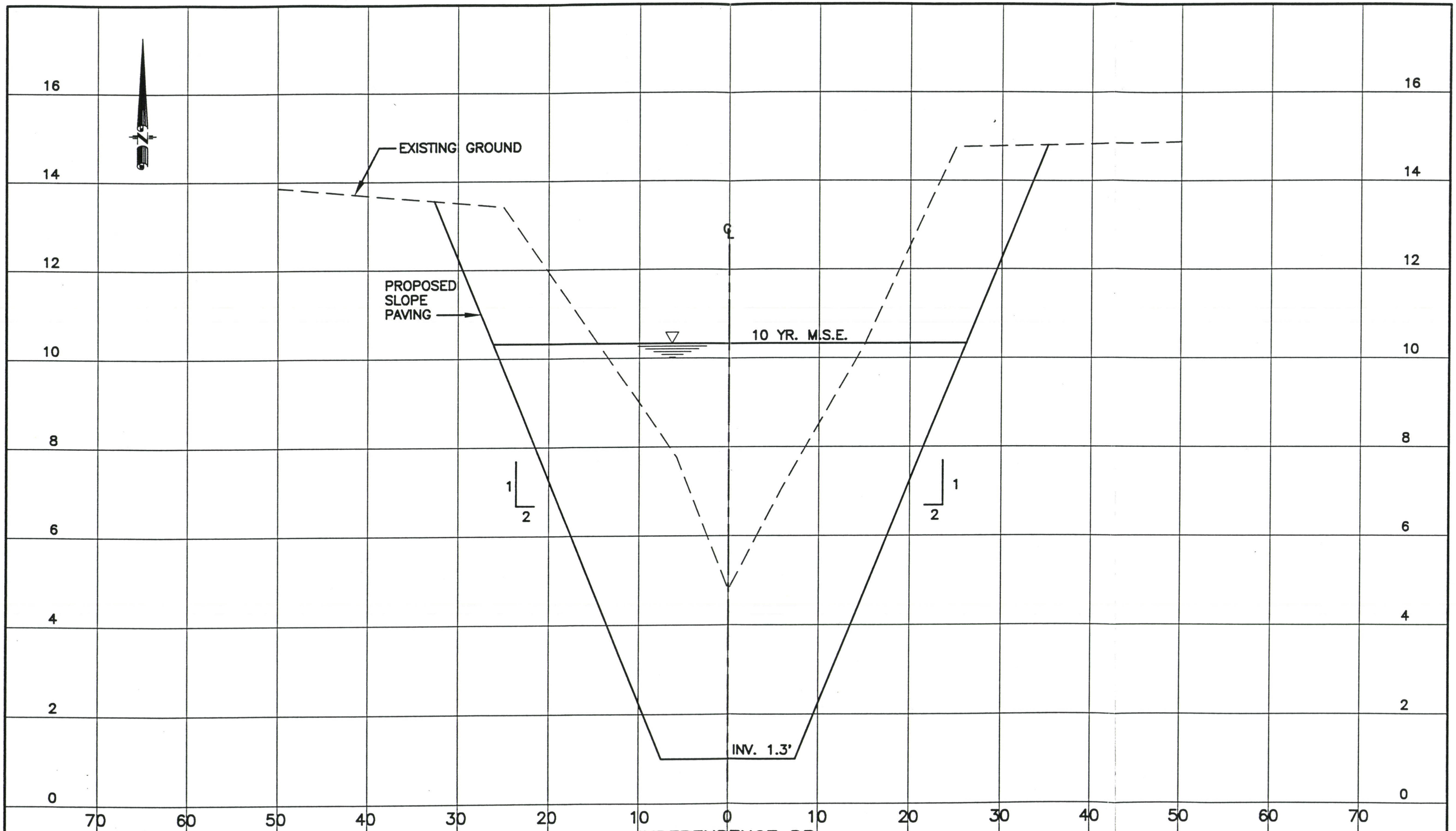


NORTH BLVD.
STA. 370+00
SECTION : XC 63-15W



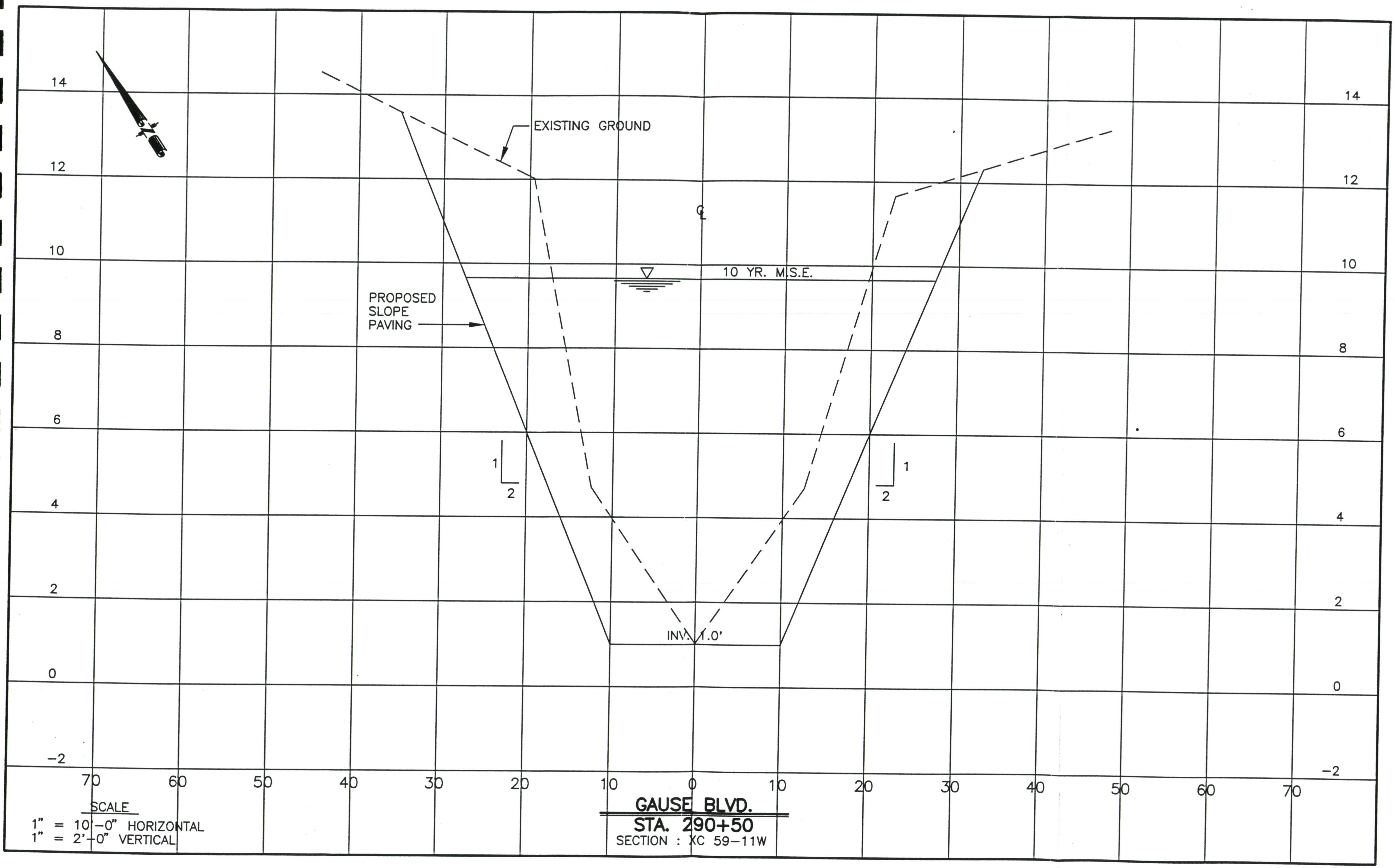
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL

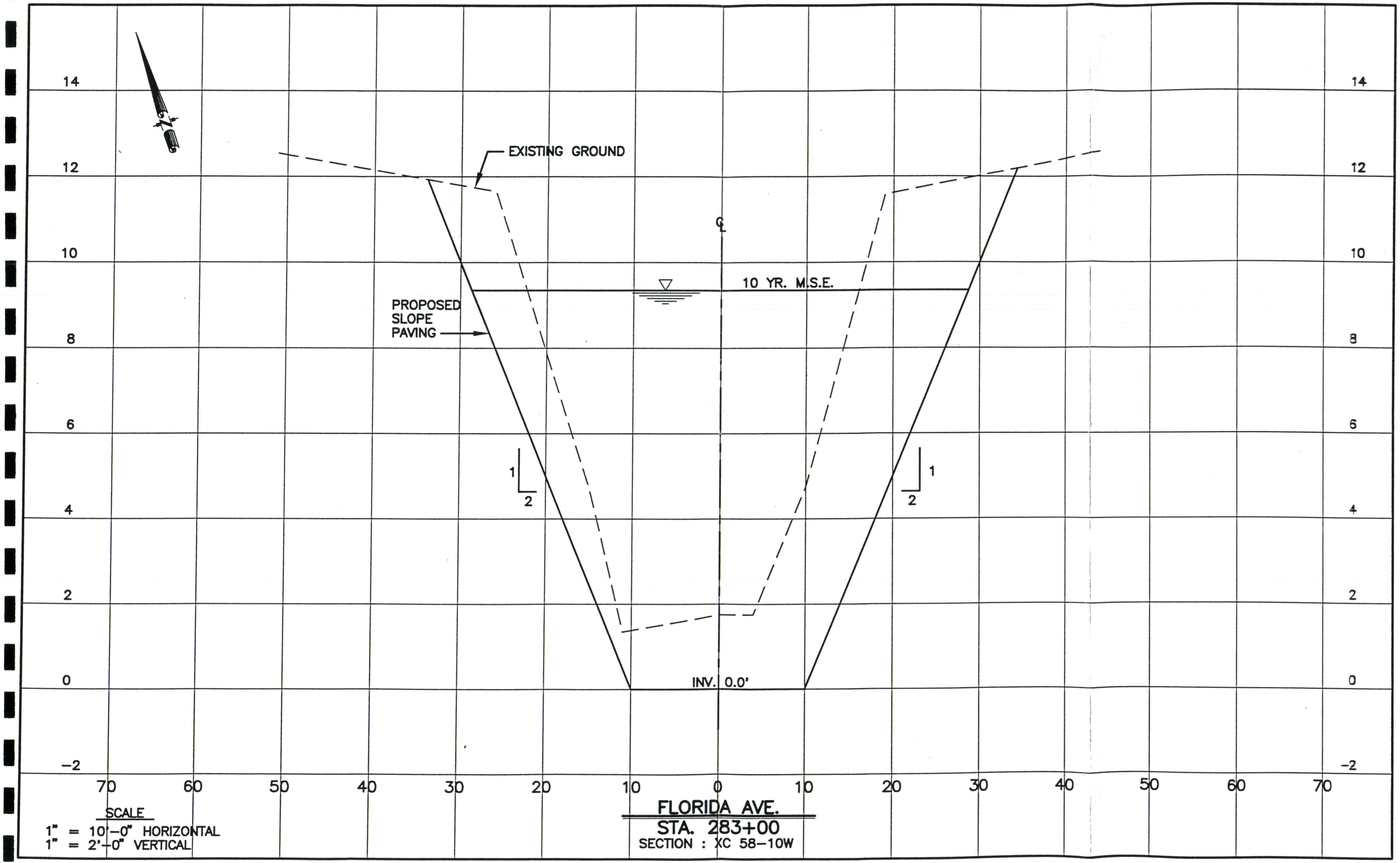
ROBERT RD.
STA. 320+30
SECTION : XC 61-13W



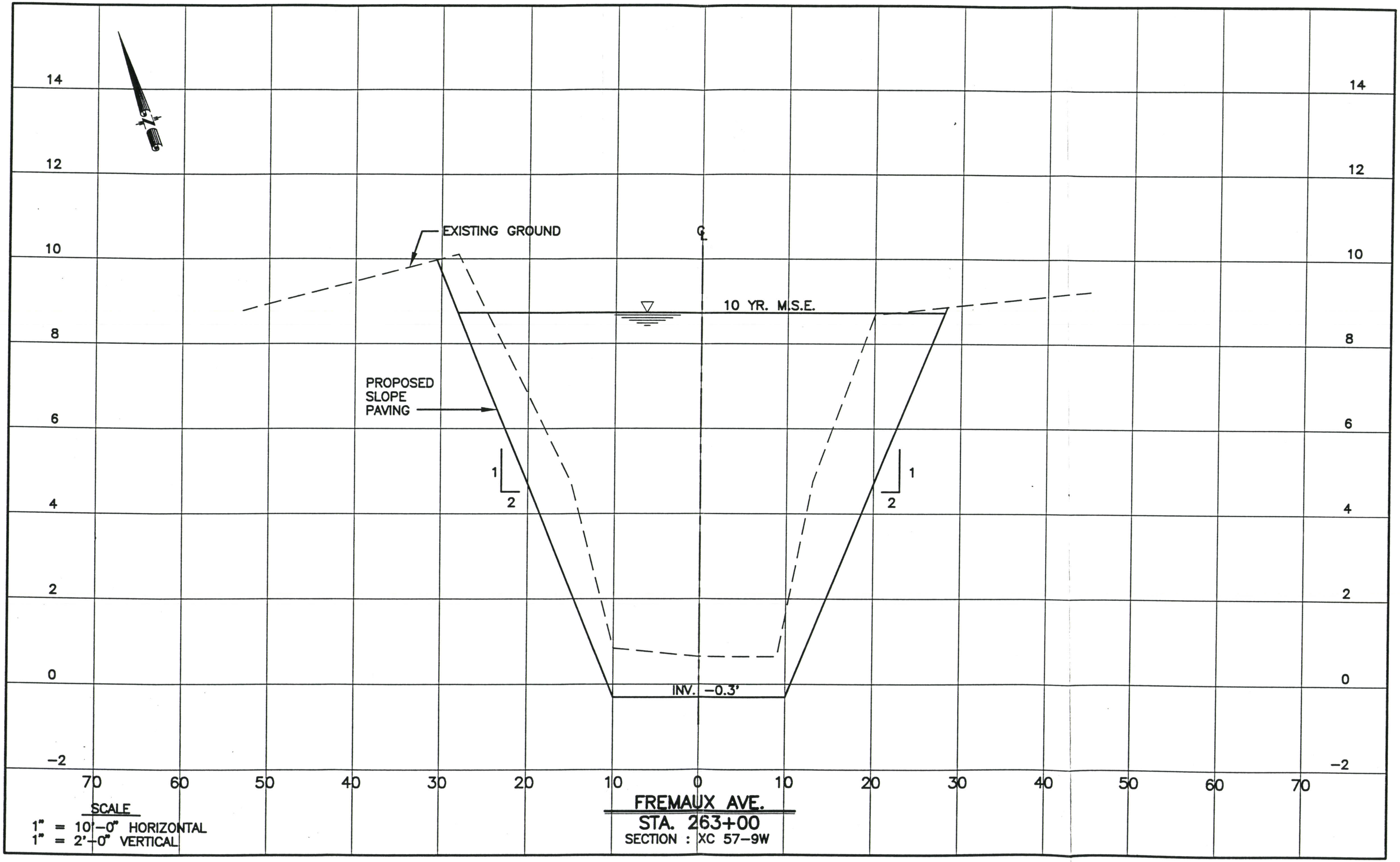
INDEPENDENCE DR.
STA. 312+50
SECTION : XC 60-12W

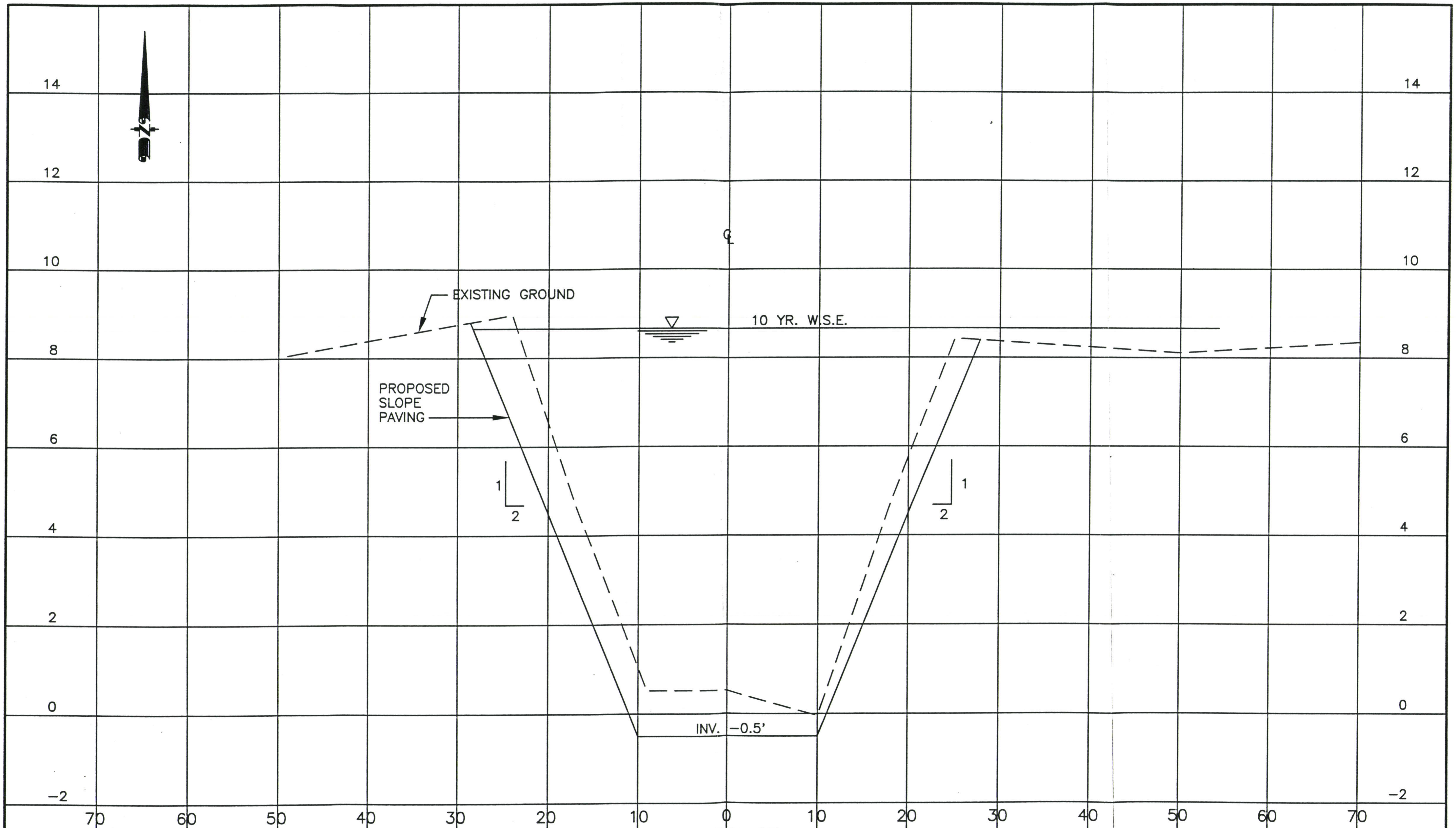
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL


EXISTING GROUND
PROPOSED SLOPE PAVING
10 YR. M.S.E.
INV. 1.0'
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL
GAUSE BLVD.
STA. 290+50
SECTION : XC 59-11W



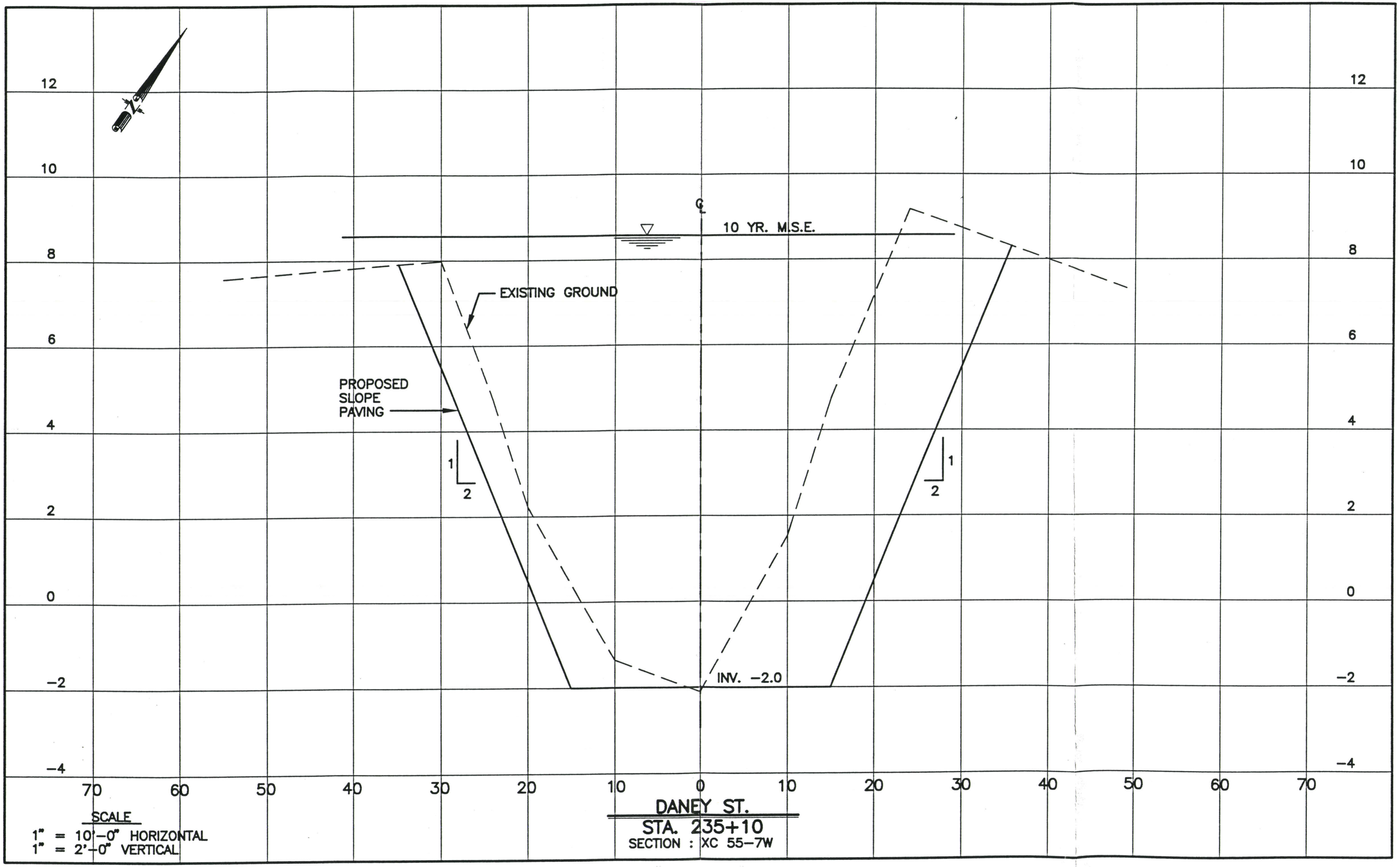
EXISTING GROUND
PROPOSED SLOPE PAVING
10 YR. M.S.E.
INV. 0.0'
FLORIDA AVE.
STA. 283+00
SECTION : XC 58-10W
SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL


EXISTING GROUND
10 YR. M.S.E.
PROPOSED SLOPE PAVING
INV. −0.3'
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
FREMAUX AVE.
STA. 263+00
SECTION : XC 57−9W

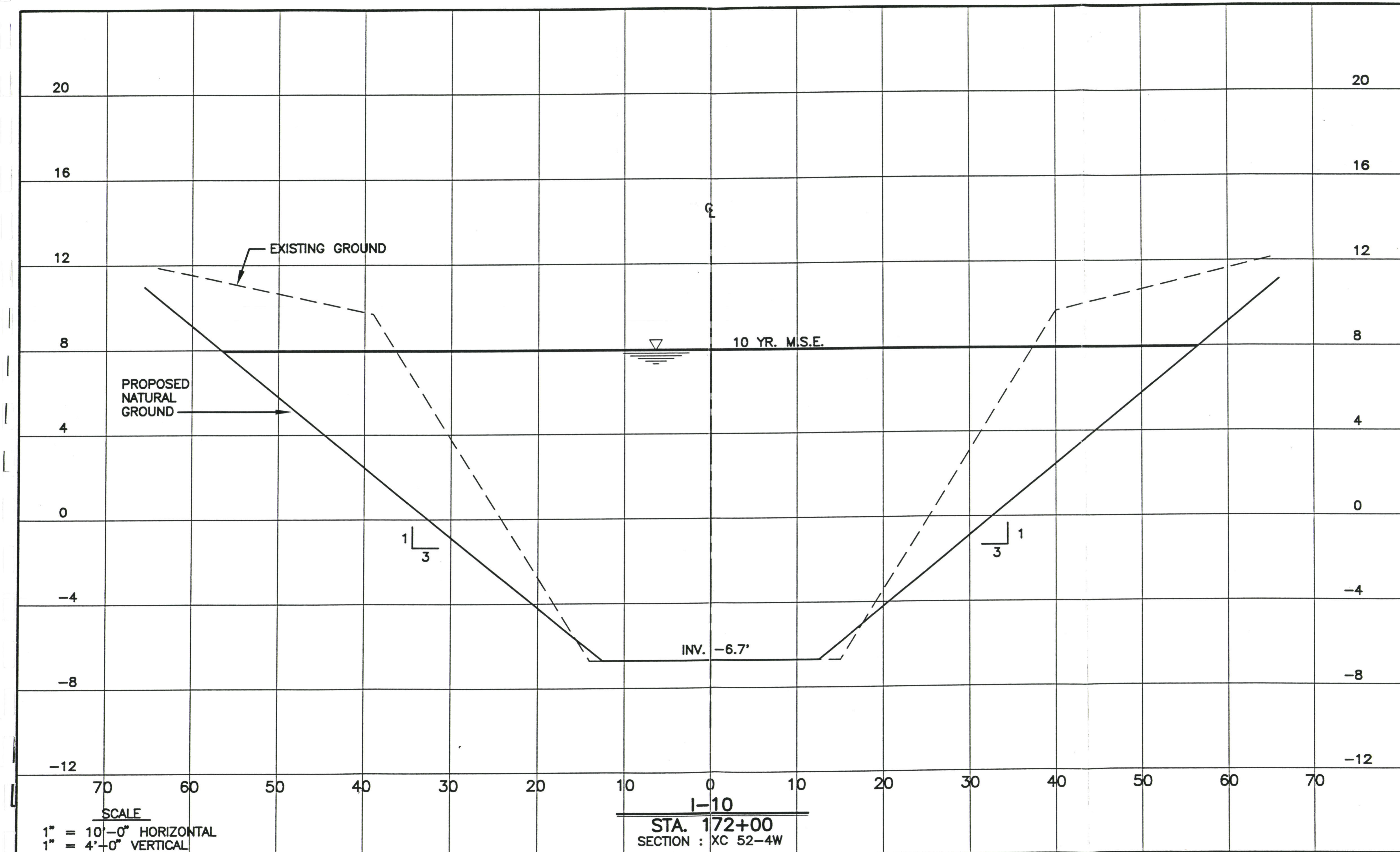


SCALE
1" = 10'−0" HORIZONTAL
1" = 2'−0" VERTICAL

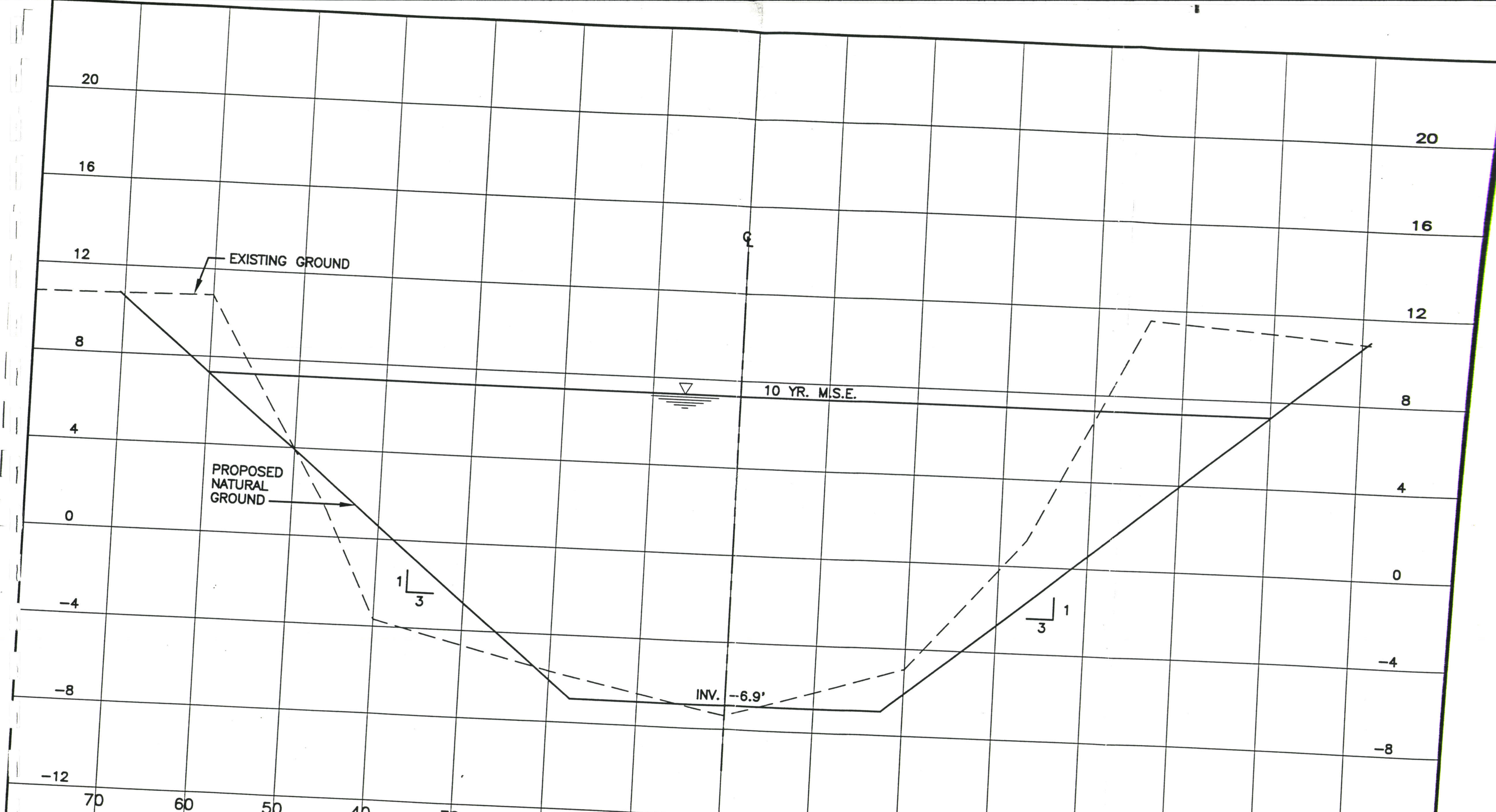
COUSIN ST.
STA. 255+50
SECTION : XC 56−8W



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

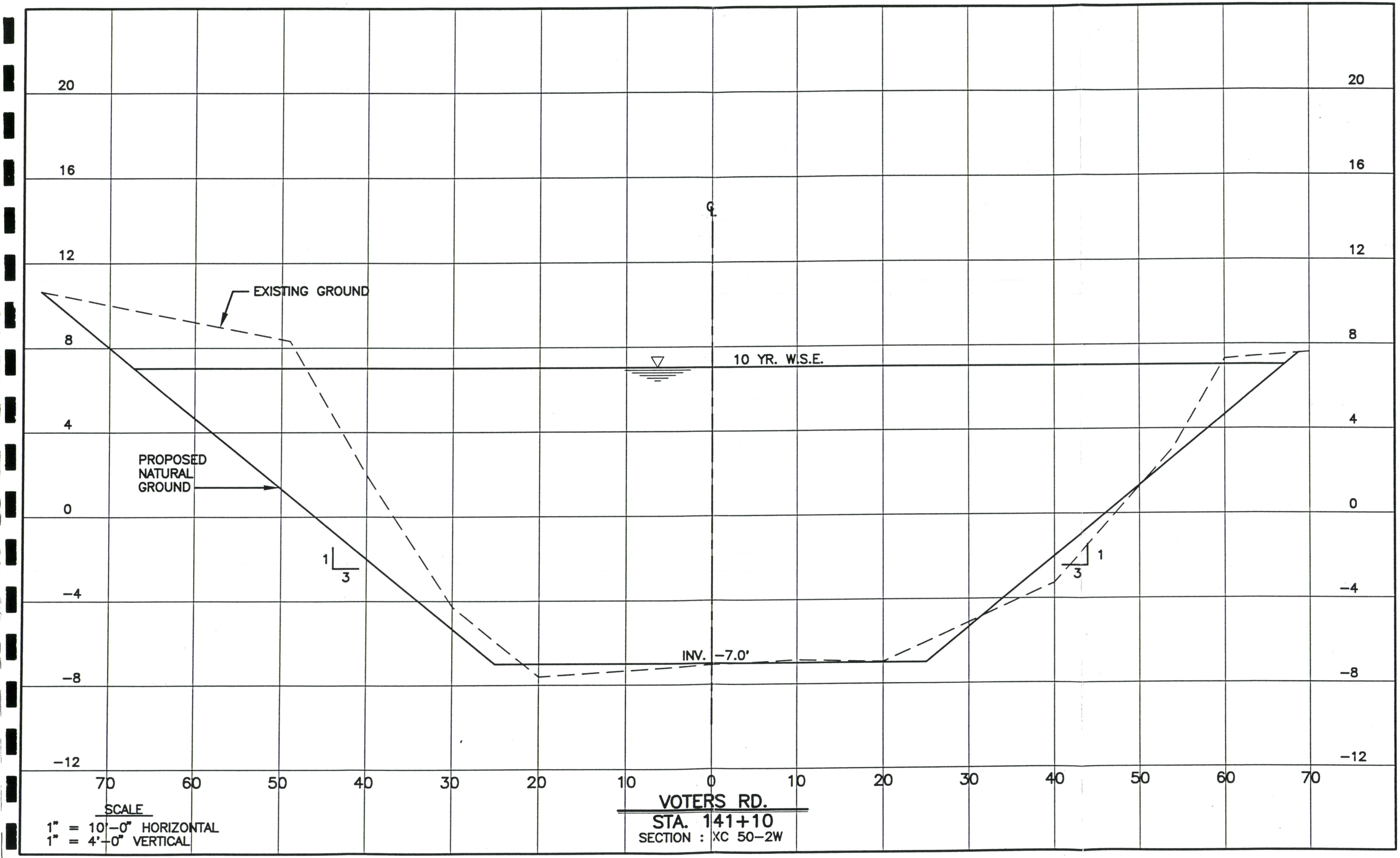


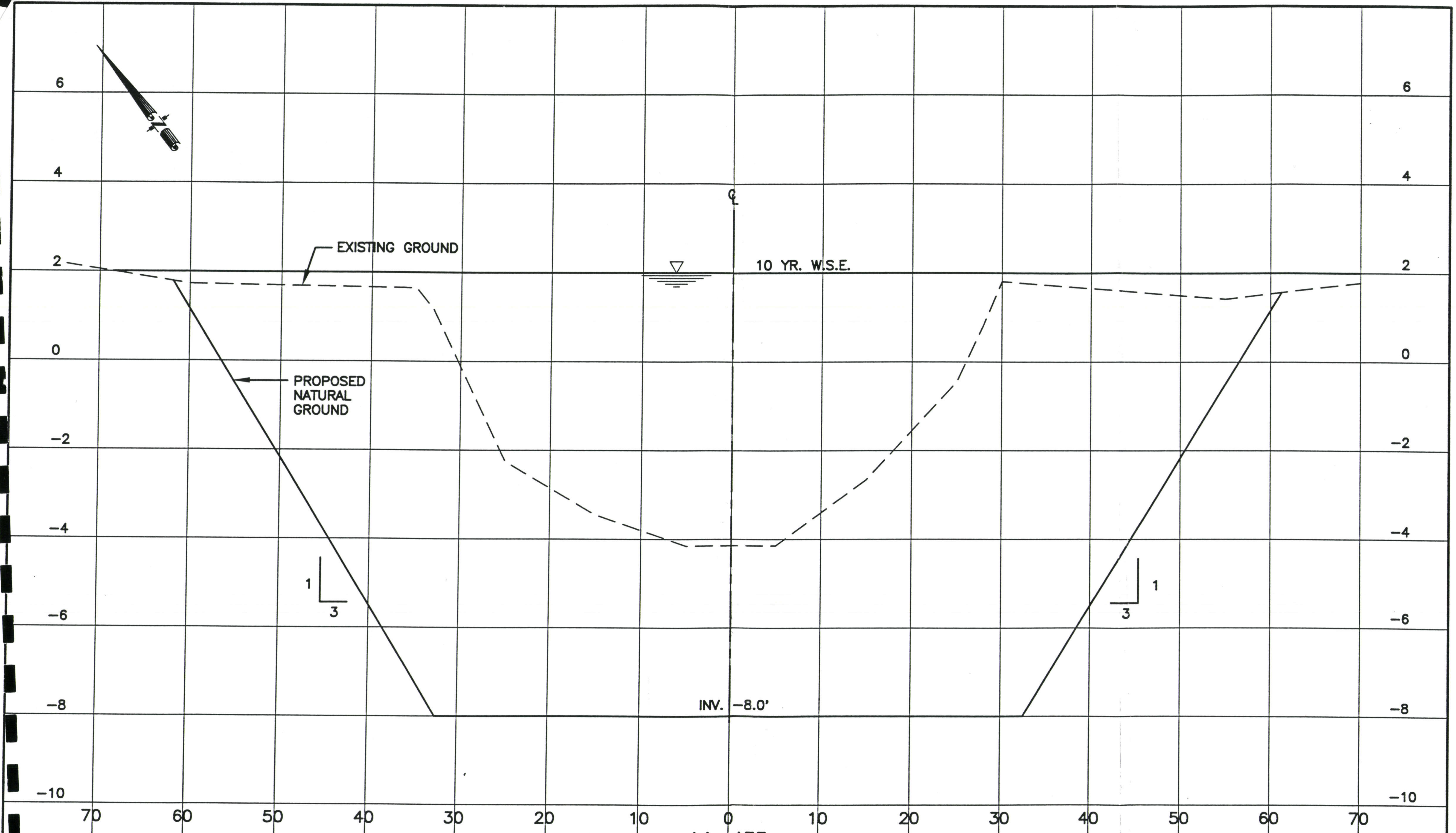
I−10
STA. 172+00
SECTION : XC 52−4W



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

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


EXISTING GROUND
PROPOSED
NATURAL
GROUND
10 YR. W.S.E.
INV. -7.0'
1
3
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



SCALE
1" = 10'-0" HORIZONTAL
1" = 2'-0" VERTICAL

LA. 433
STA. 010+00
SECTION : XC 49-1W